STRUCTURAL DESIGN CALCULATION

Aroma Joe's

Garfield and South Street
Calais, ME 04619

8/30/2024

STEVEN J PETRACEK

1100 Main St.,

Suite 2200

Kansas City, MO 64105

TKA #24626

STATE OF MAINE
STEVEN J.
PETRACEK
PE17521
LICENSED
PROFESSIONAL ENGINEER

Steven J Petracek
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INTRODUCTION

The structural design of the structure is based on a prototypical design loading with the intent of using it in several locations with minimal design changes. The structure consists of wood framing roof trusses that will be a delayed submittal, wood framed exterior, load bearing walls, and a shallow spread footing foundation. An alternate detail sheet will be provided for an option to use CMU foundation stem walls in lieu of concrete stem walls.

The project is located in Calais, ME and with the design loading parameters based on the 2015 International Building Code and use the following criteria for this structure:

Roof Live Load:	20 psf
Ground Snow Load:	70 psf
Roof Snow Load:	49 psf + drifting
Ultimate Wind Speed:	130 mph
Spectral Acceleration, Short, S_S:	0.262
Spectral Acceleration, 1 sec, S_1:	0.078

The gravity loading will be carried by pre-engineered wood trusses supported by 2x wood framed wall studs. The lateral loading will be carried by plywood or OSB panel board shear walls around the entire structure, allowing for penetrations as needed for windows and doors. The foundation system will be shallow spread footings as determined by the geotechnical engineer.

Design Criteria

Gravity Load Information

Engineer: SJP
Date: July 26, 2024
Project: Aroma Joe's Calais ME V3.0

Live Loads

Roof Live Load (RLL):	20	psf
Ground Snow Load (SL):	70	psf (ASCE 7 HAZARD TOOL)
Roof Snow Load	56	psf
Restaurant	100	psf

Dead Loads

Roof Dead Load - Top Chord (RDL-T):

Roofing:	1	psf
Rigid Insulation (6”):	2	psf
3/4" Plywood:	2	psf
Joists:	3	psf
Total:	**8**	psf

Roof Dead Load - Bottom Chord (RDL-B):

Ceiling:	1	psf
Lighting:	1	psf
Underhung mechanical:	2	psf
Misc:	3	psf
Total:	**7**	psf

Total Roof Dead Load: **15** psf

Wall Dead Load (WDL):

Wall Studs:	2	psf
5/8" Sheathing:	1.75	psf
5/8" Drywall:	2.25	psf
Misc:	2	psf
Total:	**8**	psf

The Bureau of Building Codes and Standards was created in 2010 under Title 25 §2372 to provide administrative and technical support to the Technical Building Codes and Standards Board. The BBCS also provides non-binding technical interpretation of the codes for professionals and the public.

Maine Uniform Building and Energy Code

Maine Uniform Building and Energy Code (MUBEC) applies to all towns within the State of Maine. Enforcement of MUBEC is based on population or local action for communities under 4,000 residents as outlined in Chapter 1 (see below "MUBEC Rules and Laws.")

MUBEC is made up of the following codes and standards:

- 2015 International Residential Code (IRC)
- 2015 International Building Code (IBC)
- 2015 International Existing Building Code (IEBC)
- 2015 International Energy Conservation Code (IECC)
- 2015 International Mechanical Code (IMC)

The following standards are also adopted as part of the MUBEC, and are mandatory.

The American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) Standards:

- 62.1 - 2016 (Ventilation for Acceptable Indoor Air Quality)
- 62.2 - 2016 (Ventilation and Acceptable Indoor Air Quality in Low-Rise Residential Buildings)
- 90.1 - 2016 (Energy Standard for Buildings except Low-Rise Residential Buildings) editions without addenda.
- E-1465-2008, Standard Practice for Radon Control Options for the Design and Construction of New Low-Rise Residential Buildings.

Maine has adopted the national model codes and standards with amendments. The amendments are listed in Rule Chapters 1-7. Chapters can be found under MUBEC Rules and Laws.

Fuel Gas Detector Info Sheet (Updated 12/28/2021)

CONTACT US

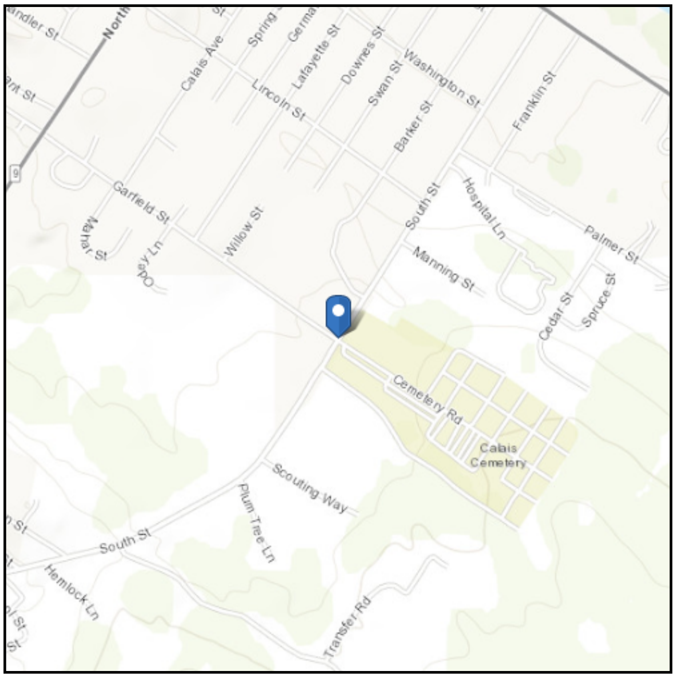
Building Codes Division
Phone: (207) 626-3870
Fax: (207) 287-6251
Email: shannon.e.quintal@maine.gov
Standard Mail:
Office of the State Fire Marshal
52 State House Station
Augusta, ME 04333

ASCE Hazards Report

Address:
Garfield St & South St
Calais, Maine
04619

Standard: ASCE/SEI 7-10
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 45.17632
Longitude: -67.274662
Elevation: 150.1696301761629 ft (NAVD 88)



Wind

Results:

Wind Speed	115 Vmph
10-year MRI	76 Vmph
25-year MRI	84 Vmph
50-year MRI	90 Vmph
100-year MRI	96 Vmph

Data Source: ASCE/SEI 7-10, Fig. 26.5-1A and Figs. CC-1–CC-4, and Section 26.5.2, incorporating errata of March 12, 2014

Date Accessed: Fri Jul 26 2024

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-10 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-10 Section 26.2.

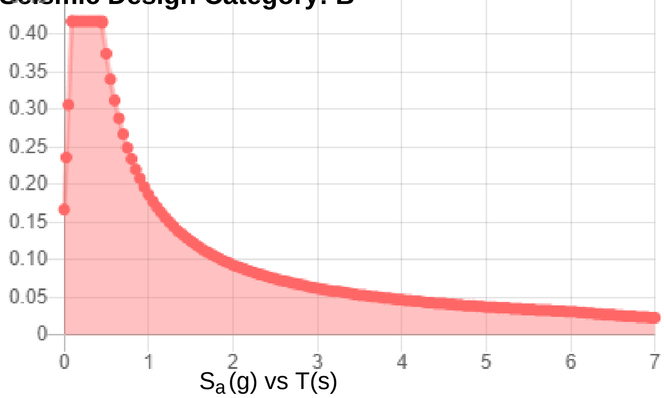
Seismic

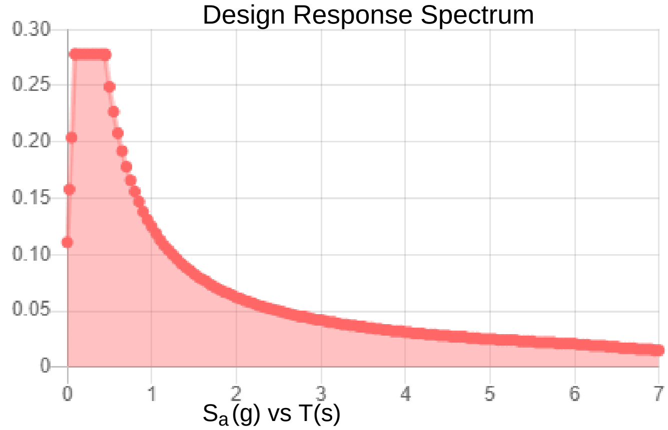
Site Soil Class: D - Stiff Soil

Results:

S_S :	0.262	S_{D1} :	0.125
S_1 :	0.078	T_L :	6
F_a :	1.59	PGA :	0.153
F_v :	2.4	PGA_M :	0.228
S_{MS} :	0.417	F_{PGA} :	1.494
S_{M1} :	0.187	I_e :	1
S_{DS} :	0.278		

Seismic Design Category: B





Data Accessed: **Fri Jul 26 2024**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-10, incorporating Supplement 1 and errata of March 31, 2013, and ASCE/SEI 7-10 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-10 Ch. 21 are available from USGS.

Ice

Results:

Ice Thickness:	1.00 in.
Concurrent Temperature:	5 F
Gust Speed	50 mph
Data Source:	Standard ASCE/SEI 7-10, Figs. 10-2 through 10-8
Date Accessed:	Fri Jul 26 2024

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain with concurrent 3-second gust speeds, for a 50-year mean recurrence interval, and temperatures concurrent with ice thicknesses due to freezing rain. Thicknesses for ice accretions caused by other sources shall be obtained from local meteorological studies. Ice thicknesses in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

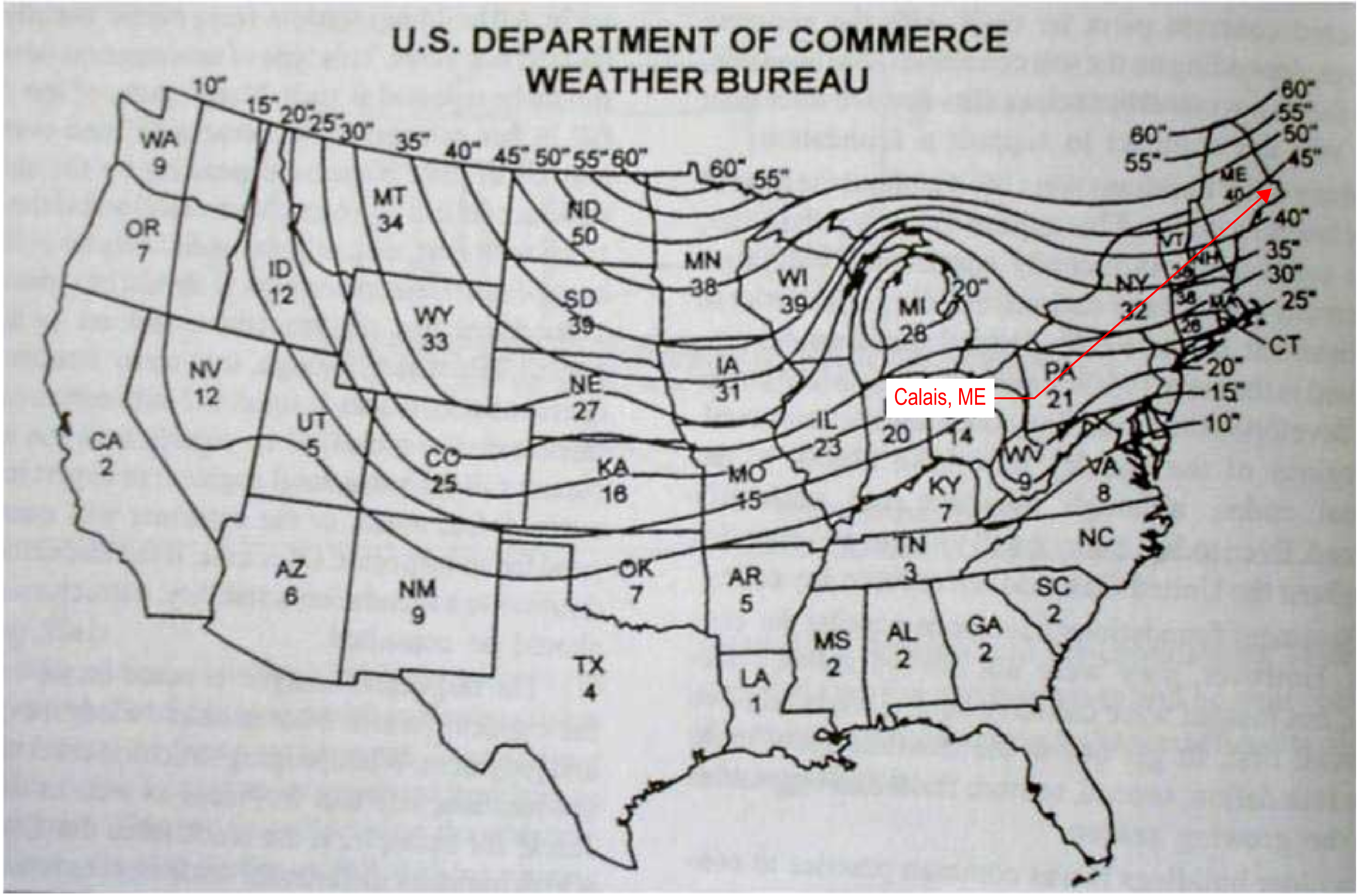
Snow

Results:

Ground Snow Load, p_g :	70 lb/ft^2
Mapped Elevation:	150.2 ft
Data Source:	ASCE/SEI 7-10, Fig. 7-1.
Date Accessed:	Fri Jul 26 2024

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.


U.S. DEPARTMENT OF COMMERCE
WEATHER BUREAU
10"
15"
20"
25"
30"
35"
40"
45"
50"
55"
60"
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WA 9
OR 7
CA 2
NV 12
ID 12
MT 34
WY 33
UT 5
AZ 6
NM 9
CO 25
ND 50
SD 39
NE 27
KA 16
OK 7
TX 4
MN 38
IA 31
MO 15
AR 5
LA 1
WI 39
IL 23
MI 28
20
14
KY 7
TN 3
MS 2
AL 2
GA 2
SC 2
NC
VA 8
WV 9
PA 21
NY 32
VT
NH
MA
ME 40
Calais, ME

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Gravity Design

SNOW LOADING ANALYSIS

Per ASCE 7-16 Code for Buildings with Flat or Low Slope Roofs (<= 5 deg. or 1 in./ft.)
for Balanced Snow, Drift, and Rain-on-Snow Surcharge Loadings

Job Name:	RMS AJS Calais ME Proto V3.0	Subject:	Drift at back of dog house - Long direction		
Job No:	24626	Originator:	LCP	Checker:	

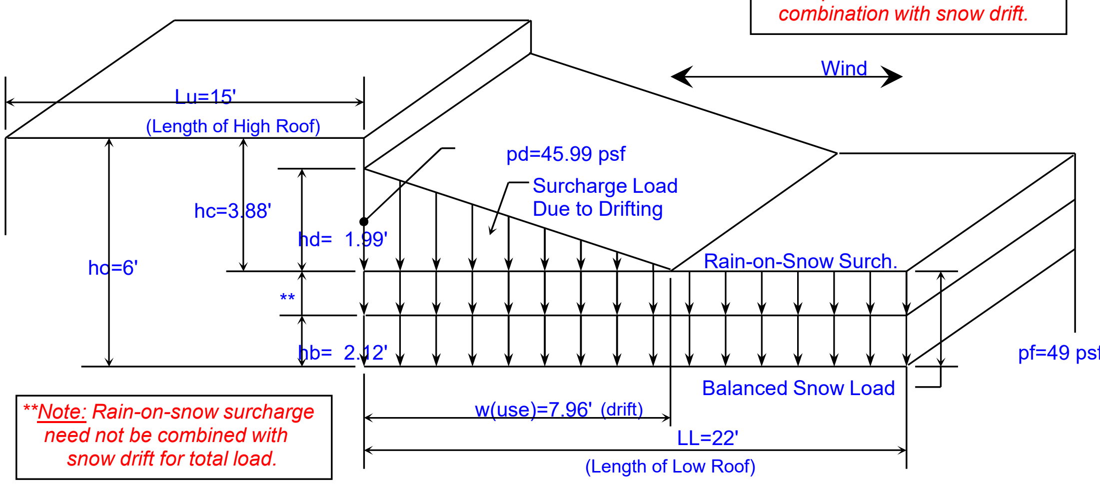
Input Data:

Building Risk Category =	II		Table 1.5-1
Ground Snow Load, p_g =	70.00	psf	Figure 7.2-1, and Table 7.2-1,
Length of High Roof, L_u =	15.00	ft.	Length of Roof Upwind of the Snow Drift
Length of Low Roof, L_L =	22.00	ft.	Length of Roof Downwind of the Snow Drift
. Dist. from Eave to Ridge, W =	15.00	ft.	Horizontal Distance from Eave to Ridge
Type of Roof =	Monoslope		Type of Roof = Monoslope, Gable, or Hip
Obstruction Height, h_o =	6.00	ft.	High Roof - Low Roof Elevations
Roof Slope, S =	0.25	in./ft.	S = Rise per foot of Run
Exposure Factor, C_e =	1.00		Table 7.3-1
Thermal Factor, C_t =	1.00		Table 7.3-2

Results:

Roof Angle, θ =	1.1935	deg.	θ = ATAN(S/12)
Importance Factor, I_s =	1.00		Table 1.5-2
Snow Density, γ =	23.10	pcf	γ = 0.13*p_g+14 <= 30 (Eqn. 7.7-1)
Flat Roof Snow Load, p_f =	49.00	psf	p_f = 0.7*C_e*C_t*I_s*p_g (Eqn. 7.3-1)
*Min. Roof Snow Load, p_m =	20.00	psf	p_m = p_g*I_s for p_g <= 20 , p_m = 20*I_s for p_g > 20 (7.3.4)
Balanced Snow Load Ht., h_b =	2.12	ft.	h_b = $p_{f(use)}/\gamma$ (Section 7.1)
Clear Height, h_c =	3.88	ft.	h_c = h_o-h_b >= 0 (Section 7.1)
Leeward Drift Height, h_{dL} =	1.99	ft.	h_{dL} = (0.43*L_u^1/3*(p_g+10)^1/4-1.5)*(I_s)^0.5, with L_u>=20' (Figure 7.6-1)
Windward Drift Height, h_{dw} =	1.58	ft.	h_{dw} = 0.75*((0.43*L_L^1/3*(p_g+10)^1/4-1.5)*(I_s)^0.5), with L_u>=20'
Max. Drift Height, $h_{d(max)}$ =	1.99	ft.	$h_{d(max)}$ = maximum of: (h_{dL} or h_{dw})
Ratio, h_c/h_b =	1.83		If hc/hb >= 0.2 , then snow drifts are required to be applied
Drift Length, w =	7.96	ft.	If $h_{d(max)}$<=h_c: w = 4*$h_{d(max)}$, if $h_{d(max)}$>h_c: w = 4*$h_{d(max)}$^2/h_c
Design Drift Height, h_d =	1.99	ft.	If $h_{d(max)}$ <= h_c: h_d = $h_{d(max)}$, if $h_{d(max)}$ > h_c: h_d = h_c
Drift Length, $w_{(max)}$ =	31.03	ft.	$w_{(max)}$ <= 8*h_c
Drift Length, $w_{(use)}$ =	7.96	ft.	$w_{(use)}$ = minimum of: w or $w_{(max)}$
Wt. of Drift at High End, p_d =	45.99	psf	p_d =h_d*γ (maximum value)
Wt. of Drift at Low End, p_{de} =	0.00	psf	p_{de} = 0, as Low Roof Length (LL) >= w(max)
Rain-on-Snow Surch., p_{rs} =	0.00	psf	p_{rs} = 5.0 psf when 0 < p_g <=20 and θ < W/50) (Sect. 7.10)
Balanced Snow Load, $p_{f(bal)}$ =	49.00	psf	$p_{f(bal)}$ = p_f+p_{rs}
**Total Snow Load, $p_{(total)}$ =	94.99	psf	$p_{(total)}$ = $p_{f(bal)}$-p_{rs}+p_d

**Note: Minimum flat roof snow load, pm, need not be used in combination with snow drift.*



***Note: Rain-on-snow surcharge need not be combined with snow drift for total load.*

Configuration of Snow Drift on Lower Roof

SNOW LOADING ANALYSIS

Per ASCE 7-16 Code for Buildings with Flat or Low Slope Roofs (<= 5 deg. or 1 in./ft.)
for Balanced Snow, Drift, and Rain-on-Snow Surcharge Loadings

Job Name:	RMS AJS Calais ME Proto V3.0	Subject:	Drift at parapets - Long direction		
Job No:	24626	Originator:	LCP	Checker:	

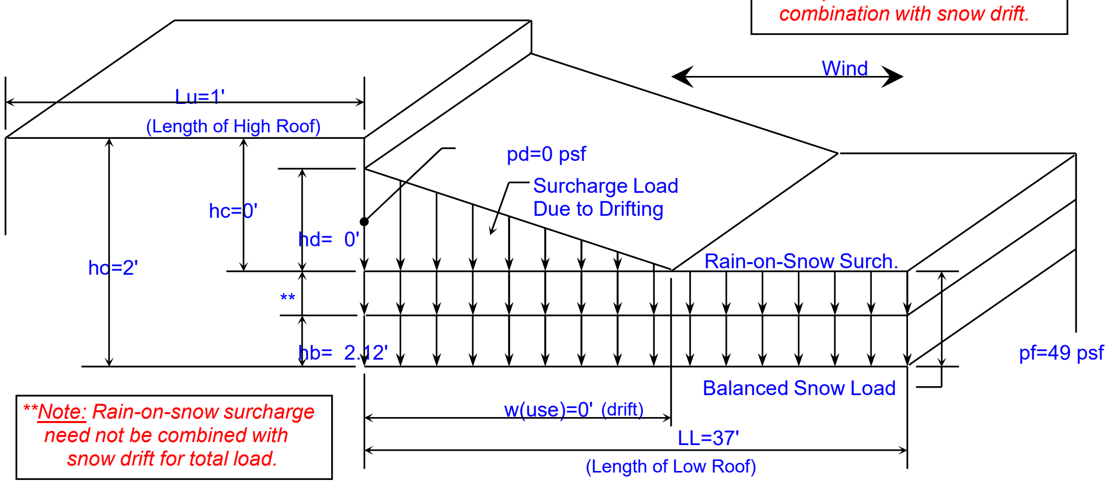
Input Data:

Building Risk Category =	II		Table 1.5-1
Ground Snow Load, p_g =	70.00	psf	Figure 7.2-1, and Table 7.2-1,
Length of High Roof, L_u =	1.00	ft.	Length of Roof Upwind of the Snow Drift
Length of Low Roof, L_L =	37.00	ft.	Length of Roof Downwind of the Snow Drift
. Dist. from Eave to Ridge, W =	37.00	ft.	Horizontal Distance from Eave to Ridge
Type of Roof =	Monoslope		Type of Roof = Monoslope, Gable, or Hip
Obstruction Height, h_o =	2.00	ft.	High Roof - Low Roof Elevations
Roof Slope, S =	0.25	in./ft.	S = Rise per foot of Run
Exposure Factor, C_e =	1.00		Table 7.3-1
Thermal Factor, C_t =	1.00		Table 7.3-2

Results:

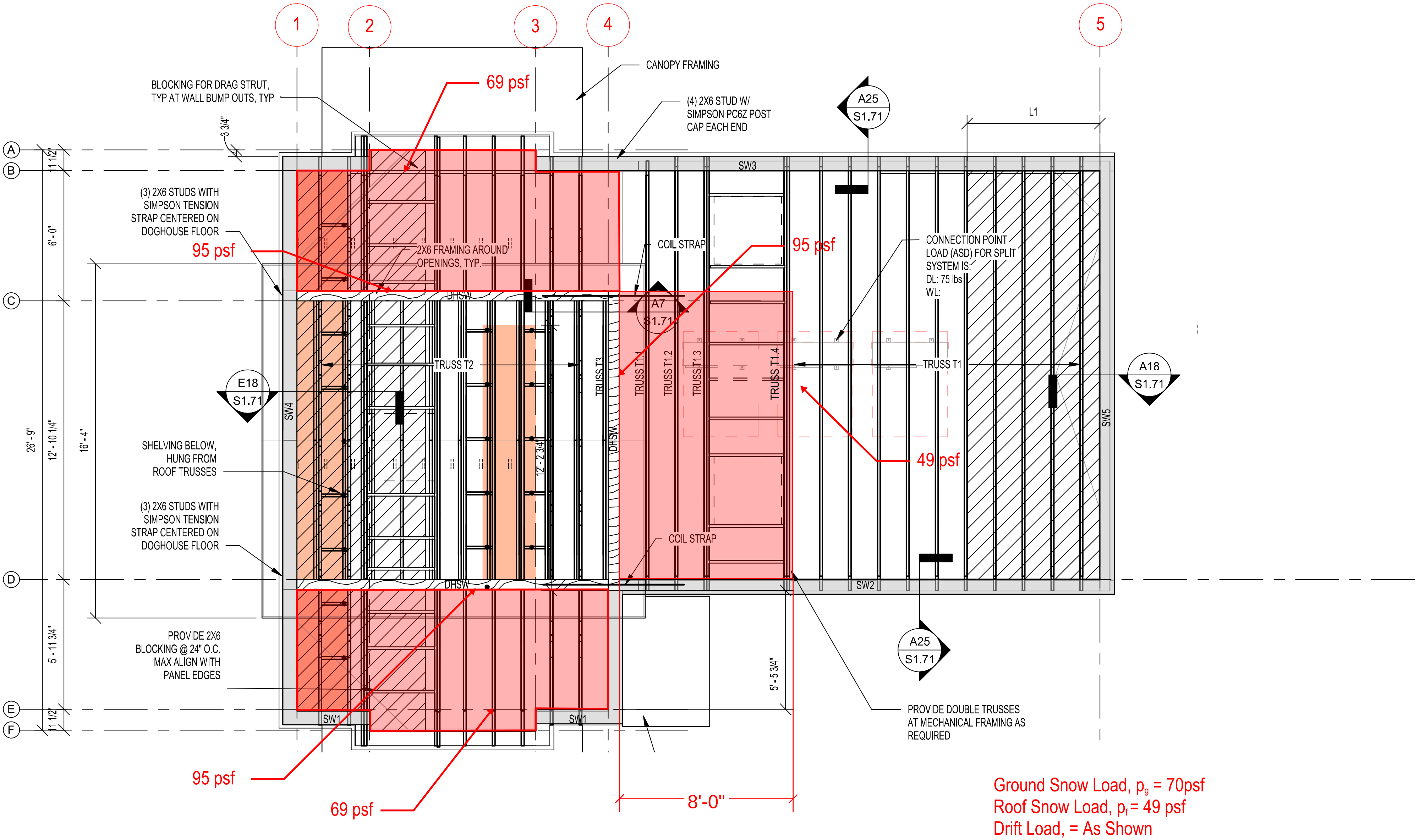
Roof Angle, θ =	1.1935	deg.	θ = ATAN(S/12)
Importance Factor, I_s =	1.00		Table 1.5-2
Snow Density, γ =	23.10	pcf	γ = 0.13*p_g+14 <= 30 (Eqn. 7.7-1)
Flat Roof Snow Load, p_f =	49.00	psf	p_f = 0.7*C_e*C_t*I_s*p_g (Eqn. 7.3-1)
*Min. Roof Snow Load, p_m =	20.00	psf	p_m = p_g*I_s for p_g <= 20 , p_m = 20*I_s for p_g > 20 (7.3.4)
Balanced Snow Load Ht., h_b =	2.12	ft.	h_b = $p_{f(use)}$/γ (Section 7.1)
Clear Height, h_c =	0.00	ft.	h_c = h_o-h_b >= 0 (Section 7.1)
Leeward Drift Height, h_{dL} =	1.99	ft.	h_{dL} = (0.43*L_u^1/3*(p_g+10)^1/4-1.5)*(I_s)^0.5, with L_u>=20' (Figure 7.6-1)
Windward Drift Height, h_{dw} =	2.09	ft.	h_{dw} = 0.75*((0.43*L_L^1/3*(p_g+10)^1/4-1.5)*(I_s)^0.5), with L_u>=20'
Max. Drift Height, $h_{d(max)}$ =	2.09	ft.	$h_{d(max)}$ = maximum of: (h_{dL} or h_{dw})
Ratio, h_c/h_b =	0.00		If h_c/h_b < 0.2 , then snow drifts are not required to be applied
Drift Length, w =	0.00	ft.	If $h_{d(max)}$<=h_c: w = 4*$h_{d(max)}$, if $h_{d(max)}$>h_c: w = 4*$h_{d(max)}$^2/h_c
Design Drift Height, h_d =	0.00	ft.	If $h_{d(max)}$ <= h_c: h_d = $h_{d(max)}$, if $h_{d(max)}$ > h_c: h_d = h_c
Drift Length, $w_{(max)}$ =	0.00	ft.	$w_{(max)}$ <= 8*h_c
Drift Length, $w_{(use)}$ =	0.00	ft.	$w_{(use)}$ = minimum of: w or $w_{(max)}$
Wt. of Drift at High End, p_d =	0.00	psf	p_d =h_d*γ (maximum value)
Wt. of Drift at Low End, p_{de} =	0.00	psf	p_{de} = 0, as Low Roof Length (LL) >= w(max)
Rain-on-Snow Surch., p_{rs} =	0.00	psf	p_{rs} = 5.0 psf when 0 < p_g <=20 and θ < W/50) (Sect. 7.10)
Balanced Snow Load, $p_{f(bal)}$ =	49.00	psf	$p_{f(bal)}$ = p_f+p_{rs}
**Total Snow Load, $p_{(total)}$ =	49.00	psf	$p_{(total)}$ = $p_{f(bal)}$-p_{rs}+p_d

Note: Minimum flat roof snow load, pm, need not be used in combination with snow drift.

