Structural Calculations

for

NEW SINGLE FAMILY DWELLING
Smersh Residence
2423 60th Ave SE
Mercer Island, WA 98040
PERMIT SUBMITTAL

prepared by:
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Job No. 22004
Date: 12/14/22

OWEN REMICK GOULD
52797

Vertical Design Loads

Typical Truss Roof		
Comp Shingle Roofing	3	psf
5/8" Plywood	2	
Trusses	3	
Batt Insulation	0.4	
5/8" Gypsum Board	2.8	
Future Solar Panels	4	*
Sum	15.2	psf
Slope:	7	:12
Slope Correction Factor	1.16	
Subtotal	17.6	psf
M/E/P/misc.	1.4	psf
DL=	19	psf
SL=	25	psf

**As required for solar-ready zone per WA State Building Building Code Amendments*

Typical Stick-Framed Roof		
Comp Shingle Roofing	3	psf
5/8" Plywood	2	
Rafters @24"o.c.	1.9	
Batt Insulation	0.4	
5/8" Gypsum Board	2.8	
Future Solar Panels	4	*
Sum	14.1	psf
Slope:	7	:12
Slope Correction Factor	1.16	
Subtotal	16.3	psf
M/E/P/misc.	1.7	psf
DL=	18	psf
SL=	25	psf

**As required for solar-ready zone per WA State Building Building Code Amendments*

Roof Deck			
Built-Up Roof Decking	8	psf	
3/4" Plywood	2.4		
Joists	2.8		
5/8" Gypsum Board	2.8		
M/E/P/misc.	2		
DL=	18	psf	
LL=	60	psf	*Decks*

Upper Floor			
Flooring	4	psf	
3/4" Plywood	2.4		
Joists	2.8		
5/8" Gypsum Board	2.8		
M/E/P/misc.	2		
DL=	14	psf	
LL=	40	psf	*Living Areas*

Main Floor			
Flooring	4	psf	
3/4" Plywood	2.4		
Joists	2.8		
Batt Insulation	0.4		
M/E/P/misc.	1.4		
DL=	11	psf	
LL=	40	psf	*Living Areas*

Exterior Walls		
Siding	3	psf
Sheathing	2	
Studs	1.4	
Batt Insulation	0.2	
1/2" Gypsum Board	2.2	
M/E/P/misc.	1.2	
DL=	10	psf

Interior Walls		
2 Layers 1/2" Gypsum Board	4.4	psf
2x4 @16"o.c.	0.9	
M/E/P/misc.	1.7	
DL=	7	psf

Seismic Design Loads

Seismic Design Parameters (ASCE 7-16 Section 12.8.1)

Approximate Fundamental Period

$T = T_a = C_t h_n^x$

where:
C_t = 0.02
h_n = 25
x = 0.75
T = 0.22 s

Seismic Response Coefficient

S_s = 1.40
S_1 = 0.49
S_{ds} = 0.93
S_{d1} = 0.49
R = 6.5
ρ = 1.3
Ω = 2.5
C_d = 4
I_e = 1

$C_s = S_{ds}/(R/I_e)$ = 0.14 W

T_L = 6 s > T

$C_{s,max} = S_{d1}/[T(R/I_e)]$ = 0.34

$C_{s,min} = 0.044 S_{ds} I_e$ = 0.041
$C_{s,min}$ = 0.01
S_1 < 0.6
$C_{s,min} = 0.5 S_1/(R/I_{e)}$ = 0.037 **Ignore**

$C_{s,min,gov}$ = 0.041

$C_{s,gov}$ = 0.14 (LRFD)

Effective Seismic Weight				
Floor	Area (sf)	w_{floor} (psf)	w_{walls} (psf)[1]	W (lbs)
Roof	1520	19	14	50160
Upper	2760	16	28	122370
			Sum:	**172530 lbs**

[1] Includes weight of interior/exterior walls as uniform area load

Base Shear (includes ρ) - LRFD Level			
$\rho V = \rho C_s W =$	0.186	W =	**32160 lbs**

Vertical Distribution of Base Shear (ASCE 7-16 Section 12.8.3) - LRFD Level						
Floor	W_x (lbs)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (lbs)	F_x (psf)
Roof	50160	25	1254000	0.48	**15510**	**10.2**
Upper	122370	11	1346070	0.52	**16649**	**6.0**
		Sum:	2600070		32160	

Where k = 1

Diaphragm Forces (ASCE 7-16 Section 12.10.1.1) - LRFD Level						
Floor	F_i (lbs)	$\sum F_i$	W_i (lbs)	$\sum W_i$	$\sum F_i/\sum W_i$	F_{px} (lbs)
Roof	11931	11931	50160	50160	0.24	11931
Upper	12807	24738	122370	172530	0.14	17546

Floor	F_{px} Min (lbs)	F_{px} Max (lbs)	F_{px} Gov (lbs)	F_{px} Gov (psf)
Roof	6545	13090	**11931**	**7.8**
Upper	15967	31934	**17546**	**6.4**

ATC Hazards by Location

Search Information

Address:	2423 60th Ave SE, Mercer Island, WA 98040, USA
Coordinates:	47.5884238, -122.2536923
Elevation:	49 ft
Timestamp:	2022-02-17T18:12:01.719Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	1.398	MCE$_R$ ground motion (period=0.2s)
S_1	0.487	MCE$_R$ ground motion (period=1.0s)
S_{MS}	1.398	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.932	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

▾Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.902	Coefficient of risk (0.2s)
CR_1	0.896	Coefficient of risk (1.0s)
PGA	0.598	MCE$_G$ peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.658	Site modified peak ground acceleration
T_L	6	Long-period transition period (s)
SsRT	1.398	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.55	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.164	Factored deterministic acceleration value (0.2s)
S1RT	0.487	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.544	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.295	Factored deterministic acceleration value (1.0s)
PGAd	1.094	Factored deterministic acceleration value (PGA)

ASCE 7-16 Wind Forces, Chapter 27, Part 1

Project File: 22004_SmershRes_.ec6

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DESCRIPTION: Smersh Residence

MWFRS

Basic Values

Risk Category	2	per ASCE 7-16 Table 1.5-1	Horizontal Dim. in North-South Direction (B or L)	30.0 ft
V : Basic Wind Speed	97.0	per ASCE 7-16 Fig. 26.5-1 & 26.5-2	Horizontal Dim. in East-West Direction (B or L)	92.0 ft
Kd : Directionality Factor	0.850	per ASCE 7-16 Table 26.6-1	h : Mean Roof height =	25.0 ft

Exposure Category per ASCE 7-16 Section 26.7

North :	Exposure C	East :	Exposure C
South :	Exposure C	West :	Exposure C

Topographic Factor per ASCE 7-16 Sec 26.8 & Figure 26.8-1

North :	K1 =	K2 =	K3 =	Kzt = 1.000
South :	K1 =	K2 =	K3 =	Kzt = 1.000
East :	K1 =	K2 =	K3 =	Kzt = 1.000
West :	K1 =	K2 =	K3 =	Kzt = 1.000

Building Period & Flexibility Category

User has specified the building frequency is >= 1 Hz, therefore considered RIGID for both North-South and East-West directions.

Building Story Data

Level Description	hi ft	Story Ht ft	E_R : X ft	E_R : X ft
Roof	25.00	14.00	0.000	0.000
UF	11.00	11.00	0.000	0.000

Gust Factor

For wind coming from direction indicated

North	=	**0.850**	South	=	**0.850**
East	=	**0.850**	West	=	**0.850**

Enclosure

Check if Building Qualifies as "Open"

	North Wall	South Wall	East Wall	West Wall	Roof	Total
Agross	1.0 ft^2	1.0 ft^2	1.0 ft^2	1.0 ft^2	1.0 ft^2	5.0 ft^2
Aopenings	0.0 ft^2	0.0 ft^2	0.0 ft^2	0.0 ft^2	0.0 ft^2	0.0 ft^2
Aopenings >= 0.8 * Agross ?	No	No	No	No		

All four Agross values must be non-zero **Building does NOT qualify as "Open"**

User has specified the Building is to be considered Enclosed when NORTH elevation receives positi

User has specified the Building is to be considered Enclosed when SOUTH elevation receives positi

User has specified the Building is to be considered Enclosed when EAST elevation receives positive

User has specified the Building is to be considered Enclosed when WEST elevation receives positive

Velocity Pressures

When the following walls experience leeward or sidewall pressures, the value of Kh shall be (per Table 26.10-1) :

North Wall = 0.9453 psf South Wall = 0.9453 psf East Wall = 0.9453 psf West Wall = 0.9453 psf

When the following walls experience leeward or sidewall pressures, the value of qh shall be (per Eq 26.10-1) :

North Wall = 19.353 psf South Wall = 19.353 psf East Wall = 19.353 psf West Wall = 19.353 psf

qz : Windward Wall Velocity Pressures at various heights per Eq. 27.3-1

	North Elevation		South Elevation		East Elevation		West Elevation	
Height Above Base (ft)	Kz	qz	Kz	qz	Kz	qz	Kz	qz
0.00	0.849	17.38	0.849	17.38	0.849	17.38	0.849	17.38
4.00	0.849	17.38	0.849	17.38	0.849	17.38	0.849	17.38
8.00	0.849	17.38	0.849	17.38	0.849	17.38	0.849	17.38
12.00	0.849	17.38	0.849	17.38	0.849	17.38	0.849	17.38

16.00	0.860	17.62	0.860	17.62	0.860	17.62	0.860	17.62
20.00	0.902	18.47	0.902	18.47	0.902	18.47	0.902	18.47
24.00	0.937	19.19	0.937	19.19	0.937	19.19	0.937	19.19

Pressure Coefficients

GCpi Values when elevation receives positive external pressure

GCpi : Internal pressure coefficient, per sec. 26.13 and Table 26.13-1

North	South	East	West
+/- 0.180	+/- 0.180	+/- 0.180	+/- 0.180

Specify Cp Values from Figure 27.3-1 for Windward, Leeward & Side Walls

Cp Values when elevation receives positive external pressure

	North	South	East	West
Windward Wall	0.80	0.80	0.80	0.80
Leeward Wall	-0.50	-0.50	-0.50	-0.50
Side Walls	-0.70	-0.70	-0.70	-0.70

Wind Pressures

Wind Pressures when NORTH Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.709 psf	-4.742 psf
Side Wall Pressures	-14.999 psf	-8.032 psf

Windward Wall Pressures . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.33	15.30
4.00	8.33	15.30
8.00	8.33	15.30
12.00	8.33	15.30
16.00	8.50	15.46
20.00	9.07	16.04
24.00	9.56	16.53

Wind Pressures when SOUTH Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.709 psf	-4.742 psf
Side Wall Pressures	-14.999 psf	-8.032 psf

Windward Wall Pressures . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.33	15.30
4.00	8.33	15.30
8.00	8.33	15.30
12.00	8.33	15.30
16.00	8.50	15.46
20.00	9.07	16.04
24.00	9.56	16.53

Wind Pressures when EAST Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.709 psf	-4.742 psf
Side Wall Pressures	-14.999 psf	-8.032 psf

Windward Wall Pressures . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)

0.00	8.33	15.30
4.00	8.33	15.30
8.00	8.33	15.30
12.00	8.33	15.30
16.00	8.50	15.46
20.00	9.07	16.04
24.00	9.56	16.53

Wind Pressures when WEST Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.709 psf	-4.742 psf
Side Wall Pressures	-14.999 psf	-8.032 psf

Windward Wall Pressures . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.33	15.30
4.00	8.33	15.30
8.00	8.33	15.30
12.00	8.33	15.30
16.00	8.50	15.46
20.00	9.07	16.04
24.00	9.56	16.53

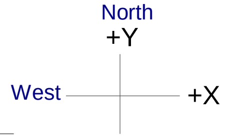
Story Forces for Design Wind Load Cases

Values below are calculated based on a building with dimensions B x L x h as defined on the "Basic Values" tab.

Load Case	Windward Wall	Building level	Ht. Range	Trib. Height	Wind Shear Components (k)		Eccentricity for (ft)		Mt, (ft-k)
					In "Y" Direction	In "X" Direction	"Y" Shear	"X" Shear	
CASE 1	North	Level 2	18.00' -> 25.0	7.00	-13.50	---	---	---	---
CASE 1	North	Level 1	5.50' -> 18.0	12.50	-23.12	---	---	---	---
CASE 1	South	Level 2	18.00' -> 25.0	7.00	13.50	---	---	---	---
CASE 1	South	Level 1	5.50' -> 18.0	12.50	23.12	---	---	---	---
CASE 1	East	Level 2	18.00' -> 25.0	7.00	---	-4.40	---	---	---
CASE 1	East	Level 1	5.50' -> 18.0	12.50	---	-7.54	---	---	---
CASE 1	West	Level 2	18.00' -> 25.0	7.00	---	4.40	---	---	---
CASE 1	West	Level 1	5.50' -> 18.0	12.50	---	7.54	---	---	---
CASE 2	North	Level 2	18.00' -> 25.0	7.00	-10.13	---	---	13.80	139.7
CASE 2	North	Level 1	5.50' -> 18.0	12.50	-17.34	---	---	13.80	239.2
CASE 2	South	Level 2	18.00' -> 25.0	7.00	10.13	---	---	13.80	139.7
CASE 2	South	Level 1	5.50' -> 18.0	12.50	17.34	---	---	13.80	239.2
CASE 2	East	Level 2	18.00' -> 25.0	7.00	---	-3.30	4.50	---	- 14.9
CASE 2	East	Level 1	5.50' -> 18.0	12.50	---	-5.65	4.50	---	- 25.4
CASE 2	West	Level 2	18.00' -> 25.0	7.00	---	3.30	4.50	---	- 14.9
CASE 2	West	Level 1	5.50' -> 18.0	12.50	---	5.65	4.50	---	- 25.4
CASE 3	North & East	Level 2	18.00' -> 25.0	7.00	-10.13	-3.30	---	---	---
CASE 3	North & East	Level 1	5.50' -> 18.0	12.50	-17.34	-5.65	---	---	---
CASE 3	North & West	Level 2	18.00' -> 25.0	7.00	-10.13	3.30	---	---	---
CASE 3	North & West	Level 1	5.50' -> 18.0	12.50	-17.34	5.65	---	---	---
CASE 3	South & West	Level 2	18.00' -> 25.0	7.00	10.13	3.30	---	---	---

CASE 3	South & West	Level 1	5.50' ->	18.0	12.50	17.34	5.65	---	---	---
CASE 3	South & East	Level 2	18.00' ->	25.0	7.00	10.13	-3.30	---	---	---
CASE 3	South & East	Level 1	5.50' ->	18.0	12.50	17.34	-5.65	---	---	---
CASE 4	North & East	Level 2	18.00' ->	25.0	7.00	-7.60	-2.48	4.50	13.80	116.1
CASE 4	North & East	Level 1	5.50' ->	18.0	12.50	-13.01	-4.24	4.50	13.80	198.7
CASE 4	North & West	Level 2	18.00' ->	25.0	7.00	-7.60	2.48	4.50	13.80	116.1
CASE 4	North & West	Level 1	5.50' ->	18.0	12.50	-13.01	4.24	4.50	13.80	198.7
CASE 4	South & West	Level 2	18.00' ->	25.0	7.00	7.60	2.48	4.50	13.80	116.1
CASE 4	South & West	Level 1	5.50' ->	18.0	12.50	13.01	4.24	4.50	13.80	198.7
CASE 4	South & East	Level 2	18.00' ->	25.0	7.00	7.60	-2.48	4.50	13.80	116.1
CASE 4	South & East	Level 1	5.50' ->	18.0	12.50	13.01	-4.24	4.50	13.80	198.7
Min per ASCE 27.1.	North	Level 2	18.00' ->	25.0	7.00	-10.30	---	---	---	---
Min per ASCE 27.1.	North	Level 1	5.50' ->	18.0	12.50	-18.40	---	---	---	---
Min per ASCE 27.1.	South	Level 2	18.00' ->	25.0	7.00	10.30	---	---	---	---
Min per ASCE 27.1.	South	Level 1	5.50' ->	18.0	12.50	18.40	---	---	---	---
Min per ASCE 27.1.	East	Level 2	18.00' ->	25.0	7.00	---	-3.36	---	---	---
Min per ASCE 27.1.	East	Level 1	5.50' ->	18.0	12.50	---	-6.00	---	---	---
Min per ASCE 27.1.	West	Level 2	18.00' ->	25.0	7.00	---	3.36	---	---	---
Min per ASCE 27.1.	West	Level 1	5.50' ->	18.0	12.50	---	6.00	---	---	---

Base Shear for Design Wind Load Cas



Values below are calculated based on a building with dimensions B x L x h as defined on the "General" tab.

