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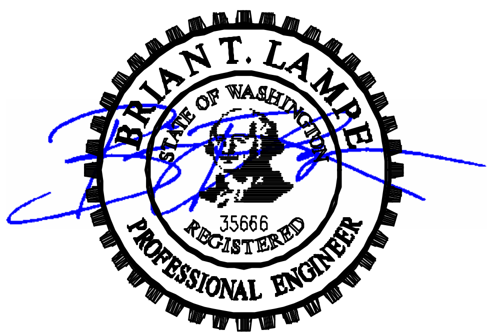
Structural Calculations

For

Walsh Residence

Covered Patio Addition
3817 80th Ave SE

July 9, 2019



Prepared by

Brian Lampe

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STRUCTURAL CALCULATIONS SHEET INDEX
Walsh Residence
3817 80th Ave SE

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Criteria

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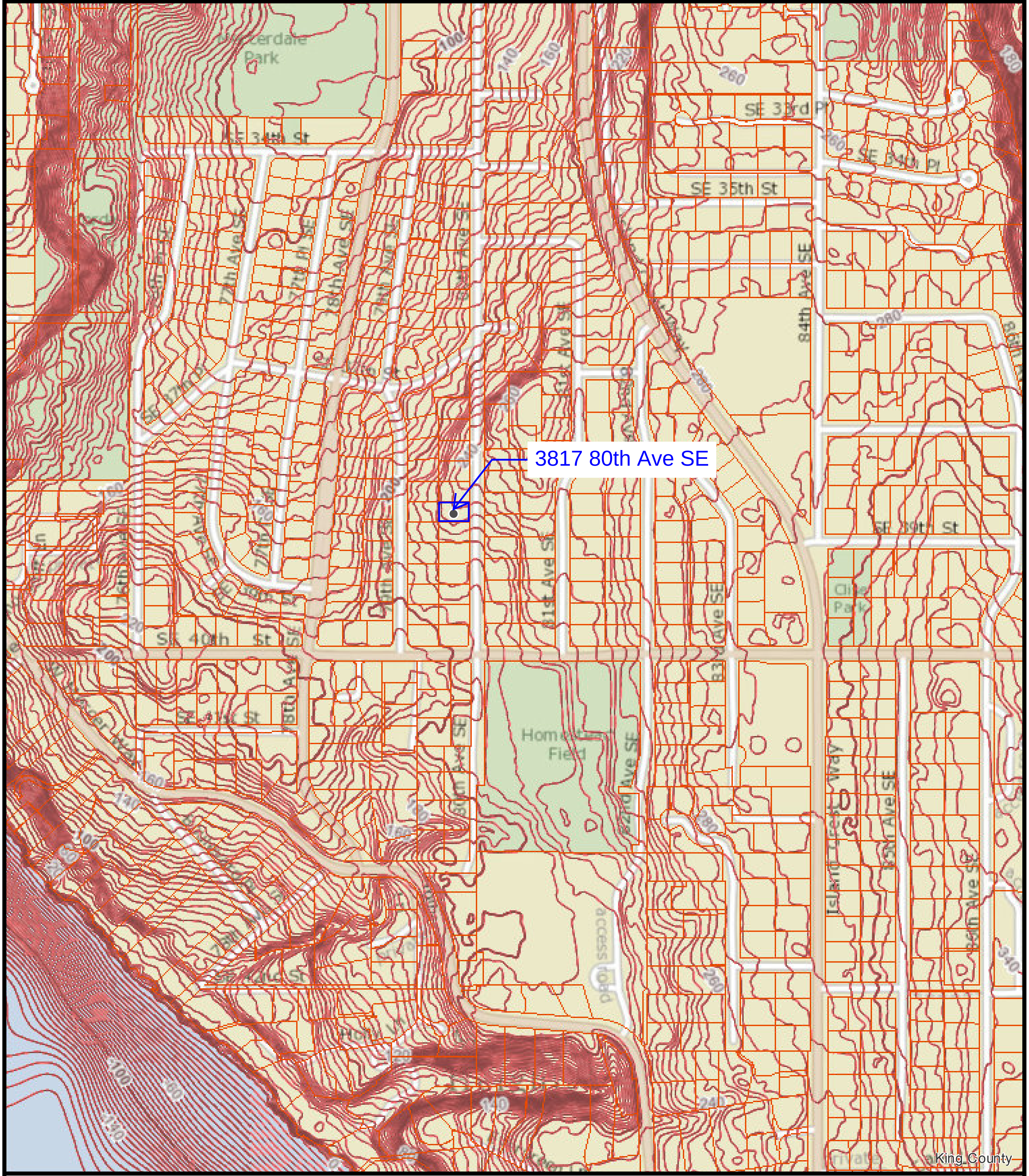
Project: Walsh Addition
Project Number: 3817 80th Ave SE

Code:	IBC 2015			
	Occupancy Category	II		
Earthquake:	Site Class	D		
	I_e =	1.00	R =	6.5
	S_S =	1.400	Ω_0 =	3.0
	S_1 =	0.538	C_d =	4.0
	ρ =	1.00		
Wind:	Ultimate Design Wind Speed, V_{ult}	110 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		1.1
	Flat Roof Snow, Pf (0.7 Ce Ct I Pg)			19 psf
	Use Snow Load			**25 psf**
Dead Load	Roofing - Composition Shingles			5.0 psf
	Sheathing - 5/8 Plywood			2.2 psf
	Framing - Trusses @ 24"oc			2.5 psf
	Soffit/Ceiling - 5/8 GWB			2.8 psf
	Misc.			2.5 psf
		Total		**15 psf**
Deflection	L/360 Live Load, L/240 Total Load			

King County iMap



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Date: 7/9/2019 Notes:

N

King County

C1.2

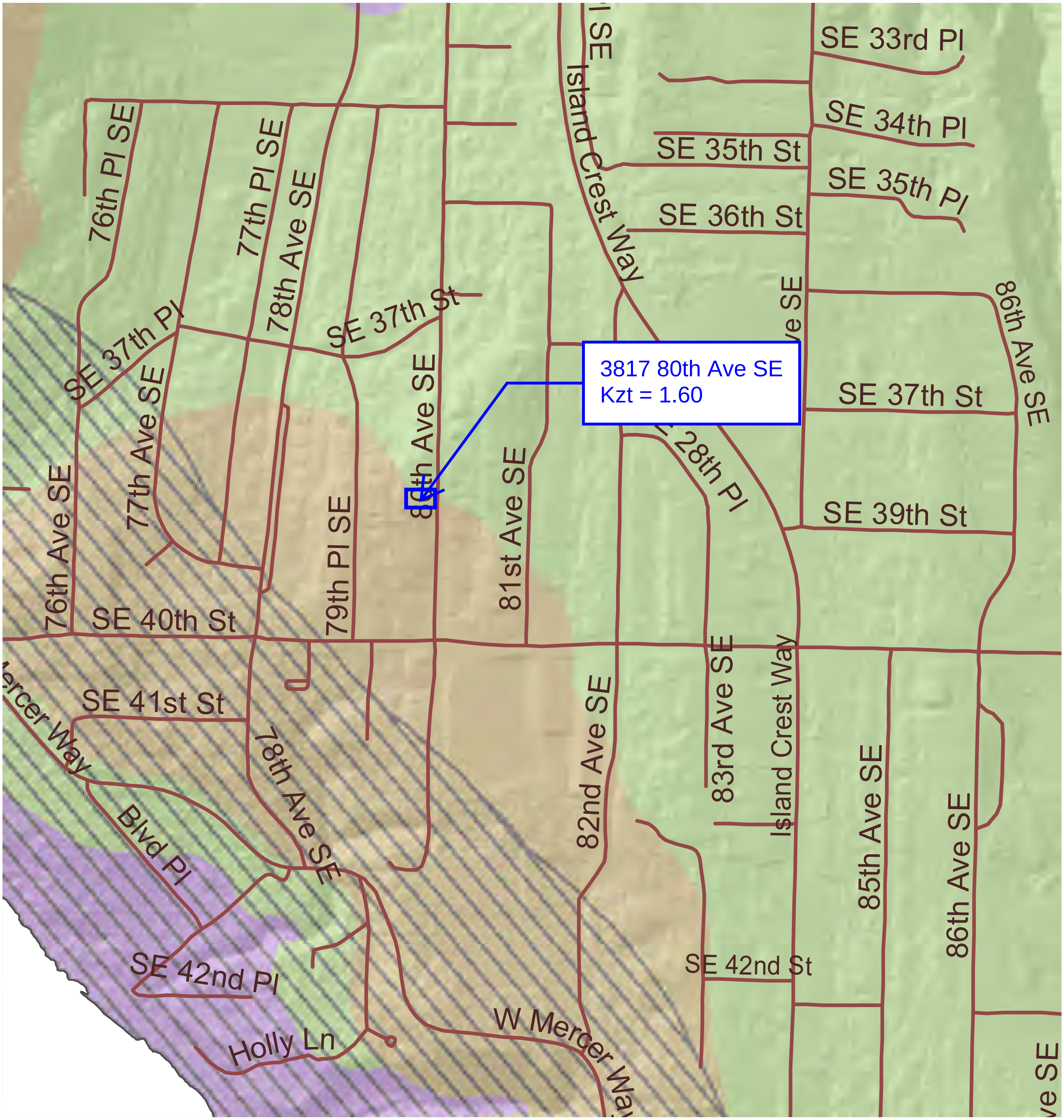
3817 80th Ave SE, Mercer Island, WA 98040, USA