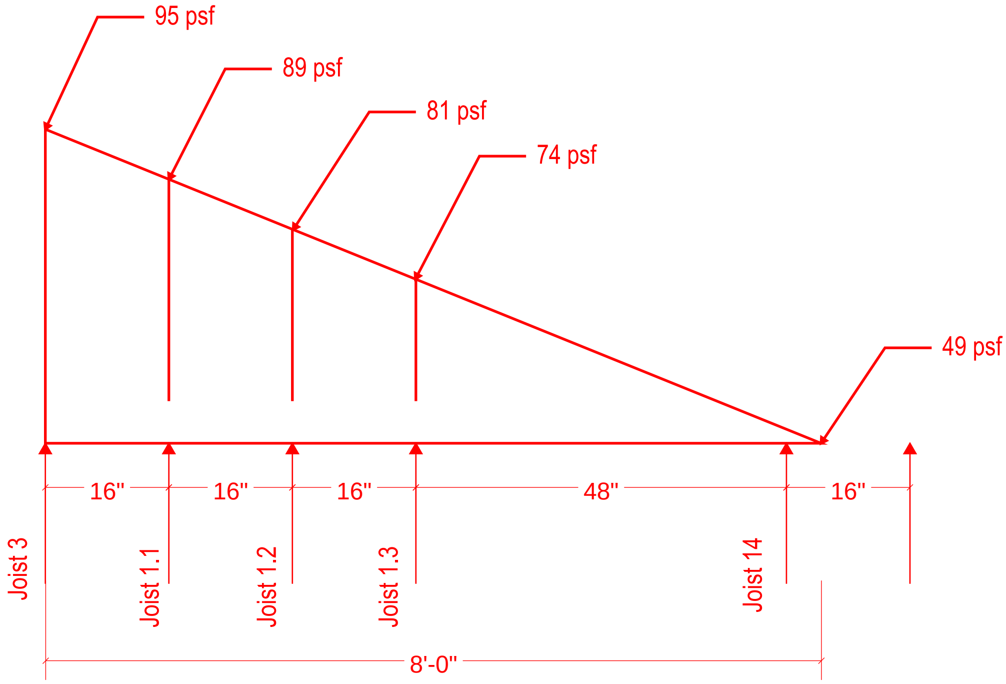


Note: Rain-on-snow surcharge need not be combined with snow drift for total load.

Configuration of Snow Drift on Lower Roof



1
2
3
4
5
69 psf
CANOPY FRAMING
BLOCKING FOR DRAG STRUT, TYP AT WALL BUMP OUTS, TYP
(4) 2X6 STUD W/ SIMPSON PC6Z POST CAP EACH END
A25
S1.71
L1
3 3/4"
A
B
C
D
E
F
SW3
11 1/2"
6' - 0"
(3) 2X6 STUDS WITH SIMPSON TENSION STRAP CENTERED ON DOGHOUSE FLOOR
95 psf
2X6 FRAMING AROUND OPENINGS, TYP.
COIL STRAP
95 psf
CONNECTION POINT LOAD (ASD) FOR SPLIT SYSTEM IS:
DL: 75 lbs
WL:
DHSW
A7
S1.71
TRUSS T2
TRUSS T3
TRUSS T1.1
TRUSS T1.2
TRUSS T1.3
TRUSS T1.4
TRUSS T1
A18
S1.71
E18
S1.71
SW4
SW5
26' - 9"
12' - 10 1/4"
16' - 4"
12' - 2 3/4"
DHSW
49 psf
SHELVING BELOW, HUNG FROM ROOF TRUSSES
(3) 2X6 STUDS WITH SIMPSON TENSION STRAP CENTERED ON DOGHOUSE FLOOR
COIL STRAP
DHSW
SW2
5' - 11 3/4"
PROVIDE 2X6 BLOCKING @ 24" O.C. MAX ALIGN WITH PANEL EDGES
A25
S1.71
5' - 5 3/4"
PROVIDE DOUBLE TRUSSES AT MECHANICAL FRAMING AS REQUIRED
11 1/2"
SW1
SW1
95 psf
69 psf
8'-0"
Ground Snow Load, p_g = 70psf
Roof Snow Load, p_f = 49 psf
Drift Load, = As Shown



Joist trib typically 16" O.C.
Determine snow loading to joists for 8'-0"
J1.1 = 116 plf
J1.2 = 106 plf
J1.3 = 180 plf
J1.4 = 140 plf

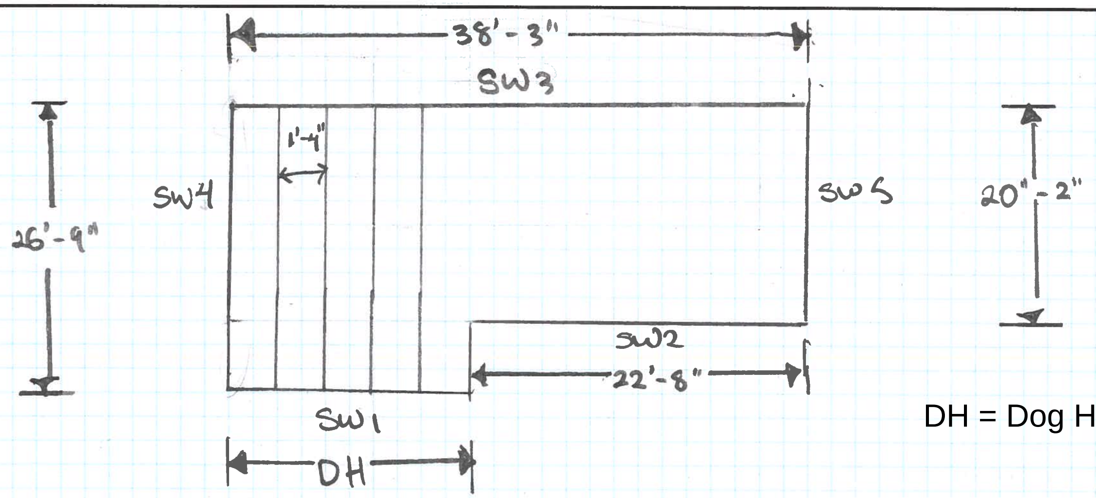
TK ARCHITECTS INTERNATIONAL

STUD ANALYSIS

Engineer ______ Date ______ Page ______

Checked ______ Date ______

Project 24622 STUD CHECK Sheet 1 of ______



DH = Dog House

With Dog House is the worse case

* Shear Wall 3 Stud Check

DL W/DH = 0.236klf

*Refer to Shear Wall Calculations to see how these loads are determined

ROOF LIVE LOAD :

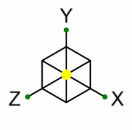
W/DH: $$\frac{20psf*(26.8')*1'\text{-}4''}{2} = 0.357k$$

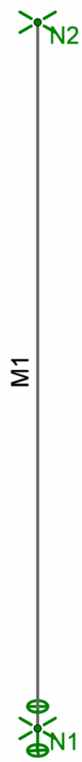
SNOW LOAD :

W/DH: $$\frac{{}^{*}65psf*(26.8')*1'\text{-}4''}{2} = 1.161k$$

WIND LOAD = 25PSF* 16"O.C. = 0.033klf

STUD ANALYSIS





	TK	Dover V3.0	SK-1
	emomah		May 16, 2024 at 03:35 PM
	24622		Stud Check.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	10	0	0	
2	N2	10	15	0	

Node Boundary Conditions

	Node Label	X [k/in]	Y [k/in]	Z [k/in]	Y Rot [k-ft/rad]
1	N2	Reaction		Reaction	
2	N1	Reaction	Reaction	Reaction	Reaction

Basic Load Cases

	BLC Description	Category	Point	Distributed
1	Dead Load	DL		1
2	Roof Live Load	RLL	1	
3	Snow Load	SL	1	
4	Wind Load	WLX		1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 21/ASCE ASD 1	Yes	Y	DL	1						
2	IBC 21/ASCE ASD 3 (a)	Yes	Y	DL	1	LL	1				
3	IBC 21/ASCE ASD 4 (a)	Yes	Y	DL	1	SL	1				
4	IBC 21/ASCE ASD 6 (a) (a)	Yes		DL	1	WLX	0.45	LL	0.75	RLL	0.75
5	IBC 21/ASCE ASD 6 (a) (c)	Yes		DL	1	WLX	-0.45	LL	0.75	RLL	0.75
6	IBC 21/ASCE ASD 6 (b) (a)	Yes		DL	1	WLX	0.45	LL	0.75	SL	0.75
7	IBC 21/ASCE ASD 6 (b) (c)	Yes		DL	1	WLX	-0.45	LL	0.75	SL	0.75
8	IBC 21/ASCE ASD 6 (c) (a)	Yes		DL	1	WLX	0.45	LL	0.75		
9	IBC 21/ASCE ASD 6 (c) (c)	Yes		DL	1	WLX	-0.45	LL	0.75		
10	IBC 21/ASCE ASD 7 (a)	Yes		DL	0.6	WLX	0.6				
11	IBC 21/ASCE ASD 7 (c)	Yes		DL	0.6	WLX	-0.6				

Member Point Loads (BLC 2 : Roof Live Load)

	Member Label	Direction	Magnitude [k, k-ft]	Location [(ft, %)]
1	M1	Y	-0.357	%100

Member Point Loads (BLC 3 : Snow Load)

	Member Label	Direction	Magnitude [k, k-ft]	Location [(ft, %)]
1	M1	Y	-1.161	%100

Member Distributed Loads (BLC 1 : Dead Load)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	Y	-0.236	-0.236	0	%100

Member Distributed Loads (BLC 4 : Wind Load)

	Member Label	Direction	Start Magnitude [k/ft, F, ksf, k-ft/ft]	End Magnitude [k/ft, F, ksf, k-ft/ft]	Start Location [(ft, %)]	End Location [(ft, %)]
1	M1	X	0.033	0.033	0	%100

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0.148	11	0	11	0	11	0	11	0	11	0	11
2		min	-0.148	10	0	1	0	1	0	1	0	1	0	1
3	N1	max	0.149	11	4.701	3	0	11	0	11	0	11	0	11
4		min	-0.149	10	2.124	10	0	1	0	1	0	1	0	1
5	Totals:	max	0.297	11	4.701	3	0	11						
6		min	-0.297	10	2.124	10	0	1						

Envelope AWC NDS-18 / SDPWS-21 ASD Member Wood Code Checks

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Fc' [ksi]	Ft' [ksi]	Fb1' [ksi]	Fb2' [ksi]	Fv' [ksi]	RB	CL	CP	Eqn
1	M1	2X6	0.826	4.688	7	0.094	15	y	11	0.814	1.404	1.78	2.08	0.288	17.963	0.856	0.308	3.9-3

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Lateral Design

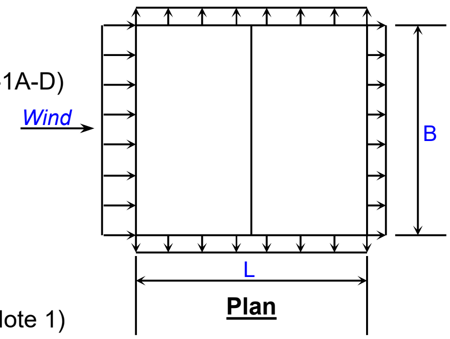
WIND LOADING ANALYSIS - Main Wind-Force Resisting System

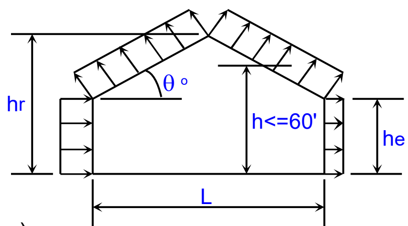
Per ASCE 7-16 Code for Enclosed or Partially Enclosed Buildings
Using Method 2: Analytical Procedure (Section 27 & 28) for Low-Rise Buildings

Job Name:	Aroma Joe's Dover Prototype	Subject:	MWFRS wind check for new framing	
Job Number:		Originator:	SJP	Checker:

Input Data:

Wind Speed, V =	130	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	25.00	ft. (hr >= he)
Eave Height, he =	13.00	ft. (he <= hr)
Building Width =	28.00	ft. (Normal to Building Ridge)
Building Length =	39.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Ground Elevation Factor Ke	1.00	(Sect. 26.9, Ke=1 based on Note 1)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.12 & Table 26.13-1)
Hurricane Region?	N	





Resulting Parameters and Coefficients:

Roof Angle, θ =	40.60	deg.
Mean Roof Ht., h =	19.00	ft. (h = (hr+he)/2, for angle >10 deg.)

Check Criteria for a Low-Rise Building:
1. Is h <= 60' ? Yes, O.K.   2. Is h <= Lesser of L or B? Yes, O.K.

External Pressure Coeff's., GCpf (Fig. 28.3-1):
(For values, see following wind load tabulations.)
Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If h < 15 then: Kh = 2.01*(15/zg)^(2/α) (Table 26.10-1)
If h >= 15 then: Kh = 2.01*(z/zg)^(2/α) (Table 26.10-1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 26.11-1)
Kh =	0.89	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Ke*Kzt*Kd*V^2 (Eq. 26.10-1)
qh = 32.81 psf   qh = 0.00256*Kh*Ke*Kzt*Kd*V^2 (qz evaluated at z = h)
Design Net External Wind Pressures (Sect. 28.3.1):
p = qh*[(GCpf) - (+/-GCpi)] (psf, Eq. 28.3-1)

Wall and Roof End Zone Widths 'a' and '2*a' (Fig. 28.3-1):

a =	3.00	ft.
2*a =	6.00	ft.

MWFRS Wind Load for Load Case A				MWFRS Wind Load for Load Case B			
Surface	GCpf	p = Net Pressures (psf)		Surface	*GCpf	p = Net Pressures (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1	0.56	12.47	24.28	Zone 1	-0.45	-20.67	-8.86
Zone 2	0.21	0.98	12.80	Zone 2	-0.69	-28.54	-16.73
Zone 3	-0.43	-20.01	-8.20	Zone 3	-0.37	-18.05	-6.23
Zone 4	-0.37	-18.05	-6.23	Zone 4	-0.45	-20.67	-8.86
Zone 5	---	---	---	Zone 5	0.40	7.22	19.03
Zone 6	---	---	---	Zone 6	-0.29	-15.42	-3.61
Zone 1E	0.69	16.73	28.54	Zone 1E	-0.48	-21.65	-9.84
Zone 2E	0.27	2.95	14.76	Zone 2E	-1.07	-41.01	-29.20
Zone 3E	-0.53	-23.30	-11.48	Zone 3E	-0.53	-23.30	-11.48
Zone 4E	-0.48	-21.65	-9.84	Zone 4E	-0.48	-21.65	-9.84
Zone 5E	---	---	---	Zone 5E	0.61	14.11	25.92
Zone 6E	---	---	---	Zone 6E	-0.43	-20.01	-8.20

*Note: Use roof angle θ = 0 degrees for Longitudinal Direction.

For Case A when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = N.A. ft.

For Case B when GCpf is neg. in Zones 2/2E:
Zones 2/2E dist. = 19.50 ft.

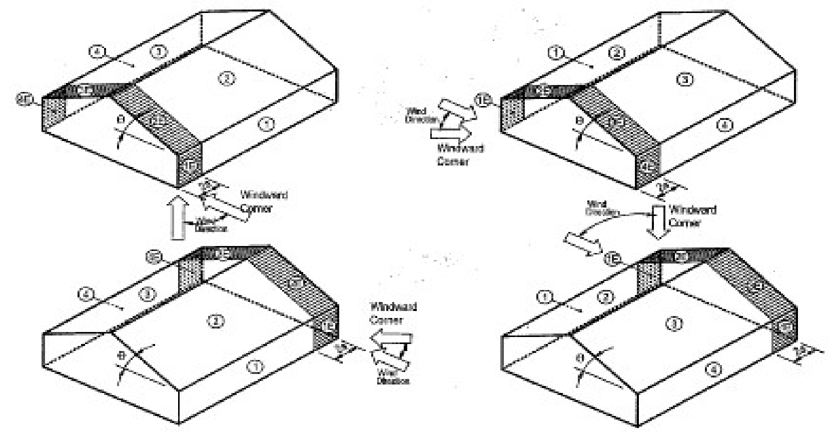
Remainder of roof Zones 2/2E extending to ridge line shall use roof Zones 3/3E pressure coefficients.

MWFRS Wind Load for Load Case A, Torsional Case				MWFRS Wind Load for Case B, Torsional Case			
Surface	GCpf	p = Net Pressure (psf)		Surface	GCpf	p = Net Pressure (psf)	
		(w/ +GCpi)	(w/ -GCpi)			(w/ +GCpi)	(w/ -GCpi)
Zone 1T	---	3.12	6.07	Zone 1T	---	-5.17	-2.21
Zone 2T	---	0.25	3.20	Zone 2T	---	-7.14	-4.18
Zone 3T	---	-5.00	-2.05	Zone 3T	---	-4.51	-1.56
Zone 4T	---	-4.51	-1.56	Zone 4T	---	-5.17	-2.21
Zone 5T	---	---	---	Zone 5T	---	1.80	4.76
Zone 6T	---	---	---	Zone 6T	---	-3.86	-0.90

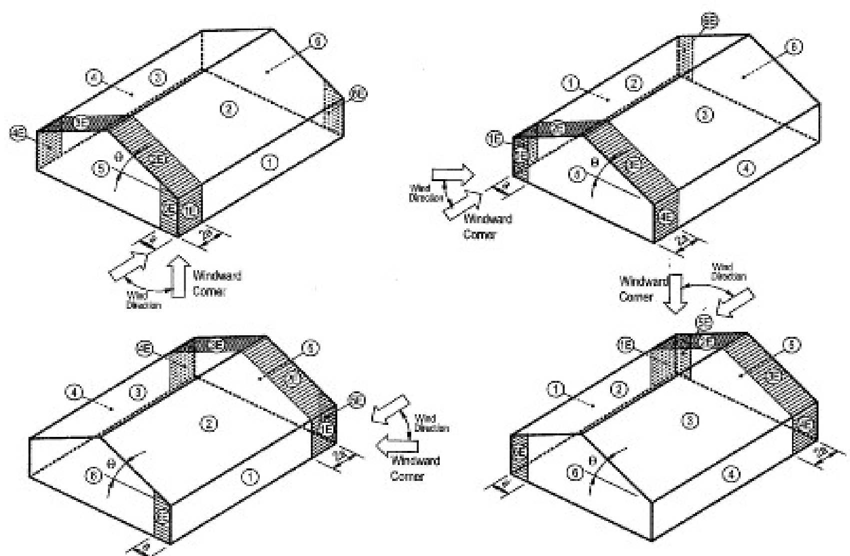
Notes: 1. For Load Case A (Transverse), Load Case B (Longitudinal), and Torsional Cases:
Zone 1 is windward wall for interior zone.
Zone 2 is windward roof for interior zone.
Zone 3 is leeward roof for interior zone.
Zone 4 is leeward wall for interior zone.
Zones 5 and 6 are sidewalls.
Zone 1T is windward wall for torsional case
Zone 3T is leeward roof for torsional case
Zones 5T and 6T are sidewalls for torsional case.
Zone 1E is windward wall for end zone.
Zone 2E is windward roof for end zone.
Zone 3E is leeward roof for end zone.
Zone 4E is leeward wall for end zone.
Zone 5E & 6E is sidewalls for end zone.
Zone 2T is windward roof for torsional case.
Zone 4T is leeward wall for torsional case.

2. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
3. Building must be designed for all wind directions using the 8 load cases shown below. The load cases are applied to each building corner in turn as the reference corner.
4. Wind loads for torsional cases are 25% of respective transverse or longitudinal zone load values. Torsional loading shall apply to all 8 basic load cases applied at each reference corner.
Exception: One-story buildings with "h" <= 30', buildings <= 2 stories framed with light frame construction, and buildings <=2 stories designed with flexible diaphragms need not be designed for torsional load cases.

Low-Rise Buildings h <= 60'

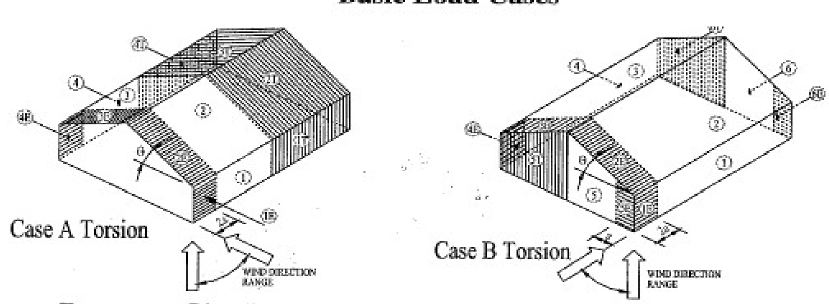


Load Case A



Load Case B

Basic Load Cases



Transverse Direction

Longitudinal Direction

Torsional Load Cases

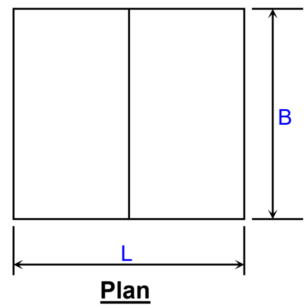
WIND LOADING ANALYSIS - Roof Components and Cladding

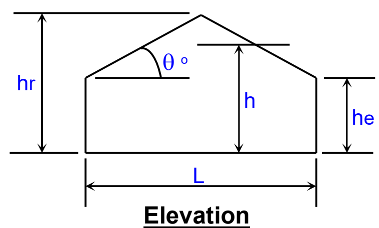
Per ASCE 7-16 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°
Using Part 1 : Analytical Procedure (Section 30.3)

Job Name:	Aroma Joe's Dover Prototype	Subject:			
Job Number:		Originator:	SJP	Checker:	

Input Data:

Wind Speed, V =	130	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	25.00	ft. (hr >= he)
Eave Height, he =	18.25	ft. (he <= hr)
Building Width =	28.00	ft. (Normal to Building Ridge)
Building Length =	39.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Ground Elevation Factor Ke	1.00	(Sect. 26.9, Ke=1 based on Note 1)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.12 & Table 26.13-1)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	10	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	Y	(if used, overhangs on all sides)





Resulting Parameters and Coefficients:

Roof Angle, θ =	25.74	deg.
Mean Roof Ht., h =	21.63	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.54	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 1 Neg. =	-2.00	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 2 Neg. =	-3.00	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 3 Neg. =	-3.94	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 26.10-1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 2611-1)
Kh =	0.92	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Ke*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

qh = 33.72 psf qh = 0.00256*Kh*Ke*Kzt*Kd*V^2 (qz evaluated at z = h)

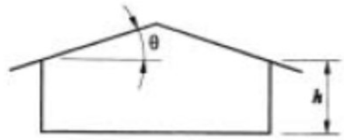
Design Net External Wind Pressures (Sect. 30.3 & 30.5):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for roof (conservatively assumed per Sect. 30.6)

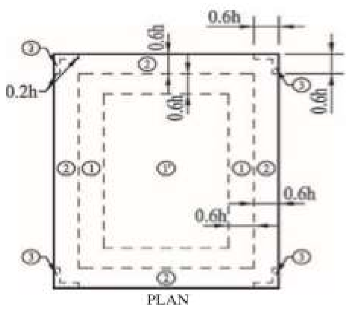
Wind Load Tabulation for Roof Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
Joist	0	0.92	33.72	24.12	-73.50	-107.22	-138.96
	15.00	0.92	33.72	24.12	-73.50	-107.22	-138.96
	20.00	0.92	33.72	24.12	-73.50	-107.22	-138.96
For z = hr:	25.00	0.92	33.72	24.12	-73.50	-107.22	-138.96
For z = he:	18.25	0.92	33.72	24.12	-73.50	-107.22	-138.96
For z = h:	21.63	0.92	33.72	24.12	-73.50	-107.22	-138.96

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.00 ft.
3. Width of Zone 3 (corner), 'a' = 3.00 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.3-2A for 'GCp' values with 'qh'.

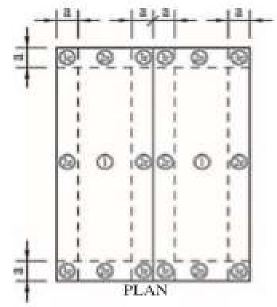
6. For all buildings with overhangs, use Fig. 30.3-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Roof Components and Cladding:

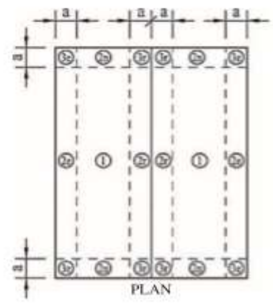




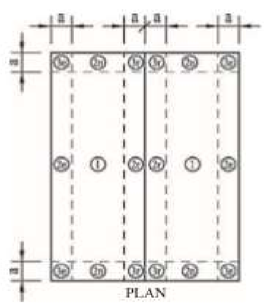
θ <= 7 deg.



7 deg. < θ <= 20 deg.



20 deg. < θ <= 27 deg.



27deg. < q <= 45 deg.

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45^{o} and Monoslope Roofs <= 3^{o})

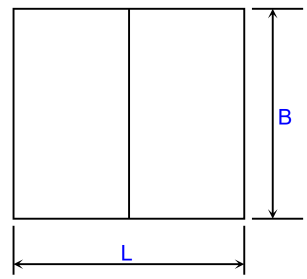
WIND LOADING ANALYSIS - Roof Components and Cladding

Per ASCE 7-16 Code for Bldgs. of Any Height with Gable Roof θ <= 45° or Monoslope Roof θ <= 3°
Using Part 1 : Analytical Procedure (Section 30.3)

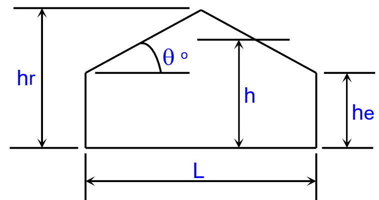
Job Name:	Aroma Joe's Dover Prototype	Subject:			
Job Number:		Originator:	SJP	Checker:	

Input Data:

Wind Speed, V =	130	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	25.00	ft. (hr >= he)
Eave Height, he =	18.25	ft. (he <= hr)
Building Width =	28.00	ft. (Normal to Building Ridge)
Building Length =	39.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Ground Elevation Factor Ke	1.00	(Sect. 26.9, Ke=1 based on Note 1)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.12 & Table 26.13-1)
Hurricane Region?	N	
Component Name =	Joist	(Purlin, Joist, Decking, or Fastener)
Effective Area, Ae =	10	ft.^2 (Area Tributary to C&C)
Overhangs? (Y/N)	N	(if used, overhangs on all sides)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	25.74	deg.
Mean Roof Ht., h =	21.63	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Roof External Pressure Coefficients, GCp:

GCp Zone 1-3 Pos. =	0.54	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 1 Neg. =	-1.50	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 2 Neg. =	-2.50	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)
GCp Zone 3 Neg. =	-2.95	(Fig. 30.3-2A, 30.3-2B,30.3-2C, and 30.3-2D)

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 26.10-1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 2611-1)
Kh =	0.92	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Ke*Kzt*Kd*V^2 (Sect. 30.3.2, Eq. 30.3-1)

qh = 33.72 psf qh = 0.00256*Kh*Ke*Kzt*Kd*V^2 (qz evaluated at z = h)

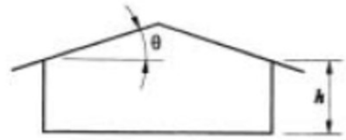
Design Net External Wind Pressures (Sect. 30.3 & 30.5):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qh for roof
qi = qh for roof (conservatively assumed per Sect. 30.6)

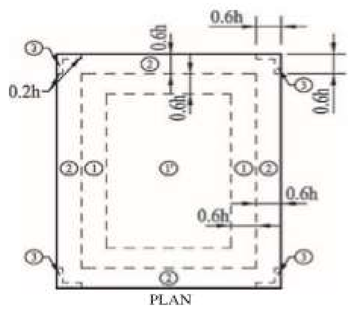
Wind Load Tabulation for Roof Components & Cladding							
Component	z	Kh	qh	p = Net Design Pressures (psf)			
	(ft.)		(psf)	Zone 1,2,3 (+)	Zone 1 (-)	Zone 2 (-)	Zone 3 (-)
Joist	0	0.92	33.72	24.12	-56.64	-90.36	-105.43
	15.00	0.92	33.72	24.12	-56.64	-90.36	-105.43
	20.00	0.92	33.72	24.12	-56.64	-90.36	-105.43
For z = hr:	25.00	0.92	33.72	24.12	-56.64	-90.36	-105.43
For z = he:	18.25	0.92	33.72	24.12	-56.64	-90.36	-105.43
For z = h:	21.63	0.92	33.72	24.12	-56.64	-90.36	-105.43

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 2 (edge), 'a' = 3.00 ft.
3. Width of Zone 3 (corner), 'a' = 3.00 ft.
4. For monoslope roofs with θ <= 3 degrees, use Fig. 30.3-2A for 'GCp' values with 'qh'.

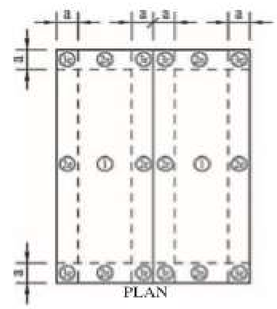
6. For all buildings with overhangs, use Fig. 30.3-2B for 'GCp' values per Sect. 30.10.
7. If a parapet >= 3' in height is provided around perimeter of roof with θ <= 10 degrees, Zone 3 shall be treated as Zone 2.
8. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
9. References : a. ASCE 7-02, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Roof Components and Cladding:

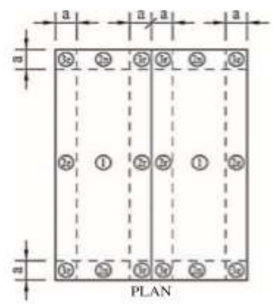




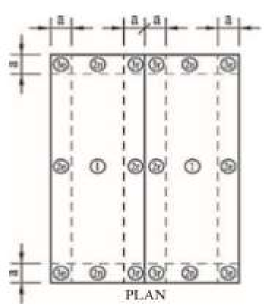
θ <= 7 deg.



7 deg. < θ <= 20 deg.



20 deg. < θ <= 27 deg.



27deg. < q <= 45 deg.