Load Case	Windward Wall	Leeward Wall	Wind Base Shear Components (k)			
			In "Y" Direction	In "X" Direction		Mt, (ft-k)
Case 1	North	South	-36.62	---		---
Case 1	South	North	36.62	---		---
Case 1	East	West	---	-11.94		---
Case 1	West	East	---	11.94		---
Case 2	North	South	-27.46	---	/-	379.0
Case 2	South	North	27.46	---	/-	379.0
Case 2	East	West	---	-8.96	-/-	40.3
Case 2	West	East	---	8.96	-/-	40.3
Case 3	North & East	South & West	-27.46	-8.96		---
Case 3	North & West	South & East	-27.46	8.96		---
Case 3	South & West	North & East	27.46	8.96		---
Case 3	South & East	North & West	27.46	-8.96		---
Case 4	North & East	South & West	-20.62	-6.72	/-	314.7
Case 4	North & West	South & East	-20.62	6.72	/-	314.7
Case 4	South & West	North & East	20.62	6.72	/-	314.7
Case 4	South & East	North & West	20.62	-6.72	/-	314.7
Min per ASCE 27.1.5	North	South	-28.70	---		---
Min per ASCE 27.1.5	South	North	28.70	---		---
Min per ASCE 27.1.5	East	West	---	-9.36		---
Min per ASCE 27.1.5	West	East	---	9.36		---

GOVERNING LATERAL LOAD ON MLFRS

SEISMIC BASE SHEAR

$V_{SEISMIC} = 0.7 \times 32160 = \underline{22510^{\#}}$

EL, ASD

WIND BASE SHEAR

$V_{WIND} = 0.6 \times 36620 = \underline{21970^{\#}}$

WL, ASD

$\rightarrow V_{SEISMIC} > V_{WIND}$

$\therefore$ SEISMIC GOVERNS MLFRS DESIGN

ROOF FRAMING

(ELSEWHERE NOT EXPLICITLY CALC'D OK BY INSPECTION)

(RB5) ROOF BEAM

SPAN = 19'-0" $w = \overset{FR}{\underset{DL}{\underline{18}} + \underset{SL}{\underline{25}}}$ psf TRIB = $\frac{12'}{2}$ = 6'

USE $3\frac{1}{2} \times 11\frac{7}{8}$ PSL

(RH6) TRANSOM HEADER

SPAN = 9'-0" $P = \underset{DL}{\underline{1030}} + \overset{RB5}{\underset{SL}{\underline{1430}}}$ # @ x = 4'-6"

USE $3\frac{1}{2} \times 9\frac{1}{2}$ PSL

(RH7) ROOF HEADER

SPAN = 6'-0" $w = \overset{TR}{\underset{DL}{\underline{19}} + \underset{SL}{\underline{25}}}$ psf TRIB = $\frac{26'}{2}$ = 13'

USE 4 x 8

(RB8) ROOF BEAM

SPAN = 12'-3" $w = \overset{TR}{\underset{DL}{\underline{19}} + \underset{SL}{\underline{25}}}$ TRIB = $\frac{23'}{2}$ = 11'-6"

USE $3\frac{1}{2} \times 11\frac{7}{8}$ PSL

$P = \underset{DL}{\underline{1030}} + \overset{RB5}{\underset{SL}{\underline{1430}}}$ # @ x = 6.125'

(RB9) ROOF BEAM

SPAN = 7'-9" $w = \overset{TR}{\underset{DL}{\underline{19}} + \underset{SL}{\underline{25}}}$ psf TRIB = $\frac{26'}{2}$ = 13'

USE $5\frac{1}{2} \times 7\frac{1}{2}$ GLB

UPPER FLOOR FRAMING

UFJ 1 UPPER FLOOR JOISTS

SPAN = 13'-3" $W = \underbrace{14}_{DL} + \underbrace{40}_{LL}$ psf Use 11 7/8 TJI 210 @ 16" o.c.

UFB 2 UPPER FLOOR BEAM

SPAN = 13'-6" $W = \left(\overset{UF}{\left(\underbrace{14}_{DL}+\underbrace{40}_{LL}\right)} \left(\frac{26'}{2}\right) + \overset{W}{\left(\underbrace{10}_{DL}\right)}(13') \right) = \underbrace{310}_{DL} + \underbrace{520}_{LL} \ \#/ft$

Use 5 1/4 x 11 7/8 PSL

UF H3 UPPER FLOOR HEADER

SPAN = 9'-0" $w = \overset{UF}{\underbrace{14}_{DL}+\underbrace{40}_{LL}}$ psf $TRIB = \frac{13'}{2} = 6'\text{-}6'$

Use 4x10

UFB 4 UPPER FLOOR BEAM

SPAN = 13'-6" (SW UF.1)

$P_1 = \overset{USD}{1.4} \times \underset{\rho}{\frac{\overset{\Omega}{2.5}}{1.3}} \times \frac{\overset{OTM}{31500 \ \#\cdot ft}}{\underset{WALL}{13'\text{-}3''}} = \frac{6400 \ \#}{EQ, USD} \ @x = 12'\text{-}9''$

Use 5 1/4 x 11 7/8 PSL

$P_2 = \overset{URB3}{\left(\underbrace{18}_{DL}+\underbrace{25}_{SL}\right)} \left(\frac{19'}{2}\right)\left(\frac{15'}{2}\right) = \underbrace{1280}_{DL} + \underbrace{1780}_{SL} \ \# \ @x = 6'$

$W = \overset{TR}{\left(\underbrace{18}_{DL}+\underbrace{25}_{SL}\right)} \left(\frac{26'}{2}\right) + \overset{W}{\left(\underbrace{10}_{DL}\right)}(8') = \underbrace{320}_{DL} + \underbrace{330}_{SL} \ \#/ft$

UFB 5 UPPER FLOOR BEAM

SPAN = 18'-6" (SW UF.2)

$P = (\pm)\, 1.4 \times \frac{2.5}{1.3} \times \frac{9000}{10'} = \frac{2420 \ \#}{EQ, USD}$ $(+) @x = 4'\text{-}6''$ $(-) @x = 14'\text{-}3''$

Use 5 1/4 x 11 7/8 PSL

$W = \overset{FR}{\left(\underbrace{18}_{DL}+\underbrace{25}_{SL}\right)} \left(\frac{14'}{2}\right) + \overset{W}{\left(\underbrace{10}_{DL}\right)}(9') + \left(\overset{UF}{\left(\underbrace{14}_{DL}+\underbrace{40}_{LL}\right)}\left(\frac{12'}{2}\right)\right)$

$= \underbrace{300}_{DL} + \underbrace{240}_{LL} + \underbrace{180}_{SL} \ \#/ft$

ROOF DECK FRAMING

RDJ1 ROOF DECK JOIST

SPAN = 7'6" $w = \underset{DL}{10} + \underset{LL}{60}$ psf USE $1\frac{3}{4} \times 7\frac{1}{4}$ (min) LVL@16" o.c.

RDH2 ROOF DECK HEADER

SPAN = 9'-0" $w = \underset{DL}{10} + \underset{LL}{60}$ psf TRIB = $\frac{8'}{2} + 1' = 5'$

USE $3\frac{1}{2} \times 9\frac{1}{2}$ LSL

LOW ROOF FRAMING

(ELEMENTS NOT EXPLICITLY CALC'D ARE BY INSPECTION)

LRB2 LOW ROOF BEAM

SPAN = 13'-0" $w = \overset{FR}{\underset{DL}{18} + \underset{SL}{25}}$ psf TRIB = $\frac{8'}{2} + 1' = 5'$

USE $5\frac{1}{2} \times 9$ GLB

LRB3 LOW ROOF BEAM

SPAN = 18'-6" $w = \overset{FR}{\underset{DL}{18} + \underset{SL}{25}}$ psf TRIB = $\frac{15'}{2} = 7'$-6"

USE $3\frac{1}{2} \times 11\frac{7}{8}$ PSL

LRB4 LOW ROOF BEAM

SPAN = 15'-0" $P = \left(\overset{FR}{\underset{DL}{18} + \underset{SL}{25}}\right)\left(\frac{15'}{2}\right)\left(\frac{13'}{2}\right) = \underset{DL}{880} + \underset{SL}{1220}$ # @ x = 7'-6"

USE $5\frac{1}{2} \times 9$ GLB

UFB7 UPPER FLOOR BEAM (SW UF.D)

SPAN = 10'-9"

$$P = 1.4 \times \frac{2.5}{1.3} \times \frac{37929}{12'\text{-}3''} = \frac{8340\#}{\text{EQ LOAD}} \ @\ x = 10'$$

Use 3 1/2 x 11 7/8 PSL

$$W = \underset{DL}{\underline{320}} + \underset{LL}{\underline{330}}\ \#/\text{ft}$$ (SAME AS UFB4)

UFH8 UPPER FLOOR HEADER (WEST SIDE OF ENTRY WINDOW)

SPAN = 6'-0"

$$w = \overset{TR}{(\underset{DL}{\underline{19}} + \underset{SL}{\underline{25}})}\left(\frac{26'}{2}\right) + \overset{FR}{(\underset{DL}{\underline{18}} + \underset{SL}{\underline{25}})}\left(\frac{5'}{2}\right) + (\underset{DL}{\underline{60}})(13')$$

Use 4x8

$$= \underset{DL}{\underline{420}} + \underset{SL}{\underline{390}}\ \#/\text{ft}$$

UFB9 UPPER FLOOR BEAM

SPAN = 12'-3"

$$w = \overset{RD}{(\underset{DL}{\underline{18}} + \underset{LL}{\underline{60}})}\left(\frac{7'\text{-}6''}{2}\right) + \overset{W}{(10)}(13') = \underset{DL}{\underline{200}} + \underset{LL}{\underline{230}}\ \#/\text{ft}$$

Use 5 1/4 x 11 7/8 PSL

$$P_1 = \overset{RH6}{\underset{DL}{\underline{520}} + \underset{SL}{\underline{720}}}\ \# \ @\ x = 1'\text{-}9'' + 10'\text{-}6''$$

(SW

$$P_2 = (\pm)\, 1.4 \times \frac{2.5}{1.3} \times \overset{\text{(ANCHOR TENSION)}}{4810\#} = \frac{12950}{\text{EQ LOAD}}\ \#\ \begin{array}{l}(+) @ x = 3'' + 10'\text{-}6' \\ (-) @ x = 1'\text{-}9'' + 12'\end{array}$$

UFH10 UPPER FLOOR HEADER

SPAN = 12'-0"

$$P = \overset{FR}{(\underset{DL}{\underline{18}} + \underset{SL}{\underline{25}})}\left(\frac{32'}{2}\right)\left(\frac{15'}{2}\right) = \underset{DL}{\underline{2160}} + \underset{SL}{\underline{3000}}\ \# \ @\ x = 6'$$

Use 5 1/4 x 9 1/2 PSL

MAIN FLOOR FRAMING

MFJ1 MAIN FLOOR JOISTS

SPAN = 14'6"

$$w = \underset{DL}{11} + \underset{LL}{40} \text{ psf}$$

USE 11 7/8 TJI 210 @ 16" o.c.

MFB2 MAIN FLOOR BEAM (WORST CASE BLW BEARING WALL)

SPAN = 8'-0" MAX

$$w = \left[\overset{UF}{\left(\underset{DL}{14} + \underset{LL}{40}\right)} + \overset{MF}{\left(\underset{DL}{11} + \underset{LL}{40}\right)}\right]\left(\frac{26'}{2}\right) = \underset{DL}{330} + \underset{LL}{1040} \text{ \#/ft}$$

USE 5 1/2 x 9 GLB

DECK FRAMING

(DJ1) DECK JOIST

SPAN = 11'-3" $w = \underset{a}{\underline{10}} + \underset{u}{\underline{60}}$ psf USE 2x10 PT @ 16" o.c.

