Structural Calculations For:

Lumpkin Residence

5401 W. Mercer Way

Mercer Island, WA 98040

Prepared for:	Suyama Peterson Deguchi
Job #:	00043-2020-04
Date:	March 17, 2021

Criteria Sheet

Codes

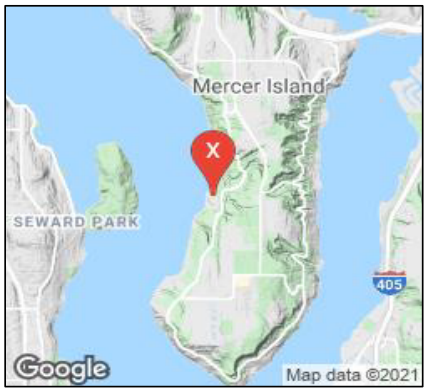
Structural	IBC 2018
Loading	ASCE 7-16
Wood:	NDS 2018
Steel:	AISC 360-16
Concrete:	ACI 318-14
Masonry:	TMS 402/602-16

Project Location

Street & Number	5401 W. Mercer Way		
City:	Mercer Island	State:	WA
ZIP:			
Latitude:	47.5541	N	
Longitude:	-122.2313	W	
Ground Elevation	73	ft	

Occupancy Category

Risk Category: II ASCE 7 Table 1.5-1



Seismic Load Summary:

Analysis Procedure: Equivalent Lateral Force Procedure

Lateral System: Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

R:	6.50	C_d=	4
Base Shear V =	21 kips	Ω_o=	2.5
S_S=	1.457	S_1=	0.506
S_{DS}=	1.00	S_{D1}=	0.57
C_s=	0.154	I_E=	1.0

Story Information

# Stories Above Grade (Including Mezzanine Levels)	2

Horizontal and Vertical Irregularities:

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

Wind Load Summary:

V= 110 K_{zt}= 1.00

Exposure = C

Dead Loads:

Roof			Floor		
Roofing	2.5	psf	Finish Floor	30	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	3.9			39.7	psf
	15	psf	Use	40	psf
Use	15	psf			

Live Loads:

Snow	25	psf	Deck	60	psf
Floor	40	psf			

Soils:

Soils Report Provided? Yes Geo Group Northwest, G-5244

Allowable Bearing	1500	psf	Active	55/35	pcf (Restrained/Unrestrained)
Sliding, μ	0.3		Seismic Surcharge	8H	
Passive	250	pcf	**3T Pin piles**		

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Seismic Design

ASCE 7-16 Seismic Analysis Equivalent Lateral Force Procedure

Seismic Force Resisting System Per Table 12.2-1	System	Bearing Wall Systems
	Type:	Light-frame (wood) Walls Sheathed with Wood Structural Panels Rated for Shear Resistance

Seismic Design Cat.	D	
Risk Category	II	I, II, or III, or IV per Table 1.5-1
Site Class	D (Default)	Assumed default soil properties, per 11.4.3.
Diaphragm Flexibility	Flexible	

Section 12.8.1.3 Exceptions

Regular Structure	Yes
≤ 5 Stories above grade	Yes
T ≤ 0.5s	Yes
ρ = 1.0	Yes
Not Site Class E or F	Yes
Risk Category I or II	Yes

If all exceptions are met, S_{DS} may be taken as 1, but not less than 0.7*(Calculated S_{DS})

S_S	1.457 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.506 g	2% in 50 yr, Latitude & Longitude lookup
R	6.50	
C_d	4.0	
Ω_o	2.5	
I_e	1.00	Table 1.5-2
h_n	20.3 ft	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.19 sec	
T	0.19 sec	Eq. 12.8-7
T_0	0.11 sec	
T_S	0.57 sec	
T_L	6.00 sec	
F_a	1.20	Table 11.4-1
F_v	1.70	Table 11.4-2
S_{MS}	1.75 g	Eq. 11.4-1
S_{M1}	0.86 g	Eq. 11.4-2
S_{DS}	1.000 g	Eq. 11.4-3
S_{D1}	0.573 g	Eq. 11.4-4
C_s	***0.154 Controls***	Eq. 12.8-2
	0.462	Eq. 12.8-3 need not exceed, T < T_L
	0.010	Eq. 12.8-5 or 12.8-6 minimum
C_s, design	0.154	
Bldg. Weight	133.9 k	
$V = C_S W$	20.6 k	Eq. 12.8-1, Strength Level Base Shear
$V = C_{Sasd} W$	14.4 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 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)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	$\gamma=F_{px}/F_x$
Roof	20.3	54.28	20.3	1099	0.599	8.63	**8.6**	8.6	7.6	15.2	**8.6**	1.00
Upper	9.3	79.61	9.3	736	0.401	5.78	**14.4**	8.6	11.1	22.3	**11.1**	1.93
Σ		133.9		1835		**14.42**						

Wind Design - MWFRS

ASCE 7 Chapter 27 - Directional Procedure

Design Method	ASD

Wind Coefficients

Exposure	C	
V=	110	mph
K_d=	0.85	Table 26.6-1
K_h=	0.90	Table 26.10-1
K_e=	1.00	Table 26.9-1
G=	0.85	26.9.4

Transverse Wind Pressures

L/B = 0.44 h/L = 0.72

Pressure Coefficients from Figure 27.3-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.63 / -0.11
Leeward Roof	-0.58

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.00	
Roof Type	Gable	
Roof Angle - Transverse Dir	18.43	degrees
Roof Angle - Long Dir	18.43	degrees
Ground to top of roof	23.75	ft
Bot of roof to top of roof	7.25	ft
Mean Roof Height, h	20.125	ft
Short Plan Dimension	28	ft
Long Plan Dimension	63.5	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade	9.25	ft

Velocity Pressure at Mean Roof Height, q_h =	23.7	psf

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	**P_{walls} (psf)**
0-15	0.85	22.32	15.18	10.08	**15.15**
15-20	0.9	23.63	16.07	10.08	**15.69**
20-25	0.94	24.68	16.79	10.08	**16.12**
25-30	0.98	25.74	17.50	10.08	**16.55**
30-40	1.04	27.31	18.57	10.08	**17.19**
41-50	1.09	28.62	19.46	10.08	**17.73**
51-60	1.13	29.67	20.18	10.08	**18.15**
61-70	1.17	30.72	20.89	10.08	**18.58**
71-80	1.21	31.77	21.61	10.08	**19.01**
81-90	1.24	32.56	22.14	10.08	**19.33**
91-100	1.26	33.09	22.50	10.08	**19.55**

