STRUCTURAL CALCULATIONS

FOR

BODEGA COUNTRY STORE

17190 BODEGA HWY
BODEGA, CA

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* DO NOT REMOVE THEM *
11/23/2021
PERMIT AND RESOURCE MANAGEMENT DEPARTMENT
BUILDING PLAN CHECK
PERMIT # BLD21-8262

VERTEX®

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PROJECT NUMBER

74756

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STRUCTURE

DESIGN CRITERIA

THE DESIGN HEREIN IS BASED ON THE FOLLOWING STANDARDS:

MODEL BUILDING CODE	=	2018	IBC
STATE CODES	=	2019	CBC & CEBC

Design loads are based on the following:

ASCE/SEI 7-16: Minimum Design Loads for Buildings and Other Structures

California Existing Building Code

Design of the Vertical and Lateral Force Resisting Systems are based on the following:

Design of Steel
- Steel Construction Manual - 15th Edition
- AISC 360-16
- AISC 341-16
- AISC 358-16

Design of Masonry
- TMS 402-16

Design of Concrete
- ACI 318-14

Design of Wood
- NDS 2018

DETERMINATION OF VERTICAL LOADS

ROOF 1	DL					Composite Shingle	=	5.0	PSF
		30	#			Roofing Felt	=	0.3	PSF
		1	/	2	"	Plywood	=	1.5	PSF
						Man. Trusses @ 24" o.c.	=	5.0	PSF
						Insulation + MEP	=	1.5	PSF
						∑	=	15.8	PSF
						Roof Pitch	=	6/12	27 deg.
						Adj. DL	=	17.7	PSF

LLr		Standard Live Load	=	20	PSF
	ASCE 7-10 [4.8]	A_t	=	24	ft^2
		R_1	=	1.0	
		R_2	=	1.0	
		LLr	=	19	PSF

FLOORS	DL					Flooring		2.5	PSF
						LWC Topping			
		3	/	4	"	Plywood/Sheathing	=	2.3	PSF
		2	x	12	@	16 " O/C	=	2.7	PSF
						MEP	=	1.5	
						Gyp Board	=	2.5	PSF
						∑	=	11.5	PSF
	LL	[ASCE 7-10 Ch. 4]					=	40	PSF

WALLS	INTERIOR					Gyp Board	=	5.0	PSF
		2	x	4	@	16 " O/C	=	0.9	PSF
						MEP	=	1.5	PSF
						∑	=	7.5	PSF
	EXTERIOR								
						Shiplap Siding	=	4	PSF
		1	/	2	"	Plywood/Sheathing	=	1.5	PSF
		2	x	6	@	16 " O/C	=	1.3	PSF
						MEP	=	1.5	PSF
						Gyp Board	=	2.5	PSF
						∑	=	11.0	PSF

LATERAL LOADS: WIND

Analysis Type	=	Directional Procedure for Bldgs of All Heights

Basic Wind Speed	V	=	95	MPH	[Figure 26.5-1A, B, C]
Wind Directionality Factor	K_d	=	0.85	for MWFRS	[Table 26.6-1]
Exposure Category		=	C		
Enclosure Classification		=	Partially Enclosed		
Internal pressure coefficient	GC_{pi}	=	0.55	(+/-)	[Table 26.11-1]

Velocity Pressure Exposure Coefficients are determined as specified in Chapter 27.3:

[From ASCE 7-10, Table 26.9-1]		
Exposure	α	z_g (ft)
B	7.0	1,200
C	9.5	900
D	11.5	700

α = 9.5

z_g = 900 ft

$$K_z = 2.01\left(\frac{15}{z_g}\right)^{2/\alpha}$$

For $z < 15$ ft [Table 27.3-1]

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$$

For $15 \text{ ft} \le z < z_g$ [Table 27.3-1]

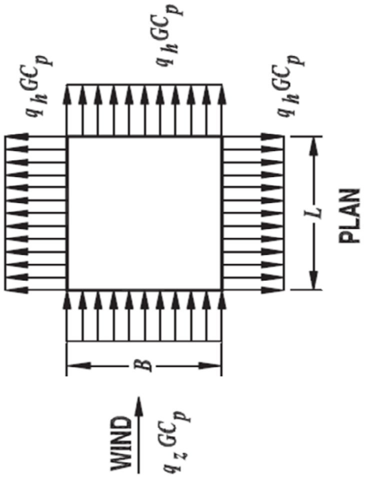
Topographic Factor	K_{zt}	=	1.00	[Section 26.8]
Ground Elevation Factor	K_e	=	1.00	[Section 26.9]

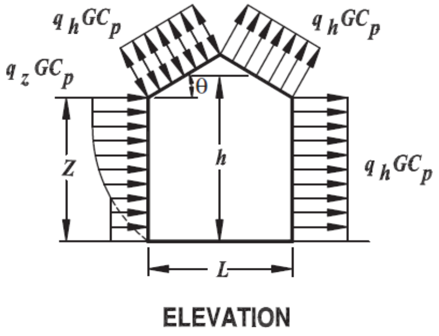
Velocity Pressure q_z = $0.00256K_zK_{zt}K_dK_eV^2$ (See table below)

Heigth	K_z	K_{zt}	K_d	V	K_e	q_z
ft				mph		lb/ft^2
0	0.85	1.00	0.85	95	1.00	16.7
15	0.85	1.00	0.85	95	1.00	16.7
20	0.90	1.00	0.85	95	1.00	17.7
25	0.95	1.00	0.85	95	1.00	18.6

Gust Effect Factor G = 0.85 [Chapter 26.9]