(DB2) DECK BEAM

SPAN = 6'-6" MAX $w = \underset{a}{\underline{10}} + \underset{u}{\underline{60}}$ psf TRIB = $\frac{19'6"}{2}$ = 9'-9"

USE 6x10 PT

POSTS

(ELEMENTS NOT EXPLICITLY CALC'D OK BY INSPECTION)

MAIN FLOOR POST SUPPORTING EAST END OF UFB5

$$H = 10'0" \quad P = \left(\underset{DL}{2730} + \overset{UFB5}{\underset{LL}{2220}} + \underset{SL}{1670}\right) + \left(\overset{RB8}{\underset{DL}{1780} + \underset{SL}{2480}}\right)$$

USE 6x6

$$+ \left(\underset{PL}{2730} + \overset{UFB4}{\underset{LL}{2230}} + \underset{SL}{\cancel{790}} + \underset{EL}{6040}^{\#}\right) = \underset{DL}{7290} + \underset{LL}{4450} + \underset{SL}{4440} + \underset{EL}{6040}^{\#}$$

MAIN FLOOR POST SUPPORTING NORTH END OF UFB7

$$H = 10'0" \quad P = \overset{RB8}{\left(\underset{DL}{19} + \underset{SL}{25}\right)}\left(\frac{24'}{2}\right)\left(\frac{12'}{2}\right) + \overset{UFB7}{\left(\underset{DL}{1720} + \underset{SL}{1770} + \underset{EL}{7760}\right)}$$

USE 3½x5½ PSL

$$= \underset{DL}{3090} + \underset{SL}{3570} + \underset{EL}{7760}^{\#}$$

FOUNDATION

(ELEMENTS NOT EXPLICITLY CALC'D OK BY INSPECTION)

(F1) EXTERIOR CRAWLSPACE FOUNDATION

$$W = \overset{TR}{\left(\underbrace{19}_{DL}+\underbrace{25}_{SL}\right)}\left(\frac{27'}{2}\right)+\overset{UF}{\left(\underbrace{14}_{DL}+\underbrace{40}_{LL}\right)}\left(\frac{13'}{2}\right)+\overset{MF}{\left(\underbrace{11}_{DL}+\underbrace{40}_{LL}\right)}\left(\frac{12'}{2}\right)+\overset{W}{\left(\underbrace{10}_{DL}\right)}\left(20'\right)$$

$$= \underbrace{610}_{DL}+\underbrace{340}_{SL}+\underbrace{500}_{LL}\ \#/\text{ft}$$

$$W_{FACTORED} = 610 + 0.75\left(340+500\right) = 1240\ \#/\text{ft}$$

USE 16" WIDE FOOTING

GOOD FOR $\frac{16}{12} \times 2500 = 3333\ \#/\text{ft}$ ok

(F4) CRAWLSPACE PAD FOOTING

$$P = \overset{MFB2}{\left(\underbrace{330}_{DL}+\underbrace{1040}_{LL}\ \#/\text{ft}\right)}\left(8'\right) = \underbrace{2640}_{DL}+\overset{0.77\times}{\underbrace{8320}_{LL}}\ \# = 9050\ \#$$

$$A_T = \frac{8320\ \#}{40\text{psf}} = 208\text{ SF} \qquad LL_{RED} = \left(0.25+\frac{15}{\sqrt{\underset{4}{K_{LL}}A_T}}\right)L_o = 0.776$$

USE 2'-0" SQ. PAD GOOD FOR $2^2 \times \underset{\text{psf}}{2500} = 10000\ \#$ ok

(F5) CRAWLSPACE PAD FOOTING

$$P = \overset{MFB2}{\left(\underbrace{2640}_{DL}+0.77\times\underbrace{8320}_{LL}\right)}+\overset{PSL\ POST}{\left(\underbrace{3040}_{DL}+\underbrace{3570}_{SL}+\underbrace{7760}_{EL}\right)}$$

$$= \underbrace{5730}_{DL}+\underbrace{6410}_{LL}+\underbrace{3570}_{SL}+\underbrace{7760}_{EL}\ \#$$

$$P_{FACTORED} = 1.098\times5730 + 0.75\left(6410+3570\right) + 0.525\times7760$$

$$= 17850\ \#$$

USE 2'-6" SQ. PAD GOOD FOR $2.5^2 \times 2500 \times 1.33 = 20780\ \#$ ok

(F6) INTERIOR CRAWLSPACE FOUNDATION

$$P_{END} = \underbrace{\overset{(6\times6\ POST)}{18290^{\#}}}_{TOTAL,\ FACTORED} + \overset{MFB2\ LEFT}{\left(\underset{D}{11}+\underset{L}{40}\right)}\left(\frac{27'}{2}\right)\left(\frac{8'}{2}\right) + \overset{MFB2\ RIGHT}{\left(\underset{D}{330}+\underset{L}{1070}\right)}\left(\frac{8'}{2}\right)$$

$$P_{FACTORED} = 18290 + 1990 + 0.75 \times 6590 - (0.3)(4450+6590) = 21910^{\#}$$

USE 16" WIDE STRIP FOOTING

$$A_T = \frac{4450+6590}{40} = 276\,SF$$

$$GOOD\ FOR\ \frac{16}{12} \times 2500 \times 1.33 \times \underbrace{5'}_{TRIB\ LENGTH} = 22170^{\#}\ \underline{\underline{ok}}$$

$$L_{RED} = 0.7\,L_o$$

(F7) DECK PAD FOOTING

$$P = \left(\underset{D}{10}+\underset{L}{60}\right)\left(\frac{10'}{2}\right)\left(\frac{20'}{2}\right) = \underset{D}{500} + \underset{L}{3000}^{\#}$$

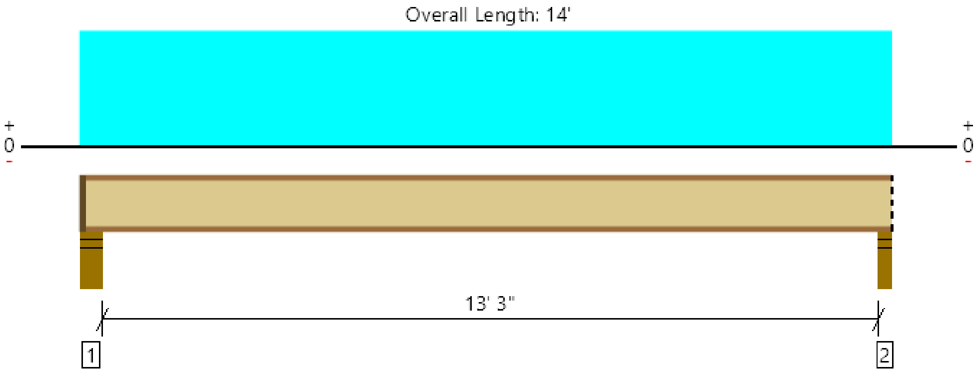
USE 18" SQ. PAD

$$GOOD\ FOR\ 1.5^2 \times 2500 = 5630^{\#}\ \underline{\underline{ok}}$$

MEMBER REPORT

PASSED

Upper Floor, UFJ1 - Upper Floor Joist
1 piece(s) 11 7/8" TJI® 210 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	501 @ 4 1/2"	1460 (3.50")	Passed (34%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	477 @ 5 1/2"	1655	Passed (29%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1620 @ 7' 1"	3795	Passed (43%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.122 @ 7' 1"	0.335	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.165 @ 7' 1"	0.671	Passed (L/978)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	57	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
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 structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Stud wall - HF	5.50"	4.00"	1.75"	132	378	510	1 1/2" Rim Board
2 - Stud wall - HF	3.50"	3.50"	1.75"	129	369	498	Blocking

- Rim Board is assumed to carry all loads applied directly above it, bypassing the member being designed.
- 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' 10" o/c	
Bottom Edge (Lu)	13' 11" o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 14'	16"	14.0	40.0	Upper Floor

Member Notes
UFJ1 - Upper Floor Joist

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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
Owen Gould O.G. Engineering, PLLC (206) 290-4608 owen@ogengineer.com	

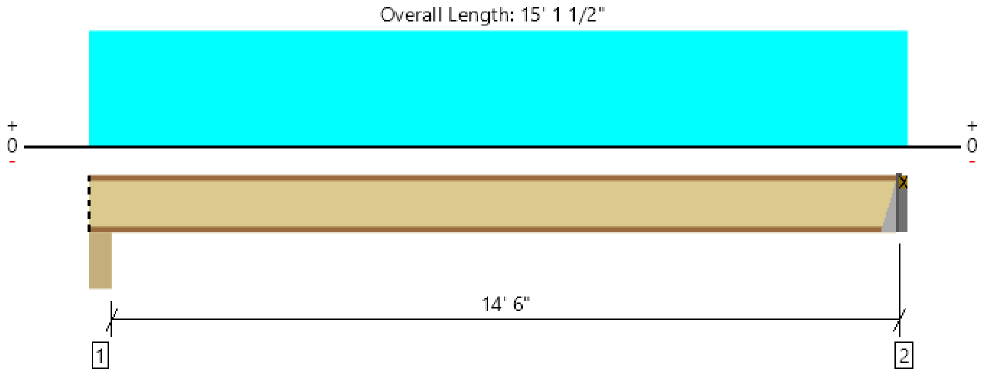
Weyerhaeuser

11/15/2022 8:29:32 PM UTC
ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0
File Name: 22004_Smersh

PASSED

MEMBER REPORT

Main Floor, MFJ1 - Main Floor Joist
1 piece(s) 11 7/8" TJI® 210 @ 16" OC



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

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	496 @ 14' 11 1/2"	1005 (1.75")	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	496 @ 14' 11 1/2"	1655	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1808 @ 7' 8"	3795	Passed (48%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.166 @ 7' 8"	0.365	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.211 @ 7' 8"	0.729	Passed (L/829)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	50	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2018
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 structural analysis of the deck has not been performed.
- Deflection analysis is based on composite action with a single layer of 23/32" Panel (24" Span Rating) that is glued and nailed down.
- Additional considerations for the TJ-Pro™ Rating include: 5/8" Gypsum ceiling.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Factored	
1 - Beam - GLB	5.50"	5.50"	1.75"	112	409	521	Blocking
2 - Hanger on Single 2X HF plate	2.00"	Hanger[1]	1.75" / - [2]	109	398	507	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.
- [2] Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 6" o/c	
Bottom Edge (Lu)	15' o/c	

- TJI joists are only analyzed using Maximum Allowable bracing solutions.
- Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie

Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Top Mount Hanger	Connector not found	N/A	N/A	N/A	N/A	

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

Vertical Load	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 1 1/2"	16"	11.0	40.0	Main Floor

Member Notes
MFJ1 - Main Floor Joist

ForteWEB Software Operator	Job Notes
Owen Gould O.G. Engineering, PLLC (206) 290-4608 owen@ogengineer.com	

Weyerhaeuser

11/17/2022 12:49:47 AM UTC
ForteWEB v3.4, Engine: V8.2.2.122, Data: V8.1.3.0
File Name: 22004_Smersh

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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
Owen Gould O.G. Engineering, PLLC (206) 290-4608 owen@ogengineer.com	

Description : Roof Framing

Wood Beam Design : RB5 - Roof Beam

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist			Wood Grade :	Parallam PSL 2.2E				
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,200.0 ksi	Density	45.070 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,118.19 ksi		

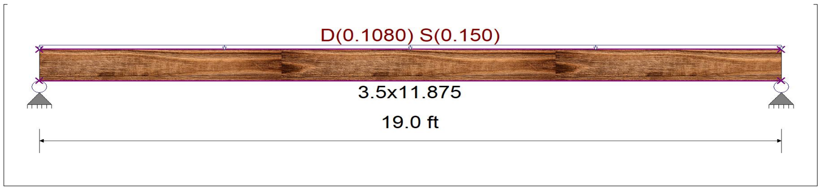
Applied Loads

Unif Load: D = 0.0180, S = 0.0250 k/ft, Trib= 6.0 ft

Design Summary

Max fb/Fb Ratio = **0.509** : 1
fb : Actual : 1,698.38 psi at 9.500 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.265** : 1
fv : Actual : 88.46 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.03			1.43			
Right Support	1.03			1.43			

Max Deflections

Transient Downward	0.412 in	Total Downward	0.708 in
Ratio	554	Ratio	322
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : RH6 - Transom Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x9.5, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species :	iLevel Truss Joist			Wood Grade :	Parallam PSL 2.2E				
Fb - Tension	2900 psi	Fc - Prll	2900 psi	Fv	290 psi	Ebend- xx	2200 ksi	Density	45.07 pcf
Fb - Compr	2900 psi	Fc - Perp	750 psi	Ft	2025 psi	Eminbend - xx	1118.19 ksi		

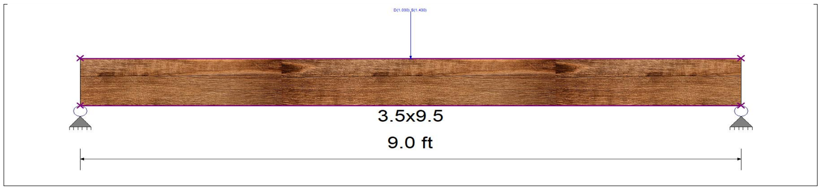
Applied Loads

1Point: D = 1.030, S = 1.430 k @ 4.50 ft

Design Summary

Max fb/Fb Ratio = **0.378** : 1
fb : Actual : 1,261.64 psi at 4.500 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.166** : 1
fv : Actual : 55.49 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.52			0.72			
Right Support	0.52			0.72			

Max Deflections

Transient Downward	0.069 in	Total Downward	0.118 in
Ratio	1575	Ratio	915
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : RH7 - Roof Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **4x8, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir-Larch Wood Grade : No.1

Fb - Tension	1000 psi	Fc - Prll	1500 psi	Fv	180 psi	Ebend- xx	1700 ksi	Density	31.21 pcf
Fb - Compr	1000 psi	Fc - Perp	625 psi	Ft	675 psi	Eminbend - xx	620 ksi		

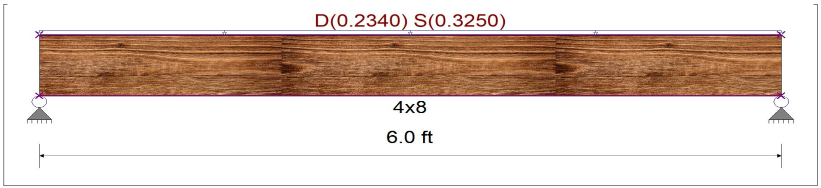
Applied Loads

Unif Load: D = 0.0180, S = 0.0250 k/ft, Trib= 13.0 ft

Design Summary

Max fb/Fb Ratio = **0.659** : 1
fb : Actual : 984.49 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,495.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.479** : 1
fv : Actual : 99.13 psi at 0.000 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.70			0.98			
Right Support	0.70			0.98			

Max Deflections

Transient Downward	0.050 in	Total Downward	0.087 in
Ratio	1427	Ratio	830
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : RB8 - Roof Beam

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **3.5x11.875, Parallam PSL, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist Wood Grade : Parallam PSL 2.2E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,200.0 ksi	Density	45.070 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	750.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,118.19 ksi		

