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Bellingham, WA 98225
360-714-8260 www.king-works.com

JOB TITLE PSE - OTC MONUMENT SIGN 03/19/25

JOB NO. 21239 SHEET NO. 1
CALCULATED BY DL DATE
CHECKED BY DATE

CS2021 Ver 2023-03-04 www.struware.com

These calculations must be on site and made available by the Permittee for all inspections.

PRSG20250392

City of Puyallup
Building
REVIEWED
FOR
COMPLIANCE
BSnowden
04/17/2025
9:09:00 AM

City of Puyallup
Development & Permitting Services
ISSUED PERMIT
Building | Planning
Engineering | Public Works
Fire | Traffic

STRUCTURAL CALCULATIONS

FOR

PSE - OTC MONUMENT SIGN

PUYALLUP, WA

Code: International Building Code 2021 Risk Category: II
Loads:

Wind: 98mph / exp B
Seismic: Ss=127.7% g / S1=43.9% g / SDC=D Ieq= 1.00
Soils: Basis: Geotech report
Allowable Bearing Stress: N/A PSF
Lateral Soil Loads (Passive): 300 PCF

Description:

Precast concrete monument sign with two pipe supports cantilevered up from drilled concrete piers.

DANIEL DAVIS LINKHART
STATE OF WASHINGTON
41390
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER
3/19/24
Daniel D. Linkhart, PE/SE
Date: 2025.03.19

DIGITAL SEAL
ON PDF FILE

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Code Search

Code: International Building Code 2021

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.25 / 12	1.2 deg
Building length	12.0 ft	
Least width	1.0 ft	
Mean Roof Ht (h)	6.0 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	

Live Loads:

Roof
0 to 200 sf: 20 psf
200 to 600 sf: 24 - 0.02Area, but not less than 12 psf
over 600 sf: 12 psf

Floor:

Typical Floor

Partitions N/A

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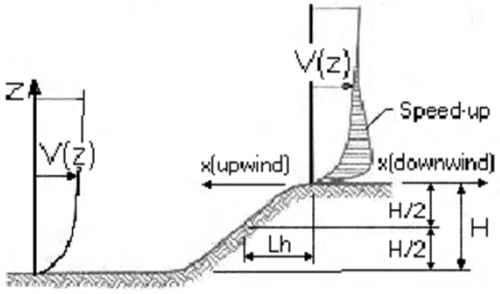
Wind Loads : ASCE 7- 16

Ultimate Wind Speed	98 mph
Nominal Wind Speed	75.9 mph
Risk Category	II
Exposure Category	B
Enclosure Classif.	Enclosed Building
Internal pressure	+/-0.18
Directionality (Kd)	0.85
Kh case 1	0.701
Kh case 2	0.575
Type of roof	Monoslope

Topographic Factor (Kzt)

Topography		Flat
Hill Height	(H)	0.0 ft
Half Hill Length (Lh)		0.0 ft
Actual H/Lh	=	0.00
Use H/Lh	=	0.00
Modified Lh	=	0.0 ft
From top of crest: x =		0.0 ft
Bldg up/down wind?		downwind

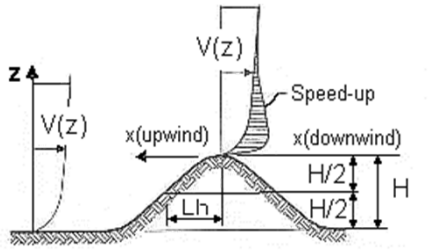
H< 60ft;exp B
∴ Kzt=1.0



ESCARPMENT

H/Lh= 0.00 K_1 = 0.000
x/Lh = 0.00 K_2 = 0.000
z/Lh = 0.00 K_3 = 1.000

At Mean Roof Ht:
Kzt = $(1+K_1K_2K_3)^2$ = 1.00



2D RIDGE or 3D AXISYMMETRICAL HILL

Gust Effect Factor

h =	6.0 ft
B =	1.0 ft
/z (0.6h) =	30.0 ft

Flexible structure if natural frequency < 1 Hz (T > 1 second).
If building h/B>4 then may be flexible and should be investigated.
h/B = 6.00 May be flexible structure

G = 0.85 Using rigid structure default

Rigid Structure

$\bar{e}$ =	0.33
ℓ =	320 ft
z_{min} =	30 ft
c =	0.30
g_Q, g_v =	3.4
L_z =	310.0 ft
Q =	0.97
I_z =	0.30
G =	**0.91** use G = 0.85

