PRCNC20231287

PCS Structural Solutions

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www.pcs-structural.com

STRUCTURAL CALCULATIONS

FOR

TACO TIME
EAST MAIN STREET
PUYALLUP, WASHINGTON

PREPARED BY
PCS STRUCTURAL *SOLUTIONS*

LUKE A. HEATH
STATE OF WASHINGTON
42292
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

City of Puyallup
Building
REVIEWED FOR COMPLIANCE
BSnowden
11/07/2023
11:31:12 AM

JULY 19, 2023
23-514

Calculations required to be provided by the Permittee on site for all Inspections

DESIGN CRITERIA

PCS Structural Solutions

Project:	Taco Time NW	Job Number:	23-514
Sheet:	of	Name:	JMB
Originating Office:	Tacoma	Date:	2023-06-16

DESIGN CRITERIA CHECKLIST

CODE: IBC 2018, ASCE 7-16 LOCATION: PUYALLUP, WA

RISK CATEGORY: II (Per ASCE 7-16 Table 1.5-1 & IBC Table 1604.5)

VERTICAL DESIGN CRITERIA

	DEAD	LIVE	PARTITION	CONCENTRATED
ROOF:	20 PSF	25 PSF		(INCLUDES 5 PSF FUTURE PV)

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) =	100 MPH	(Per ASCE 7-16 Sec. 26.5.1, Fig. 26.5-1A; 1B; 1C & 1D, or as required by Bld'g Dept.)
EXPOSURE CATEGORY:	B	(Per ACSE 7-16 Section 26.7.3)
DIRECTIONALITY FACTOR (K_d):	0.85	(Per ASCE 7-16 Table 26.6-1)
GUST EFFECT FACTOR (G):	0.85	(Per ASCE 7-16 Section 26.11)
TOPOGRAPHIC FEATURE:	None	(See ASCE 7-16 Figure 26.8-1)
HILL HEIGHT (H):	0 FT	(See ASCE 7-16 Figure 26.8-1)
UPWIND DISTANCE TO HALF HILL (Lh):	0 FT	(See ASCE 7-16 Figure 26.8-1)
DISTANCE FROM CREST TO SITE (x):	0 FT	UPWIND (See ASCE 7-16 Figure 26.8-1)
MEAN ROOF HEIGHT:	20 FT	(See ASCE 7-16 Section 26.2 - Definitions)
ELEVATION:	0 FT	(See ASCE 7-16 Section 26.9)
ENCLOSURE CLASSIFICATION:	Enclosed	(See ASCE 7-16 Secion 26.2 & Table 26.13-1)
ROOF TYPE:	Monoslope	(See ASCE 7-16 Figure 27.3-1)
ROOF SLOPE (__:12):	3.00:12	(Enter vertical rise in 12 horizontal units) θ (degrees): 14.04

SEISMIC DESIGN CRITERIA

SITE CLASS:	D	(Per IBC Section 1613.2.2, Assumed as "D" or per Geotech.)
IMPORTANCE FACTOR (I_E):	1	(Per ASCE 7-16 Table 1.5-2)
STRUCTURAL SYSTEM (R):	6.5	(Per ASCE 7-16 Table 12.2-1)
OVERSTRENGTH FACTOR (Ω_o):	3.0	(Per ASCE 7-16 Table 12.2-1)

INFORMATION BELOW FROM APPLIED TECHNOLOGY COUNCIL (ATC) "HAZARDS BY LOCATION"

LATITUDE:	47.192	S_S =	1.266	F_a =	1.200
LONGITUDE:	-122.279	S_1 =	0.436	F_v =	1.864

DEFLECTION CRITERIA

FLOOR (LIVE):	L/ 480	ROOF (LIVE):	L/ 360
FLOOR (TOTAL):	L/ 360	ROOF (TOTAL):	L/ 240
WALLS:	L/ 360	SPECIAL:	L/

SOIL DESIGN CRITERIA

REPORT: NO

ASSUMED VALUES

BEARING:	1500 PSF
ACTIVE:	35 PCF
PASSIVE:	250 PCF
COEFFICIENT OF FRICTION:	0.35

MINIMUM FOOTING DIMENSIONS:

CONTINUOUS:	1'-4"
SPREAD:	1'-6"
FROST DEPTH:	1'-6"

PILE TYPE:	NONE		
VERTICAL CAPACITY :	N/A	LATERAL CAPACITY:	N/A
UPLIFT CAPACITY:	N/A	SIZE:	N/A

MATERIALS

CONCRETE

Footings/Piles:	3000 PSI	Columns:	4000 PSI
Slabs/Walls:	4000 PSI	Beams:	4000 PSI
-	-	-	-

REINFORCING

Steel Grade =	60	f_y =	60 KSI

STRUCTURAL STEEL

W-Flange Beams	ASTM A992	f_y =	50 KSI
Shapes & Plates	ASTM A36	f_y =	36 KSI
Pipes	ASTM A53, Grade B	f_y =	35 KSI
HSS Rect.	ASTM A500, Grade C	f_y =	50 KSI
HSS Round	ASTM A500, Grade C	f_y =	46 KSI

MASONRY

ASTM C90	f'_m =	1900 PSI	SOLID GROUTED

GLULAM BEAMS

Simple Spans		Cantilevers
24F-V4	Grade =	24F-V8
1.80E+06 PSI	E =	1.80E+06 PSI
2400 PSI	$F_{b\,(BOTTOM)}$ =	2400 PSI
1850 PSI	$F_{b\,(TOP)}$ =	2400 PSI
240 PSI	F_v =	240 PSI

SCL PRODUCTS

	2x SCL	1¾" SCL	3½, 5¼ SCL
E =	1.30E+06 PSI	1.80E+06 PSI	2.00E+06 PSI
F_b =	1700 PSI	2600 PSI	2900 PSI
F_v =	285 PSI	285 PSI	285 PSI
F_c =	1400 PSI	2400 PSI	2600 PSI

FRAMING LUMBER

Joists & Studs	2x DF #2	2x HF #1	-
E =	1.60E+06 PSI	1.50E+06 PSI	-
F_b =	900 PSI	975 PSI	-
F_v =	180 PSI	150 PSI	-
F_c =	1350 PSI	1350 PSI	-
Beams & Headers	4x DF #2	4x HF #1	6x DF #1
E =	1.60E+06 PSI	1.50E+06 PSI	1.60E+06 PSI
F_b =	900 PSI	975 PSI	1350 PSI
F_v =	180 PSI	150 PSI	170 PSI
Posts & Timbers	6x DF #1	-	-
E =	1.60E+06 PSI	-	-
F_c =	1000 PSI	-	-

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DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	100 MPH	MEAN ROOF HEIGHT:	20 FT
RISK CATEGORY:	II	GROUND ELEVATION FACTOR (Ke):	1.00
EXPOSURE CATEGORY:	B	ENCLOSURE CLASSIFICATION:	Enclosed
DIRECTIONALITY FACTOR (K_d):	0.85	ROOF TYPE:	Monoslope
GUST EFFECT FACTOR (G):	0.85	ROOF SLOPE (__:12):	3.0:12
		θ (degrees):	14.04

ROOF PRESSURES (Figure 27.3-1)

		External Pressures (q_h*(GC_p)):			Internal Pressures (±q_i*(GC_{pi}))
Wind Direction:	h/L:	Windward (Positive)	Windward (Negative)	Leeward	All Roofs
Normal to Ridge for θ ≥ 10°	≤0.25	-0.4	-6.2	-5.3	2.4
	0.50	-2.1	-8.5	-5.7	
	≥1.0	-2.1	-12.1	-7.1	

			External Pressures (q*(GC_p)):		Internal Pressures (±q_i*(GC_{pi}))
	h/L:	Horizontal Distance from Windward Edge	Positive Pressure	Negative Pressure	All Roofs
Normal to Ridge for θ < 10° and Parallel to Ridge for All θ	≤0.5	0 to h	-2.1	-10.3	2.4
		h to 2h		-5.7	
		>2h		-3.4	
	≥1.0	0 to h/2	-2.1	-14.9	
		>h/2		-8.0	

ASCE 7-16 CHAPTER 27: WIND LOADS ON BUILDINGS: MWFRS (DIRECTIONAL PROCEDURE)
PART 1: ENCLOSED AND PARTIALLY ENCLOSED BUILDINGS OF ALL HEIGHTS
HORIZONTAL WALL PRESSURES (Figure 27.3-1)

Windward External Pressures (q_z*(GC_p)):			Leeward & Sidewall External Pressures (q_h*(GC_p)):			Internal Pressures (±q_i*(GC_{pi}))
Height Above Ground Level, z	K_{zt}	Windward wall	L/B:	Leeward wall	Sidewall	All walls
15	1.00	8.4	0-1	-5.7	-8.0	2.4
20	1.00	9.2	2	-3.4		
25	1.00	9.8	≥4	-2.3		
30	1.00	10.4				
40	1.00	11.2				
50	1.00	12.0				
60	1.00	12.6				
70	1.00	13.2				
80	1.00	13.8				
90	1.00	14.2				
100	1.00	14.6				
120	1.00	15.4				
140	1.00	16.1				
160	1.00	16.7				
180	1.00	17.3				
200	1.00	17.8				
250	1.00	18.9				
300	1.00	20.0				
350	1.00	20.9				
400	1.00	21.8				
450	1.00	22.5				
500	1.00	23.1				

NOTES:

1) Minimum Design Wind Loads (Per ASCE 7-16 27.1.5): The wind load used for design of the MWFRS shall not be less than 16 PSF multiplied by the wall area of the building, and 8 PSF multiplied by the roof area of the building projected on a vertical plane normal to the assumed wind direction. Wall and roof loads shall be applied simultaneously.
2) q_i has conservatively been taken equal to q_h

K_{ht} = 1.00
q_h = 13.5 PSF

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DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	100 MPH	MEAN ROOF HEIGHT:	20 FT
RISK CATEGORY:	II	GROUND ELEVATION FACTOR (Ke):	1.00
EXPOSURE CATEGORY:	B	ENCLOSURE CLASSIFICATION:	Enclosed
DIRECTIONALITY FACTOR (K_d):	0.85	ROOF TYPE:	Monoslope
GUST EFFECT FACTOR (G):	0.85	ROOF SLOPE (__:12):	3.0:12
		θ (degrees):	14.04

ASCE 7-16 CHAPTER 30: WIND LOADS: COMPONENTS AND CLADDING
PART 1: LOW-RISE BUILDINGS (h≤60 ft)

ROOF SURFACES

Effective Wind Area	POSITIVE PRESSURES	NEGATIVE PRESSURES					
		ZONE					
	ALL ZONES	1	2	3	N/A	N/A	N/A
10 SF	16.0	-20.0	-24.0	-41.6	N/A	N/A	N/A
20 SF	16.0	-18.9	-22.7	-37.5	N/A	N/A	N/A
50 SF	16.0	-18.3	-19.7	-32.9	N/A	N/A	N/A
100 SF	16.0	-17.3	-18.6	-29.4	N/A	N/A	N/A

WALL SURFACES & ROOF OVERHANGS

Effective Wind Area	WALL ZONES				ROOF OVERHANG ZONES					
	POSITIVE PRESSURES		NEGATIVE PRESSURES		NEGATIVE PRESSURES					
	4	5	4	5	1	2	3	N/A	N/A	N/A
10 SF	16.0	16.0	-17.3	-21.3	-31.0	-35.1	-52.6	N/A	N/A	N/A
20 SF	16.0	16.0	-16.6	-19.9	-29.2	-33.0	-47.9	N/A	N/A	N/A
50 SF	16.0	16.0	-16.0	-18.0	-27.7	-29.1	-42.3	N/A	N/A	N/A
100 SF	16.0	16.0	-16.0	-16.6	-25.9	-27.3	-38.1	N/A	N/A	N/A
500 SF	16.0	16.0	-16.0	-16.0	-24.3	-25.6	-36.4	N/A	N/A	N/A

NOTES:

1) ASCE 7-16 30.2.2: Minimum Design Wind Loads: The design wind pressure for C&C of buildings shall not be less than a net pressure of 16 PSF acting in either direction normal to the surface.

2) q_i has conservatively been taken equal to q_h

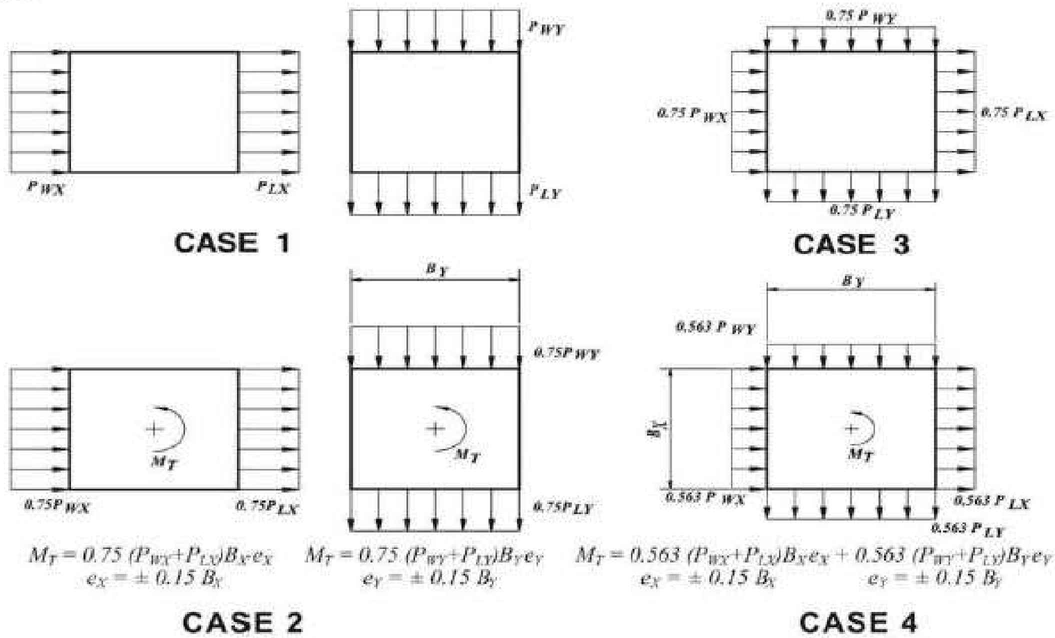
K_{ht} = 1.00

q_h = 13.5 PSF

DESIGN CRITERIA - WIND

FIGURE 27.3-8: Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases per ASCE 7-16

Diagrams



Notation

P_{WX}, P_{WY} = Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY} = Leeward face design pressure acting in the x, y principal axis, respectively.
$e(e_X, e_Y)$ = Eccentricity for the x, y principal axis of the structure, respectively.
M_T = Torsional moment per unit height acting about a vertical axis of the building.

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.
Case 2. Three-quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.
Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.
Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes

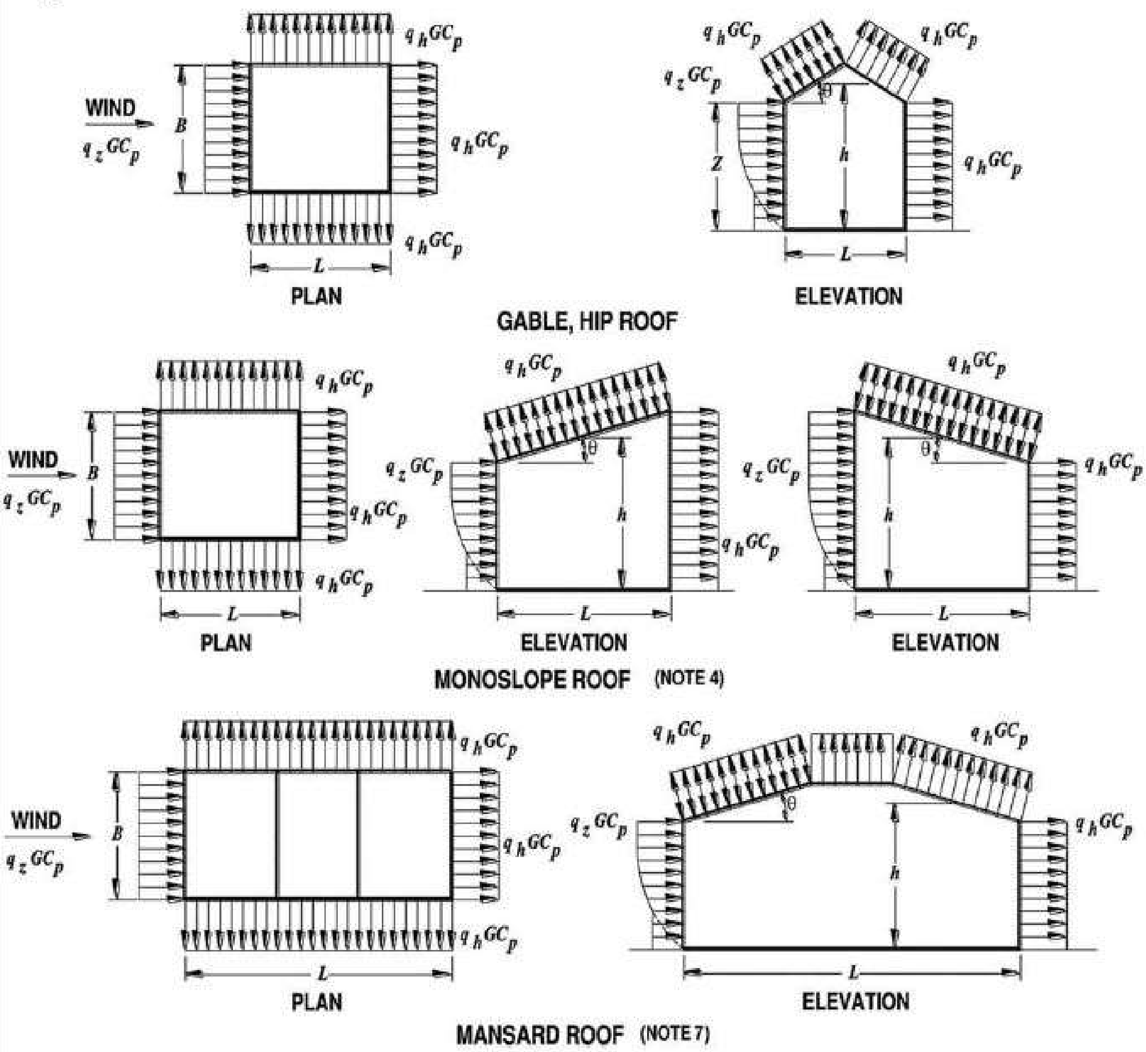
1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of Sections 27.3.1 and 27.3.2 as applicable for buildings of all heights.
2. Diagrams show plan views of buildings.

FIGURE 27.3-8 Main Wind Force Resisting System, Part 1 (All Heights): Design Wind Load Cases

DESIGN CRITERIA - WIND

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, Cp, for Enclosed and Partially Enclosed Buildings - Walls and Roofs per ASCE 7-16

Diagrams



Notation

B = Horizontal dimension of building, in ft (m), measured normal to wind direction.
L = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10$ degrees.
z = Height above ground, in ft (m).
G = Gust-effect factor.
q_z, q_h = Velocity pressure, in lb/ft^2 (N/m^2), evaluated at respective height.
θ = Angle of plane of roof from horizontal, in degrees.

FIGURE 27.3-1 Main Wind Force Resisting System, Part 1 (All Heights): External Pressure Coefficients, C_p, for Enclosed and Partially Enclosed Buildings—Walls and Roofs

Project: Taco Time NW | Job Number: 23-514

Sheet: of | Name: JMB

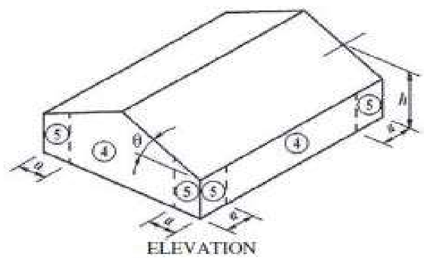
Originating Office: Tacoma | Date: 06-16-23

PCS Structural Solutions

DESIGN CRITERIA - WIND

FIGURE 30.3-1: Components and Cladding [h ≤ 60 ft]: External Pressure Coefficients, (GCp), for Enclosed and Partially Enclosed Buildings - Walls

Diagram



Notation

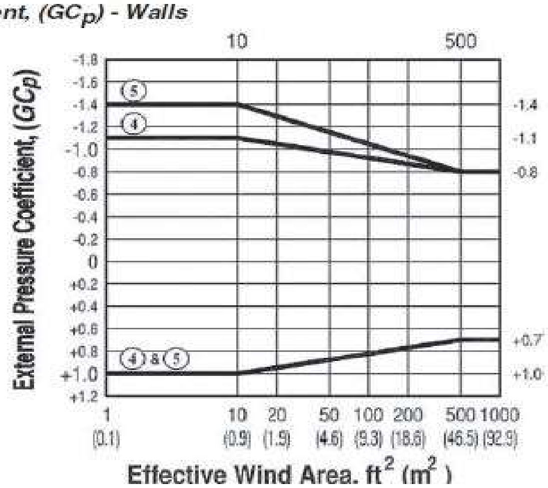
a = 10% of least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

Exception: For buildings with θ = 0° to 7° and a least horizontal dimension greater than 300 ft (90 m), dimension a shall be limited to a maximum of $0.8h$.

h = Mean roof height, in ft (m), except that eave height shall be used for θ ≤ 10°.