Latitude, Longitude: 47.5760646, -122.23273570000003

Date	7/9/2019, 9:08:11 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.4	MCE_R ground motion. (for 0.2 second period)
S_1	0.538	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.4	Site-modified spectral acceleration value
S_{M1}	0.808	Site-modified spectral acceleration value
S_{DS}	0.933	Numeric seismic design value at 0.2 second SA
S_{D1}	0.538	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.578	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.578	Site modified peak ground acceleration
T_L	6	Long-period transition period in seconds
SsRT	1.4	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.461	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.998	Factored deterministic acceleration value. (0.2 second)
S1RT	0.538	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.577	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	1.235	Factored deterministic acceleration value. (1.0 second)
PGAd	1.156	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.958	Mapped value of the risk coefficient at short periods
C_{R1}	0.933	Mapped value of the risk coefficient at a period of 1 s



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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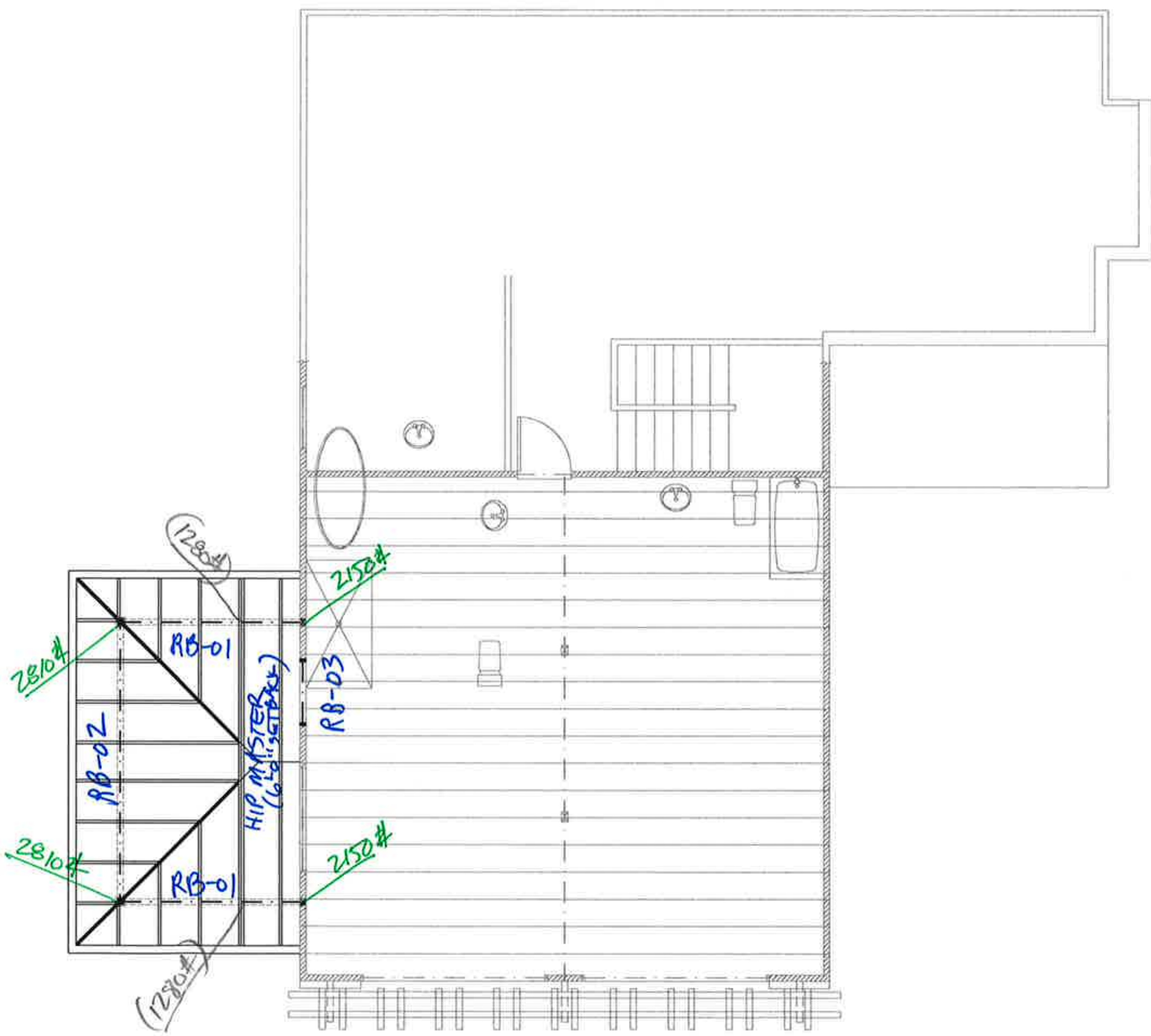
Gravity

Roof Framing



Roof Framing Key Plan

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RB-01

M = 4.75 k.ft

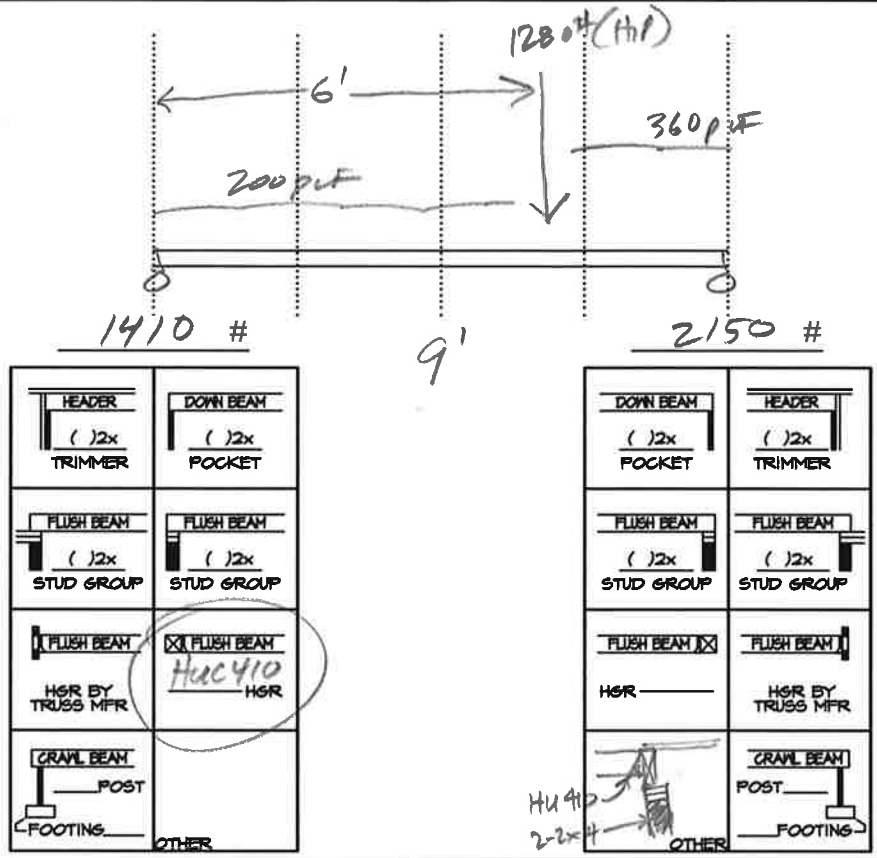
V = ______

L/360 = 0.30 in (LL)

L/240 = 0.45 in (TL)

$EI_{req'd}$ = 142 $\times 10^6$ lb.in^2

4x10



RB-02

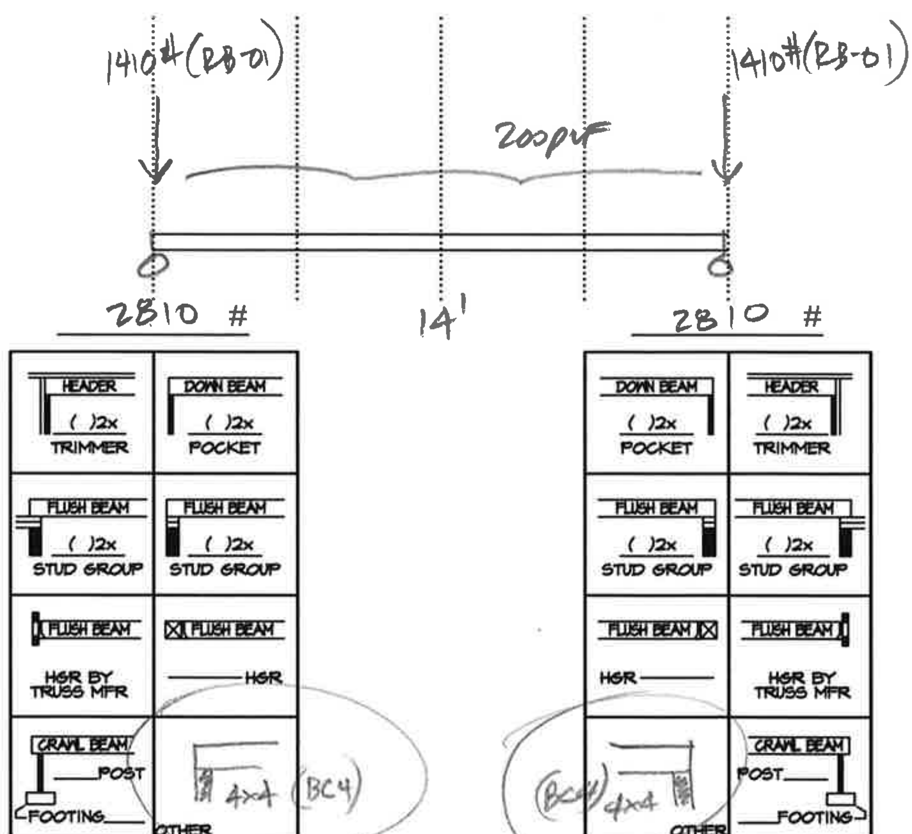
M = 4.90 k.ft

V = ______

L/360 = ______ (LL)