Flexible or Dynamically Sensitive Structure

Natural Frequency (η_1) =	0.0 Hz				
Damping ratio (β) =	0				
/b =	0.45				
/α =	0.25				
Vz =	63.2				
N_1 =	0.00				
R_n =	0.000				
R_h =	28.282	η =	0.000	h =	6.0 ft
R_B =	28.282	η =	0.000		
R_L =	28.282	η =	0.000		
g_R =	0.000				
R =	0.000				
Gf =	0.000				

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Enclosure Classification

Test for Enclosed Building: Ao < 0.01Ag or 4 sf, whichever is smaller

Test for Open Building: All walls are at least 80% open.
Ao ≥ 0.8Ag

Test for Partially Enclosed Building: Predominately open on one side only

	Input			Test	
Ao	500.0	sf	Ao ≥ 1.1Aoi	NO	
Ag	600.0	sf	Ao > 4' or 0.01Ag	YES	
Aoi	1000.0	sf	Aoi / Agi ≤ 0.20	YES	Building is NOT
Agi	10000.0	sf			Partially Enclosed

Conditions to qualify as Partially Enclosed Building. Must satisfy all of the following:
Ao ≥ 1.1Aoi
Ao > smaller of 4' or 0.01 Ag
Aoi / Agi ≤ 0.20
Where:
Ao = the total area of openings in a wall that receives positive external pressure.
Ag = the gross area of that wall in which Ao is identified.
Aoi = the sum of the areas of openings in the building envelope (walls and roof) not including Ao.
Agi = the sum of the gross surface areas of the building envelope (walls and roof) not including Ag.

Test for Partially Open Building: A building that does not qualify as open, enclosed or partially enclosed.
(This type building will have same wind pressures as an enclosed building.

Reduction Factor for large volume partially enclosed buildings (Ri) :

If the partially enclosed building contains a single room that is unpartitioned , the internal pressure coefficient may be multiplied by the reduction factor Ri.

Total area of all wall & roof openings (Aog): 0 sf
Unpartitioned internal volume (Vi) : 0 cf
Ri = 1.00

Ground Elevation Factor (Ke)

Grd level above sea level = 0.0 ft Ke = 1.0000
Constant = 0.00256 Adj Constant = 0.00256

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Wind Loads - Other Structures: ASCE 7- 16 Ultimate Wind Pressures

Wind Factor = 1.00
Gust Effect Factor (G) = 0.85 Ultimate Wind Speed = 98 mph
Kzt = 1.00 Exposure = B

A. Solid Freestanding Walls & Solid Signs (& open signs with less than 30% open)

		s/h =	0.77	**Case A & B**	
Dist to sign top (h)	6.0 ft	B/s =	2.61	C_f =	1.56
Height (s)	4.6 ft	Lr/s =	0.00	F = qz G Cf As =	**16.0 As**
Width (B)	12.0 ft	Kz =	0.575	As =	55.2 sf
Wall Return (Lr) =	0.0 ft	qz =	12.0 psf	F =	883 lbs
Directionality (Kd)	0.85				
Percent of open area		Open reduction		**CaseC**	
to gross area	0.0%	factor =	1.00	Horiz dist from windward edge	Cf F=qzGCfAs (psf)
		Case C reduction factors		0 to s	2.46 25.1 **As**
		Factor if s/h>0.8 =	1.00	s to 2s	1.62 16.6 **As**
		Wall return factor for Cf at 0 to s =	1.00	2s to 3s	1.15 16.0 **As**

B. Open Signs & Single-Plane Open Frames (openings 30% or more of gross area)

Height to centroid of Af (z)	15.0 ft			Kz =	0.575
				Base pressure (qz) =	12.0 psf
Width (zero if round)	0.0 ft				
Diameter (zero if rect)	2.0 ft	D(qz)^.5 =	6.93	$F = q_z G C_f A_f$ =	**16.0 Af**
Percent of open area		I =	0.65	Solid Area: A_f =	10.0 sf
to gross area	35.0%	C_f =	1.1	F =	160 lbs
Directionality (Kd)	0.85				

C. Chimneys, Tanks, & Similar Structures

Height to centroid of Af (z)	15.0 ft			Kz =	0.575
Cross-Section	Square			Base pressure (qz) =	12.7 psf
Directionality (Kd)	0.90				h/D = 15.00
Height (h)	15.0 ft				
Width (D)	1.0 ft				
Type of Surface	N/A				

