RB Engineers, Inc.

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EMAIL: rbe1992@gmail.com

JOB: IMANI
PROJECT #:21-8457
BY: R.B. / MJT
DATE: 05/18/2022

STRUCTURAL PLAN CHECK REPLY FOR

2405 74TH AVE SE,
MERCER ISLAND WA
PERMIT #2112-257

BASIS FOR DESIGN:

CODE: INTERNATIONAL BUILDING CODE (2018 EDITION)
WIND: 100 MPH, EXPOSURE "B" Kzt = 1.20
SEISMIC: Ss = 1.60, S1 = 0.57 (SITE CLASS D)
ROOF SNOW: 25 PSF

INDEX TO COMPUTATIONS:

RB ENGINEERS, INC. IS NOT RESPONSIBLE FOR THE SITE, SOILS, WEATHER PROOFING, TRUSSES AND/OR EXISTING CONDITIONS.

Jim Taherzadeh
STATE OF WASHINGTON
24140
REGISTERED
PROFESSIONAL ENGINEER

Expires Feb 2023

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Project: IMANI		By: RB/MJT	
Client: -		Date: 9/17/21	
Subject: Lateral		Page: G2/	

LOADING CRITERIA FOR ROOF AND/OR CEILING

- Main Roof Area
- Canopy or Mansard Roof
- Ceiling Only
- Other

Item	Material	Load PSF
Roofing	Composition	2.2
Sheathing or Decking	15/32 CDX	1.5
Insulation		2.8
Ceiling	5/8 GWB	2.6
Fixtures		1.0
Framing	Truss	2.3
Misc.		0.6
Sprinkler (Only If A>4000 sqft)		2

TOTAL DEAD LOAD = 13 PSF

LIVE LOADS

- Snow Load - 25 psf - non reducible
- Ceiling Only - 10 psf
- Increase in Fb and Fv of 15% allowed for duration of load

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LOADING CRITERIA FOR FLOOR

Item	Material	Load PSF
Floor Covering	Carpet and Pad	3.0
Floor Sheathing	3/4" T&G CDX	2.3
Ceiling	1/2" GWB	2.2
Fixtures		1.0
Framing	TJI's	3.0
Misc		1.5
Sprinkler (Only If A>4000 sqft)		2

TOTAL DEAD LOAD = 13 PSF

LIVE LOADS

- Residential - 40 psf (reducible)
- Office - 50 psf (reducible)
- Assembly - 100 psf (non-reducible)
- Corridors and Exits - 100 psf (reducible)
- Storage - 125 psf (non-reducible)

RB ENGINEE

RS, INC.

R1/

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Structural, Plan check reply

Date: 05/ 17 /2022

Project: IMANI RESIDENCE

2405 74th Ave SE, Mercer Island, WA

Permit #: 2112-257

To whom it may concern.

Structural plan-check Reply:

S1.

Item #1: See revised lateral design with updated rho value.

S2.

Item #1: Due to loading, beam needs to be 14” deep which is greater than 11-7/8” floor joists and will be upset into the wall above.

S3.

Item #1: see revised sheet S3.

SFP1.

Item #1: See detail I/S1& J/S1.

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R2/

Item #2: See revised foundation plan with top of wall elevation.

Item #3: Retaining walls on foundation per C/S1 detail and schedule.

Item #4: See provided detail E/S1 on foundation plan.

Item #5: LSL 3-1/2x9 header is added to the plan.

Item #6: See revised plan.

Item #7: Yes, retaining wall. See C/S1 called out.

Item #8: Additional (5)2x studs added @ point load above. Also see revised footings.

Item #9: By others.

Item #10: See revised plan.

SFP2.

Item #1: See revised plan.

Item #2: PSL 5-1/4x9 is specified. Strap on either side is okay.

Item #3: See clarified notes.

Item #4: See revised plan.

Item #5: Drag strut provided in two areas.

Item #6: See provided hanger notes.

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Item #7: See revised plan.

Item #8: See clarified and revised plan.

Item #9: See revised plan.

Item #10: See provided beam calculation.

