Structural Calculations

for

NEW SINGLE-FAMILY DWELLING

Plummer Residence

9212 SE 33rd Pl

Mercer Island, WA 98040

PERMIT SUBMITTAL

prepared by:
O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA, 98106
(206) 290-4608

Job No. 21006

Date: 5/14/21

Date: 5/14/2021
Job # 21006

Vertical Design Loads

Typical Roof	
Metal Roofing	1 psf
5/8" Plywood	2
Trusses @24"o.c.	4
Batt Insulation	0.5
5/8" Gypsum Board	2.8
Future Solar Panels	4 *
Sum	14.3 psf
Slope:	2 :12
Slope Correction Factor	1.01
Subtotal	14.5 psf
M/E/P/misc.	3.5 psf
DL=	18 psf
SL=	25 psf
RLL=	20 psf

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

Typical Floor		
Flooring	4 psf	
3/4" Plywood	2.4	
11 -7/8 @16"o.c.	2.8	
5/8" Gypsum Board	2.8	
M/E/P/misc.	2	
DL=	14 psf	
LL=	40 psf	*Living Areas*

Roof Deck		
Built-Up Decking	8 psf	
3/4" Plywood	2.4	
Joists	3	
Insulation	1	
M/E/P/misc.	3.6	
DL=	18 psf	
SL=	25 psf	
LL=	60 psf	*Decks*

Exterior Walls - Wood Siding	
Wood Siding	3 psf
1/2" Plywood	1.6
2x6 @16"o.c.	1.4
Batt Insulation	0.2
1/2" Gypsum Board	2.2
M/E/P/misc.	1.6
DL=	10 psf

Exterior Walls - Stone Siding	
Stone Siding	43 psf
1/2" Plywood	1.6
2x6 @16"o.c.	1.4
Batt Insulation	0.2
1/2" Gypsum Board	2.2
M/E/P/misc.	1.6
DL=	50 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

Date: 5/14/2021
Job # 21006

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 =	23		
	x =	0.75		
	T =	0.21	s	
Seismic Response Coefficient				
	S_s =	1.39		
	S_1 =	0.49		
	S_{ds} =	1.11		
	S_{d1} =	0.49		
	R =	6.5		
	ρ =	1		
	Ω =	2.5		
	C_d =	4		
	I_e =	1		
$C_s = S_{ds}/(R/I_e)$ =		0.17	W	
T_L =	6	s	>	T
$C_{s,max} = S_{d1}/[T(R/I_e)]$ =		0.36		
$C_{s,min} = 0.044 S_{ds} I_e$ =		0.049		
$C_{s,min}$ =		0.01		
S_1 < 0.6				
$C_{s,min} = 0.5 S_1/(R/I_e)$ =		0.038	**Ignore**	
$C_{s,min,gov}$ =		0.049		
$C_{s,gov}$ =		**0.17**	(LRFD)	

Effective Seismic Weight				
Floor	Area (sf)	w_{floor} (psf)	w_{walls} (psf)[1]	W (lbs)
Roof	2780	18	18	100080
Upper	3470	14	36	173500
			Sum:	**273580 lbs**

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

Base Shear (includes ρ) - LRFD Level			
$\rho V = \rho C_s W$ =	0.171	W =	**46719 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	100080	23	2301840	0.55	**25542**	**9.2**
Upper	173500	11	1908500	0.45	**21177**	**6.1**
		Sum:	4210340		46719	

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	25542	25542	100080	100080	0.26	25542
Upper	21177	46719	173500	273580	0.17	29628

Floor	F_{px} Min (lbs)	F_{px} Max (lbs)	F_{px} Gov (lbs)	F_{px} Gov (psf)
Roof	15552	31105	**25542**	**9.2**
Upper	26962	53924	**29628**	**8.5**

Hazards by Location

Search Information

Address:	9212 SE 33rd Pl, Mercer Island, WA 98040, USA
Coordinates:	47.5818239, -122.2135532
Elevation:	46 ft
Timestamp:	2021-04-03T23:49:36.644Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default

Basic Parameters

Name	Value	Description
S_S	1.392	MCE_R ground motion (period=0.2s)
S_1	0.485	MCE_R ground motion (period=1.0s)
S_{MS}	1.671	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.114	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.903	Coefficient of risk (0.2s)
CR_1	0.897	Coefficient of risk (1.0s)
PGA	0.596	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.715	Site modified peak ground acceleration

T_L	6	Long-period transition period (s)
SsRT	1.392	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.542	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	3.419	Factored deterministic acceleration value (0.2s)
S1RT	0.485	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.54	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	1.386	Factored deterministic acceleration value (1.0s)
PGAd	1.175	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

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

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: 9212 SE 33rd Pl

9212 SE 33rd Pl

Basic Values

Risk Category	2 per ASCE 7-16 Table 1.5-1	Horizontal Dim. in North-South Direction (B or L) =	112.0 ft
V : Basic Wind Speed	98.0	Horizontal Dim. in East-West Direction (B or L) =	33.0 ft
Kd : Directionality Factor	0.850 per ASCE 7-16 Table 26.6-1	h : Mean Roof height =	23.0 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	23.00	12.00	0.000	0.000
Upper	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	ft^2	ft^2	ft^2	ft^2	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 positive external pressure

User has specified the Building is to be considered Enclosed when SOUTH elevation receives positive external pressure

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

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

Velocity Pressures

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

North Wall = 0.9288 psf South Wall = 0.9288 psf East Wall = 0.9288 psf West Wall = 0.9288 psf

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

North Wall = 19.411 psf South Wall = 19.411 psf East Wall = 19.411 psf West Wall = 19.411 psf

qz : Windward Wall Velocity Pressures at various heights per Eq. 26.10-1

Height Above Base (ft)	North Elevation Kz	North Elevation qz	South Elevation Kz	South Elevation qz	East Elevation Kz	East Elevation qz	West Elevation Kz	West Elevation qz
0.00	0.849	17.74	0.849	17.74	0.849	17.74	0.849	17.74
5.00	0.849	17.74	0.849	17.74	0.849	17.74	0.849	17.74
10.00	0.849	17.74	0.849	17.74	0.849	17.74	0.849	17.74
15.00	0.849	17.74	0.849	17.74	0.849	17.74	0.849	17.74

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

DESCRIPTION: 9212 SE 33rd Pl

20.00	0.902	18.85	0.902	18.85	0.902	18.85	0.902	18.85	

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	-11.743 psf	-4.756 psf
Side Wall Pressures	-15.043 psf	-8.055 psf

Windward Wall Pressures . . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.57	15.56
5.00	8.57	15.56
10.00	8.57	15.56
15.00	8.57	15.56
20.00	9.32	16.31

Wind Pressures when SOUTH Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.743 psf	-4.756 psf
Side Wall Pressures	-15.043 psf	-8.055 psf

Windward Wall Pressures . . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.57	15.56
5.00	8.57	15.56
10.00	8.57	15.56
15.00	8.57	15.56
20.00	9.32	16.31

Wind Pressures when EAST Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.743 psf	-4.756 psf
Side Wall Pressures	-15.043 psf	-8.055 psf

Windward Wall Pressures . . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.57	15.56
5.00	8.57	15.56
10.00	8.57	15.56
15.00	8.57	15.56
20.00	9.32	16.31

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

DESCRIPTION: 9212 SE 33rd Pl

Wind Pressures when WEST Elevation receives positive external wind pressure

	Positive Internal	Negative Internal
Leeward Wall Pressures	-11.743 psf	-4.756 psf
Side Wall Pressures	-15.043 psf	-8.055 psf

Windward Wall Pressures . . . Height Above Base (ft)	Positive Internal Pressure (psf)	Negative Internal Pressure (psf)
0.00	8.57	15.56
5.00	8.57	15.56
10.00	8.57	15.56
15.00	8.57	15.56
20.00	9.32	16.31

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) In "Y" Direction	In "X" Direction	Eccentricity for (ft) "Y" Shear	"X" Shear	Mt, (ft-k)
CASE 1	North	Level 2	17.00' -> 23.00'	6.00	-4.17	---	---	---	---
CASE 1	North	Level 1	5.50' -> 17.00'	11.50	-7.72	---	---	---	---
CASE 1	South	Level 2	17.00' -> 23.00'	6.00	4.17	---	---	---	---
CASE 1	South	Level 1	5.50' -> 17.00'	11.50	7.72	---	---	---	---
CASE 1	East	Level 2	17.00' -> 23.00'	6.00	---	-14.15	---	---	---
CASE 1	East	Level 1	5.50' -> 17.00'	11.50	---	-26.20	---	---	---
CASE 1	West	Level 2	17.00' -> 23.00'	6.00	---	14.15	---	---	---
CASE 1	West	Level 1	5.50' -> 17.00'	11.50	---	26.20	---	---	---
CASE 2	North	Level 2	17.00' -> 23.00'	6.00	-3.13	---	---	4.95 +/-	15.5
CASE 2	North	Level 1	5.50' -> 17.00'	11.50	-5.79	---	---	4.95 +/-	28.7
CASE 2	South	Level 2	17.00' -> 23.00'	6.00	3.13	---	---	4.95 +/-	15.5
CASE 2	South	Level 1	5.50' -> 17.00'	11.50	5.79	---	---	4.95 +/-	28.7
CASE 2	East	Level 2	17.00' -> 23.00'	6.00	---	-10.61	16.80	--- +/-	178.3
CASE 2	East	Level 1	5.50' -> 17.00'	11.50	---	-19.65	16.80	--- +/-	330.1
CASE 2	West	Level 2	17.00' -> 23.00'	6.00	---	10.61	16.80	--- +/-	178.3
CASE 2	West	Level 1	5.50' -> 17.00'	11.50	---	19.65	16.80	--- +/-	330.1
CASE 3	North & East	Level 2	17.00' -> 23.00'	6.00	-3.13	-10.61	---	---	---
CASE 3	North & East	Level 1	5.50' -> 17.00'	11.50	-5.79	-19.65	---	---	---
CASE 3	North & West	Level 2	17.00' -> 23.00'	6.00	-3.13	10.61	---	---	---
CASE 3	North & West	Level 1	5.50' -> 17.00'	11.50	-5.79	19.65	---	---	---
CASE 3	South & West	Level 2	17.00' -> 23.00'	6.00	3.13	10.61	---	---	---
CASE 3	South & West	Level 1	5.50' -> 17.00'	11.50	5.79	19.65	---	---	---
CASE 3	South & East	Level 2	17.00' -> 23.00'	6.00	3.13	-10.61	---	---	---
CASE 3	South & East	Level 1	5.50' -> 17.00'	11.50	5.79	-19.65	---	---	---
CASE 4	North & East	Level 2	17.00' -> 23.00'	6.00	-2.35	-7.97	16.80	4.95 +/-	145.5
CASE 4	North & East	Level 1	5.50' -> 17.00'	11.50	-4.35	-14.75	16.80	4.95 +/-	269.3
CASE 4	North & West	Level 2	17.00' -> 23.00'	6.00	-2.35	7.97	16.80	4.95 +/-	145.5
CASE 4	North & West	Level 1	5.50' -> 17.00'	11.50	-4.35	14.75	16.80	4.95 +/-	269.3

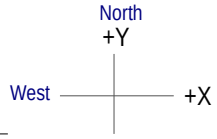
ASCE 7-16 Wind Forces, Chapter 27, Part I

DESCRIPTION: 9212 SE 33rd Pl

CASE 4	South & West	Level 2	17.00' -> 23.00'	6.00	2.35	7.97	16.80	4.95	+/-	145.5
CASE 4	South & West	Level 1	5.50' -> 17.00'	11.50	4.35	14.75	16.80	4.95	+/-	269.3
CASE 4	South & East	Level 2	17.00' -> 23.00'	6.00	2.35	-7.97	16.80	4.95	+/-	145.5
CASE 4	South & East	Level 1	5.50' -> 17.00'	11.50	4.35	-14.75	16.80	4.95	+/-	269.3
Min per ASCE 27.1.5	North	Level 2	17.00' -> 23.00'	6.00	-3.17	---	---	---		---
Min per ASCE 27.1.5	North	Level 1	5.50' -> 17.00'	11.50	-6.07	---	---	---		---
Min per ASCE 27.1.5	South	Level 2	17.00' -> 23.00'	6.00	3.17	---	---	---		---
Min per ASCE 27.1.5	South	Level 1	5.50' -> 17.00'	11.50	6.07	---	---	---		---
Min per ASCE 27.1.5	East	Level 2	17.00' -> 23.00'	6.00	---	-10.75	---	---		---
Min per ASCE 27.1.5	East	Level 1	5.50' -> 17.00'	11.50	---	-20.61	---	---		---
Min per ASCE 27.1.5	West	Level 2	17.00' -> 23.00'	6.00	---	10.75	---	---		---
Min per ASCE 27.1.5	West	Level 1	5.50' -> 17.00'	11.50	---	20.61	---	---		---

Base Shear for Design Wind Load Cases

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	-11.89	---		---
Case 1	South	North	11.89	---		---
Case 1	East	West	---	-40.35		---
Case 1	West	East	---	40.35		---
Case 2	North	South	-8.92	---	+/-	44.1
Case 2	South	North	8.92	---	+/-	44.1
Case 2	East	West	---	-30.26	+/-	508.4
Case 2	West	East	---	30.26	+/-	508.4
Case 3	North & East	South & West	-8.92	-30.26		---
Case 3	North & West	South & East	-8.92	30.26		---
Case 3	South & West	North & East	8.92	30.26		---
Case 3	South & East	North & West	8.92	-30.26		---
Case 4	North & East	South & West	-6.69	-22.72	+/-	414.8
Case 4	North & West	South & East	-6.69	22.72	+/-	414.8
Case 4	South & West	North & East	6.69	22.72	+/-	414.8
Case 4	South & East	North & West	6.69	-22.72	+/-	414.8
Min per ASCE 27.1.5	North	South	-9.24	---		---
Min per ASCE 27.1.5	South	North	9.24	---		---
Min per ASCE 27.1.5	East	West	---	-31.36		---
Min per ASCE 27.1.5	West	East	---	31.36		---

CONTROLLING LATERAL LOAD ON MLFRS

SEISMIC BASE SHEAR

$V_{SEISMIC} = \underset{EL, ADJ}{32703^{\#}}$

WIND BASE SHEAR

$V_{WIND, MAP} = 0.6 \times 40350^{\#} = \underset{WL, ADJ}{24210^{\#}}$

$V_{SEISMIC} > V_{WIND}$

∴ SEISMIC GOVERNS MLFRS DESIGN

WIND COMPONENTS + CLADDING

ASCE 7-16 § 30.4.2

$P_{NET} = \lambda \, K_{zt} \, P_{NET30}$ Exp C $H = 23'$ $A_{EFF} = 20$ SF

$\lambda = 1.32$ $K_{zt} = 1.0$ $P_{NET30} = -22.5$ PSF (WALL)
$= -38.1$ PSF (ROOF)

$\Rightarrow P_{NET} = \underset{WL, RFO}{-29.7} \text{ PSF (WALL)}$

$= \underset{WL, RFO}{-50.3} \text{ PSF (ROOF)}$

ROOF FRAMING

RM 1 ROOF TRUSS

SPAN = 12'-0" W = $\frac{18+20+25}{DL \quad LL \quad SL}$ PSF TRIB = $\left(\frac{11'}{2}+4'\right)$ = 12'

USE (2) 13/4 x 117/8 LVL

UPPER FLOOR FRAMING

UFJ1 UPPER FLOOR JOIST

SPAN = 17'-3" $w = \frac{14}{DL} + \frac{40}{LL}$ plf USE 11 7/8 TJI 560 @ 16" o.c.

UFJ2 UPPER FLOOR JOIST

SPAN = 22'-6" $w = \frac{14}{DL} + \frac{40}{LL}$ plf USE 3 1/2 x 11 7/8 LVL @ 16" o.c.

UFJ3 UPPER FLOOR JOIST

SPAN = 14'-9" $w = \frac{14}{DL} + \frac{40}{LL}$ plf USE 11 7/8 TJI 360 @ 16" o.c.

UFB4 UPPER FLOOR BEAM

SPAN = 17'-3"

USE 5 1/4 x 11 7/8 GLB

$$w = \frac{(18+20+25)^{\text{ROOF}}}{DL\ LL\ SL}\left(\frac{7'}{2}+2'\right) + \frac{(10)^{\text{WALL}}}{DL}(11') = \frac{210}{DL} + \frac{110+140}{LL\ SL}\ \#/ft$$

UFB5 UPPER FLOOR BEAM

SPAN = 18'-3"

USE W18 x 76

$$w = \frac{(18+20+25)^{\text{ROOF}}}{DL\ LL\ SL}\left(\frac{22'}{2}+\frac{2'}{2}\right) + \frac{(13+40)^{\text{UF}}}{DL\ LL}\left(\frac{17'}{2}\right) + \frac{(10)^{\text{WALL}}}{DL}(11') = \frac{440}{DL} + \frac{240+340}{LL\ SL} + \frac{300}{LL}\ \#/ft$$

RXN UFB4

$P_1 = \frac{1980}{DL} + \frac{950}{LL} + \frac{1210}{SL}\ \#$ $e_x = 16'$

SW UF.C

$P_2 = 1.4 \times 2.5 \times \frac{37637\ \#\cdot ft}{22'-6"} = \frac{5855\ \#}{EL,\ LEFT}$ $e_x = 16'$

UFB6 UPPER FLOOR BEAM

SPAN = 6'-9"

USE W10x12

$w = \frac{(18+20+25)}{DL\ RM\ SL}\left(\frac{2'}{2}+3'\right) = \frac{70+}{DL} \frac{80+}{RM} \frac{160\ \#/FT}{SL}$

RXN (UFB5)

$P = \frac{5940+}{DC} \frac{3020+}{RM} \frac{3800+}{SL} \frac{2100+}{LL} \frac{513.0\ \#}{EL}$ ecc = 2'

UFB7 UPPER FLOOR BEAM

SPAN = 8'-0"

USE 3½x11⅞ GLB

$w = \frac{(14+40)}{DC\ LL}\left(\frac{20'}{2}\right) = \frac{140+}{DL} \frac{440\ \#/FT}{LL}$

UFB8 UPPER FLOOR BEAM

SPAN = 8'-3"

USE 5¼x11⅞ GLB

$w = \frac{(18+20+25)}{DL\ RM\ SL}\left(\frac{22'}{2}\right) + \frac{(14+40)}{DL\ LL}\left(\frac{5'}{2}\right) + \frac{(18+25+60)}{DL\ SL\ LL}\left(\frac{14'}{2}\right)$

$+ \frac{840\ \#}{8'} LL + \frac{(10)}{DL}(9') = \frac{380+}{DL} \frac{220+}{RM} \frac{450+}{SL} \frac{1520\ \#/FT}{LL}$

UFB9 UPPER FLOOR BEAM

SPAN = 17'-0"

USE 5¼x11⅞ GLB

$w = \frac{(18+20+25)}{DL\ RM\ SL}\left(\frac{4'}{2}+3'\right) + \frac{(7)}{DL}(11') = \frac{170+}{DL} \frac{140+}{RM} \frac{120}{SL}$

