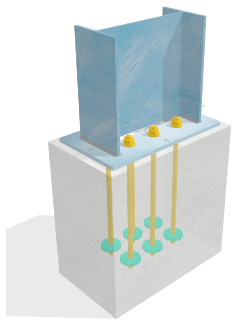
REFERENCES	CALCULATIONS	RESULTS

SkyCiv Base Plate and Anchor Rod Design

Shear and Tension Design Example for AISC

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

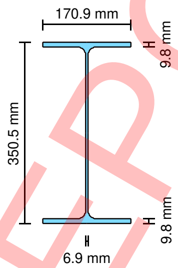


Load Combinations:

Load Combination	Axial Load, Nx (kN)	Strong-axis Shear, Vy (kN)	Weak-axis Shear, Vz (kN)
1	-30.00	3.00	6.00

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

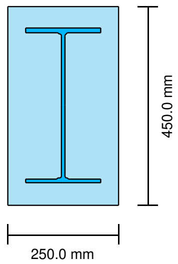
Column Properties:



Symbol	Description	Value
$Section$	Column section	W14x30
d_{col}	Column depth	350.5 mm
t_w	Column web thickness	6.9 mm
b_f	Column flange width	170.9 mm
t_f	Column flange thickness	9.8 mm
A_{col}	Column area	5709.7 mm^2
r_{col}	Column root radius	10.2 mm
F_{y_col}	Column yield stress (A992)	344.7 MPa
F_{u_col}	Column tensile stress (A992)	448.2 MPa

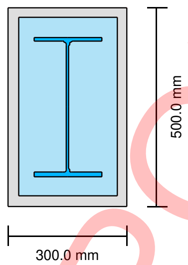
REFERENCES	CALCULATIONS	RESULTS

Base Plate Properties:



Symbol	Description	Value
B_{bp}	Base plate width	250.0 mm
L_{bp}	Base plate length	450.0 mm
t_{bp}	Base plate thickness	12.0 mm
A_{bp}	Base plate plan area	112500.0 mm^2
F_{y_bp}	Base plate yield stress (A36)	248.2 MPa
F_{u_bp}	Base plate tensile stress (A36)	399.9 MPa
t_{grout}	Grout thickness	0.0 mm

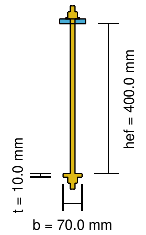
Concrete Properties:



Symbol	Description	Value
B_{conc}	Concrete width	300.0 mm
L_{conc}	Concrete length	500.0 mm
t_{conc}	Concrete thickness	500.0 mm
A_{conc}	Concrete plan area	150000.0 mm^2
f_c'	Concrete compressive strength (20.68)	20.7 MPa
λ	Factor for normal-weight concrete	1.0
—	Concrete assumption: cracked or uncracked	Cracked

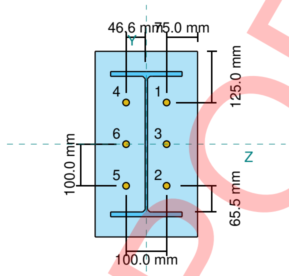
REFERENCES	CALCULATIONS	RESULTS

Anchor Information:



Symbol	Description	Value
d_a	Anchor rod diameter	16.0 mm
h_{ef}	Anchor rod effective embedment length	400.0 mm
b_{embed_plate}	Anchor embed plate diameter	70.0 mm
t_{embed_plate}	Anchor embed plate thickness	10.0 mm
F_{y_anc}	Anchor rod yield stress (F1554 Gr.55)	379.2 MPa
F_{u_anc}	Anchor rod tensile stress (F1554 Gr.55)	517.1 MPa
F_{y_ep}	Embed plate yield stress (A36)	248.2 MPa
t_{pw}	Plate washer thickness	0.0 mm

Anchor Layout:



Symbol	Description	Value
n_a	Total number of anchor rods	6.0
s_z	Spacing of anchor rods along Z-axis	100.0 mm
s_y	Spacing of anchor rods along Y-axis	100.0 mm
$l_{edge,z}$	Base plate edge distance along Z-axis	75.0 mm
$l_{edge,y}$	Base plate edge distance along Y-axis	125.0 mm

REFERENCES	CALCULATIONS	RESULTS

Anchor Data Summary:

ID	Z (mm)	Y (mm)
1	50.000	100.000
2	50.000	-100.000
3	50.000	0.000
4	-50.000	100.000
5	-50.000	-100.000
6	-50.000	0.000

ID	Anchor in Tension Zone	Breakout (Tension)	Side-Face Blowout (Y-direction)	Side-Face Blowout (Z-direction)
1	✓	BO Group 1	SFy Group 2	SFz Group 1
2	✓	BO Group 1	SFy Group 2	SFz Group 2
3	✓	BO Group 1	SFy Group 2	-
4	✓	BO Group 1	SFy Group 1	SFz Group 1
5	✓	BO Group 1	SFy Group 1	SFz Group 2
6	✓	BO Group 1	SFy Group 1	-

ID	Shear Direction Resisted	Breakout (Vy Shear)	Breakout (Vz Shear)	Pryout (Vy Shear)	Pryout (Vz Shear)
1	+Vz +Vy	BO Vy Group 1	BO Vz Group 1	PO Vy Group 1	PO Vz Group 1
2	+Vz -Vy	BO Vy Group 2	BO Vz Group 1	PO Vy Group 2	PO Vz Group 1
3	+Vz	-	BO Vz Group 1	-	PO Vz Group 1
4	-Vz +Vy	BO Vy Group 1	BO Vz Group 2	PO Vy Group 1	PO Vz Group 2
5	-Vz -Vy	BO Vy Group 2	BO Vz Group 2	PO Vy Group 2	PO Vz Group 2
6	-Vz	-	BO Vz Group 2	-	PO Vz Group 2

Weld Properties:

Symbol	Description	Value
$Type$	Weld Type	Fillet
W_{top_flange}	Fillet weld size at Top Flange	7.000 mm
W_{bottom_flange}	Fillet weld size at Bottom Flange	7.000 mm
W_{web}	Fillet weld size at Web	7.000 mm
$E_{w\_top\_flange}$	Weld throat at Top Flange	4.950 mm
$E_{w\_bottom\_flange}$	Weld throat at Bottom Flange	4.950 mm
E_{w_web}	Weld throat at Web	4.950 mm
F_{exx}	Filler metal classification strength (E70xx)	480.000 MPa

REFERENCES	CALCULATIONS	RESULTS

Summary of Detailing Checks

Check Name	Dimensions (Min/Max/Actual)	Status	Reference
Weld Size Requirement (Flange)	Min: 5.00 mm, Max: 19.00 mm, Actual: 7.00 mm	PASS	AISC 360-22 Table J2.4
Weld Size Requirement (Web)	Min: 5.00 mm, Max: 19.00 mm, Actual: 7.00 mm	PASS	AISC 360-22 Table J2.4
Anchor Clearance	Min: 30.16 mm, Actual: 39.57 mm	PASS	SkyCiv Recommendation
Base Plate Edge Distance	Min: 24.00 mm, Actual: 75.00 mm	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
Embedded Plate Dimension	Min: 64.00 mm, Actual: 70.00 mm	PASS	SkyCiv Recommendation
Min. Anchor Spacing to Prevent Concrete Splitting	Min: 64.00 mm, Actual: 100.00 mm	PASS	ACI 318-19 Table 17.9.2(a)
Min. Concrete Cover to Prevent Concrete Splitting	Min: 40.00 mm, Actual: 100.00 mm	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	0.05	0.69	0.08	PASS
Base Plate Yield Capacity (Tension Load)	1	232.86	749.02	0.31	PASS
Anchor Rod Tensile Capacity	1	5.00	43.19	0.12	PASS
Concrete Breakout Capacity (Tension Load)	1	30.00	47.75	0.63	PASS
Anchor Pullout Capacity	1	5.00	422.40	0.01	PASS
Embed Plate Flexural Capacity	1	2.99	10.00	0.30	PASS
Concrete Side-Face Blowout Capacity (Y-Direction)	1	15.00	239.51	0.06	PASS
Concrete Side-Face Blowout Capacity (Z-Direction)	1	10.00	197.86	0.05	PASS
Concrete Breakout Capacity (Vy Shear)	1	3.00	10.97	0.27	PASS
Concrete Breakout Capacity (Vz Shear)	1	6.00	17.92	0.33	PASS
Concrete Pryout Capacity (Vy Shear)	1	3.00	50.68	0.06	PASS
Concrete Pryout Capacity (Vz Shear)	1	6.00	72.53	0.08	PASS
Anchor Rod Shear Capacity	1	2.50	22.46	0.11	PASS
Anchor Rod Capacity (AISC Tension and Shear)	1	2.50	35.09	0.07	PASS
Interaction Check (ACI)	1	0.96	1.20	0.80	PASS

✔ All Design Checks Passed

The design is adequate.

REFERENCES	CALCULATIONS	RESULTS

Load Combination No. 1: Tension Load and Shear Load

Check No. 1: Weld Capacity

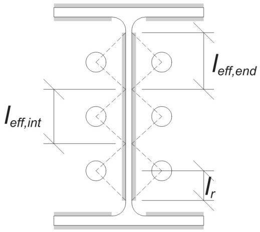


Fig. Effective welds on column to base plate.

Calculate effective weld length on column web

$d_{col} = 350.52\ \mathrm{mm}$ - Column depth

$t_f = 9.779\ \mathrm{mm}$ - Column flange thickness

$r_{col} = 10.16\ \mathrm{mm}$ - Column root radius

$s_y = 100\ \mathrm{mm}$ - Spacing of anchor rods along Y-axis

$n_{a,side} = 3$ - Number of anchor rods in tension per side

l_r - Remaining weld length near weld tip

$$l_r = \frac{d_{col} - 2t_f - 2r_{col} - s_y\left(n_{a,side} - 1\right)}{2} = \frac{350.52\ \mathrm{mm} - 2 \times 9.779\ \mathrm{mm} - 2 \times 10.16\ \mathrm{mm} - 100\ \mathrm{mm} \times (3 - 1)}{2} = 55.321\ \mathrm{mm}$$

$d_o = 46.571\ \mathrm{mm}$ - Web offset distance

$s_y = 100\ \mathrm{mm}$ - Spacing of anchor rods along Y-axis

$l_{eff,end}$ - Effective weld length on end anchors

$$l_{eff,end} = \min\left(\mathrm{d_o},\ 0.5\mathrm{s_y}\right) + \min\left(\mathrm{d_o},\ \mathrm{l_r}\right)$$

$$l_{eff,end} = \min\left(46.571\ \mathrm{mm},\ 0.5 \times 100\ \mathrm{mm}\right) + \min\left(46.571\ \mathrm{mm},\ 55.321\ \mathrm{mm}\right) = 93.142\ \mathrm{mm}$$

$l_{eff,int}$ - Effective weld length on intermediate anchors

$$l_{eff,int} = \min\left(2\mathrm{d_o},\ \mathrm{s_y}\right) = \min\left(2 \times 46.571\ \mathrm{mm},\ 100\ \mathrm{mm}\right) = 93.142\ \mathrm{mm}$$

l_{eff} - Design effective weld length

$$l_{eff} = \min\left(\mathrm{l_{eff,end}},\ \mathrm{l_{eff,int}}\right) = \min\left(93.142\ \mathrm{mm},\ 93.142\ \mathrm{mm}\right) = 93.142\ \mathrm{mm}$$

Calculate load per unit length of weld

$N_x = 30\ \mathrm{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

$l_{eff} = 93.142\ \mathrm{mm}$

$T_{u,anchor}$ - Tension load per anchor

$$T_{u,anchor} = \frac{N_x}{n_{a,t}} = \frac{30\ \mathrm{kN}}{6} = 5\ \mathrm{kN}$$