θ = Angle of plane of roof from horizontal, in degrees.

External Pressure Coefficient, (GC_p) - Walls



Notes

1. Vertical scale denotes (GCp) to be used with qh.
2. Horizontal scale denotes effective wind area, in ft² (m²).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of (GCp) for walls shall be reduced by 10% when θ ≤ 10°.

FIGURE 30.3-1 Components and Cladding [$h \leq 60$ ft ($h \leq 18.3$ m)]: External Pressure Coefficients, (GC_p), for Enclosed and Partially Enclosed Buildings—Walls

Project:	Taco Time NW	Job Number:	23-514
Sheet:	of	Name:	JMB
Originating Office:	Tacoma	Date:	06-16-23

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DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	II	LATITUDE:	47.192
SITE CLASS:	D	LONGITUDE:	-122.279
IMPORTANCE FACTOR (I_E):	1	S_S =	1.266
STRUCTURAL SYSTEM (R):	6.5	S_1 =	0.436
OVERSTRENGTH FACTOR (Ω_o):	3	F_a =	1.200
		F_v =	1.864

ASCE 7-16 SECTION 11.4 SEISMIC GROUND MOTION VALUES

Section 11.4.4 - Coefficients and Risk-Targeted Maximum Considered Earthquake (MCER) Spectral Response Acceleration Parameters

$S_{MS} = F_a{*}S_S =$ 1.519 $S_{M1} = F_v{*}S_1 =$ 0.813

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3{*}S_{MS} =$ 1.013 $S_{D1} = 2/3{*}S_{M1} =$ 0.542

ASCE 7-16 SECTION 11.6 - SEISMIC DESIGN CATEGORY - SECTION 12.8.2 - PERIOD DETERMINATION

ASCE 7-16 TABLE 11.6-1

SEISMIC DESIGN CATEGORY BASED ON S_{DS}

	RISK CATEGORY:		
	I & II	III	IV
< 0.167g	A	A	A
< 0.33g	B	B	C
< 0.50g	C	C	D
>= 0.50g	D	D	D
	D		

ASCE 7-16 TABLE 11.6-2

SEISMIC DESIGN CATEGORY BASED ON S_{D1}

	RISK CATEGORY:		
	I & II	III	IV
< 0.067g	A	A	A
< 0.133g	B	B	C
< 0.20g	C	C	D
>= 0.20g	D	D	D
	D		

Each building and structure shall be assigned to the most severe Seismic Design Category in accordance with Table 11.6-1 or Table 11.6-2, irrespective of the fundamental period of vibration of the structure.

PERIOD DETERMINATION:	
C_t =	0.02
h_n =	20 FT
x =	0.75
$T_a = C_t{*}h_n^{\ x}$ =	0.189

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

GENERAL EQUATION:	$C_S = S_{DS}/(R/I)$ =	0.156	<--CONTROLS	EQ. 12.8-2
MAXIMUM:	CS = 1.5*SD1/(T*(R/I)) =	0.661		EQ. 12.8-3
MINIMUM:	$C_S = 0.044{*}S_{DS}{*}I > 0.01$ =	0.045		EQ. 12.8-5
For structures located where S1 > 0.6g				
	$C_S = 0.5{*}S_1/(R/I)$ =	0.000		EQ. 12.8-6

ASCE 7-16 SECTION 12.8.1 - SEISMIC BASE SHEAR

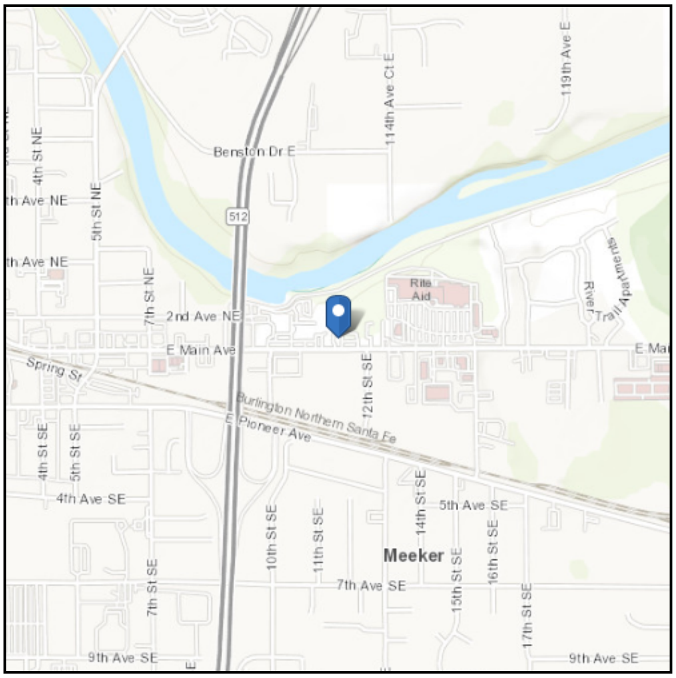
$V = C_S{*}W =$ **0.156*W**

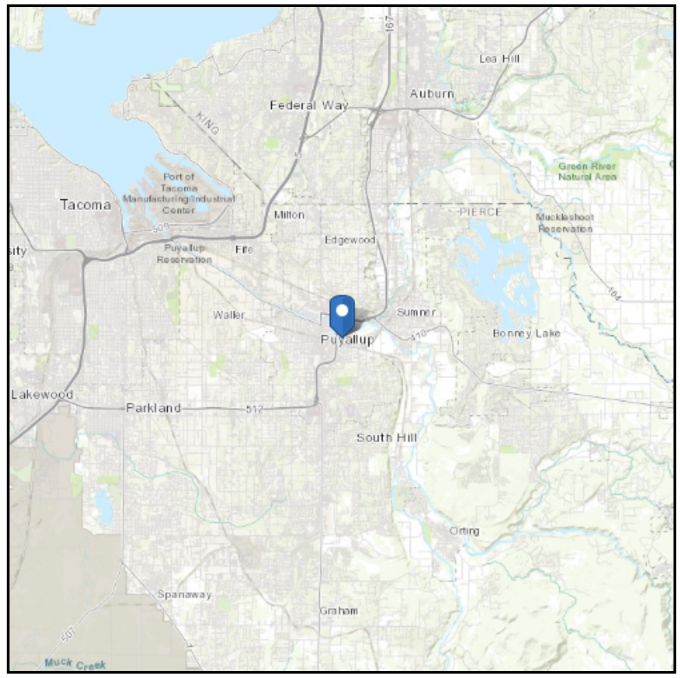
W = the total dead load and applicable portion of other loads as indicated in Section 12.7.2

ASCE 7 Hazards Report

Address:
1115 E Main
Puyallup, Washington
98372

Standard:	ASCE/SEI 7-16	**Latitude:**	47.192103
Risk Category:	II	**Longitude:**	-122.27905
Soil Class:	D - Default (see Section 11.4.3)	**Elevation:**	54.273300457218 ft (NAVD 88)





Wind

Results:

Wind Speed	98 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	83 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Fri Jun 16 2023

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

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

Seismic

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

Results:

S_S :	1.266	S_{D1} :	N/A
S_1 :	0.436	T_L :	6
F_a :	1.2	PGA :	0.5
F_v :	N/A	PGA_M :	0.6
S_{MS} :	1.519	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.013	C_v :	1.353

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

Data Accessed: Fri Jun 16 2023

Date Source: [USGS Seismic Design Maps]

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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LATERAL

Seismic

$C_s = 0.156$ (Per Design Criteria)

Roof Area = 3,212 ft² total

Includes Wall Weights

Seismic Wt. = 3,212 ft² × 25 psf effective = 80.3 K
+ 555 ft² × 20 psf eff. canopy = 11 K (Arch Revision)

Seismic Base Shear = 80.3 K × 0.156 = 12.5 K Controls
+1.7 K Canopy
14.2 K Total

Wind

Windward + Leeward Wall Pressures = 9.2 psf + 5.7 psf = 14.9 psf < 16 psf min.

⇒ 16 psf wall pressure controls
+
8 psf projected roof pressure

- South Elevation (Broad Face of Bldg)

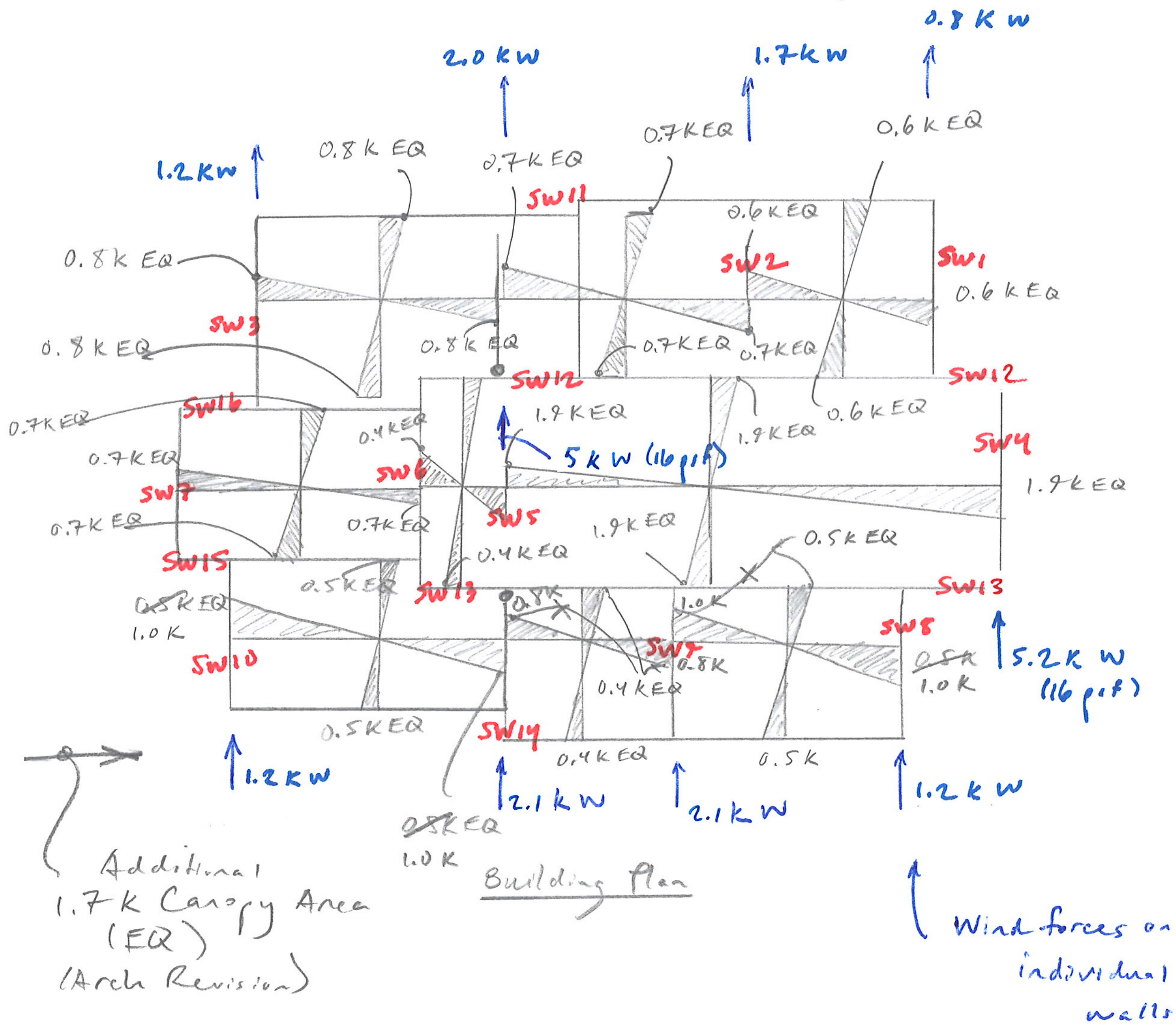
Wall Area Trib to Roof Diaphragms = 857 ft² × 16 psf

= 13.7 K

Comparing 0.6W vs. 0.7EQ, Seismic Controls Lateral Design.
↳ By inspection, the same is true in narrow direction of building.

∴ Seismic Controls Global Lateral Design

→ Review individual walls where wind may govern its design locally. w/ windward pressures. (or 16 psf whichever applicable)



Lateral Map

* All forces noted are ultimate level

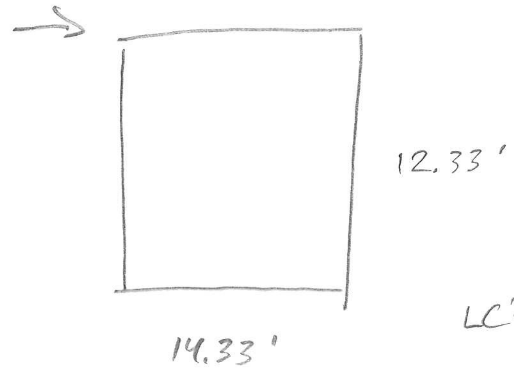
PCS Structural Solutions

Project: Taco Time NW | Job No: 23-514
Subject: Lateral | Sheet 3/19 | Name: JMB
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland | Date: 2023-06-16

SW1

0.8 k (W, ult)
0.6 (EQ, ult)

$w_d = 0.12$ klf



$V_w = \frac{0.6 \cdot 0.8 k}{14.33'} = 33$ plf

* See attached shear wall capacities schedule

10d @ 6" o.c. OK

LC's for OT:

$(0.6 - 0.14 \, S_{DS}^{1.013})D$

$= 0.46 D$

$M_{OT} = 0.6 \cdot 0.8 \cdot 12.33' = 5.9$ k-ft

Conservative for wind, captures similar seismic loading.

$M_{res} = 0.46 \cdot 0.12$ klf $\times 14.33' \times 14.33'/2 = 5.7$ k-ft

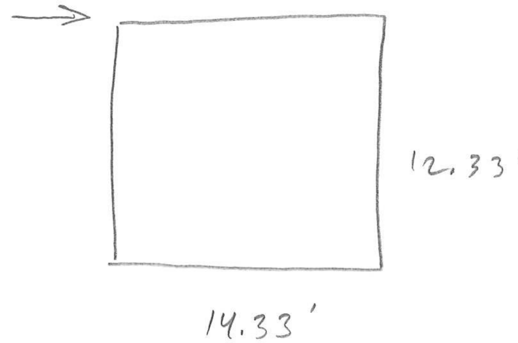
$M_{diff} = 0.2$ k·ft

$HD = 0.2$ k·ft $/ 13.33' \leq 0.1$ k   HDU2 Min

SW2

1.7 k (W, ult.)
1.3 k (EQ, ult.)

$W_d = 0.12$ klf



$V_{EQ} = \frac{0.6 \cdot 1.7\ k}{14.33'} = 71$ plf

10d @ 6" o.c. ok

$M_{or} = 0.6 \cdot 1.7k \cdot 12.33' = 12.6$ k-ft

$M_{res} = 0.46 \cdot 0.12$ klf $\times 14.33' \times 14.33'/2 = 5.6$ k-ft

$M_{diff} = 7.0$ k-ft

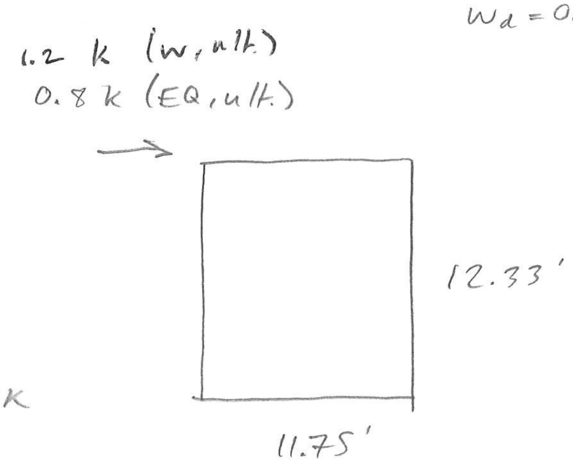
HD = 7.0 k-ft / 13.33' = 0.5 k HDU2 Min

2'-0" x 1'-0" Thickened slab Fdn

$M_{res\ fdn} = 0.46 \cdot 0.3$ klf $\cdot 15' \times 15'/2 = 15.5$ k-ft $> M_{diff}$
OT OK

SW3

$w_d = 0.12$ klf



$V_w = \frac{0.6 \cdot 1.2K}{11.75'} = 61$ plf

10d @ 6" o.c. OK

$M_{OT} = 0.6 \cdot 1.2k \times 12.33' = 8.9$ k-ft

$M_{res} = 0.46 \cdot 0.12$ klf $\times 11.75' \times 11.75'/2 = 3.8$ k-ft

$M_{diff} = 5.1$ k-ft

HD = 5.1 k-ft / 10.75' = 0.5 k HDU2 min.

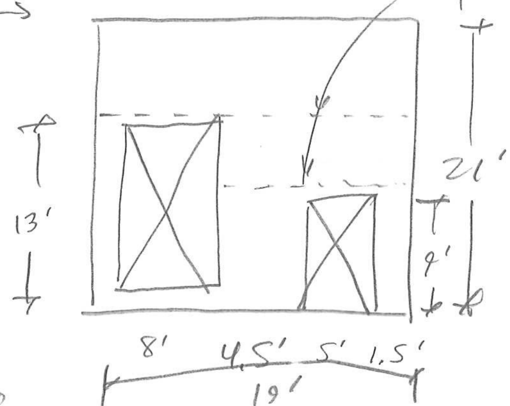
Conventional strip footing (1'-6" x 10" min)

$M_{res,\,fdn} = 0.46 \cdot 0.24$ klf $\cdot 12' \times 12'/2 = 8$ k-ft > M_{diff} OK

SW4

5.2k (w, ult.)
1.9k (EQ, ult.)

$W_D = 0.1$ klf



$v_w = \frac{0.6 \cdot 5.2k}{4.5' \cdot 0.89} = 780$ plf

Shtg (2) Sides w/ 10d @ 4" o.c.
OK

{ Aspect Ratio = 13'/4.5' = 2.9
{ 1.25 - 0.125 · 2.9' = 0.89

Global:

$M_{OT} = 0.6 \cdot 5.2k \cdot 21' = 65.5$ k-ft

$M_{res} = 0.46 \cdot 0.1$ klf $\cdot 19' \times 19'/2 = 8.3$ k-ft

$M_{diff} = 57.2$ k-ft

$HD = 57.2$ k-ft $/ 19' = 3$ k HDU4 min

M, res, ftg = 0.6 * 0.725 klf * 24' * 24* / 2 = 125 K-ft > Mdiff OK

3'x1.5" ftg

actual load factor for wind

Local:

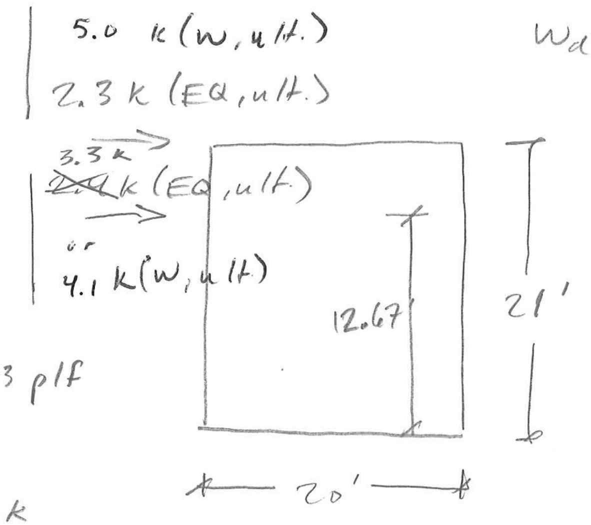
$M_{OT} = 0.6 \cdot 5.2^k \cdot 13' = 40.6$ k-ft

$HD = 40.6$ k-ft $/ 3.5' = 11.6$ k HDU14 w/ 6x6 DF#1

Strap: $T = 0.6 \cdot 5.2k = 3.1$ k Use CMSTC16 (4.7k Capacity (160))
OK

SW5

$W_d = 0.2$ klf



$V_w = \dfrac{0.6 \cdot 9.1k}{20'} = 273$ plf

10d @ 6" o.c. OK

$M_{OT} = 0.6\,(5\,k \times 21' + 4.1\,k \times 12.67') = 94.2$ k-ft

$M_{res} = 0.46 \cdot 0.2\ klf \times 20' \times 20'/2 = 18.4$ k-ft

$M_{diff} = 75.8$ k-ft

$HD = 75.8\ k\text{-}ft / 19' = 4.0\ k$ HDU5 Min

for wind loading, actual load factor applicable

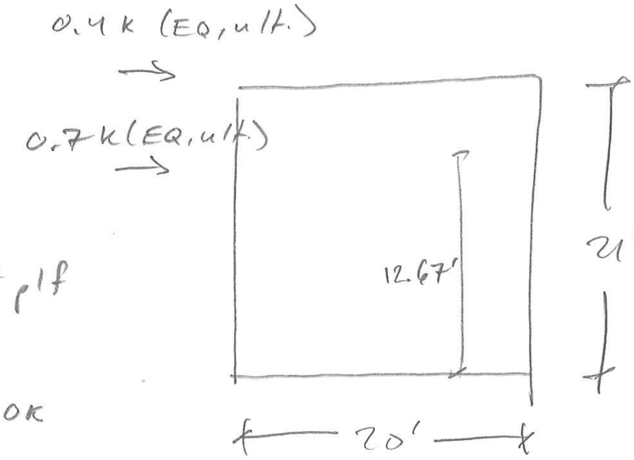
$M_{res\,ftg} = 0.6 \cdot 0.725\ klf \cdot 20' \times 20'/2 = 87$ k-ft $> M_{diff}$

OT OK

3' x 1.5' Ftg.