L/240 = ______ (TL)

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



Project: WALSH Designed By: BTL Date: 7/9/2019

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

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RB-03

M = 1.48 k.ft

V = ______

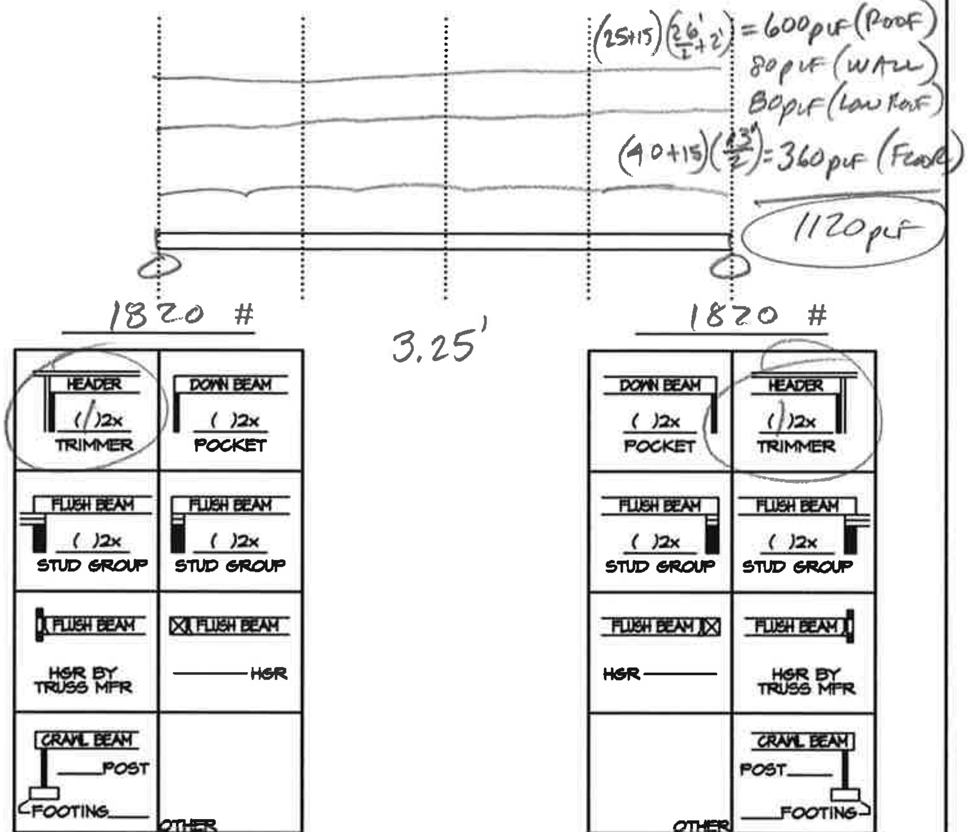
L/480 = 0.08 in

L/360 = 0.11 in (LL)

L/240 = 0.16 in (TL)

$EI_{req'd}$ = 22 $\times 10^6$ lb.in^2

4x8



RB-

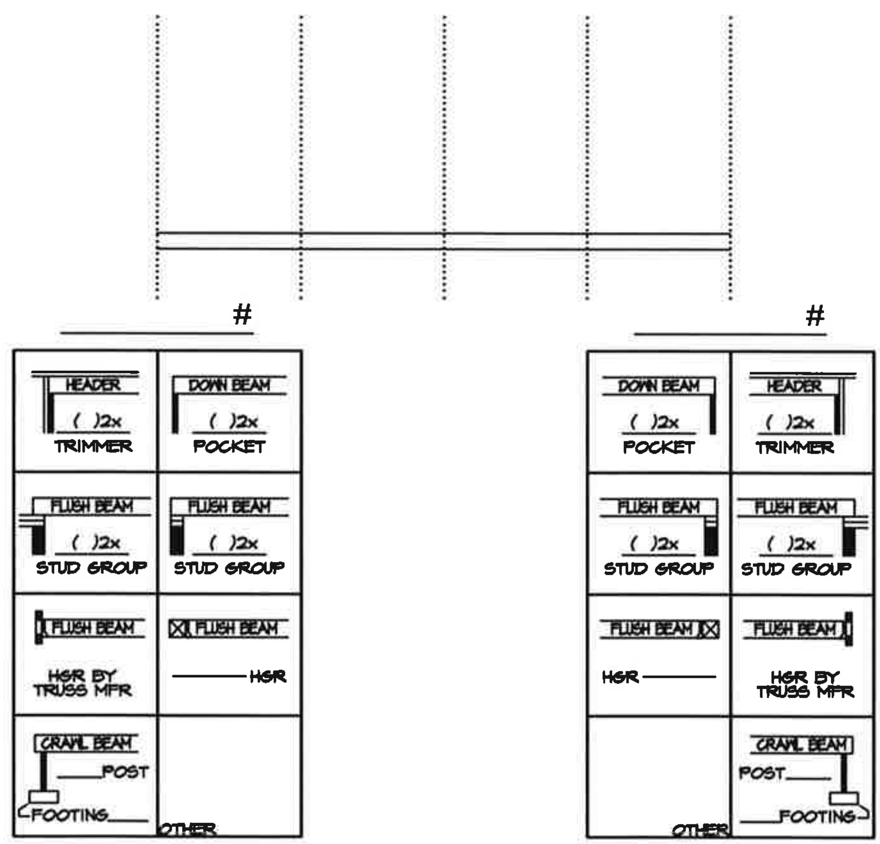
M = ______ k.ft

V = ______

L/360 = ______ (LL)

L/240 = ______ (TL)

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



Project: WALSH Designed By: BTL Date: 7/9/2019

Project Number: ______ Client: ______ Scale: ______ Page: R2.2

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Lateral

Forces

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Walsh Addition | Revision Date: 7/8/2019
3817 80th Ave SE

Criteria

Code:	2015 IBC		
	Allowable Stress Design (ASD)		
Seismic Design:	**ASCE 7-10: 12.8 Equivalent Lateral Force Procedure**		
Wind Design:	**ASCE 7-10: Ch. 28.5 Envelope 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 =	1.400	(Mapped)
Spectral Response, 1-s Period	S_1 =	0.538	(Mapped)
Site Class:	D		*Table 20.3-1*
Site Coefficient	F_a =	1.00	*Table 11.4-1*
Site Coefficient	F_v =	1.50	*Table 11.4-2*
Structural Systems:			
Light framed walls with shear panels			
All other structural systems	T_L =	6	*(Figs. 22-12 thru 22-16)*
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*
Ultimate Design Wind Speed:	110 mph		
Exposure to Wind:	Exposure B		*Section 26.7.3*
Topographical Factor	K_{ZT} =	1.60	

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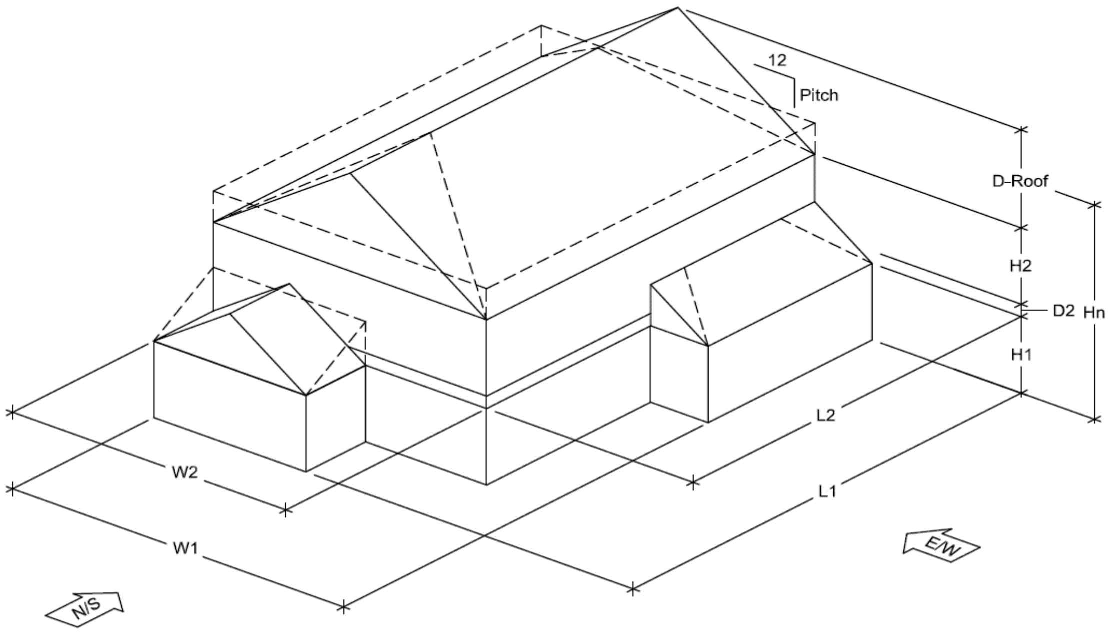
Walsh Addition
3817 80th Ave SE