Roof Zones for Buildings with h <= 60 ft.
(for Gable Roofs <= 45^o and Monoslope Roofs <= 3^o)

WIND LOADING ANALYSIS - Wall Components and Cladding

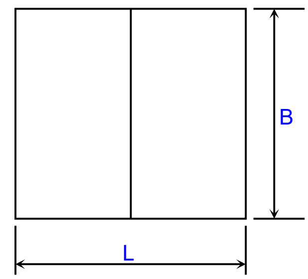
Per ASCE 7-16 Code for Buildings of Any Height

Using Part 1 & 3: Analytical Procedure (Section 30.3 & 30.5)

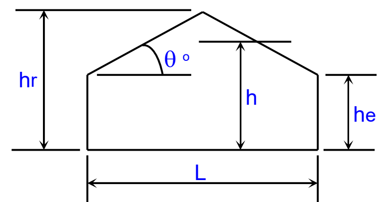
Job Name:	Aroma Joe's Dover Prototype	Subject:			
Job Number:		Originator:	SJP	Checker:	

Input Data:

Wind Speed, V =	130	mph (Wind Map, Figure 26.5-1A-D)
Bldg. Classification =	II	(Table 1.5-1 Risk Category)
Exposure Category =	C	(Sect. 26.7)
Ridge Height, hr =	25.00	ft. (hr >= he)
Eave Height, he =	13.00	ft. (he <= hr)
Building Width =	28.00	ft. (Normal to Building Ridge)
Building Length =	39.00	ft. (Parallel to Building Ridge)
Roof Type =	Gable	(Gable or Monoslope)
Topo. Factor, Kzt =	1.00	(Sect. 26.8 & Figure 26.8-1)
Ground Elevation Factor Ke	1.00	(Sect. 26.9, Ke=1 based on Note 1)
Direct. Factor, Kd =	0.85	(Table 26.6-1)
Enclosed? (Y/N)	Y	(Sect. 26.12 & Table 26.13-1)
Hurricane Region?	N	
Component Name =	Wall	(Girt, Siding, Wall, or Fastener)
Effective Area, Ae =	55	ft.^2 (Area Tributary to C&C)



Plan



Elevation

Resulting Parameters and Coefficients:

Roof Angle, θ =	40.60	deg.
Mean Roof Ht., h =	19.00	ft. (h = (hr+he)/2, for roof angle >10 deg.)

Wall External Pressure Coefficients, GCp:

GCp Zone 4 Pos. =	0.87	(Fig. 30.3-1)
GCp Zone 5 Pos. =	0.87	(Fig. 30.3-1)
GCp Zone 4 Neg. =	-0.97	(Fig. 30.3-1)
GCp Zone 5 Neg. =	-1.14	(Fig. 30.3-1)

Positive & Negative Internal Pressure Coefficients, GCpi (Table 26.13-1):

+GCpi Coef. =	0.18	(positive internal pressure)
-GCpi Coef. =	-0.18	(negative internal pressure)

If z <= 15 then: Kz = 2.01*(15/zg)^(2/α) , If z > 15 then: Kz = 2.01*(z/zg)^(2/α) (Table 26.10-1)

α =	9.50	(Table 26.11-1)
zg =	900	(Table 26.11-1)
Kh =	0.89	(Kh = Kz evaluated at z = h)

Velocity Pressure: qz = 0.00256*Kz*Ke*Kzt*Kd*V^2 (Sect. 26.10.2, Eq. 26.10-1)

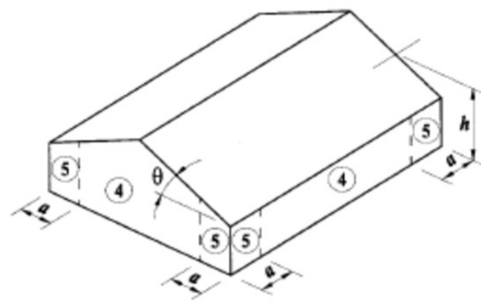
qh = 32.81 psf   qh = 0.00256*Kh*Kzt*Ke*Kd*V^2 (qz evaluated at z = h)

Design Net External Wind Pressures (Sect. 30.3 & 30.5):
For h <= 60 ft.: p = qh*((GCp) - (+/-GCpi)) (psf)
For h > 60 ft.: p = q*(GCp) - qi*(+/-GCpi) (psf)
where: q = qz for windward walls, q = qh for leeward walls and side walls
qi = qh for all walls (conservatively assumed per Sect. 30.5)

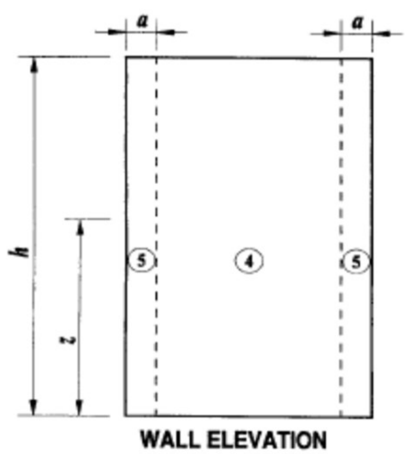
Wind Load Tabulation for Wall Components & Cladding							
Component	z (ft.)	Kh	qh (psf)	p = Net Design Pressures (psf)			
				Zone 4 (+)	Zone 4 (-)	Zone 5 (+)	Zone 5 (-)
Wall	0	0.89	32.81	34.43	-37.71	34.43	-43.26
	15.00	0.89	32.81	34.43	-37.71	34.43	-43.26
	20.00	0.89	32.81	34.43	-37.71	34.43	-43.26
For z = hr:	25.00	0.89	32.81	34.43	-37.71	34.43	-43.26
For z = he:	13.00	0.89	32.81	34.43	-37.71	34.43	-43.26
For z = h:	19.00	0.89	32.81	34.43	-37.71	34.43	-43.26

Notes: 1. (+) and (-) signs signify wind pressures acting toward & away from respective surfaces.
2. Width of Zone 5 (end zones), 'a' = 3.00 ft.
3. Per Code Section 30.2.2, the minimum wind load for C&C shall not be less than 16 psf.
4. References : a. ASCE 7-16, "Minimum Design Loads for Buildings and Other Structures".
b. "Guide to the Use of the Wind Load Provisions of ASCE 7-02"
by: Kishor C. Mehta and James M. Delahay (2004).

Wall Components and Cladding:



Wall Zones for Buildings with h <= 60 ft.



Wall Zones for Buildings with h > 60 ft.

SEISMIC BASE SHEAR AND VERTICAL SHEAR DISTRIBUTION

Per IBC 2018 and ASCE 7-16 Specifications

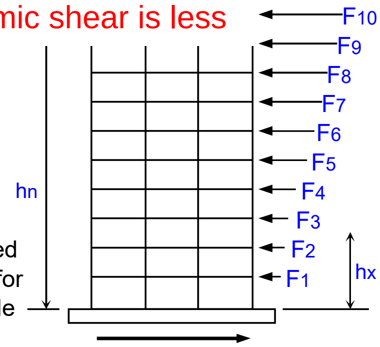
Using Equivalent Lateral Force Procedure for Regular Multi-Level Building/Structural Systems

Job Name:	Aroma Joe's Tycoga Dover Prototype V3.0	Subject:	Base Shear		
Job Number:	Top to Bottom Loading (Long dim of structure)	Originator:	SJP	Checker:	

Calais ss = 0.262 and s1 = 0.078 so the seismic shear is less

Input Data:

Risk Category =	II	IBC 2018 Table 1604.5
Importance Factor, I =	1.00	ASCE 7-16 Table 1.5-2
Soil Site Class =	D-default	ASCE 7-16 Table 20.1
Location Zip Code =		(not required)
Spectral Accel., S_S =	0.319	ASCE 7-16 Figures 22-1 to 22-11
Spectral Accel., S_1 =	0.076	ASCE 7-16 Figures 22-1 to 22-11
Long. Trans. Period, T_L =	6.000	sec. ASCE 7 Fig's. 22-12 to 22-18
Structure Height, h_n =	21.170	ft.
Actual Calc. Period, T_c =	0.197	sec. from independent analysis
Seismic Resist. System =	A15	Light-framed (wood) walls sheathed with wood structural panels rated for shear resistance (ASCE 7-16 Table 12.2-1)



V = Cs*W = Σ(Fi) = 1.17 kips

Seismic Base Shear

(Regular Bldg. Configurations Only)

Structure Weight Distribution:

No. of Seismic Levels = 2

Seismic Level x	Height, hx (ft.)	Weight, Wx (kips)
2	21.170	4.63
1	12.830	18.59

Walls = 8 psf * (235' *11') * 5
Roof = 20 psf * 21,718 sf

Total Weight, W = ΣW_x = 23.22 kips (ASCE 7-16 Section 12.7.2)

Results:

Site Coefficients:

F_a =	1.545	ASCE 7-16 Table 11.4-1
F_v =	2.400	ASCE 7-16 Table 11.4-2

Maximum Spectral Response Accelerations for Short and 1-Second Periods:

S_{MS} =	0.493	$S_{MS} = F_a*S_S$, ASCE 7 Eqn. 11.4-1
S_{M1} =	0.182	$S_{M1} = F_v*S_1$, ASCE 7 Eqn. 11.4-2

Design Spectral Response Accelerations for Short and 1-Second Periods :

S_{DS} =	0.329	$S_{DS} = 2*S_{MS}/3$, ASCE 7-16 Eqn. 11.4-3
S_{D1} =	0.122	$S_{D1} = 2*S_{M1}/3$, ASCE 7-16 Eqn. 11.4-4

(continued:)

Seismic Design Category:

Category(for S_{DS}) =	B	ASCE 7-16 Table 11.6-1
Category(for S_{D1}) =	B	ASCE 7-16 Table 11.6-2
Use Category =	B	Most critical of either category case above controls

Fundamental Period:

Period Coefficient, C_T =	0.020	ASCE 7-16 Table 12.8-2
Period Exponent, x =	0.75	ASCE 7-16 Table 12.8-2
Approx. Period, T_a =	0.197	sec., $T_a = C_T*h_n$^(x), ASCE 7 Section 12.8.2.1, Eqn. 12.8-7
Upper Limit Coef., C_u =	1.657	ASCE 7-16 Table 12.8-1
Period max., $T_{(max)}$ =	0.327	sec., $T_{(max)} = C_u*T_a$, ASCE 7 Section 12.8.2
Fundamental Period, T =	0.197	sec., T = Tc <= Cu*Ta, ASCE 7 Section 12.8.2

Seismic Design Coefficients and Factors:

Response Mod. Coef., R =	6.5	ASCE 7-16 Table 12.2-1
Overstrength Factor, Ω_o =	3	ASCE 7-16 Table 12.2-1
Defl. Amplif. Factor, C_d =	4	ASCE 7-16 Table 12.2-1
C_S =	0.051	$C_S = S_{DS}/(R/I)$, ASCE 7 Section 12.8.1.1, Eqn. 12.8-2
$C_{S(max)}$ =	0.095	For T<=TL, CS(max) = SD1/(T*(R/I)), ASCE 7 Eqn. 12.8-3
$C_{S(min)}$ =	0.014	CS(min) = 0.044*SDS*I >= 0.01, ASCE 7 Eqn. 12.8-5
Use: C_S =	0.051	$C_{S(min)} <= C_S <= C_{S(max)}$

Seismic Base Shear:

V =	1.17	kips, $V = C_S*W$, ASCE 7-16 Section 12.8.1, Eqn. 12.8-1

Seismic Shear Vertical Distribution:

Distribution Exponent, k = 1.00 — k = 1 for T<=0.5 sec., k = 2 for T>=2.5 sec.
k = (2-1)*(T-0.5)/(2.5-0.5)+1, for 0.5 sec. < T < 2.5 sec.

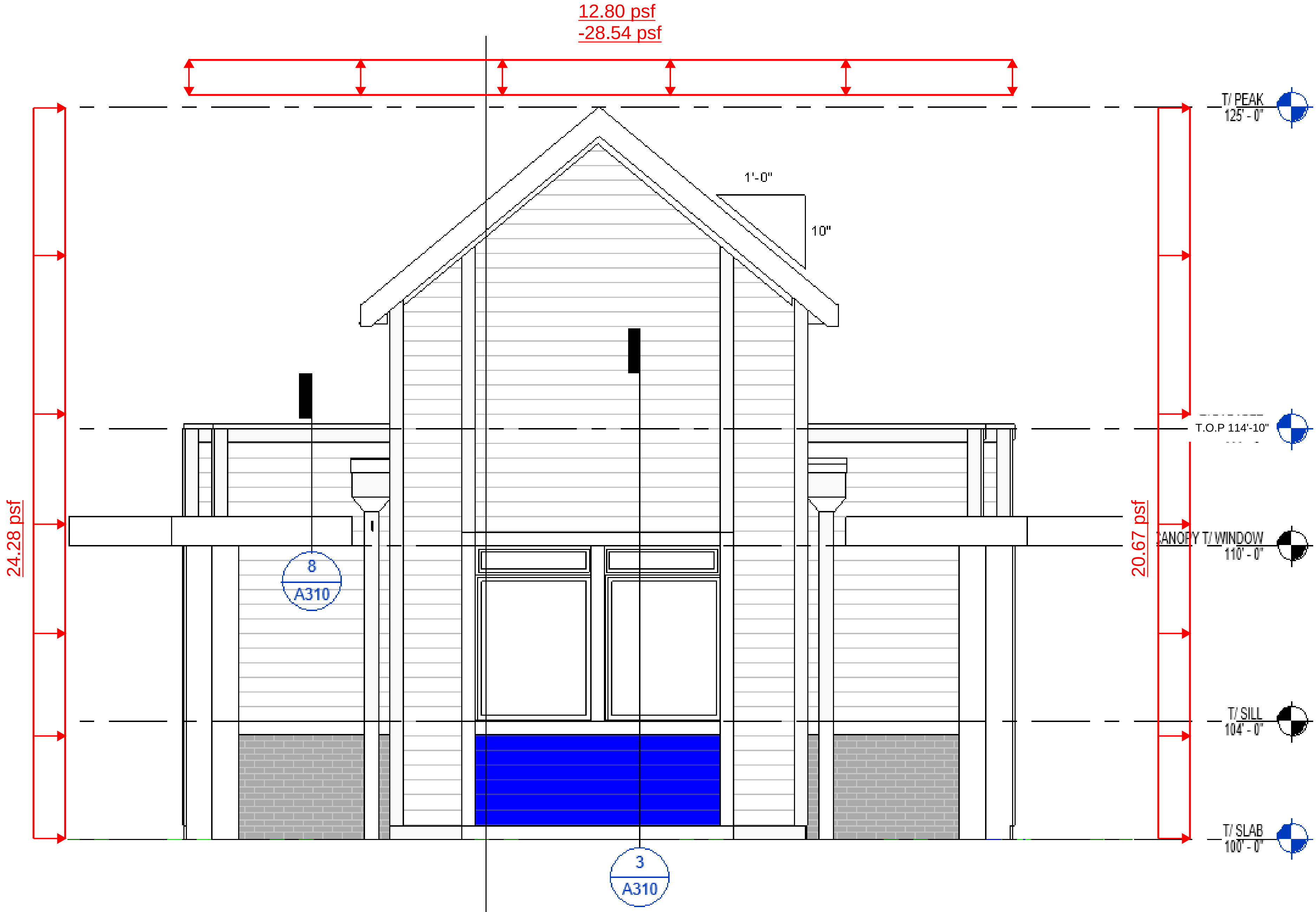
Lateral Force at Any Level: $F_x = C_{vx}*V$, ASCE 7-16 Section 12.8.3, Eqn. 12.8-11

Vertical Distribution Factor: $C_{vx} = W_x*h_x$^k/(ΣW_i*h_i^k), ASCE 7-16 Eqn. 12.8-12

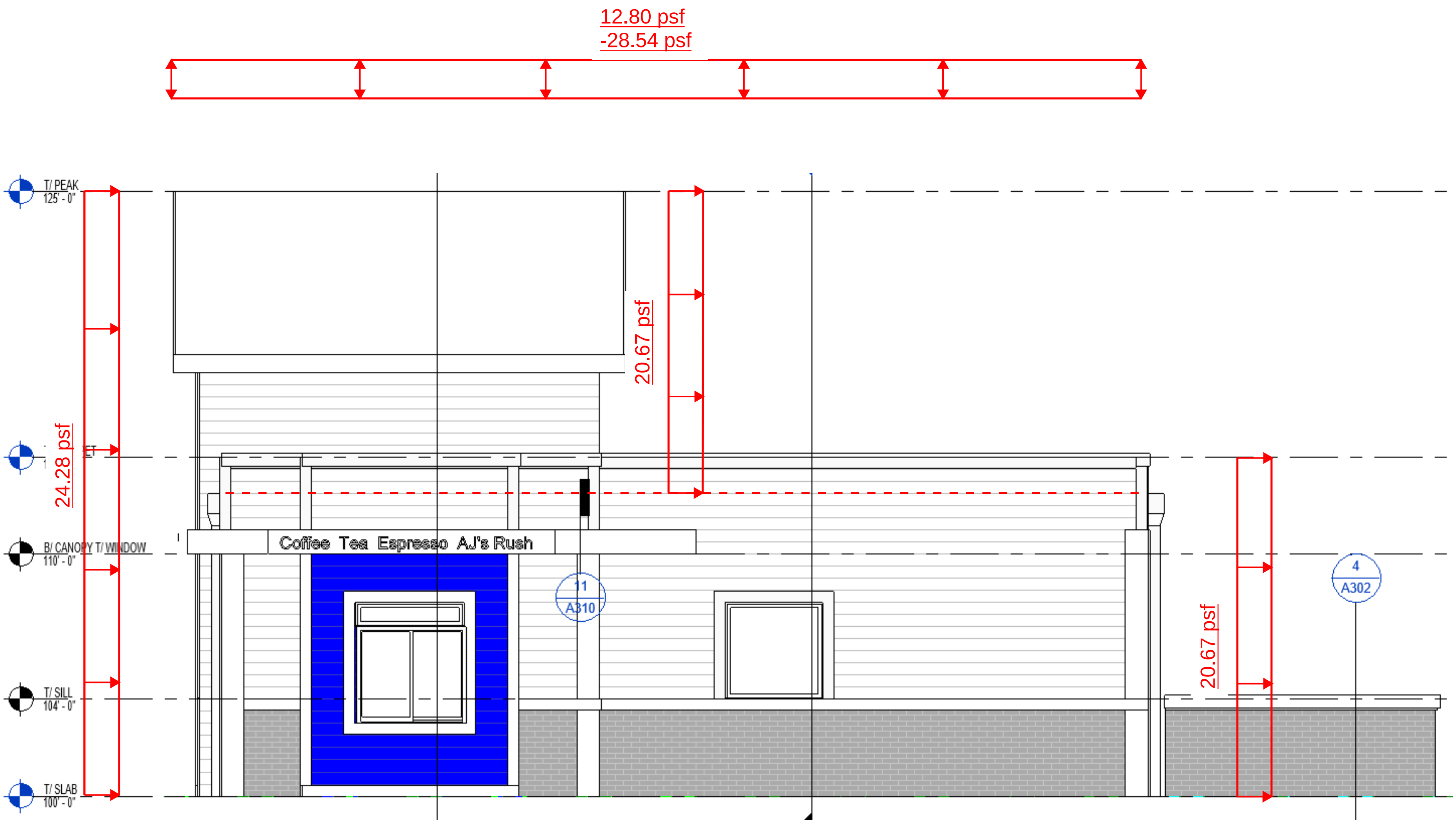
Seismic Level x	Weight, Wx (kips)	hx^k (ft.)	Wx*h^k (ft-kips)	Cvx (%)	Shear, Fx (kips)	Σ Story Shears
2	4.63	21.170	98.0	0.291	0.34	0.34
1	18.59	12.830	238.5	0.709	0.83	1.17
Σ =	23.22		336.5	1.000	1.17	

Comments:

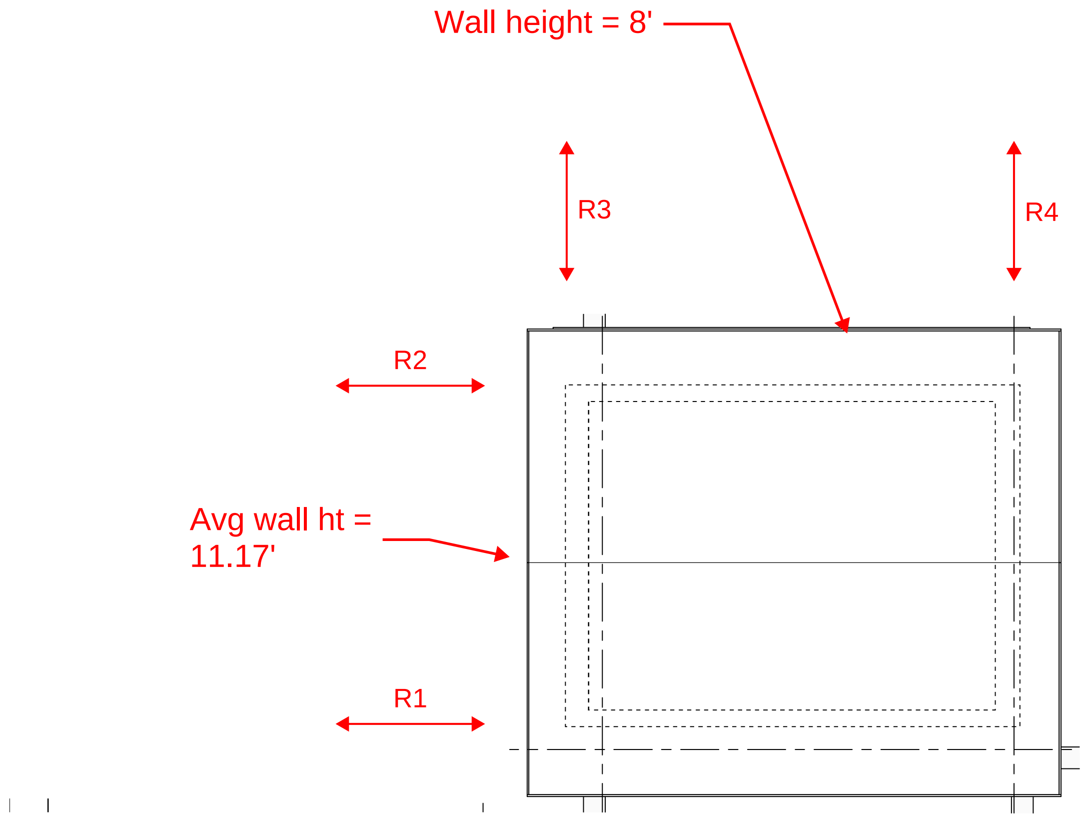
SHEAR WALL CALCULATION SETUP



Main Wind Force Resisting System Loading



Main Wind Force Resisting System Loading



Notes:
1. Wind Reactions are based on Ultimate Load pressures

<u>Wind Reactions</u>

R = Wall height*pressure*width

R1 = R2 = ((25'-13')*(25 psf+21 psf)*16.2')/2 = **4,470 lbs**
(1/2 of reaction to doghouse roof
1/2 of reaction to low roof, per wall)

R3 = R4 = ((25'-13')*(25 psf+21 psf)*17.6')/2 = **4,860 lbs**
(1/2 of reaction to doghouse roof
1/2 of reaction to low roof, per wall)

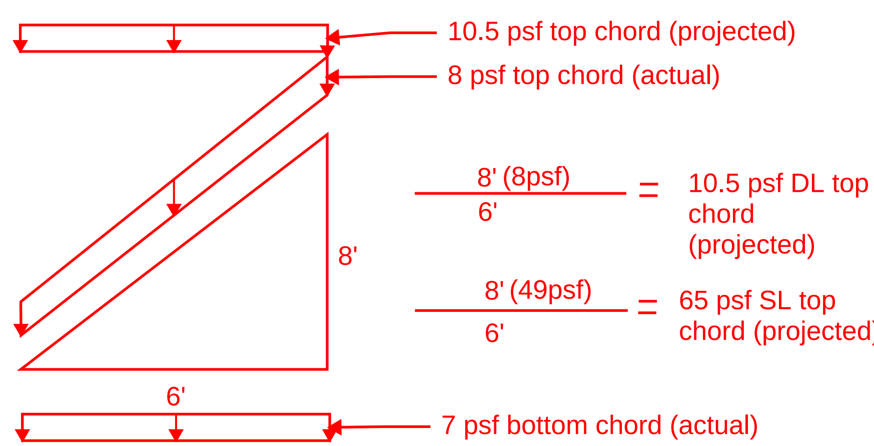
These reactions are to be added to low roof diaphragm

<u>Seismic Load at roof deck</u>

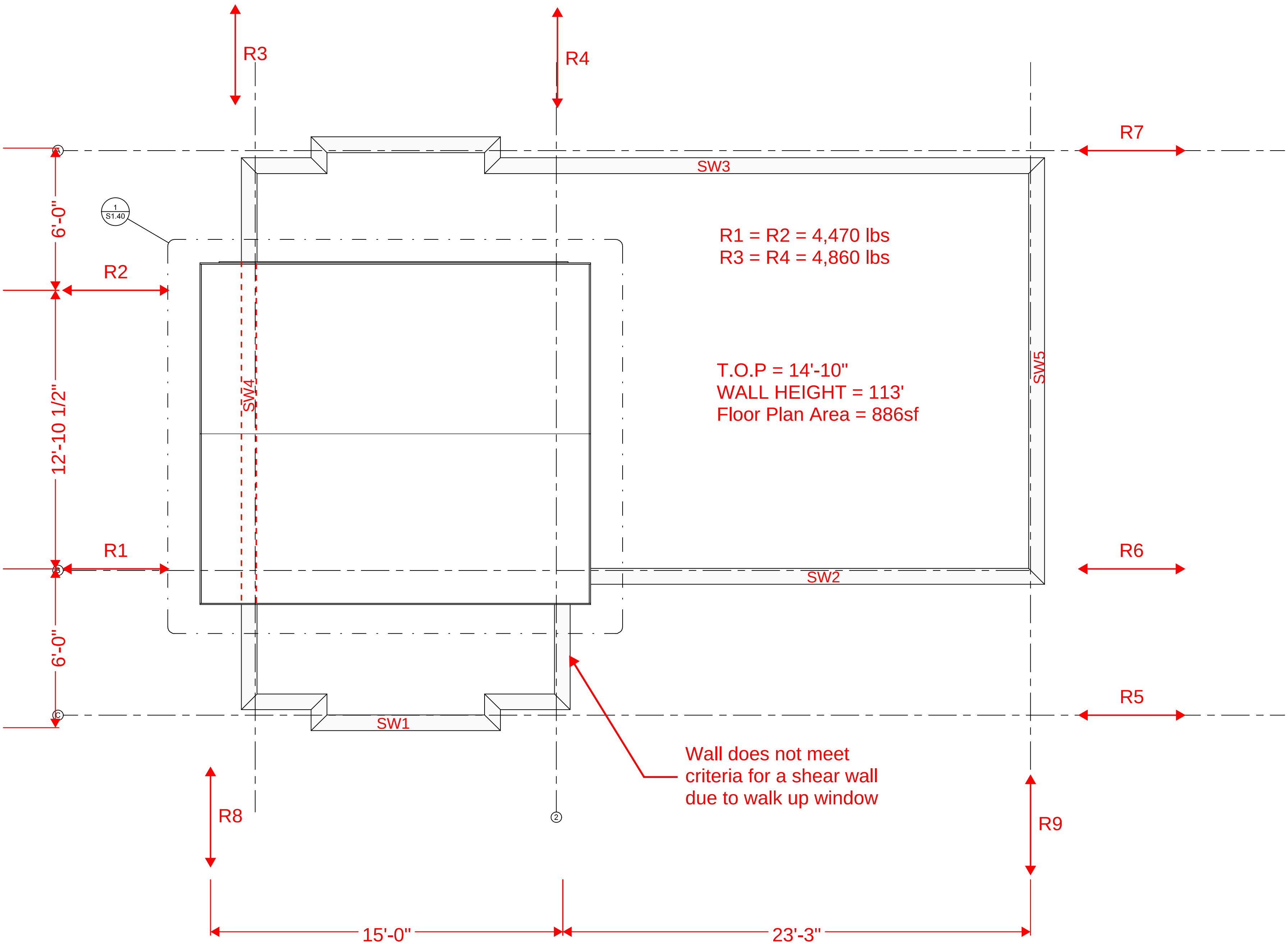
High Roof Area (along flat plan) = 300 sf
Top chord dead load weight = 8 psf
Bottom chord dead load weight = 10.5 psf (projected)
Dead load Weight at roof = 5,550 lbs

Wall Weight = 8 psf

Roof Snow Load = 49psf



Roof Load Left to Right = 5,500 + (11.17'/2*14'*8 psf)*2 = 6,750 lbs
Roof Load Top to Bottom = 5,500 + (8'/2*15.25'*8 psf)*2 = 6,475 lbs



R3
R4
R7
SW3
R1 = R2 = 4,470 lbs
R3 = R4 = 4,860 lbs
6'-0"
1
S1.40
R2
SW4
SW5
T.O.P = 14'-10"
WALL HEIGHT = 113'
Floor Plan Area = 886sf
12'-10 1/2"
R1
R6
SW2
6'-0"
R5
SW1
Wall does not meet
criteria for a shear wall
due to walk up window
R8
2
R9
15'-0"
23'-3"

Uniform Wind load at roof deck elevation
w = (25 psf+21 psf)*(13'+1.83')^2/(2*13') = 390 plf