$P_3 = 1.4x2.5x\frac{3565\ \#\cdot ft}{11'-9"} = \frac{10120\ \#}{EL,LL}$ ecc = 1'

SW (UF.7)

RXN (UFB7) $P_1 = \left(\frac{14+40}{DL\ LL}\right)\left(\frac{12'}{2}\right)\left(\frac{8'}{2}\right) = \frac{340+}{DL} \frac{910\ \#}{LL}$ ecc = 11'

RXN (UFB8) $P_2 = \frac{1650+}{DL} \frac{910+}{RM} \frac{1860+}{SL} \frac{6270\ \#}{LL}$ ecc = 15'-9"

P_4

UFB10 UPPER FLOOR BEAM

SPAN = 26'-0"

USE W18x40

$w = \frac{(18+20+25)}{DL\ RM\ SL}\left(\frac{25'}{2}\right) + \frac{(4+40)}{DL\ LL}\left(\frac{17'}{2}\right) + \frac{(18+25+60)}{DL\ SL\ LL}\left(\frac{12'}{2}\right)$

$+ \frac{(10)}{DL}(11') = \frac{560+}{DL} \frac{270+}{RM} \frac{460+}{SL} \frac{700\ \#/FT}{LL}$

$P_3 = \frac{5855\ \#}{EL,LL}$ ecc = 20'

SW (UF.6)

(IN UFB5)

RXN (UFB9) $P_1 = \frac{3360+}{DL} \frac{1640+}{RM} \frac{2830+}{SL} \frac{6430\ \#}{LL}$ ecc = 3'-6"

$P_2 = 1.4x2.5x\frac{2589\ \#\cdot ft}{25'-6"} x \frac{15'-9"}{17'} = 3300\ \#$ ecc = 3'-6"

SW (UF.B9)

UFH11 UPPER FLOOR HEADER

SPAN = 3'-6"

USE 5 1/4 x 14 PSL

RXN UFB5

$P_1 = \underset{DL}{4500} + \underset{RL}{2310} + \underset{SL}{2890} + \underset{LL}{3100} + \underset{EL}{720}\#$ @ x = 2'-0"

RXN UFB10

$P_2 = \underset{DL}{8250} + \underset{RL}{3480} + \underset{SL}{6360} + \underset{LL}{9770} + \underset{EL}{4060}\#$ @ x = 2'-0"

UFB12 UPPER FLOOR BEAM

SPAN = 17'-0"

USE 5 1/4 x 11 7/8 PSL

$w = \underset{ROOF}{(\underset{DL}{18} + \underset{RL}{20} + \underset{SL}{25})}\left(\frac{22'}{2} + \frac{4'}{2}\right) + \underset{UF}{(\underset{DL}{14} + \underset{LL}{40})}\left(\frac{4'}{2}\right) + \underset{WALL}{(\underset{DL}{10})}(9')$

$= \underset{DL}{350} + \underset{RL}{260} + \underset{SL}{330} + \underset{LL}{80}\ \#/ft$

UFB13 UPPER FLOOR BEAM

SPAN = 5'-0"

USE 5 1/4 x 11 7/8 PSL

RXN UFB8

$P_1 = \underset{DL}{1650} + \underset{RL}{910} + \underset{SL}{1860} + \underset{LL}{6270}\#$ @ x = 6"

RXN UFB12

$P_2 = \underset{DL}{3140} + \underset{RL}{2210} + \underset{SL}{2810} + \underset{LL}{680}\#$ @ x = 6"

UFH14 UPPER FLOOR HEADER

SPAN = 4'-0"

USE 5 1/4 x 11 7/8 PSL

$w = \underset{ROOF}{(\underset{DL}{18} + \underset{RL}{20} + \underset{SL}{25})}\left(\frac{28'}{2} + 1'\right) + \underset{UF}{(\underset{DL}{14} + \underset{LL}{40})}\left(\frac{22'}{2}\right) + \underset{WALL}{(\underset{DL}{10})}(9')$

$= \underset{DL}{540} + \underset{RL}{300} + \underset{SL}{375} + \underset{LL}{440}\ \#/ft$

UFB15 UPPER FLOOR BEAM

SPAN = 19'-0"

USE 5 1/4 x 11 7/8 PSL

$w = \underset{ROOF}{(\underset{DL}{18} + \underset{RL}{20} + \underset{SL}{25})}\left(\frac{7'}{2} + 2' + 2'\text{-}6"\right) + \underset{WALL}{(\underset{DL}{10})}(9') = \underset{DL}{230} + \underset{RL}{160} + \underset{SL}{200}\ \#/ft$

LOW ROOF FRAMING

LRB2 LOW ROOF RAFTERS

BACK SPAN = 1'-9"
CANTILEVER = 3'-3"

$W = \frac{18}{DL} + \frac{24}{LL} + \frac{21}{SL}$ PSF

USE 2x6 @ 24" O.C.

LRB3 LOW ROOF HIP BEAM

BACK SPAN = 2'-3"
CANTILEVER = 4'-6"

$W = \frac{(18+20+25)}{DL\ LL\ SL}(5') = \frac{90}{DL} + \frac{100}{LL} + \frac{130}{SL}$ #/FT @ X=0'

= 0 @ X = 7'

USE 2x8

LRB4 LOW ROOF BEAM

SPAN = 15'-0"

$W = \frac{(18+20+25)}{DL\ LL\ SL}(\frac{9'}{2} + 2'-6") = \frac{130}{DL} + \frac{140}{LL} + \frac{160}{SL}$ #/FT

USE 5½x10½ GLB

LRH5 LOW ROOF HEADER

SPAN = 12'-0"

$W = \frac{(18+20+25)}{DL\ LL\ SL}(\frac{3'}{2} + 2') = \frac{60}{DL} + \frac{70}{LL} + \frac{80}{SL}$ #/FT

USE (2) 2x12

LRB6 LOW ROOF BEAM

SPAN = 18'-6"

$W = \frac{(18+20+25)}{DL\ LL\ SL}(\frac{21'}{2} + 3') = \frac{70}{DL} + \frac{80}{LL} + \frac{100}{SL}$ #/FT

USE 3½x11⅞ GLB

UPPER DECK FRAMING

UDJ1 UPPER DECK JOIST

SPAN = 12'-0" $w_1 = \frac{18}{DC} + \frac{25}{LL} + \frac{60}{LL} = 103$ PSF

USE 1 3/4 x 9 1/4 (NEW) LVL @ 16" O.C. $w_2 = \frac{8000\#}{(71')^2} = 160$ PSF FROM x = 6' TO 12'

(DBC UNDER THIS TUB) → UDJ3 TUB

UDH2 UPPER DECK HEADER

SPAN = 9'-0" $w = \left(\frac{18}{DC} + \frac{25}{SC} + \frac{60}{LL}\right)\left(\frac{12'}{2}\right) = \frac{110}{DC} + \frac{150}{SC} + \frac{360}{LL}\ \#/ft$

USE (2) 1 3/4 x 9 1/2 LVL

MAIN FLOOR FRAMING

(MFJ1) MAIN FLOOR JOISTS

SPAN = 13'-8" MAX   w = 14 DL + 40 LL psf   USE 11 7/8 TJI 360 @ 16" o.c.

(MFB2) MAIN FLOOR BEAM   RXN (UFB4)

SPAN = 13'-3"   P = [(18 DL + 20 LL + 25 SL)_ROOF (22'/2 + 2'/2) + (99 DL + 40 LL)_UF (12'/2) + (10 DL)_WALL (9')] (13'/2)

USE 3 1/2 x 11 7/8 LSL   = 2576 DL + 1560 LL + 1950 SL + 1560 LL # @ x = 2'

(MFB2) MAIN FLOOR BEAM

SPAN = 8'-6"   w = (2)(14 DL + 40 LL)_MF+UF (25'/2) = 350 DL + 1000 LL #/ft

USE 5 1/2 x 10 1/2 GLB   RXN (MFB3)

P = 380 DL + 240 LL + 290 SL + 240 LL #   @ x = 3'-6"

(MFJ1) MAIN FLOOR JOISTS (SPECIAL CASE @ MUD ROOM)

SPAN = 11'-6"   w = 14 DL + 40 LL psf (MF)

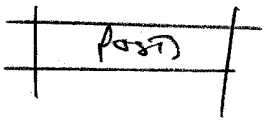
P = [(14 DL + 40 LL)_UF (25'/2) + (7 DL)_WALL (10')] (16") = 330 DL + 300 LL # @ x = 10'-6"

(MFJ4) MAIN FLOOR JOISTS BELOW HOT TUB

SPAN = 9'-3"   w = 14 DL + 40 LL psf (MF)

USE 1 3/4 x 11 7/8 LVL @ 16" o.c.   P = [(18 DL + 25 SL + 60 LL)_DECK (14'/2) + (8000# / 7')_TUB (3/4)] (16") = 170 DL + 230 SL + 1700 LL #

@ x = 9"



MAIN FLOOR POST SUPPORTING (UFB7) (WORST CASE 4x4)

H=10 1/0" $P = \frac{(14+40)}{DL}\left(\frac{14'}{2}\right)\left(\frac{20'}{2}\right) = \frac{980}{DL} + \frac{2800}{LL}$ #

USE 4x4

MAIN FLOOR POST SUPPORTING SOUTH END OF (UFB4) (WORST CASE 4x6)
(SEE UFB3 CALC)

H=10 1/0" $P = \frac{2570}{DL} + \frac{1560}{LL} + \frac{1950}{SL} + \frac{1560}{LL}$ # USE 4x6

MAIN FLOOR POST SUPPORTING WEST END OF (UFH11) (WORST CASE 3 1/2 x 5 1/4 PSL)

H=18'-0" $P = \frac{8100}{DL} + \frac{3780}{LL} + \frac{5850}{SL} + \frac{2060}{LL}$ # FROM (UFH11)

USE 3 1/2 x 5 1/4 PSL

MAIN FLOOR POST @ B-L (B/6) (WORST CASE 6x6)
FROM (UFB4)

H=10 1/0" $P = \left(\frac{18+24+25}{DL\ LL\ SL}\right)\left(\frac{120SF}{2}\right) + \left(\frac{1810}{DL} + \frac{920}{LL} + \frac{1240}{SL} + \frac{800}{LL}\right)$ # + FROM (UFB7) $\left(\frac{600}{RL} + \frac{1100}{LL}\right)$ #

USE 6x6 $P = \frac{3490}{DL} + \frac{2120}{LL} + \frac{2740}{SL} + \frac{2400}{LL}$ #

MAIN FLOOR POST SUPPORTING EAST END OF (UFH11) (WORST CASE 5 1/4 x 5 1/4 PSL)
FROM (UFH11)

H=8 1/0" $P = \frac{6280}{DL} + \frac{2900}{LL} + \frac{4490}{SL} + \frac{6200}{LL}$ #

MAIN FLOOR POST SUPPORTING SOUTH END OF (UFB10) (WORST CASE HSS 5x3x1/4)

H=11 7/0" $P = \frac{10700}{DL} + \frac{4700}{LL} + \frac{8430}{SL} + \frac{14660}{LL}$ # FROM (UFB10)

USE HSS 5x3x1/4

EXTERIOR WALL FRAMING

BFS BALLOON · FRAME STUDS ADJ. TO STAIR

$H = 20'-0"$ $w_L = \left(\frac{29.7}{WL}\right)(16") = \frac{40\ \#/FT}{WL,\ LRFD}$

USE 1¾×5½ @ 16" o.c.

$P_G = \left(\frac{18}{DL} + \frac{20}{RLL} + \frac{25}{SL}\right)\left(\frac{16'}{2} + 2'\right)(16") = \frac{240}{DL} + \frac{270}{RLL} + \frac{330\ \#}{SL}$ @ TOP

$\Delta = 3" \times 0.42 = 1.26" = H/190$ ok

TB1 TRANS ON BEAM ADJ. TO STAIR

SPAN = 6'-0" $w_G = \left(\frac{10}{DL}\right)(6') = \frac{60\ \#/FT}{DL}$

USE 5¼×5¼ PSL $w_L = \left(\frac{29.7}{WL}\right)\left(\frac{13'}{2}\right) = \frac{190\ \#/FT}{WL,\ LRFD}$

BFP BALLOON · FRAMED POST

$H = 20'-0"$ $P_{G1} = \left(\frac{18}{DL} + \frac{20}{RLL} + \frac{25}{SL}\right)\left(\frac{16'}{2} + 2'\right)\left(\frac{6'}{2}\right) = \frac{540}{DL} + \frac{600}{RLL} + \frac{750\ \#}{SL}$ @ TOP

USE 3½×5¼ PSL $P_{G2} = \left(\frac{60}{DL}\right)\left(\frac{6'}{2}\right) = \frac{180\ \#}{DL}$ @ $H = 13'-0"$ (RXN FROM TB1)

$A = 1.93" \times 0.42 = 0.81" = L/296$ ok

$P_{L1} = \frac{570\ \#}{WL,\ LRFD}$ @ $H = 13'-0"$ (RXN FROM TB1)

WIND UPLIFT @ ROOF

RT1 COMMON TRUSS

$R_{END} = 0.6\left(\frac{50.3}{WL} - \frac{14}{DL}\right)(2')\left(\frac{27'}{2}\right) = \frac{590\ \#}{ASD}$

USE 'H2' GOOD FOR 780# ok

SKELETON ROOF FRAMING

TYPICAL RAFTER / STRUT

ARM

$$M_G = \underbrace{(18+20+25)}_{DL\ RL\ SL}(3'0'')(21'0'')\left(\frac{3'0''}{2}\right) = \frac{160+180+230}{DL\ RL\ SL}\ \#\cdot ft$$

LOADS

$$T_{TOP} = \frac{M}{ARM} = \frac{(160+180+230)}{1'} = \frac{160+180+230}{DL\ RL\ SL}\ \#$$

$T_{FACTORED} = 160+230 = 390\ \#$

USE (6) #10 TOE-NAILS

$T_{all} = 6 \times 1.15 \times 135\ \#/NAIL \times 0.67 = 620\ \#$ ok

(CO, GEN)

$$M_L = 0.6\left(\frac{50.8}{WL} - \frac{14}{DL}\right)\left(3' \times 2' \times \frac{3'}{2}\right) = 200\ \#\cdot ft$$

$$T_{BOTT} = \frac{M}{ARM} = \frac{200\ \#\cdot ft}{1'} = 200\ \#$$

USE (4) #10 TOE-NAILS

$T_{all} = 4 \times 1.6 \times 135 \times 0.67 = 580\ \#$ ok

DRAG STRUTS / STRAPS (STRAPS NOT EXPLICITLY CALC'D ARE JUST LONGITUDINAL TIES)

DTR ROOF DRAG TRUSS

$T_{STRUT} = \frac{17'-6''}{31'-6''} \times 4245^{\#} = 2440^{\#}$ EL, ASD (2 STRAP TIES TO SW UF.3)

USE (4) 'A35' Good for 4 x 670#/CLIP = 2680# ok

LOAD ON DRAG TRUSS' BOTTOM CHORD = 1.4 x 2440 = 3410# EL, LRFD
(FOR TRUSS DESIGN GROUP FOR SUPPLIER)

DSJ UPPER FLOOR DRAG STRUT JOIST

$T_{STRUT} = \frac{266^{\#}}{EL, ASD}$ (SW UF.4)

USE MIN (12) 16d Good for 12 x 141#/NAIL x 1.6 CD = 2710# ok

ST1 UPPER FLOOR STRAP

$T_{STRAP} = 1704^{\#}$ SW UF.5 $\times \frac{10'}{30'} - \frac{1020^{\#}}{EL, ASD}$ USE 'LSTA24' Good for 1235# ok

ST2 UPPER FLOOR STRAP

$T_{STRAP} = 2412^{\#}$ SW UF.6 $\times \frac{12'}{32'} = \frac{904^{\#}}{EL, ASD}$ USE 'LSTA24'

ST3 UPPER FLOOR STRAP

$T_{STRAP} = \left(3151^{\#}\right.$ SW UF.7 $+ \underbrace{2 \times 1367^{\#}}_{\text{SW MF.7 1/2 DIAPHRAGM STRAP}} \times \frac{8.5}{6.1}\left.\right) \times \frac{13'}{33'} = 2740^{\#}$ ($\frac{F_{PUF}}{F_{UF}}$) USE 'MSTC40' Good for 3070# ok

ST5 UPPER FLOOR STRAP

$T_{STRAP} = 2888^{\#}$ SW UF.59 $\times \frac{26'}{42'} = 1790^{\#}$ USE 'MSTA36'

Good for 2050# ok

DTT1 UPPER FLOOR STRAP

T TIE = 3422# / EL, ASD (SW UF.C) — Use 'HDU4' Good for 4565# ok

ST8 UPPER FLOOR STRAP

$T_{STRAP} = \left(4086^{\#} + 5930^{\#} \times \frac{8.5 psf}{6.1 psf}\right)\left(\frac{14'}{58'}\right) = \frac{2980^{\#}}{EL, ASD}$

(4086 = VABV, SW MF.B6; 5930 = VCUR; 8.5 psf = FP UF; 6.1 psf = FUF)

Use (2) 'LS16' Good for 2 × 1705 = 3410# ok

ST9 UPPER FLOOR STRAP

$T_{STRAP} = \left(4086 + 5920 \times \frac{8.5}{6.1}\right)\left(\frac{26'}{58'}\right) = \frac{5530^{\#}}{EL, ASD}$ Use 'MSTC66' Good for 5580# ok

ST13 UPPER FLOOR STRAP

$T_{STRAP} = 2888^{\#} \times \frac{8'}{42'} = 550^{\#}$ (SW UF.B1) Use 'LS20' Good for 1030# ok

UPPER FLOOR DIAPHRAGM

REC ROOM FLOOR DIAPHRAGM (WORST CASE 1)

$$v_{DIA} = (0.7)\left[8.5\,psf \times 29' + 0.17 \times \frac{8''}{12''} \times \frac{10'}{2} \times 150\,PCF \times \frac{8.5}{6.1}\right]\left(\frac{31'}{29' \times 2}\right) = 140\,\#/ft$$

(FLOOR; CONCRETE)

USE 3/4" WSP w/ 10d @ 6 EN, 12 FN (1,000 FOR 215 #/FT OK)

FLOOR DIAPHRAGM @ G.L. B.1 (WORST CASE 2)

$$v_{DIA} = \left(\underbrace{408\,lb}_{SW} + \underbrace{5931}_{WALL\ DL}\right) \times \frac{8.5}{6.1}\left(\frac{14'}{58'}\right) = 213\,\#/ft$$

(F_{DIF}; F_{DIF}; COLLECTOR)

USE 3/4" WSP w/ 10d @ 6 EN, 12 FN

SEISMIC BRACING @ T-0. FULL-HEIGHT CONCRETE WALL
ASCE 7-16 § 12.11.2.1

$R_{TOP} = F_p = 0.4 S_{DS} k_a I_e W_p$

$= 0.4 \times 1.11 \times \left(1 + \frac{30'}{10'}\right) \times 1 \times \left(\frac{8"}{12} \times 150 \text{ PCF} \times \frac{10'}{2}\right) = 290 \#/\text{FT}$
EL, LRFD

USE 'L50' @ 16" O.C.

