BRENT & CHERYL STANGE RESIDENTIAL ALTERATION

76-315 WANA ST 63B,
KAILUA KONA, HI 96740

TMK: (3) 7-6-025-063-0002

STRUCTURAL DESIGN
CALCULATIONS

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION AND CONSTRUCTION OF THIS PROJECT WILL BE UNDER MY OBSERVATION.

COVER PAGES FROM 1 TO 48

	Date:	Prepared By:	Project Number:	Rev.
	08-25-2025	MSB	25798	0
Filepath:				

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 1: Design Criteria

Code : 2018 Hawaii State Building Code in reference with 2018 IBC.

A. Dead Load:

Roof Load:

Roofing	5.0psf
Insulation	2.0psf
Self-Weight of Roof Deck	4.0psf
Self-Weight of roof framing system	4.0psf
MEP	3.0psf
Misc.	2.0psf
Total	**20.0psf**

Wall Load:

Self-Weight of 2"x4" Exterior Wall	6.0psf	***(Refer Architectural Drawings)***
Insulation	3.0psf	
Total	9.0psf	

B. Live Load:

Roof Load: **20.0psf** ***(Table-1607.1,IBC-2018)***

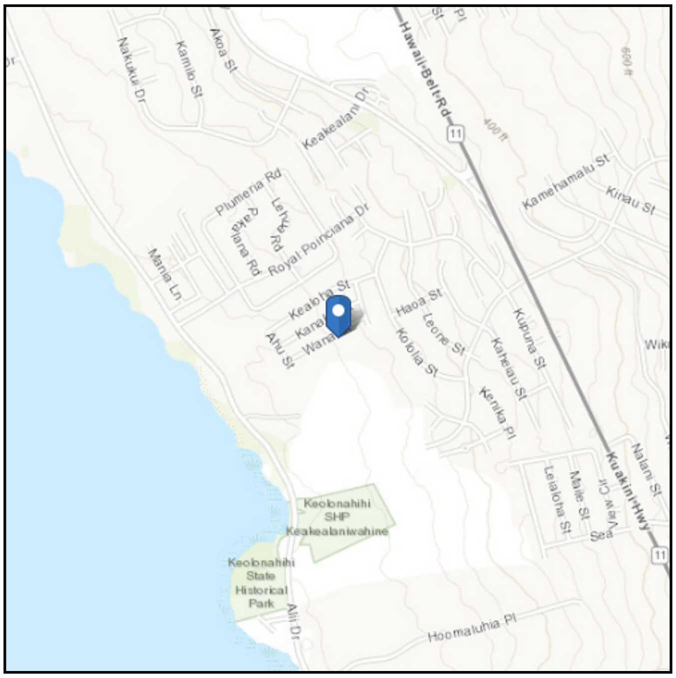
C. Wind Load:

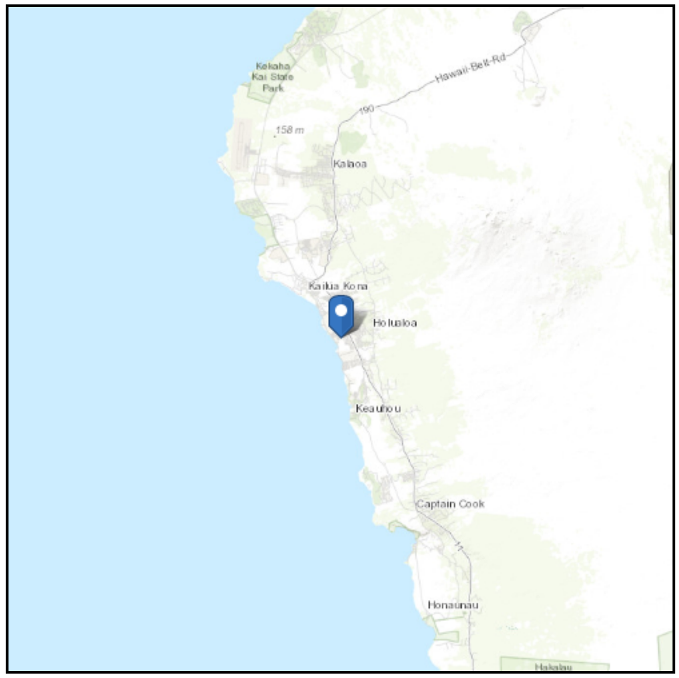
Basic Wind Speed (V_b) :	122.00 mph	***(Refer. ASCE 7 Hazards Report)***
Risk Category :	II	
Wind Exposure Category :	C	***(Refer. State Building Code)***

ASCE Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16	**Latitude:**	19.609209
Risk Category:	II	**Longitude:**	-155.972698
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	87.3359260066918 ft (National Geodetic Vertical Datum of 1929 (NGVD 29))





Wind

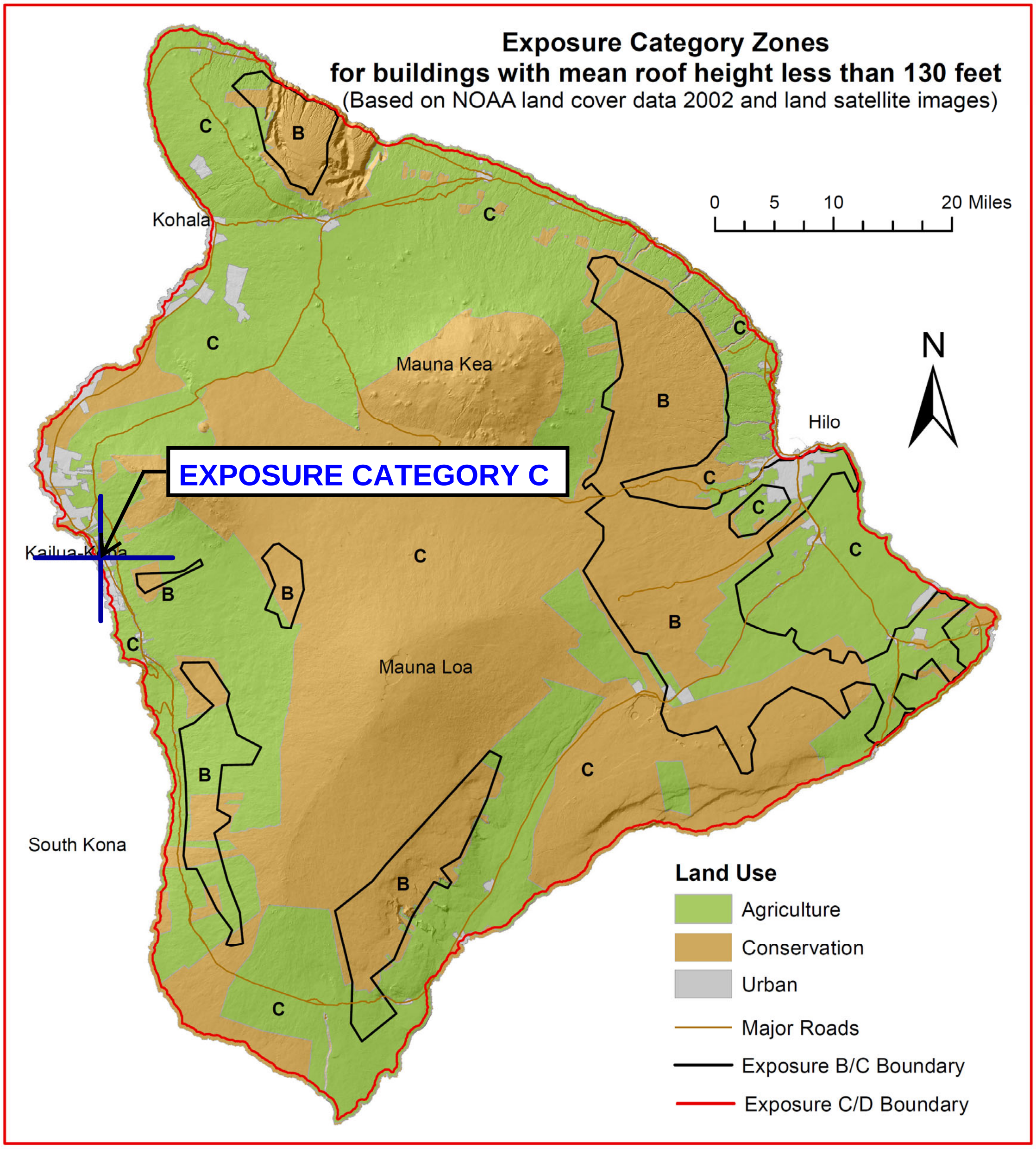
Results:

Wind Speed	122 Vmph
10-year MRI	53 Vmph
25-year MRI	59 Vmph
50-year MRI	67 Vmph
100-year MRI	86 Vmph
Special	Special Wind Region -- Mountainous terrain, gorges, and special wind regions shown in Fig. 26.5-1 shall be examined for unusual wind conditions. The Authority Having Jurisdiction shall, if necessary, adjust the values given in Fig. 26.5-1 to account for higher local wind speeds. Such adjustment shall be based on meteorological information and an estimate of the basic wind speed obtained in accordance with the provisions in Section 26.5.3.

Data Source:	ASCE/SEI 7-16, Fig. 26.5-2B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Mon Aug 25 2025

(11) R301.2.1.4 Exposure Category

Exposure Category Section R301.2.1.3 is amended to read as follows: "R301.2.1.4 Exposure Category. The exposure category shall be determined from Figures R301.2.1.4(a) through R301.2.1.4(e) or using the provisions of ASCE 7-10.



Notes:

1. Intermediate exposures, between categories B and C and between C and D, are permitted when substantiated per ASCE 7 recognized methodology.
2. Sites located within the C (coastal) zone shall be permitted to be evaluated for exposure category B for the wind directions where an adjacent B zone exists in the applicable upwind sector.
3. Sites located within 600 feet from the coastline shall be exposure category D for onshore wind directions.
4. For buildings whose height is equal to or greater than 130 ft, exposure category shall be determined per Section 1609.4.1.
5. For buildings whose mean roof height is less than or equal to 30 ft, exposure category shall be permitted to be evaluated per Section 1609.4.

Figure R301.2.1.4(a) Exposure Category Zones for Hawaii County

1609.3.3 Directionality factor.

The wind directionality factor, K_d, shall be determined from Tables 1609.3.3(a)(1) through 1609.3.3(a)(3) and 1609.3.3(b)(1) through 1609.3.3(b)(3), and Figures 1609.3.3(a)(4) and 1609.3.3(b)(4).
Exception: Basic design wind speed, V, is determined per Figures 1609.3(5) through 1609.3(8) that already include topographic effects near mountainous terrain and near gorges, which shall be used with a topographic factor Kzt of 1.0 and the directionality factors given in Table 26.6-1 of ASCE 7.

Table 1609.3.3(a)(1)
K_d Values for Main Wind Force Resisting Systems Sited in Hawaii County [a,b]

Topographic Location on the Island of Hawaii	Main Wind Force Resisting Systems		Main Wind Force Resisting Systems with totally independent systems in each orthogonal direction		Biaxially Symmetric and Axisymmetric Structures of any Height and Arched Roof Structures
	Mean Roof Height less than or equal to 100 ft.	Mean Roof Height greater than 100 ft.	Mean Roof Height less than or equal to 100 ft.	Mean Roof Height greater than 100 ft.	
Sites in North Kohala, South Kohala, South Kona, South Hilo, and Puna Districts at an elevation not greater than 3000 ft.	0.65	0.70	0.70	0.75	0.85
All other sites	0.70	0.80	0.75	0.80	0.95

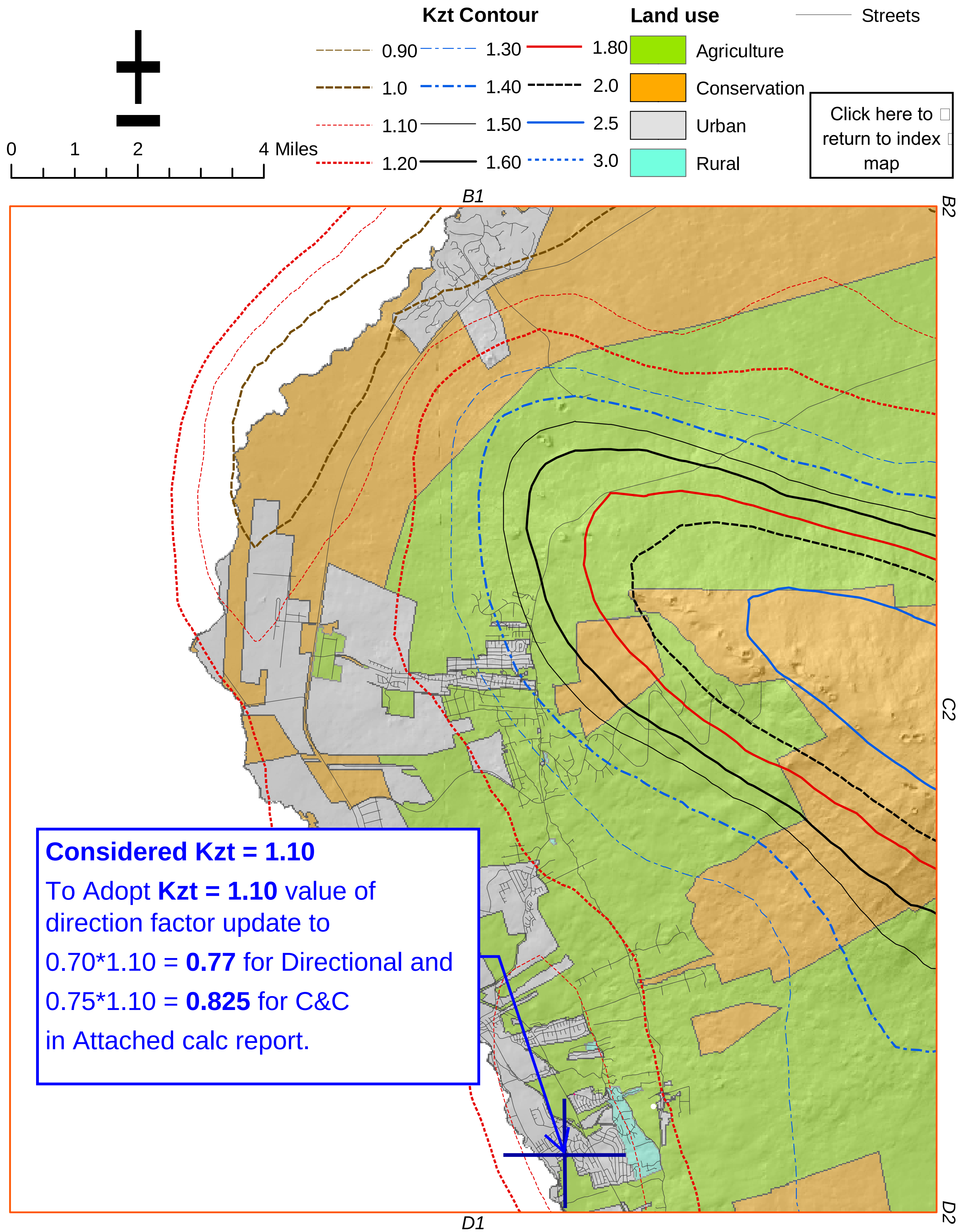
a. The values of K_d for other non-building structures indicated in ASCE-7 Table 26.6-1 shall be permitted.
b. Site-specific probabilistic analysis of K_d based on wind-tunnel testing of topography and peak gust velocity profile shall be permitted to be submitted for approval by the Building Official, but K_d shall have a value not less than 0.65.