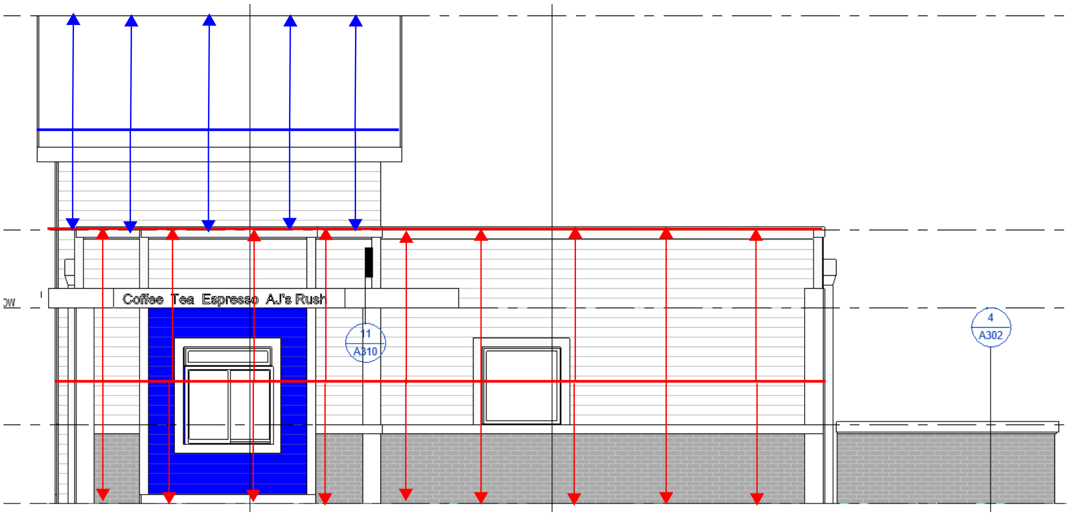
R5 = 390 plf*6'/2 = **1,170 lbs**

R6 = 390 plf*(6'+12.9')/2+R1+6'*R2/(6'+12.9)
= 3,685 lbs + 4,470 lbs + 1,420 lbs
= **9,575 lbs**

R7 = 390 plf*(6'+12.9')/2+12.9'*R2/(12.9'+6') =
= 3,685 lbs + 3,050 lbs
= **6,735 lbs**

Note: Reaction for R8 at the low roof is resisted by the inset walls while the high roof reaction is resisted by the outset wall.
R8 = (from low walls) = 390 plf*(38.25'/2) + R4*(23.25'/38.25') = **10,415 lbs**
R8 = (from dog house) = **4,860 lbs**

R9 = 390 plf*(38.25/2) + R4*(15'/38.25') = **9,365 lbs**



<u>Seismic load at roof deck elevation</u>

Low Roof Area = 880 sf
Top chord weight = 8 psf
Bottom chord weight = 7 psf
Weight at roof = 13,200 lbs

Wall weight for walls running top to bottom (short direction of building)
W = 8 psf*(28'*10'/2)*2 = 2,240 lbs

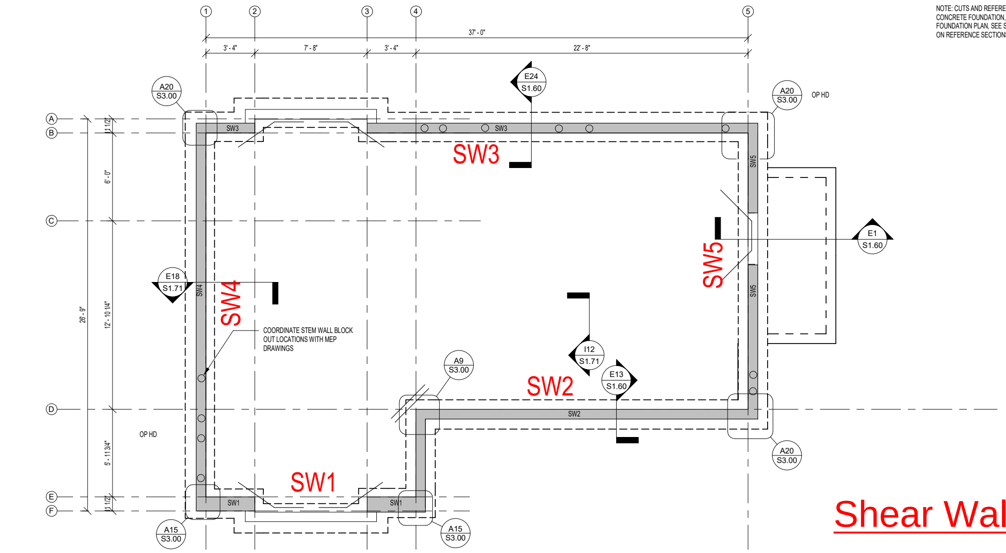
Wall weight for walls running left to right (long direction of building)
W = 8 psf*(38.25'*10'/2)*2 = 3,060 lbs

Wall weight for doghouse
W = 8 psf*(13'*11.17/2')*2 = 1,160 lbs (short direction of building)
W = 8 psf*(17.6'*8'/2)*2 = 1,130 lbs (long direction of building)

Weight Dist. to DH Diaphragm = 1/2 DH Wall Weight + DH Roof Weight
= (1,160 +1,130)/2 + 12psf * (16.2' *17.7')
= 4585lbs

Weight of Dist. to Lower Diaphragm = 1/2 Main Wall Weight + Low Roof Weight +1/2 DH Wall Weight
= (2,240 + 3,060)/2 + 15psf*(886sf) + (1,160 +1,130)/2
= 17085lbs

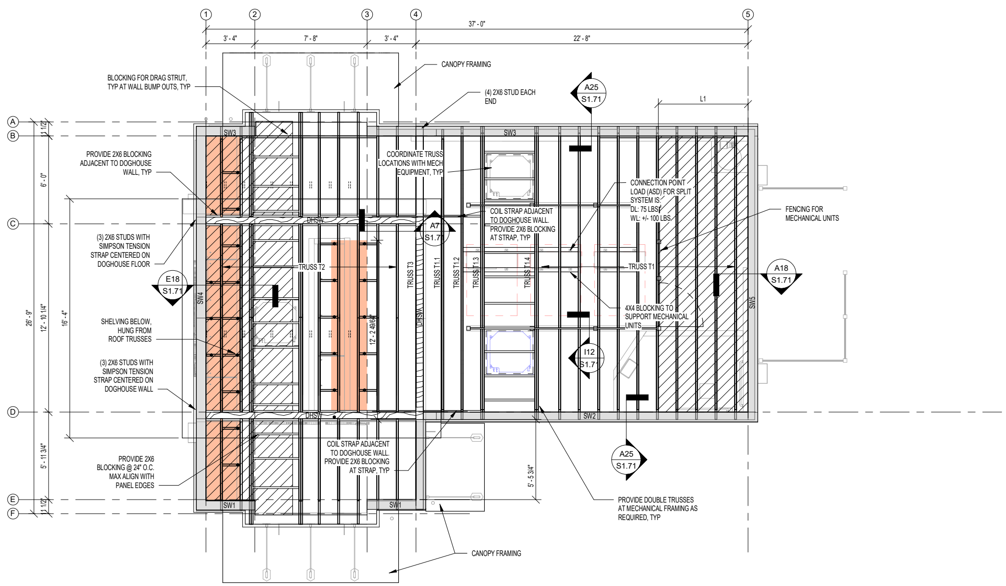
NOTE: CUTS AND REFERENCES SHOWN ARE FOR CAST-IN-PLACE CONCRETE FOUNDATION, SHEET S1.60. FOR ALTERNATE CMU FOUNDATION PLAN, SEE SHEET S1.60A. FOR THE CMU ALTERNATE, ON REFERENCE SECTIONS, SUBSTITUTE S1.60A FOR S1.60.



Shear Wall Framing Plan

F1 SCALE 1/4" = 1'-0"

FOUNDATION PLAN



A1 SCALE 1/4" = 1'-0"

ROOF FRAMING PLAN

Aroma Joes Calais
TKA #24626

FOUNDATION PLAN NOTES

1. SLAB ON GRADE SHALL BE 4" THICK MINIMUM w/ #3 BARS EACH WAY @ 12" O.C. AT MIDSLAB ON TOP OF PROPERLY PREPARED SUBGRADE PER SOIL REPORT.
   A. AS A BID ALTERNATE FOR SLAB REINFORCING, USE HELIX 5-25 MICRO REBAR AT A RATE OF 18 LBS PER CUBIC YARD.
2. MAXIMUM SPACING OF CONSTRUCTION / CONTROL JOINTS SHALL BE 12' - 0".
3. FOOTING SIZES PER PLAN
4. ALL ANCHOR BOLTS AND HOLD DOWN ANCHORS SHALL BE TIED IN PLACE PRIOR TO FOUNDATION INSPECTION.
5. ALL SILL PLATES TO BE PRESSURE TREATED DOUG.FIR.
6. FOR DIMENSIONS AND DETAILS NOT SHOWN, SEE ARCHITECTURAL DRAWINGS.
7. SEE ARCHITECTURAL DRAWINGS, FOR SLAB SLOPES AND FLOOR DRAINS.
8. INDICATES SHEAR WALL PER SCHEDULE ON SHEET S3.00.
   A. SEE SHEET S3.00 FOR SHEARWALL SCHEDULE.
   B. PROVIDE MINIMUM CONCRETE EDGE DISTANCE FOR HOLDDOWN ANCHOR BOLT PER DETAIL D15/ S3.00
9. PRIOR TO CONTRACTOR REQUESTING A BUILDING DEPT. FOUNDATION INSPECTION, THE GEOTECHNICAL ENGINEER SHALL ADVISE THE BUILDING OFFICIAL IN WRITING THAT:
   A. THE BUILDING PAD WAS PREPARED IN ACCORDANCE WITH SOILS REPORT.
   B. THE UTILITY TRENCHES HAVE BEEN PROPERLY BACKFILLED AND COMPACTED.
   C. THE FOUNDATION EXCAVATIONS, SOIL EXPANSION CHARACTERISTICS & BEARING CAPACITY CONFORM TO THE SOILS REPORT.
10. GRID LINES INDICATE INSIDE FACE OF CONCRETE FOUNDATION STEM WALL AND 2"X6" STUD WALL.
11. SEE S1.60 FOR TYPICAL FOUNDATION DETAILS.
12. TOP OF FOOTING ELEVATION 97' - 0" U.N.O.
13. FOR CMU FOUNDATION ALTERNATE, SEE SHEET S1.60A.

ROOF FRAMING PLAN NOTES:

1. ALL ROOF SHEATHING SHALL BE 3/4" THICK (40/20) CDX, TONGUE AND GROOVE W/

   L1 L2 L1

AREA A1:	AREA A2:
L1 = 6' - 0"	L2 = REMAINDER
BLOCKED	UNBLOCKED
BN 10D @ 4" O.C.	BN 10D @ 6" O.C.
EN 10D @ 6" O.C.	EN 10D @ 6" O.C.
FIELD 10D @ 12" O.C.	FIELD 10D @ 12" O.C.

2. INDICATES FRAMING DIRECTION.
3. ALL STRUCTURAL WOOD WALLS SHALL BE 2X6 VERTICAL STUDS @ 16" O.C. (UNO)
4. MINIMUM HEADERS, U.N.O. SHALL BE:
   A. 6X6 - UP TO 4'-0" OPENING SPAN
   B. 6X8 - UP TO 6'-0" OPENING SPAN
   C. 6X10 - UP TO 8'-0" OPENING SPAN
5. INDICATES BLOCKED DIAPHRAGM.
6. FOR ROOF SLOPES & OPENINGS, REFER TO ARCHITECTURAL ROOF PLAN.
7. (10) 16d NAILS EACH SIDE OF TOP PLATE SPLICES (TYPICAL, U.N.O.). SEE DETAIL H25/S1.70
8. (F) - INDICATES FLUSH BEAMS.
9. BOUNDARY NAILING IS REQUIRED AT ALL DRAG STRUTS & EDGE NAILING IS REQUIRED AT ALL SHEARWALLS, TYP.
10. INDICATES SHEAR WALL PER SHEAR WALL SCHEDULE ON SHEET S3.00.
    A. # INDICATES SHEAR WALLS AND SHEAR TRANSFERS FOR WALLS BELOW ROOF FRAMING. SEE SHEET S3.00 FOR SHEARWALL SCHEDULE
11. SHEARWALLS ARE REQUIRED TO BE BLOCKED.
12. ALL NAILS ARE COMMON, U.N.O.
13. (1) 2x IS REQUIRED MINIMUM UNDER BEAMS AND HEADERS, U.N.O.
14. MST & CS SIMPSON STRAPS SHALL BE LOCATED ON TOP OF DOUBLE TOP PLATES.
15. ENGINEERED WOOD TRUSSES:
    A. EACH TRUSS SHALL BE LEGIBLY BRANDED (MARKED) OR OTHERWISE PERMANENTLY AFFIXED WITH THE FOLLOWING INFORMATION LOCATED WITHIN TWO FEET OF THE CENTER SPAN ON THE FACE OF THE BOTTOM CHORD:
       1.) NAME OF TRUSS MANUFACTURING COMPANY
       2.) DESIGN LOAD AS SHOWN ON CRITERIA BELOW.
       3.) SPACING OF TRUSS
       4.) DESIGNED SPAN ON THE FACE OF THE BOTTOM CHORD.
    B. BOTTOM CHORD BRACING IS REQUIRED AT 10'-0" O.C. MAXIMUM.
    C. TRUSS DRAWINGS MUST BE APPROVED WITHIN FIFTY (50) DAYS OF PERMIT ISSUANCE.
    D. SUBMIT APPROVED SHOP DRAWINGS & CALCULATIONS TO CITY FOR PLAN CHECK & APPROVAL.
    E. APPROVED TRUSS DRAWINGS MUST BE ON JOB SITE FOR INSPECTION PURPOSES.
16. ROOF JOISTS ARE SPACED AT 16" O.C. UNLESS NOTED OTHERWISE. JOIST TO BEAR DIRECTLY OVER STUD WALL. ADJUST TRUSS SPACING AS REQUIRED IN DOGHOUSE TO ACCOMMODATE DUCT PENETRATIONS, MAX TRUSS SPACING 24" O.C.
17. ● INDICATES LIVE LOAD OF 650 LBS TO BE CARRIED BY TRUSS. SPACING BETWEEN REACTIONS IS 30" O.C., UNO.

NOT FOR CONSTRUCTION

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Aroma Joe's

AROMA JOE'S
6 CASCADE ROAD
SACO, ME 04072

BID AND PERMIT SET
2024.05.17

Revision Schedule

#	Date	Description

PROJECT #
24625

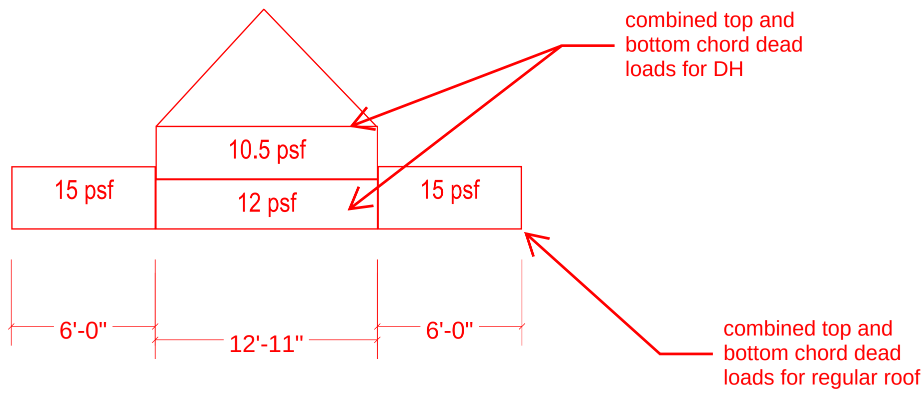
FOUNDATION PLAN & ROOF FRAIMING PLAN

SHEET NUMBER
S1.10

SHEAR WALL 1 CALCULATIONS

DH = Dog House

ROOF DEAD LOAD



TRIB: 26.8'

LENGTH: 15'

DEAD LOAD:

W/DH:

$$\frac{(2*15psf\,(6'\text{-}0") + 22.5psf(12'\text{-}11")}{2} = 0.236klf$$

ROOF LIVE LOAD :

$$\frac{20psf*(26.8')}{2} = 0.268klf$$

SNOW LOAD :

W/DH:

$$\frac{65psf*(26.8')}{2} = 0.871klf$$

WIND LOAD :

$$\frac{R5}{15'} = \frac{1170}{15'} = 0.078klf$$

SEISMIC LOAD :

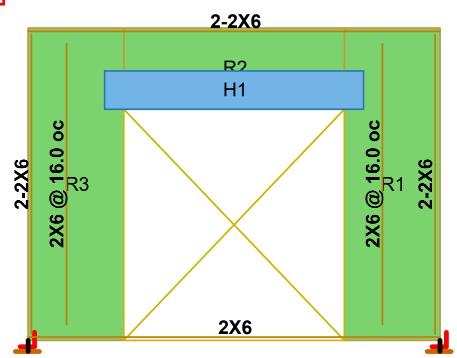
Seismic Base shear from excel spreadsheet

$$\frac{1.17+0.34}{2(15')} = 0.05klf$$

SHEAR WALL 1 (SW1)

Detail Report: WP1

Wood Wall



GENERAL		GEOMETRY		MATERIALS		
Code:	AWC NDS-18 / SDPWS-15:ASD	**Total Height (ft):**	10.83	**Top Pl.**	DF	2-2X6
		Total Length (ft):	14.33	**Sill**	DF	2X6
Design Method:	Perforated	**Wall H/W Ratio:**	0.76	**Wall Stud**	DF	2X6
Wall Material:	DF	**Max Opening Ht (ft):**	8	**Chord**	DF	2-2X6
Panel Schedule:	AWC 2015 OSB 0.469 (10d)	**Open/Wall Ht Ratio:**	0.74			
		Full Ht Sheathed (ft):	6.664			
Optimize HD:	Yes	**% Full Ht Sheathed:**	46.50			
HD Manufacturer:	SIMPSON					
HD Eccentricity (in):	0					

ENVELOPED RESULTS

Use HD5B-3

Shear Panel	Shear UC	Shear LC	Hold Down	Hold Down UC	Hold Down LC	Chord UC	Chord LC	Stud UC	Stud LC
S1_15/32_10d@6	0.519	4 (W)	LTT19_0.14 8x1.5_SPF-HF	0.989	11 (W)	0.148	11 (W)	0.400	3

STUDS

Required Cap (k):	2.705	**Governing LC:**	3	**Studs in Region:**	3
Provided Cap (k):	6.76	**Gov Region:**	1	**K:**	1.00
Ratio:	0.4	**Stud Spacing (in):**	16		

CHORDS

Max Comp Force (k):	2.004	**Gov Comp LC:**	11	**Tens Ratio:**	0.048
Comp Capacity (k):	13.52	**Max Tens Force (k):**	0.695	**Gov Tens LC:**	11
Comp Ratio:	0.148	**Tens Capacity (k):**	14.479		

SHEAR PANEL

Selected Shear Panel:	S1_15/32_10d@6	**Specific Gravity Adjustment Factor:**	1.00	**Panel Grade:**	St-I
Shear Ratio:	0.519	**Hold-Down Factor:**	1.00	**Panel Thick (in):**	0.469
Governing LC:	4 (Wind)	**Shear Stiffness Adjustment Factor:**	1.00	**Reqd Pen (in):**	1.500
Total Shear (k):	0.688	**Wall Capacity Adjustment Factor (2w/h):**	0.62	**Nail Size:**	10d
Max Unit Shear (k/ft):	0.168	**Aspect Ratio Factor:**	1.00	**Reqd. Spacing (in):**	6
Shear Cap. (k/ft):	0.340	**Governing H/W Ratio Factor:**	1.00	**Num Sides:**	One
Adjusted Shear Cap. (k/ft):	0.323	**Nailing Capacity Increase for Wind:**	1.4	**Over Gyp. Board:**	No
Specific Gravity:	0.5	**Shear Capacity Adjustment Factor (Co):**	0.68		
		Total Area of Openings (Ao) (ft^2):	61.327		
		Sheathing Area Ratio (r):	0.42		

SW1 has an additional 1/2" gyp with a shear capacity of 175 plf on the inside face of the wall plus the R9 zip system on the exterior wall with a shear capacity of 336 plf. Both combine to 511plf which is sufficient to withstand the total shear

DEFLECTIONS

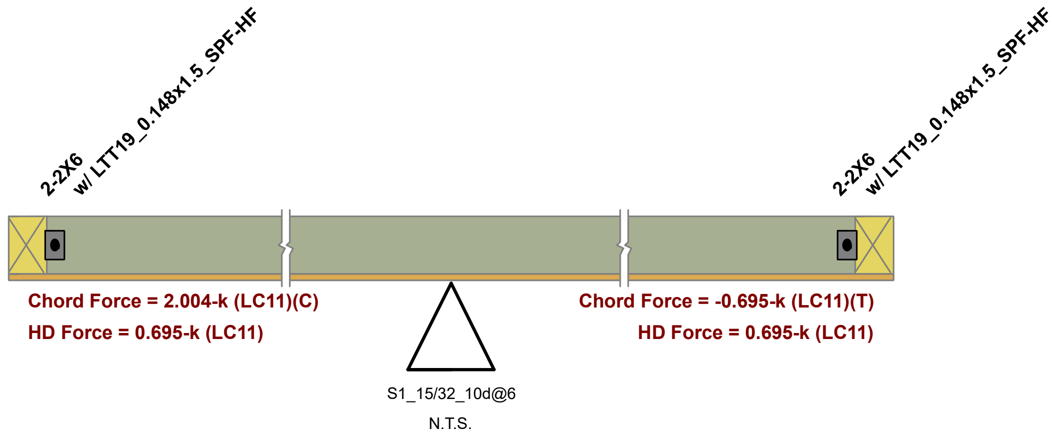
Elastic (in):	0.009	**Shear (in):**	0.083
HD (in):	0.289	**Total (in):**	0.381

HOLD-DOWNS

Selected Hold-Down:	LTT19_0.148x1.5_SPF-HF	**Governing LC:**	11	**AB Diameter (in):**	0.500
Required Cap. (k):	0.695	**CD factor:**	1	**Fastener Size:**	16d
Base Capacity (k):	0.703	**Reqd. Chord Mat.:**	Hem Fir	**Num Fasteners:**	8
Adjusted Cap. (k):	0.703	**Reqd. Chord Thk. (in):**	1.50		
Ratio:	0.989	**Raised:**	No		

Use PAB5 5/8" Dia anchor
Tall = 4,270 lbs (ASD)
de = 4"

CROSS SECTION DETAILING



Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	10.83	0	
3	N3	14.33	10.83	0	
4	N4	14.33	0	0	

Node Boundary Conditions

	Node Label	Z [k/in]
1	N2	Reaction
2	N3	Reaction

Wall Panel Distributed Loads (BLC 1 : Dead Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.236	-0.236	0	14.33

Wall Panel Distributed Loads (BLC 2 : Roof Live Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.268	-0.268	0	14.33

Wall Panel Distributed Loads (BLC 3 : Snow Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.871	-0.871	0	%100

Wall Panel Distributed Loads (BLC 4 : Wind)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.08	0.08	0	14.33

Wall Panel Distributed Loads (BLC 5 : Seismic)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.05	0.05	0	14.33

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Dead Load	Yes	Y	DL	1						
2	DL + RLL	Yes	Y	DL	1	RLL	1				
3	DL + SL	Yes	Y	DL	1	SL	1				
4	DL + 0.6WL	Yes	Y	DL	1	WLX	0.6				
5	DL - 0.6WL	Yes	Y	DL	1	WLX	-0.6				
6	DL +0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	0.45	RLL	0.75		
7	DL - 0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	-0.45	RLL	0.75		
8	DL +0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	0.45	SL	0.75		
9	DL - 0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	-0.45	SL	0.75		
10	0.6DL + 0.6WL	Yes	Y	DL	0.6	WLX	0.6				
11	0.6DL - 0.6WL	Yes	Y	DL	0.6	WLX	-0.6				
12	IBC 21/ASCE ASD 8 (a)	Yes	Y	DL	1	ELX	0.7				
13	IBC 21/ASCE ASD 8 (b)	Yes	Y	DL	1	ELZ	0.7				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
14	IBC 21/ASCE ASD 9 (a)	Yes	Y	DL	1	ELX	0.525	LL	0.75	LLS	0.75
15	IBC 21/ASCE ASD 9 (b)	Yes	Y	DL	1	ELZ	0.525	LL	0.75	LLS	0.75
16	IBC 21/ASCE ASD 10 (a)	Yes	Y	DL	0.6	ELX	0.7				
17	IBC 21/ASCE ASD 10 (b)	Yes	Y	DL	0.6	ELZ	0.7				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0	17	0	17	0	17	0	17	0	17	0	17
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N3	max	0	17	0	17	0	17	0	17	0	17	0	17
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	WP1	max	0.688	11	16.221	3	0	17	0	17	0	17	7.449	10
6		min	-0.688	4	2.244	11	0	1	0	1	0	1	-7.449	5
7	Totals:	max	0.688	11	16.221	3	0	17						
8		min	-0.688	4	2.244	11	0	1						

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Axial)

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC	Chord Size	Chord Axial Check	Gov LC
1	WP1	N/A	2X6	16	0.4	3	2-2X6	0.148	11

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (In-Plane)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Force[k/ft]	Gov LC	Hold-Down Label	Chord Strap Label	Tension Check	Tie-Down Force[k]	Gov LC
1	WP1	S1_15/32_10d@6	N/A	0.519	0.168	4	LTT19_0.148x1.5_SPF-HF	NC	0.989	0.695	11

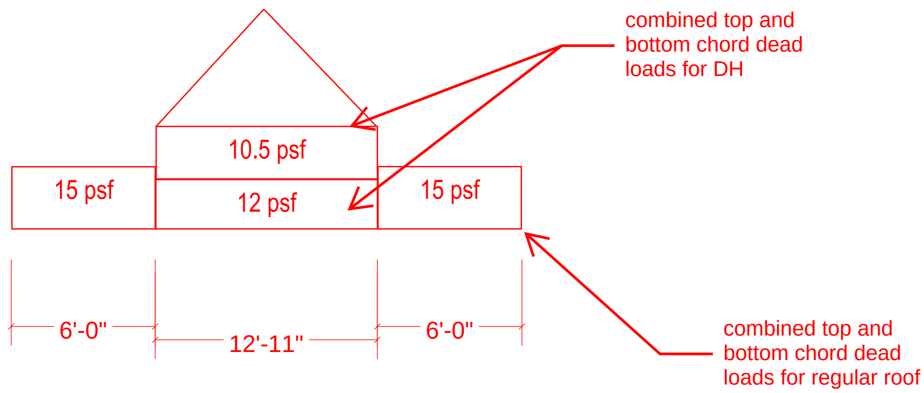
AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Header)

	Wall Panel	Header Label	Design Rule	Size	Bend UC	Shear UC	Gov LC
1	WP1	H1	Typical	6x8	0.612	0.495	3 (bending)

SHEAR WALL 2 CALCULATIONS

DH = Dog House

ROOF DEAD LOAD



TRIB: 20.3'   LENGTH: 22.67'

DEAD LOAD:

W/DH:

$$\frac{15\text{psf }(20.3')}{2} = 0.153\text{klf}$$

ROOF LIVE LOAD :

$$\frac{20\text{psf}*(20.3')}{2} = 0.203\text{klf}$$

SNOW LOAD :

W/DH:

$$\frac{65\text{psf}*(20.3')}{2} = 0.660\text{klf}$$

WIND LOAD :

$$\frac{R6}{22.67'} = \frac{9575}{22.67'} = 0.422\text{klf}$$

SEISMIC LOAD :

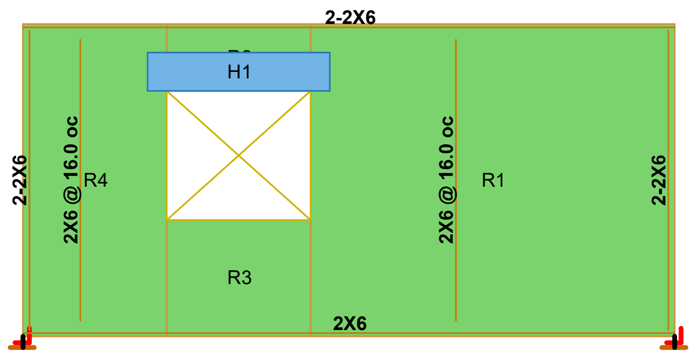
Seismic Base shear from excel spreadsheet

$$\frac{1.17}{2(22.67')} = 0.026\text{klf}$$

SHEAR WALL 2 (SW2)

Detail Report: WP1

Wood Wall



GENERAL		GEOMETRY		MATERIALS		
Code:	AWC NDS-18 / SDPWS-15:ASD	**Total Height (ft):**	10.83	**Top Pl.**	DF	2-2X6
Design Method:	Perforated	**Total Length (ft):**	22.67	**Sill**	DF	2X6
Wall Material:	DF	**Wall H/W Ratio:**	0.48	**Wall Stud**	DF	2X6
Panel Schedule:	User Selected	**Max Opening Ht (ft):**	4.5	**Chord**	DF	2-2X6
Optimize HD:	Yes	**Open/Wall Ht Ratio:**	0.42			
HD Manufacturer:	SIMPSON	**Full Ht Sheathed (ft):**	17.67			
HD Eccentricity (in):	0	**% Full Ht Sheathed:**	77.94			

Use HD5B-3

ENVELOPED RESULTS

Shear Panel	Shear UC	Shear LC	Hold Down	Hold Down UC	Hold Down LC	Chord UC	Chord LC	Stud UC	Stud LC
S1_15/32_10d@6	0.724	4 (W)	HD5B_2.5_ DF-SP	0.991	10 (W)	0.325	10 (W)	0.185	3

STUDS

Required Cap (k):	1.248	**Governing LC:**	3	**Studs in Region:**	5
Provided Cap (k):	6.76	**Gov Region:**	4	**K:**	1.00
Ratio:	0.185	**Stud Spacing (in):**	16		

CHORDS

Max Comp Force (k):	3.749	**Gov Comp LC:**	10	**Tens Ratio:**	0.188
Comp Capacity (k):	11.523	**Max Tens Force (k):**	2.323	**Gov Tens LC:**	10
Comp Ratio:	0.325	**Tens Capacity (k):**	12.34		