Revision Date: 7/8/2019



Roof		
Geometry		
Mean Roof Height	Hn =	22 ft
Roof Depth	D-Roof =	8 ft
Overhang Length		18 in
Pitch		4:12
Floor 2		
Geometry		
Width	W2 =	42 ft
Length	L2 =	48 ft
Plate Height	H2 =	8 ft
Floor Depth	D2 =	12 in
Floor 1		
Geometry		
Width	W1 =	42 ft
Length	L1 =	48 ft
Plate Height	H1 =	8 ft
Floor Depth	D1 =	12 in

Seismic Weight - Roof				
Roof Area 1	1820 SF	15 psf		27,300#
Roof Area 2				
Roof Area 3				
Exterior Wall 1	180 LF	4 ft	10 psf	7,200#
Exterior Wall 2				
Exterior Wall 3				
Interior Wall	180 LF	4 ft	8 psf	5,760#
			Total	40,260#
Seismic Weight - Floor 2				
Roof Area 1	198 SF	15 psf		2,970#
Floor Area 1	1480 SF	15 psf		22,200#
Floor Area 2				
Floor Area 3				
Exterior Wall 1	180 LF	4 ft	10 psf	7,200#
Exterior Wall 2	180 LF	4 ft	10 psf	7,200#
Exterior Wall 3				
Interior Wall1	180 LF	4 ft	8 psf	5,760#
Interior Wall2	180 LF	4 ft	8 psf	5,760#
			Total	51,090#

N/S Projected Area - Roof	
Sloped Roof Area	105 SF
Gable/Parapet Area	75 SF
Wall Area	168 SF
E/W Projected Area - Roof	
Sloped Roof Area	250 SF
Gable/Parapet Area	
Wall Area	192 SF
N/S Projected Area - Floor 2	
Sloped Roof Area	25 SF
Gable/Parapet Area	
Wall Area	378 SF
E/W Projected Area - Floor 2	
Sloped Roof Area	
Gable/Parapet Area	
Wall Area	432 SF

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

$= 1.40$ $\quad$ $= 0.81$

$S_{DS} = ⅔\, S_{MS}$ *(Eq. 11.4-3)* $\quad$ $S_{D1} = ⅔\, S_{M1}$ *(Eq. 11.4-4)*

$= 0.93$ $\quad$ $= 0.54$

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), $h_n = 22$ ft

Approximate Fundamental Period, $T_a = C_t\,(h_n)^x$ *(Eq. 12.8-7)*

$T = T_a = 0.20$

$T_L = 6$ *(Figs. 22-12 thru 22-16)*

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)

$C_s = 0.14$

The total design base shear need not exceed:

(Eq. 12.8-3)

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

$C_s = 0.41$

(Eq. 12.8-4)

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

$C_s = 12.03$

$C_s = 0.41$

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)

$C_s = 0.04$

nor where $S_1 \geq 0.6g$:

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

$C_s = 0.00$

$C_s = 0.14$

$V = 0.14\, W$

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Walsh Addition
3817 80th Ave SE

Revision Date: 7/8/2019

$\rho_S = \lambda K_{ZT} \rho_{S30}$	λ =	1.00	Exposure =	B
	K_{ZT} =	1.60	Mean Roof Ht hn (ft) =	22 ft
			a (roof) =	4.2 ft
			a (upper/main floor) =	4.2 ft
			Ultimate Wind Speed =	110 mph
			Roof Angle =	19

North/South Loading

28.6.4 Minimum Design Loads

	Zone	Area	p_{S30} (psf)	$p_{S30\,design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Roof	A_{wall}	34	26.1	26.1	41.8	1403	842	538	323
	Agable	34	26.1	26.1	41.8	1403	842	538	323
	B	67	-7.2	0.0	0.0	0	0	538	323
	Cwall	134	17.4	17.4	27.8	3733	2240	2150	1290
	Cgable	41	17.4	17.4	27.8	1150	690	662	397
	D	38	-4.0	0.0	0.0	0	0	302	181
	Total Area =	348			Total Load =	7689	4614	4728	2837
					Design:	***7689***	***4614***		
	Zone	Area	p_{S30} (psf)	$p_{S30\,design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Floor 2	Awall	76	26.1	26.1	41.8	3157	1894	1210	726
	Agable	0	26.1	26.1	41.8	0	0	0	0
	B	25	-7.2	0.0	0.0	0	0	200	120
	Cwall	302	17.4	17.4	27.8	8399	5040	4838	2903
	Cgable	0	17.4	17.4	27.8	0	0	0	0
	D	0	-4.0	0.0	0.0	0	0	0	0
	Total Area =	403			Total Load =	11557	6934	6248	3749
					Design :	***11557***	***6934***		

East/West Loading

26.6.4 Minimum Design Loads

	Zone	Area	p_{S30} (psf)	$p_{S30\,design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Roof	Awall	34	26.1	26.1	41.8	1403	842	538	323
	Agable	0	26.1	26.1	41.8	0	0	0	0
	B	67	-7.2	0.0	0.0	0	0	538	323
	Cwall	158	17.4	17.4	27.8	4400	2640	2534	1521
	Cgable	0	17.4	17.4	27.8	0	0	0	0
	D	183	-4.0	0.0	0.0	0	0	1462	877
	Total Area =	442			Total Load =	5803	3482	5072	3043
					Design :	***5803***	***3482***		
	Zone	Area	p_{S30} (psf)	$p_{S30\,design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Floor 2	Awall	71	26.1	26.1	41.8	2982	1789	1142	685
	Agable	0	26.1	26.1	41.8	0	0	0	0
	B	0	-7.2	0.0	0.0	0	0	0	0
	Cwall	361	17.4	17.4	27.8	10016	6010	5770	3462
	Cgable	0	17.4	17.4	27.8	0	0	0	0
	D	0	-4.0	0.0	0.0	0	0	0	0
	Total Area =	432			Total Load =	12998	7799	6912	4147
					Design :	***12998***	***7799***		

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Revision Date: 7/8/2019

Vertical Distribution of Lateral Forces

Base Shear:

V = 13.12 kips

Shear Walls:

$F_x = C_{vx} V$ (Eq. 12.8-11) $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (Eq. 12.8-12)

Diaphragms:

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

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

$F_{px} = 0.4 S_{DS} I_e w_{px} \ldots [Eq. 12.10-3]$ (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	21.0	40.26	845	8.50	8.50	102	40	8.50
Floor 2	9.0	51.09	460	4.62	13.12	220	91	9.54

Totals W = 91.35 Kips

$\sum w_x h_x$ = 1305 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	7.69	7.69	5.80	5.80
Floor 2	11.56	19.25	13.00	18.80

	Diaphragm (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Roof	5.95	4.61	3.48
Floor 2	6.68	6.93	7.80

	Shear Walls (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Floor 2	5.95	4.61	3.48
Floor 1	3.23	6.93	7.80

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Lateral

Shear Walls/Diaphragms

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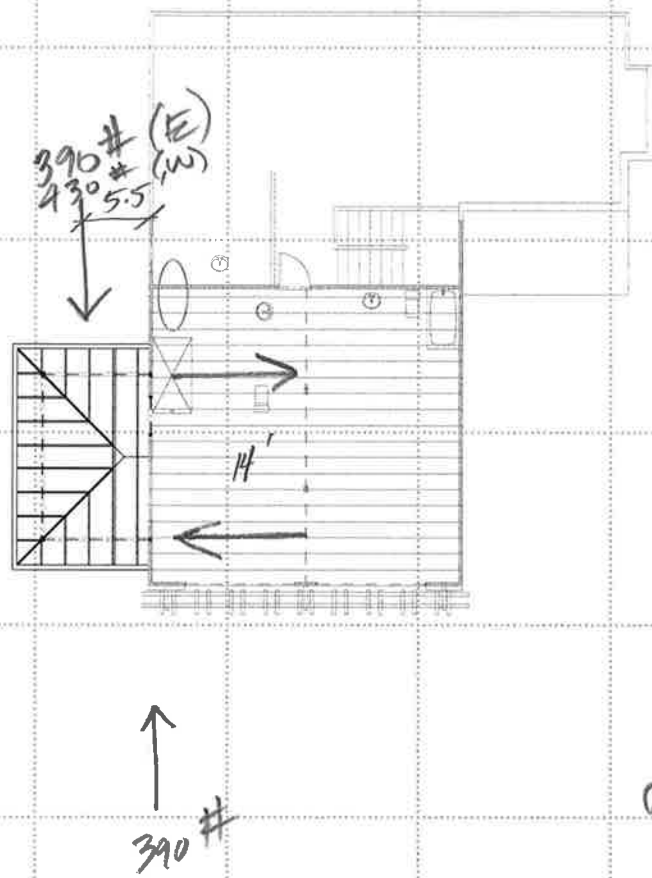
ROOF DIAPHRAGM
(PATIO ROOF)

