BTL
ENGINEERING

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Structural Calculations

For

Greisman Remodel

6511 82nd Ave SE
Mercer Island, WA 98040

August 13, 2025

Prepared by

Ryan Hartman
Dane Pollett

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Greisman Remodel
Mercer Island, WA

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Criteria

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Project: Greisman Rmeodel
Project Number: Mercer Island, WA

Code: IBC 2021

	Risk Category	II		
Earthquake:	Site Class	D		
		I_e = 1.00		R = 6.5
		S_S = 1.465		Ω_0 = 3.0
		S_1 = 0.507		C_d = 4.0
		ρ = 1.00		
Wind:	Basic Design Wind Speed, *V*		100 MPH	
	Exposure		B	
	Topographic Factor	K_{ZT} = 1.25		

Soil Bearing: 1500-psf Allowable Soil Bearing Pressure

@ existing house
(3 ton per 2" Pipe Pile @ new deck)

Concrete: 2500-psi Concrete Strength
Higher strength may be used, but special inspection and testing reports not req

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 - Asphalt Shingles	2.0 psf
	Sheathing - 7/16 OSB	2.2 psf
	Framing - Trusses @ 24"oc	2.5 psf
	Insulation - Batt.	1.0 psf
	Ceiling - 5/8 GWB	2.8 psf
	PV Panel / Misc	4.5 psf
	Total	**15 psf**
Deflection	L/360 Live Load, L/240 Total Load	

Floor Framing:

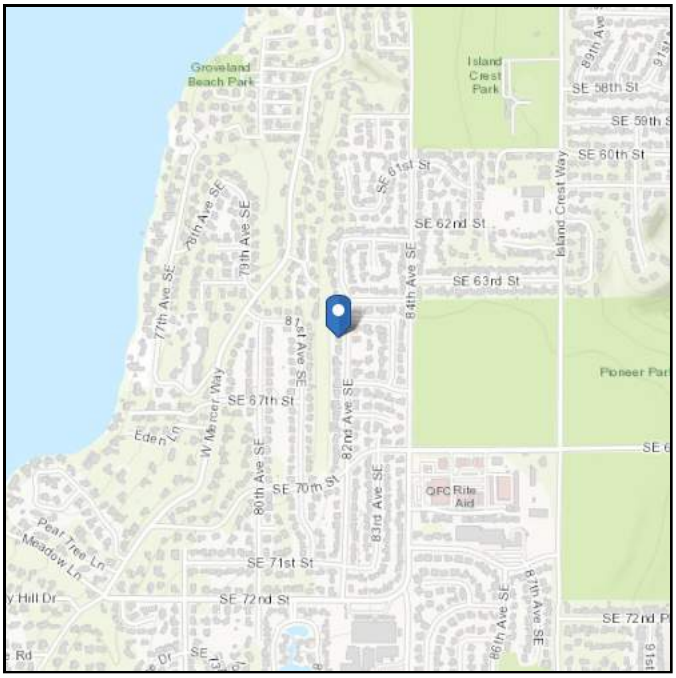
Live Load	Residential	40 psf
	Decks	60 psf
Dead Load - Floor	Finish Floor - Allowance	4.0 psf
	Sheathing - 3/4 Plywood/Edge Gold	2.5 psf
	Framing -2x10 @ 12"oc	3.75 psf
	Ceiling - 5/8 GWB	2.75 psf
	Misc.	2.0 psf
	Total	**15 psf**
Dead Load - Deck	Decking - Trex	7.5 psf
	Framing -2x8 @ 12"oc	3.0 psf
	Misc.	4.5 psf
	Total	**15 psf**
Deflection	L/480 Live Load, L/240 Total Load	

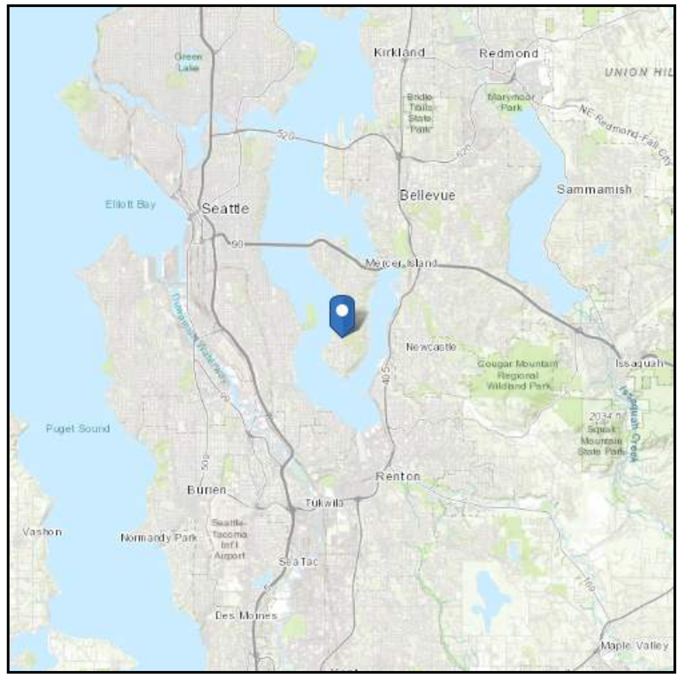
Date: 6/26/2025

ASCE Hazards Report

Address:
6511 82nd Ave SE
Mercer Island, Washington
98040

Standard:	ASCE/SEI 7-16	**Latitude:**	47.544738
Risk Category:	II	**Longitude:**	-122.229607
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	290.5672021163916 ft (NAVD 88)





Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	74 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Wed Apr 02 2025

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

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

Results:

S_S :	1.465	S_{D1} :	N/A
S_1 :	0.507	T_L :	6
F_a :	1.2	PGA :	0.627
F_v :	N/A	PGA_M :	0.753
S_{MS} :	1.758	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.172	C_v :	1.393

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

Data Accessed: Wed Apr 02 2025

Date Source: USGS Seismic Design Maps


47.54924, -122.23791
16 ft WGS84
Site: 6511 82nd Ave SE
Mercer Island, WA 98040
Exp B, B prevails for greater than 1500'
Kzt=1.25 (see C1.5)
H/2=140ft
Site=280ft
1700 ft
L=2200'
H=320ft

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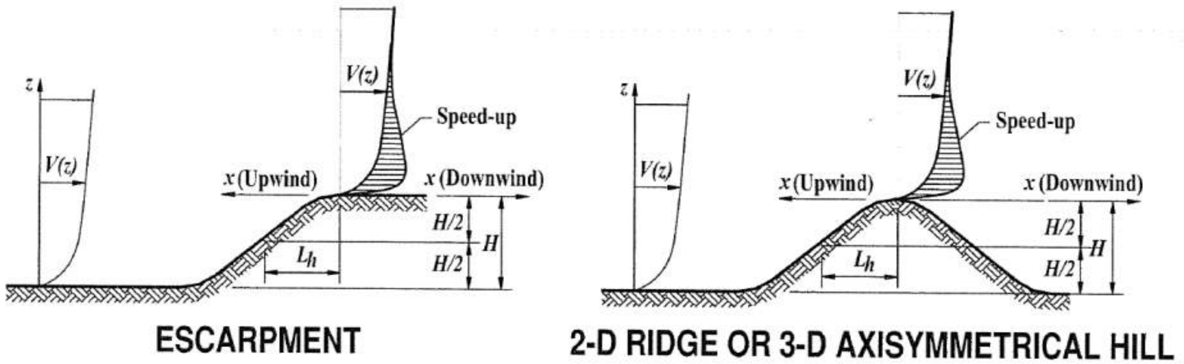
Kzt (Profile 1)

ASCE 7-16

Figure 26.8-1



K_{zt} = $(1 + K_1 K_2 K_3)^2$ (26.8-1)

$K_1/(H/L_h)$ = 0.75

K_1 = 0.15

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

μ = 1.5

K_2 = 0.78

$K_3 = e^{-\gamma z/L_h}$

y = 2.5

K_3 = 0.96

K_{zt} = (1 + 0.15 x 0.78 x 0.96)^2

Calculated:

K_{zt} =	1.25

Include exceptions:

K_{zt} =	NA

K_{zt} to use:

K_{zt} =	1.25

SPOT ELEVATIONS:

	Top of hill	Mid point	Bot of hill	Site
Elevation	320 ft	161 ft	1 ft	280 ft
Distance	2200 ft	Lh = 1550 ft	1 ft	1700 ft

Exposure =	Exposure B	
Hill type =	2-dimensional escarpments	
X refers to distance =	Upwind	(From the crest)
x =	-500 ft	(Distance from the crest to the site)
L_h =	1550 ft	(Distance upwind of crest to midpoint)
H =	319 ft	(Overall height of crest)
z =	25 ft	(Structure height above ground)

CHECK EXCEPTIONS:

(1)	Is the feature isolated from other similar features by at least: 2 miles? (100H or 2mi)		TRUE
(2)	The feature protrudes above the height of neighboring (within 2mi) features by at least 2x?		TRUE
(3)	Site Located in upper half of hill?		TRUE
(4)	H/Lh >= 0.2?	H/Lh =319/1550 = 0.2058	TRUE
(5)	H >= 60ft?	H = 319 ft >60ft	TRUE

KZT CALCULATION IS APPLICABLE: TRUE

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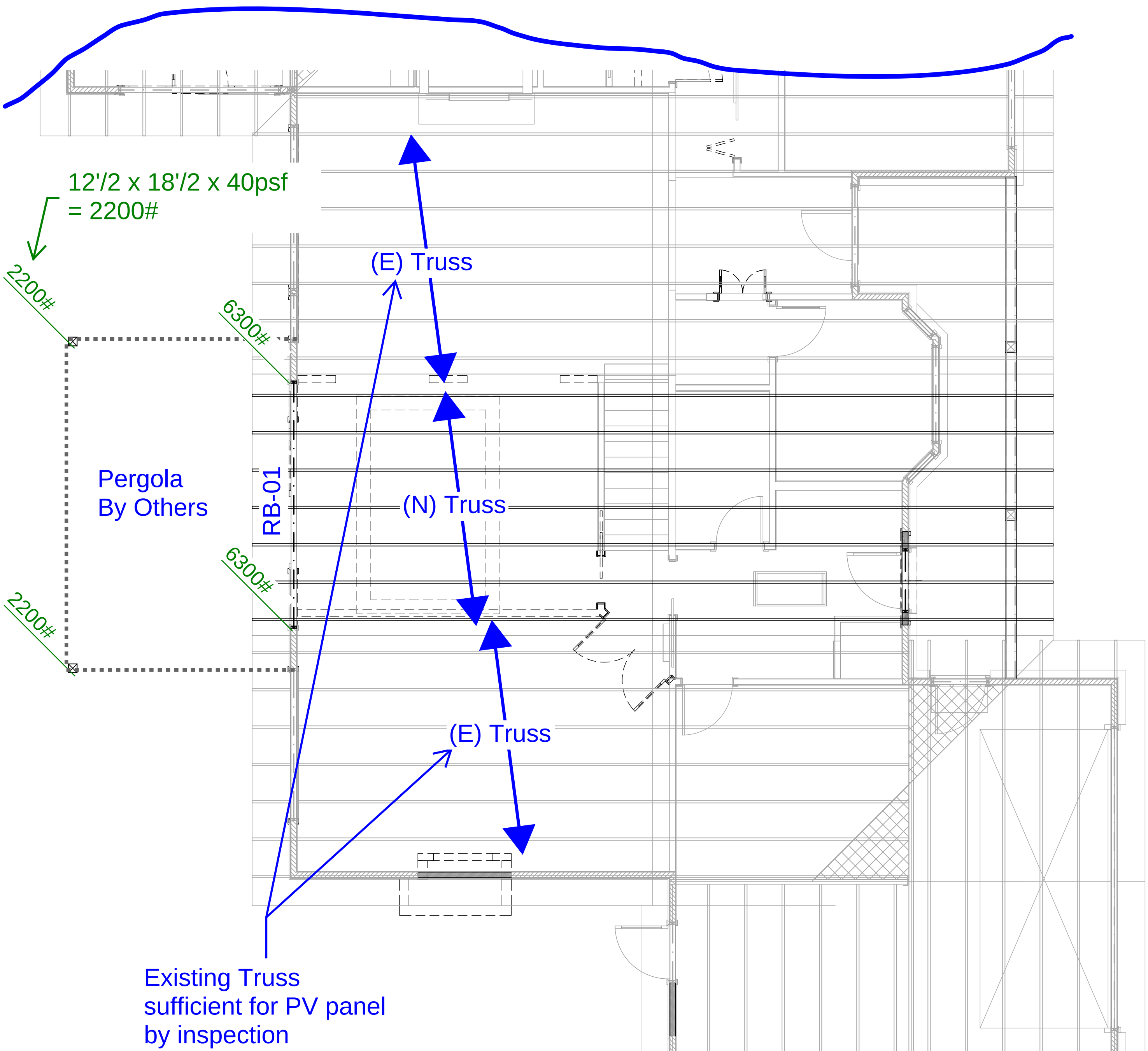
Gravity