SW 6

$W_D = 0.2$ klf



$v_{EQ} = \dfrac{0.7 \cdot 1.1k}{20'} = 39$ plf

10d @ 6" o.c. OK

Conservative

$M_{OT} = 0.7 \cdot 1.1k \times 21' = 16.2$ k-ft

$M_{res} = 0.46 \cdot 0.2$ klf $\cdot 20' \times 20'/2 = 18.4$ k-ft $> M_{OT}$

No OT

No HD's Req'd

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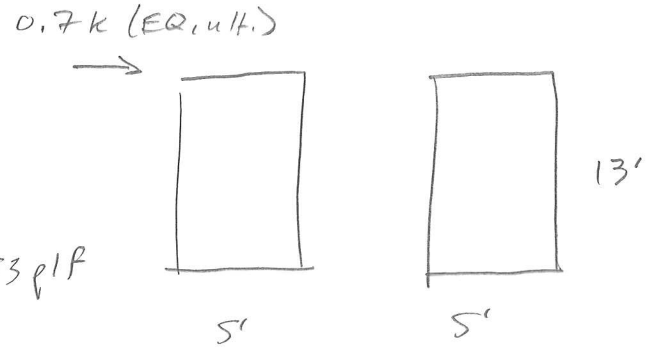
SW 7

$w_d = 0.13$ klf



$$v_{EQ} = \frac{0.7 \cdot 0.7k}{10' \cdot 0.925} = 53 \text{ plf}$$

10d @ 6" o.c. OK

$\{ 1.25 - 0.125 \, h/b = 0.925$

$$M_{OT} = 0.7 \cdot \frac{0.7k}{2} \times 13' = 3.2 \text{ k-ft}$$

$$M_{res} = 0.46 \cdot 0.13 \text{ klf} \times 5' \times 5'/2 = 0.7 \text{ k-ft}$$

$$M_{diff} = 2.5 \text{ k-ft}$$

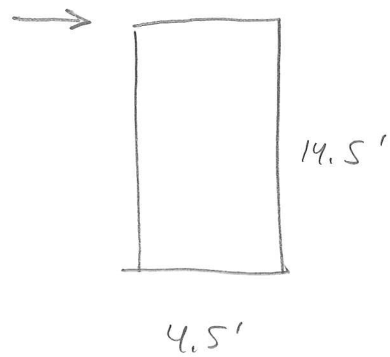
$$HD = 2.5/4' = 0.6k \quad \text{HDU2 min}$$

SW 8

$w_d = 0.14$ klf

1.2 k (W, ult)

~~0.8~~ k (EQ, ult)
1.0



$$V_w = \frac{0.6 \cdot 1.2k}{4.5' \times 0.85} = 188 \text{ plf}$$

10d @ 6" o.c. OK

$\{1.25 - 0.125\ h/b = 0.85$

$M_{OT} = 0.6 \cdot 1.2k \cdot 14.5' = 10.4$ k-ft

Use Seismic LC → Similar loading

$M_{res} = 0.46 \cdot 0.14$ klf $\times 4.5' \times 4.5'/2 = 0.6$ k-ft

$M_{diff} = 9.8$ k-ft

HD = 9.8 k-ft / 3.5' = 2.8 k   HDU4 min.

3' x 1.5' Ftg.

$M_{res\ fdn} = 0.46 \cdot 0.675$ klf $\cdot 8.5' \times 8.5'/2 = 11.2$ k-ft $> M_{diff}$

OT OK

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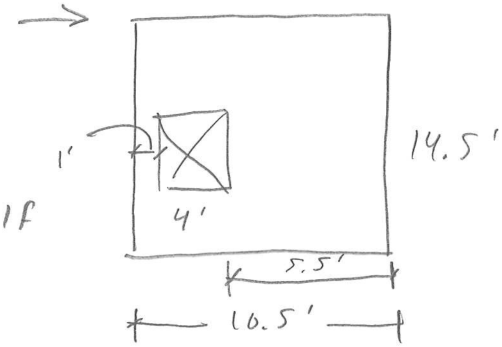
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland   Date: 2023-06-16

SW 9

2.1 k (W, ult.)

~~0.9~~ k (EQ, ult.)
1.8

$w_d = 0.14$ klf



$v_w = \dfrac{0.6 \cdot 2.1\,k}{5.5' \times 0.92} = 249$ plf

10d @ 6" o.c. OK

$\{ 1.25 - 0.125\, h/b = 0.92$

$M_{OT} = 0.6 \cdot 2.1k \cdot 14.5\,ft = 18.3$ k-ft

Use Seismic LC, similar loadings

$M_{res} = 0.46 \cdot 0.14\,klf \cdot 5.5' \cdot 5.5'/2 = 1$ k-ft

$M_{diff} = 17.3$ k-ft

$HD = 17.3\,k\text{-}ft / 4.5' = 3.8\,k$ HDU5 min

3'x1.5' Ftg.

$M_{res\ fdn} = 0.46 \cdot 0.675\,klf \times 13' \times 13'/2 = 26$ k-ft $> M_{diff}$

OT OK

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Originating Office: ☐ Seattle	☒ Tacoma	☐ Portland	Date: 2023-06-16

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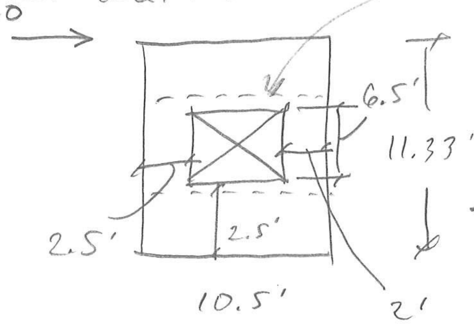
SW1D

1.2 k (w, ult)

~~0.5~~ 1.0 k (EQ, ult)

$w_d = 0.1$ klf



Strap Above & Below

P = 0.6 · 1.2 k = 0.7 k

Try Simpson CS20

2s 1 k capacity (1.60)

OK

$v_w = \dfrac{0.6 \cdot 1.2k}{4.5' \cdot 0.62} = 258$ plf

10d @ 6" o.c. OK

$\left\{ \begin{array}{l} 1.25 - 0.125\, h/b = 0.84 \\ 2b/h = 0.62 \end{array} \right.$

Global:

$M_{OT} = 0.6 \cdot 1.2K \cdot 11.33' = 8.2$ k-ft

Use Seismic LC, similar loading

$M_{res} = 0.46 \cdot 0.1$ klf $\cdot 10.5' \cdot 10.5'/2 = 2.5$ k-ft

$M_{diff} = 5.7$ k-ft

HD = 5.7 kft / 9.5' = 0.6 k

Local:

$M_{OT} = 0.6 \cdot 1.2k \left(\frac{2'}{4.5'}\right) \cdot 6.5' = 2.1$ k-ft

HD = 2.1 k-ft / 1.5' = 1.4 k

v_w = 1.4 k / 2.5' = 560 plf → Too High, Provide HD

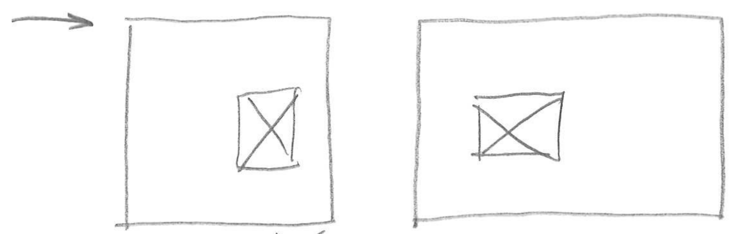
Global + Local

HD = 1.4 k + 0.6 k = 2.0 k   Provide HDU2 min

SW11

$W_d = 0.2\ klf$

2.1 k (EQ, ult.)



12'

10' ~~3'~~ 3.83' 21'

X

$$v_{EQ} = \frac{0.7 \cdot 2.1\,k}{34.83' \cdot 0.64} = 66\ plf$$

(for 3.83' pier)

$\{\ 2b/h = 0.64$

10d @ 6" o.c. OK

3.83' Pier

$M_{OT} = 0.7 \cdot 2.1\,k \left(\frac{3.83}{34.83}\right) \cdot 12' = 1.94\ k\text{-}ft$

$M_{RES} = 0.46 \cdot 0.2\ klf \times 3.83' \times 3.83'/2 = 0.67\ k\text{-}ft$

$M_{diff} = 1.27\ k\text{-}ft$

$HD = 1.27\ k\text{-}ft / 2.83' = 0.4\ k$ HDU2 Min

10' Pier

$M_{OT} = 0.7 \cdot 2.1\,k \left(\frac{10}{34.83}\right) \cdot 12' = 5.1\ k\text{-}ft$

$M_{RES} = 0.46 \cdot 0.2\ klf \cdot 10' \cdot 10'/2 = 4.6\ k\text{-}ft$

$M_{diff} = 0.5\ k\text{-}ft$

$HD = 0.5\ k\text{-}ft / 9' = 0.1\ k$ HDU2 Min

21' Pier

No OT → No HD's Req'd by inspection

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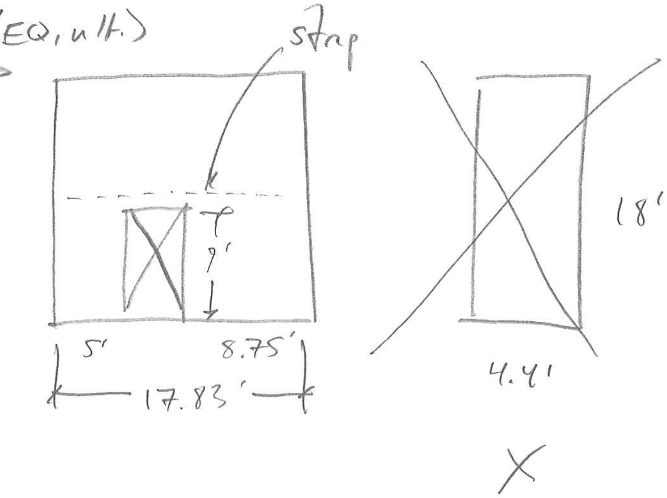
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 2023-06-16

SW12

$W_d = 0.4$ klf

4.4 k (EQ, ult.)



$$V_{EQ} = \frac{0.7 \cdot 4.4\text{ k}}{13.75'} = 224 \text{ plf}$$

10d @ 6" o.c. OK

Global:

$M_{OT} = 0.7 \cdot 4.4\text{k} \cdot 18' = 55.4$ k-ft

$M_{res} = 0.46 \cdot 0.4 \text{ klf} \cdot 17.83' \cdot 17.83'/2 = 29.2$ k-ft

$M_{diff} = 26.2$ k-ft

$HD = 26.2\text{ k-ft} / 16.83' = 1.6$ k

Local:

$$M_{OT} = 0.7 \cdot 4.4\text{ k} \cdot \left(\frac{5}{13.75}\right) \cdot 9' = 10.1 \text{ k-ft}$$

$HD = 10.1\text{ k-ft} / 4' = 2.5$ k

Global + Local:

$HD = 2.5\text{ k} + 1.6\text{ k} = 4.1$ K HDU5 Min

Strap:

$$P = 0.7 \cdot 4.4\text{ k} \cdot \left(\frac{8.75'}{13.75}\right) = 2\text{ k} \rightarrow \text{CS14 Strap} \rightarrow 2.5\text{ k Capacity (160)}$$

OK

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SW 12 Cont'd

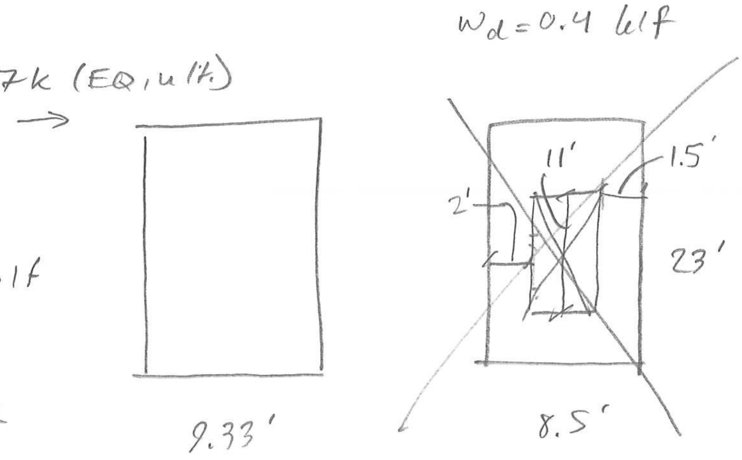
2' x 1.5' Ftg

$M_{OT\ Fdn} = 0.46 \cdot 0.45\text{ klf} \cdot 18' \times 18'/2 = 33.5\text{ k-ft} > M_{diff}$ g'.L.1

ok OT

∴ Use 2' wide x 18" Thick Ftg.

SW13



$$v_{EQ} = \frac{0.7 \cdot 3.7k}{9.33' \cdot 0.94} = 295 \text{ plf}$$

10d @ <u>4"</u> o.c. OK

$\{ \; 1.25 - 0.125 \, {}^h/_b = 0.94$

$M_{OT} = 0.7 \cdot 3.7k \cdot 23' = 59.6$ k-ft

$M_{res} = 0.46 \cdot 0.4 \text{ klf} \times 9.33' \times 9.33'/2 = 8$ k-ft

$M_{diff} = 51.6$ k-ft

$HD = 51.6 \text{ k-ft} / 8.33' = 6.2k$ HDU8 w/ (3) studs min

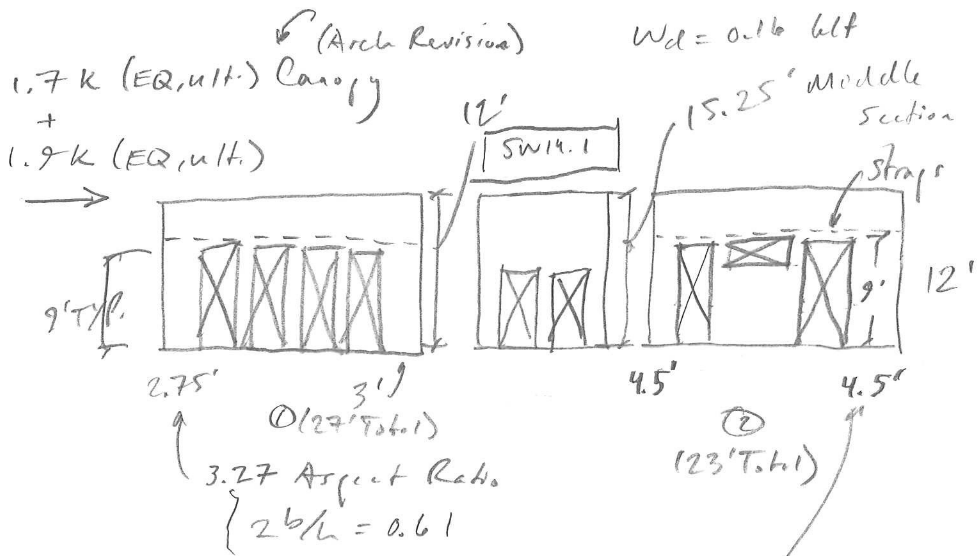
3.5' x 2' Req Ftg.

$M_{res\ fdn} = 0.46 \cdot 1.05 \text{ klf} \cdot 15' \cdot 15'/2 = 54.3 \text{ k-ft} > M_{diff}$

OT OK

∴ Use 3.5' x 2' Footing x 15' long min

SW14



* Walls act together as they are tied together w/ a low roof diaphragm.

Aspect ratio = 2.0, no adjustment req'd.

$$v_s = \frac{0.7 \cdot (1.7k + 1.9k)}{(2.75' + 3' + 4.5' + 4.5') \cdot 0.61 \text{ Adjustment.}}$$

$= 280$ plf → 10d @ 4" o.c. OK

Segment ①:

Global: $M_{OT} = 0.7 \cdot 3.6k \left(\frac{5.75}{14.75}\right) \cdot 12' = 11.8$ k-ft

$M_{res} = 0.46 \cdot 0.16$ klf $\cdot\, 27^2/2 = 26.8$ k-ft

NO Global OT

Local: $M_{OT} = 0.7 \cdot 3.6k \left(\frac{2.75}{14.75}\right) \cdot 9' = 4.3$ k-ft

$HD = \frac{4.3 \text{ k-ft}}{1.75'} = 2.5k$ HDU4 min

Segment ②

Global: $M_{OT} = 0.7 \cdot 3.6k \left(\frac{9}{14.75}\right) \cdot 12' = 18.5$ k-ft

$M_{res} = 0.46 \cdot 0.16$ klf $\cdot\, 23^2/2 = 19.5$ k-ft

NO Global OT

Local: $M_{OT} = 0.7 \cdot 3.6k \left(\frac{4.5}{14.75}\right) \cdot 9' = 6.9$ k-ft

$HD = \frac{6.9 \text{ k-ft}}{3.5'} = 2.0k$ HDU2 min

Straps: $T = 0.7 \cdot 3.6k \left(\frac{9}{14.75}\right) = 1.5k$ CS16 MIN.

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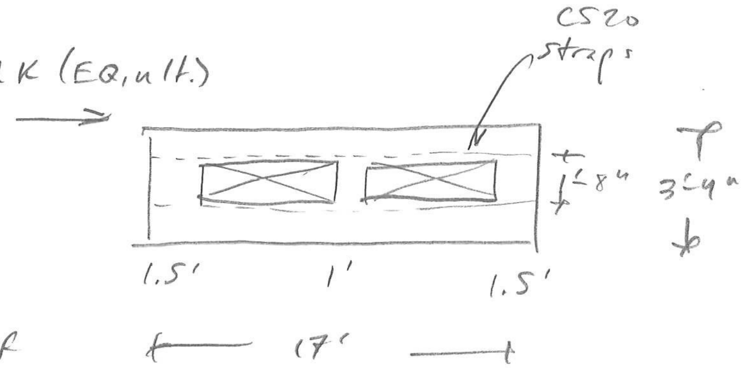
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 2023-06-20

SW14.1



$v_s = \dfrac{0.7 \cdot 0.4k}{4'} = 70$ plf

10d @ 6" o.c. OK

OT OK by inspection

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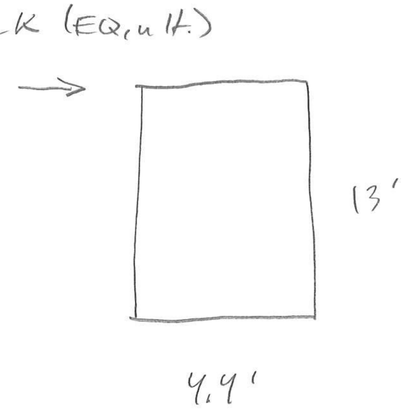
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland Date: 2023-06-16

SW15

$W_d = 0.23$ klf



$V_{EQ} = \dfrac{0.7 \cdot 1.2k}{4.4' \times 0.88} = 217$ plf

10d @ 6" o.c. OK

$\{ \; 1.25 - 0.125\ h/b = 0.88$

$M_{OT} = 0.7 \cdot 1.2k \cdot 13' = 10.9$ k-ft

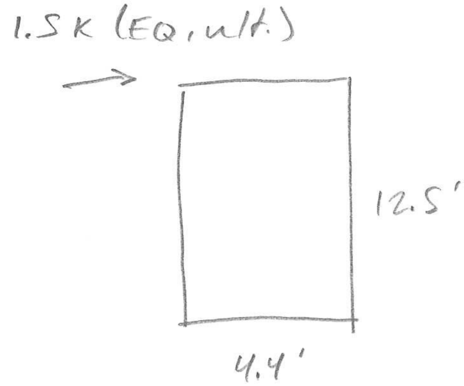
$M_{res} = 0.46 \cdot 0.23\text{klf} \cdot 4.4' \times 4.4'/2 = 1.0$ k-ft

$M_{diff} = 9.9$ k-ft

$HD = 9.9\text{ k-ft} / 3.4' = 2.9$ k HDU 4 min

SW16

$w_d = 0.23$ klf



$v_{EQ} = \dfrac{0.7 \cdot 1.5\,k}{4.4' \cdot 0.89} = 268$ plf

10 d @ 6" o.c. OK

$\{ 1.25 - 0.125\ h/b = 0.89$

$M_{OT} = 0.7 \cdot 1.5\,k \cdot 12.5' = 13.1$ k-ft

$M_{res} = 0.46 \cdot 0.23$ klf $\times 4.4' \times 4.4'/2 = 1$ k-ft

$M_{diff} = 12.1$ k-ft

HD = 12.1 k-ft / 3.4' = 3.6 k HDU5 min

Fdn => Similar to SW 8 => Use 3' x 1.5' Ftg.

