

Job: CARLOS MAURICIO & FERNANDA ROCA

LOCATION: 13125 SW 63 AVE. PINCREST, FL 33156

GREEN WALL

STRUCTURAL CALCULATIONS

DESIGN CRITERIA:

Calculations based on:

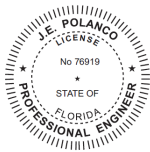
1. 2023 Florida Building Code (8TH Edition)
2. Minimum Design Loads for Buildings and Other Structures ASCE 7-22
3. Building Code Requirements for Structural Concrete ACI 318-15
4. American Institute of Steel Construction AISC-15 Ed
5. Specifications for the Design of Cold-Formed Stainless Steel Structural Members SEI/ASCE8-02

CALCULATION INDEX:

I	Cover page	1
II	Design & Analysis	2-25

CALCULATION STATEMENT:

To the best of my knowledge, ability, belief and professional judgment, I hereby attest that the manual calculations and computer generated calculations are in compliance with the existing governing codes.



Digitally signed by
Jose E Polanco
Date: 2026.07.30
16:51:29 -04'00'

Jose E. Polanco, PE
Lic. No. 76919

Wind Opening Design ($\varepsilon < 70\%$) ASCE 7-22:

$V := 175.00$ Wind Velocity (mph) as per FBC-23 (8th Edition)

$K_{zt} := 1.00$ Topographic Factor ASCE7-22

$K_d := 0.85$ Wind Directionality Factor ASCE 7-22

$G := 0.85$ Gust Factor (Rigid Structure) ASCE 7-22

$C_f := 1.6$

Net Force Coefficients C_f	(Flat-Sided Members)		(Rounded Members)	
			D $\sqrt{q_z} \leq 2.5$	D $\sqrt{q_z} > 2.5$
for $\varepsilon \leq 0.1$	-----	2.0	1.2	0.8
for 0.1 to 0.29	-----	1.8	1.3	0.9
for 0.3 to 0.7	-----	1.6	1.5	1.1

$\alpha := 9.5$

Terrain Exposure Constants α and z_g :
Exposure B----- $\alpha = 7.0$, z_g (ft) = 1200
Exposure C----- $\alpha = 9.5$, z_g (ft) = 900
Exposure D----- $\alpha = 11.5$, z_g (ft) = 700

$z_g := 900.0$

General Sign Data:

$Z := 22.0$ Height of Top of Sign (ft)

$\varepsilon := 50.0$ Solidity Ratio of Sign (%)

$Z := \text{if}(Z < 15, 15, Z)$

$K_z := 2.01 \left(\frac{Z}{z_g} \right)^{\frac{2}{\alpha}}$

$K_z = 0.92$

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2$

$q_z = 61.32$ psf

ASD Wind Loads:

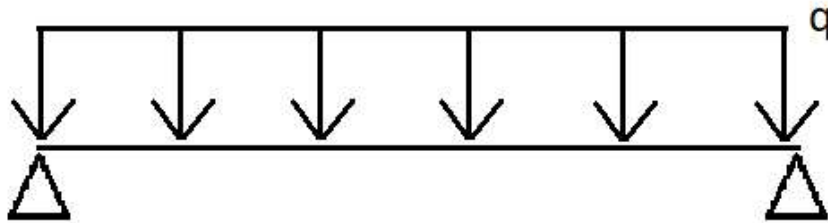
$p_z := 0.6 (q_z \cdot G \cdot C_f)$

$p_z = 50.04$ Gross Wind pressure (psf)

$p_{z,EQ} := \max \left(p_z \cdot \frac{\varepsilon}{100}, 10 \right)$

$p_{z,EQ} = 25.02$ Eq. Wind pressure (psf)

Hztal tube 3x6x1/8 Analysis & Design



Load Data:

DistXX_{Load} := 37.53 (lbs/ft) SpanLength := 5.0 (ft)

DistYY_{Load} := 0.00 (lbs/ft)

Section Data:

S_{XX} := 3.48 Section Modulus in³

S_{YY} := 2.36 Section Modulus in³

I_{XX} := 10.4 Modulus of Inertia in⁴

I_{YY} := 3.53 Modulus of Inertia in⁴

Area := 2.19 in²

Material Data:

F_b := 9500.0 (psi) Bending Stress

F_v := 5000.0 (psi) Shear Stress

E := 10100000.00 (psi) Modulus of Elasticity

Bending & Shear Analysis

$$M_{\text{DistXX.Load}} := \frac{(\text{DistXX}_{\text{Load}}) \cdot (\text{SpanLength})^2 \cdot 12}{8}$$

$$M_{\text{DistXX.Load}} = 1407.38 \quad \text{lbs} - \text{in}$$

$$V_{\text{Dist.XXLoad}} := \frac{(\text{DistXX}_{\text{Load}}) \cdot \text{SpanLength}}{2}$$

$$V_{\text{Dist.XXLoad}} = 93.825 \quad \text{lbs}$$

$$M_{\text{DistYY.Load}} := \frac{(\text{DistYY}_{\text{Load}}) \cdot (\text{SpanLength})^2 \cdot 12}{8}$$

$$M_{\text{DistYY.Load}} = 0.00 \quad \text{lbs} - \text{in}$$

$$V_{\text{Dist.YYLoad}} := \frac{(\text{DistYY}_{\text{Load}}) \cdot \text{SpanLength}}{2}$$

$$V_{\text{Dist.YYLoad}} = 0.000 \quad \text{lbs}$$

Check Bending:

$$f_{bXX} := \frac{(M_{DistXX.Load})}{S_{XX}}$$

$$f_{bXX} = 404.418 \quad \text{psi}$$

$$f_{bYY} := \frac{M_{DistYY.Load}}{S_{YY}}$$

$$f_{bYY} = 0.000 \quad \text{psi}$$

$$BENDING_{xx} := \text{if}(f_{bXX} < F_b, "OK", "N.G")$$

$$BENDING_{xx} = "OK"$$

$$BENDING_{yy} := \text{if}(f_{bYY} < F_b, "OK", "N.G")$$

$$BENDING_{yy} = "OK"$$

$$BENDING_{xxyy} := \frac{(f_{bXX})}{F_b} + \frac{(f_{bYY})}{F_b}$$

$$BENDING_{xxyy} = 0.043$$

$$BENDING_{combined} := \text{if}(BENDING_{xxyy} > 1, "N.G", "OK")$$

$$BENDING_{combined} = "OK"$$

Check Shear:

$$f_{vXX} := \frac{3 \cdot (V_{Dist.XXLoad})}{2 \cdot Area}$$

$$f_{vXX} = 64.264 \quad \text{psi}$$

$$f_{vYY} := \frac{3 \cdot (V_{Dist.YYLoad})}{2 \cdot Area}$$

$$f_{vYY} = 0.000 \quad \text{psi}$$

$$SHEAR_{xx} := \text{if}(f_{vXX} < F_v, "OK", "N.G")$$

$$SHEAR_{xx} = "OK"$$

$$SHEAR_{yy} := \text{if}(f_{vYY} < F_v, "OK", "N.G")$$

$$SHEAR_{yy} = "OK"$$

$$SHEAR_{xxyy} := \frac{(f_{vXX})}{F_v} + \frac{(f_{vYY})}{F_v}$$

$$SHEAR_{xxyy} = 0.013$$

$$SHEAR_{combined} := \text{if}(SHEAR_{xxyy} > 1, "N.G", "OK")$$

$$SHEAR_{combined} = "OK"$$

Check Deflection:

$$\Delta_{allow} := \frac{Span_{Length} \cdot 12}{360}$$

$$\Delta_{allow} = 0.167 \quad \text{in}$$

$$\Delta_{max} := \left[\frac{5 \left(\frac{DistXX_{Load}}{12} \right) \cdot (Span_{Length} \cdot 12)^4}{384 \cdot E \cdot I_{XX}} \right]$$

$$\Delta_{max} = 0.005 \quad \text{in}$$

$$DEFLECTION := \text{if}(\Delta_{max} < \Delta_{allow}, "OK", "NG")$$

$$DEFLECTION = "OK"$$

FRAME RT3x6x1/4 ANALYSIS

Load Data:

$$\text{TotalLoad} := 25.02 \quad (\text{psf})$$

$$\text{PointLoad} := 0.0 \quad (\text{lbs})$$

$$w_{\text{dist}} := (\text{TotalLoad} \cdot \text{TributaryWidth}) \quad w_{\text{dist}} = 550.440 \quad \frac{\text{lbs}}{\text{ft}}$$

$$M_{\text{Dist.Load}} := \frac{(w_{\text{dist}}) \cdot (\text{SpanLength})^2 \cdot 12}{8}$$

$$M_{\text{Dist.Load}} = 20641.500 \text{ lbs} - \text{in}$$

$$V_{\text{Dist.Load}} := \frac{(w_{\text{dist}}) \cdot \text{SpanLength}}{2}$$

$$V_{\text{Dist.Load}} = 1376.10 \text{ lbs}$$

$$\text{TributaryWidth} := 22.0 \quad (\text{ft})$$

$$\text{SpanLength} := 5.0 \quad (\text{ft})$$

$$M_{\text{Point.load}} := \frac{(\text{PointLoad}) \cdot (\text{SpanLength}) \cdot 12}{4}$$

$$M_{\text{Point.load}} = 0.000 \quad \text{lbs} - \text{in}$$

$$V_{\text{Point.Load}} := \frac{(\text{PointLoad})}{2}$$

$$V_{\text{Point.Load}} = 0.000 \quad \text{lbs}$$

Material Data:

$$F_b := 19000.00 \quad \text{Bending Stress (psi)}$$

$$F_v := 5000.0 \quad \text{Shear Stress (psi)}$$

$$E := 10100000.00 \quad \text{Modulus of Elasticity (psi)}$$

Section Data:

$$S_{\text{total}} := 6.44 \quad \text{Section Modulus in}^3$$

$$I_{\text{total}} := 19.33 \quad \text{Modulus of Inertia in}^4$$

$$\text{Area} := 4.25 \quad \text{in}^2$$

Check Bending:

$$f_b := \frac{\max(M_{\text{Dist.Load}}, M_{\text{Point.load}})}{S_{\text{total}}}$$

$$f_b = 3205.202 \quad \text{psi}$$

$$\text{BENDING} := \text{if}(f_b < F_b, \text{"OK"}, \text{"NG"})$$

BENDING = "OK"

Check Shear:

$$f_v := \frac{3 \cdot \max(V_{\text{Dist.Load}}, V_{\text{Point.Load}})}{2 \cdot \text{Area}}$$

$$f_v = 485.682 \quad \text{psi}$$

$$\text{SHEAR}_{\text{web}} := \text{if}(f_v < F_v, \text{"OK"}, \text{"NG"})$$

SHEAR_{web} = "OK"

Check Deflection:

$$\Delta_{\text{allow}} := \frac{\text{SpanLength} \cdot 12}{360}$$

$$\Delta_{\text{max}} := \max \left[\frac{5 \left(\frac{w_{\text{dist}}}{12} \right) \cdot (\text{SpanLength} \cdot 12)^4}{384 \cdot E \cdot I_{\text{total}}}, \frac{\text{PointLoad} (\text{SpanLength} \cdot 12)^3}{48 \cdot E \cdot I_{\text{total}}} \right]$$

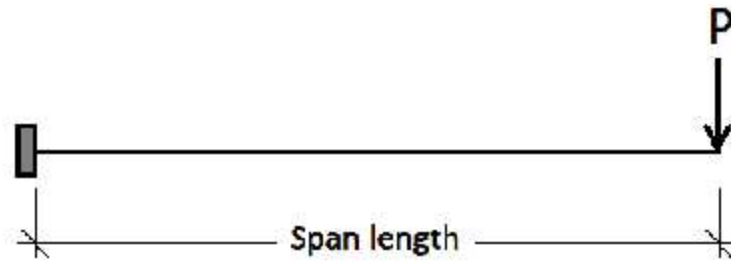
$$\Delta_{\text{allow}} = 0.167 \quad \text{in}$$

$$\Delta_{\text{max}} = 0.040 \quad \text{in}$$

$$\text{DEFLECTION} := \text{if}(\Delta_{\text{max}} < \Delta_{\text{allow}}, \text{"OK"}, \text{"NG"})$$