Roof Framing



Roof Framing Key Plan

R1.1

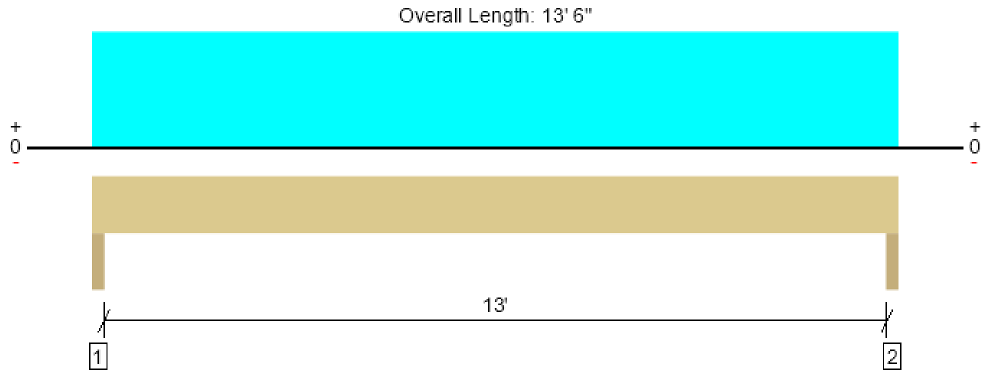
FORTEWEB

MEMBER REPORT

PASSED

Roof, RB-01

1 piece(s) 3 1/2" x 11 7/8" 2.2E Parallam® PSL



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6298 @ 1 1/2"	6563 (3.00")	Passed (96%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	5141 @ 1' 2 7/8"	9241	Passed (56%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	20475 @ 6' 9"	22888	Passed (89%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.403 @ 6' 9"	0.442	Passed (L/395)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.654 @ 6' 9"	0.663	Passed (L/243)	--	1.0 D + 1.0 S (All Spans)

Member Length : 13' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Snow	Total	
1 - Trimmer - SPF	3.00"	3.00"	2.88"	2416	3881	6297	None
2 - Trimmer - SPF	3.00"	3.00"	2.88"	2416	3881	6297	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	13' 6" o/c	
Bottom Edge (Lu)	13' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 13' 6"	N/A	13.0	--	
1 - Uniform (PSF)	0 to 13' 6"	23'	15.0	25.0	Roof + Pergola

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
Dane Pollett BTL Engineering (360) 707-1687 dane.pollett@btleng.net	

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R2.1

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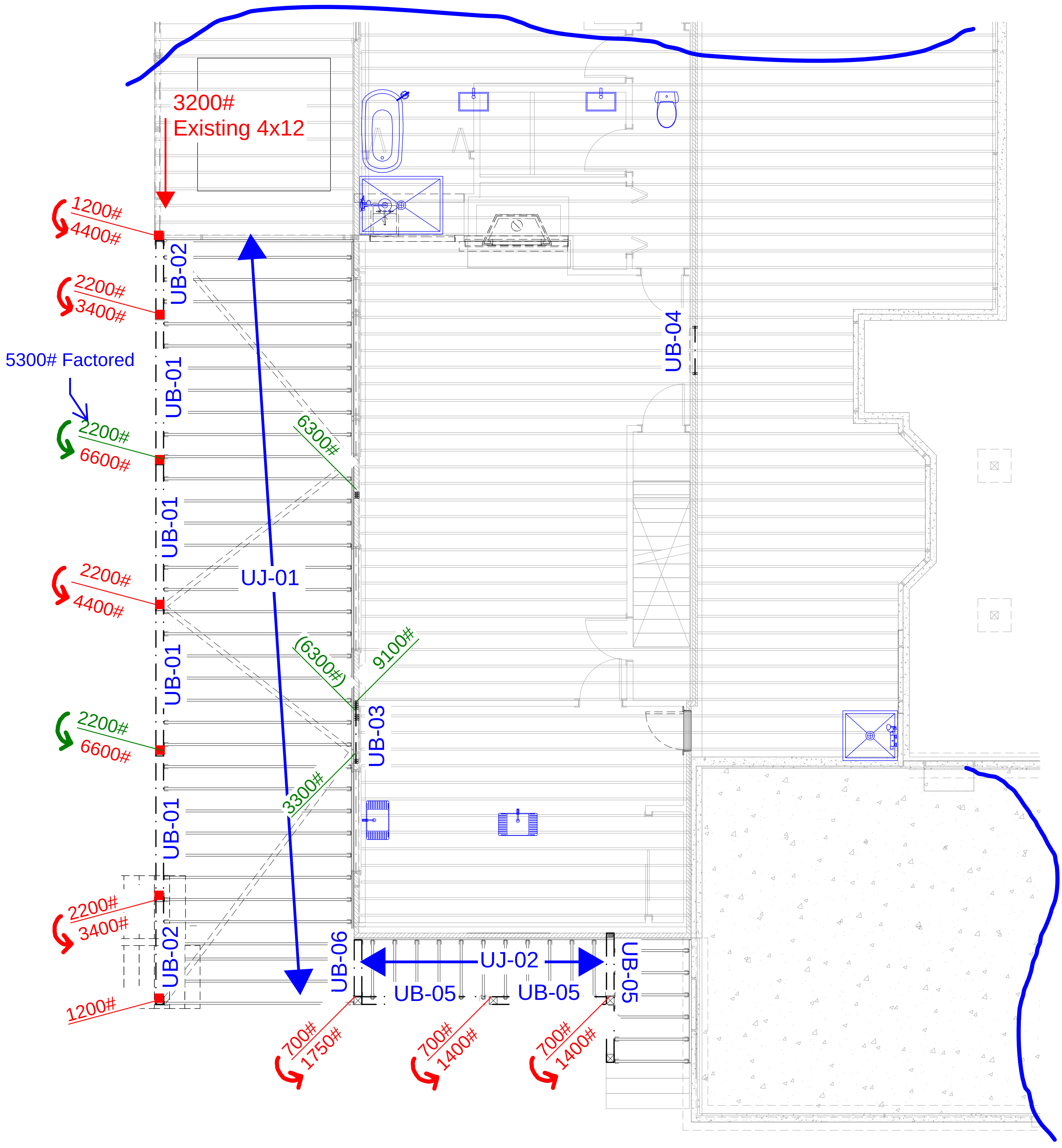
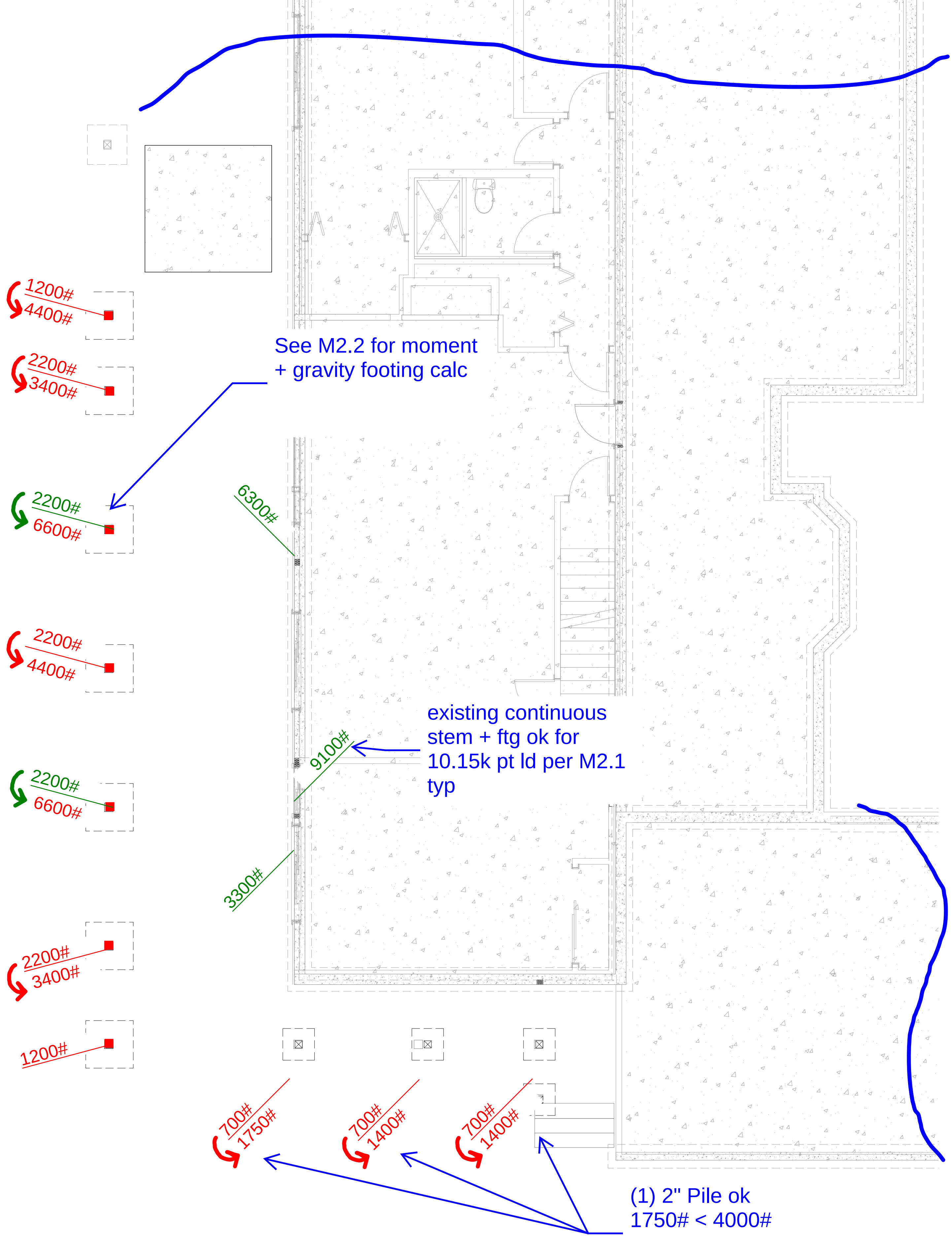
Gravity

Upper Floor Framing



Upper Floor Framing Key Plan

U1.1



Upper Floor Framing Key Plan

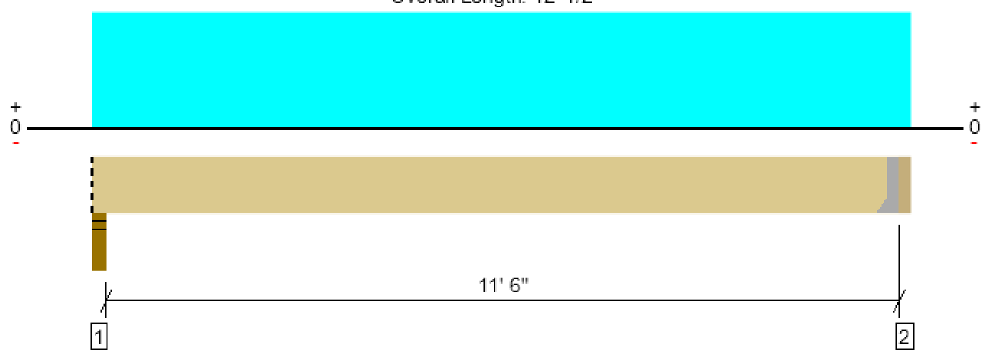
MEMBER REPORT

PASSED

Upper, UJ-01

1 piece(s) 2 x 8 DF No.1 @ 12" OC

P.T



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	434 @ 11' 9 1/2"	1406 (1.50")	Passed (31%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	389 @ 11' 2 1/4"	1305	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1258 @ 6'	1511	Passed (83%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.300 @ 6'	0.386	Passed (L/463)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.375 @ 6'	0.579	Passed (L/371)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 11' 9 1/2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member us
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

ok with incising factor

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	90	360	450	Blocking
2 - Hanger on 7 1/4" SPF beam	3.00"	Hanger[1]	1.50"	91	363	454	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	7' 1" o/c	
Bottom Edge (Lu)	11' 10" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 12' 1/2"	12"	15.0	60.0	Deck

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ForteWEB Software Operator	Job Notes
Dane Pollett BTL Engineering (360) 707-1687 dane.pollett@btleng.net	

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ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3
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MEMBER REPORT

PASSED

Upper, UJ-02

1 piece(s) 2 x 8 HF No.2 @ 16" OC

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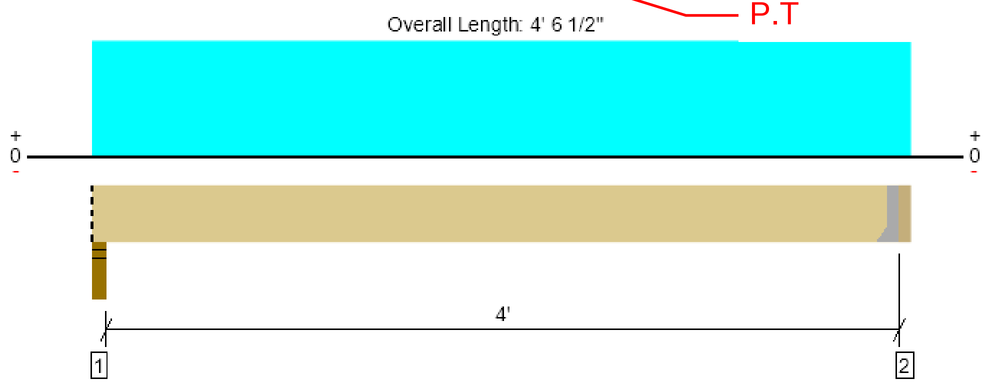

Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	**Actual @ Location**	**Allowed**	**Result**	**LDF**	**Load: Combination (Pattern)**
Member Reaction (lbs)	204 @ 4' 3 1/2"	911 (1.50")	Passed (22%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	144 @ 3' 8 1/4"	1088	Passed (13%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	208 @ 2' 3"	1284	Passed (16%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.008 @ 2' 3"	0.102	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.010 @ 2' 3"	0.204	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	N/A	N/A	N/A	--	N/A

Member Length : 4' 3 1/2"
System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- A 15% increase in the moment capacity has been added to account for repetitive member usag
- Applicable calculations are based on NDS.
- No composite action between deck and joist was considered in analysis.

ok with incising factor

Supports	**Bearing Length**			**Loads to Supports (lbs)**			**Accessories**
	Total	**Available**	**Required**	**Dead**	**Floor Live**	**Total**	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	45	180	225	Blocking
2 - Hanger on 7 1/4" SPF beam	3.00"	Hanger[1]	1.50"	46	183	229	See note [1]

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- [1] See Connector grid below for additional information and/or requirements.

