STRUCTURAL CALCULATIONS

Venetia Valley Elementary School Restroom Addition

SAN RAFAEL, CALIFORNIA

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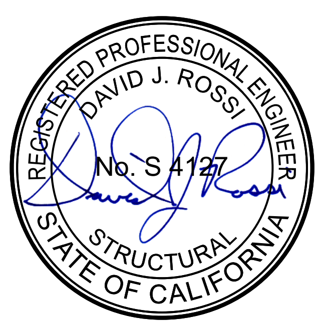

Prepared for:

SVA Architects

MAY 19th, 2023

DESCRIPTION OF PROJECT

This project includes adding a single-story restroom to the Ventia Valley elementary school in San Rafael, California.

The structure has a gable roof supported by 2x10 joists spaced at 24" o.c. A single glulam beam ridge beam is supporting the joists. The gable roof has a 5' long overhang. Roof is supported by wood stud walls along the perimeter.

Exterior wood shear walls are the main lateral force resisting system in the structure in both directions.

The structure is supported on 18" wide x 24" deep grade beams along the perimeter of the building.

A1 - LOADS & DESIGN INFORMATION

kpff Consulting Engineers
45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

Venetia Valley Restroom	by: rk		sheet no:
	date:		
SVA			job no:
	2200173		2200173.00
			Rev. No. 120.05

DESIGN CRITERIA

Design conforms to the Caliornia Building Code, 2022 Edition.

LIVE LOADS

Roofs (flat)		20	psf	

WIND ANALYSIS

Basic Wind Speed	V_{3S} =	92	mph	(ASCE 7-16 Figure 26.5)
Exposure Category	=	C		(ASCE 7-16 Section 26.7.3)
Internal Pressure Coefficient	GC_{pi} =	0.18		(ASCE 7-16 Table 26.13-1)

SEISMIC ANALYSIS

Static Lateral Force Procedure

Risk Category	=	II	(ASCE 7-16 Table 1.5-1)
Site Classification	=	C	(ASCE 7-16 Section 20.3)
Importance Factor	I_e =	1.0	(ASCE 7-16 Table 1.5-2)

Long Direction

Long Direction Lateral System		Bearing Wall: Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	
Response Modifier	R =	6.5	(ASCE 7-16 Table 12.2-1)
Overstrength Factor	Ω_0 =	3	(ASCE 7-16 Table 12.2-1)

Short Direction

Short Direction Lateral System		Bearing Wall: Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	
Response Modifier	R =	6.5	(ASCE 7-16 Table 12.2-1)
Overstrength Factor	Ω_0 =	3	(ASCE 7-16 Table 12.2-1)

Site Coefficients

Short Period Spectral Acceleration	S_S =	1.500	g
Spectral Acceleration at 1 sec.	S_1 =	0.600	g
Short-Period Site Coefficient	F_a =	1.20	
Long-Period Site Coefficient	F_v =	1.7	

Design Parameters

Short-Period Spectral Acceleration	S_{DS} =	1.200	g
Long-Period Spectral Acceleration	S_{D1} =	0.680	g

Seismic Analysis Procedure: Equiv. Lateral Force Procedure (ELF) - ASCE7 Sec 12.8
Hand Calculation

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San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

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FOUNDATION CRITERIA

Reference: CBC2022 - TABLE 1806.2

AT GRADE

Maximum Soil Pressure:

Dead	1,500	psf	
Dead + Live	1,500	psf	
Dead + Live + Lateral	1,800	psf	

Passive Earth Pressure:

Equivelent Fluid Weight	100	pcf	(FS=1.5)

Coefficient of Friction:	0.35		(FS=1.5)

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45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

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location:		date:	09/12/22
client:	SVA		
			2200173.00
			Rev. No. 120.05

EQUIVALENT LATERAL FORCE PROCEDURE CBC 2022 & ASCE 7-16, Section 12.8:

Input Data:

Risk Category =	II	(ASCE 7 Table 1.5-1)	
Importance Factor, I_e =	1.00	(ASCE 7 Table 1.5-2)	
Soil Site Class =	D	Default (No Soils Report)	ASCE 7 Ch. 20.3 Table 20.3-1
Site Latitude & Longitude	38.001° N	-122.52484	Coordinates based on site address
Spectral Accel., S_s =	1.500	g (Geotech Report or USGS Hazard Maps)	
Spectral Accel., S_1 =	0.600	g (Geotech Report or USGS Hazard Maps)	
Structure Height, h_n =	13.750	ft	
No. of Seismic Levels =	1		
Seismic Resist. System =	A15	ASCE Table 12.2-1	
Long-Period Trans., T_L =	12	sec, ASCE7 Fig. 22-14	
Fundamental Period, T =	**N/A**	sec, based on analysis per ASCE 12.8.2	

Seismic force-resisting system = Bearing Wall: Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance (ASCE Table 12.2-1)

Site Coefficients:

Ground Motion Procedure	**Code Spectrum**	ASCE 7 Sec 11.4.8	
Short-period factor, F_a =	1.2	ASCE 7 Table 11.4-1	**Fa=1.2 min per ASCE7 Sec 11.4.4**
Long-period factor, F_v =	1.7	ASCE 7 Table 11.4-2	

Maximum Spectral Response Accelerations:

S_{MS} =	**1.800**	$S_{MS} = F_a S_s$ (ASCE Eqn. 11.4-1)
S_{M1} =	1.020	$S_{M1} = F_v S_1$ (ASCE Eqn. 11.4-2)

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	1.200	S_{DS} = 2/3 S_{MS}, ASCE Eq. 11.4-3
S_{D1} =	0.680	S_{D1} = 2/3 S_{M1}, ASCE Eq. 11.4-4

Calculate Ts: T_s =	0.567	For using Code Spectrum: $T_s = S_{D1} / S_{DS,}$ ASCE7 Sec 11.4.6
Governing Period, T =	0.143	sec, $T = T_a \leq T_{max}$ (ASCE 12.8.2), see calcs below
§11.4.8 to use Code Spectrum:	**Site D - Period T ≤ 1.5Ts: Cs = Eqn. 12.8-2**	
Site-Specific Spectrum Exception:	Exception 2a	Code-Spectrum Permitted: ASCE7-16 Sec 11.4.8

Seismic Design Category:

Category (based on S_{DS}) =	D	ASCE Table 11.6-1
Category (based on S_{D1}) =	D	ASCE Table 11.6-2
Category ($S_1 \geq 0.75$) =	N.A.	ASCE Section 11.6
Sec 11.6 condition triggered?	No	
Seismic Design Category =	**D**	Governed by the most critical of all category cases above

Fundamental Period:

Period Coefficient, C_t =	0.020	ASCE Table 12.8-2 for "All other structural systems"
Period Exponent, x =	0.75	ASCE Table 12.8-2 for "All other structural systems"
Approx. Period, T_a =	0.143	sec, $T_a = C_t h_n^x$ (ASCE Eq. 12.8-7)
Upper Limit Coef., C_u =	1.400	ASCE Table 12.8-1
Period max., T_{max} =	0.200	sec, $T_{max} = C_u * T_a$ (ASCE 12.8.2)
Fundamental Period, T =	0.143	sec, $T = T_a \leq T_{max}$ (ASCE 12.8.2)

Seismic Design Factors and Coefficients:

Response Modifier, R =	6.5	ASCE Table 12.2-1
Overstrength Factor, Ω_0 =	3	ASCE Table 12.2-1 (See note d for flexible diaphragms)
Defl. Amplif. Factor, C_d =	4	ASCE Table 12.2-1
C_S (Short Period) =	0.185	$C_S = S_{DS}/(R/I)$, ASCE Eq. 12.8-2, page 89
C_s ($T \leq T_L$) =	0.733	$C_S = S_{D1}/((R/I)*T)$ for $T \leq T_L$ (ASCE Eq. 12.8-3, p. 101)
C_s ($T > T_L$) =	N/A	$C_S = S_{D1}*T_L/((R/I)*T^2)$ for $T > T_L$ (ASCE Eq. 12.8-4, p. 101)
C_s (SSS alternate) =	N/A	$C_{S,alternate}$ = Site-Specific S_a/(R/I) per ASCE Sec 21.4
$C_{s(min)}$ =	0.053	$C_{S,min} = 0.044 S_{DS} I$ or $0.5*S_1/(R/I)$ if $S_1 \geq 0.6g$ (Eq. 12.8-5&6, p. 101)
C_s (§11.4.8 exception) =	0.185	Exception 2a for Site D - Period T ≤ 1.5Ts: Cs = Eqn. 12.8-2
Use: C_S =	**0.185**	$C_{S,min} \leq C_S \leq C_{S,max}$, $C \geq 0.01$

kpff Consulting Engineers
45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

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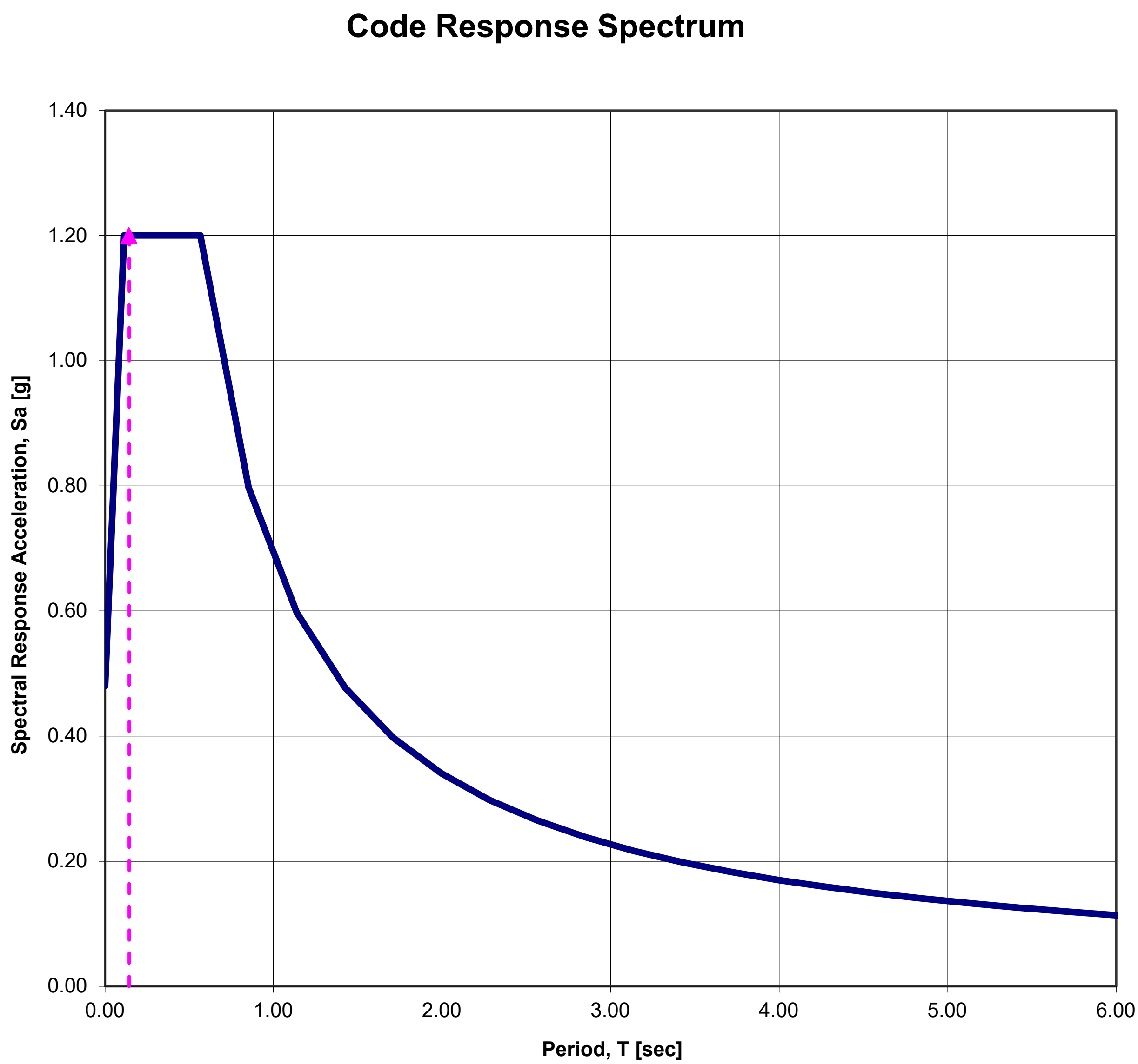
client:	SVA	date:	job no: 2200173.00
			Rev. No. 120.05

