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19011 Wood-Sno Road NE, Suite 100
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Phone: (425) 814-8448
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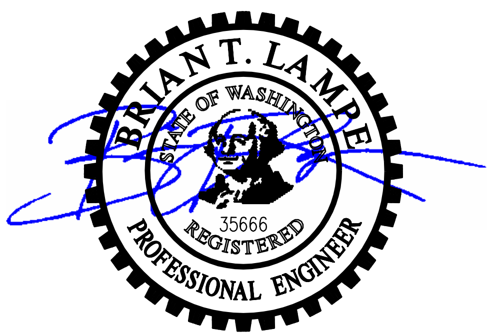
Structural Calculations

For

Walsh Addition

3817 80th Ave SE
Mercer Island, WA

February 19, 2024



Prepared by

Brian Lampe
Kelsi Bonner

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STRUCTURAL CALCULATIONS SHEET INDEX
Walsh Addition
3817 80th Ave SE, Mercer Island, WA

Item **Page #**

Criteria

Gravity

Lateral

Miscellaneous

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Criteria

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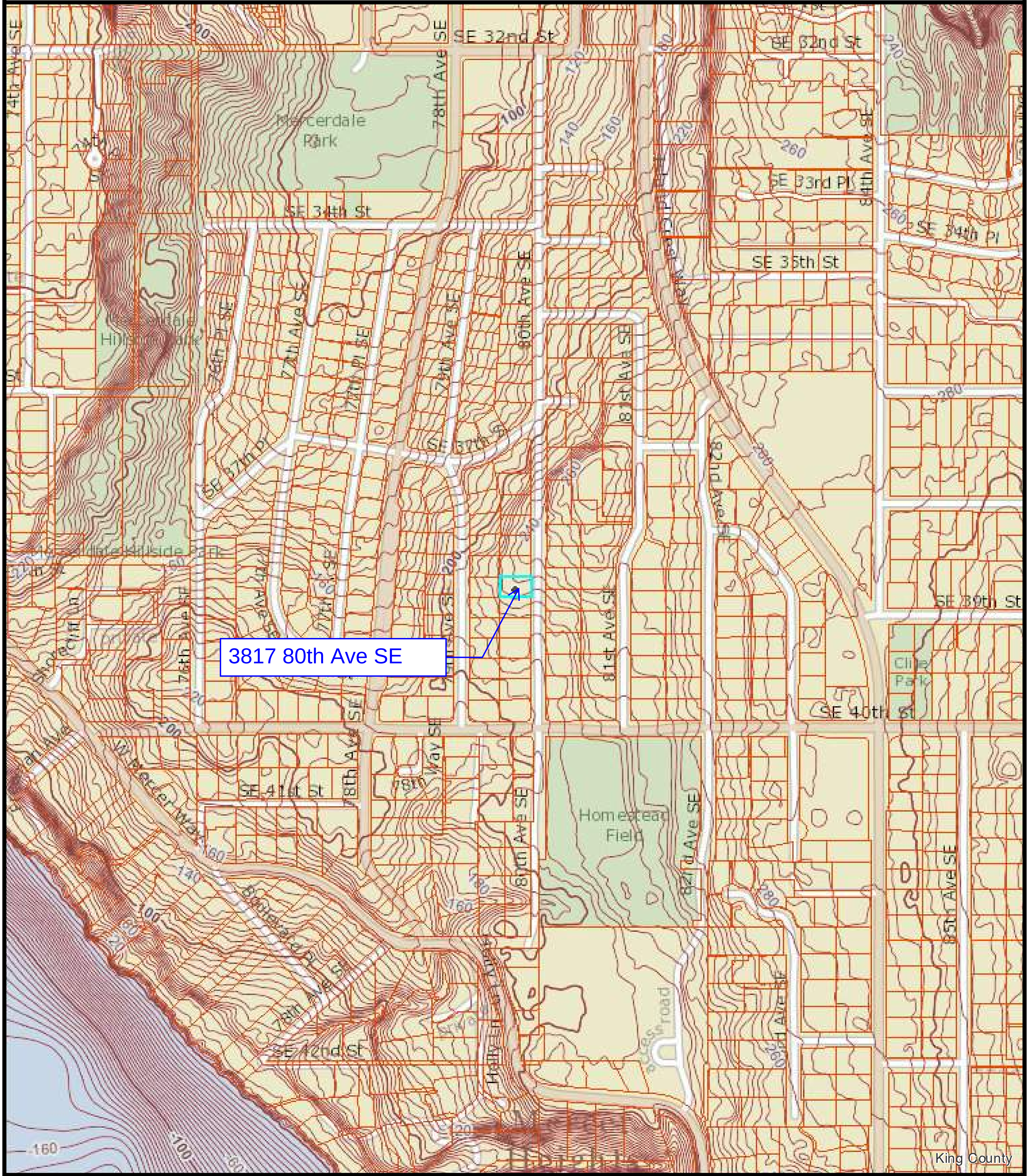
Project: Walsh Addition
Project Number: 3817 80th Ave SE, Mercer Island, WA

Code:	IBC 2018			
	Risk Category	II		
Earthquake:	Site Class	D		
	I_e =	1.00	R =	6.5
	S_S =	1.413	Ω_0 =	3.0
	S_1 =	0.492	C_d =	4.0
	ρ =	1.00		
Wind:	Basic Design Wind Speed, *V*	100 MPH		
	Exposure	B		
	Topographic Factor	K_{ZT} = 1.60		
Soil Bearing:	1500-psf Allowable Soil Bearing Pressure			
Concrete:	2500-psi Concrete Strength			
	Higher strength may be used, but special inspection and testing reports not req'd			
Nails:	Sheathing	8d common (2½" x 0.131")		
	Framing	12d box (3¼" x 0.131")		

Roof Framing:			
Snow Load	Ground Snow, Pg		25 psf
		Exposure factor, Ce	1.0
		Thermal Factor, Ct	1.2
	Flat Roof Snow, Pf (0.7 Ce Ct I Pg)		21 psf
	Use Snow Load		**25 psf**
	Attic (where accessible)		10 psf
Dead Load	Roofing - Composition Shingles		3.0 psf
	Sheathing - 7/16 OSB		2.2 psf
	(E) Framing - Trusses		2.5 psf
	Insulation - Batt.		1.0 psf
	Ceiling - 5/8 GWB		2.8 psf
	Misc.		0.5 psf
		Total	**12.0 psf**
Deflection	L/360 Live Load, L/240 Total Load		
Floor Framing:			
Live Load	Residential		40 psf
	Decks		60 psf
Dead Load	Finish Floor - Carpet/Vinyl		4.0 psf
	Sheathing - 3/4 Plywood/Edge Gold		2.5 psf
	(E) Framing - 2x10 @ 16"oc		2.9 psf
	Ceiling - 5/8 GWB		2.8 psf
	Misc.		2.8 psf
		Total	**15.0 psf**
Deflection	L/480 Live Load, L/240 Total Load		
Wall Framing:			
Dead Load	Exterior 2x Stud Walls		10 psf
	Interior 2x Stud Walls		8 psf

King County iMap



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Date: 2/5/2024

Notes:

N

King County

C1.2

⚠ This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

ℹ The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Address:	2003 82nd Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.592596, -122.22992870000001
Elevation:	119 ft
Timestamp:	2024-02-05T22:11:18.247Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.379	MCE_R ground motion (period=0.2s)
S_1	0.481	MCE_R ground motion (period=1.0s)
S_{MS}	1.655	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.104	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

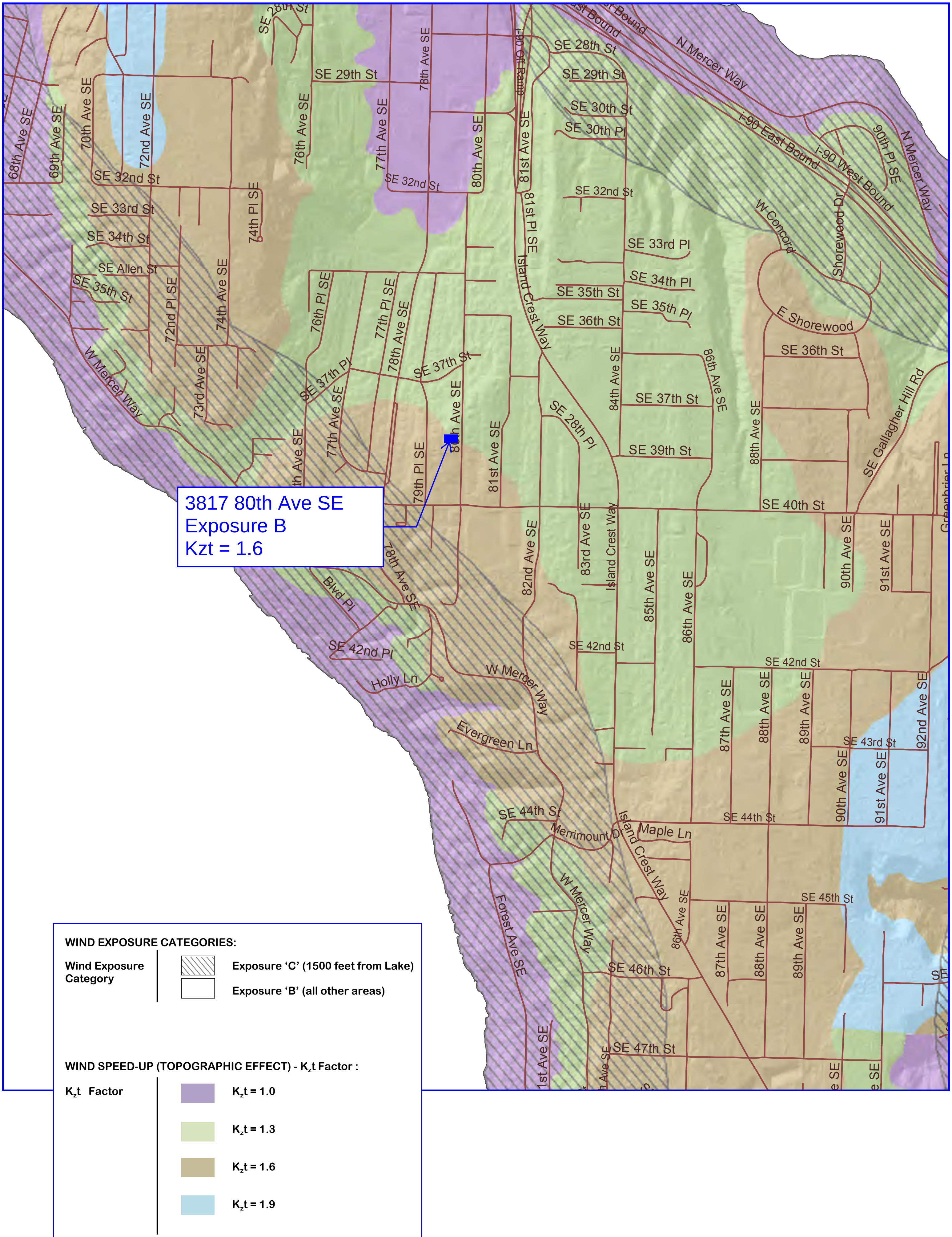
* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.903	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.59	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.708	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.379	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.528	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.092	Factored deterministic acceleration value (0.2s)
S1RT	0.481	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.536	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.278	Factored deterministic acceleration value (1.0s)
PGAd	1.076	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.


3817 80th Ave SE
Exposure B
Kzt = 1.6
SE 28th St
SE 29th St
SE 30th St
SE 30th Pl
SE 32nd St
SE 33rd St
SE 34th St
SE Allen St
SE 35th St
SE 33rd Pl
SE 34th Pl
SE 35th Pl
SE 36th St
SE 37th St
SE 37th Pl
SE 39th St
SE 40th St
SE 42nd St
SE 42nd Pl
SE 43rd St
SE 44th St
SE 45th St
SE 46th St
SE 47th St
68th Ave SE
69th Ave SE
70th Ave SE
72nd Ave SE
72nd Pl SE
73rd Ave SE
74th Ave SE
74th Pl SE
76th Ave SE
76th Pl SE
77th Ave SE
77th Pl SE
78th Ave SE
79th Pl SE
80th Ave SE
81st Ave SE
81st Pl SE
82nd Ave SE
83rd Ave SE
84th Ave SE
85th Ave SE
86th Ave SE
87th Ave SE
88th Ave SE
89th Ave SE
90th Ave SE
90th Pl SE
91st Ave SE
92nd Ave SE
I-90 Off Ramp
I-90 East Bound
I-90 West Bound
N Mercer Way
W Mercer Way
Island Crest Way
W Concord
Shorewood Dr
E Shorewood
SE Gallagher Hill Rd
Greenbrier Ln
SE 28th Pl
Blvd Pl
Holly Ln
Evergreen Ln
Merrimount Dr
Maple Ln
Forest Ave SE
WIND EXPOSURE CATEGORIES:
Wind Exposure Category
Exposure 'C' (1500 feet from Lake)
Exposure 'B' (all other areas)
WIND SPEED-UP (TOPOGRAPHIC EFFECT) - K_zt Factor :
K_zt Factor
K_zt = 1.0
K_zt = 1.3
K_zt = 1.6
K_zt = 1.9

