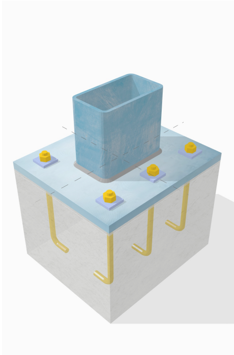
REFERENCES | CALCULATIONS | RESULTS

SkyCiv Base Plate and Anchor Rod Design

Shear Design Example for AISC

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

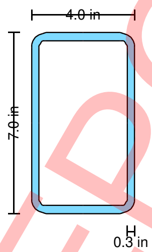


Load Combinations:

Load Combination	Axial Load, Nx (kip)	Strong-axis Shear, Vy (kip)	Weak-axis Shear, Vz (kip)
1	0.00	2.00	2.00

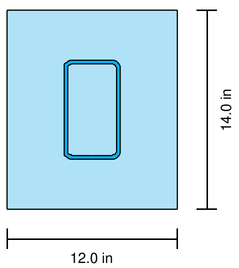
Note(s):
1. Positive Nx (+) indicates compression; negative Nx (-) indicates tension.
2. Shear forces are assumed to be distributed to all anchors in the base plate.
3. Welded plate washers are used to distribute shear forces.

Column Properties:



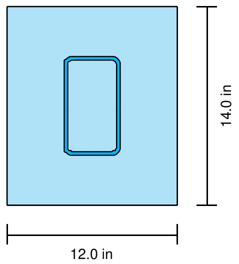
Symbol	Description	Value
$Section$	Column section	HSS7x4x5/16
d_{col}	Column depth	7.000 in
t_{col}	Column thickness	0.291 in
b_{col}	Column width	4.000 in
A_{col}	Column area	5.850 in^2
r_{col}	Column root radius	0.291 in
F_{y_col}	Column yield stress (A36)	36.000 ksi
F_{u_col}	Column tensile stress (A36)	58.000 ksi

Base Plate Properties:



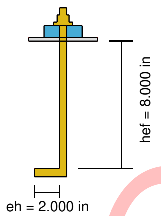
Symbol	Description	Value
B_{bp}	Base plate width	12.000 in
L_{bp}	Base plate length	14.000 in
t_{bp}	Base plate thickness	0.750 in
A_{bp}	Base plate plan area	168.000 in^2
F_{y_bp}	Base plate yield stress (A36)	36.000 ksi
F_{u_bp}	Base plate tensile stress (A36)	58.000 ksi
t_{grout}	Grout thickness	0.250 in

Concrete Properties:



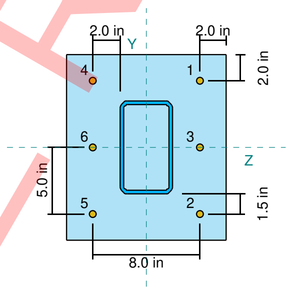
Symbol	Description	Value
B_{conc}	Concrete width	12.000 in
L_{conc}	Concrete length	14.000 in
t_{conc}	Concrete thickness	10.000 in
A_{conc}	Concrete plan area	168.000 in^2
$f_c^{'}$	Concrete compressive strength (3000)	3.000 ksi
λ	Factor for normal-weight concrete	1.000
—	Concrete assumption: cracked or uncracked	Cracked

Anchor Information:



Symbol	Description	Value
d_a	Anchor rod diameter	0.500 in
h_{ef}	Anchor rod effective embedment length	8.000 in
e_h	Anchor hook length	2.000 in
F_{y_anc}	Anchor rod yield stress (A325)	92.000 ksi
F_{u_anc}	Anchor rod tensile stress (A325)	120.000 ksi
t_{pw}	Plate washer thickness	0.250 in

Anchor Layout:



Symbol	Description	Value
n_a	Total number of anchor rods	6.000
s_z	Spacing of anchor rods along Z-axis	8.000 in
s_y	Spacing of anchor rods along Y-axis	5.000 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

Anchor Data Summary:

ID	Z (in)	Y (in)
1	4.000	5.000
2	4.000	-5.000
3	4.000	0.000
4	-4.000	5.000
5	-4.000	-5.000
6	-4.000	0.000

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

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: 1.75 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: 6.00	PASS	AISC Design Guide 1 3rd Ed. Section 4.2
Min. Anchor Spacing to Prevent Concrete Splitting	Min: 2.05 in in, Actual: 5.00 in	PASS	ACI 318-19 Table 17.9.2(a)
Min. Concrete Cover to Prevent Concrete Splitting	Min: 1.50 in in, Actual: 2.00 in	PASS	ACI 318-19 Table 20.5.1.3.1

The design geometry meets requirements!

REFERENCES	CALCULATIONS	RESULTS

Load Combination No. 1: Shear Load Only

Check No. 1: Weld Capacity

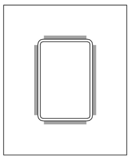


Fig. Welds on column to base plate.