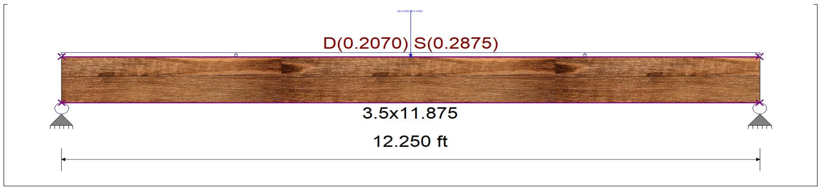
Applied Loads

Unif Load: D = 0.0180, S = 0.0250 k/ft, Trib= 11.50 ft
1Point: D = 1.030, S = 1.430 k @ 6.125 ft

Design Summary

Max fb/Fb Ratio = **0.735** : 1
fb : Actual : 2,452.18 psi at 6.125 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.461** : 1
fv : Actual : 153.70 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+S



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.78			2.48			
Right Support	1.78			2.48			

Max Deflections

Transient Downward	0.225 in	Total Downward	0.387 in
Ratio	653	Ratio	380
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : RB9 - Roof Beam

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

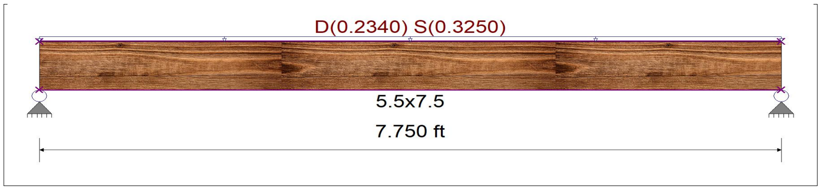
BEAM Size : **5.5x7.5, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : DF/DF
Wood Grade : 24F-V8

Fb - Tension	2,400.0 psi	Fc - Prll	1,650.0 psi	Fv	265.0 psi	Ebend- xx	1,800.0 ksi	Density	31.210 pcf
Fb - Compr	2,400.0 psi	Fc - Perp	650.0 psi	Ft	1,100.0 psi	Eminbend - xx	950.0 ksi		

Applied Loads

Unif Load: D = 0.0180, S = 0.0250 k/ft, Trib= 13.0 ft



Design Summary

Max fb/Fb Ratio = **0.354** : 1
fb : Actual : 976.73 psi at 3.875 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.258** : 1
fv : Actual : 78.77 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.91			1.26			
Right Support	0.91			1.26			

Max Deflections

Transient Downward	0.076 in	Total Downward	0.131 in
Ratio	1220	Ratio	709
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Description : Roof Deck Framing

Wood Beam Design : RDJ1 - Roof Deck Joists

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

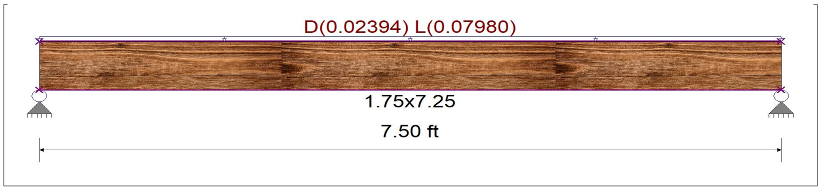
BEAM Size : **1.75x7.25, Microllam LVL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2600 psi	Fc - Prll	2510 psi	Fv	285 psi	Ebend- xx	2000 ksi	Density	42.01 pcf
Fb - Compr	2600 psi	Fc - Perp	750 psi	Ft	1555 psi	Eminbend - xx	1016.535 ksi		

Applied Loads

Unif Load: D = 0.0180, L = 0.060 k/ft, Trib= 1.330 ft

Design Summary



Max fb/Fb Ratio = **0.211** : 1
fb : Actual : 570.95 psi at 3.750 ft in Span # 1
Fb : Allowable : 2,704.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.161** : 1
fv : Actual : 45.99 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.09		0.30				
Right Support	0.09		0.30				

Max Deflections

Transient Downward	0.051 in	Total Downward	0.067 in
Ratio	1751	Ratio	1347
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : RDH2 - Roof Deck Header

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

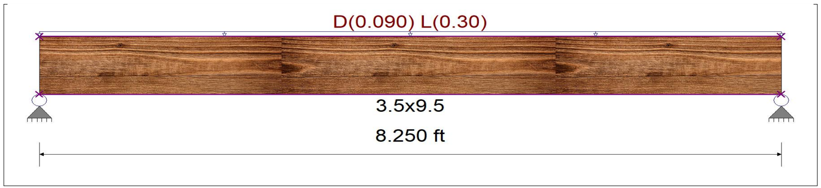
BEAM Size : **3.5x9.5, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist
Wood Grade : TimberStrand LSL 1.55E

Fb - Tension	2325 psi	Fc - Prll	2050 psi	Fv	310 psi	Ebend- xx	1550 ksi	Density	45.01 pcf
Fb - Compr	2325 psi	Fc - Perp	800 psi	Ft	1070 psi	Eminbend - xx	787.815 ksi		

Applied Loads

Unif Load: D = 0.0180, L = 0.060 k/ft, Trib= 5.0 ft

Design Summary



Max fb/Fb Ratio = **0.325** : 1
fb : Actual : 756.31 psi at 4.125 ft in Span # 1
Fb : Allowable : 2,325.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.234** : 1
fv : Actual : 72.58 psi at 0.000 ft in Span # 1
Fv : Allowable : 310.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.37		1.24				
Right Support	0.37		1.24				

Max Deflections

Transient Downward	0.081 in	Total Downward	0.105 in
Ratio	1220	Ratio	938
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Description : Main Floor Framing

Wood Beam Design : MFB2 - Main Floor Beam

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **5.5x9, GLB, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : DF/DF Wood Grade : 24F-V8

Fb - Tension	2400 psi	Fc - Prll	1650 psi	Fv	265 psi	Ebend- xx	1800 ksi	Density	31.21 pcf
Fb - Compr	2400 psi	Fc - Perp	650 psi	Ft	1100 psi	Eminbend - xx	950 ksi		

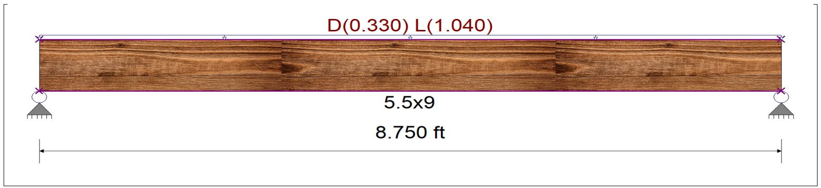
Applied Loads

Unif Load: D = 0.330, L = 1.040 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.883** : 1
fb : Actual : 2,119.00 psi at 4.375 ft in Span # 1
Fb : Allowable : 2,400.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.571** : 1
fv : Actual : 151.36 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L



Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	1.44		4.55				
Right Support	1.44		4.55				

Max Deflections

Transient Downward	0.229 in	Total Downward	0.302 in
Ratio	457	Ratio	347
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Description : Deck Framing

Wood Beam Design : DJ1 - Deck Joists

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **2x10, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

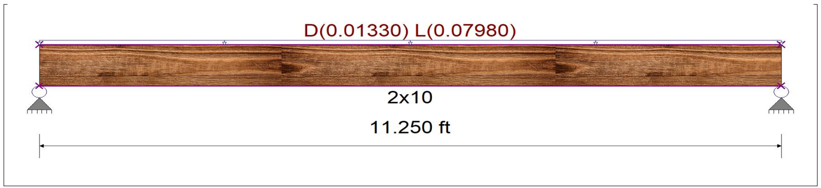
Wood Species : Hem-Fir Wood Grade : No.2

Fb - Tension	850.0 psi	Fc - Prll	1,300.0 psi	Fv	150.0 psi	Ebend- xx	1,300.0 ksi	Density	26.840 pcf
Fb - Compr	850.0 psi	Fc - Perp	405.0 psi	Ft	525.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.010, L = 0.060 k/ft, Trib= 1.330 ft

Design Summary



Max fb/Fb Ratio = **0.961** : 1
fb : Actual : 826.27 psi at 5.625 ft in Span # 1
Fb : Allowable : 860.20 psi
Load Comb : +D+L

Max fv/FvRatio = **0.486** : 1
fv : Actual : 56.61 psi at 11.250 ft in Span # 1
Fv : Allowable : 116.40 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.07		0.45				
Right Support	0.07		0.45				

Max Deflections

Transient Downward	0.250 in	Total Downward	0.291 in
Ratio	540	Ratio	463
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : DB2 - Deck Beam

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16

BEAM Size : **6x10, Sawn, Fully Braced**

Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

Wood Species : Hem-Fir Wood Grade : No.2

Fb - Tension	675 psi	Fc - Prll	500 psi	Fv	140 psi	Ebend- xx	1100 ksi	Density	26.84 pcf
Fb - Compr	675 psi	Fc - Perp	405 psi	Ft	350 psi	Eminbend - xx	400 ksi		

Applied Loads

Unif Load: D = 0.010, L = 0.060 k/ft, Trib= 9.750 ft

Design Summary

Max fb/Fb Ratio = **0.968** : 1
fb : Actual : 522.83 psi at 3.250 ft in Span # 1
Fb : Allowable : 540.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.569** : 1
fv : Actual : 63.68 psi at 6.500 ft in Span # 1
Fv : Allowable : 112.00 psi
Load Comb : +D+L

Max Reactions (k)	D	Lr	L	S	W	E	H
Left Support	0.32		1.90				
Right Support	0.32		1.90				

Max Deflections

Transient Downward	0.055 in	Total Downward	0.064 in
Ratio	1427	Ratio	1223
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Column

Project File: 22004_SmershRes_.ec6

LIC# : KW-06018000, Build:20.22.10.25 O.G. ENGINEERING, PLLC (c) ENERCALC INC 1983-2022

DESCRIPTION: Main Floor Post Supporting East End of UFB5

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Analysis Method	Allowable Stress Design	
End Fixities	Top & Bottom Pinned	
Overall Column Height		10 ft

(Used for non-slender calculations)

Wood Species	Douglas Fir-Larch		
Wood Grade	No.1		
Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	**6x6**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**5.50** in
Exact Depth	**5.50** in
Area	30.250 in^2
Ix	76.255 in^4
Iy	76.255 in^4

Allow Stress Modification Factors		
Cf or Cv for Bending	1.0	
Cf or Cv for Compressio	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Fact	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 10 ft, K
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K

Applied Loads

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

Column self weight included : 65.563 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 7.290, L = 4.450, S = 4.940, E = 6.040 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7323 : 1**

Load Combination	+1.098D+0.750L+0.750S+0.525E
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are .	
Applied Axial	18.287 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	825.53 psi

PASS Maximum Shear Stress Ratio = **0.0 : 1**

Load Combination	+0.4698D+0.70E
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a

Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

			Maximum Axial + Bending Stress Ratios			Maximum Shear Ratios		
Load Combination	C_D	C_P	Stress Ratio	Status	Location	Stress Ratio	Status	Location
D Only	0.900	0.727	0.3718	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+L	1.000	0.692	0.5644	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	0.641	0.5510	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750L	1.250	0.610	0.4634	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750L+0.750S	1.150	0.641	0.6453	PASS	0.0 ft	0.0	PASS	10.0 ft
+1.130D+0.70E	1.600	0.516	0.5022	PASS	0.0 ft	0.0	PASS	10.0 ft
+1.098D+0.750L+0.750S+0.525E	1.600	0.516	0.7323	PASS	0.0 ft	0.0	PASS	10.0 ft
+0.60D	1.600	0.516	0.1767	PASS	0.0 ft	0.0	PASS	10.0 ft
+0.4698D+0.70E	1.600	0.516	0.3077	PASS	0.0 ft	0.0	PASS	10.0 ft

Wood Column

Project File: 22004_SmershRes_.ec6

DESCRIPTION: Main Floor Post Supporting East End of UFB5

Maximum Reactions

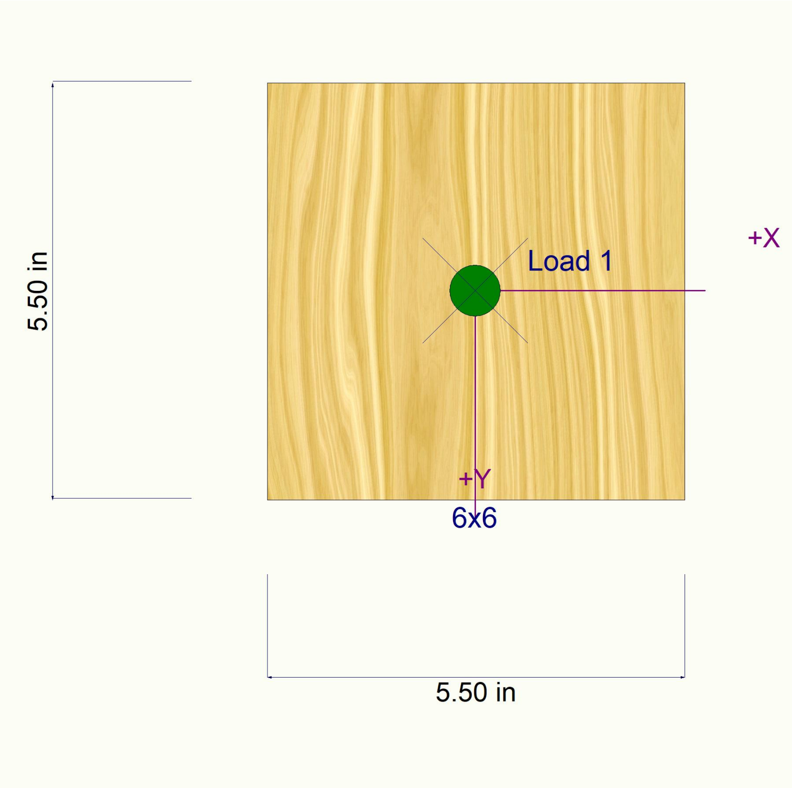
Note: Only non-zero reactions are listed.