Roof Pressures (Unfactored) ASD

Windward		Leeward	**Horiz Proj (psf)**
Max	Min		
-2.2	-12.6	-11.7	**4.80**

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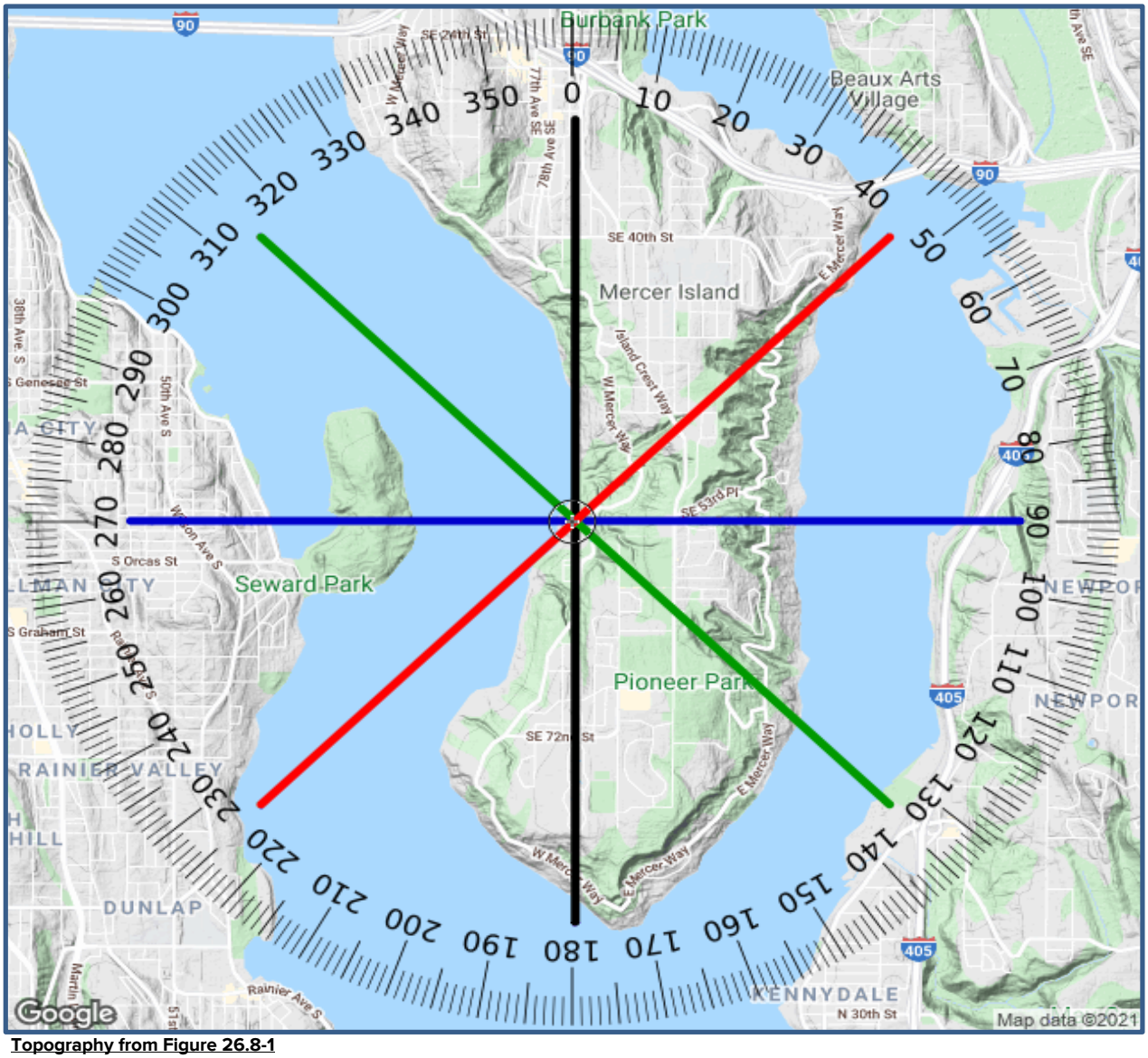
Site Address

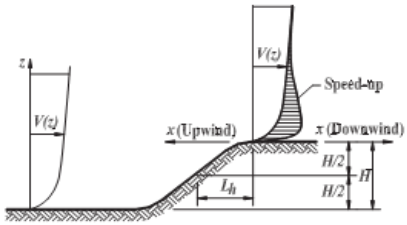
Address	5401 W. Mercer Way			Wind Radius	2.00 Miles
City:	Mercer Islanc	State:	WA	Angle	0 ˚
Lat Long	47.55412	-122.2313		Exposure	C

Profile 1 : 0 ˚ to 180 ˚
Profile 2 : 270 ˚ to 90 ˚
Profile 3 : 315 ˚ to 135 ˚
Profile 4 : 45 ˚ to 225 ˚

SITE MAP



Topography from Figure 26.8-1



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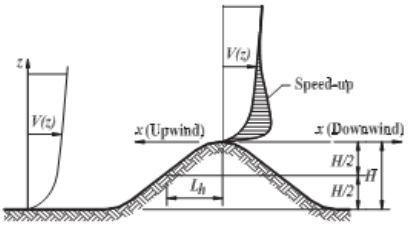

2-D RIDGE OR 3-D AXISYMMETRICAL HILL

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$
$$K_1 = \text{Per Figure}$$
$$K_2 = (1 - |x|/\mu L_h)$$
$$K_3 = e^{-\gamma z / L_h}$$
$$K_{zt} = 1, \text{ If } H/Lh \le 0.2$$

PER FIGURE 26.8-1

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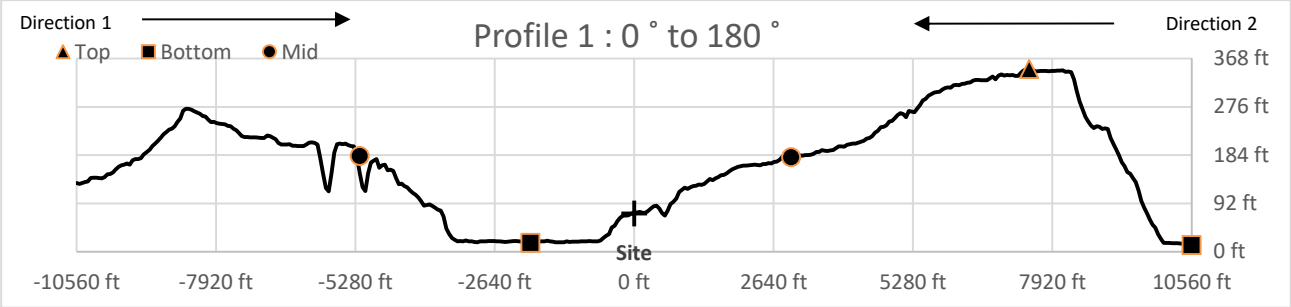

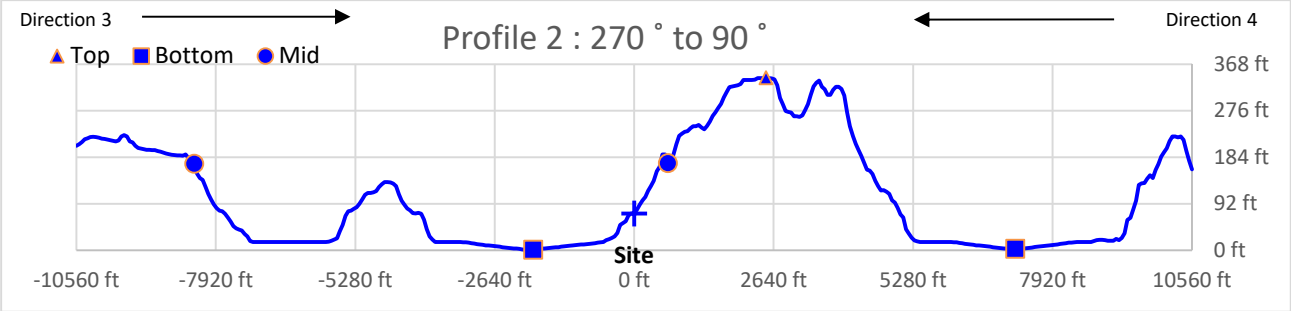

Direction 1 - 0 ˚to Site

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	7482
Bott. of Hill Dist.	-1963
L @ H/2	-5200
Site	upwind
Top of Hill Elev.	348
Bott. of Hill Elev.	17
Site Elev.	72.9
Site Dist.	0
H/2	182

Kzt Calculations

H=	331
Lh=	12682
x=	7482
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.04
K2=	0.61
k3=	1.00
H/Lh =	0.03
Kzt =	1.00

Direction 2 - Site to 180 ˚

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	7482
Bott. of Hill Dist.	10560
L @ H/2	2972
Site	downwnd
Top of Hill Elev.	348
Bott. of Hill Elev.	13
Site Elev.	72.9
Site Dist.	0
H/2	180

Kzt Calculations

H=	335
Lh=	4510
x=	7482
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.11
K2=	0.00
k3=	0.99
H/Lh =	0.07
Kzt =	1.00

Direction 3 - 270 ˚to Site

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Hill
Top of Hill Dist.	2494
Bott. of Hill Dist.	-1910
L @ H/2	-8331
Site	upwind
Top of Hill Elev.	341
Bott. of Hill Elev.	1
Site Elev.	72.9
Site Dist.	0
H/2	171

Kzt Calculations

H=	340
Lh=	10825
x=	2494
z=	20.125
μ=	1.5
γ=	4
K1 value =	1.05
K1=	0.03
K2=	0.85
k3=	0.99
H/Lh =	0.03
Kzt =	1.00