CODE RESPONSE SPECTRUM

S_{DS} =	1.200	g, S_{DS} = 2/3 S_{MS}, ASCE 7 Eq. 11.4-3
S_{D1} =	0.680	g, S_{D1} = 2/3 S_{M1}, ASCE 7 Eq. 11.4-4
T =	0.143	sec
T_0 =	0.11	sec, T_0 = 0.2 S_{D1}/S_{DS}, ASCE 7 Section 11.4.5
T_S =	0.57	sec, T_S = S_{D1}/S_{DS}, ASCE 7 Section 11.4.5
T_L =	12.00	sec, ASCE 7 Figure 22-16
Sa(T) =	1.20	g



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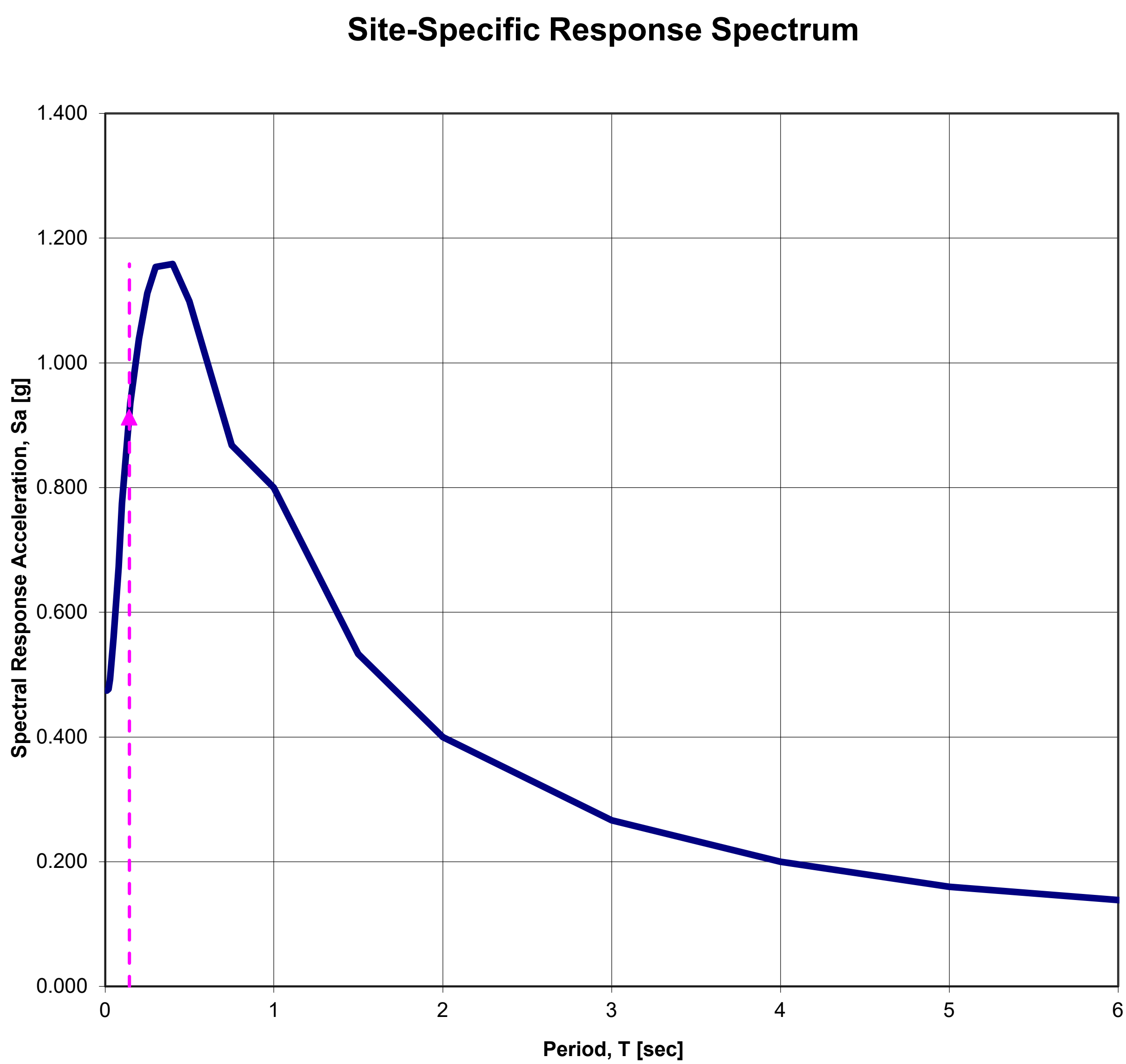
SITE-SPECIFIC RESPONSE SPECTRUM

S_{DS} =	1.200	g, S_{DS} = 2/3 S_{MS}, ASCE 7 Eq. 11.4-3		
S_{D1} =	0.680	g, S_{D1} = 2/3 S_{M1}, ASCE 7 Eq. 11.4-4		
T =	0.143	sec	0.143	0
Sa(T) =	0.91	g	0.143	1.159



C1 - LATERAL FRAMING

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45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

Number of seismic of Levels =	1
Total Structure Height, h_n =	13.75
Seismic Base Shear Coefficient, C_s =	0.185

W =	**60**	**kips**	(ASCE 7-16 Section 12.7.2)
$W*C_S$ = V =	**11.1**	**kips**	**LRFD**
$0.7*W*C_S$ = V =	**8**	**kips**	**ASD**

level	floor type	story heights (ft)	story heights (ft)	uniform loads (psf)	area (ft^2)	added loads (k)	total weight (k)
1	Roof	13.75	13.75	51	1184		60
Σ =		13.75					60

VERTICAL DISTRIBUTION OF LATERAL SEISMIC FORCE

Base Shear = 8 kips

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Fundamental Period, T = 0.143 sec

Vertical Distribution of Seismic Forces:

Distribution Exponent, k = 1.00 k = 1 for T ≤ 0.5 sec., k = 2 for T ≥ 2.5 sec.
Linear interpolation: k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < k < 2.5 sec.

Lateral Force at Any Level: $F_x = C_{vx}*V$, ASCE 7 Eq. 12.8-11 (p. 72)

Vertical Distribution Factor: $C_{vx} = W_x*h_x^k/(\Sigma W_i*h_i^k)$, ASCE 7 Eq. 12.8-12 (p. 73)

Diaphragm Design Forces:

$C_{px,max}$ = 0.480 $C_{px,max} = 0.4*S_{DS}*I$, ASCE 7 Eq. 12.10-3 (p. 75)

$C_{px,min}$ = 0.240 $C_{px,min} = 0.2*S_{DS}*I$, ASCE 7 Eq. 12.10-2 (p. 75)

Vertical Distribution Factor: $F_{px} = (\Sigma F_i / \Sigma W_i) * w_{px}$, ASCE 7 Eq. 12.10-1 (p. 75)

Seismic Level x	h_x (ft.)	h_x^k (ft.)	Weight, W_x (kips)	W_x*h^k (ft-kips)	C_{vx} (%)	Shear, F_x (kips)	Σ Story Shears	C_{px} (%)	F_{px} (kips)
1	13.75	13.75	60	824	100%	7.7	8	13%	14
Σ =			60	824	1.000	7.74			

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45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

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SHEAR WALL DESIGN

Plywood Sheathing

Level (ft)	F_x (kips)	Area, A_x (ft^2)
1	8	1184

EW DIRECTION

Zone	Level	Trib Area (ft^2)	$F_{x, zone}$ (kips)	$V_{x, zone}$ (kips)	L_{wall} (ft)	$V_{wall, ASD}$ (plf)	Shear Wall Type	Wall Capacity	DCR (%)
A-1	1	592	3.87	3.87	22.5	172	A	310	55%
A-2	1	592	3.87	3.87	18	215	A	310	69%

NS DIRECTION

Zone	Level	Trib Area (ft^2)	$F_{x, zone}$ (kips)	$V_{x, zone}$ (kips)	L_{wall} (ft)	$V_{wall, ASD}$ (plf)	Shear Wall Type	Wall Capacity	DCR (%)
1	1	592	3.87	3.87	10	387	B	460	84%
2	1	592	3.87	3.87	10	387	B	460	84%

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45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

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client: SVA		job no:	
		2200173	

SHEAR WALL OVERTURNING CHECKS

[ASD]

Notes:

ΣM_{OT} = Sum of overturning moments at story, ASD
ΣM_R = Sum of resisting moments at story
M_{NET} = net overturning moment using the LC: E - (0.6 -0.14 S_{DS})*DL
T = resulting uplift: M_{NET}/L_{eff}
$L_{w,eff}$ = Effective width of wall for resisting couple, taken as length of wall minus 2 ft
(1) Leff updated manually to account for gravity posts
T_{LRFD} = calculated using LC: 1.4E - (0.9-0.2S_{DS})*DL, used for tiedown anchorage design

S_{DS} = 1.20

Determined in other worksheets

For a sample hand calculation of how the values in each row are calculated, see following pages.

EAST-WEST DIRECTION (X)

Shear Wall	H_{wall} (ft)	L_{wall} (ft)	v_{wall} (plf)	ΣV_{WALL} (kip)	ΣM_{OT} (k-ft)	Weight (plf)	Trib DL (plf)	ΣM_R (k-ft)	M_{NET} (k-ft)	$L_{w,eff}$ (ft)	T (kip)	T_{LRFD} (kip)
WA_1	13.8	22.5	172	3.9	53	234	42	70	23	20.5	**1.1**	1.4
WA_2	13.8	10.0	215	2.2	30	234	42	14	24	8.0	**3.0**	4.0

NORTH-SOUTH DIRECTION (Y)

Shear Wall	H_{wall} (ft)	L_{wall} (ft)	v_{wall} (plf)	ΣV_{WALL} (kip)	ΣM_{OT} (k-ft)	Weight (plf)	Trib DL (plf)	ΣM_R (k-ft)	M_{NET} (k-ft)	$L_{w,eff}$ (ft)	T (kip)	T_{LRFD} (kip)
W1_1	10.0	5.0	387	1.9	19	170	231	5	17	3.0	**5.7**	7.9
W1_2	10.0	5.0	387	1.9	19	170	231	5	17	3.0	**5.7**	7.9

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[ASD]

SHEAR WALL HOLDOWN & POST DESIGN

Typ Max DCR =	0.97
ATS DSA Reduction =	0.80

Notes:

S_{DS} = 1.20
T_E = resulting uplift: M_{NET}/L_{eff}
T = C_E - gravity load, using [LC10: E-(0.6-0.14 S_{DS})D]
C_E = M_{OT}/L_{EFF}
C = C_E + gravity load, max of [LC8: (1+0.14 S_{DS})D+E] & [LC9: (1+0.14 S_{DS})D+0.75(L+E)]

(1) Seismic Load E is at ASD level
(2) Max DCR manually changed to be higher than design DCR.
(3) See following pages for Simpson Strong Tie Catalog

POST CAPACITY

	Post	Grade	Wall Thickness	L_e (ft)	C_{all} (lbs)	T_{all} (lbs)
< 27 ft	PSL 7 x 11 7/8 *	PSL 2.0E		25.8	30407	275583
	PSL 7 x 9 1/2 *	PSL 2.0E	8x	25.8	24326	220466
	PSL 7 x 7	PSL 1.8E		25.8	10391	140907
	PSL 5 1/4 x 11 7/8 *	PSL 2.0E		25.8	13913	206687
	PSL 5 1/4 x 9 1/2 *	PSL 2.0E	6x	25.8	11130	165350
	PSL 5 1/4 x 7 *	PSL 1.8E		25.8	7863	105680
< 19 ft	PSL 5 1/4 x 11 7/8 *	PSL 2.0E		17.8	25902	211030
	PSL 5 1/4 x 9 1/2 *	PSL 2.0E		17.8	20722	168824
	PSL 5 1/4 x 7 *	PSL 1.8E	6x	17.8	16305	107900
	6x12 *	No. 1		17.8	17233	68310
	6x10 *	No. 2		17.8	14236	56430
	6x8 *	No. 2		17.8	11281	44550
< 17.5 ft	PSL 5 1/4 x 11 7/8 *	PSL 2.0E		16.3	29261	212073
	6x12 *	No. 1	6x	16.3	20350	68310
	6x10 *	No. 2		16.3	16810	56430
	6x8 *	No. 2		16.3	13334	44550
	4x6		6x		7017	