$r_{u,web}$ - Required weld strength per unit length

$$r_{u,web} = \frac{T_{u,anchor}}{l_{eff}} = \frac{5\ \mathrm{kN}}{93.142\ \mathrm{mm}} = 0.053681\ \mathrm{kN/mm}$$

Calculate total weld length on column flange and web

$d_{col} = 350.52\ \mathrm{mm}$ - Column depth

$b_f = 170.94\ \mathrm{mm}$ - Column flange width

$t_f = 9.779\ \mathrm{mm}$ - Column flange thickness

$t_w = 6.858\ \mathrm{mm}$ - Column web thickness

$r_{col} = 10.16\ \mathrm{mm}$ - Column root radius

L_{weld} - Total length of weld

$$L_{weld} = 2b_f + 2\left(d_{col} - 2t_f - 2r_{col}\right) + 2\left(b_f - t_w - 2r_{col}\right)$$

$$L_{weld} = 2 \times 170.94\ \mathrm{mm} + 2 \times \left(350.52\ \mathrm{mm} - 2 \times 9.779\ \mathrm{mm} - 2 \times 10.16\ \mathrm{mm}\right) + 2 \times \left(170.94\ \mathrm{mm} - 6.858\ \mathrm{mm} - 2 \times 10.16\ \mathrm{mm}\right)$$

$$L_{weld} = 1250.7\ \mathrm{mm}$$

Calculate shear load per unit length of weld

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

$V_z = 6\ \mathrm{kN}$ - Vz shear load

v_{uy} - Required weld strength per unit length

$$v_{uy} = \frac{V_y}{L_{weld}} = \frac{3\ \mathrm{kN}}{1250.7\ \mathrm{mm}} = 0.0023987\ \mathrm{kN/mm}$$

v_{uz} - Required weld strength per unit length

$$v_{uz} = \frac{V_z}{L_{weld}} = \frac{6\text{ kN}}{1250.7\text{ mm}} = 0.0047973\text{ kN/mm}$$

Calculate resultant loads per unit length of weld at flanges

r_u - Required weld strength per unit length

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

$$r_u = \sqrt{\left((0.0023987\text{ kN/mm})^2\right) + \left((0.0047973\text{ kN/mm})^2\right)} = 0.0053636\text{ kN/mm}$$

Calculate resultant loads per unit length of weld at web

r_u - Required weld strength per unit length

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

$$r_u = \sqrt{\left((0.053681\text{ kN/mm})^2\right) + \left((0.0023987\text{ kN/mm})^2\right) + \left((0.0047973\text{ kN/mm})^2\right)}$$

$$r_u = 0.053949\text{ kN/mm}$$

Calculate fillet weld capacity on flange

$F_{exx} = 480\text{ MPa}$ - Filler metal classification strength (E70xx)

$E_w = 4.95\text{ mm}$ - Design weld throat

$\phi = 0.75$ - Weld resistance factor

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

θ - Load angle from weld axis

$$\theta = \tan^{-1}\left(\frac{\text{v}_\text{uy}}{\text{v}_\text{uz}}\right) = \tan^{-1}\left(\frac{0.0023987\text{ kN/mm}}{0.0047973\text{ kN/mm}}\right) = 0.46365\text{ rad}$$

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

k_{ds} - Directional strength increase factor

$$k_{ds} = 1.0 + 0.5(\sin(\theta))^{1.5} = 1 + 0.5 \times (\sin(0.46365\text{ rad}))^{1.5} = 1.1495$$

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

$\phi r_{n,flg}$ - Design strength of fillet welds per unit length

$$\phi r_{n,flg} = \phi 0.6 F_{exx} E_w k_{ds} = 0.75 \times 0.6 \times 480\text{ MPa} \times 4.95\text{ mm} \times 1.1495 = 1.2291\text{ kN/mm}$$

Calculate fillet weld capacity on web

$F_{exx} = 480\text{ MPa}$ - Filler metal classification strength (E70xx)

$E_w = 4.95\text{ mm}$ - Design weld throat

$\phi = 0.75$ - Weld resistance factor

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

θ - Angle of resultant load with respect to the weld axis

$$\theta = \cos^{-1}\left(\frac{\text{v}_\text{uy}}{\text{r}_\text{u}}\right) = \cos^{-1}\left(\frac{0.0023987\text{ kN/mm}}{0.053949\text{ kN/mm}}\right) = 1.5263\text{ rad}$$

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

k_{ds} - Directional strength increase factor

$$k_{ds} = 1.0 + 0.5(\sin(\theta))^{1.5} = 1 + 0.5 \times (\sin(1.5263\text{ rad}))^{1.5} = 1.4993$$

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

$\phi r_{n,web}$ - Design strength of fillet welds per unit length

$$\phi r_{n,web} = \phi 0.6 F_{exx} E_w k_{ds} = 0.75 \times 0.6 \times 480\text{ MPa} \times 4.95\text{ mm} \times 1.4993 = 1.603\text{ kN/mm}$$

Calculate base metal capacities

$F_{u\_col} = 448.2\text{ MPa}$ - Column tensile stress (A992)

$t_{col,half} = 3.429\text{ mm}$ - Half of column thickness (minimum of flange and web)

$F_{u\_bp} = 400\text{ MPa}$ - Base plate tensile stress (A36)

$t_{bp} = 12\text{ mm}$ - 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)

$$\phi r_{nbm,col} = \phi 0.6 F_{u\_col} t_{col,half} = 0.75 \times 0.6 \times 448.2\text{ MPa} \times 3.429\text{ mm} = 0.6916\text{ kN/mm}$$

AISC 360-22 Eq. J4-4

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

$$\phi r_{nbm,bp} = \phi 0.6 F_{u\_bp} t_{bp} = 0.75 \times 0.6 \times 400\text{ MPa} \times 12\text{ mm} = 2.1595\text{ kN/mm}$$

AISC 360-22 Eq. J4-4

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

$$\phi r_{nbm} = \min\left(\phi \text{r}_\text{nbm,bp},\ \phi \text{r}_\text{nbm,col}\right) = \min\left(2.1595\text{ kN/mm},\ 0.6916\text{ kN/mm}\right) = 0.6916\text{ kN/mm}$$

Result:

PASS = 0.08

DCR - Demand over capacity ratio, comparing four conditions:

$$DCR = \max\left(\frac{\text{r}_\text{u}}{\phi \text{r}_\text{nbm}}, \frac{\text{r}_\text{u}}{\phi \text{r}_\text{nbm}}, \frac{\text{r}_\text{u}}{\phi \text{r}_\text{n,web}}, \frac{\text{r}_\text{u}}{\phi \text{r}_\text{n,flg}}\right)$$

$$DCR = \max\left(\frac{0.053949\text{ kN/mm}}{0.6916\text{ kN/mm}}, \frac{0.0053636\text{ kN/mm}}{0.6916\text{ kN/mm}}, \frac{0.053949\text{ kN/mm}}{1.603\text{ kN/mm}}, \frac{0.0053636\text{ kN/mm}}{1.2291\text{ kN/mm}}\right)$$

$$DCR = 0.078006$$

Go back to summary results table.

Check No. 2: Base Plate Yield Capacity (Tension Load)

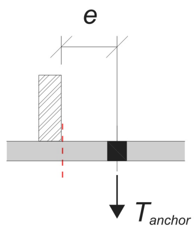


Fig. Bending of base plate from eccentric anchor load.

Calculate applied moment load on effective width of base plate

$T_{u,anchor} = 5\ \text{kN}$

$e = 46.571\ \text{mm}$ - Tensile load eccentricity equal to offset distance

AISC Design Guide 01 3rd Ed. Section 4.3.2

M_u - Required flexural yielding strength of base plate

$$M_u = T_{u,anchor} e = 5\ \text{kN} \times 46.571\ \text{mm} = 232.86\ \text{kN·mm}$$

Calculate flexural yielding capacity of base plate

$l_{eff} = 93.142\ \text{mm}$

$t_{bp} = 12\ \text{mm}$ - Base plate thickness

AISC 360-22 Chapter F2

Z_{eff} - Plastic modulus of the effective section

$$Z_{eff} = \frac{l_{eff}(t_{bp})^2}{4} = \frac{93.142\ \text{mm} \times (12\ \text{mm})^2}{4} = 3353.1\ \text{mm}^3$$

$F_{y\_bp} = 248.2\ \text{MPa}$ - Base plate yield stress (A36)

$\phi = 0.9$ - Flexural resistance factor

AISC 360-22 Eq. 2-1

ϕM_n - Design flexural yielding strength of base plate

$$\phi M_n = \phi F_{y\_bp} Z_{eff} = 0.9 \times 248.2\ \text{MPa} \times 3353.1\ \text{mm}^3 = 749.02\ \text{kN·mm}$$

Result:

PASS = 0.31

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{M_u}{\phi M_n} = \frac{232.86\ \text{kN·mm}}{749.02\ \text{kN·mm}} = 0.31088$$

Go back to summary results table.

Check No. 3: Anchor Rod Tensile Capacity

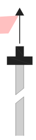


Fig. Anchor steel tension failure.

Calculate load per tension anchor

$N_x = 30\ \text{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

N_{ua} - Required tensile strength of anchor

$$N_{ua} = \frac{N_x}{n_{a,t}} = \frac{30\ \text{kN}}{6} = 5\ \text{kN}$$

Calculate specified tensile strength of anchor

$F_{u\_anc} = 517.1\ \text{MPa}$ - Anchor rod tensile stress (F1554 Gr.55)

$F_{y\_anc} = 379.2\ \text{MPa}$ - Anchor rod yield stress (F1554 Gr.55)

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

f_{uta} - Specified tensile strength of anchor steel

$$f_{uta} = \min\left(0.75\text{F}_{\text{u\_anc}},\ 1.9\text{F}_{\text{y\_anc}},\ 860\right) = \min\left(0.75 \times 517.1\ \text{MPa},\ 1.9 \times 379.2\ \text{MPa},\ 860.00\ \text{MPa}\right) = 387.82\ \text{MPa}$$

Calculate effective cross-sectional area of anchor

$d_a = 16\ \text{mm}$ - Anchor rod diameter

ASME B1.1-2019 Table 1

$n_t = 0.433\ \text{mm}^{-1}$ - 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

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

$$A_{se,N} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2 = \frac{\pi}{4} \times \left(16\ \text{mm} - \frac{0.9743}{0.433\ \text{mm}^{-1}}\right)^2 = 148.49\ \text{mm}^2$$

Calculate anchor steel tensile capacity

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

ACI 318-19 Eq. 17.6.1.2

ϕN_{sa} - Design anchor rod steel strength

$$\phi N_{sa} = \phi A_{se,N} f_{uta} = 0.75 \times 148.49\ \mathrm{mm}^2 \times 387.82\ \mathrm{MPa} = 43.19\ \mathrm{kN}$$

Result:

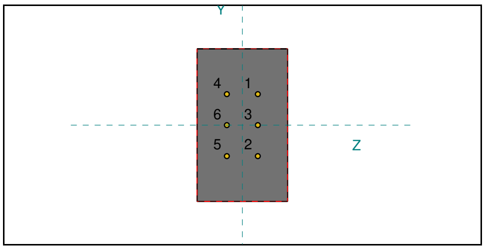
DCR - Demand-to-Capacity Ratio

$$DCR = \frac{N_{ua}}{\phi N_{sa}} = \frac{5\ \mathrm{kN}}{43.19\ \mathrm{kN}} = 0.11577$$

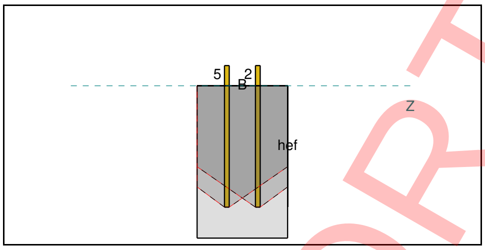
PASS = 0.12

Go back to summary results table.