Calculate total weld length on four sides of HSS

$d_{col} = 7\ \mathrm{in}$ - Column depth

$b_{col} = 4\ \mathrm{in}$ - Column width

$r_{col} = 0.291\ \mathrm{in}$ - Column root radius

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

L_{weld} - Total length of weld

$$L_{weld} = 2\left(b_{col} - 2r_{col} - 2t_{col}\right) + 2\left(d_{col} - 2r_{col} - 2t_{col}\right)$$

$$L_{weld} = 2 \times (4\ \mathrm{in} - 2 \times 0.291\ \mathrm{in} - 2 \times 0.291\ \mathrm{in}) + 2 \times (7\ \mathrm{in} - 2 \times 0.291\ \mathrm{in} - 2 \times 0.291\ \mathrm{in}) = 17.344\ \mathrm{in}$$

Calculate load per unit length of weld

$V_y = 2\ \mathrm{kip}$ - Vy shear load

$V_z = 2\ \mathrm{kip}$ - Vz shear load

v_{uy} - Required weld strength per unit length

$$v_{uy} = \frac{V_y}{L_{weld}} = \frac{2\ \mathrm{kip}}{17.344\ \mathrm{in}} = 0.11531\ \mathrm{kip/in}$$

v_{uz} - Required weld strength per unit length

$$v_{uz} = \frac{V_z}{L_{weld}} = \frac{2\ \mathrm{kip}}{17.344\ \mathrm{in}} = 0.11531\ \mathrm{kip/in}$$

r_u - Required weld strength per unit length

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

$$r_u = \sqrt{\left((0.11531\ \mathrm{kip/in})^2\right) + \left((0.11531\ \mathrm{kip/in})^2\right)} = 0.16308\ \mathrm{kip/in}$$

Calculate fillet weld capacity

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

$E_w = 0.177\ \mathrm{in}$ - 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

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

Calculate base metal capacities

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

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

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

$t_{bp} = 0.75\ \mathrm{in}$ - 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)

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

AISC 360-22 Eq. J4-4 — $\phi r_{nbm,bp}$ - Design shear rupture strength of base metal (base plate)

$$\phi r_{nbm,bp} = \phi 0.6 F_{u\_bp} t_{bp} = 0.75 \times 0.6 \times 58\ \mathrm{ksi} \times 0.75\ \mathrm{in} = 19.575\ \mathrm{kip/in}$$

AISC 360-22 Eq. J4-4 — ϕr_{nbm} - Governing design shear rupture strength of base metal

$$\phi r_{nbm} = \min\left(\phi \mathrm{r_{nbm,bp}},\ \phi \mathrm{r_{nbm,col}}\right) = \min\left(19.575\ \mathrm{kip/in},\ 7.5951\ \mathrm{kip/in}\right) = 7.5951\ \mathrm{kip/in}$$

Result: — **PASS = 0.03**

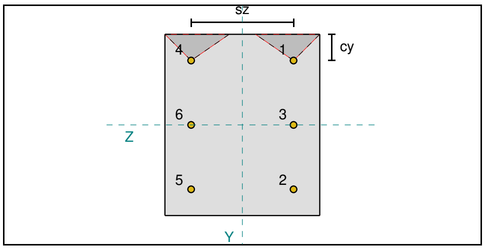
DCR - Demand over capacity ratio, comparing two conditions:

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

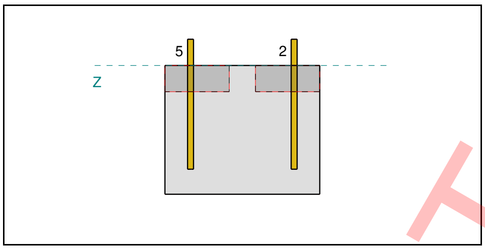
$$DCR = \max\left(\frac{0.16308\ \text{kip/in}}{7.5951\ \text{kip/in}}, \frac{0.16308\ \text{kip/in}}{5.5755\ \text{kip/in}}\right) = 0.029249$$

Check No. 2: Concrete Breakout Capacity (Vy Shear)

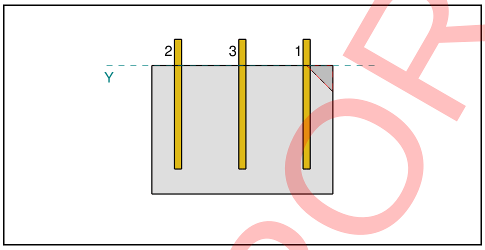
For breakout capacity on failure edge perpendicular to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_y = 2$ kip - Vy shear load

$n_a = 6$ - Total number of anchor rods

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

$$V_{fa\perp} = \frac{V_y}{n_a} = \frac{2\ \text{kip}}{6} = 0.33333\ \text{kip}$$

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)	10.000 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.000 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.000 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	12.000 in
t_{conc}	Concrete thickness	10.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vy shear)	2.000 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

$c_{a1,s1} = 2$ 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

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2\ \text{in})^2 = 18\ \text{in}^2$$

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

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

$$B_{Vc} = \min\left(10\ \text{in},\ 1.5 \times 2\ \text{in}\right) + \min\left(2\ \text{in},\ 1.5 \times 2\ \text{in}\right) = 5\ \text{in}$$

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

$$H_{Vc} = \min\left(1.5\text{c}_{\text{a1,s1}},\ \text{t}_{\text{conc}}\right) = \min\left(1.5 \times 2\ \text{in},\ 10\ \text{in}\right) = 3\ \text{in}$$

Calculate actual projected area

ACI 318-19 Clause

17.7.2.1.1 A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 5\text{ in} \times 3\text{ in} = 15\text{ in}^2$$

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

$h_{ef} = 8\text{ in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1 l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 8\text{ in}$$

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

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

$$V_{b1} = 7 \times \left(\frac{\min\left(8\text{ in},\ 8 \times 0.5\text{ in}\right)}{0.5\text{ in}}\right)^{0.2} \times \sqrt{\frac{0.5\text{ in}}{1\text{ in}}} \times 1 \times \sqrt{\frac{3\text{ ksi}}{0.001\text{ ksi}}} \times \left(\frac{2\text{ in}}{1\text{ in}}\right)^{1.5} \times 0.001\text{ kip}$$

$$V_{b1} = 1.1623\text{ kip}$$

ACI 318-19 Eq. 17.7.2.2.1b V_{b2} - Basic single anchor breakout strength condition 2