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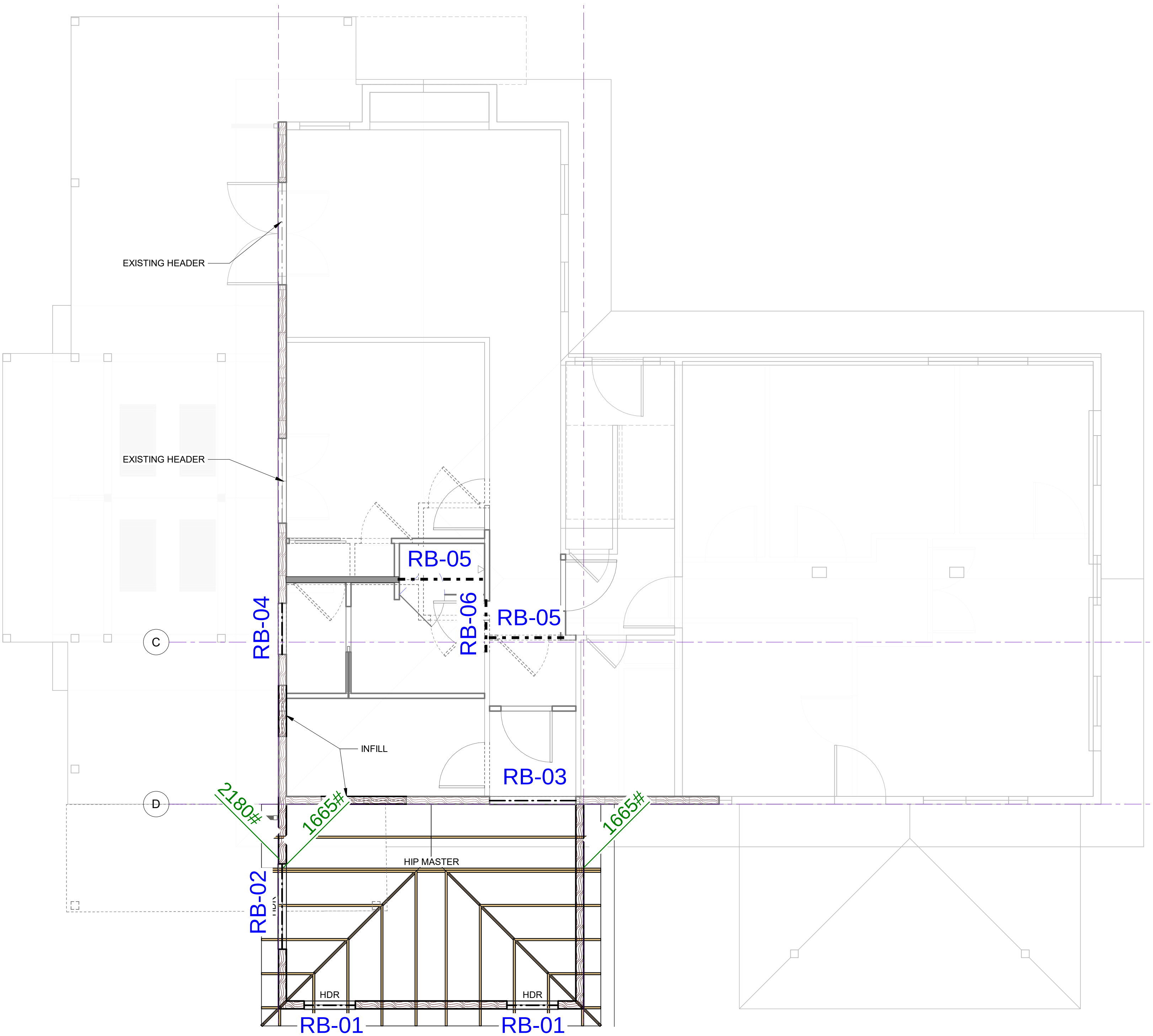
Gravity

Roof Framing



Roof Framing Key Plan

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RB-01

M = 0.29 **k.ft**

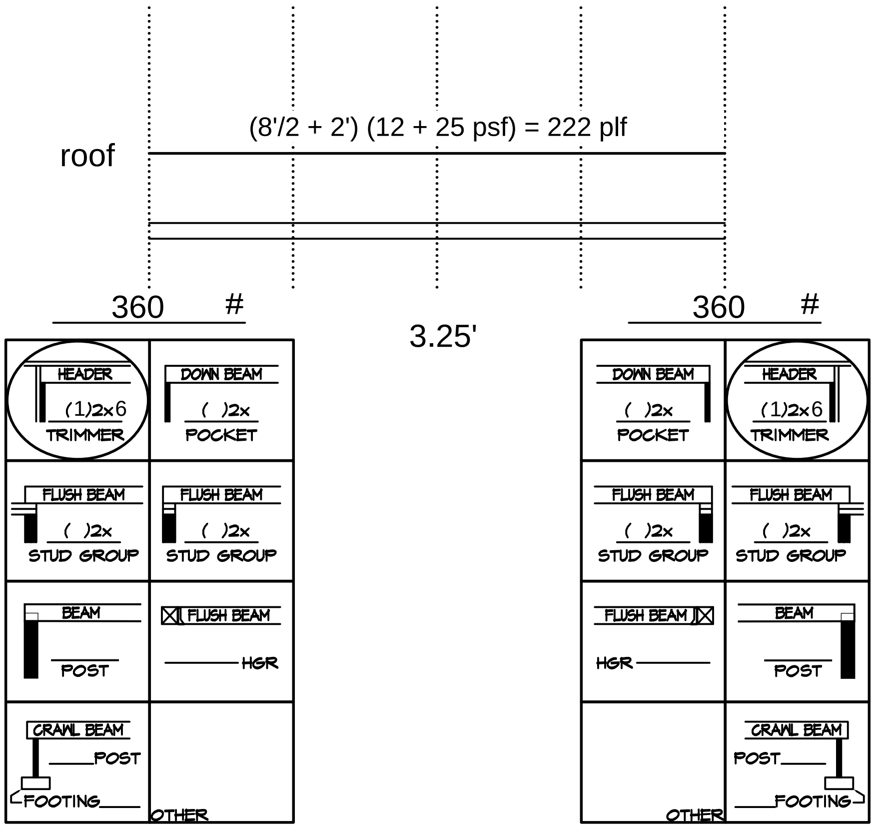
V = 0.36 k

L/ 360 = 0.11 **(LL)**

L/240 = 0.16 **(TL)**

$\mathbf{EI_{req'd}}$ = 3.5 $\mathbf{x10^6 lb.in^2}$

4 x 6 min



RB-02

M = 1.25 **k.ft**

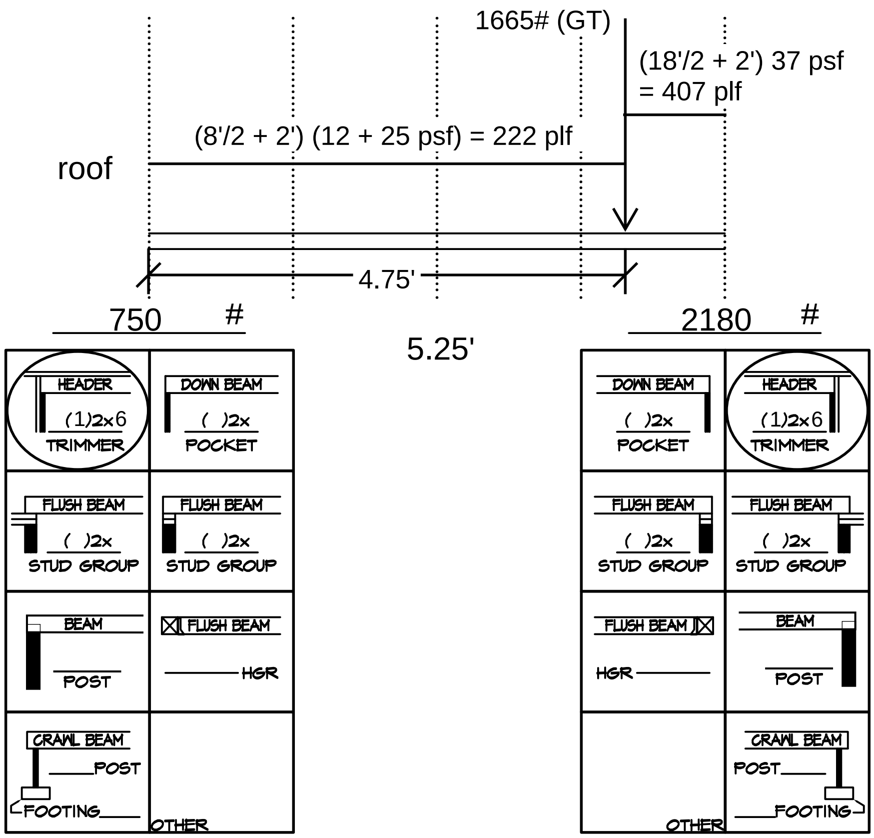
V = 2.18 k

L/ 360 = 0.18 **(LL)**

L/240 = 0.26 **(TL)**

$\mathbf{EI_{req'd}}$ = 24.3 $\mathbf{x10^6 lb.in^2}$

4 x 6 min



Project: Walsh Addition Designed By: KTB Date: 02/19/2024

Project Number: Client: Scale: Page: R2.1

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RB-03

M = 2.12 **k.ft**

V = 1.61 k

roof — (8'/2 + 2') (12 + 25 psf) = 222 plf

wall — (8') 10 psf = 80 plf

floor — (10'/2) (15 + 40 psf) = 275 plf

low roof — (2'/2) (12 + 25 psf) = 37 plf

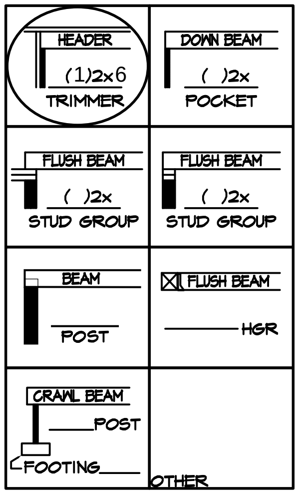
L/ 480 = 0.13 **(LL)**

L/240 = 0.26 **(TL)**

EI$_{req'd}$ = 58.2 **x10^{6}lb.in^2**

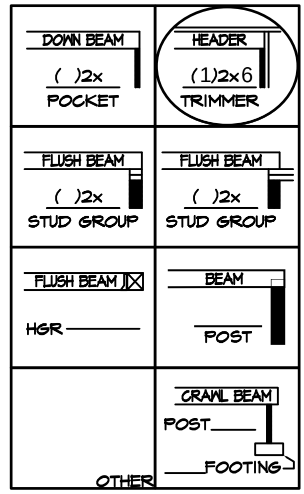
4 x 8 min

1610 #



5.25'

1610 #



RB-04

M = 0.86 **k.ft**

V = 1.06 k

roof — (18'/2 + 2') (12 + 25 psf) = 407 plf

wall — (8') 10 psf = 80 plf

floor — (2'/2) (15 + 40 psf) = 55 plf

deck — (3'/2) (15 + 60 psf) = 113 plf

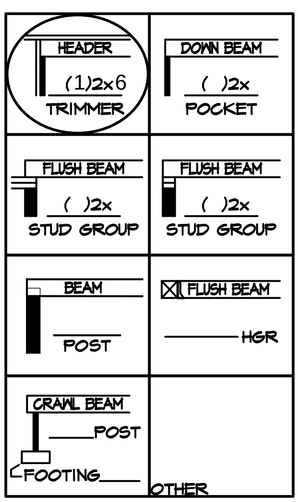
L/ 360 = 0.11 **(LL)**

L/240 = 0.16 **(TL)**

EI$_{req'd}$ = 10.3 **x10^{6}lb.in^2**

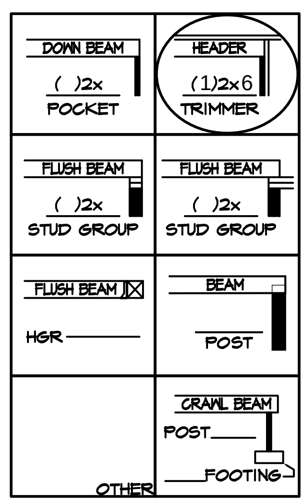
4 x 6 min

1060 #



3.25'

1060 #



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RB-05

M = 1.88 k.ft

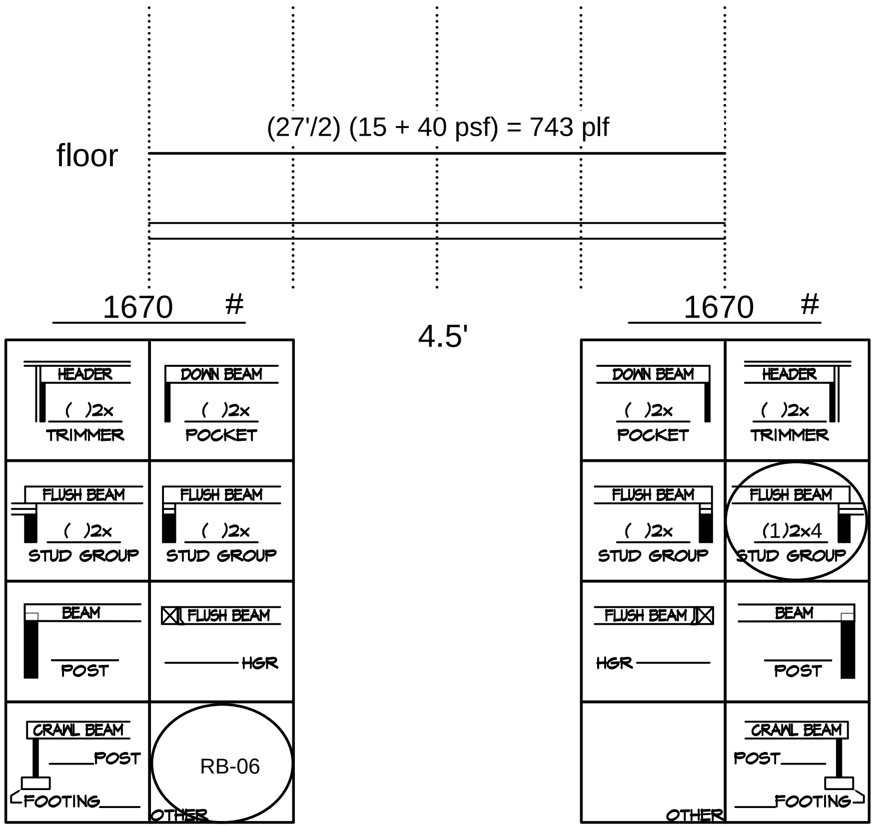
V = 1.67 k

L/480 = 0.11 (LL)

