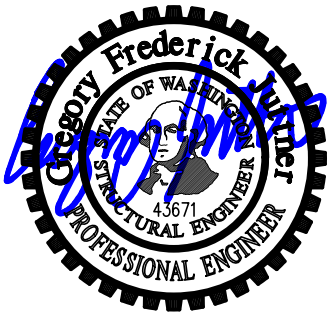
Structural Calculations For:

Cheshire House

7613 E Mercer Way

Mercer Island, WA 98040



Prepared for:	Berger Partnership
Job #:	00586-2024-01
Date:	February 7, 2025

STRUCTURAL ENGINEERING

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Criteria Sheet

Codes

Structural	IBC 2021
Loading	ASCE 7-16
Wood:	NDS 2018 / SDPWS 2021
Steel:	AISC 360-16
Concrete:	ACI 318-19
Masonry:	TMS 402/602-16

Project Location

Street & Number	7615 E Mercer Way		
City:	Mercer Island	State:	WA
ZIP:	98040		
Latitude:	47.5345	N	
Longitude:	-122.2163	W	
Ground Elevation	149	ft	

Occupancy Category

Risk Category: II — ASCE 7 Table 1.5-1

Seismic Load Summary:

Analysis Procedure: Equivalent Lateral Force Procedure

Lateral System: Steel Ordinary Cantilever Column Systems

R:	1.25	C_d=	1.25
Base Shear V =	4 kips	Ω_o=	1.25
S_S=	1.456	S_1=	0.503
S_{DS}=	1.16	S_{D1}=	0.67
C_s=	0.932	I_E=	1.0

Story Information

# Stories Above Grade (Including Mezzanine Levels)	1

Horizontal and Vertical Irregularities:

Is the building a "Regular Structure"? (No horizontal or vertical irregularities)	Yes

Wind Load Summary:

V= 98 K_{ZT}= 1.00

Exposure = C

Dead Loads:

Roof			Floor		
Roofing	1	psf	Finish Floor	2	psf
1/2" Sheathing	1.8	psf	3/4" Sheathing	2.7	psf
Trusses @ 24" oc	2.5	psf	Joists @ 16" oc	2.2	psf
Misc./Mech.	1.5	psf	Misc./Mech.	2	psf
Ceiling Finish	2.8	psf	Ceiling Finish	2.8	
Solar Panels	5	psf		11.7	psf
	15	psf	Use	12	psf
Use	15	psf	Add'l Seismic Weight	10	psf
Add'l Seismic Weight	5	psf	Seismic Weight	22	psf
Seismic Weight	20	psf			

Live Loads:

Roof	20	psf
Floor	40	psf

Snow Loading Criteria:

Ground Snow, p_g	20 psf	Flat Roof Snow Load, p_f	25.0 psf	Importance Factor, I_s	1.00
Exposure Factor, C_e	1.00	Sloped Roof Snow Load, p_s	25.0 psf		
Thermal Factor, C_t	1.00	Slope Factor, C_s	0.61		

Soils:

Allowable Bearing	2000	psf	Active	65/45 pcf (Restrained/Unrestrained)
Sliding, μ	0.3		Seismic Surcharge	8H
Passive	250	pcf		

Soils Report Provided?	Yes
Site Specific Ground Motion Hazard Analysis Provided?	Yes

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STRUCTURAL ENGINEERING

Cheshire Pergola

Criteria

DATE 2/18/2025

PROJ. #

DESIGN AD

SHEET 1

Seismic Design

ASCE 7-16 Seismic Analysis | Equivalent Lateral Force Procedure | Apply Section 12.8.1.3 (Where Applicable)? Yes

Seismic Force Resisting System Per Table 12.2-1		
	System	Cantilevered Column Systems
	Type:	Steel Ordinary Cantilever Column Systems

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	E	Per soils report.
Diaphragm Flexibility	Flexible	

Section 12.8.1.3

1. Regular Structure	Yes
2. ≤ 5 Stories above grade	Yes
3. T ≤ 0.5s	Yes
4. ρ = 1.0	No
5. Not Site Class E or F	No
6. Risk Category I or II	Yes

If all items above are met, S_{DS} may be taken as 1.0, but not less than 0.7*(Calculated S_{DS})

S_S	1.456 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.503 g	2% in 50 yr, Latitude & Longitude lookup
R	1.25	
C_d	1.3	
Ω_o	1.25	
I_e	1.00	Table 1.5-2
h_n	8.0 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.10 sec	
T	0.10 sec	Eq. 12.8-7
T_0	0.12 sec	
T_S	0.58 sec	
T_L	6.00 sec	
F_a	1.20	Table 11.4-1
F_v	2.00	Table 11.4-2
S_{MS}	1.75 g	Eq. 11.4-1
S_{M1}	1.01 g	Eq. 11.4-2
S_{DS}	1.165 g	Eq. 11.4-3
S_{D1}	0.671 g	Eq. 11.4-4
C_s	***0.932 Controls***	Eq. 12.8-2
	5.640	Eq. 12.8-3 need not exceed, T < T_L
	0.010	Eq. 12.8-5 or 12.8-6 minimum
C_s, design	0.932	
Bldg. Weight	4.8 k	
V = C_SW	4.5 k	Eq. 12.8-1, Strength Level Base Shear
V = C_{Sasd}W	3.1 k	Eq. 12.8-1 ASD Base Shear