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

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\text{ ksi}}{0.001\text{ ksi}}} \times \left(\frac{2\text{ in}}{1\text{ in}}\right)^{1.5} \times 0.001\text{ kip} = 1.3943\text{ kip}$$

ACI 318-19 Clause 17.7.2.2.1 V_b - Basic single anchor breakout strength

$$V_b = \min\left(\mathrm{V_{b1}},\ \mathrm{V_{b2}}\right) = \min\left(1.1623\text{ kip},\ 1.3943\text{ kip}\right) = 1.1623\text{ kip}$$

Calculate edge effect factor

$c_{a2,s1} = 2\text{ in}$ - 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

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a2,s1}}}{1.5\mathrm{c_{a1,s1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{2\text{ in}}{1.5 \times 2\text{ in}}\right)\right) = 0.9$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1 $\Psi_{h,V}$ - Breakout thickness factor

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,s1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 2\text{ in}}{10\text{ in}}},\ 1\right) = 1$$

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)

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

$$\phi V_{cb\perp} = 0.65 \times \left(\frac{15\text{ in}^2}{18\text{ in}^2}\right) \times 0.9 \times 1 \times 1 \times 1.1623\text{ kip} = 0.56661\text{ kip}$$

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)

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

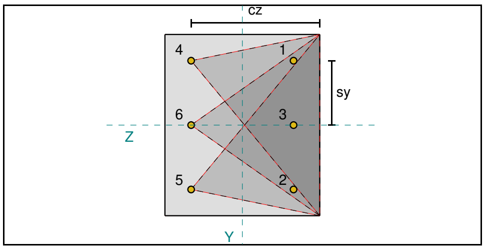
$$\phi V_{cb\perp} = 0.65 \times \left(\frac{15\text{ in}^2}{18\text{ in}^2}\right) \times 0.9 \times 1 \times 1 \times 1.1623\text{ kip} = 0.56661\text{ kip}$$

For breakout capacity on failure edge parallel to load:

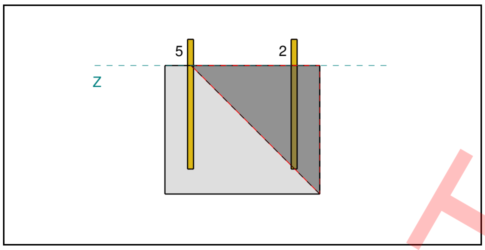
Determine cases:

Symbol	Description	Value
$s_{z,outer}$	Spacing of outer anchor rods along Z-axis	8.000 in
$c_{a1,g1}$	Anchor group distance to failure edge (+Vz shear)	2.000 in
	Is welded plate washer used?	Yes
	Applicable Cases	Case 2

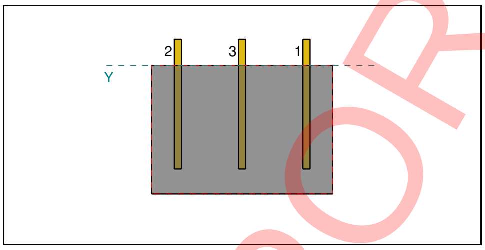
Calculations for Case 2



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group.

$V_y = 2\ \mathrm{kip}$ - Vy shear load

Assume Vy load is acting perpendicular to the anchors along the failure edge.

$V_{fa\|,case2}$ - Required concrete breakout shear strength of anchor group with failure edge parallel to load

$$V_{fa\|,case2} = V_y = 2\ \mathrm{kip}$$

ACI 318-19 Clause 17.7.2.1.2

Determine if the support is a narrow concrete member

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

Symbol	Description	Value
$c_{left,g2}$	Anchor group BO Vz 2 concrete edge distance (left)	2.000 in
$c_{right,g2}$	Anchor group BO Vz 2 concrete edge distance (right)	10.000 in
$c_{top,g2}$	Anchor group BO Vz 2 concrete edge distance (top)	2.000 in
$c_{bottom,g2}$	Anchor group BO Vz 2 concrete edge distance (bottom)	2.000 in
t_{conc}	Concrete thickness	10.000 in
s_y	Spacing of anchor rods along Y-axis	5.000 in
$c_{a1,g2}$	Rear anchor group distance to failure edge (+Vz shear)	10.000 in
	Narrow Member	TRUE
$c'_{a1,g2}$	Rear anchor group modified distance to failure edge (+Vz shear)	6.667 in

Calculation for critical anchor group: Anchor Group BO Vz 2.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c'_{a1,g2} = 6.6667\ \mathrm{in}$ - Rear anchor group modified distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5\left(c'_{a1,g2}\right)^2 = 4.5 \times (6.6667\ \mathrm{in})^2 = 200\ \mathrm{in}^2$$

Calculate width of actual projected area on failure surface

$s_{sum,y,g2} = 10\ \mathrm{in}$ - Anchor group BO Vz 2 sum of spacing along Y-axis