Square (wind along diagonal)		Square (wind normal to face)	
Cf =	1.28	C_f =	1.67
F = qz G Cf Af =	**16.0 Af**	$F = q_z G C_f A_f$ =	**18.0 Af**
Af =	sf	A_f =	10.0 sf
F =	0 lbs	F =	180 lbs

D. Trussed Towers

Height to centroid of Af (z)	15.0 ft			Kz =	0.575
∈ =	0.27			Base pressure (qz) =	13.4 psf
Tower Cross Section	triangle				
Member Shape	flat			Diagonal wind factor =	1
Directionality (Kd)	0.95			Round member factor =	1.000
				Triangular Cross Section	
				C_f =	2.38
				$F = q_z G C_f A_f$ =	**27.1 Af**
		~~10.0~~		Solid Area: A_f =	10.0 sf
				F =	271 lbs

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Seismic Loads: IBC 2021 Strength Level Forces

Risk Category : II
Importance Factor (Ie) : 1.00
Site Class : E

Ss (0.2 sec) = 127.70 %g
S1 (1.0 sec) = 43.90 %g

A site specific ground motion analysis is required for seismically isolated structures or with damping systems

Site specific ground motion analysis performed:

Fa =	1.200	use 1.20	Sms =	1.532	S_{DS} =	1.022	Design Category =	D
Fv =	0.000	use 2.32	Sm1 =	1.018	S_{D1} =	0.679	Design Category =	D

ASCE7 11.4.8 exception 1 applied and Fa taken equal to site class C

Seismic Design Category = **D**
Redundancy Coefficient ρ = 1.30
Number of Stories: 1
Structure Type: Moment-resisting frame systems of steel
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Structural steel systems not specifically detailed for seismic resistance**
Seismic resisting system: **Structural steel systems not specifically detailed for seismic resistance**
System Structural Height Limit: **System not permitted for this seismic design category**
Actual Structural Height (hn) = 6.0 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 3
Over-Strength Factor (Ωo) = 3
Deflection Amplification Factor (Cd) = 3
S_{DS} = 1.022
S_{D1} = 0.679

Seismic Load Effect (E) = Eh +/-Ev = ρ Q_E +/- 0.2S_{DS} D = 1.3Qe +/ 0.204D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = Ωo Q_E +/- 0.2S_{DS} D = 3Qe +/- 0.204D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.028 Cu = 1.40
Approx fundamental period (Ta) = $C_T h_n^{\wedge}$ = 0.117 sec x= 0.80 Tmax = CuTa = 0.164 sec
User calculated fundamental period = T = 0.117 sec
Long Period Transition Period (TL) = ASCE7 map = 6 sec
Seismic response coef. (Cs) = SdsI/R = 0.341
need not exceed Cs = Sd1 I /RT = 1.928
but not less than Cs = 0.044SdsI = 0.045
USE Cs = 0.341

Design Base Shear V = 0.341W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

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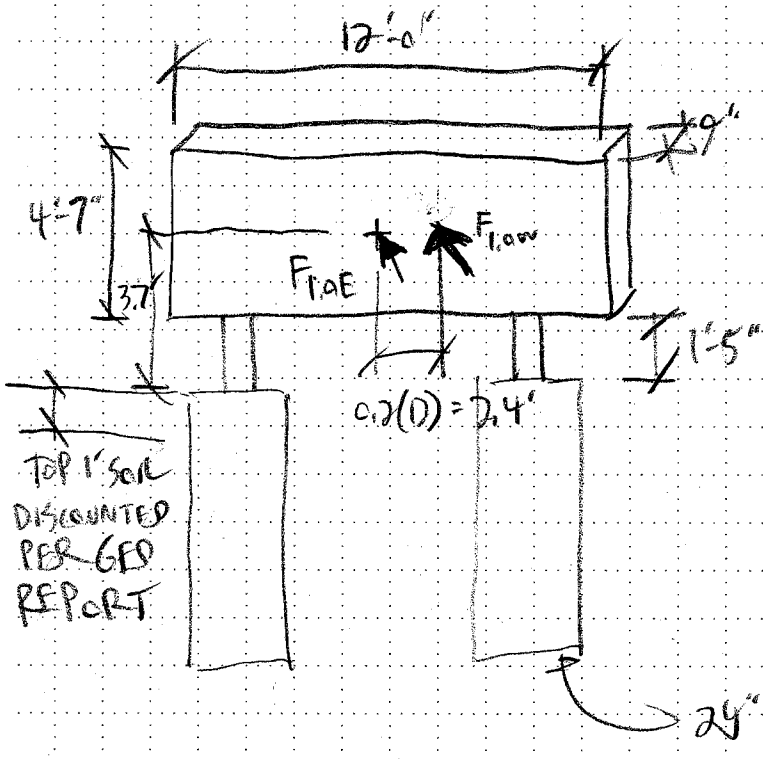
PROJECT	PSE - OTC
DESCRIPTION	MONUMENT SIGN