Building Period Per Alternate Analysis

T (sec)	

Per Geotech Report

F_a	
F_v	

$$T_a = C_t h_n^x \quad \text{Eq. 12.8.7}$$

$$S_{MS} = F_a S_S \quad \text{Eq. 11.4-1}$$
$$S_{M1} = F_v S_1 \quad \text{Eq. 11.4-2}$$
$$S_{DS} = {}^2/_3\, S_{MS} \quad \text{Eq. 11.4-3}$$
$$S_{D1} = {}^2/_3\, S_{M1} \quad \text{Eq. 11.4-4}$$

$$C_S = \frac{S_{DS}}{(R/I_e)} \quad \text{Eq. 12.8-2}$$
$$C_S = \frac{S_{D1}}{T(R/I_e)} \quad \text{Eq. 12.8-3}$$
$$C_S = \frac{S_{D1} T_L}{T^2 (R/I_e)} \quad \text{Eq. 12.8-4}$$
$$C_S \geq 0.044 S_{DS} I_e \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.01 \quad \text{Eq. 12.8-5}$$
$$C_S \geq 0.5 \frac{S_1}{(R/I_e)} \quad \text{Eq. 12.8-6}$$

$$C_{VX} = w_x h_x^k / \sum_{i=1}^{n} w_x h_i^k \quad \text{Eq. 12.8-12}$$

$$F_{px} = {}^{\sum_{i=x}^{n} F_i}/_{\sum_{i=x}^{n} w_i}\, w_{px} \quad \text{Eq. 12.10-1}$$
$$F_{px} \geq 0.2 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-2}$$
$$F_{px} \leq 0.4 S_{DS} I_e w_{px} \quad \text{Eq. 12.10-3}$$

Vertical Distribution ASD ρ= 1.3 k= 1.000

Level	h_x (ft)	W_x (k)	h_x^k (ft)	$W_x h_x^k$	Story Shear ASD			Diaphragm Force (ρ not included)				
					C_{vx} (%)	F_x (k)	SV (k)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	γ=F_{px}/F_x
Pergola	8.0	4.8	8.0	38	1.000	4.1	**4.1**	3.1	0.8	1.6	**1.6**	0.38
Σ		4.8		38		**4.1**						

Wind Design - MWFRS

ASCE 7 Chapter 27 - Directional Procedure

Design Method	ASD

Wind Coefficients

Exposure	C	
V=	98	mph
K_d=	0.85	Table 26.6-1
K_h=	0.85	Table 27.3-1
K_e=	0.99	Table 26.9-1
G=	0.85	26.9.4

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.00	
Roof Type	Monoslope	
Roof Slope - Transverse Dir	0	degrees
Roof Slope - Long Dir	0	degrees
Ground to top of roof	8	ft
Bot of roof to top of roof	0	ft
Mean Roof Height, h	8	ft
Short Plan Dimension	20	ft
Long Plan Dimension	24	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft

Velocity Pressure at Mean Roof Height, q_h =	17.6	psf

Transverse Wind Pressures

L/B = 0.83 h/L = 0.40

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.9 / -0.18
Leeward Roof	-0.42

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	17.67	12.01	7.50	**11.7**
15-20	0.9	18.71	12.72	7.50	**12.1**
20-25	0.94	19.54	13.29	7.50	**12.5**
25-30	0.98	20.37	13.85	7.50	**12.8**
30-40	1.04	21.62	14.70	7.50	**13.3**
41-50	1.09	22.66	15.41	7.50	**13.7**
51-60	1.13	23.49	15.97	7.50	**14.1**
61-70	1.17	24.32	16.54	7.50	**14.4**
71-80	1.21	25.15	17.10	7.50	**14.8**
81-90	1.24	25.77	17.53	7.50	**15.0**
91-100	1.26	26.19	17.81	7.50	**15.2**

Roof Pressures (Unfactored) ASD

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.7	-13.5	-6.3	**4.80**

Longitudinal Wind Pressures

L/B = 1.20 h/L = 0.33

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.46
Windward Roof	-0.9 / -0.18
Leeward Roof	-0.42

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	17.67	12.01	6.90	**11.35**
15-20	0.9	18.71	12.72	6.90	**11.77**
20-25	0.94	19.54	13.29	6.90	**12.11**
25-30	0.98	20.37	13.85	6.90	**12.45**
30-40	1.04	21.62	14.70	6.90	**12.96**
41-50	1.09	22.66	15.41	6.90	**13.38**
51-60	1.13	23.49	15.97	6.90	**13.72**
61-70	1.17	24.32	16.54	6.90	**14.06**
71-80	1.21	25.15	17.10	6.90	**14.40**
81-90	1.24	25.77	17.53	6.90	**14.66**
91-100	1.26	26.19	17.81	6.90	**14.82**