$n_{y,g2} = 3$ - Number of anchors along Y-axis for anchor group BO Vz 2

ACI 318-19 Clause 17.7.2.1.1

B_{Vc} - Actual width of failure surface for an anchor group

$$B_{Vc} = \min\left(\mathrm{c_{bottom,g2}},\ 1.5\mathrm{c'_{a1,g2}}\right) + \left(\min\left(\mathrm{s_{sum,y,g2}},\ 3\mathrm{c'_{a1,g2}}\left(\mathrm{n_{y,g2}} - 1\right)\right)\right) + \min\left(\mathrm{c_{top,g2}},\ 1.5\mathrm{c'_{a1,g2}}\right)$$

$$B_{Vc} = \min(2\ \mathrm{in},\ 1.5 \times 6.6667\ \mathrm{in}) + (\min(10\ \mathrm{in},\ 3 \times 6.6667\ \mathrm{in} \times (3-1))) + \min(2\ \mathrm{in},\ 1.5 \times 6.6667\ \mathrm{in})$$

$$B_{Vc} = 14\ \text{in}$$

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

$$H_{Vc} = \min\left(1.5c'_{a1,g2},\ t_{conc}\right) = \min(1.5 \times 6.6667\ \text{in},\ 10\ \text{in}) = 10\ \text{in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 14\ \text{in} \times 10\ \text{in} = 140\ \text{in}^2$$

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

$h_{ef} = 8\ \text{in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 8\ \text{in}$$

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

$$V_{b1} = 7\left(\frac{\min(l_e,\ 8d_a)}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{in}}\lambda_a\sqrt{\frac{f_c'}{psi}}\left(\frac{c'_{a1,g2}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7 \times \left(\frac{\min(8\ \text{in},\ 8 \times 0.5\ \text{in})}{0.5\ \text{in}}\right)^{0.2} \times \sqrt{\frac{0.5\ \text{in}}{1\ \text{in}}} \times 1 \times \sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}} \times \left(\frac{6.6667\ \text{in}}{1\ \text{in}}\right)^{1.5} \times 0.001\ \text{kip}$$

$$V_{b1} = 7.0733\ \text{kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 9\lambda_a\sqrt{\frac{f_c'}{psi}}\left(\frac{c'_{a1,g2}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\ \text{ksi}}{0.001\ \text{ksi}}} \times \left(\frac{6.6667\ \text{in}}{1\ \text{in}}\right)^{1.5} \times 0.001\ \text{kip} = 8.4853\ \text{kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min(V_{b1},\ V_{b2}) = \min(7.0733\ \text{kip},\ 8.4853\ \text{kip}) = 7.0733\ \text{kip}$$

Calculate eccentricity factor

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

ACI 318-19 Clause 17.7.2.3.1

$\Psi_{ec,V}$ - Breakout eccentricity factor

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e'_N}{3c'_{a1,g2}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 6.6667\ \text{in}}}\right) = 1$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5c'_{a1,g2}}{t_{conc}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 6.6667\ \text{in}}{10\ \text{in}}},\ 1\right) = 1$$

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)

$\phi V_{cbg\|}$ - Design concrete breakout strength in shear of an anchor group (parallel edge)

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

$$\phi V_{cbg\|} = 2 \times 0.65 \times \left(\frac{140\ \text{in}^2}{200\ \text{in}^2}\right) \times 1 \times 1 \times 1 \times 1 \times 7.0733\ \text{kip} = 6.4367\ \text{kip}$$

Result:

PASS = 0.59

DCR - Demand over capacity ratio, comparing two conditions:

$$DCR = \max\left(\frac{V_{fa\perp}}{\phi V_{cb\perp}},\ \frac{V_{fa\|,case2}}{\phi V_{cbg\|}}\right)$$

$$DCR = \max\left(\frac{0.33333 \text{ kip}}{0.56661 \text{ kip}}, \frac{2 \text{ kip}}{6.4367 \text{ kip}}\right) = 0.5883$$

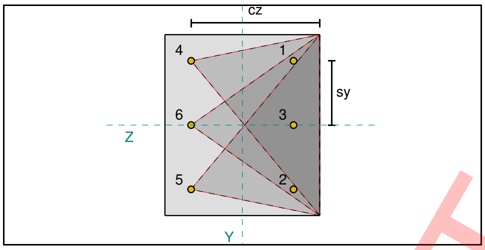
Check No. 3: Concrete Breakout Capacity (Vz Shear)

For breakout capacity on failure edge perpendicular to load:

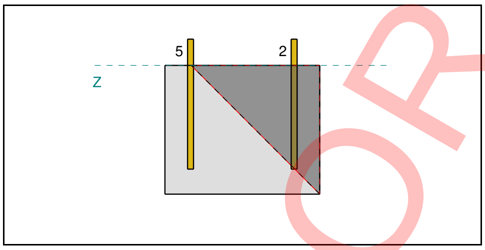
Determine cases:

Symbol	Description	Value
$s_{z,outer}$	Spacing of outer anchor rods along Z-axis	8.000 in
$c_{a1,g1}$	Anchor group distance to failure edge (+Vz shear)	2.000 in
	Is welded plate washer used?	Yes
	Applicable Cases	Case 2

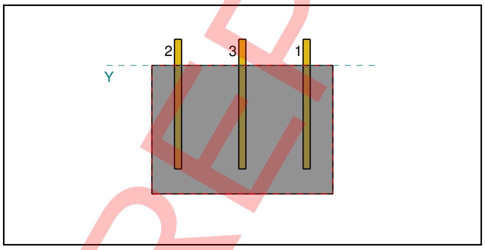
Calculations for Case 2



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group.