Diaphragms

- Check High Roof Diaphragm Shear (worst case shear)

$v_w = \frac{0.6 \cdot 5.2\,k}{20'} = 156$ plf

Try unblocked WSP Diaphragm "Rated" Sheathing w/ 8d nails (unblocked)

↳ $V_{allow} = \frac{480\ plf}{2} = 240$ plf (allow.) $> v_w$ ok

- High Roof Chord Force (worst case)

$M_w = 0.6 \cdot 213\ plf \cdot \frac{(47\ ft)^2}{8} = 35.3$ k-ft

$T/C = 35.3\ k\text{-}ft / 19.5' = 1.8$ k Cont. Top Chord Ok (18 nails min. @ splices, 100 # per nail.)

- Low Roof Chord Force (worst case)

$M_s = 0.7 \cdot 80\ plf \cdot \frac{(26\ ft)^2}{8} = 4.7$ k-ft

$T/C = 4.7\ k\text{-}ft / 7.5' = 630$ lb CS20 OK (1k capacity (160))

- SWS Low Roof Collector Straps

$T_w = 0.6 \cdot 2.1\,k = 1.3\,k$ CS16 OK (1.7k Capacity (160))

ALLOWABLE SHEAR (PLF) FOR WOOD STRUCTURAL PANEL SHEAR WALLS - RATED SHEATHING

DOUGLAS FIR-LARCH (S.G. = 0.50)

Sheathing	Edge Attachment (in. O.C.)	Field Attachment (in. O.C.)	Framing Lumber[3,4]	Sill Plate	Seismic						Wind					
					Allow (plf)	Sole Nailing[6,7]		Framing Clips[8]	5/8" A.B. (in. O.C.)	3/4" A.B. (in. O.C.)	Allow (plf)	Sole Nailing[6,7]		Framing Clips[8]	5/8" A.B. (in. O.C.)	3/4" A.B. (in. O.C.)
						Spacing	# of Rows	Spacing				Spacing	# of Rows	Spacing		
Single Sided with 10d Nails																
15/32" (1) Side	10d @ 6"	10d @ 12"	2x D.F.	2x D.F.	**310**	16d @ 8"	2	24"	48	48	**434**	16d @ 8"	2	18	32	48
15/32" (1) Side	10d @ 4"	10d @ 12"	3x D.F.	2x D.F.	**460**	16d @ 8"	2	16"	32	48	**644**	SDS @ 6"	1	12"	24	32
15/32" (1) Side	10d @ 3"	10d @ 12"	3x D.F.	2x D.F.	**600**	SDS @ 6"	1	12"	24	32	**840**	SDS @ 6"	2	9"	16	24
Double Sided with 10d Nails																
15/32" (2) Sides	10d @ 6"	10d @ 12"	2x D.F.	2x D.F.	**620**	SDS @ 6"	1	12"	24	32	**868**	SDS @ 6"	2	9"	16	24
15/32" (2) Sides	10d @ 4"	10d @ 12"	3x D.F.	3x D.F.	**920**	SDS @ 6"	2	8"	24	32	**1288**	SDS @ 6"	2	6"	16	24
15/32" (2) Sides	10d @ 3"	10d @ 12"	3x D.F.	3x D.F.	**1200**	SDS @ 6"	2	6"	16	24	**1680**	SDS @ 6"	3	4"	12	16

HEM-FIR (S.G. = 0.43)

Sheathing	Edge Attachment (in. O.C.)	Field Attachment (in. O.C.)	Framing Lumber[3,4]	Sill Plate	Seismic						Wind					
					Allow (plf)	Sole Nailing[6,7]		Framing Clips[8]	5/8" A.B. (in. O.C.)	3/4" A.B. (in. O.C.)	Allow (plf)	Sole Nailing[6,7]		Framing Clips[8]	5/8" A.B. (in. O.C.)	3/4" A.B. (in. O.C.)
						Spacing	# of Rows	Spacing				Spacing	# of Rows	Spacing		
Single Sided with 10d Nails																
15/32" (1) Side	10d @ 6"	10d @ 12"	2x H.F.	2x H.F.	**288**	16d @ 8"	2	22"	48	48	**404**	16d @ 8"	2	16"	32	48
15/32" (1) Side	10d @ 4"	10d @ 12"	3x H.F.	2x H.F.	**428**	16d @ 8"	2	16"	32	48	**599**	SDS @ 6"	1	10"	24	32
15/32" (1) Side	10d @ 3"	10d @ 12"	3x H.F.	2x H.F.	**558**	SDS @ 6"	1	12"	24	32	**781**	SDS @ 6"	2	8"	16	24
Double Sided with 10d Nails																
15/32" (2) Sides	10d @ 6"	10d @ 12"	2x H.F.	2x H.F.	**577**	SDS @ 6"	1	12"	24	32	**807**	SDS @ 6"	2	8	16	24
15/32" (2) Sides	10d @ 4"	10d @ 12"	3x H.F.	3x H.F.	**856**	SDS @ 6"	2	8"	24	32	**1198**	SDS @ 6"	2	5	16	16
15/32" (2) Sides	10d @ 3"	10d @ 12"	3x H.F.	3x H.F.	**1116**	SDS @ 6"	2	6"	16	24	**1562**	SDS @ 6"	2	4	12	16

NOTES:

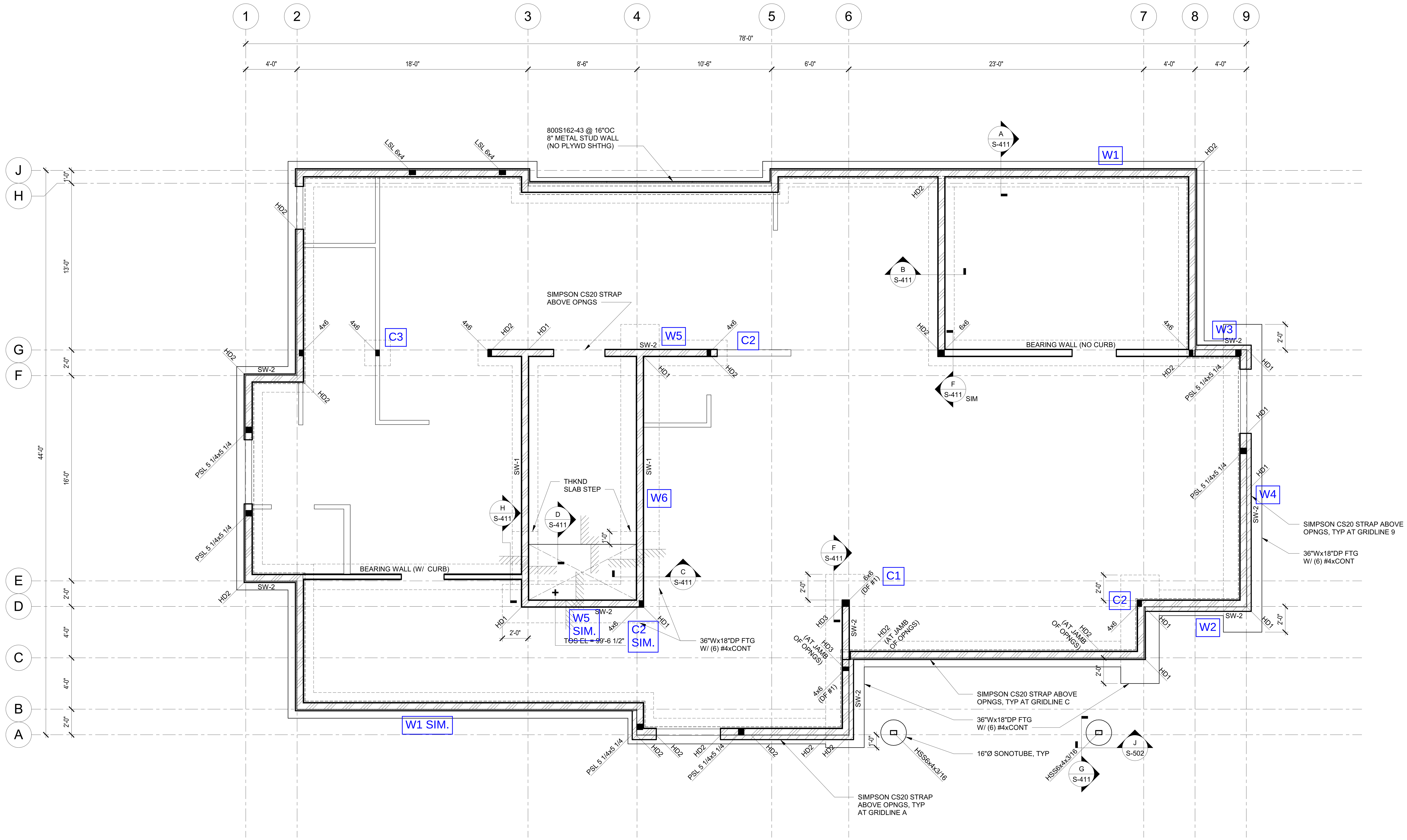
1. DESIGN AND CONSTRUCTION OF SHEAR WALLS PER **2018 IBC & 2018 SDPWS**
2. CAPACITIES BASED ON SDPWS TABLE 4.3A.
3. 3x OR (2) 2x MEMBERS ARE REQUIRED AT ABUTTING PANEL EDGES WHERE SEISMIC LOAD EXCEEDS 350 PLF/SIDE. (SDPWS 4.3.7.1.5c)
4. 3x OR (2) 2x MEMBERS ARE REQUIRED WHERE NAILING IS LESS THAN 6" FROM BOTH SIDES INTO COMMON MEMBER. (NOTE 6 OF SDPWS TABLE 4.3A)
5. SHEAR WALLS ARE BLOCKED AT ALL PANEL EDGES
6. STAGGER SOLE NAIL ROWS 1/2"
7. SDS SCREWS ARE 1/4"x4 1/2" MINIMUM (ASD CAPACITY 350# DF/ 250# HF). 16d COMMON NAILS ARE 0.148"x3.5" (ASD CAPACITY 100# DF/86# HF).
8. FRAMING CLIPS ARE SIMPSON A35 OR LTP5. (ASD CAPACITY A35-670#DF/575#HF LTP5-620#DF/535# HF). CONSIDER CLIP LENGTH & OPTIONS AT TIGHT SPACING.
9. SDPWS 4.3.6.4.3 - SILL PLATE WITH 0.229x3"x3" PLATE WASHER ON AB's 1/2" MAX FROM SHEATHED EDGE. PCS STANDARD USES SIMPSON SLOTTED WASHER.
10. ANCHOR BOLTS BASED ON NDS TABLE 12E. 5/8" ASD CAPACITY 930# 2xDF/1180# 3xDF/860# 2xHF/1070#3xHF. 3/4" ASD CAPACITY 1270#/1540#/1200#/1400#.
11. REQUIRES 3x or (2) 2x RIMBOARD AND/OR BLOCKING - DISCUSS WITH PM.

VERTICAL

VERTICAL MAPS

THIS MAP IS FOR REFERENCE OF LOCATION ONLY AND SIZES NOTED ARE PRELIMINARY

NOTES:
1. SEE S4.01, S4.02, AND S8.01 FOR TYPICAL DETAILS.
2. TOP OF FOOTING ELEVATIONS AT 99'-0", UNLESS NOTED OTHERWISE
3. TOP OF SLAB ELEVATION AT 100'-0", UNLESS NOTED OTHERWISE. ELEVATIONS ARE RELATIVE, SEE CIVIL FOR ACTUAL FINISH FLOOR ELEVATIONS.
4. ALL EXTERIOR WALLS TO BE SW-1, UNLESS NOTED OTHERWISE.
5. TYPICAL EXTERIOR FOOTING 24"x12" DEEP WITH (4) #4xCONTINUOUS.
6. WALL FRAMING TO BE HF NO. 2 2x6 @ 16"OC, TYPICAL, UNLESS NOTED OTHERWISE.
7. SLAB ON GRADE TO BE 4" CONCRETE WITH #4 @ 24"OC EACH WAY.
8. SLAB ON GRADE TO BE SUPPORTED BY A MINIMUM OF 12 INCH PROPERLY COMPACTED FILL WITH A MINIMUM 4 INCH THICK CAPILARY BREAK CONSISTING OF FREE-DRAINING, CRUSHED ROCK OR WELL-GRADED GRAVEL" PER GEOTECHNICAL RECOMMENDATIONS.



1 FOUNDATION PLAN
1/4" = 1'-0"

bcra
T 253.627.4367 F 253.627.4395 WWW.BCRADESIGN.COM
2106 PACIFIC AVENUE, SUITE 300, TACOMA, WA 98402

SEAL

PROJECT
NEW CONSTRUCTION
TACO TIME
EAST MAIN STREET
PUYALLUP, WA 98372

REVISIONS

DATE
6.30.2022
BCRA NO.
19110
DRAWN BY: Author
REVIEWED BY:
SHEET TITLE
FOUNDATION PLAN

bcra

SHEET
S-121
PERMIT SET

24x36 3/30/2022 3:22:59 PM

IF SHEET MEASURES LESS THAN 24"X36", IT IS A REDUCED PRINT. REDUCE SCALE ACCORDINGLY

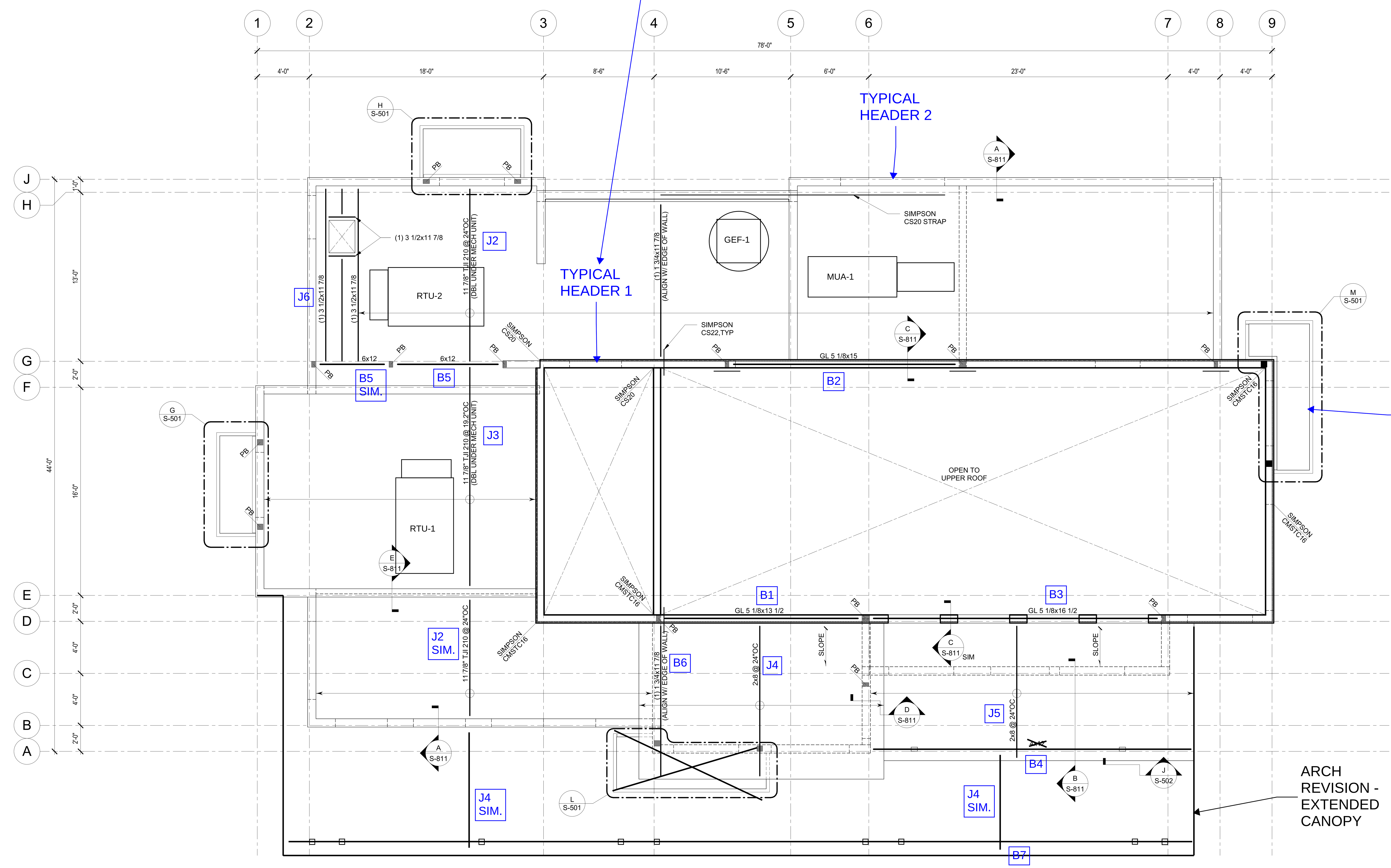
VERTICAL MAPS

THIS MAP IS FOR REFERENCE OF LOCATION ONLY AND SIZES NOTED ARE PRELIMINARY

TYPICAL JAMB BEARING STUD
-> CAPACITY = 625 PSI x 1.5" x 3" WIDE = 2.8 K
-> Compare to Reactions per Enercalc
OK

NOTES:
1. SEE S4.01, S4.02, AND S8.01 FOR TYPICAL DETAILS.
2. 1/2" APA RATED SHEATHING TYPICAL FOR ROOF SHEATHING, NAIL WITH 10d @ 6"OC AT PANEL EDGES AND 12"OC IN THE FIELD.
3. SEE ARCHITECTURAL DRAWINGS FOR TOP PLATE ELEVATIONS.
4. TYPICAL HEADERS TO BE (2) 2x12 FOR OPENINGS UPTO 8'-0". (1) 4x12 ACCEPTABLE.
5. SEE ARCH FOR PLATE HEIGHT.
6. BLOCK BELOW SIMPSON CHORD STRAPS.