Table 1609.3.3(b)(1)
***K_d* Values for Components and Cladding of Buildings Sited in Hawaii County [a,b]**

Topographic Location on the Island of Hawaii	Components and Cladding		
	Mean Roof Height less than or equal to 100 ft.	Mean Roof Height greater than 100 ft.	Risk Category IV Buildings and Structures
Sites in North Kohala, South Kohala, South Kona, South Hilo, and Puna Districts at an elevation not greater than 3000 ft.	0.65	0.70	0.75
All other sites	0.75	0.80	0.85

a. The values of K_d for other non-building structures indicated in ASCE-7 Table 26.6-1 shall be permitted.
b. Site-specific probabilistic analysis of K_d based on wind-tunnel testing of topography and peak gust velocity profile shall be permitted to be submitted for approval by the Building Official, but in any case, subject to a minimum value of 0.65.

Wind Topographic Factor (Kzt) for the Island of Hawaii (C1)



Chapter 1: Design Criteria Cont'd...

MWFRS and Corner Pressure Inputs:

Building Data:

Type of Roof :	Hip
Length of Building (L) :	63.81 ft
Width of Building (B) :	27.00 ft
Height of Building/Eaves (H) :	9.00 ft
Height of Parapet (Hp) :	0.00 ft
Mean Height (h) :	10.97 ft

Design Wind Pressure (MWFRS):

Core Pressure :

Windward Pressure, P.ww :	16.80 psf
Leeward Pressure, P.lw :	-5.30 psf
Design Wind Pressure, Pw :	**22.10 psf**

Corner Pressure :

Windward pressure, P.ww.c :	23.20 psf
Leeward pressure, P.lw.c :	-10.10 psf
Design wind pressure, Pw.c :	**33.30 psf**
Corner region width, 2a :	6.00 ft

Mean pressure Along N-S Direction :

Windward pressure, P.mww :	**17.40 psf**
Leeward pressure, P.mlw :	**-5.75 psf**

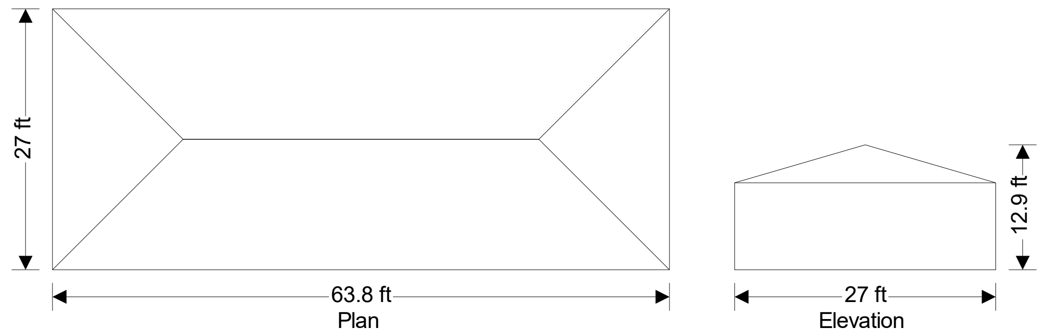
Chapter 1: Design Criteria Cont'd...

C.1 WIND LOAD

In accordance with ASCE7-16

Using the envelope design method

Tedds calculation version 2.1.16



Building data

Type of roof	Hipped
Length of building	b = **63.81** ft
Width of building	d = **27.00** ft
Height to eaves	H = **9.00** ft
Pitch of main slope	α_0 = **16.3** deg
Pitch of gable slope	α_{90} = **16.3** deg
Mean height	h = **10.97** ft
End zone width	$a = \max(\min(0.1\times\min(b, d), 0.4\times h), 0.04\times\min(b, d), 3\text{ft})$ = **3.00** ft
Plan length of Zone 2/2E when GC_{pf} negative	$L_{Z2} = \min(0.5 \times d, 2.5 \times H)$ = **13.50** ft
Plan length of Zone 3/3E encroachment on zone 2	$L_{Z3} = \max(0\text{ ft}, 0.5 \times d - L_{Z2})$ = **0.00** ft

General wind load requirements

Basic wind speed	V = **122.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.77**
Ground elevation above sea level	z_{gl} = **87** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1\text{ft})$ = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **24.9** psf

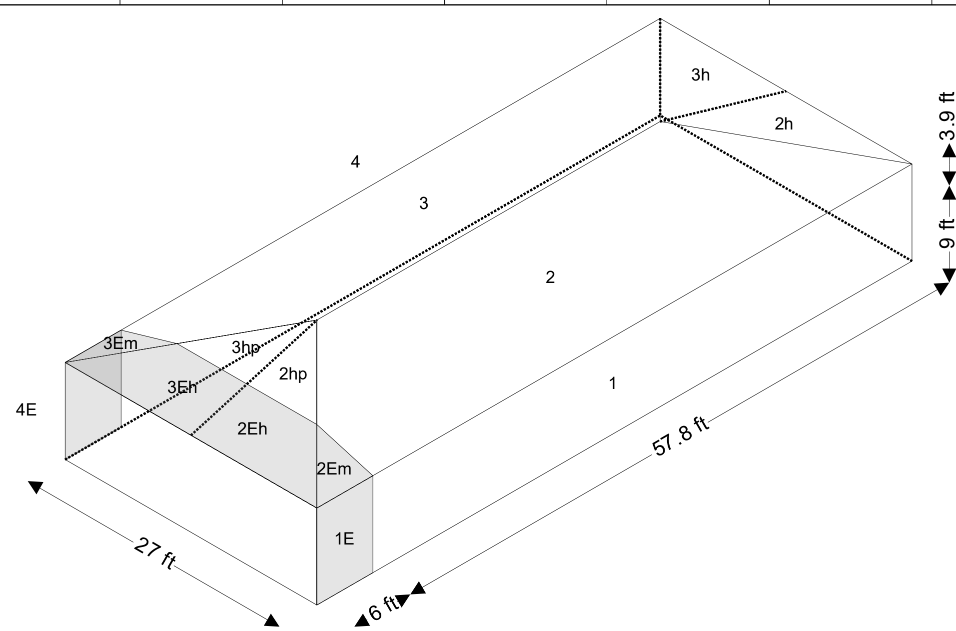
Design wind pressures

Design wind pressure equation	$p = q_h \times ((GC_{pf}) - (GC_{pi}))$

Chapter 1: Design Criteria Cont'd...

Design wind pressures – Loadcase A

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.50	7.9	16.8	520	4.1	8.8
2	-0.69	-21.6	-12.7	689	-14.9	-8.7
3	-0.45	-15.7	-6.8	689	-10.8	-4.7
4	-0.39	-14.3	-5.3	520	-7.4	-2.8
1E	0.75	14.2	23.2	54	0.8	1.3
2Em	-1.07	-31.1	-22.1	19	-0.6	-0.4
2Eh	-1.07	-31.1	-22.1	66	-2.0	-1.5
3Eh	-0.65	-20.6	-11.7	66	-1.4	-0.8
3Em	-0.65	-20.6	-11.7	19	-0.4	-0.2
4E	-0.59	-19.1	-10.1	54	-1.0	-0.5
2hp	-0.69	-21.6	-12.7	29	-0.6	-0.4
3hp	-0.45	-15.7	-6.8	29	-0.5	-0.2
2h	-0.69	-21.6	-12.7	95	-2.1	-1.2
3h	-0.45	-15.7	-6.8	95	-1.5	-0.6



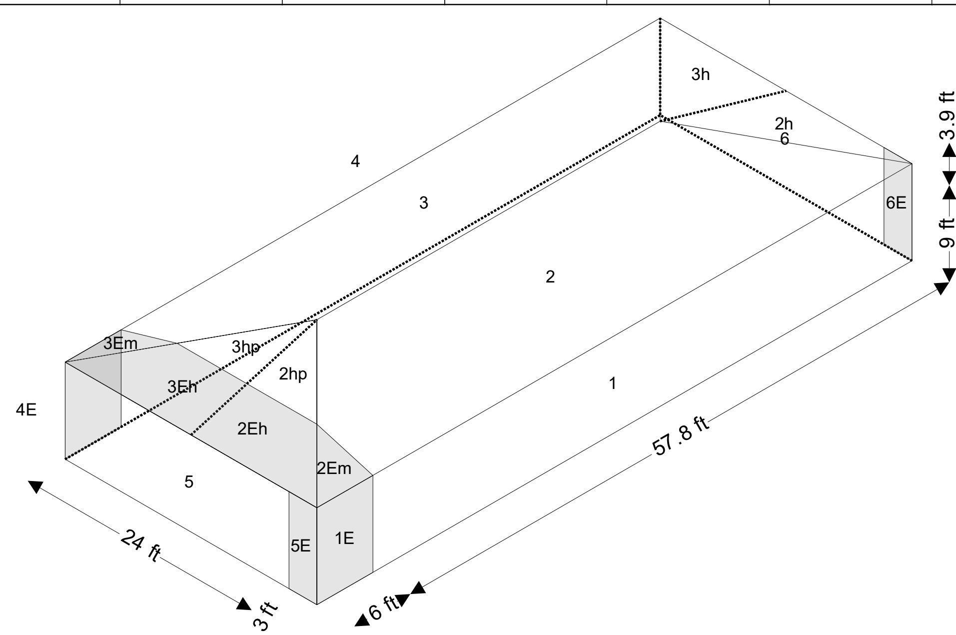
Loadcase A

Design wind pressures – Loadcase B

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-15.7	-6.7	520	-8.1	-3.5
2	-0.69	-21.6	-12.7	689	-14.9	-8.7
3	-0.37	-13.7	-4.7	689	-9.4	-3.3

Chapter 1: Design Criteria Cont'd...

4	-0.45	-15.7	-6.7	520	-8.1	-3.5
1E	-0.48	-16.4	-7.5	54	-0.9	-0.4
2Em	-1.07	-31.1	-22.1	19	-0.6	-0.4
2Eh	-1.07	-31.1	-22.1	66	-2.0	-1.5
3Eh	-0.53	-17.7	-8.7	66	-1.2	-0.6
3Em	-0.53	-17.7	-8.7	19	-0.3	-0.2
4E	-0.48	-16.4	-7.5	54	-0.9	-0.4
2hp	-0.69	-21.6	-12.7	29	-0.6	-0.4
3hp	-0.37	-13.7	-4.7	29	-0.4	-0.1
2h	-0.69	-21.6	-12.7	95	-2.1	-1.2
3h	-0.37	-13.7	-4.7	95	-1.3	-0.4
5	0.40	5.5	14.4	216	1.2	3.1
6	-0.29	-11.7	-2.7	216	-2.5	-0.6
5E	0.61	10.7	19.6	27	0.3	0.5
6E	-0.43	-15.2	-6.2	27	-0.4	-0.2



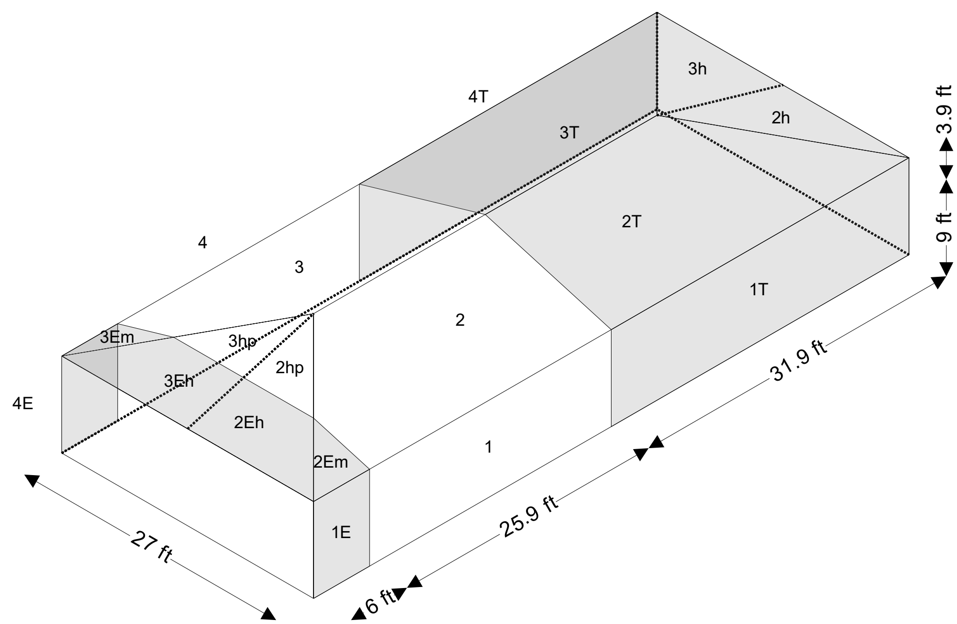
Loadcase B

Design wind pressures – Loadcase AT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	0.50	7.9	16.8	233	1.8	3.9
2	-0.69	-21.6	-12.7	335	-7.2	-4.2
3	-0.45	-15.7	-6.8	335	-5.3	-2.3
4	-0.39	-14.3	-5.3	233	-3.3	-1.2

Chapter 1: Design Criteria Cont'd...