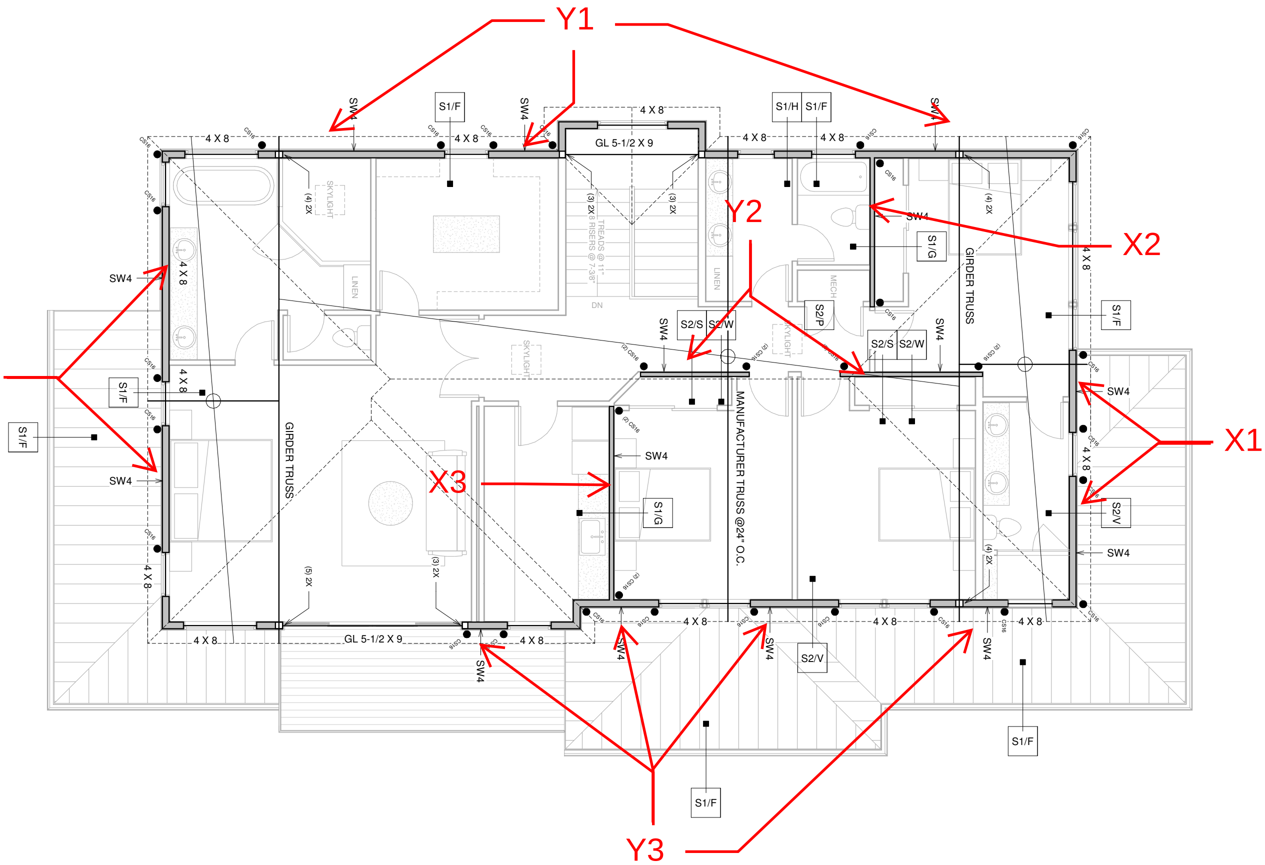
Thanks for your comments,

Sincerely yours,

Ross Baharmast

Jim Taherzadeh, PE

SHEARWALL KEY PLAN



MILTON LAM ARCHITECTS

ARCHITECT
MILTON LAM ARCHITECTS
PO BOX 523, KIRKLAND, WA 98083

Contact:
MILTON LAM 206-303-7877
MILTON@MLARC.COM

Client Name IMANI

Project Address 2405 74TH AVE SE MERCER ISLAND, WA 98040

No.	Description	Date

IMANI

MERCER ISLAND

ROOF FRAMING PLAN

Project number	2124
Date	12/27/21
Drawn by	RB
Checked by	MLA

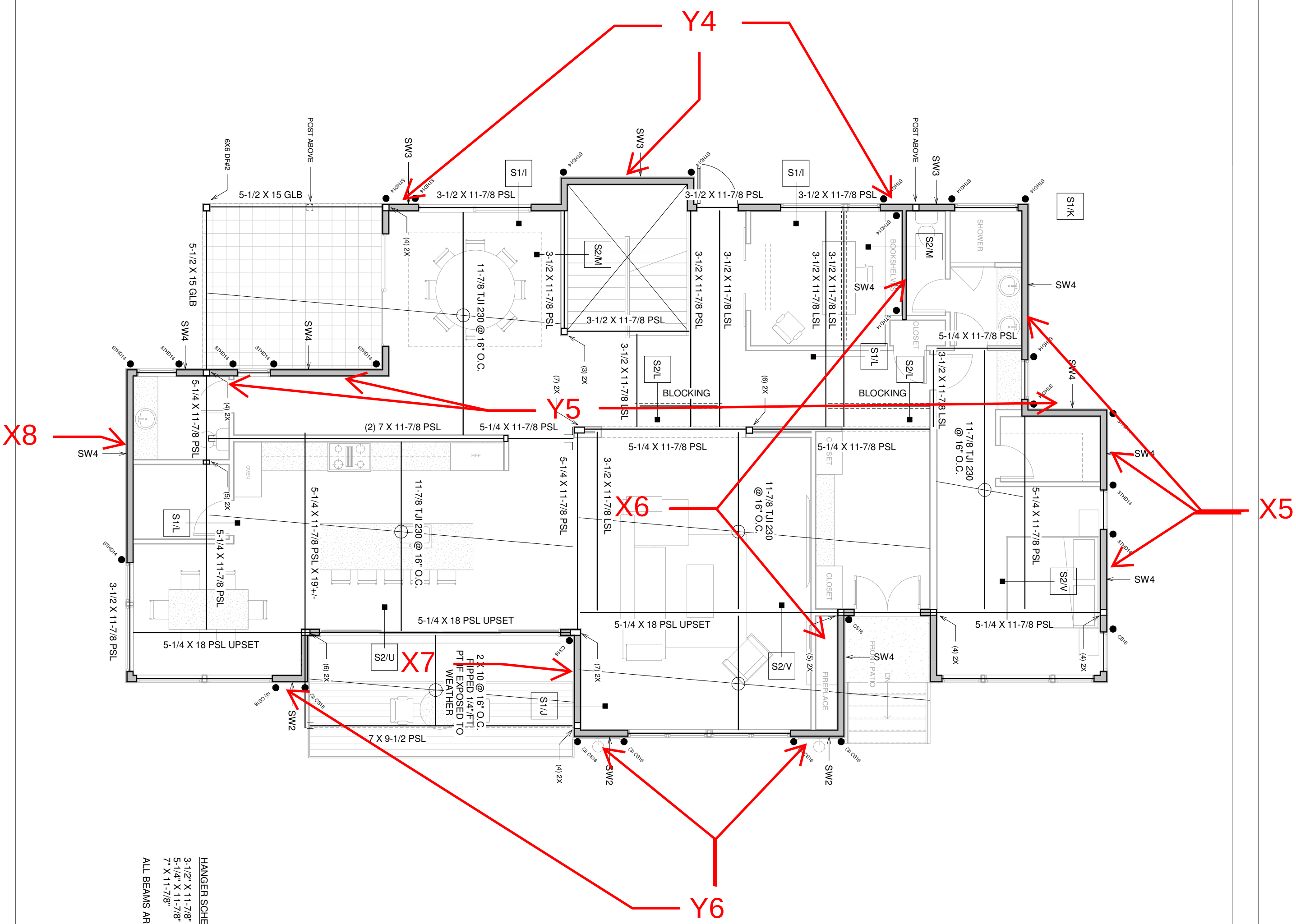
SFP3

Scale 1/4" = 1'-0"

12/27/2021 1:16:14 AM

L2/

SHEARWALL KEY PLAN



HANGER SCHEDULE
3-1/2" X 11-7/8" - HGUS412
5-1/4" X 11-7/8" - HGUS5.5/12
7" X 11-7/8" - HGUS7.25/12
ALL BEAMS ARE FLUSH U.N.O.

N

MILTON LAM ARCHITECTS	
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Contact:	MILTON LAM 206-303-7877, MILTON@MLARC.COM
Client Name	IMANI
Project Address	2405 74TH AVE SE, MERCER ISLAND, WA 98040
Project	IMANI, MERCER ISLAND
Sheet	UPPER FLOOR FRAMING PLAN
Project number	2124
Date	12/27/21
Drawn by	RB
Checked by	MLA
Scale	1/4" = 1'-0"
Sheet number	SFP2

No.	Description	Date

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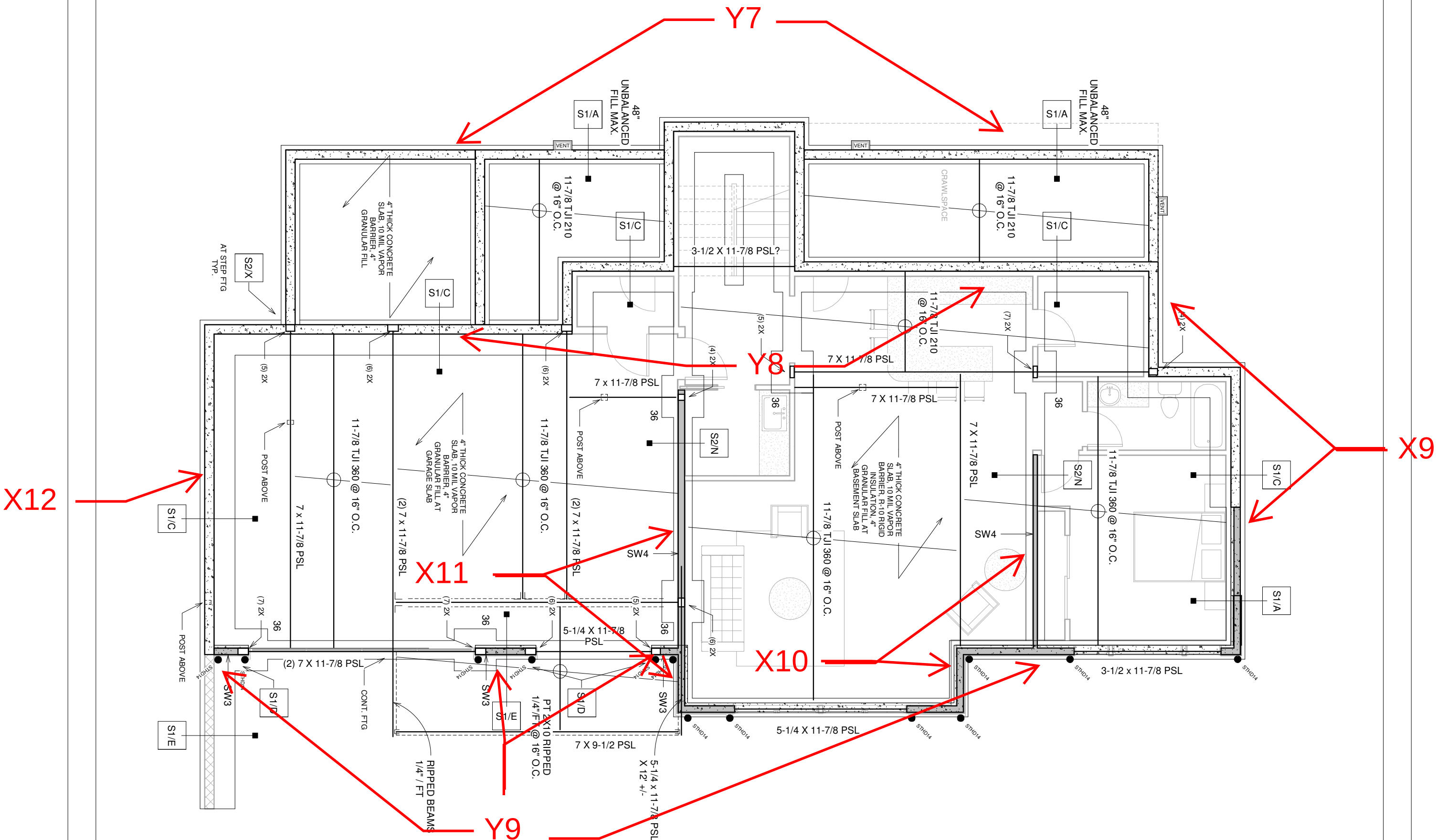
L3/

SHEARWALL KEY PLAN



36" X 36" X 8" W/ (5) #4 B.E.W.

30" X 30" X 8" W/ (4) #4 B.E.W.

24" X 24" X 8" W/ (3) #4 B.E.W.

HANGER SCHEDULE

3-1/2" X 11-7/8" - HGUS412
5-1/4" X 11-7/8" - HGUS5.5/12
7" X 11-7/8" - HGUS7.25/12

ALL BEAMS ARE FLUSH U.N.O.

