Structural Calculations For:

Marshall Residence (Phase 2)

4307 E. Mercer Way

Mercer Island, WA 98040

Prepared for:	Sturman Architects
Job #:	10315-2019-02
Date:	October 2, 2019

Criteria Sheet

Codes:

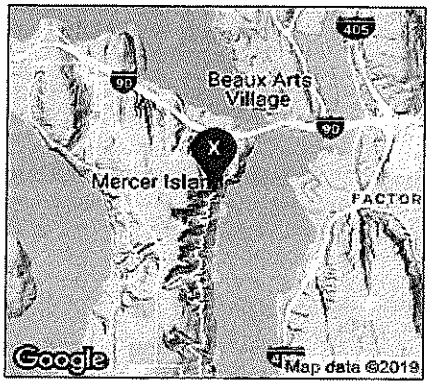
Structural:	IBC 2015
Loading:	ASCE 7-10
Wood:	NDS 2015
Steel:	AISC 360-10
Concrete:	ACI 318-14
Masonry:	TMS 402/602-13

Project Location:

Street & Number:	4307 E Mercer Way
City:	Mercer Islanc State: WA
ZIP:	98040
Latitude:	47.5692 N
Longitude:	-122.2104 W

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 =	12 kips	Ω_o=	2.5
S_S=	1.405	S_1=	0.539
S_{DS}=	0.94	S_{D1}=	0.54
C_s=	0.144	I_E=	1.0

Wind Load Summary:

V=	110	K_{zt}=	1.32
Exposure =	C		

Dead Loads:

Roof	
Roofing	2.5 psf
1/2" Sheathing	1.8 psf
Trusses @ 24" oc	2.5 psf
Misc./Mech.	1.5 psf
Ceiling Finish	2.8 psf
Solar Panels	3.9
	15 psf
Use	15 psf

Floor	
Finish Floor	1 psf
3/4" Sheathing	2.7 psf
Joists @ 16" oc	2.2 psf
Misc./Mech.	2 psf
Ceiling Finish	2.8
	10.7 psf
Use	12 psf

Live Loads:

Snow	25	psf
Floor	40	psf

Soils:

Allowable Bearing	1500	psf

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Seismic Design

ASCE 7-10 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	per soils report (D assumed, without soils report)
Diaphragm Flexibility	Flexible	Bearing Wall Systems

Ω_o	2.5	
S_S	1.405 g	2% in 50 yr, Latitude & Longitude lookup
S_1	0.539 g	2% in 50 yr, Latitude & Longitude lookup
h_n	20.3 ft	
R	6.50	
I_e	1.0	Table 1.5-2
C_d	4	
Ct	0.02	Table 12.8-2
x	0.75	Table 12.8-2
T	0.19 sec	Eq. 12.8-7
T_0	0.12 sec	
T_s	0.58 sec	
k	1.000	
Fa	1.00	Table 11.4-1
Fv	1.50	Table 11.4-2
S_{MS}	1.41 g	Eq. 11.4-1
S_{M1}	0.81 g	Eq. 11.4-2
S_{DS}	0.937 g	Eq. 11.4-3
S_{D1}	0.539 g	Eq. 11.4-4
Cs	***0.144 Controls***	Eq. 12.8-2
	0.434	Eq. 12.8-3 need not exceed, $T < T_L$
	0.010	Eq. 12.8-5 or 12.8-6 minimum
Cs, design	0.144	
Bldg. Weight	85.6 k	
$V = C_S W$	12.3 k	Eq. 12.8-1, Strength Level Base Shear
$V = C_{Sasd} W$	8.6 k	Eq. 12.8-1 ASD Base Shear

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

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

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

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

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

Vertical Distribution ASD ρ= 1.3

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)	$F_{px,calc}$	$F_{px,min}$	$F_{px,max}$	$F_{px,design}$	$\gamma=F_{px}/F_x$
High Roof	20.3	18.24	20.3	369	0.354	3.97	**4.0**	3.1	2.4	4.8	**3.1**	0.77
Up. Fl. / L. Rf	10.0	67.35	10.0	674	0.646	7.25	**11.2**	6.8	8.8	17.7	**8.8**	1.22
Σ		85.6		1043		**11.22**						

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Wind Design - MWFRS

ASCE 7-10 Chapter 27 - Directional Procedure

Design Method	ASD

Wind Coefficients

Exposure	C	
V=	110	mph
K_d=	0.85	Table 26.6-1
K_h=	0.92	Table 27.3-1
G=	0.85	26.9.4

Location and Building Dimensions

Calculate Kzt?	Yes	
Kzt	1.23	
Roof Type	Hip	
Roof Angle - Transverse Dir	25	degrees
Roof Angle - Long Dir	25	degrees
Ground to top of roof	25	ft
Bot of roof to top of roof	5	ft
Mean Roof Height, h	22.5	ft
Short Plan Dimension	55.25	ft
Long Plan Dimension	96.25	ft
Parapet ?	No	
Ground to top of parapet		ft
Average Parapet Height		ft
Ht of 2nd Level Above Grade	15	ft

Velocity Pressure at Mean Roof Height, q_h =	32.5	psf

Transverse Wind Pressures

L/B = 0.57 h/L = 0.41

Pressure Coefficients from Figure 27.4-1:

Bldg Face	C_p
Windward Wall	0.8
Leeward Wall	-0.50
Windward Roof	-0.26 / 0.24
Leeward Roof	-0.60

Wall Pressures (Unfactored): ASD

Ht	K_z	q_z	$P_{ww\ walls}$	$P_{lwwalls}$	P_{walls} (psf)
0-15	0.85	27.53	18.72	13.83	**19.53**
15-20	0.9	29.15	19.82	13.83	**20.19**
20-25	0.94	30.44	20.70	13.83	**20.72**
25-30	0.98	31.74	21.58	13.83	21.25
30-40	1.04	33.68	22.90	13.83	22.04
41-50	1.09	35.30	24.00	13.83	22.70
51-60	1.13	36.60	24.88	13.83	23.23
61-70	1.17	37.89	25.77	13.83	23.76
71-80	1.21	39.19	26.65	13.83	24.28
81-90	1.24	40.16	27.31	13.83	24.68
91-100	1.26	40.81	27.75	13.83	24.95

Roof Pressures (Unfactored) ASD

Windward		Leeward	Horiz Proj (psf)
Max	Min		
6.6	-7.3	-16.6	**5.87**

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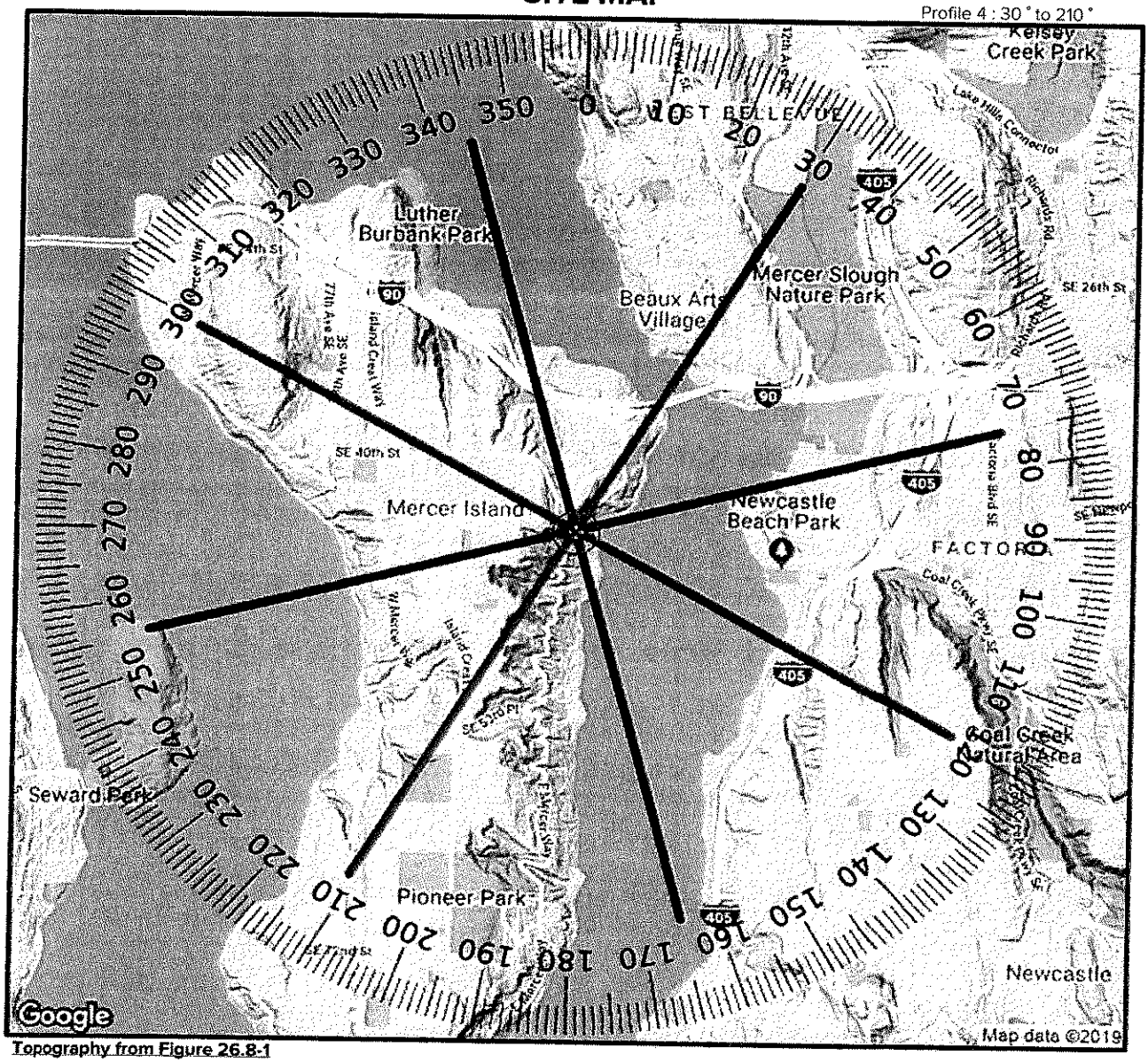
Site Address