ENGINEER	PROJECT NO.	DATE	PAGE
DL	2239		



$F_{1.aW} = 16\,PSF(55.25F) = 883\#$

$WT = 12(0.75)(4.6)(0.15) = 6.21K$

$F_p = 0.341\,(6.21K) = 2.12K$

GOVS BY INSP

24"ø x (5'+1') = 6' EMBED PER SPRDSHT

POLE $M_u = \left(\frac{2.12K}{2}\right)(4') = 4.24K'$

$Z_{MIN} = \frac{4.24(12)}{35(0.9)} = 1.62$ SAY 3" STD PIPE ($Z = 2.19$)

SIGN REINF — SAY 2' EFF STRIP @ EA POLE

$M_u = \left(\frac{2.12K}{2}\right)(2.4') = 2.54K'$ $A_s \approx \frac{2.54}{4(4.5)} = 0.14in^2$

USE #4 CENTERED @ 12" O/C EA WAY

EMBED REINF: $M_u = 2.12K(2.3')\left(\frac{1}{2}\right) = 2.44K'$ @ T/POST

$A_{s(REQ)} \approx \frac{2.44}{4(5")} = 0.13in^2 \rightarrow$ (2) #4 EA SIDE OK

BASE PLATE: $M_u = \left(\frac{2.12}{2}\right)(3.7) = 3.93K'$ SEE ANCHPSN OUTPUT

$\times 1.75\,\Omega_a = 6.88K'$, $V_u = 1.86K$ ↳ $\frac{5}{8}$" MN BP

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IBC 1807.3 Pole Foundation

Project	PSE - OTC		#21239
Description	monument sign pier		
Engineer	DL	Date	

		case 1		case 2	case 3	case 4	case 5	case 6
Nonconstraned:		TRUE		TRUE				
b=Diameter of round or diagonal dimension of square footing:		2	FT	2				
h=Distance in feet from ground surface to point of load application:		4.7	FT	4.7				
P=Applied load		1060	LB	742				
Presumptive Lateral Bearing Pressure:		300		300				
Nonconstrained only:								
S1=Allowable lateral soil bearing pressure as set forth in section 1806.2 based on a depth of one - third the depth of embedment								
	S1=	500	PSF	440	381	360	350	
	A=	2.48		1.97	N/A	N/A	N/A	N/A
d=Depth of embedment in earth but not over 12 feet		5.01	FT	4.32	N/A	N/A	N/A	N/A
	calculated S1	501		432	N/A	N/A	N/A	N/A
Contrained only:	covergence test	TRUE		TRUE	N/A	N/A	N/A	N/A
S3=Allowable lateral soil bearing pressure as set for the in sectio n1806.2 based on a depth equal to the depth of embedment								
	S3=		PSF					
d=Depth of embedment in earth but not over 12 feet		N/A	FT	N/A	N/A	N/A	N/A	N/A
	calculated S3	N/A		N/A	N/A	N/A	N/A	N/A
	covergence test	N/A		N/A	N/A	N/A	N/A	N/A

Table 1806.2 Presumptive Load Bearing Values

Lateral Bearing Pressure	
Bedrock	400 to 1200 psf/ft
Sandy gravel or or gravel	200 psf/ft
sand, silty sand, silty gravel and glayey gravel	150 psf/ft
clay sandy clay, silty clay, clayey silt, silt, and sandy silt	100 psf/ft

1806.1 Lateral loads may be increased by 1/3 where used with the alternative basic load combinations of section 1605.3.2 that include wind or earthquake loads

1806.3.3 Lateral loads may be increased by the value above for each additional foot of depth to a maximum of 15 times the tabular value

1806.3.4 Increase for poles: Isolated poles for uses such as flagpoles or signs and poles usded to support buildings that are not adversely affected by a 1/2 inch motion at the ground surface due to short tem lateral loads shall be permitted to be designed unsing lateral bearing pressures equal to two times the tabular values