MILTON LAM ARCHITECTS

ARCHITECT
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Client Name: IMANI

Project Address: 2405 74TH AVE SE, MERCER ISLAND, WA 98040

No.	Description	Date

IMANI

MERCER ISLAND

FOUNDATION PLAN

Project number: 2124
Date: 12/27/21
Drawn by: RB
Checked by: MLA

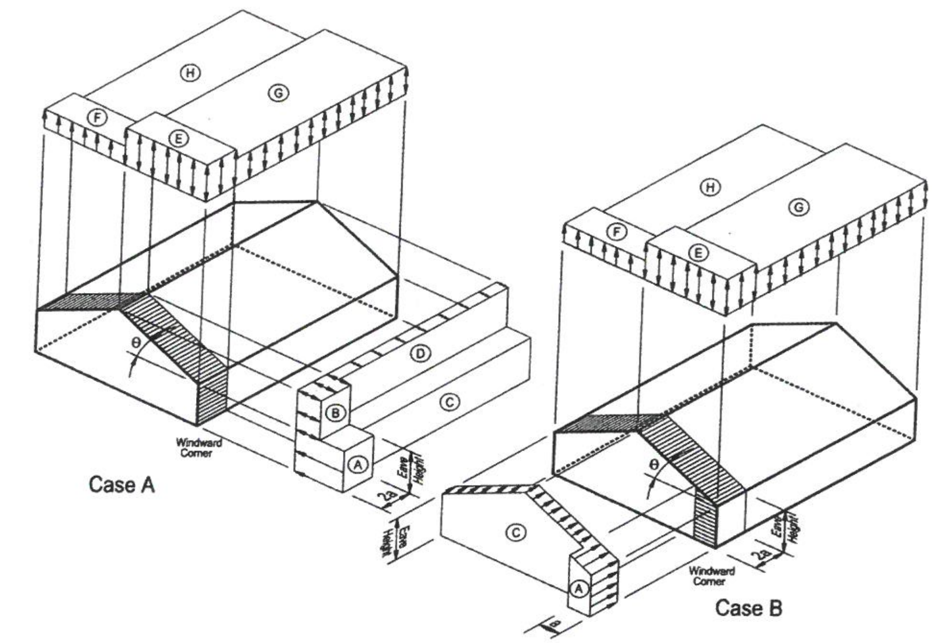
SFP1

Scale: 1/4" = 1'-0"

12/27/2021 1:16:13 AM

L4/

Diagrams



Notation

a 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).
EXCEPTION: For buildings with $\theta = 0$ to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h Mean roof height, in ft (m), except that eave height shall be used for roof angles < 10°.

θ Angle of plane of roof from horizontal, in degrees.

Notes

1. Pressures shown are applied to the horizontal and vertical projections, for Exposure B, at $h = 30$ ft ($h = 9.1$ m). Adjust to other exposures and heights with adjustment factor λ.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner (See Fig. 28.3-1).
3. For Case B, use $\theta = 0°$.
4. Load cases 1 and 2 must be checked for $25° < \theta \leq 45°$. Load case 2 at 25° is provided only for interpolation between 25° and 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_s = 0$ in Zones B and D.
8. Where Zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
9. Unit conversions for tables:

Adjustment Factor for Building Height and Exposure, λ

Mean roof height (ft)	Exposure B	Exposure C	Exposure D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Note: Unit conversions for tables: 1.0 ft = 0.3048 m; 1.0 lb/ft^2 = 0.0479 kN/m^2; 1 mph = 1.6 km/h

FIGURE 28.5-1 Main Wind Force Resisting System, Part 2 [$h \leq 60$ ft ($h \leq 18.3$ m)]: Design Wind Pressures for Enclosed Buildings—Walls and Roofs

continues

L5/

Simplified Design Wind Pressure, P_{s30} (psf) for Exposure B at h = 30 ft (h = 9.1 m)

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
85	0 to 5°	1	11.5	−5.9	7.6	−3.5	−13.8	−7.8	−9.6	−6.1	−19.3	−15.1
	10°	1	12.9	−5.4	8.6	−3.1	−13.8	−8.4	−9.6	−6.5	−19.3	−15.1
	15°	1	14.4	−4.8	9.6	−2.7	−13.8	−9.0	−9.6	−6.9	−19.3	−15.1
	20°	1	15.9	−4.2	10.6	−2.3	−13.8	−9.6	−9.6	−7.3	−19.3	−15.1
	25°	1	14.4	2.3	10.4	2.4	−6.4	−8.7	−4.6	−7.0	−11.9	−10.1
		2	—	—	—	—	−2.4	−4.7	−0.7	−3.0	—	—
	30 to 45	1	12.9	8.8	10.2	7.0	1.0	−7.8	0.3	−6.7	−4.5	−5.2
		2	12.9	8.8	10.2	7.0	5.0	−3.9	4.3	−2.8	−4.5	−5.2
90	0 to 5°	1	12.8	−6.7	8.5	−4.0	−15.4	−8.8	−10.7	−6.8	−21.6	−16.9
	10°	1	14.5	−6.0	9.6	−3.5	−15.4	−9.4	−10.7	−7.2	−21.6	−16.9
	15°	1	16.1	−5.4	10.7	−3.0	−15.4	−10.1	−10.7	−7.7	−21.6	−16.9
	20°	1	17.8	−4.7	11.9	−2.6	−15.4	−10.7	−10.7	−8.1	−21.6	−16.9
	25°	1	16.1	2.6	11.7	2.7	−7.2	−9.8	−5.2	−7.8	−13.3	−11.4
		2	—	—	—	—	−2.7	−5.3	−0.7	−3.4	—	—
	30 to 45	1	14.4	9.9	11.5	7.9	1.1	−8.8	0.4	−7.5	−5.1	−5.8
		2	14.4	9.9	11.5	7.9	5.6	−4.3	4.8	−3.1	−5.1	−5.8
95	0 to 5°	1	14.3	−7.4	9.5	−4.4	−17.2	−9.8	−12.0	−7.6	−24.1	−18.8
	10°	1	16.1	−6.7	10.7	−3.9	−17.2	−10.5	−12.0	−8.1	−24.1	−18.8
	15°	1	18.0	−6.0	12.0	−3.4	−17.2	−11.2	−12.0	−8.6	−24.1	−18.8
	20°	1	19.8	−5.2	13.2	−2.9	−17.2	−12.0	−12.0	−9.1	−24.1	−18.8
	25°	1	18.0	2.9	13.0	3.0	−8.0	−10.9	−5.8	−8.7	−14.9	−12.7
		2	—	—	—	—	−3.0	−5.9	−0.8	−3.8	—	—
	30 to 45	1	16.1	11.0	12.8	8.8	1.2	−9.8	0.4	−8.4	−5.6	−6.5
		2	16.1	11.0	12.8	8.8	6.2	−4.8	5.4	−3.4	−5.6	−6.5
100	0 to 5°	1	15.9	−8.2	10.5	−4.9	−19.1	−10.8	−13.3	−8.4	−26.7	−20.9
	10°	1	17.9	−7.4	11.9	−4.3	−19.1	−11.6	−13.3	−8.9	−26.7	−20.9
	15°	1	19.9	−6.6	13.3	−3.8	−19.1	−12.4	−13.3	−9.5	−26.7	−20.9
	20°	1	22.0	−5.8	14.6	−3.2	−19.1	−13.3	−13.3	−10.1	−26.7	−20.9
	25°	1	19.9	3.2	14.4	3.3	−8.8	−12.0	−6.4	−9.7	−16.5	−14.0
		2	—	—	—	—	−3.4	−6.6	−0.9	−4.2	—	—
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	−10.8	0.5	−9.3	−6.3	−7.2
		2	17.8	12.2	14.2	9.8	6.9	−5.3	5.9	−3.8	−6.3	−7.2

FIGURE 28.5-1 (*Continued*). Main Wind Force Resisting System, Part 2 [$h \leq 60$ ft ($h \leq 18.3$ m)]: Design Wind Pressures for Enclosed Buildings—Walls and Roofs

continues

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Client:	-	Date:	9/17/21
Subject:	Lateral	Page:	L6/

LATERAL WIND FORCES
ENVELOPE PROCEDURE (ASCE 7-16 Chapter 28)

Design Wind Pressures

Roof Pitch:	3:17 (10°)		Wind Speed:	100 mph		
Wind Exposure:	B λ = 1.0	ASCE 7-16 p.316	A:	17.9	C:	11.9
Minimum Pressure:	16	psf (wall) 28.5.4	B:	0.0	D:	0.0
Minimum Pressure:	8	psf (roof) 28.5.4				
Kzt:	1.2					

(ASCE 7-16) Using Allowable Stress Design, 2.4.5 Basic Combinations option 7: 0.6 D + 0.6 W

X – X Direction

Σ Fw Roof = 17.9x(2x4.5x12.5) + 11.9x(4.5x37.5) /1000 =

4.02 4.02 x 1.20 Kzt x 0.6 = 2.90 kip

Σ Fw Upper = 17.9x(2x(4.5x12.5+5.5x15)) + 11.9x(4.5x37.5+5.5x45) /1000 =

9.94 9.94 x 1.20 Kzt x 0.6 = 7.16 kip

Σ Fw Lower = 17.9x(2x10x15) + 11.9x(10x45) /1000 =

10.73 10.73 x 1.20 Kzt x 0.6 = 7.72 kip

Roof Min = [(112.5+168.75)x16+(140)x8)]x1.20x0.6 = $\boxed{4.05}$ kip

Upper Min = [(278+417)x16+(0+0)x8)]x1.20x0.6 = $\boxed{8.01}$ kip

Main Min = [(300+450)x16+(0+0)x8)]x1.20x0.6 = $\boxed{8.64}$ kip

Y – Y Direction

Σ Fw Roof = 17.9x(2x4.5x7) + 11.9x(4.5x21) /1000 =

2.25 2.25 x 1.20 Kzt x 0.6 = 1.62 kip

Σ Fw Upper = 17.9x(2x(4.5x7+5.5x8.5)) + 11.9x(4.5x21+5.5x25.5) /1000 =

	5.60	5.60 x 1.20 Kzt x 0.6 =	4.03	kip
Σ Fw Lower =	17.9x(2x6x8.5) + 11.9x(6x25.5) /1000 =			
	3.65	3.65 x 1.20 Kzt x 0.6 =	2.63	kip
Roof Min =	[(63+94.5)x16+(73)x8)]x1.20x0.6 =		$\boxed{2.23}$	kip
Upper Min =	[(156.5+235)x16+(0+0)x8)]x1.20x0.6 =		$\boxed{4.51}$	kip
Main Min =	[(102+153)x16+(0+0)x8)]x1.20x0.6 =		$\boxed{2.94}$	kip