TYPICAL HEADER



TYPICAL AWNING

ARCH REVISION - EXTENDED CANOPY

1 LOWER ROOF FRAMING PLAN
1/4" = 1'-0"

24x36 3/30/2022 3:22:59 PM

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SEAL

PROJECT
NEW CONSTRUCTION
TACO TIME
EAST MAIN STREET
PUYALLUP, WA 98372

REVISIONS

DATE
6.30.2022

BCRA NO.
19110

DRAWN BY: Author

REVIEWED BY:

SHEET TITLE
LOWER ROOF FRAMING PLAN

bcra

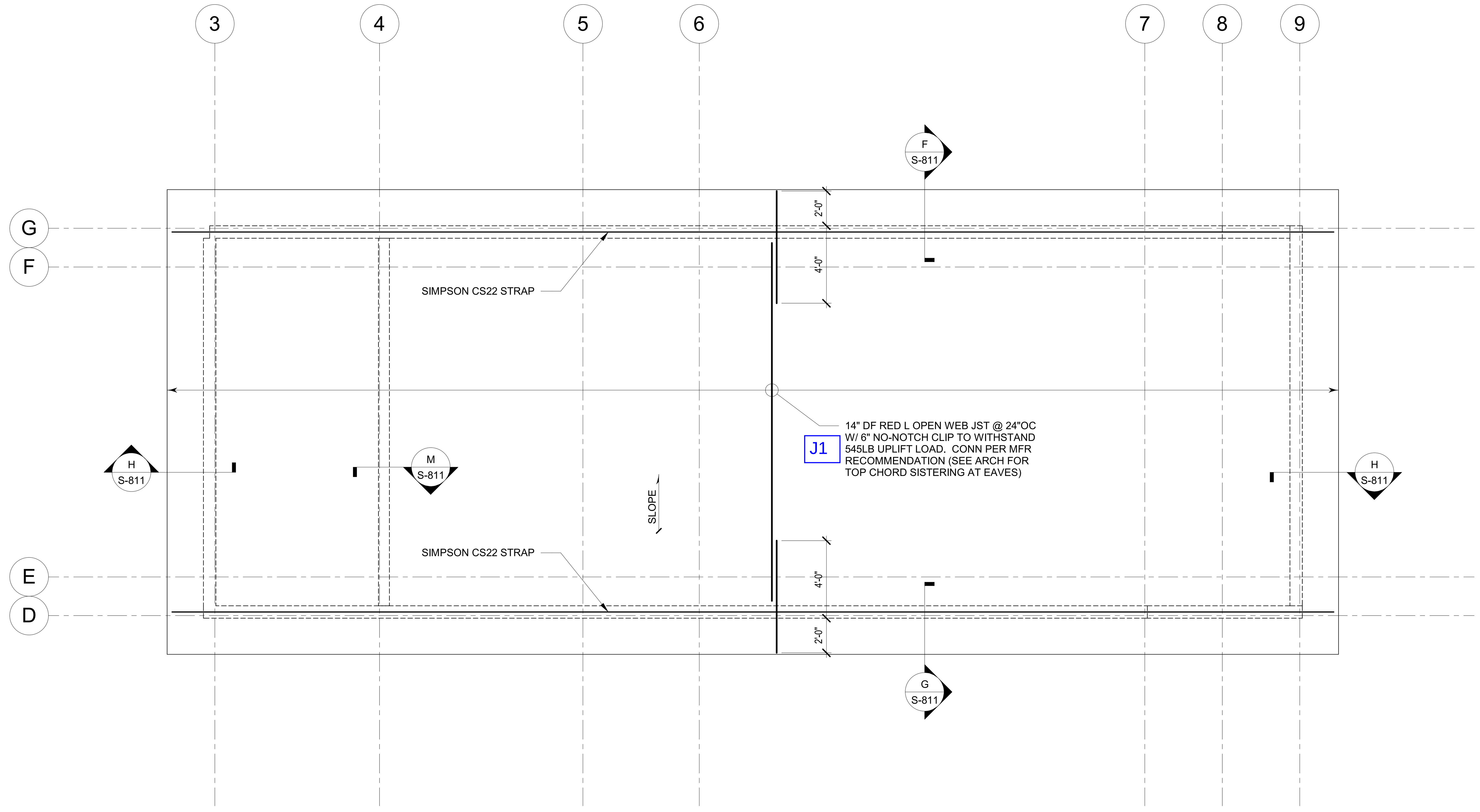
SHEET
S-122
PERMIT SET

IF SHEET MEASURES LESS THAN 24"X36", IT IS A REDUCED PRINT. REDUCE SCALE ACCORDINGLY

VERTICAL MAPS

THIS MAP IS FOR REFERENCE OF LOCATION ONLY AND SIZES NOTED ARE PRELIMINARY

NOTES:
1. SEE S4.01, S4.02, AND S8.01 FOR TYPICAL DETAILS.
2. 1/2" APA RATED SHEATHING TYPICAL FOR ROOF SHEATHING, NAIL WITH 10d @ 6"OC AT PANEL EDGES AND 12"OC IN THE FIELD.
3. SEE ARCHITECTURAL DRAWINGS FOR TOP PLATE ELEVATIONS.
4. TYPICAL HEADERS TO BE (2) 2x12 FOR OPENINGS UPTO 8'-0". (1) 4x12 ACCEPTABLE.
5. SEE ARCH FOR PLATE HEIGHT.
6. BLOCK BELOW SIMPSON CHORD STRAPS.



1 UPPER ROOF FRAMING PLAN
1/4" = 1'-0"

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2106 PACIFIC AVENUE, SUITE 300, TACOMA, WA 98402

SEAL

PROJECT
NEW CONSTRUCTION
TACO TIME
EAST MAIN STREET
PUYALLUP, WA 98372

REVISIONS

DATE
6.30.2022
BCRA NO.
19110
DRAWN BY: Author
REVIEWED BY:
SHEET TITLE
UPPER ROOF FRAMING PLAN

bcra

SHEET
S-123
PERMIT SET

IF SHEET MEASURES LESS THAN 24"X36", IT IS A REDUCED PRINT. REDUCE SCALE ACCORDINGLY

24x36 3/30/2022 3:23:00 PM

RedSpec™ by RedBuilt™
v7.1.14

Project: Project
Location: Puyallup, WA
Folder: Folder
Date: 7-19-23 11:20 AM
Designer: JMB
Comment:

Type: J1

14" Red-L™ @ 24" o.c.

This product meets or exceeds the set design controls for the application and loads listed

This truss design is feasible. The finished design shall be produced by RedBuilt Engineering. All open-web trusses are custom designed to carry the specific design loads for each project. Actual truss capacity when fabricated is limited to that required to resist the specific loads. Do not use this analysis to verify the capacity of existing trusses.

DEFLECTIONS (in)	%	Design	Allow.	Design	Allow.	Pass/Fail
Span Live	36%	0.226	0.633	L / 999+	L / 360	PASS
Span Total	55%	0.525	0.950	L / 434	L / 240	PASS

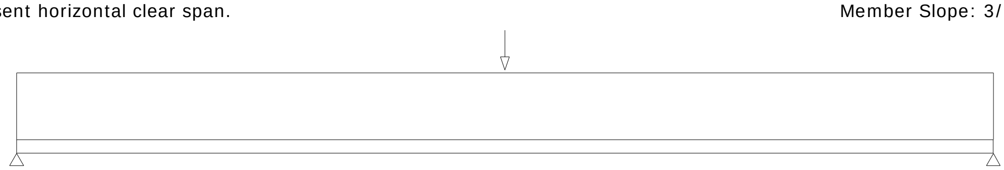
SUPPORTS

	Support 1	Support 2
Live Reaction (lb) (DOL%)	489 (115)	490 (115)
Dead Reaction (lb)	541	543
Total Reaction (lb) (DOL%)	1030 (115)	1033 (115)
Bearing	Top Chord	Top Chord
Support	Wall	Wall
Bearing Clip	6" No-Notch Clip	6" No-Notch Clip
Approx. Clip Height	1.5"	1.5"
Approx. Clip Width	7.1875"	7.1875"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Member Slope: 3/12 í



19'- 0.0"

APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Snow(115%)	25	20	0	24"	Snow Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Point	lb	Snow(115%)	0	300	9'-6.0"	Adds To	

NOTES

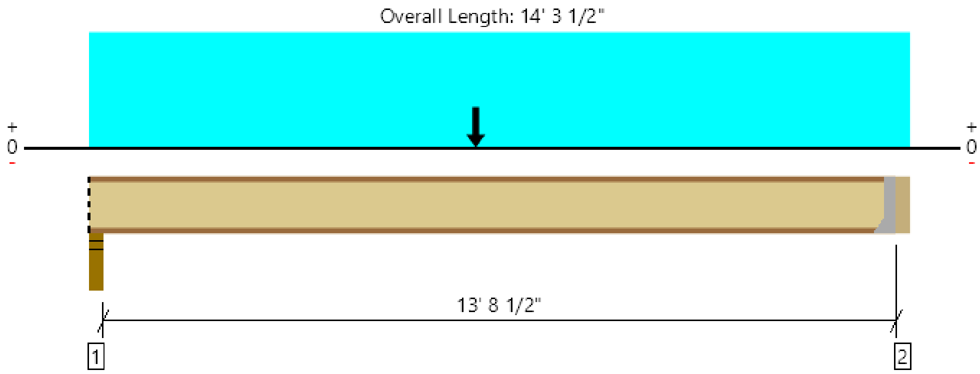
- Building code and design methodology: 2018 IBC ASD (US).
- Repetitive member increase applied in design.
- Beveled plate required at left support.
- Beveled plate required at right support.
- Truss design includes consideration for partial span application live load.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.
- Pricing Load (plf) = 105
- Pricing Index (plf) = 121

G:\2023 Jobs\23514 Taco Time NW\Calcs\23514 redspec 2023-06-17 jmb.red

Roof			
Member Name	Results	Current Solution	Comments
J2	Passed	1 piece(s) 11 7/8" TJI® 210 @ 24" OC	
J3 Dbl	Passed	2 piece(s) 11 7/8" TJI® 210 @ 19.2" OC	
J3	Passed	1 piece(s) 11 7/8" TJI® 210 @ 19.2" OC	
J6	Passed	1 piece(s) 3 1/2" x 11 7/8" 1.55E TimberStrand® LSL @ 36" OC	
J3 - Option 2	Passed	1 piece(s) 11 7/8" TJI® 360 @ 24" OC	
J3 Dbl - Option 2	Passed	2 piece(s) 11 7/8" TJI® 360 @ 24" OC	

Roof, J2

1 piece(s) 11 7/8" TJI® 210 @ 24" 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)	862 @ 14'	1156 (1.75")	Passed (75%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	872 @ 3 1/2"	1903	Passed (46%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	3859 @ 6' 10 1/4"	4364	Passed (88%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.151 @ 7' 1 1/4"	0.460	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.451 @ 6' 10 1/4"	0.690	Passed (L/367)	--	1.0 D + 1.0 S (All Spans)

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

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.75"	543	284	355	898	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	528	288	359	888	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 7" o/c	
Bottom Edge (Lu)	14' 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	IUS2.06/11.88	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 Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 14' 3 1/2"	24"	20.0	20.0	25.0	Default Load
2 - Point (lb)	6' 10 1/4"	N/A	500	-	-	Assumed Mech Unit Load

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by JMB

ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

Weyerhaeuser

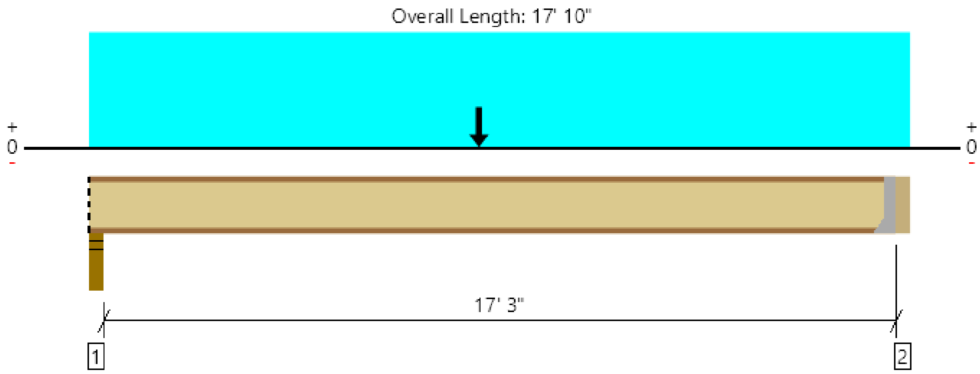
FORTEWEB

MEMBER REPORT

PASSED

Roof, J3 Dbl

2 piece(s) 11 7/8" TJI® 210 @ 19.2" 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)	867 @ 17' 6 1/2"	2312 (1.75")	Passed (37%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	875 @ 3 1/2"	3807	Passed (23%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4867 @ 8' 7 1/2"	8729	Passed (56%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.142 @ 8' 10 1/2"	0.578	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.424 @ 8' 7 1/2"	0.867	Passed (L/490)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.75"	541	284	355	896	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	529	287	358	888	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 8" o/c	
Bottom Edge (Lu)	17' 7" 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	MIU4.28/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	

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

Vertical Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 17' 10"	19.2"	20.0	20.0	25.0	Default Load
2 - Point (lb)	8' 7 1/2"	N/A	500	-	-	Assumed Mech Unit Load

Weyerhaeuser Notes

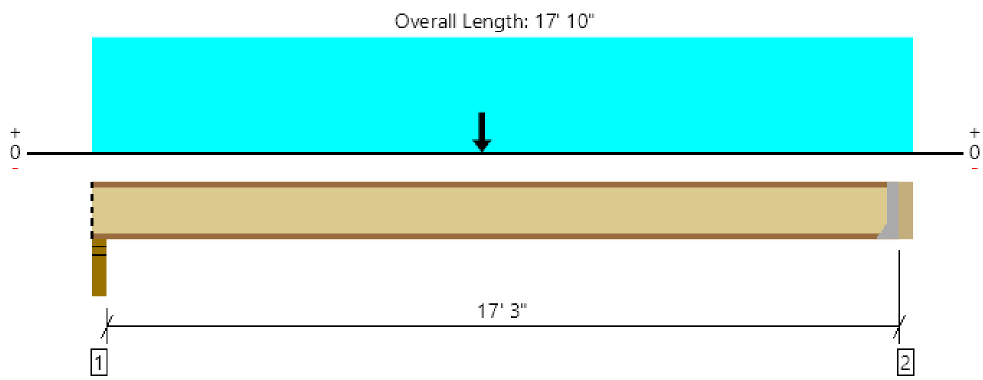
The product application, input design loads, dimensions and support information have been provided by JMB

ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

Weyerhaeuser

Roof, J3

1 piece(s) 11 7/8" TJI® 210 @ 19.2" 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)	770 @ 17' 6 1/2"	1156 (1.75")	Passed (67%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	772 @ 3 1/2"	1903	Passed (41%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4001 @ 8' 7 1/2"	4364	Passed (92%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.285 @ 8' 10 1/2"	0.578	Passed (L/730)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.714 @ 8' 7 1/2"	0.867	Passed (L/291)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.75"	438	284	355	793	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	432	287	358	791	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	3' 6" o/c	
Bottom Edge (Lu)	17' 7" 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	IUS2.06/11.88	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 Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 17' 10"	19.2"	20.0	20.0	25.0	Default Load
2 - Point (lb)	8' 7 1/2"	N/A	300	-	-	

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by JMB

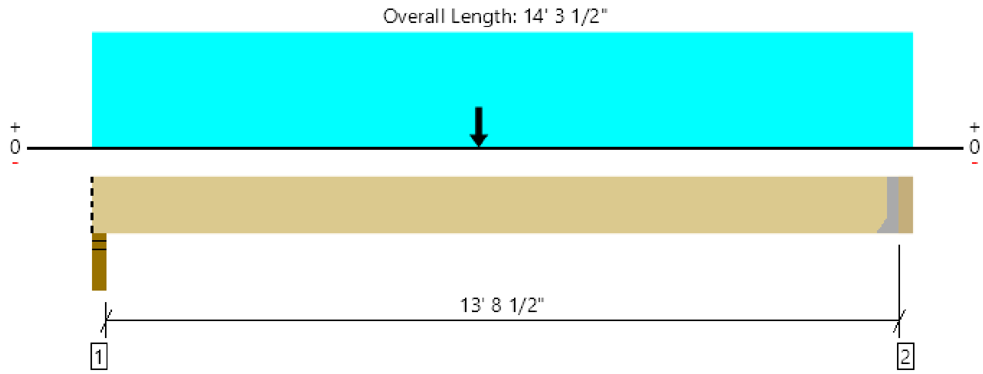
ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

MEMBER REPORT

PASSED

Roof, J6

1 piece(s) 3 1/2" x 11 7/8" 1.55E TimberStrand® LSL @ 36" 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)	1172 @ 14'	4725 (1.50")	Passed (25%)	--	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1045 @ 1' 3 3/8"	9878	Passed (11%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4927 @ 6' 10 1/4"	18346	Passed (27%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.087 @ 7' 1 1/4"	0.460	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.225 @ 7' 1"	0.690	Passed (L/736)	--	1.0 D + 1.0 S (All Spans)

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

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.50"	685	426	533	1218	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger¹	1.50"	672	431	539	1211	See note ¹

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	14' o/c	
Bottom Edge (Lu)	14' o/c	

•Maximum allowable bracing intervals based on applied load.

Connector: Simpson Strong-Tie						
Support	Model	Seat Length	Top Fasteners	Face Fasteners	Member Fasteners	Accessories
2 - Face Mount Hanger	LUS410	2.00"	N/A	8-10dx1.5	6-10d	

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

Vertical Loads	Location (Side)	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 14' 3 1/2"	36"	20.0	20.0	25.0	Default Load
2 - Point (lb)	6' 10 1/4"	N/A	500	-	-	Assumed Mech Unit Load

Weyerhaeuser Notes

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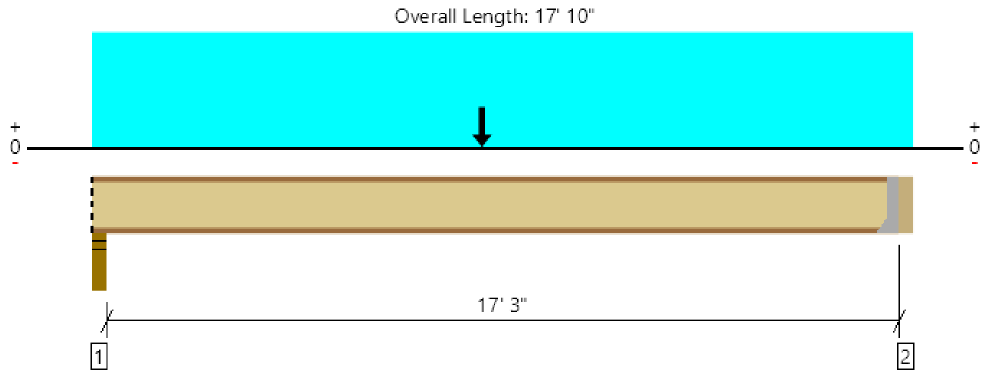
The product application, input design loads, dimensions and support information have been provided by JMB

ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

Roof, J3 - Option 2

1 piece(s) 11 7/8" TJI® 360 @ 24" 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)	926 @ 17' 6 1/2"	1242 (1.75")	Passed (75%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	927 @ 3 1/2"	1961	Passed (47%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	4676 @ 8' 7 1/2"	7107	Passed (66%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.276 @ 8' 10 1/2"	0.578	Passed (L/753)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.654 @ 8' 7 1/2"	0.867	Passed (L/318)	--	1.0 D + 1.0 S (All Spans)

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

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.75"	509	355	444	953	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger¹	1.75" / - ²	504	358	448	952	See note ¹

- Blocking Panels are assumed to carry no loads applied directly above them and the full load is applied to the member being designed.
- At hanger supports, the Total Bearing dimension is equal to the width of the material that is supporting the hanger
- ¹ See Connector grid below for additional information and/or requirements.
- ² Required Bearing Length / Required Bearing Length with Web Stiffeners

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	4' 3" o/c	
Bottom Edge (Lu)	17' 7" 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 Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 17' 10"	24"	20.0	20.0	25.0	Default Load
2 - Point (lb)	8' 7 1/2"	N/A	300	-	-	

Weyerhaeuser Notes

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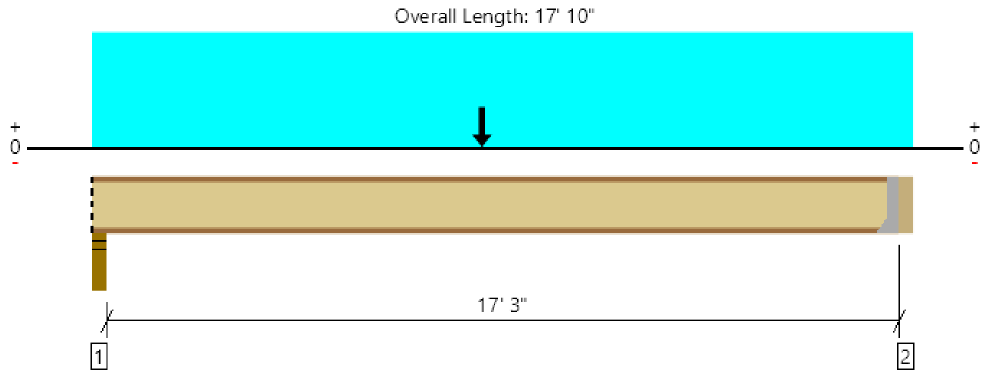
The product application, input design loads, dimensions and support information have been provided by JMB

ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

Roof, J3 Dbl - Option 2

2 piece(s) 11 7/8" TJI® 360 @ 24" 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)	1023 @ 17' 6 1/2"	2484 (1.75")	Passed (41%)	1.15	1.0 D + 1.0 S (All Spans)
Shear (lbs)	1030 @ 3 1/2"	3922	Passed (26%)	1.15	1.0 D + 1.0 S (All Spans)
Moment (Ft-lbs)	5542 @ 8' 7 1/2"	14214	Passed (39%)	1.15	1.0 D + 1.0 S (All Spans)
Live Load Defl. (in)	0.138 @ 8' 10 1/2"	0.578	Passed (L/999+)	--	1.0 D + 1.0 S (All Spans)
Total Load Defl. (in)	0.379 @ 8' 7 1/2"	0.867	Passed (L/548)	--	1.0 D + 1.0 S (All Spans)

System : Roof
Member Type : Joist
Building Use : Commercial
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

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

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Snow	Factored	
1 - Stud wall - HF	3.50"	3.50"	1.75"	612	355	444	1056	Blocking
2 - Hanger on 11 7/8" DF beam	3.50"	Hanger[1]	1.75" / - [2]	601	358	448	1049	See note [1]

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

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 8" o/c	
Bottom Edge (Lu)	17' 7" 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	MIU4.75/11	2.50"	N/A	20-10dx1.5	2-10dx1.5	

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

Vertical Loads	Location	Spacing	Dead (0.90)	Roof Live (non-snow: 1.25)	Snow (1.15)	Comments
1 - Uniform (PSF)	0 to 17' 10"	24"	20.0	20.0	25.0	Default Load
2 - Point (lb)	8' 7 1/2"	N/A	500	-	-	Assumed Mech Unit Load