Direction 4 - Site to 90 ˚

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	2494
Bott. of Hill Dist.	7217
L @ H/2	637
Site	downwnd
Top of Hill Elev.	341
Bott. of Hill Elev.	3
Site Elev.	72.9
Site Dist.	0
H/2	172

Kzt Calculations

H=	338
Lh=	1857
x=	2494
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.26
K2=	0.10
k3=	0.97
H/Lh =	0.18
Kzt =	1.00

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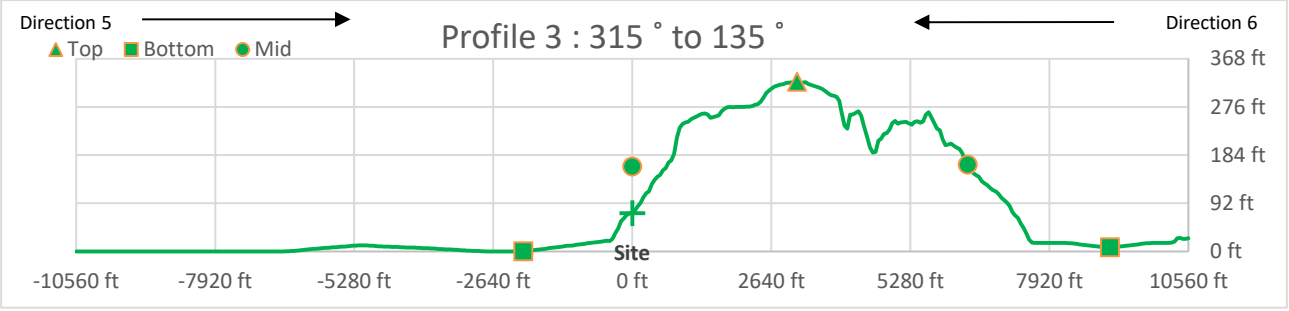

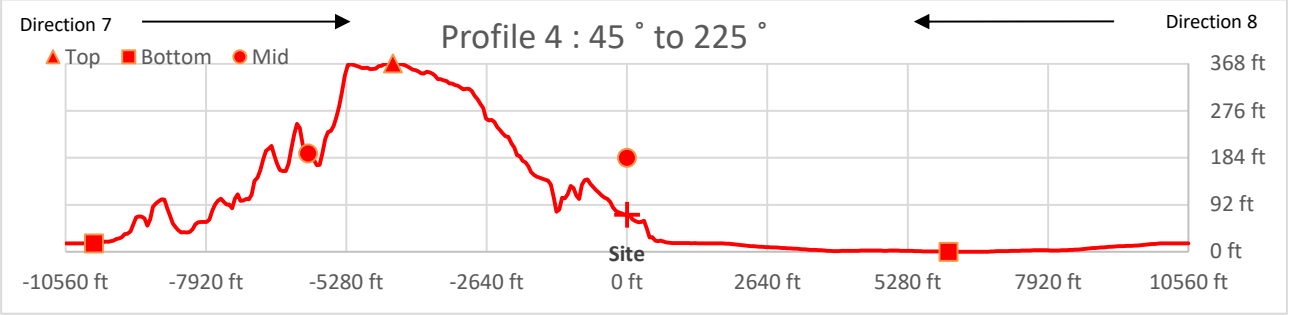

Direction 5 - 315 ˚to Site

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	3131
Bott. of Hill Dist.	-2070
L @ H/2	0
Site	upwind
Top of Hill Elev.	324
Bott. of Hill Elev.	0
Site Elev.	72.9
Site Dist.	0
H/2	162

Kzt Calculations

H=	324
Lh=	3131
x=	3131
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.15
K2=	0.33
k3=	0.98
H/Lh =	0.10
Kzt =	1.00

Direction 6 - Site to 135 ˚

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated		
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	3131
Bott. of Hill Dist.	9074
L @ H/2	6368
Site	downwnd
Top of Hill Elev.	324
Bott. of Hill Elev.	8
Site Elev.	72.9
Site Dist.	0
H/2	166

Kzt Calculations

H=	316
Lh=	3237
x=	3131
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.14
K2=	0.36
k3=	0.98
H/Lh =	0.10
Kzt =	1.00

Direction 7 - 45 ˚to Site

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	Yes	
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	-4404
Bott. of Hill Dist.	-10029
L @ H/2	-5996
Site	downwnd
Top of Hill Elev.	369
Bott. of Hill Elev.	16
Site Elev.	72.9
Site Dist.	0
H/2	193

Kzt Calculations

H=	352
Lh=	1592
x=	4404
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.32
K2=	0.00
k3=	0.96
H/Lh =	0.22
Kzt =	1.00

Direction 8 - Site to 225 ˚

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	No	Kzt=1
4. H/Lh ≥ 0.2	No	Kzt=1
5. H≥ 15'	Yes	

Terrain Data

Terrain	Ridge
Top of Hill Dist.	-4404
Bott. of Hill Dist.	6049
L @ H/2	0
Site	upwind
Top of Hill Elev.	369
Bott. of Hill Elev.	0
Site Elev.	72.9
Site Dist.	0
H/2	184

Kzt Calculations

H=	369
Lh=	4404
x=	4404
z=	20.125
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.12
K2=	0.33
k3=	0.99
H/Lh =	0.08
Kzt =	1.00

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SEISMIC WEIGHTS

W_S (ROOF) = ROOF + WALL
@ $h = 20.25'$ = 15 (3075 S.F) + 5 (1620 S.F)
= 54275 lbs

W_S (FLOOR) = FLOOR + WALL + TOPPING
@ $h = 9.25'$ = 12 (1630 S.F) + 10 (1630 S.F) + 25 (1630 SF)
= 76610 lbs
+ DECK || + 15 (200 S.F) || + 3000 -
= 79610 lbs

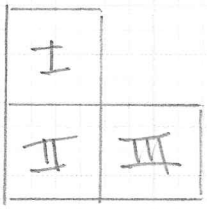
SEISMIC STORY SHEAR

ASD, $\rho = 1.0$, $C_s = 0.154$

V_S (ROOF) = 8.63 k
V_S (MAIN) = 5.79 k
ΣV_S = 14.42 k

DISTRIBUTION OF SEISMIC SHEAR

A (I) = 1220 S.F V(%) = 0.397
A (II) = 1000 S.F V(%) = 0.325
A (III) = 855 S.F V(%) = 0.278
ΣA = 3075 S.F

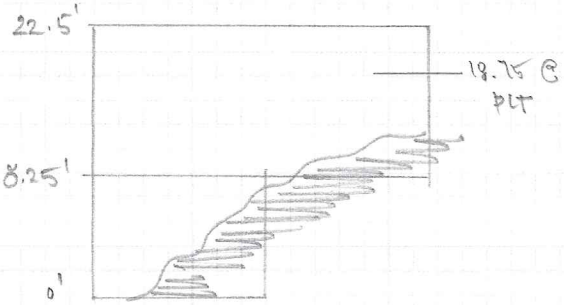


WIND LOADING

NOTE: GABLE END VISIBLE AT NORTH, SOUTH & EAST ELEVATIONS. FOR SIMPLICITY, USE 'BOX' APPROACH W/ FLAT WALL LOADS, IN LIEU OF THE LOWER ROOF WAS

W_W (ROOF) = WALL
= 2.5 (16.19) + 5 (15.76)
+ 1.75 (15.23)
= 146 PLF

W_W (MAIN) = WALL
= 9.125 (15.23)
= 139 PLF
= 84 PLF (WINDWARD ONLY, DAYLIGHTED)



WIND STORY SHEAR

V_W (ROOF - N/S) = 146 (28) = 4.09 k
V_W (MAIN - N/S) = 139 (28) = 3.89 k
ΣV_W = 7.98 k

V_W (ROOF) - E/W = 146 (63.5) = 9.27 k
V_W (MAIN) - E/W = 84 (63.5) = 5.33 k
ΣV_W = 14.60 k

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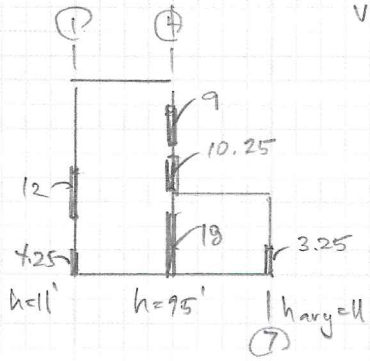
LATERAL DESIGN (NORTH / SOUTH @ ROOF)