Wind Direction:	=	EAST - WEST

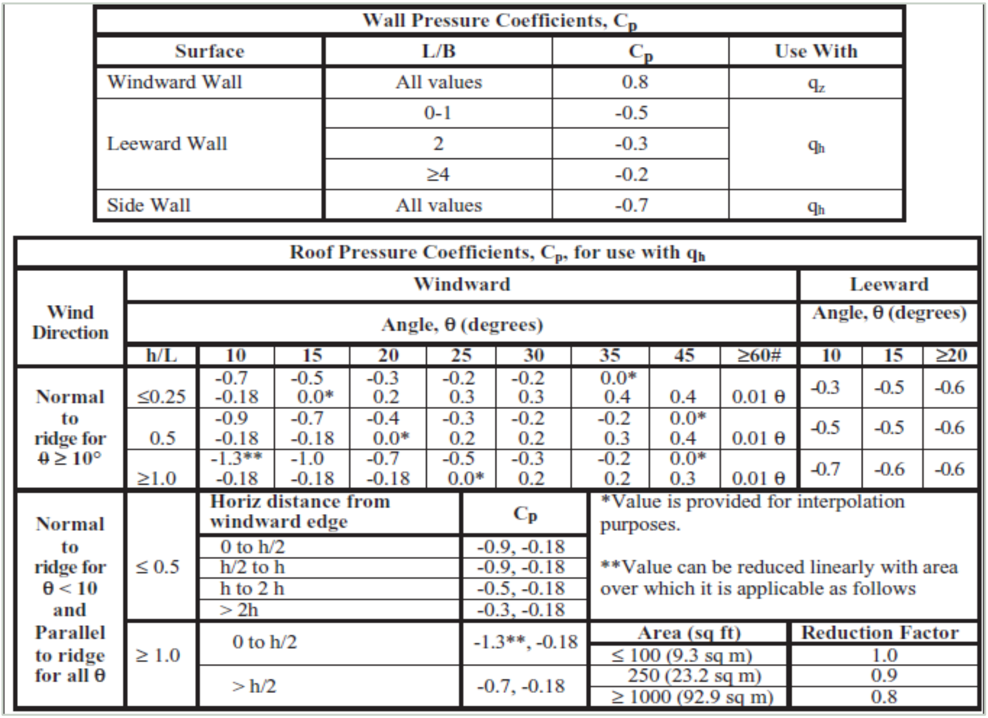




Building Geometry

B	=	58	ft
L	=	30	ft
h	=	25	ft

θ	=	23 deg.
L/B	=	0.5
h/L	=	0.8



Wall Pressure Coefficients, C_p			
Surface	**L/B**	**C_p**	**Use With**
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	**Windward**									**Leeward**		
	Angle, θ (degrees)									**Angle, θ (degrees)**		
	h/L	**10**	**15**	**20**	**25**	**30**	**35**	**45**	**≥60#**	**10**	**15**	**≥20**
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Normal to ridge for θ < 10 and Parallel to ridge for all θ		**Horiz distance from windward edge**	**Cp**
	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
250 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Windward Surfaces			
Walls	C_p	=	0.8
Roofs	C_p	=	-0.5
Roofs	C_p	=	0.1

Leeward Surfaces			
Walls	C_p	=	-0.5
Roofs	C_p	=	-0.6

Design Wind Loads for the MWFRS

Wind Direction:	=	EAST - WEST

Design wind pressures for the MWFRS are determined as shown below:

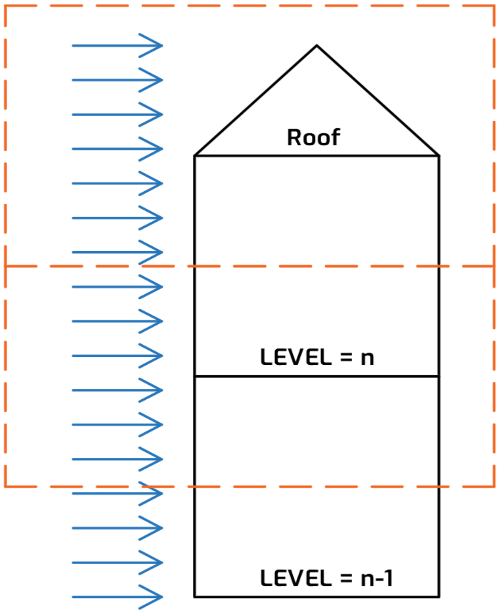
$$p = qGC_p - q_i(GC_{pi}) \qquad \text{[Equation 27.4-1]}$$

Wind Pressures							
		Windward				Leeward	
	h	q	G	C_p	p (psf)	C_p	p (psf)
Roofs	25	18.6	0.85	-0.50	-7.9	-0.6	-9.5
Roofs	25	18.6	0.85	0.1	1.6		
	z	q	G	C_p	p (psf)	C_p	p (psf)
Walls	20	17.7	0.85	0.8	12.0	-0.5	-7.5
Walls	20	17.7	0.85	0.8	12.0	-0.5	-7.5

Wind Loads							
Roofs		p x sin(θ)					
	θ	Windward	Leeward	Sum	p_{gov}	$A_{proj.}$	Load
	deg.	psf	psf	psf	psf	ft^2	kips
	27	0.7	4.2	4.9	**8.0**	174	1.39
Walls	Height	Windward	Leeward	Sum	p_{gov}	$A_{proj.}$	Load
	ft	psf	psf	psf	psf	ft^2	kips
Story 2	12	12.0	7.5	19.6	**19.6**	348	6.81
Story 1	11	12.0	7.5	19.6	**19.6**	667	13.05

Walls	Height	Windward	Parapet		p_{gov}	$A_{proj.}$	Load
	ft	psf	Coefficient	psf	psf	ft^2	kips
Parapets	5	12.0	2.25	27.1	**27.1**		0.00

Wind Direction:	=	EAST - WEST



Note: Graphic shown above is for illustration only, it does not necessarily depict the actual number of stories/levels in the structure.

Diaphragm		Load kips
Roof		1.39
Story 2		6.81
Story 1		13.05

Wind Base Shear	V_{wind_NS}	=	21.3	kips

Note: Applicable load factors will be applied later on during the design phase.

LATERAL LOADS: SEISMIC

Analysis Procedure	=	Equivalent Lateral Force

S_s	=	1.772
S_1	=	0.723
S_{DS}	=	1.418
S_{D1}	=	1.205

Risk Cat.	=	II	Risk Category
I_e	=	1.00	[Table 1.5-2]
Site Class	=	D	
SDC	=	D	

R	=	6.5	[Table 12.2-1]
Ω	=	2.5	[Table 12.2-1]
C_d	=	4	

T_L	=	8	sec	[Fig. 22-12]

Approximate Fundamental Period:

	HEIGHT
DIAPHRAGM	ft
STORY 2	13.0
STORY 1	11.0
Σ =	24.0

h_n = 24.0 ft

C_t = 0.02

x = 0.75

$T_a = C_t h_n^x$ [Eq. 12.8-7]

= 0.02(24^0.75)

= 0.217 sec

Seismic Response Coefficient:

$$C_s = \frac{S_{DS}}{(R/I_e)} = \frac{1.42}{(6.5/1)} = \boxed{0.218}$$ [Eq. 12.8-2]

$$C_{s,max} = \frac{S_{D1}}{T(R/I_e)} = \frac{1.21}{0.217(6.5/1)} = \boxed{0.854}$$ [Eq. 12.8-3]

$$C_{s,min} = 0.044 S_{DS} I_e \geq 0.01 = 0.044(1.418)1 = \boxed{0.062}$$ [Eq. 12.8-5]

$$C_{s,min} = \frac{0.5 S_1}{(R/I_e)} = \frac{0.5(0.723)}{(6.5/1)} = \boxed{0.056}$$ [Eq. 12.8-6]

C_s	=	0.218

$C_{s,ASD}$	=	0.153

LATERAL LOADS: SEISMIC

SEISMIC WEIGHT

		LOAD	ADJ. LOAD	PERIMETER	Height	AREA	WEIGHT
		psf	psf	ft	ft	ft^2	kips
STORY 2	ROOF 1	15.8	17.7	-		1,740	30.7
	INT. WALLS	7.5	-	206	5.5	1,133	8.5
	EXT. WALLS	11.0	-	176	5.5	968	10.6
						∑ =	49.9
STORY 1	FLOOR	11.5	-	-		1,740	20.0
	INT. WALLS	7.5	-	206	5.5	1,133	8.5
	EXT. WALLS	11.0	-	176	5.5	968	10.6
	INT. WALLS	7.5	-	88	5.5	484	3.6
	EXT. WALLS	11.0	-	176	5.5	968	10.6
						∑ =	53.4