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	2/18/2025
Engineer:		Page:	2
Project:			
Address:			
Phone:			
E-mail:			

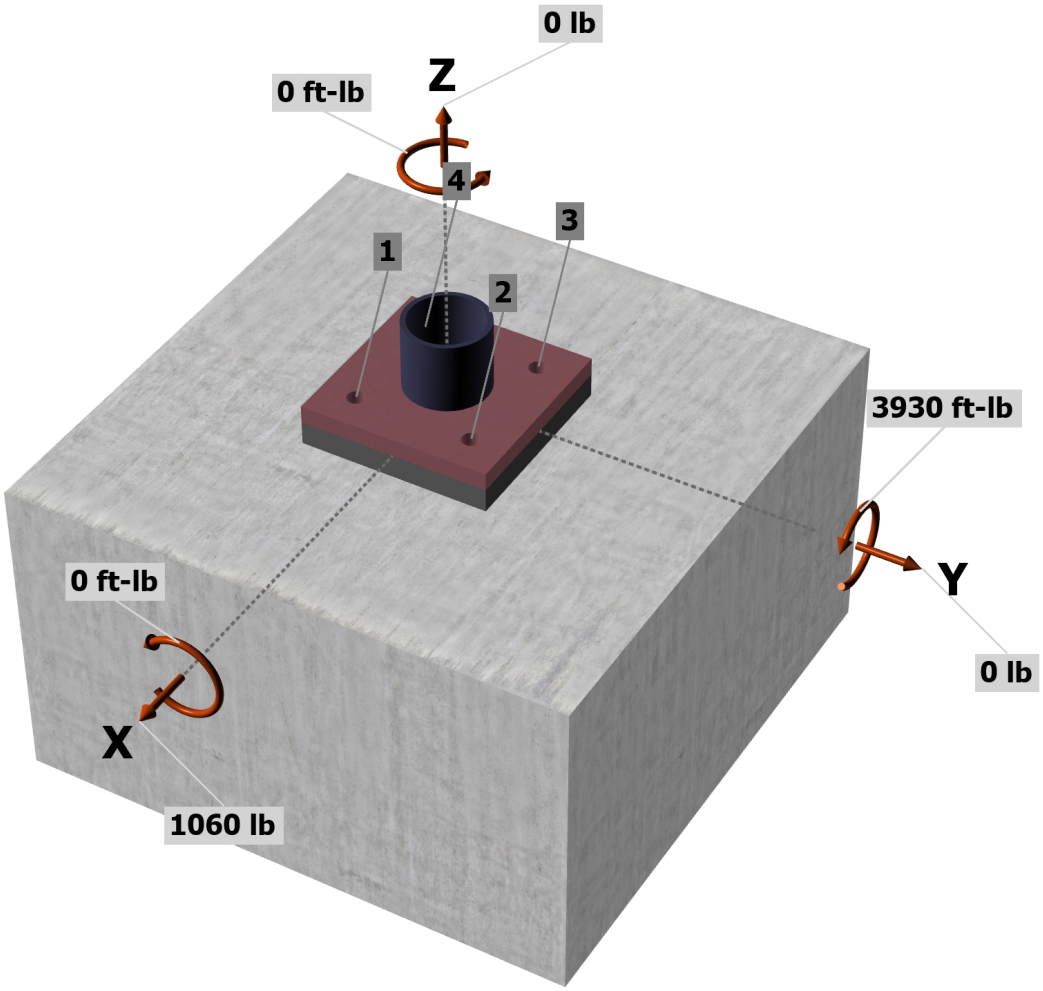
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

This calc is for baseplate thickness only, not for anchors

Strength level loads:

N_{ua} [lb]:	0
V_{uax} [lb]:	1060
V_{uay} [lb]:	0
M_{ux} [ft-lb]:	0
M_{uy} [ft-lb]:	3930
M_{uz} [ft-lb]:	0