ASSUMED NORTH

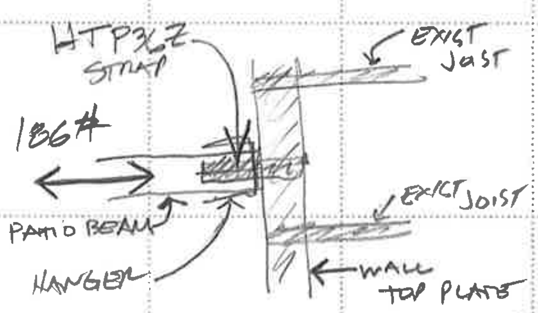


$\sigma = \frac{430\#}{18'} = 24\,plf \ll 150\,plf$

ACCIDENTAL TORSION

ROTATION: $(430\#)(5.5')(1.10) = 2600\,ft\#$

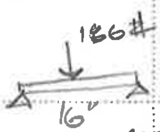
RESISTING: $\frac{2600\,ft\#}{14'} = 186\#$



CHECK DIAPHRAGM STRESS

PATIO ROOF: $\frac{186\#}{11'} = 17\,plf \ll 150\,plf$ ✓

CHECK PLATE BENDING:



EXIST. FLOOR: $\frac{186\#}{13'} = 14\,plf \ll 150\,plf$ ✓

$M = 0.06\,k\cdot ft$

↳ EXIST 2x4 TOP PLATE OK

TOENAILS: $(80\#)(1.60)(0.83)$

$= 106\#/\text{TOENAIL}$

EXIST (2) TOENAILS $= 212\# > 186\#$ ✓ OK

Project: WALSH Designed By: BTL Date: 7/9/2019

Project Number: Client: Scale: Page: L2.1

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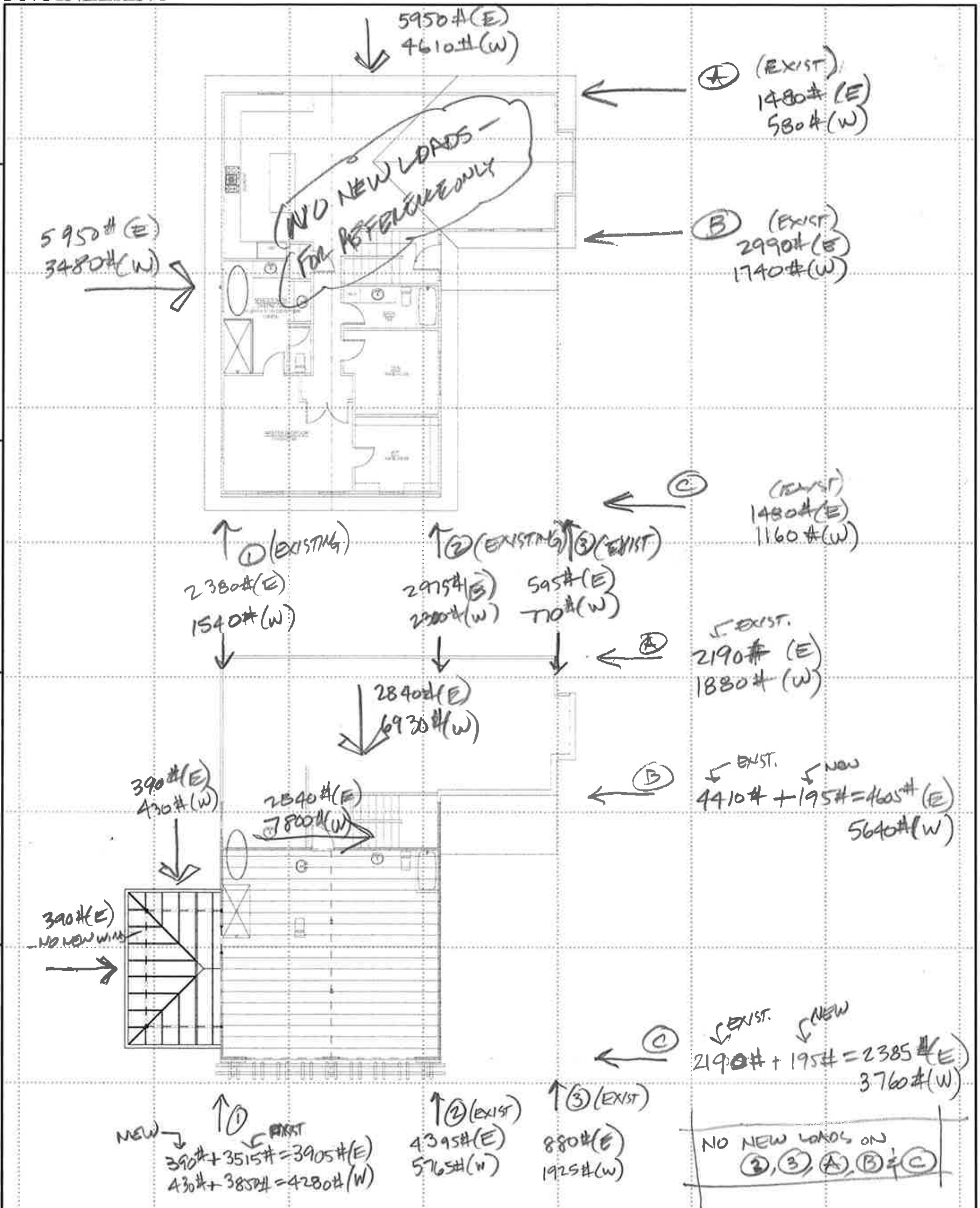
UPPER FLOOR SHEAR WALL

MAIN FLOOR SHEAR WALLS



Project: WALSH
Designed By: BTL
Date: 7/9/2019

Project Number: ______ Client: ______ Scale: ______ Page: L2.2

BTL ENGINEERING

19011 Wood-Sno Road NE, Suite 100
Woodinville, WA 98072-4436
Phone: 425-814-8448
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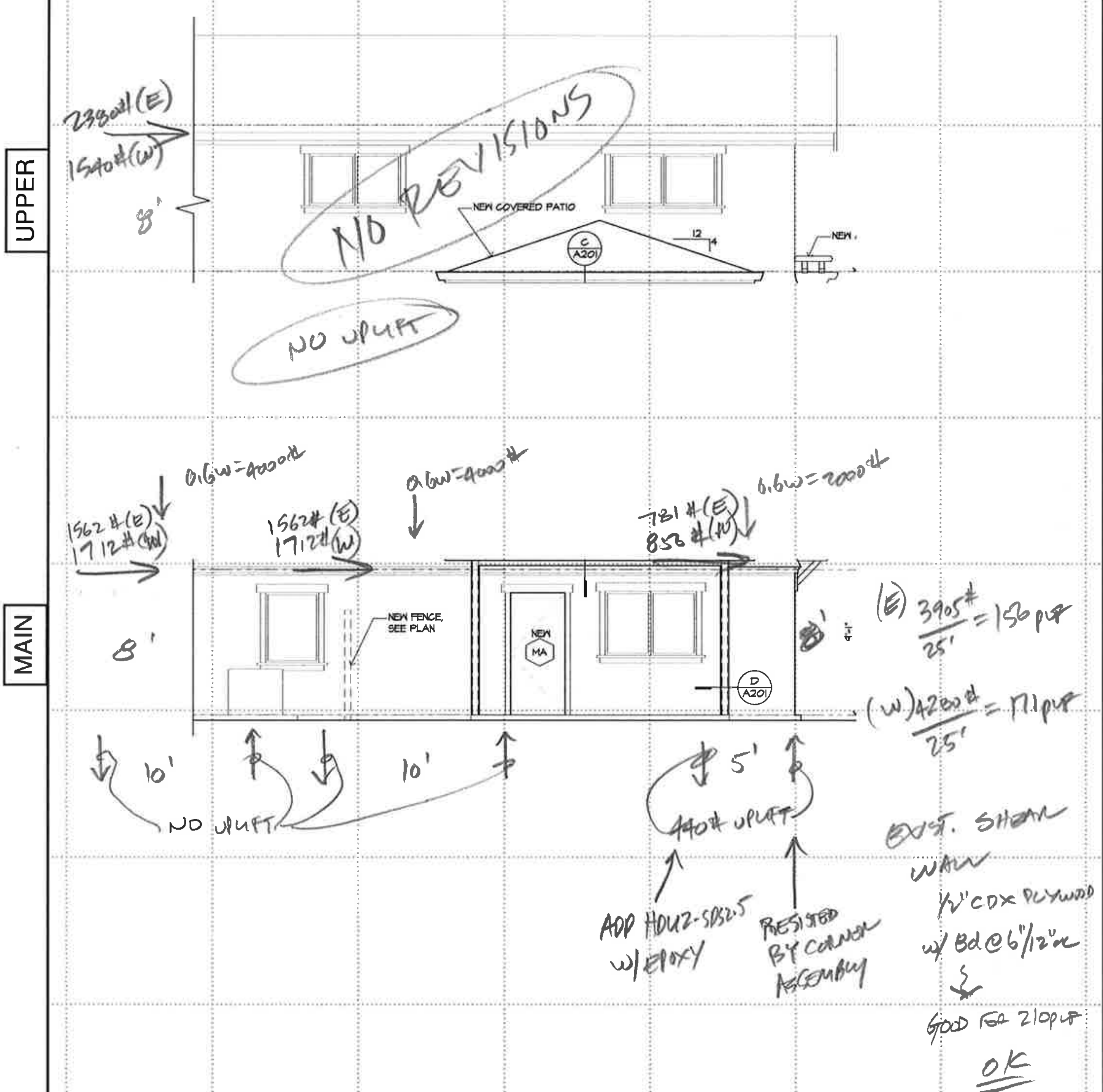
SHEAR WALL LINE 1