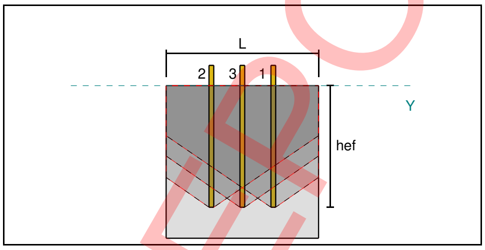
Check No. 4: Concrete Breakout Capacity (Tension Load)



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total tension load on anchor group

$N_x = 30\ \mathrm{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

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

N_{ua} - Required concrete breakout tensile strength of an anchor group

$$N_{ua} = \left(\frac{N_x}{n_{a,t}}\right) n_{a,g1} = \left(\frac{30\ \mathrm{kN}}{6}\right) \times 6 = 30\ \mathrm{kN}$$

Determine if the support is a narrow concrete member

ACI 318-19 Clause 17.6.2.1.2

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 BO 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group BO 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group BO 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group BO 1 concrete edge distance (bottom)	150.0 mm
h_{ef}	Anchor rod effective embedment length	400.0 mm
	Narrow Member	TRUE
$h'_{ef,g1}$	Anchor group BO 1 modified effective embedment length	100.0 mm

Calculation for critical anchor group: Anchor Group BO 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.6.2.1.2

$h'_{ef,g1} = 100\ \mathrm{mm}$ - Anchor group BO 1 modified effective embedment length

ACI 318-19 Clause 17.6.2

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

$$A_{Nco} = 9\left(h'_{ef,g1}\right)^2 = 9 \times (100\ \mathrm{mm})^2 = 90000\ \mathrm{mm}^2$$

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

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

SkyCiv

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

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

$$L_{Nc} = \min(100\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm}) + (\min(100\ \mathrm{mm},\ 3 \times 100\ \mathrm{mm} \times (2-1))) + \min(100\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm})$$

$$L_{Nc} = 300\ \mathrm{mm}$$

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

$n_{y,g1} = 3$ - Number of anchors along Y-axis for anchor group BO 1

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

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

$$B_{Nc} = \min(150\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm}) + (\min(200\ \mathrm{mm},\ 3 \times 100\ \mathrm{mm} \times (3-1))) + \min(150\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm})$$

$$B_{Nc} = 500\ \mathrm{mm}$$

Calculate actual projected area of the anchor group

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

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

$$A_{Nc} = \min\left(\mathrm{n_{a,g1}A_{Nco}},\ \mathrm{L_{Nc}B_{Nc}}\right) = \min\left(6 \times 90000\ \mathrm{mm}^2,\ 300\ \mathrm{mm} \times 500\ \mathrm{mm}\right) = 150000\ \mathrm{mm}^2$$

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

$f_c' = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

ACI 318-19 Clause 17.6.2.2.1 $k_c = 10$ - Factor for cast-in anchors breakout strength

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

$$N_b = k_c \lambda_a \sqrt{\frac{f_c'}{MPa}} \left(\frac{h'_{ef,g1}}{mm}\right)^{1.5} N$$

$$N_b = 10 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{100\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 45.475\ \mathrm{kN}$$

Calculate minimum edge distance of anchor

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

$$c_{a,min} = \min\left(\mathrm{c_{left,g1}},\ \mathrm{c_{right,g1}},\ \mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min(100\ \mathrm{mm},\ 100\ \mathrm{mm},\ 150\ \mathrm{mm},\ 150\ \mathrm{mm}) = 100\ \mathrm{mm}$$

Calculate edge effect factor

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

$$\Psi_{ed,N} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a,min}}}{1.5\mathrm{h'_{ef,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{100\ \mathrm{mm}}{1.5 \times 100\ \mathrm{mm}}\right)\right) = 0.9$$

Calculate eccentricity factor

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

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

$$\Psi_{ec,N} = \min\left(1.0,\ \frac{1}{1 + \frac{2\mathrm{e'_N}}{3\mathrm{h'_{ef,g1}}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 100\ \mathrm{mm}}}\right) = 1$$

Calculate concrete breakout capacity of the anchor group

ACI 318-19 Clause 17.6.2.4.1 $\Psi_{ed,N} = 0.9$

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 Clause 17.6.2.4.1 $\Psi_{ec,N} = 1$

ACI 318-19 Eq. 17.6.2.1b ϕN_{cbg} - Design concrete breakout strength in tension of an anchor group

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

$$\phi N_{cbg} = 0.7 \times \left(\frac{150000\ \mathrm{mm}^2}{90000\ \mathrm{mm}^2}\right) \times 1 \times 0.9 \times 1 \times 1 \times 45.475\ \mathrm{kN} = 47.749\ \mathrm{kN}$$

Result:

DCR - Demand-to-Capacity Ratio

PASS = 0.63

$$DCR = \frac{N_{ua}}{\phi N_{cbg}} = \frac{30\ \mathrm{kN}}{47.749\ \mathrm{kN}} = 0.62828$$

[Go back to summary results table.]

Check No. 5: Anchor Pullout Capacity

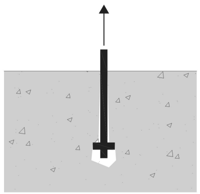


Fig. Headed anchor concrete pullout failure.

Calculate load per tension anchor

$N_x = 30\ \mathrm{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

N_{ua} - Required pull-out strength of cast-in anchor

$$N_{ua} = \frac{N_x}{n_{a,t}} = \frac{30\ \mathrm{kN}}{6} = 5\ \mathrm{kN}$$

Calculate area of bearing on embedded head

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

A_{rod} - Area of anchor rod

$$A_{rod} = \frac{\pi}{4}(d_a)^2 = \frac{\pi}{4} \times (16\ \mathrm{mm})^2 = 201.06\ \mathrm{mm}^2$$

$b_{embed\_plate} = 70\ \mathrm{mm}$ - Anchor embed plate diameter

A_{brg} - Net area of bearing for circular end

$$A_{brg} = \frac{\pi}{4}(b_{embed\_plate})^2 - A_{rod} = \frac{\pi}{4} \times (70\ \mathrm{mm})^2 - 201.06\ \mathrm{mm}^2 = 3647.4\ \mathrm{mm}^2$$

Calculate basic anchor pullout strength

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

ACI 318-19 Eq. 17.6.3.2.2a N_b - Basic single anchor pullout strength for cast-in headed anchors

$$N_b = 8A_{brg}\left(f_c^{'}\right) = 8 \times 3647.4\ \mathrm{mm}^2 \times (20.68\ \mathrm{MPa}) = 603.42\ \mathrm{kN}$$

Calculate anchor pullout strength in tension

$\phi = 0.7$ - Concrete tension resistance factor

ACI 318-19 Clause 17.6.3.3 $\Psi_{c,P} = 1$ - Pullout cracking factor

ACI 318-19 Eq. 17.6.3.2.2a $N_b = 603.42\ \mathrm{kN}$

ACI 318-19 Eq. 17.6.3.1 ϕN_{pn} - Design pull-out strength of cast-in anchor in tension

$$\phi N_{pn} = \phi \Psi_{c,P} N_b = 0.7 \times 1 \times 603.42\ \mathrm{kN} = 422.4\ \mathrm{kN}$$

Result: **PASS = 0.01**

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{N_{ua}}{\phi N_{pn}} = \frac{5\ \mathrm{kN}}{422.4\ \mathrm{kN}} = 0.011837$$

[Go back to summary results table.]

Check No. 6: Embed Plate Flexural Capacity

This additional check is to confirm the embed plate has enough thickness to resist flexural yielding.

Calculate load per tension anchor

T_a - Load per anchor

$$T_a = \frac{N_x}{n_{a,t}} = \frac{30\ \mathrm{kN}}{6} = 5\ \mathrm{kN}$$

Calculate one side width of embed plate

$b_{embed\_plate} = 70\ \mathrm{mm}$ - Anchor embed plate diameter

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

b' - Remaining embed plate dimension

$$b' = \frac{b_{embed\_plate} - d_a}{2} = \frac{70\ \mathrm{mm} - 16\ \mathrm{mm}}{2} = 27\ \mathrm{mm}$$

Calculate flexural demand on embed plate (taken as moment per unit length)

$A_{brg} = 3647.4\ \mathrm{mm}^2$

m_f - Moment force from uniform pressure

$$m_f = \frac{\left(\frac{T_a}{A_{brg}}\right)(b')^2}{2} = \frac{\left(\frac{5\ \mathrm{kN}}{3647.4\ \mathrm{mm}^2}\right) \times (27\ \mathrm{mm})^2}{2} = 0.49967\ \mathrm{kN}$$

Calculate the minimum embed plate thickness for flexural yielding

The embed plate yield grade is assumed to match the base plate grade.

$F_{y\_ep} = 248.2\ \mathrm{MPa}$ - Embed plate yield stress (A36)

$\phi = 0.9$ - Flexural resistance factor

t_{min} - Required thickness of embed plate

$$t_{min} = \sqrt{\frac{4m_f}{\phi F_{y\_ep}}} = \sqrt{\frac{4 \times 0.49967\ \text{kN}}{0.9 \times 248.2\ \text{MPa}}} = 2.9912\ \text{mm}$$

Check actual embed plate thickness

t_{actual} - Provided embed plate thickness

$$t_{actual} = t_{embed\_plate} = 10\ \text{mm}$$

Result:

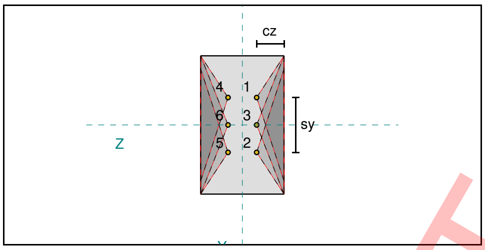
DCR - Demand-to-Capacity Ratio

$$DCR = \frac{t_{min}}{t_{actual}} = \frac{2.9912\ \text{mm}}{10\ \text{mm}} = 0.29912$$

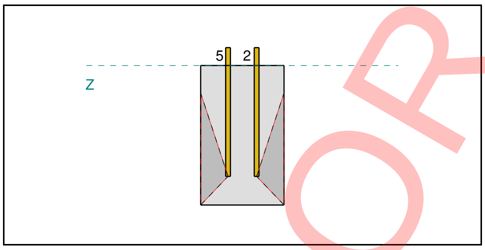
PASS = 0.30

Go back to summary results table.

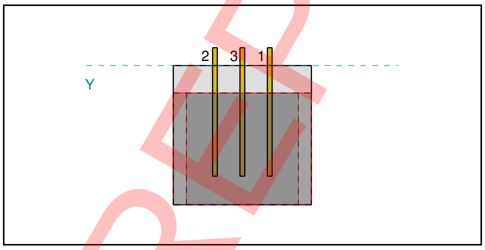
Check No. 7: Concrete Side-Face Blowout Capacity (Y-Direction)



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total tension load on anchor group in Y-direction

$N_x = 30\ \text{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

$n_{y,g1} = 3$ - Number of anchors along Y-axis for anchor group SFy 1

N_{ua} - Required concrete side-face blowout strength of an anchor group along Y-axis

$$N_{ua} = \left(\frac{N_x}{n_{a,t}}\right) n_{y,g1} = \left(\frac{30\ \text{kN}}{6}\right) \times 3 = 15\ \text{kN}$$

Symbol	Description	Value
$c_{left,g1}$	Anchor group SFy 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group SFy 1 concrete edge distance (right)	200.0 mm
$c_{top,g1}$	Anchor group SFy 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group SFy 1 concrete edge distance (bottom)	150.0 mm
$s_{sum,y,g1}$	Anchor group SFy 1 sum of spacing along Y-axis	200.0 mm

Calculation for critical anchor group: Anchor Group SFy 1.