<Figure 1>



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Company:		Date:	2/18/2025
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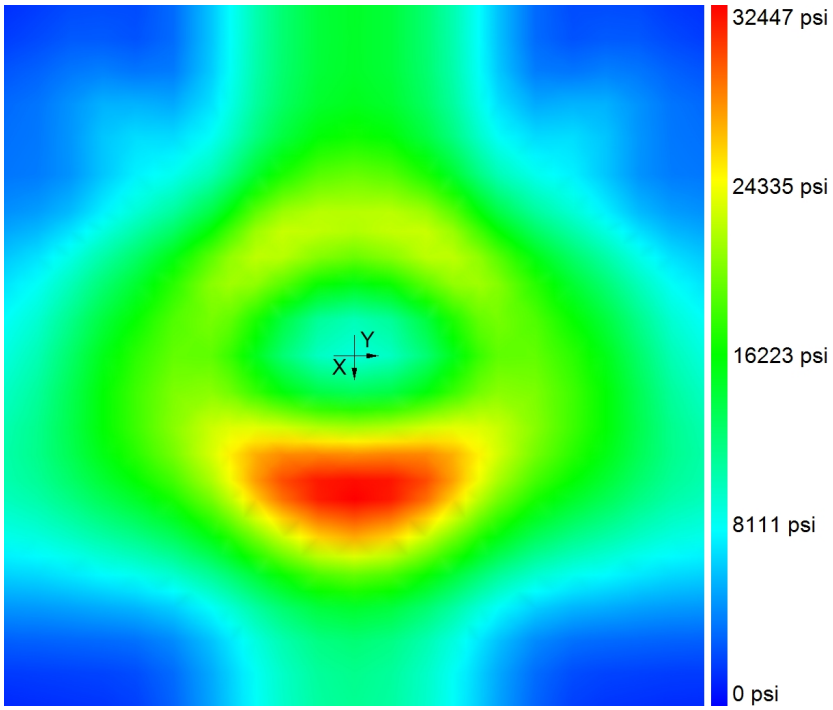
Project:			
Address:			
Phone:			
E-mail:			

Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32447 psi**
Calculated plate thickness	**0.675 inch**

Stress distribution



For ACI and CSA design methods, maximum base plate stress is limited to 0.9 times yield stress.
For ETAG design method, maximum base plate stress is limited to yield stress divide by 1.5.
Plate stress is derived using Von Mises theory.

$$\sigma_{xx} = \frac{F_{xx}}{t} + \frac{6M_{xx}}{t^2} \text{ (@ bottom) or } \sigma_{xx} = \frac{F_{xx}}{t} - \frac{6M_{xx}}{t^2} \text{ (@ top)}$$

$$\sigma_{yy} = \frac{F_{yy}}{t} + \frac{6M_{yy}}{t^2} \text{ (@bottom) or } \sigma_{yy} = \frac{F_{yy}}{t} - \frac{6M_{yy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xy} = \frac{F_{xy}}{t} + \frac{6M_{xy}}{t^2} \text{ (@bottom) or } \sigma_{xy} = \frac{F_{xy}}{t} - \frac{6M_{xy}}{t^2} \text{ (@ top)}$$

$$\sigma_{xz} = \frac{V_x}{t}$$

$$\sigma_{yz} = \frac{V_y}{t}$$

$\sigma_{xx}, \sigma_{yy}, \sigma_{xy}$ as follows:

$$S_1 = \frac{\sigma_{xx} + \sigma_{yy}}{2} + \sqrt{(\frac{\sigma_{xx} - \sigma_{yy}}{2})^2 + \sigma_{xy}^2}$$

$$S_2 = \frac{\sigma_{xx} + \sigma_{yy}}{2} - \sqrt{(\frac{\sigma_{xx} - \sigma_{yy}}{2})^2 + \sigma_{xy}^2}$$

$$S_3 = 0$$

$$\sigma_{Von\,Mises} = \sqrt{\frac{(S_1 - S_2)^2 + (S_1 - S_3)^2 + (S_2 - S_3)^2}{2}}$$

12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

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Anchor Designer™ for Concrete Software
Version 3.3.2410.2

Company:		Date:	2/18/2025
Engineer:		Page:	1
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Design name: Design

Comment: This calc is for anchors only, using omega seismic loads

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.38
C_{min} (inch): 3.75
S_{min} (inch): 3.75

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Cracked
Compressive strength, f'_c (psi): 4500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 8.00 x 8.00 x 0.75
Yield stress: 36000 psi

Profile type/size: 3STD

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 5/8"Ø Heavy Hex Bolt, F1554 Gr. 36