DEFLECTION = "OK"

Alum. Beams RT3x6x1/4 Analysis & Design



Load Data:

$P_{y.Live} := 688.05$ lbs

$P_{x.Live} := 0.00$ lbs

$Span_{Length} := 3.50$ (ft)

Section Data:

$S_{XX} := 6.44$ Section Modulus in³

$S_{YY} := 4.22$ Section Modulus in³

$I_{XX} := 19.33$ Modulus of Inertia in⁴

$I_{YY} := 6.33$ Modulus of Inertia in⁴

Area := 4.25 in²

Material Data:

$F_b := 9500.0$ (psi) Bending Stress

$F_v := 5000.00$ (psi) Shear Stress

$E := 10100000.00$ (psi) Modulus of Elasticity

Bending & Shear Analysis

$M_{YY.Load.Beam} := P_{y.Live} \cdot Span_{Length} \cdot 12$

$M_{YY.Load.Beam} = 28898.100$ Lbs-in

$V_{Dist.YYLoad} := P_{y.Live}$

$V_{Dist.YYLoad} = 688.050$ Lbs

$M_{XX.Load.Beam} := P_{x.Live} \cdot Span_{Length} \cdot 12$

$M_{XX.Load.Beam} = 0.000$ Lbs-in

$V_{Dist.XXLoad} := P_{x.Live}$

$V_{Dist.XXLoad} = 0.000$ Lbs

Check Bending:

$$f_{bXX} := \frac{M_{XX.Load.Beam}}{S_{XX}}$$

$$f_{bXX} = 0.000 \quad \text{psi}$$

$$f_{bYY} := \frac{M_{YY.Load.Beam}}{S_{YY}}$$

$$f_{bYY} = 6847.891 \quad \text{psi}$$

$$BENDING_{xx} := \text{if}(f_{bXX} < F_b, "OK", "N.G")$$

$$BENDING_{xx} = "OK"$$

$$BENDING_{yy} := \text{if}(f_{bYY} < F_b, "OK", "N.G")$$

$$BENDING_{yy} = "OK"$$

$$BENDING_{xxyy} := \frac{(f_{bXX})}{F_b} + \frac{(f_{bYY})}{F_b}$$

$$BENDING_{xxyy} = 0.721$$

$$BENDING_{combined} := \text{if}(BENDING_{xxyy} > 1, "N.G", "OK")$$

$$BENDING_{combined} = "OK"$$

Check Shear:

$$f_{vXX} := \frac{3 \cdot (V_{Dist.XXLoad})}{2 \cdot Area}$$

$$f_{vXX} = 0.000 \quad \text{psi}$$

$$f_{vYY} := \frac{3 \cdot (V_{Dist.YYLoad})}{2 \cdot Area}$$

$$f_{vYY} = 242.841 \quad \text{psi}$$

$$SHEAR_{xx} := \text{if}(f_{vXX} < F_v, "OK", "N.G")$$

$$SHEAR_{xx} = "OK"$$

$$SHEAR_{yy} := \text{if}(f_{vYY} < F_v, "OK", "N.G")$$

$$SHEAR_{yy} = "OK"$$

$$SHEAR_{xxyy} := \frac{(f_{vXX})}{F_v} + \frac{(f_{vYY})}{F_v}$$

$$SHEAR_{xxyy} = 0.049$$

$$SHEAR_{combined} := \text{if}(SHEAR_{xxyy} > 1, "N.G", "OK")$$

$$SHEAR_{combined} = "OK"$$

Check Deflection:

$$\Delta_{allow} := \frac{Span_{Length} \cdot 12}{360}$$

$$\Delta_{allow} = 0.117 \quad \text{in}$$

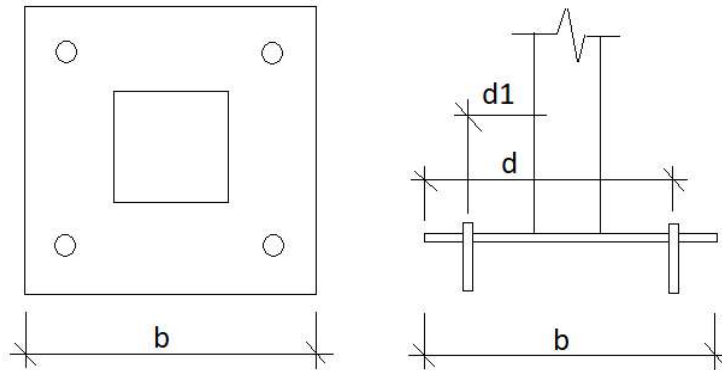
$$\Delta_{max} := \left(\frac{P_{y.Live} \cdot Span_{Length}^3}{3 \cdot E \cdot I_{YY}} \right)$$

$$\Delta_{max} = 0.000 \quad \text{in}$$

$$DEFLECTION := \text{if}(\Delta_{max} < \Delta_{allow}, "OK", "NG")$$

$$DEFLECTION = "OK"$$

Alum. Plate Analysis & Design



Material Data:

$$F_{b_{plate}} := 11800.0 \quad \text{psi}$$

Load Data:

$$M_{Post} := 20641.5 \quad \text{Lbs-in}$$

$$d := 5.0 \quad \text{in}$$

$$b := 6.0 \quad \text{in}$$

$$d_1 := 0.5 \quad \text{in}$$

$$T_{Bolt} := \frac{M_{Post}}{0.9 \cdot d} \quad T_{Bolt} = 4.587 \times 10^3 \quad \text{Lbs}$$

$$M_{Plate} := T_{Bolt} \cdot d_1 \quad M_{Plate} = 2.293 \times 10^3 \text{ Lbs-in}$$

$$t_{plate} := \sqrt{\frac{4 \cdot M_{Plate}}{F_{b_{plate}} \cdot b}} \quad t_{plate} = 0.36 \quad \text{in}$$

Using Alum. Plate PL3/8"x6"x6"