Lateral Bracing	**Bracing Intervals**	**Comments**
Top Edge (Lu)	4' 4" o/c	
Bottom Edge (Lu)	4' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	**Model**	**Seat Length**	**Top Fasteners**	**Face Fasteners**	**Member Fasteners**	**Accessories**
2 - Face Mount Hanger	LU26	1.50"	N/A	6-10dx1.5	4-10dx1.5	

- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	**Location (Side)**	**Spacing**	**Dead (0.90)**	**Floor Live (1.00)**	**Comments**
1 - Uniform (PSF)	0 to 4' 6 1/2"	16"	15.0	60.0	Deck

Weyerhaeuser Notes

ForteWEB Software Operator	**Job Notes**
Dane Pollett BTL Engineering (360) 707-1687 dane.pollett@btleng.net	

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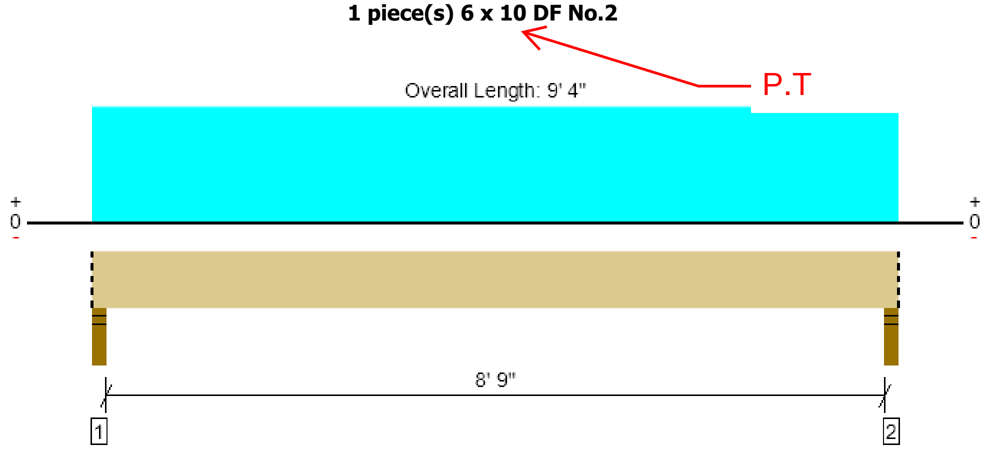
MEMBER REPORT

PASSED

Upper, UB-01

1 piece(s) 6 x 10 DF No.2



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2162 @ 2"	8181 (3.50")	Passed (26%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1660 @ 1' 1"	5922	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4690 @ 4' 8"	6032	Passed (78%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.104 @ 4' 8"	0.300	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.134 @ 4' 8"	0.450	Passed (L/807)	--	1.0 D + 1.0 L (All Spans)

Member Length : 9' 4"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	482	1680	2162	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	482	1680	2162	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	9' 4" o/c	
Bottom Edge (Lu)	9' 4" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 9' 4"	N/A	13.2	--	
1 - Uniform (PSF)	0 to 9' 4" (Front)	6'	15.0	60.0	Deck

• Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

File Name: Greisman Remodel

FORTEWEB

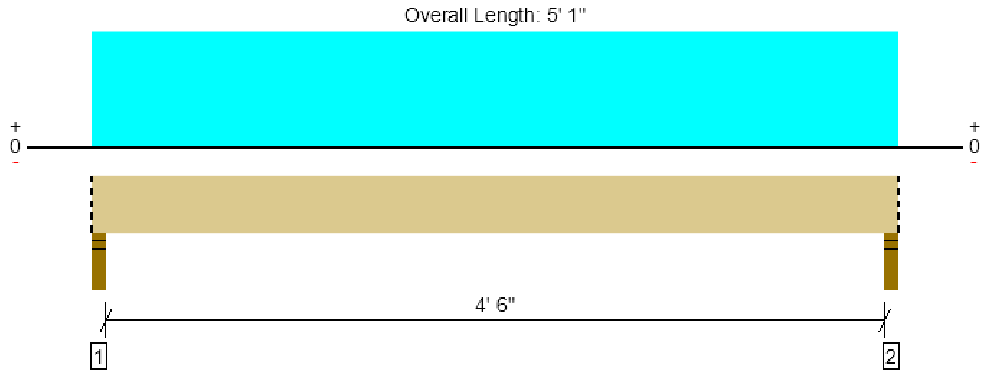
MEMBER REPORT

PASSED

Upper, UB-02

1 piece(s) 6 x 10 DF No.2

P.T



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Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1177 @ 2"	8181 (3.50")	Passed (14%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	676 @ 1' 1"	5922	Passed (11%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1306 @ 2' 6 1/2"	6032	Passed (22%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.008 @ 2' 6 1/2"	0.158	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.010 @ 2' 6 1/2"	0.237	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Lumber grading provisions must be extended over the length of the member per NDS 4.2.5.5.
- Applicable calculations are based on NDS.

Member Length : 5' 1"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	3.50"	1.50"	262	915	1177	Blocking
2 - Stud wall - SPF	3.50"	3.50"	1.50"	262	915	1177	Blocking

• Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 1" o/c	
Bottom Edge (Lu)	5' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 5' 1"	N/A	13.2	--	
1 - Uniform (PSF)	0 to 5' 1" (Front)	6'	15.0	60.0	Deck

• Side loads are assumed to not induce cross-grain tension.

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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U2.4

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MEMBER REPORT

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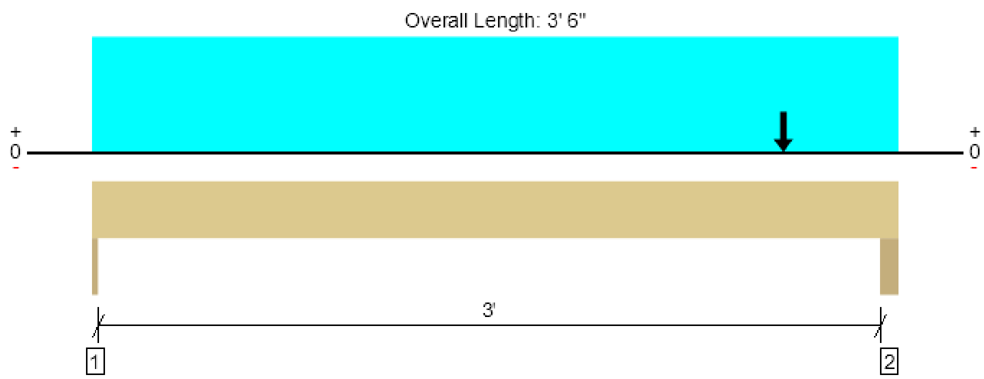
Upper, UB-03

1 piece(s) 4 x 8 DF No.2 alt: 3-1/2x7-1/2 GLB



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2784 @ 0	3281 (1.50")	Passed (85%)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Shear (lbs)	2326 @ 2' 6 1/4"	3502	Passed (66%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Moment (Ft-lbs)	2652 @ 1' 10 7/8"	3438	Passed (77%)	1.15	1.0 D + 0.75 L + 0.75 S (All Spans)
Live Load Defl. (in)	0.017 @ 1' 8 7/16"	0.108	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)
Total Load Defl. (in)	0.029 @ 1' 8 7/16"	0.162	Passed (L/999+)	--	1.0 D + 0.75 L + 0.75 S (All Spans)

Member Length : 3' 6"
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Floor Live	Snow	Total	
1 - Trimmer - SPF	1.50"	1.50"	1.50"	1143	1138	1050	3331	None
2 - Trimmer - SPF	4.50"	4.50"	3.56"	3334	1313	4450	9097	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	3' 6" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Snow (1.15)	Comments
0 - Self Weight (PLF)	0 to 3' 6"	N/A	6.4	--	--	
1 - Uniform (PSF)	0 to 3' 6"	10'	15.0	40.0	--	Upper
2 - Uniform (PLF)	0 to 3' 6"	N/A	80.0	--	--	Wall
3 - Uniform (PSF)	0 to 3' 6"	18' 6"	15.0	--	25.0	Roof
4 - Point (lb)	3'	N/A	2416	--	3881	Linked from: RB-01, Support 2
5 - Uniform (PSF)	0 to 3' 6"	5'	15.0	60.0	--	Deck

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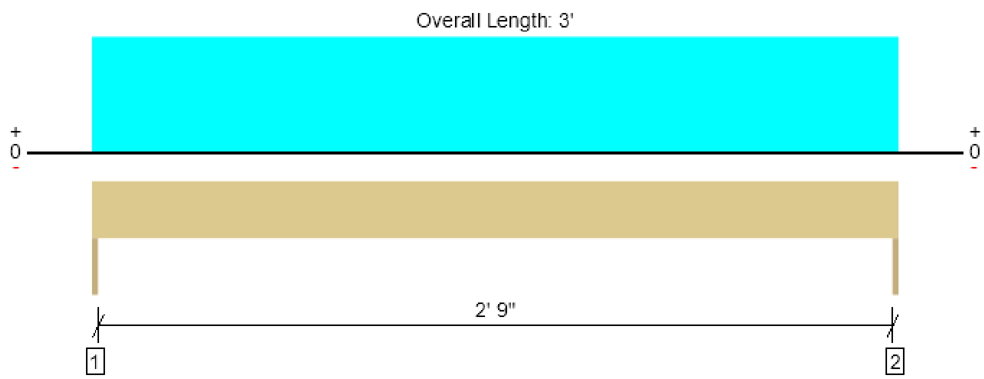
Upper, UB-04

1 piece(s) 4 x 6 DF No.2



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1245 @ 0	3281 (1.50")	Passed (38%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	761 @ 7"	2310	Passed (33%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	934 @ 1' 6"	1720	Passed (54%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.014 @ 1' 6"	0.100	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.019 @ 1' 6"	0.150	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 3'
System : Wall
Member Type : Header
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Trimmer - SPF	1.50"	1.50"	1.50"	345	900	1245	None
2 - Trimmer - SPF	1.50"	1.50"	1.50"	345	900	1245	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' o/c	
Bottom Edge (Lu)	3' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 3'	N/A	4.9	--	
1 - Uniform (PSF)	0 to 3'	15'	15.0	40.0	Upper

Weyerhaeuser Notes

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7/2/2025 8:41:26 PM UTC

ForteWEB v3.9, Engine: V8.4.3.94, Data: V8.1.7.3

File Name: Greisman Remodel

FORTEWEB

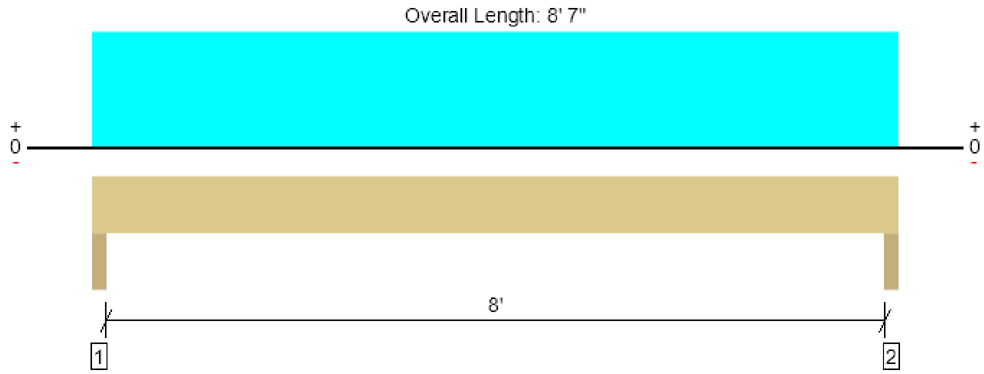
MEMBER REPORT

PASSED

Upper, UB-05

1 piece(s) 6 x 8 DF No.2

P.T



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	689 @ 2"	8181 (3.50")	Passed (8%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	542 @ 11"	4675	Passed (12%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1365 @ 4' 3 1/2"	3222	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.050 @ 4' 3 1/2"	0.275	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.067 @ 4' 3 1/2"	0.412	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Member Length : 8' 7"
System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Beam - SPF	3.50"	3.50"	1.50"	174	515	689	None
2 - Beam - SPF	3.50"	3.50"	1.50"	174	515	689	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 7" o/c	
Bottom Edge (Lu)	8' 7" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 8' 7"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 8' 7" (Front)	2'	15.0	60.0	Deck

• Side loads are assumed to not induce cross-grain tension.