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Subject: Lateral	Page: L8/

QUAKE FORCES (ASCE 7-16)

Site Class "D" (Table 11.4.2)

Ss = Critical Values per Latest SEAOC website; 1.6

S1 = Critical Values per Latest SEAOC website; 0.57

Fa =per Table 11.4-1	1	
Fv = per Table 11.4-2	1.8	
Sms = Fa * Ss = 1.2 (1.6)	1.6	(11.4-1)
Sm1 = Fv * S1 = 1.8 (0.57)	1.03	(11.4-2)
Sds = 2/3 * Sms = 2/3 (1.92)	1.07	(11.4-3), Seismic Design Category "D", Table 11.6-1
Sd1 = 2/3 * Sm1 = 2/3 (1.03)	0.68	(11.4-4); Seismic Design Category D, Table 11.6-2

SEISMIC RESPONSE COEFFICIENT: Use Section (12.8.1.1) ASCE 7-16 Except as Noted

To = 0.2 (Sd1/Sds) =0.2(0.68/1.28)	0.13	Section 11.4.6
Ts = Sd1 /Sds =(0.68/1.28)	0.64	Section 11.4.6
Ct	0.02	
Hn (Bldg. height-ft))	35.00	
Tstruc = Ct * (Hn)^3/4 = 0.02(35^.75)	0.29	Table 12.8-2
Where To ≤ Tstruc ≤ Ts; Sa = Sds	1.066667	Section 11.4.6, item 2
R	6.5	ASCE 7-16 Table 12.2-1, item 15
Ie	1	Section 11.5.1, Table 1.5-1 and 1.5-2)
Cs = Sds /(R/I) = 1.28 / (6.5/1) =	0.164	(12.8-2)

$\boxed{Cs = 0.16}$

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Subject:	Lateral	Page:	L9/

LATERAL QUAKE FORCES

Cs:	0.16		ρ:	1.3	
Roof Area:	2280	$feet^2$	Roof Dead Load:	13	psf
Upper Area:	3330	$feet^2$	Floor Dead Load:	13	psf
Main Area:	2115	$feet^2$	Wall Dead Load:	8	psf

ASCE 7-16 Using Allowable Stress Design, 2.4.5 Basic Combination option 10: $0.6\ D + 0.7\ E_h - 0.7E_v$

ASCE 7-16 Table 12.2-1: Overstrength factor, Ω = 2.5

DEAD LOAD OF STRUCTURE

Roof Weight =	2280 x (13+8) =	=	47.88	kip
Upper Weight =	3330 x (13+8) =	=	69.93	kip
Main Weight =	2115 x (13+8) =	=	44.42	kip
			162.23	kip

V_{base} = Cs x Σ Weight = 0.16 x 162.23 = 25.96 kip

V_{asd} = V_{base} x 0.7 x ρ = $\boxed{23.62}$ kip

LATERAL FORCES

	WEIGHT		HEIGHT		WEIGHT x HEIGHT
Roof:	47.88	kip	29	Feet	1388.52
Upper:	69.93	kip	20	Feet	1398.60
Main:	44.42	kip	9	Feet	399.74
					3186.86

Fq Roof:	23.62 x (1388.52/3186.86) =	=	10.29	kip
Fq Upper:	23.62 x (1398.60/3186.86) =	=	10.37	kip
Fq Main:	23.62 x (399.74/3186.86) =	=	2.96	kip

<u>Therefore quake governs all.</u>

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Client:		Date:	9/17/2021
Subject:	Lateral	Page:	L10/

WIND FORCES ON SHEAR WALLS

Shearwalls in X – X Direction

Fw X – X @ Roof: 4.05 kips — 4.05k / 62.5 ft = 64.7 #/Ft

Location	Calculation	Force / Length		Result	Units
V @ X 1 =	(4.05/62.5)x(14/2) =	0.45 / 9'+5.5'	=	31	# / Ft
SW-4					
V @ X 2 =	(4.05/62.5)x(14/2+17.5/2) =	1.02 / 9.5'	=	107	# / Ft
SW-4					
V @ X 3 =	(4.05/62.5)x(17.5/2+31/2) =	1.57 / 13.5'	=	116	# / Ft
SW-4					
V @ X 4 =	(4.05/62.5)x(31/2) =	1.00 / 8.5'+12'	=	49	# / Ft
SW-4					

Fw X – X @ Upper: 8.01 kips — 8.01 k / 74 ft = 108.2 #/Ft

Location	Calculation	Force / Length		Result	Units
V @ X 5 =	(8.01/74)x(6+10/2)	1.64 / 11.75'+6'+8'	=	64	# / Ft
SW-4	+ 0.45 =				
V @ X 6 =	(8.01/74)x(10/2+4+20/2)	3.31 / 9'+8'	=	195	# / Ft
SW-4	+ 1.02 +1.57x(3/20)=				
V @ X 7 =	(8.01/74)x(20/2+34/2)	4.26 / 8'	=	532	# / Ft
(2)SW-3	+ 1.57x(17/20) =				
V @ X 8 =	(8.01/74)x(34/2)	3.41 / 14.5'	=	235	# / Ft
SW-3	+ 1.00 =				

Fw X – X @ Main: 8.64 kips — 8.64 k / 74 ft = 116.8 #/Ft

Location	Calculation	Force / Length		Result	Units
V @ X 9 =	(8.64/74)x(6+8.5/2)	2.84 / 28'	=	101	# / Ft
SW-4	+ 1.64 =				

V @ X 10 SW-3	(8.64/74)x(8.5/2+5+20/2) + 3.31 =	$\frac{5.56}{14'+4'}$	=	309	$\frac{\#}{Ft}$
V @ X 11 SW-4	(8.64/74)x(20/2+34.5/2) + 4.26 =	$\frac{7.44}{19'+4'}$	=	323	$\frac{\#}{Ft}$
V @ X 12 SW-4	(8.64/74)x(34.5/2) + 3.41 =	$\frac{5.42}{(23.5')(12''x8'')}$	=	2	psi

RB Engineers, Inc.
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WIND FORCES ON SHEAR WALLS

Shearwalls in Y – Y Direction

Fw Y – Y @ Roof: 2.23 kips 6.56 k / 40.5 ft = 55.2 #/Ft

V @ Y 1 = (2.23/34.5)x(2+15/2) = 0.62 / 12.5'+4.75'+15' = 19 #/Ft
SW-4

V @ Y 2 = (2.23/34.5)x(15/2+16/2) = 1.00 / 8.25'+6.25' = 69 #/Ft
SW-4

V @ Y 3 = (2.23/34.5)x(16/2+1.5) = 0.62 / 10'+6'+5.5'+3.25' = 29 #/Ft
SW-4

Fw Y – Y @ Upper: 4.51 kips 4.51 k / 42 ft = 107.4 #/Ft

V @ Y 4 = (4.51/42)x(2+12.5/2) + 0.62 = 1.50 / 5.5'+10'+2.75' = 82 #/Ft
SW-3

V @ Y 5 = (4.51/42)x(12.5/2+3+20/2) +1.00 = 3.07 / 6'+9'+5' = 146 #/Ft
SW-3

V @ Y 6 = (4.51/42)x(20/2+4) +0.62 = 2.12 / 2.75'+4'+4' = 197 #/Ft
(2)SW-3

Fw Y – Y @ Main: 2.94 kips 2.94 k / 40 ft = 73.4 #/Ft

V @ Y 7 = (2.94/40)x(12/2) + 1.50 = 1.94 / (25'+26')(12"/1'x8") = 0 psi
CONC
Concrete wall treated as regular shear wall for conservative purposes

V @ Y 8 = (2.94/40)x(12/2+3+20.5/2) + 3.07 = 4.48 / (25'+6')(12"/1'x8") = 2 psi
CONC

V @ Y 9 = (2.94/40)x(20.5/2+4) + 2.12 = 3.12 < 4.6k LRP capacity.
SW-3
(2)(24"x8')+(18"x8') LRP : 2 x 1675lb + 1250lb = 4.6k LRP capacity.