Seismic Weight			
W	=	103	kips

Seismic Base Shear			
ρ	=	1.30	[Redundancy Factor]
C_s	=	0.218	
ρC_s	=	0.284	

ρ (North-South)	=	1.30
ρ (East-West)	=	1.30
$\rho C_{s,ASD}$	=	0.199

V	=	ρCsW	
V	=	29.3	kips
0.75V	=	22.0	kips

[CEBC A403.3]	=	Yes
$0.75\rho C_{s,ASD}$	=	0.149

Vertical Distribution of Seismic Forces

T_a	=	0.217	sec
k	=	1.00	

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \quad \text{[Eq. 12.8-12]}$$

$$F_x = C_{vx}V \quad \text{[Eq. 12.8-11]}$$

					LRFD	ASD
	w_i	h_i	$w_i h_i^k$	C_{vx}	F_x	$0.7F_x$
	kips	ft			kips	kips
STORY 2	49.9	23	1,147	0.661	14.5	10.2
STORY 1	53.4	11	588	0.339	7.4	5.2
	103.3	∑ =	1,735	1.00	22.0	15.4

LATERAL ANALYSIS: WIND & SEISMIC BASE SHEARS

	SEISMIC		
	Load	ASD	V_{ASD}
	kips	Load Factor	kips
V_E	29.30	0.70	20.51

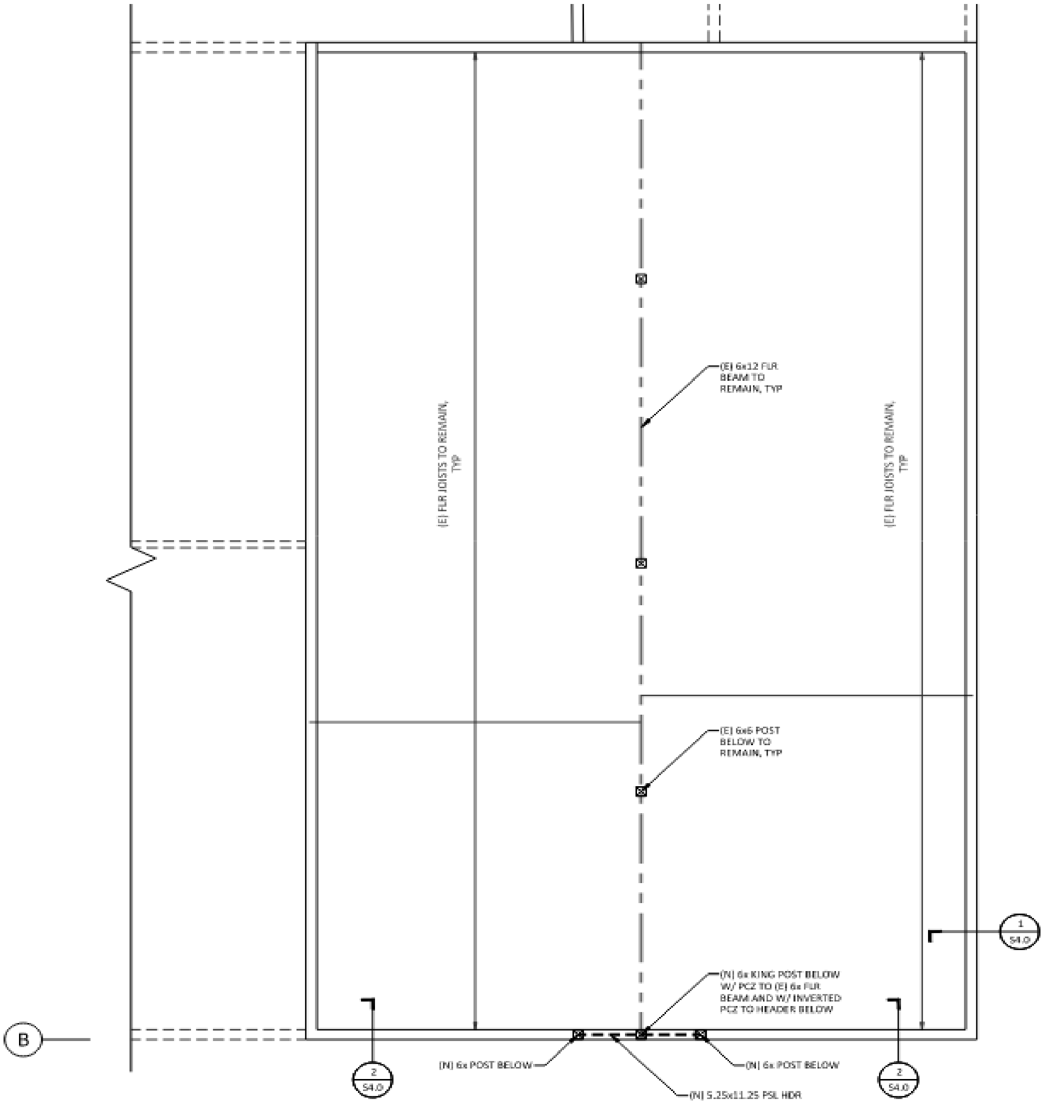
	WIND		
	Load	ASD	V_{ASD}
	kips	Load Factor	kips
V_w	21.26	0.60	12.75

NOTES:

1. Wind load shown is for the governing direction.
2. Seismic base shear governs.

LATERAL ANALYSIS & DESIGN

SEISMIC LINE LOADS:	:	



					LOAD	LOAD
		FORCE	AREA	*LOAD*	LRFD	ASD
ρ		kips	ft^2	*psf*	psf	psf
1.30	STORY 2	14.5	1,740	*8*	8	6
1.30	STORY 1	7.4	1,740	*4*	4	3
	∑ =	22.0				