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Company:		Date:	2/18/2025
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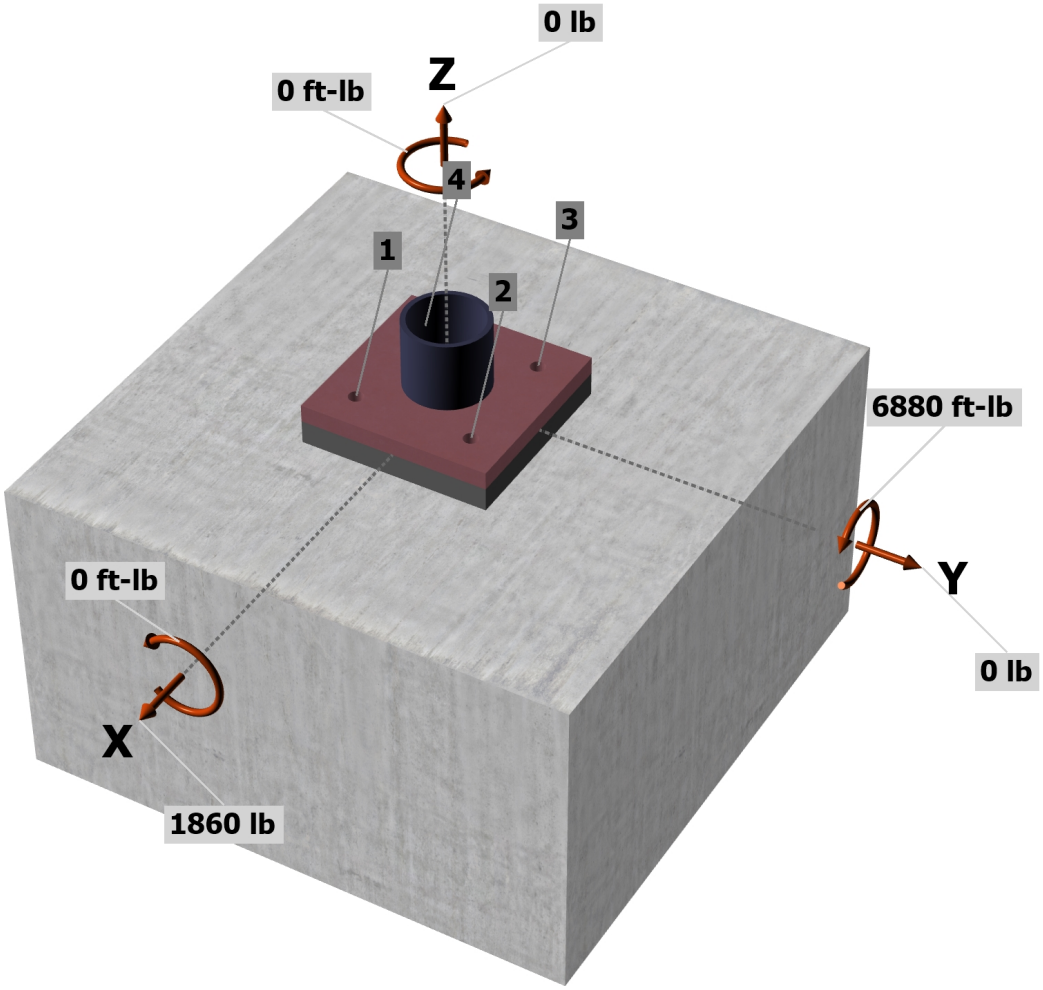
Project:			
Address:			
Phone:			
E-mail:			

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.10.5.3 (d) is satisfied
Ductility section for shear: 17.10.6.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]:	0
V_{uax} [lb]:	1860
V_{uay} [lb]:	0
M_{ux} [ft-lb]:	0
M_{uy} [ft-lb]:	6880
M_{uz} [ft-lb]:	0