Roof Pressures (Unfactored) ASD

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.7	-13.5	-6.3	**4.80**

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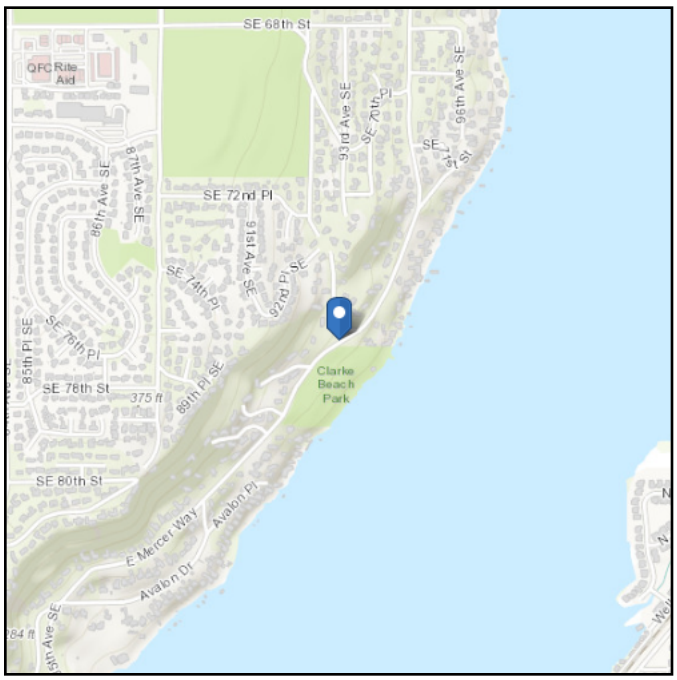
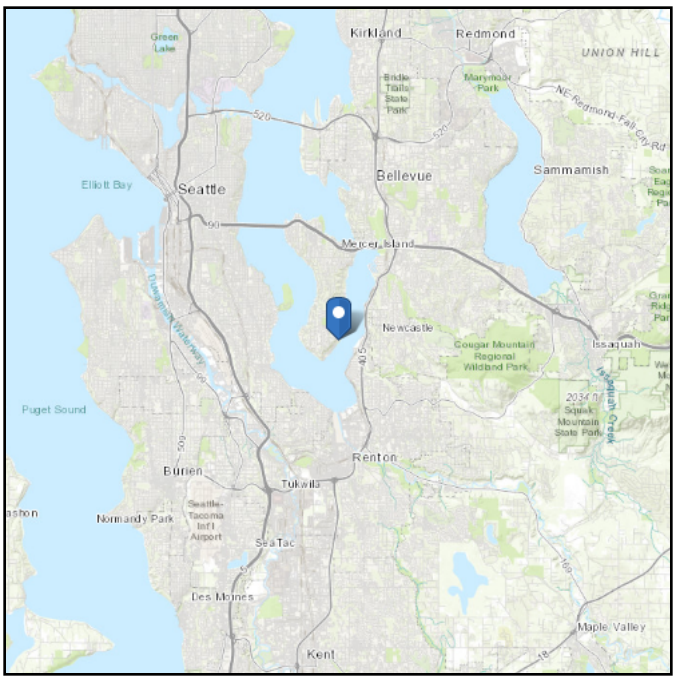
ASCE Hazards Report

Address:
7613 E Mercer Way
Mercer Island, Washington
98040

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: DE

Latitude: 47.534508
Longitude: -122.215356
Elevation: 103.97892005632795 ft (NAVD 88)





Seismic

Site Soil Class: DE

Results:

PGA_M :	0.7	T_L :	6
S_{MS} :	1.67	S_S :	1.63
S_{M1} :	1.6	S_1 :	0.62
S_{DS} :	1.11	V_{S30} :	185
S_{D1} :	1.07		

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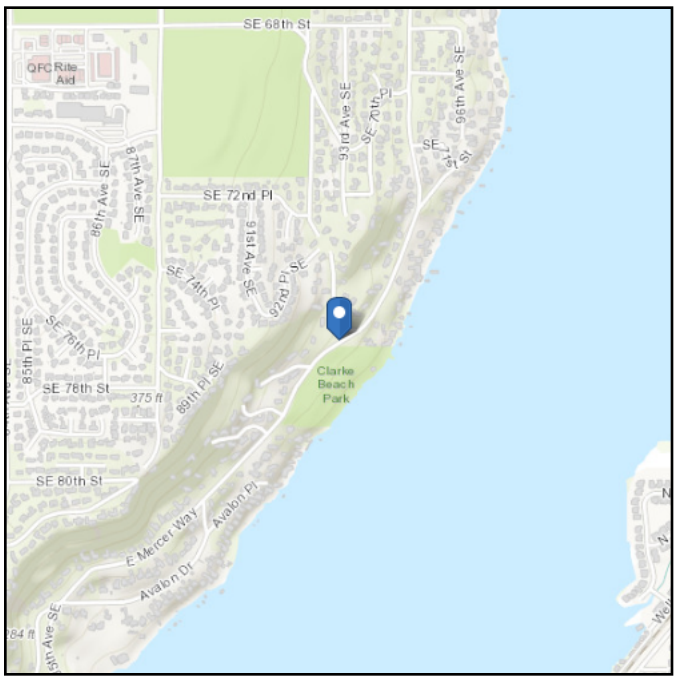
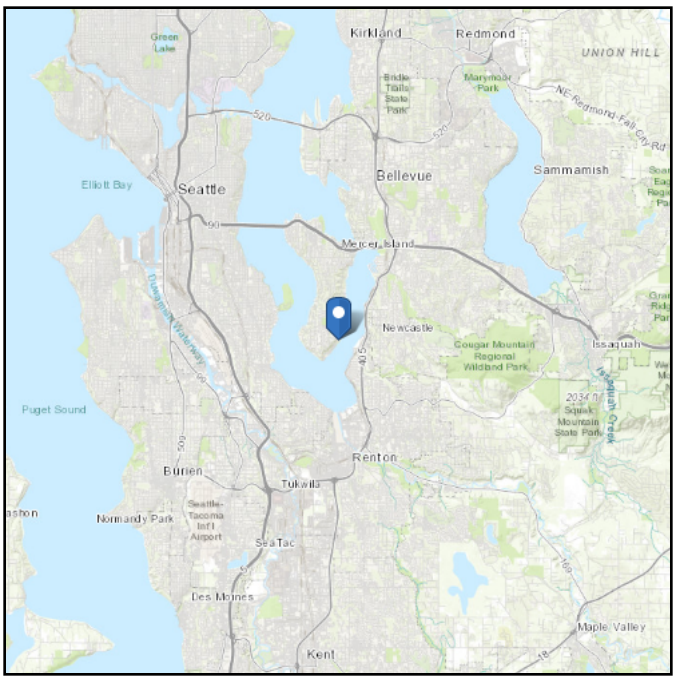




Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	74 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph
300-year MRI	92 Vmph
700-year MRI	98 Vmph
1,700-year MRI	105 Vmph
3,000-year MRI	109 Vmph
10,000-year MRI	118 Vmph
100,000-year MRI	136 Vmph
1,000,000-year MRI	154 Vmph

Lateral Analysis

seismic:

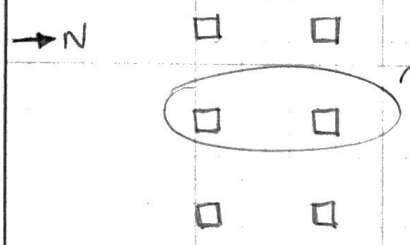
Level	Area	psf	Wx	hx	Wx·hx	Cvx	Fx
Pergola	480 ft²	10	4.8K	8ft	38.4k-ft.	1.0	4.1K

ordinary cantilever column system → $V = Cs \cdot \rho \cdot w = \left(\frac{1.105}{(1.25/1.0)}\right)(1.3)(4.8K)$

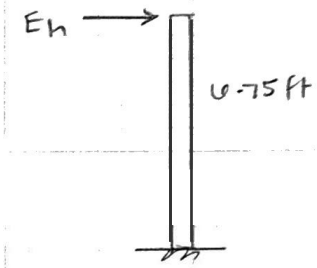
$V = (0.932)(1.3)(4.8K) = 5.8K$

$V(ASD) = 0.7(5.8K) = \underline{4.1K}$

Worst case: E-W Direction



$V = \frac{4.1K}{2} = \frac{2.05K}{2\,col} \rightarrow 1.025K/column$ (allowable)



check deflection:

Allowable story drift = $0.02 \cdot hx = 0.02(82in) = 1.64in$

$E_h = 1.025K/0.7 = 1.46/1.3 = 1.13K$ (strength level seismic force per ASCE 12.8.6)

$\Delta = 0.62in \rightarrow \delta = \frac{Cd \cdot \Delta}{I} = \frac{(1.25)(0.65in)}{1.0} = 0.78in < 1.64in$ OK.
(amplified)

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STRUCTURAL ENGINEERING

PROJECT: Cheshire Pergola, Mercer Island, WA

DATE: 01/24/2025

PROJ #: AD

DESIGN: L-1

SHEET

Member Design (continued)

Load combination 8: $1.0D + 0.7\cancel{E_v} + 0.7E_h$

$1.0D = 1.0(13.91 plf(4.75ft) + 1343 lb) = 1437 lb.$

$0.7E_h = 1025 lb$ (already factored)

→ $V_a = 1025 lb.$
$P_a = 1437 lb.$
$M_a = 6920 lb\text{-}ft.$

Load combination 9: $1.0D + 0.525\cancel{E_v} + 0.525E_h + 0.75L + 0.75S$

$1.0D = 1.0(13.91 plf(4.75ft) + 1343 lb) = 1437 lb.$

$0.525E_h = 0.525(1025 lb / 0.7) = 769 lb.$

$0.75L = 0.75(1785 lb) = 1339 lb.$

$0.75S = 0.75(2231 lb) = 1748 lb.$

→ $V_a = 769 lb.$
$P_a = 4524 lb$
$M_a = 5191 lb\text{-}ft.$

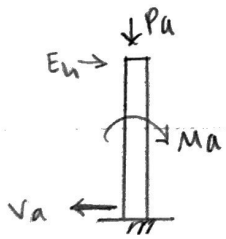
Load combination 10: $0.6D - 0.7\cancel{E_v} + 0.7E_h$

$0.6D = 0.6(13.91 plf(4.75ft) + 1343 lb) = 862 lb.$

$0.7E_h = 1025 lb$

→ $V_a = 1025 lb.$
$P_a = 862 lb$
$M_a = 6920 lb\text{-}ft.$

⇒ Load combination 8 controls:



$\sigma_{Pa} = \frac{1437 lb}{2.42 in^2} = 594 psi$

$\sigma_{Va} = \frac{1025 lb}{2.42 in^2} = 424 psi$

$\sigma_{Ma} = \frac{(6920 lb\text{-}ft)(12 in/1ft)(2 in)}{11.9 in^4} = 13960 psi$

→ Combined stress (unity check) $= \frac{594 psi}{30000 psi} + \frac{424 psi}{30000 psi} + \frac{13960 psi}{30000 psi}$

$= 0.50 < 1.0$ OK.