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by JMB

ForteWEB Software Operator	Job Notes
Jacob Baker PCS Structural Solutions (253) 383-2797 JBaker@pcs-structural.com	Taco Time NW, Puyallup, WA

Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

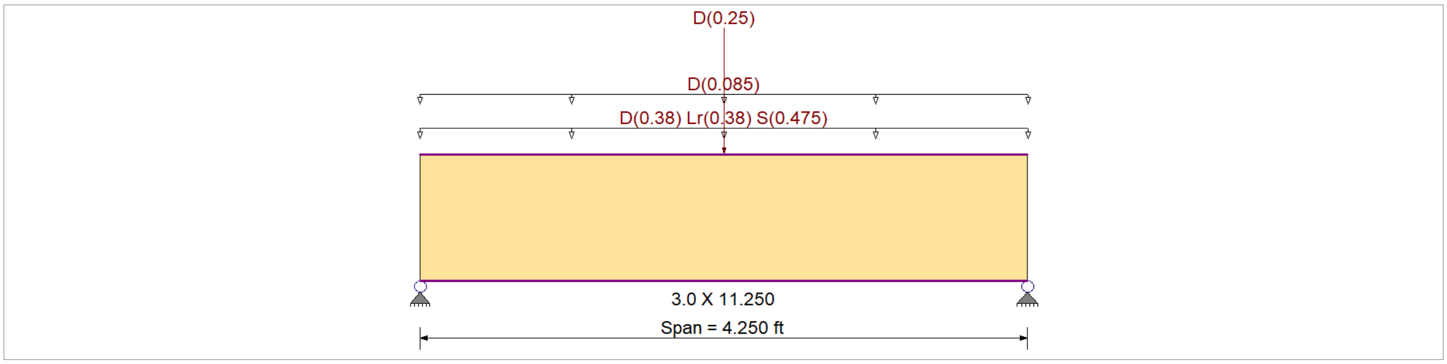
DESCRIPTION: Typical Header 1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 19.0 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 8.50 ft
Point Load : D = 0.250 k @ 2.125 ft, (Assumed Mech Unit Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.400** : 1	Maximum Shear Stress Ratio	=	**0.270** : 1
Section used for this span		**3.0 X 11.250**	Section used for this span		**3.0 X 11.250**
fb: Actual	=	455.96 psi	fv: Actual	=	55.84 psi
F'b	=	1,138.50 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	2.125 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.006 in	Ratio =	8281 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.014 in	Ratio =	3777 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.250 ft	1	0.283	0.189	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.33	252.6	891.0	0.69	30.6	162.0
+D+Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.250 ft	1	0.336	0.226	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.19	415.3	1,237.5	1.14	50.8	225.0
+D+S					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.250 ft	1	0.400	0.270	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.40	456.0	1,138.5	1.26	55.8	207.0
+D+0.750Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.250 ft	1	0.303	0.203	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.98	374.6	1,237.5	1.03	45.8	225.0
+D+0.750S					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0

DESCRIPTION: Typical Header 1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 4.250 ft	**1**	0.356	0.239	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.14	405.1	1,138.5	1.11	49.5	207.0
+0.60D					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.250 ft	**1**	0.096	0.064	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	0.80	151.6	1,584.0	0.41	18.4	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0135	2.141		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	2.138	2.138
Max Upward from Load Combinations	2.138	2.138
Max Upward from Load Cases	1.129	1.129
D Only	1.129	1.129
+D+Lr	1.936	1.936
+D+S	2.138	2.138
+D+0.750Lr	1.734	1.734
+D+0.750S	1.886	1.886
+0.60D	0.677	0.677
Lr Only	0.808	0.808
S Only	1.009	1.009

Structural Solutions

Project Title:	Taco Time NW
Engineer:	JMB
Project ID:	23514
Project Descr:	

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

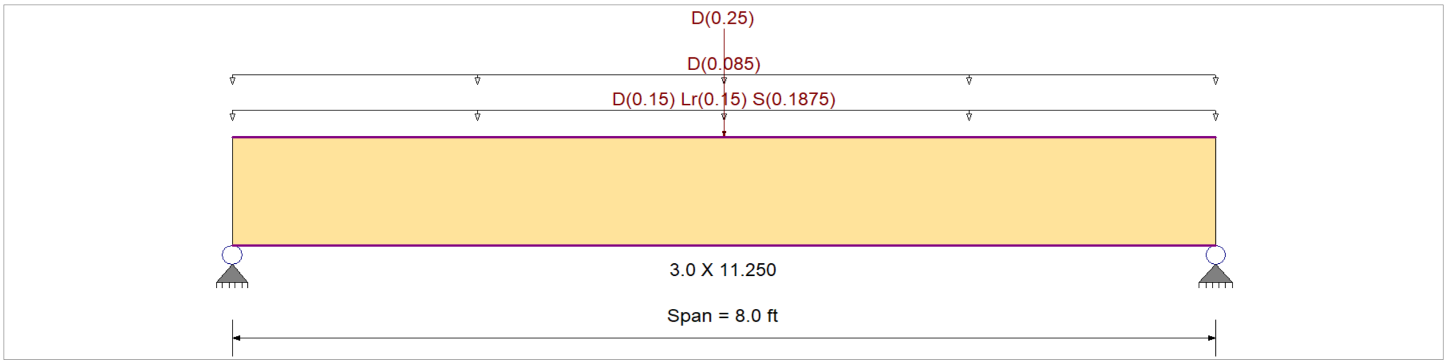
DESCRIPTION: Typical Header 2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 7.50 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 8.50 ft
Point Load : D = 0.250 k @ 4.0 ft, (Assumed Mech Unit Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.656** : 1	Maximum Shear Stress Ratio	=	**0.310** : 1
Section used for this span		**3.0 X 11.250**	Section used for this span		**3.0 X 11.250**
fb: Actual	=	746.86 psi	fv: Actual	=	64.12 psi
F'b	=	1,138.50 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	7.066 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.031	in	Ratio =	3145	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0	in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.078	in	Ratio =	1229	>=240	Span: 1 : +D+S
Max Upward Total Deflection	0	in	Ratio =	0	<240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.519	0.238	0.90	1.00	1.00	1.00	1.100	1.00	1.00	1.00	2.44	462.4	891.0	0.87	38.6	162.0
+D+Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.558	0.262	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.64	690.0	1,237.5	1.33	59.0	225.0
+D+S					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.656	0.310	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.94	746.9	1,138.5	1.44	64.1	207.0
+D+0.750Lr					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	1	0.512	0.240	1.25	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.34	633.1	1,237.5	1.21	53.9	225.0
+D+0.750S					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0

PCS Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: Typical Header 2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 8.0 ft	**1**	0.594	0.279	1.15	1.00	1.00	1.00	1.100	1.00	1.00	1.00	3.56	675.7	1,138.5	1.30	57.7	207.0
+0.60D					1.00	1.00	1.00	1.100	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.0 ft	**1**	0.175	0.080	1.60	1.00	1.00	1.00	1.100	1.00	1.00	1.00	1.46	277.4	1,584.0	0.52	23.1	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0781	4.029		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.844	1.844
Max Upward from Load Combinations	1.844	1.844
Max Upward from Load Cases	1.094	1.094
D Only	1.094	1.094
+D+Lr	1.694	1.694
+D+S	1.844	1.844
+D+0.750Lr	1.544	1.544
+D+0.750S	1.657	1.657
+0.60D	0.657	0.657
Lr Only	0.600	0.600
S Only	0.750	0.750

PCS Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 | PCS STRUCTURAL SOLUTIONS | (c) ENERCALC INC 1983-2023

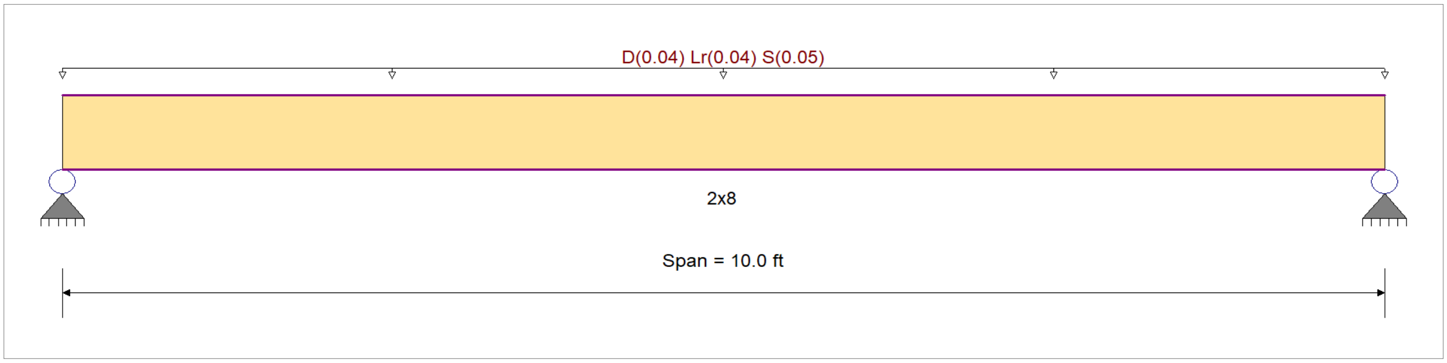
DESCRIPTION: J4

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.827**: 1	Maximum Shear Stress Ratio	=	**0.265** : 1
Section used for this span		**2x8**	Section used for this span		**2x8**
fb: Actual	=	1,027.35 psi	fv: Actual	=	54.82 psi
F'b	=	1,242.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.148	in	Ratio =	808 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0	in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.267	in	Ratio =	449 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0	in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.470	0.150	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.50	456.6	972.0	0.18	24.4	162.0
+D+Lr					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.676	0.217	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	1.00	913.2	1,350.0	0.35	48.7	225.0
+D+S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.827	0.265	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	1.13	1,027.3	1,242.0	0.40	54.8	207.0
+D+0.750Lr					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.592	0.190	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.88	799.0	1,350.0	0.31	42.6	225.0
+D+0.750S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.712	0.228	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.97	884.7	1,242.0	0.34	47.2	207.0

Wood Beam

DESCRIPTION: J4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+0.60D					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	**1**	0.159	0.051	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.30	274.0	1,728.0	0.11	14.6	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.2672	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.450	0.450
Max Upward from Load Combinations	0.450	0.450
Max Upward from Load Cases	0.250	0.250
D Only	0.200	0.200
+D+Lr	0.400	0.400
+D+S	0.450	0.450
+D+0.750Lr	0.350	0.350
+D+0.750S	0.388	0.388
+0.60D	0.120	0.120
Lr Only	0.200	0.200
S Only	0.250	0.250

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

PCS Structural Solutions

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

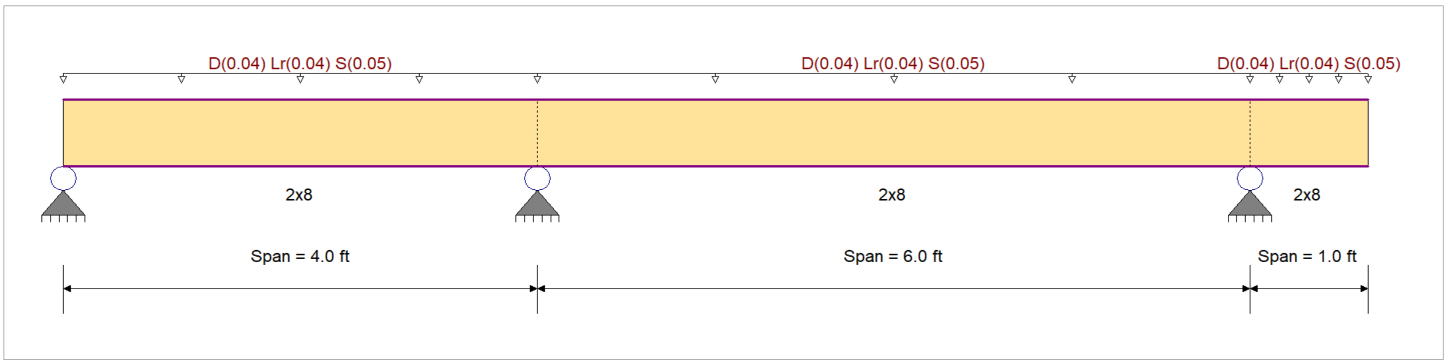
DESCRIPTION: J5

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	900.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.2	Fv	180.0 psi		
		Ft	575.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight NOT internally calculated and added
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 2.0 ft
Load for Span Number 2
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 2.0 ft
Load for Span Number 3
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 2.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.222**: 1	Maximum Shear Stress Ratio	=	**0.175** : 1
Section used for this span		**2x8**	Section used for this span		**2x8**
fb: Actual	=	275.33 psi	fv: Actual	=	36.25 psi
F'b	=	1,242.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.000 ft	Location of maximum on span	=	4.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.009 in	Ratio =	7601 >=360	Span: 2 : S Only
Max Upward Transient Deflection	-0.005 in	Ratio =	4676 >=360	Span: 3 : S Only
Max Downward Total Deflection	0.017 in	Ratio =	4223 >=240	Span: 2 : +D+S
Max Upward Total Deflection	-0.009 in	Ratio =	2598 >=240	Span: 3 : +D+S

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.126	0.099	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.13	122.4	972.0	0.12	16.1	162.0
Length = 6.0 ft	2	0.126	0.099	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.13	122.4	972.0	0.12	16.1	162.0
Length = 1.0 ft	3	0.019	0.099	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.02	18.3	972.0	0.02	16.1	162.0
+D+Lr					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	1	0.181	0.143	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.27	244.7	1,350.0	0.23	32.2	225.0

Project Title:	Taco Time NW
Engineer:	JMB
Project ID:	23514
Project Descr:	

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: J5

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 6.0 ft	**2**	0.181	0.143	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.27	244.7	1,350.0	0.23	32.2	225.0
Length = 1.0 ft	**3**	0.027	0.143	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.04	36.5	1,350.0	0.03	32.2	225.0
+D+S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.222	0.175	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.30	275.3	1,242.0	0.26	36.3	207.0
Length = 6.0 ft	**2**	0.222	0.175	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.30	275.3	1,242.0	0.26	36.3	207.0
Length = 1.0 ft	**3**	0.033	0.175	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.04	41.1	1,242.0	0.04	36.3	207.0
+D+0.750Lr					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.159	0.125	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.23	214.1	1,350.0	0.20	28.2	225.0
Length = 6.0 ft	**2**	0.159	0.125	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.23	214.1	1,350.0	0.20	28.2	225.0
Length = 1.0 ft	**3**	0.024	0.125	1.25	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.03	32.0	1,350.0	0.03	28.2	225.0
+D+0.750S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.191	0.151	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.26	237.1	1,242.0	0.23	31.2	207.0
Length = 6.0 ft	**2**	0.191	0.151	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.26	237.1	1,242.0	0.23	31.2	207.0
Length = 1.0 ft	**3**	0.028	0.151	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.04	35.4	1,242.0	0.03	31.2	207.0
+0.60D					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 4.0 ft	**1**	0.042	0.034	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.08	73.4	1,728.0	0.07	9.7	288.0
Length = 6.0 ft	**2**	0.042	0.034	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.08	73.4	1,728.0	0.07	9.7	288.0
Length = 1.0 ft	**3**	0.006	0.034	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.01	11.0	1,728.0	0.01	9.7	288.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0006	1.042	+D+S	-0.0012	3.261
+D+S	2	0.0170	3.277		0.0000	3.261
	3	0.0000	3.277	+D+S	-0.0092	1.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	0.105	0.568	0.317	
Max Upward from Load Combinations	0.105	0.568	0.317	
Max Upward from Load Cases	0.058	0.316	0.176	
D Only	0.047	0.252	0.141	
+D+Lr	0.093	0.505	0.282	
+D+S	0.105	0.568	0.317	
+D+0.750Lr	0.081	0.442	0.247	
+D+0.750S	0.090	0.489	0.273	
+0.60D	0.028	0.151	0.085	
Lr Only	0.047	0.252	0.141	
S Only	0.058	0.316	0.176	

Wood Beam

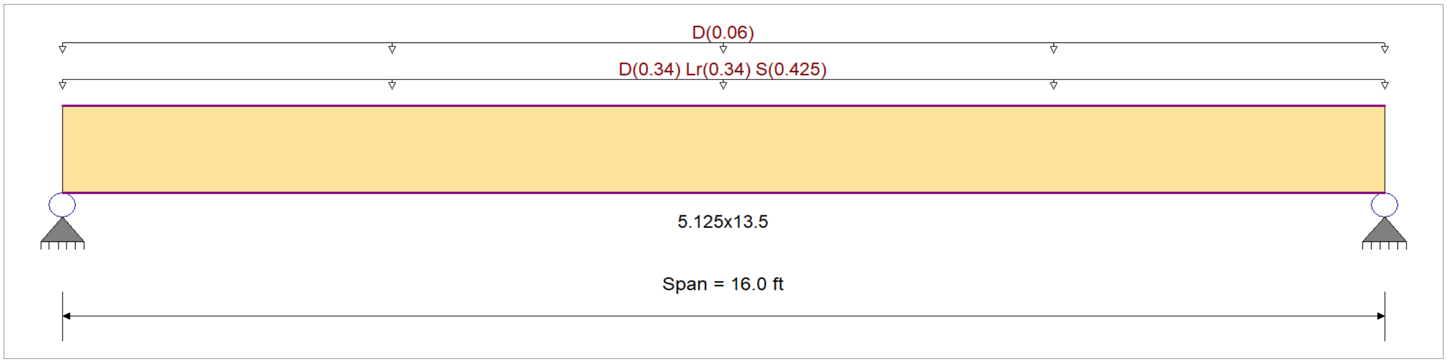
DESCRIPTION: B1

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 17.0 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.751**: 1	Maximum Shear Stress Ratio	=	**0.412** : 1
Section used for this span		**5.125x13.5**	Section used for this span		**5.125x13.5**
fb: Actual	=	2,072.04 psi	fv: Actual	=	125.49 psi
F'b	=	2,760.00 psi	F'v	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	8.000 ft	Location of maximum on span	=	14.891 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.333 in	Ratio =	576 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.659 in	Ratio =	291 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.474	0.260	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	13.28	1,023.7	2,160.0	2.86	62.0	238.5
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.621	0.340	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	24.16	1,862.4	3,000.0	5.20	112.8	331.3
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.751	0.412	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	26.88	2,072.0	2,760.0	5.79	125.5	304.8
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.551	0.302	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	21.44	1,652.7	3,000.0	4.62	100.1	331.3
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

PCS Structural Solutions

Project Title:	Taco Time NW
Engineer:	JMB
Project ID:	23514
Project Descr:	

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: B1

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 16.0 ft	**1**	0.656	0.360	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	23.48	1,809.9	2,760.0	5.06	109.6	304.8
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.160	0.088	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.97	614.2	3,840.0	1.72	37.2	424.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.6587	8.058		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	6.720	6.720
Max Upward from Load Combinations	6.720	6.720
Max Upward from Load Cases	3.400	3.400
D Only	3.320	3.320
+D+Lr	6.040	6.040
+D+S	6.720	6.720
+D+0.750Lr	5.360	5.360
+D+0.750S	5.870	5.870
+0.60D	1.992	1.992
Lr Only	2.720	2.720
S Only	3.400	3.400

PCS Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS

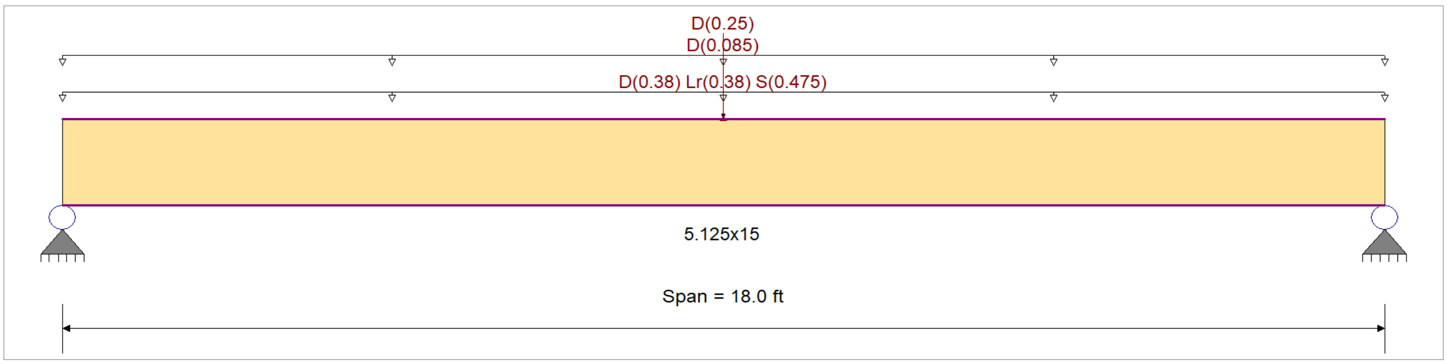
DESCRIPTION: B2

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 19.0 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 8.50 ft
Point Load : D = 0.250 k @ 9.0 ft, (Assumed Mech Unit Load)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.908**: 1	Maximum Shear Stress Ratio	=	**0.483** : 1
Section used for this span		**5.125x15**	Section used for this span		**5.125x15**
fb: Actual	=	2,489.43 psi	fv: Actual	=	147.14 psi
F'b	=	2,741.02 psi	F'v	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	9.000 ft	Location of maximum on span	=	16.752 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.435 in	Ratio =	496 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.896 in	Ratio =	240 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.601	0.316	0.90	1.00	1.00	1.00	0.993	1.00	1.00	1.00	20.63	1,288.3	2,145.1	3.86	75.3	238.5
+D+Lr					1.00	1.00	1.00	0.993	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.755	0.401	1.25	1.00	1.00	1.00	0.993	1.00	1.00	1.00	36.02	2,249.2	2,979.4	6.80	132.8	331.3
+D+S					1.00	1.00	1.00	0.993	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.908	0.483	1.15	1.00	1.00	1.00	0.993	1.00	1.00	1.00	39.87	2,489.4	2,741.0	7.54	147.1	304.8
+D+0.750Lr					1.00	1.00	1.00	0.993	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.674	0.357	1.25	1.00	1.00	1.00	0.993	1.00	1.00	1.00	32.17	2,009.0	2,979.4	6.07	118.4	331.3
+D+0.750S					1.00	1.00	1.00	0.993	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B2