$V_z = 2 \text{ kip}$ - Vz shear load

$V_{fa\perp,case2}$ - Required concrete breakout shear strength of anchor group with failure edge perpendicular to load

$$V_{fa\perp,case2} = V_z = 2 \text{ kip}$$

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
$c_{left,g2}$	Anchor group BO Vz 2 concrete edge distance (left)	2.000 in
$c_{right,g2}$	Anchor group BO Vz 2 concrete edge distance (right)	10.000 in
$c_{top,g2}$	Anchor group BO Vz 2 concrete edge distance (top)	2.000 in
$c_{bottom,g2}$	Anchor group BO Vz 2 concrete edge distance (bottom)	2.000 in
t_{conc}	Concrete thickness	10.000 in
s_y	Spacing of anchor rods along Y-axis	5.000 in
$c_{a1,g2}$	Rear anchor group distance to failure edge (+Vz shear)	10.000 in
	Narrow Member	TRUE
$c'_{a1,g2}$	Rear anchor group modified distance to failure edge (+Vz shear)	6.667 in

Calculation for critical anchor group: Anchor Group BO Vz 2.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c'_{a1,g2} = 6.6667 \text{ in}$ - Rear anchor group modified distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5\left(c'_{a1,g2}\right)^2 = 4.5 \times (6.6667 \text{ in})^2 = 200 \text{ in}^2$$

ACI 318-19 Clause 17.7.2.1.1

Calculate width of actual projected area on failure surface

$s_{sum,y,g2} = 10$ in - Anchor group BO Vz 2 sum of spacing along Y-axis

$n_{y,g2} = 3$ - Number of anchors along Y-axis for anchor group BO Vz 2

B_{Vc} - Actual width of failure surface for an anchor group

$$B_{Vc} = \min\left(c_{bottom,g2},\ 1.5c'_{a1,g2}\right) + \left(\min\left(s_{sum,y,g2},\ 3c'_{a1,g2}\left(n_{y,g2} - 1\right)\right)\right) + \min\left(c_{top,g2},\ 1.5c'_{a1,g2}\right)$$

$$B_{Vc} = \min\left(2\text{ in},\ 1.5 \times 6.6667\text{ in}\right) + \left(\min\left(10\text{ in},\ 3 \times 6.6667\text{ in} \times (3 - 1)\right)\right) + \min\left(2\text{ in},\ 1.5 \times 6.6667\text{ in}\right)$$

$$B_{Vc} = 14\text{ in}$$

ACI 318-19 Clause 17.7.2.1.1

Calculate height of actual projected area on failure surface

H_{Vc} - Actual height of projected area

$$H_{Vc} = \min\left(1.5c'_{a1,g2},\ t_{conc}\right) = \min\left(1.5 \times 6.6667\text{ in},\ 10\text{ in}\right) = 10\text{ in}$$

ACI 318-19 Clause 17.7.2.1.1

Calculate actual projected area

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 14\text{ in} \times 10\text{ in} = 140\text{ in}^2$$

ACI 318-19 Table 17.2.4.1

Calculate modification factor for lightweight concrete

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

λ_a - Modification factor for lightweight concrete

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

ACI 318-19 Clause 17.7.2.2.1

Calculate load bearing length of the anchor

$h_{ef} = 8$ in - Anchor rod effective embedment length

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 8\text{ in}$$

ACI 318-19 Eq. 17.7.2.2.1a

Calculate basic single anchor breakout strength

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

$d_a = 0.5$ in - Anchor rod diameter

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

$$V_{b1} = 7\left(\frac{\min\left(l_e,\ 8d_a\right)}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{in}}\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c'_{a1,g2}}{in}\right)^{1.5} lbf$$

$$V_{b1} = 7 \times \left(\frac{\min\left(8\text{ in},\ 8 \times 0.5\text{ in}\right)}{0.5\text{ in}}\right)^{0.2} \times \sqrt{\frac{0.5\text{ in}}{1\text{ in}}} \times 1 \times \sqrt{\frac{3\text{ ksi}}{0.001\text{ ksi}}} \times \left(\frac{6.6667\text{ in}}{1\text{ in}}\right)^{1.5} \times 0.001\text{ kip}$$

$$V_{b1} = 7.0733\text{ kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 9\lambda_a\sqrt{\frac{f_c^{'}}{psi}}\left(\frac{c'_{a1,g2}}{in}\right)^{1.5} lbf$$

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\text{ ksi}}{0.001\text{ ksi}}} \times \left(\frac{6.6667\text{ in}}{1\text{ in}}\right)^{1.5} \times 0.001\text{ kip} = 8.4853\text{ kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(V_{b1},\ V_{b2}\right) = \min\left(7.0733\text{ kip},\ 8.4853\text{ kip}\right) = 7.0733\text{ kip}$$

ACI 318-19 Clause 17.7.2.3.1

Calculate eccentricity factor

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

$\Psi_{ec,V}$ - Breakout eccentricity factor

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e'_N}{3c'_{a1,g2}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 6.6667\text{ in}}}\right) = 1$$

ACI 318-19 Clause 17.7.2.4.1a and 17.7.2.4.1b

Calculate edge effect factor

$c_{a2,g2} = 2$ in - Anchor group distance to failure edge (+Vz shear)

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

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{c_{a2,g2}}{1.5c'_{a1,g2}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{2\text{ in}}{1.5 \times 6.6667\text{ in}}\right)\right) = 0.76$$

ACI 318-19 Eq. 17.7.2.6.1

Calculate thickness factor

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5c'_{a1,g2}}{t_{conc}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 6.6667\text{ in}}{10\text{ in}}},\ 1\right) = 1$$

SkyCiv

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)

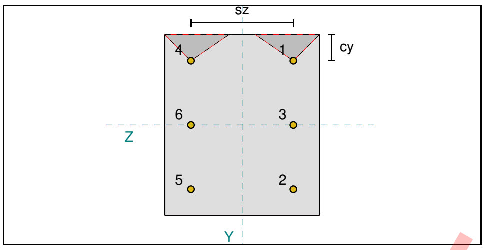
ACI 318-19 Clause 17.7.2.1(b)