1E	0.75	14.2	23.2	54	0.8	1.3
2Em	-1.07	-31.1	-22.1	19	-0.6	-0.4
2Eh	-1.07	-31.1	-22.1	66	-2.0	-1.5
3Eh	-0.65	-20.6	-11.7	66	-1.4	-0.8
3Em	-0.65	-20.6	-11.7	19	-0.4	-0.2
4E	-0.59	-19.1	-10.1	54	-1.0	-0.5
2hp	-0.69	-21.6	-12.7	29	-0.6	-0.4
3hp	-0.45	-15.7	-6.8	29	-0.5	-0.2
2h	-0.69	-21.6	-12.7	95	-2.1	-1.2
3h	-0.45	-15.7	-6.8	95	-1.5	-0.6
1T	-	2.0	4.2	287	0.6	1.2
2T	-	-5.4	-3.2	354	-1.9	-1.1
3T	-	-3.9	-1.7	354	-1.4	-0.6
4T	-	-3.6	-1.3	287	-1.0	-0.4



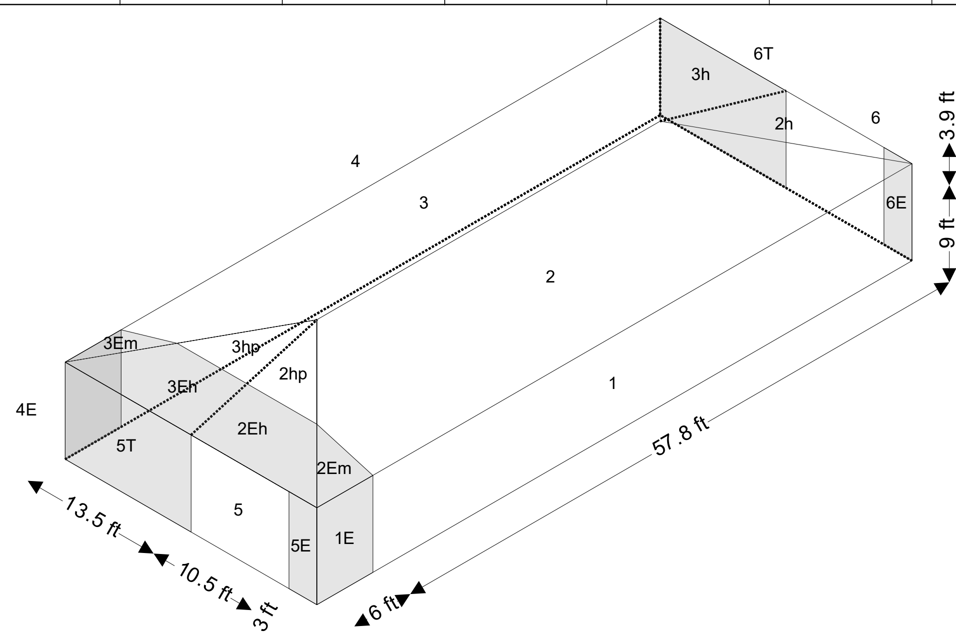
Loadcase AT

Design wind pressures – Loadcase BT

Zone	GC_{pf}	$p_{(+GCpi)}$ (psf)	$p_{(-GCpi)}$ (psf)	Area (ft^2)	$+F_{wi}$ (kips)	$-F_{wi}$ (kips)
1	-0.45	-15.7	-6.7	520	-8.1	-3.5
2	-0.69	-21.6	-12.7	689	-14.9	-8.7
3	-0.37	-13.7	-4.7	689	-9.4	-3.3
4	-0.45	-15.7	-6.7	520	-8.1	-3.5
1E	-0.48	-16.4	-7.5	54	-0.9	-0.4

Chapter 1: Design Criteria Cont'd...

2Em	-1.07	-31.1	-22.1	19	-0.6	-0.4
2Eh	-1.07	-31.1	-22.1	66	-2.0	-1.5
3Eh	-0.53	-17.7	-8.7	66	-1.2	-0.6
3Em	-0.53	-17.7	-8.7	19	-0.3	-0.2
4E	-0.48	-16.4	-7.5	54	-0.9	-0.4
2hp	-0.69	-21.6	-12.7	29	-0.6	-0.4
3hp	-0.37	-13.7	-4.7	29	-0.4	-0.1
2h	-0.69	-21.6	-12.7	95	-2.1	-1.2
3h	-0.37	-13.7	-4.7	95	-1.3	-0.4
5	0.40	5.5	14.4	95	0.5	1.4
6	-0.29	-11.7	-2.7	95	-1.1	-0.3
5E	0.61	10.7	19.6	27	0.3	0.5
6E	-0.43	-15.2	-6.2	27	-0.4	-0.2
5T	-	1.4	3.6	122	0.2	0.4
6T	-	-2.9	-0.7	122	-0.4	-0.1



Loadcase BT

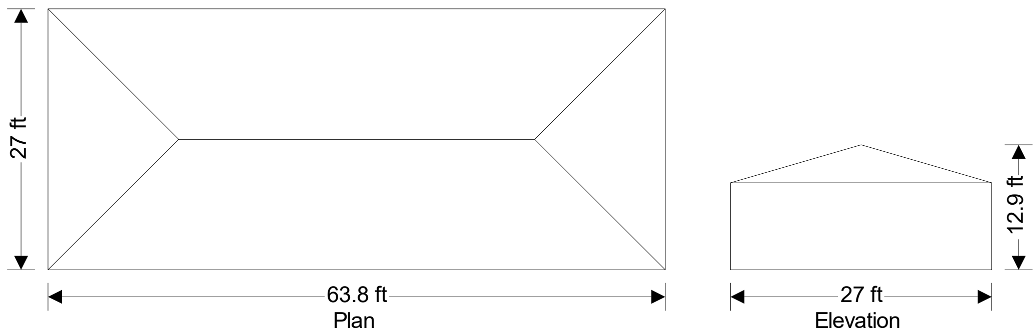
Chapter 1: Design Criteria Cont'd...

C.2 WIND LOAD

In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.16



Building data

Type of roof	Hipped
Length of building	b = **63.81** ft
Width of building	d = **27.00** ft
Height to eaves	H = **9.00** ft
Pitch of main slope	α_0 = **16.3** deg
Pitch of gable slope	α_{90} = **16.3** deg
Mean height	h = **10.97** ft
End zone width	a = max(min(0.1×min(b, d), 0.4×h), 0.04×min(b, d), 3ft) = **3.00** ft

General wind load requirements

Basic wind speed	V = **122.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.83**
Ground elevation above sea level	z_{gl} = **87** ft
Ground elevation factor	K_e = exp(-0.0000362 × z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	$q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **26.6** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **26.64** psf

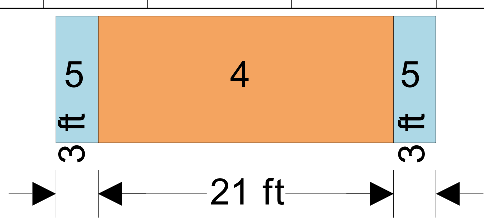
Chapter 1: Design Criteria Cont'd...

Equations used in tables

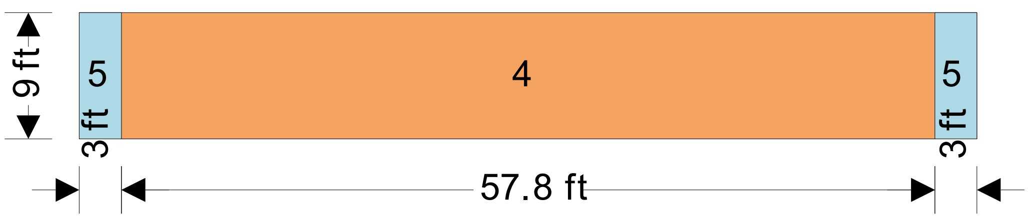
Net pressure $p = q_h \times (GC_p - GC_{pi})$

Components and cladding pressures - Wall (Table 30.3-1)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	-	-	10.0	1.00	-1.10	31.4	-34.1
50 sf	4	-	-	50.0	0.88	-0.98	28.1	-30.8
200 sf	4	-	-	200.0	0.77	-0.87	25.3	-28.0
>500 sf	4	-	-	500.1	0.70	-0.80	23.4	-26.1
<=10 sf	5	-	-	10.0	1.00	-1.40	31.4	-42.1
50 sf	5	-	-	50.0	0.88	-1.15	28.1	-35.5
200 sf	5	-	-	200.0	0.77	-0.94	25.3	-29.8
>500 sf	5	-	-	500.1	0.70	-0.80	23.4	-26.1



Elevation of gable wall



Elevation of side wall

Components and cladding pressures - Roof (Figure Figure 30.3-2E)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	$+GC_p$	$-GC_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1m	-	-	10.0	0.70	-1.30	23.4	-39.4
20 sf	1m	-	-	20.0	0.58	-1.30	20.2	-39.4
50 sf	1m	-	-	50.0	0.42	-1.13	16.0 #	-34.9
>100 sf	1m	-	-	100.1	0.30	-1.00	12.8 #	-31.4
<=10 sf	1g	-	-	10.0	0.70	-1.30	23.4	-39.4
20 sf	1g	-	-	20.0	0.58	-1.30	20.2	-39.4
50 sf	1g	-	-	50.0	0.42	-1.13	16.0 #	-34.9
>100 sf	1g	-	-	100.1	0.30	-1.00	12.8 #	-31.4

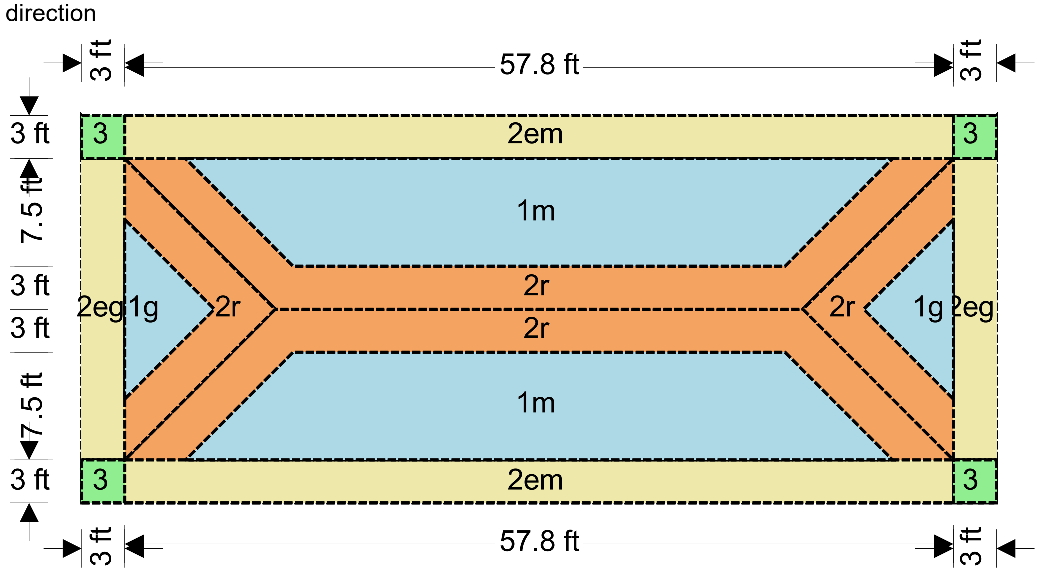
Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 1: Design Criteria Cont'd...

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft²)	+GC$_p$	-GC$_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	2em	-	-	10.0	0.70	-1.80	23.4	-52.7
50 sf	2em	-	-	50.0	0.42	-1.42	16.0 #	-42.7
100 sf	2em	-	-	100.0	0.30	-1.26	12.8 #	-38.4
>200 sf	2em	-	-	200.1	0.30	-1.10	12.8 #	-34.1
<=10 sf	2eg	-	-	10.0	0.70	-1.80	23.4	-52.7
50 sf	2eg	-	-	50.0	0.42	-1.42	16.0 #	-42.7
100 sf	2eg	-	-	100.0	0.30	-1.26	12.8 #	-38.4
>200 sf	2eg	-	-	200.1	0.30	-1.10	12.8 #	-34.1
<=10 sf	2r	-	-	10.0	0.70	-2.40	23.4	-68.7
50 sf	2r	-	-	50.0	0.42	-1.81	16.0 #	-53.0
100 sf	2r	-	-	100.0	0.30	-1.55	12.8 #	-46.2
>200 sf	2r	-	-	200.1	0.30	-1.30	12.8 #	-39.4
<=10 sf	3	-	-	10.0	0.70	-1.80	23.4	-52.7
50 sf	3	-	-	50.0	0.42	-1.42	16.0 #	-42.7
100 sf	3	-	-	100.0	0.30	-1.26	12.8 #	-38.4
>200 sf	3	-	-	200.1	0.30	-1.10	12.8 #	-34.1

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction



Plan on roof

Chapter 1: Design Criteria Cont'd...