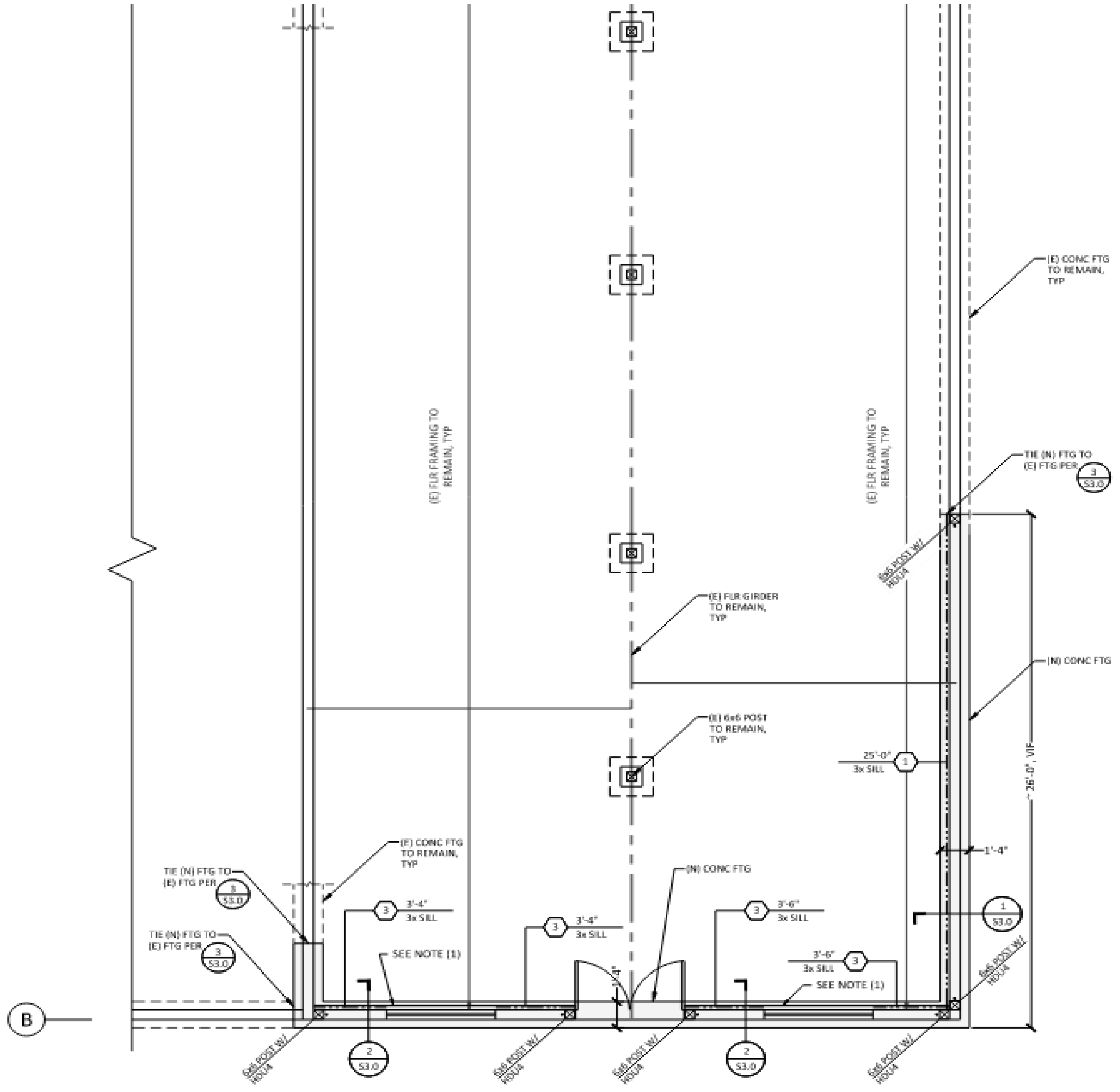
SEISMIC LINE LOADS: :

DIAPHRAGM = STORY 2

Line = 2

w	=	*8*	psf	x	58	ft	=	484	lb/ft
V	=	$\frac{1}{2}$	x	484 lb/ft	x	30	=	7,265	lbs
Reaction(s) from shearwall(s) at story above:									lbs
						∑V	=	7,265	lbs

SEISMIC LINE LOADS: :



					LOAD	LOAD
		FORCE	AREA	*LOAD*	LRFD	ASD
ρ		kips	ft^2	*psf*	psf	psf
1.30	STORY 2	14.5	1,740	*8*	8	6
1.30	STORY 1	7.4	1,740	*4*	4	3
	∑ =	22.0				

SEISMIC LINE LOADS:	:	

DIAPHRAGM	=	STORY 1
Line	=	2

w	=	*4*	psf	x	58	ft	=	248	lb/ft
V	=	1 2	x	248 lb/ft	x	30	=	3,722	lbs

Reaction(s) from shearwall(s) at story above: 7,265 lbs

∑V = 10,988 lbs

LATERAL ANALYSIS & DESIGN

SHEARWALL DESIGN	:	

DIAPHRAGM	=	STORY 1
Line	=	2

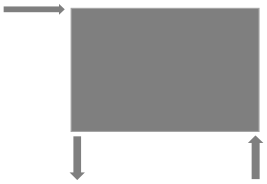
0.7E = 0.7 x 10,988 lbs = 7,691 lbs [Governs]

$(0.6-.14S_{DS})D$	=	0.40	x	17.7 psf	x	7.5 ft	=	53.2	lb/ft
$(0.6-.14S_{DS})D$	=	0.40	x	11.5 psf		7.5 ft	=	34.6	lb/ft
$(0.6-.14S_{DS})D$	=	0.40	x	11. psf	x	25. ft	=	110.4	lb/ft
							w =∑ =	198.2	lb/ft

$$v = \frac{7{,}691\text{ lbs}}{25.\text{ ft}} = 308 \text{ plf}$$

H	=	12.0	ft
L	=	25.0	ft
L'	=	24.5	ft

H/L =	0.48
Factor =	1.00



$$T = \frac{vLH-wL(L'/2)}{L'} = 1{,}289 \text{ lbs}$$

$$C = wL+T = 6{,}245 \text{ lbs}$$

Eonly	=	3,767	lbs
Eonly/(0.7)	=	5,382	lbs

USE TYPE 1 SHEARWALL

USE DTT2Z HOLDOWN

MIN POST SIZE	=	2-2x4

Shearwall Deflection

A	=	10.5	in.^2
Nail Spacing	=	6	in. o.c.
G_a	=	16	kips/in.
Holdown	=	DTT2Z	
Δ_a	=	0.128	in.

v	=	440	lb/ft
h	=	12.0	ft
b	=	25.0	ft
E	=	1,600,000	psi

Elastic Displacement:

$$\delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} = 0.406 \text{ in.}$$

Inelastic Displacement:

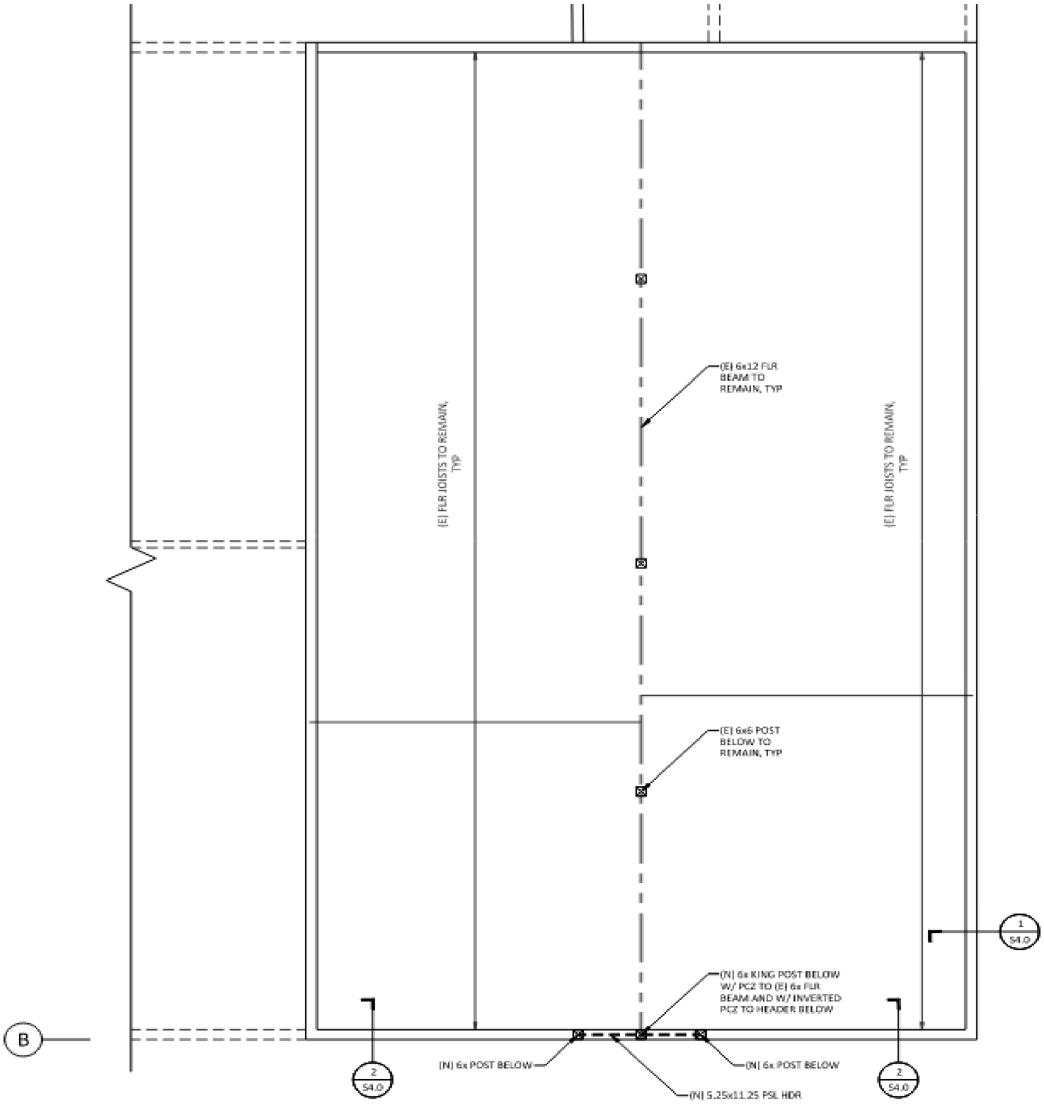
C_d = 4

I_e = 1.00

$$\delta_M = \frac{C_d\delta_{max}}{I_e} = \frac{1.622\text{ in.}}{1.00} = 1.622 \text{ in.}$$

Drift Ratio	=	1.13%

SEISMIC LINE LOADS: | :



					LOAD	LOAD
		FORCE	AREA	*LOAD*	LRFD	ASD
ρ		kips	ft^2	*psf*	psf	psf
1.30	STORY 2	14.5	1,740	*8*	8	6
1.30	STORY 1	7.4	1,740	*4*	4	3
	∑ =	22.0				

SEISMIC LINE LOADS:	:	

DIAPHRAGM = STORY 2

Line = B

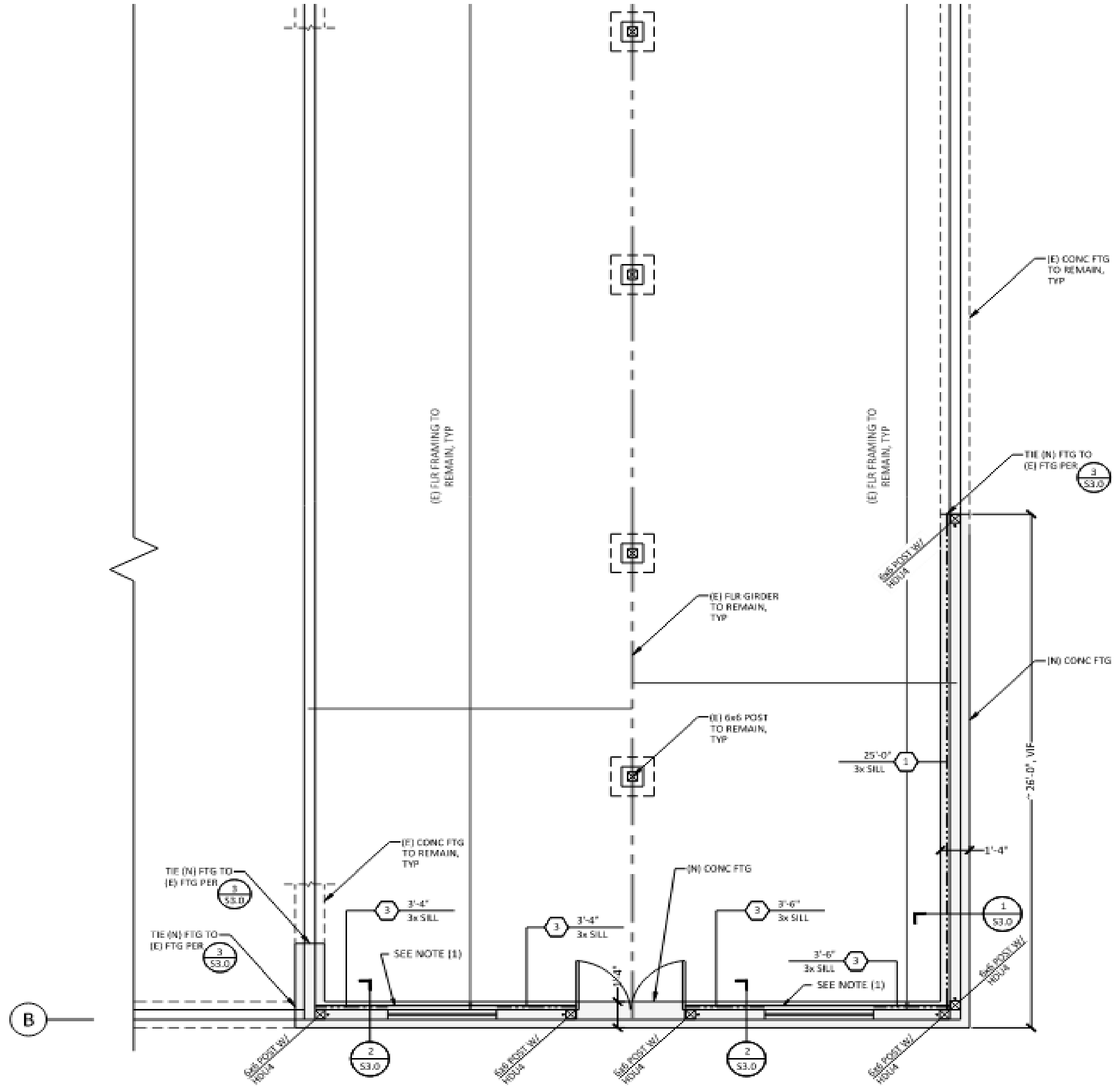
w = 8 psf x 30 ft = 251 lb/ft

V = $\frac{1}{2}$ x 251 lb/ft x 58 = 7,265 lbs

Reaction(s) from shearwall(s) at story above: lbs

∑V = 7,265 lbs

SEISMIC LINE LOADS:	:	