1. Project Information

Company:
Project Engineer: -

Address: -

Phone: -

Email: -

Project Name: -

Project Address: -

Notes:

2. Selected Anchor Information

Selected Anchor : KH-EZ

Brand: Hilti®

Material: 3/8" Ø Medium Carbon Steel

Embedment: h_{ef} 2.50 in h_{nom} 3.25 in

Approval: ICC-ES ESR-3027

Issued/Revision: Dec,2025 -

Drill method: Hammer Drilled


3. Design Principles

Design Method: ACI 318-19

Load Combinations: Section 5.3 User Defined Loads

4. Base Material Information

Concrete:

Type: Uncracked Normal Weight Concrete
Strength 4000 psi

Reinforcement:

Supplemental Reinforcement False
Spacing Tension No (Condition B) Shear No (Condition B)
Controls Breakout Tension False Shear False

Base Plate:

Sizing Thickness 0.25 in Length 6 in Width 6 in
Standoff None Height 0 in
Strength 36000 psi

Profile: HSS Rectangular 3 X 3 X 0.250

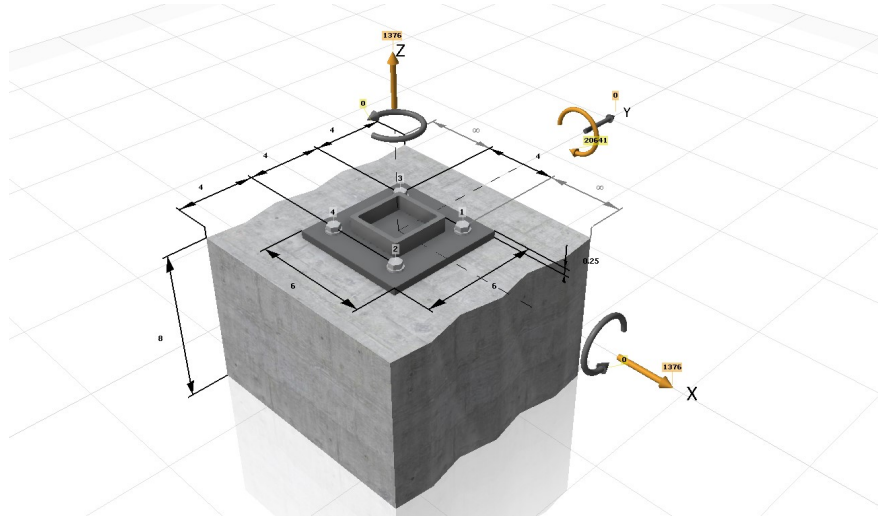
Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

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5. Geometric Conditions



h_{slab}	8	in	h_{min}	4.750	in
Edge Cx-	∞	in	c_{min}	1.500	in
Edge Cx+	∞	in	c_{ac}	3.750	in
Edge Cy-	4	in	s_{min}	2.250	in
Edge Cy+	4	in			

6. Summary Results

Tension Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength:	2615.30	6718.00	0.389	OK	
Concrete Breakout Strength:	5231.00	5980.00	0.875	OK	Controls

Shear Loading

Design Proof	Demand(lb)	Capacity(lb)	Utilization	Status	Critical
Steel Strength	344.00	3111.00	0.111	OK	Controls
Concrete Breakout Strength:	688.00	8283.00	0.083	OK	
Pryout Strength	1376.00	19749.00	0.070	OK	

Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

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7. Warnings and Remarks

ANCHOR DESIGN CRITERIA IS SATISFIED

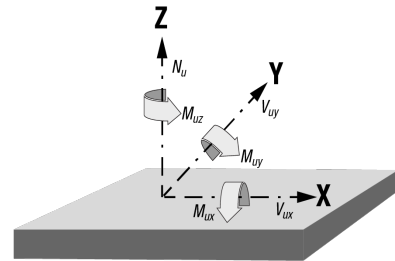


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8. Load Condition

Design Loads / Actions

Nu	1376.0	lb	Vux	1376.0	lb	Vuy	0.0	lb
Muz	0.0	in-lb	Mux	0.0	in-lb	Muy	20641.	in-lb
Consider Load Reversal			X Direction			Y Direction		
			0%			0%		



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9. Load Distribution

Max. concrete compressive strain:	0.274	%	<u>Anchor Eccentricity</u>					
Max. concrete compressive stress:	1192.859	psi	ex	0	in	ey	0	in
Resulting tension force:	5230.593	lb	<u>Profile Eccentricity</u>					
Resulting compression force:	3854.593	lb	ex	0	in	ey	0	in

Resulting anchor forces / Load distribution

Anchor	Tension Load (lb)	Shear Load (lb)	Component Shear Load (lb)		Anchor Coordinates (in)	
			Shear X	Shear Y	X	Y
1	0.00	344.0	344.0	0.0	2.000	2.000
2	0.00	344.0	344.0	0.0	2.000	-2.000
3	2615.30	344.0	344.0	0.0	-2.000	2.000
4	2615.30	344.0	344.0	0.0	-2.000	-2.000

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10.Design Proof Tension Loading

Steel Strength

ACI 318-19 17.6.1

Variables

N_{sa} (lb)	ϕ
10335.000	0.65

Results

ϕN_{sa}	=	6718.0	lb
N_{ua}	=	2615.3	lb
Utilization	=	38.9%	



Table 17.5.2

Concrete Breakout Strength

ACI 318-19 17.6.2

Equations

$$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$$

Eqn. 17.6.2.1b

$$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$$

Eqn. 17.6.2.2.1

Variables

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
86.250	56.250	1.000	1.000	1.000	1.000
c_{ac} (in)	k_c	λ_a	f'_c (psi)	h_{ef} (in)	C_{amin} (in)
3.750	24.000	1.000	4000	2.50	4
N_b (lb)	ϕ				
6000.000	0.65				

Results

ϕN_{cbg}	=	5980	lb
N_{ua}	=	5231.0	lb
Utilization	=	87.5%	

Table 17.5.2



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Reference

A diagram showing a yellow pile foundation embedded in grey soil. A horizontal arrow labeled V points to the right, indicating a lateral load applied to the pile head. The soil is represented by a grey area with small black triangles, and a shadow is cast by the pile into the soil.