L/240 = 0.23 (TL)

$EI_{req'd}$ = 44.3 $\times 10^6$ lb.in^2

4 x 10



RB-06

M = 1.05 k.ft

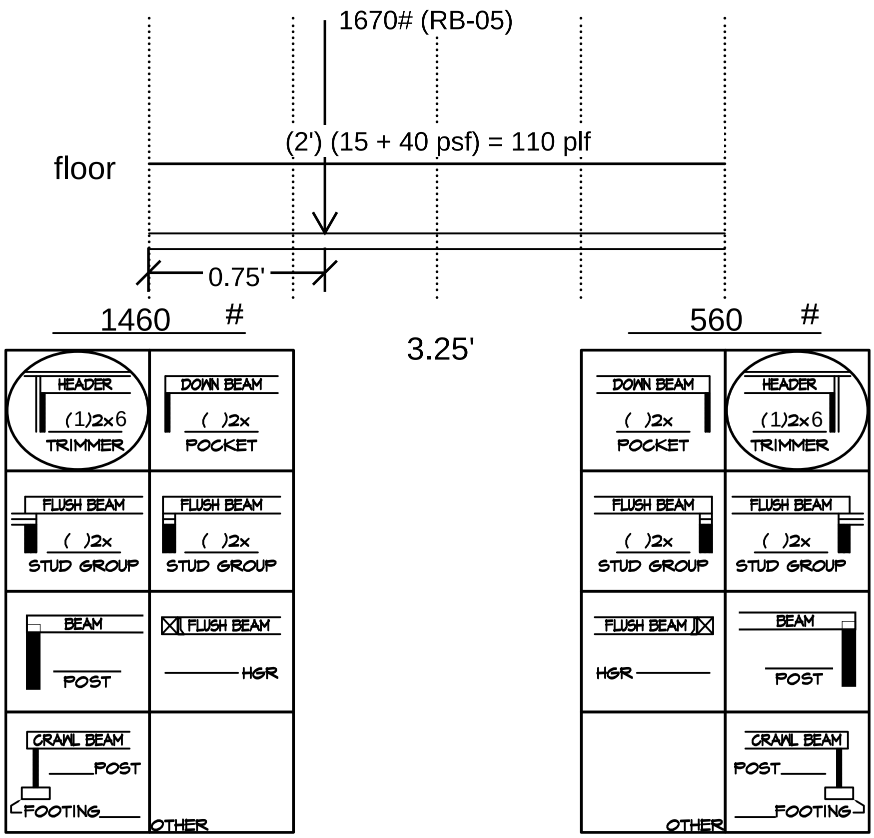
V = 1.46 k

L/480 = 0.08 (LL)

L/240 = 0.16 (TL)

$EI_{req'd}$ = 14.5 $\times 10^6$ lb.in^2

4 x 6 min



Project: Walsh Addition Designed By: KTB Date: 02/19/2024

Project Number: Client: Scale: Page: R2.3

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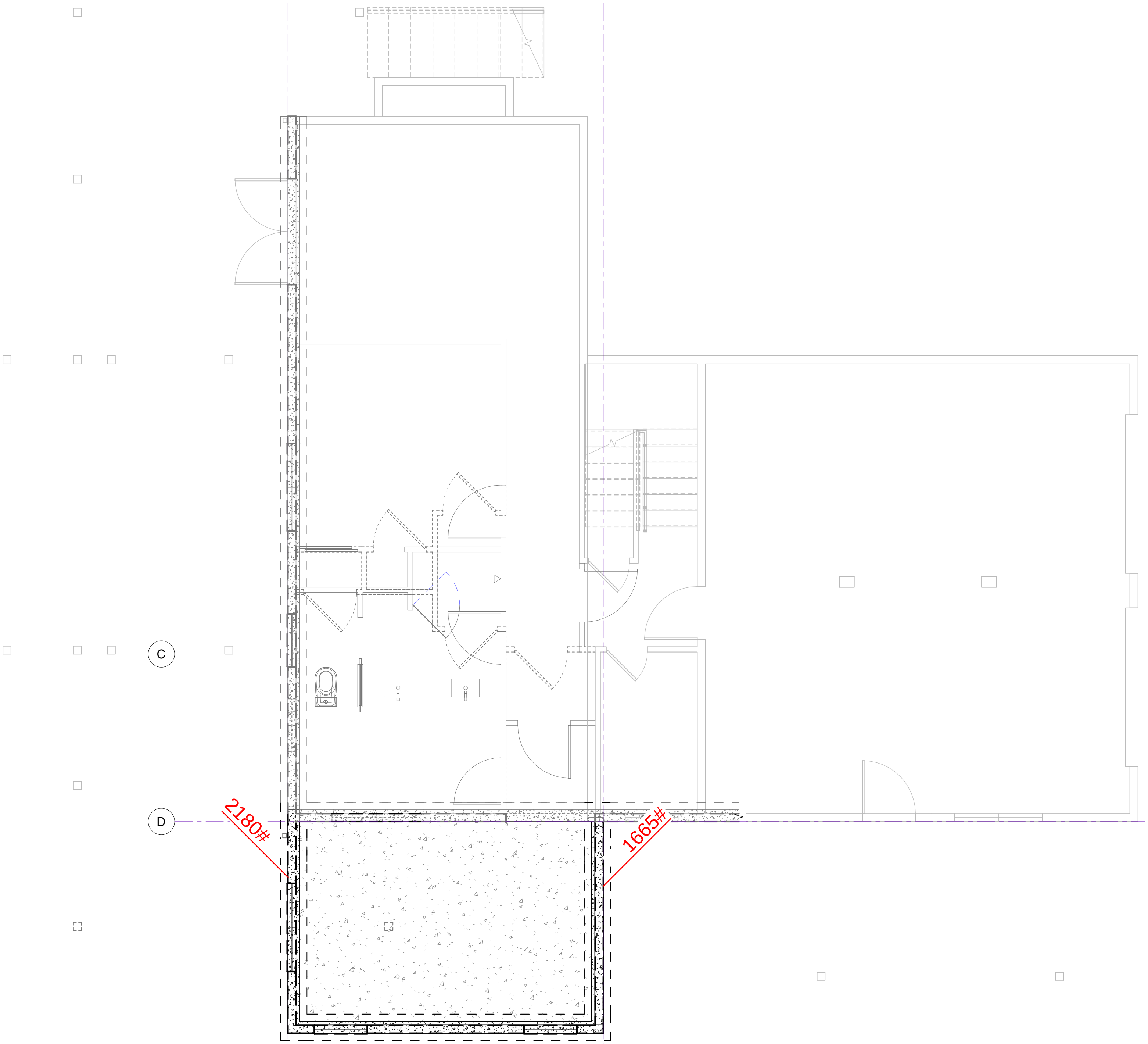
Gravity

Main Floor Framing



Main Floor Framing Key Plan

MF1.1

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Lateral

Forces

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Walsh Addition — Revision Date: 2/9/2024

3817 80th Ave SE, Mercer Island, WA

Criteria

Code: 2018 IBC

Allowable Stress Design (ASD)

Seismic Design: **ASCE 7-16: 12.8 Equivalent Lateral Force Procedure**

Wind Design: **ASCE 7-16: Ch. 28 Envelope Procedure, Low Rise**

Risk Category:	II - Other Structures		*Table 1.5-1*
Snow Importance Factor	I_S =	1.00	*Table 1.5-2*
Ice Importance Factor - Thickness	I_i =	1.00	*Table 1.5-2*
Ice Importance Factor - Wind	I_w =	1.00	*Table 1.5-2*
Seismic Importance Factor	I_e =	1.00	*Table 1.5-2*
Spectral Response, Short Period	S_S =	1.413	(Mapped)
Spectral Response, 1-s Period	S_1 =	0.492	(Mapped)

Site Class assumed, no Geotechnical Report

Site Class:	D		*Table 20.3-1*
Site Coefficient	F_a =	1.20	*Table 11.4-1*
Site Coefficient	F_v =	1.81	*Table 11.4-2*

Structural Systems:

Light framed walls with shear panels

All other structural systems	T_L =	6	*(Figs. 22-14 thru 22-17)*
Response Modification Coefficient	R =	6.5	*Table 12.2-1*
Overstrength Factor	Ω_0 =	3	*Table 12.2-1*
Deflection Amplification Factor	C_d =	4	*Table 12.2-1*

Basic Wind Speed:	100 mph		
Exposure to Wind:	Exposure B		*Section 26.7.3*
Topographical Factor	K_{ZT} =	1.60	

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Walsh Addition
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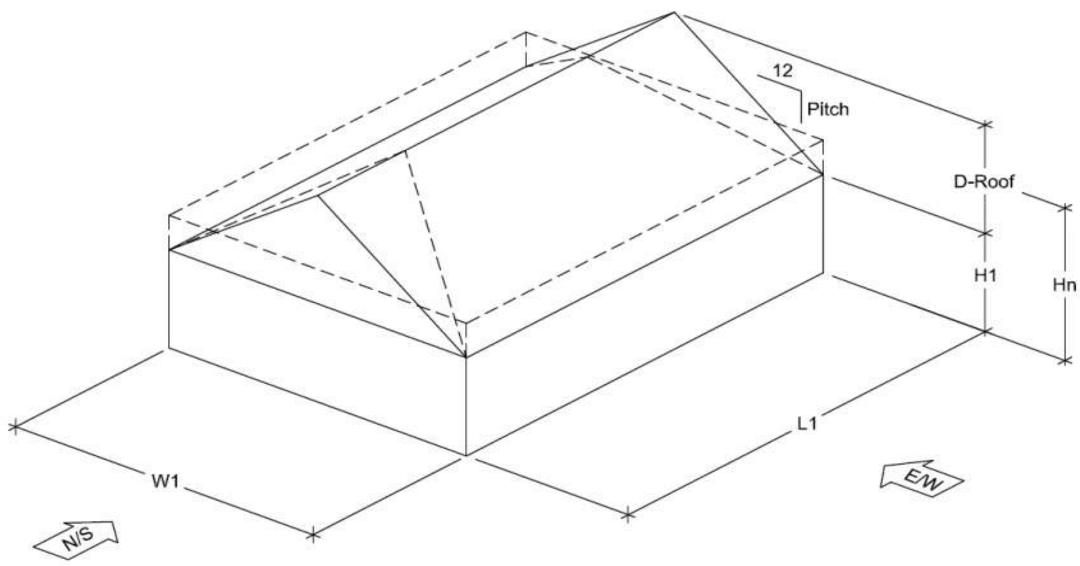
Revision Date: 2/9/2024



Roof		
Geometry		
Mean Roof Height	Hn =	10.75 ft
Roof Depth	D-Roof =	3.5 ft
Overhang Length		18 in
Pitch		4:12
Floor 1		
Geometry		
Width	W3 =	18 ft
Length	L3 =	12 ft
Plate Height	H3 =	9 ft
Floor Depth	D3 =	0 in

Seismic Weight - Roof				
Roof Area 1	260 SF	12 psf		3,120#
Roof Area 2				
Roof Area 3				
Exterior Wall 1	60 LF	4.5 ft	10 psf	2,700#
Exterior Wall 2				
Exterior Wall 3				
Interior Wall	60 LF	4.5 ft	8 psf	2,160#
			Total	7,980#

N/S Projected Area - Roof	
Sloped Roof Area	35 SF
Gable/Parapet Area	0 SF
Wall Area	81 SF
E/W Projected Area - Roof	
Sloped Roof Area	30 SF
Gable/Parapet Area	0 SF
Wall Area	54 SF

Date: 2/9/2024

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Redundancy, ρ 1.0 *(Section 12.3.4)*

Design Base Shear

$S_{MS} = F_a S_S$ *(Eq. 11.4-1)*
$= 1.696$

$S_{M1} = F_v S_1$ *(Eq. 11.4-2)*
$= 0.890$

$S_{DS} = ⅔ S_{MS}$ *(Eq. 11.4-3)*
$= 1.130$

$S_{D1} = ⅔ S_{M1}$ *(Eq. 11.4-4)*
$= 0.593$

Seismic Design Category:

Short Period -- D
1-Second Period -- D

Structure Period and Weight:

$C_t = 0.020$ *Table 12.8-2*
$x = 0.75$
Building Height (Mean Roof), $h_n = 11$ ft
Approximate Fundamental Period, $T_a = C_t (h_n)^x$ *(Eq. 12.8-7)*
$T = T_a = 0.12$
$T_L = 6$ *(Figs. 22-14 thru 22-17)*

Calculated design base shear:

$V = C_s W$ (Eq. 12.8-1)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$ (Eq. 12.8-2)

$C_s = 0.174$

The total design base shear need not exceed:

(Eq. 12.8-3)

for $T \leq T_L$ $$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

$C_s = 0.768$

(Eq. 12.8-4)

for $T > T_L$ $$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)}$$

$C_s = 38.827$

$C_s = 0.768$ *T ≤ TL*

$C_s = 1.153$ *1.5 times Cs in accordance with 11.4.8*

The total design base shear shall not be less than:

$C_s = 0.044 S_{DS} I_e \geq 0.01$ (Eq. 12.8-5)

$C_s = 0.050$

nor where $S_1 \geq 0.6g$:

$C_s = 0.5 S_1/(R/I_e)$ (Eq. 12.8-6)

$C_s = 0.000$

$C_s = 0.174$

$V = 0.174\ W$

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Revision Date: 2/9/2024

$\rho_S = \lambda K_{ZT} \rho_{S30}$	(28.5-1)	Exposure =	B
λ = 1.00	(Fig. 28.5.1)	Mean Roof Ht hn (ft) =	11 ft
K_{ZT} = 1.60	(Section 26.8)	a (roof) =	3.0 ft
		Basic Wind Speed =	100 mph
		Roof Angle =	19