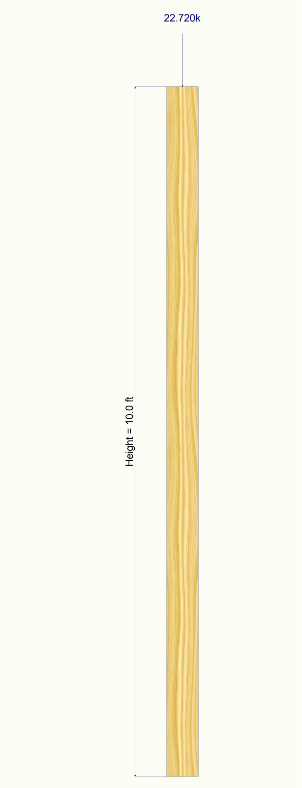
Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments k-ft @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only					7.356				
+D+L					11.806				
+D+S					12.296				
+D+0.750L					10.693				
+D+0.750L+0.750S					14.398				
+D+0.70E					11.584				
+D+0.750L+0.750S+0.5250E					17.569				
+0.60D					4.413				
+0.60D+0.70E					8.641				
L Only					4.450				
S Only					4.940				
E Only					6.040				

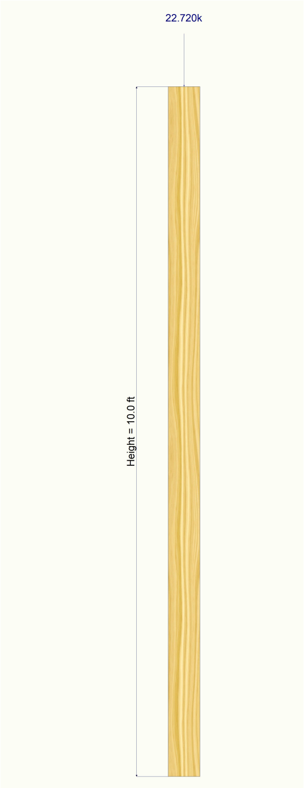
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.5250E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches







Wood Column

Project File: 22004_SmershRes_.ec6

LIC# : KW-06018000, Build:20.22.10.25 O.G. ENGINEERING, PLLC (c) ENERCALC INC 1983-2022

DESCRIPTION: Main Floor Post Supporting South End of UFB7

Code References

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Analysis Method	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height		10	ft

(Used for non-slender calculations)

Wood Species	iLevel Truss Joist		
Wood Grade	Parallam PSL 1.8E		

Fb +	2,400.0 psi	Fv	190.0	psi
Fb -	2,400.0 psi	Ft	1,755.0	psi
Fc - Prll	2,500.0 psi	Density	45.070	pcf
Fc - Perp	425.0 psi			

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,800.0	1,800.0	1,800.0 ksi
Minimum	914.88	914.88	

Wood Section Name	**3.5x5.25**	
Wood Grading/Manuf.	Trus Joist	
Wood Member Type	Parallam PSL	
Exact Width	**3.50**	in
Exact Depth	**5.250**	in
Area	18.375	in^2
Ix	42.205	in^4
Iy	18.758	in^4

Allow Stress Modification Factors		
Cf or Cv for Bending	1.0	
Cf or Cv for Compressio	1.0	
Cf or Cv for Tension	1.0	
Cm : Wet Use Factor	1.0	
Ct : Temperature Fact	1.0	
Cfu : Flat Use Factor	1.0	
Kf : Built-up columns	1.0	*NDS 15.3.2*
Use Cr : Repetitive ?	No	

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 10 ft, K
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K

Applied Loads

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

Column self weight included : 57.511 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 3.090, S = 3.570, E = 7.760 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.8844 : 1**
	Load Combination	+1.098D+0.750S+0.5250E
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are .	
	Applied Axial	10.206 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	628.05 psi
PASS	Maximum Shear Stress Ratio =	**0.0 : 1**
	Load Combination	+0.4698D+0.70E
	Location of max.above base	10.0 ft
	Applied Design Shear	0.0 psi
	Allowable Shear	304.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a

Along X-X 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios Stress Ratio	Status	Location	Maximum Shear Ratios Stress Ratio	Status	Location
D Only	0.900	0.274	0.2779	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+S	1.150	0.217	0.5872	PASS	0.0 ft	0.0	PASS	10.0 ft
+D+0.750S	1.150	0.217	0.5092	PASS	0.0 ft	0.0	PASS	10.0 ft
+1.130D+0.70E	1.600	0.157	0.7789	PASS	0.0 ft	0.0	PASS	10.0 ft
+1.098D+0.750S+0.5250E	1.600	0.157	0.8844	PASS	0.0 ft	0.0	PASS	10.0 ft
+0.60D	1.600	0.157	0.1636	PASS	0.0 ft	0.0	PASS	10.0 ft
+0.4698D+0.70E	1.600	0.157	0.5988	PASS	0.0 ft	0.0	PASS	10.0 ft

DESCRIPTION: Main Floor Post Supporting South End of UFB7

Maximum Reactions

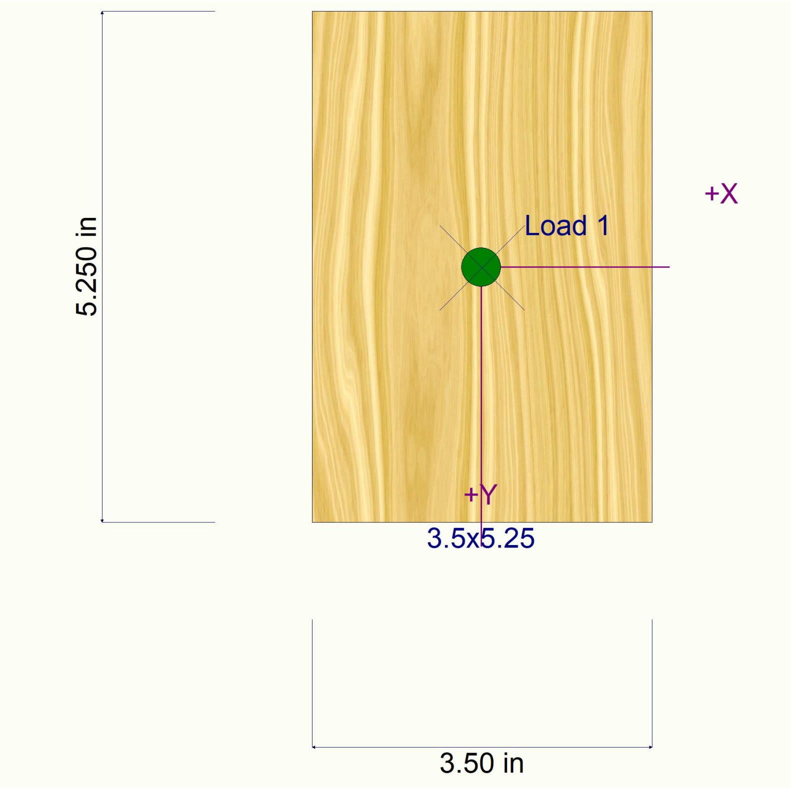
Note: Only non-zero reactions are listed.

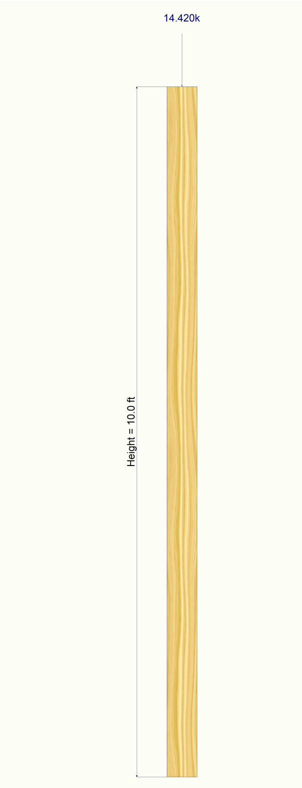
Load Combination	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments @ Base	k-ft @ Top	Mx - End Moments @ Base	@ Top
D Only					3.148				
+D+S					6.718				
+D+0.750S					5.825				
+D+0.70E					8.580				
+D+0.750S+0.5250E					9.899				
+0.60D					1.889				
+0.60D+0.70E					7.321				
S Only					3.570				
E Only					7.760				

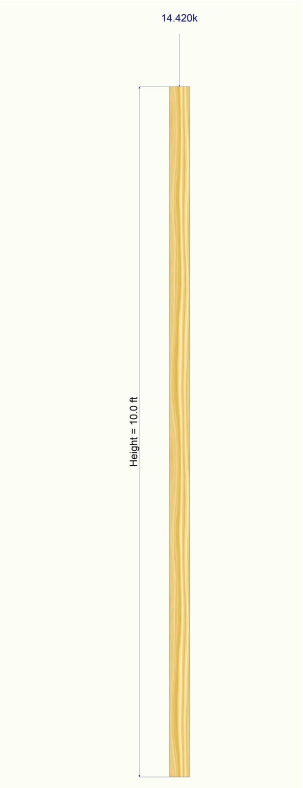
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

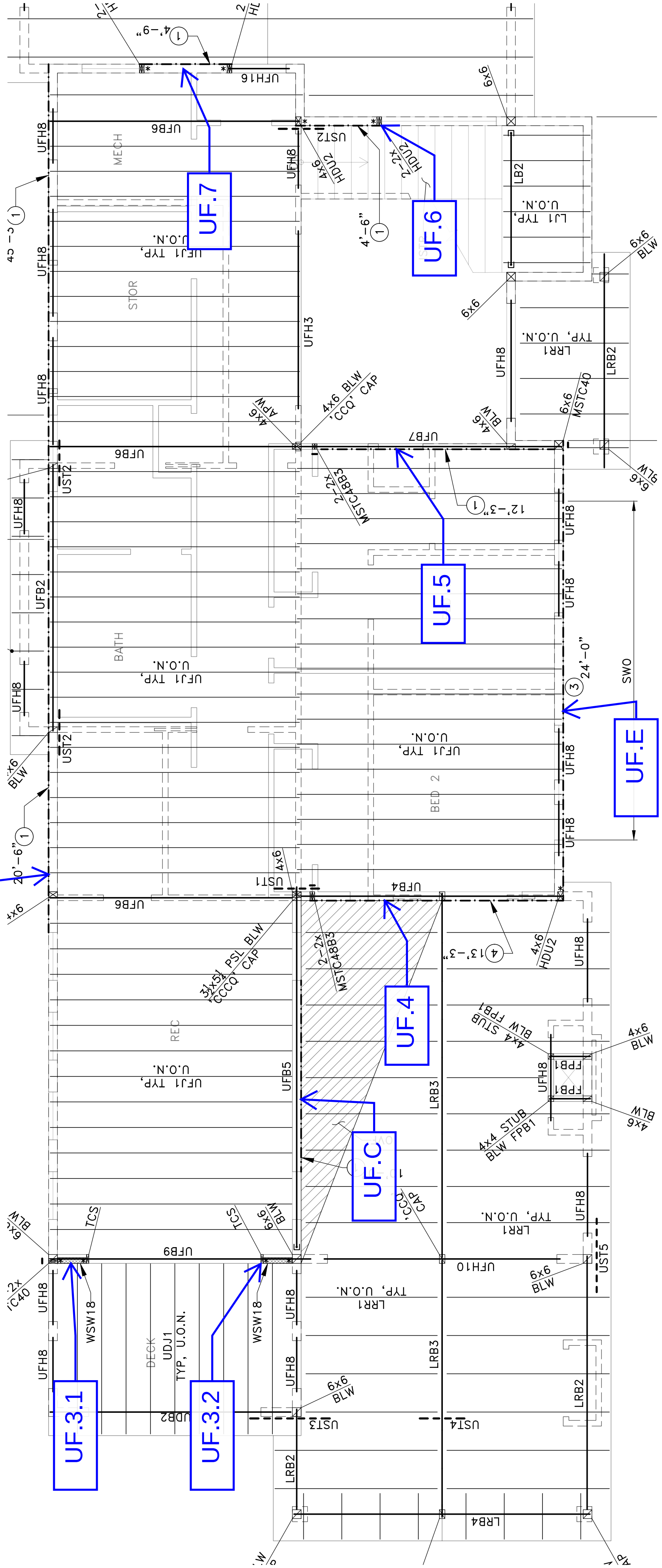
Sketches

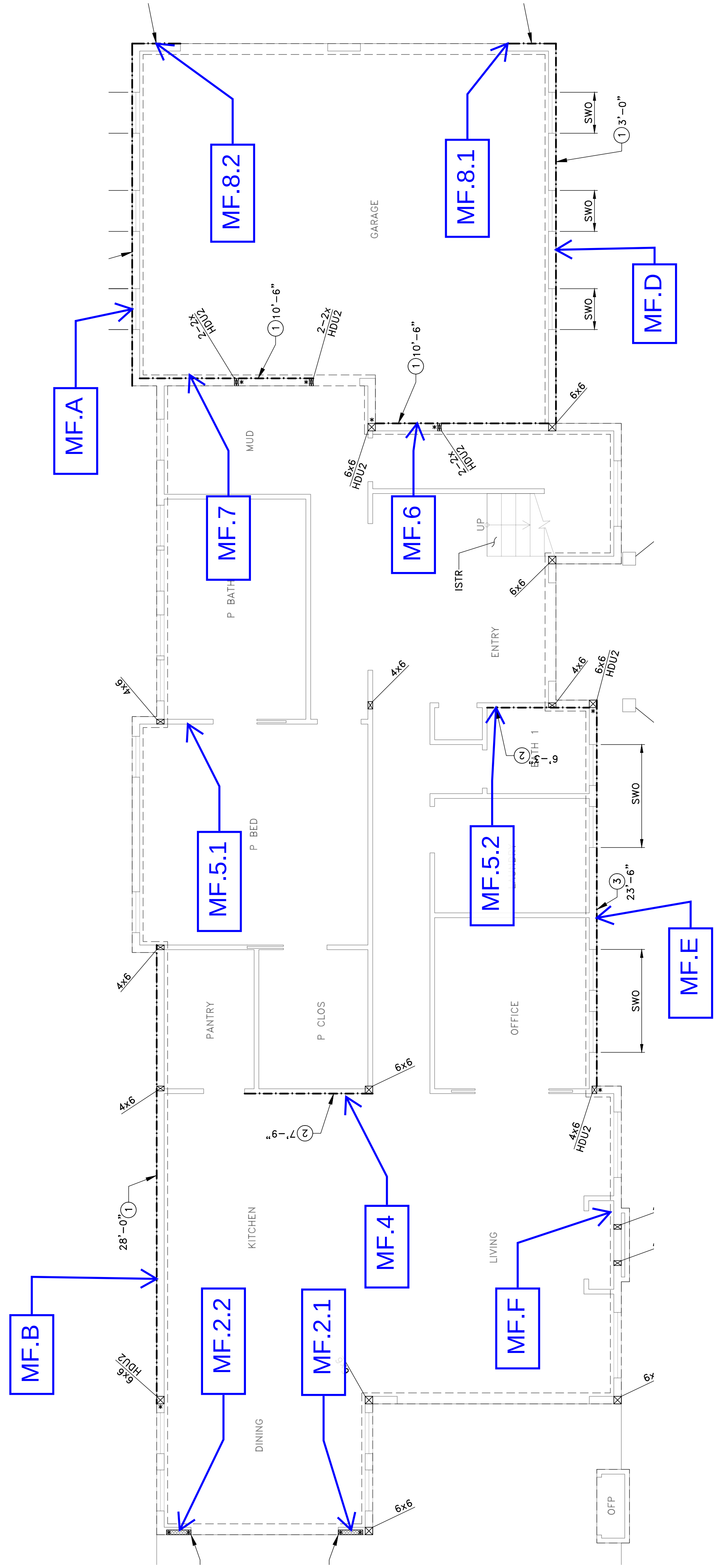






UPPER FLOOR SHEAR WALL KEY PLAN





MAIN FLOOR SHEAR WALL KEY PLAN

Plywood Shear Wall Design

Refer to Shear Wall Key Plans

Story Forces - ASD Level	
Floor	F_x (psf)
Roof	7.1
UF	4.2

Plywood Grade
CD-X

Struct 1 or CD-X

15/32" Plywood, w/ 10d nails, min. 1-1/2" penetration into framing members

R_d (Dead Load Resistance Factor) = 0.6-0.14S_{ds} = 0.47

Wall Mark Capacity (Grade Struct 1)		
Wall Mark	Edge Nailing	Capacity (plf)
1	6"o.c.	340
2	4"o.c.	510
3	3"o.c.	665
4	2"o.c.	870
Dbl 2	4"o.c. Both Sides	1020
Dbl 3	3"o.c. Both Sides	1330
Dbl 4	2"o.c. Both Sides	1740