$V_a = 1025 lb.$
$P_a = 1437 lb$
$M_a = 6920 lb\text{-}ft.$

HSS 4x4x1/2

$V_n/\Omega = 40K$
$P_n/\Omega = 128K$
$M_n/\Omega = 19.2 k\text{-}ft.$

Check worst case $P_a = 4524 lb < 0.15(P_n/\Omega)$ (per ASCE 12.2.5.2)

$4524 lb < 0.15(128 K)$ (AISC Table 4-4 @ 8ft)

$4524 lb << 19.2K$ OK.

⇒ Use HSS 4x4x1/2

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PROJECT: Classmire Pergola, Mercer Island, WA

DATE: 01/24/2025

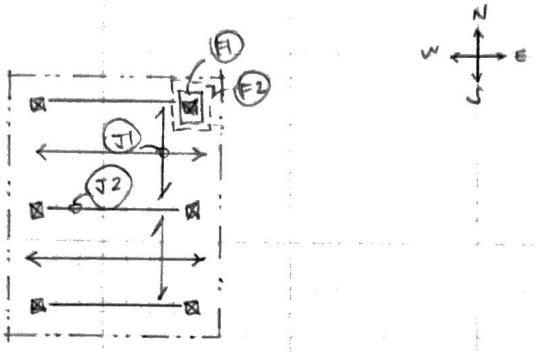
PROJ #: AD

DESIGN: L-2

SHEET

Gravity Design

Key Plan



Pergola Framing:

DL = 15psf
LL = 25psf → TL = 40psf

(J1) Rafters

w = (24in/12)(40psf) = 80 plf
L = 10.5 ft.
R = 420 lb.
M = 1103 lb-ft.

2x10 @ 24" oc
f_b = 618 psi
fv = 39 psi
Δ = 0.17 in

(J2) Beam

w = (10.5 ft)(40psf) = 420 plf.
L = 17 ft.
R = 3570 lb.
M = 15173 lb-ft.

GL 5 1/2 x 12
f_b = 1379 psi
fv = 72 psi
Δ = 0.554 in

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SSF STRUCTURAL ENGINEERING

PROJECT: Cheshire Pergola, Mercer Island, WA
DATE: 01/24/2025
PROJ. #: AD
DESIGN: G-1
SHEET

(F1) Base Plate Design

loading: $E_{mh} = \Omega \cdot Q$ (ASCE 12.4-7)
$= 1.25\,(1.025K)$
$E_{mh} = 1.28K$

Load combination 8: $1.0D + 0.7E_v + 0.7E_{mh}$
$1.0D = 1.0(13.91plf(6.75ft) + 1343lb) = 1437lb.$
$0.7E_{mh} = 1280lb.$
→ $V_a = 1280lb.$
$P_a = 1437lb.$
$M_a = 8640lb\text{-}ft$

Load combination 9: $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.75S$
$1.0D = 1.0(13.91plf(6.75ft) + 1343lb) = 1437lb.$
$0.525E_{mh} = 0.525(1280lb/0.7) = 960lb.$
$0.75L = 0.75(1785lb) = 1339lb.$
$0.75S = 0.75(2231lb) = 1748lb.$
→ $V_a = 960lb.$
$P_a = 4524lb$
$M_a = 6480lb\text{-}ft$

Load combination 10: $0.6D - 0.7E_v + 0.7E_{mh}$
$0.6D = 0.6(13.91plf(6.75ft) + 1343lb) = 862lb.$
$0.7E_{mh} = 1280lb.$
→ $V_a = 1280lb.$
$P_a = 862lb$
$M_a = 8640lb\text{-}ft.$

⇒ Load combination 8 controls

⇒ see next page for base plate design and calculations

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Company:		Date:	2/3/2025
Engineer:		Page:	1
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Design name: Design

Comment:

2. Input Data & Anchor Parameters

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

Anchor Information:
Anchor type: Cast-in-place
Material: AB
Diameter (inch): 0.500
Effective Embedment depth, h_{ef} (inch): 8.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 9.88
C_{min} (inch): 3.00
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 13.78
State: Uncracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.4
Reinforcement condition: A tension, A shear
Supplemental edge reinforcement: Yes
Reinforcement provided at corners: Yes
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): 12.00 x 12.00 x 0.80
Yield stress: 36000 psi

Profile type/size: 4X4X1/2

Recommended Anchor
Anchor Name: PAB Pre-Assembled Anchor Bolt - PAB4 (1/2"Ø)