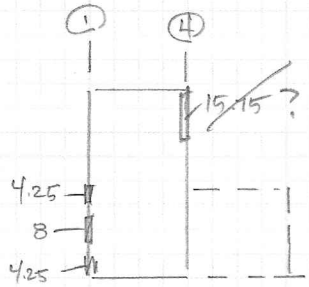


$V(1) = 3.12\ k\ (E)$
$2.05\ k\ (W)$
$L_w = 16.25\ ft$
$v = 192\ \#/ft$
S.W = (W6)
$OT = 2.11\ k$
$H_b = (H_{bu2})$

$V(4) = 7.43\ k\ (E)$
$2.05\ k\ (W)$
$L_w = 37.25\ ft$
$v = 199\ \#/ft$
S.W = (W6)
$OT = 1.89\ k$
$H_b = (H_{bu2})$

$V(7) = 1.20\ k\ (E)$
$1.42\ k\ (W)$
$L_w = 3.25$
$v = 523\ \#/ft$
S.W = (W2)
$OT = 5.81\ k$
$H_b = (H_{bu3})$

LATERAL DESIGN (NORTH / SOUTH @ MAIN)



$V(1) = 6.01\ k\ (E)$
$3.99\ k\ (W)$
$L_w = 16.5\ ft$
$v = 364\ \#/ft$
S.W = (W3)
$OT = 3.46\ k$
$\Sigma OT = 5.57\ k$
$OTR = 0.49\ k$
$OT' = 5.08\ k$
$H_b = (H_{bu8})$

$V(4)$ = FULL HEIGHT CONCRETE WALL

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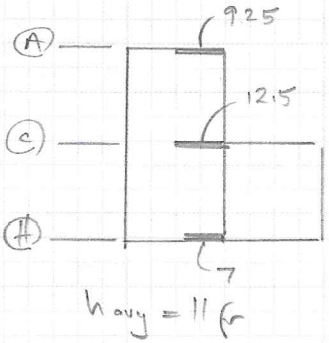
LATERAL DESIGN (EAST/WEST @ Roof)

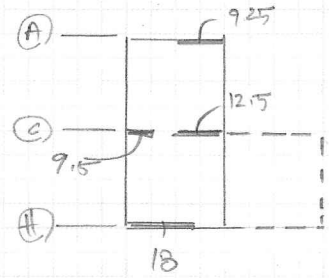


V(A) = 1.71 k (E)
2.32 k (W)
L_w = 9.25 ft
w = 185 #/ft
S.W = W4
OT = 2.03 k
Hb = Hbu2 CS14

V(C) = 4.32 k (E)
4.64 k (W)
L_w = 12.5 ft
w = 346 #/ft
S.W = W3
OT = 4.08 k
Hb = Hbu5 (2) CS14

V(4) = 2.66 k (E)
2.32 k (W)
L_w = 7 ft
w = 371 #/ft
S.W = W3
OT = 4.09 k
Hb = Hbu5 (2) CS14

LATERAL DESIGN (EAST/WEST @ MAIN)



V(A) = 3.16 k (E)
3.65 k (W)
L_w = 9.25 ft
w = 342 #/ft
S.W = W3
OT = 3.75 k
ΣOT = 6.51 k
OTR = 0.83 k
OT' = 5.68 k
Hb = Hbu8

V(C) = 7.20 k (E)
7.31 k (W)
L_w = 12.5 ft
w = 576 #/ft
S.W = W2
OT = 5.47 k
ΣOT = 9.55 k
OTR = 1.13 k
OT' = 8.42 k
Hb = Hbu11

↓

USING ADJACENT SEGMENT TOO
L' = 12.5 + 9.5
= 22 ft
w' = 327 #/ft
S.W = W3
OT = 3.10 k
ΣOT = 7.19 k
OTR = 1.13 k
OT' = 6.06 k
Hb = Hbu8

V(4) = 4.05 k (E)
3.65 k (W)
L_w = 18 ft
w = 225 #/ft
S.W = W3
OT = 2.14 k
ΣOT = 6.23 k
OTR = 1.62 k
OT' = 4.61 k
Hb = Hbu5

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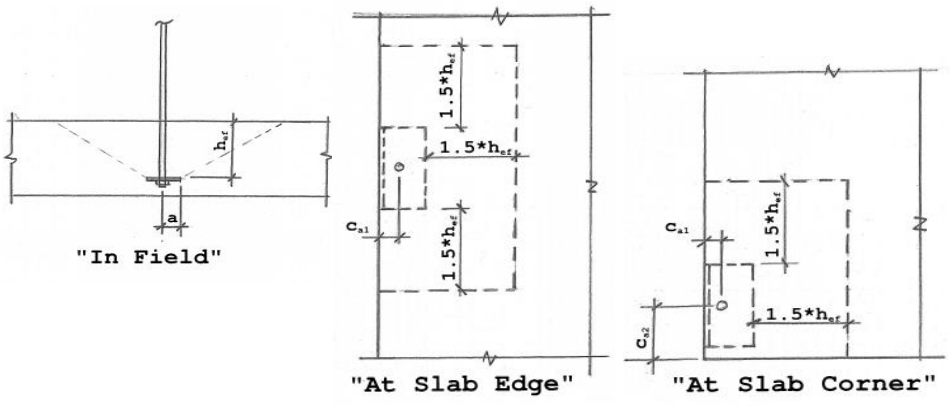
Holdown anchor cast in concrete slab (ACI 318-14, 17.2.3.4.3 (a))

Anchor: HD perpendicular to slab edge

$P_{up,ASD}$	11.2	k
Rod $_{diam}$	7/8"	
$A_{s,gross}$	0.601	in^2
$A_{s,eff}$	0.462	in^2
f_c'	4000	psi
h_{ef}	4.5	in
A_{Nco}	182.25	in^2
c_{a1}	24	in
c_{a2}	24	in
$c_{a,min}$	24	in
ϕ	0.7	
f_{uta}	54	ksi
λ_a	1	
k_c	24	
Plate size	6	in



Anchor Steel strength			Concrete break out strength			Pullout strength			Side face blowout strength		
N_{sa} =	24.9	k	A_{Nc} =	380.25	in^2	A_{brg} =	36	in^2	Req'd (h_{ef}>2.5c_{a1}) ?		NO
			$\psi_{ed,N}$ =	1.00		$\psi_{c,P}$ =	1		N_{sb} =	30.2	k
			$\psi_{c,N}$ =	1.00		N_p =	1152.0	k			
			$\psi_{cp,N}$ =	1.00		N_{pn} =	1152.0	k			
			N_b =	14.5	k						
			N_{cb} =	30.2	k						

Ductility check			LRFD check				Steel plate check		
1.2 Nsa =	29.9	k	P_u =	15.6	k		F_y =	50	ksi
Critical strength =	30.2	k	ϕN_{sa} =	17.5	k	**OK**	f_u =	0.83	ksi
	Ductile ?	**YES**	$0.75\phi N_{cb}$ =	15.9	k	**OK**	f_{ASD} =	0.59	ksi
			$0.75\phi N_{pn}$ =	604.8	k	**OK**	a_{max} =	3.00	in
NOTE: DUCTILE FAILURE MECHANISM, THEREFORE NO NEED TO INCLUDE OVERSTRENGTH = 2.5 FACTOR.			$0.75\phi N_{sb}$ =	15.9	k	**OK**	M_{plate} =	2.67	in-k/in
							t_{min} =	0.65	in
							B_u (concr bearing)	29.9	k
							ϕB_n =	159.1	k
Pmax, asd =	11.18						$\phi B_n > B_u$?	**OK**	