Calculate minimum edge distance to failure edge

$c_{left,g1} = 100\ \text{mm}$ - Anchor group SFy 1 concrete edge distance (left)

$c_{right,g1} = 200\ \text{mm}$ - Anchor group SFy 1 concrete edge distance (right)

$c_{z,min}$ - Minimum edge distance along Z-axis

$$c_{z,min} = \min\left(\text{c}_{\text{left,g1}},\ \text{c}_{\text{right,g1}}\right) = \min\left(100\ \text{mm},\ 200\ \text{mm}\right) = 100\ \text{mm}$$

Calculate minimum edge distance to the orthogonal edge

$c_{top,g1} = 150\ \text{mm}$ - Anchor group SFy 1 concrete edge distance (top)

$c_{bottom,g1} = 150\ \text{mm}$ - Anchor group SFy 1 concrete edge distance (bottom)

$c_{y,min}$ - Minimum edge distance along Y-axis

$$c_{y,min} = \min\left(\mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min\left(150\ \mathrm{mm},\ 150\ \mathrm{mm}\right) = 150\ \mathrm{mm}$$

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 nominal side-face blowout of the group anchor along Y-direction

$s_{sum,y,g1} = 200\ \mathrm{mm}$ - Anchor group SFy 1 sum of spacing along Y-axis

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

$A_{brg} = 3647.4\ \mathrm{mm}^2$

ACI 318M-19 Eq. 17.6.4.1

N_{sbg} - Nominal side-face blowout strength of multiple headed anchors (close to perpendicular edge)

$$N_{sbg} = \left(\frac{1 + \frac{c_{y,min}}{c_{z,min}}}{4} + \frac{s_{sum,y,g1}}{6c_{z,min}}\right) 13 \left(\frac{c_{z,min}}{mm}\right) \sqrt{\frac{A_{brg}}{mm^2}} \lambda_a \sqrt{\frac{f_c^{'}}{MPa}} N$$

$$N_{sbg} = \left(\frac{1 + \frac{150\ \mathrm{mm}}{100\ \mathrm{mm}}}{4} + \frac{200\ \mathrm{mm}}{6 \times 100\ \mathrm{mm}}\right) \times 13 \times \left(\frac{100\ \mathrm{mm}}{1\ \mathrm{mm}}\right) \times \sqrt{\frac{3647.4\ \mathrm{mm}^2}{1\ \mathrm{mm}^2}} \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times 0.001\ \mathrm{kN}$$

$$N_{sbg} = 342.16\ \mathrm{kN}$$

Calculate design side-face blowout of anchor group along Y-direction

$\phi = 0.7$ - Concrete tension resistance factor

ACI 318-19 Clause 17.6.4.1

ϕN_{sbg} - Design concrete side-face blowout strength of headed anchors

$$\phi N_{sbg} = \phi N_{sbg} = 0.7 \times 342.16\ \mathrm{kN} = 239.51\ \mathrm{kN}$$

Result:

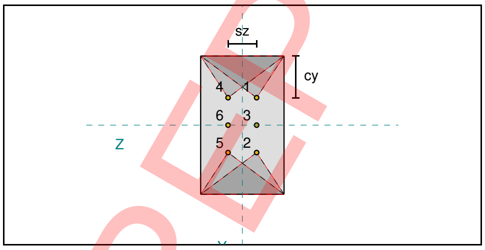
PASS = 0.06

DCR - Demand-to-Capacity Ratio

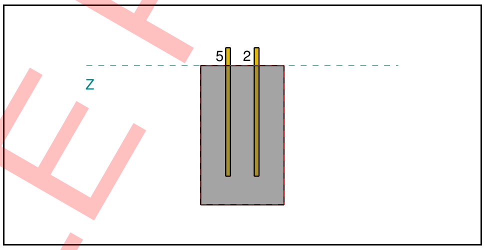
$$DCR = \frac{N_{ua}}{\phi N_{sbg}} = \frac{15\ \mathrm{kN}}{239.51\ \mathrm{kN}} = 0.062628$$

[Go back to summary results table](#).

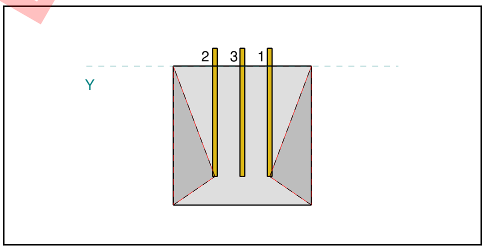
Check No. 8: Concrete Side-Face Blowout Capacity (Z-Direction)



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total tension load on anchor group in Z-direction

$N_x = 30\ \mathrm{kN}$ - Tension load

$n_{a,t} = 6$ - Number of anchors in tension

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

N_{ua} - Required concrete side-face blowout strength of an anchor group along Z-axis

$$N_{ua} = \left(\frac{N_x}{n_{a,t}}\right) n_{z,g1} = \left(\frac{30\ \mathrm{kN}}{6}\right) \times 2 = 10\ \mathrm{kN}$$

Symbol	Description	Value
$c_{left,g1}$	Anchor group SFz 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group SFz 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group SFz 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group SFz 1 concrete edge distance (bottom)	350.0 mm
$s_{sum,z,g1}$	Anchor group SFz 1 sum of spacing along Z-axis	100.0 mm

Calculation for critical anchor group: Anchor Group SFz 1.

Calculate minimum edge distance to the failure edge

$c_{top,g1} = 150\ \mathrm{mm}$ - Anchor group SFz 1 concrete edge distance (top)

$c_{bottom,g1} = 350\ \mathrm{mm}$ - Anchor group SFz 1 concrete edge distance (bottom)

$c_{y,min}$ - Minimum edge distance along Y-axis

$$c_{y,min} = \min\left(\mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min\left(150\ \mathrm{mm},\ 350\ \mathrm{mm}\right) = 150\ \mathrm{mm}$$

Calculate minimum edge distance to the orthogonal edge

$c_{left,g1} = 100\ \mathrm{mm}$ - Anchor group SFz 1 concrete edge distance (left)

$c_{right,g1} = 100\ \mathrm{mm}$ - Anchor group SFz 1 concrete edge distance (right)

$c_{z,min}$ - Minimum edge distance along Z-axis

$$c_{z,min} = \min\left(\mathrm{c_{left,g1}},\ \mathrm{c_{right,g1}}\right) = \min\left(100\ \mathrm{mm},\ 100\ \mathrm{mm}\right) = 100\ \mathrm{mm}$$

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 nominal side-face blowout of the group anchor along Z-direction

$s_{sum,z,g1} = 100\ \mathrm{mm}$ - Anchor group SFz 1 sum of spacing along Z-axis

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

$A_{brg} = 3647.4\ \mathrm{mm}^2$

ACI 318M-19 Eq. 17.6.4.1

N_{sbg} - Nominal side-face blowout strength of multiple headed anchors (close to perpendicular edge)

$$N_{sbg} = \left(\frac{1 + \frac{c_{z,min}}{c_{y,min}}}{4} + \frac{s_{sum,z,g1}}{6c_{y,min}}\right) 13 \left(\frac{c_{y,min}}{mm}\right) \sqrt{\frac{A_{brg}}{mm^2}} \lambda_a \sqrt{\frac{f_c^{'}}{MPa}} N$$

$$N_{sbg} = \left(\frac{1 + \frac{100\ \mathrm{mm}}{150\ \mathrm{mm}}}{4} + \frac{100\ \mathrm{mm}}{6 \times 150\ \mathrm{mm}}\right) \times 13 \times \left(\frac{150\ \mathrm{mm}}{1\ \mathrm{mm}}\right) \times \sqrt{\frac{3647.4\ \mathrm{mm}^2}{1\ \mathrm{mm}^2}} \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times 0.001\ \mathrm{kN}$$

$$N_{sbg} = 282.65\ \mathrm{kN}$$

Calculate design side-face blowout of anchor group along Z-direction

$\phi = 0.7$ - Concrete tension resistance factor

ACI 318-19 Clause 17.6.4.1

ϕN_{sbg} - Design concrete side-face blowout strength of headed anchors

$$\phi N_{sbg} = \phi N_{sbg} = 0.7 \times 282.65\ \mathrm{kN} = 197.86\ \mathrm{kN}$$

Result:

PASS = 0.05

DCR - Demand-to-Capacity Ratio

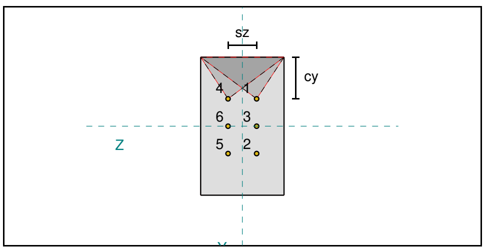
$$DCR = \frac{N_{ua}}{\phi N_{sbg}} = \frac{10\ \mathrm{kN}}{197.86\ \mathrm{kN}} = 0.050542$$

Go back to summary results table.

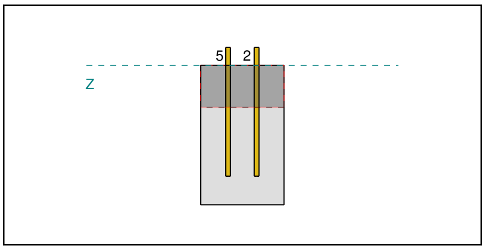
Check No. 9: Concrete Breakout Capacity (Vy Shear)

Only the anchors nearest to the failure edge are assumed to resist the shear load.

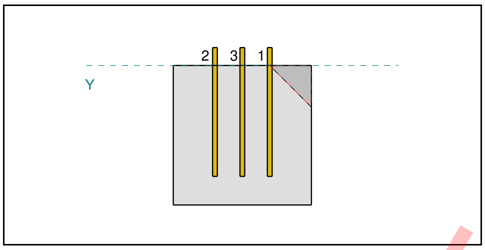
For breakout capacity on failure edge perpendicular to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group. Consider one side only.