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

MEMBER REPORT

PASSED

Upper, UB-06

1 piece(s) 6 x 8 DF No.2

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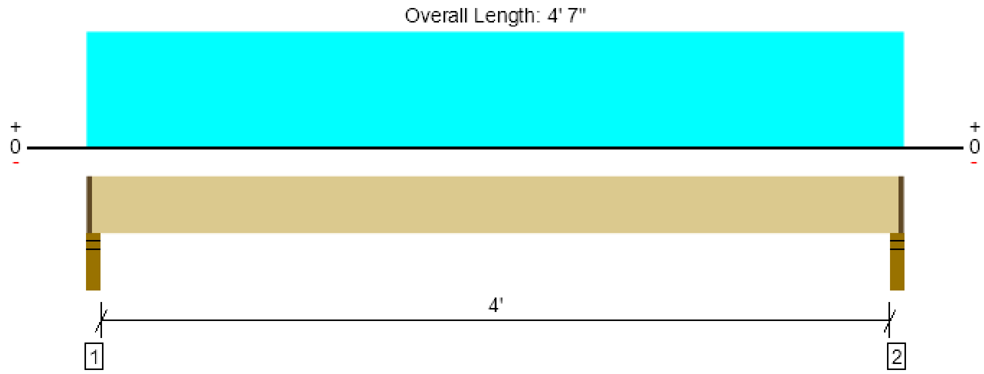

Drawing is Conceptual. All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal (typ.).

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1007 @ 2"	5259 (2.25")	Passed (19%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	633 @ 11"	4675	Passed (14%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1040 @ 2' 3 1/2"	3222	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.011 @ 2' 3 1/2"	0.106	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.013 @ 2' 3 1/2"	0.213	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)

Member Length : 4' 4 1/2"
System : Floor
Member Type : Flush Beam
Building Use : Residential
Building Code : IBC 2021
Design Methodology : ASD

- Deflection criteria: LL (L/480) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.50"	229	825	1054	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.50"	229	825	1054	1 1/4" Rim Board

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 5" o/c	
Bottom Edge (Lu)	4' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	1 1/4" to 4' 5 3/4"	N/A	10.4	--	
1 - Uniform (PSF)	0 to 4' 7" (Front)	6'	15.0	60.0	Deck

- Side loads are assumed to not induce cross-grain tension.

Weyerhaeuser Notes

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Lateral
Forces

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Greisman Remodel
Mercer Island, WA

Revision Date: 1/23/2023

Criteria

Code:	2021 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 =	1.465	(Mapped)
Spectral Response, 1-s Period	S_1 =	0.507	(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 =	1.79	*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} =	1.25	

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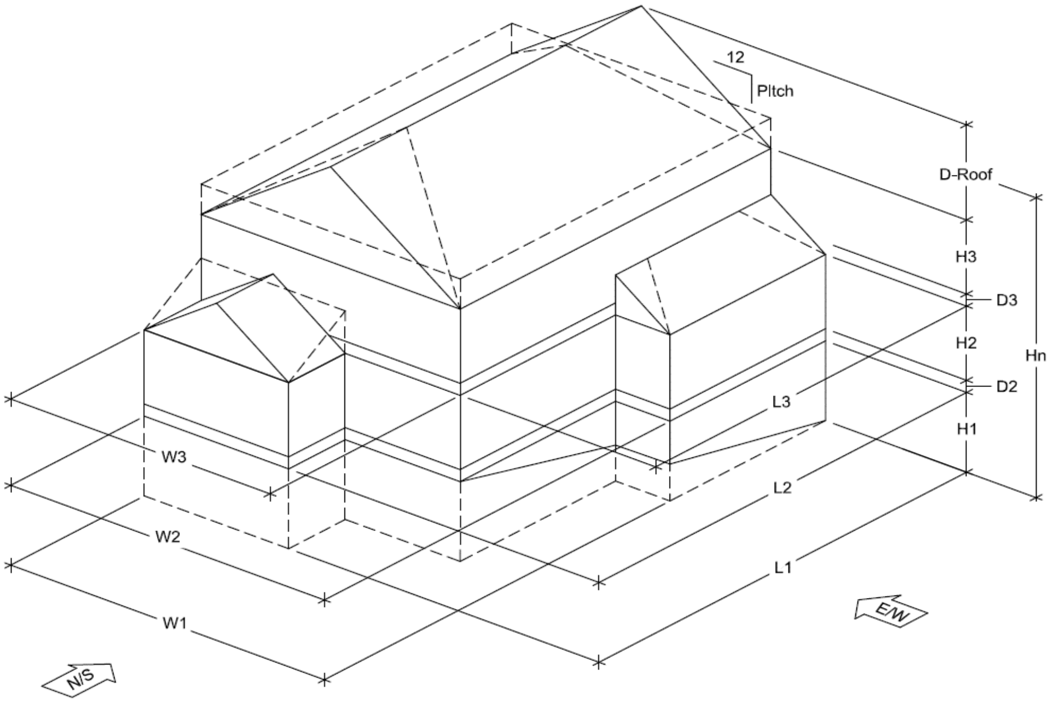
Greisman Remodel
Mercer Island, WA

Revision Date: 1/23/2023



Roof		
Geometry		
Mean Roof Height	Hn =	20.5 ft
Roof Depth	D-Roof =	7 ft
Overhang Length		24 in
Pitch		6:12
Floor 1		
Geometry		
Width	W2 =	75 ft
Length	L2 =	56 ft
Plate Height	H2 =	8 ft
Floor Depth	D2 =	12 in
Basement		
Geometry		
Width	W1 =	64 ft
Length	L1 =	39 ft
Plate Height	H1 =	8.0 ft

Seismic Weight - Roof				
Roof Area 1	3050 SF	15 psf		45,750#
Roof Area 2	560 SF	8 psf		4,480#
Roof Area 3				
Exterior Wall 1	280 LF	4 ft	10 psf	11,200#
Exterior Wall 2				
Exterior Wall 3				
Interior Wall1	200 LF	4 ft	8 psf	6,400#
Interior Wall2				
			Total	67,830#
Seismic Weight - Floor 1				
Roof Area 1				
Floor Area 1	2000 SF	15 psf		30,000#
Floor Area 2				
Floor Area 3				
Exterior Wall 1	280 LF	4 ft	10 psf	11,200#
Exterior Wall 2	105 LF	4 ft	10 psf	4,200#
Exterior Wall 3				
Interior Wall1	200 LF	4 ft	8 psf	6,400#
Interior Wall2	60 LF	4 ft	8 psf	1,920#
			Total	53,720#

N/S Projected Area - Roof	
Sloped Roof Area	400 SF
Gable/Parapet Area	50 SF
Wall Area	300 SF
E/W Projected Area - Roof	
Sloped Roof Area	100 SF
Gable/Parapet Area	135 SF
Wall Area	224 SF
N/S Projected Area - Floor 1	
Sloped Roof Area	
Gable/Parapet Area	
Wall Area	
E/W Projected Area - Floor 1	
Sloped Roof Area	
Gable/Parapet Area	
Wall Area	345 SF

Date: 7/21/2025

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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)*
$= 1.758$

$S_{M1} = F_v S_1$ *(Eq. 11.4-2)*
$= 0.909$

$S_{DS} = ⅔\, S_{MS}$ *(Eq. 11.4-3)*
$= 1.172$

$S_{D1} = ⅔\, S_{M1}$ *(Eq. 11.4-4)*
$= 0.606$

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), $h_n = 21$ ft
Approximate Fundamental Period, $T_a = C_t\,(h_n)^x$ *(Eq. 12.8-7)*
$T = T_a = 0.19$
$T_L = 6$ *(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)

$C_s = 0.180$

The total design base shear need not exceed:

(Eq. 12.8-3)

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

$C_s = 0.484$

(Eq. 12.8-4)

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

$C_s = 15.068$

$C_s = 0.484$ *$T \le TL$*
$C_s = 0.726$ *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)
$C_s = 0.052$

nor where $S_1 \geq 0.6g$:

$C_s = 0.5 S_1/(R/I_e)$ (Eq. 12.8-6)
$C_s = 0.000$

$C_s = 0.180$
$V = 0.180\,W$

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Greisman Remodel | Revision Date: 1/23/2023
Mercer Island, WA

$\rho_S = \lambda K_{ZT} \rho_{S30}$	(28.5-1)	Exposure =	B
λ = 1.00	(Fig. 28.5.1)	Mean Roof Ht hn (ft) =	21 ft
K_{ZT} = 1.25	(Section 26.8)	a (roof) =	3.0 ft
		a (upper/main floor) =	3.9 ft
		Basic Wind Speed =	100 mph
		Roof Angle =	27

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}	24	19.1	19.1	23.8	572	343	384	230
	Agable	21	19.1	19.1	23.8	500	300	336	202
	B	42	6.8	6.8	8.5	357	214	336	202
	Cwall	276	14.3	14.3	17.9	4940	2964	4416	2650
	Cgable	29	14.3	14.3	17.9	519	311	464	278
	D	358	5.9	5.9	7.4	2640	1584	2864	1718
	Total Area =	750			Total Load =	9529	5717	8800	5280
					Design:	***9529***	***5717***		

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	24	19.1	19.1	23.8	572	343	384	230
	Agable	21	19.1	19.1	23.8	500	300	336	202
	B	42	6.8	6.8	8.5	357	214	336	202
	Cwall	200	14.3	14.3	17.9	3580	2148	3200	1920
	Cgable	114	14.3	14.3	17.9	2041	1224	1824	1094
	D	58	5.9	5.9	7.4	428	257	464	278
	Total Area =	459			Total Load =	7477	4486	6544	3926
					Design :	***7477***	***4486***		
	Zone	Area	p_{S30} (psf)	$p_{S30\,design}$ (psf)	ρ (psf)	*Force (#)*	*ASD Force (#)*	*Force (#)*	*ASD Force (#)*
Floor 1	Awall	59	19.1	19.1	23.8	1408	845	946	567
	Agable	27	19.1	19.1	23.8	650	390	437	262
	B	0	6.8	6.8	8.5	0	0	0	0
	Cwall	286	14.3	14.3	17.9	5118	3071	4574	2745
	Cgable	108	14.3	14.3	17.9	1928	1157	1723	1034
	D	0	5.9	5.9	7.4	0	0	0	0
	Total Area =	480			Total Load =	9104	5462	7680	4608
					Design :	***9104***	***5462***		

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Vertical Distribution of Lateral Forces

Base Shear:

$V =$ 21.92 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	11.5	67.83	780	12.23	12.23	141	68	15.90
Floor 1	-	53.72	-	9.69	21.92	-	122	12.59

Totals W = 121.55 Kips

$\sum w_x h_x =$ 780 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	9.53	9.53	7.48	7.48
Floor 1	-	-	9.10	16.58

	Diaphragm (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Roof	11.13	5.72	4.49
Floor 1	8.81	-	5.46

	Shear Walls (ASD)		
	Seismic, [0.7E] (kips)	Wind N/S [0.6W] (kips)	Wind E/W [0.6W] (kips)
Floor 1	8.56	5.72	4.49
Basement	6.78	-	5.46