Table 17.5.2

V _{sa} (lb)	φ
5185.00	0.60

ϕ_{sa}^V	=	3111	lb
V_{ua}	=	344	lb
Utilization	=	11.1%	

A diagram showing a pile foundation embedded in soil. A horizontal load V is applied to the pile head, pointing to the right. The soil is represented by a grey area with small black triangles indicating internal friction or resistance.

Eqn. 17.7.2.1b

$$V_{cbg} = (A_{Vc} / A_{Vc0}) \cdot \psi_{ec,V} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot \psi_{h,v} \cdot V_b$$

$$V_b = (7 \cdot (l_e / d_a)^{0.2} \cdot (d_a)^{0.5}) \cdot \lambda_a \cdot (f_c)^{0.5} \cdot (c_{al})^{1.5}$$

Eqn.
17.7.2.2.1a

A_{Vc} (in ²)	A_{Vc0} (in ²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$
96.000	72.000	1.000	1.000	1.400	1.000
$\Psi_{a,V}$	l_e (in)	d_a (in)	λ_a	f_c (psi)	c_{al} (in)
2.000	2.500	0.375	1.000	4000.000	4.000
V_b (lb)	φ			V_{cbg} (lb)	
3169.680	0.70			11833.472	

ϕV_{cbg}	=	8283	lb
Direction	=	Y+	
V_{ua}	=	688	lb
Utilization	=	8.3%	

Table 17.5.2

Shear Breakout Case 1 Controls



6/17/2026

Pryout Strength

ACI 318-19 17.7.3

Equations

$$V_{cpg} = k_{cp} \cdot N_{cpg}$$

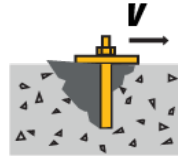
Eqn. 17.7.3.1b

$$N_{cbg} = (A_{Nc} / A_{Nc0}) \cdot \Psi_{ec,N} \cdot \Psi_{ed,N} \cdot \Psi_{c,N} \cdot \Psi_{cp,N} \cdot N_b$$

Eqn. 17.6.2.1b

$$N_b = k_c \cdot \lambda_a \cdot (f'_c)^{0.5} \cdot h_{ef}^{1.5}$$

Eqn. 17.6.2.2.1



Variables

A_{Nc} (in ²)	A_{Nc0} (in ²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$
132.250	56.250	1.000	1.000	1.000	1.000
c_{ac} (in)	k_c	λ_a	h_{ef} (in)	f'_c (psi)	C_{amin} (in)
3.750	24.000	1.000	2.50	4000.000	4.000
N_b (lb)	k_{cp}	N_{cpg} (lb)	ϕ		
6000.000	2.000	14106.667	0.70		

Results

ϕV_{cpg}	=	19749.00 lb
V_{ua}	=	1376 lb
Utilization	=	7.0%

Table 17.5.2

Input data and results must be checked for agreement with the existing conditions, the standards and guidelines and must be checked for plausibility

12. Interaction of Tension and Shear Loads

Reference

ACI 318-19 17.8

Equations

$$(N_{ua} / \phi \cdot N_n)^{5/3} + (V_{ua} / \phi \cdot V_n)^{5/3} \leq 1.0$$

Eqn. 17.8.3

Variables

$N_{ua} / \phi \cdot N_n$	$V_{ua} / \phi \cdot V_n$
0.875	0.111

Results

0.825	≤	1.0
Status	:	OK

ANCHOR DESIGN CRITERIA IS SATISFIED



Steel Bolts Threaded Tension & Shear Design 14th

Data:

$T_{\text{bolt}} := 0.688$	Tension in Anchor (kips)	$\Omega := 2.00$	ASD Factor=2.00
$V_{\text{bolt}} := 0.688$	Shear in Anchor (kips)		
$F_{\text{nv}} := 45.0$	A325 Nominal Shear Strength (ksi) Table J3.2 AISC 16.1-1200.45*Fu		
$F_{\text{nt}} := 90.0$	A325 Nominal Tensile Strength (ksi) Table J3.2 AISC 16.1-120.....=0.75*Fu		
$D := 0.375$	Screw Diameter (in)	$n := 10.00$	Threads per in..... (AISC-Table 7-17 page 7-81)

$$F_v := \frac{F_{\text{nv}}}{\Omega} \quad F_v = 22.500 \quad \text{ksi}$$

$$F_t := \frac{F_{\text{nt}}}{\Omega} \quad F_t = 45.000 \quad \text{ksi}$$

$$A_{\text{bolt}} := 0.7854 \cdot \left(D - \frac{0.9743}{n} \right)^2 \quad A_{\text{bolt}} = 0.061 \quad \text{in}^2$$

Check Shear

$$f_v := \frac{V_{\text{bolt}}}{A_{\text{bolt}}} \quad f_v = 11.370 \quad \text{ksi}$$

$$V_{\text{allow}} := F_v \cdot A_{\text{bolt}} \quad V_{\text{allow}} = 1.362 \quad \text{kips}$$

$$\text{SHEAR} := \text{if}(f_v \leq F_v, \text{"OK"}, \text{"NG"}) \quad \boxed{\text{SHEAR} = \text{"OK"}}$$

Check Tension

$$f_t := \frac{T_{\text{bolt}}}{A_{\text{bolt}}} \quad f_t = 11.370 \quad \text{ksi}$$

$$F'_{\text{nt}} := \min \left(F_t, \max \left(0, 1.3F_t - \Omega \cdot \frac{F_t}{F_v} \cdot f_v \right) \right) \quad F'_{\text{nt}} = 13.021 \quad \text{ksi}$$