					LOAD	LOAD
		FORCE	AREA	*LOAD*	LRFD	ASD
ρ		kips	ft^2	*psf*	psf	psf
1.30	STORY 2	14.5	1,740	*8*	8	6
1.30	STORY 1	7.4	1,740	*4*	4	3
	∑ =	22.0				

SEISMIC LINE LOADS:	:	

DIAPHRAGM	=	STORY 1
Line	=	B

w	=	4	psf	x	30	ft	=	128	lb/ft
V	=	1 2	x	128 lb/ft	x	58	=	3,722	lbs

Reaction(s) from shearwall(s) at story above: 7,265 lbs

∑V = 10,988 lbs

SHEARWALL DESIGN	:	

DIAPHRAGM = STORY 1

Line = B

0.7E	=	0.7	x	10,988	lbs	=	7,691	lbs	[Governs]
$(0.6-.14S_{DS})D$	=	0.40	x	17.7 psf	x	3. ft	=	21.3	lb/ft
$(0.6-.14S_{DS})D$	=	0.40	x	11.5 psf	x	2. ft	=	9.2	lb/ft
$(0.6-.14S_{DS})D$	=	0.40	x	11. psf	x	25. ft	=	110.4	lb/ft
							w =∑ =	140.9	lb/ft

Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

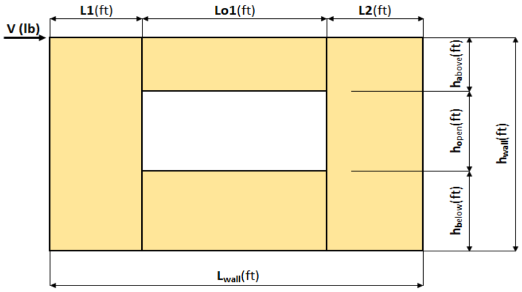
Code: **Date:**

Designer:

Client:

Project:

Wall Line: STORY 1 - GRIDLINE A



Shear Wall Calculation Variables

V	3850 lbf
L1	3.33 ft
L2	3.33 ft
h_{wall}	10.67 ft
L_{wall}	11.66 ft

Opening 1	
ha1	2.67 ft
ho1	5.50 ft
hb1	2.50 ft
Lo1	5.00 ft

Adj. Factor Method = 2bs/h

Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	1.65	N/A
P2=ho2/L2=	1.65	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 3523 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 681 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3407 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1704 lbf

F2 = O1(L2)/(L1+L2) = 1704 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.50 ft

T2 = (L2*Lo1)/(L1+L2) = 2.50 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 578 plf

V2 = (V/L)(T2+L2)/L2 = 578 plf

Check V1*L1+V2*L2=V? 3850 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 1925 lbf

R2 = V2*L2 = 1925 lbf

8. Difference corner force + resistance

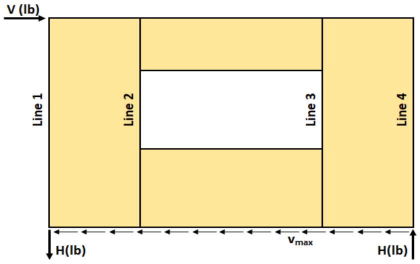
R1-F1 = 221 lbf

R2-F2 = 221 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 66 plf

vc2 = (R2-F2)/L2 = 66 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		344	3179	3523 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	3523	344	3179	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	3523	344	3179	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		344	3179	3523 lbf

Design Summary*

Req. Sheathing Capacity	681 plf	**4-Term Deflection**	0.749 in.	**3-Term Deflection**	0.783 in.
Req. Strap Force	1704 lbf	**4-Term Story Drift %**	0.023 %	**3-Term Story Drift %**	0.024 %
Req. HD Force (H)	3523 lbf		See Page 2		See Page 3
Req. Shear Wall Anchorage Force (v_{max})	330 plf				

*The Design Summary assumes that the shear wall is designed as blocked.

Project Information

Code: **Date:**

Designer:

Client:

Project:

Wall Line: STORY 1 - GRIDLINE A

Shear Wall Deflection Calculation Variables

Induced Shear Load $V_{induced}$: 5500 (lbf)

Sheathing:

OSB	Sheathing Material
15/32	Performance Category
APA Structural 1	Grade
	Gt Override
	Ga Overide

Wood End Post Values:

Species:	DF		
E:	1.60E+06	(psi)	
	Qty	Stud Size	
Dimensions:	2	2x6	
A:	16.5	$(in.^2)$	
A Override:		$(in.^2)$	

Nail Type: 10d common (penny weight)

	Pier 1	Pier 2	
Nail Spacing:	2	2	(in.)
HD Capacity:	4565	4565	(lbf)
HD Deflection:	0.115	0.115	(in.)

Four-Term Equation Deflection Check

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b} \qquad \text{(Equation 23-2)}$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	15/32	15/32	15/32	15/32	
Nail:	10d common	10d common	10d common	10d common	
$v_{induced}$:	826	826	826	826	(plf)
E:	1.60E+06	1.60E+06	1.60E+06	1.60E+06	(psi)
h:	10.67	8.17	8.17	10.67	(ft)
A:	16.5	16.5	16.5	16.5	$(in.^2)$
Gt:	83,500	83,500	83,500	83,500	(lbf/in.)
Nail Spacing:	2	2	2	2	(in.)
Vn:	138	138	138	138	(plf)
e_n:	0.0036	0.0036	0.0036	0.0036	(in.)
b:	3.33	3.33	3.33	3.33	(ft)
HD Capacity:	4565	4565	4565	4565	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	(in.)

Check Total Deflection of Wall System

Pier 1 (left)				Pier 1 (right)			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.091	0.106	0.029	0.711	0.041	0.081	0.022	0.417
		Sum	0.937			Sum	0.561
Pier 2 (left)				**Pier 2 (right)**			
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-1	Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 4 HD-2
0.041	0.081	0.022	0.417	0.091	0.106	0.029	0.711
		Sum	0.561			Sum	0.937

Total Defl.	
0.749	(in.)
0.0234	%drift

Project Information

Code: **Date:**

Designer:

Client:

Project:

Wall Line: STORY 1 - GRIDLINE A

Three-Term Equation Deflection Check

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad (4.3\text{-}1)$$

	Pier 1-L	Pier 1-R	Pier 2-L	Pier 2-R	
Sheathing:	15/32	15/32	15/32	15/32	
Nail:	10d common	10d common	10d common	10d common	
$v_{induced}$:	826	826	826	826	(plf)
E:	1.60E+06	1.60E+06	1.60E+06	1.60E+06	(psi)
h:	10.67	8.17	8.17	10.67	(ft)
A:	16.5	16.5	16.5	16.5	(in.^{2})
Ga:	51.0	51.0	51.0	51.0	(kips/in.)
b:	3.33	3.33	3.33	3.33	(ft)
HD Capacity:	4565	4565	4565	4565	(lbf)
HD Defl:	0.115	0.115	0.115	0.115	(in.)