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Lateral

Shear Walls/Diaphragms

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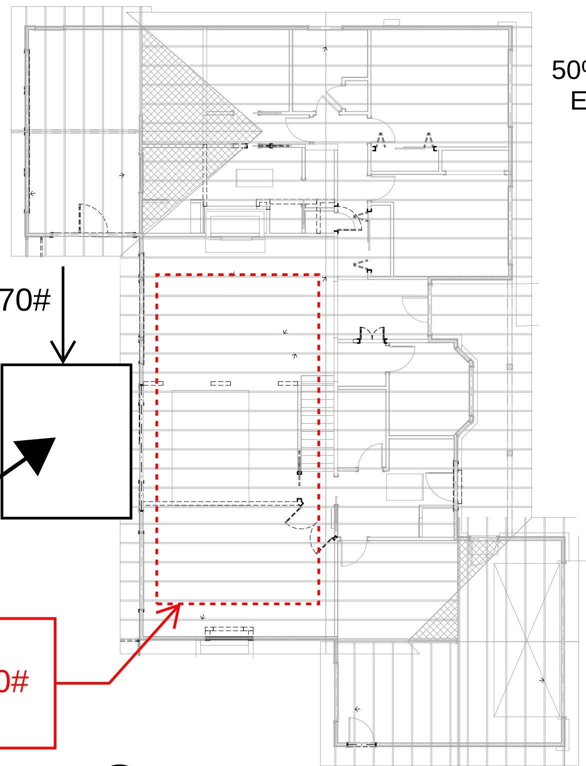
Upper Floor Shear Forces

N

E = 8560#
W = 4490#



50% Res + 50% Pergola/Solar
E = 4280# + 380# = 4660#
W = 2860#

E = 8560#
W = 5720#

E = 270#

E = 270#

Pergola : W=1500#
V= .18 x W = 270#

No change to existing capacity, demand change = 9%
No New Analysis Req'd Per IEBC1103.2 Exception 2

Solar:
W= 4.5 psf x 600 sf = 2700#
V = 485#

50% Res + 50% Pergola/Solar
E = 4280# + 380# = 4660#
W = 2860#

2

50% Residence + 100% Pergola/Solar
E = 4280# + 755# = 5035#
W = 2245#

50% Res
E = 4280#
W = 2245#

no change to existing capacity or demand

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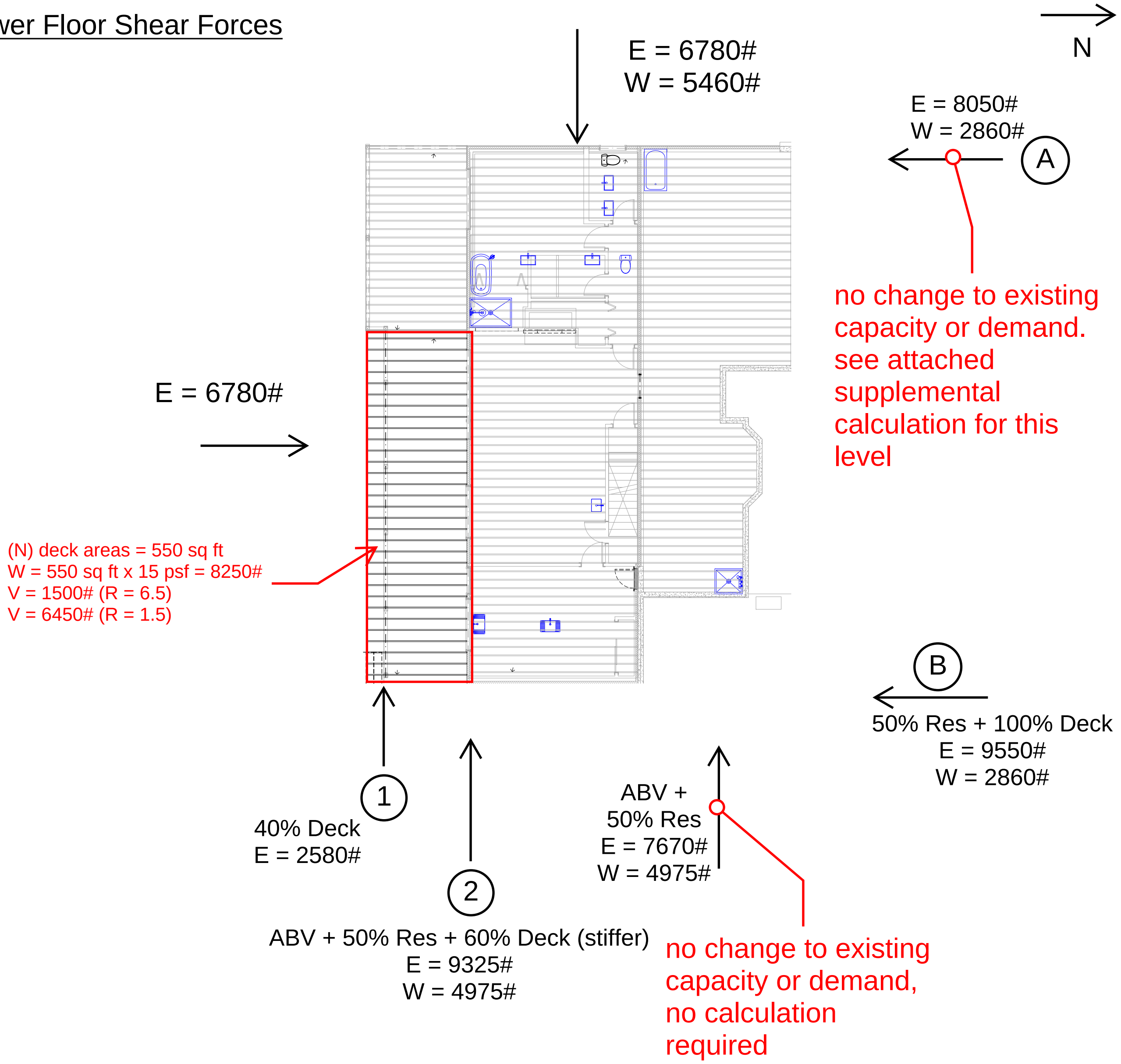
Lower Floor Shear Forces



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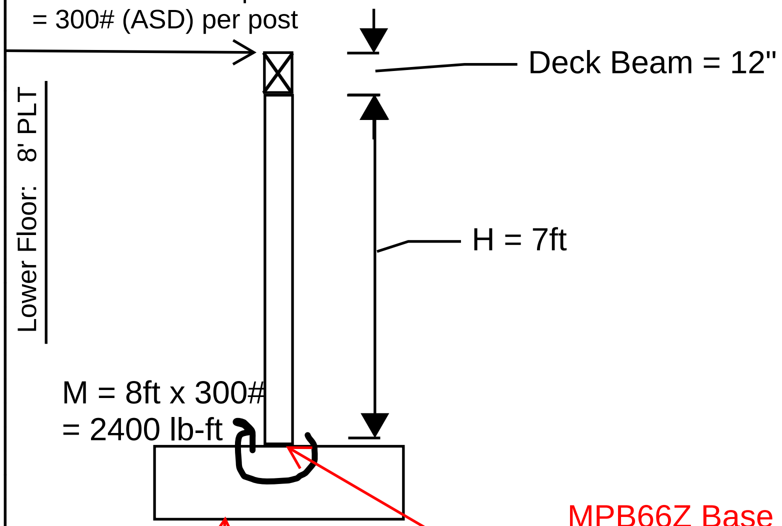
Shear Wall Line 1

E = 2580# x .7 / 6 posts
= 300# (ASD) per post



MPB66Z Base Sufficient
with Reinforced Footing
2400 lb-ft < 2795 lb-ft

See M2.3 for footing
gravity + moment
calculations

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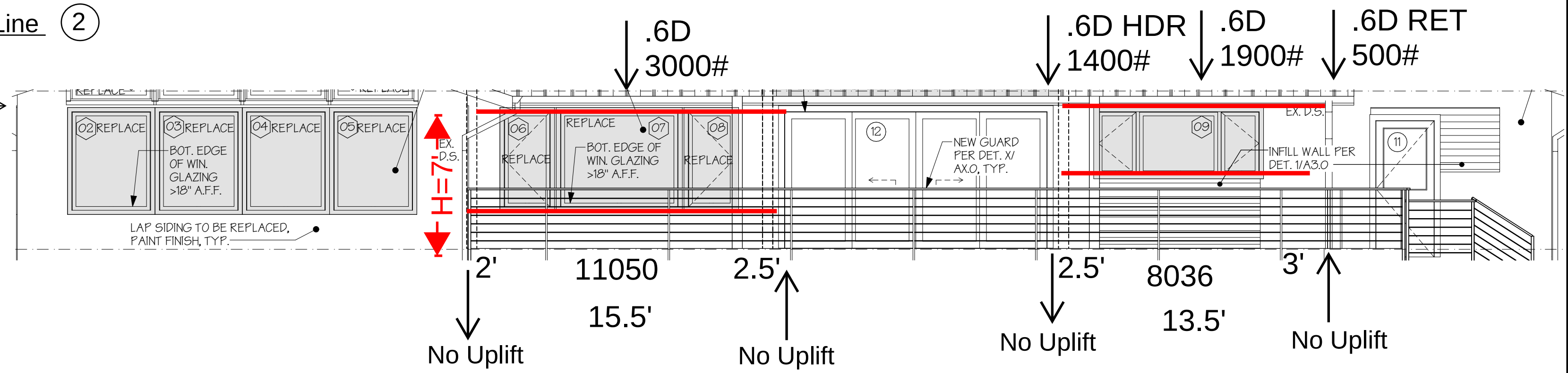
Shear Wall Line (2)

E = 5035#
W = 2245#

Upper Floor: 8' PLT



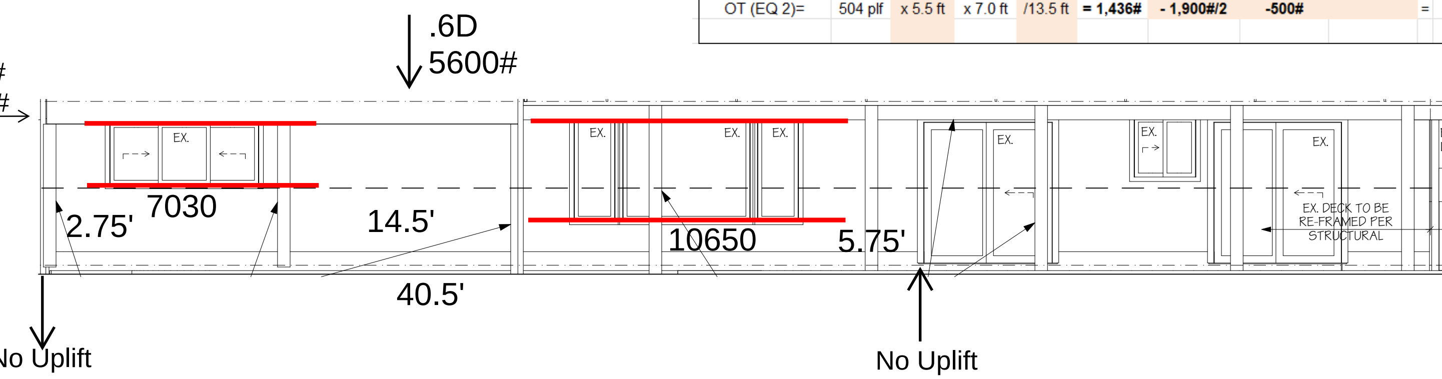
Dead: 10psf x 8ft x 15.5ft + 15psf x 16ft x 15.5ft = 5000#
Dead: 10psf x 8ft x 13.5ft + 15psf x 10ft x 13.5ft = 3100#

Dead: 10psf x 8ft x 40.5ft + 15psf x 10ft x 40.5ft = 9300#

8 ft	[Plate Height]	= 10.0 ft	[Total pier widths]	=2+2.5+2.5+3						
2 ft	[FTAO Min Panel Width]	5 ft	[FTAO Max Panel Height]	FTAO	[Analysis type]					
Ve =	5.04k	/ 10 ft	=	503.5 plf	==>	P1-2 SW OK (Vert OK)				
Vw =	2.25k	/ 10 ft	=	224.5 plf	==>	P1-2 SW OK				
SW Capacity x 1.25-0.125*(h/b)=	(E)[553 plf]	(W)[769 plf]	[Reduced Capacity]				Overturning Load			
OT (EQ 1)=	504 plf	x 4.5 ft	x 7.0 ft	/15.5 ft	= 1,023#	- 3,000#/2		=	-477#	No Net Uplift OK
OT (EQ 2)=	504 plf	x 5.5 ft	x 7.0 ft	/13.5 ft	= 1,436#	- 1,900#/2	-500#	=	-14#	No Net Uplift OK

E = 9325#
W = 4975#

Lower Floor: 8' PLT



8 ft	[Plate Height]	= 23.0 ft	[Total pier widths]	=2.75+14.5+5.75					
2.8 ft	[FTAO Min Panel Width]	5 ft	[FTAO Max Panel Height]	FTAO	[Analysis type]				
Ve =	9.33k	/ 23 ft	=	405.4 plf	==>	P1-2 SW OK Vert OK			
Vw =	4.98k	/ 23 ft	=	216.3 plf	==>	P1-2 SW OK			
SW Capacity =	(E)[590 plf]	(W)[820 plf]	[Unreduced Capacity]			Overturning Load			
OT (EQ)=	405 plf	x 23.0 ft	x 8.0 ft	/40.5 ft	= 1,842#	- 5,600#/2	=	-958#	No Net Uplift OK