$\phi V_{cbg\perp}$ - Design concrete breakout strength in shear of an anchor group (perpendicular edge)

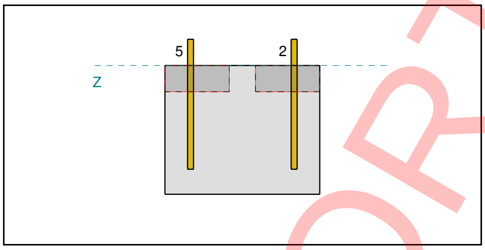
$$\phi V_{cbg\perp} = \phi \left(\frac{A_{Vc}}{A_{Vco}} \right) \Psi_{ec,V} \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_b$$

$$\phi V_{cbg\perp} = 0.65 \times \left(\frac{140 \text{ in}^2}{200 \text{ in}^2} \right) \times 1 \times 0.76 \times 1 \times 1 \times 7.0733 \text{ kip} = 2.446 \text{ kip}$$

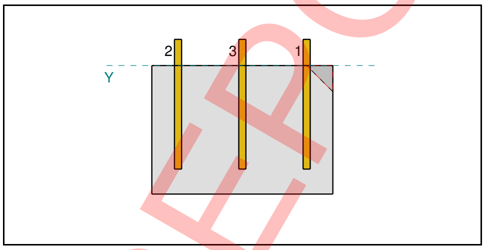
For breakout capacity on failure edge parallel to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate shear load per single anchor

$V_z = 2$ kip - Vz shear load

$n_a = 6$ - 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)	10.000 in
$c_{right,s1}$	Single anchor concrete edge distance (right)	2.000 in
$c_{top,s1}$	Single anchor concrete edge distance (top)	2.000 in
$c_{bottom,s1}$	Single anchor concrete edge distance (bottom)	12.000 in
t_{conc}	Concrete thickness	10.000 in
$c_{a1,s1}$	Single anchor distance to failure edge (+Vy shear)	2.000 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

$c_{a1,s1} = 2$ 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

$$A_{Vco} = 4.5(c_{a1,s1})^2 = 4.5 \times (2 \text{ in})^2 = 18 \text{ in}^2$$

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

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

$$B_{Vc} = \min(10 \text{ in},\ 1.5 \times 2 \text{ in}) + \min(2 \text{ in},\ 1.5 \times 2 \text{ in}) = 5 \text{ in}$$

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

$$H_{Vc} = \min\left(1.5\mathrm{c_{a1,s1}},\ \mathrm{t_{conc}}\right) = \min\left(1.5 \times 2\ \mathrm{in},\ 10\ \mathrm{in}\right) = 3\ \mathrm{in}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 5\ \mathrm{in} \times 3\ \mathrm{in} = 15\ \mathrm{in}^2$$

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

$h_{ef} = 8\ \mathrm{in}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 8\ \mathrm{in}$$

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

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

$$V_{b1} = 7 \times \left(\frac{\min\left(8\ \mathrm{in},\ 8 \times 0.5\ \mathrm{in}\right)}{0.5\ \mathrm{in}}\right)^{0.2} \times \sqrt{\frac{0.5\ \mathrm{in}}{1\ \mathrm{in}}} \times 1 \times \sqrt{\frac{3\ \mathrm{ksi}}{0.001\ \mathrm{ksi}}} \times \left(\frac{2\ \mathrm{in}}{1\ \mathrm{in}}\right)^{1.5} \times 0.001\ \mathrm{kip}$$

$$V_{b1} = 1.1623\ \mathrm{kip}$$

ACI 318-19 Eq. 17.7.2.2.1b

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

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

$$V_{b2} = 9 \times 1 \times \sqrt{\frac{3\ \mathrm{ksi}}{0.001\ \mathrm{ksi}}} \times \left(\frac{2\ \mathrm{in}}{1\ \mathrm{in}}\right)^{1.5} \times 0.001\ \mathrm{kip} = 1.3943\ \mathrm{kip}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(\mathrm{V_{b1}},\ \mathrm{V_{b2}}\right) = \min\left(1.1623\ \mathrm{kip},\ 1.3943\ \mathrm{kip}\right) = 1.1623\ \mathrm{kip}$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,s1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 2\ \mathrm{in}}{10\ \mathrm{in}}},\ 1\right) = 1$$

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)

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

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

$$\phi V_{cb||} = 2 \times 0.65 \times \left(\frac{15\ \mathrm{in}^2}{18\ \mathrm{in}^2}\right) \times 1 \times 1 \times 1 \times 1.1623\ \mathrm{kip} = 1.2591\ \mathrm{kip}$$

Result:

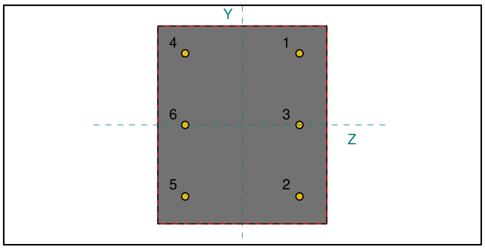
PASS = 0.82

DCR - Demand over capacity ratio, comparing two conditions:

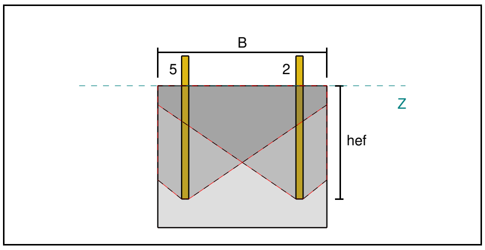
$$DCR = \max\left(\frac{\mathrm{V_{fa\perp,case2}}}{\phi\mathrm{V_{cbg\perp}}},\ \frac{\mathrm{V_{fa||}}}{\phi\mathrm{V_{cb||}}}\right)$$

$$DCR = \max\left(\frac{2\ \mathrm{kip}}{2.446\ \mathrm{kip}},\ \frac{0.33333\ \mathrm{kip}}{1.2591\ \mathrm{kip}}\right) = 0.81767$$

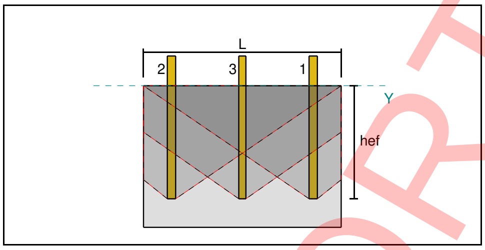
Check No. 4: Concrete Pryout Capacity



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group

$V_y = 2\ \text{kip}$ - Vy shear load

$V_z = 2\ \text{kip}$ - Vz shear load

V_{ua} - Resultant shear load

$$V_{ua} = \sqrt{\left((V_y)^2\right) + \left((V_z)^2\right)} = \sqrt{\left((2\ \text{kip})^2\right) + \left((2\ \text{kip})^2\right)} = 2.8284\ \text{kip}$$

$n_a = 6$ - Total number of anchor rods

$n_{a,g1} = 6$ - Number of anchors in anchor group PO 1

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

$$V_{ua} = \left(\frac{V_{ua}}{n_a}\right) n_{a,g1} = \left(\frac{2.8284\ \text{kip}}{6}\right) \times 6 = 2.8284\ \text{kip}$$

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)	2.000 in
$c_{right,g1}$	Anchor group PO 1 concrete edge distance (right)	2.000 in
$c_{top,g1}$	Anchor group PO 1 concrete edge distance (top)	2.000 in
$c_{bottom,g1}$	Anchor group PO 1 concrete edge distance (bottom)	2.000 in
h_{ef}	Anchor rod effective embedment length	8.000 in
$s_{sum,y,g1}$	Anchor group PO 1 sum of spacing along Y-axis	10.000 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	3.000
$n_{z,g1}$	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

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

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

$$L_{Nc} = 12\ \text{in}$$

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

$s_{sum,y,g1} = 10\ \text{in}$ - Anchor group PO 1 sum of spacing along Y-axis

$n_{y,g1} = 3$ - 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

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

$$B_{Nc} = \min\left(2\ \text{in},\ 1.5 \times 2.6667\ \text{in}\right) + \left(\min\left(10\ \text{in},\ 3 \times 2.6667\ \text{in} \times (3-1)\right)\right) + \min\left(2\ \text{in},\ 1.5 \times 2.6667\ \text{in}\right)$$

$$B_{Nc} = 14\ \text{in}$$

Calculate actual projected area of the anchor group

$n_{a,g1} = 6$ - Number of anchors in anchor group PO 1

ACI 318-19 Clause 17.6.2

A_{Nc} - Actual projected area for an anchor group

$$A_{Nc} = \min\left(\mathrm{n_{a,g1}A_{Nco}},\ \mathrm{L_{Nc}B_{Nc}}\right) = \min\left(6 \times 64\ \text{in}^2,\ 12\ \text{in} \times 14\ \text{in}\right) = 168\ \text{in}^2$$

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

$f_c^{'} = 3\ \text{ksi}$ - 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

$$c_{a,min} = \min\left(\mathrm{c_{left,g1}},\ \mathrm{c_{right,g1}},\ \mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min\left(2\ \text{in},\ 2\ \text{in},\ 2\ \text{in},\ 2\ \text{in}\right) = 2\ \text{in}$$

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

$$\Psi_{ed,N} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a,min}}}{1.5\mathrm{h'_{ef,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{2\ \text{in}}{1.5 \times 2.6667\ \text{in}}\right)\right) = 0.85$$

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

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

$$N_{cbg} = \left(\frac{168\ \text{in}^2}{64\ \text{in}^2}\right) \times 1 \times 0.85 \times 1 \times 1 \times 5.7243\ \text{kip} = 12.772\ \text{kip}$$

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

$$\phi V_{cpg} = \phi k_{cp} N_{cbg} = 0.65 \times 2 \times 12.772\ \text{kip} = 16.604\ \text{kip}$$

Result: **PASS = 0.17**

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{V_{ua}}{\phi V_{cpg}} = \frac{2.8284 \text{ kip}}{16.604 \text{ kip}} = 0.17034$$

Check No. 5: Anchor Rod Shear Capacity

Calculate portion of Vy shear load on most critical anchor

$V_y = 2 \text{ kip}$ - Vy shear load

$n_a = 6$ - Total number of anchor rods

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

$$v_{ua,y} = \frac{V_y}{n_a} = \frac{2 \text{ kip}}{6} = 0.33333 \text{ kip}$$

Calculate portion of Vz shear load on most critical anchor

Governing case from concrete breakout calculation for Vz shear: **Case 2 (perpendicular)**. Therefore, check the perpendicular anchors for a portion of Vz load

$V_z = 2 \text{ kip}$ - Vz shear load

$n_a = 6$ - Total number of anchor rods

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

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

$$v_{ua,z} = \left(\frac{V_z}{n_a}\right) n_{z,g1} = \left(\frac{2 \text{ kip}}{6}\right) \times 1 = 0.33333 \text{ kip}$$