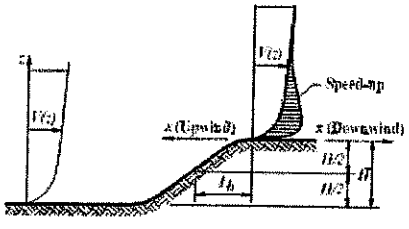
Address	4307 East Mercer Way			Wind Radius	2.00 Miles
City:	Seattle	State:	WA	Angle	15 °
Lat Long	47.56919	-122.2104		Exposure	C

Profile 1 : -15 ° to 165 °
Profile 2 : 255 ° to 75 °
Profile 3 : 300 ° to 120 °
Profile 4 : 30 ° to 210 °

SITE MAP



Topography from Figure 26.8-1



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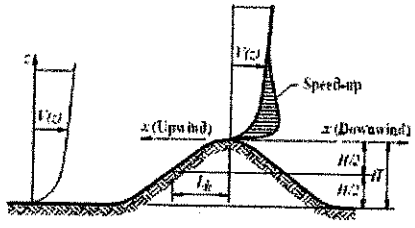

2-D RIDGE OR 3-D AXISYMMETRICAL HILL

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

PER FIGURE 26.8-1

MARSHALL RESIDENCE

4307 East Mercer Island

Kzt Calculations

DATE 8/8/2019

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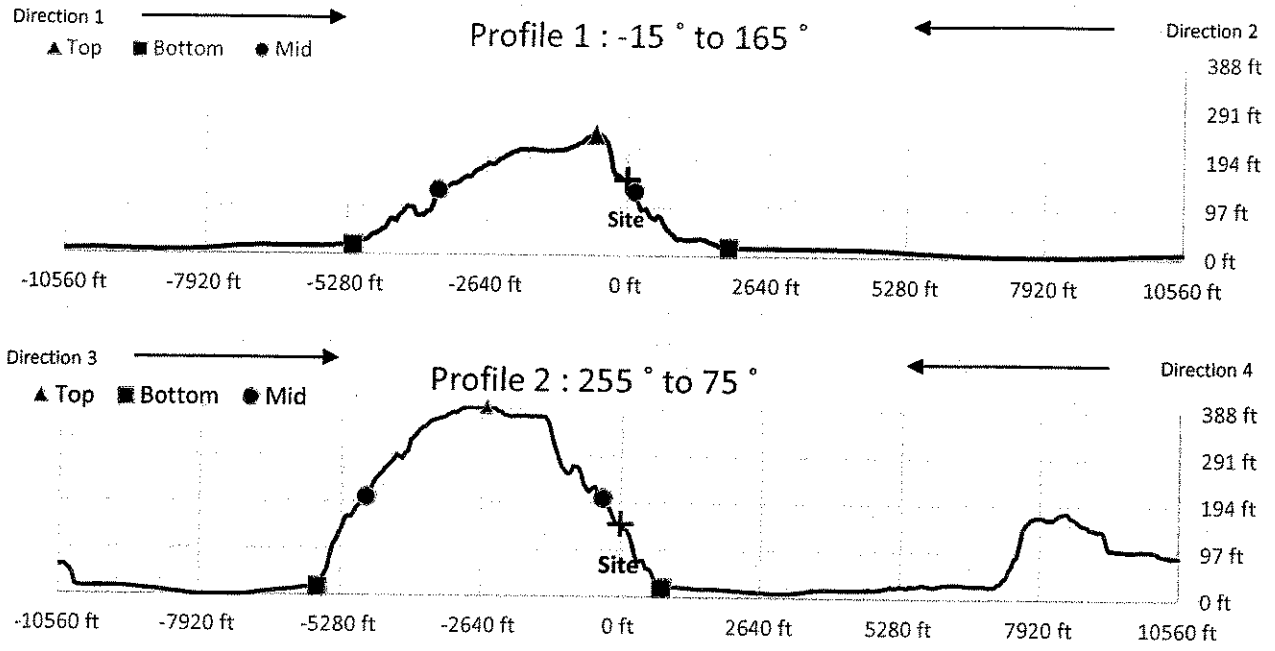

Direction 1 - -15 °to Site

Site Conditions (26.8.1)

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

Terrain Data

Terrain	Hill
Top of Hill Dist.	-584
Bott. of Hill Dist.	-5150
L @ H/2	-3555
Site	downwnd
Top of Hill Elev.	239
Bott. of Hill Elev.	18
Site Elev.	150.9
Site Dist.	0
H/2	128

Kzt Calculations

H=	221
Lh=	2971
x=	584
z=	60
μ=	1.5
γ=	4
K1 value =	1.05
K1=	0.08
K2=	0.87
k3=	0.92
H/Lh =	0.07
Kzt =	1.00

Direction 2 - Site to 165 °

Site Conditions (26.8.1)

1. Unubstructed	Yes	
2. Isolated	Yes	
3. Upper Half Hill	Yes	
4. H/Lh ≥ 0.2	Yes	
5. H≥ 15'	Yes	

Terrain Data

Terrain	Hill
Top of Hill Dist.	-584
Bott. of Hill Dist.	1930
L @ H/2	159
Site	upwind
Top of Hill Elev.	239
Bott. of Hill Elev.	17
Site Elev.	150.9
Site Dist.	0
H/2	128

Kzt Calculations

H=	222
Lh=	743
x=	584
z=	60
μ=	1.5
γ=	4
K1 value =	1.05
K1=	0.31
K2=	0.48
k3=	0.72
H/Lh =	0.30
Kzt =	1.23

Direction 3 - 255 °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.	-2547
Bott. of Hill Dist.	-5680
L @ H/2	-4800
Site	downwnd
Top of Hill Elev.	391
Bott. of Hill Elev.	16
Site Elev.	150.9
Site Dist.	0
H/2	204

Kzt Calculations

H=	374
Lh=	2253
x=	2547
z=	60
μ=	1.5
γ=	4
K1 value =	1.05
K1=	0.17
K2=	0.25
k3=	0.90
H/Lh =	0.17
Kzt =	1.00

Direction 4 - Site to 75 °

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.	-2547
Bott. of Hill Dist.	800
L @ H/2	-320
Site	upwind
Top of Hill Elev.	391
Bott. of Hill Elev.	18
Site Elev.	150.9
Site Dist.	0
H/2	204