D.SEISMIC FORCE

In accordance with ASCE 7-16

Tedds calculation version 3.1.05

Site parameters

Site class	D, Soil properties not known
Mapped acceleration parameters (Section 11.4.2)	
at short period	S_S = **2.31**
at 1 sec period	S_1 = **1.05**
Site coefficient	
at short period (Table 11.4-1)	F_a = **1.200**
at 1 sec period (Table 11.4-2)	F_v = **1.700**

Spectral response acceleration parameters

at short period (Eq. 11.4-1)	$S_{MS} = F_a \times S_S$ = **2.772**
at 1 sec period (Eq. 11.4-2)	$S_{M1} = F_v \times S_1$ = **1.785**

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3)	$S_{DS} = 2 / 3 \times S_{MS}$ = **1.848**
at 1 sec period (Eq. 11.4-4)	$S_{D1} = 2 / 3 \times S_{M1}$ = **1.190**

Seismic design category

Seismic risk category	II
Seismic design category (Sect. 11.6)	E

Approximate fundamental period

Height above base to highest level of building	h_n = **9** ft
From Table 12.8-2:	
Structure type	All other systems
Building period parameter C_t	C_t = **0.02**
Building period parameter x	x = **0.75**
Approximate fundamental period (Eq 12.8-7)	$T_a = C_t \times (h_n)^x \times 1\text{sec} / (1\text{ft})^x$ = **0.104** sec
Building fundamental period (Sect 12.8.2)	$T = T_a$ = **0.104** sec
Long-period transition period	T_L = **12** sec
Limiting period	$T_S = S_{D1} / S_{DS} \times 1$ sec = **0.644** sec

Alternative design spectral response parameter at short period for seismic response coefficient (Sect. 12.8.1.3)

Design spectral resp. accel. params. at short per.	$S_{DSalt} = \max(1.0, 0.7 \times S_{DS})$ = **1.29**

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1)	A. Bearing_Wall_Systems 15. Light-frame (wood) walls sheathed with wood structural panels
Response modification factor (Table 12.2-1)	R = **6.5**
Seismic importance factor (Table 1.5-2)	I_e = **1.000**
Seismic response coefficient (Sect 11.4.8)	
Calculated (Eq 12.8-2)	$C_{s\_calc} = S_{DSalt} / (R / I_e)$ = **0.1990**
Minimum:	

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	2.31	S_{D1} :	N/A
S_1 :	1.05	T_L :	12
F_a :	1.2	PGA :	1.03
F_v :	N/A	PGA_M :	1.236
S_{MS} :	2.772	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.848	C_v :	1.5

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Mon Aug 25 2025

Date Source: USGS Seismic Design Maps

Chapter 1: Design Criteria Cont'd...

Eq.12.8-5	C_{s_min1} = max(0.044 × S_{DSalt} × I_e,0.01) = **0.0569**
Eq 12.8-6 (where S_1 >= 0.6)	C_{s_min2} = (0.5 × S_1) / (R / I_e) = **0.0808**
	C_{s_min} = **0.0808**
Seismic response coefficient	C_s = **0.1990**

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 2: Lateral Load Take-Down

2.1 Wind Load Calculation

N-S Direction :

At Roof Level

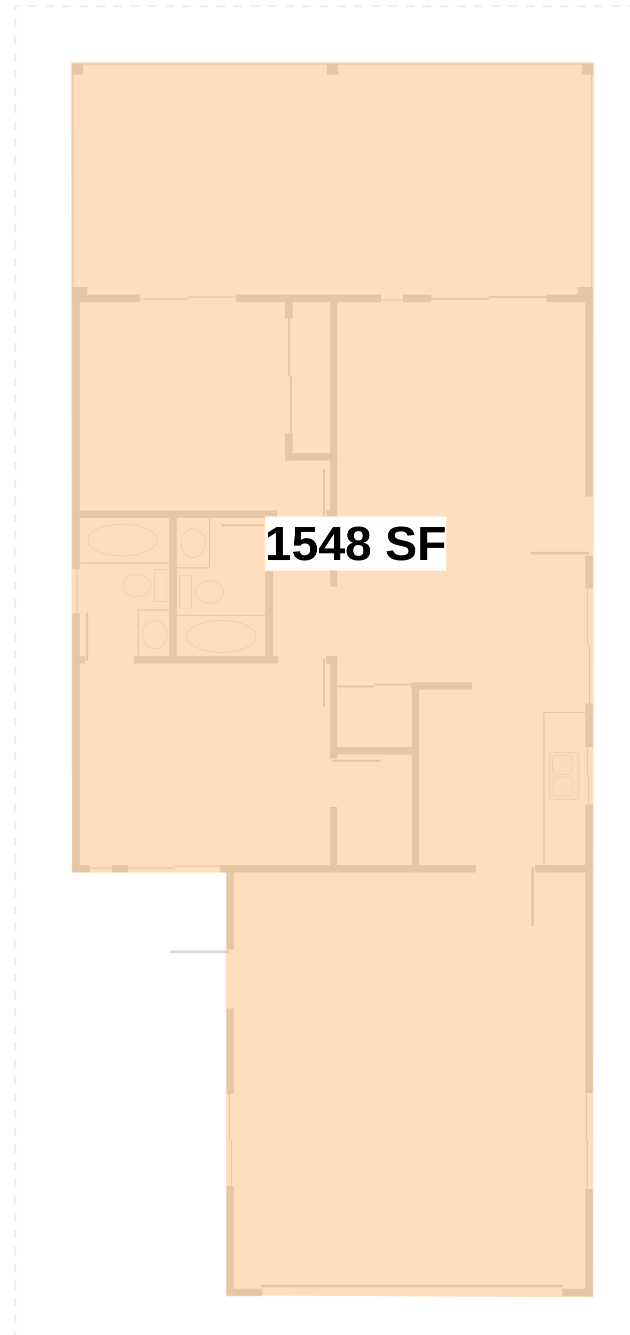
Wind Resisting Trib Area of Wall ($A_{NS,W}$) :	272.00 sq.ft	
Wind Resisting Trib Area of Roof ($A_{NS,R}$) :	232.00 sq.ft	
Windward Wind Pressure on Wall (P_w) :	17.40 psf	***(Refer Design Criteria For Value)***
Leeward Wind Pressure on Wall (P_L) :	-5.75 psf	
Wind Pressure on Roof Projection (P_R) :	10.00 psf	
Wind Force on Building @ Roof Level (1.0W) : $P^*A_{NS,W} + A_{NS,R}{}^*P_R$:	**8.62 kip**	

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 2: Lateral Load Take-Down Cont'd...

2.2 - Area Plan



- Roof Area @Eave Level = 1548 sq.ft

Chapter 2: Lateral Load Take-Down Cont'd...

2.3 Seismic Load Calculation

Area of	:	1548.00 sq.ft
Seismic weight due to dead load	:	20.0 psf
Seismic Weight of Roof	:	30.96 kips
Seismic Response Coefficient (Cs)	:	0.199
Seismic base shear	:	6.16 kips

Hence, Wind Force Governs Over Seimic.

Chapter 3: Lateral Elements Design

4.1 Segmented Shear wall - Design Basis

Shear walls with an aspect ratio less than or equal to 2:1 need to be designed for the applied force.
The shear walls with aspect ratio more than 2:1 and less than 3.5:1 needs to be designed for The adjusted shear load is described in section 4.3.4/SDPWS_2015 edition (refer to the following snip).
If the aspect ratio of the wall is more than 3.5:1, that segment of the wall shall not be considered in lateral
Load-resisting system.

Refer to Following Snap Taken From SDPWS_2015

4.3.4 Shear Wall Aspect Ratios and Capacity Adjustments

4.3.4.1 The size and shape of shear walls shall be limited to the aspect ratios in Table 4.3.4.

4.3.4.2 For wood structural panel shear walls with aspect ratios (h/b_s) greater than 2:1, the nominal shear capacity shall be multiplied by the Aspect Ratio Factor (WSP) = $1.25 - 0.125h/b_s$. For structural fiberboard shear walls with aspect ratios (h/b_s) greater than 1:1, the nominal shear capacity shall be multiplied by the Aspect Ratio Factor (fiberboard) = $1.09 - 0.09\ h/b_s$.

4.3.4.3 Aspect Ratio of Perforated Shear Wall Segments: The aspect ratio limitations of Table 4.3.4 shall apply to perforated shear wall segments within a perforated shear wall as illustrated in Figure 4C. Portions of walls with aspect ratios exceeding 3.5:1 shall not be considered in the sum of shear wall segments. In the design of perforated shear walls, the length of each perforated shear wall segment with an aspect ratio greater than 2:1 shall be multiplied by $2b_s/h$ for the purposes of determining L_i and ΣL_i. The provisions of Section 4.3.4.2 and the exceptions to Section 4.3.3.4.1 shall not apply to perforated shear wall segments.

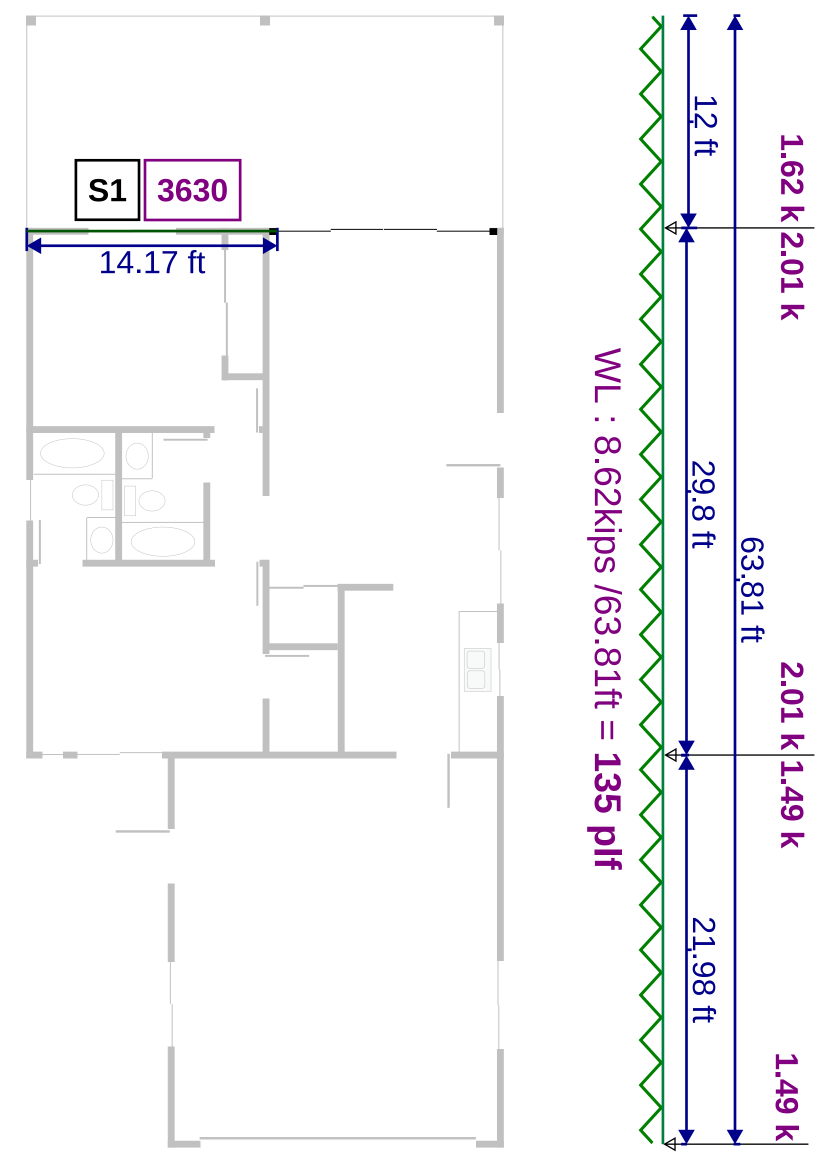
Refer to the below pages for the Shear Wall Force Calculation.

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 3: Lateral Elements Design Cont'd...

3.2- Shear Wall Layout



ROOF PLAN

*Refer Attached **Below** For detail Shear Wall Design Calculation*

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 3: Lateral Elements Design Cont'd...

3.2.1-Shear Wall Design Summary

Wood Shear Wall Schedule															
SR. NO.	TAG	HOLD DOWN FORCE (KIPS)	TAG	SHEATHING	FASTENER	FASTENER SPACING		FRAMING MEMBER	END POST	GRADE	HOLD DOWN ASSEMBLY WITH SDS SCREWS	ANCHORS	HOLDDOWN MARK	REMARK
						PANEL EDGE	FIELD							
1	S1	3.54	SW1	15/32" THK STRUCTURAL 1 - ONE SIDE	8d NAIL	4" OC	12" OC	2X @ 16" OC	(2) 2X4	DF#2	HDU4-SDS2.5	5/8" Ø THREADED ROD THRU FOOTING,BOLTED AT BOTTOM WITH 3"X3"X1/2" BRNG PLATE	H1	-

*Refer Attached **Below Pages** For detail Shear Wall Design Calculation*

Chapter 3: Lateral Elements Design Cont'd...