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

$n_{a,vy} = 2$ - Number of anchors resisting +Vy shear

$n_{a,g1} = 2$ - Number of anchors in anchor group BO Vy 1

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

$$V_{fa\perp} = V_y\left(\frac{n_{a,g1}}{n_{a,vy}}\right) = 3\ \mathrm{kN} \times \left(\frac{2}{2}\right) = 3\ \mathrm{kN}$$

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
$c_{left,g1}$	Anchor group BO Vy 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group BO Vy 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group BO Vy 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group BO Vy 1 concrete edge distance (bottom)	350.0 mm
t_{conc}	Concrete thickness	500.0 mm
s_z	Spacing of anchor rods along Z-axis	100.0 mm
$c_{a1,g1}$	Anchor group distance to failure edge (+Vy shear)	150.0 mm
	Narrow Member	FALSE

Calculation for critical anchor group: Anchor Group BO Vy 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,g1} = 150\ \mathrm{mm}$ - Anchor group distance to failure edge (+Vy shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5(c_{a1,g1})^2 = 4.5 \times (150\ \mathrm{mm})^2 = 101250\ \mathrm{mm}^2$$

Calculate width of actual projected area on failure surface

$s_{sum,z,g1} = 100\ \mathrm{mm}$ - Anchor group BO Vy 1 sum of spacing along Z-axis

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

ACI 318-19 Clause 17.7.2.1.1

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

$$B_{Vc} = \min\left(\mathrm{c_{left,g1}},\ 1.5\mathrm{c_{a1,g1}}\right) + \left(\min\left(\mathrm{s_{sum,z,g1}},\ 3\mathrm{c_{a1,g1}}\left(\mathrm{n_{z,g1}} - 1\right)\right)\right) + \min\left(\mathrm{c_{right,g1}},\ 1.5\mathrm{c_{a1,g1}}\right)$$

$$B_{Vc} = \min\left(100\ \mathrm{mm},\ 1.5 \times 150\ \mathrm{mm}\right) + \left(\min\left(100\ \mathrm{mm},\ 3 \times 150\ \mathrm{mm} \times (2-1)\right)\right) + \min\left(100\ \mathrm{mm},\ 1.5 \times 150\ \mathrm{mm}\right)$$

$$B_{Vc} = 300\ \mathrm{mm}$$

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

$$H_{Vc} = \min\left(1.5\mathrm{c_{a1,g1}},\ \mathrm{t_{conc}}\right) = \min\left(1.5 \times 150\ \mathrm{mm},\ 500\ \mathrm{mm}\right) = 225\ \mathrm{mm}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 300\ \mathrm{mm} \times 225\ \mathrm{mm} = 67500\ \mathrm{mm}^2$$

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

$h_{ef} = 400\ \mathrm{mm}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 400\ \mathrm{mm}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

ACI 318M-19 Eq. 17.7.2.2.1a

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

$$V_{b1} = 0.6\left(\frac{\min\left(\mathrm{l_e},\ 8\mathrm{d_a}\right)}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{mm}}\lambda_a\sqrt{\frac{f_c^{'}}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b1} = 0.6 \times \left(\frac{\min\left(400\ \mathrm{mm},\ 8 \times 16\ \mathrm{mm}\right)}{16\ \mathrm{mm}}\right)^{0.2} \times \sqrt{\frac{16\ \mathrm{mm}}{1\ \mathrm{mm}}} \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{150\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN}$$

$$V_{b1} = 30.391\ \mathrm{kN}$$

ACI 318M-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 3.7\lambda_a\sqrt{\frac{f_c^{'}}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b2} = 3.7 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{150\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 30.911\ \mathrm{kN}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(\mathrm{V_{b1}},\ \mathrm{V_{b2}}\right) = \min\left(30.391\ \mathrm{kN},\ 30.911\ \mathrm{kN}\right) = 30.391\ \mathrm{kN}$$

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

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e_N^{'}}{3c_{a1,g1}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 150\ \mathrm{mm}}}\right) = 1$$

Calculate edge effect factor

$c_{a2,g1} = 100\ \mathrm{mm}$ - Anchor group distance to failure 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

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a2,g1}}}{1.5\mathrm{c_{a1,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{100\ \mathrm{mm}}{1.5 \times 150\ \mathrm{mm}}\right)\right) = 0.83333$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,g1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 150\ \mathrm{mm}}{500\ \mathrm{mm}}},\ 1\right) = 1$$

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)

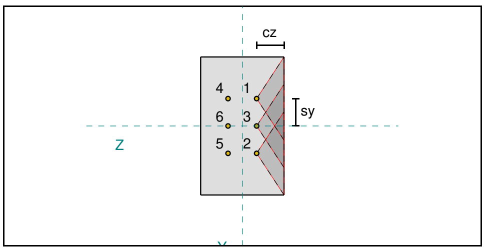
ACI 318-19 Clause 17.7.2.1(b)

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

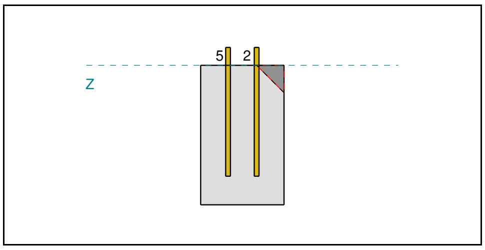
$$\phi V_{cbg\perp} = \phi\left(\frac{A_{Vc}}{A_{Vco}}\right)\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

$$\phi V_{cbg\perp} = 0.65 \times \left(\frac{67500\ \mathrm{mm}^2}{101250\ \mathrm{mm}^2}\right) \times 1 \times 0.83333 \times 1 \times 1 \times 30.391\ \mathrm{kN} = 10.974\ \mathrm{kN}$$

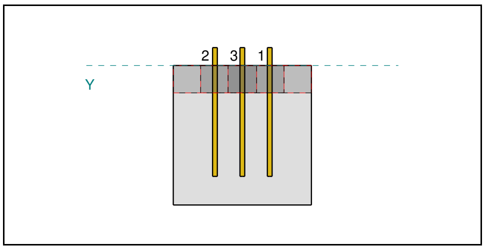
For breakout capacity on failure edge parallel to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group. Consider one side only.

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

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

$n_{a,vz} = 3$ - Number of anchors resisting +Vz shear

$n_{a,g1} = 3$ - Number of anchors in anchor group BO Vz 1

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

$$V_{fa\|} = V_y \left(\frac{n_{a,g1}}{n_{a,vz}}\right) = 3\ \mathrm{kN} \times \left(\frac{3}{3}\right) = 3\ \mathrm{kN}$$

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
$c_{left,g1}$	Anchor group BO Vz 1 concrete edge distance (left)	200.0 mm
$c_{right,g1}$	Anchor group BO Vz 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group BO Vz 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group BO Vz 1 concrete edge distance (bottom)	150.0 mm
t_{conc}	Concrete thickness	500.0 mm
s_y	Spacing of anchor rods along Y-axis	100.0 mm
$c_{a1,g1}$	Anchor group distance to failure edge (+Vz shear)	100.0 mm
	Narrow Member	FALSE

Calculation for critical anchor group: Anchor Group BO Vz 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,g1} = 100\ \mathrm{mm}$ - Anchor group distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5(c_{a1,g1})^2 = 4.5 \times (100\ \mathrm{mm})^2 = 45000\ \mathrm{mm}^2$$

Calculate width of actual projected area on failure surface

$s_{sum,y,g1} = 200\ \mathrm{mm}$ - Anchor group BO Vz 1 sum of spacing along Y-axis

$n_{y,g1} = 3$ - Number of anchors along Y-axis for anchor group BO Vz 1

ACI 318-19 Clause 17.7.2.1.1

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

$$B_{Vc} = \min\left(\mathrm{c_{top,g1}},\ 1.5\mathrm{c_{a1,g1}}\right) + \left(\min\left(\mathrm{s_{sum,y,g1}},\ 3\mathrm{c_{a1,g1}}\left(\mathrm{n_{y,g1}} - 1\right)\right)\right) + \min\left(\mathrm{c_{bottom,g1}},\ 1.5\mathrm{c_{a1,g1}}\right)$$

$$B_{Vc} = \min\left(150\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm}\right) + \left(\min\left(200\ \mathrm{mm},\ 3 \times 100\ \mathrm{mm} \times (3 - 1)\right)\right) + \min\left(150\ \mathrm{mm},\ 1.5 \times 100\ \mathrm{mm}\right)$$

$$B_{Vc} = 500\ \mathrm{mm}$$

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

$$H_{Vc} = \min\left(1.5\mathrm{c_{a1,g1}},\ \mathrm{t_{conc}}\right) = \min\left(1.5 \times 100\ \mathrm{mm},\ 500\ \mathrm{mm}\right) = 150\ \mathrm{mm}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc} H_{Vc} = 500 \text{ mm} \times 150 \text{ mm} = 75000 \text{ mm}^2$$

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

$h_{ef} = 400 \text{ mm}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 400 \text{ mm}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 20.68 \text{ MPa}$ - Concrete compressive strength (20.68)

$d_a = 16 \text{ mm}$ - Anchor rod diameter

ACI 318M-19 Eq. 17.7.2.2.1a

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

$$V_{b1} = 0.6 \left(\frac{\min(\mathrm{l_e},\ 8\mathrm{d_a})}{d_a}\right)^{0.2} \sqrt{\frac{d_a}{mm}} \lambda_a \sqrt{\frac{f_c^{'}}{MPa}} \left(\frac{c_{a1,g1}}{mm}\right)^{1.5} N$$

$$V_{b1} = 0.6 \times \left(\frac{\min(400 \text{ mm},\ 8 \times 16 \text{ mm})}{16 \text{ mm}}\right)^{0.2} \times \sqrt{\frac{16 \text{ mm}}{1 \text{ mm}}} \times 1 \times \sqrt{\frac{20.68 \text{ MPa}}{1 \text{ MPa}}} \times \left(\frac{100 \text{ mm}}{1 \text{ mm}}\right)^{1.5} \times 0.001 \text{ kN}$$

$$V_{b1} = 16.543 \text{ kN}$$

ACI 318M-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 3.7\lambda_a \sqrt{\frac{f_c^{'}}{MPa}} \left(\frac{c_{a1,g1}}{mm}\right)^{1.5} N$$

$$V_{b2} = 3.7 \times 1 \times \sqrt{\frac{20.68 \text{ MPa}}{1 \text{ MPa}}} \times \left(\frac{100 \text{ mm}}{1 \text{ mm}}\right)^{1.5} \times 0.001 \text{ kN} = 16.826 \text{ kN}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min(\mathrm{V_{b1}},\ \mathrm{V_{b2}}) = \min(16.543 \text{ kN},\ 16.826 \text{ kN}) = 16.543 \text{ kN}$$

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

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e_N'}{3c_{a1,g1}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 100 \text{ mm}}}\right) = 1$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,g1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 100 \text{ mm}}{500 \text{ mm}}},\ 1\right) = 1$$

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)

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

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

$$\phi V_{cbg\parallel} = 2 \times 0.65 \times \left(\frac{75000 \text{ mm}^2}{45000 \text{ mm}^2}\right) \times 1 \times 1 \times 1 \times 1 \times 16.543 \text{ kN} = 35.842 \text{ kN}$$

Result:

PASS = 0.27

DCR - Demand over capacity ratio, comparing two conditions:

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

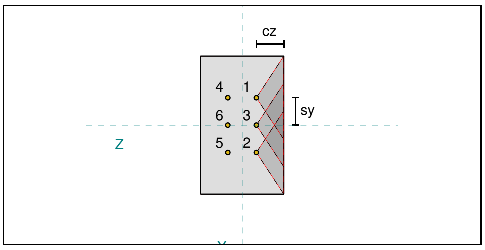
$$DCR = \max\left(\frac{3 \text{ kN}}{10.974 \text{ kN}},\ \frac{3 \text{ kN}}{35.842 \text{ kN}}\right) = 0.27336$$

Go back to summary results table.

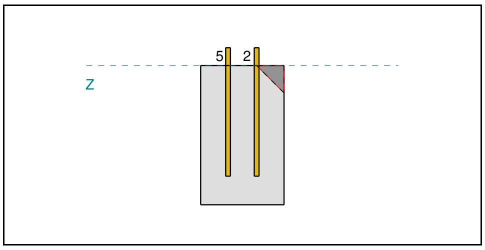
Check No. 10: Concrete Breakout Capacity (Vz Shear)

Only the anchors nearest to the failure edge are assumed to resist the shear load.

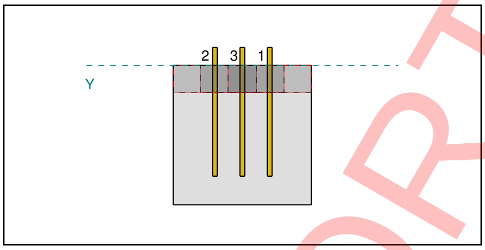
For breakout capacity on failure edge perpendicular to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group. Consider one side only.