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Project: Imani	By: RB/MJT
Client:	Date: 9/17/2021
Subject: Lateral	Page: L13/

QUAKE FORCES ON SHEAR WALLS

Shearwalls in X – X Direction

Fw X – X @ Roof: 10.29 kips — 10.29 k / 62.5 ft = 164.7 #/Ft

V @ X 1 = (10.29/62.5)x(14/2) = SW-4	1.15 / 9'+5.5'	=	79	# / Ft
V @ X 2 = (10.29/62.5)x(14/2+17.5/2) = SW-4	2.59 / 9.5'	=	273	# / Ft
V @ X 3 = (10.29/62.5)x(17.5/2+31/2) = SW-4	3.99 / 13.5'	=	296	# / Ft
V @ X 4 = (10.29/62.5)x(31/2) = SW-4	2.55 / 8.5'+12'	=	124	# / Ft

Fw X – X @ Upper: 10.37 kips — 10.37 k / 74 ft = 140.1 #/Ft

V @ X 5 = (10.37/74)x(6+10/2) SW-4 + 1.15 =	2.69 / 11.75'+6'+8'	=	105	# / Ft
V @ X 6 = (10.37/74)x(10/2+4+20/2) SW-4 + 2.59 +3.99x(3/20)=	5.85 / 9'+8'	=	344	# / Ft
V @ X 7 = (10.37/74)x(20/2+34/2) (2)SW-3 + 3.99x(17/20) =	7.18 / 8'	=	897	# / Ft
V @ X 8 = (10.37/74)x(34/2) SW-3 + 2.55 =	6.37 / 14.5'	=	440	# / Ft

Fw X – X @ Main: 2.96 kips — 2.96 k / 74 ft = 40.0 #/Ft

V @ X 9 = (2.94/74)x(6+8.5/2) SW-4 + 2.69 =	3.10 / 28'	=	111	# / Ft

V @ X 10	(2.94/74)x(8.5/2+5+20/2)	$\frac{6.62}{14'+4'}$	=	368	$\frac{\#}{Ft}$
$\boxed{\text{SW-3}}$	+ 5.85 =				
V @ X 11	(2.94/74)x(20/2+34.5/2)	$\frac{8.27}{19'+4'}$	=	359	$\frac{\#}{Ft}$
$\boxed{\text{SW-4}}$	+ 7.18 =				
V @ X 12	(2.94/74)x(34.5/2)	$\frac{7.07}{(23.5')(12''x8'')}$	=	3	psi
$\boxed{\text{SW-4}}$	+ 6.37 =				

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QUAKE FORCES ON SHEAR WALLS

Shearwalls in Y – Y Direction

Fw Y – Y @ Roof: 10.29 kips 10.29 k / 34.5 ft = 298.3 #/Ft

V @ Y 1 = (10.29/34.5)x(2+15/2) = 2.83 / 12.5'+4.75'+15' = 88 #/Ft
SW-4

V @ Y 2 = (10.29/34.5)x(15/2+16/2) = 4.62 / 8.25'+6.25' = 319 #/Ft
SW-4

V @ Y 3 = (10.29/34.5)x(16/2+1.5) = 2.83 / 10'+6'+5.5'+3.25' = 132 #/Ft
SW-4
(1.25-0.125x(9/3.25))x350 #/Ft =305 #/Ft capacity per table 4.3.4 ANSI/AF+PA SDPWS

Fw Y – Y @ Upper: 10.37 kips 10.37 k / 42 ft = 259.1 #/Ft

V @ Y 4 = (10.37/42)x(2+12.5/2) + 2.83 = 4.87 / 5.5'+10'+2.75' = 267 #/Ft
SW-3
(1.25-0.125x(10/3))x450 #/Ft =375 #/Ft capacity per table 4.3.4 ANSI/AF+PA SDPWS

V @ Y 5 = (10.37/42)x(12.5/2+3+20/2) +4.62= 9.37 / 6'+9'+5' = 446 #/Ft
SW-3

V @ Y 6 = (10.37/42)x(20/2+4) +2.83= 6.29 / 2.75'+4'+4' = 585 #/Ft
(2)SW-3
Restrain Top& Bottom of Window Openings to reduce H:W ratio.

Fw Y – Y @ Main: 2.96 kips 2.96 k / 40 ft = 74.1 #/Ft

V @ Y 7 = (2.96/40)x(12/2) + 4.87 = 5.31 / (25'+26')(12"/1'x8") = 1 psi
CONC
Concrete wall treated as regular shear wall for conservative purposes

V @ Y 8 = (2.96/40)x(12/2+3+20.5/2) + 9.37 = 10.80 / (25'+6')(12"/1'x8") = 4 psi
CONC

V @ Y 9 = (2.96/40)x(20.5/2+4) + 6.29 -4.6k = 2.69 / (7.5'+4'+4')(12"/1'x8') = 2 #/Ft
CONC
(2)(24"x8')+(18"x8') LRP : 2 x 1675lb + 1250lb = 4.6k LRP capacity.

RB Engineers, Inc.

Project:	Imani	By:	RB/MJT
Client:		Date:	9/17/2021
Subject:	Lateral	Page:	L16/

CHECK OVERTURNING FOR: X 3 (Quake)

L =	13.5	ft
P =	296	lb/ft
P x L =	296x13.5	4.00 kip

MoT = 4.00x9 35.96 kip - ft

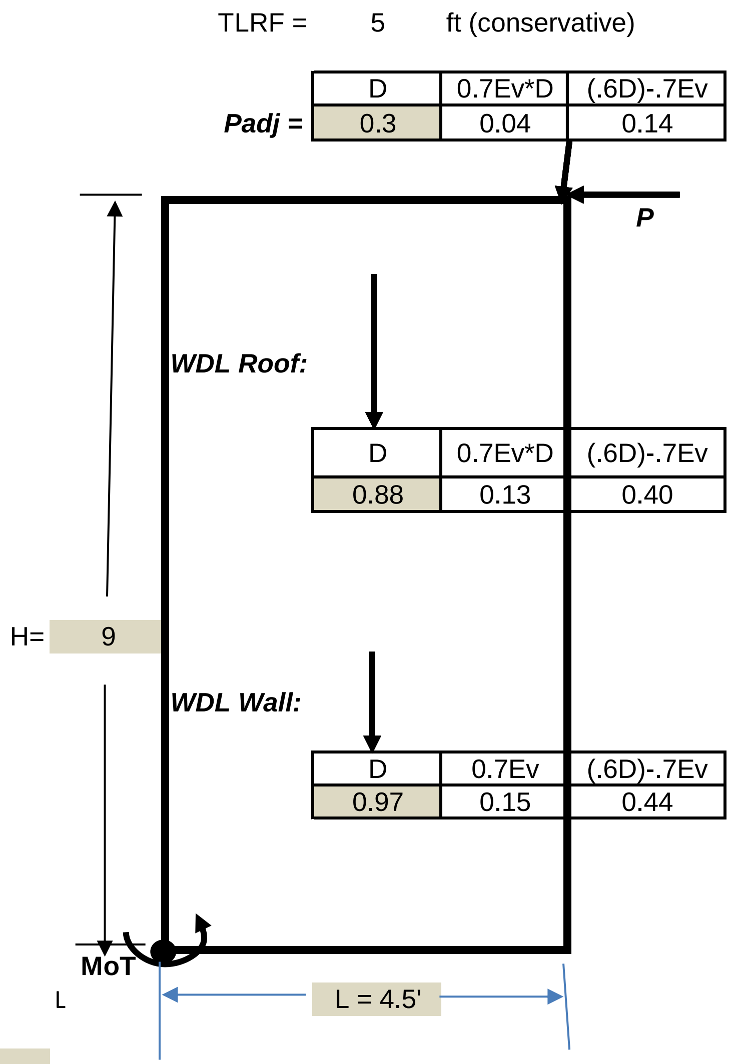
S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15

MR = [(0.40+0.44)x0.5x13.5+0.14x13.5] = 7.45 kip - ft

F = MoT - MR / L = **2.11** kip

Positive # : Hold down required
Negative # : No Hold down Required



Therefore use (1)CS16 hold downs at each end

RB Engineers, Inc.