3.3 Sill Anchorage Design for in-plane shear

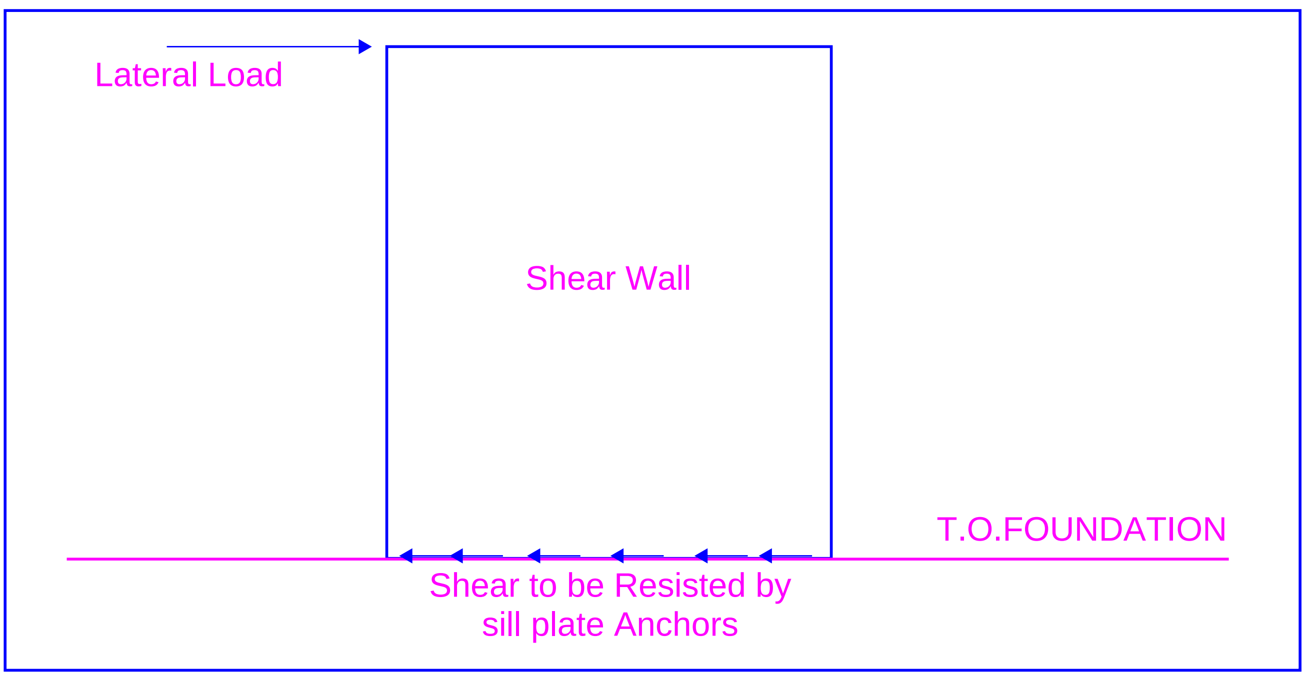
Maximum Shear load in SW = 256 lb/ft in S1 for Seismic Loading.

Shear force on anchors spaced at 36" OC = 0.77 kips (LRFD) < 1.65 kips Hence OK

HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 5/8"Ø Anchors with 7" embed in to Foundation/SOG.

Foundation Is Working Fine

Refer following attached detail report



www.hilti.com

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4IN WALL- SILL ANCHOR CHECK	Date:	7/25/2025
Fastening point:			

Specifier's comments:

1 Input data

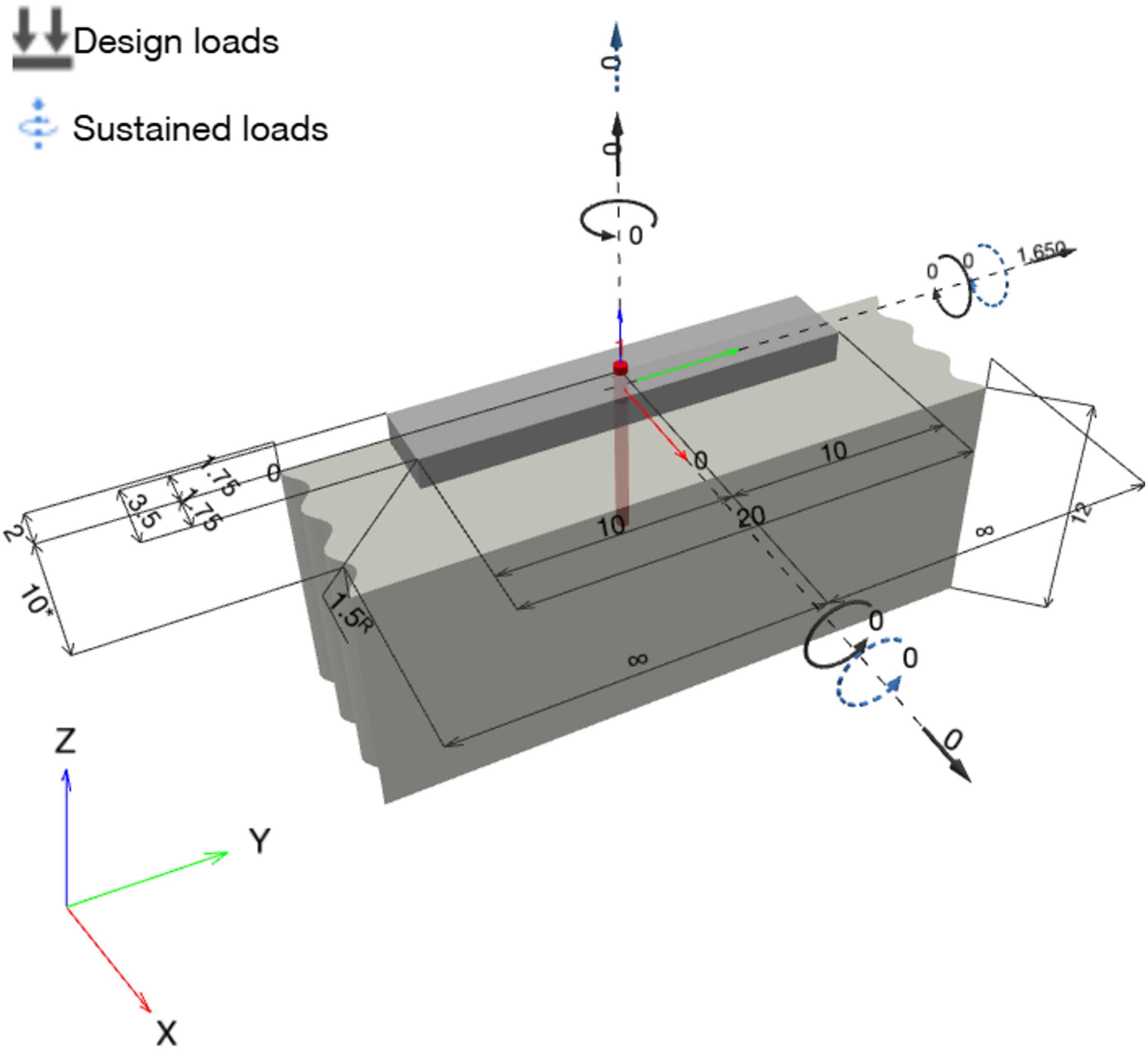
Anchor type and diameter:	**HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 5/8**
Item number:	2198026 HAS-V-36 5/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Specification text:	Hilti ∅ 5/8 in HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) with 7 in nominal embedment depth per ICC-ES ESR-4868 , Hammer drill bit installation per MPII,
Effective embedment depth:	$h_{ef,act}$ = 7.000 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F1554 Grade 36
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2024 | 11/1/2026
Proof:	Design Method ACI 318-19 / Chem
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 1.500 in.
Anchor plate^R :	l_x x l_y x t = 3.500 in. x 20.000 in. x 1.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 12.000 in., Temp. short/long: 32/32 °F
Installation:	**Hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

www.hilti.com

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4IN WALL- SILL ANCHOR CHECK	Date:	7/25/2025
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]

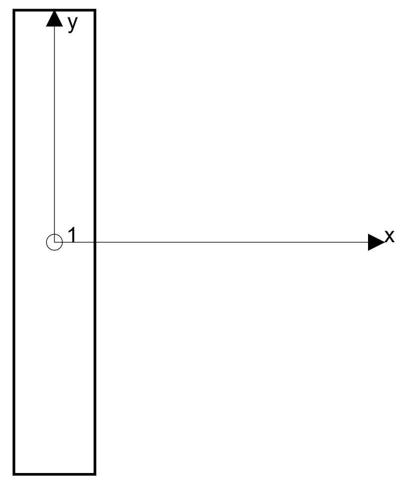


Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4IN WALL- SILL ANCHOR CHECK	Date:	7/25/2025
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	S9 - SW Chord Uplift	$N = 0; V_x = 0; V_y = 1{,}650;$ $M_x = 0; M_y = 0; M_z = 0;$	yes	100

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,650	0	1,650

Max. concrete compressive strain: - [‰]
Max. concrete compressive stress: - [psi]
Resulting tension force in (x/y)=(-/-): 0 [lb]
Resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	**Capacity ϕ N_n [lb]**	**Utilization $\beta_N = N_{ua}/\phi$ N_n**	**Status**
Steel Strength*	N/A	N/A	N/A	N/A
Bond Strength**	N/A	N/A	N/A	N/A
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	N/A	N/A	N/A	N/A

* highest loaded anchor   **anchor group (anchors in tension)

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4IN WALL- SILL ANCHOR CHECK	Date:	7/25/2025
Fastening point:			

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,650	3,067	54	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	1,650	9,535	18	OK
Concrete edge failure in direction x-**	1,650	1,661	100	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-4868
ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.23	58,000	0.600

Calculations

$V_{sa,eq}$ [lb]
4,719

Results

$V_{sa,eq}$ [lb]	ϕ $_{steel}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
4,719	0.650	3,067	1,650

www.hilti.com

Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	4IN WALL- SILL ANCHOR CHECK	Date:	7/25/2025
Fastening point:			

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cp} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ed,N} \ \psi_{c,N} \ \psi_{cp,N} \ N_b \right]$ ACI 318-19 Eq. (17.7.3.1a)

$\phi \ V_{cp} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \ \lambda_a \ \sqrt{f'_c} \ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
2	7.000	2.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
15.190	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
252.00	441.00	0.757	1.000	15,742

Results

V_{cp} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \ V_{cp}$ [lb]	V_{ua} [lb]
13,622	0.700	1.000	1.000	9,535	1,650

4.3 Concrete edge failure in direction x-

$V_{cb} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ed,V} \; \psi_{c,V} \; \psi_{h,V} \; \psi_{parallel,V} \; V_b$	ACI 318-19 Eq. (17.7.2.1a)
$\phi \; V_{cb} \geq V_{ua}$	ACI 318-19 Table 17.5.2
A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*	
$A_{Vc0} = 4.5 \; c_{a1}^2$	ACI 318-19 Eq. (17.7.2.1.3)
$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$	ACI 318-19 Eq. (17.7.2.4.1b)
$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$	ACI 318-19 Eq. (17.7.2.6.1)
$V_b = \left(7 \left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} \; c_{a1}^{1.5}$	ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	$\psi_{c,V}$	h_a [in.]	l_e [in.]
2.000	-	1.000	12.000	5.000

λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
1.000	0.625	2,500	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
18.00	18.00	1.000	1.000	1,186

Results

V_{cb} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \; V_{cb}$ [lb]	V_{ua} [lb]
2,372	0.700	1.000	1.000	1,661	1,650

*Anchor row defined by: Anchor 1; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

6 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 1.500 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) 5/8

Item number: 2198026 HAS-V-36 5/8"x10" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

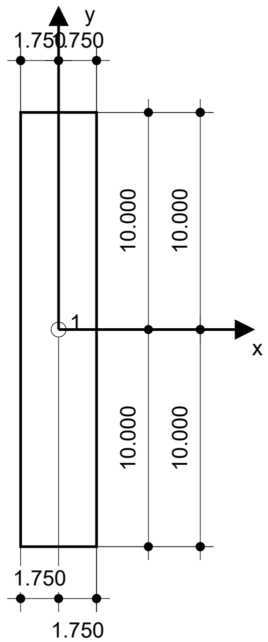
Hole depth in the base material: 7.000 in.

Minimum thickness of the base material: 8.500 in.

Hilti ∅ 5/8 in HIT-HY 200 V3 + HAS-V-36 (ASTM F1554 Gr.36) with 7 in nominal embedment depth per ICC-ES ESR-4868 , Hammer drill bit installation per MPII

6.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	0.000	-0.000	2.000	10.000	-	-

7 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

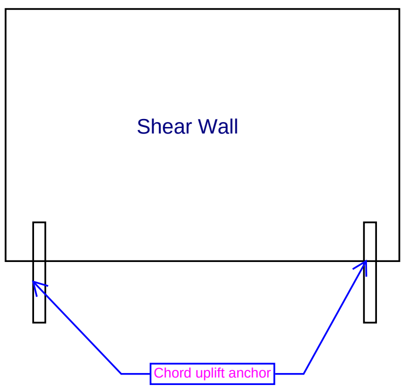
Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 3: Lateral Elements Design Cont'd...