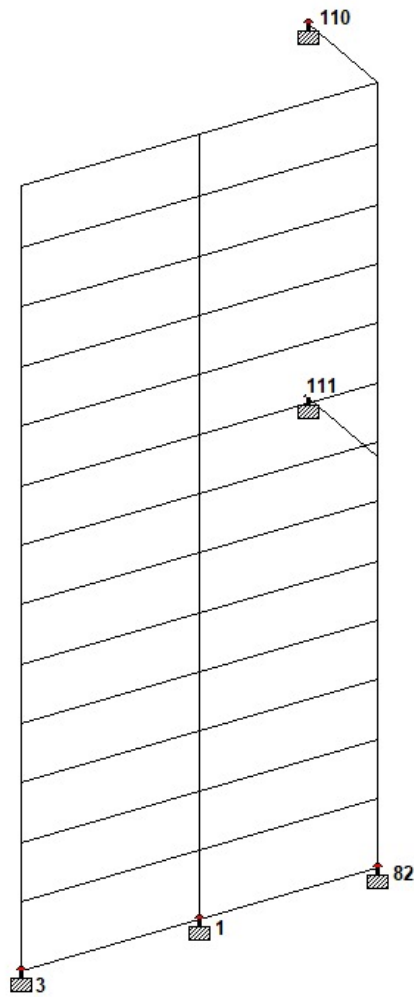
$$T_{\text{allow}} := F'_{\text{nt}} \cdot A_{\text{bolt}} \quad T_{\text{allow}} = 0.788 \quad \text{kips}$$

$$\text{TENSION} := \text{if}(T_{\text{bolt}} \leq T_{\text{allow}}, \text{"OK"}, \text{"NG"}) \quad \boxed{\text{TENSION} = \text{"OK"}}$$

Minimum Thread Length

$$TL_{\text{min}} := \frac{2 \cdot T_{\text{bolt}}}{F_v \cdot \pi \cdot \left(D - \frac{0.9743}{n} \right)} \quad TL_{\text{min}} = 0.070 \quad \text{in}$$

Reactions



Nodes 3

REACCIONES JARDINERA3.std - Support Reactions: 3								
All Summary Envelope								
	Node	L/C	Horizontal Fx lb	Vertical Fy lb	Horizontal Fz lb	Moment		
						Mx lb-ft	My lb-ft	Mz lb-ft
Max Fx	3	1 DEAD LOAD	2.362	239.807	-0.007	-0.015	-0.008	9.069
Min Fx	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Max Fy	3	1 DEAD LOAD	2.362	239.807	-0.007	-0.015	-0.008	9.069
Min Fy	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Max Fz	3	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min Fz	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Max Mx	3	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min Mx	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Max My	3	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min My	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Max Mz	3	6 VIENTO EN Z (UPLIFT)	-227.226	-1527.858	-378.467	-1511.964	-151.786	415.657
Min Mz	3	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000

Node 1

REACCIONES JARDINERA3.std - Support Reactions: 1								
All Summary Envelope								
	Node	L/C	Horizontal Fx lb	Vertical Fy lb	Horizontal Fz lb	Moment		
						Mx lb-ft	My lb-ft	Mz lb-ft
Max Fx	1	1 DEAD LOAD	0.009	392.612	0.043	0.197	-0.064	0.005
Min Fx	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Max Fy	1	1 DEAD LOAD	0.009	392.612	0.043	0.197	-0.064	0.005
Min Fy	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Max Fz	1	1 DEAD LOAD	0.009	392.612	0.043	0.197	-0.064	0.005
Min Fz	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Max Mx	1	1 DEAD LOAD	0.009	392.612	0.043	0.197	-0.064	0.005
Min Mx	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Max My	1	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min My	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Max Mz	1	6 VIENTO EN Z (UPLIFT)	-367.278	-406.683	-599.507	-3205.434	-672.158	651.655
Min Mz	1	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000

Nodos 82

REACCIONES JARDINERA3.std - Support Reactions: 82								
All Summary Envelope								
	Node	L/C	Horizontal Fx lb	Vertical Fy lb	Horizontal Fz lb	Moment		
						Mx lb-ft	My lb-ft	Mz lb-ft
Max Fx	82	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min Fx	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Max Fy	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Min Fy	82	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Max Fz	82	4 WX WIND X (UPLIFT)	-146.001	201.312	5.191	10.134	-8.205	245.176
Min Fz	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Max Mx	82	4 WX WIND X (UPLIFT)	-146.001	201.312	5.191	10.134	-8.205	245.176
Min Mx	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Max My	82	2 LIVE LOAD	0.000	0.000	0.000	0.000	0.000	0.000
Min My	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Max Mz	82	6 VIENTO EN Z (UPLIFT)	-218.540	1061.896	-2.303	-19.250	-177.588	410.704
Min Mz	82	1 DEAD LOAD	-2.351	243.755	0.056	0.201	-0.035	-9.064

Foundation Design

Data:

$h_{\text{post}} := 22.0$ Pole Height (ft)
 $M_{\text{base}} := 3205.434$ Base Moment (lbs*ft)

Soil Data:

$S := 200.0$ Soil allowable lateral pressure (psf/ft)
 $I_F := 2.00$ Isolated Poles Short-Term Loads Increase Factor=2 FBC 1819.6.1
 $b := 3.0$ Diameter of round post or diagonal dimension of square post or footing (ft)

Then:

$h := h_{\text{post}}$ $h = 22.00$ ft
 $P := \frac{M_{\text{base}}}{h_{\text{post}}}$ $P = 145.70$ lbs

Nonconstrained Depth of Embedment:

$d :=$
 $x \leftarrow 0.00001$
 $\text{while } 0.5 \cdot \frac{2.34 \cdot P}{\left(\frac{x \cdot S \cdot I_F}{3}\right) \cdot b} \cdot \left[1 + \left[1 + 4.36 \cdot \frac{h}{\frac{2.34 \cdot P}{\left(\frac{x \cdot S \cdot I_F}{3}\right) \cdot b}} \right]^{\frac{1}{2}} \right] > x$
 $x \leftarrow x + 0.1$
 $d = 2.90$ ft

$S_1 := \frac{d \cdot S \cdot I_F}{3}$ $S_1 = 386.67$ psf

Constrained Depth of Embedment:

$d_1 :=$
 $x \leftarrow 0.01$
 $\text{while } \sqrt{4.25 \cdot \frac{P \cdot h}{d \cdot S \cdot I_F \cdot b}} > x$
 $x \leftarrow x + 0.1$
 $d_1 = 2.01$ ft