Kzt Calculations

H=	373
Lh=	2227
x=	2547
z=	60
μ=	1.5
γ=	4
K1 value =	1.05
K1=	0.18
K2=	0.24
k3=	0.90
H/Lh =	0.17
Kzt =	1.00

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Project MARSHALL RESIDENCE
Kzt Calculations

DATE 8/8/2019
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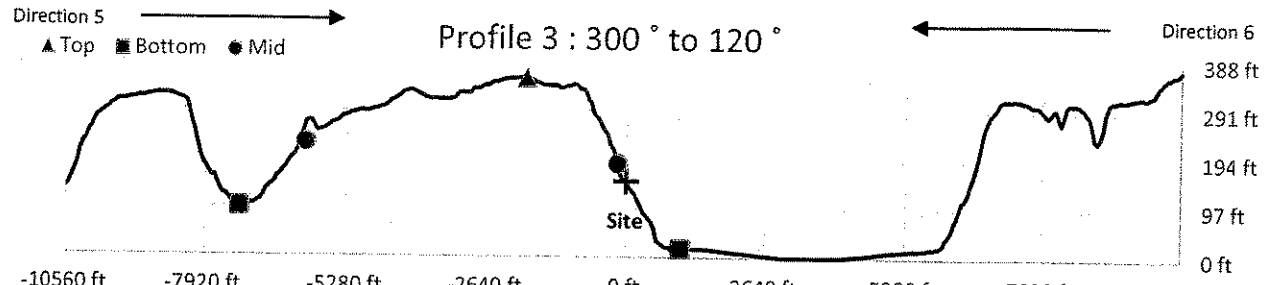

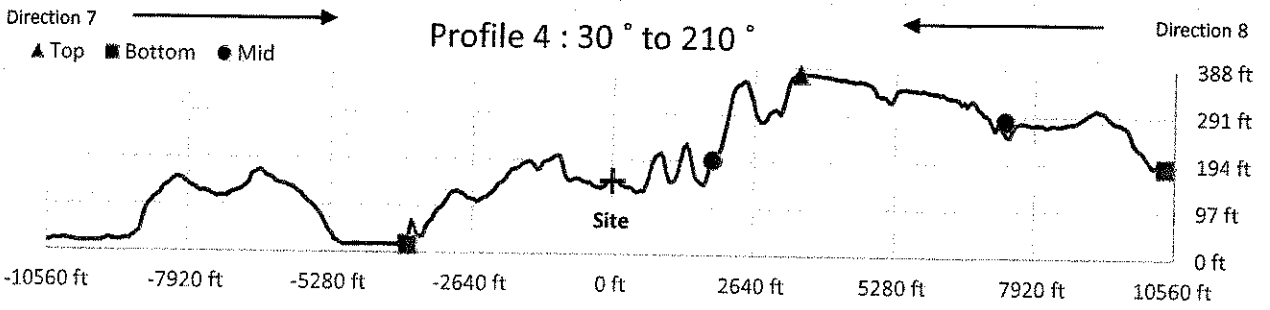

Direction 5 - 300 °to Site

Site Conditions (26.8.1)

1. Unubstructed	No	Kzt=1
2. Isolated	No	Kzt=1

Ridge

Terrain Data

Terrain	Ridge
Top of Hill Dist.	-1910
Bott. of Hill Dist.	1050
L @ H/2	-160
Site	upwind
Top of Hill Elev.	356
Bott. of Hill Elev.	16
Site Elev.	150.9
Site Dist.	0
H/2	186

Kzt Calculations

H=	339
Lh=	1750
x=	1910
z=	60
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.28
K2=	0.27
k3=	0.90
H/Lh =	0.19
Kzt =	1.00

Direction 6 - Site to 120 °

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.	3502
Bott. of Hill Dist.	-3821
L @ H/2	1870
Site	upwind
Top of Hill Elev.	374
Bott. of Hill Elev.	16
Site Elev.	150.9
Site Dist.	0
H/2	195

Kzt Calculations

H=	357
Lh=	1632
x=	3502
z=	60
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.32
K2=	0.00
k3=	0.90
H/Lh =	0.22
Kzt =	1.00

Direction 7 - 30 °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.	3502
Bott. of Hill Dist.	10400
L @ H/2	7376
Site	downwnd
Top of Hill Elev.	374
Bott. of Hill Elev.	187
Site Elev.	150.9
Site Dist.	0
H/2	280

Kzt Calculations

H=	187
Lh=	3874
x=	3502
z=	60
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.07
K2=	0.40
k3=	0.95
H/Lh =	0.05
Kzt =	1.00

Direction 8 - Site to 210 °

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.	3502
Bott. of Hill Dist.	10400
L @ H/2	7376
Site	downwnd
Top of Hill Elev.	374
Bott. of Hill Elev.	187
Site Elev.	150.9
Site Dist.	0
H/2	280

Kzt Calculations

H=	187
Lh=	3874
x=	3502
z=	60
μ=	1.5
γ=	3
K1 value =	1.45
K1=	0.07
K2=	0.40
k3=	0.95
H/Lh =	0.05
Kzt =	1.00

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Kzt Calculations

DATE 8/8/2019

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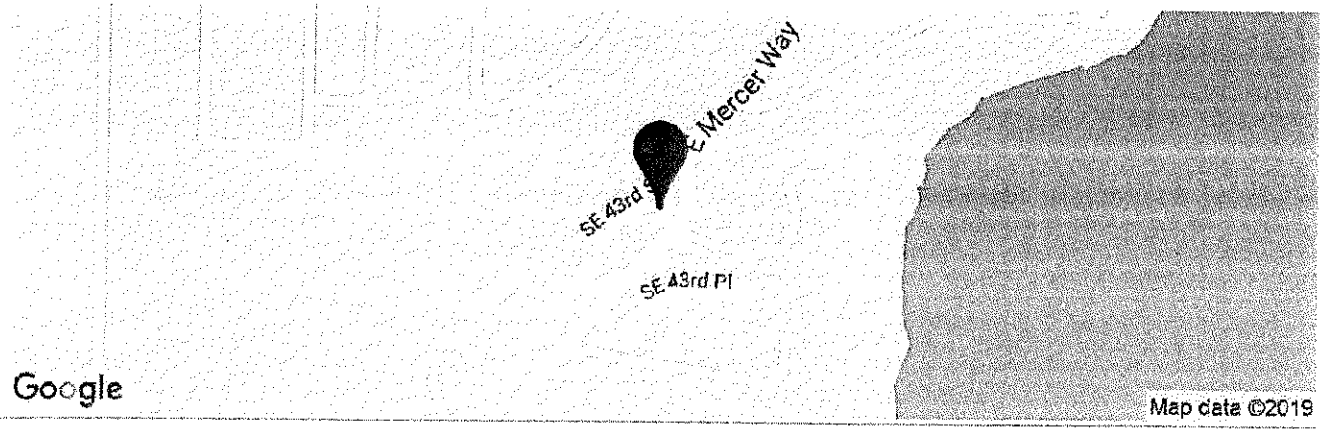
SHEET 6