3.4 Uplift Anchor Check (5/8" Dia)

Maximum Chord Uplift forces for rest existing shear wall S1 is as below:



Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1	-5974;	0;	81;	81;	0;	27;	0;	0;
Ch2	5974;	0;	81;	81;	0;	27;	0;	0;

Uplift Load Case (LRFD) 0.9DL-1.0EQ = 5901 lbs of Uplift

Washer Plate Bearing Capacity Check

Size of Plate (L X B X T) : 3" X 3" X 1/2" _ A36 Washer Plate
Considering A36 Plate, Capacity of plate in bearing/Shear = 0.6fy*A
= 0.6*36*0.5*3
= 32.4 kips (LRFD)

Demand (LRFD) = 5.901 kips << 32.4 kips

UC = 0.18 << 1.0

Hence OK

Tension Capacity Check for Anchor

Anchor : 5/8" Dia through A36 anchor bolt
Capacity of anchor bolt intension =Φ * 0.75 * Fu * Ag) (LRFD)
= 0.9 * (0.75 * 58 * (0.785 *$5/8^2$))
= 12 kips (LRFD)

Demand (LRFD) =5.901 kips <<12 kips

UC = 0.5 << 1.0

Hence OK

Chapter 3: Lateral Elements Design Cont'd...

S1

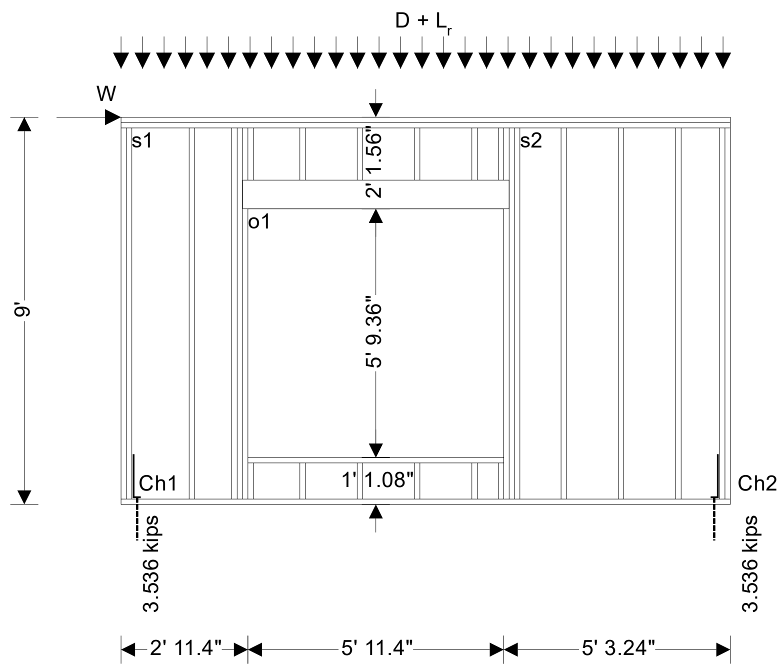
In accordance with NDS2018 and SDPWS2015 allowable stress design and the perforated shear wall method

Tedds calculation version 1.2.13

Panel details

Structural I wood panel sheathing on one side

Panel height	h = **9** ft
Panel length	b = **14.17** ft



Panel opening details

Width of opening	w_{o1} = **5.95** ft
Height of opening	h_{o1} = **5.78** ft
Height to underside of lintel over opening	l_{o1} = **6.87** ft
Position of opening	P_{o1} = **2.95** ft
Total area of wall	$A = h \times b - w_{o1} \times h_{o1}$ = **93.139** ft^2

Panel construction

Nominal stud size	2" x 4"
Dressed stud size	1.5" x 3.5"
Cross-sectional area of studs	A_s = **5.25** in^2
Stud spacing	s = **16** in
Nominal end post size	2 x 2" x 4"
Dressed end post size	2 x 1.5" x 3.5"
Cross-sectional area of end posts	A_e = **10.5** in^2
Hole diameter	Dia = **1** in
Net cross-sectional area of end posts	A_{en} = **7.5** in^2
Nominal collector size	2 x 2" x 4"

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 3: Lateral Elements Design Cont'd...

Dressed collector size	2 x 1.5" x 3.5"
Service condition	Dry
Temperature	100 degF or less
Vertical anchor stiffness	k_a = **30000** lb/in

From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)

Species, grade and size classification	Douglas Fir-Larch, no.2 grade, 2" & wider
Specific gravity	G = **0.50**
Tension parallel to grain	F_t = **575** lb/in^2
Compression parallel to grain	F_c = **1350** lb/in^2
Compression perpendicular to grain	F_{c_perp} = **625** lb/in^2
Modulus of elasticity	E = **1600000** lb/in^2
Minimum modulus of elasticity	E_{min} = **580000** lb/in^2

Sheathing details

Sheathing material	15/32" wood panel structural I oriented strandboard sheathing
Fastener type	8d common nails at 4"centers

From SDPWS Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Wood-based Panels

Nominal unit shear capacity for seismic design	v_s = min(860 plf, 1740 plf) = **860** lb/ft
Nominal unit shear capacity for wind design	v_w = min(1205 plf, 2435 plf) = **1205** lb/ft
Apparent shear wall shear stiffness	G_a = **18** kips/in

Loading details

Dead load acting on top of panel	D = **40** lb/ft
Roof live load acting on top of panel	L_r = **40** lb/ft
Self weight of panel	S_{wt} = **9** lb/ft^2
In plane wind load acting at head of panel	W = **3630** lbs
Wind load serviceability factor	f_{Wserv} = **0.60**

From IBC 2018 cl.1605.3.1 Basic load combinations

Load combination no.1	D + 0.6W
Load combination no.2	D + 0.7E
Load combination no.3	D + 0.45W + 0.75L_f + 0.75(L_r or S or R)
Load combination no.4	D + 0.525E + 0.75L_f + 0.75S
Load combination no.5	0.6D + 0.6W
Load combination no.6	0.6D + 0.7E

Adjustment factors

Load duration factor – Table 2.3.2	C_D = **1.60**
Size factor for tension – Table 4A	C_{Ft} = **1.50**
Size factor for compression – Table 4A	C_{Fc} = **1.15**
Wet service factor for tension – Table 4A	C_{Mt} = **1.00**
Wet service factor for compression – Table 4A	C_{Mc} = **1.00**
Wet service factor for modulus of elasticity – Table 4A	C_{ME} = **1.00**
Temperature factor for tension – Table 2.3.3	C_{tt} = **1.00**
Temperature factor for compression – Table 2.3.3	C_{tc} = **1.00**
Temperature factor for modulus of elasticity – Table 2.3.3	

Chapter 3: Lateral Elements Design Cont'd...

	C_{tE} = **1.00**
Incising factor – cl.4.3.8	C_i = **1.00**
Buckling stiffness factor – cl.4.4.2	C_T = **1.00**
Bearing area factor - cl. 3.10.4	C_b = **1.0**
Adjusted modulus of elasticity	$E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi
Critical buckling design value	$F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **501** psi
Reference compression design value	$F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2484** psi
For sawn lumber	c = **0.8**
Column stability factor – eqn.3.7-1	$C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$
	= **0.19**
From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios	
Maximum shear wall aspect ratio	3.5
Perforated wall length	b_1 = **2.95** ft
Shear wall aspect ratio	h / b_1 = **3.051**
Perforated wall length	b_2 = **5.27** ft
Shear wall aspect ratio	h / b_2 = **1.708**
Shear capacity adjustment factor – cl.4.3.3.5	
Sum of perforated shear wall lengths	$\Sigma L_i = b_1 \times 2 \times b_s / h + b_2$ = **7.204** ft
Total length of perforated shear wall	$L_{tot} = b_1 + w_{o1} + b_2$ = **14.17** ft
Total area of openings	$A_o = w_{o1} \times h_{o1}$ = **34.39** ft^2
Sheathing area ratio (eqn. 4.3-6)	$r = 1 / (1 + A_o /(h \times \Sigma L_i))$ = **0.653**
Shear capacity adjustment factor (eqn. 4.3-5)	C_o = **0.759**
Perforated shear wall capacity	
Maximum shear force under wind loading	$V_{w\_max} = 0.6 \times W$ = **2.178** kips
Shear capacity for wind loading	$V_w = v_w \times C_o \times \Sigma L_i / 2$ = **3.295** kips
	V_{w_max} / V_w = **0.661**
	PASS - Shear capacity for wind load exceeds maximum shear force
Chord capacity for chords 1 and 2	
Load combination 5	
Shear force for maximum tension	$V = 0.6 \times W$ = **2.178** kips
Axial force for maximum tension	$P = (0.6 \times (D + S_{wt} \times h)) \times s / 2$ = **0.048** kips
Maximum tensile force in chord	$T = V \times h / (C_o \times \Sigma L_i) - P$ = **3.536** kips
Maximum applied tensile stress	$f_t = T / A_{en}$ = **472** lb/in^2
Design tensile stress	$F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2
	f_t / F_t' = **0.342**
	PASS - Design tensile stress exceeds maximum applied tensile stress
Load combination 1	
Shear force for maximum compression	$V = 0.6 \times W$ = **2.178** kips
Axial force for maximum compression	$P = ((D + S_{wt} \times h)) \times s / 2$ = **0.081** kips
Maximum compressive force in chord	$C = V \times h / (C_o \times \Sigma L_i) + P$ = **3.665** kips
Maximum applied compressive stress	$f_c = C / A_e$ = **349** lb/in^2
Design compressive stress	$F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **478** lb/in^2
	f_c / F_c' = **0.730**

Chapter 3: Lateral Elements Design Cont'd...

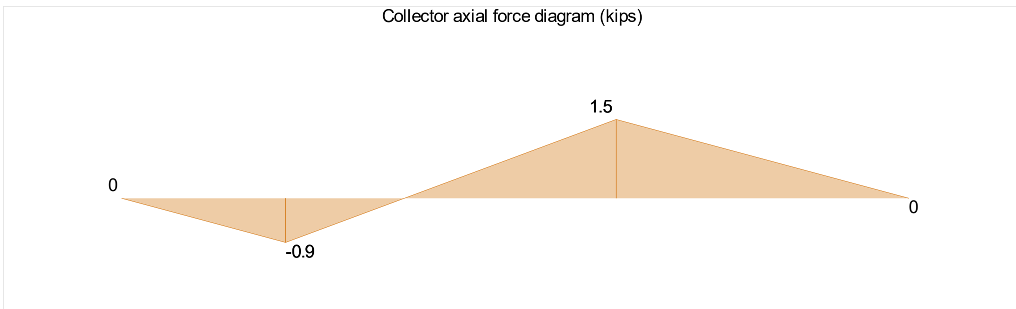
PASS - Design compressive stress exceeds maximum applied compressive stress

Design bearing compr. stress, bottom plate $F_{c\_perp}' = F_{c\_perp} \times C_{Mc} \times C_{tc} \times C_i \times C_b$ = **625** lb/in^2

f_c / F_{c_perp}' = **0.559**

PASS - Design bearing compressive stress exceeds maximum applied bearing compressive stress

Collector capacity



Maximum shear force on wall $V_{max} = V_{w\_max}$ = **2.178** kips

Uniform shear applied to wall $v_a = V_{max} / (C_o \times \Sigma L_i)$ = **398.3** plf

Shear resisted by wall segments $v_b = v_a \times b / (b_1 + b_2)$ = **686.6** plf

Maximum force in collector P_{coll} = **1.519** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s)$ = **145** lb/in^2

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1380** lb/in^2

f_t / F_t' = **0.105**

PASS - Design tensile stress exceeds maximum applied tensile stress

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s)$ = **145** lb/in^2

Column stability factor C_P = **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2484** lb/in^2

f_c / F_c' = **0.058**

PASS - Design compressive stress exceeds maximum applied compressive stress

Hold down force

Chord 1 T_1 = **3.536** kips

Chord 2 T_2 = **3.536** kips

Chord reactions by load type

Chord	$W_{ch[i]R}$ (lbs)	$E_{q\_ch[i]R}$ (lbs)	$D_{C\_ch[i]R}$ (lbs)	$D_{T\_ch[i]R}$ (lbs)	$L_{f\_ch[i]R}$ (lbs)	$L_{r\_ch[i]R}$ (lbs)	$S_{ch[i]R}$ (lbs)	$R_{ch[i]R}$ (lbs)
Ch1	-5974;	0;	81;	81;	0;	27;	0;	0;
Ch2	5974;	0;	81;	81;	0;	27;	0;	0;

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 4: Gravity Elements Design

4.1- Gravity Framing Layout

LEGEND	
ATT##	Design ID
	Wood Beam
	Wood Column

ROOF PLAN

Refer Attached ***Appendix-A.2*** *For Detail Gravity Member Design Calculation*

Chapter 4: Gravity Elements Design Cont'd...