$S_3 := d \cdot S \cdot I_F$ $S_3 = 1160.00$ psf

Structural Plain Concrete Foundation

Foundation Concrete Data:

$$f_c := 3000.00 \quad \text{Concrete Compressive Strength (psi)}$$

$$M_u := 1.7 \cdot M_{\text{base}}$$

$$M_u = 5449.24 \quad \text{lbs} - \text{ft}$$

For Circular Foundation:

$$S_{m.C} := \frac{\pi \cdot b^3}{32}$$

$$S_{m.C} = 2.65 \quad \text{ft}^3$$

$$M_{n.C} := 5 \cdot S_{m.C} \cdot \sqrt{f_c} \cdot 144$$

$$M_{n.C} = 104533.81 \quad \text{lbs} - \text{ft}$$

$$\text{FLEXURE}_C := \text{if}(M_u \leq 0.85 \cdot M_{n.C}, \text{"OK"}, \text{"N.G."})$$

$$\text{FLEXURE}_C = \text{"OK"}$$

For Square Foundation:

$$S_{m.S} := \frac{b^3}{6}$$

$$S_{m.S} = 4.50 \quad \text{ft}^3$$

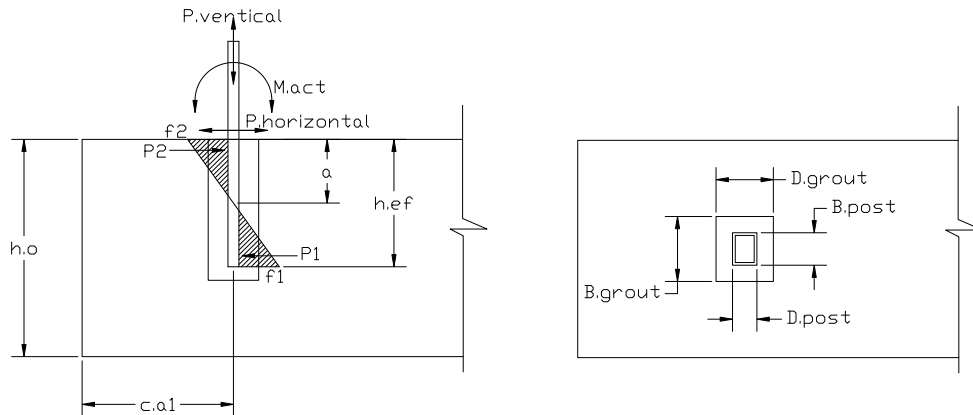
$$M_{n.S} := 5 \cdot S_{m.S} \cdot \sqrt{f_c} \cdot 144$$

$$M_{n.S} = 177462.11 \quad \text{lbs} - \text{ft}$$

$$\text{FLEXURE}_S := \text{if}(M_u \leq 0.85 \cdot M_{n.S}, \text{"OK"}, \text{"N.G."})$$

$$\text{FLEXURE}_S = \text{"OK"}$$

Moment Connection:



Load Data:

$P_{\text{horizontal}} := 145.70$ Horizontal Load (lbs)

$P_{\text{vertical}} := 0.00$ Vertical Load (lbs)

$M_{\text{act}} := 38465.208$ Moment (#-in)

Post Data:

$B_{\text{post}} := 3.0$ Post Width (in)

$D_{\text{post}} := 6.0$ Post Depth (in)

$h_{\text{ef}} := 30.0$ Post Embedment (in)

$C_{a1} := 9.0$ Minimum Edge Distance (in)

Grout Size Data:

$B_{\text{grout}} := 3.0$ Equivalent Grout Width (in)

$D_{\text{grout}} := 3.0$ Equivalent Grout Depth (in)

$f_{c_{\text{grout}}} := 3000.00$ Grout Strength (psi)

Concrete Slab Data:

$f_{c_{\text{concrete}}} := 3000.00$ Concrete Strength (psi)

$d_o := 36.0$ Hold diameter (Grout Diameter) (in)

$h_a := 36.0$ Concrete Slab Deep (in)

Arm Effect:

$$a := \frac{2 \cdot h_{ef}^2 + 3 \cdot \left(\frac{M_{act}}{P_{horizontal}} \cdot h_{ef} \right)}{3 \cdot h_{ef} + 6 \cdot \frac{M_{act}}{P_{horizontal}}}$$

$$a = 15.27 \quad \text{in}$$

$$P_2 := \frac{P_{horizontal} \cdot a^2}{h_{ef} \cdot (2 \cdot a - h_{ef})}$$

$$P_2 = 2106.04 \quad \text{lbs}$$

$$P_1 := P_2 - P_{horizontal}$$

$$P_1 = 1960.34 \quad \text{lbs}$$

$$M_{external} := M_{act} + (P_{horizontal} \cdot a)$$

$$M_{external} = 40689.87 \quad \text{lbs} - \text{in}$$

$$M_{internal} := P_2 \cdot \frac{2}{3} \cdot a + P_1 \cdot \frac{2}{3} \cdot (h_{ef} - a)$$

$$M_{internal} = 40689.87 \quad \text{lbs} - \text{in}$$

$$Vu_{horizontal} := 1.6 \cdot (\max(P_1, P_2))$$

$$Vu_{horizontal} = 3369.66 \quad \text{lbs}$$

$$Vu_{vertical} := 1.6 \cdot P_{vertical}$$

$$Vu_{vertical} = 0.00 \quad \text{lbs}$$

Stress Analysis Between Post and Grout:

$$f_{2,grout} := 1.6 \cdot \frac{2 \cdot P_2}{a \cdot B_{post}}$$

$$f_{2,grout} = 147.13 \quad \text{psi}$$

$$f_{1,grout} := 1.6 \cdot \frac{2 \cdot P_1}{(h_{ef} - a) \cdot B_{post}}$$

$$f_{1,grout} = 141.95 \quad \text{psi}$$

Bearing Stress for Grout :

$$Fp_{grout} := [0.65 \cdot (0.85 f_{c,grout})] \cdot 1.5$$

$$Fp_{grout} = 2486.25 \quad \text{psi}$$

$$\text{Stress}_{grout} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} \quad \text{if } \max(f_{1,grout}, f_{2,grout}) \leq Fp_{grout} \end{cases}$$