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 18.0 ft	**1**	0.799	0.424	1.15	1.00	1.00	1.00	0.993	1.00	1.00	1.00	35.06	2,189.1	2,741.0	6.62	129.2	304.8
+0.60D					1.00	1.00	1.00	0.993	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.203	0.107	1.60	1.00	1.00	1.00	0.993	1.00	1.00	1.00	12.38	773.0	3,813.6	2.32	45.2	424.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.8963	9.066		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.735	8.735
Max Upward from Load Combinations	8.735	8.735
Max Upward from Load Cases	4.460	4.460
D Only	4.460	4.460
+D+Lr	7.880	7.880
+D+S	8.735	8.735
+D+0.750Lr	7.025	7.025
+D+0.750S	7.666	7.666
+0.60D	2.676	2.676
Lr Only	3.420	3.420
S Only	4.275	4.275

Wood Beam

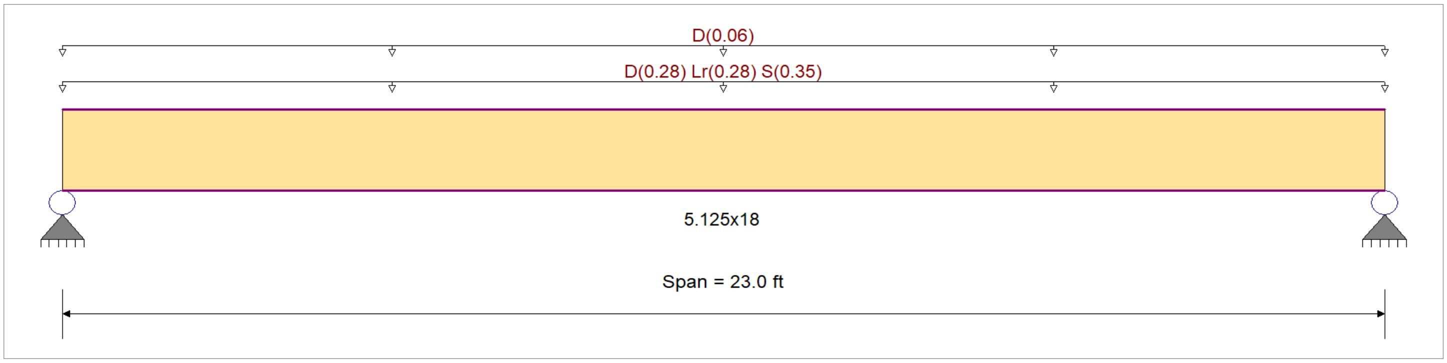
DESCRIPTION: B3

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 14.0 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 6.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.775** : 1	Maximum Shear Stress Ratio	=	**0.382** : 1
Section used for this span		**5.125x18**	Section used for this span		**5.125x18**
fb: Actual	=	2,035.70 psi	fv: Actual	=	116.29 psi
F'b	=	2,626.33 psi	F'v	=	304.75 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	11.500 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.494 in	Ratio =	558	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.003 in	Ratio =	275	>=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 23.0 ft	1	0.502	0.247	0.90	1.00	1.00	1.00	0.952	1.00	1.00	1.00	23.80	1,032.2	2,055.4	3.63	59.0	238.5
+D+Lr					1.00	1.00	1.00	0.952	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 23.0 ft	1	0.643	0.316	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	42.32	1,835.0	2,854.7	6.45	104.8	331.3
+D+S					1.00	1.00	1.00	0.952	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 23.0 ft	1	0.775	0.382	1.15	1.00	1.00	1.00	0.952	1.00	1.00	1.00	46.95	2,035.7	2,626.3	7.15	116.3	304.8
+D+0.750Lr					1.00	1.00	1.00	0.952	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 23.0 ft	1	0.572	0.282	1.25	1.00	1.00	1.00	0.952	1.00	1.00	1.00	37.69	1,634.3	2,854.7	5.74	93.4	331.3
+D+0.750S					1.00	1.00	1.00	0.952	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B3

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 23.0 ft	**1**	0.680	0.335	1.15	1.00	1.00	1.00	0.952	1.00	1.00	1.00	41.16	1,784.8	2,626.3	6.27	102.0	304.8
+0.60D					1.00	1.00	1.00	0.952	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 23.0 ft	**1**	0.169	0.083	1.60	1.00	1.00	1.00	0.952	1.00	1.00	1.00	14.28	619.3	3,654.0	2.18	35.4	424.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.0029	11.584		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	8.165	8.165
Max Upward from Load Combinations	8.165	8.165
Max Upward from Load Cases	4.140	4.140
D Only	4.140	4.140
+D+Lr	7.360	7.360
+D+S	8.165	8.165
+D+0.750Lr	6.555	6.555
+D+0.750S	7.159	7.159
+0.60D	2.484	2.484
Lr Only	3.220	3.220
S Only	4.025	4.025

PCS Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 | PCS STRUCTURAL SOLUTIONS |

DESCRIPTION: B4

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

Wood Species : DF/DF
Wood Grade : 24F-V4

Beam Bracing : Beam bracing is defined Beam-by-Beam

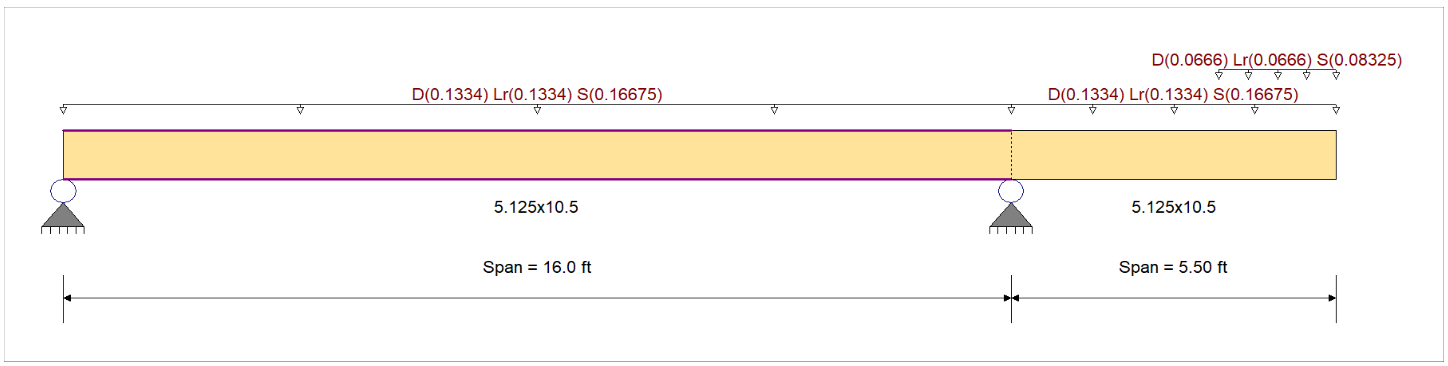
Fb +	2,400.0	psi
Fb -	1,850.0	psi
Fc - Prll	1,650.0	psi
Fc - Perp	650.0	psi
Fv	265.0	psi
Ft	1,100.0	psi

E : Modulus of Elasticity

Ebend- xx	1,800.0	ksi
Eminbend - xx	950.0	ksi
Ebend- yy	1,600.0	ksi
Eminbend - yy	850.0	ksi
Density	31.210	pcf

Unbraced Lengths

Span # 1, Fully Braced
Span # 2, Defined Brace Locations, First Brace at ft, Second Brace at ft, Third Brace at ft



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 6.670 ft
Load for Span Number 2
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 6.670 ft
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Extent = 3.50 -->> 5.50 ft, Tributary Width = 3.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.365** : 1	Maximum Shear Stress Ratio	=	**0.240** : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	772.82 psi	fv: Actual	=	73.11 psi
F'b	=	2,115.63 psi	F'v	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	0.000 ft	Location of maximum on span	=	15.196 ft
Span # where maximum occurs	=	Span # 2	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.189 in	Ratio =	698	>=360	Span: 2 : Lr Only, LL Comb Run (*L)
Max Upward Transient Deflection	-0.243 in	Ratio =	542	>=360	Span: 2 : Lr Only, LL Comb Run (L*)
Max Downward Total Deflection	0.379 in	Ratio =	507	>=240	Span: 2 : +D+Lr+H, LL Comb Run (*L)
Max Upward Total Deflection	-0.306 in	Ratio =	430	>=240	Span: 2 : +D+Lr+H, LL Comb Run (L*)

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+H														0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.198	0.142	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	2,160.0	1.22	34.0	238.5
Length = 5.50 ft	**2**	0.215	0.142	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.79	356.0	1,657.9	0.81	34.0	238.5
+D+L+H, LL Comb Run (*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 16.0 ft	**1**	0.178	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	2,400.0	1.22	34.0	265.0
Length = 5.50 ft	**2**	0.193	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.79	356.0	1,841.1	0.81	34.0	265.0
+D+L+H, LL Comb Run (L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.178	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	2,400.0	1.22	34.0	265.0
Length = 5.50 ft	**2**	0.193	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.79	356.0	1,841.1	0.81	34.0	265.0
+D+L+H, LL Comb Run (LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.178	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	2,400.0	1.22	34.0	265.0
Length = 5.50 ft	**2**	0.193	0.128	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.79	356.0	1,841.1	0.81	34.0	265.0
+D+Lr+H, LL Comb Run (*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.298	0.131	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.41	689.4	2,312.5	1.56	43.4	331.3
Length = 5.50 ft	**2**	0.300	0.131	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.41	689.4	2,298.4	1.56	43.4	331.3
+D+Lr+H, LL Comb Run (L*)					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.321	0.183	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.57	964.4	3,000.0	2.18	60.7	331.3
Length = 5.50 ft	**2**	0.155	0.183	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.79	356.0	2,298.4	0.81	60.7	331.3
+D+Lr+H, LL Comb Run (LL)					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.272	0.197	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.41	816.9	3,000.0	2.34	65.3	331.3
Length = 5.50 ft	**2**	0.300	0.197	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.41	689.4	2,298.4	1.56	65.3	331.3
+D+S+H					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.331	0.240	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.18	914.4	2,760.0	2.62	73.1	304.8
Length = 5.50 ft	**2**	0.365	0.240	1.15	1.00	1.00	0.99	1.000	1.00	1.00	1.00	6.06	772.8	2,115.6	1.75	73.1	304.8
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.262	0.115	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.76	606.1	2,312.5	1.37	38.2	331.3
Length = 5.50 ft	**2**	0.264	0.115	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	4.76	606.1	2,298.4	1.37	38.2	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.276	0.163	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.51	829.4	3,000.0	1.94	54.0	331.3
Length = 5.50 ft	**2**	0.155	0.163	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.79	356.0	2,298.4	0.81	54.0	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.240	0.173	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.65	719.4	3,000.0	2.06	57.4	331.3
Length = 5.50 ft	**2**	0.264	0.173	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	4.76	606.1	2,298.4	1.37	57.4	331.3
+D+0.750L+0.750S+H, LL Co					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.287	0.208	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	2,760.0	2.27	63.3	304.8
Length = 5.50 ft	**2**	0.316	0.208	1.15	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,115.6	1.51	63.3	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.287	0.208	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	2,760.0	2.27	63.3	304.8
Length = 5.50 ft	**2**	0.316	0.208	1.15	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,115.6	1.51	63.3	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.287	0.208	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	2,760.0	2.27	63.3	304.8
Length = 5.50 ft	**2**	0.316	0.208	1.15	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,115.6	1.51	63.3	304.8
+D+0.60W+H					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.111	0.080	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	3,840.0	1.22	34.0	424.0
Length = 5.50 ft	**2**	0.121	0.080	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.79	356.0	2,936.1	0.81	34.0	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.205	0.090	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.76	606.1	2,960.0	1.37	38.2	424.0
Length = 5.50 ft	**2**	0.206	0.090	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	4.76	606.1	2,936.1	1.37	38.2	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	**1**	0.216	0.127	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.51	829.4	3,840.0	1.94	54.0	424.0
Length = 5.50 ft	**2**	0.121	0.127	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.79	356.0	2,936.1	0.81	54.0	424.0

Wood Beam

DESCRIPTION: B4

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.187	0.135	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	5.65	719.4	3,840.0	2.06	57.4	424.0
Length = 5.50 ft	2	0.206	0.135	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	4.76	606.1	2,936.1	1.37	57.4	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+0.60D+0.60W+0.60H					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.067	0.048	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.01	256.2	3,840.0	0.73	20.4	424.0
Length = 5.50 ft	2	0.073	0.048	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.68	213.6	2,936.1	0.48	20.4	424.0
+D+0.70E+0.60H					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.111	0.080	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.35	426.9	3,840.0	1.22	34.0	424.0
Length = 5.50 ft	2	0.121	0.080	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.79	356.0	2,936.1	0.81	34.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.206	0.149	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.22	792.5	3,840.0	2.27	63.3	424.0
Length = 5.50 ft	2	0.228	0.149	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	5.25	668.6	2,936.1	1.51	63.3	424.0
+0.60D+0.70E+H					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 16.0 ft	1	0.067	0.048	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.01	256.2	3,840.0	0.73	20.4	424.0
Length = 5.50 ft	2	0.073	0.048	1.60	1.00	1.00	0.99	1.000	1.00	1.00	1.00	1.68	213.6	2,936.1	0.48	20.4	424.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr+H, LL Comb Run (L*)	1	0.3785	7.777		0.0000	0.000
	2	0.0000	7.777	+D+Lr+H, LL Comb Run (L*)	-0.3059	5.500

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Max Upward from all Load Conditions	2.115	4.888	
Max Upward from Load Combinations	2.115	4.888	
Max Upward from Load Cases	1.130	2.622	
Max Downward from all Load Conditio	-0.164		
Max Downward from Load Cases (Resis	-0.164		
+D+H	0.986	2.266	
+D+L+H, LL Comb Run (*L)	0.986	2.266	
+D+L+H, LL Comb Run (L*)	0.986	2.266	
+D+L+H, LL Comb Run (LL)	0.986	2.266	
+D+Lr+H, LL Comb Run (*L)	0.822	3.297	
+D+Lr+H, LL Comb Run (L*)	2.053	3.333	
+D+Lr+H, LL Comb Run (LL)	1.890	4.364	
+D+S+H	2.115	4.888	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.863	3.039	

Wood Beam

DESCRIPTION: B4

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.786	3.067	
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.664	3.839	
+D+0.750L+0.750S+H, LL Comb Run (*L	1.833	4.233	
+D+0.750L+0.750S+H, LL Comb Run (L*	1.833	4.233	
+D+0.750L+0.750S+H, LL Comb Run (LL	1.833	4.233	
+D+0.60W+H	0.986	2.266	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.863	3.039	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.786	3.067	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.664	3.839	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.833	4.233	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.833	4.233	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.833	4.233	
+0.60D+0.60W+0.60H	0.592	1.360	
+D+0.70E+0.60H	0.986	2.266	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.833	4.233	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.833	4.233	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.833	4.233	
+0.60D+0.70E+H	0.592	1.360	
D Only	0.986	2.266	
Lr Only, LL Comb Run (*L)	-0.164	1.030	
Lr Only, LL Comb Run (L*)	1.067	1.067	
Lr Only, LL Comb Run (LL)	0.904	2.098	
S Only	1.130	2.622	
H Only			

Wood Beam

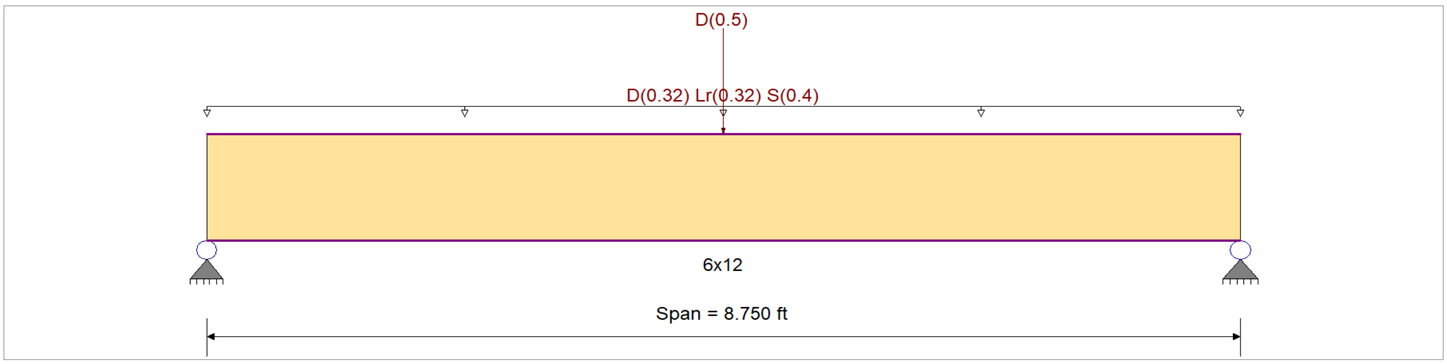
DESCRIPTION: B5

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.210 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 16.0 ft
Point Load : D = 0.50 k @ 4.375 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.517**: 1	Maximum Shear Stress Ratio	=	**0.334** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb: Actual	=	803.33 psi	fv: Actual	=	65.38 psi
F'b	=	1,552.50 psi	F'v	=	195.50 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	4.375 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.048 in	Ratio =	2206 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.098 in	Ratio =	1069 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 8.750 ft	1	0.349	0.215	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.29	424.4	1,215.0	1.39	33.0	153.0
+D+Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.750 ft	1	0.431	0.277	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	727.5	1,687.5	2.48	58.9	212.5
+D+S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.750 ft	1	0.517	0.334	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.12	803.3	1,552.5	2.76	65.4	195.5
+D+0.750Lr					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.750 ft	1	0.386	0.247	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.58	651.8	1,687.5	2.21	52.4	212.5
+D+0.750S					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B5

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 8.750 ft	**1**	0.456	0.293	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.16	708.6	1,552.5	2.42	57.3	195.5
+0.60D					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 8.750 ft	**1**	0.118	0.073	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.57	254.6	2,160.0	0.83	19.8	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0981	4.407		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	3.460	3.460
Max Upward from Load Combinations	3.460	3.460
Max Upward from Load Cases	1.750	1.750
D Only	1.710	1.710
+D+Lr	3.110	3.110
+D+S	3.460	3.460
+D+0.750Lr	2.760	2.760
+D+0.750S	3.022	3.022
+0.60D	1.026	1.026
Lr Only	1.400	1.400
S Only	1.750	1.750

Wood Beam

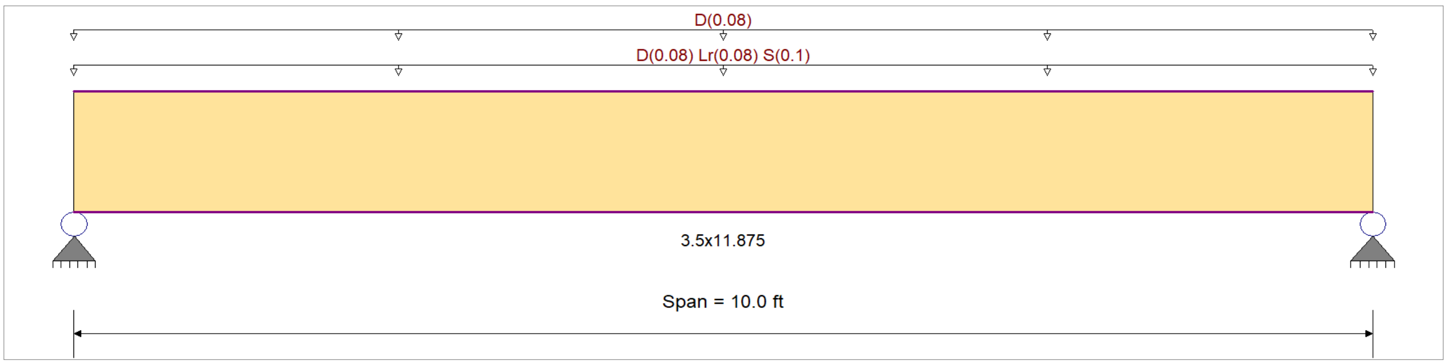
DESCRIPTION: B6

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1700 psi	*E : Modulus of Elasticity*		
Load Combination :	ASCE 7-16	Fb -	1700 psi	Ebend- xx	1300	ksi
		Fc - Prll	1400 psi	Eminbend - xx	660.75	ksi
Wood Species :	iLevel Truss Joist	Fc - Perp	680 psi			
Wood Grade :	TimberStrand LSL 1.3E - Beam/Col	Fv	400 psi			
		Ft	1075 psi	Density	45.01	pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling					