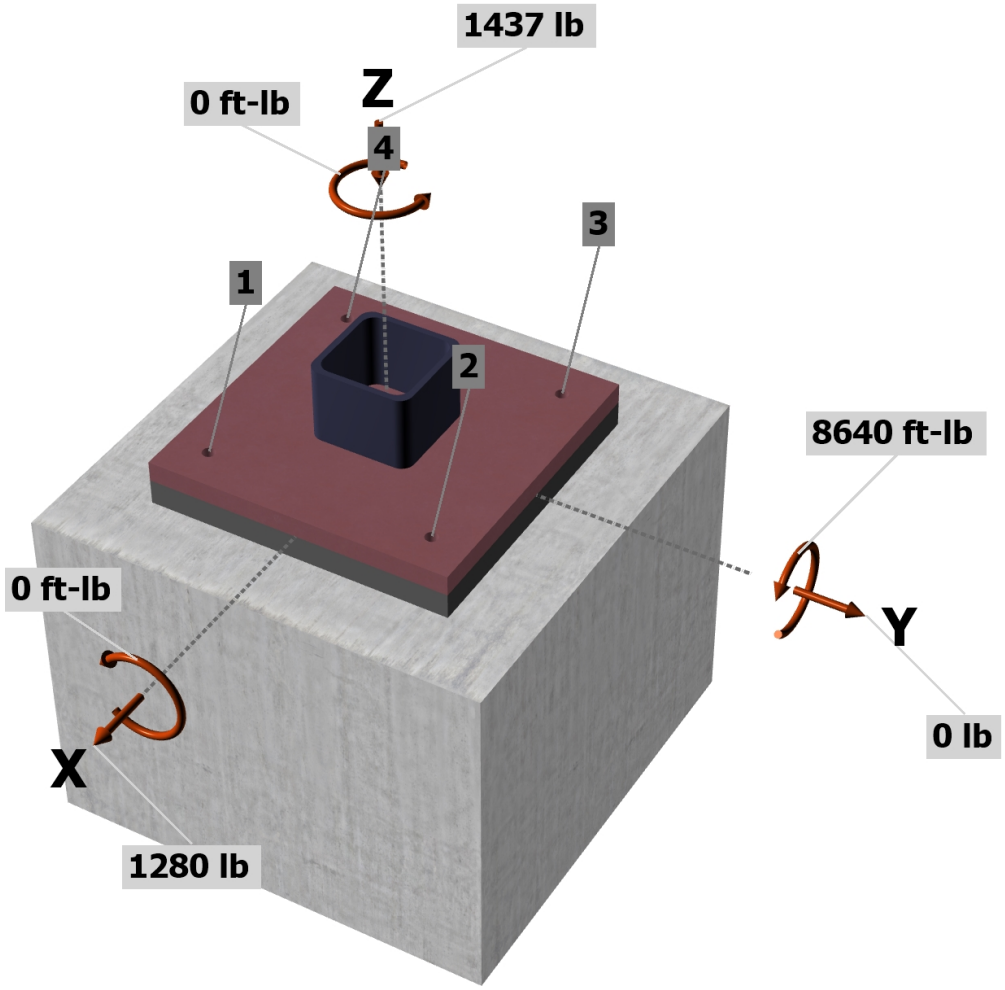
G-3

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: 17.10.5.2 not applicable
Ductility section for shear: 17.10.6.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

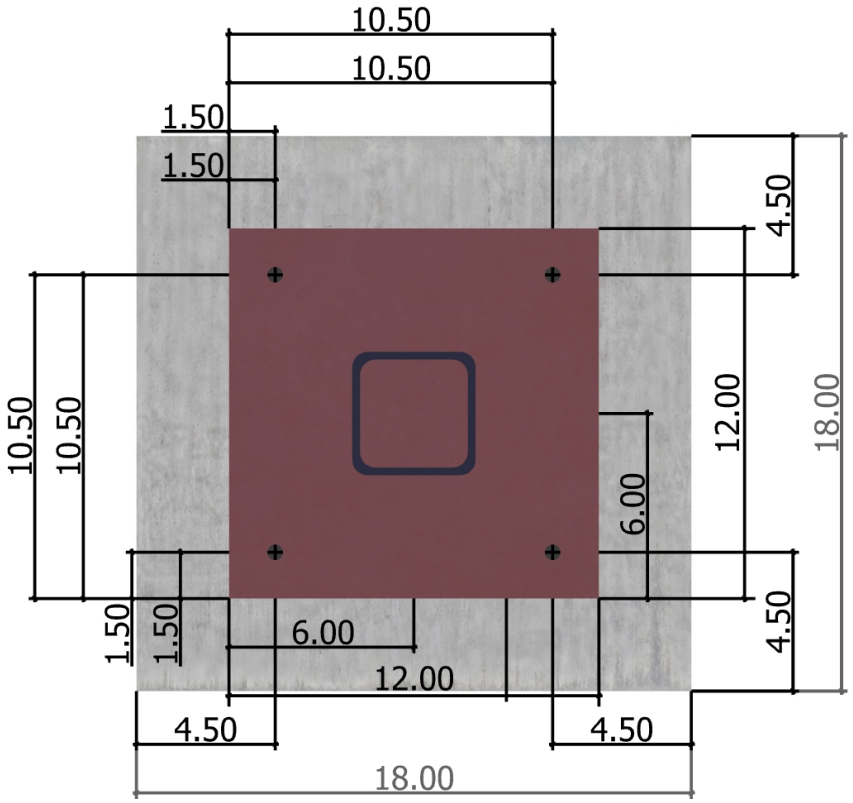
Strength level loads:

N_{ua} [lb]:	-1437
V_{uax} [lb]:	1280
V_{uay} [lb]:	0
M_{ux} [ft-lb]:	0
M_{uy} [ft-lb]:	8640
M_{uz} [ft-lb]:	0

<Figure 1>



<Figure 2>

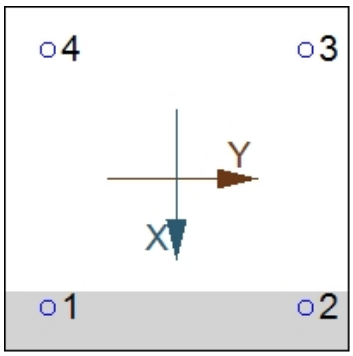


3. Resulting Anchor Forces

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	320.0	0.0	320.0
2	0.0	320.0	0.0	320.0
3	4894.2	320.0	0.0	320.0
4	4894.2	320.0	0.0	320.0
Sum	9788.4	1280.0	0.0	1280.0

Maximum concrete compression strain (‰): 0.21
Maximum concrete compression stress (psi): 907
Resultant tension force (lb): 9788
Resultant compression force (lb): 11225
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>



4. Steel Strength of Anchor in Tension (Sec. 17.6.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
8235	0.75	6176

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	3000	3.000	6831

$0.75\phi N_{cbg} = 0.75\phi \left(A_{Nc} / A_{Nco} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (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)
175.50	81.00	4.50	1.000	1.000	1.25	1.000	6831	0.75	10406

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)
1.4	1.57	3000	0.70	27712

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

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
4940	0.8	0.65	2569

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)
4.00	0.500	1.00	3000	13.50	20383

$\phi V_{cbgx} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (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)
364.50	820.13	1.000	0.767	1.400	1.000	20383	0.75	7292

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)
4.00	0.500	1.00	3000	4.50	3923

$\phi V_{cbgy} = \phi\ (2)(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}V_{bx}$ (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)
121.50	91.13	1.000	1.000	1.400	1.000	3923	0.75	10983

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	324.00	81.00	1.000	1.000	1.250	1.000	6831	0.70	47814

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, øN_n (lb)	Ratio	Status
Steel	4894	6176	0.79	Pass
Concrete breakout	**9788**	**10406**	**0.94**	**Pass (Governs)**
Pullout	4894	27712	0.18	Pass

Shear	Factored Load, V_{ua} (lb)	Design Strength, øV_n (lb)	Ratio	Status
Steel	320	2569	0.12	Pass
T Concrete breakout x+	**1280**	**7292**	**0.18**	**Pass (Governs)**
|| Concrete breakout y+	**640**	**10983**	**0.06**	**Pass (Governs)**
Pryout	1280	47814	0.03	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Utilization Ratio	Permissible	Status
Sec. 17.8.3	0.94	0.18	93.0%	1.0	Pass

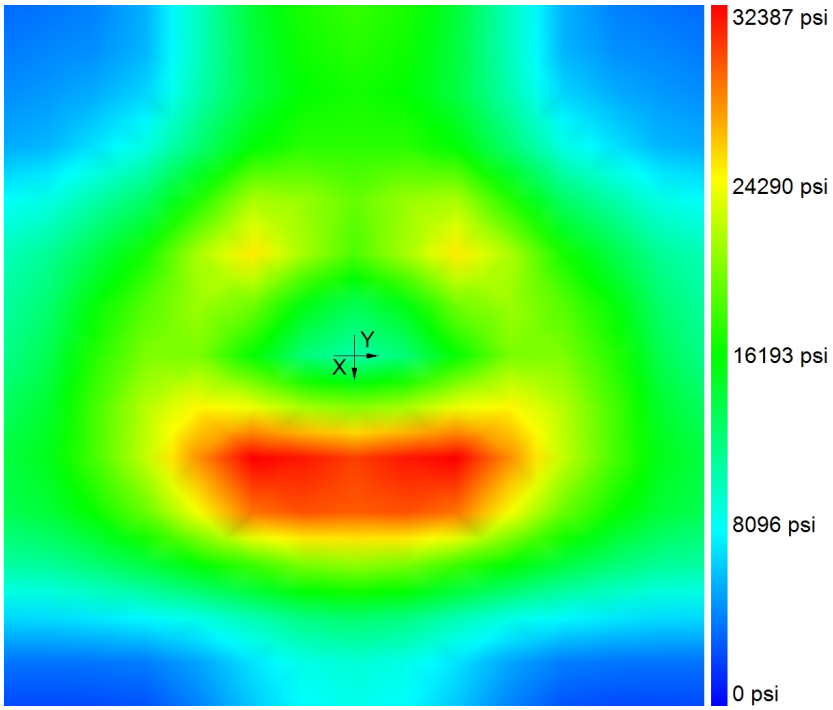
PAB4 (1/2"Ø) with hef = 8.000 inch meets the selected design criteria.