OSHPD

MARSHALL RESIDENCE

4307 E Mercer Way, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.569191, -122.21035740000002



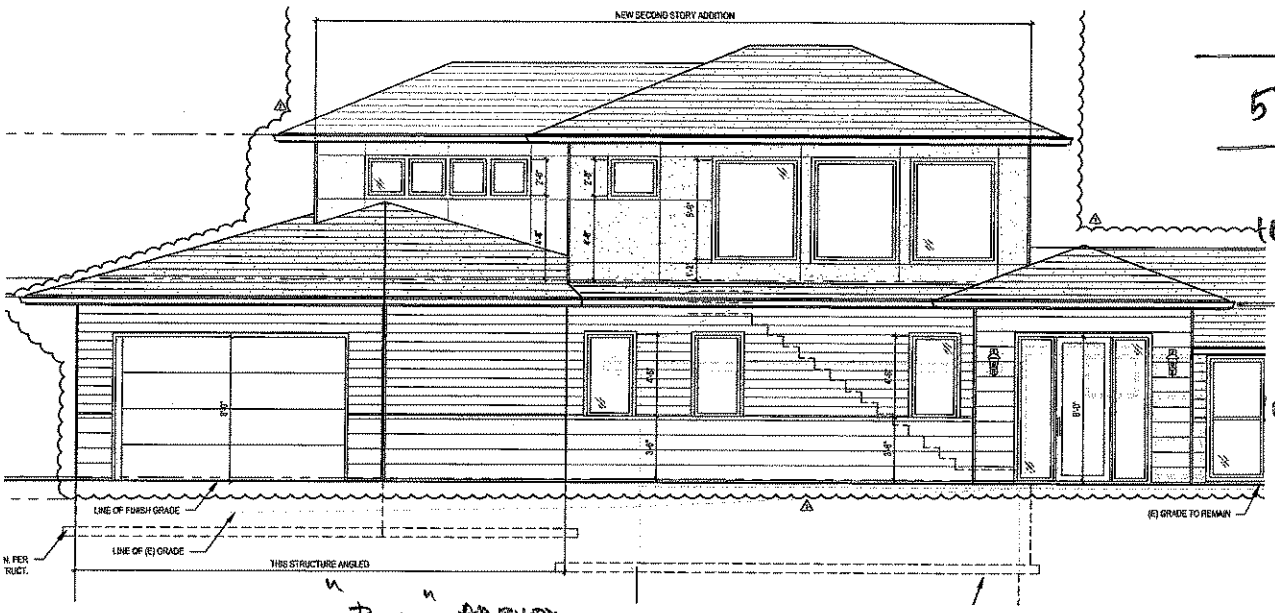
Date	8/16/2019, 2:14:58 PM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.405	MCE_R ground motion. (for 0.2 second period)
S_1	0.539	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.405	Site-modified spectral acceleration value
S_{M1}	0.809	Site-modified spectral acceleration value
S_{DS}	0.937	Numeric seismic design value at 0.2 second SA
S_{D1}	0.539	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.58	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.58	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.405	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.469	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	3.298	Factored deterministic acceleration value. (0.2 second)
S1RT	0.539	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.578	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.309	Factored deterministic acceleration value. (1.0 second)
PGAd	1.278	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.957	Mapped value of the risk coefficient at short periods
C_{R1}	0.933	Mapped value of the risk coefficient at a period of 1 s

Marshall Residence

8/16/19



"Box" approx.

Wind loading (based on this pic)

W_w (Roof) = Roof + wall

= 5 ft (5.87 psf) + 5 (20.19)

= 130 PLF

W_w (upp) = wall

= 10 (19.53)

= 195 PLF } under 2nd floor

= Roof + wall

= 5 (5.87) + 5 (19.53)

= 127 PLF } w/o 2nd floor this one

North / South, L = 31.75 ft, L = 96.25 ft

East / West, L = 20 ft @ Roof, L = 55.25 ft

C Roof = 5.17 k = 130 PLF

2.60 k = 130 PLF

C upp = 17.75 k = 195 PLF under 2nd floor

7.18 k = 127 PLF under low roof } totals

3.90 k = 195 PLF under 2nd floor

4.52 k = 127 PLF under low roof

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Markham P2 — PROJECT

Wind loading

9/12/19 — DATE

VMB — PROJ. #

DESIGN

8 — SHEET

Seismic Weight

W_x (Roof) = Roof + Walls

= 15 psf (980 SF) + 5 psf (725)

C_h = 20.25 ft = 18.235 k

W_x (Low roof / up. floor) = Roof + Walls + Floor + Walls

C_h = 10 ft = [15 (2300) + 5 (1880)] + [12 (725) + 10 (725)]

= 67.350 k

Story Shears

	North/South, L = 96.25 ft	East/West, L = 55.25 ft
	L = 39.75 ft @ Roof	L = 20 ft @ Roof
V_s (Roof) = 3.97 k	3.97 k = 100 PLF	3.97 k = 199 PLF
V_s (Upp) = 7.25 k	7.25 k = 75 PLF	7.25 k = 131 PLF

Wind Loading

V_w (Roof) = 5.17 k (N/S) = 130 PLF

260 k (E/W) = 130 PLF

V_w (Upp) = 795 PIF @ Floor

127 PIF @ Roof Low

(2) Numbers based on Upper Roof or Upper Floor

Note: Only 1/2 of V_s (Upper) occurs at area of remodel based on proportional seismic mass. So seismic force at addition is (V_s/2)

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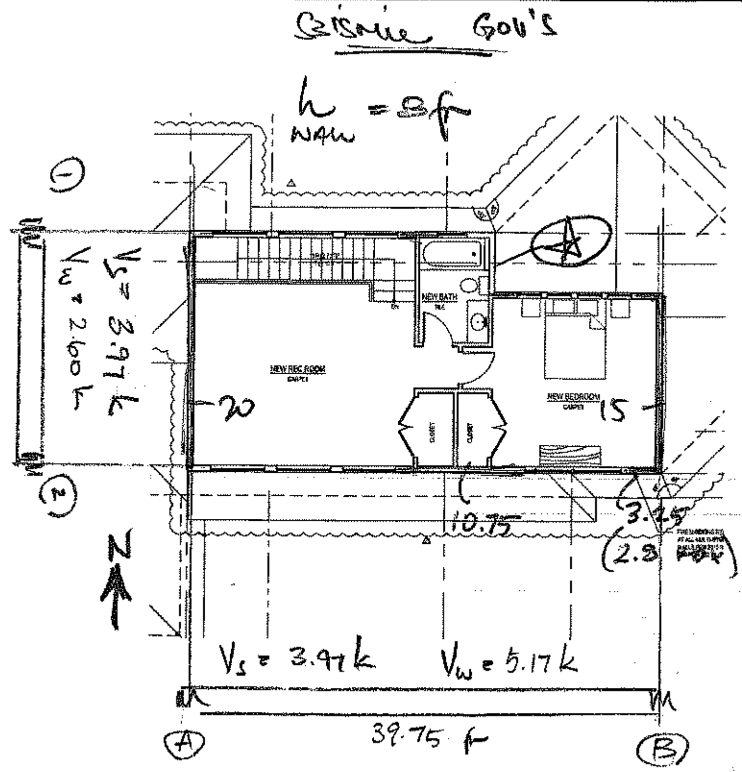
SSF STRUCTURAL ENGINEERING

PROJECT	Marshall Phase 2
DATE	9/12/19
PROJ. #	
DESIGN	VMB
SHEET	9

SEISMIC GOV'S

$h_{WALL} = 8 f$



V
L3
5c
OT
OTR
OT'
HD

1.99k
FTAO REQ'D

1.99k
13.55 f
145 #/f
(W6)
1.17 k
0.18 k
0.99 k
OT NOT CRIT
HORZ EXN REQ'D

	A	B
V	1.99 k (E), 2.59 k (W)	1.99 / 2.59
L_W	20 f	15 f
v	100 (E) / 130 (W) #/f	133 / 173
S.W	(W6)	(W6)
OT	1.46 k	1.38 k
OTR	1.17 k	0.88 k
OT'	0.29 k	0.50 k
HD	NOT CRIT	NOT CRIT
OT (S20)	1.99 k (S20)	2.65 k (S21)

(★) If using this wall:

V = 2.59 k (W), 1.94 k (E)

L = 4.75 f

v = 545 #/f (W)

419 #/f (E)

W3

OT = 4.36 k

(2) CS14

NOTE: SHEAR WALL v IS EQUIVALENT TO DIAPHRAGM v SINCE $L_W = L_{diaph}$. MUST BLOCK DIA w/ 8d @ 4" oc NAILING AT WALL B

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SSF STRUCTURAL ENGINEERING

PROJECT: MARSHALL RESIDENCE

LATERAL @ ROOF

DATE: 9/12/19

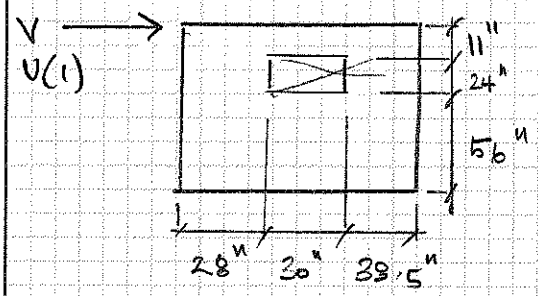
PROJ. #: VMB

DESIGN

SHEET: 10

FTA0-1 NORTH ELEVATION
EAST/WEST DIRECTION

V_E 1.99 k (SEISMIC) IN WALL



V_E = 1.99 k

v = 359 #/f

S.W = (W3)