North/South Loading

28.5.4 Minimum Design Loads

	Zone	Area	p_{S30} (psf)	$p_{S30\ design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Roof	A_{wall}	27	21.6	21.6	34.5	932	559	432	259
	Agable	0	21.6	21.6	34.5	0	0	0	0
	B	21	-6.0	0.0	0.0	0	0	168	101
	Cwall	54	14.3	14.3	22.9	1239	743	864	518
	Cgable	0	14.3	14.3	22.9	0	0	0	0
	D	14	-3.3	0.0	0.0	0	0	112	67
	Total Area =	116			Total Load =	2171	1303	1576	946
					Design:	***2171***	***1303***		

East/West Loading

28.5.4 Minimum Design Loads

	Zone	Area	p_{S30} (psf)	$p_{S30\ design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Roof	Awall	27	21.6	21.6	34.5	932	559	432	259
	Agable	0	21.6	21.6	34.5	0	0	0	0
	B	21	-6.0	0.0	0.0	0	0	168	101
	Cwall	27	14.3	14.3	22.9	619	372	432	259
	Cgable	0	14.3	14.3	22.9	0	0	0	0
	D	9	-3.3	0.0	0.0	0	0	72	43
	Total Area =	84			Total Load =	1552	931	1104	662
					Design :	***1552***	***931***		

Date: 2/9/2024

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Vertical Distribution of Lateral Forces

Base Shear:

$V =$ 1.39 kips

Shear Walls:

$F_x = C_{vx} V$ *(Eq. 12.8-11)*

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$ *(Eq. 12.8-12)*

Diaphragms:

$$F_{px} = \left(\sum_{i=x}^{n} F_i \Big/ \sum_{i=x}^{n} w_i\right)(w_{px}) \ldots [Eq.\,12.10-1]$$

$$F_{px} = 0.2 S_{DS} I_e w_{px} \ldots [Eq.\,12.10-2] \text{ (min)}$$

$$F_{px} = 0.4 S_{DS} I_e w_{px} \ldots [Eq.\,12.10-3] \text{ (max)}$$

	Strength Design Seismic Forces (E)							
Floor Level (from base)	Height, h_x (ft)	Story Weight, w_x (Kips)	$w_x h_x$ (ft-Kips)	Lateral Force, F_x (Kips)	Story Shear, $\sum F_x$ (Kips)	Story Moment (ft-Kips)	Portion of Weight at *i*, $\sum w_i$ (Kips)	Diaphragm Force, Fpx (Kips)
Roof	10.8	7.98	86	1.39	1.39	15	8	1.80

Totals W = 7.98 Kips

$\sum w_x h_x =$ 86 ft-Kips

	Strength Design Wind Forces (W)			
Floor Level (from base)	Lateral Force N/S, H_x (Kips)	Story Shear N/S, $\sum$Hx (Kips)	Lateral Force E/W, H_x (Kips)	Story Shear E/W, $\sum H_x$ (Kips)
Roof	2.17	2.17	1.55	1.55

	Diaphragm (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Roof	1.26	1.30	0.93

	Shear Walls (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Floor 1	0.97	1.30	0.93

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Lateral

Shear Walls/Diaphragms

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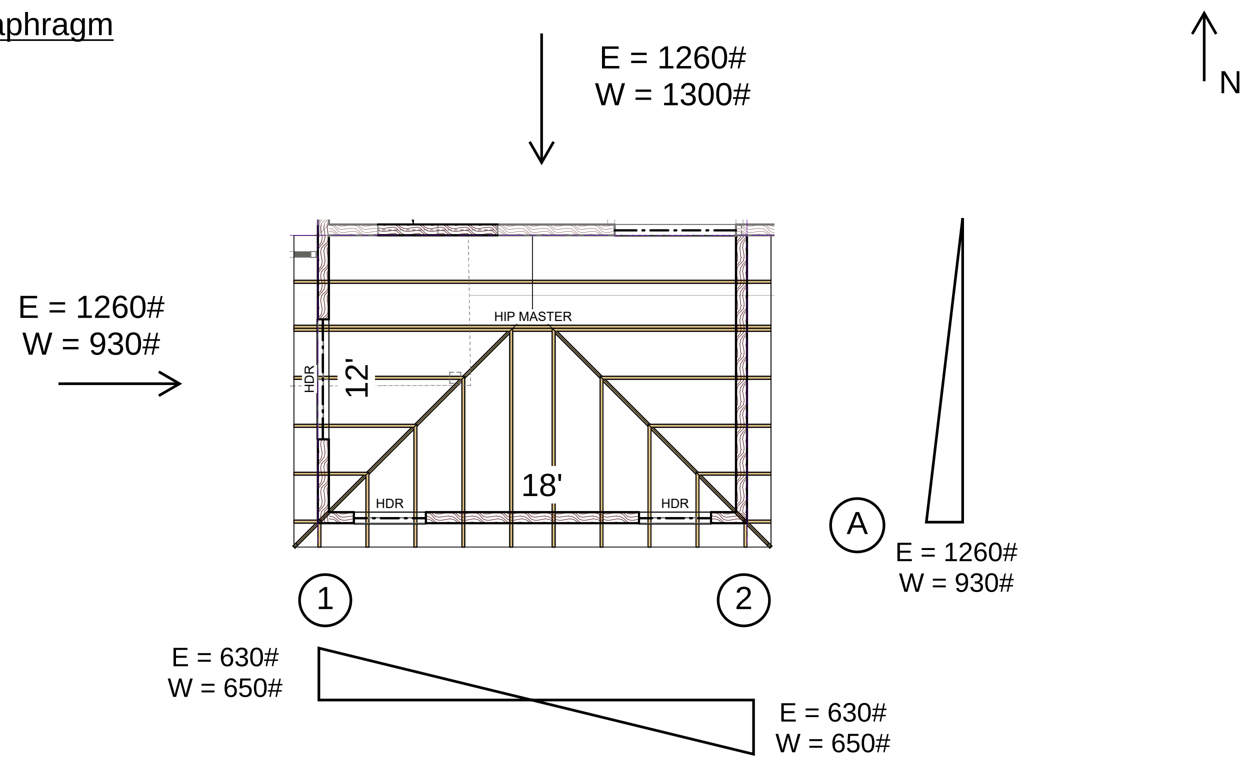
Fax: 425-821-2120

Roof Diaphragm



1 2 (E) 630# / 12' = 53 plf
(W) 650# / 12' = 54 plf

A (E) 1260# / 18' = 70 plf
(W) 930# / 18' = 52 plf

unblocked

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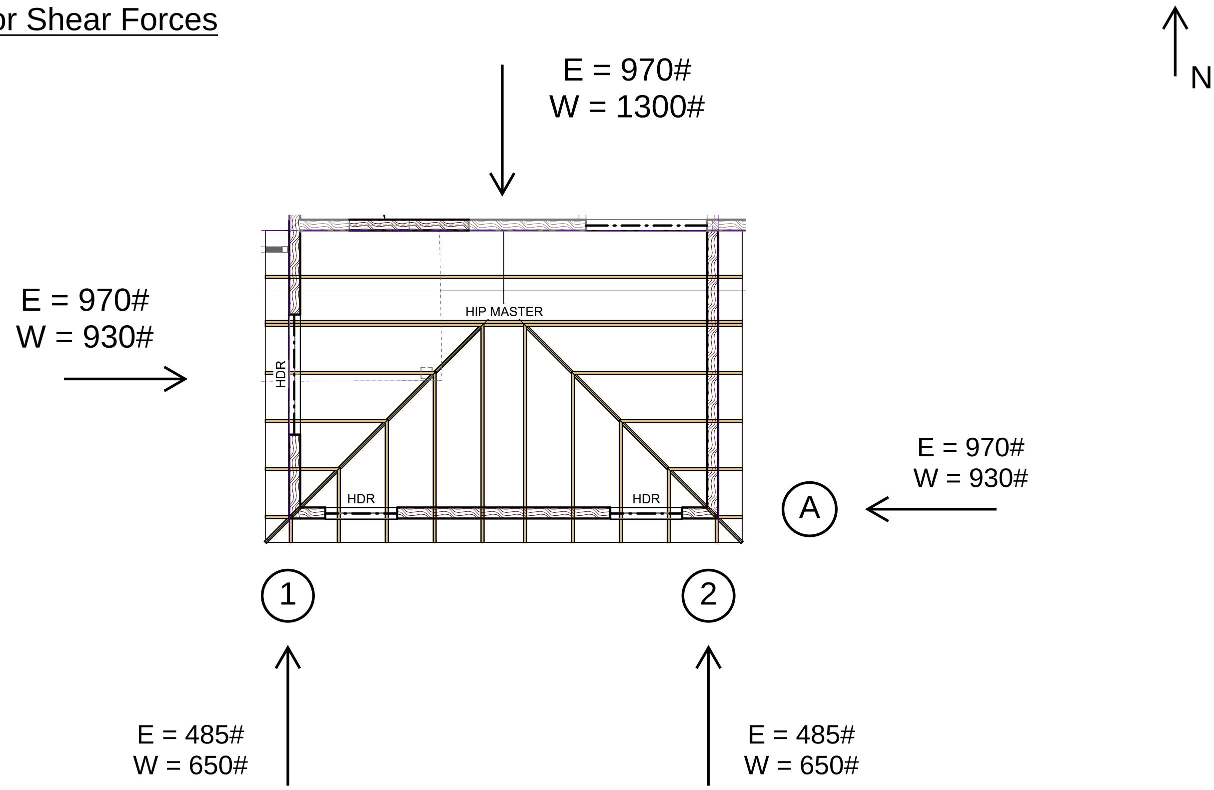
Main Floor Shear Forces



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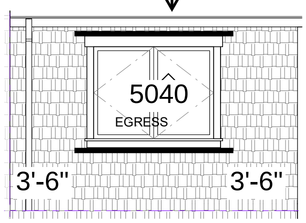
Shear Wall Line 1

E = 485#
W = 650#

Main Floor: 8' PLT

0.6D = 1000#



12'

(E) -175#
(W) -65#

no uplift

(E) -175#
(W) -65#

no uplift

(E) 485# / 7' = 69-plf
(W) 650# / 7' = 93-plf

h/b < 2.0

P1-6

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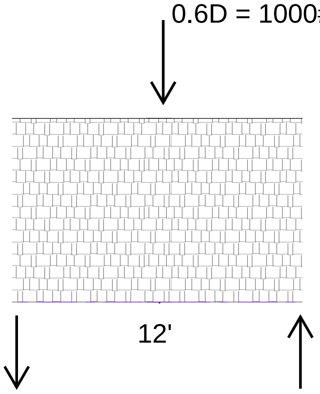
Shear Wall Line 2

E = 485#
W = 650#

Main Floor: 8' PLT



(E) 485# / 12' = 40-plf
(W) 650# / 12' = 54-plf

h/b < 2.0

P1-6

Project: Walsh Addition | Designed By: KTB | Date: 02/19/2024

Project Number: | Client: | Scale: | Page: L2.4

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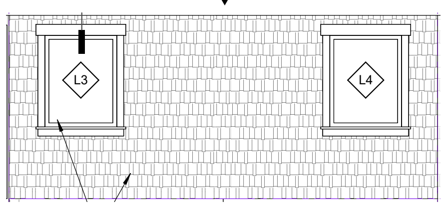
Fax: 425-821-2120

Shear Wall Line (A)

E = 970#
W = 930#

Main Floor: 8' PLT

0.6D = 670#



8'

(E) 635#
(W) 595#
HDU2

(E) 635#
(W) 595#
HDU2

(E) 970# / 8' = 121-plf
(W) 930# / 8' = 116-plf

h/b < 2.0

P1-6

Project: Walsh Addition
Designed By: KTB
Date: 02/19/2024

Project Number:
Client:
Scale:
Page: L2.5

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Lateral

Shear Wall/Diaphragm Capacities

2018 IBC/SDPWS 2015 – Diaphragms (8d Nailing)

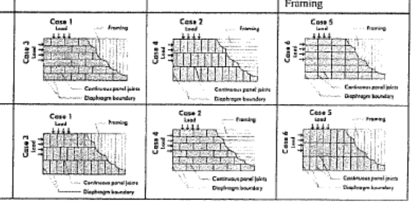
Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_s (plf)	Case 1 – G_a (kips/in.) OSB	Case 1 – G_a (kips/in.) PLY	Cases 2,3,4,5,6 – v_s (plf)	Cases 2,3,4,5,6 – G_a (kips/in.) OSB	Cases 2,3,4,5,6 – G_a (kips/in.) PLY	B WIND – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges – Case 1 – v_w (plf)	Cases 2,3,4,5,6 – v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

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. See Appendix A for common nail dimensions.
2. For species and grades of framing 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* (Table 12.3.3A). 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 with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied 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.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	Cases 2&4: Continuous Panel Joints Parallel to Framing	Cases 5&6: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			



(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

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

Blocked Wood Structural Panel Diaphragms[1,2,3,4,5]

A SEISMIC – 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 other panel edges (Cases 1, 2, 3, & 4)

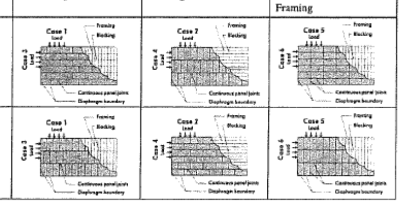
Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	6 / 6: v_s (plf)	6 / 6: G_a OSB	6 / 6: G_a PLY	4 / 6: v_s (plf)	4 / 6: G_a OSB	4 / 6: G_a PLY	2-1/2 / 4: v_s (plf)	2-1/2 / 4: G_a OSB	2-1/2 / 4: G_a PLY	2 / 3: v_s (plf)	2 / 3: G_a OSB	2 / 3: G_a PLY
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18
Sheathing and Single-Floor	6d	1-1/4	5/16	2	340	15	10	450	9.0	7.0	670	13	9.5	760	21	13
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15