NOTE: THIS CONDITION OCCURS AT THE 8' THICK SLAB AT GRID LINE C BETWEEN LINES 1 AND 4

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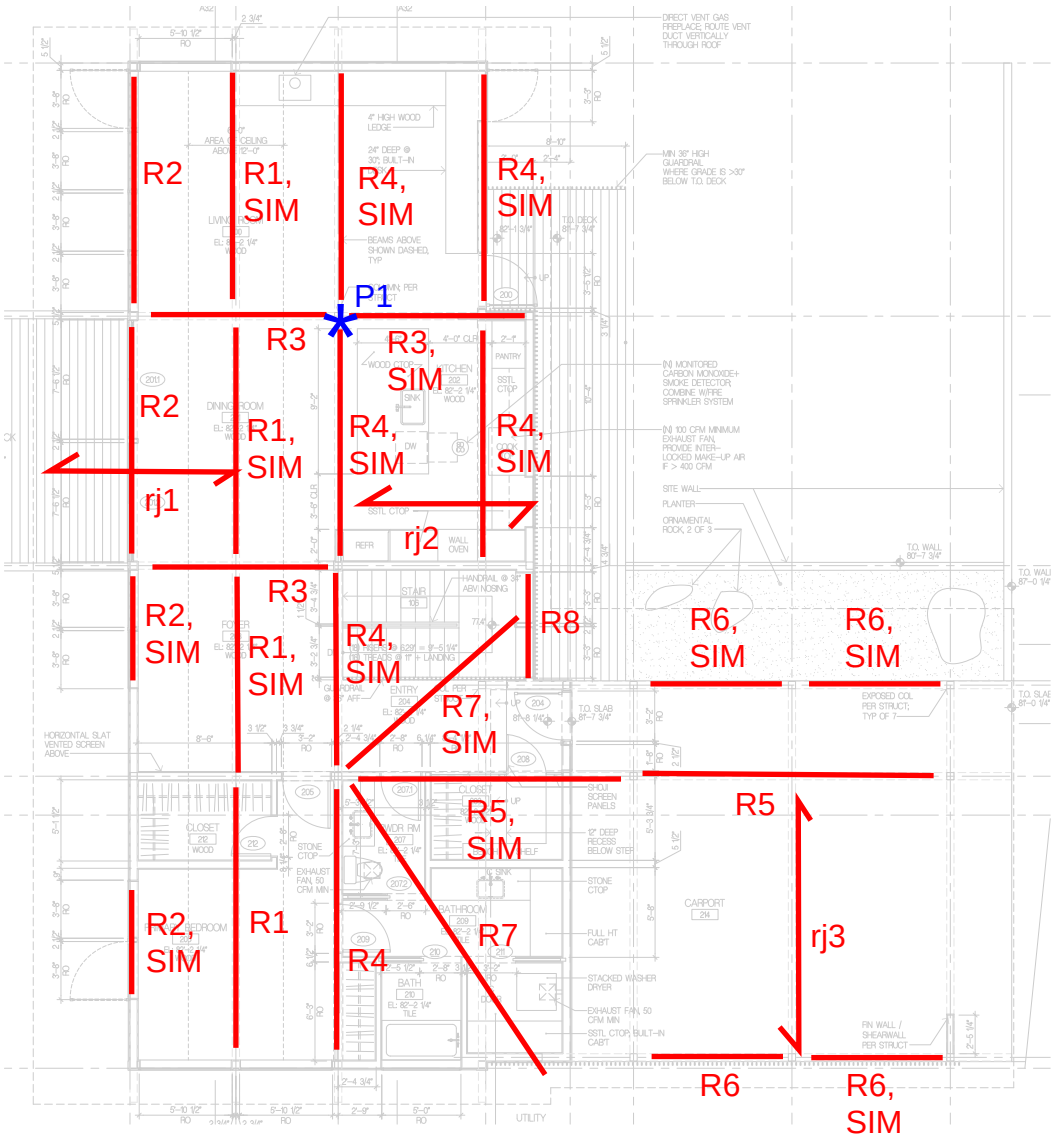
Date: March 16, 2021
Design: VMB
Sheet:



ROOF FRAMING KEY PLAN

Snow / Roof live = 25
Roof Dead = 15 (includes 4 psf for solar panels)

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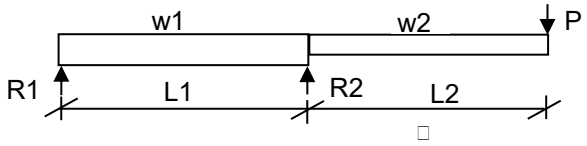
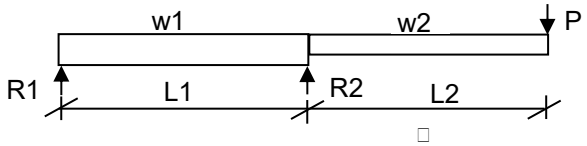
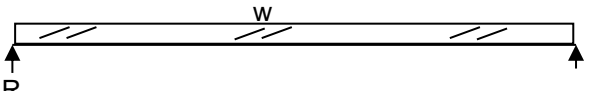
Beam		RJ1		HF	2	x 12	
w1=	80	plf		R1=	0	lbs
w2=	80	plf		R2=	960	lbs
L1=	6	ft		M+=	-	lb-ft
L2=	6	ft		M-=	1,440	lb-ft
X=	3.00	ft		Fb=	546	psi
P=		lbs		Fv=	36	psi
b=	1.50	in		Δspan=	(0.014)	in
d=	11.25	in		l span/	(5,101)	
E=	1,300	ksi		Δcant=	0.19	in
Cv=	1.00			l cant/	744	



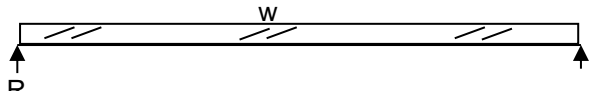
Beam		RJ1-UNBAL		HF	2	x 12	
w1=	20	plf		R1=	-180	lbs
w2=	80	plf		R2=	780	lbs
L1=	6	ft		M+=	-	lb-ft
L2=	6	ft		M-=	1,440	lb-ft
X=	3.00	ft		Fb=	546	psi
P=		lbs		Fv=	36	psi
b=	1.50	in		Δspan=	(0.022)	in
d=	11.25	in		l span/	(3,321)	
E=	1,300	ksi		Δcant=	0.22	in
Cv=	1.00			l cant/	661	



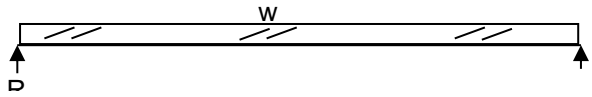
Beam		RJ2		HF	2	x 12	
w=	80	plf		R=	490	lbs
L=	12.25	ft		M=	1,501	ft-lbs
b=	1.50	in		Fb=	569	psi
d=	11.25	in		Fv=	37	psi
E=	1300	ksi		Δ=	0.18	in
Cv=	1.00	≤1.0		l /	839	



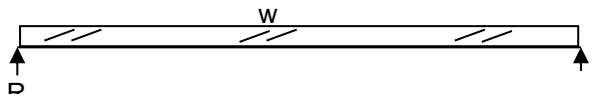
Beam		RJ3		HF	2	x 12	
w=	80	plf		R=	720	lbs
L=	18	ft		M=	3,240	ft-lbs
b=	1.50	in		Fb=	1,229	psi
d=	11.25	in		Fv=	57	psi
E=	1300	ksi		Δ=	0.82	in
Cv=	1.00	≤1.0		l /	264	



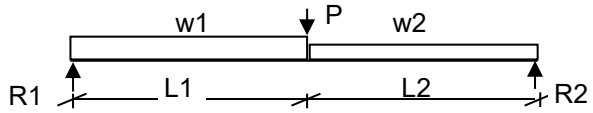
Beam		R1		GL	5 1/2	x 11 7/8	
w=	270	plf		R=	2,498	lbs
L=	18.5	ft		M=	11,551	ft-lbs
b=	5.50	in		Fb=	1,072	psi
d=	11.88	in		Fv=	51	psi
E=	1800	ksi		Δ=	0.52	in
Cv=	1.00	≤1.0		l /	431	