GOOD FOR $525\# \times 12/16 = 390 \#/\text{FT}$ OK

$\times 0.7 = 200 \#/\text{FT}$
EL, ASD

USE 4" AT MASONRY W/ 5/8" ϕ BOLT @ 16" O.C.

GOOD FOR $490\# \times 1.6^{CD} \times 12/16$
$= 590 \#/\text{FT}$ OK

FOUNDATIONS

(F1/F2/F3) EXTERIOR FOUNDATION WALL (WORST CASE @ GL Ⓐ STAIR ⑥-⑦)

$$w = \left(\underset{DL}{18} + \underset{ML}{20} + \underset{SL}{25}\right)_{ROOF}\left(\frac{25'}{2}\right) + \left(\underset{DL}{14} + \underset{LL}{40}\right)_{FLOORS}\left(\frac{21'}{2} + \frac{11'}{2}\right) + \left(\underset{DL}{10}\right)_{WALL}(26')$$

$$= \underset{DL}{710} + \underset{ML}{250} + \underset{SL}{310} + \underset{LL}{640} \ \#/ft \qquad W_{FACTORED} = 710 + 0.75(310 + 640)$$

$$= 1420 \ \#/ft$$

USE 18" WIDE STRIP FTG

GOOD FOR 1.5' x 2000 PSF = 3000 #/ft ok

(F4) EXTERIOR FOUNDATION WALL w/ FULL HEIGHT CONCRETE

$$w = \left(\underset{DL}{18} + \underset{ML}{20} + \underset{SL}{25}\right)_{ROOF}\left(\frac{28'}{2}\right) + \left(\underset{DL}{14} + \underset{LL}{40}\right)_{UF}\left(\frac{24'}{2}\right) + \left(\underset{DL}{10}\right)_{WALL}(8') + \left(\underset{DL}{100}\right)_{CONC. WALL}(10')$$

$$= \underset{DL}{1570} + \underset{ML}{280} + \underset{SL}{350} + \underset{LL}{480} \ \#/ft \qquad W_{FACTORED} = 1570 + 0.75(350 + 480)$$

$$= 2130 \ \#/ft$$

USE ~~2'6"~~ WIDE STRIP ~~FTG~~

GOOD FOR 2.5' x 2000 = 5000 #/ft ok

(F5) INTERIOR FOUNDATION WALL (WORST CASE @ GL (5.5))

$$P = \underset{DL}{10710} + \underset{ML}{1771} + \underset{SL}{843} + \underset{LL}{14660}\ \# \ @\ x = 8'\text{-}3" \qquad RXN\ (VFB10)$$

L = 15'

USE 8" x 48" STEM w/ 18" WIDE STRIP FTG

(F6) PATIO RETAINING WALL

RETAINED HEIGHT = 4'-0" ACTIVE PRESSURE = 35 PCF

SEISMIC PRESSURE = 8H = 32 PSF

USE 8" STEM, 2'-0" TOE, 1'-0" HEEL, 8" KEY

(F7) INTERIOR PAD FOOTING (WORST CASE)

$$P = \left(\underset{DL}{14} + \underset{LL}{40}\right)_{MF}\left(\frac{21'}{2}\right)\left(\frac{11'}{2}\right) + \left(\underset{DL}{380} + \underset{ML}{240} + \underset{SL}{240} + \underset{LL}{240}\ \#\right)_{(MFB3)} = \underset{DL}{1410} + \underset{ML}{240} + \underset{SL}{240} + \underset{LL}{3100}\ \#$$

USE 2'0" SQ. PAD GOOD FOR 2^2 x 2000 = 8000 # ok

(F8) INTERIOR PAD FOOTING

FROM UF813

$P = \underset{PC}{4310} + \underset{LL}{2810} + \underset{SL}{4200} + \underset{W}{626^{\#}}$

$P_{AT\ FOOTING} = 4310 + 0.75(4200 + 626)$
$= 12240^{\#}$

U_{ULT} 24" SQ. PAD

GOOD FOR $2.5^2 \times 2000 = 12500^{\#}$ OK

(F9) EXTERIOR PAD FOOTING

FROM (LB4)

$P = \underset{PC}{980} + \underset{LL}{1150} + \underset{SL}{1350^{\#}}$

U_{ULT} 24" SQ. PAD

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 2 APR 2021, 4:06PM

Multiple Simple Beam

File: 21006_Plummer.ec6

Lic. # : KW-06011183

O.G. Engineering, PLLC

Description : Roof Framing

Wood Beam Design : RH1 - Roof Header

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

BEAM Size : **2-1.75x11.87, Microllam LVL, Fully Braced**

Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	2,000.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	1,016.54 ksi		

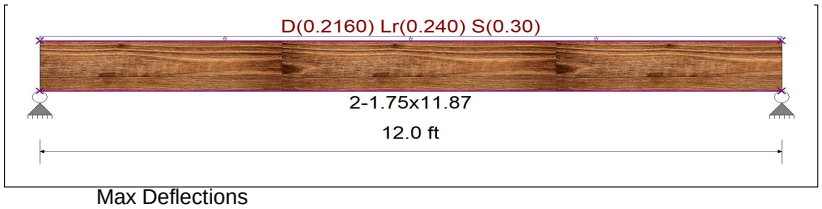
Applied Loads

Unif Load: D = 0.0180, Lr = 0.020, S = 0.0250 k/ft, Trib= 12.0 ft

Design Summary

Max fb/Fb Ratio = **0.453** : 1
fb : Actual : 1,354.94 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,990.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.286** : 1
fv : Actual : 93.86 psi at 11.040 ft in Span # 1
Fv : Allowable : 327.75 psi
Load Comb : +D+S



Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	1.30		1.44	1.80			
Right Support	1.30		1.44	1.80			

Max Deflections

Transient Downward	0.144 in	Total Downward	0.248 in
Ratio	999	Ratio	581
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Description : Upper Floor Framing

Wood Beam Design : UFB4 - Upper Floor Beam

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

BEAM Size : **5.5x11.875, GLB, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

Beam self weight calculated and added to loads
Unif Load: D = 0.210, Lr = 0.110, S = 0.140 k/ft, Trib= 1.0 ft

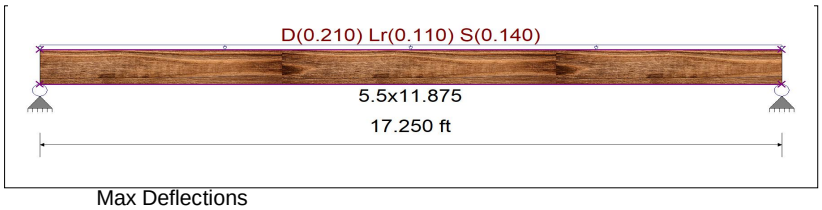
Design Summary

Max fb/Fb Ratio = **0.456** : 1
fb : Actual : 1,257.41 psi at 8.625 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.237** : 1
fv : Actual : 72.13 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	1.93		0.95	1.21			
Right Support	1.93		0.95	1.21			



Max Deflections

Transient Downward	0.203 in	Total Downward	0.528 in
Ratio	1019	Ratio	392
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Steel Beam Design : UFB5 - Upper Floor Beam

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

STEEL Section : **W10x26, Fully Braced**
Using Allowable Strength Design with ASCE 7-16 Load Combinations, Major Axis Bending — Fy = 50.0 ksi E = 29,000.0 ksi

Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.440, Lr = 0.240, L = 0.340, S = 0.30 k/ft, Trib= 1.0 ft
Point: D = 1.980, Lr = 0.950, S = 1.210, E = 5.855 k @ 16.0 ft

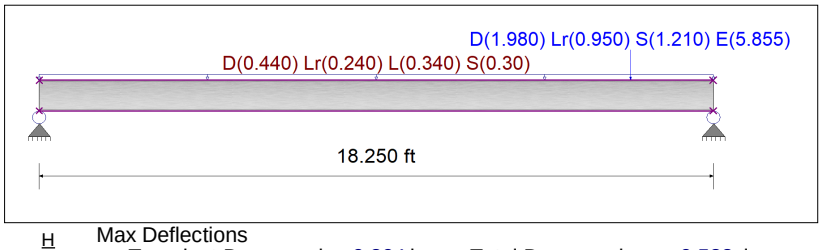
Design Summary

Max fb/Fb Ratio = **0.626** : 1
Mu : Applied 48.898 k-ft at 9.916 ft in Span # 1
Mn / Omega : Allow 78.094 k-ft
Load Comb : +1.117D+0.750L+0.750S+0.5250

Max fv/FvRatio = **0.272** : 1
Vu : Applied 14.556 k at 18.250 ft in Span # 1
Vn / Omega : Allow 53.560 k
Load Comb : +1.117D+0.750L+0.750S+0.5250

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	4.50	3.10	2.31	2.89		0.72	
Right Support	5.99	3.10	3.02	3.80		5.13	



Max Deflections

Transient Downward	0.204 in	Total Downward	0.522 in
Ratio	1071		419
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Steel Beam Design : UFB6 - Upper Floor Beam

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

STEEL Section : **W10x19, Fully Braced**
Using Allowable Strength Design with ASCE 7-16 Load Combinations, Major Axis Bending

Fy = 50.0 ksi E = 29,000.0 ksi

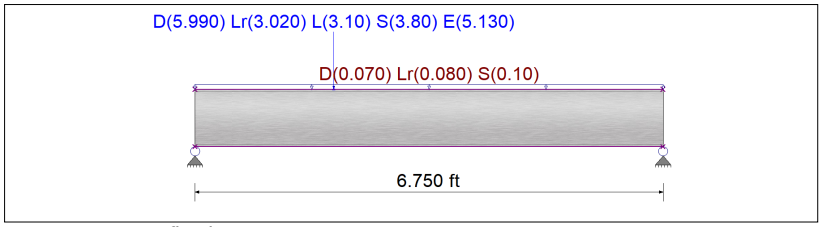
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.070, Lr = 0.080, S = 0.10 k/ft, Trib= 1.0 ft
Point: D = 5.990, Lr = 3.020, L = 3.10, S = 3.80, E = 5.130 k @ 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.395** : 1
Mu : Applied 21.305 k-ft at 2.003 ft in Span # 1
Mn / Omega : Allow 53.892 k-ft
Load Comb : +1.117D+0.750L+0.750S+0.5250

Max fv/FvRatio = **0.212** : 1
Vu : Applied 10.832 k at 0.000 ft in Span # 1
Vn / Omega : Allow 51.0 k
Load Comb : +1.117D+0.750L+0.750S+0.5250



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	4.52	2.18	2.40	3.01		3.61	
Right Support	2.08	0.92	1.16	1.46		1.52	

Max Deflections

Transient Downward	0.014 in	Total Downward	0.034 in
Ratio	5908	Ratio	2370
	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 : UFB7 - Upper Floor Beam

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

BEAM Size : **3.5x11.875, GLB, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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		

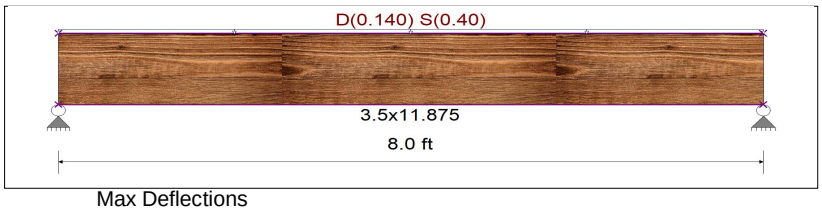
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.140, S = 0.40 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.232** : 1
fb : Actual : 640.72 psi at 4.000 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.260** : 1
fv : Actual : 79.26 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.60			1.60			
Right Support	0.60			1.60			

Max Deflections

Transient Downward	0.042 in	Total Downward	0.058 in
Ratio	2277	Ratio	1659
	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 : UFB8 - Upper Floor Beam

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

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

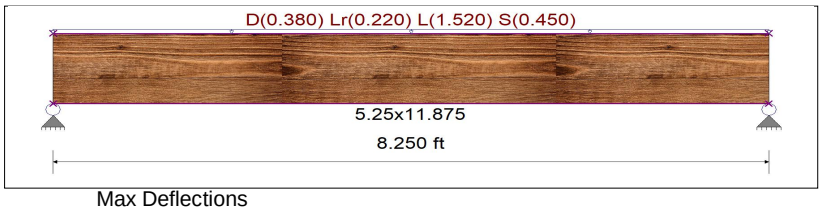
Beam self weight calculated and added to loads
Unif Load: D = 0.380, Lr = 0.220, L = 1.520, S = 0.450 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.548** : 1
fb : Actual : 1,588.24 psi at 4.125 ft in Span # 1
Fb : Allowable : 2,900.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.657** : 1
fv : Actual : 190.51 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.65	6.27	0.91	1.86			
Right Support	1.65	6.27	0.91	1.86			



Max Deflections

Transient Downward	0.099 in	Total Downward	0.125 in
Ratio	1001	Ratio	793
	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 : UFB9 - Upper Floor Beam

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

BEAM Size : **5.5x11.875, GLB, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

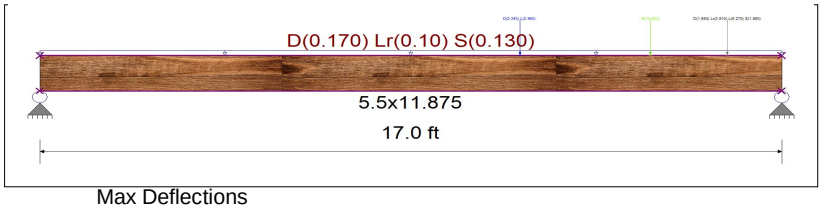
Beam self weight calculated and added to loads
Unif Load: D = 0.170, Lr = 0.10, S = 0.130 k/ft, Trib= 1.0 ft
Point: D = 0.340, L = 0.960 k @ 11.0 ft
Point: D = 1.650, Lr = 0.910, L = 6.270, S = 1.860 k @ 15.750 ft
Point: E = 10.620 k @ 14.0 ft

Design Summary

Max fb/Fb Ratio = **0.762** : 1
fb : Actual : 2,927.30 psi at 11.333 ft in Span # 1
Fb : Allowable : 3,840.00 psi
Load Comb : +1.117D+0.750L+0.750S+0.5250

Max fv/FvRatio = **0.844** : 1
fv : Actual : 223.79 psi at 17.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.81	0.80	0.92	1.24		1.87	
Right Support	3.31	6.43	1.69	2.83		8.75	



Max Deflections

Transient Downward	0.291 in	Total Downward	0.626 in
Ratio	702	Ratio	325
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Steel Beam Design : UFB10 - Upper Floor Beam

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16

STEEL Section : **W18x40, Fully Braced**
Using Allowable Strength Design with ASCE 7-16 Load Combinations, Major Axis Bending

Fy = 50.0 ksi E = 29,000.0 ksi

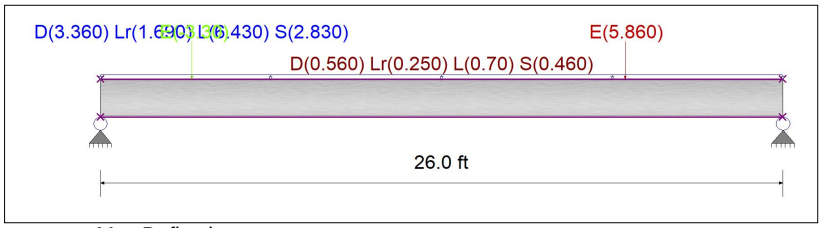
Applied Loads

Beam self weight calculated and added to loads
Unif Load: D = 0.560, Lr = 0.250, L = 0.70, S = 0.460 k/ft, Trib= 1.0 ft
Point: D = 3.360, Lr = 1.690, L = 6.430, S = 2.830 k @ 3.50 ft
Point: E = -3.30 k @ 3.50 ft
Point: E = 5.860 k @ 20.0 ft

Design Summary

Max fb/Fb Ratio = **0.793** : 1
Mu : Applied 155.136 k-ft at 12.653 ft in Span # 1
Mn / Omega : Allow 195.609 k-ft
Load Comb : +1.117D+0.750L+0.750S+0.5250

Max fv/FvRatio = **0.253** : 1
Vu : Applied 28.489 k at 0.000 ft in Span # 1
Vn / Omega : Allow 112.770 k
Load Comb : +1.117D+0.750L+0.750S+0.5250



Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	10.71	14.66	4.71	8.43		-1.50	
Right Support	8.25	9.97	3.48	6.36		4.06	

Max Deflections

Transient Downward	0.499 in	Total Downward	0.896 in
Ratio	625		348
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Wood Beam Design : UFH11 - Upper Floor Beam

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

BEAM Size : **5.25x16.0, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with ASCE 7-16 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

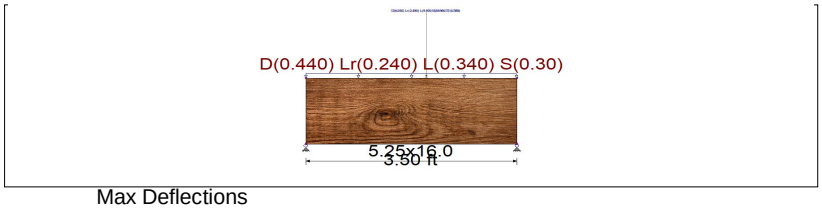
Beam self weight calculated and added to loads
Unif Load: D = 0.440, Lr = 0.240, L = 0.340, S = 0.30 k/ft, Trib= 1.0 ft
Point: D = 4.50, Lr = 2.310, L = 3.10, S = 2.890, E = -0.720 k @ 2.0 ft
Point: D = 8.250, Lr = 3.480, L = 9.970, S = 6.360, E = 4.060 k @ 2.0 ft

Design Summary

Max fb/Fb Ratio = **0.447** : 1
fb : Actual : 1,247.49 psi at 1.995 ft in Span # 1
Fb : Allowable : 2,792.04 psi
Load Comb : +D+L

Max fv/FvRatio = **0.995** : 1
fv : Actual : 288.67 psi at 3.500 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	6.28	6.20	2.90	4.49		1.43	
Right Support	8.10	8.06	3.73	5.81		1.91	



Max Deflections

Transient Downward	0.005 in	Total Downward	0.011 in
Ratio	7929	Ratio	3970
	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 : UFB12 - Upper Floor Beam

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

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

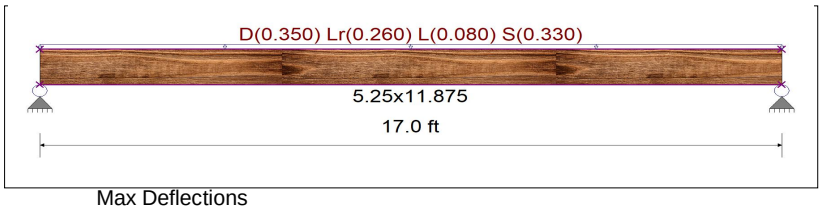
Beam self weight calculated and added to loads
Unif Load: D = 0.350, Lr = 0.260, L = 0.080, S = 0.330 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.737** : 1
fb : Actual : 2,457.59 psi at 8.500 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.429** : 1
fv : Actual : 143.06 psi at 17.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	3.14	0.68	2.21	2.81			
Right Support	3.14	0.68	2.21	2.81			