Project: Imani	By: RB/MJT
Client:	Date: 9/17/2021
Subject: Lateral	Page: L17/

CHECK OVERTURNING FOR: X 7 (Quake)

L =	8	ft
P =	897	lb/ft
P x L =	897x8	7.18 kip

TLRF = 5 ft (conservative)

	D	0.7Ev*D	(.6D)-.7Ev
Padj =	0.6	0.09	0.27

MoT = 7.18x10 71.76 kip - ft

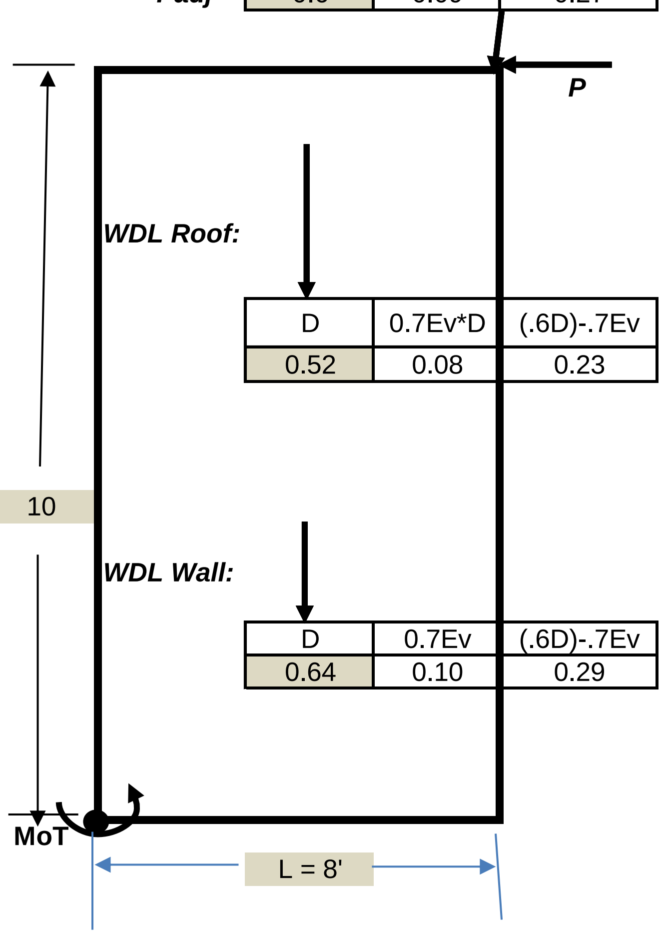
S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15

MR = [(0.23+0.29)x0.5x8+0.27x8] = 4.25 kip - ft

F = <u>MoT - MR</u> / L = **8.44** kip

Positive # : Hold down required
Negative # : No Hold down Required



WDL Roof:

D	0.7Ev*D	(.6D)-.7Ev
0.52	0.08	0.23

WDL Wall:

D	0.7Ev	(.6D)-.7Ev
0.64	0.10	0.29

<u>Therefore use (4)CS16 hold downs at each end Within.</u>

RB Engineers, Inc.

Project: Imani	By: RB/MJT
Client:	Date: 9/17/2021
Subject: Lateral	Page: L18/

CHECK OVERTURNING FOR: X 8 (Quake)

L =	14.5	ft
P =	440	lb/ft
P x L =	440x14.5	6.38 kip

TLRF = 5 ft (conservative)

MoT = 6.38x10 63.80 kip - ft

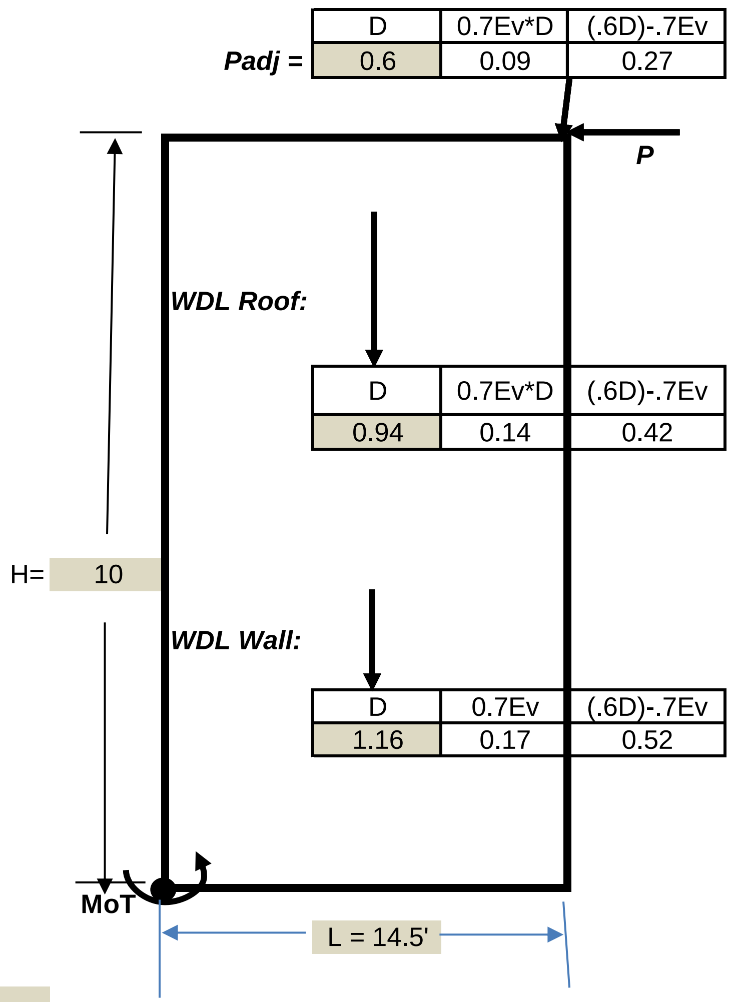
S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15

MR = [(0.42+0.52)x0.5x14.5+0.27x14.5] = 10.79 kip - ft

F = MoT - MR / L = **3.66** kip

Positive # : Hold down required
Negative # : No Hold down Required



Therefore use STHD14 hold downs at each end

CHECK OVERTURNING FOR: X 11 (Quake)

L =	4	ft
P =	359	lb/ft
P x L =	359x4	1.44 kip

TLRF = 5 ft (conservative)

Padj =

D	0.7Ev*D	(.6D)-.7Ev
0.9	0.13	0.41

MoT = 1.44x8.5 12.21 kip - ft

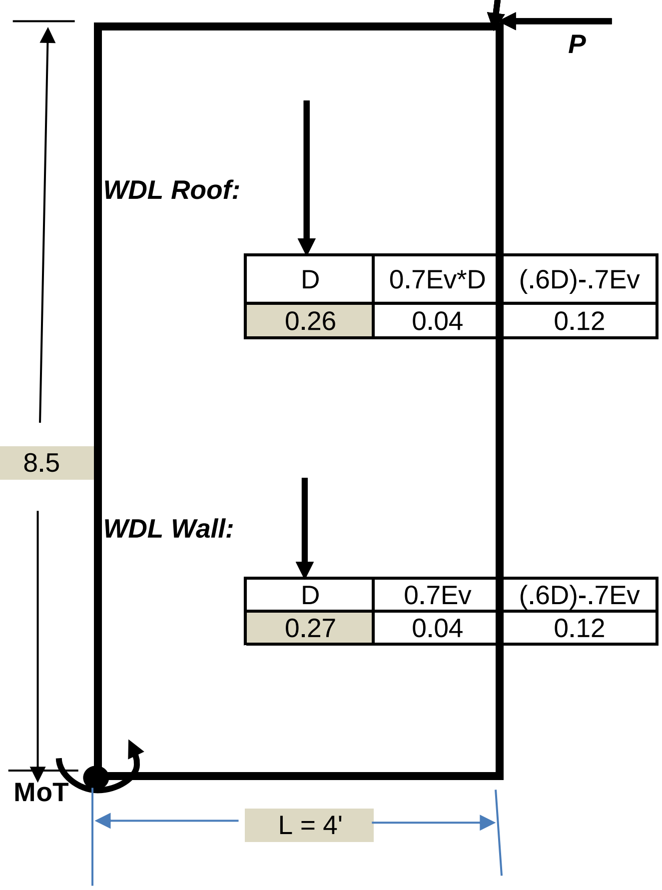
S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15

MR = [(0.12.+0.12)x0.5x4+0.41x4] = 2.10 kip - ft

F = MoT - MR / L = **2.53** kip

Positive # : Hold down required
Negative # : No Hold down Required



WDL Roof:

D	0.7Ev*D	(.6D)-.7Ev
0.26	0.04	0.12

WDL Wall:

D	0.7Ev	(.6D)-.7Ev
0.27	0.04	0.12

Therefore use STHD14 hold downs at each end

CHECK OVERTURNING FOR: Y2 (Quake)

L =	6.25	ft
P =	319	lb/ft
P x L =	319x6.25	1.99 kip

TLRF = 5 ft (conservative)