Wall Mark Capacity (Grade CD-X)		
Wall Mark	Edge Nailing	Capacity (plf)
1	6"o.c.	310
2	4"o.c.	460
3	3"o.c.	600
4	2"o.c.	770
Dbl 2	4"o.c. Both Sides	920
Dbl 3	3"o.c. Both Sides	1200
Dbl 4	2"o.c. Both Sides	1540

Holdown Schedule	
Holdown	Capacity (lb)
HDU2	3075
HDU4	4565
HDU5	5645
HDU8	7870
MSTC40	2690
MSTC52	4225
MSTC66	5850

Notes

1) $Wall_{abv}$ = Shear wall on story above that adds shear to subject wall
2) V_{abv} = Shear demand from wall on story above
3) V_{cur} = Shear demand from current story = A_T x F_x
4) V = Total shear demand in wall = V_{abv} + V_{cur}
5) v = unit shear demand = V / L
6) Allowable shear reduction multiplier of 2xL/h for walls w/ h>2L (=1 if h<2L)
7) OTM = Wall overturning moment = V x h
8) w_{DL} = Distributed resisting dead load on top of wall
9) $P_{DL,END}$ = Minimum resisting point dead load on end of wall
10) RM = Resisting Moment from w_{DL} & $P_{DL,END}$, mulitplied by R_d above
11) T_{end} = Tension at end of wall from current story shear = (OTM - RM) / L
12) T_{abv} = Tension from wall holdown on story above
13) T = T_{end} + T_{abv}

Roof Diaphragm

Walls in North-South Direction												
Wall	L (ft)	h (ft)	A_T (sf)	$Wall_{abv}{}^1$	$V_{abv}{}^2$ (lbs)	$V_{cur}{}^3$ (lbs)	V^4 (lb)	v^5 (plf)	Wall Mark	h>2L?	$2xL/h^6$	Capacity (plf)
UF.3.1/2	WSW18	10	80	none	0	571	571	N/A	WSW18	N/A	N/A	1715 lbs
UF.4*	13.25	9	490	none	0	3500	3500	984	DBL 3	no	1	1200
UF.5	12.25	9	590	none	0	4214	4214	344	2	no	1	460
UF.6	4.5	9	140	none	0	1000	1000	222	1	no	1	310
UF.7	4.75	9	150	none	0	1071	1071	226	1	no	1	310

Holdowns for Walls in North-South Direction									
Wall	OTM^7 (lb-ft)	$w_{DL}{}^8$ (plf)	$P_{DL,END}{}^9$ (lb)	RM^{10} (lb-ft)	$T_{end}{}^{11}$ (lb)	$T_{abv}{}^{12}$ (lb)	T^{13} (lb)	Holdown	Capacity
UF.3.1/2	5714			N/A	4811		4811	PL0.229x3SQ	8310
UF.4*	31500	324		13354	1370		1370	HDU2	3075
UF.5	37929	324		11414	2164		2164	HDU2	3075
UF.6	9000	120		570	1873		1873	HDU2	3075
UF.7	9643	120		636	1896		1896	HDU2	3075

Bearing capacity of plate washer governs

Walls in East-West Direction												
Wall	L (ft)	h (ft)	A_T (sf)	$Wall_{abv}{}^1$	$V_{abv}{}^2$ (lbs)	$V_{cur}{}^3$ (lbs)	V^4 (lb)	v^5 (plf)	Wall Mark	h>2L?	$2xL/h^6$	Capacity (plf)
UF.B*	45.25	9	780	none	0	5571	5571	297	1	no	1	310
UF.C	10	9	140	none	0	1000	1000	100	1	no	1	310
UF.E*	24	9	600	none	0	4286	4286	430	2	no	1	460

Holdowns for Walls in East-West Direction									
Wall	OTM^7 (lb-ft)	$w_{DL}{}^8$ (plf)	$P_{DL,END}{}^9$ (lb)	RM^{10} (lb-ft)	$T_{end}{}^{11}$ (lb)	$T_{abv}{}^{12}$ (lb)	T^{13} (lb)	Holdown	Capacity
UF.B*	50143	120		57682	-167		-167	NONE	#N/A
UF.C	9000	162		3803	520		520	MSTC48B3	3380
UF.E*	38572	120		16227	931		931	HDU2	3075

Upper Floor Diaphragm

Walls in North-South Direction												
Wall	L (ft)	h (ft)	A_T (sf)	$Wall_{abv}{}^{1}$	$V_{abv}{}^{2}$ (lbs)	$V_{cur}{}^{3}$ (lbs)	V^4 (lb)	v^5 (plf)	Wall Mark	h>2L?	$2xL/h^6$	Capacity (plf)
MF.2.1/2	WSW24	8	260	UF.3	286	1098	1384	N/A	WSW24	N/A	N/A	4945 LBS
MF.4	7.75	10	730	UF.3&4	4071	3083	7154	923	DBL 3	no	1	1200
MF.5.1	6.25	10	333	UF.5	2341	1408	3749	600	3	no	1	600
MF.5.2	5	10	267	UF.5	1873	1126	2999	600	3	no	1	600
MF.6	10.5	10	310	UF.6	1000	1309	2309	220	1	no	1	310
MF.7	10.5	10	310	UF.7	1071	1309	2380	227	1	no	1	310
MF.8.1	3	10	140	none	0	591	591	197	2	yes	0.60	276
MF.8.2	3	10	140	none	0	591	591	197	2	yes	0.60	276

Holdowns for Walls in North-South Direction										
Wall	OTM^7 (lb-ft)	$w_{DL}{}^{8}$ (plf)	$P_{DL,END}{}^{9}$ (lb)	RM^{10} (lb-ft)	$T_{end}{}^{11}$ (lb)	$T_{abv}{}^{12}$ (lb)	T^{13} (lb)	Holdown	Capacity	
MF.2.1/2	11069	N/A	N/A	N/A	6814		6814	SB1x30	8315	*Use in lieu of strong wall anchor*
MF.4	71540	70		987	9104		9104	HDU11	9335	
MF.5.1	37488	70		642	5895	2,164	8060	HDU8	7870	*Close enough*
MF.5.2	29991	70		411	5916		5916	HDU5	5645	*Close enough*
MF.6	23090	70		1812	2027	1,873	3900	HDU4	4565	
MF.7	23805	70		1812	2095	1,896	3991	HDU4	4565	
MF.8.1	5912			0	1971		1971	HDU2	3075	
MF.8.2	5912			0	1971		1971	HDU2	3075	

Walls in East-West Direction												
Wall	L (ft)	h (ft)	A_T (sf)	$Wall_{abv}{}^{1}$	$V_{abv}{}^{2}$ (lbs)	$V_{cur}{}^{3}$ (lbs)	V^4 (lb)	v^5 (plf)	Wall Mark	h>2L?	$2xL/h^6$	Capacity (plf)
MF.A*	21	10	449	UF.B&C	1975	1896	3871	321	2	no	1	460
MF.B	28	10	931	UF.B&C	4096	3932	8028	287	2	no	1	460
MF.D*	23.25	10	621	UF.C&E	2154	2622	4776	358	2	no	1	460
MF.E*	23.5	10	434	UF.C&E	1504	1831	3336	348	2	no	1	460
MF.F	8.25	10	325	UF.C&E	1128	1374	2502	303	2	no	1	460

Holdowns for Walls in East-West Direction										
Wall	OTM^7 (lb-ft)	$w_{DL}{}^{8}$ (plf)	$P_{DL,END}{}^{9}$ (lb)	RM^{10} (lb-ft)	$T_{end}{}^{11}$ (lb)	$T_{abv}{}^{12}$ (lb)	T^{13} (lb)	Holdown	Capacity	
MF.B	80281	194		35706	1592		1592	HDU2	3075	
MF.A*	38707	366		37892	39		39	NONE	#N/A	*Close enough*
MF.F	25016	181		2892	2682		2682	HDU2	3075	
MF.E*	33355	208		26966	272	931	1203	HDU2	3075	
MF.D*	47758	366		46446	56		56	NONE	#N/A	*Close enough*

**Shear wall with force-transfer around openings; see additional spreadsheet to follow.*

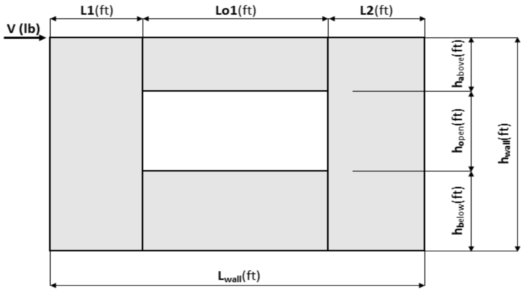
Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

Code: **Date:**
Designer:
Client:
Project:
Wall Line: UF.C



Shear Wall Calculation Variables

V	3500 lbf		Opening 1	Adj. Factor Method =	2bs/h	
L1	2.00 ft	h_a	1.40 ft	Wall Pier Aspect Ratio		Adj. Factor
L2	2.00 ft	h_o	4.50 ft	P1=h_o/L1=	2.25	0.889
h_{wall}	9.40 ft	h_b	3.50 ft	P2=h_o/L2=	2.25	0.889
L_{wall}	13.00 ft	Lo1	9.00 ft			

1. Hold-down forces: H = Vh_{wall}/L_{wall} 2531 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a+h_b) = 516 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 4648 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 2324 lbf
F2 = O1(L2)/(L1+L2) = 2324 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 4.50 ft
T2 = (L2*Lo1)/(L1+L2) = 4.50 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 875 plf
v2 = (V/L)(T2+L2)/L2 = 875 plf
Check v1*L1+v2*L2=V? 3500 lbf **OK**

7. Resistance to corner forces

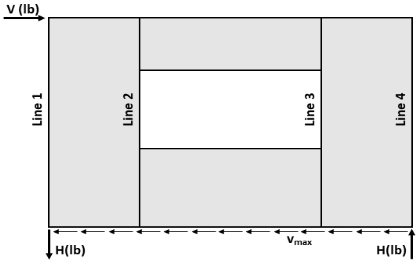
R1 = v1*L1 = 1750 lbf
R2 = v2*L2 = 1750 lbf

8. Difference corner force + resistance

R1-F1 = -574 lbf
R2-F2 = -574 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -287 plf
vc2 = (R2-F2)/L2 = -287 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(h_a+h_b)+v1(h_o)=H?		-1407	3938	2531 lbf
Line 2: va1(h_a+h_b)-vc1(h_a+h_b)-v1(h_o)=0?	2531	-1407	3938	0
Line 3: va1(h_a+h_b)-vc2(h_a+h_b)-v1(h_o)=0?	2531	-1407	3938	0
Line 4: vc2(h_a+h_b)+v2(h_o)=H?		-1407	3938	2531 lbf

Design Summary*

Req. Sheathing Capacity	984 plf	**
Req. Strap Force	2324 lbf	
Req. HD Force (H)	2531 lbf	
Req. Shear Wall Anchorage Force (v_{max})	269 plf	

****Req. Sheathing Capacity has been adjusted per the Aspect Ratio Adjustment Factor**

***The Design Summary assumes that the shear wall is designed as blocked.**

Force Transfer Around Openings Calculator

ONE OPENING

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

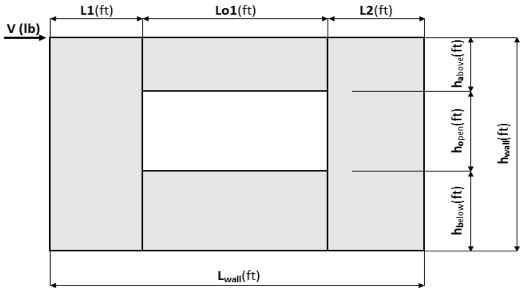
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.B



Shear Wall Calculation Variables

V	5571 lbf		Opening 1	
L1	18.25 ft	h_a	1.40 ft	
L2	20.50 ft	h_o	5.50 ft	
h_{wall}	9.40 ft	h_b	2.50 ft	
L_{wall}	45.25 ft	Lo1	6.50 ft	

Adj. Factor Method = 2bs/h

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	0.30	N/A
P2=h_o/L2=	0.27	N/A

1. Hold-down forces: H = Vh

Force Transfer Around Openings Calculator

TWO OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

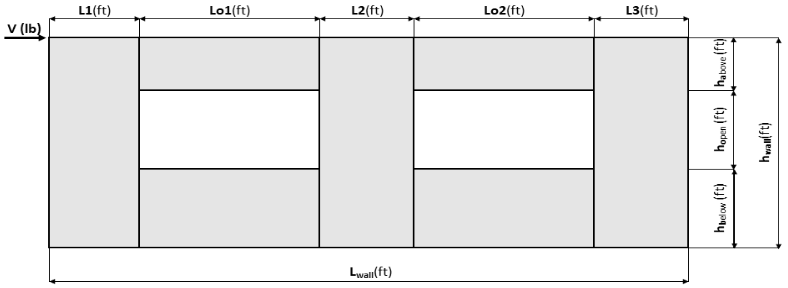
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.E