Max Deflections

Transient Downward	0.387 in	Total Downward	0.820 in
Ratio	527	Ratio	248
	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 : UFB13 - Upper Floor Beam

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

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

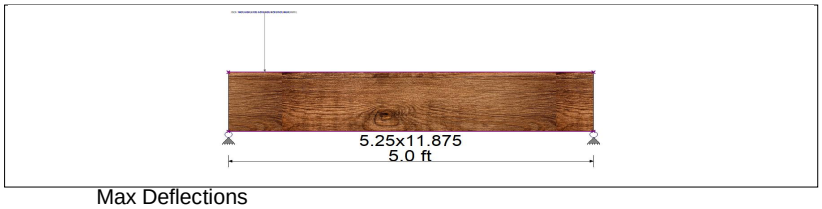
Beam self weight calculated and added to loads
Point: D = 1.650, Lr = 0.910, L = 6.270, S = 1.860 k @ 0.50 ft
Point: D = 3.140, Lr = 2.210, L = 0.680, S = 2.810, E = -2.820 k @ 0.50 ft

Design Summary

Max fb/Fb Ratio = **0.178** : 1
fb : Actual : 515.93 psi at 0.500 ft in Span # 1
Fb : Allowable : 2,900.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.881** : 1
fv : Actual : 255.39 psi at 0.000 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	4.36	6.26	2.81	4.20		-2.54	
Right Support	0.53	0.70	0.31	0.47		-0.28	



Max Deflections

Transient Downward	0.006 in	Total Downward	0.010 in
Ratio	9999	Ratio	5891
	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 : UFH14 - Upper Floor Header

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

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

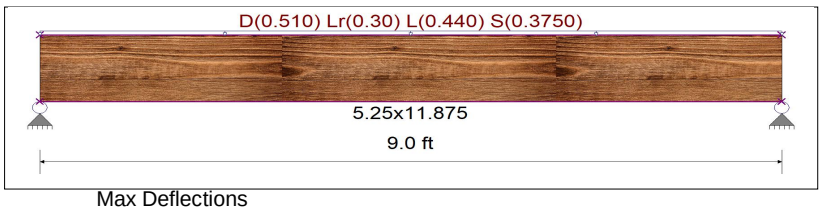
Beam self weight calculated and added to loads
Unif Load: D = 0.510, Lr = 0.30, L = 0.440, S = 0.3750 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.337** : 1
fb : Actual : 1,123.30 psi at 4.500 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.370** : 1
fv : Actual : 123.51 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.38	1.98	1.35	1.69			
Right Support	2.38	1.98	1.35	1.69			



Max Deflections

Transient Downward	0.041 in	Total Downward	0.089 in
Ratio	2665	Ratio	1209
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Wood Beam Design : UFB12 - Upper Floor Beam

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

BEAM Size : **5.25x11.875, Parallam PSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

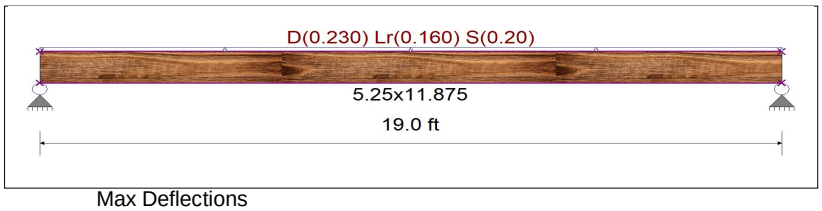
Beam self weight calculated and added to loads
Unif Load: D = 0.230, Lr = 0.160, S = 0.20 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.592** : 1
fb : Actual : 1,972.72 psi at 9.500 ft in Span # 1
Fb : Allowable : 3,335.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.308** : 1
fv : Actual : 102.75 psi at 0.000 ft in Span # 1
Fv : Allowable : 333.50 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	2.37		1.52	1.90			
Right Support	2.37		1.52	1.90			



Max Deflections

Transient Downward	0.366 in	Total Downward	0.822 in
Ratio	623	Ratio	277
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Description : Low Roof Framing

Wood Beam Design : LRR2 - Low Roof Rafters

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

BEAM Size : **2x6, Sawn, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch — Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0180, Lr = 0.020, S = 0.0250 k/ft, Trib= 2.0 ft

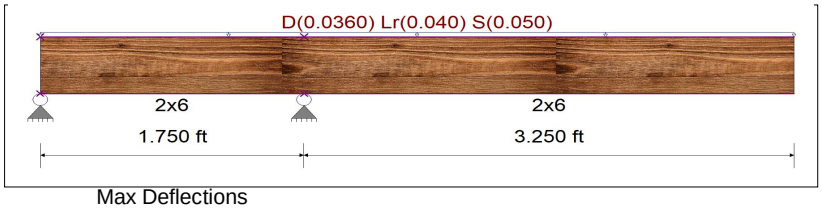
Design Summary

Max fb/Fb Ratio = **0.466** : 1
fb : Actual : 720.69 psi at 1.750 ft in Span # 1
Fb : Allowable : 1,547.33 psi
Load Comb : +D+S

Max fv/FvRatio = **0.260** : 1
fv : Actual : 53.76 psi at 1.295 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	-0.08		-0.09	-0.11			
Right Support	0.26		0.29	0.36			



Max Deflections

Transient Downward	0.060 in	Total Downward	0.104 in
Ratio	1294	Ratio	752
	LC: S Only		LC: +D+S
Transient Upward	-0.002 in	Total Upward	-0.004 in
Ratio	8741	Ratio	5082
	LC: S Only		LC: +D+S

Wood Beam Design : LRB3 - Low Roof Hip Beam

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

BEAM Size : **2x8, Sawn, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch — Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D= 0.090->0.0, Lr = 0.140->0.0, S = 0.180->0.0 k/ft, 0.0 to 6.750 ft

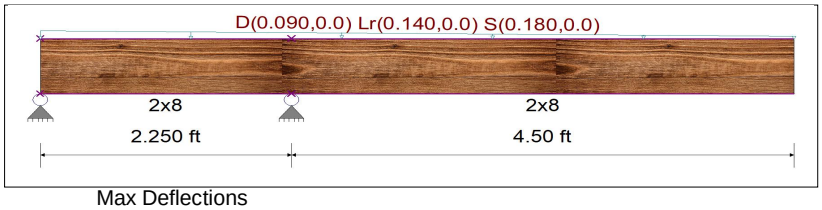
Design Summary

Max fb/Fb Ratio = **0.447** : 1
fb : Actual : 554.77 psi at 2.250 ft in Span # 1
Fb : Allowable : 1,242.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.337** : 1
fv : Actual : 69.83 psi at 2.250 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	0.00		-0.00	0.00			
Right Support	0.30		0.47	0.61			



Max Deflections

Transient Downward	0.061 in	Total Downward	0.091 in
Ratio	1772	Ratio	1182
	LC: S Only		LC: +D+S
Transient Upward	-0.002 in	Total Upward	-0.003 in
Ratio	9999	Ratio	9399
	LC: S Only		LC: +D+S

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Wood Beam Design : LRB4 - Low Roof Beam

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

BEAM Size : **5.5x10.5, GLB, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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

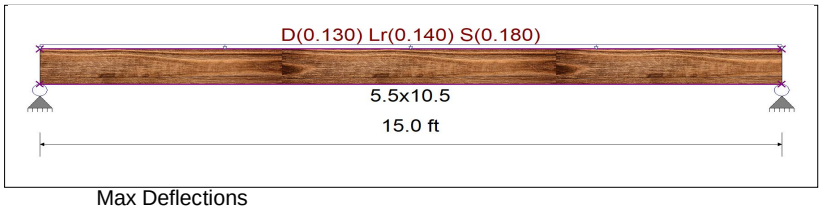
Unif Load: D = 0.130, Lr = 0.140, S = 0.180 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.375** : 1
fb : Actual : 1,035.25 psi at 7.500 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.176** : 1
fv : Actual : 53.55 psi at 0.000 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.98		1.05	1.35			
Right Support	0.98		1.05	1.35			



Max Deflections

Transient Downward	0.216 in	Total Downward	0.372 in
Ratio	833	Ratio	484
	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 : LRH5 - Low Roof Header

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

BEAM Size : **2-2x12, Sawn, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch — Wood Grade : No.2

Fb - Tension	900.0 psi	Fc - Prll	1,350.0 psi	Fv	180.0 psi	Ebend- xx	1,600.0 ksi	Density	31.210 pcf
Fb - Compr	900.0 psi	Fc - Perp	625.0 psi	Ft	575.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

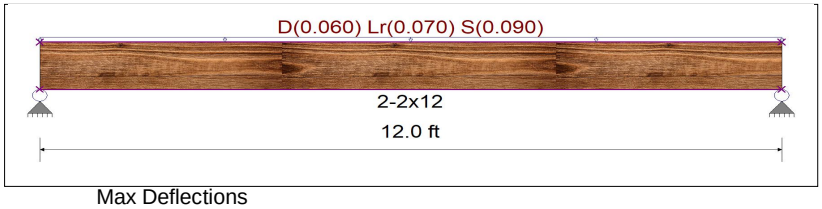
Unif Load: D = 0.060, Lr = 0.070, S = 0.090 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.495** : 1
fb : Actual : 512.00 psi at 6.000 ft in Span # 1
Fb : Allowable : 1,035.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.164** : 1
fv : Actual : 33.87 psi at 11.080 ft in Span # 1
Fv : Allowable : 207.00 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.36		0.42	0.54			
Right Support	0.36		0.42	0.54			



Max Deflections

Transient Downward	0.074 in	Total Downward	0.124 in
Ratio	1942	Ratio	1165
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 11:59PM

Multiple Simple Beam

File: 21006_Plummer.ec6

Lic. # : KW-06011183

O.G. Engineering, PLLC

Wood Beam Design : LRB6 - Low Roof Beam

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

BEAM Size : **3.5x11.875, GLB, Fully Braced**

Using Allowable Stress Design with ASCE 7-16 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

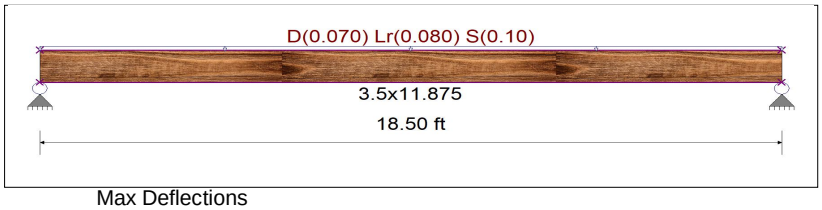
Unif Load: D = 0.070, Lr = 0.080, S = 0.10 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.384** : 1
fb : Actual : 1,060.96 psi at 9.250 ft in Span # 1
Fb : Allowable : 2,760.00 psi
Load Comb : +D+S

Max fv/FvRatio = **0.166** : 1
fv : Actual : 50.70 psi at 17.513 ft in Span # 1
Fv : Allowable : 304.75 psi
Load Comb : +D+S

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.65		0.74	0.93			
Right Support	0.65		0.74	0.93			



Max Deflections

Transient Downward	0.301 in	Total Downward	0.512 in
Ratio	736	Ratio	433
	LC: S Only		LC: +D+S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Description : Upper Deck Framing

Wood Beam Design : UDJ1 - Upper Deck Joists

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

BEAM Size : **1.750 X 8.0, Microllam LVL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	2,000.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

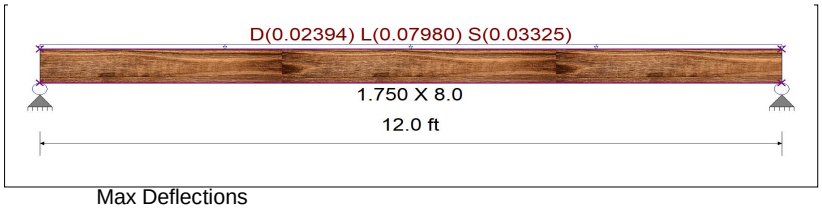
Unif Load: D = 0.0180, L = 0.060, S = 0.0250 k/ft, Trib= 1.330 ft

Design Summary

Max fb/Fb Ratio = **0.462** : 1
fb : Actual : 1,200.42 psi at 6.000 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.234** : 1
fv : Actual : 66.69 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.14	0.48		0.20			
Right Support	0.14	0.48		0.20			



Max Deflections

Transient Downward	0.251 in	Total Downward	0.342 in
Ratio	574	Ratio	421
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : UDH2 - Upper Deck Header

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

BEAM Size : **2-1.75x9.5, Microllam LVL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	2,000.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

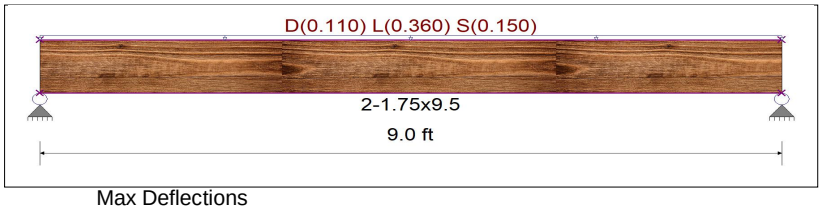
Unif Load: D = 0.110, L = 0.360, S = 0.150 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.417** : 1
fb : Actual : 1,084.70 psi at 4.500 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.335** : 1
fv : Actual : 95.41 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.50	1.62		0.68			
Right Support	0.50	1.62		0.68			



Max Deflections

Transient Downward	0.107 in	Total Downward	0.146 in
Ratio	1010	Ratio	738
	LC: L Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Wood Beam Design : UDJ3 - Upper Deck Joists

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

BEAM Size : **3.50 X 8.0, Microllam LVL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	2,000.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

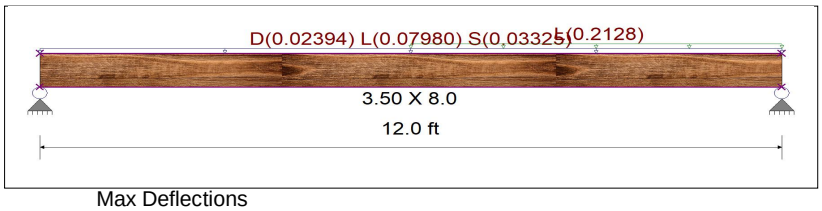
Unif Load: D = 0.0180, L = 0.060, S = 0.0250 k/ft, Trib= 1.330 ft
Unif Load: L = 0.160 k/ft, 6.0 to 12.0 ft, Trib= 1.330 ft

Design Summary

Max fb/Fb Ratio = **0.488** : 1
fb : Actual : 1,267.54 psi at 7.000 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.297** : 1
fv : Actual : 84.65 psi at 12.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.14	0.80		0.20			
Right Support	0.14	1.44		0.20			



Max Deflections

Transient Downward	0.293 in	Total Downward	0.331 in
Ratio	491	Ratio	435
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Multiple Simple Beam

Lic. # : KW-06011183 — O.G. Engineering, PLLC

Description : Main Floor Framing

Wood Beam Design : MFB3 - Main Floor Beam

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

BEAM Size : **3.5x11.875, TimberStrand LSL, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending
Wood Species : iLevel Truss Joist — Wood Grade : TimberStrand LSL 1.55E

Fb - Tension	2,325.0 psi	Fc - Prll	2,050.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	45.010 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	800.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Point: D = 2.510, Lr = 1.560, L = 1.560, S = 1.950 k @ 2.0 ft

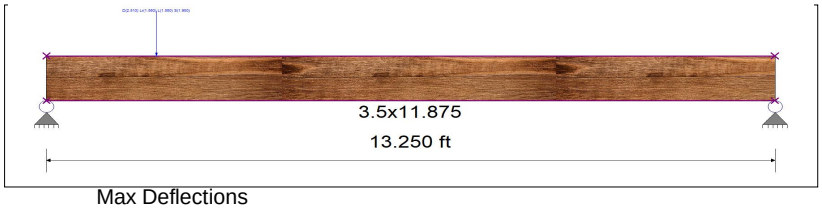
Design Summary

Max fb/Fb Ratio = **0.475** : 1
fb : Actual : 1,270.32 psi at 2.032 ft in Span # 1
Fb : Allowable : 2,673.75 psi
Load Comb : +D+0.750L+0.750S

Max fv/FvRatio = **0.442** : 1
fv : Actual : 157.58 psi at 0.000 ft in Span # 1
Fv : Allowable : 356.50 psi
Load Comb : +D+0.750L+0.750S

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	2.13	1.32	1.32	1.66			
Right Support	0.38	0.24	0.24	0.29			



Max Deflections

Transient Downward	0.098 in	Total Downward	0.257 in
Ratio	1630	Ratio	618
	LC: S Only		LC: +D+0.750L+0.750S
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

Wood Beam Design : MFB2 - Main Floor Beam

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

BEAM Size : **5.5x10.5, GLB, Fully Braced**
Using Allowable Stress Design with ASCE 7-16 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 = 0.360, L = 1.0 k/ft, Trib= 1.0 ft
Point: D = 0.380, Lr = 0.240, L = 0.240, S = 0.290 k @ 3.50 ft

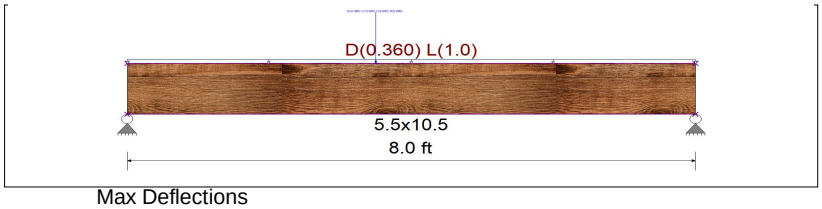
Design Summary

Max fb/Fb Ratio = **0.593** : 1
fb : Actual : 1,423.90 psi at 3.813 ft in Span # 1
Fb : Allowable : 2,400.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.567** : 1
fv : Actual : 150.36 psi at 0.000 ft in Span # 1
Fv : Allowable : 265.00 psi
Load Comb : +D+L

Max Reactions (k)

	D	L	Lr	S	W	E	H
Left Support	1.65	4.14	0.14	0.16			
Right Support	1.61	4.11	0.11	0.13			



Max Deflections

Transient Downward	0.102 in	Total Downward	0.144 in
Ratio	945	Ratio	668
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 5 MAY 2021, 8:24PM

Multiple Simple Beam

File: 21006_Plummer.ec6

Lic. # : KW-06011183

O.G. Engineering, PLLC

Wood Beam Design : MFJ4 - Main Floor Joists Blw Hot Tub

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

BEAM Size : **1.75x11.87, Microllam LVL, Fully Braced**

Using Allowable Stress Design with ASCE 7-16 Load Combinations, Major Axis Bending

Wood Species : iLevel Truss Joist
Wood Grade : MicroLam LVL 2.0 E

Fb - Tension	2,600.0 psi	Fc - Prll	2,510.0 psi	Fv	285.0 psi	Ebend- xx	2,000.0 ksi	Density	42.010 pcf
Fb - Compr	2,600.0 psi	Fc - Perp	750.0 psi	Ft	1,555.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.0140, L = 0.040 k/ft, Trib= 1.330 ft