<Figure 1>



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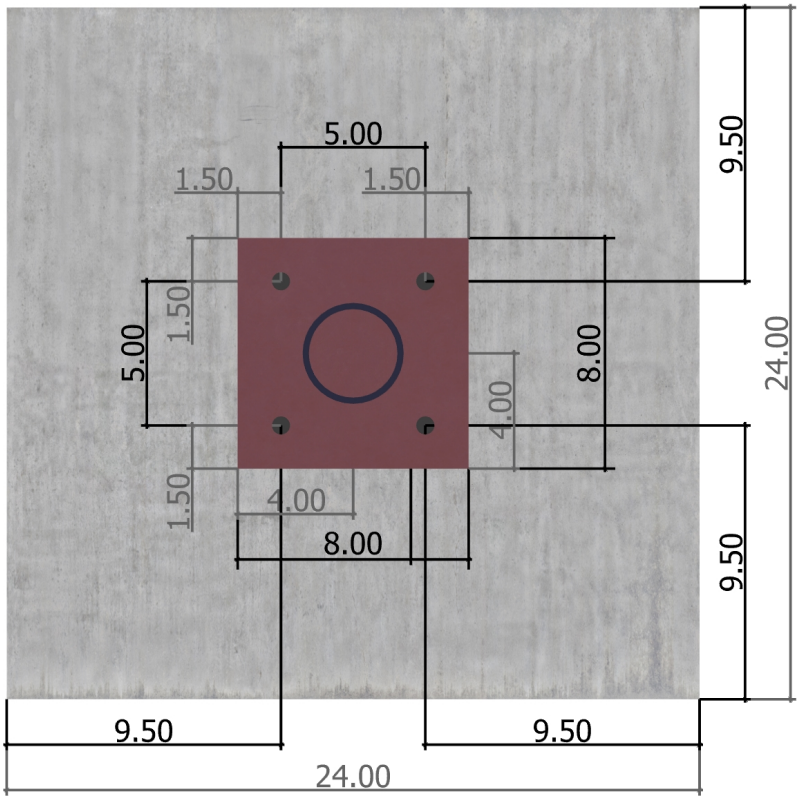
Company:		Date:	2/18/2025
Engineer:		Page:	3
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>

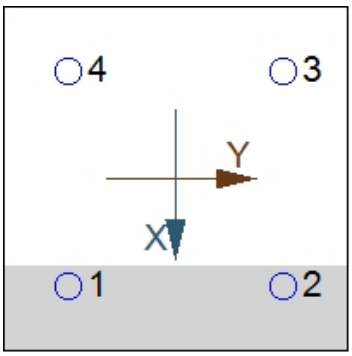


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	465.0	0.0	465.0
2	0.0	465.0	0.0	465.0
3	7064.2	465.0	0.0	465.0
4	7064.2	465.0	0.0	465.0
Sum	14128.3	1860.0	0.0	1860.0

Maximum concrete compression strain (‰): 0.41
Maximum concrete compression stress (psi): 1793
Resultant tension force (lb): 14128
Resultant compression force (lb): 14128
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
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4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13100	0.75	9825

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.6.2.2.1)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
24.0	1.00	4500	6.333	25661

$0.75\phi N_{cbg} = 0.75\phi \left(A_{Nc} / A_{Nco} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
456.00	361.00	9.50	1.000	1.000	1.00	1.000	25661	0.70	17017

6. Pullout Strength of Anchor in Tension (Sec. 17.6.3)

$0.75\phi N_{pn} = 0.75\phi \Psi_{c,P} N_p = 0.75\phi \Psi_{c,P} 8 A_{brg} f'_c$ (Sec. 17.5.1.2, Eq. 17.6.3.1 & 17.6.3.2.2a)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	0.67	4500	0.70	12682

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8. Steel Strength of Anchor in Shear (Sec. 17.7.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
7865	0.8	0.65	4090

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.7.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	4500	14.50	31068

$\phi V_{cbgx} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. 17.5.1.2 & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
522.00	946.13	1.000	0.831	1.000	1.000	31068	0.70	9971

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.7.2.2.1a & Eq. 17.7.2.2.1b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
5.00	0.625	1.00	4500	9.50	16476

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (Sec. 17.5.1.2, 17.7.2.1(c) & Eq. 17.7.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
342.00	406.13	1.000	1.000	1.000	1.000	16476	0.70	19424

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.7.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp}(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N}N_b$ (Sec. 17.5.1.2 & Eq. 17.7.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	576.00	361.00	1.000	1.000	1.000	1.000	25661	0.70	57320

11. Results

Interaction of Tensile and Shear Forces (Sec. R17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	7064	9825	0.72	Pass
Concrete breakout	**14128**	**17017**	**0.83**	**Pass (Governs)**
Pullout	7064	12682	0.56	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	465	4090	0.11	Pass
T Concrete breakout x+	**1860**	**9971**	**0.19**	**Pass (Governs)**
|| Concrete breakout y+	**930**	**19424**	**0.05**	**Pass (Governs)**
Pryout	1860	57320	0.03	Pass

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status
Sec. R17.8	0.73	0.06	79.4%	1.0	Pass

5/8"Ø Heavy Hex Bolt, F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

- Designer must exercise own judgement to determine if this design is suitable.