MoT = 1.99x9 17.94 kip - ft

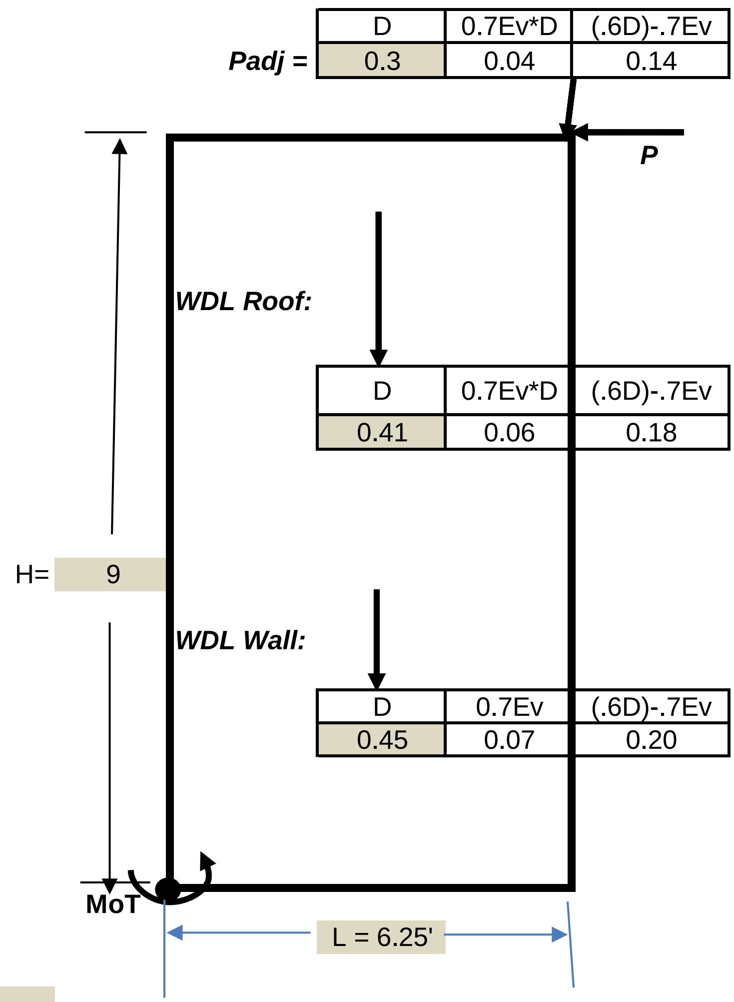
S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15

MR = [(0.18+0.20)x0.5x6.25+0.14x6.25] =
2.05 kip - ft

F = MoT - MR / L = **2.54** kip

Positive # : Hold down required
Negative # : No Hold down Required



Therefore use (2)CS16 hold downs at each end

RB Engineers, Inc.

Project: Imani	By: RB/MJT
Client:	Date: 9/17/2021
Subject: Lateral	Page: L21/21

CHECK OVERTURNING FOR: Y6 (Quake)

L =	2.75	ft
P =	585	lb/ft
P x L =	585x2.75	

1.61 kip

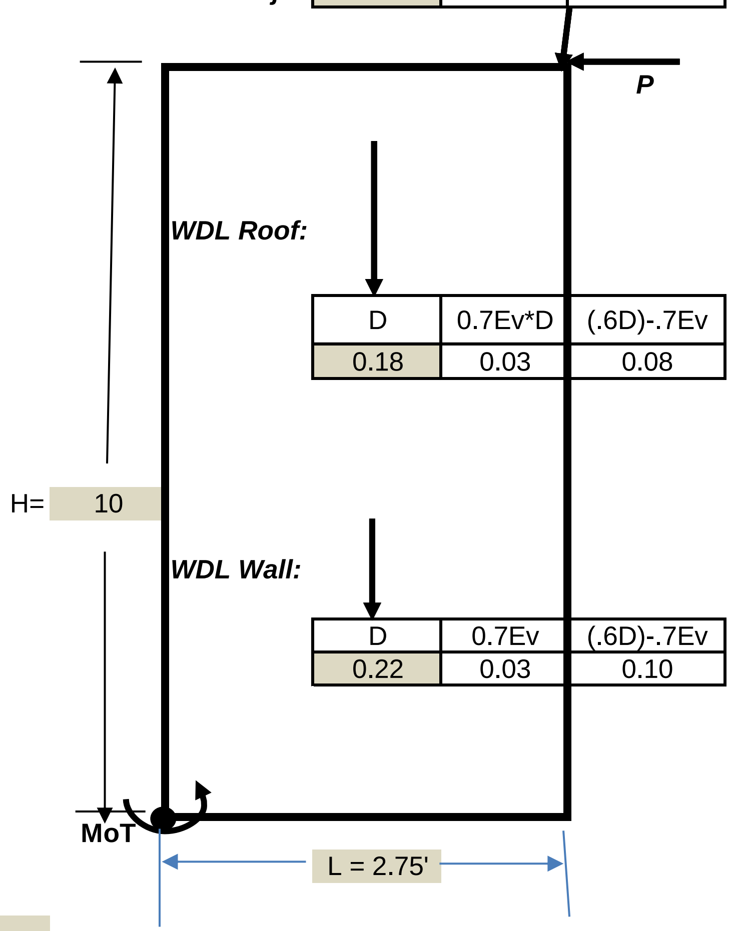
TLRF = 5 ft (conservative)

	D	0.7Ev*D	(.6D)-.7Ev
Padj =	0.6	0.09	0.27

MoT = 1.61x10 16.09 kip - ft

S_ds= 1.07 From Cs Calculations

.7(0.2*S_ds)= 0.15



WDL Roof:

D	0.7Ev*D	(.6D)-.7Ev
0.18	0.03	0.08

WDL Wall:

D	0.7Ev	(.6D)-.7Ev
0.22	0.03	0.10

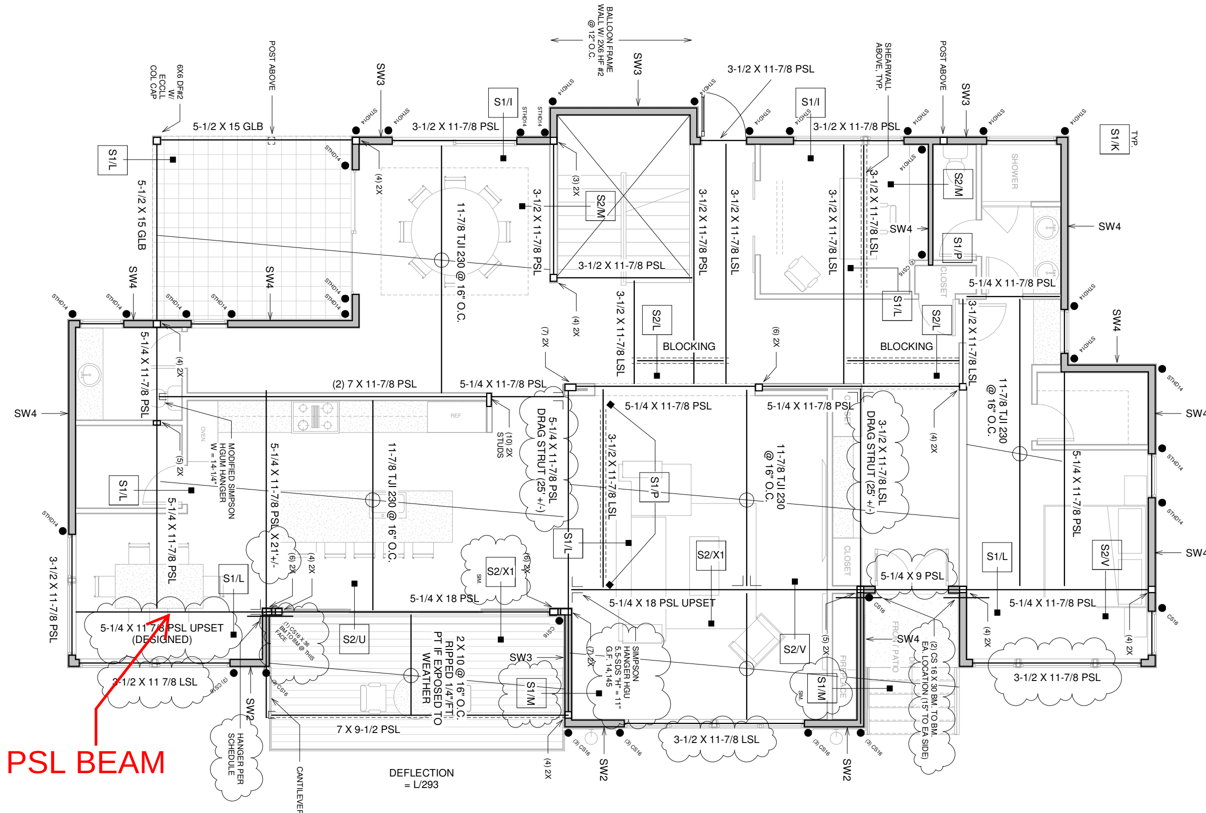
MR = [(0.08+0.10)x0.5x2.75+0.27x2.75] =
0.99 kip - ft

F = <u>MoT - MR</u> / L = **3.74** kip

Positive # : Hold down required
Negative # : No Hold down Required

<u>Therefore use (3)CS16 or STHD14 hold downs at each end</u>

BEAM KEY PLAN



HANGER SCHEDULE
3-1/2" X 11-7/8" - HGUS412
5-1/4" X 11-7/8" - HGUS5.5/12
7" X 11-7/8" - HGUS7.25/12
ALL BEAMS ARE FLUSH U.N.O.