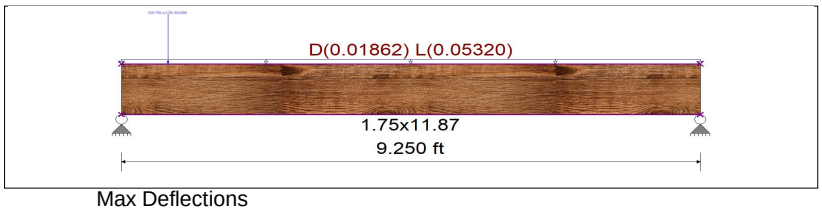
Point: D = 0.170, L = 1.70, S = 0.230 k @ 0.750 ft

Design Summary

Max fb/Fb Ratio = **0.183** : 1
fb : Actual : 475.40 psi at 2.528 ft in Span # 1
Fb : Allowable : 2,600.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.519** : 1
fv : Actual : 148.01 psi at 0.000 ft in Span # 1
Fv : Allowable : 285.00 psi
Load Comb : +D+L

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.24	1.81		0.21			
Right Support	0.10	0.38		0.02			



Max Deflections

Transient Downward	0.042 in	Total Downward	0.051 in
Ratio	2613	Ratio	2168
	LC: L Only		LC: +D+L
Transient Upward	0.000 in	Total Upward	0.000 in
Ratio	9999	Ratio	9999
	LC:		LC:

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 8:41PM

Wood Column

File: 21006_Plummer.ec6

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Post Supporting UFB7 (Worst Case 4x4)

Code References

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

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.2

Fb +	900 psi	Fv	180 psi
Fb -	900 psi	Ft	575 psi
Fc - Prll	1350 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1600	1600	1600 ksi
Minimum	580	580	

Wood Section Name	**4x4**		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	**3.50** in	Allow Stress Modification Factors	
Exact Depth	**3.50** in	Cf or Cv for Bending	1.50
Area	12.250 in^2	Cf or Cv for Compression	1.150
Ix	12.505 in^4	Cf or Cv for Tension	1.50
Iy	12.505 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 26.550 lbs * Dead Load Factor
AXIAL LOADS . . .
UFB7: Axial Load at 10.0 ft, D = 0.980, L = 2.80 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8160** : 1

Load Combination	+D+L
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	3.807 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	380.822 psi

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

Load Combination	+0.60D
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	288.0 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

Printed: 9 MAY 2021, 8:47PM

Wood Column

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Post Supporting South End of UFB4 (Worst Case 4x6)

Code References

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

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.2		
Fb +	900.0 psi	Fv	180.0 psi
Fb -	900.0 psi	Ft	575.0 psi
Fc - Prll	1,350.0 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	**4x6**		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	**3.50** in	Allow Stress Modification Factors	
Exact Depth	**5.50** in	Cf or Cv for Bending	1.30
Area	19.250 in^2	Cf or Cv for Compression	1.10
Ix	48.526 in^4	Cf or Cv for Tension	1.30
Iy	19.651 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 41.722 lbs * Dead Load Factor
AXIAL LOADS . . .
UFB4: Axial Load at 10.0 ft, D = 2.510, Lr = 1.560, L = 1.560, S = 1.950 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.7025** : 1

Load Combination	+D+0.750L+0.750S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	5.184 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	383.381 psi

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

Load Combination	+0.60D
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	288.0 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

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 9:03PM

Wood Column

File: 21006_Plummer.ec6

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Post @ GL B/6 (Worst Case 6x6)

Code References

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

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.2

Fb +	750 psi	Fv	170 psi
Fb -	750 psi	Ft	475 psi
Fc - Prll	700 psi	Density	31.21 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1300	1300	1300 ksi
Minimum	470	470	

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 Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	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 = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0

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 = 3.490, Lr = 2.120, L = 2.40, S = 2.740 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4386** : 1

Load Combination	+D+0.750L+0.750S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	7.411 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	558.50 psi

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

Load Combination	+0.60D
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

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 9:11PM

Wood Column

File: 21006_Plummer.ec6

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Post Supporting East End of UFH11 (Worst Case 5-1/4x5-1/4 PSL)

Code References

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

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 2.2E		
Fb +	2,900.0 psi	Fv	290.0 psi
Fb -	2,900.0 psi	Ft	2,025.0 psi
Fc - Prll	2,900.0 psi	Density	45.070 pcf
Fc - Perp	750.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	2,200.0	2,200.0	2,200.0 ksi
Minimum	1,118.19	1,118.19	

Wood Section Name	**5.25x5.25**		
Wood Grading/Manuf.	Trus Joist		
Wood Member Type	Parallam PSL		
Exact Width	**5.250** in	Allow Stress Modification Factors	
Exact Depth	**5.250** in	Cf or Cv for Bending	1.0
Area	27.563 in^2	Cf or Cv for Compression	1.0
Ix	63.308 in^4	Cf or Cv for Tension	1.0
Iy	63.308 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 10 ft, K = 1.0

Applied Loads

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

Column self weight included : 86.268 lbs * Dead Load Factor
AXIAL LOADS . . .
UFB13: Axial Load at 10.0 ft, D = 6.280, Lr = 2.90, L = 6.20, S = 4.490 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3243** : 1

Load Combination	+D+0.750L+0.750S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	14.384 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,609.25 psi

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

Load Combination	+0.60D
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	464.0 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

Printed: 9 MAY 2021, 11:14PM

Wood Column

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Post Supporting West End of UFH11 (Worst Case 3-1/2x5-1/4 PSL)

Code References

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

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 8 ft
(Used for non-slender calculations)

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

Fb +	2,900.0 psi	Fv	290.0 psi
Fb -	2,900.0 psi	Ft	2,025.0 psi
Fc - Prll	2,900.0 psi	Density	45.070 pcf
Fc - Perp	750.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	2,200.0	2,200.0	2,200.0 ksi
Minimum	1,118.19	1,118.19	

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 Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	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 = 8 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 8 ft, K = 1.0

Applied Loads

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

Column self weight included : 46.009 lbs * Dead Load Factor
AXIAL LOADS . . .
UFB13: Axial Load at 8.0 ft, D = 8.10, Lr = 3.730, L = 5.810, S = 8.060 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.8703** : 1

Load Combination	+D+0.750L+0.750S
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	
Applied Axial	18.549 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,159.89 psi

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

Load Combination	+0.60D
Location of max.above base	8.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	464.0 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

Steel Column

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Main Floor Post Supporting South End of UFB10

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

General Information

Steel Section Name : **HSS5x3x1/4**
Analysis Method : Allowable Strength
Steel Stress Grade
Fy : Steel Yield 46.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 11.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for buckling ABOUT Y-Y Axis = 11.0 ft, K = 1.0
Y-Y (depth) axis :
Unbraced Length for buckling ABOUT X-X Axis = 11.0 ft, K = 1.0

Applied Loads

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

Column self weight included : 133.989 lbs * Dead Load Factor
AXIAL LOADS . . .
UFB10: Axial Load at 11.0 ft, D = 10.710, LR = 4.710, L = 14.660, S = 8.430 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.6941** : 1
Load Combination +D+0.750L+0.750S
Location of max.above base 0.0 ft
At maximum location values are . . .
Pa : Axial 28.161 k
Pn / Omega : Allowable 40.571 k
Ma-x : Applied 0.0 k-ft
Mn-x / Omega : Allowable 12.349 k-ft
Ma-y : Applied 0.0 k-ft
Mn-y / Omega : Allowable 8.654 k-ft

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination 0.0
Location of max.above base 0.0 ft
At maximum location values are . . .
Va : Applied 0.0 k
Vn / Omega : Allowable 0.0 k

Maximum Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.0 in at 0.0 ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Rx	KyLy/Ry	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.267	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+D+L	0.629	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+D+Lr	0.383	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+D+S	0.475	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+D+0.750Lr+0.750L	0.625	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+D+0.750L+0.750S	0.694	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft
+0.60D	0.160	PASS	0.00 ft	1.00	1.00	74.16	110.92	0.000	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	10.844									
+D+L	25.504									
+D+Lr	15.554									
+D+S	19.274									
+D+0.750Lr+0.750L	25.371									
+D+0.750L+0.750S	28.161									
+0.60D	6.506									
Lr Only	4.710									

Steel Column

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: Main Floor Post Supporting South End of UFB10

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	k-ft	My - End Moments @ Base	My - End Moments @ Top
L Only	14.660										
S Only	8.430										

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	k	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top	k-ft	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	28.161										
"	Minimum	4.710										
Reaction, X-X Axis Base	Maximum	10.844										
"	Minimum	10.844										
Reaction, Y-Y Axis Base	Maximum	10.844										
"	Minimum	10.844										
Reaction, X-X Axis Top	Maximum	10.844										
"	Minimum	10.844										
Reaction, Y-Y Axis Top	Maximum	10.844										
"	Minimum	10.844										
Moment, X-X Axis Base	Maximum	10.844										
"	Minimum	10.844										
Moment, Y-Y Axis Base	Maximum	10.844										
"	Minimum	10.844										
Moment, X-X Axis Top	Maximum	10.844										
"	Minimum	10.844										
Moment, Y-Y Axis Top	Maximum	10.844										
"	Minimum	10.844										

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+Lr	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+0.750Lr+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
+0.60D	0.0000	in	0.000	ft	0.000	in	0.000	ft
Lr Only	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

Steel Section Properties : HSS5x3x1/4

Depth	=	5.000	in	I xx	=	10.70	in^4	J	=	11.000 in^4
Design Thick	=	0.233	in	S xx	=	4.29	in^3	Cw	=	6.10 in^6
Width	=	3.000	in	R xx	=	1.780	in			
Wall Thick	=	0.250	in	Zx	=	5.380	in^3			
Area	=	3.370	in^2	I yy	=	4.810	in^4	C	=	6.100 in^3
Weight	=	12.181	plf	S yy	=	3.210	in^3			
				R yy	=	1.190	in			
				Zy	=	3.770	in^3			
Ycg	=	0.000	in							

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 11:30PM

Steel Column

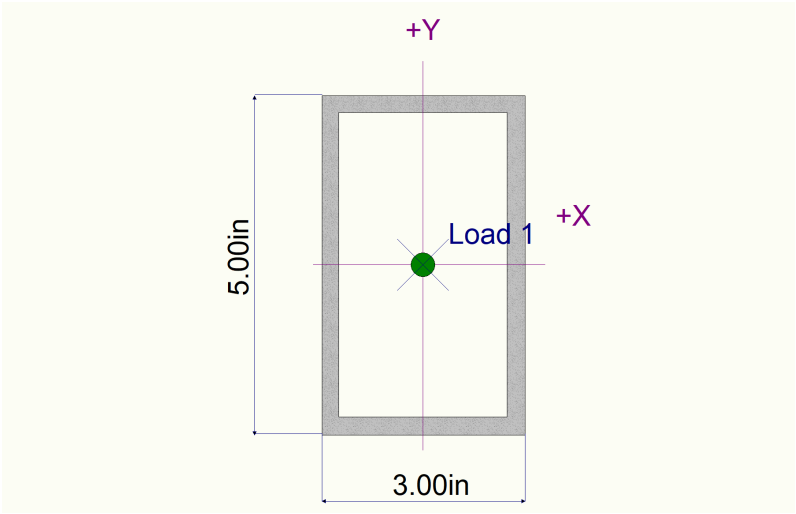
File: 21006_Plummer.ec6

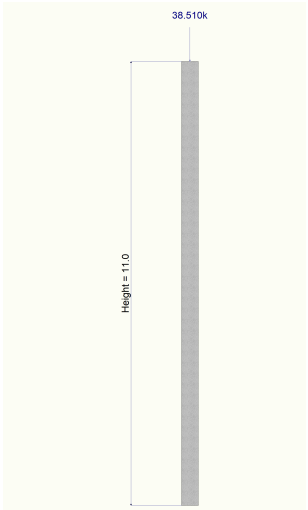
Lic. # : KW-06011183

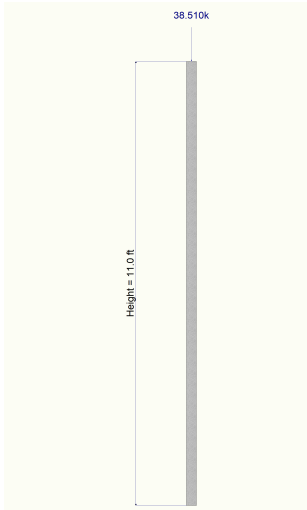
O.G. Engineering, PLLC

DESCRIPTION: Main Floor Post Supporting South End of UFB10

Sketches







Printed: 9 MAY 2021, 11:35PM

Beam on Elastic Foundation

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: F5 - Interior Foundation Wall

CODE REFERENCES

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : ASCE 7-16

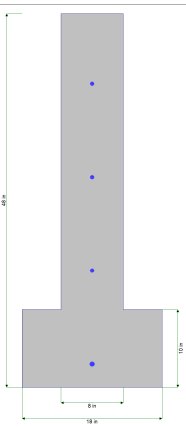
Material Properties

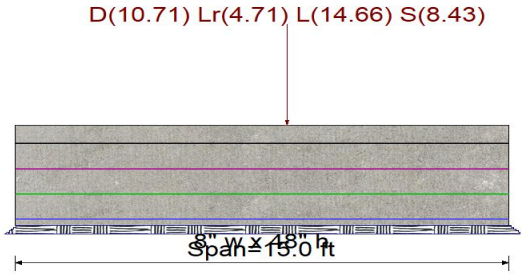
f'c	= 3.0 ksi	ϕ Phi Values	Flexure :	0.90	
fr = f'c$^{1/2}$ * 7.50	= 410.792 psi		Shear :	0.750	
ψ Density	= 145.0 pcf		β_1 =	0.850	
λ Lt Wt Factor	= 1.0				
Elastic Modulus	= 3,122.0 ksi				

Soil Subgrade Modulus = 100.0 psi / (inch deflection)
Load Combination : ASCE 7-16

fy - Main Rebar	= 60.0 ksi	Fy - Stirrups	= 60.0 ksi
E - Main Rebar	= 29,000.0 ksi	E - Stirrups	= 29,000.0 ksi
		Stirrup Bar Size #	= # 4
		Number of Resisting Legs Per Stirrup	1.0

Beam is supported on an elastic foundation.





Cross Section & Reinforcing Details

Inverted Tee Section, Stem Width = 8.0 in, Total Height = 48.0 in, Top Flange Width = 18.0 in, Flange Thickness = 10.0 in
Span #1 Reinforcing....
1-#5 at 3.0 in from Bottom, from 0.0 to 15.0 ft in this span
1-#4 at 21.0 in from Top, from 0.0 to 15.0 ft in this span
1-#4 at 15.0 in from Bottom, from 0.0 to 15.0 ft in this span
1-#4 at 9.0 in from Top, from 0.0 to 15.0 ft in this span

Applied Loads

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

Point Load : D = 10.710, Lr = 4.710, L = 14.660, S = 8.430 k @ 8.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.648**: 1	Maximum Deflection	
Section used for this span	**Typical Section**	Max Downward L+Lr+S Deflection	0.000 in
Mu : Applied	73.813 k-ft	Max Upward L+Lr+S Deflection	0.000 in
Mn * Phi : Allowable	113.995 k-ft	Max Downward Total Deflection	0.110 in
Load Combination	+1.20D+L+1.60S	Max Upward Total Deflection	0.000 in
Location of maximum on span	8.294 ft		
Span # where maximum occurs	Span # 1		

Maximum Soil Pressure = **1.585** ksf at 15.00 ft LdComb: +D+0.750L+0.750S
Allowable Soil Pressure = **2.0** ksf OK

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
Span 1	1	0.1100	15.000		0.0000	0.000

O.G. Engineering, PLLC
8645 22nd Ave SW
Seattle, WA 98106

Project Title:
Engineer:
Project ID:
Project Descr:

Printed: 9 MAY 2021, 11:53PM

Cantilevered Retaining Wall

File: 21006_Plummer.ec6

Lic. # : KW-06011183 — O.G. Engineering, PLLC

DESCRIPTION: F6 - Patio Retaining Wall

Calculations per ACI 318-14, TMS 402-16, IBC 2018, CBC 2019, ASCE 7-16

Criteria

Retained Height	=	5.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00 : 1
Height of Soil over Toe	=	12.00 in
Water height over heel	=	0.0 ft
Vertical component of active Lateral soil pressure options:		
NOT USED for Soil Pressure.		
NOT USED for Sliding Resistance.		
NOT USED for Overturning Resistance.		

Soil Data

Allow Soil Bearing	=	2,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	35.0 psf/ft
Toe Active Pressure	=	0.0 psf/ft
Passive Pressure	=	250.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Friction Coeff btwn Ftg & Soil	=	0.300
Soil height to ignore for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Lateral Load Applied to Stem

Lateral Load	=	40.0 plf
...Height to Top	=	5.00 ft
...Height to Bottom	=	0.00 ft
Wind on Exposed Stem	=	0.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft
Poisson's Ratio	=	0.300

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Wall Stability Ratios			
Overturning	=	2.13	OK
Sliding	=	1.60	OK
Total Bearing Load	=	1,889 lbs	
...resultant ecc.	=	8.00 in	
Soil Pressure @ Toe	=	1,078 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	2,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	1,293 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	8.9 psi	OK
Footing Shear @ Heel	=	7.4 psi	OK
Allowable	=	75.0 psi	
Sliding Calcs (Vertical Component NOT Used)			
Lateral Sliding Force	=	830.0 lbs	
less 100% Passive Force	= -	763.9 lbs	
less 100% Friction Force	= -	560.0 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	0.0 lbs	OK

Load Factors

Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.600
Seismic, E	1.000

Stem Construction

Top Stem

Stem OK

Design Height Above Ftg	ft =	0.00
Wall Material Above "Ht"	=	Concrete
Thickness	in =	8.00
Rebar Size	=	# 5
Rebar Spacing	in =	12.00
Rebar Placed at	=	User Spec
Design Data		
fb/FB + fa/Fa	=	0.222
Total Force @ Section	lbs =	900.0
Moment....Actual	ft-l =	1,666.7
Moment.....Allowable	ft-l =	7,512.3
Shear.....Actual	psi =	13.2
Shear.....Allowable	psi =	82.2
Wall Weight	psf =	100.0
Rebar Depth 'd'	in =	5.69
Lap splice if above	in =	12.00
Lap splice if below	in =	6.00
Hook embed into footing	in =	6.00
Concrete Data		
f'c	psi =	3,000.0
Fy	psi =	

Printed: 9 MAY 2021, 11:53PM

Cantilevered Retaining Wall

File: 21006_Plummer.ec6

Lic. # : KW-06011183
O.G. Engineering, PLLC

DESCRIPTION: F6 - Patio Retaining Wall

Footing Dimensions & Strengths

Toe Width	=	2.00 ft
Heel Width	=	1.67
Total Footing Width	=	3.67
Footing Thickness	=	12.00 in
Key Width	=	8.00 in
Key Depth	=	8.00 in
Key Distance from Toe	=	0.00 ft

f'c = 2,500 psi Fy = 60,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top 2.00 @ Btm.= 3.00 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	1,293	0 psf
Mu' : Upward	=	2,095	0 ft-lb
Mu' : Downward	=	624	423 ft-lb
Mu: Design	=	1,471	423 ft-lb
Actual 1-Way Shear	=	8.85	7.39 psi
Allow 1-Way Shear	=	75.00	75.00 psi
Toe Reinforcing	=	None Spec'd	
Heel Reinforcing	=	None Spec'd	
Key Reinforcing	=	None Spec'd	

Other Acceptable Sizes & Spacings

Toe: Not req'd, Mu < S * Fr
Heel: Not req'd, Mu < S * Fr
Key: Not req'd, Mu < S * Fr

Summary of Overturning & Resisting Forces & Moments

		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-lb			.....RESISTING..... Force lbs	Distance ft	Moment ft-lb
Heel Active Pressure	=	630.0	2.00	1,260.0	Soil Over Heel	=	551.8	3.17	1,748.4
Surcharge over Heel	=				Sloped Soil Over Heel	=			
Toe Active Pressure	=		0.67		Surcharge Over Heel	=			
Surcharge Over Toe	=				Adjacent Footing Load	=			
Adjacent Footing Load	=				Axial Dead Load on Stem	=			
Added Lateral Load	=	200.0	3.50	700.0	* Axial Live Load on Stem	=			
Load @ Stem Above Soil	=				Soil Over Toe	=	220.0	1.00	220.0
					Surcharge Over Toe	=			
					Stem Weight(s)	=	500.0	2.33	1,166.7
					Earth @ Stem Transitions	=			
Total	=	830.0	O.T.M. =	1,960.0	Footing Weight	=	550.5	1.84	1,010.2
					Key Weight	=	66.7	0.33	22.2
					Vert. Component	=			
					Total	=	1,889.0 lbs	R.M. =	4,167.4

Resisting/Overturning Ratio = 2.13
Vertical Loads used for Soil Pressure = 1,889.0 lbs

* Axial live load NOT included in total displayed, or used for overturning resistance, but is included for soil pressure calculation.