$V_z = 6 \text{ kN}$ - Vz shear load

$n_{a,vz} = 3$ - Number of anchors resisting +Vz shear

$n_{a,g1} = 3$ - Number of anchors in anchor group BO Vz 1

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

$$V_{fa\perp} = V_z \left(\frac{n_{a,g1}}{n_{a,vz}}\right) = 6 \text{ kN} \times \left(\frac{3}{3}\right) = 6 \text{ kN}$$

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
$c_{left,g1}$	Anchor group BO Vz 1 concrete edge distance (left)	200.0 mm
$c_{right,g1}$	Anchor group BO Vz 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group BO Vz 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group BO Vz 1 concrete edge distance (bottom)	150.0 mm
t_{conc}	Concrete thickness	500.0 mm
s_y	Spacing of anchor rods along Y-axis	100.0 mm
$c_{a1,g1}$	Anchor group distance to failure edge (+Vz shear)	100.0 mm
	Narrow Member	FALSE

Calculation for critical anchor group: Anchor Group BO Vz 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,g1} = 100 \text{ mm}$ - Anchor group distance to failure edge (+Vz shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5(c_{a1,g1})^2 = 4.5 \times (100 \text{ mm})^2 = 45000 \text{ mm}^2$$

Calculate width of actual projected area on failure surface

$s_{sum,y,g1} = 200 \text{ mm}$ - Anchor group BO Vz 1 sum of spacing along Y-axis

$n_{y,g1} = 3$ - Number of anchors along Y-axis for anchor group BO Vz 1

ACI 318-19 Clause 17.7.2.1.1

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

$$B_{Vc} = \min\left(\mathrm{c_{top,g1}},\ 1.5\mathrm{c_{a1,g1}}\right) + \left(\min\left(\mathrm{s_{sum,y,g1}},\ 3\mathrm{c_{a1,g1}}\left(\mathrm{n_{y,g1}} - 1\right)\right)\right) + \min\left(\mathrm{c_{bottom,g1}},\ 1.5\mathrm{c_{a1,g1}}\right)$$

$$B_{Vc} = \min\left(150 \text{ mm},\ 1.5 \times 100 \text{ mm}\right) + \left(\min\left(200 \text{ mm},\ 3 \times 100 \text{ mm} \times (3-1)\right)\right) + \min\left(150 \text{ mm},\ 1.5 \times 100 \text{ mm}\right)$$

$$B_{Vc} = 500 \text{ mm}$$

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

$$H_{Vc} = \min\left(1.5\mathrm{c_{a1,g1}},\ \mathrm{t_{conc}}\right) = \min\left(1.5 \times 100\ \mathrm{mm},\ 500\ \mathrm{mm}\right) = 150\ \mathrm{mm}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 500\ \mathrm{mm} \times 150\ \mathrm{mm} = 75000\ \mathrm{mm}^2$$

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

$h_{ef} = 400\ \mathrm{mm}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 400\ \mathrm{mm}$$

Calculate basic single anchor breakout strength

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

ACI 318M-19 Eq. 17.7.2.2.1a

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

$$V_{b1} = 0.6\left(\frac{\min\left(\mathrm{l_e},\ 8\mathrm{d_a}\right)}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{mm}}\lambda_a\sqrt{\frac{f_c^{'}}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b1} = 0.6 \times \left(\frac{\min\left(400\ \mathrm{mm},\ 8 \times 16\ \mathrm{mm}\right)}{16\ \mathrm{mm}}\right)^{0.2} \times \sqrt{\frac{16\ \mathrm{mm}}{1\ \mathrm{mm}}} \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{100\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN}$$

$$V_{b1} = 16.543\ \mathrm{kN}$$

ACI 318M-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 3.7\lambda_a\sqrt{\frac{f_c^{'}}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b2} = 3.7 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{100\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 16.826\ \mathrm{kN}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min\left(\mathrm{V_{b1}},\ \mathrm{V_{b2}}\right) = \min\left(16.543\ \mathrm{kN},\ 16.826\ \mathrm{kN}\right) = 16.543\ \mathrm{kN}$$

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

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e_N^{'}}{3\mathrm{c_{a1,g1}}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 100\ \mathrm{mm}}}\right) = 1$$

Calculate edge effect factor

$c_{a2,g1} = 150\ \mathrm{mm}$ - Anchor group distance to failure 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

$$\Psi_{ed,V} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a2,g1}}}{1.5\mathrm{c_{a1,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{150\ \mathrm{mm}}{1.5 \times 100\ \mathrm{mm}}\right)\right) = 1$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,g1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 100\ \mathrm{mm}}{500\ \mathrm{mm}}},\ 1\right) = 1$$

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)

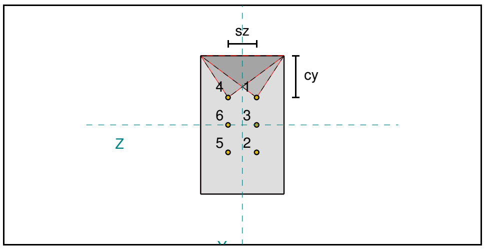
ACI 318-19 Clause 17.7.2.1(b)

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

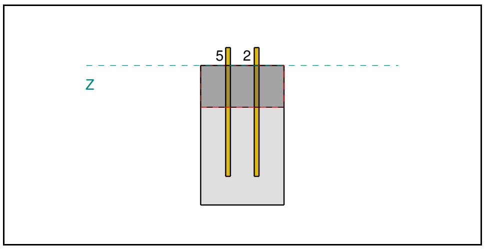
$$\phi V_{cbg\perp} = \phi\left(\frac{A_{Vc}}{A_{Vco}}\right)\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_b$$

$$\phi V_{cbg\perp} = 0.65 \times \left(\frac{75000\ \mathrm{mm}^2}{45000\ \mathrm{mm}^2}\right) \times 1 \times 1 \times 1 \times 1 \times 16.543\ \mathrm{kN} = 17.921\ \mathrm{kN}$$

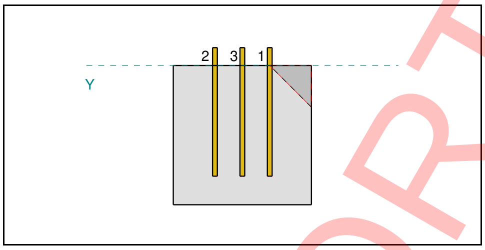
For breakout capacity on failure edge parallel to load:



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group. Consider one side only.

$V_z = 6\ \mathrm{kN}$ - Vz shear load

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

$n_{a,vy} = 2$ - Number of anchors resisting +Vy shear

$n_{a,g1} = 2$ - Number of anchors in anchor group BO Vy 1

$V_{f a\parallel}$ - Required concrete breakout shear strength of anchor group with failure edge parallel to load

$$V_{f a\parallel} = V_z \left(\frac{n_{a,g1}}{n_{a,vy}}\right) = 6\ \mathrm{kN} \times \left(\frac{2}{2}\right) = 6\ \mathrm{kN}$$

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
$c_{left,g1}$	Anchor group BO Vy 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group BO Vy 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group BO Vy 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group BO Vy 1 concrete edge distance (bottom)	350.0 mm
t_{conc}	Concrete thickness	500.0 mm
s_z	Spacing of anchor rods along Z-axis	100.0 mm
$c_{a1,g1}$	Anchor group distance to failure edge (+Vy shear)	150.0 mm
	Narrow Member	FALSE

Calculation for critical anchor group: Anchor Group BO Vy 1.

Calculate maximum projected area for a single anchor

ACI 318-19 Clause 17.7.2.1.2

$c_{a1,g1} = 150\ \mathrm{mm}$ - Anchor group distance to failure edge (+Vy shear)

ACI 318-19 Eq. 17.7.2.1.3

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

$$A_{Vco} = 4.5(c_{a1,g1})^2 = 4.5 \times (150\ \mathrm{mm})^2 = 101250\ \mathrm{mm}^2$$

Calculate width of actual projected area on failure surface

$s_{sum,z,g1} = 100\ \mathrm{mm}$ - Anchor group BO Vy 1 sum of spacing along Z-axis

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

ACI 318-19 Clause 17.7.2.1.1

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

$$B_{Vc} = \min\left(\mathrm{c_{left,g1}},\ 1.5\mathrm{c_{a1,g1}}\right) + \left(\min\left(\mathrm{s_{sum,z,g1}},\ 3\mathrm{c_{a1,g1}}\left(\mathrm{n_{z,g1}} - 1\right)\right)\right) + \min\left(\mathrm{c_{right,g1}},\ 1.5\mathrm{c_{a1,g1}}\right)$$

$$B_{Vc} = \min\left(100\ \mathrm{mm},\ 1.5 \times 150\ \mathrm{mm}\right) + \left(\min\left(100\ \mathrm{mm},\ 3 \times 150\ \mathrm{mm} \times (2-1)\right)\right) + \min\left(100\ \mathrm{mm},\ 1.5 \times 150\ \mathrm{mm}\right)$$

$$B_{Vc} = 300\ \mathrm{mm}$$

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

$$H_{Vc} = \min(1.5\mathrm{c_{a1,g1}},\ \mathrm{t_{conc}}) = \min(1.5 \times 150\ \mathrm{mm},\ 500\ \mathrm{mm}) = 225\ \mathrm{mm}$$

Calculate actual projected area

ACI 318-19 Clause 17.7.2.1.1

A_{Vc} - Actual projected area

$$A_{Vc} = B_{Vc}H_{Vc} = 300\ \mathrm{mm} \times 225\ \mathrm{mm} = 67500\ \mathrm{mm}^2$$

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

$h_{ef} = 400\ \mathrm{mm}$ - Anchor rod effective embedment length

ACI 318-19 Clause 17.7.2.2.1

l_e - Load bearing length (equal to embedment height)

$$l_e = h_{ef} = 400\ \mathrm{mm}$$

Calculate basic single anchor breakout strength

$f_c' = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

ACI 318M-19 Eq. 17.7.2.2.1a

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

$$V_{b1} = 0.6\left(\frac{\min(\mathrm{l_e},\ 8\mathrm{d_a})}{d_a}\right)^{0.2}\sqrt{\frac{d_a}{mm}}\lambda_a\sqrt{\frac{f_c'}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b1} = 0.6 \times \left(\frac{\min(400\ \mathrm{mm},\ 8 \times 16\ \mathrm{mm})}{16\ \mathrm{mm}}\right)^{0.2} \times \sqrt{\frac{16\ \mathrm{mm}}{1\ \mathrm{mm}}} \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{150\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN}$$

$$V_{b1} = 30.391\ \mathrm{kN}$$

ACI 318M-19 Eq. 17.7.2.2.1b

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

$$V_{b2} = 3.7\lambda_a\sqrt{\frac{f_c'}{MPa}}\left(\frac{c_{a1,g1}}{mm}\right)^{1.5}N$$

$$V_{b2} = 3.7 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{150\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 30.911\ \mathrm{kN}$$

ACI 318-19 Clause 17.7.2.2.1

V_b - Basic single anchor breakout strength

$$V_b = \min(\mathrm{V_{b1}},\ \mathrm{V_{b2}}) = \min(30.391\ \mathrm{kN},\ 30.911\ \mathrm{kN}) = 30.391\ \mathrm{kN}$$

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

$$\Psi_{ec,V} = \min\left(1.0,\ \frac{1}{1 + \frac{2e_N'}{3c_{a1,g1}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2\times0}{3\times150\ \mathrm{mm}}}\right) = 1$$

Calculate thickness factor

ACI 318-19 Eq. 17.7.2.6.1

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

$$\Psi_{h,V} = \max\left(\sqrt{\frac{1.5\mathrm{c_{a1,g1}}}{\mathrm{t_{conc}}}},\ 1.0\right) = \max\left(\sqrt{\frac{1.5 \times 150\ \mathrm{mm}}{500\ \mathrm{mm}}},\ 1\right) = 1$$

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)

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

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

$$\phi V_{cbg\parallel} = 2 \times 0.65 \times \left(\frac{67500\ \mathrm{mm}^2}{101250\ \mathrm{mm}^2}\right) \times 1 \times 1 \times 1 \times 1 \times 30.391\ \mathrm{kN} = 26.339\ \mathrm{kN}$$

Result:

PASS = 0.33

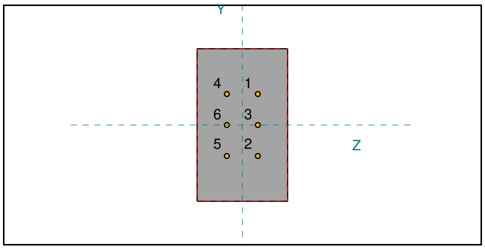
DCR - Demand over capacity ratio, comparing two conditions:

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

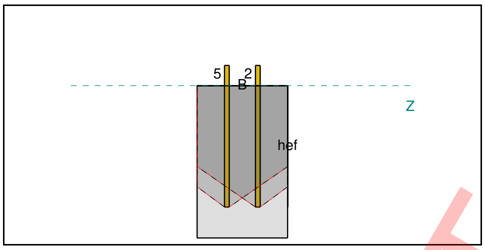
$$DCR = \max\left(\frac{6\ \mathrm{kN}}{17.921\ \mathrm{kN}},\ \frac{6\ \mathrm{kN}}{26.339\ \mathrm{kN}}\right) = 0.3348$$

Go back to summary results table.