SHEAR PANEL

Selected Shear Panel:	S1_15/32_10d@6	**Specific Gravity Adjustment Factor:**	1.00	**Panel Grade:**	St-I
Shear Ratio:	0.724	**Hold-Down Factor:**	1.00	**Panel Thick (in):**	0.469
Governing LC:	4 (Wind)	**Shear Stiffness Adjustment Factor:**	1.00	**Reqd Pen (in):**	1.500
Total Shear (k):	5.74	**Wall Capacity Adjustment Factor (2w/h):**	0.92	**Nail Size:**	10d
Max Unit Shear (k/ft):	0.332	**Aspect Ratio Factor:**	1.00	**Reqd. Spacing (in):**	6
Shear Cap. (k/ft):	0.340	**Governing H/W Ratio Factor:**	1.00	**Num Sides:**	One
Adjusted Shear Cap. (k/ft):	0.459	**Nailing Capacity Increase for Wind:**	1.4	**Over Gyp. Board:**	No
Specific Gravity:	0.5	**Shear Capacity Adjustment Factor (Co):**	0.96		
		Total Area of Openings (Ao) (ft^2):	22.5		
		Sheathing Area Ratio (r):	0.89		

SW2 has an additional 1/2" gyp with a shear capacity of 175 plf on the inside face of the wall plus the R9 zip system on the exterior wall with a shear capacity of 336 plf. Both combine to 511plf which is sufficient to withstand the total shear

DEFLECTIONS

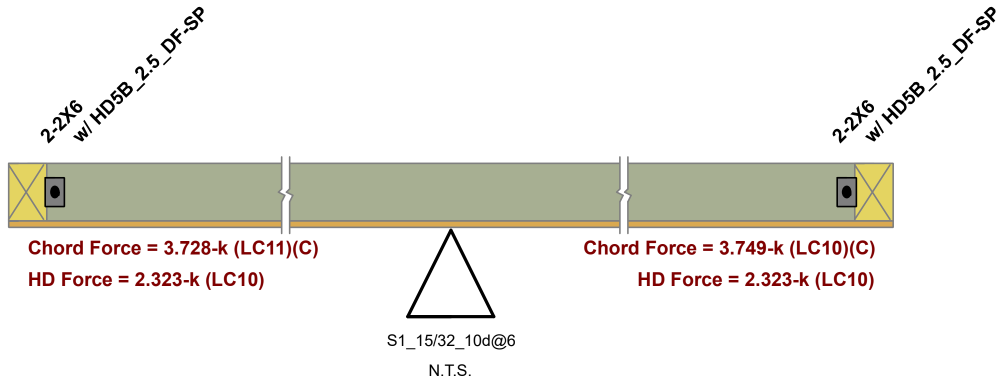
Elastic (in):	0.007	**Shear (in):**	0.163
HD (in):	0.078	**Total (in):**	0.249

HOLD-DOWNS

Selected Hold-Down:	HD5B_2.5_DF-SP	**Governing LC:**	10	**AB Diameter (in):**	0.500
Required Cap. (k):	2.323	**CD factor:**	1	**Fastener Size:**	16d
Base Capacity (k):	2.344	**Reqd. Chord Mat.:**	Douglas Fir	**Num Fasteners:**	8
Adjusted Cap. (k):	2.344	**Reqd. Chord Thk. (in):**	2.50		
Ratio:	0.991	**Raised:**	No		

Use PAB5 5/8" Dia anchor
Tall = 4,270 lbs (ASD)
de = 4"

CROSS SECTION DETAILING



Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	10.83	0	
3	N3	22.67	10.83	0	
4	N4	22.67	0	0	

Node Boundary Conditions

	Node Label	Z [k/in]
1	N2	Reaction
2	N3	Reaction

Wall Panel Distributed Loads (BLC 1 : Dead Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.153	-0.153	0	22.67

Wall Panel Distributed Loads (BLC 2 : Roof Live Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.203	-0.203	0	22.67

Wall Panel Distributed Loads (BLC 3 : Snow Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.66	-0.66	0	%100

Wall Panel Distributed Loads (BLC 4 : Wind)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.422	0.422	0	22.67

Wall Panel Distributed Loads (BLC 5 : Seismic)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.02	0.02	0	22.67

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Dead Load	Yes	Y	DL	1						
2	DL + RLL	Yes	Y	DL	1	RLL	1				
3	DL + SL	Yes	Y	DL	1	SL	1				
4	DL + 0.6WL	Yes	Y	DL	1	WLX	0.6				
5	DL - 0.6WL	Yes	Y	DL	1	WLX	-0.6				
6	DL +0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	0.45	RLL	0.75		
7	DL -0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	-0.45	RLL	0.75		
8	DL +0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	0.45	SL	0.75		
9	DL -0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	-0.45	SL	0.75		
10	0.6DL + 0.6WL	Yes	Y	DL	0.6	WLX	0.6				
11	0.6DL - 0.6WL	Yes	Y	DL	0.6	WLX	-0.6				
12	IBC 21/ASCE ASD 8 (a)	Yes		DL	1	ELX	0.7				
13	IBC 21/ASCE ASD 8 (b)	Yes		DL	1	ELZ	0.7				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
14	IBC 21/ASCE ASD 9 (a)	Yes		DL	1	ELX	0.525	LL	0.75	LLS	0.75
15	IBC 21/ASCE ASD 9 (b)	Yes		DL	1	ELZ	0.525	LL	0.75	LLS	0.75
16	IBC 21/ASCE ASD 10 (a)	Yes		DL	0.6	ELX	0.7				
17	IBC 21/ASCE ASD 10 (b)	Yes		DL	0.6	ELZ	0.7				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0	17	0	17	0	17	0	17	0	17	0	17
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N3	max	0	17	0	17	0	17	0	17	0	17	0	17
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	WP1	max	5.74	11	19.24	3	0	17	0	17	0	17	62.477	4
6		min	-5.74	4	2.567	11	0	1	0	1	0	1	-61.977	11
7	Totals:	max	5.74	11	19.24	3	0	17						
8		min	-5.74	4	2.567	11	0	1						

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Axial)

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC	Chord Size	Chord Axial Check	Gov LC
1	WP1	N/A	2X6	16	0.185	3	2-2X6	0.325	10

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (In-Plane)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Force[k/ft]	Gov LC	Hold-Down Label	Chord Strap Label	Tension Check	Tie-Down Force[k]	Gov LC
1	WP1	S1_15/32_10d@6	N/A	0.724	0.332	4	HD5B_2.5_DF-SP	NC	0.991	2.323	10

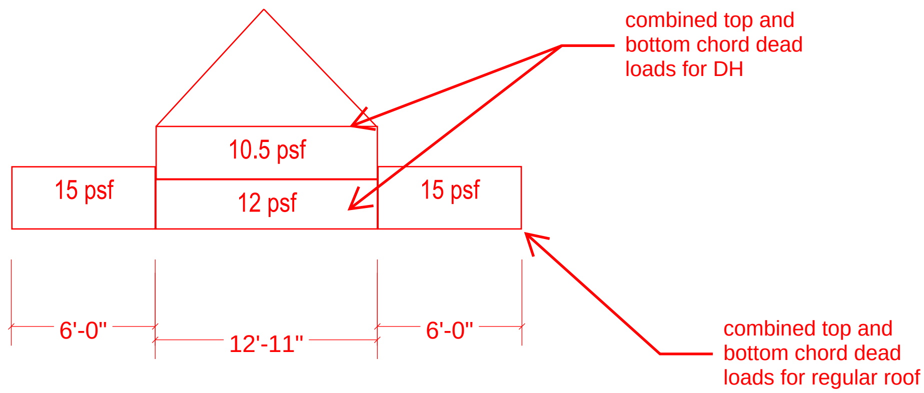
AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Header)

	Wall Panel	Header Label	Design Rule	Size	Bend UC	Shear UC	Gov LC
1	WP1	H1	Typical	6x8	0.209	0.25	3 (shear)

SHEAR WALL 3 CALCULATIONS

DH = Dog House

ROOF DEAD LOAD



BDH TRIB: 26.8'

LENGTH: 38.25'

ADH TRIB: 20.3'

DEAD LOAD:

W/DH: $$\frac{(2*15psf\ (6'\text{-}0'') + 22.5psf(12'\text{-}11''))}{2} = 0.236klf$$

W/O DH: $$\frac{15psf(20.2')}{2} = 0.152klf$$

ROOF LIVE LOAD :

W/DH: $$\frac{20psf*(26.8')}{2} = 0.268klf$$

W/O DH: $$\frac{20psf*(20.3')}{2} = 0.203klf$$

SNOW LOAD :

W/DH: $$\frac{65psf*(26.8')}{2} = 0.871klf$$

W/O DH: $$\frac{49psf*(20.3')}{2} = 0.497klf$$

WIND LOAD :

$$\frac{R7}{38.25'} = \frac{6735}{38.25'} = 0.176klf$$

SEISMIC LOAD :

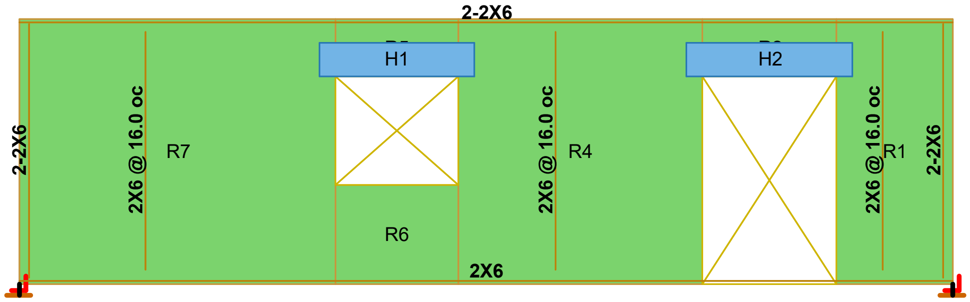
Seismic Base shear from excel spreadsheet

$$\frac{1.17+0.34}{2(38.25')} = 0.02klf$$

SHEAR WALL 3 (SW3)

Detail Report: WP1

Wood Wall



GENERAL

Code:	AWC NDS-18 / SDPWS-15:ASD
Design Method:	Perforated
Wall Material:	DF
Panel Schedule:	AWC 2015 OSB 0.469 (10d)
Optimize HD:	Yes
HD Manufacturer:	SIMPSON
HD Eccentricity (in):	0

GEOMETRY

Total Height (ft):	10.83
Total Length (ft):	38.25
Wall H/W Ratio:	0.28
Max Opening Ht (ft):	8.5
Open/Wall Ht Ratio:	0.78
Full Ht Sheathed (ft):	27.75
% Full Ht Sheathed:	72.55

MATERIALS

Top Pl.	DF	2-2X6
Sill	DF	2X6
Wall Stud	DF	2X6
Chord	DF	2-2X6

Use HD5B-3

ENVELOPED RESULTS

Shear Panel	Shear UC	Shear LC	Hold Down	Hold Down UC	Hold Down LC	Chord UC	Chord LC	Stud UC	Stud LC
S1_15/32_10d@6	0.378	4 (W)	Not Req'd	NC	NC	0.296	3	0.255	3

STUDS

Required Cap (k):	1.726	**Governing LC:**	3	**Studs in Region:**	8
Provided Cap (k):	6.76	**Gov Region:**	3	**K:**	1.00
Ratio:	0.255	**Stud Spacing (in):**	16		

CHORDS

Max Comp Force (k):	4.008	**Gov Comp LC:**	3	**Tens Ratio:**	0
Comp Capacity (k):	13.52	**Max Tens Force (k):**	0	**Gov Tens LC:**	N/A
Comp Ratio:	0.296	**Tens Capacity (k):**	14.479		

SHEAR PANEL

Selected Shear Panel:	S1_15/32_10d@6
Shear Ratio:	0.378
Governing LC:	4 (Wind)
Total Shear (k):	4.039
Max Unit Shear (k/ft):	0.149
Shear Cap. (k/ft):	0.340
Adjusted Shear Cap. (k/ft):	0.393
Specific Gravity:	0.5

Use R-9 Zip system with
3/12 fastening pattern with
0.131" dia shank nails.
Vall = 336 plf

Specific Gravity Adjustment Factor:	1.00
Hold-Down Factor:	1.00
Shear Stiffness Adjustment Factor:	1.00
Wall Capacity Adjustment Factor (2w/h):	0.88
Aspect Ratio Factor:	1.00
Governing H/W Ratio Factor:	1.00
Nailing Capacity Increase for Wind:	1.4
Shear Capacity Adjustment Factor (Co):	0.83
Total Area of Openings (Ao) (ft^2):	69.25
Sheathing Area Ratio (r):	0.81

Panel Grade:	St-I
Panel Thick (in):	0.469
Reqd Pen (in):	1.500
Nail Size:	10d
Reqd. Spacing (in):	6
Num Sides:	One
Over Gyp. Board:	No

DEFLECTIONS

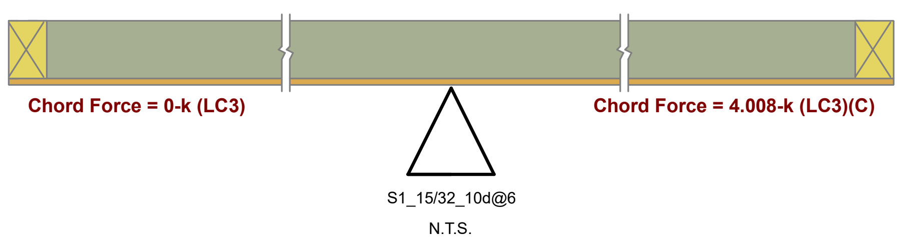
Elastic (in):	0.002	**Shear (in):**	0.073
HD (in):	0	**Total (in):**	0.075

HOLD-DOWNS

Hold-Downs Not Required.

Use PAB5 5/8" Dia anchor
Tall = 4,270 lbs (ASD)
de = 4"

CROSS SECTION DETAILING



Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	10.83	0	
3	N3	38.25	10.83	0	
4	N4	38.25	0	0	

Node Boundary Conditions

	Node Label	Z [k/in]
1	N2	Reaction
2	N3	Reaction

Wall Panel Distributed Loads (BLC 1 : Dead Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.152	-0.152	0	22.867
2	WP1(10.83ft)	Y	-0.236	-0.236	22.867	38.25

Wall Panel Distributed Loads (BLC 2 : Roof Live Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.202	-0.202	0	22.867
2	WP1(10.83ft)	Y	-0.268	-0.268	22.867	38.25

Wall Panel Distributed Loads (BLC 3 : Snow Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	Y	-0.497	-0.497	0	22.867
2	WP1(10.83ft)	Y	-0.871	-0.871	22.867	38.25

Wall Panel Distributed Loads (BLC 4 : Wind)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.176	0.176	0	38.25

Wall Panel Distributed Loads (BLC 5 : Seismic)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(10.83ft)	X	0.02	0.02	0	38.25

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Dead Load	Yes	Y	DL	1						
2	DL + RLL	Yes	Y	DL	1	RLL	1				
3	DL + SL	Yes	Y	DL	1	SL	1				
4	DL + 0.6WL	Yes	Y	DL	1	WLX	0.6				
5	DL +0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	0.45	RLL	0.75		
6	DL +0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	0.45	SL	0.75		
7	0.6DL + 0.6WL	Yes	Y	DL	0.6	WLX	0.6				
8	IBC 21/ASCE ASD 8 (a)	Yes		DL	1	ELX	0.7				
9	IBC 21/ASCE ASD 8 (b)	Yes		DL	1	ELZ	0.7				
10	IBC 21/ASCE ASD 9 (a)	Yes		DL	1	ELX	0.525	LL	0.75	LLS	0.75

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
11	IBC 21/ASCE ASD 9 (b)	Yes		DL	1	ELZ	0.525	LL	0.75	LLS	0.75
12	IBC 21/ASCE ASD 10 (a)	Yes		DL	0.6	ELX	0.7				
13	IBC 21/ASCE ASD 10 (b)	Yes		DL	0.6	ELZ	0.7				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0	13	0	13	0	13	0	13	0	13	0	13
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N3	max	0	13	0	13	0	13	0	13	0	13	0	13
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	WP1	max	0	3	33.092	3	0	13	0	13	0	13	95.28	6
6		min	-4.039	7	4.997	7	0	1	0	1	0	1	7.882	13
7	Totals:	max	0	3	33.092	3	0	13						
8		min	-4.039	7	4.997	7	0	1						

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Axial)

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC	Chord Size	Chord Axial Check	Gov LC
1	WP1	N/A	2X6	16	0.255	3	2-2X6	0.296	3

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (In-Plane)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Force[k/ft]	Gov LC	Hold-Down Label	Chord Strap Label	Tension Check	Tie-Down Force[k]	Gov LC
1	WP1	S1_15/32_10d@6	N/A	0.378	0.149	4	Not Req'd	NC	NC	NC	NC

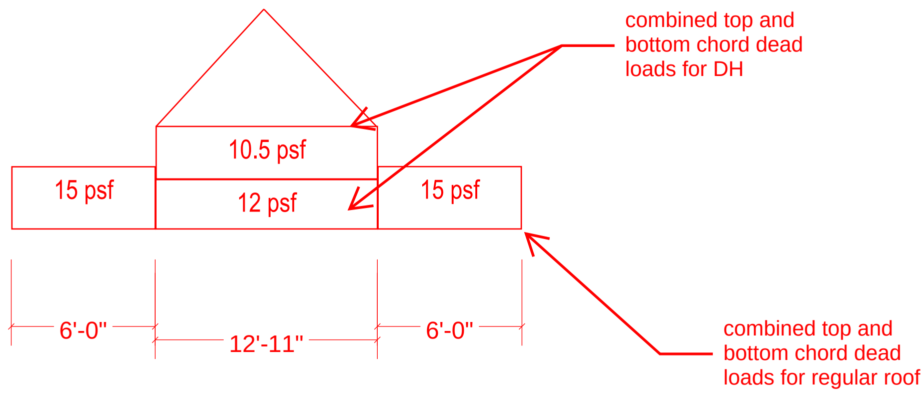
AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Header)

	Wall Panel	Header Label	Design Rule	Size	Bend UC	Shear UC	Gov LC
1	WP1	H1	Typical	6x8	0.17	0.201	3 (shear)
2		H2	Typical	6x8	0.286	0.346	3 (shear)

SHEAR WALL 4 CALCULATIONS

DH = Dog House

ROOF DEAD LOAD



TRIB: 8" (Non Load bearing)   LENGTH: 26.8'

DEAD LOAD:

W/DH: $$\frac{(2*15psf\ (0.67')) + 22.5psf(0.67')}{2} = 0.018klf$$

ROOF LIVE LOAD :

W/DH: $$\frac{20psf*(0.67')}{2} = 0.0067klf$$

SNOW LOAD :

W/DH: $$\frac{65psf*(0.67')}{2} = 0.022klf$$

WIND LOAD :

$$\frac{R8}{26.8'} = \frac{10415+4860}{26.8'} = 0.570klf$$

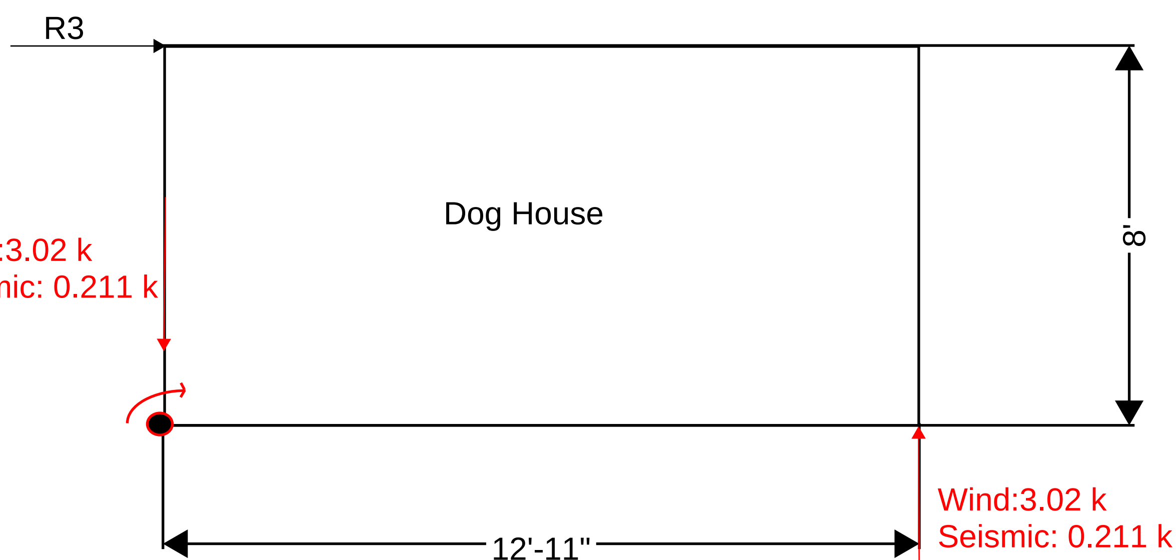
SEISMIC LOAD :

Seismic Base shear from excel spreadsheet

$$\frac{1.17+0.34}{2(38.25')} = 0.02klf$$

Calculating the Overturning due to the Dog House

DH Height Top to Bottom: 8'



Wind Moment:

R3:4,860lbs

Wind Moment = 4860*(8) = 39kft

$$\frac{39\text{kft}}{12'\text{-}11''} = +/3.02\text{ k}$$

Use MSTC40 Simpson strap tie between dog house and SW3 centered on rim joist.

Seismic Moment:

R3 = 0.34k (Base Shear From Excel)

Seismic Moment = 0.34k*(8) = 2.72kft

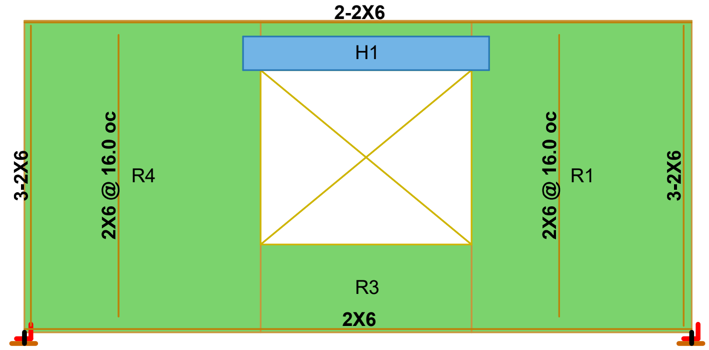
$$\frac{2.72\text{kft}}{12'\text{-}11''} = +/-0.211\text{ k}$$

Follow that line of action down to the foundation and use PAB5 5/8" Dia anchor alongside HD5B-3.5

SHEAR WALL 4 (SW4)

Detail Report: WP4

Wood Wall



GENERAL		GEOMETRY		MATERIALS		
Code:	AWC NDS-18 / SDPWS-15:ASD	**Total Height (ft):**	12.5	**Top Pl.**	DF	2-2X6
		Total Length (ft):	26.8	**Sill**	DF	2X6
Design Method:	Perforated	**Wall H/W Ratio:**	0.47	**Wall Stud**	DF	2X6
Wall Material:	DF	**Max Opening Ht (ft):**	7	**Chord**	DF	3-2X6
Panel Schedule:	AWC 2015 OSB	**Open/Wall Ht Ratio:**	0.56			
Optimize HD:	Yes	**Full Ht Sheathed (ft):**	18.3			
HD Manufacturer:	SIMPSON	**% Full Ht Sheathed:**	68.28			
HD Eccentricity (in):	0					

ENVELOPED RESULTS

Shear Panel	Shear UC	Shear LC	Hold Down	Hold Down UC	Hold Down LC	Chord UC	Chord LC	Stud UC	Stud LC
RS_19/32_10d@4	0.853	4 (W)	HD9B_4.5_DF-SP	0.857	11 (W)	0.595	5 (W)	0.091	3

STUDS

Required Cap (k):	0.483	**Governing LC:**	3	**Studs in Region:**	8
Provided Cap (k):	5.33	**Gov Region:**	4	**K:**	1.00
Ratio:	0.091	**Stud Spacing (in):**	16		

CHORDS

Max Comp Force (k):	7.898	**Gov Comp LC:**	5	**Tens Ratio:**	0.295
Comp Capacity (k):	13.268	**Max Tens Force (k):**	5.312	**Gov Tens LC:**	11
Comp Ratio:	0.595	**Tens Capacity (k):**	18.016		

SHEAR PANEL

Selected Shear Panel:	RS_19/32_10d@4	**Specific Gravity Adjustment Factor:**	1.00	**Panel Grade:**	RS
Shear Ratio:	0.853	**Hold-Down Factor:**	1.00	**Panel Thick (in):**	0.594
Governing LC:	4 (Wind)	**Shear Stiffness Adjustment Factor:**	1.00	**Reqd Pen (in):**	1.500
Total Shear (k):	9.166	**Wall Capacity Adjustment Factor (2w/h):**	1.00	**Nail Size:**	10d
Max Unit Shear (k/ft):	0.501	**Aspect Ratio Factor:**	1.00	**Reqd. Spacing (in):**	4
Shear Cap. (k/ft):	0.510	**Governing H/W Ratio Factor:**	1.00	**Num Sides:**	One
Adjusted Shear Cap. (k/ft):	0.587	**Nailing Capacity Increase for Wind:**	1.4	**Over Gyp. Board:**	No
Specific Gravity:	0.5	**Shear Capacity Adjustment Factor (Co):**	0.82		
		Total Area of Openings (Ao) (ft^2):	59.5		
		Sheathing Area Ratio (r):	0.79		

R-9 Zip system provides an additional shear capacity of 336 plf on the exterior combine with the 5/8" plywood (additional 640 plf capacity) on the interior to withstand the total shear

DEFLECTIONS

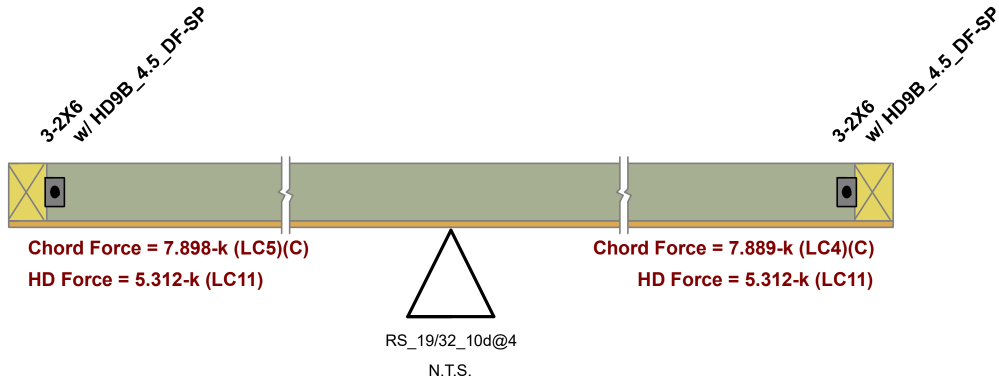
Elastic (in):	0.01	**Shear (in):**	0.241
HD (in):	0.104	**Total (in):**	0.355

HOLD-DOWNS

Selected Hold-Down:	HD9B_4.5_DF-SP	**Governing LC:**	11	**AB Diameter (in):**	0.500
Required Cap. (k):	5.312	**CD factor:**	1	**Fastener Size:**	16d
Base Capacity (k):	6.200	**Reqd. Chord Mat.:**	Douglas Fir	**Num Fasteners:**	8
Adjusted Cap. (k):	6.2	**Reqd. Chord Thk. (in):**	4.50		
Ratio:	0.857	**Raised:**	No		

Use PAB8 1" Dia anchor
Tall = 11,340 lbs (ASD)
de = 8" due to hold down
anchor requirement

CROSS SECTION DETAILING



Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N5	0	12.5	0	
2	N6	26.8	12.5	0	
3	N9	0	0	0	
4	N10	26.8	0	0	
5	N11	6	12.5	0	
6	N14	18.92	12.5	0	

Node Boundary Conditions

	Node Label	Z [k/in]
1	N5	Reaction
2	N6	Reaction

Wall Panel Distributed Loads (BLC 1 : Dead Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP4(12.5ft)	Y	-0.236	-0.236	0	26.8

Wall Panel Distributed Loads (BLC 2 : Roof Live Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP4(12.5ft)	Y	-0.007	-0.007	0	26.8

Wall Panel Distributed Loads (BLC 3 : Snow Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP4(12.5ft)	Y	-0.022	-0.022	0	%100

Wall Panel Distributed Loads (BLC 4 : Wind)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP4(12.5ft)	X	0.57	0.57	0	26.8

Wall Panel Distributed Loads (BLC 5 : Seismic)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP4(12.5ft)	X	0.02	0.02	0	26.8

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Dead Load	Yes	Y	DL	1						
2	DL + RLL	Yes	Y	DL	1	RLL	1				
3	DL + SL	Yes	Y	DL	1	SL	1				
4	DL + 0.6WL	Yes	Y	DL	1	WLX	0.6				
5	DL - 0.6WL	Yes	Y	DL	1	WLX	-0.6				
6	DL +0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	0.45	RLL	0.75		
7	DL -0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	-0.45	RLL	0.75		
8	DL +0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	0.45	SL	0.75		
9	DL -0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	-0.45	SL	0.75		
10	0.6DL + 0.6WL	Yes	Y	DL	0.6	WLX	0.6				
11	0.6DL - 0.6WL	Yes	Y	DL	0.6	WLX	-0.6				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
12	IBC 21/ASCE ASD 8	Yes		DL	1	EL	0.7				
13	IBC 21/ASCE ASD 9	Yes		DL	1	EL	0.525	LL	0.75	LLS	0.75
14	IBC 21/ASCE ASD 10	Yes		DL	0.6	EL	0.7				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N5	max	0	14	0	14	0	14	0	14	0	14	0	14
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N6	max	0	14	0	14	0	14	0	14	0	14	0	14
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	WP4	max	9.166	11	8.024	3	0	14	0	14	0	14	137.931	10
6		min	-9.166	10	4.461	11	0	1	0	1	0	1	-138.064	5
7	Totals:	max	9.166	11	8.024	3	0	14						
8		min	-9.166	10	4.461	11	0	1						

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Axial)

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC	Chord Size	Chord Axial Check	Gov LC
1	WP4	N/A	2X6	16	0.091	3	3-2X6	0.595	5

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (In-Plane)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Force[k/ft]	Gov LC	Hold-Down Label	Chord Strap Label	Tension Check	Tie-Down Force[k]	Gov LC
1	WP4	RS_19/32_10d@4	N/A	0.853	0.501	4	HD9B_4.5_DF-SP	NC	0.857	5.312	11

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Header)

	Wall Panel	Header Label	Design Rule	Size	Bend UC	Shear UC	Gov LC
1	WP4	H1	Typical	6x8	0.244	0.19	3 (bending)

SHEAR WALL 5 CALCULATIONS

ROOF DEAD LOAD

TRIB: 8" (Non Load bearing) LENGTH: 20.2'

DEAD LOAD:

$$\frac{15\text{psf }(0.67')}{2} = 0.005\text{klf}$$

ROOF LIVE LOAD :

$$\frac{20\text{psf}*(0.67')}{2} = 0.0067\text{klf}$$

SNOW LOAD :

Flat Roof Snow

$$\frac{49\text{psf}*(0.67')}{2} = 0.017\text{ klf}$$

WIND LOAD :

$$\frac{R9}{20.2'} = \frac{9365}{20.2'} = 0.464\text{klf}$$

SEISMIC LOAD :

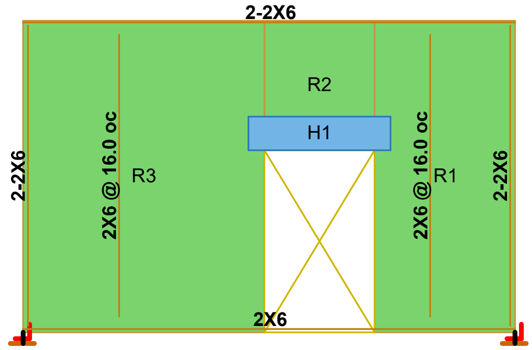
Seismic Base shear from excel spreadsheet

$$\frac{0.34}{2(38.25')} = 0.004\text{klf}$$

SHEAR WALL 5 (SW5)

Detail Report: WP1

Wood Wall



GENERAL

Code:	AWC NDS-18 / SDPWS-15:ASD
Design Method:	Perforated
Wall Material:	DF
Panel Schedule:	AWC 2015 OSB 0.469 (10d)
Optimize HD:	Yes
HD Manufacturer:	SIMPSON
HD Eccentricity (in):	0

GEOMETRY

Total Height (ft):	12.83
Total Length (ft):	20.3
Wall H/W Ratio:	0.63
Max Opening Ht (ft):	7.5
Open/Wall Ht Ratio:	0.58
Full Ht Sheathed (ft):	15.8
% Full Ht Sheathed:	77.83

MATERIALS

Top Pl.	DF	2-2X6
Sill	DF	2X6
Wall Stud	DF	2X6
Chord	DF	2-2X6

Use HD9B (same as SW4 for corners only.)