SST HOLDOWN CAPACITY [3]

HDU Type	Model	Rod Dia. [in]	Capacity [lbs]	0.8*T_{all} [kip]	d_a [in]
HD-7	(2) HDU19	1 1/4	38720	30.976	0.180
HD-6	(2) HDU14	1	28890	23.112	0.172
HD-5	(2) HDU11	1	22350	17.880	0.137
HD-4	(2) HDU8	7/8	15740	12.592	0.113
HD-3	HDU14	1	14445	11.556	0.172
HD-2	HDU11	1	11175	8.940	0.137
HD-1	HDU8	7/8	7870	6.296	0.113
			38140	30512.000	

EAST-WEST DIRECTION (X)

			GRAVITY LOADS		COMPRESSION				TENSION						
Shear Wall	Wall Thickness	Height	Add'l DL	Add'l LL	C_E [lbs]	C (lbs)	Post	DCR	T_E [kip]	T [kip]	$d_{a,POST}$ [in]	*INCLUDE DL?*	HDU Type	DCR	**NOTES**
WA_1	6x	13.75			1197.3	1197.3	4x6	0.17	1.1	1.1	0.01	NO	HD-1	0.18	
WA_2	6x	13.75			295.6	295.6	4x6	0.04	3.0	3.0	0.02	NO	HD-1	0.47	

NORTH-SOUTH DIRECTION (Y)

			GRAVITY LOADS		COMPRESSION				TENSION						
Shear Wall	Wall Thickness	Height	Add'l DL	Add'l LL	C_E [kip]	C (kip)	Post	DCR	T_E [kip]	T [kip]	$d_{a,POST}$ [in]	*INCLUDE DL?*	HDU Type	DCR	**NOTES**
W1_1	6x	10.5			96.8	96.8	4x6	0.01	5.7	5.7	0.02	NO	HD-1	0.91	

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45 Fremont Street, 28th Floor
San Francisco, California 94105
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TIEDOWN ANCHORAGE DESIGN

ACI318-19 Ch. 17

Tiedown anchors are sized using the ductile anchor method of ACI 318-14, 17.2.34.3 9(a)

INPUT:

f'c =	3	ksi	[Concrete Strength]
Fy =	36	ksi	[Anchor yield strength]
Fu =	58	ksi	[Anchor ultimate strength]
Fy,bar =	60	ksi	[Rebar yield strength]
Φ =	0.75		[Breakout, Steel element]
Φ =	0.70		[Pullout & Pryout]

Anchor Steel Strength:
$N_{sa} = Ae*Fu$

Pullout Strength:
$N_{pn} = \Psi_{c,p}*8*A_{brg}*f'c$
$A_{brg} = l_{washer}^2 - \pi Ø_{A.B.}^2/4$

Breakout Strength:
Based on anchor reinforcement capacity
$N_n = As*Fy$

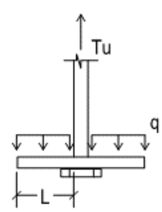
ANCHORAGE CHECKS:

HDU Type	N_{ua} (kip) ASD	N_{ua} (kip) LRFD	$Ø_{A.B.}$ (in)	Plate Washer (in)	No. of Legs	Reinf. #	Steel strength: N_{sa} (kip)	Steel strength: $N_{ua}/\Phi N_{sa}$	Pullout Strength: N_{pn} (kip)	Pullout Strength: $N_{ua}/\Phi N_{pn}$	Breakout strength: N_n (kip)	Breakout strength: $N_{ua}/\Phi N_n$	Ductility Check: (a) $N_{ua}/1.2N_{sa}$	Ductility Check: (b) N_{ua}/N_c	Ductility Check: (a)>(b)?
HD-1	8	11	0.875	3	2	4	26.8	0.55	151.2	0.10	24.0	0.61	0.34	0.07	O.K.

PLATE WASHER CHECK:
Check plate washer for bending

T_u =	11	kip
w =	3	in
t =	1	in
F_y =	36	ksi
$Ø_{A.B.}$ =	1	in
A_{brg} =	8.21	in^2
$L = (w-Ø_{A.B.})/2$ =	1.00	in
$Z = w*t^2/4$ =	0.75	in^3

$q_u = T_u/A_{brg}$ =	1.3	ksi
$M_u = w*q_u L^2/2$ =	2.01	kip-in
Φ =	0.9	
$\Phi M_n = \Phi Fy*Z$ =	24.3	kip-in
DCR =	0.08	OK



D1 - FOUNDATION

Foundation Design

Design Criteria

Building Dimension: $L := 34.8\ ft$ $B := 22.5\ ft$ $H := 12.5\ ft$

Building Weight: $W := 60\ kip$

Grade Beam Dimension: $Bgb := 1.5\ ft$ $Dgb := 2\ ft$

Slab on Grade Thickness: $tslab := 5\ in$

Concrete Property: $f'c := 3000\ psi$ $\gamma c := 150\ pcf$

Seismic

Seismic Parameter: $Sds := 1.2$ $\rho := 1.0$ $Cs := 0.185$ $\Omega o := 3$

Base Shear: $V\_E := 1 \cdot Cs \cdot W = 11.1\ kip$

Overturning Moment: $M\_E := V\_E \cdot H = 138.75\ ft \cdot kip$

Seismic governs the design for foudnation.

Overturnning Seismic governs the design for overturning.

Consider the entire building as one unit

Min. effect ASD load combination (0.6 - 0.14Sds)D + 0.7E

$OTM := 0.7 \cdot M\_E = 97.125\ kip \cdot ft$

Total weight of the building including foundation:

$Ptotal := W + \gamma c \cdot ((2 \cdot L + 4 \cdot (B - Bgb)) \cdot Dgb \cdot Bgb + (B + Bgb) \cdot (L + Bgb) \cdot tslab) = 183.57\ kip$

Resisting Moment:

$Mo := (0.6 - 0.14\ Sds) \cdot Ptotal \cdot \dfrac{(B + Bgb)}{2} = 951.627\ kip \cdot ft$

Factor of Safety against overturning moment:

$FS := \dfrac{Mo}{OTM} = 9.798$ OK

kpff Consulting Engineers	Project: Venetia Valley	By: RK	Page D - 3
45 Fremont Street, 28th Floor		date: 5/15/2023	
Foundations design conform to California	Client: SVA	Job No.: 2200173	
(415) 989-1004 Fax (415) 989-1552		Rev. No. 121.01	

Gra = 1500 psf DL + LL   CBC2022 - TABLE 1806.2

Late = 100 pcf

Late = 130 psf   Basic Load Combinations

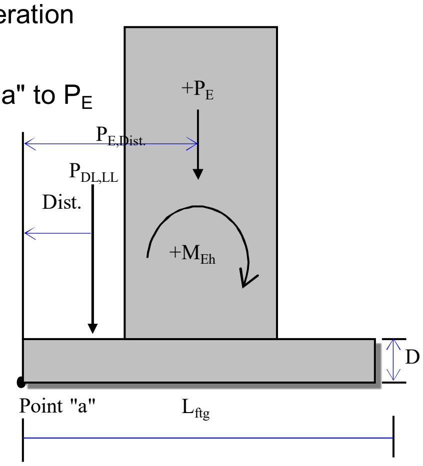
Footing Information

Weight   60 kips

Location   Foundation/Second Floor

Earthquake Loads

S_{DS} =	1.20	g	Seismic Design Spectral Acceleration
P_E =	7	kips	Vertical seismic force resultant
$P_{E,Dist.}$ =	12	ft	Horizontal distance from point "a" to P_E
$M_{E,PE}$ =	86	k-ft	(taken at Point a)
M_{Eh} =	53	k-ft	(taken at Point a)
$M_{E,Total}$ =	140	k-ft	(taken at Point a)



Foundation Dimensions

Component	Depth (ft)	Width (ft)	Length (ft)	Notes
Footing	2.00	1.50	24.00	
Surcharge	1.00			

Gravity Loads: Forces and Moments (D + L)

Load	Dead (k)	Live (k)	Dist. (ft)	Dead (k-ft)	Live (k-ft)	Notes
Footing	11		12.00	130		
Surcharge	4		12.00	52		
Roof LL		1	12.00		6	
P_1	5	4	13.25	66	52	
P_2						
P_3						
P_4						
P_5						
P_6						
P_7						
P_8						
P_9						
P_{10}						
Sum	20	4	----	248	52	Sum excludes L_r

Allowable Soil Bearing Pressures

Soil density ρ =   120   pcf   CBC2022 - TABLE 1806.2

Add displaced soil?   No

Loads	$q_{allow,net}$	$q_{allow,gross}$
D + L	1500 psf	1500 psf
D + L + E	1500 psf	1500 psf

W/O added displaced soil (depth accounted in $q_{allow,net}$)

$q_{allow,gross} = q_{allow,net}$

With additional displaced soil increase

$q_{allow,gross} = q_{allow,net}$ + (displaced soil wt/ftg area)

Values for q_{allow} obtained from soils report

Required Footing Width

W_{reqd} = (if inside kern, e ≤ L_{ftg} / 6) (if outside kern, e > L_{ftg} / 6)

$$W_{reqd} = \frac{R}{q_{allow} L_{ftg}}\left(1 + \frac{6e}{L_{ftg}}\right) \qquad \text{(Eq. 1)}$$

$$W_{reqd} = \frac{2R}{3q_{allow}(L-x)}, \text{ or } \frac{2R}{3q_{allow}x} \qquad \text{(Eq. 2)}$$

(x > L/2) (x ≤ L/2)

ASD Load Combinations for Soil Bearing

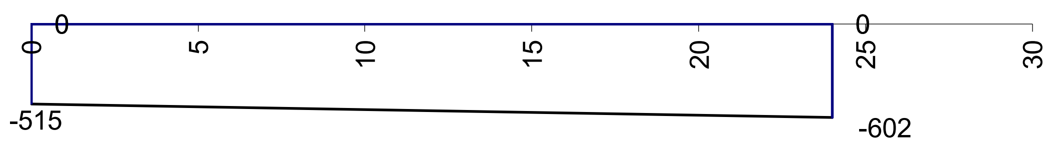
Load Combination 1: D (CBC 2022 Eq. 16-8) **Check: OK**

M_a =	248	kip-ft	Sum of the moments about left end
R =	20	kips	Factored load reaction
x =	12.31	ft	Resultant location from left end, x = M_a / R
e =	0.31	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.60**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 602 at dist. = 24.00 ft

q_{min} = 515 at dist. = 0.00 ft

Soil pressure distribution



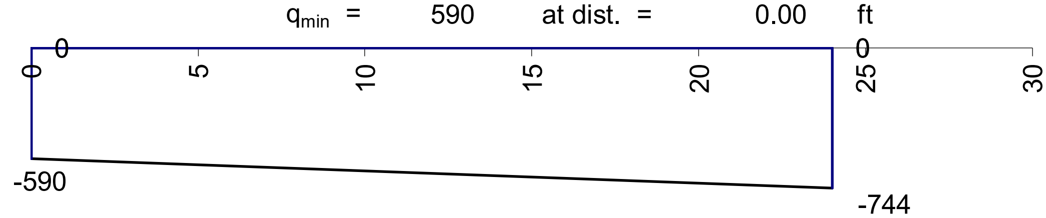
Load Combination 2: D + L (CBC 2022 Eq. 16-9) **Check: OK**