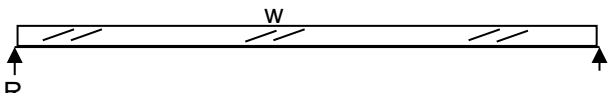
Beam		R2		GL	5 1/2	x 11 7/8	
w=	480	plf		R=	3,840	lbs
L=	16	ft		M=	15,360	ft-lbs
b=	5.50	in		Fb=	1,426	psi
d=	11.88	in		Fv=	77	psi
E=	1800	ksi		Δ=	0.51	in
Cv=	1.00	≤1.0		l /	375	



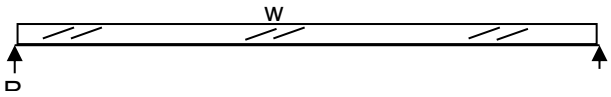
Beam		R3			GL 5 1/2 x 11 7/8	
w1=	25	plf		R1 =	2,313	lbs
w2=	25	plf		R2 =	2,313	lbs
L1=	6.50	ft		M =	14,503	lb-ft
L2=	6.50	ft		Fb =	1,346	psi
X=	6.5	ft		Fv =	53	psi
P=	4,300	lbs		Δ=	0.26	in
b=	5.50	in		I /	605	
d=	11.88	in		Cv=	1.00	
E=	1,800	ksi				



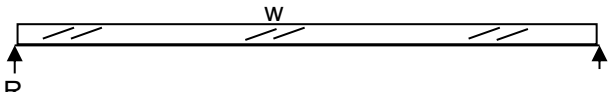
Beam		R4			GL 5 1/2 x 11 7/8	
w=	310	plf		R=	2,829	lbs
L=	18.25	ft		M=	12,906	ft-lbs
b=	5.50	in		Fb=	1,198	psi
d=	11.88	in		Fv=	58	psi
E=	1800	ksi		Δ=	0.56	in
Cv=	1.00	≤1.0		I /	391	



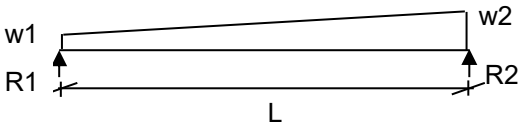
Beam		R5			GL 5 1/2 x 15	
w=	480	plf		R=	4,800	lbs
L=	20	ft		M=	24,000	ft-lbs
b=	5.50	in		Fb=	1,396	psi
d=	15.00	in		Fv=	76	psi
E=	1800	ksi		Δ=	0.62	in
Cv=	0.98	≤1.0		I /	387	



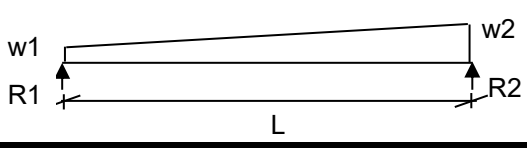
Beam		R6			GL 5 1/2 x 9	
w=	360	plf		R=	1,800	lbs
L=	10	ft		M=	4,500	ft-lbs
b=	5.50	in		Fb=	727	psi
d=	9.00	in		Fv=	46	psi
E=	1800	ksi		Δ=	0.13	in
Cv=	1.00	≤1.0		I /	891	



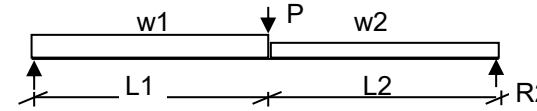
Beam		R7			PSL 5 1/4 x 11 7/8	
w1=	-	plf		R1 =	2,240	lbs
w2=	625	plf		R2 =	4,479	lbs
L=	22	ft		M =	17,580	lb-ft
b=	5.25	in		Fb =	1,710	psi
d=	11.88	in		Fv =	54	psi
E=	2,000	ksi		Δ=	1.03	in
Cv=	1.00			I /	251	



Beam		R7, sim			PSL 5 1/4 x 11 7/8	
w1=	-	plf		R1 =	1,148	lbs
w2=	475	plf		R2 =	2,296	lbs
L=	15	ft		M =	6,077	lb-ft
b=	5.25	in		Fb =	591	psi
d=	11.88	in		Fv =	27	psi
E=	2,000	ksi		Δ=	0.16	in
Cv=	1.00			I /	1,078	



Beam		R8			GL 5 1/2 x 11 7/8	
w1=	200	plf		R1 =	1,848	lbs
w2=	200	plf		R2 =	1,848	lbs
L1=	3.5	ft		M =	5,243	lb-ft
L2=	3.5	ft		Fb =	487	psi
X=	3.5	ft		Fv =	38	psi
P=	2,296	lbs		Δ=	0.03	in
b=	5.50	in		I /	2,964	
d=	11.88	in		Cv=	1.00	
E=	1,800	ksi				



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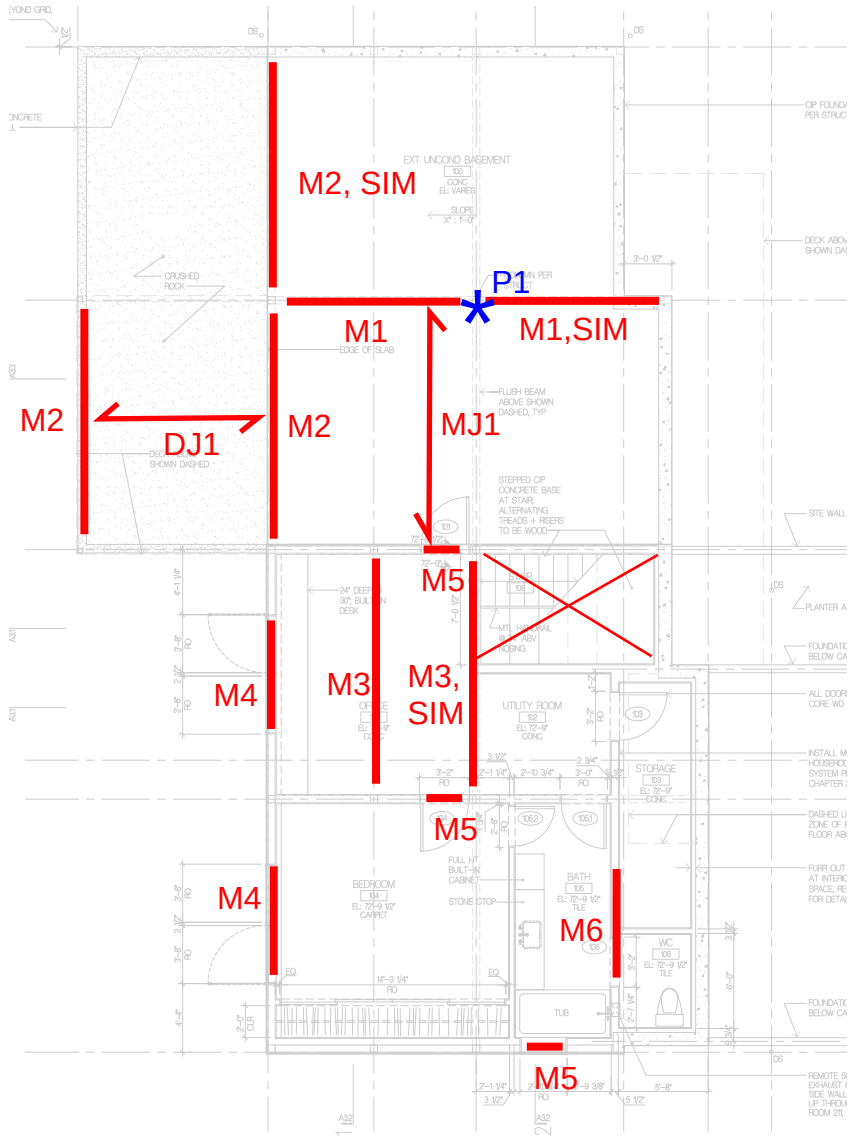
Project: Lumpkin Residence *Date:* 03/16/21

Project #:

GRAVITY DESIGN - ROOF

Design: VMB



FLOOR FRAMING KEY PLAN

Floor Live = 40 psf
Floor Dead = 35 psf (20 psf topping slab)