Printed: 14 MAY 2021, 6:00AM
File: 21006_Plummer.ec6

Lic. # : KW-06011183
O.G. Engineering, PLLC

Wood Column

DESCRIPTION: BFS - Balloon Framed Studs Adjacent To Stair

Code References

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

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	20 ft		
(Used for non-slender calculations)			
Wood Species	iLevel Truss Joist		
Wood Grade	MicroLam LVL 2.0 E		
Fb +	2,600.0 psi	Fv	285.0 psi
Fb -	2,600.0 psi	Ft	1,555.0 psi
Fc - Prll	2,510.0 psi	Density	42.010 pcf
Fc - Perp	750.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	2,000.0	2,000.0	2,000.0 ksi
Minimum	1,016.54	1,016.54	

Wood Section Name	**1.75x5.5**		
Wood Grading/Manuf.	Trus Joist		
Wood Member Type	Microllam LVL		
Exact Width	**1.750** in	Allow Stress Modification Factors	
Exact Depth	**5.50** in	Cf or Cv for Bending	1.0
Area	9.625 in^2	Cf or Cv for Compression	1.0
Ix	24.263 in^4	Cf or Cv for Tension	1.0
Iy	2.456 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 20 ft, K = 1.0

Applied Loads

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

Column self weight included : 56.159 lbs * Dead Load Factor
AXIAL LOADS . . .
Gravity: Axial Load at 20.0 ft, D = 0.240, Lr = 0.270, S = 0.330 k
BENDING LOADS . . .
Lateral: Lat. Uniform Load creating Mx-x, W = 0.040 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.4269** : 1
Load Combination +D+0.60W
Governing NDS Forumla 1Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 9.933 ft
At maximum location values are . . .
Applied Axial 0.2962 k
Applied Mx 1.20 k-ft
Applied My 0.0 k-ft
Fc : Allowable 433.585 psi

PASS Maximum Shear Stress Ratio = **0.08202** : 1
Load Combination +D+0.60W
Location of max.above base 20.0 ft
Applied Design Shear 37.403 psi
Allowable Shear 456.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.40 k	Bottom along Y-Y	0.40 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 3.0 in at 10.067 ft above base
for load combination : W Only
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

Printed: 14 MAY 2021, 6:01AM

Wood Column

File: 21006_Plummer.ec6

Lic. # : KW-06011183
O.G. Engineering, PLLC

DESCRIPTION: TB1 - Transom Beam Adjacent To Stair

Code References

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

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	6 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	**5.25x5.25**		
Wood Grading/Manuf.	Trus Joist		
Wood Member Type	Parallam PSL		
Exact Width	**5.250** in	Allow Stress Modification Factors	
Exact Depth	**5.250** in	Cf or Cv for Bending	1.0
Area	27.563 in^2	Cf or Cv for Compression	1.0
Ix	63.308 in^4	Cf or Cv for Tension	1.0
Iy	63.308 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 6 ft, K = 1.0

Applied Loads

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

Column self weight included : 51.761 lbs * Dead Load Factor
BENDING LOADS . . .
Lateral: Lat. Uniform Load creating Mx-x, W = 0.190 k/ft
Lateral: Lat. Uniform Load creating My-y, D = 0.060 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.1015** : 1
Load Combination +D+0.60W
Governing NDS Forumla np + Mxx + Myy, NDS Eq. 3.9-
Location of max.above base 3.020 ft
At maximum location values are . . .
Applied Axial 0.05176 k
Applied Mx 0.5130 k-ft
Applied My 0.270 k-ft
Fc : Allowable 3,038.39 psi

PASS Maximum Shear Stress Ratio = **0.06122** : 1
Load Combination +D+0.60W
Location of max.above base 6.0 ft
Applied Design Shear 18.612 psi
Allowable Shear 304.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.570 k	Bottom along Y-Y	0.570 k
Top along X-X	0.180 k	Bottom along X-X	0.180 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 0.04914 in at 3.020 ft above base
for load combination : W Only
Along X-X 0.01552 in at 3.020 ft above base
for load combination : D Only

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Printed: 14 MAY 2021, 6:04AM
File: 21006_Plummer.ec6

Lic. # : KW-06011183
O.G. Engineering, PLLC

Wood Column

DESCRIPTION: BFP - Balloon Framed Posts Adjacent To Stair

Code References

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

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	20 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	Allow Stress Modification Factors	
Exact Depth	**5.250** in	Cf or Cv for Bending	1.0
Area	18.375 in^2	Cf or Cv for Compression	1.0
Ix	42.205 in^4	Cf or Cv for Tension	1.0
Iy	18.758 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	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 : Fully braced against buckling ABOUT Y-Y Axis
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 20 ft, K = 1.0

Applied Loads

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

Column self weight included : 115.022 lbs * Dead Load Factor
AXIAL LOADS . . .
Gravity: Axial Load at 20.0 ft, D = 0.540, Lr = 0.60, S = 0.750 k
Gravity: Axial Load at 13.0 ft, D = 0.180 k
BENDING LOADS . . .
Lateral: Lat. Point Load at 13.0 ft creating Mx-x, W = 0.570 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3594** : 1
Load Combination +D+0.60W
Governing NDS Forumla 1Comp + Mxx, NDS Eq. 3.9-3
Location of max.above base 12.886 ft
At maximum location values are . . .
Applied Axial 0.8350 k
Applied Mx 1.542 k-ft
Applied My 0.0 k-ft
Fc : Allowable 356.373 psi

PASS Maximum Shear Stress Ratio = **0.05969** : 1
Load Combination +D+0.60W
Location of max.above base 20.0 ft
Applied Design Shear 18.147 psi
Allowable Shear 304.0 psi

Maximum SERVICE Lateral Load Reactions . .

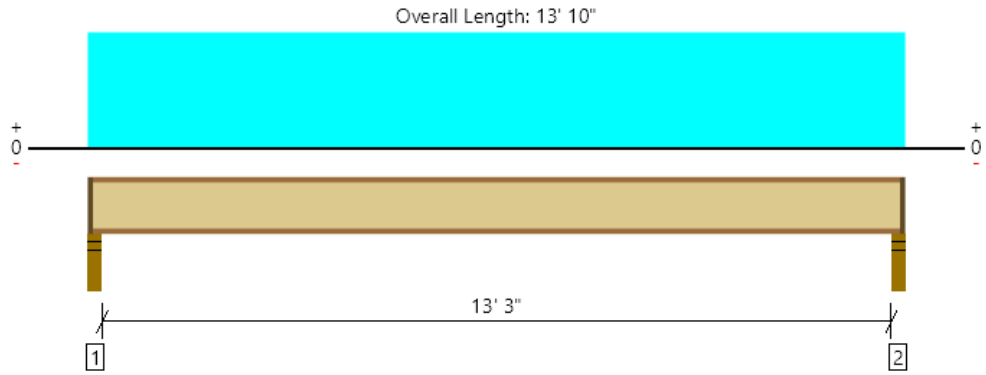
Top along Y-Y	0.3705 k	Bottom along Y-Y	0.1995 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y 1.932 in at 10.872 ft above base
for load combination : W Only
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

Permit, MFJ1 - Main Floor Joist
1 piece(s) 11 7/8" TJI® 360 @ 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)	491 @ 2 1/2"	1202 (2.25")	Passed (41%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	477 @ 3 1/2"	1705	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1620 @ 6' 11"	6180	Passed (26%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.100 @ 6' 11"	0.335	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.135 @ 6' 11"	0.671	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	60	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	129	369	498	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	1.75"	129	369	498	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)	7' 6" o/c	
Bottom Edge (Lu)	13' 8" 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 13' 10"	16"	14.0	40.0	Typical Floor

Member Notes

MFJ1 - Main Floor Joist

Weyerhaeuser Notes

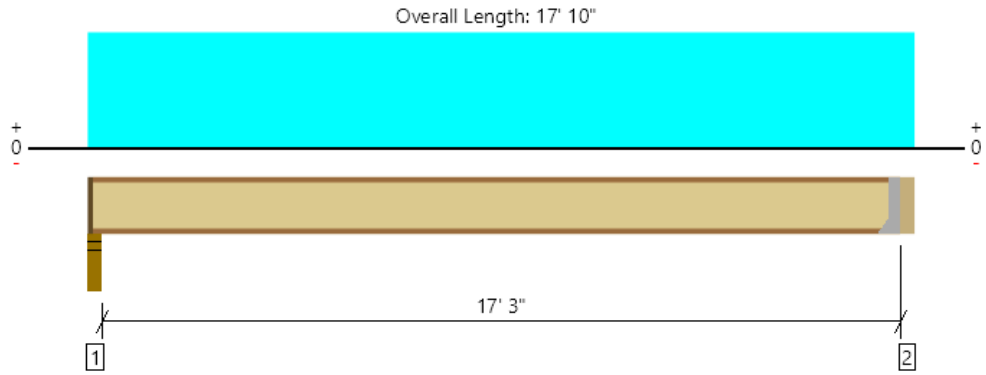
Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

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

Permit, UFJ1 - Upper Floor Joist
1 piece(s) 11 7/8" TJI® 560 @ 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)	624 @ 17' 6 1/2"	1265 (1.75")	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	624 @ 17' 6 1/2"	2050	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	2704 @ 8' 10 1/2"	9500	Passed (28%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.181 @ 8' 10 1/2"	0.433	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.244 @ 8' 10 1/2"	0.867	Passed (L/851)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	53	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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	Total	
1 - Stud wall - DF	3.50"	2.25"	1.75"	166	473	639	1 1/4" Rim Board
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	167	478	645	See note [1]

- Rim Board is assumed to carry all loads applied directly above it, bypassing 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)	10' 11" o/c	
Bottom Edge (Lu)	17' 5" 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 - Face Mount Hanger	IUS3.56/11.88	2.00"	N/A	12-10dx1.5	2-Strong-Grip	

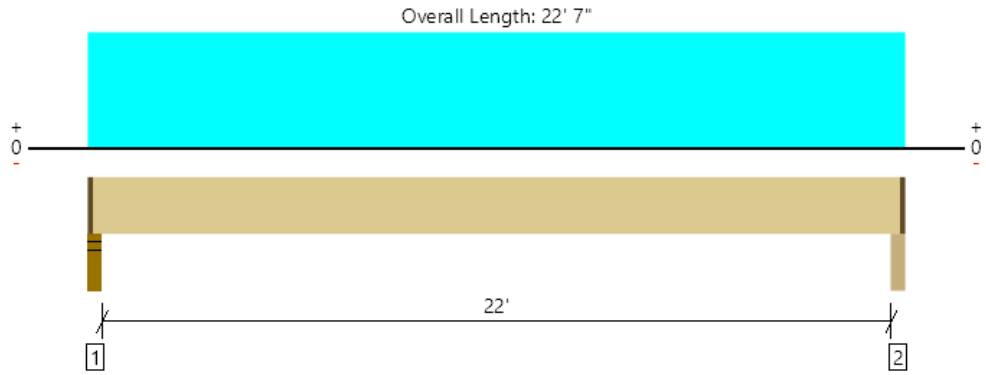
- Refer to manufacturer notes and instructions for proper installation and use of all connectors.

Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 17' 10"	16"	14.0	40.0	Typical Floor

Member Notes
UFJ1 - Upper Floor Joist

Permit, UFJ2 - Upper Floor Joist

1 piece(s) 3 1/2" x 11 7/8" 1.55E TimberStrand® LSL @ 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)	806 @ 2 1/2"	4922 (2.25")	Passed (16%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	721 @ 1' 3 3/8"	8590	Passed (8%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	4422 @ 11' 3 1/2"	16591	Passed (27%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.351 @ 11' 3 1/2"	0.554	Passed (L/759)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.473 @ 11' 3 1/2"	1.108	Passed (L/562)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	46	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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 4% increase in the moment capacity has been added to account for repetitive member usage.
- 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	Total	
1 - Stud wall - DF	3.50"	2.25"	1.50"	211	602	813	1 1/4" Rim Board
2 - Beam - DF	3.50"	2.25"	1.50"	211	602	813	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)	22' 5" o/c	
Bottom Edge (Lu)	22' 5" o/c	

•Maximum allowable bracing intervals based on applied load.

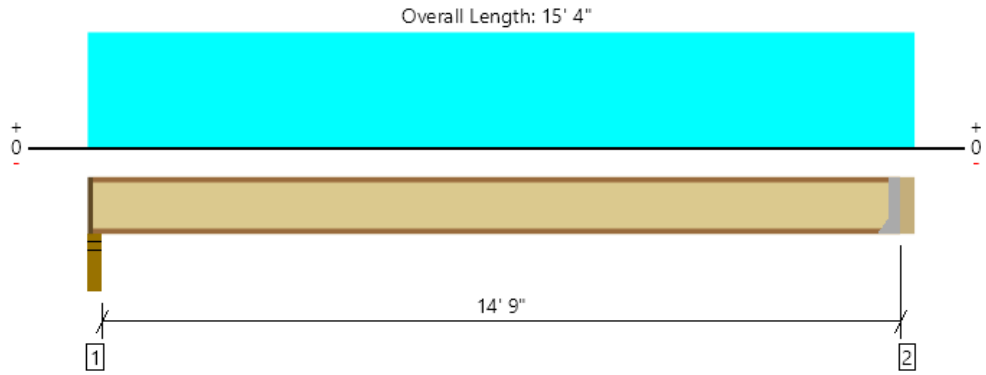
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 22' 7"	16"	14.0	40.0	Typical Floor

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

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

Permit, UFJ3 - Upper Floor Joist
1 piece(s) 11 7/8" TJI® 360 @ 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)	534 @ 15' 1/2"	1080 (1.75")	Passed (49%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	534 @ 15' 1/2"	1705	Passed (31%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1980 @ 7' 7 1/2"	6180	Passed (32%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.143 @ 7' 7 1/2"	0.371	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.193 @ 7' 7 1/2"	0.742	Passed (L/920)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	53	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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	Total	
1 - Stud wall - DF	3.50"	2.25"	1.75"	142	407	549	1 1/4" Rim Board
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	144	411	555	See note [1]

- Rim Board is assumed to carry all loads applied directly above it, bypassing 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)	6' 9" o/c	
Bottom Edge (Lu)	14' 11" 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 - Face Mount Hanger	IUS2.37/11.88	2.00"	N/A	10-10dx1.5	2-Strong-Grip	

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

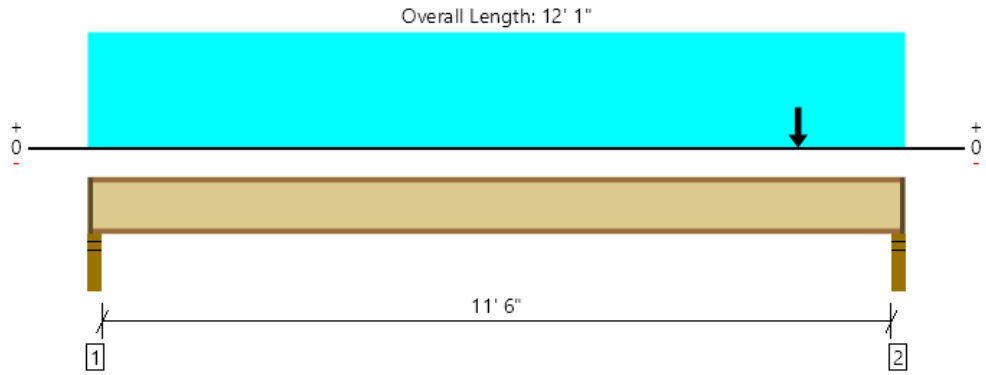
Vertical Load	Location (Side)	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 15' 4"	16"	14.0	40.0	Typical Floor

Member Notes
UFJ3 - Upper Floor Joist

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

Weyerhaeuser

Permit, MFJ1 - Main Floor Joist (Special case @ mudroom)
1 piece(s) 11 7/8" TJI® 360 @ 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)	1160 @ 11' 10 1/2"	1202 (2.25")	Passed (97%)	1.00	1.0 D + 1.0 L (All Spans)
Shear (lbs)	1146 @ 11' 9 1/2"	1705	Passed (67%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	1862 @ 7' 4 13/16"	6180	Passed (30%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.088 @ 6' 3 5/16"	0.292	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.127 @ 6' 3 3/4"	0.583	Passed (L/999+)	--	1.0 D + 1.0 L (All Spans)
TJ-Pro™ Rating	64	Any	Passed	--	--

System : Floor
Member Type : Joist
Building Use : Residential
Building Code : IBC 2015
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	Total	
1 - Stud wall - SPF	3.50"	2.25"	1.75"	152	381	533	1 1/4" Rim Board
2 - Stud wall - SPF	3.50"	2.25"	2.08"	404	763	1167	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)	7' o/c	
Bottom Edge (Lu)	11' 11" o/c	

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

Vertical Loads	Location	Spacing	Dead (0.90)	Floor Live (1.00)	Comments
1 - Uniform (PSF)	0 to 12' 1"	16"	14.0	40.0	Main Floor
2 - Point (lb)	10' 6"	N/A	330	500	Upper Floor