ENVELOPED RESULTS

Shear Panel	Shear UC	Shear LC	Hold Down	Hold Down UC	Hold Down LC	Chord UC	Chord LC	Stud UC	Stud LC
S1_15/32_10d@6	0.888	4 (W)	HD7B_3.5_DF-SP	0.984	11 (W)	0.556	5 (W)	0.019	3

STUDS

Required Cap (k):	0.097	**Governing LC:**	3	**Studs in Region:**	5
Provided Cap (k):	5.09	**Gov Region:**	1	**K:**	1.00
Ratio:	0.019	**Stud Spacing (in):**	16		

CHORDS

Max Comp Force (k):	4.83	**Gov Comp LC:**	5	**Tens Ratio:**	0.364
Comp Capacity (k):	8.683	**Max Tens Force (k):**	4.494	**Gov Tens LC:**	11
Comp Ratio:	0.556	**Tens Capacity (k):**	12.34		

SHEAR PANEL

Selected Shear Panel:	S1_15/32_10d@6	**Specific Gravity Adjustment Factor:**	1.00	**Panel Grade:**	St-I
Shear Ratio:	0.888	**Hold-Down Factor:**	1.00	**Panel Thick (in):**	0.469
Governing LC:	4 (Wind)	**Shear Stiffness Adjustment Factor:**	1.00	**Reqd Pen (in):**	1.500
Total Shear (k):	5.652	**Wall Capacity Adjustment Factor (2w/h):**	0.90	**Nail Size:**	10d
Max Unit Shear (k/ft):	0.371	**Aspect Ratio Factor:**	1.00	**Reqd. Spacing (in):**	6
Shear Cap. (k/ft):	0.340	**Governing H/W Ratio Factor:**	1.00	**Num Sides:**	One
Adjusted Shear Cap. (k/ft):	0.418	**Nailing Capacity Increase for Wind:**	1.4	**Over Gyp. Board:**	No
Specific Gravity:	0.5	**Shear Capacity Adjustment Factor (Co):**	0.88		
		Total Area of Openings (Ao) (ft^2):	33.75		
		Sheathing Area Ratio (r):	0.85		

SW5 has an additional 1/2" gyp with a shear capacity of 175 plf on the inside face of the wall plus the R9 zip system on the exterior wall with a shear capacity of 336 plf. Both combine to 511plf which is sufficient to withstand the total shear

DEFLECTIONS

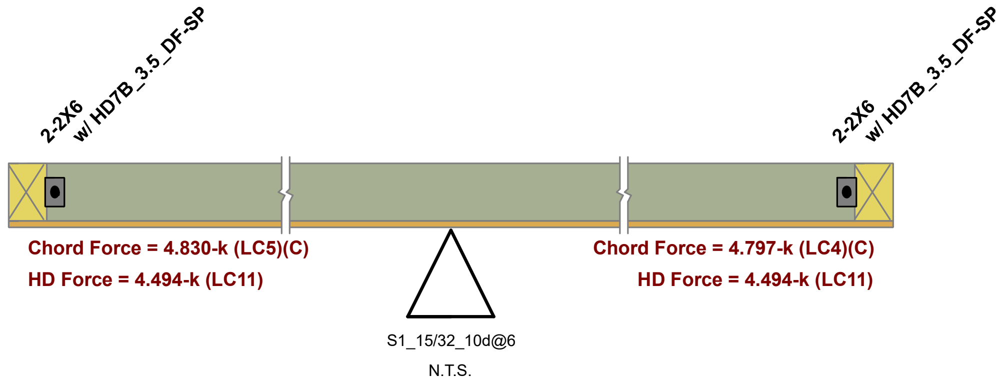
Elastic (in):	0.014	**Shear (in):**	0.216
HD (in):	0.123	**Total (in):**	0.353

HOLD-DOWNS

Selected Hold-Down:	HD7B_3.5_DF-SP	**Governing LC:**	11	**AB Diameter (in):**	0.500
Required Cap. (k):	4.494	**CD factor:**	1	**Fastener Size:**	16d
Base Capacity (k):	4.569	**Reqd. Chord Mat.:**	Douglas Fir	**Num Fasteners:**	8
Adjusted Cap. (k):	4.569	**Reqd. Chord Thk. (in):**	3.50		
Ratio:	0.984	**Raised:**	No		

Use PAB8 1" Dia anchor
Tall = 11,340 lbs (ASD)
de = 8" due to hold down
anchor requirement

CROSS SECTION DETAILING



Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	12.83	0	
3	N3	20.3	12.83	0	
4	N4	20.3	0	0	

Node Boundary Conditions

	Node Label	Z [k/in]
1	N2	Reaction
2	N3	Reaction

Wall Panel Distributed Loads (BLC 1 : Dead Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(12.83ft)	Y	-0.005	-0.005	0	20.3

Wall Panel Distributed Loads (BLC 2 : Roof Live Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(12.83ft)	Y	-0.007	-0.007	0	%100

Wall Panel Distributed Loads (BLC 3 : Snow Load)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(12.83ft)	Y	-0.017	-0.017	0	%100

Wall Panel Distributed Loads (BLC 4 : Wind)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(12.83ft)	X	0.464	0.464	0	20.3

Wall Panel Distributed Loads (BLC 5 : Seismic)

	Wall Label	Direction	Start Magnitude [k/ft, F]	End Magnitude [k/ft, F]	Start Location [(ft, %)]	End Location [(ft, %)]
1	WP1(12.83ft)	X	0.004	0.004	0	20.3

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	Dead Load	Yes	Y	DL	1						
2	DL + RLL	Yes	Y	DL	1	RLL	1				
3	DL + SL	Yes	Y	DL	1	SL	1				
4	DL + 0.6WL	Yes	Y	DL	1	WLX	0.6				
5	DL - 0.6WL	Yes	Y	DL	1	WLX	-0.6				
6	DL +0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	0.45	RLL	0.75		
7	DL -0.75(0.6)WL + 0.75RLL	Yes	Y	DL	1	WLX	-0.45	RLL	0.75		
8	DL +0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	0.45	SL	0.75		
9	DL -0.75(0.6)WL + 0.75SL	Yes	Y	DL	1	WLX	-0.45	SL	0.75		
10	0.6DL + 0.6WL	Yes	Y	DL	0.6	WLX	0.6				
11	0.6DL - 0.6WL	Yes	Y	DL	0.6	WLX	-0.6				
12	IBC 21/ASCE ASD 8 (a)	Yes		DL	1	ELX	0.7				
13	IBC 21/ASCE ASD 8 (b)	Yes		DL	1	ELZ	0.7				

Load Combinations (Continued)

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
14	IBC 21/ASCE ASD 9 (a)	Yes		DL	1	ELX	0.525	LL	0.75	LLS	0.75
15	IBC 21/ASCE ASD 9 (b)	Yes		DL	1	ELZ	0.525	LL	0.75	LLS	0.75
16	IBC 21/ASCE ASD 10 (a)	Yes		DL	0.6	ELX	0.7				
17	IBC 21/ASCE ASD 10 (b)	Yes		DL	0.6	ELZ	0.7				

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N2	max	0	17	0	17	0	17	0	17	0	17	0	17
2		min	0	1	0	1	0	1	0	1	0	1	0	1
3	N3	max	0	17	0	17	0	17	0	17	0	17	0	17
4		min	0	1	0	1	0	1	0	1	0	1	0	1
5	WP1	max	5.652	5	1.254	3	0	17	0	17	0	17	72.358	10
6		min	-5.652	4	0.545	10	0	1	0	1	0	1	-72.76	5
7	Totals:	max	5.652	5	1.254	3	0	17						
8		min	-5.652	4	0.545	10	0	1						

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Axial)

	Wall Panel	Region	Stud Size	Stud Spacing[in]	Axial Check	Gov LC	Chord Size	Chord Axial Check	Gov LC
1	WP1	N/A	2X6	16	0.019	3	2-2X6	0.556	5

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (In-Plane)

	Wall Panel	Shear Panel Label	Region	Shear Check	Shear Force[k/ft]	Gov LC	Hold-Down Label	Chord Strap Label	Tension Check	Tie-Down Force[k]	Gov LC
1	WP1	S1_15/32_10d@6	N/A	0.888	0.371	4	HD7B_3.5_DF-SP	NC	0.984	4.494	11

AWC NDS-18 / SDPWS-15 ASD Wall Panel Wood Code Checks (Header)

	Wall Panel	Header Label	Design Rule	Size	Bend UC	Shear UC	Gov LC
1	WP1	H1	Typical	6x8	0.009	0.013	3 (shear)

Diaphragm Shear Loading (segmented design)											
Seismic Load	Long Dir (lbs)										
Roof Load at Doghouse =	340										
Roof Load at Low Roof =	1170										
	Wind	Seismic									
Shear Wall	**Strength Shear Load**	**Shear Load**	**Diaphragm Length**	**Diaphragm Shear**	**Shear Wall Length**	**Wall Shear**	**Drag Strut Force**	**Plywood Thickness**	**Chord size**	**Oustide Edges Required Hold Down Anchor or Strap**	**Opening Required Hold Down Anchor or Strap**	**Shear Panel**
	(lbs)	(lbs)	(ft)	(plf)	(ft)	(plf)	(lbs)					
SW1	1,170	755	15	78.0	7.3	160.3	600.6	15/32	(2) 2x6	HD5B 3 DF-SP	HD5B 3 DF-SP	S1 15/32 10d@3
SW2	9,575	585	38.25	250.3	22.67	422.4	3,900.1	15/32	(2) 2x6	HD5B 3 DF-SP	HD5B 3 DF-SP	S1 15/32 10d@6
SW3	6,735	755	38.25	176.1	30.58	220.2	1,350.5	15/32	(2) 2x6	Not Required but use (HD5B 3 DF-SP)	Not Required but use (HD5B 3 DF-SP)	S1 15/32 10d@6
SW4	15,275	755	26.8	570.0	26.8	570.0	0.0	19/32	(3) 2x6	HD9B 4.5 DF-SP	HD9B 4.5 DF-SP	RS 19/32 10d@4
SW5	9,365	170	20.2	463.6	20.2	463.6	0.0	15/32	(2) 2x6	HD9B 4.5 DF-SP	HD9B 4.5 DF-SP	S1 15/32 10d@4

SW1, SW2 & SW3
Use MSTC40

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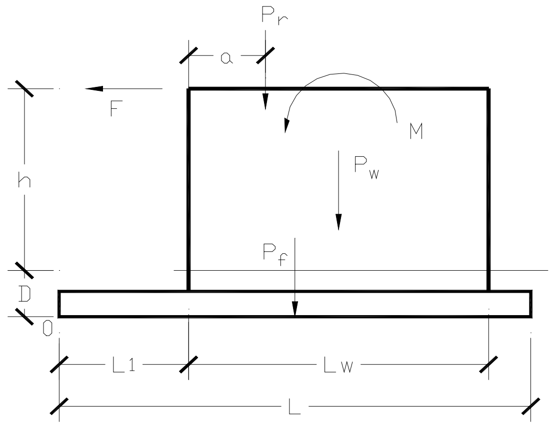
Foundations

PROJECT :	Aroma Joes Saco Dover Prototype		PAGE :	
SUBJECT :	SW1		DESIGN BY :	EKM
JOB NO. :	24625	DATE : 5/23/2024	REVIEW BY :	

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA

WALL LENGTH	L_w =	15	ft		
WALL HEIGHT	h =	12.83	ft		
WALL THICKNESS	t =	6	in		
FOOTING LENGTH	L =	16.33	ft		
	L_1 =	0.665	ft		
FOOTING WIDTH	B =	2	ft		
FOOTING THICKNESS	T =	12	in		
FOOTING EMBEDMENT DEPTH	D =	4.5	ft		
ALLOWABLE SOIL PRESSURE	q_a =	1.5	ksf		
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	3.54	kips		
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	4.02	kips		
TOP LOAD LOCATION	a =	7.5	ft		
WALL SELF WEIGHT	P_w =	6.02	kips		
LATERAL LOAD TYPE (0=wind,1=seismic)		0	Wind - Service		
WIND LOADS AT WALL TOP	F =	1.2	kips		
	M =	0	ft-kips		
CONCRETE STRENGTH	f_c' =	4	ksi		
REBAR YIELD STRESS	f_y =	60	ksi		
TOP BARS, LONGITUDINAL		1	#	5	< == Not Required
BOTTOM BARS, LONGITUDINAL		3	#	5	
BOTTOM BARS, TRANSVERSE	#	3	@	12 in o.c.	< == Not Required

THE FOOTING DESIGN IS ADEQUATE.

Uniform load acting on the middle of the wall

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 18 1605.3.1, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 5.61 > 1.5 for wind [Satisfactory]

Where P_f = 4.7357 kips (footing self weight)

$M_O = F (h + D) + M =$ 21 ft-kips (overturning moment)

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 117 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 3.266 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 15.05 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 150 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ -0.39 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = \quad 0.39 \text{ ksf} \quad < \quad 4/3\, q_a$$

[Satisfactory]

Where e = -0.39 ft, < (L / 6)

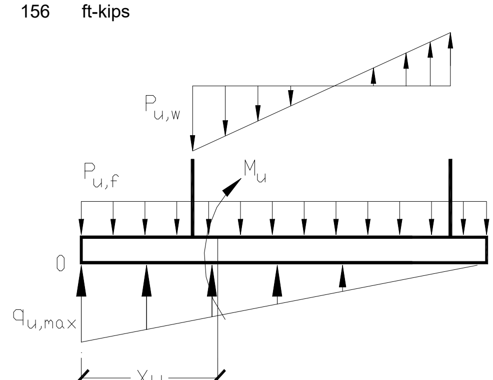
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R}$ = $1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r,LL}(L_1 + a) =$ 156 ft-kips

$M_{u,o}$ = $1.6 [F(h + D) + M] =$ 33 ft-kips

P_u = $1.2 (P_{r,DL} + P_f + P_w) + 0.5 P_{r,LL} =$ 19 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 1.74 ft

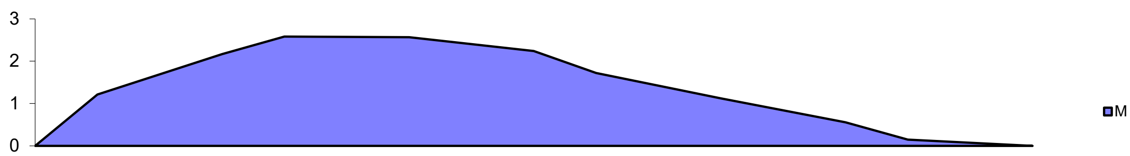


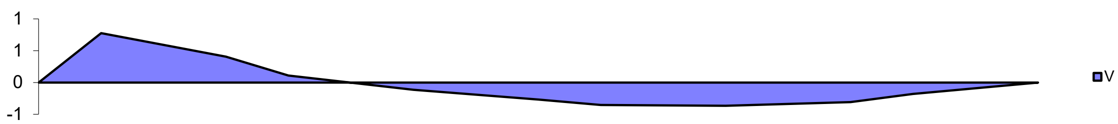
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} = \quad 0.96 \text{ ksf}$$

(cont'd)

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	1.63	3.27	4.90	6.53	8.17	9.80	11.43	13.06	14.70	16.33
$P_{u,w}$ (klf)	0.0	1.7	1.5	1.3	1.1	0.9	0.7	0.5	0.3	0.1	0.0
$M_{u,w}$ (ft-k)	0	-1	-6	-15	-27	-42	-59	-79	-100	-121	-143
$V_{u,w}$ (kips)	0	-2	-4	-7	-8	-10	-11	-12	-13	-13	-13
$P_{u,f}$ (ksf)	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
$M_{u,f}$ (ft-k)	0	0	-2	-4	-7	-12	-17	-23	-30	-38	-46
$V_{u,f}$ (kips)	0	-1	-1	-2	-2	-3	-3	-4	-5	-5	-6
q_u (ksf)	-1.0	-0.9	-0.8	-0.7	-0.7	-0.6	-0.5	-0.4	-0.4	-0.3	-0.2
$M_{u,q}$ (ft-k)	0	2	10	21	37	56	78	103	130	159	190
$V_{u,q}$ (kips)	0	3	6	8	11	13	14	16	17	18	19
ΣM_u (ft-k)	**0**	**1**	**2**	**3**	**3**	**2**	**2**	**1**	**1**	**0**	**0**
ΣV_u (kips)	**0**	**1**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	8.69	0.0000	0.0000	1	kips	20	kips
Bottom Longitudinal	3	ft-k	8.69	0.0018	0.0045	1	kips	20	kips
Bottom Transverse	0	ft-k / ft	8.19	0.0000	0.0000	0	kips / ft	9	kips / ft

Where $$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

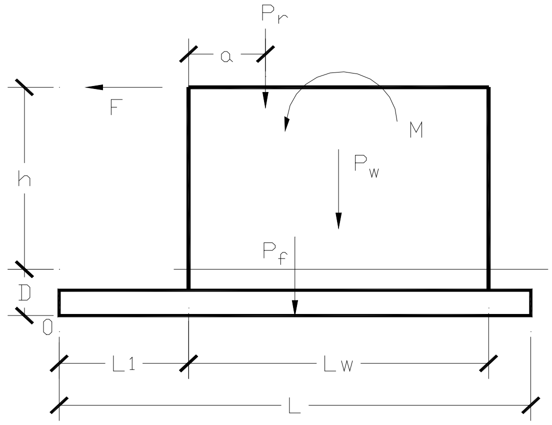
ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206 [Satisfactory]

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA

WALL LENGTH	L_w =	22.67	ft			
WALL HEIGHT	h =	12.83	ft			
WALL THICKNESS	t =	6	in			
FOOTING LENGTH	L =	24	ft			
	L_1 =	0.665	ft			
FOOTING WIDTH	B =	3	ft			
FOOTING THICKNESS	T =	12	in			
FOOTING EMBEDMENT DEPTH	D =	4.5	ft			
ALLOWABLE SOIL PRESSURE	q_a =	2	ksf			
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	3.47	kips			
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	4.6	kips			
TOP LOAD LOCATION	a =	11.335	ft			
WALL SELF WEIGHT	P_w =	10.66	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		0	Wind - Service			
WIND LOADS AT WALL TOP	F =	9.6	kips			
	M =	0	ft-kips			
CONCRETE STRENGTH	f_c' =	4	ksi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		0	#	4		< == Not Required
BOTTOM BARS, LONGITUDINAL		4	#	5		
BOTTOM BARS, TRANSVERSE	#	5	@	12	in o.c.	



THE FOOTING DESIGN IS ADEQUATE.

Uniform load acting on the middle of the wall

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 18 1605.3.1, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 1.80 > 1.5 for wind [Satisfactory]

Where P_f = 10.8 kips (footing self weight)

$M_O = F\,(h + D) + M =$ 166 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 299 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 7.2 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 22.33 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 354 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 3.58 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases} = 0.59 \text{ ksf} < 4/3\, q_a$$

[Satisfactory]

Where e = 3.58 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

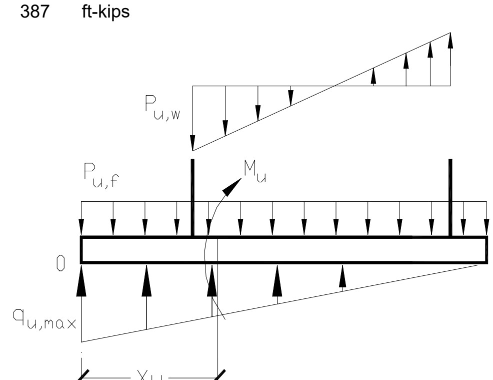
$M_{u,R} =$ $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,\,LL}(L_1 + a) =$ 387 ft-kips

$M_{u,o} =$ $1.6\,[F(h + D) + M] =$ 266 ft-kips

$P_u =$ $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,\,LL} =$ 32 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 8.26 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u > \dfrac{L}{6} \end{cases} = 1.92 \text{ ksf}$$



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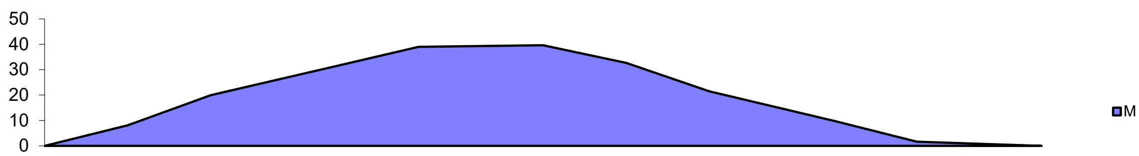
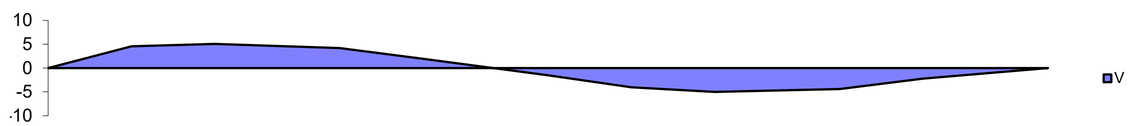
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.40	4.80	7.20	9.60	12.00	14.40	16.80	19.20	21.60	24.00
$P_{u,w}$ (klf)	0.0	3.5	2.8	2.2	1.5	0.8	0.2	-0.5	-1.1	-1.8	0.0
$M_{u,w}$ (ft-k)	0	-6	-31	-72	-125	-188	-255	-323	-389	-448	-497
$V_{u,w}$ (kips)	0	-6	-14	-20	-24	-27	-28	-28	-26	-23	-19
$P_{u,f}$ (ksf)	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
$M_{u,f}$ (ft-k)	0	-2	-6	-14	-25	-39	-56	-76	-100	-126	-156
$V_{u,f}$ (kips)	0	-1	-3	-4	-5	-6	-8	-9	-10	-12	-13
q_u (ksf)	-1.9	-1.5	-1.1	-0.7	-0.3	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	15	57	117	189	266	344	421	498	575	653
$V_{u,q}$ (kips)	0	12	22	28	32	32	32	32	32	32	32
ΣM_u (ft-k)	**0**	**8**	**20**	**31**	**39**	**40**	**33**	**21**	**10**	**2**	**0**
ΣV_u (kips)	**0**	**5**	**5**	**4**	**2**	**-2**	**-4**	**-5**	**-4**	**-2**	**0**



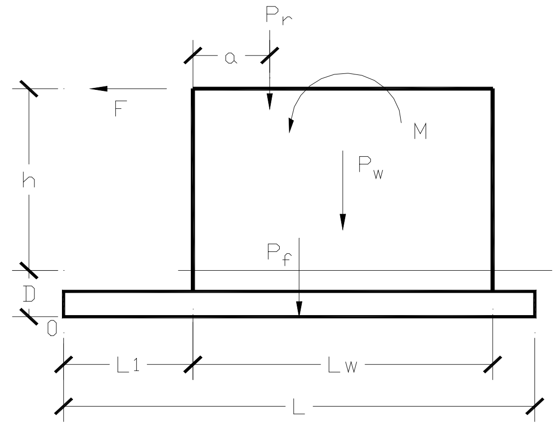


Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	8.75	0.0000	0.0000	5	kips	30	kips
Bottom Longitudinal	40	ft-k	8.69	0.0033	0.0040	5	kips	30	kips
Bottom Transverse	0	ft-k / ft	8.06	0.0018	0.0032	1	kips / ft	9	kips / ft

Where $$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$ ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206 [Satisfactory]

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA

WALL LENGTH	L_w =	38.25	ft			
WALL HEIGHT	h =	12.83	ft			
WALL THICKNESS	t =	6	in			
FOOTING LENGTH	L =	39.5833	ft			
	L_1 =	0.66665	ft			
FOOTING WIDTH	B =	2	ft			
FOOTING THICKNESS	T =	12	in			
FOOTING EMBEDMENT DEPTH	D =	4.5	ft			
ALLOWABLE SOIL PRESSURE	q_a =	1.5	ksf			
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	9.027	kips			
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	10.25	kips			
TOP LOAD LOCATION	a =	19.125	ft			
WALL SELF WEIGHT	P_w =	15	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		0	Wind - Service			
WIND LOADS AT WALL TOP	F =	6.7	kips			**THE FOOTING DESIGN IS ADEQUATE.**
	M =	0	ft-kips			
CONCRETE STRENGTH	f_c' =	4	ksi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		2	#	5		**< == Not Required**
BOTTOM BARS, LONGITUDINAL		3	#	5		
BOTTOM BARS, TRANSVERSE	#	3	@	12	in o.c.	**< == Not Required**

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 18 1605.3.1, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 6.05 > 1.5 for wind [Satisfactory]

Where P_f = 11.47916 kips (footing self weight)

$M_O = F\,(h + D) + M =$ 116 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 703 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 7.91666 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 37.84 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 906 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ -1.07 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.40 \text{ ksf} < 4/3\, q_a$$

[Satisfactory]

Where e = -1.07 ft, < (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

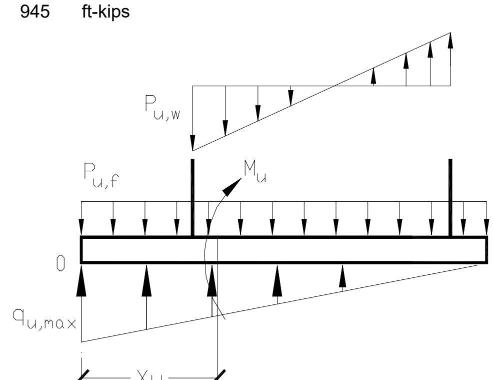
$M_{u,R}$ = $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 945 ft-kips

$M_{u,o}$ = $1.6\,[F(h + D) + M] =$ 186 ft-kips

P_u = $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 48 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 3.89 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} = 0.96 \text{ ksf}$$



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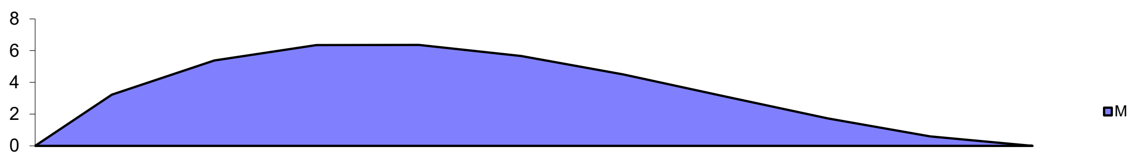
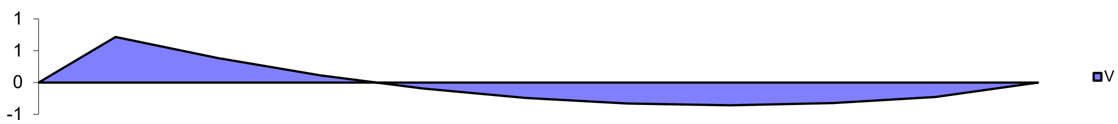
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	3.96	7.92	11.87	15.83	19.79	23.75	27.71	31.67	35.62	39.58
$P_{u,w}$ (klf)	0.0	1.5	1.4	1.2	1.0	0.9	0.7	0.6	0.4	0.3	0.0
$M_{u,w}$ (ft-k)	0	-9	-41	-94	-167	-255	-358	-472	-595	-724	-858
$V_{u,w}$ (kips)	0	-5	-11	-16	-20	-24	-27	-30	-32	-33	-34
$P_{u,f}$ (ksf)	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
$M_{u,f}$ (ft-k)	0	-3	-11	-25	-44	-68	-98	-134	-174	-221	-273
$V_{u,f}$ (kips)	0	-1	-3	-4	-6	-7	-8	-10	-11	-12	-14
q_u (ksf)	-1.0	-0.9	-0.8	-0.7	-0.7	-0.6	-0.5	-0.5	-0.4	-0.3	-0.2
$M_{u,q}$ (ft-k)	0	15	57	125	217	329	460	609	771	946	1130
$V_{u,q}$ (kips)	0	7	14	20	26	31	35	39	43	45	48
ΣM_u (ft-k)	**0**	**3**	**5**	**6**	**6**	**6**	**4**	**3**	**2**	**1**	**0**
ΣV_u (kips)	**0**	**1**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	8.69	0.0000	0.0000	1	kips	20	kips
Bottom Longitudinal	6	ft-k	8.69	0.0018	0.0045	1	kips	20	kips
Bottom Transverse	0	ft-k / ft	8.19	0.0000	0.0000	0	kips / ft	9	kips / ft