STRAP F = 0.99 k

STRAP = CS16

OT = 1.03 k

OTR = 0.20 k

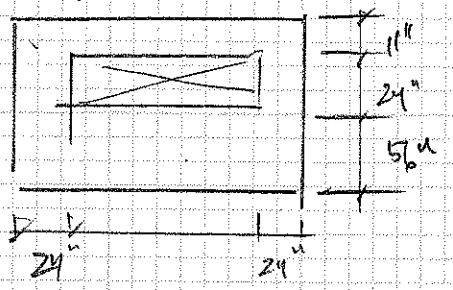
OT' = 0.73 k

OT NOT CMU

NOTE: TO FORCE TRANSFER FOR REDUNDANCY ON OTHER WALL

EAST ELEVATION

V = 1.0 k (SEISMIC)
USING SHEARWALL ② ★



V = 1.0 k

v = 494 ~ W3

SW = (W2)

STRAP F = 0.49 k

STRAP = CS16

OT = 0.52 k

OT NOT CMU

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SSF STRUCTURAL ENGINEERING

MAKAH RESIDENCE
PROJECT

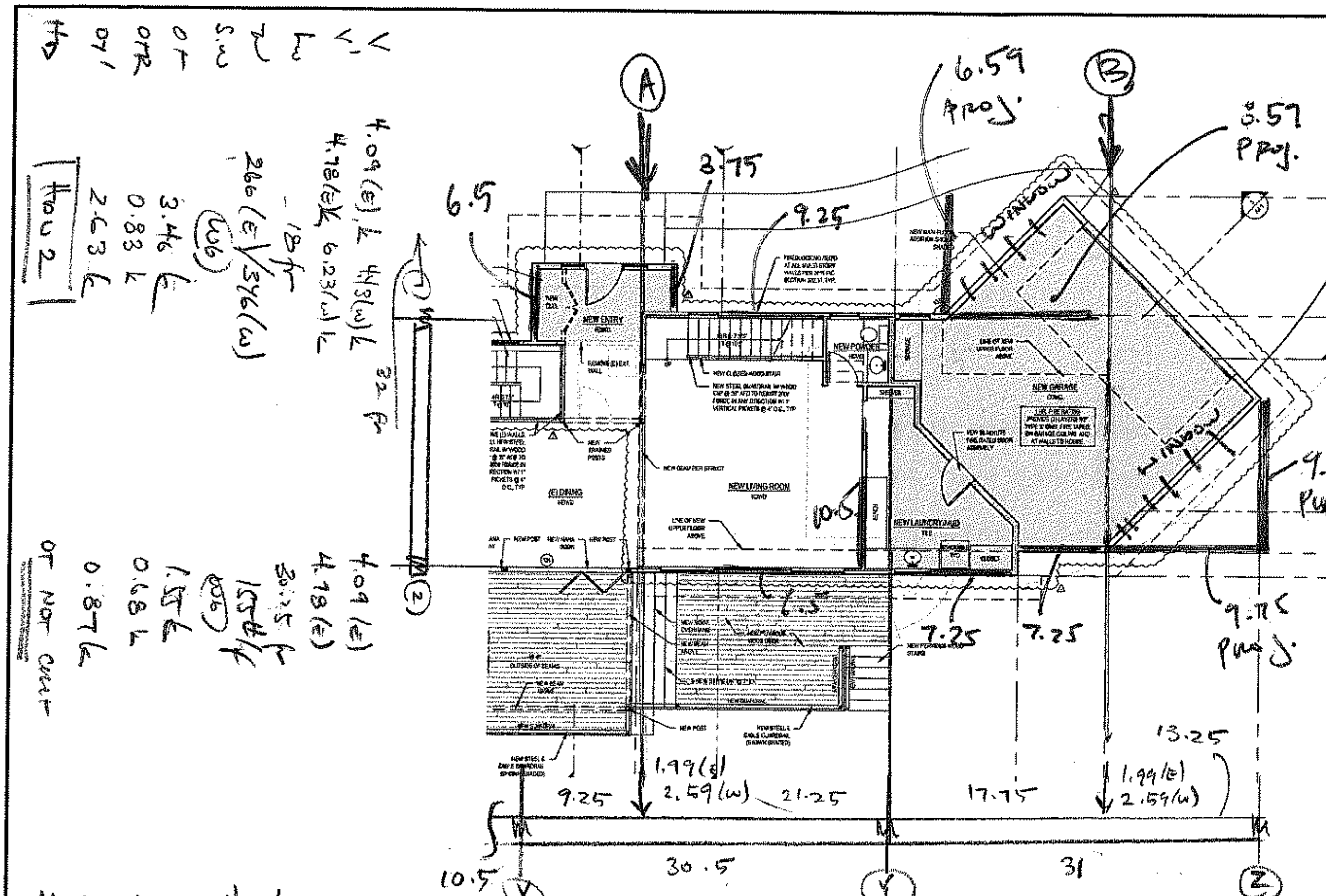
FTA0 - LATERAL @ Roof

10/2/19
DATE

PROJ. # VMB

DESIGN

SHEET 11



Note: Type 4 Horiz. Irregularity w/ Type 5 @ End.

Increase loads 25% North/South (Y+Z)(B) & East/West (2) Direction. Add 30% load of orthog. Directions per ASCE 7, 12.5

Shown as V', Seismic load only.

	X	Y	Z
V	3.92(E)k, 6.23(w)k	3.53(E)k, 7.29(w)	4.38(E), 4.32(w)
V'	4.90(E)k, 6.23(w)k	4.42(E)k, 7.29(w)	5.47(E), 4.32(w)
Lwall	10.25 ft	17 ft	9.75 ft
v	478(E), 608(w) #/f	260(E), 429(w) #/f	561(E), 443(w)
S.W	W2	W4	W2
OT	6.08 k	4.29 k	5.61 k
OTR	—	0.21 k	2.56 k
OT'	6.08 k	4.08 k	3.05 k
HD	HDU8 w/ 4x6 or (3)2x	HDU5 @ 7ft wall segment	HDU2 w/ 7.ft

Sim ↑ @10' proj. HDU

Note: Block DIA @ V(B)

See next page for FTG design

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PROJECT: Marshall Residence P2
DATE: 9/12/19
PROJ. #: VMB
DESIGN: 12
SHEET

Spread Footing Soil Bearing Design

Service Loads Loading

Dead Load =	0.0 kips
Live Load =	0.0 kips
Wind/EQ Load =	4.3 kips
Wind/EQ Moment (M_Y) =	30 ft-kips
Gravity Load Eccentricity (±X)=	0.88 ft.
Footing Weight =	3.7 kips
Total Load =	8.0 kips
Total Moment =	34 ft-kips

Service Load Factors

DL	1
LL	1
EQ/Wind	1

Soil Properties

Allowable Soil Brg. (Qa) =	2.00 ksf
Overburdan Density (γs) =	120 psf
Net Ftg Wt? (γ_c-γ_s)	No

Column Dimensions and Location

Column Xc Dimension (Dx) =	5.50 in.
Column Yc Dimension (Dy) =	5.50 in.
Column Face from right (Cr) =	5.02 ft.
Column Face from left (Cl) =	6.77 ft.

Soil Bearing Check (Allowable)

Eccentricity =	4.24 ft.	
Leng. Soil Brg. Under Ftg.=	5.65 ft.	
qmax =	1.41 ksf	OK
qmin =	0.00 ksf	

1 ksf

Footing Dimensions

L Dimension (X) =	12.25 ft.
B Dimension (Y) =	2.00 ft.
Footing Thickness (t) =	12.00 in.
Ftg Overburden (Ot) =	0.00 ft.