Deck Live = 60 psf
Deck Dead = 12 psf

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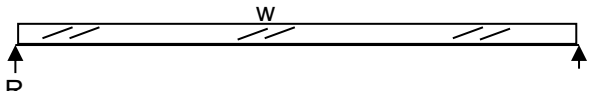
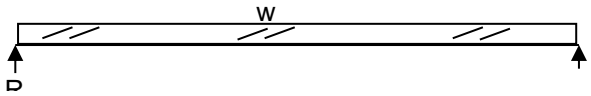
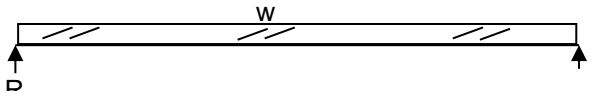
Beam		DJ1		HF	2	x 12
w=	96	plf		R=	576	lbs
L=	12	ft		M=	1,728	ft-lbs
b=	1.50	in		Fb=	655	psi
d=	11.25	in		Fv=	43	psi
E=	1300	ksi		Δ=	0.19	in
Cv=	1.00	≤1.0		l /	744	



Beam		M2		GL	5 1/2	x 15
w=	582	plf		R=	4,656	lbs
L=	16	ft		M=	18,624	ft-lbs
b=	5.50	in		Fb=	1,084	psi
d=	15.00	in		Fv=	71	psi
E=	1800	ksi		Δ=	0.31	in
Cv=	1.00	≤1.0		l /	623	

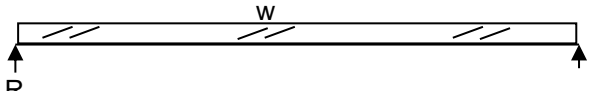


Beam		MJ1		LSL	1 3/4	x 11 7/8
w=	100	plf		R=	788	lbs
L=	15.75	ft		M=	3,101	ft-lbs
b=	1.75	in		Fb=	905	psi
d=	11.88	in		Fv=	50	psi
E=	1550	ksi		Δ=	0.37	in
Cv=	1.00	≤1.0		l /	517	

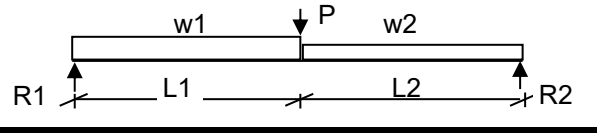


TJI Size	11.88	in		RE 1.75	11.875	TJI 360
EI =	419	in^4		Ma=	6180	lb-ft
Δ =	0.386	in		Va=	1705	lbs
l /	489			Ra=	1080	lbs

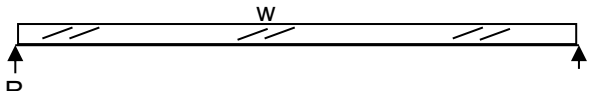
Beam		M1		GL	5 1/2	x 15
w=	1200	plf		R=	7,800	lbs
L=	13	ft		M=	25,350	ft-lbs
b=	5.50	in		Fb=	1,475	psi
d=	15.00	in		Fv=	115	psi
E=	1800	ksi		Δ=	0.28	in
Cv=	1.00	≤1.0		l /	563	



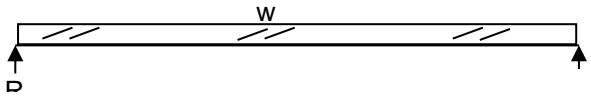
Beam		M3		PSL	5 1/4	x 11 7/8
w1=	100	plf		R1 =	1,476	lbs
w2=	100	plf		R2 =	4,449	lbs
L1=	13.5	ft		M =	10,811	lb-ft
L2=	2.5	ft		Fb =	1,051	psi
X=	13.5	ft		Fv =	105	psi
P=	4,325	lbs		Δ=	0.17	in
b=	5.25	in		l /	1,135	
d=	11.88	in		Cv=	1.00	
E=	2,000	ksi				



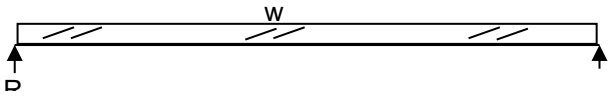
Beam		M4		LSL	3 1/2	x 11 7/8
w=	225	plf		R=	900	lbs
L=	8	ft		M=	1,800	ft-lbs
b=	3.50	in		Fb=	263	psi
d=	11.88	in		Fv=	24	psi
E=	1550	ksi		Δ=	0.03	in
Cv=	1.00	≤1.0		l /	3505	



Beam		M5		HF	4	x 12
w=	1200	plf		R=	2,400	lbs
L=	4	ft		M=	2,400	ft-lbs
b=	3.50	in		Fb=	390	psi
d=	11.25	in		Fv=	49	psi
E=	1300	ksi		Δ=	0.01	in
Cv=	1.00	≤1.0		l /	3749	



Beam		M6		HF	4	x 12
w=	225	plf		R=	788	lbs
L=	7	ft		M=	1,378	ft-lbs
b=	3.50	in		Fb=	224	psi
d=	11.25	in		Fv=	22	psi
E=	1300	ksi		Δ=	0.02	in
Cv=	1.00	≤1.0		I /	3731	



STRUCTURAL ENGINEERING

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Project: Lumpkin Residence *Date:* 03/16/21

Project #:

GRAVITY DESIGN - ROOF Design: VMB

Sheet

TYPICAL POSTS (WORST CASES)

P_1 (ROOF) · ROOF

$= 40\left(\frac{780\ S.F.}{4}\right) = 7800\#$ @ h = 13 ft

6x6 POST

$f_t = 258$ PSI

$D_{CR} = 0.49$

P_1 (FLOOR) = P_1 (ROOF) + FLOOR

$= 7800\# + 75\left(\frac{780\ S.F.}{4}\right) = 22425\#$ @ h = 8 ft

6x6 POST

$f_t = 741$ PSI ∴ NOT OK

$D_{CR} = 0.74$

6x8 POST

$f_t = 562$ PSI ∴ OK

$D_{CR} = 0.56$

PIN PILES @ P_1, $P_1 = 22425\#$

3" ϕ PILES = 6 TON CAPACITY

$\frac{22425}{12000} = 1.86 =$ 2 PILES MIN (USE 3, OPTIONAL)

NOTE: P_1 IS WORST CASE, USE 6x6 BELOW ROOF

6x8 BELOW FLOOR & (2) PIN PILES @ EA. POST

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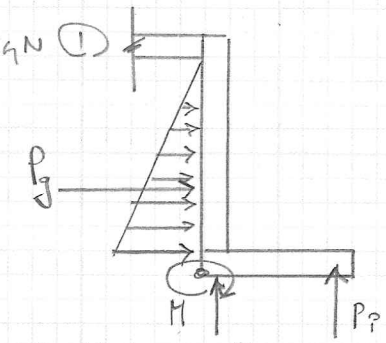
PIPE PILE SUPPORTED RETAINING WALL

TWO DESIGNS: ① 9'-0" WALL, RESTRAINED BY SLAB
② 6'-0" WALL, UNRESTRAINED

DESIGN ①



NOTE: NO MOMENT, RESTRAINED PIN PILES SIZED FOR GRAVITY ONLY

NOTE: NO SLIDING, (4) SIDED "PLANTER" BOX DESIGNED, RESTRAINED ALL (4) SIDES

P_p = GRAVITY
= (8" WALL x 10 ft) + WALL + FLOOR + ROOF + SLAB
$= 150\left(\frac{8}{12}\right)(10) + 150 + (52+40)\left(\frac{15}{2}\right) + 150\left(\frac{8}{12}\right)(4)$
= 2240 PLF