M_a =	299	kip-ft	Sum of the moments about left end
R =	24	kips	Factored load reaction
x =	12.46	ft	Resultant location from left end, x = M_a / R
e =	0.46	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.74**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 744 at dist. = 24.00 ft

q_{min} = 590 at dist. = 0.00 ft

Soil pressure distribution



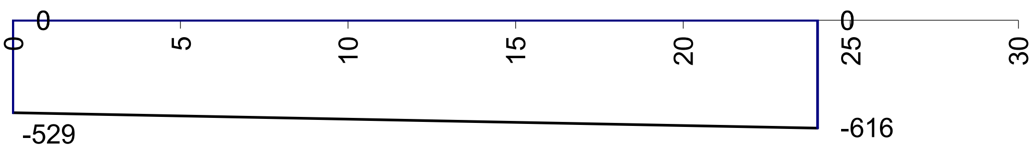
Load Combination 3: D + Lr (CBC 2022 Eq. 16-10) **Check: OK**

M_a =	254	kip-ft	Sum of the moments about left end
R =	21	kip	Factored load reaction
x =	12.30	ft	Resultant location from left end, x = M_a / R
e =	0.30	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.62**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 616 at dist. = 24.00 ft

q_{min} = 529 at dist. = 0.00 ft

Soil pressure distribution



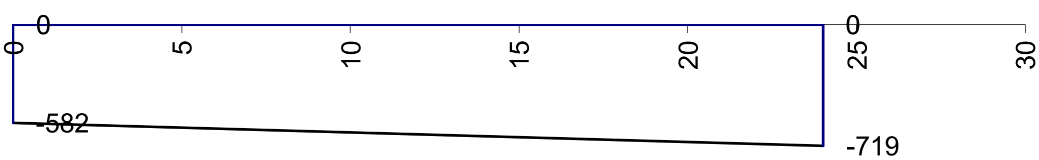
Load Combination 4: D + 0.75(L) + 0.75(Lr) (CBC 2022 Eq. 16-11) **Check: OK**

M_a =	291	kip-ft	Sum of the moments about left end
R =	23	kips	Factored load reaction
x =	12.42	kips	Resultant location from left end, x = M_a / R
e =	0.42	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.72**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 719 at dist. = 24.00 ft

q_{min} = 582 at dist. = 0.00 ft

Soil pressure distribution



Load Combination 8a: (1 + 0.14 SDS) D + 0.7(Eh) (CBC 2022 Eq. 16-12) **Check: OK**

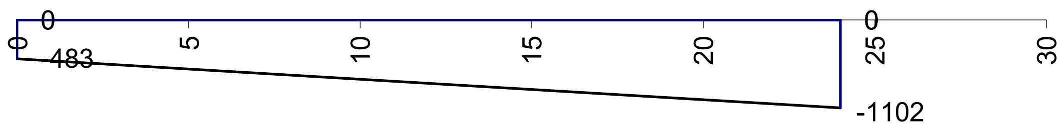
M_a =	387	kip-ft	Sum of the moments about left end
R =	29	kips	Factored load reaction
x =	13.56	ft	Resultant location from left end, x = M_a / R
e =	1.56	ft	Eccentricity, e = x - (L_{ftg} / 2)

e_{kern} =	4.00	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**1.10**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 1102 at dist. = 24.00 ft

q_{min} = 483 at dist. = 0.00 ft

Soil pressure distribution



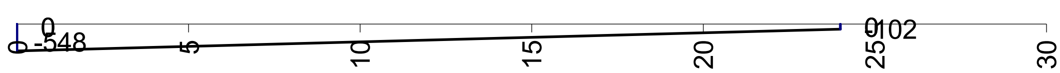
Load Combination 8b: (1 - 0.14 SDS) D - 0.7(Eh) (CBC 2022 Eq. 16-12) **Check: OK**

M_a =	108	kip-ft	Sum of the moments about left end
R =	12	kips	Factored load reaction
x =	9.26	ft	Resultant location from left end, $x = M_a / R$
e =	-2.74	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	4.00	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.55**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 548 at dist. = 0.00 ft

q_{min} = 102 at dist. = 24.00 ft

Soil pressure distribution



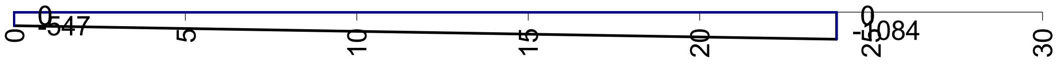
Load Combination 9a: (1 + 0.105 SDS) D + 0.525(Eh) + 0.75(L) (CBC 2022 Eq. 16-14) **Check: OK**

M_a =	391	kip-ft	Sum of the moments about left end
R =	29	kips	Factored load reaction
x =	13.32	ft	Resultant location from left end, $x = M_a / R$
e =	1.32	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	4.00	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**1.08**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 1084 at dist. = 24.00 ft

q_{min} = 547 at dist. = 0.00 ft

Soil pressure distribution

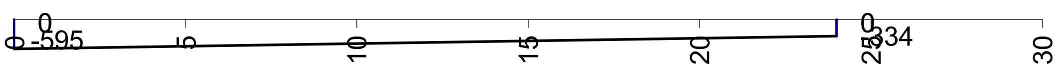


Load Combination 9b: (1 - 0.105 SDS) D - 0.525(Eh) + 0.75(L) (CBC 2022 Eq. 16-14) **Check: OK**

M_a =	182	kip-ft	Sum of the moments about left end
R =	17	kips	Factored load reaction
x =	10.88	ft	Resultant location from left end, x = M_a / R
e =	-1.12	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.60**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 595 at dist. = 0.00 ft
q_{min} = 334 at dist. = 24.00 ft

Soil pressure distribution

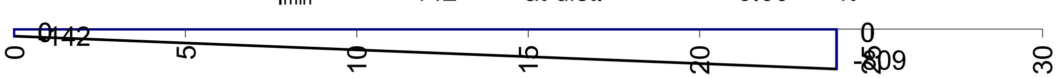


Load Combination 10a: (0.6)(D) + 0.7(Eh) (CBC 2022 Eq. 16-16) **Check: OK**

M_a =	253	kip-ft	Sum of the moments about left end
R =	17	kips	Factored load reaction
x =	14.80	ft	Resultant location from left end, x = M_a / R
e =	2.80	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.81**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 809 at dist. = 24.00 ft
q_{min} = 142 at dist. = 0.00 ft

Soil pressure distribution

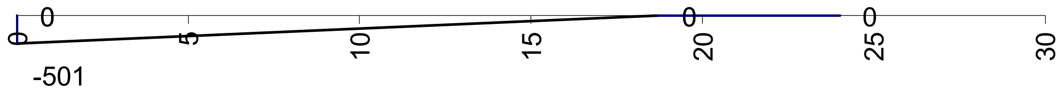


Load Combination 10b: (0.6)(D) - 0.7(Eh) (CBC 2022 Eq. 16-16) **Check: OK**

M_a =	44	kip-ft	Sum of the moments about left end
R =	7	kips	Factored load reaction
x =	6.24	ft	Resultant location from left end, x = M_a / R
e =	-5.76	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	4.00	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	No		Resultant is inside kern if e ≤ e_{kern}

W_{reqd} = 0.50 ft Required width (see Eq. 1 and Eq. 2 above)
Sufficient width? OK

For W_{ftg} = 1.50 ft., q_{max} = 501 at dist. = 0.00 ft
q_{min} = 0 at dist. = 18.73 ft



Soil pressure distribution

Grade Beam - Short Side:

Consider the grade beam in the long direction resisting half of the seismic moment.

Grade Beam Length in Long Direction: $Lgb := B + Bgb = 24\ ft$

From previous calculation, the highest uniform bearing load is 837 psf from load combination: (1 + 0.14 SDS) D + 0.7(Eh)
Using conversion factor of 1.4 to convert the max soil pressure into LRFD

Max. Soil Pressure: $qu := 1.4 \cdot 837\ psf = 1171.8\ psf$

Uniform Linear Loads Along Grade Beam: $wu := qu \cdot Bgb = 1.758\ klf$

Max. Momont per AISC Table 3-22c: $Mu := 0.1 \cdot wu \cdot Lgb^2 = 101.244\ ft \cdot kip$

Max. Shear per AISC Table 3-22c: $Vu := 1.1 \cdot wu \cdot Lgb = 46.403\ kip$

<u>Flexural</u>
Use (3) #8bars at bottom & #4 Ties

$db := 1\ in$ $A\_bar := 0.79\ in^2$ $n := 3$ $\lambda := 1.0$ $dbv := 0.5\ in$

$d := Dgb - 3\ in - 0.5 \cdot db = 20.5\ in$ $fy := 60\ ksi$

$$Asmin := \max\left(\frac{3 \cdot \sqrt{3000}\ psi \cdot Bgb \cdot d}{fy}, 200\ psi \cdot Bgb \cdot \frac{d}{fy}\right) = 1.23\ in^2$$

$As := n \cdot A\_bar = 2.37\ in^2 > Asmin = 1.23\ in^2$ OK

$$a := \frac{As \cdot fy}{0.85\ Bgb \cdot f'c} = 3.098\ in$$

$$\phi Mn := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{a}{2}\right) = 202.112\ kip \cdot ft > Mu = 101.244\ kip \cdot ft \text{ OK}$$

<u>Shear</u>
Provide shear reinforcing: #4 @ 12" o.c.

$$\phi Vc := 2 \cdot 0.75 \cdot d \cdot 1\ ft \cdot \lambda \cdot \sqrt{3000}\ psi = 20.211\ kip$$

$s := 12\ in$ $Av := 0.2\ in^2$

$$\phi Vs := \frac{2 \cdot Av \cdot fy \cdot d}{s} = 41\ kip$$

$\phi V := \phi Vs + \phi Vc = 61.211\ kip > Vu = 46.403\ kip$ OK

Gra = 1500 psf DL + LL CBC2022 - TABLE 1806.2

Late = 100 pcf

Late = 130 psf Basic Load Combinations

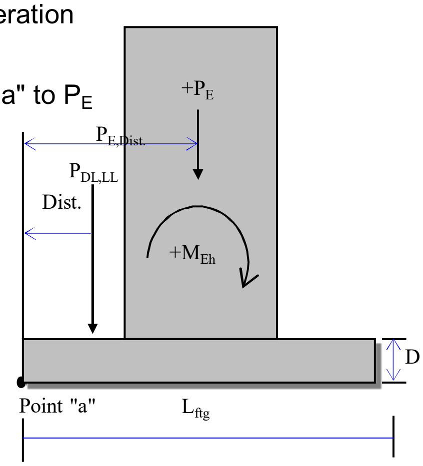
Footing Information

Weight 60 kips

Location Foundation/Second Floor

Earthquake Loads

S_{DS} =	1.20	g	Seismic Design Spectral Acceleration
P_E =	7	kips	Vertical seismic force resultant
$P_{E,Dist.}$ =	18	ft	Horizontal distance from point "a" to P_E
$M_{E,PE}$ =	131	k-ft	(taken at Point a)
M_{Eh} =	38	k-ft	(taken at Point a)
$M_{E,Total}$ =	169	k-ft	(taken at Point a)



Foundation Dimensions

Component	Depth (ft)	Width (ft)	Length (ft)	Notes
Footing	2.00	1.50	36.50	
Surcharge	1.00			

Gravity Loads: Forces and Moments (D + L)

Load	Dead (k)	Live (k)	Dist. (ft)	Dead (k-ft)	Live (k-ft)	Notes
Footing	16		18.25	300		
Surcharge	7		18.25	120		
Roof LL		1	18.25		9	
P_1						
P_2						
P_3						
P_4						
P_5						
P_6						
P_7						
P_8						
P_9						
P_{10}						
Sum	23	0	----	420	0	Sum excludes L_r

Allowable Soil Bearing Pressures

Soil density ρ = 120 pcf CBC2022 - TABLE 1806.2

Add displaced soil? No

Loads	$q_{allow,net}$	$q_{allow,gross}$
D + L	1500 psf	1500 psf
D + L + E	1500 psf	1500 psf

W/O added displaced soil (depth accounted in $q_{allow,net}$)

$q_{allow,gross} = q_{allow,net}$

With additional displaced soil increase

$q_{allow,gross} = q_{allow,net}$ + (displaced soil wt/ftg area)