Member Notes

MFJ1 - Main Floor Joist (special case @ mudroom)

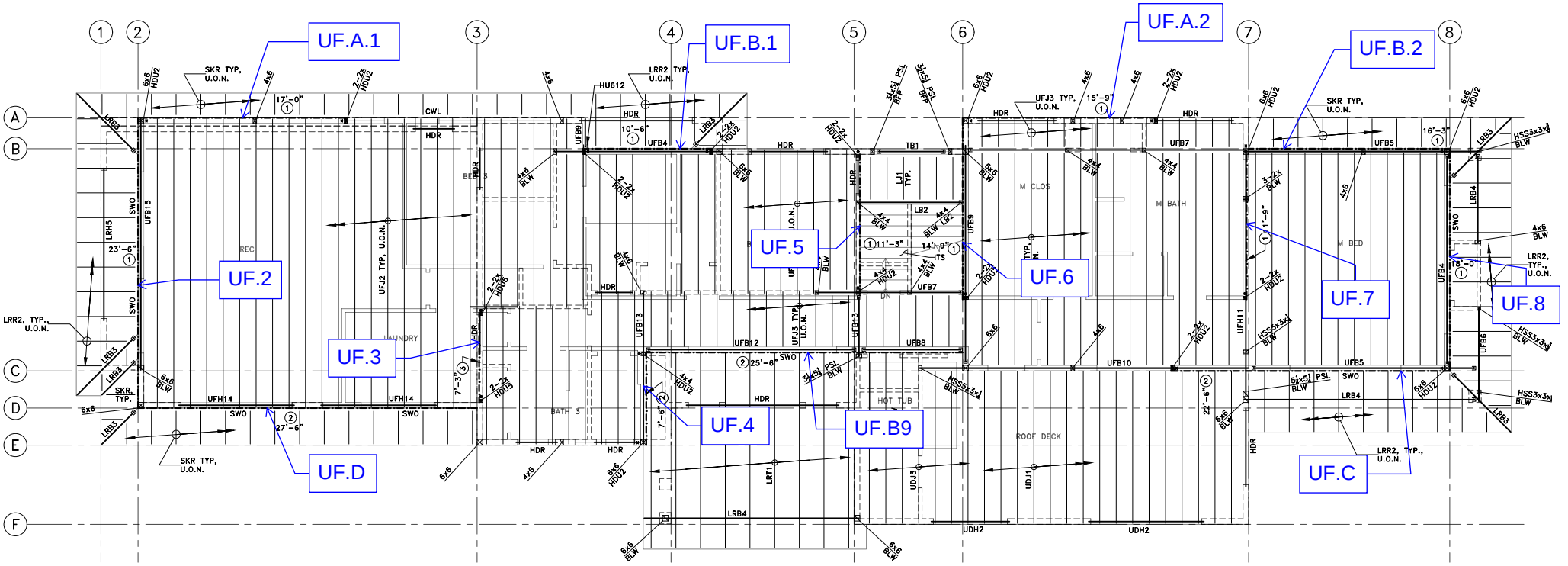
Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

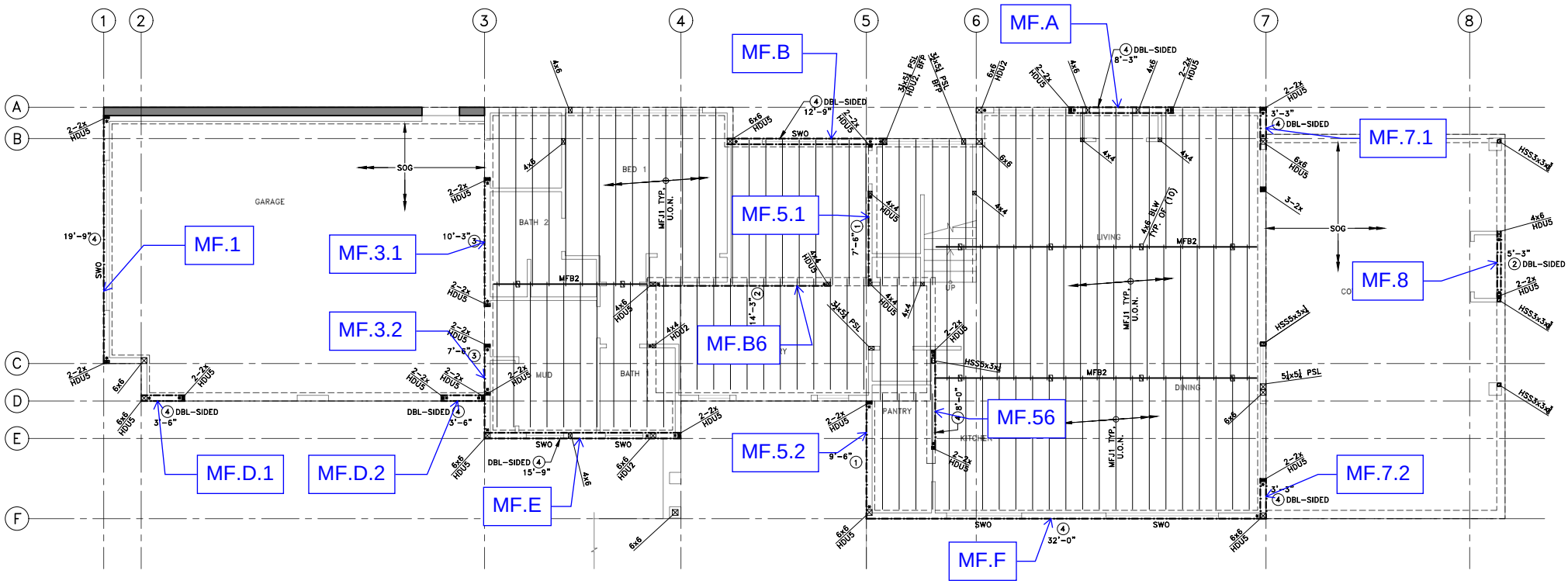
The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

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

Weyerhaeuser



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	6.4
Upper	4.3

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} = 0.44

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
MSTC28	1540
MSTC40	3080
MSTC52	4620

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 *(negative means no uplift)*
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.A.1	17	9	397	none	0	2554	2554	150	1	no	1	310
UF.B.1	10.5	9	245	none	0	1578	1578	150	1	no	1	310
UF.A.2	15.75	11	368	none	0	2366	2366	150	1	no	1	310
UF.B.2	16.25	11	380	none	0	2441	2441	150	1	no	1	310
UF.D*	27.5	9	431	none	0	2772	2772	269	1	no	1	310
UF.B9*	25.5	9	449	none	0	2888	2888	269	1	no	1	310
UF.C*	22.5	11	532	none	0	3422	3422	269	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.A.1	22988	340	1360	32122	-537		-537	NONE	#N/A
UF.B.1	14198	330	1320	14250	-5		-5	NONE	#N/A
UF.A.2	26030	315	1260	26194	-10		-10	NONE	#N/A
UF.B.2	26856	310	1240	27156	-18		-18	NONE	#N/A
UF.D*	24947	340	1360	73787	-1776		-1776	NONE	#N/A
UF.B9*	25989	300	1200	56970	-1215		-1215	NONE	#N/A
UF.C*	37637	290	1160	44240	-294		-294	NONE	#N/A

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.2*	23.5	9	410	none	0	2637	2637	305	1	no	1	310
UF.3	7.25	9	660	none	0	4245	4245	585	3	no	1	600
UF.4	7.5	9	415	none	0	2669	2669	356	2	no	1	460
UF.5	11.25	9	265	none	0	1704	1704	151	1	no	1	310
UF.6	14.75	11	375	none	0	2412	2412	164	1	no	1	310
UF.7	11.75	11	490	none	0	3151	3151	268	1	no	1	310
UF.8*	18	11	190	none	0	1222	1222	303	1	no	1	310

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.2*	23732	200	800	32912	-391		-391	NONE	#N/A
UF.3	38202	60	240	1475	5066		5066	HDU5	5645
UF.4	24021	90	360	2326	2893		2893	HDU2	3075
UF.5	15339	60	240	2889	1107		1107	HDU2	3075
UF.6	26530	80	320	5968	1394		1394	HDU2	3075
UF.7	34665	80	320	4127	2599		2599	HDU2	3075
UF.8*	13442	200	800	20807	-409		-409	NONE	#N/A

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.A	8.25	10	325	UF.A.2, B.1/2	3512	1386	4898	594	3	no	1	600
MF.B*	12.75	10	266	UF.A.2, B.1/2	2873	1134	4008	629	4	no	1	770
MF.B6	14.25	10	1388	UF.B9,C	4086	5930	10015	703	4	no	1	770
MF.D.1/2	3.5	10	220	UF.D	1386	940	2326	665	DBL 3	yes	0.70	840
MF.E*	15.75	10	270	UF.B9	689	1151	1840	330	2	no	1	460
MF.F*	32	10	468	UF.B9,C	1534	1999	3534	247	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
MF.A	48983	120	480	3576	5504		5504	HDU5	5645
MF.B*	40077	180	720	10586	2313		2313	HDU2	3075
MF.B6	100154	170	680	11982	6188		6188	HDU8	7870
MF.D.1/2	23258	310	1240	2774	5853		5853	HDU8	7870
MF.E*	18404	180	720	14968	218		218	HDU2	3075
MF.F*	35336	210	840	59754	-763		-763	NONE	#N/A

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.1*	19.75	10	459	UF.2	5023	1959	6981	901	DBL 2	no	1	920
MF.3.1	10.25	10	549	UF.2-4	4902	2344	7246	707	4	no	1	770
MF.3.2	7.5	10	401	UF.2-4	3587	1715	5302	707	4	no	1	770
MF.5.1	7.5	10	259	UF.4,5	1303	1107	2410	321	2	no	1	460
MF.5.2	9	10	311	UF.4,5	1564	1328	2892	321	2	no	1	460
MF.56	8	10	510	UF.6	2144	2179	4323	540	3	no	1	600
MF.7.1/2	3.25	8	320	UF.6-8	1774	1367	3141	966	DBL 3	yes	0.81	975
MF.8	5.5	10	330	UF.8	1093	1410	2503	455	2	no	1	460

Note: Seismic load from full-height concrete wall added to MF.1, 3.1 & 3.2 Vabv

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.1*	69814	100	400	12183	2918		2918	HDU2	3075
MF.3.1	72456	70	280	2911	6785		6785	HDU8	7870
MF.3.2	53016	70	280	1809	6828		6828	HDU8	7870
MF.5.1	24099	70	280	1809	2972	1107	4079	HDU4	4565
MF.5.2	28918	70	280	2381	2949		2949	HDU4	4565
MF.56	43225	70	280	1992	5154		5154	HDU5	5645
MF.7.1/2	25128	100	400	813	7482		7482	HDU8	7870
MF.8	25031	100	400	1651	4251		4251	HDU4	4565

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

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

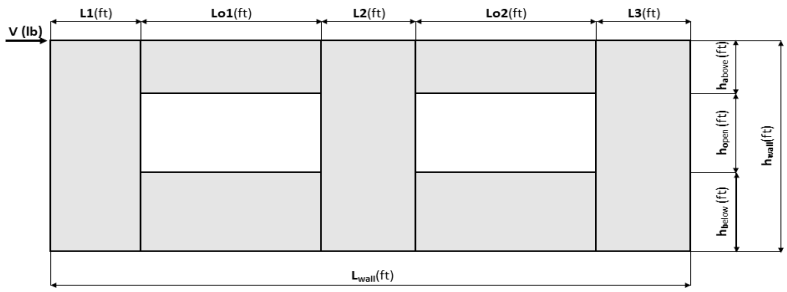
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.D



Shear Wall Calculation Variables

		Opening 1		Opening 2	
V	2772 lbf				
L1	3.50 ft	ha1	1.50 ft	ha2	1.50 ft
L2	5.50 ft	ho1	5.00 ft	ho2	5.00 ft
L3	3.50 ft	hb1	3.00 ft	hb2	3.00 ft
h_{wall}	9.50 ft	Lo1	9.00 ft	Lo2	6.00 ft
L_{wall}	27.50 ft				

Adj. Factor Method =	2bs/h
Wall Pier Aspect Ratio	Adj. Factor
P1=ho1/L1= 1.43	N/A
P2=ho2/L2= 0.91	N/A
P3=ho3/L3= 1.43	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 958 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 213 plf

Second opening: va2 = vb2 = H/(ha2+hb2) = 213 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1915 lbf

Second opening: O2 = va2 x (Lo2) = 1277 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 745 lbf

F2 = O1(L2)/(L1+L2) = 1170 lbf

F3 = O2(L2)/(L2+L3) = 780 lbf

F4 = O2(L3)/(L2+L3) = 497 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.50 ft

T2 = (L2*Lo1)/(L1+L2) = 5.50 ft

T3 = (L2*Lo2)/(L2+L3) = 3.67 ft

T4 = (L3*Lo2)/(L2+L3) = 2.33 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 202 plf

V2 = (V/L)(T2+L2+T3)/L2 = 269 plf

V3 = (V/L)(T4+L3)/L3 = 168 plf

Check V1*L1+V2*L2+V3*L3=V? 2772 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 706 lbf

R2 = V2*L2 = 1478 lbf

R3 = V3*L3 = 588 lbf

8. Difference corner force + resistance

R1-F1 = -39 lbf

R2-F2-F3 = -472 lbf

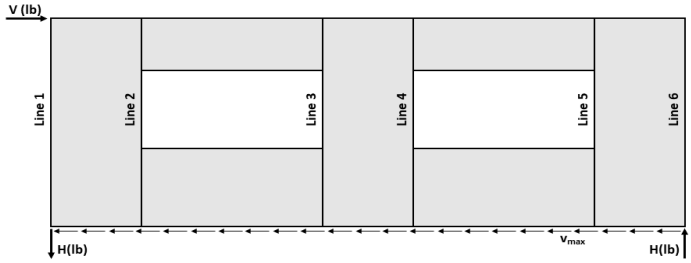
R3-F4 = 91 lbf

9. Unit shear in corner zones

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

vc2 = (R2-F2-F3)/L2 = -86 plf

vc3 = (R3-F4)/L3 = 26 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-50	1008	958 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	958	-50	1008	0
Line 3: vc2(ha1+hb1)+V2(ho1)-va1(ha1+hb1)=0?	-386	1344	958	0
Line 4: va2(ha2+hb2)-V2(ho2)-vc2(ha2+hb2)=0?	958	1344	-386	0
Line 5: va2(ha2+hb2)-vc3(ha2+hb2)-V3(ho2)=0?	958	118	840	0
Line 6: vc3(ha2+hb2)+ V3(ho2) = H?		118	840	958 lbf

Design Summary*

Req. Sheathing Capacity	269 plf
Req. Strap Force	1170 lbf
Req. HD Force	958 lbf
Req. Shear Wall Anchorage Force	101 plf

*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

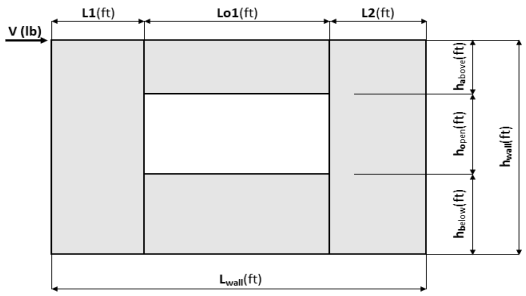
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.B9



Shear Wall Calculation Variables

V	2888 lbf	Opening 1	
L1	3.50 ft	ha1	1.50 ft
L2	10.00 ft	ho1	5.50 ft
h_{wall}	9.50 ft	hb1	2.50 ft
L_{wall}	25.50 ft	Lo1	12.00 ft

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	1.57	N/A
P2=ho2/L2=	0.55	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 1076 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 269 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3227 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 837 lbf

F2 = O1(L2)/(L1+L2) = 2391 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.11 ft

T2 = (L2*Lo1)/(L1+L2) = 8.89 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 214 plf

V2 = (V/L)(T2+L2)/L2 = 214 plf

Check V1*L1+V2*L2=V? 2888 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 749 lbf

R2 = V2*L2 = 2139 lbf

8. Difference corner force + resistance

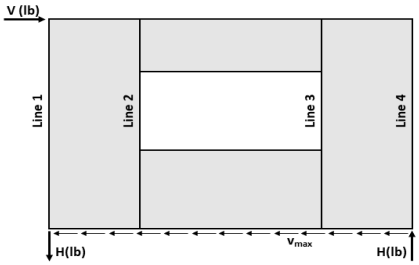
R1-F1 = -88 lbf

R2-F2 = -252 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -25 plf

vc2 = (R2-F2)/L2 = -25 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-101	1176	1076 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1076	-101	1176	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	1076	-101	1176	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		-101	1176	1076 lbf

Design Summary*

Req. Sheathing Capacity	269 plf
Req. Strap Force	2391 lbf
Req. HD Force (H)	1076 lbf
Req. Shear Wall Anchorage Force (v_{max})	113 plf

*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

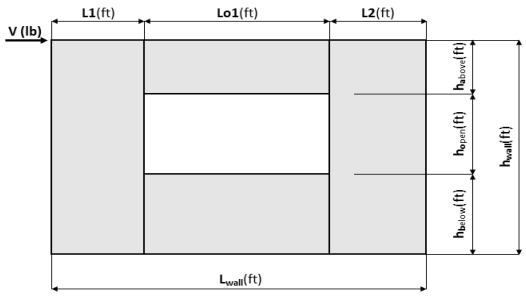
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.C



Shear Wall Calculation Variables

V	3422 lbf	Opening 1	
L1	9.50 ft	ha1	3.50 ft
L2	4.00 ft	ho1	5.00 ft
h_{wall}	11.50 ft	hb1	3.00 ft
L_{wall}	22.50 ft	Lo1	9.00 ft

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	0.53	N/A
P2=ho2/L2=	1.25	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1749 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 269 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2421 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1704 lbf

F2 = O1(L2)/(L1+L2) = 717 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 6.33 ft

T2 = (L2*Lo1)/(L1+L2) = 2.67 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 253 plf

V2 = (V/L)(T2+L2)/L2 = 253 plf

Check V1*L1+V2*L2=V? 3422 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 2408 lbf

R2 = V2*L2 = 1014 lbf

8. Difference corner force + resistance

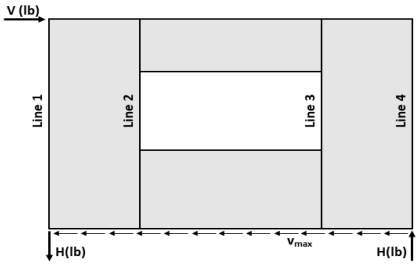
R1-F1 = 704 lbf

R2-F2 = 296 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 74 plf

vc2 = (R2-F2)/L2 = 74 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		482	1267	1749 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1749	482	1267	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	1749	482	1267	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		482	1267	1749 lbf

Design Summary*

Req. Sheathing Capacity	269 plf
Req. Strap Force	1704 lbf
Req. HD Force (H)	1749 lbf
Req. Shear Wall Anchorage Force (v_{max})	152 plf