Soil Pressure Equations:

$e \le L/6$

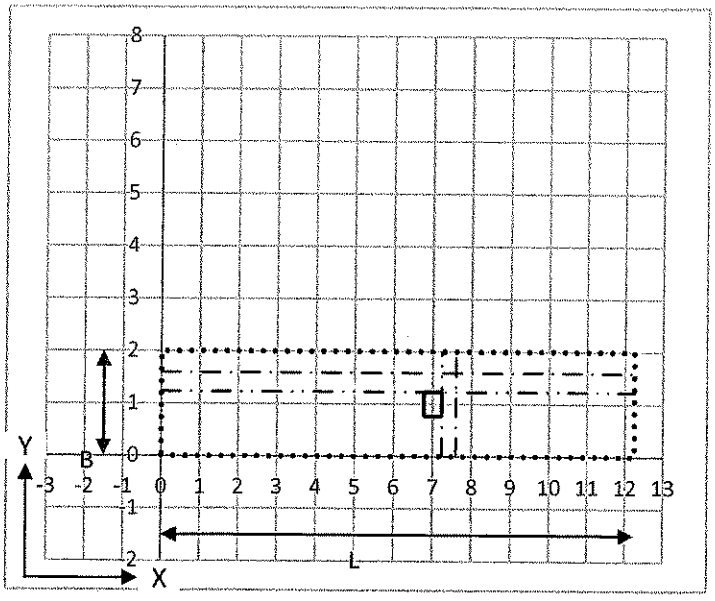
$$qmax = \frac{Q}{LB}\left(1 + \frac{6e}{L}\right)$$

$$qmin = \frac{Q}{LB}\left(1 - \frac{6e}{L}\right)$$

$e > L/6$

$$qmax = \frac{4Q}{3L(L-2e)}$$

$$qmin = 0$$



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STRUCTURAL ENGINEERING

Marshall Residence P2 — PROJECT

New Footing at Shear Wall

9/16/2019 — DATE

PROJ. #

DESIGN VMB

SHEET 13

Spread Footing Concrete Design - ACI 318-14

Footing Properties

Concrete Strength (f'c) =	2500 psi
Rebar Yield Strength (f_y) =	60000 psi
Reinforcing Clear Cover (cvr) =	3.00 in.
Reinforcing Depth (d) =	8.75 in.

Strength Load Factors

DL	1.2
LL	1.6
EQ/Wind	1

Factored Loads

Factored Total Load =	8.7 kips
Factored Total moment =	30 ft-kips

Factored Moments and Shears

	Mu k-ft	Vu kips
X Right Side	22	7
X Left Side	1	0
Y Both Sides	1	2

Factored Bearing

Eccentricity =	3.45 ft.
Length of Soil Brg. Under Ftg. =	8.02 ft.
qmax =	1.08 ksf
qcolr =	0.41 ksf
qcoll =	0.34 ksf
qmin =	0.00 ksf

Check Negitive Moment Reinforcing Top of Footing

Flexural Design - X Direction

Bar Size =	#4	
Bars =	12	
Mu =	22 ft-kips	
ØMn =	79 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0004	
A_s Required =	2.32 sq. in.	12" oc
A_s Provided =	2.40 sq. in.	OK

Flexural Design - Y Direction

Bar Size =	#4	
Bars =	2	
Mu =	1 ft-kips	
ØMn =	16 ft-kips	OK
ρ_{min} =	0.0018	*Controls*
ρ_{req} =	0.0002	
A_s Required =	0.38 sq. in.	
A_s Provided =	0.40 sq. in.	OK

One-Way Shear Design - X Direction

Vu =	7 kips	
ØVn =	16 kips	OK

β =	6.125
$\gamma_s = 2/(\beta+1)$ =	0.28
Provide $A_{s,req}\gamma_s$ =	0.11 sq. in.

Provide 1 bars in center 2 ft of the ftg

and the remainder outside of those extents.

One-Way Shear Design - Y Direction

Vu =	2 kips	
ØVn =	96 kips	OK

Two-Way (Punching) Shear Design

b_o =	57 in	
vu =	8 kips	
Øvn =	73 kips	OK

Concrete Capacity Equations:

$$Mn = A_s F_y \left[d - \frac{1}{2}\left(\frac{A_s F_y}{0.85 f'c\, b} \right) \right]$$

$$vn = min \begin{pmatrix} 4\sqrt{f'c} \\ \left(2 + \frac{4}{\beta}\right)\sqrt{f'c} \\ \left(2 + \frac{\alpha_s d}{b_o}\right)\sqrt{f'c} \end{pmatrix} b_o d$$

$$Vn = 2\,\gamma\sqrt{f'c}\, b_w d$$

$$b_o = 2(Dx + d) + 2(Dy + d)$$

$$\beta = \max(Dx, Dy)/\min(Dx, Dy)$$

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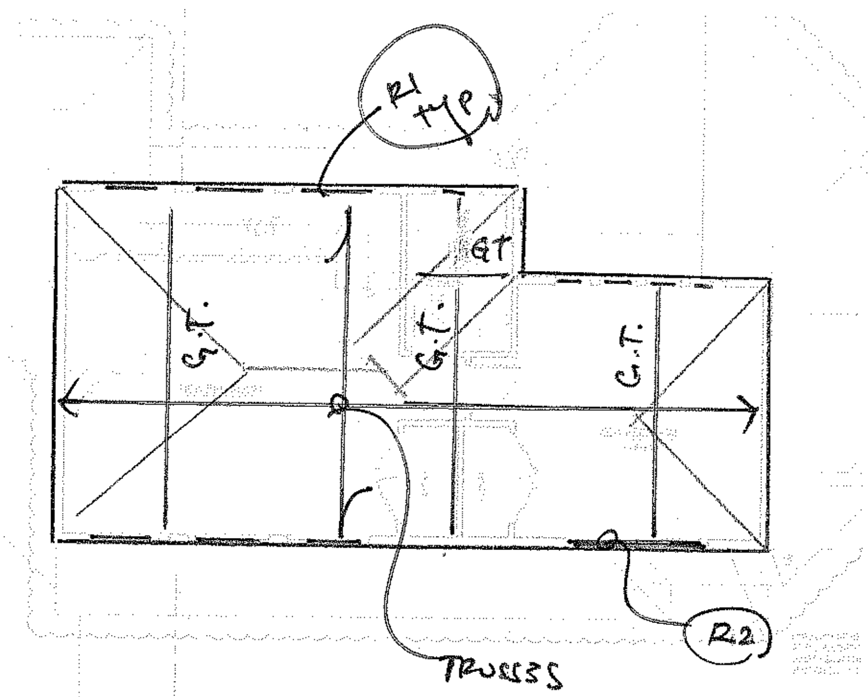

R1 - TYP. EXT. HDR.

W = ROOF = 42(20/2) = 420 PLF

SPAN = 4.75 ft — (2)2x6 MIN.

V = 998 # — fv = 73 PSI

M = 1185 lb-ft — fb = 940 PSI

Δ = 0.09" = L/641

R2 - BEDROOM HDR.

W = ROOF = 42(15/2) = 315 PLF

SPAN = 8.5 ft — (2)2x10 HDR.