Values for q_{allow} obtained from soils report

Required Footing Width

W_{reqd} = (if inside kern, e ≤ L_{ftg} / 6)

$$W_{reqd} = \frac{R}{q_{allow} L_{ftg}} \left(1 + \frac{6e}{L_{ftg}} \right) \qquad \text{(Eq. 1)}$$

(if outside kern, e > L_{ftg} / 6)

$$W_{reqd} = \frac{2R}{3q_{allow}(L-x)}, \text{ or } \frac{2R}{3q_{allow}x} \qquad \text{(Eq. 2)}$$

(x > L/2) (x ≤ L/2)

ASD Load Combinations for Soil Bearing

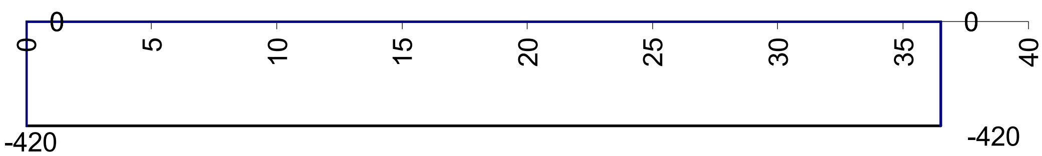
Load Combination 1: D (CBC 2022 Eq. 16-8) **Check: OK**

M_a =	420	kip-ft	Sum of the moments about left end
R =	23	kips	Factored load reaction
x =	18.25	ft	Resultant location from left end, x = M_a / R
e =	0.00	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	6.08	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.42**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 420 at dist. = 36.50 ft

q_{min} = 420 at dist. = 0.00 ft

Soil pressure distribution



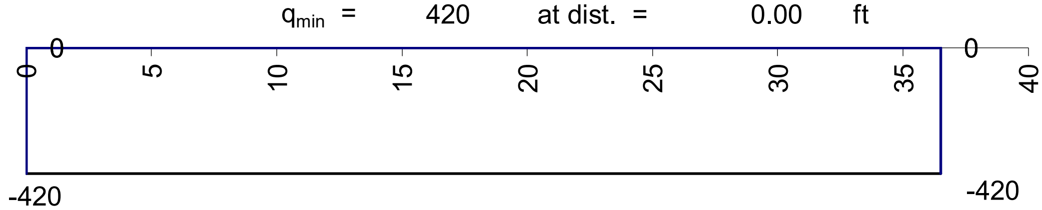
Load Combination 2: D + L (CBC 2022 Eq. 16-9) **Check: OK**

M_a =	420	kip-ft	Sum of the moments about left end
R =	23	kips	Factored load reaction
x =	18.25	ft	Resultant location from left end, x = M_a / R
e =	0.00	ft	Eccentricity, e = x - (L_{ftg} / 2)
e_{kern} =	6.08	ft	Kern distance, e_{kern} = L_{ftg} / 6
Inside kern?	Yes		Resultant is inside kern if e ≤ e_{kern}
W_{reqd} =	**0.42**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 420 at dist. = 36.50 ft

q_{min} = 420 at dist. = 0.00 ft

Soil pressure distribution



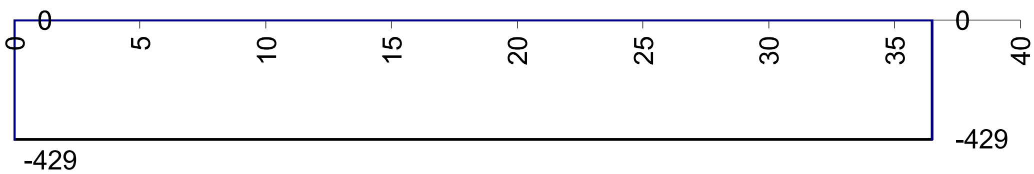
Load Combination 3: D + Lr (CBC 2022 Eq. 16-10) **Check: OK**

M_a =	429	kip-ft	Sum of the moments about left end
R =	23	kip	Factored load reaction
x =	18.25	ft	Resultant location from left end, $x = M_a / R$
e =	0.00	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.43**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 429 at dist. = 36.50 ft

q_{min} = 429 at dist. = 0.00 ft

Soil pressure distribution



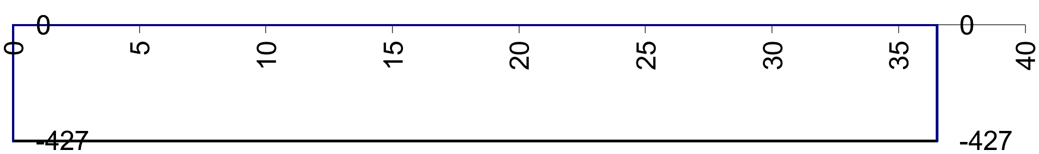
Load Combination 4: D + 0.75(L) + 0.75(Lr) (CBC 2022 Eq. 16-11) **Check: OK**

M_a =	427	kip-ft	Sum of the moments about left end
R =	23	kips	Factored load reaction
x =	18.25	kips	Resultant location from left end, $x = M_a / R$
e =	0.00	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.43**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 427 at dist. = 36.50 ft

q_{min} = 427 at dist. = 0.00 ft

Soil pressure distribution



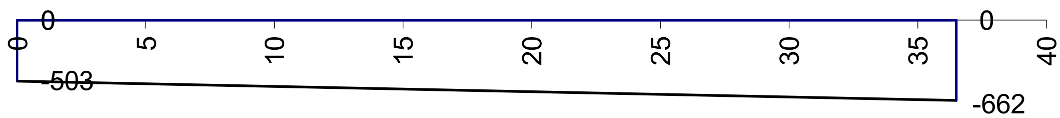
Load Combination 8a: (1 + 0.14 SDS) D + 0.7(Eh) (CBC 2022 Eq. 16-12) **Check: OK**

M_a =	609	kip-ft	Sum of the moments about left end
R =	32	kips	Factored load reaction
x =	19.08	ft	Resultant location from left end, $x = M_a / R$
e =	0.83	ft	Eccentricity, $e = x - (L_{ftg} / 2)$

e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.66**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 662 at dist. = 36.50 ft
q_{min} = 503 at dist. = 0.00 ft

Soil pressure distribution

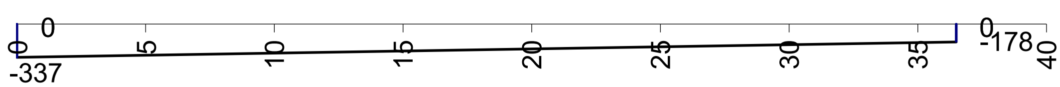


Load Combination 8b: (1 - 0.14 SDS) D - 0.7(Eh) (CBC 2022 Eq. 16-12) **Check: OK**

M_a =	231	kip-ft	Sum of the moments about left end
R =	14	kips	Factored load reaction
x =	16.36	ft	Resultant location from left end, $x = M_a / R$
e =	-1.89	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.34**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 337 at dist. = 0.00 ft
q_{min} = 178 at dist. = 36.50 ft

Soil pressure distribution

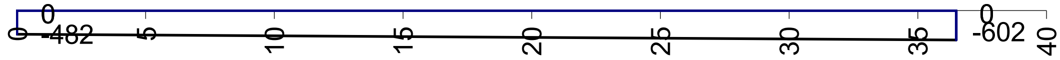


Load Combination 9a: (1 + 0.105 SDS) D + 0.525(Eh) + 0.75(L) (CBC 2022 Eq. 16-14) **Check: OK**

M_a =	561	kip-ft	Sum of the moments about left end
R =	30	kips	Factored load reaction
x =	18.92	ft	Resultant location from left end, $x = M_a / R$
e =	0.67	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.60**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 602 at dist. = 36.50 ft
q_{min} = 482 at dist. = 0.00 ft

Soil pressure distribution



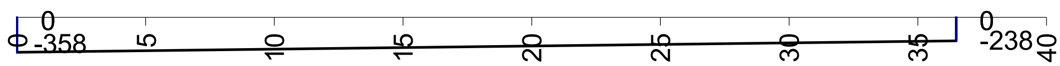
Load Combination 9b: (1 - 0.105 SDS) D - 0.525(Eh) + 0.75(L) (CBC 2022 **Eq. 16-14)** **Check: OK**

M_a =	278	kip-ft	Sum of the moments about left end
R =	16	kips	Factored load reaction
x =	17.03	ft	Resultant location from left end, $x = M_a / R$
e =	-1.22	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.36**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 358 at dist. = 0.00 ft

q_{min} = 238 at dist. = 36.50 ft

Soil pressure distribution



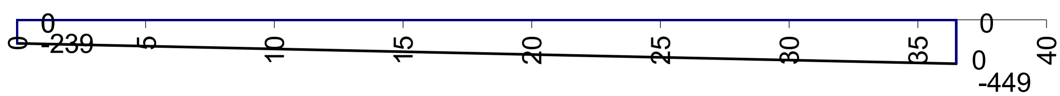
Load Combination 10a: (0.6)(D) + 0.7(Eh) (CBC 2022 **Eq. 16-16)** **Check: OK**

M_a =	379	kip-ft	Sum of the moments about left end
R =	19	kips	Factored load reaction
x =	20.11	ft	Resultant location from left end, $x = M_a / R$
e =	1.86	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$
W_{reqd} =	**0.45**	**ft**	Required width (see Eq. 1 and Eq. 2 above)
Sufficient width?	**OK**		

For W_{ftg} = 1.50 ft., q_{max} = 449 at dist. = 36.50 ft

q_{min} = 239 at dist. = 0.00 ft

Soil pressure distribution



Load Combination 10b: (0.6)(D) - 0.7(Eh) (CBC 2022 **Eq. 16-16)** **Check: OK**

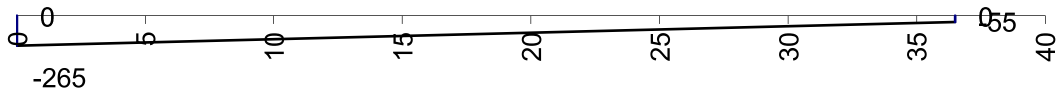
M_a =	125	kip-ft	Sum of the moments about left end
R =	9	kips	Factored load reaction
x =	14.25	ft	Resultant location from left end, $x = M_a / R$
e =	-4.00	ft	Eccentricity, $e = x - (L_{ftg} / 2)$
e_{kern} =	6.08	ft	Kern distance, $e_{kern} = L_{ftg} / 6$
Inside kern?	Yes		Resultant is inside kern if $e \le e_{kern}$

W_{reqd} = 0.27 ft Required width (see Eq. 1 and Eq. 2 above)

Sufficient width? OK

For W_{ftg} = 1.50 ft., q_{max} = 265 at dist. = 0.00 ft

q_{min} = 55 at dist. = 36.50 ft



Soil pressure distribution

Grade Beam - Long Side:

Consider the grade beam in the long direction resisting half of the seismic moment.

Grade Beam Length in Long Direction: $Lgb := L + Bgb = 36.3\ ft$

From previous calculation, the highest uniform bearing load is 613 psf from load combination: (1 + 0.14 SDS) D + 0.7(Eh)
Using conversion factor of 1.4 to convert the max soil pressure into LRFD

Max. Soil Pressure: $qu := 1.4 \cdot 613\ psf = 858.2\ psf$

Uniform Linear Loads Along Grade Beam: $wu := qu \cdot Bgb = 1.287\ klf$

Max. Momont per AISC Table 3-22c: $Mu := 0.1 \cdot wu \cdot Lgb^2 = 169.626\ ft \cdot kip$

Max. Shear per AISC Table 3-22c: $Vu := 1.1 \cdot wu \cdot Lgb = 51.402\ kip$

Flexural
Use (3) #8bars at bottom & #4 Ties

$db := 1\ in$ $\quad A\_bar := 0.79\ in^2$ $\quad n := 3$ $\quad \lambda := 1.0$ $\quad dbv := 0.5\ in$

$d := Dgb - 3\ in - 0.5 \cdot db = 20.5\ in$ $\quad fy := 60\ ksi$

$$Asmin := \max\left(\frac{3 \cdot \sqrt{3000}\ psi \cdot Bgb \cdot d}{fy}, 200\ psi \cdot Bgb \cdot \frac{d}{fy}\right) = 1.23\ in^2$$

$As := n \cdot A\_bar = 2.37\ in^2 > Asmin = 1.23\ in^2$ OK

$$a := \frac{As \cdot fy}{0.85\ Bgb \cdot f'c} = 3.098\ in$$

$$\phi Mn := 0.9 \cdot As \cdot fy \cdot \left(d - \frac{a}{2}\right) = 202.112\ kip \cdot ft > Mu = 169.626\ kip \cdot ft$$ OK

Shear
Provide shear reinforcing: #4 @ 12" o.c.