4.1.1 Gravity Element Design Summary

Level			
Member Name	**Results**	**Current Solution**	**Comments**
ATT#1	Passed	1 piece(s) 4 x 10 DF No.1	
ATT#2	Passed	1 piece(s) 4 x 4 DF No.1	

Note:

Refer Below Pages for the detailed design calculation

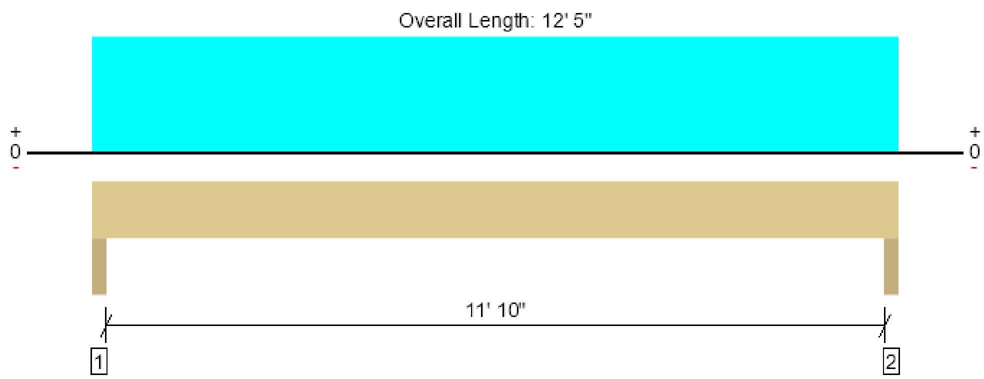
FORTEWEB

MEMBER REPORT

PASSED

Level, ATT#1

1 piece(s) 4 x 10 DF No.1



Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1156 @ 2"	7656 (3.50")	Passed (15%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	958 @ 1' 3/4"	4856	Passed (20%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	3398 @ 6' 2 1/2"	6239	Passed (54%)	1.25	1.0 D + 1.0 Lr (All Spans)
Vert Live Load Defl. (in)	0.098 @ 6' 2 1/2"	0.403	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Vert Total Load Defl. (in)	0.228 @ 6' 2 1/2"	0.604	Passed (L/637)	--	1.0 D + 1.0 Lr (All Spans)
Lat Member Reaction (lbs)	425 @ 12' 3"	N/A	Passed (N/A)	1.60	1.0 D + 0.6 W
Lat Shear (lbs)	396 @ 7"	6216	Passed (6%)	1.60	1.0 D + 0.6 W
Lat Moment (Ft-lbs)	1284 @ mid-span	3324	Passed (39%)	1.60	1.0 D + 0.6 W
Lat Deflection (in)	0.420 @ mid-span	1.208	Passed (L/345)	--	1.0 D + 0.6 W
Bi-Axial Bending	0.70	1.00	Passed (70%)	1.60	1.0 D + 0.45 W + 0.75 L + 0.75 Lr

Member Length : 12' 5"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Wall deflection criteria: TL (L/120)
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Factored	
1 - Trimmer - DF	3.50"	3.50"	1.50"	659	497	1156	None
2 - Trimmer - DF	3.50"	3.50"	1.50"	659	497	1156	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	12' 5" o/c	
Bottom Edge (Lu)	12' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Lateral Connections

Supports	Stud Size	Stud Material	Connector	Type/Model	Quantity	Nailing
Left	2X	Douglas Fir-Larch	Nails	10d (0.128" x 3") (End)	5	
Right	2X	Douglas Fir-Larch	Nails	10d (0.128" x 3") (End)	5	

Vertical Loads	Location	Tributary Width	Dead (0.90)	Roof Live (1.25)	Comments
0 - Self Weight (PLF)	0 to 12' 5"	N/A	8.2	--	
1 - Uniform (PSF)	0 to 12' 5"	4'	20.0	20.0	ROOF LOAD
2 - Uniform (PSF)	0 to 12' 5"	2'	9.0	--	WALL LOAD

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	4' 6"	26.0	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (10' 3 5/8"), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (122), Risk Category(II), Wind Zone (4), GCpi (+/- 0.18), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

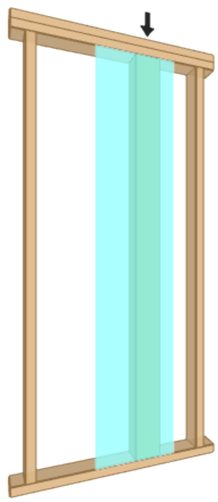
MEMBER REPORT

PASSED

Level, ATT#2

1 piece(s) 4 x 4 DF No.1

Wall Height: 9' Member Height: 8' 7 1/2" Tributary Width: 1'



Drawing is Conceptual

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	30	50	Passed (59%)	--	--
Compression (lbs)	1156	6686	Passed (17%)	1.25	1.0 D + 1.0 Lr
Plate Bearing (lbs)	1156	7656	Passed (15%)	--	1.0 D + 1.0 Lr
Lateral Reaction (lbs)	71	--	--	1.60	1.0 D + 0.6 W
Lateral Shear (lbs)	66	2352	Passed (3%)	1.60	1.0 D + 0.6 W
Lateral Moment (ft-lbs)	153 @ mid-span	1429	Passed (11%)	1.60	1.0 D + 0.6 W
Total Deflection (in)	0.07 @ mid-span	0.86	Passed (L/1532)	--	1.0 D + 0.6 W
Bending/Compression	0.13	1	Passed (13%)	1.60	1.0 D + 0.6 W

- Wall deflection criteria: TL (L/120)
- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.
- This product has a square cross section. The analysis engine has checked both edge and plank orientations to allow for either installation.

Supports	Type	Material
Top	Dbl 2X	Douglas Fir-Larch
Base	2X	Douglas Fir-Larch

System : Wall
Member Type : Column
Building Code : IBC 2021
Design Methodology : ASD

Max Unbraced Length	Comments
1'	

Lateral Connections				
Supports	**Connector**	**Type/Model**	**Quantity**	**Connector Nailing**
Top	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A
Base	Nails	8d (0.113" x 2 1/2") (Toe)	2	N/A

- Nailed connection at the top of the member is assumed to be nailed through the bottom 2x plate prior to placement of the top 2x of the double top plate assembly.

Vertical Load	Tributary Width	Dead (0.90)	Roof Live (1.25)	Comments
1 - Point (lb)	N/A	659	497	Linked from: Wall: Header, Support 1

Lateral Load	Location	Tributary Width	Wind (1.60)	Comments
1 - Uniform (PSF)	Full Length	1'	27.5	

• ASCE/SEI 7 Sec. 30.4: Exposure Category (B), Mean Roof Height (10' 3 5/8"), Topographic Factor (1.0), Wind Directionality Factor (0.85), Basic Wind Speed (122), Risk Category(II), Wind Zone (4), GCpi (+/- 0.18), Effective Wind Area determined using full member span and trib. width.
• IBC Table 1604.3, footnote f: Deflection checks are performed using 42% of this lateral wind load.

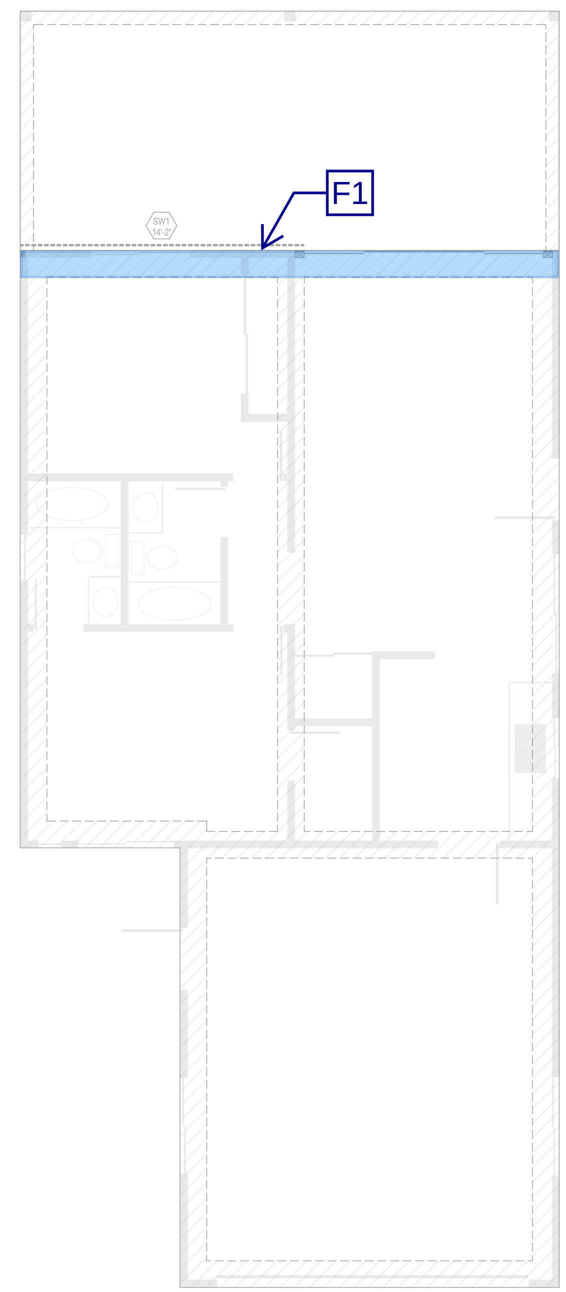
Weyerhaeuser Notes

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 5: Foundation Design

5.1- Foundation Design (Strip Ftg)



Refer Attached Reports For foundation Design Calculation

Project : Brent & Cheryl Stange Residential Alteration

Date :

Chapter 5: Foundation Design Cont'd...

Beam on Elastic Foundation

LIC# : KW-06021401, Build:20.25.07.16 (c) ENERCALC, LLC 1982-2025

DESCRIPTION: F1

CODE REFERENCES

Calculations per ACI 318-14, IBC 2018
Load Combinations Used : ASCE 7-16

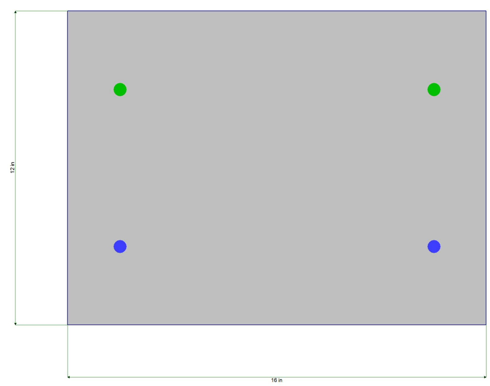
Material Properties

f'c = 2.50 ksi
fr = $f'c^{1/2}$ * 7.50 = 375.0 psi
ψ Density = 145.0 pcf
λ Lt Wt Factor = 1.0
Elastic Modulus = 2,850.0 ksi
Soil Subgrade Modulus = 250.0 psi / (inch deflection)
Load Combination ASCE 7-16

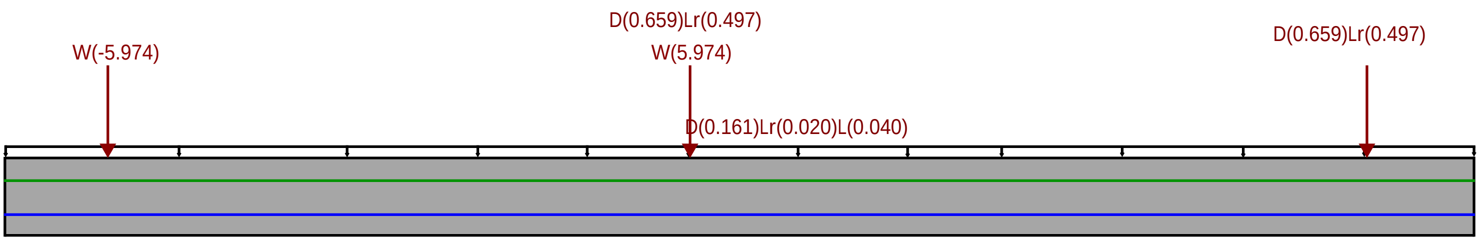
ϕ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # = # 4
Number of Resisting Legs Per Stirrup 2



Beam is supported on an elastic foundation.



16" w x 12" h
Span = 33 ft

Cross Section & Reinforcing Details

Rectangular Section, Width = 16.0 in, Height = 12.0 in
Span #1 Reinforcing....
2-#4 at 3.0 in from Bottom, from 0.0 to 33.0 ft in this span 2-#4 at 3.0 in from Top, from 0.0 to 33.0 ft in this span

Applied Loads

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

Load for Span Number 1
Uniform Load : D = 0.161, Lr = 0.020, L = 0.040 k/ft, Tributary Width = 1.0 ft, (DL)
Point Load : W = -5.974 k @ 3.50 ft
Point Load : D = 0.6590, Lr = 0.4970, W = 5.974 k @ 16.670 ft
Point Load : D = 0.6590, Lr = 0.4970 k @ 29.50 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.392** : 1
Section used for this span **Typical Section**
Mu : Applied 7.462 k-ft
Mn * Phi : Allowable 19.059 k-ft
Load Combination +1.20D+0.50Lr+L+W
Location of maximum on span 16.694 ft
Span # where maximum occurs Span # 1

Maximum Deflection
Max Downward L+Lr+S Deflection 0.000 in
Max Upward L+Lr+S Deflection 0.000 in
Max Downward Total Deflection 0.031 in
Max Upward Total Deflection 0.000 in

Maximum Soil Pressure = **1.113** ksf at 1.83 ft LdComb: +D-0.60W
Allowable Soil Pressure = **1.50** ksf **OK**

Cross Section Strength & Inertia

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Btm Tension	Top Tension	Moment of Inertia (in^4) I gross	Icr - Btm Tension	Icr - Top Tension
Section 1	2-#4 @ d=9", 2-#4 @ d=3"	19.06	19.06	2,304.00	246.17	246.17

Shear Stirrup Requirements

Entire Beam Span Length : Vu < Phi*Vc / 2, Req'd Vs = Not Reqd per 9.6.3.1, use stirrups spaced at 0.000 in

Maximum Forces & Stresses for Load Combinatior

Chapter 5: Foundation Design Cont'd...