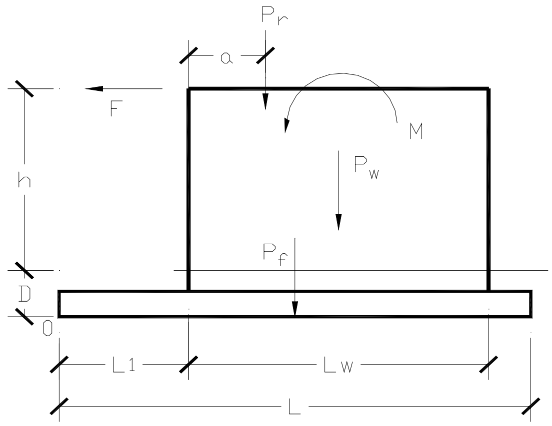
Where $$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206 [Satisfactory]

Footing Design of Shear Wall Based on ACI 318-14

INPUT DATA



WALL LENGTH	L_w =	26.8	ft			
WALL HEIGHT	h =	12.83	ft			
WALL THICKNESS	t =	6	in			
FOOTING LENGTH	L =	28.13	ft			
	L_1 =	0.665	ft			
FOOTING WIDTH	B =	3	ft			
FOOTING THICKNESS	T =	21	in			
FOOTING EMBEDMENT DEPTH	D =	4.5	ft			
ALLOWABLE SOIL PRESSURE	q_a =	2	ksf			
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.1	kips			
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0.1	kips			
TOP LOAD LOCATION	a =	13.4	ft			Uniform load acting on the middle of the wall
WALL SELF WEIGHT	P_w =	11	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		0	Wind - Service			
WIND LOADS AT WALL TOP	F =	15.3	kips			THE FOOTING DESIGN IS ADEQUATE.
	M =	34.85	ft-kips			Overturning From Dog House
CONCRETE STRENGTH	f_c' =	4	ksi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		0	#	4		< == Not Required
BOTTOM BARS, LONGITUDINAL		4	#	6		
BOTTOM BARS, TRANSVERSE	#	5	@	12	in o.c.	< == Not Required

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 18 1605.3.1, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 1.56 > 1.5 for wind [Satisfactory]

Where P_f = 22.15238 kips (footing self weight)

$M_O = F\,(h + D) + M =$ 300 ft-kips (overturning moment)

$M_R = (P_{r,DL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 468 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 14.76825 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 18.58 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL})\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w) =$ 469 ft-kips (resisting moment with live load)

$e = 0.5\,L - (M_R - M_O) / P =$ 4.97 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for \quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for \quad e > \dfrac{L}{6} \end{cases} = 0.45 \text{ ksf} < 4/3\, q_a$$

[Satisfactory]

Where e = 4.97 ft, > (L / 6)

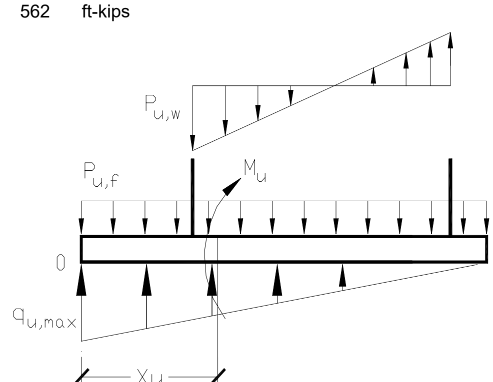
CHECK FOOTING CAPACITY (STRENGTH DESIGN)

$M_{u,R}$ = $1.2\,[P_{r,DL}\,(L_1 + a) + P_f\,(0.5\,L) + P_w\,(L_1 + 0.5L_w)] + 0.5\,P_{r,LL}(L_1 + a) =$ 562 ft-kips

$M_{u,o}$ = $1.6\,[F(h + D) + M] =$ 480 ft-kips

P_u = $1.2\,(P_{r,DL} + P_f + P_w) + 0.5\,P_{r,LL} =$ 40 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 12.01 ft

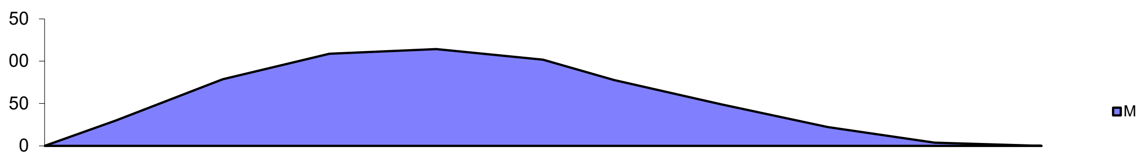


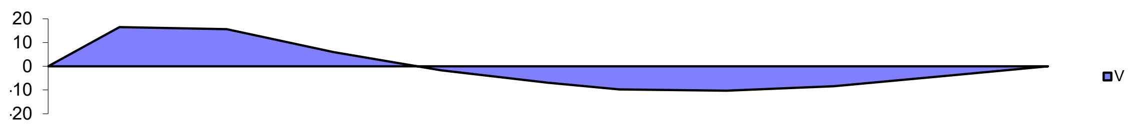
$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for \quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for \quad e_u > \dfrac{L}{6} \end{cases} = 4.33 \text{ ksf}$$

(cont'd)

BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.81	5.63	8.44	11.25	14.07	16.88	19.69	22.50	25.32	28.13
$P_{u,w}$ (klf)	0.0	3.9	3.0	2.2	1.3	0.5	-0.3	-1.2	-2.0	-2.9	0.0
$M_{u,w}$ (ft-k)	0	-10	-49	-113	-193	-285	-380	-473	-556	-623	-668
$V_{u,w}$ (kips)	0	-9	-19	-26	-31	-34	-34	-32	-27	-20	-13
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-4	-15	-34	-60	-93	-135	-183	-239	-303	-374
$V_{u,f}$ (kips)	0	-3	-5	-8	-11	-13	-16	-19	-21	-24	-27
q_u (ksf)	-4.3	-2.3	-0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	44	143	255	368	480	592	705	817	930	1042
$V_{u,q}$ (kips)	0	28	40	40	40	40	40	40	40	40	40
ΣM_u (ft-k)	**0**	**30**	**79**	**109**	**114**	**102**	**78**	**49**	**22**	**4**	**0**
ΣV_u (kips)	**0**	**17**	**16**	**6**	**-2**	**-7**	**-10**	**-10**	**-8**	**-4**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	17.75	0.0000	0.0000	17	kips	61	kips
Bottom Longitudinal	114	ft-k	17.63	0.0023	0.0028	17	kips	60	kips
Bottom Transverse	0	ft-k / ft	16.94	0.0000	0.0000	1	kips / ft	19	kips / ft

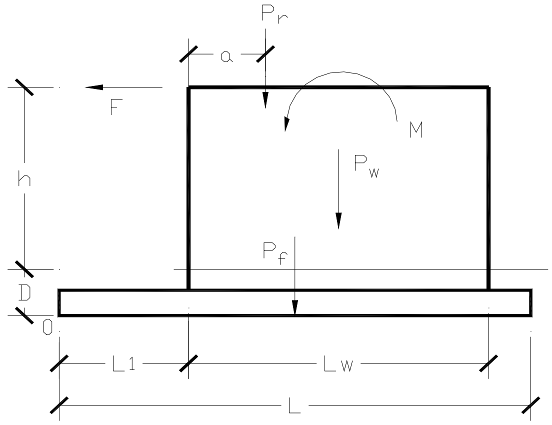
Where
$$\rho = \frac{0.85 f_c' \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f_c'}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206

[Satisfactory]

Footing Design of Shear Wall Based on ACI 318-14



INPUT DATA

WALL LENGTH	L_w =	20.2	ft			
WALL HEIGHT	h =	12.83	ft			
WALL THICKNESS	t =	6	in			
FOOTING LENGTH	L =	21.5333	ft			
	L_1 =	0.66665	ft			
FOOTING WIDTH	B =	3	ft			
FOOTING THICKNESS	T =	18	in			
FOOTING EMBEDMENT DEPTH	D =	4.5	ft			
ALLOWABLE SOIL PRESSURE	q_a =	1.5	ksf			
DEAD LOAD AT TOP WALL	$P_{r,DL}$ =	0.1	kips			
LIVE LOAD AT TOP WALL	$P_{r,LL}$ =	0.1	kips			
TOP LOAD LOCATION	a =	10.1	ft			
WALL SELF WEIGHT	P_w =	9.5	kips			
LATERAL LOAD TYPE (0=wind,1=seismic)		0	Wind - Service			
WIND LOADS AT WALL TOP	F =	9.4	kips			THE FOOTING DESIGN IS ADEQUATE.
	M =		ft-kips			
CONCRETE STRENGTH	f_c' =	4	ksi			
REBAR YIELD STRESS	f_y =	60	ksi			
TOP BARS, LONGITUDINAL		0	#	4		< == Not Required
BOTTOM BARS, LONGITUDINAL		4	#	6		
BOTTOM BARS, TRANSVERSE	#	5	@	12	in o.c.	< == Not Required

Uniform load acting on the middle of the wall

ANALYSIS

CHECK OVERTURNING FACTOR (IBC 18 1605.3.1, 1808.3.1, & ASCE 7-16 12.13.4)

$F = M_R / M_O =$ 1.56 > 1.5 for wind [Satisfactory]

Where P_f = 14.05048 kips (footing self weight)

$M_O = F (h + D) + M =$ 163 ft-kips (overturning moment)

$M_R = (P_{r,DL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 255 ft-kips (resisting moment without live load)

CHECK SOIL CAPACITY (ALLOWABLE STRESS DESIGN)

P_s = 9.689985 kips (soil weight in footing size)

$P = (P_{r,DL} + P_{r,LL}) + P_w + (P_f - P_s) =$ 14.06 kips (total vertical net load)

$M_R = (P_{r,DL} + P_{r,LL}) (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w) =$ 256 ft-kips (resisting moment with live load)

$e = 0.5 L - (M_R - M_O) / P =$ 4.17 ft (eccentricity from middle of footing)

$$q_{MAX} = \begin{cases} \dfrac{P\left(1+\dfrac{6e}{L}\right)}{BL}, & for\quad e \le \dfrac{L}{6} \\ \dfrac{2P}{3B(0.5L-e)}, & for\quad e > \dfrac{L}{6} \end{cases} = 0.47 \text{ ksf} < 4/3\, q_a$$

[Satisfactory]

Where e = 4.17 ft, > (L / 6)

CHECK FOOTING CAPACITY (STRENGTH DESIGN)

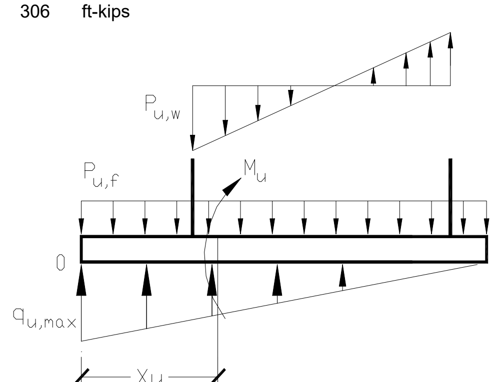
$M_{u,R}$ = $1.2 [P_{r,DL} (L_1 + a) + P_f (0.5 L) + P_w (L_1 + 0.5L_w)] + 0.5 P_{r,LL}(L_1 + a) =$ 306 ft-kips

$M_{u,o}$ = $1.6 [F(h + D) + M] =$ 261 ft-kips

P_u = $1.2 (P_{r,DL} + P_f + P_w) + 0.5 P_{r,LL} =$ 28 kips

$e_u = 0.5L - (M_{u,R} - M_{u,O}) / P_u =$ 9.17 ft

$$q_{u,MAX} = \begin{cases} \dfrac{P_u\left(1+\dfrac{6e_u}{L}\right)}{BL}, & for\quad e_u \le \dfrac{L}{6} \\ \dfrac{2P_u}{3B(0.5L-e_u)}, & for\quad e_u > \dfrac{L}{6} \end{cases} = 3.95 \text{ ksf}$$



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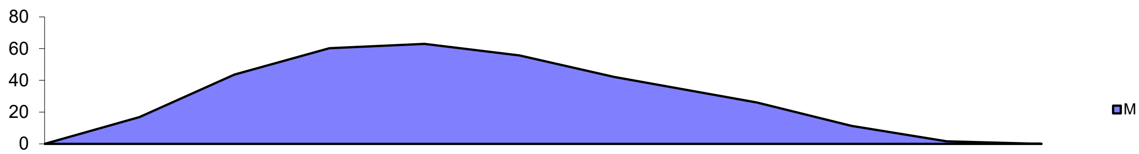
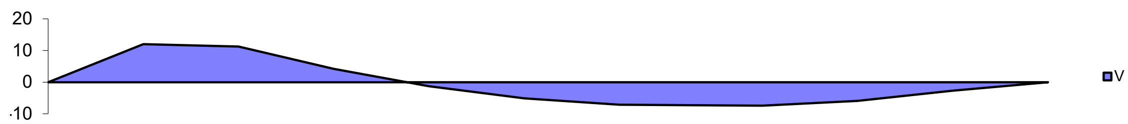
BENDING MOMENT & SHEAR AT EACH FOOTING SECTION

Section	0	1/10 L	2/10 L	3/10 L	4/10 L	5/10 L	6/10 L	7/10 L	8/10 L	9/10 L	L
X_u (ft)	0	2.15	4.31	6.46	8.61	10.77	12.92	15.07	17.23	19.38	21.53
$P_{u,w}$ (klf)	0.0	3.8	3.0	2.2	1.4	0.6	-0.2	-1.1	-1.9	-2.7	0.0
$M_{u,w}$ (ft-k)	0	-5	-26	-62	-107	-160	-214	-268	-317	-357	-385
$V_{u,w}$ (kips)	0	-6	-14	-19	-23	-25	-25	-24	-21	-16	-12
$P_{u,f}$ (ksf)	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
$M_{u,f}$ (ft-k)	0	-2	-7	-16	-29	-45	-65	-89	-116	-147	-182
$V_{u,f}$ (kips)	0	-2	-3	-5	-7	-8	-10	-12	-13	-15	-17
q_u (ksf)	-4.0	-2.2	-0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
$M_{u,q}$ (ft-k)	0	23	77	138	199	261	322	383	444	506	567
$V_{u,q}$ (kips)	0	20	28	28	28	28	28	28	28	28	28
ΣM_u (ft-k)	**0**	**17**	**44**	**60**	**63**	**56**	**42**	**26**	**11**	**2**	**0**
ΣV_u (kips)	**0**	**12**	**11**	**4**	**-1**	**-5**	**-7**	**-7**	**-6**	**-3**	**0**





Location	$M_{u,max}$		d (in)	ρ_{reqD}	ρ_{provD}	$V_{u,max}$		$\phi V_c = 2\,\phi\, b\, d\, (f_c')^{0.5}$	
Top Longitudinal	0	ft-k	14.75	0.0000	0.0000	12	kips	50	kips
Bottom Longitudinal	63	ft-k	14.63	0.0018	0.0033	12	kips	50	kips
Bottom Transverse	0	ft-k / ft	13.94	0.0000	0.0000	1	kips / ft	16	kips / ft

Where
$$\rho = \frac{0.85 f'_c \left(1 - \sqrt{1 - \frac{M_u}{0.383 b d^2 f'_c}}\right)}{f_y}$$

ρ_{min} = 0.0018

$$\rho_{MAX} = \frac{0.85 \beta_1 f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} =$$ 0.0206

[Satisfactory]

Design Method	Allowable Stress Design (ASD)
Connection Type	Lateral loading
Fastener Type	Bolt
Loading Scenario	Single Shear - Wood Main Member

Main Member Type	Douglas Fir-Larch
Main Member Thickness	1.5 in.
Main Member: Angle of Load to Grain	0
Side Member Type	Steel
Side Member Thickness	1/4 in.
Side Member: Angle of Load to Grain	0
Fastener Diameter	5/8 in.
Load Duration Factor	C_D = 1.6
Wet Service Factor	C_M = 1.0
Temperature Factor	C_t = 1.0

Connection Yield Modes

Im	2100 lbs.
Is	5438 lbs.
II	1166 lbs.
IIIm	1821 lbs.
IIIs	1930 lbs.
IV	2454 lbs.

Adjusted ASD Capacity	**1166 lbs.**

- Bolt bending yield strength of 45,000 psi is assumed.
- The Adjusted ASD Capacity is only applicable for bolts with adequate end distance, edge distance and spacing per NDS chapter 11.
- ASTM A36 Steel is assumed for steel side members 1/4 in. thick, and ASTM A653 Grade 33 Steel is assumed for steel side members less than 1/4 in. thick.

While every effort has been made to insure the accuracy of the information presented, and special effort has been made to assure that the information reflects the state-of-the-art, neither the American Wood Council nor its members assume any responsibility for any particular design prepared from this on-line Connection Calculator. Those using this on-line Connection Calculator assume all liability from its use.

The Connection Calculator was designed and created by Cameron Knudson, Michael Dodson and David Pollock at Washington State University. Support for development of the Connection Calculator was provided by American Wood Council.

ANCHOR BOLT SPACING SUMMARY

SW	Max Unit Shear (k)	Adjusted ASD Anchor Capacity (k)	Anchor Bolt Spacing
SW1	0.168	1.166	48" O.C.
SW2	0.332	1.166	40" O.C.
SW3	0.149	1.166	48" O.C.
SW4	0.501	1.166	24" O.C.
SW5	0.371	1.166	32" O.C.

$$SW1 = \frac{1.166 * 12}{0.168} = 83'' \xrightarrow{\text{Use}} 48''\text{O.C}$$

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Supporting Information

PAB

Pre-Assembled Anchor Bolt

The PAB anchor bolt is a versatile cast-in-place anchor bolt ideal for high-tension-load applications, such as rod systems and shearwalls. It features a plate washer, at the embedded end, sandwiched between two fixed hex nuts and a head stamp for easy identification after the pour.

- Available in diameters from ½" to 1¼" in lengths from 12" to 36" (in 6" increments)
- Available in standard and high-strength steel
- Head stamp contains the No Equal sign, diameter designation and an "H" on high-strength rods

Material:
Standard Steel — ASTM F1554 Grade 36, A36 or A307; F_u = 58 ksi
High-Strength Steel (up to 1" dia.) — ASTM A449; F_u = 120 ksi
High-Strength Steel (1⅛" and 1¼" dia.) — ASTM A193 B7 or F1554 Grade 105; F_u = 125 ksi

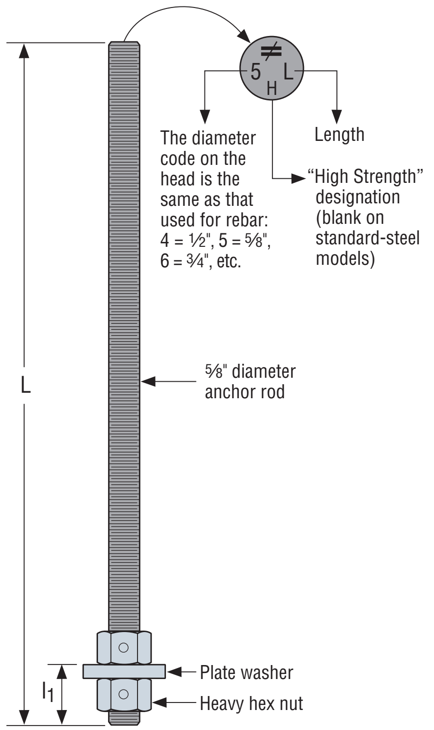
Finish: None. May be ordered in HDG; contact Simpson Strong-Tie.

Installation:

- On HDG PABs, chase the threads to use standard nuts or couplers or use overtapped products in accordance with ASTM A563; for example, Simpson Strong-Tie® NUT⅝-OST, NUT⅞-OST, CNW⅝-OST, CNW⅞-OST. OST couplers are typically oversized on one end of the coupler nut only and will be marked with an "O" on oversized side. Couplers may be special ordered with both ends oversized. Contact Simpson Strong-Tie.

Related Software

The Simpson Strong-Tie Anchor Designer™ Software analyzes and suggests anchor solutions using the ACI 318 strength-design methodology (or CAN/CSA A23.3 Annex D Limit States Design methodology). It provides cracked and uncracked-concrete anchorage solutions for numerous Simpson Strong-Tie mechanical and adhesive anchors as well as the PAB anchor bolt. With its easy-to-use graphical user interface, the software makes it easy for the designer to identify anchorage solutions without having to perform time-consuming calculations by hand. See **strongtie.com/software**.



PAB

Naming Legend

PAB5H-12

PAB Anchor Bolt — Diameter* and Grade — Length (12", 18", 24", 30" or 36")

*Units in ⅛" Increments (Ex: 9 = ⁹⁄₈" or 1⅛")

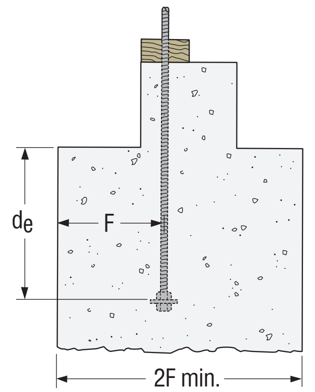
How to Specify and Order:

- When calling out PAB anchor bolts, substitute the desired length for the "XX" in the Root Model Number
- For a ⅝" x 18" anchor bolt, the model number would be PAB5-18 (or PAB5H-18 for high strength)

PAB Anchor Bolt

Diameter (in.)	Plate Washer Size (in.)	l_1 (in.)	Root Model No. Standard Strength	Root Model No. High Strength	Lengths (in.)
½	⅜ x 1½ x 1½	1⅛	PAB4—XX	PAB4H—XX	12" to 36" (in 6" increments)
⅝	½ x 1¾ x 1¾	1⅜	PAB5—XX	PAB5H—XX	
¾	½ x 2¼ x 2¼	1½	PAB6—XX	PAB6H—XX	
⅞	½ x 2½ x 2½	1⅝	PAB7—XX	PAB7H—XX	
1	⅝ x 3 x 2¾	1⅞	PAB8—XX	PAB8H—XX	
1⅛	⅝ x 3½ x 3¼	2	PAB9—XX	PAB9H—XX	
1¼	¾ x 3½ x 3½	2¼	PAB10—XX	PAB10H—XX	

1. Lengths greater than 36" are available as a special order.
2. Plate washers are designed to develop the capacity of the bolt.



Design loads are calculated using a full shear cone. Coverage on each side of the bolt shall be a minimum of F or reductions must be taken.

PAB

Pre-Assembled Anchor Bolt (cont.)

PAB Anchor Bolt – Anchorage Solutions

Design Criteria	Diameter (in.)	Anchor Bolt	2,500 psi Concrete: Dimensions (in.) d_e	2,500 psi Concrete: Dimensions (in.) F	2,500 psi Concrete: Tension Load ASD	2,500 psi Concrete: Tension Load LRFD	3,000 psi Concrete: Dimensions (in.) d_e	3,000 psi Concrete: Dimensions (in.) F	3,000 psi Concrete: Tension Load ASD	3,000 psi Concrete: Tension Load LRFD
	½	PAB4	4½	7	4,270	6,405	4	6	4,270	6,405
SW1 SW2 SW3	⅝	PAB5	4	6	4,030	6,720	4	6	4,415	7,360
			6	9	6,675	10,010	5½	8½	6,675	10,010
	¾	PAB6	5½	8½	6,500	10,835	5	7½	6,175	10,290
			7½	11½	9,610	14,415	7	10½	9,610	14,415
SW4 SW5 Wind	⅞	PAB7	6	9	7,405	12,345	5½	8½	7,120	11,870
			9	13½	13,080	19,620	8½	13	13,080	19,620
		PAB7H	9	13½	13,610	22,680	8½	13	13,680	22,805
			14	21	27,060	40,590	13½	20½	27,060	40,590
	1	PAB8	8	12	11,405	19,005	7½	11½	11,340	18,900
			10½	16	17,080	25,565	10	15	17,080	25,560
		PAB8H	10½	16	17,150	28,580	10	15	17,460	29,100
			16½	25	35,345	53,015	15½	23½	35,345	53,015
	1⅛	PAB9	9	13½	13,610	22,680	8	12	12,495	20,820
			12½	19	21,620	32,430	12	18	21,620	32,430
	1¼	PAB10	14	21	26,690	40,035	13½	20½	26,690	40,035
Seismic	½	PAB4	5	7½	4,270	6,405	4½	7	4,270	6,405
	⅝	PAB5	6½	10	6,675	10,010	6	9	6,675	10,010
	¾	PAB6	7½	11½	9,060	12,940	7	10½	8,945	12,780
			8	12	9,610	14,415	7½	11½	9,610	14,415
	⅞	PAB7	9	13½	11,905	17,010	8½	13	11,970	17,100
			10	15	13,080	19,620	9½	14½	13,080	19,620
		PAB7H	14½	22	25,350	36,215	13½	20½	24,650	35,215
			15½	23½	27,060	40,590	14½	22	27,060	40,590
	1	PAB8	11	16½	15,996	22,850	10½	16	16,435	23,480
			11½	17½	17,080	25,625	11	16½	17,080	25,625
		PAB8H	17	25½	33,045	47,205	16	24	32,720	46,740
			18	27	35,345	53,015	17	25½	35,345	53,015
	1⅛	PAB9	12½	19	19,795	28,275	12	18	20,255	28,940
			13½	20½	21,620	32,430	12½	19	21,620	32,430
	1¼	PAB10	14½	22	25,350	36,215	14	21	26,190	37,415
			15	22½	26,690	40,035	14½	22	26,690	40,035

1. Anchorage designs conform to ACI 318-14 and assume cracked concrete with no supplementary reinforcement.
2. Seismic indicates Seismic Design Category C-F and designs comply with ACI 318-14, Section 17.2.3.4. Per Section 1613 of the 2012/2015/2018/2021 IBC, detached one- and two-family dwellings in SDC C may use wind values.
3. Wind includes Seismic Design Category A and B.
4. Foundation dimensions are for anchorage only. Foundation design (size and reinforcement) by designer. The registered design professional may specify alternative embedment, footing size, and anchor bolt.
5. Where tension loads are governed by anchor steel, the design provisions from AISC 360 are used to determine the tensile steel limit. LRFD values are calculated by multiplying the nominal AISC steel capacity by a 0.75 phi factor, and allowable values are calculated by dividing the AISC nominal capacity by a 2.0 omega factor.
6. Where tension loads are governed by ACI 318 concrete limit, the Allowable Stress Design (ASD) values are obtained by multiplying Load Resistance Factor Design (LRFD) capacities by 0.7 for Seismic and by 0.6 for Wind.

HDB/HD

SIMPSON Strong-Tie®

Holdowns

Simpson Strong-Tie offers a wide variety of bolted holdowns offering low-deflection performance for a range of load requirements.

The HD3B is a light-duty holdown designed for use in shearwalls and braced-wall panels, as well as other lateral applications.

The HD5B, HD7B and HD9B bolted holdowns incorporate the proven design of our HDQ8 SDS-style holdown and feature a unique seat design which greatly minimizes deflection under load. HDB and HD holdowns are self-jigging, ensuring that the code-required minimum of seven bolt diameters from the end of the post is met. They can be installed directly on the sill plate or raised above it and are suitable for back-to-back applications where eccentricity is a concern. HDBs and HDs are designed to provide loads for intermediate-load-range shearwalls, braced-wall panels and lateral applications.

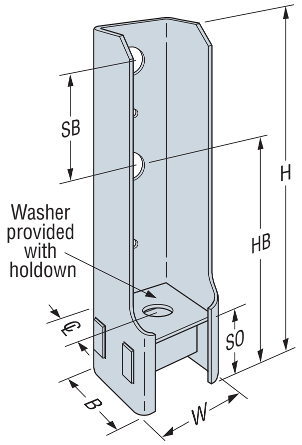
Material: See table

Finish: HD3B/HD5B/HD7B/HD9B — Galvanized;
HD — Simpson Strong-Tie gray paint; HDG available.
For stainless steel options, see engineering letter L-C-SSHD at **strongtie.com**.