Project: WALSH Designed By: BTL Date: 7/9/2019

Project Number: Client: Scale: Page: L2.3

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 8.000
Code report: ICC-ES ESR-2508
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.13
c_{ac} (inch): 15.82
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, f'c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: A tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: Yes
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Periodic
Temperature range, Short/Long: 150/110°F
Ignore 6do requirement: Not applicable
Build-up grout pad: No

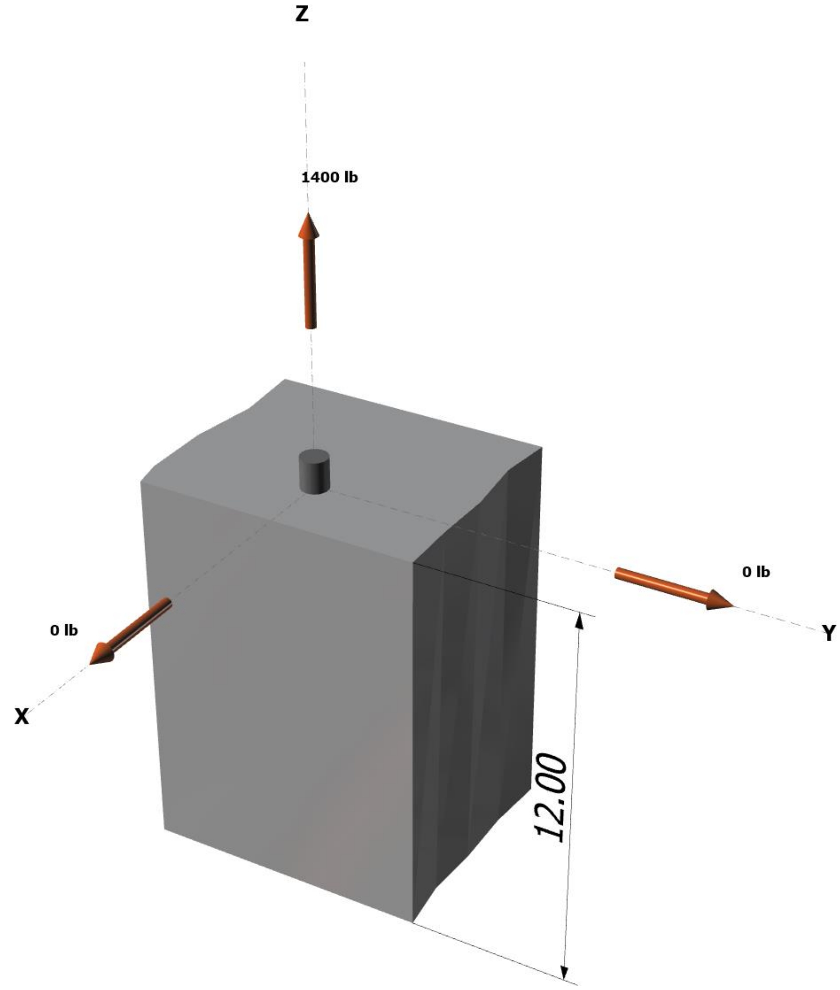
Recommended Anchor
Anchor Name: SET-XP® - SET-XP w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-2508

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: Yes
Anchors only resisting wind and/or seismic loads: No

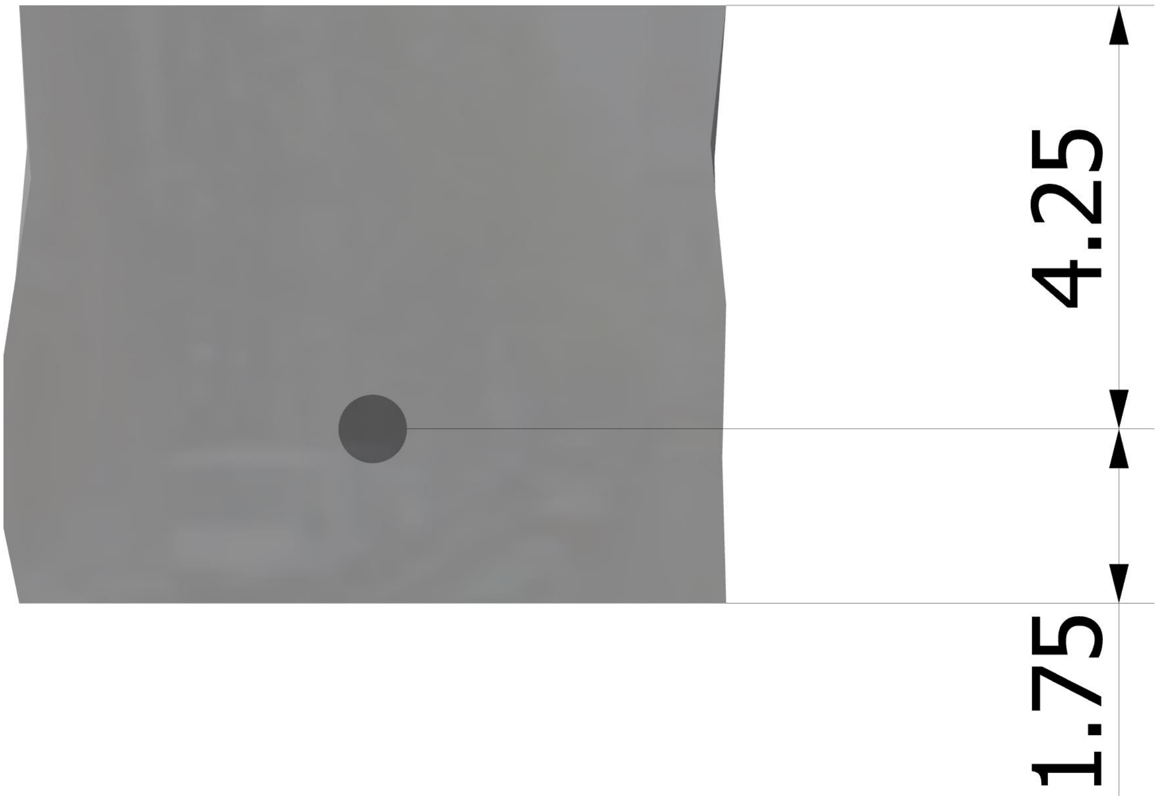
Strength level loads:

N_{ua} [lb]: 1400
V_{uax} [lb]: 0
V_{uay} [lb]: 0

<Figure 1>



<Figure 2>



3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	1400.0	0.0	0.0	0.0
Sum	1400.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1400
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
13110	0.75	9833

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	8.000	19233

$\phi N_{cb} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cb} (lb)
144.00	576.00	1.75	0.744	1.00	1.000	19233	0.75	2682

6. Adhesive Strength of Anchor in Tension (Sec. 17.4.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\tau_{k,cr}$ (psi)
435	1.00	1.00	435

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.4.5.2)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	435	0.63	8.000	6833

$\phi N_a = \phi\,(A_{Na} / A_{Na0})\,\Psi_{ed,Na}\,\Psi_{cp,Na} N_{ba}$ (Sec. 17.3.1 & Eq. 17.4.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	ϕ	ϕN_a (lb)
73.62	150.57	6.14	1.75	0.786	1.000	6833	0.55	1444

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	1400	9833	0.14	Pass
Concrete breakout	1400	2682	0.52	Pass
Adhesive	**1400**	**1444**	**0.97**	**Pass (Governs)**

SET-XP w/ 5/8"Ø F1554 Gr. 36 with hef = 8.000 inch meets the selected design criteria.

12. Warnings

- When cracked concrete is selected, concrete compressive strength used in concrete breakout strength in tension, adhesive strength in tension and concrete pryout strength in shear for SET-XP adhesive anchor is limited to 2,500 psi per ICC-ES ESR-2508 Section 5.3.

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

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Lateral

Shear Wall/Diaphragm Capacities

2015 IBC – Diaphragms (8d Nailing)

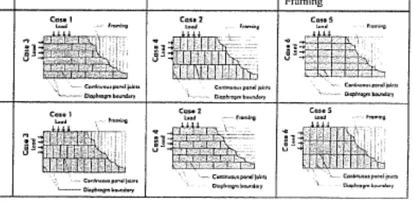
Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

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						B WIND – 6 in. Nail Spacing at diaphragm boundaries and supported panel edges	
					Case 1			Cases 2,3,4,5,6			Case 1	Cases 2,3,4,5,6
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	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)

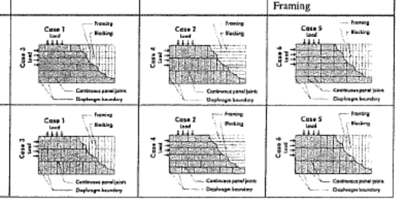
Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms

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

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.)	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)												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)			
					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 (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
Structural I	6d	1-1/4	5/16	2	370	15	12	500	8.5	7.5	750	12	10	840	20	15	520	700	1050	1175
				3	420	12	9.5	560	7.0	6.0	840	9.5	8.5	950	17	13	590	785	1175	1330
	8d	1-3/8	3/8	2	540	14	11	720	9.0	7.5	1060	13	10	1200	21	15	755	1010	1485	1680
				3	600	12	10	800	7.5	6.5	1200	10	9.0	1350	18	13	840	1120	1680	1890
	10d	1-1/2	15/32	2	640	24	17	850	15	12	1280	20	15	1460	31	21	895	1190	1790	2045
				3	720	20	15	960	12	9.5	1440	16	13	1640	26	18	1010	1345	2015	2295
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	475	630	940	1065
				3	380	12	9.0	500	7.0	6.0	760	10	8.0	860	17	12	530	700	1065	1205
			3/8	2	370	13	9.5	500	7.0	6.0	750	10	8.0	840	18	12	520	700	1050	1175
				3	420	10	8.0	560	5.5	5.0	840	8.5	7.0	950	14	10	590	785	1175	1330
	8d	1-3/8	3/8	2	480	15	11	640	9.5	7.5	960	13	9.5	1090	21	13	670	895	1345	1525
				3	540	12	9.5	720	7.5	6.0	1080	11	8.5	1220	18	12	755	1010	1510	1710
			7/16	2	510	14	10	680	8.5	7.0	1010	12	9.5	1150	20	13	715	950	1415	1610
				3	570	11	9.0	760	7.0	6.0	1140	10	8.0	1290	17	12	800	1065	1595	1805
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680
				3	600	10	8.5	800	6.0	5.5	1200	9.0	7.5	1350	15	11	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045
				3	720	17	12	960	10	8.0	1440	14	11	1640	24	15	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

2015 IBC – 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			4			3			2			B WIND – Panel Edge Fastener Spacing (in.) 6	4	3	2
				v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_s (plf)	G_a (kips/in.)		v_w (plf)	v_w (plf)	v_w (plf)	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 $h/b_s = 3.5$
P1-6U	N	1	6”/12” oc	2x	2x	144-plf	117-plf	201-plf	164-plf
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

2015 IBC – Shear Wall Framing Clips

Model No.	Type of Connection	Fasteners (in.)	Direction of Load	DF/SP Allowable Loads Floor (100)	DF/SP Allowable Loads Roof (125)	DF/SP Allowable Loads (160)	SPF/HF Allowable Loads Floor (100)	SPF/HF Allowable Loads Roof (125)	SPF/HF Allowable Loads (160)
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
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

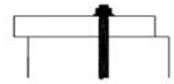
1. Allowable loads are for one angle. When angles are installed on each side of the joist, the minimum joist thickness is 3".
2. Some illustrations show connections that could cause cross-grain tension or bending of the wood during loading if not reinforced sufficiently. In this case, mechanical reinforcement 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

2015 IBC – Shear Wall Bolts

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

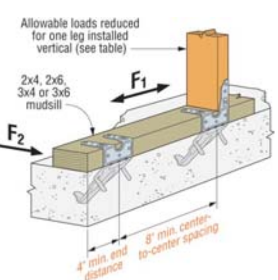
for sawn lumber or SCL to concrete



Embedment Depth in Concrete t_m in.	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	Uncracked Wind and SDC A&B[5,6] F_1	Uncracked Wind and SDC A&B[5,6] F_2	Uncracked SDC C–F[6] Uplift	Uncracked SDC C–F[6] F_1	Uncracked SDC C–F[6] F_2	Cracked Wind and SDC A&B[5,6] Uplift	Cracked Wind and SDC A&B[5,6] F_1	Cracked Wind and SDC A&B[5,6] F_2	Cracked SDC C–F[6] Uplift	Cracked SDC C–F[6] F_1	Cracked SDC C–F[6] 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

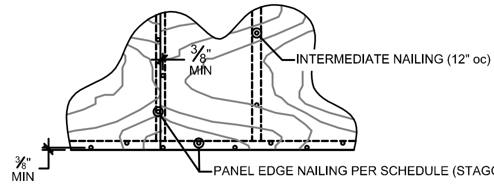
(IN ACCORDANCE w/ ANSI/AF&PA SDPWS-2015 SECTION 4.3)
Updated 5/28/2019

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 LTP5 @ 27" oc	⅝"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 LTP5 @ 18" oc	⅝"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 LTP5 @ 14" oc	⅝"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 @ 7½" oc or LTP5 @ 7½" oc	⅝"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 LTP5 @ 18" oc	⅝"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 LTP5 @ 14" oc	⅝"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 LTP5 @ 8" oc	⅝"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.

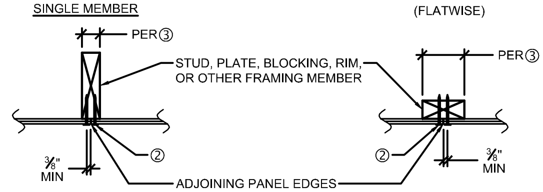
② (SECTION 4.3.7.1.2. & SECTION 4.3.7.1.3)
PANEL EDGE NAILING APPLIES TO ALL SHEATHING PANEL EDGES. NAIL SHEATHING TO INTERMEDIATE FRAMING MEMBERS WITH SHEATHING NAILS @ 12"oc. MAXIMUM STUD SPACING SHALL BE 16"oc. SHEATHING NAILS SHALL BE 0.131"Ø x 2½". PLYWOOD EDGE NAILING SHALL BE STAGGERED. NAILS SHALL BE LOCATED AT LEAST ⅜" FROM THE PANEL EDGES.



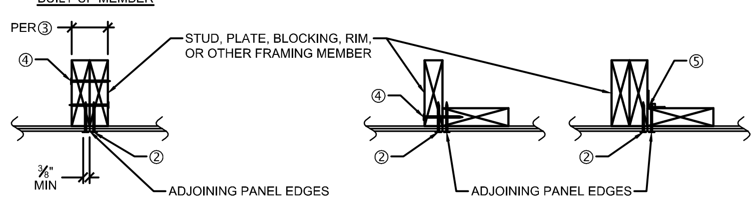
③ (SECTION 4.3.7.1.4)
THE MINIMUM NOMINAL WIDTH OF THE NAILED FACE OF FRAMING AND BLOCKING AT ADJOINING PANEL EDGES SHALL BE AS INDICATED IN THE SCHEDULE.

SINGLE MEMBER

(FLATWISE)

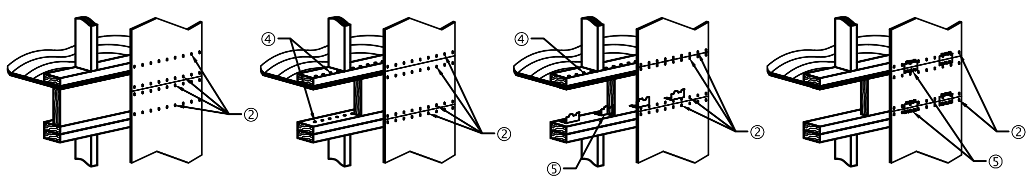


BUILT-UP MEMBER



④ FACE NAILING APPLIES TO CONDITIONS WHERE FRAMING NAILS CAN BE STRAIGHT DRIVEN THRU FIRST MEMBER AND PENETRATE MAIN MEMBER MINIMUM OF 1½". FRAMING NAILS SHALL BE 0.131"Ø x 3¼". 0.131"Øx3" NAILS MAY BE USED WHEN STITCHING TOGETHER (2)2x MEMBERS WITH NO SPACERS.

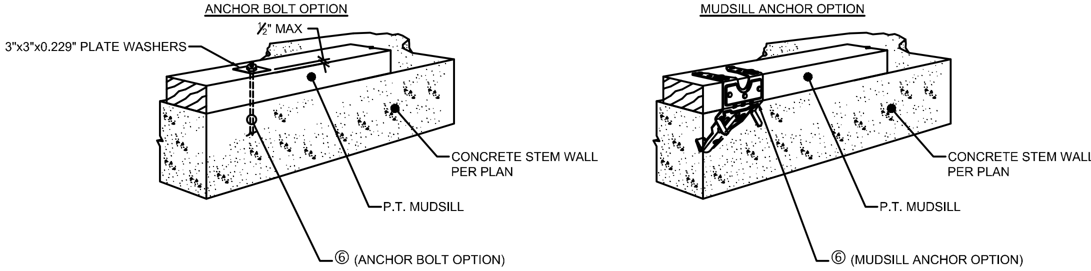
⑤ AT ADJOINING PANEL EDGES WHERE SHEATHING CANNOT LAP ON SINGLE MEMBER AND FACE NAILING CANNOT BE ACCOMPLISHED, FRAMING CLIPS SHALL BE USED TO FASTEN BUILT-UP MEMBERS.



LAP RIM OPTION | LAP PLATE OPTION | A35 OPTION | LTP5 OPTION

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 ½" 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 ½" OF THE EDGE OF THE BOTTOM PLATE ON EACH SIDE. HOLE IN PLATE WASHERS MAY BE DIAGONALLY SLOTTED.