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 other panel edges (Cases 1, 2, 3, & 4)

Sheathing Grade	Common Nail Size	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width (in.)	6 / 6: v_w (plf)	4 / 6: v_w (plf)	2-1/2 / 4: v_w (plf)	2 / 3: v_w (plf)
Structural I	6d	5/16	2	520	700	1050	1175
			3	590	785	1175	1330
	8d	3/8	2	755	1010	1485	1680
			3	840	1120	1680	1890
	10d	15/32	2	895	1190	1790	2045
			3	1010	1345	2015	2295
Sheathing and Single-Floor	6d	5/16	2	475	630	940	1065
			3	530	700	1065	1205
		3/8	2	520	700	1050	1175
			3	590	785	1175	1330
	8d	3/8	2	670	895	1345	1525
			3	755	1010	1510	1710
		7/16	2	715	950	1415	1610
			3	800	1065	1595	1805
		15/32	2	755	1010	1485	1680
			3	840	1120	1680	1890
	10d	15/32	2	810	1080	1610	1835
			3	910	1205	1820	2060
		19/32	2	895	1190	1790	2045
			3	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. See Appendix A for common nail dimensions.
2. For species and grades of framing 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* (Table 12.3.3A). 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 with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied 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.
5. Diaphragm resistance depends on the direction of continuous panel joints with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

	Cases 1&3: Continuous Panel Joints Perpendicular to Framing	Cases 2&4: Continuous Panel Joints Parallel to Framing	Cases 5&6: Continuous Panel Joints Perpendicular and Parallel to Framing
Long Panel Direction Perpendicular to Supports			
Long Panel Direction Parallel to Supports[a]			



(a) Panel span rating for out-of-plane loads may be lower than the span rating with the long panel direction perpendicular to supports (See Section 3.2.2 and Section 3.2.3)

1. Reduction Factor = 2
2. G = 0.42 (SPF or Hem Fir)… Adjustment Factor = [1-(0.5-0.42)] = 0.92 or 0.5 (I-Joists or Douglas Fir)… Adjustment Factor = 1.0

Diaphragm	Sheathing Thickness	Nail Spacing Edge/Intermediate	Blocked	Framing	Seismic Capacity (Case 1/Other)	Wind Capacity (Case 1/Other)
Roof – Unblocked	7/16”	6”/12” oc	N	2x (SPF/HF)	212-plf/156-plf	297-plf/219-plf
Roof – Blocked	7/16”	4”/12” oc	Y	2x (SPF/HF)	313-plf	437-plf
Floor – Unblocked	3/4“	6”/12” oc	N	2x (DF) or 3x (HF)	240-plf/180-plf	335-plf/252-plf
Floor – Blocked	3/4“	4”/12” oc,	Y	2x (DF) or 3x (HF)	360-plf	505-plf

2018 IBC/SDPWS 2015 – Shear Wall Schedule

7/16"OSB; 0.131"ϕ Nails; SPF or HF Studs @ 16"oc

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC – Panel Edge Fastener Spacing (in.) 6: v_s (plf)	6: G_a (kips/in.) OSB	6: G_a (kips/in.) PLY	4: v_s (plf)	4: G_a OSB	4: G_a PLY	3: v_s (plf)	3: G_a OSB	3: G_a PLY	2: v_s (plf)	2: G_a OSB	2: G_a PLY	B WIND – Panel Edge Fastener Spacing (in.) 6: v_w (plf)	4: v_w (plf)	3: v_w (plf)	2: v_w (plf)
			Nail (common or galvanized box)		OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY				
Wood Structural Panels - Structural I[4,5]	5/16	1-1/4	6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8[2]	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16[2]			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435
Wood Structural Panels – Sheathing[4,5]	5/16	1-1/4	6d	360	13	9.5	540	18	12	700	24	14	900	37	18	505	755	980	1260
	3/8			400	11	8.5	600	15	11	780	20	13	1020	32	17	560	840	1090	1430
	3/8[2]	1-3/8	8d	440	17	12	640	25	15	820	31	17	1060	45	20	615	895	1150	1485
	7/16[2]			480	15	11	700	22	14	900	28	17	1170	42	21	670	980	1260	1640
	15/32			520	13	10	760	19	13	980	25	15	1280	39	20	730	1065	1370	1790
	15/32	1-1/2	10d	620	22	14	920	30	17	1200	37	19	1540	52	23	870	1290	1680	2155
	19/32			680	19	13	1020	26	16	1330	33	18	1740	48	22	950	1430	1860	2435
Plywood Siding			Nail (galvanized casing)																
	5/16	1-1/4	6d	280	13		420	16		550	17		720	21		390	590	770	1010
	3/8	1-3/8	8d	320	16		480	18		620	20		820	22		450	670	870	1150
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")			Nail (common or galvanized box)																
	3/8		6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

1. Nominal unit shear capacities shall be adjusted in accordance with 4.3.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.2 for particleboard shear walls, and 4.3.7.3 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
2. Shears are permitted to be increased to values shown for 15/32 inch (nominal) sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
3. For species and grades of framing 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 (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
4. Apparent shear stiffness values G_a, are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be multiplied by 1.2.
5. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members as shown below. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
7. Galvanized nails shall be hot-dipped or tumbled.

1. Reduction Factor = 2
2. 16"oc studs – use values for 15/32
3. G = 0.42 (SPF or Hem Fir)… Adjustment Factor = [1-(0.5-0.42)] = 0.92

Wall Type	Blocked	Sheathing (1) or (2) Sides	Nail Spacing Edge/Intermediate	Framing	Sill Plate	Seismic Capacity $h/b_s = 2$	Seismic Capacity $h/b_s = 3.5$	Wind Capacity $h/b_s = 2$	Wind Capacity $h/b_s = 3.5$
P1-6	Y	1	6"/12" oc	2x	2x	240-plf	194-plf	335-plf	272-plf
P1-4	Y	1	4"/12" oc	2x	2x	350-plf	284-plf	490-plf	398-plf
P1-3	Y	1	3"/12" oc	2-2x	2x	450-plf	366-plf	630-plf	512-plf
P1-2	Y	1	2"12" oc	2-2x	2x	590-plf	478-plf	820-plf	669-plf
P2-4	Y	2	4"/12" oc, ea.side	2-2x	3x	700-plf	568-plf	980-plf	796-plf
P2-3	Y	2	3"/12" oc, ea. side	2-2x	3x	900-plf	733-plf	1260-plf	1024-plf
P2-2	Y	2	2"/12" oc, ea. side	2-2x	3x	1180-plf	957-plf	1640-plf	1338-plf

2018 IBC/NDS 2015 – Shear Wall Framing Clips

Model No.	Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads			SPF/HF Allowable Loads		
				Floor (100)	Roof (125)	(160)	Floor (100)	Roof (125)	(160)
SS A34	1	(8) 0.131 x 1½	F_1	395	465	465	340	400	400
			F_2[6]	395	430	430	340	370	370
	1	(8) #9 x 1½ SD	F_1	640	640	640	550	550	550
			F_2	495	495	495	425	425	425
			Uplift	240	240	240	170	170	170
SS A35	2	(9) 0.131 x 1½	A_1	295	350	350	255	300	300
			E	295	360	385	255	310	330
			C_1	185	185	185	160	160	160
	3	(12) 0.131 x 1½	A_2	295	325	325	255	280	280
			C_2	295	330	330	255	285	285
			D	225	225	225	195	195	195
	4	(12) 0.131 x 1½	F_1	590	650	650	510	560	560
			F_2[6]	590	670	670	510	575	575
	5	(12) PH612I	F_1	420	420	420	360	360	360
LTP4	6	(12) 0.131 x 1½	G	580	625	625	500	540	540
			H	580	525	525	500	450	450
LTP5	7	(12) 0.131 x 1½	G	580	565	565	500	485	485
			H	545	490	490	470	420	420

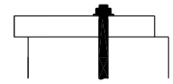
1. Allowable loads are for one angle. When angles are installed on each side of the joist, the minimum joist thickness is 3".
2. Some illustrations show connections that could cause cross-grain tension or bending of the wood during loading if not reinforced sufficiently. In this case, mechanical reinforcement should be considered.
3. LTP4 can be installed over 3/8" wood structural panel sheathing with 0.131" x 1 1/2" nails and achieve 0.72 of the listed load, or over 1/2" sheathing and achieve 0.64 of the listed load. 0.131" x 2 1/2" nails will achieve 100% load.
4. LTP4 satisfies the IRC continuously sheathed portal frame (CS-PF) framing anchor requirements when installed over raised wood floor framing per Figure R602.10.6.4.
5. The LTP5 may be installed over wood structural panel sheathing up to 1/2" thick using 0.131" x 1 1/2" nails with no reduction in load.
6. Connectors are required on both sides to achieve F_2 loads in both directions.
7. Fasteners: Nail dimensions in the table are diameter by length. SD screws are Simpson Strong-Tie® Strong-Drive® screws. PH612I is a pan-head #6 x 1/2" screw available from Simpson Strong-Tie. For additional information, see Fastener Types and Sizes Specified for Simpson Strong-Tie Connectors.

Wall Type	Capacity	A35 Capacity	A35 Spacing	LTP4 Capacity	LTP4 Spacing
P1-6U	144-plf (E)	560#	44" oc	540#	44" oc
P1-6	240-plf (E)	560#	27" oc	540#	27" oc
P1-4	350-plf (E)	560#	18" oc	540#	18" oc
P1-3	450-plf (E)	560#	14" oc	540#	14" oc
P1-2	820-plf (W)	560#	7½" oc	540#	7½" oc
P2-4	700-plf (E)	560#	9" oc	540#	LTP5 18" oc + A35 18" oc
P2-3	900-plf (E)	560#	7" oc	540#	LTP5 14" oc + A35 14" oc
P2-2	1640-plf (W)	560#	2 rows 8" oc	540#	LTP5 8" oc + A35 8" oc

2018 IBC/NDS 2018 – Shear Wall Bolts

Table 12E **BOLTS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]**

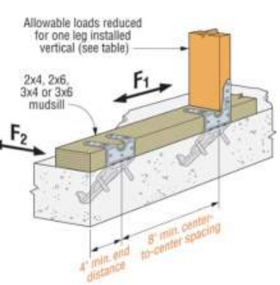
for sawn lumber or SCL to concrete



Thickness: Embedment Depth in Concrete t_m in.	Thickness: Side Member t_s in.	Bolt Diameter D in.	G=0.43 Hem-Fir $Z_{\parallel}$ lbs.	G=0.43 Hem-Fir $Z_{\perp}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\parallel}$ lbs.	G=0.42 Spruce-Pine-Fir $Z_{\perp}$ lbs.	G=0.37 Redwood (open grain) $Z_{\parallel}$ lbs.	G=0.37 Redwood (open grain) $Z_{\perp}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\parallel}$ lbs.	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods $Z_{\perp}$ lbs.	G=0.35 Northern Species $Z_{\parallel}$ lbs.	G=0.35 Northern Species $Z_{\perp}$ lbs.
6.0 and greater	1-1/2	1/2	590	340	590	340	550	310	540	290	530	290
		5/8	860	420	850	410	810	350	800	330	780	320
		3/4	1200	460	1190	450	1130	370	1120	360	1100	350
		7/8	1580	500	1540	490	1360	410	1330	390	1280	370
		1	1800	540	1760	530	1560	440	1520	420	1460	410
	1-3/4	1/2	640	360	630	350	580	320	580	310	560	310
		5/8	910	490	900	480	840	400	830	380	810	370
		3/4	1230	540	1220	530	1160	430	1140	420	1120	410
		7/8	1630	580	1610	570	1540	470	1520	460	1490	430
		1	2090	630	2060	610	1820	510	1770	490	1710	470
	2-1/2	1/2	730	410	730	400	700	360	690	340	680	340
		5/8	1070	540	1060	530	980	480	960	470	940	460
		3/4	1400	710	1380	700	1290	620	1270	600	1240	580
		7/8	1790	830	1770	810	1660	680	1640	660	1600	610
		1	2230	900	2210	880	2080	730	2060	700	2030	680
	3-1/2	1/2	730	470	730	470	700	430	690	410	690	400
		5/8	1140	620	1140	610	1090	550	1080	530	1070	520
		3/4	1650	780	1640	770	1540	680	1510	670	1470	660
		7/8	2100	960	2070	950	1910	870	1880	850	1840	820
		1	2550	1190	2520	1180	2340	1020	2310	980	2260	950

1. Tabulated lateral design values, Z, for bolted connections shall be multiplied by all applicable adjustment factors (see Table 11.3.1).
2. Tabulated lateral design values, Z, are for "full-body diameter" bolts (see Appendix Table L1) with bolt bending yield strength, F_{yb}, of 45,000 psi.
3. Tabulated lateral design values, Z, are based on dowel bearing strength, F_e, of 7,500 psi for concrete with minimum f_c'=2,500 psi.
4. Six inch anchor embedment assumed.