$\phi Vc := 2 \cdot 0.75 \cdot d \cdot 1\ ft \cdot \lambda \cdot \sqrt{3000}\ psi = 20.211\ kip$

$s := 12\ in$ $\quad Av := 0.2\ in^2$

$$\phi Vs := \frac{2 \cdot Av \cdot fy \cdot d}{s} = 41\ kip$$

$\phi V := \phi Vs + \phi Vc = 61.211\ kip > Vu = 51.402\ kip$ OK

E1 MISCELLANEOUS

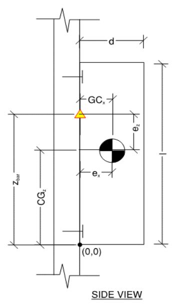
kpff Consulting Engineers
45 Fremont Street, 28th Floor
San Francisco, California 94105
(415) 989-1004 FAX (415) 989-1552

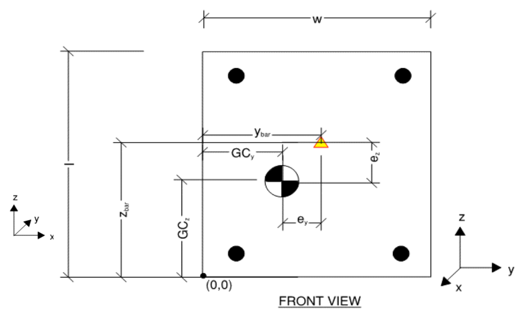
project:	
location:	
client:	
EQUIPMENT ANCHORAGE-WALL MOUNTED	

EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

Equipment ID = Water Heater
Equipment Description = Water Heater - Wall Mounted
Base Material = Wall Mounted
W_p = weight of equipment = 230 lb
l = overall length = 25 in
w = overall width = 18 in
d = overall depth = 18 in





Center of Mass Location

CG_x = dist. in x-dir from Origin = 9 in
CG_y = dist. in y-dir from Origin = 9 in
CG_z = dist. in z-dir from Origin = 16.6666667 in
n = # of anchors = 4
m = # of fastener @ each anchor = 1

Seismic Accelerations

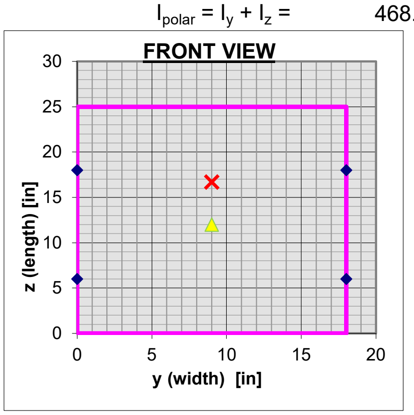
a_p = amplification factor = 1
R_p = response factor = 2.5
S_{DS} = spectral acceleration = 1.2
$\Omega_0 Fh$=ASCE 7-10: 13.3-1, 13.3-2, 13.3-3 = 264.96 lb = 1.16 Wp ***LRFD DESIGN***
$\Omega_0 F_v$ = vertical force = $0.2 S_{DS} W_p$ = 110.4 lb = 0.48 Wp ***LRFD DESIGN***

Ip = Imp. Factor = 1
Ω_o = Over. Factor = 2
Apply Ω_0= Yes

z= 1 ft
h= 1 ft

Bolt Group Properties

x_{bar} = y-dist. of C.R.from Origin = 0 in
y_{bar} = y-dist. of C.R.from Origin = 9 in
z_{bar} = z-dist. of C.R.from Origin = 12 in
e_x = x-eccen. of C.G.from C.R. = 9 in
e_y = y-eccen. of C.G.from C.R. = 0 in
e_z = z-eccen. of C.G.from C.R. = 4.66666667 in
$I_y = \sum(d_y^2)$ = 324.0 in^2
$I_z = \sum(d_z^2)$ = 144.0 in^2
$I_{polar} = I_y + I_z$ = 468.0 in^2



ANCHOR LOCATIONS (UP TO 20 ANCHORS)

Bolt #	Y (W)	Z (L)	d_y	d_z	d_y^2	d_z^2	d^2
1	0.00	6.00	-9.0	-6.0	81.0	36.0	117.0
2	0.00	18.00	-9.0	6.0	81.0	36.0	117.0
3	18.00	6.00	9.0	-6.0	81.0	36.0	117.0
4	18.00	18.00	9.0	6.0	81.0	36.0	117.0
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

SUM=$\sum d^2$ 468

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San Francisco, California 94105
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EQUIPMENT ANCHORAGE-WALL MOUNTED	

EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

FASTENER FORCES: LOAD CASE 1: SEISMIC LOADING IN X-X DIRECTION + Z-Z DIRECTION

LRFD LOAD CASE:	1.2	DL+	1.0	EL
ASD LOAD CASE:	1.0	DL+	0.7	EL

	LRFD	ASD	
$M_x = (F_v{*}EL_{factor}+W_p{*}DL_{factor}){*}{-}e_y=$	**0**	**0**	lb*in
$M_y = (F_v{*}EL_{factor}+Wp{*}DL_{factor}){*}e_x+(F_h{*}EL_{factor}){*}{-}e_z=$	**2241.1**	**1900**	lb*in
$M_z = (F_h{*}EL_{factor}){*}e_y=$	**0**	**0**	lb*in

$T_{fastener}=F_{ph}{*}EL_{factor}/(m{*}n)+M_y{*}d_z/I_z-M_z{*}d_y/I_y$ (negative tension indicates compression)

$V_{y\ fastener}=-M_x{*}d_z/I_p$

$V_{z\ fastener}=M_x{*}d_y/I_p-(W_p{*}DL_{factor}+F_{pv}{*}EL_{factor})/(m{*}n)$

$V_r=(v_y{}^2+V_z{}^2)^{0.5}$

			LRFD LEVEL				ASD LEVEL			
Bolt #	d_y (in)	d_z (in)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)
1	-9	-6	-27.14	0.00	-96.60	96.60	-32.80	0.00	-76.82	76.82
2	-9	6	159.62	0.00	-96.60	96.60	125.53	0.00	-76.82	76.82
3	9	-6	-27.14	0.00	-96.60	96.60	-32.80	0.00	-76.82	76.82
4	9	6	159.62	0.00	-96.60	96.60	125.53	0.00	-76.82	76.82
5										
20										

MAX LOAD CASE 1:		LRFD		ASD	
(X-X LOADING)	Bolt 2	Tension (T)=	159.62 lbs	Tension (T)=	125.53 lbs
	Bolt 1	Shear (V)=	96.60 lbs	Shear (V)=	76.82 lbs

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EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

FASTENER FORCES: LOAD CASE 2: SEISMIC LOADING IN Y-Y DIRECTION + Z-Z DIRECTION

LRFD LOAD CASE:	1.2	DL+	1.0	EL
ASD LOAD CASE:	1.00	DL+	0.7	EL

	LRFD	ASD	
$M_x = (F_v*EL_{factor}+W_p*DL_{factor})*-e_y+(F_h*EL_{factor})*-e_z=$	-1236	-865.5	lb*in
$M_y = (F_v*EL_{factor}+W_p*DL_{factor})*e_x=$	3477.6	2765.5	lb*in
$M_z = (F_h*EL_{factor})*e_x=$	2384.6	1669.2	lb*in

$T_{fastener}=M_y*d_z/I_z-M_z*d_y/I_y$ (negative tension indicates compression)

$V_{y\ fastener}=F_h*EL_{factor}/(m*n)-M_x*d_z/I_p$

$V_{z\ fastener}=M_x*d_y/I_p-(W_p*DL_{factor}+F_{pv}*EL_{factor})/(m*n)$

$V_r=(v_y^2+V_z^2)^{0.5}$

			LRFD LEVEL				ASD LEVEL			
Bolt #	d_y (in)	d_z (in)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)
1	-9	-6	-78.66	50.39	-72.82	88.55	-68.86	35.27	-60.18	69.75
2	-9	6	211.14	82.09	-72.82	109.74	161.60	57.46	-60.18	83.21
3	9	-6	-211.14	50.39	-120.38	130.50	-161.60	35.27	-93.46	99.90
4	9	6	78.66	82.09	-120.38	145.71	68.86	57.46	-93.46	109.72
5										
6										
20										

MAX LOAD CASE 2:		LRFD		ASD	
(X-X LOADING)	Bolt 2	Tension (T)=	211.14 lbs	Tension (T)=	161.60 lbs
	Bolt 4	Shear (V)=	145.71 lbs	Shear (V)=	109.72 lbs

GOVERNING LOADS:			LRFD		ASD	
	Load Case 2	Bolt 2	Tension (T)=	211.14 lbs	Tension (T)=	161.60 lbs
	Load Case 2	Bolt 4	Shear (V)=	145.71 lbs	Shear (V)=	109.72 lbs

CHANNEL NUT LOAD DATA
FOR 1⅝" (41 MM) WIDTH SERIES CHANNEL

UNISTRUT

MAXIMUM ALLOWABLE PULL-OUT AND SLIP LOADS

Channel Nut Size/Thread	Gage	Channel	Allowable Pull-Out Strength		Resistance to Slip		Torque	
			Lbs	kN	Lbs	kN	Ft Lbs	N•m
¾" - 10	12	P1000 P3000 P5000 P5500	2500	11.1	1700	7.6	125*	170
⅝" - 11			2500	11.1	1500	6.7	100*	135
½" - 13			2000	8.9	1500	6.7	50	70
7⁄16" - 14			1400	6.2	1000	4.4	35	50
⅜" - 16			1000	4.4	800	3.6	19	25
5⁄16" - 18			800	3.6	500	2.2	11	15
¼" - 20			600	2.7	300	1.3	6	8

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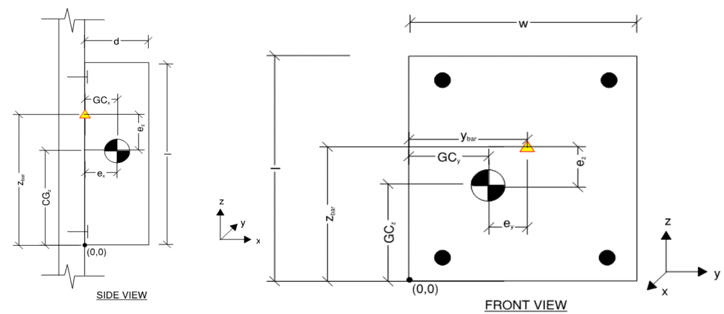
EQUIPMENT ANCHORAGE-WALL MOUNTED

EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

Equipment ID = Water Heater
Equipment Description = Water Heater - Wall Mounted
Base Material = Wall Mounted
W_p = weight of equipment = 25 lb
l = overall length = 12 in
w = overall width = 6 in
d = overall depth = 12 in



Center of Mass Location

CG_x = dist. in x-dir from Origin = 6 in
CG_y = dist. in y-dir from Origin = 3 in
CG_z = dist. in z-dir from Origin = 8 in
n = # of anchors = 2
m = # of fastener @ each anchor = 1

Seismic Accelerations

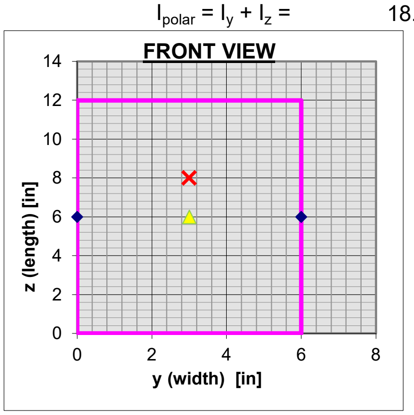
a_p = amplification factor = 1
R_p = response factor = 2.5
S_{DS} = spectral acceleration = 1.2
$\Omega_0 Fh$=ASCE 7-10: 13.3-1, 13.3-2, 13.3-3 = 28.8 lb = 1.16 Wp ***LRFD DESIGN***
$\Omega_0 F_v$ = vertical force = $0.2S_{DS}W_p$ = 12 lb = 0.48 Wp ***LRFD DESIGN***