Project: Greisman Remodel Designed By: DKP Date: 6-27-2025

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

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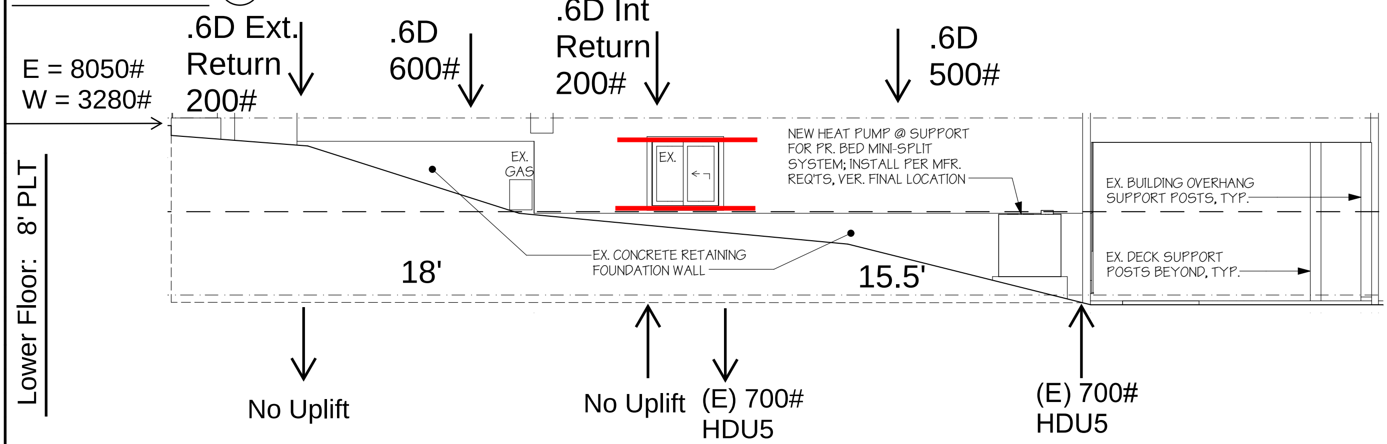
Shear Wall Line A



Dead: 10psf x 4ft x 18ft + 15psf x 1ft x 18 = 1000#

4 ft	[Plate Height]		= 33.5 ft	[Total pier widths]	=18+15.5							
						Segmented	[Analysis type]					
Ve =	8.05k	/ 34 ft	=	**239.9 plf**	==>	P1-6	SW OK					
Vw =	3.28k	/ 34 ft	=	**97.9 plf**	==>	P1-6	SW OK					
		SW Capacity =	**(E)[240 plf]**		**(W)[240 plf]**	[Unreduced Capacity]						
									Overturning Load			
OT (EQ 1)=	240 plf	x 18.0 ft	x 2.0 ft	/18.0 ft	**= 480#**	**- 600#/2**	**-200#**	=	**-20#**	**No Net Uplift**	**OK**	
OT (EQ 2)=	240 plf	x 15.5 ft	x 4.0 ft	/15.5 ft	**= 960#**	**- 500#/2**		=	**710#**	**==>HDU5**	**OK**	

Project: Greisman Remodel Designed By: DKP Date: 6-27-2025

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

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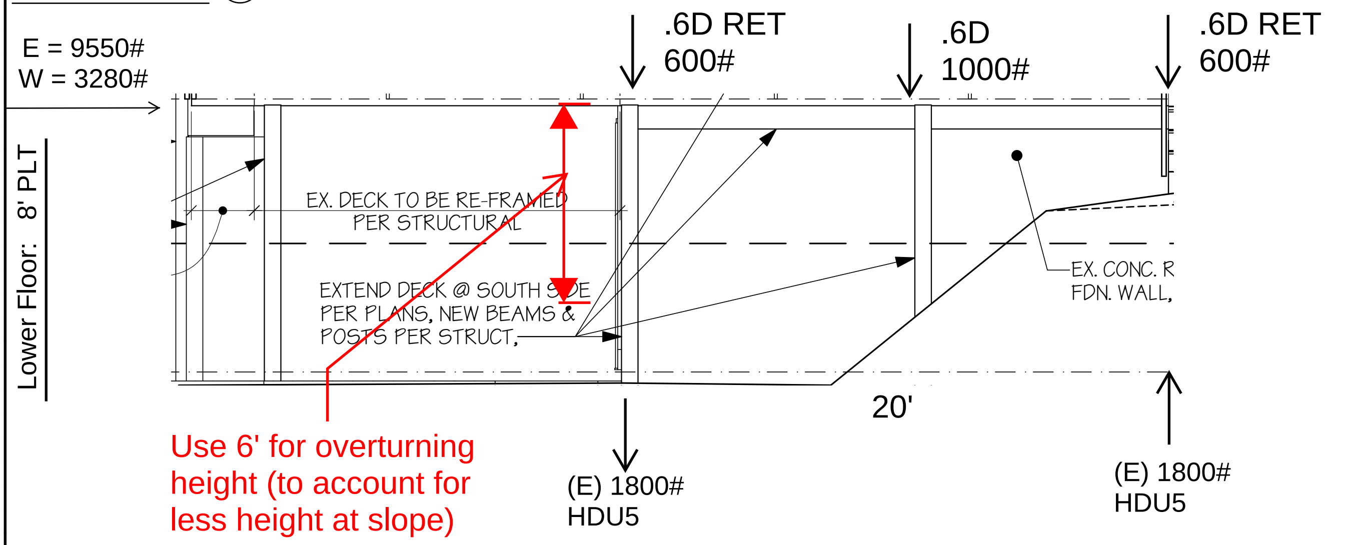
Shear Wall Line (B)



Dead: 10psf x 8ft x 20ft = 1600#
Return Dead: 10psf x 8ft x 4ft + 15psf x 10ft x 4ft = 1000#

6 ft	[Plate Height]		= 20.0 ft	[Total pier widths]	=20							
						Segmented	[Analysis type]					
Ve =	9.55k	/	20 ft	=	**477.5 plf**	==>	P1-2	SW OK				
Vw =	3.28k	/	20 ft	=	**164.0 plf**	==>	P1-2	SW OK				
		SW Capacity =		**(E)[590 plf]**		**(W)[820 plf]**	[Unreduced Capacity]					
									Overturning Load			
OT (EQ)=	478 plf	x 20.0 ft	x 6.0 ft	/20.0 ft	**= 2,865#**	**- 1,000#/2**	**-600#**	=	**1,765#**	**==>HDU5**		**OK**

Project: Greisman Remodel Designed By: DKP Date: 6-27-2025

Project Number: Client: Scale: Page: L2.6

1.Project information

Project description:
Location:
Fastening description:

Comment:

2. Input Data & Anchor Parameters

General
Design method:ACI 318-19
Units: Imperial units

Anchor Information:
Anchor type: Bonded anchor
Material: F1554 Grade 36
Diameter (inch): 0.625
Effective Embedment depth, h_{ef} (inch): 10.000
Code report: ICC-ES ESR-4057
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 11.38
c_{ac} (inch): 22.57
C_{min} (inch): 1.75
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 18.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: Supplementary reinforcement not present
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Hole condition: Dry concrete
Inspection: Continuous
Temperature range, Short/Long: 150/110°F
Reduced installation torque (for AT-3G): Not applicable
Ignore 6do requirement: Not applicable
Build-up grout pad: No

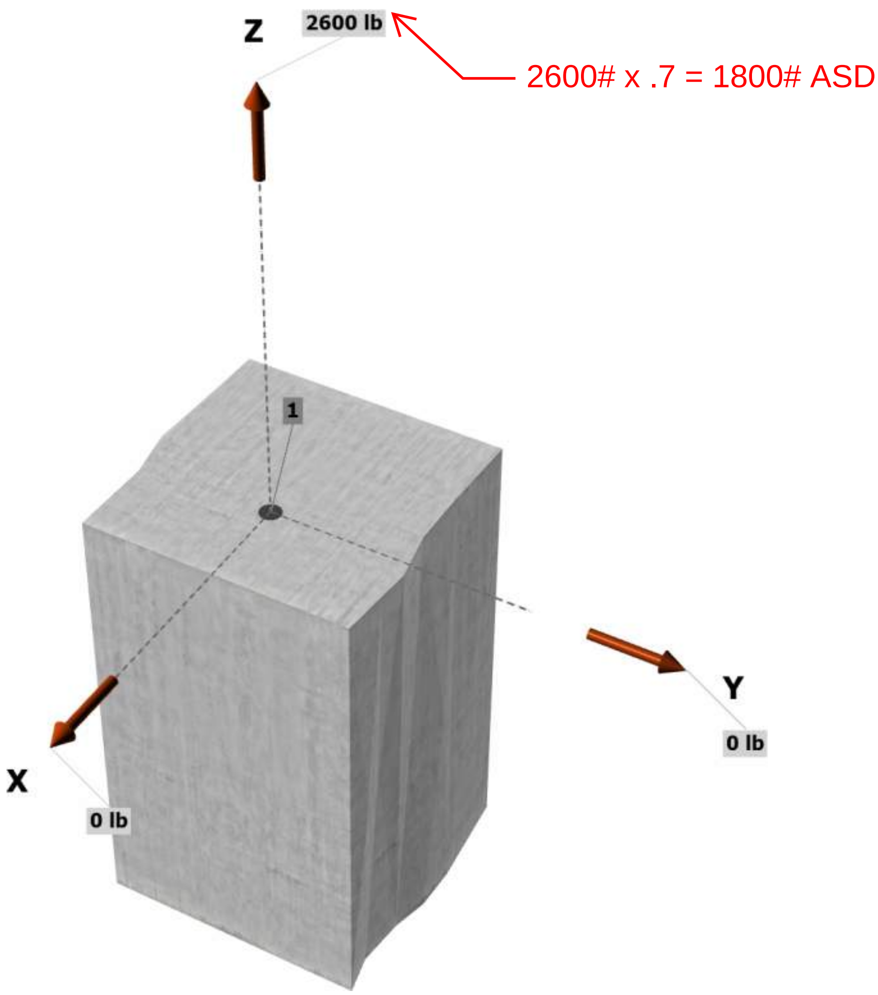
Recommended Anchor
Anchor Name: SET-3G™ - SET-3G w/ 5/8"Ø F1554 Gr. 36
Code Report: ICC-ES ESR-4057

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.10.5.2 not applicable
Ductility section for shear: 17.10.6.2 not applicable
Ω_0 factor: not set
Apply entire shear load at front row: Yes
Anchors only resisting wind and/or seismic loads: Yes

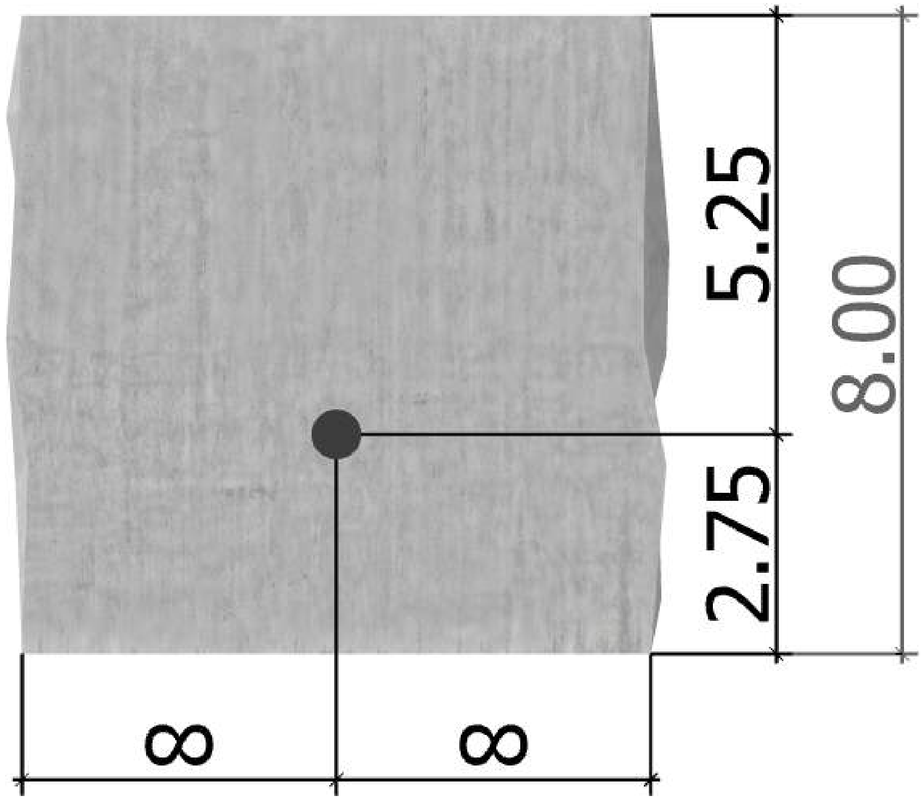
Strength level loads:

N_{ua} [lb]: 2600
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	2600.0	0.0	0.0	0.0
Sum	2600.0	0.0	0.0	0.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 2600
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. 17.6.1)

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

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.6.2)

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

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	10.000	26879

$0.75\phi N_{cb} = 0.75\phi \left(A_{Nc} / A_{Nco}\right) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.5.1.2 & Eq. 17.6.2.1a)

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	$0.75\phi N_{cb}$ (lb)
240.00	900.00	2.75	0.755	1.00	1.000	26879	0.65	2638

6. Adhesive Strength of Anchor in Tension (Sec. 17.6.5)

$\tau_{k,cr} = \tau_{k,cr} f_{short\text{-}term} K_{sat} (f'_c / 2{,}500)^n \alpha_{N.seis}$

$\tau_{k,cr}$ (psi)	$f_{short\text{-}term}$	K_{sat}	$\alpha_{N.seis}$	f'_c (psi)	n	$\tau_{k,cr}$ (psi)
1356	1.00	1.00	1.00	2500	0.24	1356

$N_{ba} = \lambda_a \tau_{cr} \pi d_a h_{ef}$ (Eq. 17.6.5.2.1)

λ_a	τ_{cr} (psi)	d_a (in)	h_{ef} (in)	N_{ba} (lb)
1.00	1356	0.63	10.000	26625

$0.75\phi N_a = 0.75\phi \left(A_{Na} / A_{Na0}\right) \Psi_{ed,Na} \Psi_{cp,Na} N_{ba}$ (Sec. 17.5.1.2 & Eq. 17.6.5.1a)

A_{Na} (in^2)	A_{Na0} (in^2)	c_{Na} (in)	$c_{a,min}$ (in)	$\Psi_{ed,Na}$	$\Psi_{p,Na}$	N_{a0} (lb)	ϕ	$0.75\phi N_a$ (lb)
140.19	307.10	8.76	2.75	0.794	1.000	26625	0.65	4706

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.8)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	2600	9833	0.26	Pass
Concrete breakout	**2600**	**2638**	**0.99**	**Pass (Governs)**

Adhesive	2600	4706	0.55	Pass

SET-3G w/ 5/8"Ø F1554 Gr. 36 with hef = 10.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, the tensile component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor tensile force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.5.2 for tension need not be satisfied – designer to verify.

- Per designer input, the shear component of the strength-level earthquake force applied to anchors does not exceed 20 percent of the total factored anchor shear force associated with the same load combination. Therefore the ductility requirements of ACI 318 17.10.6.2 for shear need not be satisfied – designer to verify.

- 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

2021 IBC/SDPWS 2021 – Diaphragms (8d Nailing)

Table 4.2C Nominal Unit Shear Values for Sheathed Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,6]

Sheathing Grade	Common Nail Size[5] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length in Framing Member, ℓ_m (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	Case 1 v_n (plf)	Case 1 G_a (kips/in) OSB	Case 1 G_a (kips/in) PLY	Cases 2,3,4,5,6 v_n (plf)	Cases 2,3,4,5,6 G_a (kips/in) OSB	Cases 2,3,4,5,6 G_a (kips/in) PLY
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	460	9.0	7.0	350	6.0	4.5
				3	520	7.0	6.0	390	4.5	4.0
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	670	8.5	7.0	505	6.0	4.5
				3	740	7.5	6.0	560	5.0	4.0
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	800	14	10	600	9.5	7.0
				3	895	12	9.0	670	8.0	6.0
Sheathing and Single-Floor	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	420	9.0	6.5	310	6.0	4.0
				3	475	7.0	5.5	350	5.0	3.5
			3/8	2	460	7.5	5.5	350	5.0	4.0
				3	520	6.0	4.5	390	4.0	3.0
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	600	9.0	6.5	450	6.0	4.5
				3	670	7.5	5.5	505	5.0	3.5
			7/16	2	645	8.5	6.0	475	5.5	4.0
				3	715	7.0	5.5	530	4.5	3.5
			15/32	2	670	7.5	5.5	505	5.0	4.0
				3	740	6.5	5.0	560	4.0	3.5
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	715	15	9.0	530	10	6.0
				3	810	12	8.0	600	8.0	5.5
			19/32	2	800	13	8.5	600	8.5	5.5
				3	895	10	7.5	670	7.0	5.0

1. Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.7. For specific requirements, see 4.2.8.1 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 increased by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Tabulated nominal unit shear capacities are applicable for carbon steel smooth shank nails of the specified type and size.
6. Diaphragm resistance depends on the direction of continuous adjoining panel edges with respect to the loading direction and direction of framing members, and is independent of the panel orientation.

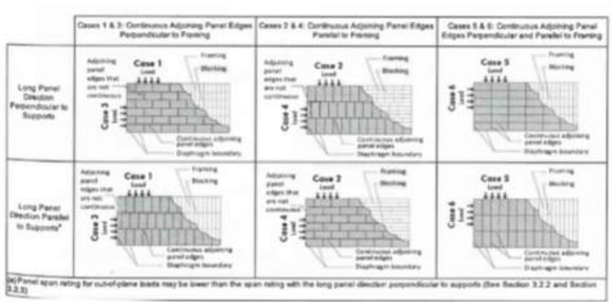
Table 4.2A Nominal Unit Shear Capacities for Sheathed Wood-Frame Diaphragms

Blocked Wood Structural Panel Diaphragms[1,2,3,4,6]

Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6): 6 / 4 / 2-1/2 / 2. Nail Spacing (in.) at other panel edges (Cases 1,2,3, & 4): 6 / 6 / 4 / 3.

Sheathing Grade	Common Nail Size[5] Length (in.) x Shank diameter (in.) x Head diameter (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)	6/6 v_n (plf)	6/6 G_a OSB	6/6 G_a PLY	4/6 v_n (plf)	4/6 G_a OSB	4/6 G_a PLY	2-1/2/4 v_n (plf)	2-1/2/4 G_a OSB	2-1/2/4 G_a PLY	2/3 v_n (plf)	2/3 G_a OSB	2/3 G_a PLY
Structural I	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	520	15	12	700	8.5	7.5	1050	12	10	1175	20	15
				3	590	12	9.5	785	7.0	6.0	1175	9.5	8.5	1330	17	13
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	755	14	11	1010	9.0	7.5	1485	13	10	1680	21	15
				3	840	12	10	1120	7.5	6.5	1680	10	9.0	1890	18	13
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	895	24	17	1190	15	12	1790	20	15	2045	31	21
				3	1010	20	15	1345	12	9.5	2015	16	13	2295	26	18
Sheathing and Single-Floor	6d (2 x 0.113 x 0.266)	1-1/4	5/16	2	475	15	10	630	9.0	7.0	940	13	9.5	1065	21	13
				3	530	12	9.0	700	7.0	6.0	1065	10	8.0	1205	17	12
			3/8	2	520	13	9.5	700	7.0	6.0	1050	10	8.0	1175	18	12
				3	590	10	8.0	785	5.5	5.0	1175	8.5	7.0	1330	14	10
	8d (2-1/2 x 0.131 x 0.281)	1-3/8	3/8	2	670	15	11	895	9.5	7.5	1345	13	9.5	1525	21	13
				3	755	12	9.5	1010	7.5	6.0	1510	11	8.5	1710	18	12
			7/16	2	715	14	10	950	8.5	7.0	1415	12	9.5	1610	20	13
				3	800	11	9.0	1065	7.0	6.0	1595	10	8.0	1805	17	12
			15/32	2	755	13	9.5	1010	7.5	6.5	1485	11	8.5	1680	19	13
				3	840	10	8.5	1120	6.0	5.5	1680	9.0	7.5	1890	15	11
	10d (3 x 0.148 x 0.312)	1-1/2	15/32	2	810	25	15	1080	15	11	1610	21	14	1835	33	18
				3	910	21	14	1205	12	9.5	1820	17	12	2060	28	16
			19/32	2	895	21	14	1190	13	9.5	1790	18	12	2045	28	17
				3	1010	17	12	1345	10	8.0	2015	14	11	2295	24	15

1. Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.7. For specific requirements, see 4.2.8.1 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 increased by 1.2.
4. Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
5. Tabulated nominal unit shear capacities are applicable for carbon steel smooth shank nails of the specified type and size.
6. Diaphragm resistance depends on the direction of continuous adjoining panel edges with respect to the loading direction and direction of framing members, and is independent of the panel orientation.



1. Reduction Factor = 2.8 for seismic and 2.0 for wind per SDPWS 2021 4.1.4
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

2021 IBC/SDPWS 2021 – Shear Wall Schedule

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

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

Wood-based Panels [4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Nail Type & Size [9]: Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.): 6			4			3			2		
				v_n (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_n (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_n (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_n (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY
Wood Structural Panels - Structural I [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	560	13	10	840	18	13	1090	23	16	1430	35	22
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	645	19	14	1010	24	17	1290	30	20	1710	43	24
	7/16 [2]			715	16	13	1105	21	16	1415	27	19	1875	40	24
	15/32			785	14	11	1205	18	14	1540	24	17	2045	37	23
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	950	22	16	1430	29	20	1860	36	22	2435	51	28
Wood Structural Panels - Sheathing [4,5]	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) [8]	505	13	9.5	755	18	12	980	24	14	1260	37	18
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17
	3/8 [2]	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) [8]	615	17	12	895	25	15	1150	31	17	1485	45	20
	7/16 [2]			670	15	11	980	22	14	1260	28	17	1640	42	21
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) [8,10]	870	22	14	1290	30	17	1680	37	19	2155	52	23
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22
Plywood Siding	5/16	1-1/4	6d galv.[7] casing nail (2 x 0.099 x 0.142)	390	13		590	16		770	17		1010	21	
	3/8	1-3/8	8d galv.[7] casing nail (2-1/2 x 0.113 x 0.155)	450	16		670	18		870	20		1150	22	
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")	3/8		6d common nail (2 x 0.113 x 0.266) [8]	335	15		505	17		645	19		840	22	
	3/8		8d common nail (2-1/2 x 0.131 x 0.281) [8]	365	18		530	20		670	21		880	23	
	1/2			390	18		590	20		755	22		980	24	
	1/2		10d common nail (3 x 0.148 x 0.312) [8]	520	21		770	23		1010	24		1290	25	
	5/8			560	21		855	23		1105	24		1455	26	
Structural Fiberboard Sheathing	1/2		11 ga. galv.[7] roofing nail (1-1/2 x 0.120 x 7/16)				475	4.0		645	5.0		730	5.5	
	25/32		11 ga. galv.[7] roofing nail (1-3/4 x 0.120 x 3/8)				475	4.0		645	5.0		730	5.5	

1. Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 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, 4.3.7.3 for particleboard shear walls, and 4.3.7.4 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
2. Nominal unit shear capacities 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 increased 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. 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 mechanically deposited.
8. Galvanized box nails shall be permitted to be substituted for the specified common nails as shown in the table below.

Nail Type & Size: Length (in.) x Shank diameter (in.) x Head diameter (in.)	
Common Nail	Galvanized Box Nail [7]
6d (2 x 0.113 x 0.266)	6d (2 x 0.099 x 0.266)
8d (2-1/2 x 0.131 x 0.281)	8d (2-1/2 x 0.113 x 0.297)
10d (3 x 0.148 x 0.312)	10d (3 x 0.128 x 0.312)

9. Tabulated nominal unit shear capacities are applicable for carbon steel smooth shank nails of the specified type and size.
10. Where tension force induced by shear wall overturning is resisted by a hold-down attached to the inside face of the end post, nominal unit shear capacity for shear walls using 10d common nails shall be multiplied by 0.92.