ANCHOR BOLT OPTION | MUDSILL ANCHOR OPTION



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Miscellaneous

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Stud Wall Design

Based on 2015 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:	16	in oc
Design Axial Dead Load:	700	plf
Design Axial Live Load:	800	plf
Design Axial Snow Load:	700	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)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] =$	0.32	<	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)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] =$	0.45	<	1	OK
$f_c/F_{cE2} + (f_b/F_{bE})^2 =$	0.00	<	1	OK
D+0.75L+0.75S (C_D = 1.15)				
$f_c/F_c' =$	0.48	<	1	OK
D+L (C_D = 1.0)				
$f_c/F_c' =$	0.43	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 240 = 0.45	0.12	<	0.45	OK
SPF Stud 2x6 @ 16 oc				OK

PLATE CRUSHING CHECK[1] Checks Crushing for Stud Spacing[2] No Stress Increase for Load Duration				
Hem Fir Plates: $f_c/F_{c\perp}' =$	0.58	<	1	OK
Douglas Fir Plates: $f_c/F_{c\perp}' =$	0.38	<	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: 12/14/2016

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Stud Wall Design

Based on 2015 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:	400	plf
Design Axial Live Load:	800	plf
Design Axial Snow Load:	0	plf
Design Lateral Pressure (0.6W):	5	psf
Deflection Criteria: L/	240	

STUD CHECK	ℓ_e/d	<	50	OK
D+0.6W (C_D = 1.60)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.35	<	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)				
$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})]$ =	0.96	<	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' =	0.75	<	1	OK
D+L (C_D = 1.0)				
f_c/F_c' =	0.92	<	1	OK
Deflection (No Increase for Load Duration):				
Defl: L/ 240 = 0.45	0.15	<	0.45	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: $f_c/F_{c\perp}'$ =	0.50	<	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.

Date: 12/14/2016

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2015 NDS

3.7-SOLID COLUMNS and 15.3-BUILT-UP COLUMNS

Solid Column

Visually graded lumber (Dimensional)

No Fire Rating

Hem-Fir Stud

F_c =	800 psi	E_{min} =	440 ksi
C_D =	1.00	E_{min}' =	440 ksi
C_M =	1.00	l =	9.0 ft
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

Date: 12/14/2016

BTL ENGINEERING

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2015 NDS

3.7-SOLID COLUMNS and 15.3-BUILT-UP COLUMNS

Solid Column
Visually graded lumber (Dimensional)
No Fire Rating
Hem-Fir Stud

F_c = 800 psi
C_D = 1.00
C_M = 1.00
C_t = 1.00
C_F = 1.00

E_{min} = 440 ksi
E_{min}' = 440 ksi
l = 9.0 ft
d = 3 1/2 in
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

F_c' = 333 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

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

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

Date: 12/14/2016

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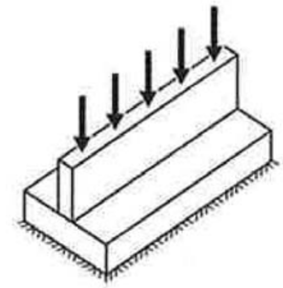
Project: Continuous Strip Footing
18" wide x 8" thick

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] $V_u \leq \phi V_c$

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

$q_u =$ 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

$M_u \leq \phi M_n$

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

$$a = \frac{A_s f_y}{0.85 f'_c b} =$$ 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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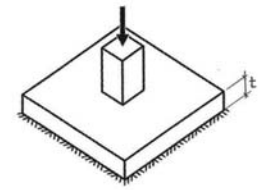
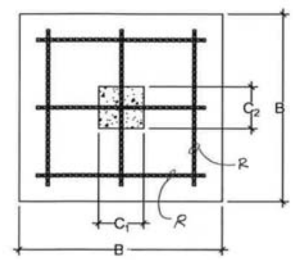
Project: Typical Footing
Footing: 18" x 18" x 8" thick



Isolated footing



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

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

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

$q_u =$ 10392 psf or 10392 psf $P_u = q_u B^2 =$ 23383 #

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_o = 2(C₁+d)+2(C₂+d)
31

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

$V_u \leq \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 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

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

$M_u \leq \emptyset M_n$ $\emptyset M_n =$ 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	14614	14614	14614	14614	14614	14614
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**

Date: 3/19/2018

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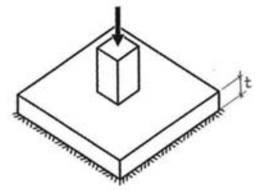
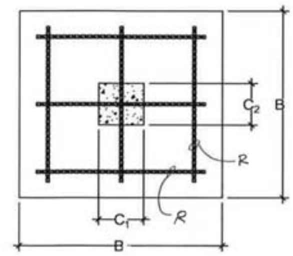
Project: Typical Footing
Footing: 24" x 24" x 8" thick



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
$V_c = 2\lambda\sqrt{f_c'}Bd =$ 10.20 k
$V_u \leq \emptyset V_c$ $\emptyset V_c =$ 7.65 k

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

$q_u =$ 5649 psf or 5649 psf $P_u = q_u B^2 =$ 22597 #

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
[22.6.5.2(c)] $v_c = \left(2+\frac{\alpha_x d}{b_0}\right)\lambda\sqrt{f_c'} =$ 374 psi
b_0 = 2(C_1+d)+2(C_2+d)
31
$V_u \leq \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 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
$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) =$ 5.5 k-ft
$a = A_s f_y/(0.85f'_c B) =$ 0.31 in
$M_u \leq \emptyset M_n$ $\emptyset M_n =$ 4.9 k-ft

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

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

Development of Reinforcement:

$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b =$$ 7 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	14123	14123	14123	14123	14123	14123
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**

Date: 3/19/2018

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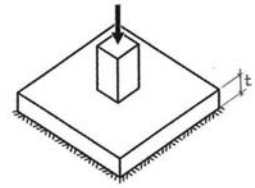
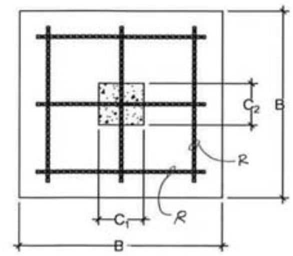
Project: Typical Footing
Footing: 30" x 30" x 8" thick



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

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

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

$q_u =$ 3974 psf or 3974 psf $P_u = q_u B^2 =$ 24838 #

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(C₁+d)+2(C₂+d)
31

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

$V_u \leq \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 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

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

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

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

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

$q_u =$ 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	15524	15524	15524	15524	15524	15524
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**

Date: 3/19/2018

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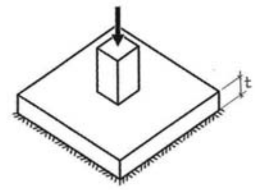
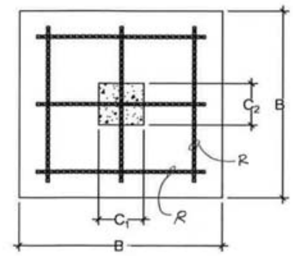
Project: Typical Footing
Footing: 36" x 36" x 12" thick



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

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

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

$q_u =$ 7128 psf or 7128 psf $P_u = q_u B^2 =$ 64152 #

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

$V_u \leq \phi V_c$ $\phi V_c = \phi v_c b_0 d =$ 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

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

$M_u \leq \emptyset M_n$ $\emptyset M_n =$ 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**

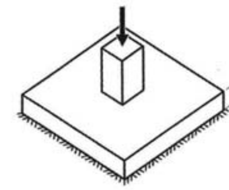
Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Load (lbs), Soil	13140	17640	22140	26640	31140	35640
Max Load (lbs), One-Way Shear	40095	40095	40095	40095	40095	40095
Max Load (lbs), Two-Way Shear	49805	49805	49805	49805	49805	49805
Max Load (lbs), 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: 3/19/2018

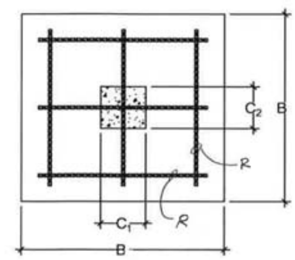
BTL ENGINEERING

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 $\lambda =$ 1.00
f_y = 40000 psi Uncoated $\psi_e =$ 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
$V_c = 2\lambda\sqrt{f'_c}Bd =$ 34.65 k
$V_u \le \emptyset V_c$ $\emptyset V_c =$ 25.99 k

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

$q_u =$ 5606 psf or 5606 psf $P_u = q_u B^2 =$ 68677 #

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

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

$q_u =$ 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
$a = A_s f_y/(0.85f'_c B) =$ 0.36 in
$M_u \le \emptyset M_n$ $\emptyset M_n =$ 19.4 k-ft

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

$q_u =$ 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	42923	42923	42923	42923	42923	42923
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**

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BTL ENGINEERING

19011 Wood-Sno Road NE, Suite 100
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Phone: 425-814-8448
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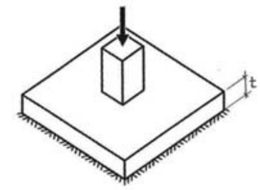
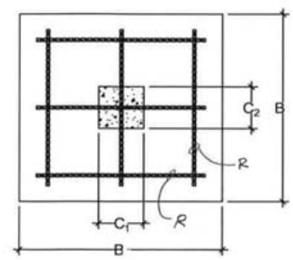
Project: Typical Footing
Footing: 48" x 48" x 12" thick



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: $\emptyset$ = 0.75
$V_c = 2\lambda\sqrt{f_c'}Bd =$ 39.60 k
$V_u \leq \emptyset V_c$ $\emptyset V_c =$ 29.70 k

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

$q_u =$ 4644 psf or 4644 psf $P_u = q_u B^2 =$ 74298 #

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

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

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

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

Moment: $\emptyset$ = 0.90
$M_n = A_s f_y\left(d - {}^{a}/_{2}\right) =$ 26.8 k-ft
$a = A_s f_y/(0.85f'_c B) =$ 0.39 in
$M_u \leq \emptyset M_n$ $\emptyset M_n =$ 24.2 k-ft

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

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

Development of Reinforcement:
$$l_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right)d_b =$$ 12 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	46436	46436	46436	46436	46436	46436
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: 3/19/2018