Model No.	Sill Size	Fasteners (in.) Sides	Fasteners (in.) Top	Uncracked Wind and SDC A&B[5,6] Uplift	F_1	F_2	Uncracked SDC C–F[6] Uplift	F_1	F_2	Cracked Wind and SDC A&B[5,6] Uplift	F_1	F_2	Cracked SDC C–F[6] Uplift	F_1	F_2
Standard Installation – Attached to DF/SP Sill Plate															
MASA or MASAP	2x4, x6, x8, x10	(3) 0.148 x 1½	(6) 0.148 x 1½	920	1,475	1,095	745	1,235	1,045	750	1,475	875	660	1,235	765
	3x4, 3x6	(5) 0.148 x 1½	(4) 0.148 x 1½	630	1,165	725	550	1,020	725	475	1,165	725	415	1,020	640
One-Leg-Up Installation – Attached to DF/SP Sill Plate															
MASA or MASAP	2x4, x6, x8, x10	(6) 0.148 x 1½	(3) 0.148 x 1½	755	965	995	660	845	995	570	965	930	500	845	810
	3x4, 3x6	(7) 0.148 x 1½	(2) 0.148 x 1½	—	760	—	—	685	—	—	760	—	—	685	—
Two-Legs-Up Installation – Attached to DF/SP Sill Plate and Rimboard															
MASA or MASAP	2x4, x6, x8, x10	(9) 0.148 x 1½	—	810	1,105	865	740	965	755	620	1,105	630	560	965	550
Double 2x Installation – Attached to DF/SP Sill Plate															
MASA or MASAP	Double 2x4, Double 2x6	(5) 0.148 x 1½	(2) 0.148 x 1½	840	1,030	785	735	900	785	635	1,030	785	555	900	785
Standard Installation – Attached to Hem Fir Sill Plate															
MASA or MASAP	2x4, x6, x8, x10	(3) 0.148 x 1½	(6) 0.148 x 1½	790	1,250	940	640	1,060	900	650	1,250	755	570	1,060	660
	3x4, 3x6	(5) 0.148 x 1½	(4) 0.148 x 1½	535	1,005	625	475	875	625	410	1,005	625	355	875	550
One-Leg-Up Installation – Attached to Hem Fir Sill Plate and HF/SPF Stud															
MASA or MASAP	2x4, x6, x8, x10	(6) 0.148 x 1½	(3) 0.148 x 1½	650	830	855	565	725	855	490	830	795	430	725	695
	3x4, 3x6	(7) 0.148 x 1½	(2) 0.148 x 1½	—	670	—	—	590	—	—	670	—	—	590	—
Two-Legs-Up Installation – Hem Fir Sill Plate and HF/SPF Rimboard															
MASA or MASAP	2x4, x6, x8, x10	(9) 0.148 x 1½	—	700	950	745	635	830	650	545	950	540	480	830	475
Double 2x Installation – Attached to Hem Fir Sill Plate															
MASA or MASAP	Double 2x4, Double 2x6	(5) 0.148 x 1½	(2) 0.148 x 1½	720	890	675	630	775	675	545	890	675	555	775	675



Wall Type	Capacity	Sill Plate	Single 5/8"ϕ Bolt Capacity	5/8"ϕ Anchor Bolt Spacing	MASAP Anchor Capacity	MASAP Anchor Spacing
P1-6U	144-plf (E)	2x	1376#	60" oc	1060#	60" oc
P1-6	240-plf (E)	2x	1376#	60" oc	1060#	52" oc
P1-4	350-plf (E)	2x	1376#	46" oc	1060#	36" oc
P1-3	450-plf (E)	2x	1376#	36" oc	1060#	28" oc
P1-2	820-plf (W)	2x	1376#	20" oc	1250#	18" oc
P2-4	700-plf (E)	3x	1712#	28" oc	875#	15" oc
P2-3	900-plf (E)	3x	1712#	22" oc	875#	11" oc
P2-2	1640-plf (W)	3x	1712#	12" oc	1005#	7" oc

SHEAR WALL SCHEDULE

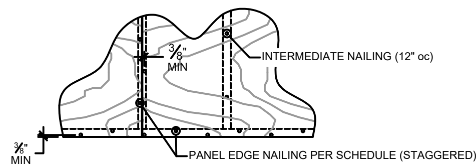
(IN ACCORDANCE w/ ANSI/AF&PA SDPWS-2015 SECTION 4.3)
Updated 1/20/2021

WALL TYPE	SHEATHING	PANEL EDGE NAILING ②	MINIMUM WIDTH OF NAILED FACE OF FRAMING @ ADJOINING PANEL EDGES ③		MUDSILL PLATE	FACE NAILING ④	FRAMING CLIPS ⑤	ANCHORAGE TO CONCRETE ⑥		SEISMIC CAPACITY - h/b = 2 h/b = 3.5	WIND CAPACITY - h/b = 2 h/b = 3.5
			SINGLE MEMBER	BUILT-UP MEMBER				ANCHOR BOLTS	MUDSILL ANCHORS		
P1-6	1 SIDE	6" oc	2x	2x	2x	6" oc	A35 @ 27" oc or LTP4 @ 27" oc	5/8"Ø @ 60" oc	MASAP @ 52" oc	240-plf 194-plf	240-plf 194-plf
P1-4	1 SIDE	4" oc	2x	2x	2x	4" oc	A35 @ 18" oc or LTP4 @ 18" oc	5/8"Ø @ 46" oc	MASAP @ 36" oc	350-plf 284-plf	350-plf 284-plf
P1-3	1 SIDE	3" oc	3x	(2)2x	2x	3" oc	A35 @ 14" oc or LTP4 @ 14" oc	5/8"Ø @ 36" oc	MASAP @ 28" oc	450-plf 366-plf	450-plf 366-plf
P1-2	1 SIDE	2" oc	3x	(2)2x	2x	2" oc	A35 @ 7 1/2" oc or LTP4 @ 7 1/2" oc	5/8"Ø @ 20" oc	MASAP @ 18" oc	590-plf 478-plf	820-plf 669-plf
P2-4	2 SIDES	4"oc	3x	(2)2x	3x	(2) Rows, 4" oc	A35 @ 18" oc and LTP4 @ 18" oc	5/8"Ø @ 28" oc	MASAP @ 15" oc	700-plf 568-plf	700-plf 568-plf
P2-3	2 SIDES	3" oc	3x	(2)2x	3x	(2) Rows, 3" oc	A35 @ 14" oc and LTP4 @ 14" oc	5/8"Ø @ 22" oc	MASAP @ 11" oc	900-plf 733-plf	900-plf 733-plf
P2-2	2 SIDES	2" oc	3x	(2)2x	3x	(2) Rows, 2" oc	A35 @ 8" oc and LTP4 @ 8" oc	5/8"Ø @ 12" oc	MASAP @ 7" oc	1180-plf 957-plf	1640-plf 1338-plf

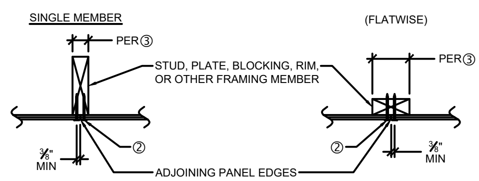
SHEAR WALL SCHEDULE NOTES

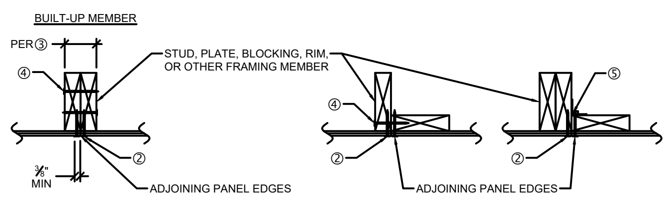
(SECTION 4.3.7.1.1)
7/16" OSB or 15/32" PLYWOOD SHEATHING OR SIDING EXCEPT GROUP 5 SPECIES. MINIMUM PANEL SPAN RATING OF (24/0). PANELS SHALL NOT BE LESS THAN 4'x8', EXCEPT AT BOUNDARIES AND CHANGES IN FRAMING. ALL EDGES OF ALL PANELS SHALL BE SUPPORTED BY AND FASTENED TO FRAMING MEMBERS OR BLOCKING.

② (SECTION 4.3.7.1.2. & SECTION 4.3.7.1.3)
PANEL EDGE NAILING APPLIES TO ALL SHEATHING PANEL EDGES. NAIL SHEATHING TO INTERMEDIATE FRAMING MEMBERS WITH SHEATHING NAILS @ 12"oc. MAXIMUM STUD SPACING SHALL BE 16"oc. SHEATHING NAILS SHALL BE 0.131"Ø x 2 1/2". PLYWOOD EDGE NAILING SHALL BE STAGGERED. NAILS SHALL BE LOCATED AT LEAST 3/8" FROM THE PANEL EDGES.



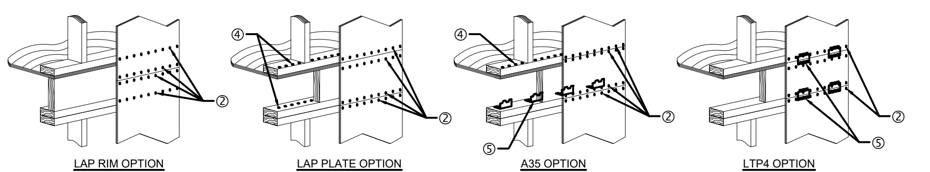
③ (SECTION 4.3.7.1.4)
THE MINIMUM NOMINAL WIDTH OF THE NAILED FACE OF FRAMING AND BLOCKING AT ADJOINING PANEL EDGES SHALL BE AS INDICATED IN THE SCHEDULE.



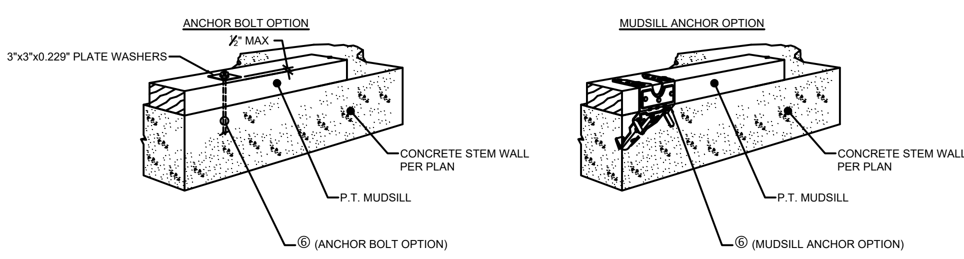


④ FACE NAILING APPLIES TO CONDITIONS WHERE FRAMING NAILS CAN BE STRAIGHT DRIVEN THRU FIRST MEMBER AND PENETRATE MAIN MEMBER MINIMUM OF 1 1/2". FRAMING NAILS SHALL BE 0.131"Ø x 3 1/4". 0.131"Ø x 3" NAILS MAY BE USED WHEN STITCHING TOGETHER (2)2x MEMBERS WITH NO SPACERS.

⑤ AT ADJOINING PANEL EDGES WHERE SHEATHING CANNOT LAP ON SINGLE MEMBER AND FACE NAILING CANNOT BE ACCOMPLISHED, FRAMING CLIPS SHALL BE USED TO FASTEN BUILT-UP MEMBERS. USE 0.131"Ø x 2 1/2" NAILS AT LTP4 CLIP WHEN INSTALLED OVER 1/2" SHEATHING.



⑥ (SECTION 4.3.6.4.3)
ANCHOR BOLTS EMBEDMENT SHALL BE 7", U.O.N. ALL ANCHORS SHALL HAVE 3" x 3" x 0.229" PLATE WASHERS. PLATE WASHER SHALL EXTEND TO WITHIN 1/2" OF THE EDGE OF THE BOTTOM PLATE ON THE SIDE WITH SHEATHING. IF SHEATHING IS ON BOTH SIDES OF THE WALL, STAGGER THE ANCHOR BOLTS, AS REQUIRED, SO THAT HALF OF THE PLATE WASHERS ARE WITHIN 1/2" OF THE EDGE OF THE BOTTOM PLATE ON EACH SIDE. HOLE IN PLATE WASHERS MAY BE DIAGONALLY SLOTTED.



BTL
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Miscellaneous

BT ENGINE

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Stud Wall Design

Based on 2018 NDS Combined axial and bending formula:

$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] < 1$ in which: $F_{cE} = 0.822(Emin')/(\ell_e/d)^2$

Wall: Exterior Walls

No Fire Rating

2x6

SPF Stud

Wall Height:	9	ft
Desired Stud Spacing:	24	in oc
Design Axial Dead Load:	683	plf
Design Axial Live Load:	960	plf
Design Axial Snow Load:	538	plf
Design Lateral Pressure (0.6W):	15	psf
Deflection Criteria: L/	240	

STUD CHECK	ℓ_e/d	<	50	OK
D+0.6W (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.53	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75(0.6W)+0.75S (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.92	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75S (C_D = 1.15)				
f_c/F_c' =	0.72	<	1	OK
D+L (C_D = 1.0)				
f_c/F_c' =	0.71	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 240 = 0.45	0.18	<	0.45	OK
SPF Stud 2x6 @ 24 oc				OK

PLATE CRUSHING CHECK[1] Checks Crushing for Stud Spacing[2] No Stress Increase for Load Duration				
Hem Fir Plates: $f_c/F_{c\perp}'$ =	0.87	<	1	OK
Douglas Fir Plates: $f_c/F_{c\perp}'$ =	0.56	<	1	OK

[1] Plate must also be checked for bending.

[2] Check on crushing only applies to stud spacing. Joists above must also be checked for crushing effect on plate.
Also, no stress increase is allowed due to load duration.

Date: 1/27/2021

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Stud Wall Design

Based on 2018 NDS Combined axial and bending formula:

$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] < 1$ in which: $F_{cE} = 0.822(Emin')/(\ell_e/d)^2$

Wall: Exterior Walls	Wall Height:	19.25	ft
	Desired Stud Spacing:	16	in oc
No Fire Rating	Design Axial Dead Load:	323	plf
(2)2x6	Design Axial Live Load:	0	plf
SPF Stud	Design Axial Snow Load:	538	plf
	Design Lateral Pressure (0.6W):	15	psf
	Deflection Criteria: L/	180	

STUD CHECK	ℓ_e/d	<	50	OK
D+0.6W (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.70	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75(0.6W)+0.75S (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.71	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75S (C_D = 1.15)				
f_c/F_c' =	0.30	<	1	OK
D+L (C_D = 1.0)				
f_c/F_c' =	0.14	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 180 = 1.28	1.24	<	1.28	OK
SPF Stud (2)2x6 @ 16 oc				OK

PLATE CRUSHING CHECK[1]				
Checks Crushing for Stud Spacing[2]				
No Stress Increase for Load Duration				
Hem Fir Plates: $f_c/F_{c\perp}'$ =	0.13	<	1	OK
Douglas Fir Plates: $f_c/F_{c\perp}'$ =	0.08	<	1	OK

[1] Plate must also be checked for bending.