V = 1260 # — fv = 59 PSI

M = 2844 lb-ft — fb = 798 PSI

Δ = 0.14" = L/709

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SSF STRUCTURAL ENGINEERING

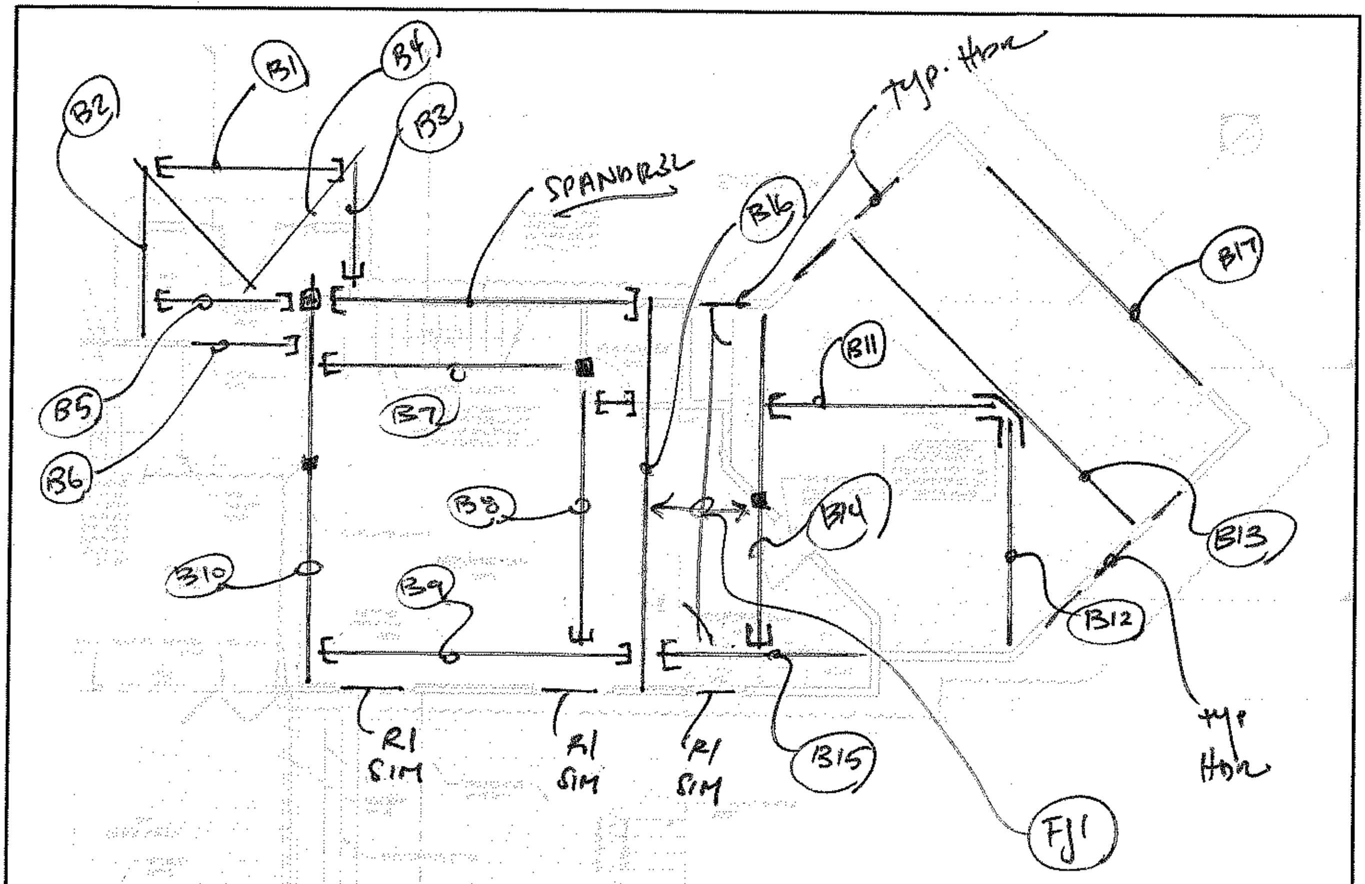
PROJECT: MARSHALL DR.

DATE: 9/13/19

PROJ. #: VMB

DESIGN: 15

SHEET: BRACING FRAMING @ ROOF



FJ1 – TYP FLOOR JOISTS

w = FLOOR = 52(16/12) = 70 PLF

SPAN = 20 ft OUT-OUT   **TJI 230 @ 16" o.c.**

V = 700 # V_{allow} = 1360 #

M = 3500 lb-ft M_{allow} = 4847 lb-ft

Δ_{TL} = 0.604" = L/389

TYP. HDR

w = ROOF = 40(775/2) = 160 PLF

OR

1300 W = ROOF + FLOOR + WALL = 840 PLF

SPAN = 3 ft   (2)2x8

V = 1260 # f_v = 87 PSI

M = 945 lb.ft f_b = 432 PSI   ok

PROJECT: MARSHALL P2

GRAVITY FRAMING @ UPP. FLOOR

DATE: 9/3/19

PROJ. #: VMB

DESIGN

SHEET: 16

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B1 – UPTURNED BEAM @ POINT

w = ROOF = 40(4 ft) = 160 PLF
SPAN = 11.5 ft 4x8
V = 920 # f_v = 49 PSI
M = 2645 ft-# f_b = 1035 PSI OK w/ C_b = 1.15
Δ = 0.44" = L/317

B2 – WEST POINT CANTILEVER

w = ROOF = 40(6½) = 120 PLF P = R_{B1} = 920 # CFMS
SPAN = 1.75 ft CANT, 6.5 ft BACK
V_{MAX} = 920 #
M_{MAX} = 1610 ft-# 4x8, MATCH B1

B3 – EAST POINT CANTILEVER

SIM to B2, CHECK UPLIFT

$R_B = \frac{920\#(1.75)}{3.5} = 460\#$

D5/6

B4 – GIRDER TRUSS HIP

(BY OTHERS)

B5 – GIRDER TRUSS HIP MASTER

(BY OTHERS)

B6 – ROOF BEAMS @ EXIST

w = ROOF = 40($\frac{19.5}{2}$) = 390 PLF
SPAN = 6.5 ft (2) 2x12
V = 1268 # f_v = 57 PSI
M = 2060 ft-# f_b = 390 PSI
Δ = 0.03" = L/2304

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SSF STRUCTURAL ENGINEERING

MARSHALL P2
PROJECT

10/2/19
DATE

PROJ. # VMB

DESIGN

17
SHEET

B7 – STAIR PLM

W = floor = 52(12.5/2) = 481 PLF

SPAN = 15 ft

PSL 3 1/2 x 11 7/8

Vc 3608#

Mc 13528 lb-ft

fvc 113 PSI

fb = 1973 PSI

Δ_T 0.56" = L/320 TOTAL

Δ_L = 0.43" = L/417 LL

4x4 POST OK @ h=10 ft

B8 – FLUSH BEAM @ STAIR CORNER

W = floor = 52(1 6/12) = 76 PLF

SPAN =

MATCH B7

B9 – LIVING ROOM FLUSH BEAM

W = ROOF + WALL + FLOOR

= 250 S.L + 320 LL + 400 DL

= 828 TL @ 1.0D + 0.75(S+L)

= 650 TL @ 1.0D + 1.0S

= 720 TL @ 1.0S + 1.0L

= 828 TL

SPAN = 15.25 ft

PSL 7" x 11 7/8

V = 6314 #

fv = 99 PSI

M = 24070 lb-ft

fb = 1756 PSI

Δ = 0.52" = L/355

B10 – FLUSH BEAM @ WEST UPP WALL

W = ROOF + WALL

= 1/2 (8/12) + 150 = 318 PLF = (100 S.L + 218 DL) PLF

P_1 = ROOF + R_{B7} = 750 + 3608 = 4555 # @ 4 ft = 594 # S.L + 2775 # LL + 1186 # DL

P_2 = R_{B9} = 6325 # @ 20 ft → 1906 # SL + 2440 # LL + 3050 # DL

SPAN = 22 ft

P_1 ↓ 9.25' A B 12.75' C ↓ P_2

CONTINUOUS

PSL 5 1/4 x 11 7/8

fvc 155 PSI

fb = 1190 / -1089

↑ Δ^+ = —

↓ Δ^- = 0.15" = L/1021

POST = 8131 # @ h = 10 ft

4L 3 1/2 x 3 1/2

Vc 6428 #

M = 12249 / -11160

R_A = 2436 #

R_B = 8131 #

R_C = 6428 #

1.0 D + 0.75 S + 0.75 L Gov's

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PROJECT: MAXWELL P2

DATE: 9/13/19

PROJ. #:

DESIGN: VMF

SHEET: 18

GRADING FOUNDING @ UPP FLOOR

3/10 – ALTERNATE CONTINUOUS

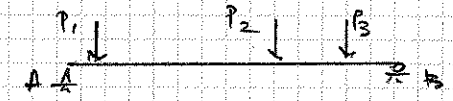
W_{ROOF} WALL = 318 PLF (100 SL + 318 DL) PLF

P_1 = R_{BM} @ 2 ft = 6314 # (1906 SL + 2440 W + 1968 DL)

P_2 = ROOF = 40(19.5/2)(7½) = 1265 # @ 12.5 ft (853 SL + 512 DL)

P_3 = ROOF · 40(19.5/4)(7½) = 683 # @ 19 ft (427 SL + 256 DL) + 3324 # (2536 L + 788 D)

SPAN = 21.75 ft



PSL 5¼ x 18 UPTURNED

V_{MAX} = 8836 #

M_{MAX} = 32668 lb-ft

R_A = 8.835 # (0.75 S+L)

R_B = 7.198 # (0.75 S+L)

f_v = 140 PSI

f = 1383 PSI

Δ_{MAX} = 0.56" = L/465

CHECK COPE @ A

11.5" @ WALL A' = 60.4

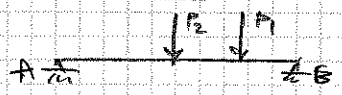
f_v' = 2# PSI ∴ OK

CHECK CONT. RIM – SPANDREL

P_1 = 7198 # @ 9 ft

P_2 = ROOF + 1000 # @ L/2

SPAN = 11.5 ft



PSL 5¼ x 11⅞

V = 6117 #

M = 15286 lb-ft

R_A = 2060 #

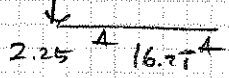
R_B = 6117 #

f_v = 144 PSI

f_b = 1487 PSI

Δ = 0.20" = L/678

CHECK AS CANTILEVER



V = 7198 #

M = 16200 lb-ft

f_{VC} = 173 PSI

f_b = 1576 PSI

Δ = 0.19" = 2L/291

NOTE: THIS BEAM OPTION REPLACES B5 + SPANDREL SINCE SPANDREL HERE IS CONTINUOUS.