Shear Wall Calculation Variables

			Opening 1		Opening 2	Adj. Factor Method =		2bs/h
						Wall Pier Aspect Ratio		Adj. Factor
V	4286 lbf	h_a1	1.40 ft	h_a2	1.40 ft	P1=h_o/L1=	1.69	N/A
L1	3.25 ft	h_o1	5.50 ft	h_o2	5.50 ft	P2=h_o/L2=	0.96	N/A
L2	5.75 ft	h_b1	2.50 ft	h_b2	2.50 ft	P3=h_o/L3=	1.83	N/A
L3	3.00 ft	Lo1	6.00 ft	Lo2	6.00 ft			
h_{wall}	9.40 ft							
L_{wall}	24.00 ft							

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1679 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a1+h_b1) = 430 plf

Second opening: va2 = vb2 = H/(h_a2+h_b2) = 430 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2582 lbf

Second opening: O2 = va2 x (Lo2) = 2582 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 933 lbf

F2 = O1(L2)/(L1+L2) = 1650 lbf

F3 = O2(L2)/(L2+L3) = 1697 lbf

F4 = O2(L3)/(L2+L3) = 885 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.17 ft

T2 = (L2*Lo1)/(L1+L2) = 3.83 ft

T3 = (L2*Lo2)/(L2+L3) = 3.94 ft

T4 = (L3*Lo2)/(L2+L3) = 2.06 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 298 plf

v2 = (V/L)(T2+L2+T3)/L2 = 420 plf

v3 = (V/L)(T4+L3)/L3 = 301 plf

Check v1*L1+v2*L2+v3*L3=V? 4286 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 967 lbf

R2 = v2*L2 = 2415 lbf

R3 = v3*L3 = 903 lbf

8. Difference corner force + resistance

R1-F1 = 35 lbf

R2-F2-F3 = -932 lbf

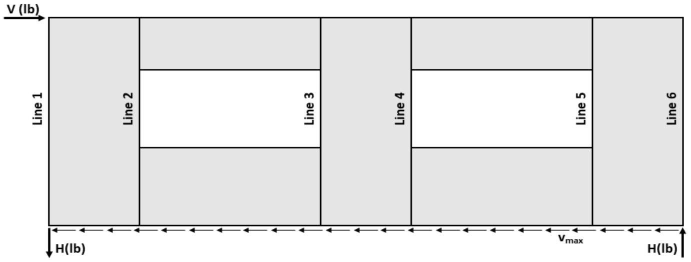
R3-F4 = 18 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 11 plf

vc2 = (R2-F2-F3)/L2 = -162 plf

vc3 = (R3-F4)/L3 = 6 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(h_a1+h_b1)+v1(h_o1)=H?		42	1637	1679 lbf
Line 2: va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?	1679	42	1637	0
Line 3: vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?	-632	2310	1679	0
Line 4: va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?	1679	2310	-632	0
Line 5: va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?	1679	23	1656	0
Line 6: vc3(h_a2+h_b2)+ v3(h_o2) = H?		23	1656	1679 lbf

Design Summary*

Req. Sheathing Capacity	430 plf
Req. Strap Force	1697 lbf
Req. HD Force	1679 lbf
Req. Shear Wall Anchorage Force	179 plf

***The Design Summary assumes that the shear wall is designed as blocked.**

Force Transfer Around Openings Calculator

THREE OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

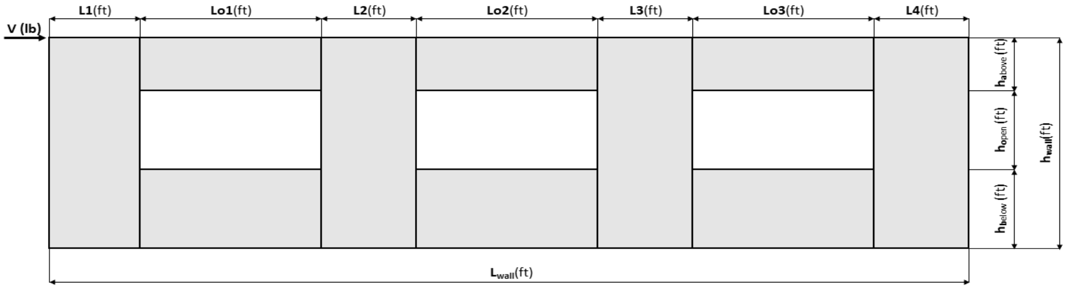
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.A



Shear Wall Calculation Variables

V	3871 lbf		Opening 1		Opening 2		Opening 3
L1	3.00 ft	h_a1	2.00 ft	h_a2	2.00 ft	h_a3	2.00 ft
L2	3.50 ft	h_o1	2.50 ft	h_o2	2.50 ft	h_o3	2.50 ft
L3	3.50 ft	h_b1	5.50 ft	h_b2	5.50 ft	h_b3	5.50 ft
L4	3.50 ft	Lo1	2.50 ft	Lo2	2.50 ft	Lo3	2.50 ft
h_{wall}	10.00 ft						
L_{wall}	21.00 ft						

Adj. Factor Method = 2bs/h

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	0.83	N/A
P2=h_o/L2=	0.71	N/A
P3=h_o/L3=	0.71	N/A
P4=h_o/L4=	0.71	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1843 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a1+h_b1) = 246 plf

Second opening: va2 = vb2 = H/(h_a2+h_b2) = 246 plf

Third opening: va3 = vb3 = H/(h_a3+h_b3) = 246 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 614 lbf

Second opening: O2 = va2 x (Lo2) = 614 lbf

Third opening: O3 = va3 x (Lo3) = 614 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 284 lbf

F2 = O1(L2)/(L1+L2) = 331 lbf

F3 = O2(L2)/(L2+L3) = 307 lbf

F4 = O2(L3)/(L2+L3) = 307 lbf

F5 = O3(L3)/(L3+L4) = 307 lbf

F6 = O3(L4)/(L3+L4) = 307 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.15 ft

T2 = (L2*Lo1)/(L1+L2) = 1.35 ft

T3 = (L2*Lo2)/(L2+L3) = 1.25 ft

T4 = (L3*Lo2)/(L2+L3) = 1.25 ft

T5 = (L3*Lo3)/(L3+L4) = 1.25 ft

T6 = (L4*Lo3)/(L3+L4) = 1.25 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 255 plf

v2 = (V/L)(T2+L2+T3)/L2 = 321 plf

v3 = (V/L)(T4+L3+T5)/L3 = 316 plf

v4 = (V/L)(T6+L4)/L4 = 250 plf

Check v1*L1+v2*L2+v3*L3+v4*L4=V? 3871 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 766 lbf

R2 = v2*L2 = 1124 lbf

R3 = v3*L3 = 1106 lbf

R4 = v4*L4 = 876 lbf

8. Difference corner force + resistance

R1-F1 = 482 lbf

R2-F2-F3 = 486 lbf

R3-F4-F5 = 492 lbf

R4-F6 = 568 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 161 plf

vc2 = (R2-F2-F3)/L2 = 139 plf

vc3 = (R3-F4-F5)/L3 = 140 plf

vc4 = (R4-F6)/L4 = 162 plf

Project Information

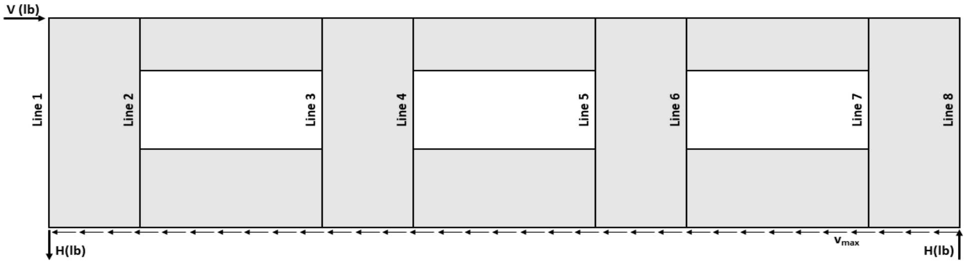
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.A



Check Summary of Shear Values for Three Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H?$		1205	638	1843 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?$	1843	1205	638	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?$	1041	803	1843	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?$	1843	803	1041	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?$	1843	1053	790	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0?$	1843	790	1053	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0?$	1843	1218	625	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H?$		1218	625	1843 lbf

Design Summary*

Req. Sheathing Capacity	321 plf
Req. Strap Force	331 lbf
Req. HD Force (H)	1843 lbf
Req. Shear Wall Anchorage Force (v_{max})	184 plf

***The Design Summary assumes that the shear wall is designed as blocked.**

APA

Force Transfer Around Openings Calculator

THREE OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

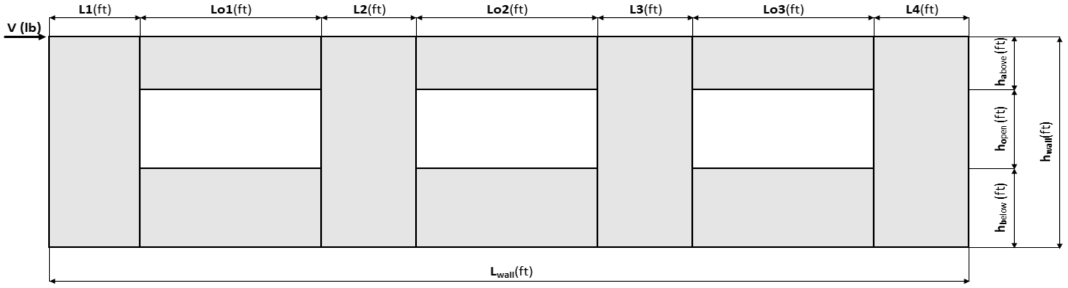
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.3.3



Shear Wall Calculation Variables

V	4776 lbf	Opening 1		Opening 2		Opening 3	
L1	5.75 ft	h_a1	2.00 ft	h_a2	2.00 ft	h_a3	2.00 ft
L2	3.50 ft	h_o1	2.50 ft	h_o2	2.50 ft	h_o3	2.50 ft
L3	3.50 ft	h_b1	5.50 ft	h_b2	5.50 ft	h_b3	5.50 ft
L4	3.00 ft	Lo1	2.50 ft	Lo2	2.50 ft	Lo3	2.50 ft
h_{wall}	10.00 ft						
L_{wall}	23.25 ft						

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
$P1=h_o/L1=$	0.43	N/A
$P2=h_o/L2=$	0.71	N/A
$P3=h_o/L3=$	0.71	N/A
$P4=h_o/L4=$	0.83	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 2054 lbf

2. Unit shear above + below opening

First opening: $va1 = vb1 = H/(h_a1+h_b1) =$ 274 plf

Second opening: $va2 = vb2 = H/(h_a2+h_b2) =$ 274 plf

Third opening: $va3 = vb3 = H/(h_a3+h_b3) =$ 274 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 685 lbf

Second opening: O2 = va2 x (Lo2) = 685 lbf

Third opening: O3 = va3 x (Lo3) = 685 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 426 lbf

F2 = O1(L2)/(L1+L2) = 259 lbf

F3 = O2(L2)/(L2+L3) = 342 lbf

F4 = O2(L3)/(L2+L3) = 342 lbf

F5 = O3(L3)/(L3+L4) = 369 lbf

F6 = O3(L4)/(L3+L4) = 316 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.55 ft

T2 = (L2*Lo1)/(L1+L2) = 0.95 ft

T3 = (L2*Lo2)/(L2+L3) = 1.25 ft

T4 = (L3*Lo2)/(L2+L3) = 1.25 ft

T5 = (L3*Lo3)/(L3+L4) = 1.35 ft

T6 = (L4*Lo3)/(L3+L4) = 1.15 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 261 plf

v2 = (V/L)(T2+L2+T3)/L2 = 334 plf

v3 = (V/L)(T4+L3+T5)/L3 = 358 plf

v4 = (V/L)(T6+L4)/L4 = 284 plf

Check v1*L1+v2*L2+v3*L3+v4*L4=V? 4776 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 1500 lbf

R2 = v2*L2 = 1170 lbf

R3 = v3*L3 = 1252 lbf

R4 = v4*L4 = 853 lbf

8. Difference corner force + resistance

R1-F1 = 1075 lbf

R2-F2-F3 = 569 lbf

R3-F4-F5 = 541 lbf

R4-F6 = 537 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 187 plf

vc2 = (R2-F2-F3)/L2 = 162 plf

vc3 = (R3-F4-F5)/L3 = 155 plf

vc4 = (R4-F6)/L4 = 179 plf

Project Information

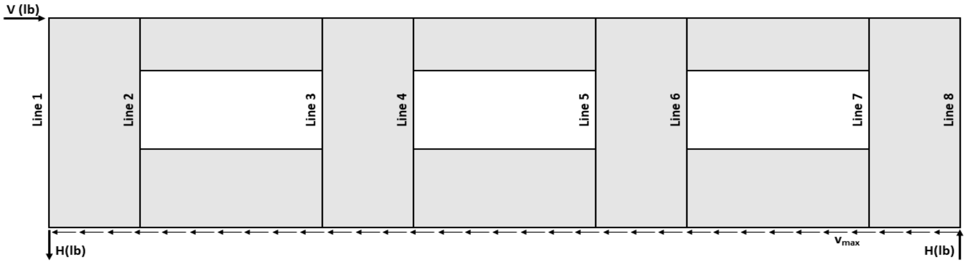
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.3.3



Check Summary of Shear Values for Three Openings

Line 1: $vc1(h_a1+h_b1)+v1(h_o1)=H$?		1402	652	2054 lbf
Line 2: $va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0$?	2054	1402	652	0
Line 3: $vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0$?	1218	836	2054	0
Line 4: $va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0$?	2054	836	1218	0
Line 5: $va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0$?	2054	1160	894	0
Line 6: $va3(h_a3+h_b3)-v3(h_o3)-vc3(h_a3+h_b3)=0$?	2054	894	1160	0
Line 7: $va3(h_a3+h_b3)-vc4(h_a3+h_b3)-v4(h_o3)=0$?	2054	1343	711	0
Line 8: $vc4(h_a3+h_b3)+v4(h_o3)=H$?		1343	711	2054 lbf

Design Summary*

Req. Sheathing Capacity	358 plf
Req. Strap Force	426 lbf
Req. HD Force (H)	2054 lbf
Req. Shear Wall Anchorage Force (v_{max})	205 plf

***The Design Summary assumes that the shear wall is designed as blocked.**

APA

Force Transfer Around Openings Calculator

TWO OPENINGS

The force transfer around openings (FTAO) method of shear wall analysis is an approach that aims to reinforce the wall such that it performs as if there was no opening. This approach lends certain advantages over segmented shear walls: more versatility, because it allows for narrower wall segments while still meeting the height-to-width ratios and, often, fewer required hold-downs.