[2] Check on crushing only applies to stud spacing. Joists above must also be checked for crushing effect on plate.
Also, no stress increase is allowed due to load duration.

Date: 1/27/2021

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Stud Wall Design

Based on 2018 NDS Combined axial and bending formula:

$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] < 1$ in which: $F_{cE} = 0.822(Emin')/(\ell_e/d)^2$

Wall: Interior Walls

No Fire Rating

2x4

SPF Stud

Wall Height:	9	ft
Desired Stud Spacing:	24	in oc
Design Axial Dead Load:	203	plf
Design Axial Live Load:	540	plf
Design Axial Snow Load:	0	plf
Design Lateral Pressure (0.6W):	5	psf
Deflection Criteria: L/	180	

STUD CHECK	ℓ_e/d < 50			OK
D+0.6W (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] =$	0.41	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2 =$	0.00	<	1	OK
D+0.75L+0.75(0.6W)+0.75S (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] =$	0.99	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2 =$	0.00	<	1	OK
D+0.75L+0.75S (C_D = 1.15)				
$f_c/F_c' =$	0.69	<	1	OK
D+L (C_D = 1.0)				
$f_c/F_c' =$	0.86	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 180 = 0.60	0.23	<	0.60	OK
SPF Stud 2x4 @ 24 oc				OK

PLATE CRUSHING CHECK[1]				
Checks Crushing for Stud Spacing[2]				
No Stress Increase for Load Duration				
Hem Fir Plates: $f_c/F_{c\perp}' =$	0.46	<	1	OK
Douglas Fir Plates: $f_c/F_{c\perp}' =$	0.30	<	1	OK

[1] Plate must also be checked for bending.

[2] Check on crushing only applies to stud spacing. Joists above must also be checked for crushing effect on plate. Also, no stress increase is allowed due to load duration.

Date: 1/27/2021

BT ENGINE

19011 Wood-Sno Road NE, Suite 100
Woodinville, WA 98072-4436
Phone: 425-814-8448
Fax: 425-821-2120

Stud Wall Design

Based on 2018 NDS Combined axial and bending formula:

$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] < 1$ in which: $F_{cE} = 0.822(Emin')/(\ell_e/d)^2$

Wall: Interior Walls

No Fire Rating

2x4

SPF Stud

Wall Height:	9	ft
Desired Stud Spacing:	16	in oc
Design Axial Dead Load:	338	plf
Design Axial Live Load:	900	plf
Design Axial Snow Load:	0	plf
Design Lateral Pressure (0.6W):	5	psf
Deflection Criteria: L/	180	

STUD CHECK	ℓ_e/d	<	50	OK
D+0.6W (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.31	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75(0.6W)+0.75S (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.99	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2$ =	0.00	<	1	OK
D+0.75L+0.75S (C_D = 1.15)				
f_c/F_c' =	0.76	<	1	OK
D+L (C_D = 1.0)				
f_c/F_c' =	0.95	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 180 = 0.60	0.15	<	0.60	OK
SPF Stud 2x4 @ 16 oc				OK

PLATE CRUSHING CHECK[1]				
Checks Crushing for Stud Spacing[2]				
No Stress Increase for Load Duration				
Hem Fir Plates: $f_c/F_{c\perp}'$ =	0.51	<	1	OK
Douglas Fir Plates: $f_c/F_{c\perp}'$ =	0.33	<	1	OK

[1] Plate must also be checked for bending.

[2] Check on crushing only applies to stud spacing. Joists above must also be checked for crushing effect on plate. Also, no stress increase is allowed due to load duration.

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2018 NDS

3.7-SOLID COLUMNS and 15.3-BUILT-UP COLUMNS

Solid Column	$F_c =$ 800 psi	$E_{min} =$ 440 ksi
Visually graded lumber (Dimensional)	$C_D =$ 1.00	$E_{min}' =$ 440 ksi
No Fire Rating	$C_M =$ 1.00	$l =$ 9.0 ft
Hem-Fir Stud	$C_t =$ 1.00	$d =$ 5 1/2 in
	$C_F =$ 1.00	$K_e =$ 1.0
		$l_e =$ 108.0 in
		$l_e/d =$ 19.6

$F_c' = F_c^* C_P$

$F_c^* = F_c C_D C_M C_t C_F$

$F_c^* =$ 800 psi

$C_p =$ 0.743

$\boxed{F_c' = 594 \text{ psi}}$

$$C_p = K_f \left[\frac{1 + \left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1 + \left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \right]$$

$F_{cE} =$ 938

$$F_{cE} = \frac{0.822\, E_{min}'}{\left(l_e/d\right)^2}$$

$c =$ 0.8

$K_f =$ 1.0

	STUD	HF Plate Crushing	DF Plate Crushing
(1) 2x6	4904	3341	5156
(2) 2x6	9807	6683	10313
(3) 2x6	14711	10024	15469
(4) 2x6	19614	13365	20625
(5) 2x6	24518	16706	25781

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2018 NDS

3.7-SOLID COLUMNS and 15.3-BUILT-UP COLUMNS

Solid Column	F_c =	800 psi	E_{min} =	440 ksi
Visually graded lumber (Dimensional)	C_D =	1.00	E_{min}' =	440 ksi
No Fire Rating	C_M =	1.00	l =	9.0 ft
Hem-Fir Stud	C_t =	1.00	d =	3 1/2 in
	C_F =	1.00	K_e =	1.0
			l_e =	108.0 in
			l_e/d =	30.9

$F_c' = F_c^* C_P$

$F_c^* = F_c\, C_D\, C_M\, C_t\, C_F$

F_c^* = 800 psi

C_p = 0.416

$\boxed{F_c' = 333 \text{ psi}}$

$$C_p = K_f\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}\right]$$

F_{cE} = 380

$$F_{cE} = \frac{0.822\, E\, \text{min}'}{\left(l_e/d\right)^2}$$

c = 0.8

K_f = 1.0

	STUD	HF Plate Crushing	DF Plate Crushing
(1) 2x4	1746	2126	3281
(2) 2x4	3492	4253	6563
(3) 2x4	5237	6379	9844
(4) 2x4	6983	8505	13125
(5) 2x4	8729	10631	16406

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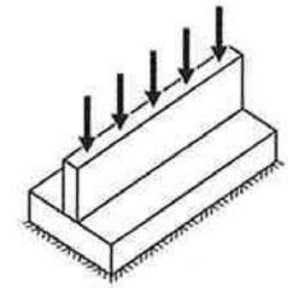
Project: Continuous Strip Footing
18" wide x 8" thick

IBC Section 13.3.2: One-way shallow foundations



Strip footing

Footing width, B = 18 in
Footing Thickness, t = 8 in
Stem Wall width, C = 8 in
Stem Wall Height = 24 in

Normalweight f'_c = 2500 psi
Uncoated f_y = 40000 psi

Longintudinal Reinforcement: (2) #4
Bar Diameter = 0.500 in
Bar Area = 0.20 in^2
A_s = 0.40 in^2
Cover: 3 in

Stem Wall Reinforcement: #4 @ 24 "oc Straight Dowels
Bar Diameter = 0.500 in
Bar Area = 0.20 in^2
A_s = 0.00 in^2
Cover: 3 in
$b_w =$ 12 in (per ft)
$d =$ 4.75 in

Footing + Stem Wall Weight - Weight of Displaced Soil = 240 plf

One-way shear, no shear reinforcement:

[22.5.5.1] $V_c = 2\lambda\sqrt{f'_c}b_w d =$ 5700 # per foot length $\phi =$ 0.75

[22.5.10.1] $V_u \le \phi V_c$

$$V_u = q_u b_w\left(\frac{B-C}{2} - d\right) \rightarrow q_u = \frac{\phi V_c}{b_w\left(\frac{B-C}{2} - d\right)}$$

$q_u =$ 51300 psf

Max Uniform Load on Stem = 76950 plf [Ultimate]
48094 plf [Service]

Moment:

[22.2.1.1] $M_n = A_s f_y(d - a/2) =$ 0.000 k-ft per foot length $\phi =$ 0.90

$M_u \le \phi M_n$

$$M_u = \frac{q_u b_w\left(\frac{B-C}{2}\right)^2}{2} \rightarrow q_u = \frac{2\phi M_n}{b_w\left(\frac{B-C}{2}\right)^2}$$

$a = \frac{A_s f_y}{0.85 f'_c b} =$ 0.00 in

$q_u =$ NO MOMENT

Max Uniform Load on Stem = 12000 plf [Ultimate]
7500 plf [Service]

Development of Reinforcement:

[25.4.2.3] $l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b =$ N/A

OK

Allowable Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Uniform Load, Soil	2010 plf	2760 plf	3510 plf	4260 plf	5010 plf	5760 plf
Max Uniform Load, Shear	48094 plf	48094 plf	48094 plf	48094 plf	48094 plf	48094 plf
Max Uniform Load, Moment	7500 plf	7500 plf	7500 plf	7500 plf	7500 plf	7500 plf
Max Uniform Load (Service)	**2010 plf**	**2760 plf**	**3510 plf**	**4260 plf**	**5010 plf**	**5760 plf**
Max Uniform Load (Ultimate)	**3216 plf**	**4416 plf**	**5616 plf**	**6816 plf**	**8016 plf**	**9216 plf**
Max Point Load (Service)	**16080 #**	**22080 #**	**28080 #**	**34080 #**	**40080 #**	**46080 #**
Max Point Load (Ultimate)	**25728 #**	**35328 #**	**44928 #**	**54528 #**	**64128 #**	**73728 #**

Date: 3/19/2018

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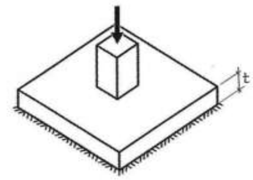
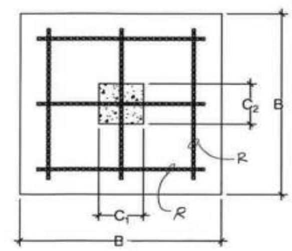
Project: Typical Footing
Footing: 18" x 18" x 8" thick





Footing B = 1.50 ft
t = 8 in

Reinforcement R = (2) #4
A_{s1} = 0.40 in²
d = 4.25 in Cover: 3 in

Column C_1 = 3.50 in C_2 = 3.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00
f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight
P_{FTG} = 0.06 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: $\emptyset$ = 0.75

$V_c = 2\lambda\sqrt{f'_c}Bd =$ 7.65 k

$V_u \le \emptyset V_c$ $\emptyset V_c =$ 5.74 k

$$V_u = q_u B\left(\frac{B-C_2}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_2}{2} - d\right)} \qquad V_u = q_u B\left(\frac{B-C_1}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_1}{2} - d\right)}$$

$q_u =$ 15300 psf or 15300 psf $P_u = q_u B^2 =$ 34425 #

Two-way shear: $\emptyset$ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 374 psi

β = 1.00
α_x = 40
b_0 = 2(C_1+d)+2(C_2+d)
31

$V_u \le \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 19.76 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\emptyset V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

$q_u =$ 10782 psf $P_u = q_u B^2 =$ 24260 #

Moment: $\emptyset$ = 0.90

$M_n = A_s f_y(d - {}^{a}/_{2}) =$ 5.4 k-ft

$a = A_s f_y/(0.85f'_c B) =$ 0.42 in

$M_u \le \emptyset M_n$ $\emptyset M_n =$ 4.8 k-ft

$$M_u = \frac{q_u B\left(\frac{B-C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B-C_2)/2\right)^2} \qquad M_u = \frac{q_u B\left(\frac{B-C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B-C_1)/2\right)^2}$$

$q_u =$ 17712 psf or 17712 psf $P_u = q_u B^2 =$ 39853 #

Development of Reinforcement:

$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b =$ 4 in ...4 in available **OK**

Adjusted

Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	3315	4440	5565	6690	7815	8940
Max Load (lbs), One-Way Shear	21516	21516	21516	21516	21516	21516
Max Load (lbs), Two-Way Shear	15162	15162	15162	15162	15162	15162
Max Load (lbs), Moment	24908	24908	24908	24908	24908	24908
Max Load (ASD)	**3315**	**4440**	**5565**	**6690**	**7815**	**8940**
Max Load (Factored)	**5304**	**7104**	**8904**	**10704**	**12504**	**14304**

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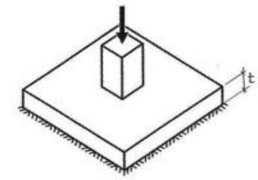
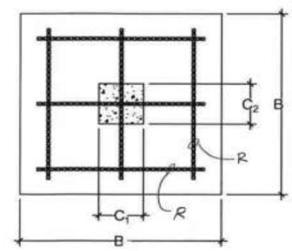
Project: Typical Footing
Footing: 24" x 24" x 8" thick



Isolated footing



Footing B = 2.00 ft

t = 8 in

Reinforcement R = (2) #4

A_{s1} = 0.40 in²

d = 4.25 in Cover: 3 in

Column C_1 = 3.50 in C_2 = 3.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00

f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight

P_{FTG} = 0.11 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: $\emptyset$ = 0.75

$$V_c = 2\lambda\sqrt{f'_c}Bd = \quad 10.20 \text{ k}$$

$$V_u \le \emptyset V_c \qquad \emptyset V_c = \quad 7.65 \text{ k}$$

$$V_u = q_u B\left(\frac{B-C_2}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_2}{2} - d\right)} \qquad V_u = q_u B\left(\frac{B-C_1}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_1}{2} - d\right)}$$

q_u = 7650 psf or 7650 psf $P_u = q_u B^2$ = 30600 #

Two-way shear: $\emptyset$ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 374 psi

β = 1.00
α_x = 40
b_0 = 2(C1+d)+2(C2+d)
31

$V_u \le \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 19.76 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\emptyset V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

q_u = 5516 psf $P_u = q_u B^2$ = 22063 #

Moment: $\emptyset$ = 0.90

$$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) = \quad 5.5 \text{ k-ft}$$