3" ϕ PIN PILES, 6T CAPACITY

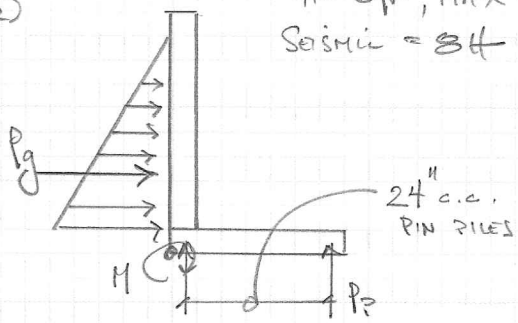
$\frac{12000\#}{2240} = 5.35$ ft oc

NOTE: THIS IS WORST CASE GRAVITY LOADING, MATCH ALL PILES

48" oc MAX

DESIGN ②

H = 6 ft, MAX
SEISMIC = 8H



P_g = 817.2 lbs ← SLIDING FORCE/ft

$M_{BASE} = 817.2(2.28\text{ ft}) = 1861.3$ lb-ft/ft

$P_p = \frac{M_{BASE}}{d} = \frac{1861.3}{2\text{ ft}} = 930.65$ lb/ft

3" ϕ PIN PILES, 6T CAPACITY

$\frac{12000\#}{930.65} = 12.89$ ft oc

USE F.S = 1.5, SPACING = 8.5 ft oc

P_g (w/ SEISMIC) = 1018.1 lbs ← SLIDING FORCE

M_{BASE} (w/ SEISMIC) = 2685 lb-ft/ft

$P_p = \frac{M_{BASE}}{d} = \frac{2685}{2\text{ ft}} = 1343$ lb/ft

3" ϕ PIN PILES, 6T CAPACITY

$\frac{12000\#}{1343\text{ lb/ft}} = 8.93$ ft oc

USE F.S = 1.5, SPACING = 5.96 ft oc

NOTE: THIS IS WORST CASE RETAINING LOADING, MATCH ALL PILES

48" oc MAX

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CHECK RESISTING OVERTURNING, LIMIT RATIO TO 1.1 FOR SEISMIC OTM PER IBC SECTION 1807.2.3

M_{BASE} = 2685 lb-ft (SEISMIC) / ft WIDTH

M_{RESIST} = 1.1(2685) = 2954 lb-ft ← SELF WEIGHT OF WALL REQUIRED TO RESIST

WALL WEIGHT = 150 (8/12)(9 ft) = 900 #/ft WIDTH

M = Pd, d_{REQ} = M/P = 2954/900 = 3.28 ft = 40", CL PILE TO CL WALL

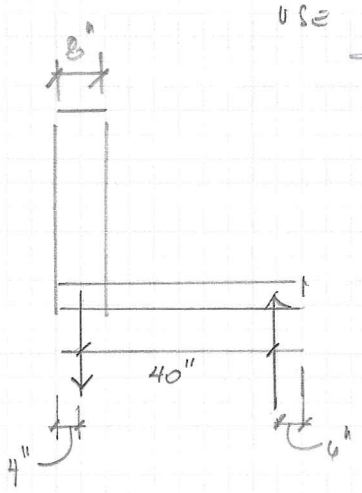
M_{BASE} = 1861.3 lb-ft (NO SEISMIC) / ft WIDTH, OTM RATIO OF 1.5, MIN

M_{RESIST} = 1.5(1861.3) = 2792 lb-ft / ft WIDTH ←

WALL WEIGHT = 900 #/ft WIDTH

M = Pd, d_{REQ} = M/P = 2792/900 = 3.10 ft = 38" CL PILE TO CL WALL

USE 40" AS DIMENSION ON PILES



BY OBSERVATION, SEISMIC GOV'S OVERTURNING

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SLAB DESIGN

8'-0" GRID, ONE WAY SLAB, #5@12" o.c E.W CNTRD

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.306(60)}{0.85(2.5)(12)} = 0.721"$$

$d' = 3.68"$

$$\phi M_n = 0.9(A_s F_y)(d - a/2)$$

$$= 0.9\left(0.306\right)(60)\left(3.68" - \frac{0.721}{2}\right)$$

$$= 54.84 \text{ k-in}$$

$$= 4.57 \text{ k-ft}$$

CHECK QUICK SIMPLE SPAN, 8'-0" GRID

W = floor + conc. $= 1.6L + 1.2D$

$$= 40(1.6) + 5(1.2) + 150\left(\frac{8}{12}\right)(1.2)$$

$$= 190 \text{ PSF (LRFD)}$$

$$M_u = \frac{wL^2}{8} = 1520 \text{ lb-ft} = 1.52 \text{ k-ft}$$

$$= 18.24 \text{ k-in}$$

NOTE: $\phi M_n > M_u \Rightarrow 4.57 \text{ k-ft} > 1.52 \text{ k-ft}$

∴ SLAB ok FLEXURE

BY OBSERVATION, ONE-WAY SLAB DESIGN MORE CONSERVATIVE THAN TWO-WAY SLAB, THEREFORE TWO-WAY SLAB ACTION OK

MAX REACTON ON PIN PILE

$$= 145 \text{ PSF}\left(\frac{8' \times 8'}{4}\right)$$

$$= 2320\# = 2.32 \text{ k}$$

CHECK PUNCHING SHEAR

$$\phi V_c = 4\lambda\sqrt{f'_c} \quad \text{or} \quad \left(2 + \frac{A_s' d}{b_o}\right)\lambda\sqrt{f'_c}$$

ACI 318-14 22.6.5.2

NOTE: $\left(2 + \frac{\alpha_s d}{b_o}\right)$

$$= \left(2 + \frac{40(3)}{5}\right)$$

$d = 3"$
$b_o = 5"$
$\alpha_s = 40$

$V_c = 4\lambda\sqrt{f'_c}$ GOV'S

$$= 4(1.0)\left(\sqrt{2500}\right)$$

$$= 200 \text{ PSI}$$

$\phi V_c = 0.75(200) = 150$ PSI MAX

$V_u(MAX) = v_c(b_o)(d)$

$b_o = 4(5 + 3.5) = 28.2$ in

$\phi V_u(MAX) = 150 \text{ PSI}\left(28.2 \text{ in}\right)(3.5")$

$= 14805$ lbs

$= 14.81$ kips

$\phi V_u > V_u$ ∴ OK IN SHEAR

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TWO WAY SLAB DESIGN: ACI 318-14, CHAPTER 8

NOTE: MINIMUM THICKNESS PER TABLE 8.3.1; $t_{MIN} = l_n/30 = 8F(12")/30 = 3.2"$
W/O EDGE BEAMS & DROP PANELS

8" SLAB MEETS MIN. REQUIREMENT

NOTE: MINIMUM REINFORCING PER TABLE 8.6.1; $A_{s_{MIN}} = 0.0018 A_g = 0.1728 in^2$

#5@12" oc = $0.307 in^2$

#5@12" MEETS MIN. REQUIREMENT

TOTAL FACTORED MOMENT: $M_o = \frac{q_u L_2 l_n^2}{8} = \frac{190 PSF (8)(8)^2}{8}$
(8'-0" GRID; 96" oc)
q_u = 184 PSF → 190 PSF
$= 12.16$ k-f

NOTE: PROJECT SATISFIES THE REQUIREMENTS OF 8.10.2 *DM ok

DIRECT DESIGN METHOD

INTERIOR SPAN: $M^- = 0.65 M_o$, $M^+ = 0.35 M_o$ } 8.10.4.1

EXTERIOR (INT −) $M^- = 0.75 M_o$
(INT +) $M^+ = 0.63 M_o$
(EXT −) $M^- = 0.65 M_o$ } 8.10.4.2

BY OBSERVATION, INTERIOR NEGATIVE MOMENT GOV'S, $M_{DES} = 0.75 M_o$

$M = 0.75 M_o = 0.75(12.16$ k-f$)$
$= 9.12$ k-in

NOTE: 1.00 OF M_u IS IN THE COLUMN STRIP PER 8.10.5.1 & 2

DESIGN OF COLUMN STRIP: (48" WIDE)

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4(0.306)(60)}{0.85(2.5)(48")} = 0.72"$ ‖ $\phi M_n = 0.9\left(4.(0.306).(60)\right)\left(d - \frac{a}{2}\right)$
$= 219.44$ k-in
$= 18.29$ k-f

LUMPKIN RESIDENCE
PROJECT

SLAB DESIGN

3/15/21
DATE

PROJ. # VMB
DESIGN

SHEET