Calculate resultant load on most critical anchor

V_{ua} - Required shear strength on anchor

$$V_{ua} = \sqrt{\left((v_{ua,y})^2\right) + \left((v_{ua,z})^2\right)}$$

$$V_{ua} = \sqrt{\left((0.33333 \text{ kip})^2\right) + \left((0.33333 \text{ kip})^2\right)} = 0.4714 \text{ kip}$$

Calculate eccentricity of shear load

$t_{bp} = 0.75 \text{ in}$ - Base plate thickness

$t_{pw} = 0.25 \text{ in}$ - Plate washer thickness

AISC Design Guide 1 3rd Ed. Section 4.3.3

e - Eccentricity of shear load due to plate washer

$$e = 0.5\left(\frac{t_{pw}}{2} + t_{bp}\right) = 0.5 \times \left(\frac{0.25 \text{ in}}{2} + 0.75 \text{ in}\right) = 0.4375 \text{ in}$$

Calculate shear stress on anchor

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

f_v - Shear stress on anchor

$$f_v = \frac{V_{ua}}{A_{rod}} = \frac{0.4714 \text{ kip}}{0.19635 \text{ in}^2} = 2.4008 \text{ ksi}$$

Calculate section modulus of the anchor rod

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

Z_{rod} - Section modulus of anchor rod

$$Z_{rod} = \frac{\pi}{32}(d_a)^3 = \frac{\pi}{32} \times (0.5 \text{ in})^3 = 0.012272 \text{ in}^3$$

Calculate tensile stress on anchor due to lever arm

f_t - Tensile stress on anchor due to shear load with lever arm

$$f_t = \frac{V_{ua} e}{Z_{rod}} = \frac{0.4714 \text{ kip} \times 0.4375 \text{ in}}{0.012272 \text{ in}^3} = 16.806 \text{ ksi}$$

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 area of the anchor rod $d_a = 0.5\ \text{in}$ - Anchor rod diameter A_{rod} - Nominal area of anchor rod $$A_{rod} = \frac{\pi}{4}(d_a)^2 = \frac{\pi}{4} \times (0.5\ \text{in})^2 = 0.19635\ \text{in}^2$$	
	Calculate nominal shear stress of the anchor rod $F_{u\_anc} = 120\ \text{ksi}$ - Anchor rod tensile stress (A325)	
AISC 360-22 Table J3.2	F_{nv} - Nominal shear stress of the anchor rod for threads not excluded on shear plane (N) $$F_{nv} = 0.45 F_{u\_anc} = 0.45 \times 120\ \text{ksi} = 54\ \text{ksi}$$	
	Calculate nominal tensile stress of the anchor rod	
AISC 360-22 Table J3.2	F_{nt} - Nominal tensile stress of anchor rod $$F_{nt} = 0.75 F_{u\_anc} = 0.75 \times 120\ \text{ksi} = 90\ \text{ksi}$$	
	Calculate nominal modified shear stress of the anchor rod to include effects of tensile stress $f_t = 16.806\ \text{ksi}$ $\phi = 0.75$ - Anchor steel resistance factor (AISC)	
AISC 360-22 Eq. J3-3a (rewritten per Sect J3.8 note)	F'_{nv} - Nominal shear stress modified to include the effects of tensile stress $$F'_{nv} = \min\left(1.3\mathrm{F_{nv}} - \left(\frac{\mathrm{F_{nv}}}{\phi \mathrm{F_{nt}}}\right)\mathrm{f_t},\ \mathrm{F_{nv}}\right)$$ $$F'_{nv} = \min\left(1.3 \times 54\ \text{ksi} - \left(\frac{54\ \text{ksi}}{0.75 \times 90\ \text{ksi}}\right) \times 16.806\ \text{ksi},\ 54\ \text{ksi}\right) = 54\ \text{ksi}$$	
	Calculate design shear and tensile strength of the anchor rod per AISC provisions	
AISC 360-22 Eq. J3-2	$\phi R_{n,aisc}$ - Design anchor rod combined tensile and shear strength per AISC $$\phi R_{n,aisc} = \phi F'_{nv} A_{rod} = 0.75 \times 54\ \text{ksi} \times 0.19635\ \text{in}^2 = 7.9522\ \text{kip}$$	
	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 $$\phi V_n = \min\left(\phi \mathrm{V_{sa,aci}},\ \phi \mathrm{R_{n,aisc}}\right) = \min\left(3.9845\ \text{kip},\ 7.9522\ \text{kip}\right) = 3.9845\ \text{kip}$$	
	Result: DCR - Demand-to-Capacity Ratio $$DCR = \frac{V_{ua}}{\phi V_n} = \frac{0.4714\ \text{kip}}{3.9845\ \text{kip}} = 0.11831$$	**PASS = 0.12**

REFERENCES	CALCULATIONS	RESULTS
	Summary of Design Checks	The governing combination is Load Combination 1.

Load Combination	Design Check	Demand	Capacity	DCR	Result
1	Weld Capacity	0.16	5.58	0.03	**PASS**
1	Concrete Breakout Capacity (Vy Shear)	0.33	0.57	0.59	**PASS**
1	Concrete Breakout Capacity (Vz Shear)	2.00	2.45	0.82	**PASS**
1	Concrete Pryout Capacity	2.83	16.60	0.17	**PASS**
1	Anchor Rod Shear Capacity	0.47	3.98	0.12	**PASS**

The design is adequate!