Ip = Imp. Factor = 1
Ω_o = Over. Factor = 2
Apply Ω_0= Yes

z= 1 ft
h= 1 ft

Bolt Group Properties

x_{bar} = y-dist. of C.R.from Origin = 0 in
y_{bar} = y-dist. of C.R.from Origin = 3 in
z_{bar} = z-dist. of C.R.from Origin = 6 in
e_x = x-eccen. of C.G.from C.R. = 6 in
e_y = y-eccen. of C.G.from C.R. = 0 in
e_z = z-eccen. of C.G.from C.R. = 2 in
$I_y = \sum(d_y^2)$ = 18.0 in^2
$I_z = \sum(d_z^2)$ = 0.0 in^2
$I_{polar} = I_y + I_z$ = 18.0 in^2



ANCHOR LOCATIONS (UP TO 20 ANCHORS)

Bolt #	Y (W)	Z (L)	d_y	d_z	d_y^2	d_z^2	d^2
1	0.00	6.00	-3.0	0.0	9.0	0.0	9.0
2	6.00	6.00	3.0	0.0	9.0	0.0	9.0
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							

SUM=$\sum d^2$ 18

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EQUIPMENT ANCHORAGE-WALL MOUNTED

EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

FASTENER FORCES: LOAD CASE 1: SEISMIC LOADING IN X-X DIRECTION + Z-Z DIRECTION

LRFD LOAD CASE:	1.2	DL+	1.0	EL
ASD LOAD CASE:	1.0	DL+	0.7	EL

	LRFD	ASD	
$M_x = (F_v{*}EL_{factor}+W_p{*}DL_{factor}){*}{-}e_y=$	**0**	**0**	lb*in
$M_y = (F_v{*}EL_{factor}+Wp{*}DL_{factor}){*}e_x+(F_h{*}EL_{factor}){*}{-}e_z=$	**194.4**	**160.08**	lb*in
$M_z = (F_h{*}EL_{factor}){*}e_y=$	**0**	**0**	lb*in

$T_{fastener}=F_{ph}{*}EL_{factor}/(m{*}n)+M_y{*}d_z/I_z-M_z{*}d_y/I_y$ (negative tension indicates compression)

$V_{y\ fastener}=-M_x{*}d_z/I_p$

$V_{z\ fastener}=M_x{*}d_y/I_p-(W_p{*}DL_{factor}+F_{pv}{*}EL_{factor})/(m{*}n)$

$V_r=(v_y^{\ 2}+V_z^{\ 2})^{0.5}$

			LRFD LEVEL				ASD LEVEL			
Bolt #	d_y (in)	d_z (in)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)
1	-3	0	#DIV/0!	0.00	-21.00	21.00	#DIV/0!	0.00	-16.70	16.70
2	3	0	#DIV/0!	0.00	-21.00	21.00	#DIV/0!	0.00	-16.70	16.70
3										
4										
5										
20										

		LRFD		ASD
MAX LOAD CASE 1:				
(X-X LOADING)				lbs
	Bolt 1	Shear (V)= 21.00 lbs		Shear (V)= 16.70 lbs

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EQUIPMENT ANCHORAGE-WALL MOUNTED

REFERENCE: 2022 CBC, ASCE 7-16 Chapter 13

FASTENER FORCES: LOAD CASE 2: SEISMIC LOADING IN Y-Y DIRECTION + Z-Z DIRECTION

LRFD LOAD CASE:	1.2	DL+	1.0	EL
ASD LOAD CASE:	1.00	DL+	0.7	EL

	LRFD	ASD	
$M_x = (F_v{*}EL_{factor}+W_p{*}DL_{factor}){*}{-e_y}+(F_h{*}EL_{factor}){*}{-e_z}=$	**-57.6**	**-40.32**	lb*in
$M_y = (F_v{*}EL_{factor}+W_p{*}DL_{factor}){*}e_x=$	**252**	**200.4**	lb*in
$M_z = (F_h{*}EL_{factor}){*}e_x=$	**172.8**	**120.96**	lb*in

$T_{fastener}=M_y{*}d_z/I_z-M_z{*}d_y/I_y$ (negative tension indicates compression)

$V_{y\ fastener}=F_h{*}EL_{factor}/(m{*}n)-M_x{*}d_z/I_p$

$V_{z\ fastener}=M_x{*}d_y/I_p-(W_p{*}DL_{factor}+F_{pv}{*}EL_{factor})/(m{*}n)$

$V_r=(v_y^2+V_z^2)^{0.5}$

			LRFD LEVEL				ASD LEVEL			
Bolt #	d_y (in)	d_z (in)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)	T (lbs)	V_y (lbs)	V_z (lbs)	V_r (lbs)
1	-3	0	#DIV/0!	14.40	-11.40	18.37	#DIV/0!	10.08	-9.98	14.18
2	3	0	#DIV/0!	14.40	-30.60	33.82	#DIV/0!	10.08	-23.42	25.50
3										
4										
5										
6										
20										

MAX LOAD CASE 2:		LRFD			ASD	
(X-X LOADING)	#DIV/0!	Tension (T)=	#DIV/0!	lbs	Tension (T)=	#DIV/0! lbs
	Bolt 2	Shear (V)=	33.82	lbs	Shear (V)=	25.50 lbs

GOVERNING LOADS:		LRFD		ASD	
					lbs
Load Case 2	Bolt 2	Shear (V)=	33.82 lbs	Shear (V)=	25.50 lbs

CHANNEL NUT LOAD DATA
FOR 1⅝" (41 MM) WIDTH SERIES CHANNEL

UNISTRUT

MAXIMUM ALLOWABLE PULL-OUT AND SLIP LOADS

Channel Nut Size/Thread	Gage	Channel	Allowable Pull-Out Strength		Resistance to Slip		Torque	
			Lbs	kN	Lbs	kN	Ft Lbs	N•m
¾" - 10	12	P1000 P3000 P5000 P5500	2500	11.1	1700	7.6	125*	170
⅝" - 11			2500	11.1	1500	6.7	100*	135
½" - 13			2000	8.9	1500	6.7	50	70
7/16" - 14			1400	6.2	1000	4.4	35	50
⅜" - 16			1000	4.4	800	3.6	19	25
5/16" - 18			800	3.6	500	2.2	11	15
¼" - 20			600	2.7	300	1.3	6	8

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project	by		sheet no.
location	date		
client	job no.		

Max Weight = 135 lbs

per ASCE7-16 Table 13.6-1 , plumbing
ap = 1
Rp = 2.5
Omega = 2
SDS = 1.2
z/h = 0.9

Fp = 0.4ap*SDS*Wp*(1+2z/h)/(Rp/Ip) = 0.53Wp
= 72 lbs

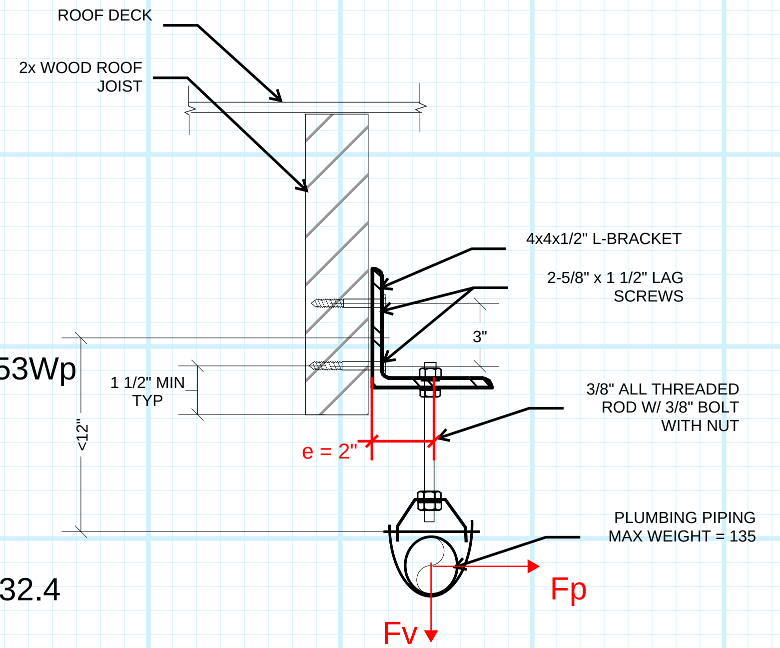
Fv = 0.2*SDS*Wp = 32.4 lbs

Threaded Rod Tensile Strength:
60ksi x pi x (3/16)^2 = 6.6 kips >> 135lbs+32.4 lbs --> ok!

Withdrawal Check:
M_Fp = Fp*12" + (Wp + Fv)*2" = 1200 lb-inch
T_Fp = 1200 lb-in / 3" = 400 lbs pull out

5/8" x 1 1/2" lag screw withdrawal capacity per NDS Table 12.2A = 447 per inch x 0.9 (permanent load) = 402.6 lbs > 400 lbs --> ok!

Shear Check:
Vu = 135+33=168 lbs
5/8" x 1 1/2" lag screw shear capacity per NDS Table 12K = 440 lbs x 0.9 x 0.5 = 198 lbs --> ok!
Minimum end distance for C_delta = 0.5 is 2D = 2x5/8" = 1.25" < 1.5"



PLUMBING PIPE SUPPORT DETAIL

WIND LOADING ANALYSIS - Roof Components and Cladding

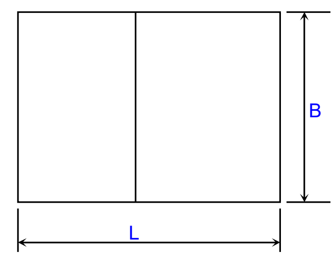
Per ASCE 7-16 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°

Using Part 1 & 3: (Chapter 30.3) for Low-Rise Buidings and (Chapter 30.5) for Buildings > 60 ft

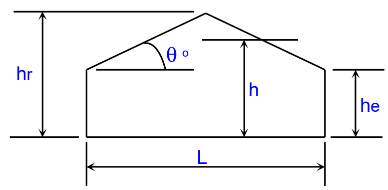
Job Name:		Subject:			
Job Number:		Originator:		Checker:	

Input Data:

Wind Speed, V =	92	mph (Wind Map, Fig 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Occupancy Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	13.00	ft. (hr >= he)
Eave Height, he =	10.75	ft. (he <= hr)
Building Width =	22.50	ft. (Normal to Building Ridge)
Building Length =	35.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Gnd. Elev. Factor, Ke =	1.00	(Table 26.9-1)
Enclosed? (Y/N)	Y	(Sect. 28.6-1 & Figure 26.11-1)
Hurricane Region?	N	
Component Name =	Decking	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	16.875	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	11.31	deg.
Mean Roof Ht., h =	11.88	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Roof External Pressure Coefficients, GCp:

GCp All Zones (+) =	0.48	(Fig. 30.3-2A, 30.3-2B, 30.3-2C, and 30.3-2D)
GCp Zone 1,2e (-) =	-2.50	(Fig. 30.3-2A, 30.3-2B, 30.3-2C, and 30.3-2D)
GCp Zone 2n,2r (-) =	-3.26	(Fig. 30.3-2A, 30.3-2B, 30.3-2C, and 30.3-2D)
GCp Zone 3e (-) =	-3.68	(Fig. 30.3-2A, 30.3-2B, 30.3-2C, and 30.3-2D)
GCp Zone 3r (-) =	-4.15	(Fig. 30.3-2A, 30.3-2B, 30.3-2C, and 30.3-2D)

Positive & Negative Internal Pressure Coefficients, GCpi (Figure 26.11-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 30.3-1)

α =	9.50	(Table 26.9-1)
zg =	900	(Table 26.9-1)
Kh =	0.85	(Kh = Kz evaluated at z = h)

Velocity Pressure: qh = 0.00256*Kh*Kzt*Kd*Ke*V^2 (Eq. 26.10-1)

qh = 15.63 psf   qh = 0.00256*Kh*Kzt*Kd*Ke*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.3 & 30.5):

For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)

For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)

where: q = qh for roof

qi = qh for roof (conservatively assumed per Sect. 30.5)

Wind Load Tabulation for Roof Components & Cladding								
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)				
				All Zones (+)	Zone 1,2e (-)	Zone 2n,2r (-)	Zone 3e (-)	Zone 3r (-)
Decking	0.00	0.85	15.63	10.35	-41.90	-53.72	-60.31	-67.77
For z = hr:	13.00	0.85	15.63	10.35	-41.90	-53.72	-60.31	-67.77
For z = he:	10.75	0.85	15.63	10.35	-41.90	-53.72	-60.31	-67.77
For z = h:	11.88	0.85	15.63	10.35	-41.90	-53.72	-60.31	-67.77

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 1 & 2 (edge), '0.6*a' = 1.80 ft.
3. Width of Zone 3 (corner), '0.6*a' & '0.2*a'= 1.80 0.60 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.4-2A for 'GCp' values with 'qh'.
5. For buildings with h > 60' and θ > 10 degrees, use Fig. 30.6-1 for 'GCpi' values with 'qh'.
6. For all buildings with overhangs, use Fig. 30.4-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 7 degrees, Negative Zone 3 shall be treated as Zone 2; Positive Zone 2 & 3 shall be treated as Zone 4 & 5
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".