1. Reduction Factor = 2.8 for seismic and 2.0 for wind per SDPWS 2021 4.1.4
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

2021 IBC/NDS 2018 – 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)	Code Ref.
SS	A34	1	(8) 0.131 x 1½	F_1	395	480	545	340	415	480	IBC, FL, LA
				F_2[6]	395	430	430	340	370	370	
			(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	IBC, FL, LA
				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) 0.131 x 1½	F_1	555	555	555	475	475	475	
		6	(12) PH612I	F_1	420	420	420	360	360	360	—
	LTP4	7	(12) 0.131 x 1½	G	580	715	715	500	615	615	IBC, FL, LA
				H	525	525	525	450	450	450	
	LTP5	8	(12) 0.131 x 1½	G	565	565	565	485	485	485	
				H	490	490	490	420	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 ½" 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 ½" 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 ½" nails with no reduction in load.
6. Connectors are required on both sides to achieve F_2 loads in both directions.
7. A34 and A35 installed with 0.131" x 1 ½" nails onto 1 ¼" LSL material will achieve 0.90 of the listed F_1 and F_2 loads.
8. 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
P1-6	240-plf (E)	560#	28" oc	615#	28" oc
P1-4	350-plf (E)	560#	19" oc	615#	19" oc
P1-3	450-plf (E)	560#	14" oc	615#	14" oc
P1-2	820-plf (W)	560#	8" oc	615#	8" oc
P2-4	700-plf (E)	560#	LTP4 19" oc + A35 19" oc	615#	LTP4 19" oc + A35 19" oc
P2-3	900-plf (E)	560#	LTP4 14" oc + A35 14" oc	615#	LTP4 14" oc + A35 14" oc
P2-2	1640-plf (W)	560#	LTP4 8" oc + A35 8" oc	615#	LTP4 8" oc + A35 8" oc

2021 IBC/NDS 2018 – Shear Wall Bolts

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

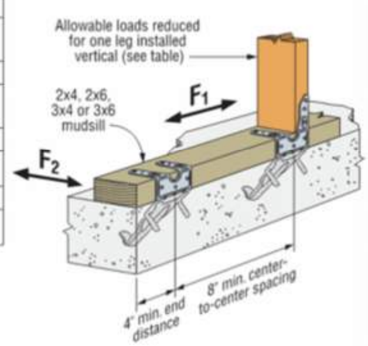
for sawn lumber or SCL to concrete

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	Uncracked Wind and SDC A&B F1	Uncracked Wind and SDC A&B F2	Uncracked SDC C–F[6] Uplift	Uncracked SDC C–F F1	Uncracked SDC C–F F2	Cracked Wind and SDC A&B[5,6] Uplift	Cracked Wind and SDC A&B F1	Cracked Wind and SDC A&B F2	Cracked SDC C–F[6] Uplift	Cracked SDC C–F F1	Cracked SDC C–F F2	Code Ref.
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	IBC, FL, LA
	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 and DF/SP Stud																
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	IBC, FL, LA
	3x4, 3x6	(7) 0.148 x 1½	(2) 0.148 x 1½	—	760	—	—	665	—	—	760	—	—	665	—	
Two-Legs-Up Installation — Attached to DF/SP Sill Plate and Rimboard or Blocking																
MASA or MASAP	2x4, x6, x8, x10, 3x4, 3x6	(9) 0.148 x 1½	—	810	1,105	865	740	965	755	620	1,105	630	560	965	550	IBC, FL, LA
Double 2x Installation — Attached to DF/SP Sill Plates																
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	IBC, FL, LA
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	—	—	570	—	—	670	—	—	570	—	
Two-Legs-Up Installation — Attached to Hem-Fir Sill Plate and HF/SPF Rimboard or Blocking																
MASA or MASAP	2x4, x6, x8, x10, 3x4, 3x6	(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 Plates																
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	475	775	675	—

1. Loads have been increased for wind or earthquake loading, with no further increase allowed. Reduce where other loads govern.
2. Concrete shall have a minimum compressive strength of f'_c = 2,500 psi.
3. Allowable loads are based on a minimum stem wall width of 6".
4. For simultaneous loads in more than one direction, the connector must be evaluated using the Unity Equation, as described in General Instructions for the Designer.
5. Per Section 1613 of the 2012/2015/2018/2021 IBC, detached one- and two-family dwellings in SDC C may use the "Wind and SDC A&B" allowable loads.
6. For designs under the 2012/2015/2018/2021 IBC, sill plate size shall comply with the shearwall requirements of the 2015/2021 Special Design Provisions for Wind and Seismic.
7. **Fasteners:** Nail dimensions are listed diameter by length. See pp. 21–22 for fastener information.



Typical MASA/MASAP Installation on Sill Plate

Wall Type	Capacity	Sill Plate	Single 5/8"ϕ Bolt Capacity	5/8"ϕ Anchor Bolt Spacing	MASAP Anchor Capacity	MASAP Anchor Spacing
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

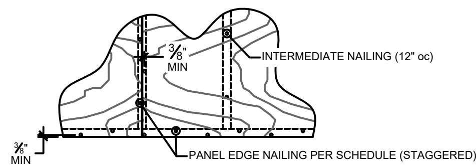
(IN ACCORDANCE w/ ANSI/AF&PA SDPWS-2021 SECTION 4.3)
Updated 11/15/2023

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 @ 28" oc or LTP4 @ 28" 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 @ 19" oc or LTP4 @ 19" 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 LTP4 @ 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 @ 8" oc or LTP4 @ 8" 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 @ 19" oc and LTP4 @ 19" 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 LTP4 @ 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 LTP4 @ 8" oc	⅝"Ø @ 12" oc	MASAP @ 7" oc	1180-plf 957-plf	1640-plf 1338-plf

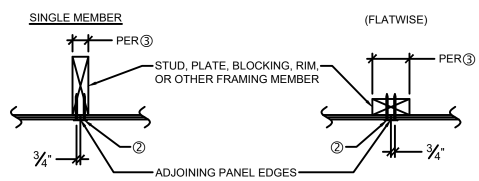
SHEAR WALL SCHEDULE NOTES

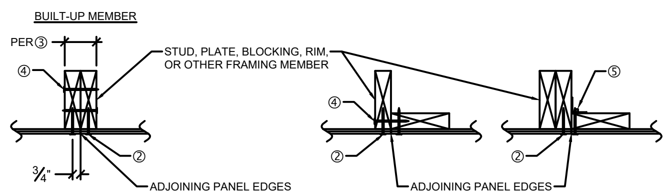
(SECTION 4.3.7.1.1)
7/16" OSB or 15/32" PLYWOOD SHEATHING OR SIDING EXCEPT GROUP 5 SPECIES. MINIMUM PANEL SPAN RATING OF (24/0). PANELS SHALL NOT BE LESS THAN 4'x8', EXCEPT AT BOUNDARIES AND CHANGES IN FRAMING. ALL EDGES OF ALL PANELS SHALL BE SUPPORTED BY AND FASTENED TO FRAMING MEMBERS OR BLOCKING.

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.



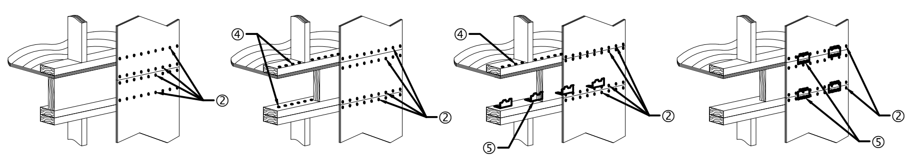
③ (SECTION 4.3.7.1.4)
THE MINIMUM NOMINAL WIDTH OF THE NAILED FACE OF FRAMING AND BLOCKING AT ADJOINING PANEL EDGES SHALL BE AS INDICATED IN THE SCHEDULE.





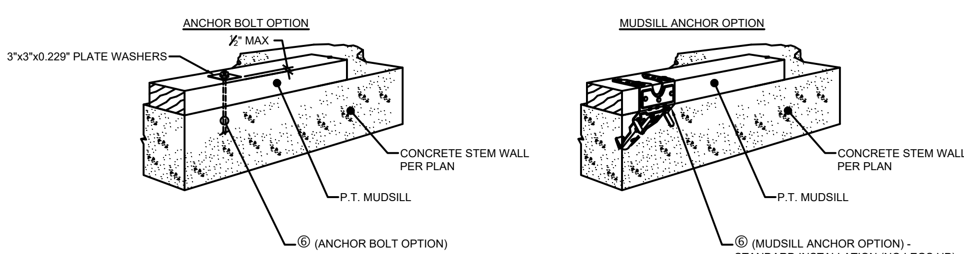
④ FACE NAILING APPLIES TO CONDITIONS WHERE FRAMING NAILS CAN BE STRAIGHT DRIVEN THRU FIRST MEMBER AND PENETRATE MAIN MEMBER MINIMUM OF 1½". FRAMING NAILS SHALL BE 0.131"Ø x 3¼". 0.131"Ø x 3" NAILS MAY BE USED WHEN STITCHING TOGETHER (2)2x MEMBERS WITH NO SPACERS.

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 2½" NAILS AT LTP4 CLIP WHEN INSTALLED OVER ½" SHEATHING.



LAP RIM OPTION | LAP PLATE OPTION | A35 OPTION | LTP4 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.



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Miscellaneous

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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:		9	ft
	Desired Stud Spacing:		24	in oc
No Fire Rating	Design Axial Dead Load:		683	plf
2x6	Design Axial Live Load:		960	plf
SPF Stud	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: 4/18/2024

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Stud Wall Design

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$[f_c/F_c']^2 + f_b/F_b'[1-(f_c/F_{cE})] < 1$ 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: 4/18/2024

Page: M1.2

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Stud Wall Design

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

Wall: Exterior Walls	Wall Height:	9	ft
	Desired Stud Spacing:	24	in oc
No Fire Rating	Design Axial Dead Load:	203	plf
2x4	Design Axial Live Load:	540	plf
SPF Stud	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: 4/18/2024

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Stud Wall Design

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

Wall: Exterior Walls	Wall Height:	9	ft
	Desired Stud Spacing:	16	in oc
No Fire Rating	Design Axial Dead Load:	338	plf
2x4	Design Axial Live Load:	900	plf
SPF Stud	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.

Date: 4/18/2024

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2018 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 = 5 1/2 in

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

F_c' = 594 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

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

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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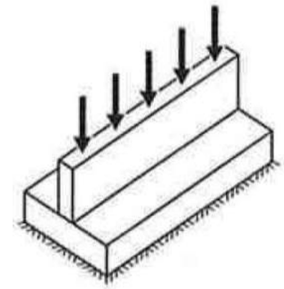
Project: (E) Continuous Strip Footing
15" wide x 8" thick

IBC Section 13.3.2: One-way shallow foundations

Footing width, B = 15 in
Footing Thickness, t = 8 in
Stem Wall width, C = 8 in
Stem Wall Height = 18 in



Strip footing

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

Longintudinal Reinforcement: (1) #4
Bar Diameter = 0.500 in
Bar Area = 0.20 in^2
A_s = 0.20 in^2
Cover: 3 in

Stem Wall Reinforcement: #4 @ 48 "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 = 183 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 = 64125 plf [Ultimate]
40078 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

$M_u \le \phi M_n$

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

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

$q_u =$

NO MOMENT

Max Uniform Load on Stem = 10000 plf [Ultimate]
6250 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

Allowable Soil Bearing Pressure	**1500 psf**	**2000 psf**	**2500 psf**	**3000 psf**	**3500 psf**	**4000 psf**
Max Uniform Load, Soil	1692 plf	2317 plf	2942 plf	3567 plf	4192 plf	4817 plf
Max Uniform Load, Shear	40078 plf	40078 plf	40078 plf	40078 plf	40078 plf	40078 plf
Max Uniform Load, Moment	6250 plf	6250 plf	6250 plf	6250 plf	6250 plf	6250 plf
Max Uniform Load (Service)	**1692 plf**	**2317 plf**	**2942 plf**	**3567 plf**	**4192 plf**	**4817 plf**
Max Uniform Load (Ultimate)	**2707 plf**	**3707 plf**	**4707 plf**	**5707 plf**	**6707 plf**	**7707 plf**
Max Point Load (Service)	**10150 #**	**13900 #**	**17650 #**	**21400 #**	**25150 #**	**28900 #**
Max Point Load (Ultimate)	**16240 #**	**22240 #**	**28240 #**	**34240 #**	**40240 #**	**46240 #**

Date: 7/2/2025

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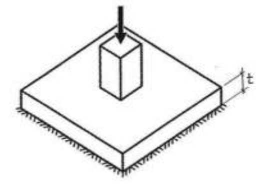
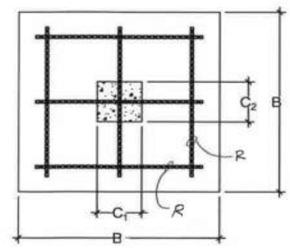
Project: Typical Footing
Footing: 36" x 36" x 12" thick



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: ϕ = 0.75

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

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

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

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

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(C1+d)+2(C2+d)
55

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

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

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

Moment: ϕ = 0.90

$M_n = A_s f_y (d - a/2) =$ 16.2 k-ft

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

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

Footing Moment Capacity Check
6600# / 33825# = 20% (Grav Used Moment)
80% x 14.6k-ft = 11.68 k-ft (Avail. Mom.)
11.68k-ft > 2.4k-ft
Footing Sufficient for Lateral + Grav Moment

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

$q_u =$ 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**
Max Load (lbs), Soil	13140
Max Load (lbs), One-Way Shear	71598
Max Load (lbs), Two-Way Shear	49805
Max Load (lbs), Moment	33825
Max Load (ASD)	**13140**
Max Load (Factored)	**21024**

Pile w/ Moment Check
e = M / (P + Wf) = 2.4k-ft / (6.6k + .36k) = .35ft
e < B/6? .35ft < .5ft ... OK
Pile Load = P/2 + M/d
5300#/2 + 2400#-ft / (3ft - 1ft) = 3850# < 6000#/pile
... (2) Piles per footing OK

Date: 4/18/2024