Base Plate Thickness

Required base plate thickness: 0.5 inches

Steel	**36000 psi**
Maximum stress	**32387 psi**
Calculated plate thickness	**0.800 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

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.5.2 for tension need not be satisfied – designer to verify.

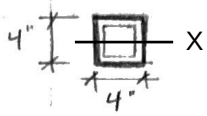
- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

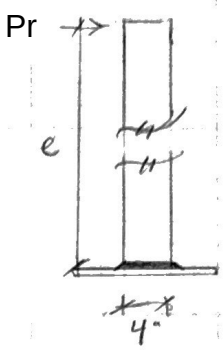
- Designer must exercise own judgement to determine if this design is suitable.

Base plate - column weld connection

Loading: Va = Pr = 1280 lb (per load combination 8)

e = 6.75 ft





$I_o = I_x$

$V_p = \frac{P_r}{\ell}$

$V_e = \frac{P_r \cdot e_x}{I_o}$ → $R = \sqrt{(V_p + V_e)^2 + H_e^2}$

$H_e = \frac{P_r \cdot e_y}{I_o}$

$\bar{I} = 4\ell = 4(4in) = 16in$ (in x-direction)

$I_x = 2\left(\frac{\ell^3}{12}\right) + 2(\ell)\left(\frac{\ell}{2}\right)^2 = 2\left(\frac{4^3}{12}\right) + 2(4)\left(\frac{4}{2}\right)^2 = 42.67\ in^3$

$V_p = \frac{P_r}{\bar{I}} = \frac{1280lb}{16in} = 80lb/in = 0.08\ kip/in.$

$V_e = 0$

$H_e = \frac{P_r \cdot e \cdot y}{I_x} = \frac{(1.28k)(6.75ft \cdot 12in/1ft)(4in/2)}{42.67in^3} = 4.86\ k/in$

$R = \sqrt{(0.08)^2 + (4.86)^2} = 4.86\ k/in$

$D = R/q = \frac{4.86\ k/in}{0.928\ k/in\ per\ 1/16in.} = 5.24\ sixteenths = 0.33\ in.$

→ We weld size of 3/8 in

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SSF STRUCTURAL ENGINEERING

PROJECT: Cheshire Pergola, Mercer Island, WA

DATE: 01/29/2025

PROJ. # AD

DESIGN G-10

SHEET

(F2) Pergola column footing Design

Loading:

Load combination 8: 1.0D + 0.7Ev + 0.7Emh

→ P: 7511 lb. 1.0D: 1.0[150pcf (4.5ft·4.5ft·2.0ft) + 13.91plf (6.75ft) + 1343 lb) = 7511 lb.

M: 8640 lb-ft. 0.7Emh: 1280 lb.

V: 1280 lb

Load combination 9: 1.0D + 0.525Ev + 0.525Emh + 0.75L + 0.75S

→ P: 10523 lb. 1.0D: 1.0[150 pcf (4.5 ft·4.5 ft·2.0 ft) + 13.91plf (6.75ft) + 1343) = 7511 lb.

M: 6480 lb-ft. 0.525Emh: 0.525(1.25((1025 lb/0.7)) = 960 lb

V: 960 lb 0.75L: 0.75(17851b) = 13391b.

0.75S: 0.75(2231b) = 1673 lb

Load combination 10: 0.6D + 0.7Ev + 0.7Emh

→ P: 4507 lb. 0.6D: 0.6[150pcf·4.5ft·4.5ft·2.0ft) + 13.91plf(6.75ft) + 1343) = 4507 lb.

M: 8640 lb-ft. 0.7Emh: 1280 lb.

V: 1280 lb.

Bearing Pressure:

→ Load combination 10 controls

$$e = \frac{M_{OT}}{P_T} = \frac{8640\ lb\text{-}ft}{4507\ lb.} = 1.91\ ft.$$

L/6: 4.5 ft/6 = 0.75ft → e > L/6

Therefore: $q_{max} = \frac{2 \cdot P}{3 \cdot B \cdot x}$ where $x = L/2 - e = 4.5\ ft/2 - 1.91\ ft = 0.33$

$$q_{max} = \frac{2(4507\ lb.)}{3(4.5ft)(0.33)} = 2023\ psf < 1.33(2000\ psf)$$ per geotech report

2023 psf < 2660 psf OK.

→ 4'-6" □ x 2'-0" dp. conc. ftg

Check sliding: (worst case)

$$\frac{P_{resisting}}{V_a} = \frac{\mu(P_{ftg} + P_{column} + P_{roof}) + P_{passive}}{1280\ lb.} = \frac{0.3\left(4800\ lb + 941\ lb + 1343\ lb\right) + 250pcf(2.2ft)}{1280\ lb.}$$

$$F.S. = \frac{P_R}{V_a} = \frac{2871\ lb}{1250\ lb} = 2.2 > 1.5 \quad OK$$

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STRUCTURAL ENGINEERING

PROJECT Cheshire Pergola, Mercer Island, WA

DATE 01/29/2025

PROJ. # 1D

DESIGN

SHEET