Project Information

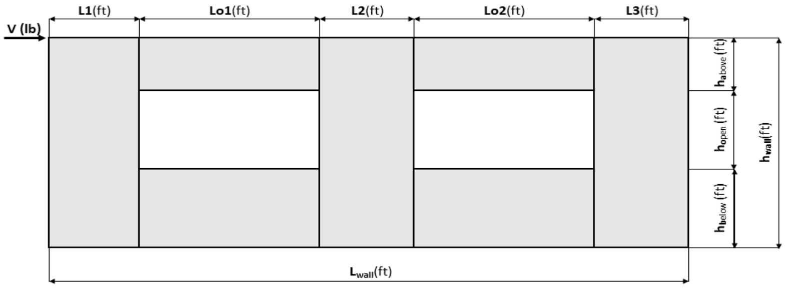
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.3.2



Shear Wall Calculation Variables

V	3336 lbf
L1	2.00 ft
L2	6.25 ft
L3	2.75 ft
h_{wall}	10.00 ft
L_{wall}	23.50 ft

Opening 1	
h_a1	2.00 ft
h_o1	4.50 ft
h_b1	3.50 ft
Lo1	6.25 ft

Opening 2	
h_a2	2.00 ft
h_o2	4.50 ft
h_b2	3.50 ft
Lo2	6.25 ft

Adj. Factor Method = 2bs/h

Wall Pier Aspect Ratio		Adj. Factor
P1=h_o/L1=	2.25	0.889
P2=h_o/L2=	0.72	N/A
P3=h_o/L3=	1.64	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1419 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(h_a1+h_b1) = 258 plf

Second opening: va2 = vb2 = H/(h_a2+h_b2) = 258 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1613 lbf

Second opening: O2 = va2 x (Lo2) = 1613 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 391 lbf

F2 = O1(L2)/(L1+L2) = 1222 lbf

F3 = O2(L2)/(L2+L3) = 1120 lbf

F4 = O2(L3)/(L2+L3) = 493 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.52 ft

T2 = (L2*Lo1)/(L1+L2) = 4.73 ft

T3 = (L2*Lo2)/(L2+L3) = 4.34 ft

T4 = (L3*Lo2)/(L2+L3) = 1.91 ft

6. Unit shear beside opening

v1 = (V/L)(L1+T1)/L1 = 249 plf

v2 = (V/L)(T2+L2+T3)/L2 = 348 plf

v3 = (V/L)(T4+L3)/L3 = 241 plf

Check v1*L1+v2*L2+v3*L3=V? 3336 lbf **OK**

7. Resistance to corner forces

R1 = v1*L1 = 499 lbf

R2 = v2*L2 = 2175 lbf

R3 = v3*L3 = 661 lbf

8. Difference corner force + resistance

R1-F1 = 108 lbf

R2-F2-F3 = -167 lbf

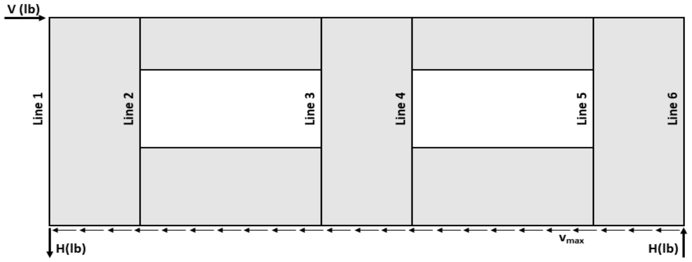
R3-F4 = 169 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 54 plf

vc2 = (R2-F2-F3)/L2 = -27 plf

vc3 = (R3-F4)/L3 = 61 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(h_a1+h_b1)+v1(h_o1)=H?		297	1123	1419 lbf
Line 2: va1(h_a1+h_b1)-vc1(h_a1+h_b1)-v1(h_o1)=0?	1419	297	1123	0
Line 3: vc2(h_a1+h_b1)+v2(h_o1)-va1(h_a1+h_b1)=0?	-147	1566	1419	0
Line 4: va2(h_a2+h_b2)-v2(h_o2)-vc2(h_a2+h_b2)=0?	1419	1566	-147	0
Line 5: va2(h_a2+h_b2)-vc3(h_a2+h_b2)-v3(h_o2)=0?	1419	337	1082	0
Line 6: vc3(h_a2+h_b2)+ v3(h_o2) = H?		337	1082	1419 lbf

Design Summary*

Req. Sheathing Capacity	348 plf
Req. Strap Force	1222 lbf
Req. HD Force	1419 lbf
Req. Shear Wall Anchorage Force	142 plf

***The Design Summary assumes that the shear wall is designed as blocked.**

Standard and Balloon Framing on Concrete Foundations

Strong-Wall® Wood Shearwall Standard Application on Concrete Foundation

Strong-Wall Wood Shearwall Model[9]	Allowable Vertical Load, P (lb.)[4]	2,500 psi Concrete						3,000 psi Concrete					
		Seismic[3]			Wind			Seismic[3]			Wind		
		Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[10]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[10]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[10]	Anchor Tension at Allowable Shear, T (lb.)[11]	Allowable ASD Shear Load, V (lb.)	Drift at Allowable Shear, Δ (in.)[10]	Anchor Tension at Allowable Shear, T (lb.)[11]
WSW12x7	1,000	1,065	0.31	10,285	1,380	0.43	13,375	1,065	0.31	10,285	1,380	0.43	13,375
	4,000	1,065	0.31	10,285	1,380	0.43	13,375	1,065	0.31	10,285	1,380	0.43	13,375
	7,500	1,065	0.31	10,285	1,380	0.43	13,370	1,065	0.31	10,285	1,380	0.43	13,375
WSW18x7	1,000	2,475	0.31	13,865	2,980	0.4	16,675	2,475	0.31	13,865	3,225	0.43	18,040
	4,000	2,475	0.31	13,865	2,710	0.36	15,160	2,475	0.31	13,865	3,225	0.43	18,040
	7,500	2,475	0.31	13,865	2,395	0.32	13,395	2,475	0.31	13,865	2,910	0.39	16,280
WSW24x7[9]	1,000	5,515	0.29	22,710	5,515	0.32	22,710	5,515	0.29	22,710	5,515	0.32	22,710
	4,000	5,515	0.29	22,710	5,400	0.31	22,240	5,515	0.29	22,710	5,515	0.32	22,710
	7,500	5,515	0.29	22,710	4,950	0.29	20,390	5,515	0.29	22,710	5,515	0.32	22,710
WSW12x8	1,000	960	0.39	11,125	1,245	0.53	14,420	960	0.39	11,125	1,245	0.53	14,420
	4,000	960	0.39	11,125	1,245	0.53	14,420	960	0.39	11,125	1,245	0.53	14,420
	7,500	960	0.39	11,125	1,155	0.49	13,370	960	0.39	11,125	1,245	0.53	14,420
WSW18x8	1,000	2,430	0.39	16,245	2,490	0.42	16,675	2,430	0.39	16,245	2,925	0.50	19,560
	4,000	2,430	0.39	16,245	2,265	0.38	15,160	2,430	0.39	16,245	2,695	0.46	18,045
	7,500	2,430	0.39	16,245	2,000	0.34	13,395	2,430	0.39	16,245	2,435	0.41	16,280
WSW24x8	1,000	4,945	0.37	24,355	4,840	0.40	23,830	4,945	0.37	24,355	5,515	0.45	27,150
	4,000	4,945	0.37	24,355	4,515	0.37	22,240	4,945	0.37	24,355	5,360	0.44	26,395
	7,500	4,945	0.37	24,355	4,140	0.34	20,390	4,945	0.37	24,355	4,985	0.41	24,540
WSW12x9	1,000	790	0.43	10,310	1,020	0.60	13,335	790	0.43	10,310	1,020	0.60	13,335
	4,000	790	0.43	10,310	1,020	0.60	13,335	790	0.43	10,310	1,020	0.60	13,335
	7,500	790	0.43	10,310	1,020	0.60	13,335	790	0.43	10,310	1,020	0.60	13,335
WSW18x9	1,000	1,920	0.43	14,505	2,210	0.53	16,675	1,920	0.43	14,505	2,515	0.60	18,980
	4,000	1,920	0.43	14,505	2,010	0.48	15,160	1,920	0.43	14,505	2,390	0.57	18,045
	7,500	1,920	0.43	14,505	1,775	0.42	13,395	1,920	0.43	14,505	2,155	0.51	16,280
WSW24x9	1,000	4,190	0.43	23,275	4,290	0.46	23,830	4,190	0.43	23,275	5,035	0.54	27,985
	4,000	4,190	0.43	23,275	4,000	0.43	22,240	4,190	0.43	23,275	4,750	0.51	26,395
	7,500	4,190	0.43	23,275	3,670	0.40	20,390	4,190	0.43	23,275	4,415	0.48	24,540
WSW12x10	1,000	630	0.50	9,175	810	0.67	11,810	630	0.50	9,175	810	0.67	11,810
	4,000	630	0.50	9,175	810	0.67	11,810	630	0.50	9,175	810	0.67	11,810
	7,500	630	0.50	9,175	810	0.67	11,810	630	0.50	9,175	810	0.67	11,810
WSW18x10	1,000	1,715	0.49	14,440	1,980	0.59	16,675	1,715	0.49	14,440	2,225	0.67	18,715
	4,000	1,715	0.49	14,440	1,800	0.54	15,160	1,715	0.49	14,440	2,145	0.64	18,045
	7,500	1,715	0.49	14,440	1,590	0.48	13,395	1,715	0.49	14,440	1,935	0.58	16,280
WSW24x10	1,000	3,675	0.48	22,740	3,850	0.54	23,830	3,675	0.48	22,740	4,520	0.63	27,985
	4,000	3,675	0.48	22,740	3,590	0.50	22,240	3,675	0.48	22,740	4,265	0.60	26,395
	7,500	3,675	0.48	22,740	3,295	0.46	20,390	3,675	0.48	22,740	3,965	0.55	24,540
WSW12x11	1,000	575	0.55	9,190	735	0.73	11,810	575	0.55	9,190	735	0.73	11,810
	4,000	575	0.55	9,190	735	0.73	11,810	575	0.55	9,190	735	0.73	11,810
	7,500	575	0.55	9,190	735	0.73	11,810	575	0.55	9,190	735	0.73	11,810
WSW18x11	1,000	1,510	0.53	14,010	1,800	0.67	16,675	1,510	0.53	14,010	1,975	0.73	18,335
	4,000	1,510	0.53	14,010	1,635	0.61	15,160	1,510	0.53	14,010	1,945	0.72	18,045
	7,500	1,510	0.53	14,010	1,445	0.54	13,395	1,510	0.53	14,010	1,755	0.65	16,280
WSW24x11	1,000	3,295	0.53	22,485	3,490	0.58	23,830	3,295	0.53	22,485	4,100	0.69	27,985
	4,000	3,295	0.53	22,485	3,260	0.55	22,240	3,295	0.53	22,485	3,865	0.65	26,395
	7,500	3,295	0.53	22,485	2,985	0.50	20,390	3,295	0.53	22,485	3,595	0.60	24,540

See foonotes on page 13.

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Anchorage Solutions

Strong-Wall® Wood Shearwall Tension Anchorage Solutions – 2,500 psi Concrete[1,5,6]

Design Criteria	Concrete Condition	Anchor Strength[2]	WSW-AB⅞ Anchor Bolt			WSW-AB1 Anchor Bolt		
			ASD Allowable Tension (lb.)	W (in.)	d_e (in.)	ASD Allowable Tension (lb.)	W (in.)	d_e (in.)
Seismic[3]	Cracked	Standard	11,900	27	9	16,100	33	11
			13,100	29	10	17,100	35	12
		High Strength	24,900	43	15	33,000	51	17
			27,100	46	16	35,300	54	18
	Uncracked	Standard	12,500	24	8	15,700	28	10
			13,100	25	9	17,100	30	10
		High Strength	25,300	38	13	32,300	44	15
			27,100	40	14	35,300	47	16
Wind[4]	Cracked	Standard	5,100	14	6	6,200	16	6
			8,700	20	7	11,400	24	8
			13,100	27	9	17,100	32	11
		High Strength	15,900	30	10	21,100	36	12
			18,400	33	11	27,300	42	14
			23,100	38	13	31,800	46	16
			27,100	42	14	35,300	50	17
	Uncracked	Standard	5,000	12	6	6,400	14	6
			9,300	18	6	12,500	22	8
			13,100	23	8	17,100	28	10
		High Strength	15,200	25	9	21,900	32	11
			19,900	30	10	26,400	36	12
			24,000	34	12	31,500	40	14
			27,100	37	13	35,300	43	15

Strong-Wall® Wood Shearwall Tension Anchorage Solutions – 3,000 psi Concrete[1,5,6]

Design Criteria	Concrete Condition	Anchor Strength[2]	WSW-AB⅞ Anchor Bolt			WSW-AB1 Anchor Bolt		
			ASD Allowable Tension (lb.)	W (in.)	d_e (in.)	ASD Allowable Tension (lb.)	W (in.)	d_e (in.)
Seismic[3]	Cracked	Standard	12,300	26	9	16,000	31	11
			13,100	28	10	17,100	33	11
		High Strength	25,200	41	14	32,700	48	16
			27,100	43	15	35,300	51	17
	Uncracked	Standard	12,000	22	8	16,300	27	9
			13,100	24	8	17,100	28	10
		High Strength	25,300	36	12	32,700	42	14
			27,100	38	13	35,300	44	15
Wind[4]	Cracked	Standard	5,000	13	6	5,600	14	6
			8,800	19	7	10,200	21	7
			13,100	25	9	17,100	30	10
		High Strength	15,700	28	10	20,100	33	11
			19,200	32	11	25,300	38	13
			23,200	36	12	32,300	44	15
			27,100	40	14	35,300	47	16
	Uncracked	Standard	5,500	12	6	6,200	13	6
			8,500	16	6	12,800	21	7
			13,100	22	8	17,100	26	9
		High Strength	16,600	25	9	21,800	30	10
			19,700	28	10	25,200	33	11
			24,000	32	11	31,700	38	13
			27,100	35	12	35,300	41	14

See footnotes on page 24.

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