$a = A_s f_y/(0.85f'_c B)$ = 0.31 in

$$M_u \le \emptyset M_n \qquad \emptyset M_n = \quad 4.9 \text{ k-ft}$$

$$M_u = \frac{q_u B\left(\frac{B-C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B-C_2)/2\right)^2} \qquad M_u = \frac{q_u B\left(\frac{B-C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B-C_1)/2\right)^2}$$

q_u = 6732 psf or 6732 psf $P_u = q_u B^2$ = 26929 #

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b = \quad 7 \text{ in}$$

...7 in available **OK**

	Adjusted					
Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	5893	7893	9893	11893	13893	15893
Max Load (lbs), One-Way Shear	19125	19125	19125	19125	19125	19125
Max Load (lbs), Two-Way Shear	13789	13789	13789	13789	13789	13789
Max Load (lbs), Moment	16830	16830	16830	16830	16830	16830
Max Load (ASD)	**5893**	**7893**	**9893**	**11893**	**12710**	**12710**
Max Load (Factored)	**9429**	**12629**	**15829**	**19029**	**20337**	**20337**

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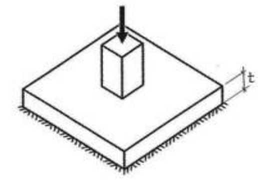
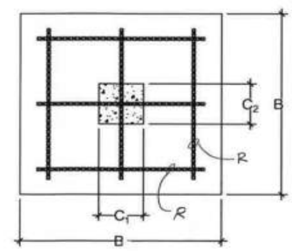
Project: Typical Footing
Footing: 30" x 30" x 8" thick



Isolated footing



Footing B = 2.50 ft
t = 8 in

Reinforcement R = (3) #4

A_{s1} = 0.60 in²

d = 4.25 in Cover: 3 in

Column C_1 = 3.50 in C_2 = 3.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00

f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight

P_{FTG} = 0.17 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: $\emptyset$ = 0.75

$V_c = 2\lambda\sqrt{f_c'}Bd =$ 12.75 k

$V_u \le \emptyset V_c$ $\emptyset V_c =$ 9.56 k

$$V_u = q_u B\left(\frac{B - C_2}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B - C_2}{2} - d\right)}$$

$$V_u = q_u B\left(\frac{B - C_1}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B - C_1}{2} - d\right)}$$

$q_u =$ 5100 psf or 5100 psf $P_u = q_u B^2 =$ 31875 #

Two-way shear: $\emptyset$ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f_c'} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f_c'} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f_c'} =$ 374 psi

β = 1.00
α_x = 40
b_0 = 2(C_1+d)+2(C_2+d)
31

$V_u \le \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 19.76 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\emptyset V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

$q_u =$ 3388 psf $P_u = q_u B^2 =$ 21176 #

Moment: $\emptyset$ = 0.90

$M_n = A_s f_y(d - {}^{a}/_{2}) =$ 8.1 k-ft

$a = A_s f_y/(0.85 f'_c B) =$ 0.38 in

$M_u \le \emptyset M_n$ $\emptyset M_n =$ 7.3 k-ft

$$M_u = \frac{q_u B\left(\frac{B - C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_2)/2\right)^2}$$

$$M_u = \frac{q_u B\left(\frac{B - C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_1)/2\right)^2}$$

$q_u =$ 4797 psf or 4797 psf $P_u = q_u B^2 =$ 29984 #

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b =$$ 10 in ...10 in available **OK**

Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	9208	12333	15458	18583	21708	24833
Max Load (lbs), One-Way Shear	19922	19922	19922	19922	19922	19922
Max Load (lbs), Two-Way Shear	13235	13235	13235	13235	13235	13235
Max Load (lbs), Moment	18740	18740	18740	18740	18740	18740
Max Load (ASD)	**9208**	**12333**	**13235**	**13235**	**13235**	**13235**
Max Load (Factored)	**14733**	**19733**	**21176**	**21176**	**21176**	**21176**

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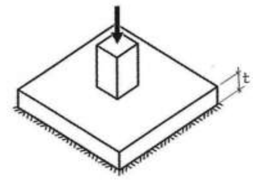
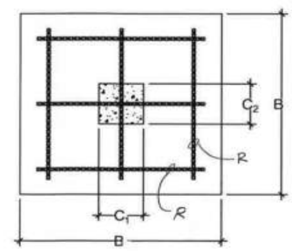
Project: Typical Footing
Footing: 36" x 36" x 12" thick



Isolated footing



Footing B = 3.00 ft
t = 12 in

Reinforcement R = (3) #4
A_{s1} = 0.60 in²
d = 8.25 in Cover: 3 in

Column C_1 = 5.50 in C_2 = 5.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00
f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight
P_{FTG} = 0.36 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: $\emptyset$ = 0.75

$V_c = 2\lambda\sqrt{f'_c}Bd =$ 29.70 k

$V_u \le \emptyset V_c$ $\emptyset V_c =$ 22.28 k

$$V_u = q_u B\left(\frac{B - C_2}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B - C_2}{2} - d\right)} \qquad V_u = q_u B\left(\frac{B - C_1}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B - C_1}{2} - d\right)}$$

$q_u =$ 12729 psf or 12729 psf $P_u = q_u B^2 =$ 114557 #

Two-way shear: $\emptyset$ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 400 psi

β = 1.00
α_x = 40
b_0 = 2(C_1+d)+2(C_2+d)
55

$V_u \le \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 68.06 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\emptyset V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

$q_u =$ 8854 psf $P_u = q_u B^2 =$ 79687 #

Moment: $\emptyset$ = 0.90

$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) =$ 16.2 k-ft

$a = A_s f_y/(0.85 f'_c B) =$ 0.31 in

$M_u \le \emptyset M_n$ $\emptyset M_n =$ 14.6 k-ft

$$M_u = \frac{q_u B\left(\frac{B - C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_2)/2\right)^2} \qquad M_u = \frac{q_u B\left(\frac{B - C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_1)/2\right)^2}$$

$q_u =$ 6013 psf or 6013 psf $P_u = q_u B^2 =$ 54121 #

Development of Reinforcement:

$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b =$ 12 in ...12 in available **OK**

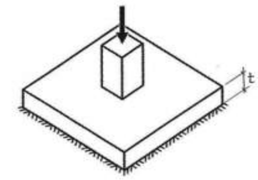
Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	13140	17640	22140	26640	31140	35640
Max Load (lbs), One-Way Shear	71598	71598	71598	71598	71598	71598
Max Load (lbs), Two-Way Shear	49805	49805	49805	49805	49805	49805
Max Load (lbs), Moment	33825	33825	33825	33825	33825	33825
Max Load (ASD)	**13140**	**17640**	**22140**	**26640**	**31140**	**33825**
Max Load (Factored)	**21024**	**28224**	**35424**	**42624**	**49824**	**54121**

Date: 1/23/2023

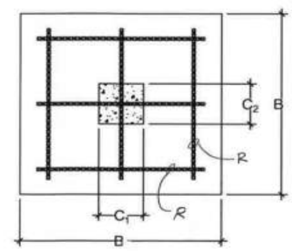
BTL ENGINEERING

19011 Wood-Sno Road NE, Suite 100
Woodinville, WA 98072-4436
Phone: 425-814-8448
Fax: 425-821-2120

Project: Typical Footing
Footing: 42" x 42" x 12" thick



Isolated footing



Footing B = 3.50 ft
t = 12 in

Reinforcement R = (4) #4
A_{s1} = 0.80 in²
d = 8.25 in Cover: 3 in

Column C_1 = 5.50 in C_2 = 5.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00
f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight
P_{FTG} = 0.49 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: $\emptyset$ = 0.75

$$V_c = 2\lambda\sqrt{f'_c}Bd = \quad 34.65 \text{ k}$$

$$V_u \leq \emptyset V_c \qquad \emptyset V_c = \quad 25.99 \text{ k}$$

$$V_u = q_u B\left(\frac{B - C_2}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_2}{2} - d\right)} \qquad V_u = q_u B\left(\frac{B - C_1}{2} - d\right) \rightarrow q_u = \frac{\emptyset V_c}{B\left(\frac{B-C_1}{2} - d\right)}$$

$q_u =$ 8910 psf or 8910 psf $P_u = q_u B^2 =$ 109148 #

Two-way shear: $\emptyset$ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

β = 1.00
α_x = 40
b_0 = 2(C_1+d)+2(C_2+d)
55

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 400 psi

$V_u \leq \phi V_c \qquad \phi V_c = \phi v_c b_0 d =$ 68.06 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\emptyset V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

$q_u =$ 6223 psf $P_u = q_u B^2 =$ 76233 #

Moment: $\emptyset$ = 0.90

$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) =$ 21.5 k-ft

$a = A_s f_y/(0.85 f'_c B) =$ 0.36 in

$M_u \leq \emptyset M_n \qquad \emptyset M_n =$ 19.4 k-ft

$$M_u = \frac{q_u B\left(\frac{B - C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_2)/2\right)^2} \qquad M_u = \frac{q_u B\left(\frac{B - C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\emptyset M_n}{B\left((B - C_1)/2\right)^2}$$

$q_u =$ 4785 psf or 4785 psf $P_u = q_u B^2 =$ 58622 #

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b =$$ 12 in ...15 in available **OK**

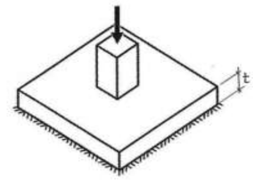
Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	17885	24010	30135	36260	42385	48510
Max Load (lbs), One-Way Shear	68217	68217	68217	68217	68217	68217
Max Load (lbs), Two-Way Shear	47646	47646	47646	47646	47646	47646
Max Load (lbs), Moment	36639	36639	36639	36639	36639	36639
Max Load (ASD)	**17885**	**24010**	**30135**	**36260**	**36639**	**36639**
Max Load (Factored)	**28616**	**38416**	**48216**	**58016**	**58622**	**58622**

Date: 1/23/2023

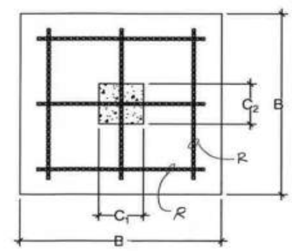
BTL ENGINEERING

19011 Wood-Sno Road NE, Suite 100
Woodinville, WA 98072-4436
Phone: 425-814-8448
Fax: 425-821-2120

Project: Typical Footing
Footing: 48" x 48" x 12" thick



Isolated footing



Footing B = 4.00 ft

t = 12 in

Reinforcement R = (5) #4

A_{s1} = 1.00 in²

d = 8.25 in Cover: 3 in

Column C_1 = 5.50 in C_2 = 5.50 in

Materials f'_c = 2500 psi Normalweight λ = 1.00

f_y = 40000 psi Uncoated ψ_e = 1.00

Net Footing Weight

P_{FTG} = 0.64 k

Soil Pressure:

$$P_{ASD} = q_a B^2 - P_{FTG} =$$

One-way shear: ϕ = 0.75

$$V_c = 2\lambda\sqrt{f'_c}Bd = \quad 39.60\text{ k}$$

$$V_u \le \phi V_c \quad \phi V_c = \quad 29.70\text{ k}$$

$$V_u = q_u B\left(\frac{B-C_2}{2} - d\right) \rightarrow q_u = \frac{\phi V_c}{B\left(\frac{B-C_2}{2} - d\right)} \qquad V_u = q_u B\left(\frac{B-C_1}{2} - d\right) \rightarrow q_u = \frac{\phi V_c}{B\left(\frac{B-C_1}{2} - d\right)}$$

$q_u =$ 6854 psf or 6854 psf $P_u = q_u B^2 =$ 109662 #

Two-way shear: ϕ = 0.75

[22.6.5.2(a)] $v_c = 4\lambda\sqrt{f'_c} =$ 200 psi ⇦

[22.6.5.2(b)] $v_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c} =$ 300 psi

[22.6.5.2(c)] $v_c = \left(2 + \frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f'_c} =$ 400 psi

β = 1.00
α_x = 40
b_0 = 2(C_1+d)+2(C_2+d)
55

$V_u \le \phi V_c \quad \phi V_c = \phi v_c b_0 d =$ 68.06 k

$$V_u = q_u[B^2 - (C_1 + d)(C_2 + d)] \rightarrow q_u = \frac{\phi V_c}{[B^2 - (C_1 + d)(C_2 + d)]}$$

$q_u =$ 4634 psf $P_u = q_u B^2 =$ 74147 #

Moment: ϕ = 0.90

$$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) = \quad 26.8\text{ k-ft}$$

$a = A_s f_y/(0.85f'_c B) =$ 0.39 in

$M_u \le \phi M_n \quad \phi M_n =$ 24.2 k-ft

$$M_u = \frac{q_u B\left(\frac{B-C_2}{2}\right)^2}{2} \rightarrow q_u = \frac{2\phi M_n}{B\left((B-C_2)/2\right)^2} \qquad M_u = \frac{q_u B\left(\frac{B-C_1}{2}\right)^2}{2} \rightarrow q_u = \frac{2\phi M_n}{B\left((B-C_1)/2\right)^2}$$

$q_u =$ 3853 psf or 3853 psf $P_u = q_u B^2 =$ 61640 #

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b = \quad 12\text{ in}$$

...18 in available **OK**

Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	23360	31360	39360	47360	55360	63360
Max Load (lbs), One-Way Shear	68538	68538	68538	68538	68538	68538
Max Load (lbs), Two-Way Shear	46342	46342	46342	46342	46342	46342
Max Load (lbs), Moment	38525	38525	38525	38525	38525	38525
Max Load (ASD)	**23360**	**31360**	**38525**	**38525**	**38525**	**38525**
Max Load (Factored)	**37376**	**50176**	**61640**	**61640**	**61640**	**61640**

Date: 1/23/2023