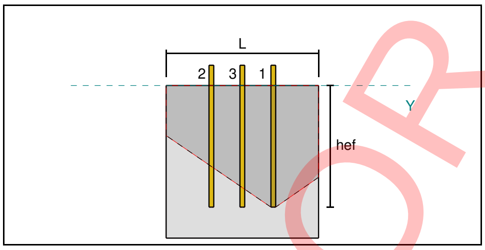
Check No. 11: Concrete Pryout Capacity (Vy Shear)



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group due to Vy shear

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

$n_{a,vy} = 2$ - Number of anchors resisting +Vy shear

$n_{a,g1} = 2$ - Number of anchors in anchor group PO Vy 1

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

$$V_{ua} = \left(\frac{V_y}{n_{a,vy}}\right) n_{a,g1} = \left(\frac{3\ \text{kN}}{2}\right) \times 2 = 3\ \text{kN}$$

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 Vy 1 concrete edge distance (left)	100.0 mm
$c_{right,g1}$	Anchor group PO Vy 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group PO Vy 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group PO Vy 1 concrete edge distance (bottom)	350.0 mm
h_{ef}	Anchor rod effective embedment length	400.0 mm
$s_{sum,y,g1}$	Anchor group PO Vy 1 sum of spacing along Y-axis	0.0 mm
$s_{sum,z,g1}$	Anchor group PO Vy 1 sum of spacing along Z-axis	100.0 mm
$n_{y,g1}$	Number of anchors along Y-axis for anchor group PO Vy 1	1.0
$n_{z,g1}$	Number of anchors along Z-axis for anchor group PO Vy 1	2.0
	Narrow Member	TRUE
$h'_{ef,g1}$	Anchor group PO Vy 1 modified effective embedment length	233.3 mm

Calculation for critical anchor group: Anchor Group PO Vy 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

$$A_{Nco} = 9\left(h'_{ef,g1}\right)^2 = 9 \times (233.33\ \text{mm})^2 = 490000\ \text{mm}^2$$

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

$s_{sum,z,g1} = 100\ \text{mm}$ - Anchor group PO Vy 1 sum of spacing along Z-axis

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

ACI 318-19 Clause 17.6.2

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

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

$$L_{Nc} = \min(100\ \mathrm{mm},\ 1.5 \times 233.33\ \mathrm{mm}) + (\min(100\ \mathrm{mm},\ 3 \times 233.33\ \mathrm{mm} \times (2-1))) + \min(100\ \mathrm{mm},\ 1.5 \times 233.33\ \mathrm{mm})$$

$$L_{Nc} = 300\ \mathrm{mm}$$

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

$s_{sum,y,g1} = 0\ \mathrm{mm}$ - Anchor group PO Vy 1 sum of spacing along Y-axis

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

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

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

$$B_{Nc} = \min(150\ \mathrm{mm},\ 1.5 \times 233.33\ \mathrm{mm}) + (\min(0\ \mathrm{mm},\ 3 \times 233.33\ \mathrm{mm} \times (1-1))) + \min(350\ \mathrm{mm},\ 1.5 \times 233.33\ \mathrm{mm})$$

$$B_{Nc} = 500\ \mathrm{mm}$$

Calculate actual projected area of the anchor group

$n_{a,g1} = 2$ - Number of anchors in anchor group PO Vy 1

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

$$A_{Nc} = \min\left(\mathrm{n_{a,g1}A_{Nco}},\ \mathrm{L_{Nc}B_{Nc}}\right) = \min\left(2 \times 490000\ \mathrm{mm}^2,\ 300\ \mathrm{mm} \times 500\ \mathrm{mm}\right) = 150000\ \mathrm{mm}^2$$

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

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

ACI 318-19 Clause 17.6.2.2.1 $k_c = 10$ - Factor for cast-in anchors breakout strength

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

$$N_b = k_c \lambda_a \sqrt{\frac{f_c^{'}}{MPa}} \left(\frac{h'_{ef,g1}}{mm}\right)^{1.5} N$$

$$N_b = 10 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{233.33\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 162.08\ \mathrm{kN}$$

Calculate minimum edge distance of anchor

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

$$c_{a,min} = \min\left(\mathrm{c_{left,g1}},\ \mathrm{c_{right,g1}},\ \mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min(100\ \mathrm{mm},\ 100\ \mathrm{mm},\ 150\ \mathrm{mm},\ 350\ \mathrm{mm}) = 100\ \mathrm{mm}$$

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

$$\Psi_{ec,N} = \min\left(1.0,\ \frac{1}{1 + \frac{2e'_N}{3h'_{ef,g1}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2\times 0}{3\times 233.33\ \mathrm{mm}}}\right) = 1$$

Calculate edge effect factor

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

$$\Psi_{ed,N} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a,min}}}{1.5\mathrm{h'_{ef,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{100\ \mathrm{mm}}{1.5 \times 233.33\ \mathrm{mm}}\right)\right) = 0.78571$$

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

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

$$N_{cbg} = \left(\frac{150000\ \mathrm{mm}^2}{490000\ \mathrm{mm}^2}\right) \times 1 \times 0.78571 \times 1 \times 1 \times 162.08\ \mathrm{kN} = 38.985\ \mathrm{kN}$$

Calculate design concrete pryout capacity

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.6.2.2.1 $k_{cp} = 2$ - Factor for pryout strength

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

$$\phi V_{cpg} = \phi k_{cp} N_{cbg} = 0.65 \times 2 \times 38.985\ \mathrm{kN} = 50.681\ \mathrm{kN}$$

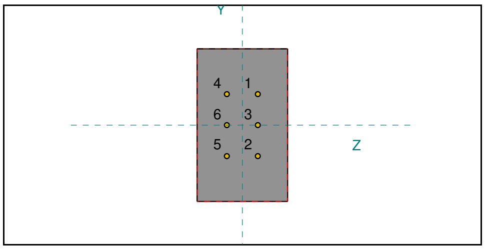
Result: **PASS = 0.06**

DCR - Demand-to-Capacity Ratio

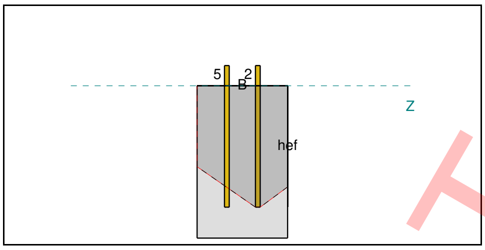
$$DCR = \frac{V_{ua}}{\phi V_{cpg}} = \frac{3 \text{ kN}}{50.681 \text{ kN}} = 0.059194$$

Go back to summary results table.

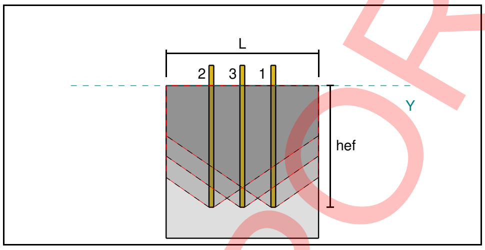
Check No. 12: Concrete Pryout Capacity (Vz Shear)



PLAN VIEW



FRONT VIEW



SIDE VIEW

Calculate total shear load on anchor group due to Vz shear

$V_z = 6 \text{ kN}$ - Vz shear load

$n_{a,vz} = 3$ - Number of anchors resisting +Vz shear

$n_{a,g1} = 3$ - Number of anchors in anchor group PO Vz 1

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

$$V_{ua} = \left(\frac{V_z}{n_{a,vz}}\right) n_{a,g1} = \left(\frac{6 \text{ kN}}{3}\right) \times 3 = 6 \text{ kN}$$

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 Vz 1 concrete edge distance (left)	200.0 mm
$c_{right,g1}$	Anchor group PO Vz 1 concrete edge distance (right)	100.0 mm
$c_{top,g1}$	Anchor group PO Vz 1 concrete edge distance (top)	150.0 mm
$c_{bottom,g1}$	Anchor group PO Vz 1 concrete edge distance (bottom)	150.0 mm
h_{ef}	Anchor rod effective embedment length	400.0 mm
$s_{sum,y,g1}$	Anchor group PO Vz 1 sum of spacing along Y-axis	200.0 mm
$s_{sum,z,g1}$	Anchor group PO Vz 1 sum of spacing along Z-axis	0.0 mm
$n_{y,g1}$	Number of anchors along Y-axis for anchor group PO Vz 1	3.0
$n_{z,g1}$	Number of anchors along Z-axis for anchor group PO Vz 1	1.0
	Narrow Member	TRUE
$h'_{ef,g1}$	Anchor group PO Vz 1 modified effective embedment length	133.3 mm

Calculation for critical anchor group: Anchor Group BO Vz 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

$$A_{Nco} = 9\left(h'_{ef,g1}\right)^2 = 9 \times (133.33 \text{ mm})^2 = 160000 \text{ mm}^2$$

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

$s_{sum,z,g1} = 0 \text{ mm}$ - Anchor group PO Vz 1 sum of spacing along Z-axis

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

ACI 318-19 Clause 17.6.2

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

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

$$L_{Nc} = \min(200\ \mathrm{mm},\ 1.5 \times 133.33\ \mathrm{mm}) + (\min(0\ \mathrm{mm},\ 3 \times 133.33\ \mathrm{mm} \times (1-1))) + \min(100\ \mathrm{mm},\ 1.5 \times 133.33\ \mathrm{mm})$$

$$L_{Nc} = 300\ \mathrm{mm}$$

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

$s_{sum,y,g1} = 200\ \mathrm{mm}$ - Anchor group PO Vz 1 sum of spacing along Y-axis