TABLE 1.2 – Fastener Wind Uplift Design Load – Wood Substrates:

	T1.2 - Fastener Wind Uplift Design Strength, PSF Consealor #14-13 DP1 Screws in Wood Substrates											
	Nominal Thickness of Wood Deck	Clip Spacing (Span) in Feet										
		6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
2 Screws per Clip	7/16" OSB	13.6	14.8	16.3	18.1	20.4	23.3	27.2	32.6	40.8	54.4	81.6
	19/32" OSB	29.1	31.7	34.9	38.8	43.6	49.8	58.1	69.8	87.2	116.3	174.4
	23/32" OSB	40.6	44.3	48.7	54.2	60.9	69.6	81.2	97.5	121.9	162.5	243.7
	1/2" Plywood	38.6	42.1	46.3	51.4	57.9	66.1	77.2	92.6	115.7	154.3	231.5
	5/8" Plywood	42.2	46.1	50.7	56.3	63.3	72.4	84.4	101.3	126.7	168.9	253.3
	3/4" Plywood	55.6	60.7	66.8	74.2	83.5	95.4	111.3	133.5	166.9	222.6	333.9
	2x4 SYP	88.1	96.1	105.7	117.5	132.1	151.0	176.2	211.4	264.3	352.4	528.5

2. Design loads are factored loads for use with LRFD Load Combinations. A resistance factor, Φ, of 0.40 has been applied in accordance with AWC-NDS.

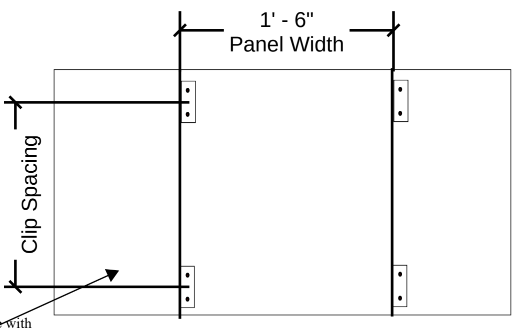
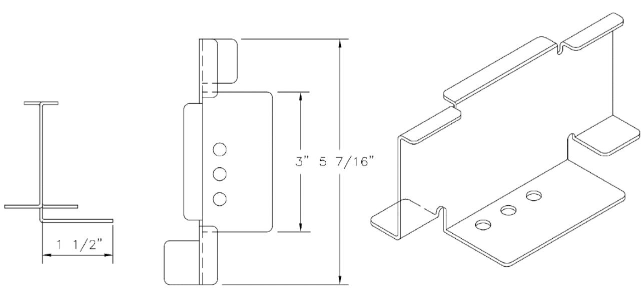


FIGURE 1.3: No. 16 Gauge Standard Clip – Material Properties and Figures (Typical)



16 Gauge Standard Clip Properties

Thickness	0.0566 in. (min)	S_{xe} (top)	0.1520 in^3	ϕP_n (E-TF)	0.473 k
Type	A653/A792 SS-33	S_{xe} (bot)	0.2454 in^3		
F_y	33 ksi (min)	I_x	0.2634 in^4		
E	29,500 ksi				

ZONE 3r
Penetration Depth = 5/8"
Nominal Capacity at 3' - 0" spacing = 84.4 psf
Penetration reduction factor = 0.625
LRFD to ASD conversion = 1/0.4 = 2.5
Actual capacity = 84.4 x 0.625 x 2.5 = 131.875 psf

Factored max wind demand (3r) = 0.6 x 67.77 psf = 40.66 psf
Factor of Safety = 2

DCR = 40.66x2/131.875 < 1

Spacing 3' - 0" acceptable for zone 3r.

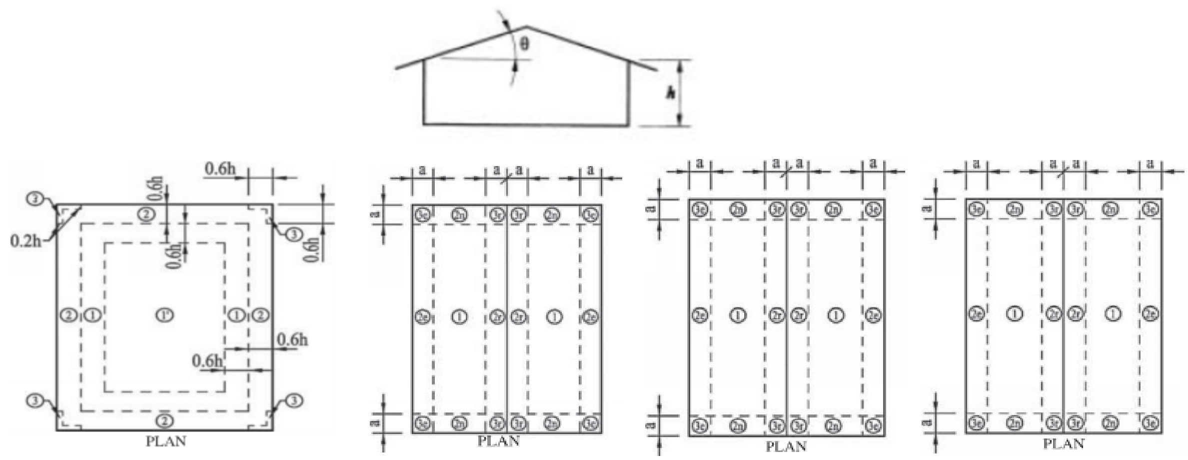
All other zones:
Penetration Depth = 5/8"
Nominal Capacity at 3' - 6" spacing = 72.4 psf
Penetration reduction factor = 0.625
LRFD to ASD conversion = 1/0.4 = 2.5
Actual capacity = 72.4 x 0.625 x 2.5 = 113.125 psf

Factored max wind demand (3r) = 0.6 x 60.31 psf = 36.2 psf
Factor of Safety = 2

DCR = 36.2x2/131.875 < 1

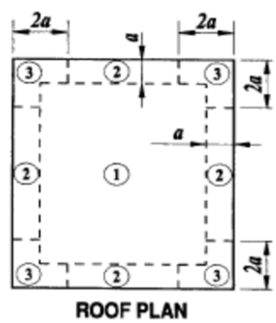
Spacing 3' - 6" acceptable for zone 3r.

Roof Components and Cladding:



θ <= 7°   7° < θ <= 20°   20° < θ <= 27°   27° < θ <= 45°

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45° and Monoslope Roofs <= 3°)



Roof Zones for Buildings with h > 60 ft.
(for Roofs <= 7°)

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FIGURE1.6 – Series 300 – 0.040-inch and 0.050-inch thick Aluminum:

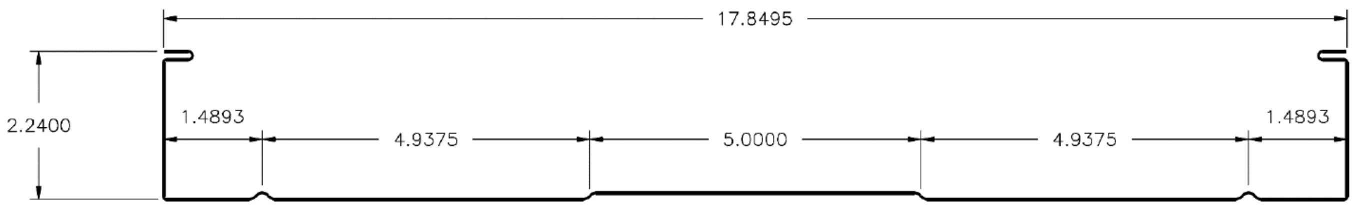

TABLE 1.13 – 0.040-thick Aluminum Section Properties:
.040" Aluminum x 18" Panel Properties

Thickness	0.040 in. (nom)	Ix (top)	0.450 in4	ϕM_n (top)	4.02 k-in
Type	3105-H25	Ix (bot)	0.430 in4	ϕM_n (bot)	5.42 k-in
Width	18 in. (nom)			ϕV_n	1.340 k
F_y	19 ksi				
E	10,100 ksi				

For SI: 1 inch = 2.54 mm; 1 ksi = 6.89 MPa; 1 kip = 1000 lbs.

Notes:

1. Section properties are calculated in accordance with the ADM1-2015, Aluminum Design Manual: Part 1-A Specification for Aluminum Structures.
2. The section properties also shall be used for the 0.050-inch panels.
3. The 0.050- and 0.040-inch aluminum panel loads may be designed by a registered design professional using the Section Properties in Table 1.13 of this report.
4. E is the modulus of elasticity.
5. F_y is the yield strength.
6. I_{xe} is the effective moment of inertia about the cross-section about the x-axis.
7. M_n is the nominal bending strength.
8. V_n is the nominal shear strength.

Check Panel Strength:
Trib Width of Panel = Building Width / 2 = 22.5' / 2 = 11.25'
W = 66.8 psf x 11.25' = 751.5 plf
Mu = WL^2/8 = 751.5 x (1.5)^2 / 8 = 211 lbs-ft = 2.5 kip-in < Mn listed in Table 1.13 above --> OK!

For clip capacity, see Table 1.16 in ER 686 shown below:
Allowed wind load = 141 psf >> 66.7 psf

TABLE 1.16 – Clip Wind Negative (Uplift) Design Load:

	T1.16 Series 300 Panel/Clip Wind Uplift Design Load, PSF										
	.040" or .050" Aluminum with 18" o.c. Seam Spacing										
	Standard 16 GA Clip Anchors										
	Clip Spacing (Span), Feet										
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Max. Design Load, PSF	44.8	55.5	66.2	77.0	87.7	98.4	109.1	119.8	130.6	141.3	152.0

For SI: 1 inch = 25.4 mm; 1 foot =305 mm; 1 psf = 47.9 Pa

1. The wind uplift nominal strength has been determined according to the procedures of AISI S906, with a resistance factor, Φ =0.80 in accordance with AISI S100-12 D6.2.1.
2. Design loads shown are factored loads for use with LRFD load combinations.
3. Intermediate design values have been determined based on linear interpolation between tested values based on Section 5.4.3.2 of EC011-2019.
4. The allowable service load deflections for metal roof panels shall be taken as L/60.
5. Wind uplift design loads may be further limited by the design strength of the anchor fasteners into the roof substrate. The tests conducted in accordance with ASTM E1592, shown in this table, utilized two ¼" -14 x 1.25" HWH self-drilling tapping screws per anchor/ purlin connection. For other fastener types or roof substrates, the faster design strength shall be designed by the registered design professional.
6. Tables 1.1 and 1.2 of this report have been provided for the fastener design strength of a frequently used screw type into typical metal and wood roof substrates.