Beam on Elastic Foundation

LIC# : KW-06021401, Build:20.25.07.16 (c) ENERCALC, LLC 1982-2025

DESCRIPTION: F1

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft: Mu : Max	Phi*Mnx	Stress Ratio
MAXimum Bending Envelope					
Span # 1	1	16.694	7.46	19.06	0.39
+1.40D					
Span # 1	1	16.694	0.90	19.06	0.05
+1.20D+0.50Lr+1.60L					
Span # 1	1	16.694	1.02	19.06	0.05
+1.20D+1.60L					
Span # 1	1	16.694	0.77	19.06	0.04
+1.20D+1.60Lr+L					
Span # 1	1	16.694	1.56	19.06	0.08
+1.20D+1.60Lr+0.50W					
Span # 1	1	16.694	4.78	19.06	0.25
+1.20D+1.60Lr-0.50W					
Span # 1	1	3.494	2.65	19.06	0.14
+1.20D+L					
Span # 1	1	16.694	0.77	19.06	0.04
+1.20D+0.50W					
Span # 1	1	16.694	3.99	19.06	0.21
+1.20D-0.50W					
Span # 1	1	3.494	2.68	19.06	0.14
+1.20D+0.50Lr+L+W					
Span # 1	1	16.694	7.46	19.06	0.39
+1.20D+0.50Lr+L-W					
Span # 1	1	29.506	1.12	19.06	0.06
+1.20D+L+W					
Span # 1	1	16.694	7.22	19.06	0.38
+1.20D+L-W					
Span # 1	1	23.294	0.97	19.06	0.05
+0.90D+W					
Span # 1	1	16.694	7.02	19.06	0.37
+0.90D-W					
Span # 1	1	23.294	1.07	19.06	0.06
+1.570D+L					
Span # 1	1	16.694	1.01	19.06	0.05
+0.5304D					
Span # 1	1	16.694	0.34	19.06	0.02

Overall Maximum Deflections - Unfactored Lo

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.0309	1.833		0.0000	0.000

Maximum Deflections for Load Combinations - Unfactored

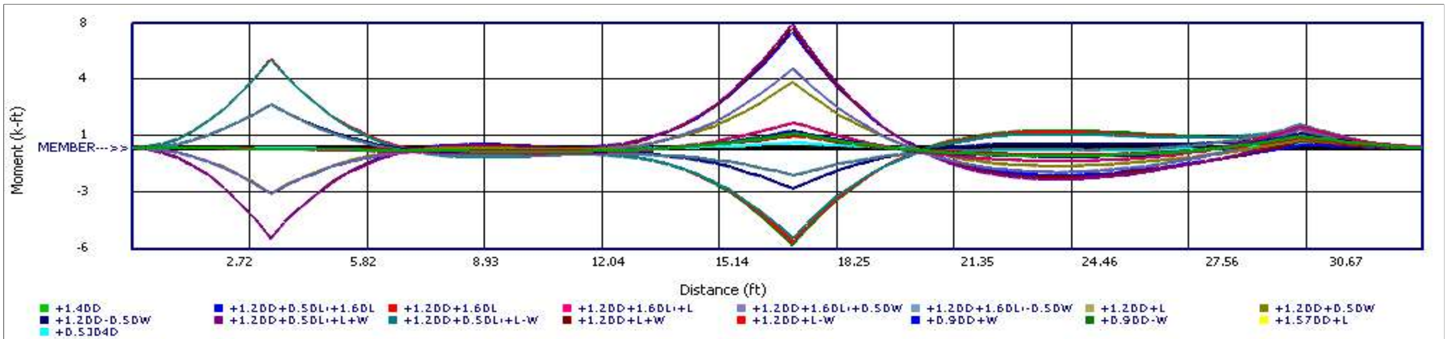
Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
D Only	1	0.0217	30.433	0.0000	0.000
+D+L	1	0.0225	30.433	0.0000	0.000
+D+Lr	1	0.0235	30.433	0.0000	0.000
+D+0.750Lr+0.750L	1	0.0237	30.433	0.0000	0.000
+D+0.750L	1	0.0223	30.433	0.0000	0.000
+D+0.60W	1	0.0306	16.500	0.0000	0.000
+D-0.60W	1	0.0309	1.833	0.0000	0.000
+D+0.750Lr+0.750L+0.450W	1	0.0301	16.500	0.0000	0.000
+D+0.750Lr+0.750L-0.450W	1	0.0290	2.567	0.0000	0.000
+D+0.750L+0.450W	1	0.0289	16.500	0.0000	0.000
+D+0.750L-0.450W	1	0.0287	2.567	0.0000	0.000
+0.60D+0.60W	1	0.0220	16.500	0.0000	0.000
+0.60D-0.60W	1	0.0230	1.833	0.0000	0.000
+0.60D	1	0.0130	30.433	0.0000	0.000

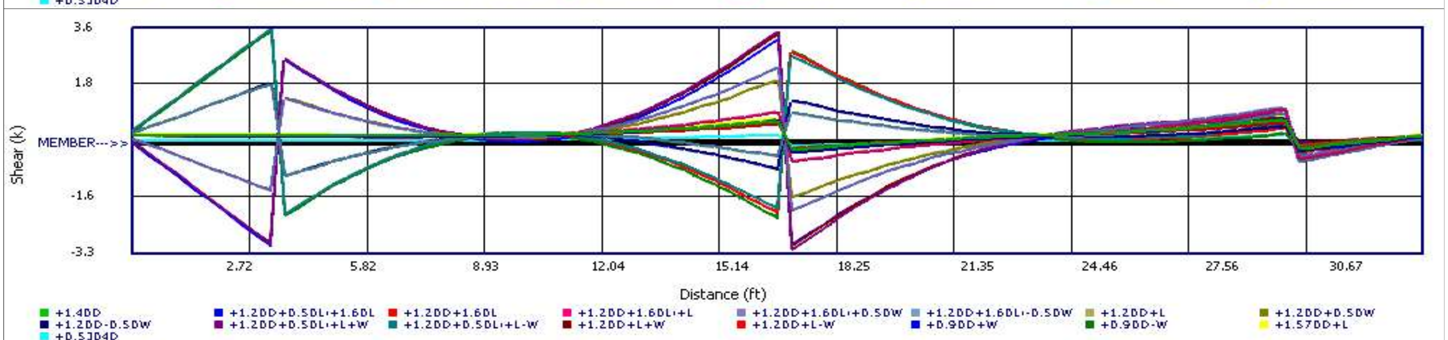
Chapter 5: Foundation Design Cont'd...

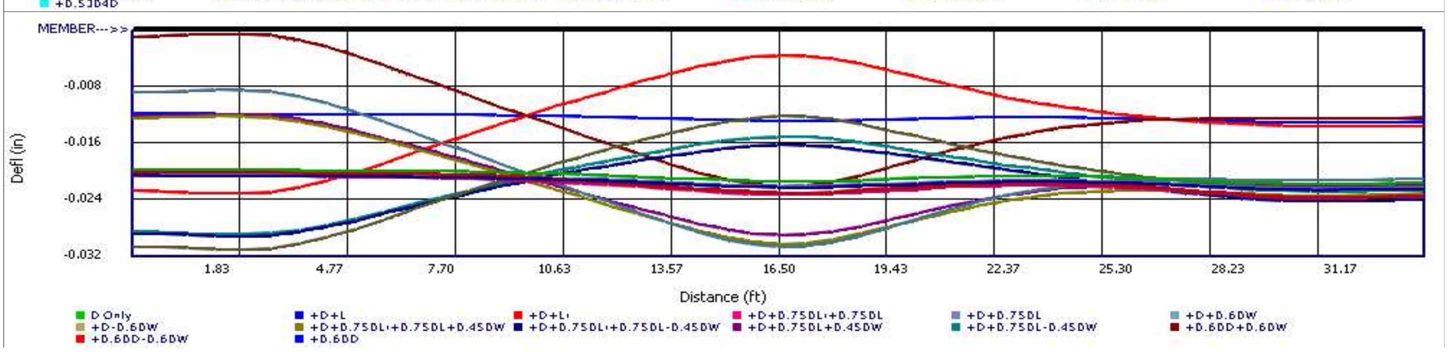
Beam on Elastic Foundation

LIC# : KW-06021401, Build:20.25.07.16

DESCRIPTION: F1







Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Spacing (in) Req'd	Spacing (in) Suggest
+1.20D+0.50Lr+L-W	1	0.00	6.00	0.40	0.40	0.00	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L-W	1	0.39	6.00	0.74	0.74	0.07	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L-W	1	0.78	6.00	1.08	1.08	0.26	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	1.16	6.00	1.42	1.42	0.59	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	1.55	6.00	1.77	1.77	1.05	0.84	8.10	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	1.94	6.00	2.12	2.12	1.65	0.64	7.80	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	2.33	6.00	2.46	2.46	2.38	0.52	7.62	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	2.72	6.00	2.81	2.81	3.25	0.43	7.49	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	3.11	6.00	3.15	3.15	4.25	0.37	7.40	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	3.49	6.00	3.49	3.49	5.39	0.32	7.33	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	3.88	6.00	2.58	2.58	4.45	0.29	7.27	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	4.27	6.00	2.27	2.27	3.54	0.32	7.32	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	4.66	6.00	1.97	1.97	2.75	0.36	7.38	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	5.05	6.00	1.69	1.69	2.07	0.41	7.45	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	5.44	6.00	1.44	1.44	1.51	0.48	7.55	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	5.82	6.00	1.20	1.20	1.04	0.58	7.71	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	6.21	6.00	0.99	0.99	0.66	0.75	7.96	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	6.60	6.00	0.80	0.80	0.37	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	6.99	6.00	0.63	0.63	0.15	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	7.38	6.00	0.49	0.49	0.01	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	7.76	6.00	0.37	0.37	0.11	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	8.15	6.00	0.28	0.28	0.17	0.82	8.07	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	8.54	6.00	0.27	0.27	0.21	0.65	7.82	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	8.93	6.00	0.28	0.28	0.22	0.64	7.79	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	9.32	6.00	0.31	0.31	0.49	0.32	7.32	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00

Chapter 5: Foundation Design Cont'd...

Beam on Elastic Foundation

DESCRIPTION: F1

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Spacing (in) Req'd	Spacing (in) Suggest
+1.20D+0.50Lr+L-W	1	9.71	6.00	0.33	0.33	0.51	0.33	7.33	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L-W	1	10.09	6.00	0.33	0.33	0.47	0.35	7.37	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	10.48	6.00	0.31	0.31	0.23	0.68	7.86	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	10.87	6.00	0.33	0.33	0.22	0.73	7.94	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	11.26	6.00	0.34	0.34	0.21	0.81	8.05	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	11.65	6.00	0.46	0.46	0.01	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	12.04	6.00	0.59	0.59	0.10	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	12.42	6.00	0.75	0.75	0.24	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	12.81	6.00	0.93	0.93	0.44	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	13.20	6.00	1.13	1.13	0.71	0.80	8.04	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	13.59	6.00	1.36	1.36	1.06	0.64	7.80	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	13.98	6.00	1.61	1.61	1.50	0.54	7.64	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	14.36	6.00	1.87	1.87	2.03	0.46	7.53	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	14.75	6.00	2.16	2.16	2.67	0.40	7.45	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	15.14	6.00	2.46	2.46	3.42	0.36	7.38	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	15.53	6.00	2.78	2.78	4.29	0.32	7.33	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	15.92	6.00	3.10	3.10	5.27	0.29	7.28	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	16.31	6.00	3.43	3.43	6.39	0.27	7.24	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	16.69	6.00	-3.24	3.24	7.46	0.22	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	17.08	6.00	-2.91	2.91	6.11	0.24	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	17.47	6.00	-2.59	2.59	4.89	0.26	7.24	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	17.86	6.00	-2.28	2.28	3.80	0.30	7.29	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	18.25	6.00	-1.98	1.98	2.82	0.35	7.37	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	18.64	6.00	-1.70	1.70	1.96	0.43	7.49	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	19.02	6.00	-1.44	1.44	1.21	0.59	7.73	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	19.41	6.00	1.23	1.23	0.66	0.94	8.25	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	19.80	6.00	1.06	1.06	0.27	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	20.19	6.00	0.91	0.91	0.06	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	20.58	6.00	0.77	0.77	0.32	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	20.96	6.00	0.64	0.64	0.53	0.61	7.76	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	21.35	6.00	0.54	0.54	0.69	0.39	7.42	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	21.74	6.00	0.44	0.44	0.81	0.27	7.25	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	22.13	6.00	0.36	0.36	0.89	0.20	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L-W	1	22.52	6.00	0.29	0.29	0.94	0.15	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	22.91	6.00	0.28	0.28	0.51	0.28	7.25	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	23.29	6.00	0.30	0.30	0.52	0.29	7.28	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	23.68	6.00	0.35	0.35	1.86	0.09	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	24.07	6.00	0.41	0.41	1.82	0.11	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	24.46	6.00	0.46	0.46	1.75	0.13	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	24.85	6.00	0.51	0.51	1.66	0.15	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	25.24	6.00	0.55	0.55	1.55	0.18	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	25.62	6.00	0.59	0.59	1.43	0.21	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	26.01	6.00	0.63	0.63	1.29	0.24	7.21	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+0.50Lr+L+W	1	26.40	6.00	0.66	0.66	1.13	0.29	7.28	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	26.79	6.00	0.71	0.71	0.65	0.55	7.66	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	27.18	6.00	0.77	0.77	0.46	0.84	8.10	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	27.56	6.00	0.83	0.83	0.25	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	27.95	6.00	0.89	0.89	0.02	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	28.34	6.00	0.96	0.96	0.24	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	28.73	6.00	1.03	1.03	0.53	0.98	8.30	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr+0.50W	1	29.12	6.00	1.10	1.10	0.84	0.66	7.83	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr-0.50W	1	29.51	6.00	-0.55	0.55	1.44	0.19	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr-0.50W	1	29.89	6.00	-0.46	0.46	1.14	0.20	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr-0.50W	1	30.28	6.00	-0.37	0.37	0.87	0.21	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr-0.50W	1	30.67	6.00	-0.28	0.28	0.64	0.22	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+1.60Lr-0.50W	1	31.06	6.00	-0.18	0.18	0.44	0.21	7.20	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	31.45	6.00	0.15	0.15	0.07	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.20D+L+W	1	31.84	6.00	0.17	0.17	0.04	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	32.22	6.00	0.22	0.22	0.04	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00
+1.570D+L	1	32.61	6.00	0.27	0.27	0.01	1.00	8.34	Vu < Phi*Vc / 2	Not Reqd per 9.6.3.1	0.00	0.00