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

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

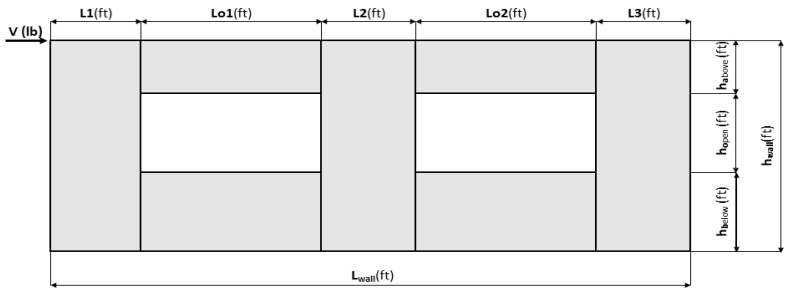
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.2



Shear Wall Calculation Variables

			Opening 1		Opening 2
V	2637 lbf				
L1	4.50 ft	ha1	1.50 ft	ha2	1.50 ft
L2	2.50 ft	ho1	5.00 ft	ho2	5.00 ft
L3	4.50 ft	hb1	3.00 ft	hb2	3.00 ft
h_{wall}	9.50 ft	Lo1	6.00 ft	Lo2	6.00 ft
L_{wall}	23.50 ft				

Adj. Factor Method =	2bs/h
Wall Pier Aspect Ratio	Adj. Factor
P1=ho1/L1= 1.11	N/A
P2=ho2/L2= 2.00	N/A
P3=ho3/L3= 1.11	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1066 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 237 plf

Second opening: va2 = vb2 = H/(ha2+hb2) = 237 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1421 lbf

Second opening: O2 = va2 x (Lo2) = 1421 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 914 lbf

F2 = O1(L2)/(L1+L2) = 508 lbf

F3 = O2(L2)/(L2+L3) = 508 lbf

F4 = O2(L3)/(L2+L3) = 914 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.86 ft

T2 = (L2*Lo1)/(L1+L2) = 2.14 ft

T3 = (L2*Lo2)/(L2+L3) = 2.14 ft

T4 = (L3*Lo2)/(L2+L3) = 3.86 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 208 plf

V2 = (V/L)(T2+L2+T3)/L2 = 305 plf

V3 = (V/L)(T4+L3)/L3 = 208 plf

Check V1*L1+V2*L2+V3*L3=V? 2637 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 938 lbf

R2 = V2*L2 = 761 lbf

R3 = V3*L3 = 938 lbf

8. Difference corner force + resistance

R1-F1 = 24 lbf

R2-F2-F3 = -254 lbf

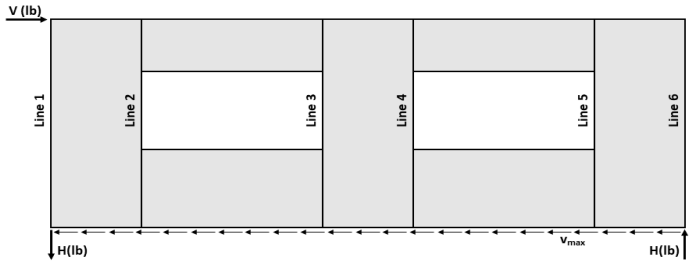
R3-F4 = 24 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 5 plf

vc2 = (R2-F2-F3)/L2 = -102 plf

vc3 = (R3-F4)/L3 = 5 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		24	1042	1066 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1066	24	1042	0
Line 3: vc2(ha1+hb1)+V2(ho1)-va1(ha1+hb1)=0?	-457	1523	1066	0
Line 4: va2(ha2+hb2)-V2(ho2)-vc2(ha2+hb2)=0?	1066	1523	-457	0
Line 5: va2(ha2+hb2)-vc3(ha2+hb2)-V3(ho2)=0?	1066	24	1042	0
Line 6: vc3(ha2+hb2)+ V3(ho2) = H?		24	1042	1066 lbf

Design Summary*

Req. Sheathing Capacity	305 plf
Req. Strap Force	914 lbf
Req. HD Force	1066 lbf
Req. Shear Wall Anchorage Force	112 plf

*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

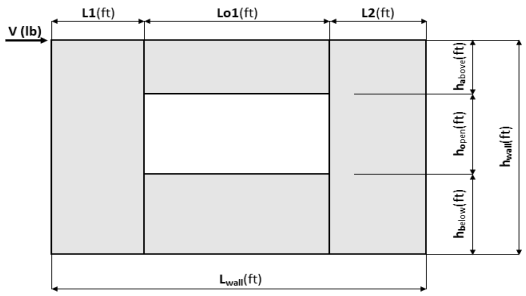
Code: **Date:**

Designer:

Client:

Project:

Wall Line: UF.8



Shear Wall Calculation Variables

V	1222 lbf	Opening 1	
L1	2.75 ft	ha1	2.00 ft
L2	2.75 ft	ho1	7.50 ft
h_{wall}	11.50 ft	hb1	2.00 ft
L_{wall}	18.00 ft	Lo1	12.50 ft

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	2.73	0.733
P2=ho2/L2=	2.73	0.733

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 781 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 195 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 2440 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 1220 lbf

F2 = O1(L2)/(L1+L2) = 1220 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 6.25 ft

T2 = (L2*Lo1)/(L1+L2) = 6.25 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 222 plf

V2 = (V/L)(T2+L2)/L2 = 222 plf

Check V1*L1+V2*L2=V? 1222 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 611 lbf

R2 = V2*L2 = 611 lbf

8. Difference corner force + resistance

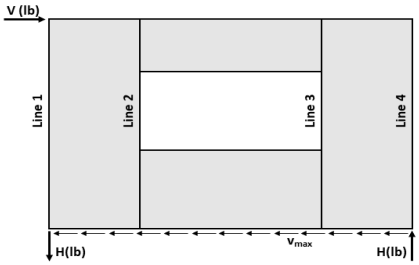
R1-F1 = -609 lbf

R2-F2 = -609 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = -221 plf

vc2 = (R2-F2)/L2 = -221 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		-886	1666	781 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	781	-886	1666	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	781	-886	1666	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		-886	1666	781 lbf

Design Summary*

Req. Sheathing Capacity	303 plf
Req. Strap Force	1220 lbf
Req. HD Force (H)	781 lbf
Req. Shear Wall Anchorage Force (v_{max})	68 plf

Req. Sheathing Capacity has been adjusted per the Aspect Ratio Factor in SDPWS 4.3.4.2

*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

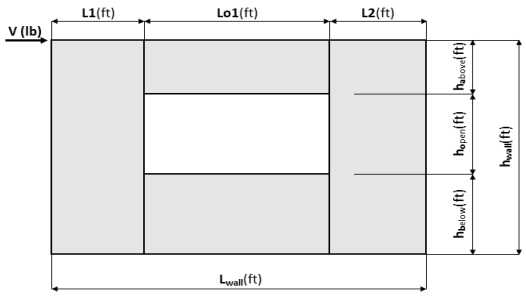
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.B



Shear Wall Calculation Variables

V	4008 lbf	Opening 1	
L1	4.00 ft	ha1	2.00 ft
L2	2.75 ft	ho1	5.00 ft
h_{wall}	10.00 ft	hb1	3.00 ft
L_{wall}	12.75 ft	Lo1	6.00 ft

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	1.25	N/A
P2=ho2/L2=	1.82	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 3143 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 629 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 3772 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 2235 lbf

F2 = O1(L2)/(L1+L2) = 1537 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.56 ft

T2 = (L2*Lo1)/(L1+L2) = 2.44 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 594 plf

V2 = (V/L)(T2+L2)/L2 = 594 plf

Check V1*L1+V2*L2=V? 4008 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 2375 lbf

R2 = V2*L2 = 1633 lbf

8. Difference corner force + resistance

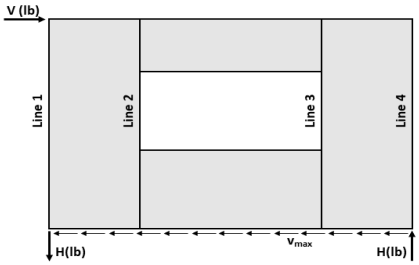
R1-F1 = 140 lbf

R2-F2 = 96 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 35 plf

vc2 = (R2-F2)/L2 = 35 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		175	2969	3143 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	3143	175	2969	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	3143	175	2969	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		175	2969	3143 lbf

Design Summary*

Req. Sheathing Capacity	629 plf
Req. Strap Force	2235 lbf
Req. HD Force (H)	3143 lbf
Req. Shear Wall Anchorage Force (v_{max})	314 plf

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

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

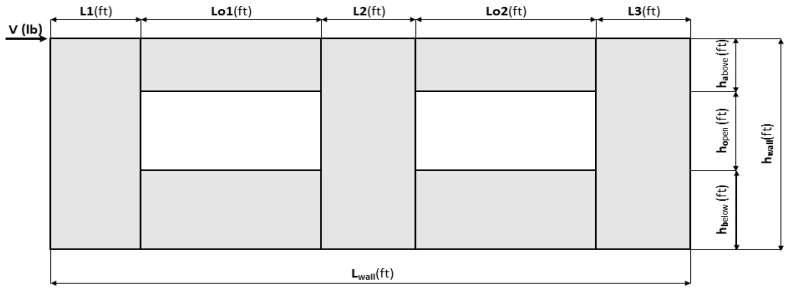
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.E



Shear Wall Calculation Variables

			Opening 1		Opening 2
V	2637 lbf				
L1	3.25 ft	ha1	2.00 ft	ha2	2.00 ft
L2	3.25 ft	ho1	4.50 ft	ho2	4.50 ft
L3	3.00 ft	hb1	3.50 ft	hb2	3.50 ft
h_{wall}	10.00 ft	Lo1	3.00 ft	Lo2	3.00 ft
L_{wall}	15.50 ft				

Adj. Factor Method =	2bs/h
Wall Pier Aspect Ratio	Adj. Factor
P1=ho1/L1= 1.38	N/A
P2=ho2/L2= 1.38	N/A
P3=ho3/L3= 1.50	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1701 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 309 plf

Second opening: va2 = vb2 = H/(ha2+hb2) = 309 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 928 lbf

Second opening: O2 = va2 x (Lo2) = 928 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 464 lbf

F2 = O1(L2)/(L1+L2) = 464 lbf

F3 = O2(L2)/(L2+L3) = 483 lbf

F4 = O2(L3)/(L2+L3) = 445 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 1.50 ft

T2 = (L2*Lo1)/(L1+L2) = 1.50 ft

T3 = (L2*Lo2)/(L2+L3) = 1.56 ft

T4 = (L3*Lo2)/(L2+L3) = 1.44 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 249 plf

V2 = (V/L)(T2+L2+T3)/L2 = 330 plf

V3 = (V/L)(T4+L3)/L3 = 252 plf

Check V1*L1+V2*L2+V3*L3=V? 2637 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 808 lbf

R2 = V2*L2 = 1073 lbf

R3 = V3*L3 = 755 lbf

8. Difference corner force + resistance

R1-F1 = 344 lbf

R2-F2-F3 = 127 lbf

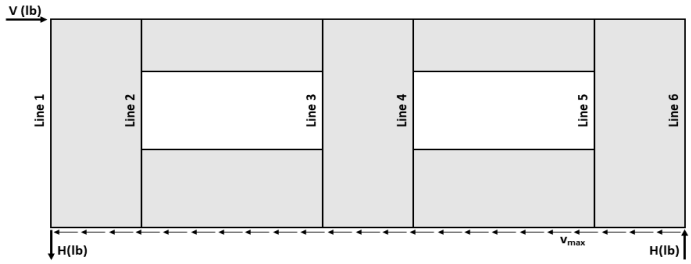
R3-F4 = 310 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 106 plf

vc2 = (R2-F2-F3)/L2 = 39 plf

vc3 = (R3-F4)/L3 = 103 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		582	1119	1701 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1701	582	1119	0
Line 3: vc2(ha1+hb1)+V2(ho1)-va1(ha1+hb1)=0?	215	1486	1701	0
Line 4: va2(ha2+hb2)-V2(ho2)-vc2(ha2+hb2)=0?	1701	1486	215	0
Line 5: va2(ha2+hb2)-vc3(ha2+hb2)-V3(ho2)=0?	1701	568	1133	0
Line 6: vc3(ha2+hb2)+ V3(ho2) = H?		568	1133	1701 lbf

Design Summary*

Req. Sheathing Capacity	330 plf
Req. Strap Force	483 lbf
Req. HD Force	1701 lbf
Req. Shear Wall Anchorage Force	170 plf

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

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

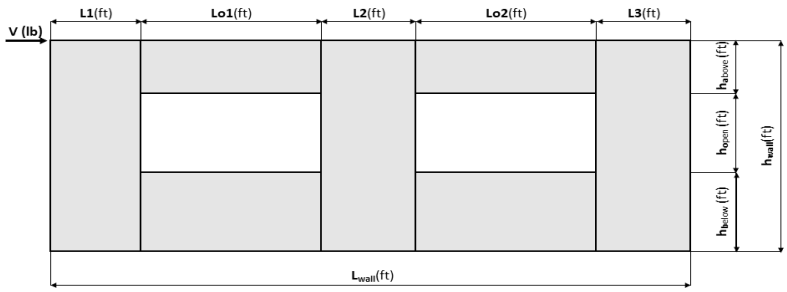
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.F



Shear Wall Calculation Variables

			Opening 1		Opening 2
V	3534 lbf				
L1	6.50 ft	ha1	2.00 ft	ha2	2.00 ft
L2	6.75 ft	ho1	5.00 ft	ho2	5.00 ft
L3	3.75 ft	hb1	3.00 ft	hb2	3.00 ft
h_{wall}	10.00 ft	Lo1	6.00 ft	Lo2	6.00 ft
L_{wall}	29.00 ft				

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	0.77	N/A
P2=ho2/L2=	0.74	N/A
P3=ho3/L3=	1.33	N/A

1. Hold-down forces: H = Vh_{wall}/L_{wall} 1218 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 244 plf

Second opening: va2 = vb2 = H/(ha2+hb2) = 244 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 1462 lbf

Second opening: O2 = va2 x (Lo2) = 1462 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 717 lbf

F2 = O1(L2)/(L1+L2) = 745 lbf

F3 = O2(L2)/(L2+L3) = 940 lbf

F4 = O2(L3)/(L2+L3) = 522 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 2.94 ft

T2 = (L2*Lo1)/(L1+L2) = 3.06 ft

T3 = (L2*Lo2)/(L2+L3) = 3.86 ft

T4 = (L3*Lo2)/(L2+L3) = 2.14 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 177 plf

V2 = (V/L)(T2+L2+T3)/L2 = 247 plf

V3 = (V/L)(T4+L3)/L3 = 191 plf

Check V1*L1+V2*L2+V3*L3=V? 3534 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 1151 lbf

R2 = V2*L2 = 1665 lbf

R3 = V3*L3 = 718 lbf

8. Difference corner force + resistance

R1-F1 = 433 lbf

R2-F2-F3 = -20 lbf

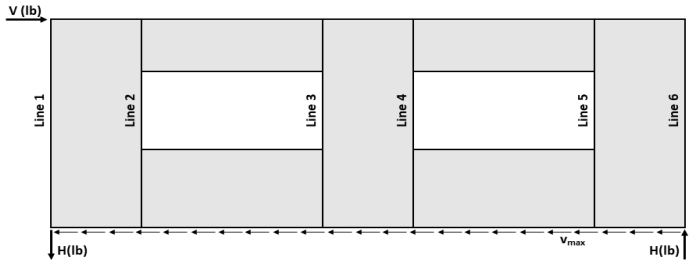
R3-F4 = 196 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 67 plf

vc2 = (R2-F2-F3)/L2 = -3 plf

vc3 = (R3-F4)/L3 = 52 plf



Check Summary of Shear Values for Two Openings

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		333	885	1218 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	1218	333	885	0
Line 3: vc2(ha1+hb1)+V2(ho1)-va1(ha1+hb1)=0?	-15	1233	1218	0
Line 4: va2(ha2+hb2)-V2(ho2)-vc2(ha2+hb2)=0?	1218	1233	-15	0
Line 5: va2(ha2+hb2)-vc3(ha2+hb2)-V3(ho2)=0?	1218	261	957	0
Line 6: vc3(ha2+hb2)+ V3(ho2) = H?		261	957	1218 lbf

Design Summary*

Req. Sheathing Capacity	247 plf
Req. Strap Force	940 lbf
Req. HD Force	1218 lbf
Req. Shear Wall Anchorage Force	122 plf

*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

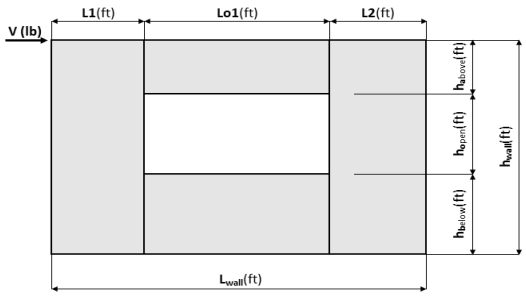
Code: **Date:**

Designer:

Client:

Project:

Wall Line: MF.1



Shear Wall Calculation Variables

V	6981 lbf	Opening 1	
L1	4.00 ft	ha1	2.50 ft
L2	3.75 ft	ho1	3.50 ft
h_{wall}	10.00 ft	hb1	4.00 ft
L_{wall}	20.75 ft	Lo1	13.00 ft

Adj. Factor Method =		2bs/h
Wall Pier Aspect Ratio		Adj. Factor
P1=ho1/L1=	0.88	N/A
P2=ho2/L2=	0.93	N/A

1. Hold-down forces: $H = Vh_{wall}/L_{wall}$ 3365 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 518 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 6729 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 3473 lbf

F2 = O1(L2)/(L1+L2) = 3256 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 6.71 ft

T2 = (L2*Lo1)/(L1+L2) = 6.29 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 901 plf

V2 = (V/L)(T2+L2)/L2 = 901 plf

Check V1*L1+V2*L2=V? 6981 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 3603 lbf

R2 = V2*L2 = 3378 lbf

8. Difference corner force + resistance

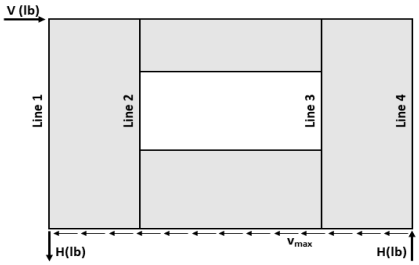
R1-F1 = 130 lbf

R2-F2 = 122 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 33 plf

vc2 = (R2-F2)/L2 = 33 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		212	3153	3365 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	3365	212	3153	0
Line 3: va1(ha1+hb1)-vc2(ha1+hb1)-V1(ho1)=0?	3365	212	3153	0
Line 4: vc2(ha1+hb1)+V2(ho1)=H?		212	3153	3365 lbf

Design Summary*

Req. Sheathing Capacity	901 plf
Req. Strap Force	3473 lbf
Req. HD Force (H)	3365 lbf
Req. Shear Wall Anchorage Force (v_{max})	336 plf

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