MILTON LAM ARCHITECTS

ARCHITECT
MILTON LAM ARCHITECTS
PO BOX 523, KIRKLAND, WA 98083
Contact:
MILTON LAM 206-303-7877
MILTON@MLARC.COM

Client Name: IMANI

Project Address: 2405 74TH AVE SE, MERCER ISLAND, WA 98040

No.	Description	Date

IMANI
MERCER ISLAND

UPPER FLOOR FRAMING PLAN

Project number	2124
Date	5/16/22
Drawn by	RB
Checked by	MLA

SFP2

Scale 1/4" = 1'-0"

5/17/2022 7:40:33 AM

Wood Beam

Project File: Imani.ec6

LIC# : KW-06015928, Build:20.22.4.26 | RB Engineers, Inc. | (c) ENERCALC INC 1983-2022

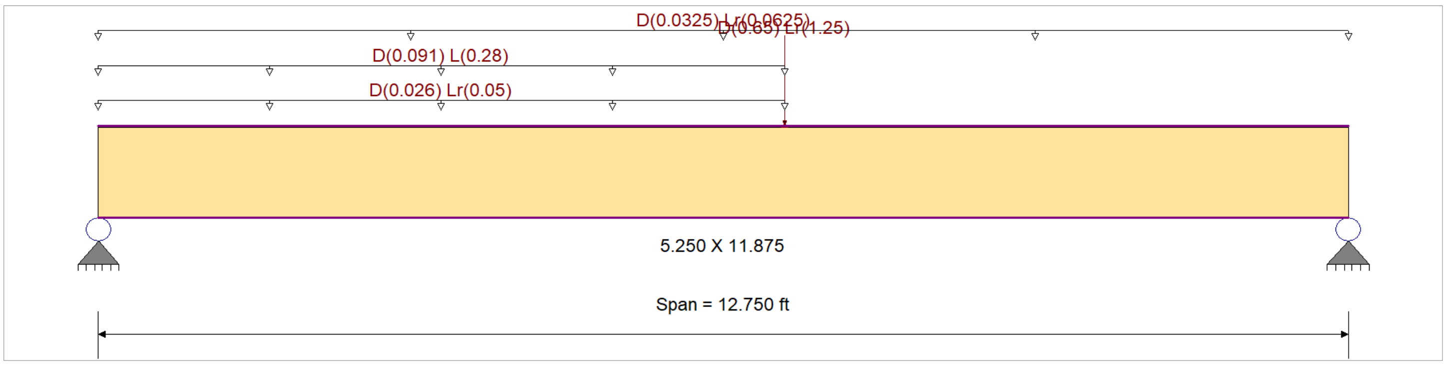
DESCRIPTION: PSL beam

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	750.0 psi		
Wood Grade :	Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.070 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Point Load : D = 0.650, Lr = 1.250 k @ 7.0 ft
Uniform Load : D = 0.0130, Lr = 0.0250 ksf, Extent = 0.0 -->> 7.0 ft, Tributary Width = 2.0 ft
Uniform Load : D = 0.0130, L = 0.040 ksf, Extent = 0.0 -->> 7.0 ft, Tributary Width = 7.0 ft
Uniform Load : D = 0.0130, Lr = 0.0250 ksf, Tributary Width = 2.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.285 : 1	Maximum Shear Stress Ratio	=	0.175 : 1
Section used for this span		**5.250 X 11.875**	Section used for this span		**5.250 X 11.875**
fb: Actual	=	1,034.55 psi	fv: Actual	=	63.50 psi
Fb: Allowable	=	3,625.00 psi	Fv: Allowable	=	362.50 psi
Load Combination		+D+0.750Lr+0.750L	Load Combination		+D+0.750Lr+0.750L
Location of maximum on span	=	6.933 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.100 in	Ratio =	1523 >=360	Span: 1 : Lr Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.199 in	Ratio =	770 >=180	Span: 1 : +D+0.750Lr+0.750L
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
D Only													0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.149	0.087	0.90	1.000	1.00	1.00	1.00	1.00	1.00	4.00	388.80	2610.00	0.95	22.82	261.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.247	0.174	1.00	1.000	1.00	1.00	1.00	1.00	1.00	7.37	716.96	2900.00	2.10	50.45	290.00
+D+Lr					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.261	0.136	1.25	1.000	1.00	1.00	1.00	1.00	1.00	9.75	947.78	3625.00	2.05	49.43	362.50
+D+0.750Lr+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.285	0.175	1.25	1.000	1.00	1.00	1.00	1.00	1.00	10.64	1,034.55	3625.00	2.64	63.50	362.50
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	1	0.189	0.131	1.15	1.000	1.00	1.00	1.00	1.00	1.00	6.49	631.65	3335.00	1.81	43.54	333.50

B3/4

Wood Beam

Project File: Imani.ec6

LIC# : KW-06015928, Build:20.22.4.26 RB Engineers, Inc.

DESCRIPTION: PSL beam

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.750 ft	**1**	0.050	0.030	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.40	233.28	4640.00	0.57	13.69	464.00

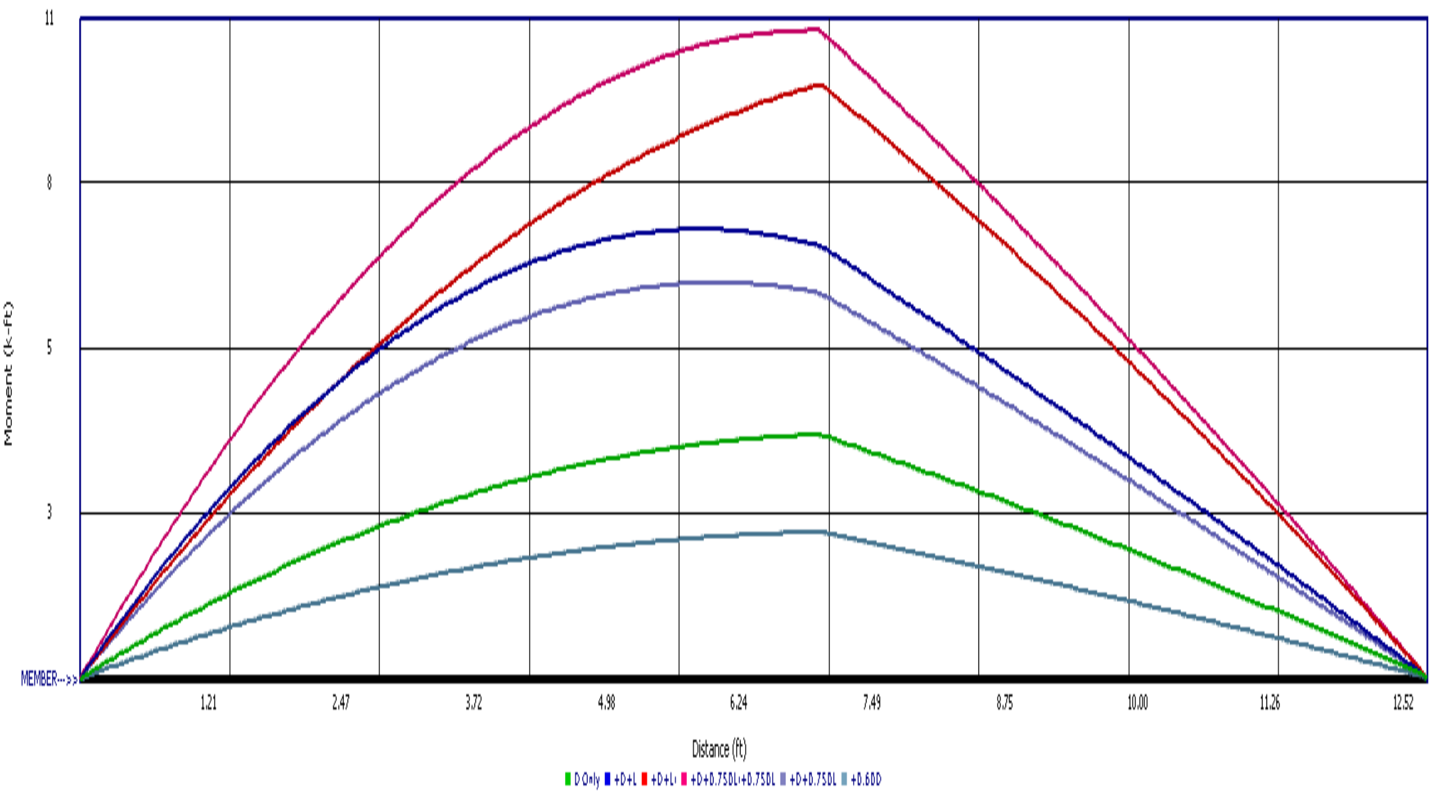
Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+0.750Lr+0.750L	1	0.1986	6.282		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.073	2.078
Overall MINimum	1.422	0.538
D Only	1.095	0.789
+D+L	2.516	1.327
+D+Lr	2.311	1.970
+D+0.750Lr+0.750L	3.073	2.078
+D+0.750L	2.161	1.192
+0.60D	0.657	0.473
Lr Only	1.216	1.181
L Only	1.422	0.538



Wood Beam

Project File: Imani.ec6

LIC# : KW-06015928, Build:20.22.4.26 RB Engineers, Inc. (c) ENERCALC INC 1983-2022

DESCRIPTION: PSL beam