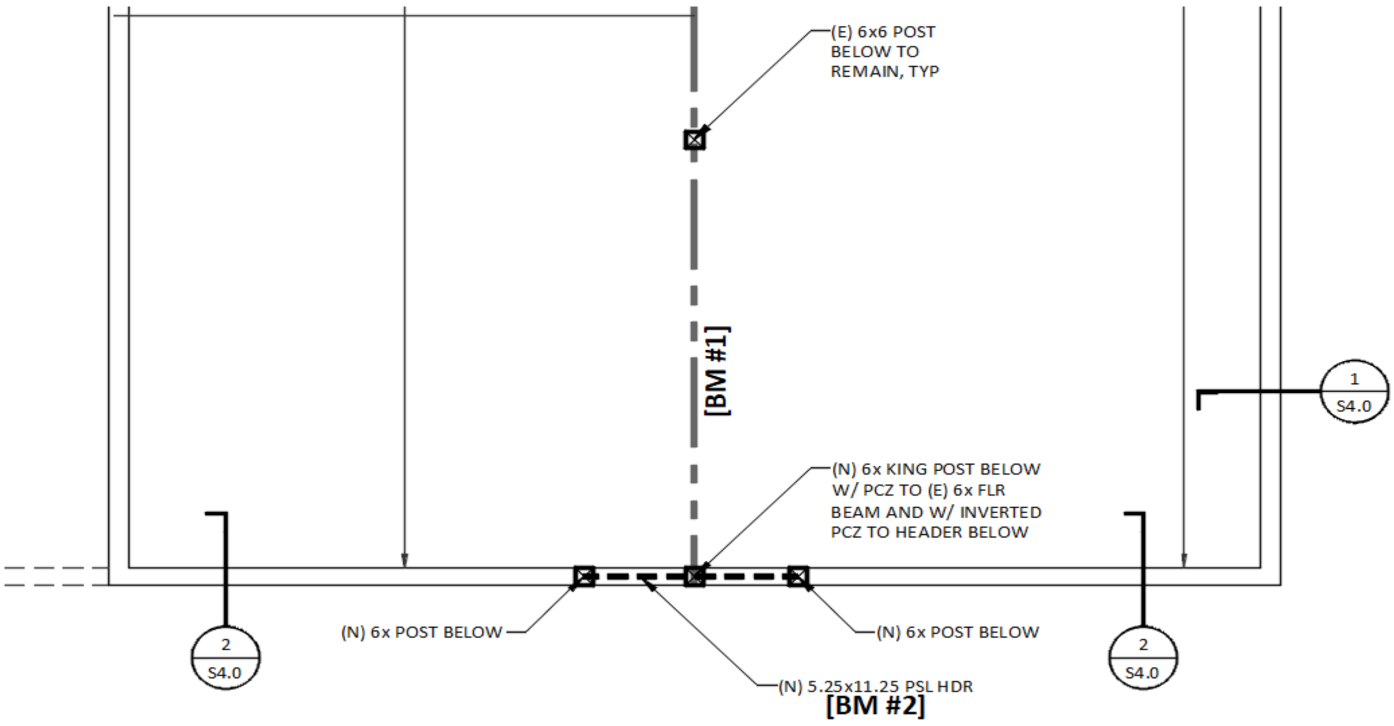
Check Total Deflection of Wall System

Pier 1 (left)			Pier 1 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.091	0.173	0.711	0.041	0.132	0.417
	Sum	0.975		Sum	0.590
Pier 2 (left)			Pier 2 (right)		
Term 1 Bending	Term 2 Shear	Term 3 Fastener	Term 1 Bending	Term 2 Shear	Term 3 Fastener
0.041	0.132	0.417	0.091	0.173	0.711
	Sum	0.590		Sum	0.975

Total Defl.	
0.783	(in.)
0.0245	%drift

Comment: The 3-term equation is calibrated to be approximately equal to 4-term equation at 1.4*ASD capacity.

BEAM DESIGN/CALCS: 2ND FLOOR



2ND FLOOR FRAMING PLAN

DESCRIPTION: BM #1: EXIST FLR 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
Load Combination : ASCE 7-16

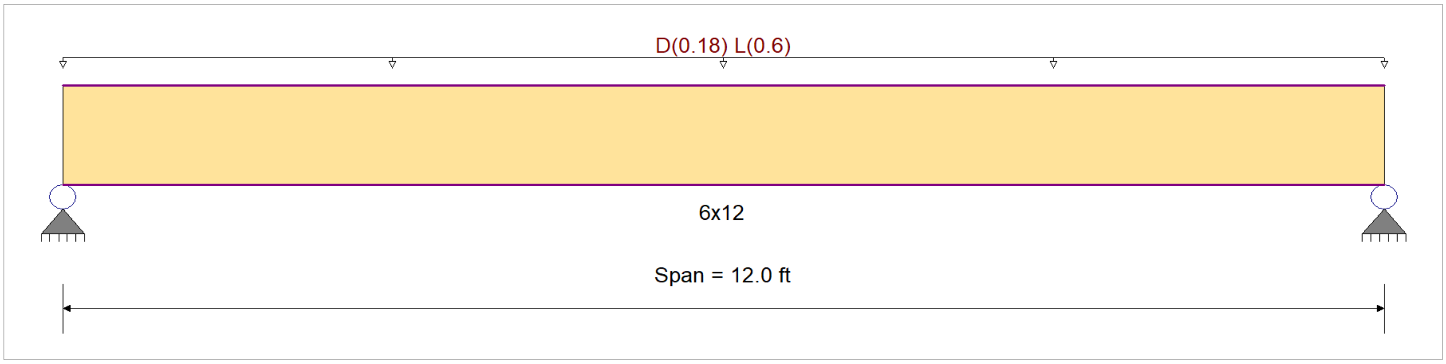
Wood Species : Douglas Fir-Larch
Wood Grade : Dense No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	1550	psi
Fb -	1550	psi
Fc - Prll	1100	psi
Fc - Perp	730	psi
Fv	170	psi
Ft	775	psi

E : Modulus of Elasticity

Ebend- xx	1700	ksi
Eminbend - xx	620	ksi
Density	31.21	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0120, L = 0.040 ksf, Tributary Width = 15.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.912** : 1	Maximum Shear Stress Ratio	=	**0.563** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	1,414.19 psi	fv: Actual	=	95.63 psi
Fb: Allowable	=	1,550.00 psi	Fv: Allowable	=	170.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	6.000 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	0.238 in	Ratio =	606 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.314 in	Ratio =	458 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

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+H													0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.247	0.153	0.90	1.000	1.00	1.00	1.00	1.00	1.00	3.49	345.14	1395.00	0.98	23.34	153.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.912	0.563	1.00	1.000	1.00	1.00	1.00	1.00	1.00	14.29	1,414.19	1550.00	4.03	95.63	170.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.178	0.110	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.49	345.14	1937.50	0.98	23.34	212.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.194	0.119	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.49	345.14	1782.50	0.98	23.34	195.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.592	0.365	1.25	1.000	1.00	1.00	1.00	1.00	1.00	11.59	1,146.93	1937.50	3.27	77.55	212.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.643	0.397	1.15	1.000	1.00	1.00	1.00	1.00	1.00	11.59	1,146.93	1782.50	3.27	77.55	195.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