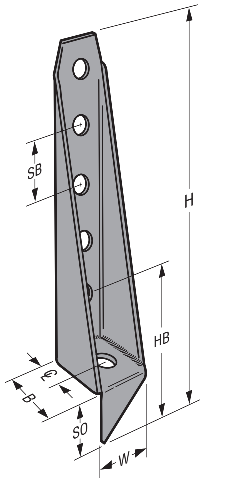
Installation:

- See Holdown and Tension Tie General Notes on pp. 49–50
- Bolt holes shall be a minimum of 1⁄32" to a maximum of 1⁄16" larger than the bolt diameter (per 2015/2018 NDS, section 12.1.3.2)
- Stud bolts should be snugly tightened with standard cut washers between the wood and nut (BPs are required in the City and County of Los Angeles)
- HD and HDB holdowns are self-jigging and will ensure minimum bolt end distance when installed flush with the sill plate
- Standard cut washer is required under the anchor nut for HD12 with 1" anchor and HD19 with 1⅛" anchors

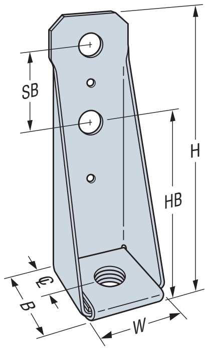
Codes: See p. 11 for Code Reference Key Chart



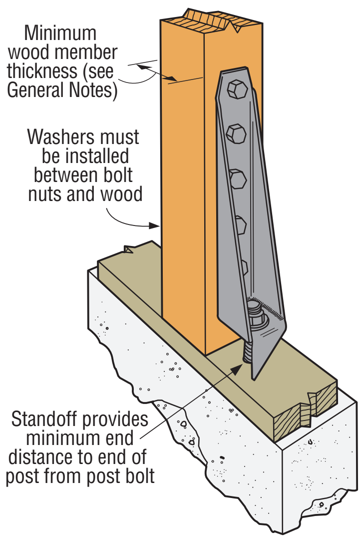
HD5B
(HD7B and HD9B similar)



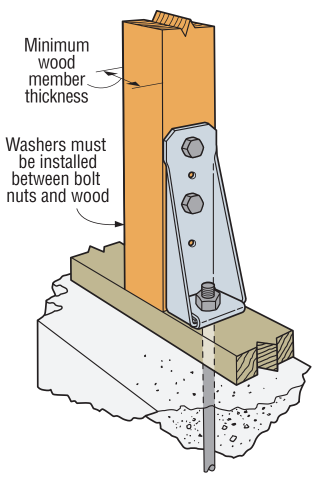
HD19
(HD12 similar)



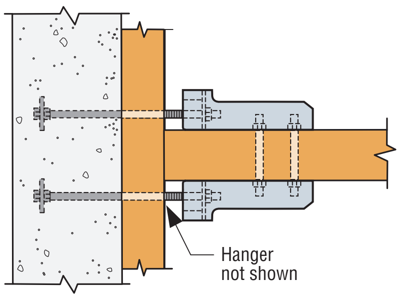
HD3B



Vertical HD19 Installation



Vertical HD3B Installation



Horizontal HDB Installation
(plan view)

Holdowns (cont.)

These products are available with additional corrosion protection. For more information, see p. 14.

	Model No.	Material		Dimensions (in.)							Fasteners (in.)		Minimum Wood Member Size (in.)	Allowable Tension Loads (160)		Deflection at Highest Allowable Load	Code Ref.
		Base (in.)	Body (ga.)	HB	SB	W	H	B	CL	SO	Anchor Dia. Bolt	Stud Bolts		DF/SP	SPF/HF		
	HD3B	—	12	4¾	2½	2½	8⅝	2¼	1 5/16	⅜	⅝	(2) ⅝	1½ x 3½	1,895	1,610	0.156	IBC, FL, LA
													2½ x 3½	2,525	2,145	0.169	
													3 x 3½	3,130	3,050	0.12	
													3½ x 3½	3,130	3,050	0.12	
SW1 SW2 SW3	HD5B	3/16	10	5¼	3	2½	9⅜	2½	1¼	2	⅝	(2) ¾	1½ x 3½	2,405	2,070	0.153	
													2½ x 3½	3,750	3,190	0.129	
													3 x 3½	4,505	3,785	0.156	
													3½ x 3½	4,935	4,195	0.15	
	HD7B	3/16	10	5¼	3	2½	12⅜	2½	1¼	2	⅞	(3) ¾	3 x 3½	6,645	5,650	0.142	
													3½ x 3½	7,310	6,215	0.154	
													3½ x 4½	7,345	6,245	0.155	
SW4 SW5	HD9B	⅜	7	6⅛	3½	2⅞	14	2½	1¼	2⅜	⅞	(3) ⅞	3½ x 3½	7,740	6,580	0.159	
													3½ x 4½	9,920	8,430	0.178	
													3½ x 5½	9,920	8,430	0.178	
													3½ x 7¼	10,035	8,530	0.179	
	HD12	⅜	3	7	4	3½	20 5/16	4¼	2⅛	3⅝	1	(4) 1	3½ x 3½	11,350	9,215	0.171	
													3½ x 4½	12,665	10,765	0.171	
													5½ x 5½	14,220	12,085	0.162	
											1⅛	(4) 1	3½ x 3½	11,775	9,215	0.171	
													3½ x 4½	13,335	11,055	0.177	
													3½ x 7¼	15,435	13,120	0.194	
													5½ x 5½	15,510	12,690	0.162	
	HD19	⅜	3	7	4	3½	24½	4¼	2⅛	3⅝	1⅛	(5) 1	3½ x 7¼	16,735	14,225	0.191	
													5½ x 5½	16,775	12,690	0.2	
											1¼	(5) 1	3½ x 7¼	19,360	15,270	0.18	
													5½ x 5½	19,070	16,210	0.137	

1. To achieve published loads, machine bolts shall be installed with the nut on the opposite side of the holdown. If this orientation is reversed, the designer shall reduce the allowable loads shown per NDS requirements when bolt threads are in the shear plane.
2. All references to bolts are for structural quality through bolts (not lag screw or carriage bolts) equal to or better than ASTM A307, Grade A.
3. HD19 with 1¼" anchor rod requires No.1 post (or better) to achieve published loads.

HRS/ST/HTP/LSTA/LSTI/MST/MSTA/MSTC/MSTI

SIMPSON Strong-Tie

Strap Ties

Straps are designed to transfer tension loads in a wide variety of applications.

HRS — Heavy strap designed for installation on the edge of 2x members. The HRS416Z installs with Strong-Drive® SDS Heavy-Duty Connector screws.

HTP — Heavy tie plate designed for installation on the side of 2x4 or larger members.

LSTA and MSTA — Designed for use on the edge of 2x members, with a nailing pattern that reduces the potential for splitting.

LSTI and MSTI — Light and medium straps that are suitable where pneumatic-nailing is necessary through diaphragm decking and wood chord open-web trusses.

MST — High-capacity strap that can be installed with either nails or bolts. Suitable for double 2x member connections or greater.

MSTC — High-capacity strap that utilizes a staggered nail pattern to help minimize wood splitting. Nail slots have been countersunk to provide a lower nail head profile.

Finish: Galvanized. Some products are available in stainless steel, ZMAX® coating or black powder coat (add PC to SKU); contact Simpson Strong-Tie. See Corrosion Information, pp. 12–15.

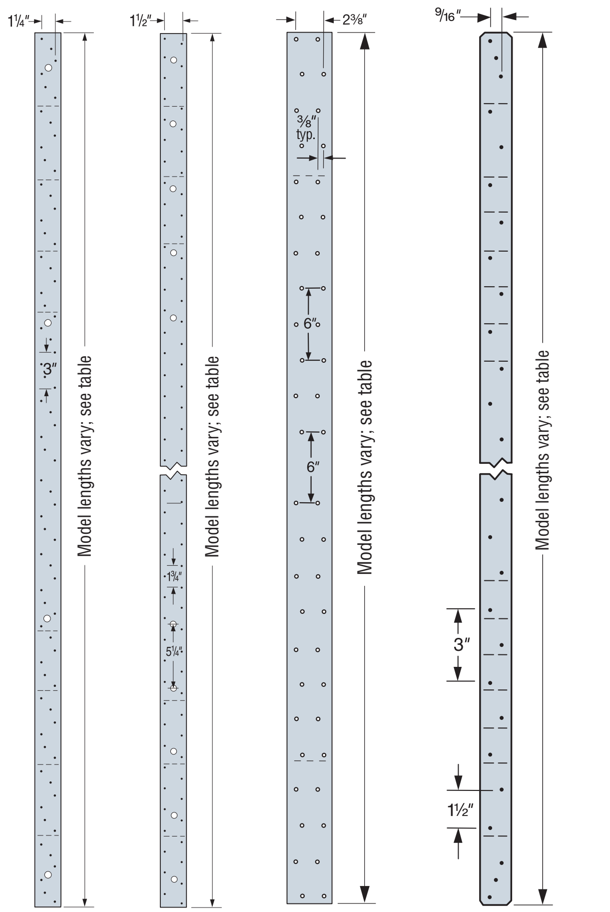
Installation: Use all specified fasteners; see General Notes

Options: Special sizes can be made to order; contact Simpson Strong-Tie

Codes: See p. 11 for Code Reference Key Chart

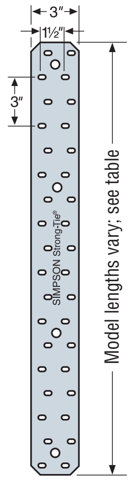
MSTC and RPS meet code requirements for reinforcing cut members (16 gauge) at top plate and RPS at sill plate.
International Residential Code® — 2012/2015/2018/2021 R602.6.1
International Building Code® — 2012 2308.9.8; 2015/2018/2021 2308.5.8

(For RPS, refer to p. 309. For CTS218 compression and tension strap, see p. 307.)

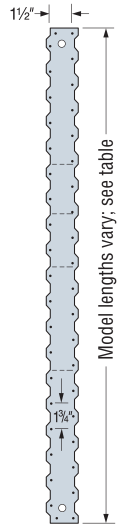


MSTI MST LSTI LSTA and MSTA

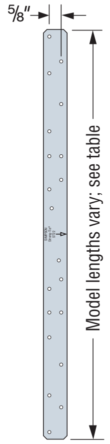
(pilot holes not shown)



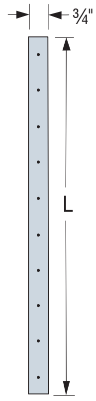
MSTC



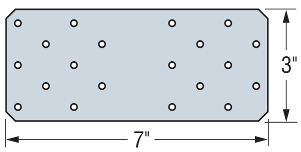
ST292, ST2122, ST2215, ST6215, ST6224, ST6236



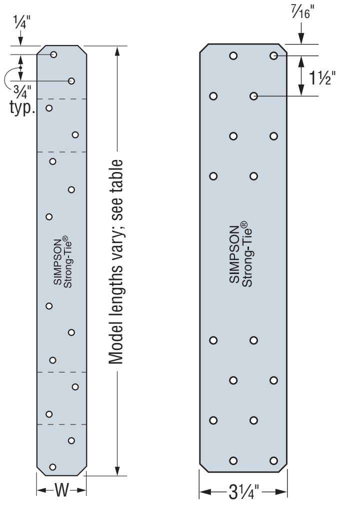
ST9, ST12, ST18, ST22



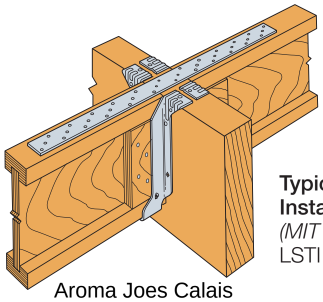
ST2115



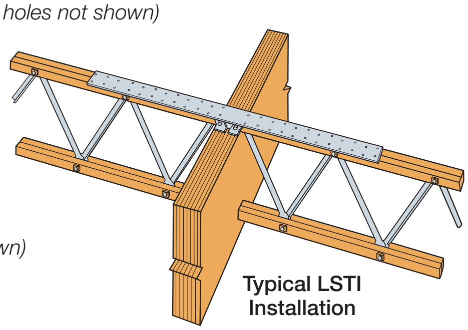
HTP37Z



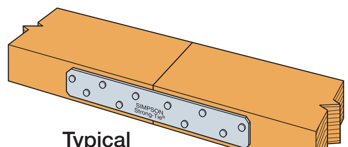
HRS HRS416Z



Typical MSTI Installation
(MIT hanger shown)
LSTI similar



Typical LSTI Installation



Typical HRS Installation

Aroma Joes Calais
TKA #24626

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Straps and Ties

Strap Ties (cont.)

Codes: See p. 11 for Code Reference Key Chart

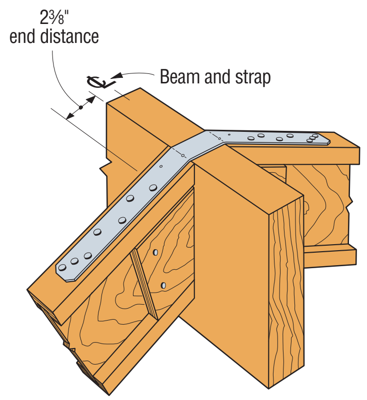
These products are available with additional corrosion protection. For more information, see p. 14.

SS For stainless-steel fasteners, see p. 21.

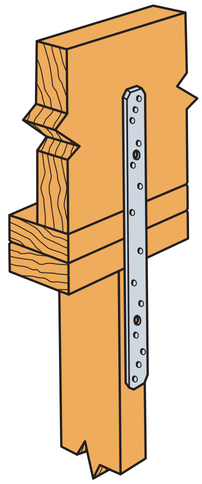
SD Many of these products are approved for installation with Strong-Drive® SD Connector screws. See pp. 348–352 for more information.

	Model No.	Ga.	Dimensions (in.)		Fasteners (Total) (in.)	Allowable Tension Loads (DF/SP)	Allowable Tension Loads (SPF/HF)	Code Ref.
			W	L		(160)	(160)	
	ST2115	20	¾	16 5/16	(10) 0.162 x 2½	660	660	IBC, FL, LA
	LSTA9		1¼	9	(8) 0.148 x 2½	740	635	
	LSTA12		1¼	12	(10) 0.148 x 2½	925	795	
	LSTA15		1¼	15	(12) 0.148 x 2½	1,110	955	
	LSTA18		1¼	18	(14) 0.148 x 2½	1,235	1,115	
	LSTA21		1¼	21	(16) 0.148 x 2½	1,235	1,235	
	LSTA24		1¼	24	(18) 0.148 x 2½	1,235	1,235	
	LSTA30	18	1¼	30	(22) 0.148 x 2½	1,640	1,640	
	LSTA36		1¼	36	(24) 0.148 x 2½	1,640	1,640	
	MSTA9		1¼	9	(8) 0.148 x 2½	750	650	
SS	MSTA12		1¼	12	(10) 0.148 x 2½	940	810	
	MSTA15		1¼	15	(12) 0.148 x 2½	1,130	970	
SS	MSTA18		1¼	18	(14) 0.148 x 2½	1,315	1,135	
	MSTA21		1¼	21	(16) 0.148 x 2½	1,505	1,295	
SS	MSTA24		1¼	24	(18) 0.148 x 2½	1,640	1,460	
	MSTA30	16	1¼	30	(22) 0.148 x 2½	2,050	1,825	
SS	MSTA36		1¼	36	(26) 0.148 x 2½	2,050	2,050	
	MSTA49		1¼	49	(26) 0.148 x 2½	2,020	2,020	
	ST9		1¼	9	(8) 0.162 x 2½	885	765	
	ST12		1¼	11⅝	(10) 0.162 x 2½	1,105	955	
	ST18		1¼	17¾	(14) 0.162 x 2½	1,420	1,335	
	ST22		1¼	21⅝	(18) 0.162 x 2½	1,420	1,420	
	HRS6	12	1⅜	6	(6) 0.148 x 2½	605	530	
	HRS8		1⅜	8	(10) 0.148 x 2½	1,010	880	
	HRS12		1⅜	12	(14) 0.148 x 2½	1,415	1,230	
	ST292	20	2 1/16	9 5/16	(12) 0.162 x 2½	1,260	1,120	
	ST2122		2 1/16	12 13/16	(16) 0.162 x 2½	1,530	1,510	
	ST2215		2 1/16	16 5/16	(20) 0.162 x 2½	1,875	1,875	
	ST6215	16	2 1/16	16 5/16	(20) 0.162 x 2½	2,090	1,910	
	ST6224		2 1/16	23 5/16	(28) 0.162 x 2½	2,535	2,535	
	ST6236	14	2 1/16	33 13/16	(40) 0.162 x 2½	3,845	3,845	
	MSTI26	12	2 1/16	26	(26) 0.148 x 1½	2,745	2,380	
	MSTI36		2 1/16	36	(36) 0.148 x 1½	3,800	3,295	
	MSTI48		2 1/16	48	(48) 0.148 x 1½	5,070	4,390	
	MSTI60		2 1/16	60	(60) 0.148 x 1½	5,070	5,070	
	MSTI72		2 1/16	72	(72) 0.148 x 1½	5,070	5,070	
Drag	HTP37Z	16	3	7	(20) 0.148 x 1½	900	690	
Strut	MSTC28		3	28¼	(36) 0.148 x 3¼	3,460	2,990	
SW1	MSTC40		3	40¼	(52) 0.148 x 3¼	4,735	4,315	
SW2	MSTC52		3	52¼	(62) 0.148 x 3¼	4,735	4,735	
SW3	MSTC66	14	3	65¾	(76) 0.148 x 3¼	5,850	5,850	
	MSTC78		3	77¾	(76) 0.148 x 3¼	5,850	5,850	
	HRS416Z	12	3¼	16	(16) ¼ x 1½ SDS	2,835	2,305	—
	LSTI49	18	3¾	49	(32) 0.148 x 1½	2,970	2,560	IBC, FL, LA
	LSTI73		3¾	73	(48) 0.148 x 1½	4,205	3,840	

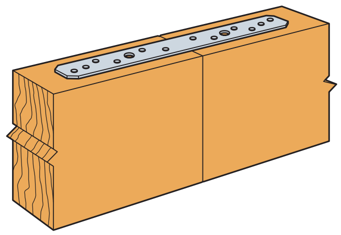
1. See pp. 266–267 for Straps and Ties General Notes.
2. **Fasteners:** Nail dimensions are listed diameter by length. SDS screws are Simpson Strong-Tie® Strong-Drive SDS Heavy-Duty Connector screws. See pp. 21–22 for fastener information.



Typical LSTA Installation
(hanger not shown)
Bend strap one time only, max. 12/12 joist pitch.



Typical LSTA18 Installation



Typical MSTA15 Installation

Straps and Ties

ZIP SYSTEM

1", 1-1/2", 2", 2-1/2" ZIP SYSTEM® R-SHEATHING						
PANEL TYPE[2]	TOTAL THICKNESS	PANEL SIZE	PANEL COUNT	R-VALUE	CODE EVALUATION REPORT	AIR BARRIER
R-3	1"	4' x 8' 4' x 9' 4' x 10'	32	3.6	ESR 3373 ER 482	ASTM E 2178 <0.02 L/(s·m²) @ 75 Pa ASTM E 2357 <0.2 L/(s·m²) @ 75 Pa
R-6	1-1/2"		31	6.6		
R-9	2"		23	9.6		
R-12	2-1/2"		18	12.6		

FOAM PERFORMANCE		
PROPERTY	TEST METHOD	TYPICAL RESULTS
Dimensional Stability	ASTM D 2126	< 2%
Compressive Strength	ASTM D 1621	20 psi
Water Absorption	ASTM C 209 ASTM D 2842	< 1% < 3.5%
Water Vapor Transmission	ASTM E 96	< 1.0 perm
Density	ASTM D 1622	Nominal 2.0 pcf
Flame Spread	ASTM E 84	40-60
Smoke Developed	ASTM E 84	50-170
Tensile Strength	ASTM D 1623	> 730 psf
Service Temperature		-40°F – 200°F

ZIP SYSTEM® PERFORMANCE		
PROPERTY	TEST METHOD	TYPICAL RESULTS
Water Resistance of Coatings	ASTM D 2247 (for 14 days)	Passed
Drainage Efficiency	ASTM E 2273	> 90%
Water Vapor Transmission	ASTM E 96B	12-16 perms (overlay)
Water Penetration	ASTM E 331	Passed
Air Barrier Assembly	ASTM E 2357 at 75 Pa	0.037 L/(s·m²)
Wind-Driven Rain	TAS-100	Passed 100mph
Accelerated Weathering	ASTM G 154	Passed

Long term thermal resistance values of the foam were determined in accordance with ASTM C 1289-02. The R-Value of 0.55 for 7/16" OSB was obtained from ASHRAE Handbook, Fundamentals.

FASTENING REQUIREMENTS FOR PRESCRIPTIVE BRACING[2,3] AND ENGINEERED SHEAR WALL DESIGN[4]

DESIGN RATED
ESR-3373
IAPMO ES ER-482

	FRAMING		FASTENERS			SHEAR VALUES[5]	
ZIP SYSTEM® R-SHEATHING TYPE	NOMINAL STUD SPACING (MIN.)	MAXIMUM STUD SPACING (IN.)	FASTENER SPECIFICATIONS[6]	EDGE/FIELD SPACING (IN.)	MINIMUM PENETRATION INTO FRAMING (IN.)	ALLOWABLE SEISMIC CONTROLLED SHEAR VALUES[7,8] (PLF)	ALLOWABLE WIND CONTROLLED SHEAR VALUES[7] (PLF)
R-3	2-by-4	24	0.131" shank nails	4/12	1.5	245	343
R-3	2-by-4	24	0.131" shank nails	3/12	1.5	280	393
R-3	2-by-4	16	16ga staples, 7/16" crown, 2" length	3/6	1.0	210	294
R-6	2-by-4	24	0.131" shank nails	4/12	1.5	230	322
R-6	2-by-4	24	15ga staples, 7/16" crown, 2.5" length	3/6	1.0	NA[9]	NA
R-6	2-by-4	24	0.131" shank nails	3/12	1.5	255	357
R-9	2-by-4	24	0.131" shank nails	3/12	1.5	240	336
R-12	2-by-4	24	0.131" shank nails	3/12	1.5	215	301

For SI: Inch = 25.4mm; 1 pound per foot (ppf) = 14.59 N/m.

1. Limitations and restrictions apply. Visit HuberWood.com/warranties for details.
2. Prescriptive bracing requirements under the 2018, 2015, 2012 and 2009 IRC.
3. Not approved for use as prescriptive wall bracing where wind design is required by R301.2.1.1.
4. Engineered shear wall requirements with Douglas Fir-Larch Framing under the 2015, 2012 and 2009 IBC.
5. For framing with other than Douglas Fir-Larch, the shear value must be multiplied by the Specific Gravity Adjustment Factor = [1 - (0.50 - SG)], where SG=Specific Gravity of the framing lumber in accordance with the ANSI/AWC NDS. This adjustment factor must not be greater than 1.
6. Fasteners must be common nails or equivalent, or staples, of a type generally used to attach wood sheathing.
7. The shearwalls must have a maximum height-to-width aspect ratio of 2:1.
8. ZIP System R-sheathing used as the lateral resistance system in seismic zones D_0, D_1, D_2 and E should be designed in accordance to ER-482.
9. This panel and fastening configuration is only applicable to the prescriptive bracing requirements under the 2015 IRC.

Alabama Building Code 2009

2306.7 Shear Walls Sheathed With Other Materials

Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing or gypsum board shall be designed and constructed in accordance with AF&PA SDPWS. Shear walls sheathed with these materials are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.7. Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing or gypsum board shall not be used to resist seismic forces in structures assigned to *Seismic Design Category* E or F.

TABLE 2306.7
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES

TYPE OF MATERIAL	THICKNESS OF MATERIAL	WALL CONSTRUCTION	FASTENER SPACING[b] MAXIMUM (inches)	SHEAR VALUE[a, e] (plf)	MINIMUM FASTENER SIZE[c, d, j, k]
1. Expanded metal or woven wire lath and portland cement plaster	7/8"	Unblocked	6	180	No. 11 gage 1 1/2" long, 7/16" head No. 16 gage galv. staple, 7/8" legs
2. Gypsum lath, plain or perforated with vertical joints staggered	3/8" lath and 1/2" plaster	Unblocked	5	180	No. 13 gage galv. 1 1/8" long, 19/64" head, plasterboard nail
3. Gypsum lath, plain or perforated	3/8'' lath and 1/2''plaster	Unblocked	5	100	No. 16 gage galv. staple 11/8''long, 0.120'' nail min. 3/8'' head, 11/4'' long
4. Gypsum sheathing	1/2" x 2' x 8'	Unblocked	4	75	No. 11 gage, 1 3/4" long, 7/16" head, diamond-point, galvanized
	1/2" x 4'	Blocked [d]	4	175	
		Unblocked	7	100	16 Ga. Galv. Stable, 1 3/4" long
	5/8" x 4'	Blocked	4" edge/ 7" field	200	6d galvanized 0.120" Nail, min. 3/8" head, 1 3/4" long
5. Gypsum board, gypsum veneer base or water-resistant gypsum backing board	1/2"	Unblocked[f]	7	75	5d cooler (1 5/8" × 0.086") or wallboard 0.120" nail, min. 3/8 " head, 1 1/2 " long No. 16 gage galv. staple, 1 1/2" long
		Unblocked[f]	4	110	
		Unblocked	7	100	
		Unblocked	4	125	
		Blocked[g]	7	125	
		Blocked[g]	4	150	
		Unblocked	8/12[h]	60	No. 6—1 1/4" screws[i]
		Blocked[g]	4/16[h]	160	
		Blocked[f, g]	4/12[h]	155	
		Blocked[g]	8/12[h]	70	

		Blocked[g]	6/12[h]	90	
	$^5/_8$"	Unblocked[f]	7	115	6d cooler ($1^7/_8$" × 0.092") or wallboard 0.120" nail, min. $^3/_8$" head, $1^3/_4$" long No. 16 gage galv. staple, $1^1/_2$" legs, $1^5/_8$" long
			4	145	
		Blocked[g]	7	145	
			4	175	
		Blocked[g] Two-ply	Base ply: 9 Face ply: 7	250	Base ply-6d cooler ($1^7/_8$" × 0.092") or wallboard $1^3/_4$" × 0.120" nail, min. $^3/_8$" head $1^5/_8$" 16 gage galv. staple Face ply-8d cooler ($2^3/_8$" × 0.113") or wallboard 0.120" nail, min. $^3/_8$" head, $2^3/_8$" long No. 15 gage galv. staple, $2^1/_4$" long
		Unblocked	8/12[h]	70	No. 6—$1^1/_4$" screws[i]
		Blocked[g]	8/12[h]	90	

For SI: 1 inch = 25.4 mm, 1 pound per linear foot = 14.5939 N/m.

a. These shear walls shall not be used to resist loads imposed by masonry or concrete walls (see Section 4.1.5 of AF & PA SDPWS). Values shown are for short-term loading due to wind or seismic loading. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7. Values shown shall be reduced 25 percent for normal loading.

b. Applies to fastening at studs, top and bottom plates and blocking.

c. Alternate fasteners are permitted to be used if their dimensions are not less than the specified dimensions. Drywall screws are permitted to substitute for the 5d ($1^5/_8$" × 0.086"), and 6d ($1^7/_8$" × 0.092")(cooler) nails listed above, and No. 6 $1^1/_4$ inch Type S or W screws for 6d ($1^7/_8$" × 0.092) (cooler) nails.

d. For properties of cooler nails, see ASTM C 514.

e. Except as noted, shear values are based on a maximum framing spacing of 16 inches on center.

f. Maximum framing spacing of 24 inches on center.

g. All edges are blocked, and edge fastening is provided at all supports and all panel edges.

h. First number denotes fastener spacing at the edges; second number denotes fastener spacing at intermediate framing members.

i. Screws are Type W or S.

j. Staples shall have a minimum crown width of $^7/_{16}$ inch, measured outside the legs, and shall be installed with their crowns parallel to the long dimension of the framing members.

k. Staples for the attachment of gypsum lath and woven-wire lath shall have a minimum crown width of 3/4 inch, measured outside the legs.

Table A.4.2A Nominal Unit Shear Capacities for Wood-Frame Plywood Diaphragms

Blocked Wood Structural Panel Diaphragms[1,2,3,4]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Framing Width (in.)	A SEISMIC								B WIND			
					Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)								Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
					6		4		2-1/2		2		6	4	2-1/2	2
					Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)								Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
					6		6		4		3		6	6	4	3
					v_s	G_a	v_s	G_a	v_s	G_a	v_s	G_a	v_w	v_w	v_w	v_w
					(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(plf)	(plf)	(plf)
Structural I	6d	1-1/4	5/16	2	370	12.0	500	7.5	750	10.0	840	15.0	520	700	1050	1175
				3	420	9.5	560	6.0	840	8.5	950	13.0	590	785	1175	1330
	8d	1-3/8	3/8	2	540	11.0	720	7.5	1060	10.0	1200	15.0	755	1010	1485	1680
				3	600	10.0	800	6.5	1200	9.0	1350	13.0	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	17.0	850	12.0	1280	15.0	1460	21.0	895	1190	1790	2045
				3	720	15.0	960	9.5	1440	13.0	1640	18.0	1010	1345	2015	2295
Sheathing and Single-Floor	6d	1-1/4	5/16	2	340	10.0	450	7.0	670	9.5	760	13.0	475	630	940	1065
				3	380	9.0	500	6.0	760	8.0	860	12.0	530	700	1065	1205
			3/8	2	370	9.5	500	6.0	750	8.0	840	12.0	520	700	1050	1175
				3	420	8.0	560	5.0	840	7.0	950	10.0	590	785	1175	1330
	8d	1-3/8	3/8	2	480	11.0	640	7.5	960	9.5	1090	13.0	670	895	1345	1525
				3	540	9.5	720	6.0	1080	8.5	1220	12.0	755	1010	1510	1710
			7/16	2	510	10.0	680	7.0	1010	9.5	1150	13.0	715	950	1415	1610
				3	570	9.0	760	6.0	1140	8.0	1290	12.0	800	1065	1595	1805
			15/32	2	540	9.5	720	6.5	1060	8.5	1200	13.0	755	1010	1485	1680
				3	600	8.5	800	5.5	1200	7.5	1350	11.0	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	15.0	770	11.0	1150	14.0	1310	18.0	810	1080	1610	1835
				3	650	14.0	860	9.5	1300	12.0	1470	16.0	910	1205	1820	2060
			19/32	2	640	14.0	850	9.5	1280	12.0	1460	17.0	895	1190	1790	2045
				3	720	12.0	960	8.0	1440	11.0	1640	15.0	1010	1345	2015	2295

1. Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms.
2. For framing grades other than Douglas Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = [1 – (0.5 – G)], where G = Specific Gravity of the framing lumber from the *NDS*. The Specific Gravity Adjustment Factor shall not be greater than 1.
3. Apparent shear stiffness values, G_a, are based on nail slip in framing and panel stiffness values for diaphragms constructed with 3-ply plywood with moisture content less than or equal to 19% at time of fabrication. When 4-ply, 5-ply, or COM-PLY are used, G_a values shall be permitted to be increased by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.