$$\text{Stress}_{grout} = \text{"O.K."}$$

Stress Analysis Between Grout and Concrete:

$$f_{2,concrete} := 1.6 \cdot \frac{2 \cdot P_2}{a \cdot B_{grout}}$$

$$f_{2,concrete} = 147.13 \quad \text{psi}$$

$$f_{1,concrete} := 1.6 \cdot \frac{2 \cdot P_1}{(h_{ef} - a) \cdot B_{grout}}$$

$$f_{1,concrete} = 141.95 \quad \text{psi}$$

Bearing Stress for Concrete :

$$F_{p_{\text{concrete}}} := \left[0.65 \cdot (0.85 \cdot f_{c_{\text{concrete}}}) \right] \cdot 1.5$$

$$F_{p_{\text{concrete}}} = 2486.25 \quad \text{psi}$$

$$\text{Stress}_{\text{concrete}} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} & \text{if } \max(f_{1,\text{concrete}}, f_{2,\text{concrete}}) \leq F_{p_{\text{concrete}}} \end{cases}$$

$$\text{Stress}_{\text{concrete}} = \text{"O.K."}$$

Shear Analysis-Concrete Breakout Strength:

$$A_{V_{co}} := 4.5 \cdot C_{a1}^2$$

$$A_{V_{co}} = 364.5 \quad \text{in}^2$$

$$A_{V_c} := \begin{cases} 2 \cdot (1.5 \cdot C_{a1}) \cdot h_a & \text{if } h_a < 1.5 \cdot C_{a1} \\ 2 \cdot (1.5 \cdot C_{a1}) \cdot (1.5 \cdot C_{a1}) & \text{if } h_a \geq 1.5 \cdot C_{a1} \end{cases}$$

$$A_{V_c} = 364.50 \quad \text{in}^2$$

$$\psi_{ed,V} := 1.00 \quad \text{Modification factor for edge effect}$$

$$\psi_{c,V} := 1.40 \quad \begin{array}{l} \text{Modification Factor in No cracked concrete at service loads} \\ \psi_{c,V}=1.4 \\ \text{Modification Factor in cracked concrete with} \\ \psi_{c,V}=1 \text{ w/ No supplementary reinforcement.} \\ \psi_{c,V}=1.2 \text{ w/ Supplementary reinforcement of a No 4 Bar between the anchor and the edge.} \\ \psi_{c,V}=1.4 \text{ w/ Supplementary reinforcement of No 4 Bar and stirrups spaced at not more than 4 in.} \end{array}$$

$$l_e := \text{if}(h_{ef} > 8 \cdot d_o, 8 \cdot d_o, h_{ef})$$

$$l_e = 30.00 \quad \text{in}$$

$$V_b := 7 \cdot \left[\left(\frac{l_e}{d_o} \right)^{0.2} \sqrt{d_o} \cdot \sqrt{f_{c_{\text{concrete}}}} \cdot C_{a1}^{1.5} \right]$$

$$V_b = 59887.67 \quad \text{lbs.}$$

$$V_{cb} := \frac{A_{V_c}}{A_{V_{co}}} \cdot \psi_{ed,V} \cdot \psi_{c,V} \cdot V_b$$

$$V_{cb} = 83842.74 \quad \text{lbs.}$$

$$\phi := 0.70 \quad \begin{array}{l} \text{Conditions} \\ \phi=0.75 \text{ w/ Supplementary reinforcement are crossing} \\ \text{concrete failure prism.} \\ \phi=0.70 \text{ w/ No supplementary reinforcement are crossing} \\ \text{concrete failure prism.} \end{array}$$

$$\phi \cdot V_{cb} = 58689.92 \quad \text{lbs}$$

$$\text{BREAKOUT}_{\text{shear}} := \text{if}(V_{u_{\text{horizontal}}} > \phi \cdot V_{cb}, \text{"N.G."}, \text{"OK"})$$

$$\text{BREAKOUT}_{\text{shear}} = \text{"OK"}$$

Shear Analysis-Concrete Burst-Out Strength:

$$C_{a1c} := \begin{cases} C_{a1} \\ \text{"N/A"} & \text{if } h_a < C_{a1} \end{cases}$$

$$C_{a1c} = 9.00 \quad \text{in}$$

Total Cone Area for Failure Plane

$$A_{\text{cone.total}} := \sqrt{2} \cdot C_{\text{alc}} \cdot \pi \cdot \left(C_{\text{alc}} - \frac{D_{\text{grout}}}{2} + \frac{B_{\text{grout}}}{2} \right) \quad A_{\text{cone.total}} = 359.87 \quad \text{in}^2$$

Cone Area for Failure Plane at Post

$$A_{\text{cone.post}} := \sqrt{2} \cdot \left(\frac{B_{\text{grout}}}{2} \right)^2 \cdot \pi \quad A_{\text{cone.post}} = 10.00 \quad \text{in}^2$$

Final Cone Area for Failure Plane

$$A_{\text{cone.final}} := \frac{A_{\text{cone.total}} - A_{\text{cone.post}}}{2} \quad A_{\text{cone.final}} = 174.94 \quad \text{in}^2$$

Final Area for Failure Plane

$$V_{\text{cone.cb}} := 4 \cdot \sqrt{f_{\text{c.concrete}}} \cdot A_{\text{cone.final}} \quad V_{\text{cone.cb}} = 38327.11 \quad \text{lbs}$$

$$\phi := 0.70 \quad \phi \cdot V_{\text{cone.cb}} = 26828.98 \quad \text{lbs}$$

Conditions
 $\phi=0.75$ w/ Supplementary reinforcement are crossing concrete failure prism.
 $\phi=0.70$ w/ No supplementary reinforcement are crossing concrete failure prism.

$$\text{BURSTOUT}_{\text{shear}} := \text{if}(V_{\text{u.horizontal}} > \phi \cdot V_{\text{cone.cb}}, \text{"N.G."}, \text{"OK"})$$

$\text{BURSTOUT}_{\text{shear}} = \text{"OK"}$
--