DESCRIPTION: BM #1: EXIST FLR BEAM

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
Length = 12.0 ft	1	0.139	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.49	345.14	2480.00	0.98	23.34	272.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.462	0.285	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.59	1,146.93	2480.00	3.27	77.55	272.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.462	0.285	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.59	1,146.93	2480.00	3.27	77.55	272.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.084	0.051	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.09	207.08	2480.00	0.59	14.00	272.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.139	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.49	345.14	2480.00	0.98	23.34	272.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.462	0.285	1.60	1.000	1.00	1.00	1.00	1.00	1.00	11.59	1,146.93	2480.00	3.27	77.55	272.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.0 ft	1	0.084	0.051	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.09	207.08	2480.00	0.59	14.00	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.3143	6.044		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	4.762	4.762
Overall MINimum	3.600	3.600
+D+H	1.162	1.162
+D+L+H	4.762	4.762
+D+Lr+H	1.162	1.162
+D+S+H	1.162	1.162
+D+0.750Lr+0.750L+H	3.862	3.862
+D+0.750L+0.750S+H	3.862	3.862
+D+0.60W+H	1.162	1.162
+D+0.750Lr+0.750L+0.450W+H	3.862	3.862
+D+0.750L+0.750S+0.450W+H	3.862	3.862
+0.60D+0.60W+0.60H	0.697	0.697
+D+0.70E+0.60H	1.162	1.162
+D+0.750L+0.750S+0.5250E+H	3.862	3.862
+0.60D+0.70E+H	0.697	0.697
D Only	1.162	1.162
L Only	3.600	3.600
H Only		

Wood Beam

DESCRIPTION: BM #2: NEW FLR HEADER

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
Load Combination : ASCE 7-16

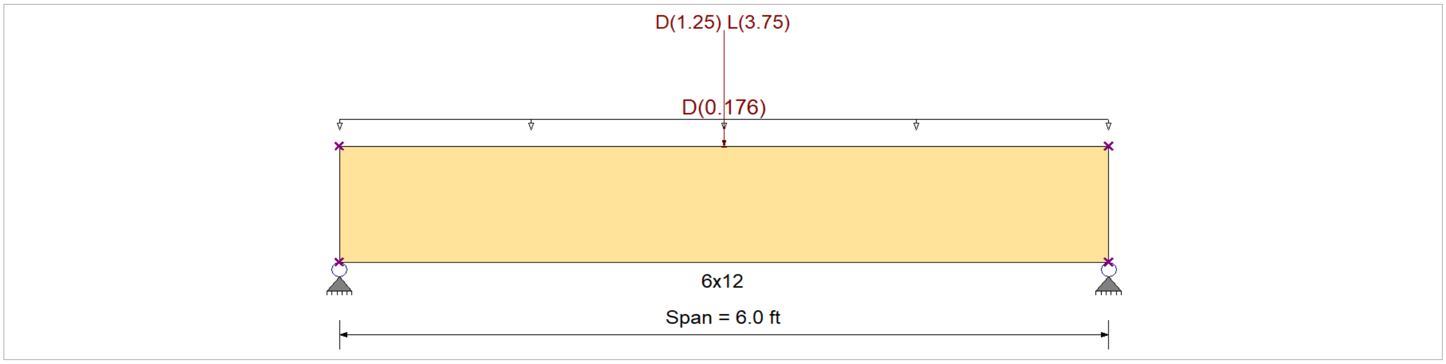
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

Beam Bracing : Completely Unbraced

Fb +	875	psi
Fb -	875	psi
Fc - Prll	600	psi
Fc - Perp	625	psi
Fv	170	psi
Ft	425	psi

E : Modulus of Elasticity

Ebend- xx	1300	ksi
Eminbend - xx	470	ksi
Density	31.21	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.0110 ksf, Tributary Width = 16.0 ft, (Ext Wall)
Point Load : D = 1.250, L = 3.750 k @ 3.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.950**: 1	Maximum Shear Stress Ratio	=	**0.403** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	826.90psi	fv: Actual	=	68.55 psi
Fb: Allowable	=	870.85psi	Fv: Allowable	=	170.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	3.000ft	Location of maximum on span	=	0.000ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.032 in	Ratio =	2225 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.049 in	Ratio =	1461 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

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+H													0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.344	0.157	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.73	270.10	784.16	1.02	24.08	153.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.950	0.403	1.00	1.000	1.00	1.00	1.00	1.00	1.00	8.35	826.90	870.85	2.89	68.55	170.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.248	0.113	1.25	1.000	1.00	1.00	1.00	1.00	0.99	2.73	270.10	1087.12	1.02	24.08	212.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.270	0.123	1.15	1.000	1.00	1.00	1.00	1.00	0.99	2.73	270.10	1000.69	1.02	24.08	195.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.633	0.270	1.25	1.000	1.00	1.00	1.00	1.00	0.99	6.95	687.70	1087.12	2.42	57.43	212.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.687	0.294	1.15	1.000	1.00	1.00	1.00	1.00	0.99	6.95	687.70	1000.69	2.42	57.43	195.50

DESCRIPTION: BM #2: NEW FLR HEADER

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+0.60W+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.194	0.089	1.60	1.000	1.00	1.00	1.00	1.00	0.99	2.73	270.10	1388.80	1.02	24.08	272.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.495	0.211	1.60	1.000	1.00	1.00	1.00	1.00	0.99	6.95	687.70	1388.80	2.42	57.43	272.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.495	0.211	1.60	1.000	1.00	1.00	1.00	1.00	0.99	6.95	687.70	1388.80	2.42	57.43	272.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.117	0.053	1.60	1.000	1.00	1.00	1.00	1.00	0.99	1.64	162.06	1388.80	0.61	14.45	272.00
+D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.194	0.089	1.60	1.000	1.00	1.00	1.00	1.00	0.99	2.73	270.10	1388.80	1.02	24.08	272.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.495	0.211	1.60	1.000	1.00	1.00	1.00	1.00	0.99	6.95	687.70	1388.80	2.42	57.43	272.00
+0.60D+0.70E+H					1.000	1.00	1.00	1.00	1.00	0.99			0.00	0.00	0.00	0.00
Length = 6.0 ft	1	0.117	0.053	1.60	1.000	1.00	1.00	1.00	1.00	0.99	1.64	162.06	1388.80	0.61	14.45	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.0493	3.022		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.069	3.069
Overall MINimum	1.875	1.875
+D+H	1.194	1.194
+D+L+H	3.069	3.069
+D+Lr+H	1.194	1.194
+D+S+H	1.194	1.194
+D+0.750Lr+0.750L+H	2.600	2.600
+D+0.750L+0.750S+H	2.600	2.600
+D+0.60W+H	1.194	1.194
+D+0.750Lr+0.750L+0.450W+H	2.600	2.600
+D+0.750L+0.750S+0.450W+H	2.600	2.600
+0.60D+0.60W+0.60H	0.716	0.716
+D+0.70E+0.60H	1.194	1.194
+D+0.750L+0.750S+0.5250E+H	2.600	2.600
+0.60D+0.70E+H	0.716	0.716
D Only	1.194	1.194
L Only	1.875	1.875
H Only		