$n_{y,g1} = 3$ - Number of anchors along Y-axis for anchor group PO Vz 1

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

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

$$B_{Nc} = \min(150\ \mathrm{mm},\ 1.5 \times 133.33\ \mathrm{mm}) + (\min(200\ \mathrm{mm},\ 3 \times 133.33\ \mathrm{mm} \times (3-1))) + \min(150\ \mathrm{mm},\ 1.5 \times 133.33\ \mathrm{mm})$$

$$B_{Nc} = 500\ \mathrm{mm}$$

Calculate actual projected area of the anchor group

$n_{a,g1} = 3$ - Number of anchors in anchor group PO Vz 1

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

$$A_{Nc} = \min\left(\mathrm{n_{a,g1}A_{Nco}},\ \mathrm{L_{Nc}B_{Nc}}\right) = \min\left(3 \times 160000\ \mathrm{mm}^2,\ 300\ \mathrm{mm} \times 500\ \mathrm{mm}\right) = 150000\ \mathrm{mm}^2$$

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

$f_c^{'} = 20.68\ \mathrm{MPa}$ - Concrete compressive strength (20.68)

ACI 318-19 Clause 17.6.2.2.1 $k_c = 10$ - Factor for cast-in anchors breakout strength

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

$$N_b = k_c \lambda_a \sqrt{\frac{f_c^{'}}{MPa}} \left(\frac{h'_{ef,g1}}{mm}\right)^{1.5} N$$

$$N_b = 10 \times 1 \times \sqrt{\frac{20.68\ \mathrm{MPa}}{1\ \mathrm{MPa}}} \times \left(\frac{133.33\ \mathrm{mm}}{1\ \mathrm{mm}}\right)^{1.5} \times 0.001\ \mathrm{kN} = 70.014\ \mathrm{kN}$$

Calculate minimum edge distance of anchor

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

$$c_{a,min} = \min\left(\mathrm{c_{left,g1}},\ \mathrm{c_{right,g1}},\ \mathrm{c_{top,g1}},\ \mathrm{c_{bottom,g1}}\right) = \min(200\ \mathrm{mm},\ 100\ \mathrm{mm},\ 150\ \mathrm{mm},\ 150\ \mathrm{mm}) = 100\ \mathrm{mm}$$

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

$$\Psi_{ec,N} = \min\left(1.0,\ \frac{1}{1 + \frac{2e'_N}{3\mathrm{h'_{ef,g1}}}}\right) = \min\left(1,\ \frac{1}{1 + \frac{2 \times 0}{3 \times 133.33\ \mathrm{mm}}}\right) = 1$$

Calculate edge effect factor

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

$$\Psi_{ed,N} = \min\left(1.0,\ 0.7 + 0.3\left(\frac{\mathrm{c_{a,min}}}{1.5\mathrm{h'_{ef,g1}}}\right)\right) = \min\left(1,\ 0.7 + 0.3 \times \left(\frac{100\ \mathrm{mm}}{1.5 \times 133.33\ \mathrm{mm}}\right)\right) = 0.85$$

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

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

$$N_{cbg} = \left(\frac{150000\ \mathrm{mm}^2}{160000\ \mathrm{mm}^2}\right) \times 1 \times 0.85 \times 1 \times 1 \times 70.014\ \mathrm{kN} = 55.792\ \mathrm{kN}$$

Calculate design concrete pryout capacity

$\phi = 0.65$ - Concrete shear resistance factor

ACI 318-19 Clause 17.6.2.2.1 $k_{cp} = 2$ - Factor for pryout strength

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

$$\phi V_{cpg} = \phi k_{cp} N_{cbg} = 0.65 \times 2 \times 55.792\ \mathrm{kN} = 72.53\ \mathrm{kN}$$

Result: **PASS = 0.08**

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{V_{ua}}{\phi V_{cp}} = \frac{6\ \text{kN}}{72.53\ \text{kN}} = 0.082725$$

Go back to summary results table.

Check No. 13: Anchor Rod Shear Capacity

Calculate shear load on most critical anchor (corner anchor)

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

$n_{a,vy} = 2$ - Number of anchors resisting +Vy shear

$V_z = 6\ \text{kN}$ - Vz shear load

$n_{a,vz} = 3$ - Number of anchors resisting +Vz shear

V_{ua} - Required shear strength on anchor

$$V_{ua} = \sqrt{\left(\left(\frac{V_y}{n_{a,vy}}\right)^2\right) + \left(\left(\frac{V_z}{n_{a,vz}}\right)^2\right)}$$

$$V_{ua} = \sqrt{\left(\left(\frac{3\ \text{kN}}{2}\right)^2\right) + \left(\left(\frac{6\ \text{kN}}{3}\right)^2\right)} = 2.5\ \text{kN}$$

Calculations according to ACI provisions:

Calculate specified tensile strength of anchor

$F_{u\_anc} = 517.1\ \text{MPa}$ - Anchor rod tensile stress (F1554 Gr.55)

$F_{y\_anc} = 379.2\ \text{MPa}$ - Anchor rod yield stress (F1554 Gr.55)

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

f_{uta} - Specified tensile strength of anchor steel

$$f_{uta} = \min\left(0.75\text{F}_{\text{u\_anc}},\ 1.9\text{F}_{\text{y\_anc}},\ 860\right) = \min\left(0.75 \times 517.1\ \text{MPa},\ 1.9 \times 379.2\ \text{MPa},\ 860.00\ \text{MPa}\right) = 387.82\ \text{MPa}$$

Calculate effective cross-sectional area of anchor

$d_a = 16\ \text{mm}$ - Anchor rod diameter

ASME B1.1-2019 Table 1

$n_t = 0.433\ \text{mm}^{-1}$ - 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

$$A_{se,V} = \frac{\pi}{4}\left(d_a - \frac{0.9743}{n_t}\right)^2 = \frac{\pi}{4} \times \left(16\ \text{mm} - \frac{0.9743}{0.433\ \text{mm}^{-1}}\right)^2 = 148.49\ \text{mm}^2$$

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

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

$$\phi V_{sa,aci} = \phi 0.6 A_{se,V} f_{uta} = 0.65 \times 0.6 \times 148.49\ \text{mm}^2 \times 387.82\ \text{MPa} = 22.459\ \text{kN}$$

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)

$$F_{nv} = 0.45 F_{u\_anc} = 0.45 \times 517.1\ \text{MPa} = 232.69\ \text{MPa}$$

Calculate design shear strength of the anchor rod per AISC provisions

$A_{rod} = 201.06\ \text{mm}^2$ - 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

$$\phi R_{nv,aisc} = \phi F_{nv} A_{rod} = 0.75 \times 232.69\ \text{MPa} \times 201.06\ \text{mm}^2 = 35.09\ \text{kN}$$

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

$$\phi V_n = \min\left(\phi \text{V}_{\text{sa,aci}},\ \phi \text{R}_{\text{nv,aisc}}\right) = \min\left(22.459\ \text{kN},\ 35.09\ \text{kN}\right) = 22.459\ \text{kN}$$

Result:

PASS = 0.11

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{V_{ua}}{\phi V_n} = \frac{2.5\ \text{kN}}{22.459\ \text{kN}} = 0.11131$$

Go back to summary results table.

Check No. 14: Anchor Rod Capacity (AISC Tension and Shear)

Recall resultant shear load on anchor

V_{ua} - Required shear strength on anchor

$$V_{ua} = V_{ua} = 2.5\ \text{kN}$$

Calculate tensile stress on anchor due to direct tension

$A_{rod} = 201.06\ \text{mm}^2$ - Anchor rod area

f_{ut} - Required tensile stress

$$f_{ut} = \frac{N_{ua}}{A_{rod}} = \frac{5\ \text{kN}}{201.06\ \text{mm}^2} = 24.868\ \text{MPa}$$

Calculations for shear and direct tension according to AISC provisions:

Calculate nominal area of the anchor rod

$d_a = 16\ \mathrm{mm}$ - Anchor rod diameter

A_{rod} - Nominal area of anchor rod

$$A_{rod} = \frac{\pi}{4}(d_a)^2 = \frac{\pi}{4} \times (16\ \mathrm{mm})^2 = 201.06\ \mathrm{mm}^2$$

Calculate nominal shear stress of the anchor rod

$F_{u\_anc} = 517.1\ \mathrm{MPa}$ - Anchor rod tensile stress (F1554 Gr.55)

AISC 360-22 Table J3.2

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

$$F_{nv} = 0.45F_{u\_anc} = 0.45 \times 517.1\ \mathrm{MPa} = 232.69\ \mathrm{MPa}$$

Calculate nominal tensile stress of the anchor rod

AISC 360-22 Table J3.2

F_{nt} - Nominal tensile stress of anchor rod

$$F_{nt} = 0.75F_{u\_anc} = 0.75 \times 517.1\ \mathrm{MPa} = 387.82\ \mathrm{MPa}$$

Calculate nominal modified shear stress of the anchor rod to include effects of tensile stress

$f_{ut} = 24.868\ \mathrm{MPa}$

$\phi = 0.75$ - 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

$$F'_{nv} = \min\left(1.3\mathrm{F_{nv}} - \left(\frac{\mathrm{F_{nv}}}{\phi \mathrm{F_{nt}}}\right)\mathrm{f_{ut}},\ \mathrm{F_{nv}}\right)$$

$$F'_{nv} = \min\left(1.3 \times 232.69\ \mathrm{MPa} - \left(\frac{232.69\ \mathrm{MPa}}{0.75 \times 387.82\ \mathrm{MPa}}\right) \times 24.868\ \mathrm{MPa},\ 232.69\ \mathrm{MPa}\right) = 232.69\ \mathrm{MPa}$$

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

$$\phi R_{n,aisc} = \phi F'_{nv} A_{rod} = 0.75 \times 232.69\ \mathrm{MPa} \times 201.06\ \mathrm{mm}^2 = 35.09\ \mathrm{kN}$$

Result:

PASS = 0.07

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{V_{ua}}{\phi R_{n,aisc}} = \frac{2.5\ \mathrm{kN}}{35.09\ \mathrm{kN}} = 0.071246$$

Go back to summary results table.

Check No. 15: Interaction Check (ACI)

Ratios of Anchor Checks for Tension Load:

Check Name	Demand	Capacity	DCR
Anchor Rod Tensile Capacity	5.00	43.19	0.116
Concrete Breakout Capacity (Tension Load)	30.00	47.75	0.628
Anchor Pullout Capacity	5.00	422.40	0.012
Embed Plate Flexural Capacity	2.99	10.00	0.299
Concrete Side-Face Blowout Capacity (Y-Direction)	15.00	239.51	0.063
Concrete Side-Face Blowout Capacity (Z-Direction)	10.00	197.86	0.051
Anchor Rod Capacity (ACI Tension)	5.00	43.19	0.116

Ratios of Anchor Checks For Shear Load:

Check Name	Demand	Capacity	DCR
Concrete Breakout Capacity (Vy Shear)	3.00	10.97	0.273
Concrete Breakout Capacity (Vy Shear)	3.00	35.84	0.084
Concrete Breakout Capacity (Vz Shear)	6.00	17.92	0.335
Concrete Breakout Capacity (Vz Shear)	6.00	26.34	0.228
Concrete Pryout Capacity (Vy Shear)	3.00	50.68	0.059
Concrete Pryout Capacity (Vz Shear)	6.00	72.53	0.083
Anchor Rod Shear Capacity	2.50	22.46	0.111
Anchor Rod Capacity (ACI Shear)	2.50	22.46	0.111

Maximum interaction ratio

ACI 318-19 Eq. 17.8.3

$I_{max} = 1.2$

Calculate shear and axial interaction ratio

ACI 318-19 Eq. 17.8.3

I_{int} - Shear and axial interaction check

$$I_{int} = \frac{N_{ua}}{\phi N_n} + \frac{V_{ua}}{\phi V_n} = \frac{30}{47.749} + \frac{6}{17.921} = 0.96308$$

Result:

PASS = 0.80

DCR - Demand-to-Capacity Ratio

$$DCR = \frac{I_{int}}{I_{max}} = \frac{0.96308}{1.2} = 0.80257$$

Go back to summary results table.