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PROJECT: MARSHALL P2

DATE: 10/3/19

PROJ. #: UMB

DESIGN: 19

SHEET

B11 - North Flush Beam @ Garage

w = Roof$_1$ + Roof$_2$ + Wall + Floor
= (94 SL + 56 DL) + (188 SL + 113 DL) + 150 DL
+ 300 LL + 90 DL
= 282 SL + 409 DL + 300 LL = <u>991 PLF</u>
= 691 TL @ 1.0D + 1.0S
= 709 TL @ 1.0D + 1.0L
= 846 TL @ 1.0D + 0.75(S+L)

SPAN = 16 ft
V = 6768 #
M = 27072 lb-ft
≈ 75%

PSL 7" x 11 7/8"
fv = 107 PSI
fb = 1975 PSI
Δ_{TL} = 0.64" = L/301
Δ_{LL} = 0.33" = L/582

SKEWED MGU7-SDS

CONNECTION to SE BM
P = R_{B11} + R_{B12} = 6768 + 3330
= 10100 #
Z⊥ @ NAWW = 510 # EA. 3/4"ϕ
10100 / 510 = 20 <u>BOLTS</u> REQ'D
Z⊥ @ NAWW = 670 # EA. 1"ϕ
10100 / 670 = 16 BOLTS REQ'D

Just do KERF PLATE

B12 - East Flush Beam @ Garage

w = Roof + Wall + Roof
= 42(8/2) + 190 + 42(4/2)
= 444 PLF

SPAN = 15 ft
V = 3330 #
M = 12488 lb-ft

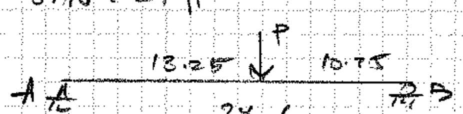
PSL 5 1/4 x 11 7/8
fv = 70 PSI
fb = 1214 PSI
Δ = 0.35" = L/521

PSL 3 1/2 x 11 7/8
fv = 104 PSI
fb = 1822 PSI
Δ = 0.52" = L/348

B13 - Garage Backbone

P = R_{B11} + R_{B12}
= 2256 SL + 3272 DL + 2400 LL + 1313 SL + 2018 DL
= 3569 SL + 5290 DL + 2400 LL = 11259 # TL
= 8859 # @ 1.0D + 1.0S
= 7690 # @ 1.0D + 1.0L
= 9767 # @ 1.0D + 0.75(S+L)
@ 13.25 ft

SPAN = 24 ft



V = 5934 #
M = 61161 lb-ft
R_A = 4915 #
R_B = 5934 #

W10x45
V_n/Ω = 70.7 k
M_n/Ω = 93.3 k-ft @ L_b = 24
Δ = 0.71" = L/409

PROJECT: Marshall P2
Gravity framing @ upper floor

DATE: 9/13/19
PROJ. #: VMB
DESIGN
SHEET: 20

STRUCTURAL ENGINEERING
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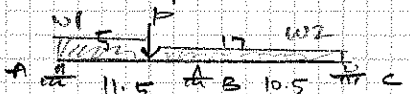
B14 – FLUSH BEAM @ UPPER INCLUDE cont

w_1 = ROOF + WALL
= 40(8/2) + 130 = 100 SL + 210 DL

w_2 = FLOOR' = 52(16/12) = 54 LL + 16 DL

P_1 = R_{B1} = 6768 # @ 1.0D + 0.75(S+L) @ 5 ft

SPAN = 22 ft



	PSL 5 1/4 x 11 7/8 AS SEPARATE SPANS	PSL 5 1/4 x 11 7/8 AS CONT. (2) SPAN
V = 4242 #	fv = 119 PSI	fv = 102 PSI
M = 17629 lb-ft	fb = 2052 PSI	fb = 1710 PSI
R_A = 4242 #	Δ = 0.28" = L/497	Δ = 0.20" = L/687
R_B = 4710 #	R_A = 4938 #	
R_C = −763 # ⇒ 1356 SUST. UPL.	R_B = 3251 #	

(V, M, R_A, R_B, R_C AS 1.0D + 0.75(S+L))

B15 – SOUTH FLUSH BEAM AT LAUNDRY

w_1 = ROOF + WALL + FLOOR = 42(15/2) + 150 + 52(7.5)
= 188 SL + 400 LL + 383 DL

w_2 = ROOF$_1$ + ROOF$_2$ + WALL + FLOOR
= 206 SL + 300 LL + 363 DL

P_1 = R_{B14} = −256 SL, −242 LL, −389 DL @ 6.75 ft

SPAN = 13.25 ft

V = 5136 #

M = 17013 lb-ft

@ 1.0D + 0.75(S+L)

PSL 5 1/4 x 11 7/8

fv = 124 PSI

fb = 1655 PSI

Δ = 0.37" = L/433

POST @ END ⇒

P = R_{B9-B} + R_{B15-A}

= 6314 + 5136

= 11450 #

B16 – DRAG STRUT @ STAIR

w = FLOOR' = 52(16/12) = 70 PLF

NOTE: JUST A TYP. JOIST,

MAKE 1 3/4 LSL

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B17 – GARAGE DOOR HDR

W = ROOF = 42(8.5/2) + 50
= 230 PLF

SPAN = 18.25 ft PSL $3\frac{1}{2} \times 11\frac{7}{8}$

V = 2099 # f_v = 68 PSI

M = 9576 lb-ft f_b = 1397 PSI

Δ = 0.59" = L/373

SPANDREL BEAM @ STAIR

W = COMPONENTS & CLADDING WINDWARD/LEEWARD WIND
ASCE 7-10 30.5 – SIMPLIFIED

$p_{net} = \lambda K_{zt} p_{net30}$

$p_{net} = 1.35(1.23)(+18.5/-22.6)$
= +30.7, −37.5 PSF

W = −37.5(19/2) = 356 PLF

SPAN = 12.75 ft PSL $5\frac{1}{4} \times 11\frac{7}{8}$ RIM

V = −2270 # f_v = 55 PSI

M = 7224 lb-ft f_b = 1591 PSI

Δ = 0.74" = L/207
OK > L/180

DTT2Z OK

NOTE: EXP. C

λ = 1.35 @ Avg height = 25 ft

K_{zt} = 1.23 TRIB AREA = (19/2)(13.25/2) = 63 ft²

ZONE 5: CORNER

p_{net30} = +18.5/−22.6

NOTE: LEEWARD GOV'S

WEAK AXIS BENDING

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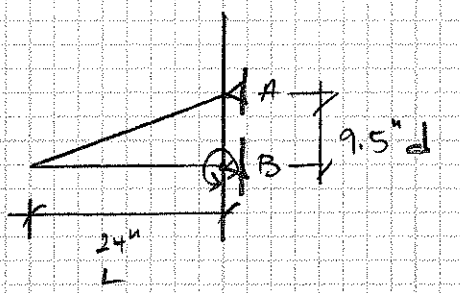
Roof Skirt Design @ RIM

40 PSF @ 24" = 80 PLF



$$M = \frac{wL^2}{2} = Td = Cd$$

$$\Sigma M_B = \frac{wL^2}{2} - R_{Ax}\,d$$

$$= \frac{80\left(\frac{24"}{12}\right)^2}{2} - R_{Ax}\left(\frac{9.5"}{12}\right)$$

R_{Ax} = 202 # TENSION

A34 w/ 1½" SD Screws @ EA.

(2) 16d @ EA, COX to RIM

LUS HNG @ SOFFIT

OR A35

OR SDS × 3½ @ 12" oc

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STRUCTURAL ENGINEERING

PROJECT Marc Haven PZ

DATE 9/19/19

PROJ. # VMR

DESIGN 23

SHEET