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 4.0 ft
Uniform Load : D = 0.010 ksf, Tributary Width = 8.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.254 : 1	Maximum Shear Stress Ratio	=	0.086 : 1
Section used for this span		**3.5x11.875**	Section used for this span		**3.5x11.875**
fb: Actual	=	497.80 psi	fv: Actual	=	39.55 psi
F'b	=	1,957.27 psi	F'v	=	460.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.036 in	Ratio =	3366	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.097 in	Ratio =	1233	>=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.206	0.070	0.90	1.00	1.00	1.00	1.001	1.00	1.00	1.00	2.16	315.5	1,531.8	0.69	25.1	360.0
+D+Lr					1.00	1.00	1.00	1.001	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.217	0.073	1.25	1.00	1.00	1.00	1.001	1.00	1.00	1.00	3.16	461.3	2,127.5	1.02	36.7	500.0
+D+S					1.00	1.00	1.00	1.001	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.254	0.086	1.15	1.00	1.00	1.00	1.001	1.00	1.00	1.00	3.41	497.8	1,957.3	1.10	39.6	460.0
+D+0.750Lr					1.00	1.00	1.00	1.001	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	1	0.200	0.068	1.25	1.00	1.00	1.00	1.001	1.00	1.00	1.00	2.91	424.9	2,127.5	0.94	33.8	500.0
+D+0.750S					1.00	1.00	1.00	1.001	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B6

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 10.0 ft	**1**	0.231	0.078	1.15	1.00	1.00	1.00	1.001	1.00	1.00	1.00	3.10	452.2	1,957.3	1.00	35.9	460.0
+0.60D					1.00	1.00	1.00	1.001	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 10.0 ft	**1**	0.070	0.023	1.60	1.00	1.00	1.00	1.001	1.00	1.00	1.00	1.30	189.3	2,723.2	0.42	15.0	640.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0973	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.365	1.365
Max Upward from Load Combinations	1.365	1.365
Max Upward from Load Cases	0.865	0.865
D Only	0.865	0.865
+D+Lr	1.265	1.265
+D+S	1.365	1.365
+D+0.750Lr	1.165	1.165
+D+0.750S	1.240	1.240
+0.60D	0.519	0.519
Lr Only	0.400	0.400
S Only	0.500	0.500

Structural Solutions

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: B7

CODE REFERENCES

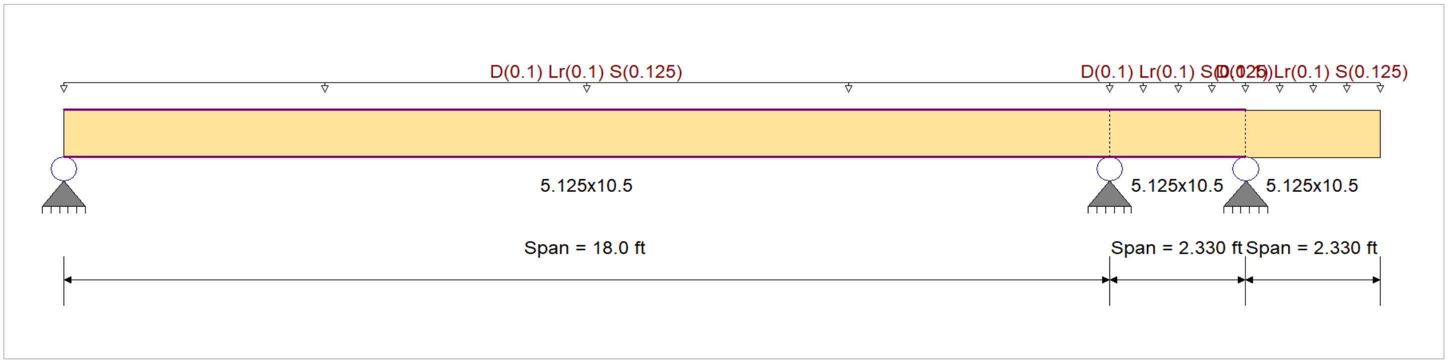
Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	1,850.0 psi	Ebend- xx	1,800.0 ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0 ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0 ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0 ksi
		Ft	1,100.0 psi	Density	31.210 pcf
Beam Bracing :	Beam bracing is defined Beam-by-Beam				

Unbraced Lengths

Span # 1, Fully Braced
Span # 2, Fully Braced
Span # 3, Defined Brace Locations, First Brace at ft, Second Brace at ft, Third Brace at ft



Applied Loads

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

Beam self weight calculated and added to loading
Load for Span Number 1
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 5.0 ft
Load for Span Number 2
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 5.0 ft
Load for Span Number 3
Uniform Load : D = 0.020, Lr = 0.020, S = 0.0250 ksf, Tributary Width = 5.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.507**: 1	Maximum Shear Stress Ratio	=	**0.314** : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	1,079.04 psi	fv: Actual	=	95.62 psi
F'b	=	2,127.50 psi	F'v	=	304.75 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	18.000 ft	Location of maximum on span	=	18.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.162 in	Ratio =	1332 >=360	Span: 3 : S Only
Max Upward Transient Deflection	-0.003 in	Ratio =	8712 >=360	Span: 3 : Lr Only, LL Comb Run (*L*)
Max Downward Total Deflection	0.307 in	Ratio =	703 >=240	Span: 3 : +D+S+H
Max Upward Total Deflection	-0.006 in	Ratio =	4601 >=240	Span: 2 : +D+S+H

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+H														0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.306	0.189	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,665.0	1.62	45.1	238.5
Length = 2.330 ft	2	0.306	0.189	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,665.0	1.62	45.1	238.5

DESCRIPTION: B7

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 2.330 ft	3	0.023	0.189	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,662.1	0.16	45.1	238.5
+D+L+H, LL Comb Run (**L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (*L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (*LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (L**)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (L*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (LL*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+L+H, LL Comb Run (LLL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	2	0.275	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	1,850.0	1.62	45.1	265.0
Length = 2.330 ft	3	0.021	0.170	1.00	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	1,846.4	0.16	45.1	265.0
+D+Lr+H, LL Comb Run (**L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.219	0.126	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.1	2,312.5	1.50	41.7	331.3
Length = 2.330 ft	2	0.219	0.126	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.1	2,312.5	1.50	41.7	331.3
Length = 2.330 ft	3	0.032	0.126	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	73.2	2,306.9	0.31	41.7	331.3
+D+Lr+H, LL Comb Run (*L*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.221	0.139	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	510.1	2,312.5	1.65	46.1	331.3
Length = 2.330 ft	2	0.221	0.139	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	510.1	2,312.5	1.65	46.1	331.3
Length = 2.330 ft	3	0.017	0.139	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	46.1	331.3
+D+Lr+H, LL Comb Run (*LL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.220	0.129	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.1	2,312.5	1.53	42.6	331.3
Length = 2.330 ft	2	0.220	0.129	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.1	2,312.5	1.53	42.6	331.3
Length = 2.330 ft	3	0.032	0.129	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	73.2	2,306.9	0.31	42.6	331.3
+D+Lr+H, LL Comb Run (L**)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.418	0.266	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.58	966.0	2,312.5	3.16	88.0	331.3
Length = 2.330 ft	2	0.418	0.266	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.58	966.0	2,312.5	3.16	88.0	331.3
Length = 2.330 ft	3	0.017	0.266	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	88.0	331.3
+D+Lr+H, LL Comb Run (L*L)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.417	0.255	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.57	964.1	2,312.5	3.03	84.6	331.3

Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Structural Solutions

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: B7

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 2.330 ft	**2**	0.417	0.255	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.57	964.1	2,312.5	3.03	84.6	331.3
Length = 2.330 ft	**3**	0.032	0.255	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	73.2	2,306.9	0.31	84.6	331.3
+D+Lr+H, LL Comb Run (LL*)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.418	0.269	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.59	967.0	2,312.5	3.19	89.0	331.3
Length = 2.330 ft	**2**	0.418	0.269	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.59	967.0	2,312.5	3.19	89.0	331.3
Length = 2.330 ft	**3**	0.017	0.269	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	89.0	331.3
+D+Lr+H, LL Comb Run (LLL)					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.417	0.258	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.57	965.1	2,312.5	3.07	85.5	331.3
Length = 2.330 ft	**2**	0.417	0.258	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.57	965.1	2,312.5	3.07	85.5	331.3
Length = 2.330 ft	**3**	0.032	0.258	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.57	73.2	2,306.9	0.31	85.5	331.3
+D+S+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.507	0.314	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.47	1,079.0	2,127.5	3.43	95.6	304.8
Length = 2.330 ft	**2**	0.507	0.314	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	8.47	1,079.0	2,127.5	3.43	95.6	304.8
Length = 2.330 ft	**3**	0.039	0.314	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.64	81.9	2,122.7	0.35	95.6	304.8
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.220	0.128	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.6	2,312.5	1.53	42.5	331.3
Length = 2.330 ft	**2**	0.220	0.128	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.6	2,312.5	1.53	42.5	331.3
Length = 2.330 ft	**3**	0.028	0.128	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,306.9	0.27	42.5	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.220	0.138	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.9	2,312.5	1.64	45.8	331.3
Length = 2.330 ft	**2**	0.220	0.138	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.9	2,312.5	1.64	45.8	331.3
Length = 2.330 ft	**3**	0.017	0.138	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	45.8	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.220	0.131	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.4	2,312.5	1.55	43.2	331.3
Length = 2.330 ft	**2**	0.220	0.131	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.4	2,312.5	1.55	43.2	331.3
Length = 2.330 ft	**3**	0.028	0.131	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,306.9	0.27	43.2	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.368	0.233	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.8	2,312.5	2.77	77.3	331.3
Length = 2.330 ft	**2**	0.368	0.233	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.8	2,312.5	2.77	77.3	331.3
Length = 2.330 ft	**3**	0.017	0.233	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	77.3	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.368	0.226	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.67	850.3	2,312.5	2.68	74.7	331.3
Length = 2.330 ft	**2**	0.368	0.226	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.67	850.3	2,312.5	2.68	74.7	331.3
Length = 2.330 ft	**3**	0.028	0.226	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,306.9	0.27	74.7	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.369	0.235	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.69	852.6	2,312.5	2.80	78.0	331.3
Length = 2.330 ft	**2**	0.369	0.235	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.69	852.6	2,312.5	2.80	78.0	331.3
Length = 2.330 ft	**3**	0.017	0.235	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,306.9	0.16	78.0	331.3
+D+0.750Lr+0.750L+H, LL Cc					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.368	0.228	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.1	2,312.5	2.71	75.4	331.3
Length = 2.330 ft	**2**	0.368	0.228	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.1	2,312.5	2.71	75.4	331.3
Length = 2.330 ft	**3**	0.028	0.228	1.25	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,306.9	0.27	75.4	331.3
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0

Wood Beam

DESCRIPTION: B7

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.750L+0.750S+H, LL Co					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**2**	0.440	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,127.5	2.98	83.0	304.8
Length = 2.330 ft	**3**	0.033	0.272	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,122.7	0.30	83.0	304.8
+D+0.60W+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.172	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	2,960.0	1.62	45.1	424.0
Length = 2.330 ft	**2**	0.172	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	2,960.0	1.62	45.1	424.0
Length = 2.330 ft	**3**	0.013	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,950.7	0.16	45.1	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.171	0.100	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.6	2,960.0	1.53	42.5	424.0
Length = 2.330 ft	**2**	0.171	0.100	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.98	507.6	2,960.0	1.53	42.5	424.0
Length = 2.330 ft	**3**	0.022	0.100	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,950.7	0.27	42.5	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.172	0.108	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.9	2,960.0	1.64	45.8	424.0
Length = 2.330 ft	**2**	0.172	0.108	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.9	2,960.0	1.64	45.8	424.0
Length = 2.330 ft	**3**	0.013	0.108	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,950.7	0.16	45.8	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.172	0.102	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.4	2,960.0	1.55	43.2	424.0
Length = 2.330 ft	**2**	0.172	0.102	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	3.99	508.4	2,960.0	1.55	43.2	424.0
Length = 2.330 ft	**3**	0.022	0.102	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,950.7	0.27	43.2	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.288	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.8	2,960.0	2.77	77.3	424.0
Length = 2.330 ft	**2**	0.288	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.8	2,960.0	2.77	77.3	424.0
Length = 2.330 ft	**3**	0.013	0.182	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,950.7	0.16	77.3	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.287	0.176	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.67	850.3	2,960.0	2.68	74.7	424.0
Length = 2.330 ft	**2**	0.287	0.176	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.67	850.3	2,960.0	2.68	74.7	424.0
Length = 2.330 ft	**3**	0.022	0.176	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,950.7	0.27	74.7	424.0

Wood Beam

DESCRIPTION: B7

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.288	0.184	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.69	852.6	2,960.0	2.80	78.0	424.0
Length = 2.330 ft	2	0.288	0.184	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.69	852.6	2,960.0	2.80	78.0	424.0
Length = 2.330 ft	3	0.013	0.184	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,950.7	0.16	78.0	424.0
+D+0.750Lr+0.750L+0.450W-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.288	0.178	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.1	2,960.0	2.71	75.4	424.0
Length = 2.330 ft	2	0.288	0.178	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.68	851.1	2,960.0	2.71	75.4	424.0
Length = 2.330 ft	3	0.022	0.178	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.51	64.6	2,950.7	0.27	75.4	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.450W+					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	3	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+0.60D+0.60W+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.103	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.40	305.5	2,960.0	0.97	27.1	424.0
Length = 2.330 ft	2	0.103	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.40	305.5	2,960.0	0.97	27.1	424.0
Length = 2.330 ft	3	0.008	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.18	23.2	2,950.7	0.10	27.1	424.0
+D+0.70E+0.60H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.172	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	2,960.0	1.62	45.1	424.0
Length = 2.330 ft	2	0.172	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.00	509.1	2,960.0	1.62	45.1	424.0
Length = 2.330 ft	3	0.013	0.106	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.30	38.6	2,950.7	0.16	45.1	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	1	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	2	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0

DESCRIPTION: B7

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_V	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+D+0.750L+0.750S+0.5250E-					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**2**	0.316	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.35	936.6	2,960.0	2.98	83.0	424.0
Length = 2.330 ft	**3**	0.024	0.196	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.56	71.1	2,950.7	0.30	83.0	424.0
+0.60D+0.70E+H					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 18.0 ft	**1**	0.103	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.40	305.5	2,960.0	0.97	27.1	424.0
Length = 2.330 ft	**2**	0.103	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.40	305.5	2,960.0	0.97	27.1	424.0
Length = 2.330 ft	**3**	0.008	0.064	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.18	23.2	2,950.7	0.10	27.1	424.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3070	7.866		0.0000	0.000
	2	0.0000	7.866	+D+S+H	-0.0061	1.018
+D+S+H	3	0.0183	2.330		0.0000	1.018

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
Max Upward from all Load Conditions	1.660	6.235	0.469	
Max Upward from Load Combinations	1.660	6.235		
Max Upward from Load Cases	0.877	3.293	0.469	
Max Downward from all Load Conditio	-0.000	-0.124	-2.733	
Max Downward from Load Combinations			-2.733	
Max Downward from Load Cases (Resis	-0.000	-0.124	-1.539	
+D+H	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (**L)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (*L*)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (*LL)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (L**)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (L*L)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (LL*)	0.783	2.942	-1.194	
+D+L+H, LL Comb Run (LLL)	0.783	2.942	-1.194	

Wood Beam

DESCRIPTION: B7

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3	Support 4
+D+Lr+H, LL Comb Run (**L)	0.784	2.818	-0.838	
+D+Lr+H, LL Comb Run (*L*)	0.783	3.062	-1.081	
+D+Lr+H, LL Comb Run (*LL)	0.783	2.938	-0.725	
+D+Lr+H, LL Comb Run (L**)	1.484	5.580	-2.733	
+D+Lr+H, LL Comb Run (L*L)	1.485	5.456	-2.377	
+D+Lr+H, LL Comb Run (LL*)	1.483	5.700	-2.620	
+D+Lr+H, LL Comb Run (LLL)	1.484	5.576	-2.264	
+D+S+H	1.660	6.235	-2.531	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.784	2.849	-0.927	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.783	3.032	-1.110	
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.783	2.939	-0.842	
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.309	4.920	-2.349	
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.309	4.827	-2.082	
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.308	5.011	-2.264	
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.309	4.918	-1.997	
+D+0.750L+0.750S+H, LL Comb Run (**	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (*L	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (*L	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (L*	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (L*	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (LL	1.440	5.411	-2.197	
+D+0.750L+0.750S+H, LL Comb Run (LL	1.440	5.411	-2.197	
+D+0.60W+H	0.783	2.942	-1.194	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.784	2.849	-0.927	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.783	3.032	-1.110	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.783	2.939	-0.842	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.309	4.920	-2.349	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.309	4.827	-2.082	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.308	5.011	-2.264	
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.309	4.918	-1.997	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.450W+H, LL Comb	1.440	5.411	-2.197	
+0.60D+0.60W+0.60H	0.470	1.765	-0.717	
+D+0.70E+0.60H	0.783	2.942	-1.194	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.440	5.411	-2.197	
+0.60D+0.70E+H	0.470	1.765	-0.717	
D Only	0.783	2.942	-1.194	
Lr Only, LL Comb Run (**L)	0.001	-0.124	0.356	
Lr Only, LL Comb Run (*L*)	-0.000	0.120	0.113	
Lr Only, LL Comb Run (*LL)	0.000	-0.004	0.469	
Lr Only, LL Comb Run (L**)	0.701	2.638	-1.539	
Lr Only, LL Comb Run (L*L)	0.702	2.514	-1.183	
Lr Only, LL Comb Run (LL*)	0.700	2.758	-1.426	
Lr Only, LL Comb Run (LLL)	0.701	2.634	-1.070	
S Only	0.877	3.293	-1.337	
H Only				

Wood Column

DESCRIPTION: C1

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	13.5 ft
(Used for non-slender calculations)	
Wood Species	Douglas Fir-Larch
Wood Grade	No.1

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

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

Wood Section Name	**6x10**
Wood Grading/Manuf.	Graded Lumber
Wood Member Type	Sawn
Exact Width	**5.50** in
Exact Depth	**9.50** in
Area	52.250 in^2
Ix	392.964 in^4
Iy	131.714 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 Factc	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 = 13.5 ft, K
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 13.5 ft, K

Applied Loads

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

Column self weight included : 152.880 lbs * Dead Load Factor
AXIAL LOADS . . .
B1+B3 Reactions: Axial Load at 13.50 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 7.460, Lr = 5.940, S = 7.430 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.6735 : 1**
	Load Combination	+D+S
	Governing NDS Forumla	Comp + Mxx + Myy, NDS Eq. 3.9-
	Location of max.above base	13.409 ft
	At maximum location values are . . .	
	Applied Axial	15.043 k
	Applied Mx	-0.6163 k-ft
	Applied My	-0.6163 k-ft
	Fc : Allowable	480.546 psi
PASS	Maximum Shear Stress Ratio =	**0.006749 : 1**
	Load Combination	+D+S
	Location of max.above base	13.50 ft
	Applied Design Shear	1.979 psi
	Allowable Shear	195.50 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.04596 k	Bottom along Y-Y	0.04596 k
Top along X-X	0.04596 k	Bottom along X-X	0.04596 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.02011 in at 7.883 ft above base
for load combination : +D+S
Along X-X -0.05999 in at 7.883 ft above base
for load combination : +D+S

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.507	0.3196	PASS	0.0 ft	0.004320	PASS	13.50 ft
+D+Lr	1.250	0.390	0.5323	PASS	0.0 ft	0.005587	PASS	13.50 ft
+D+S	1.150	0.418	0.6735	PASS	13.409 ft	0.006749	PASS	13.50 ft
+D+0.750Lr	1.250	0.390	0.4740	PASS	0.0 ft	0.004968	PASS	13.50 ft
+D+0.750S	1.150	0.418	0.5251	PASS	0.0 ft	0.005907	PASS	13.50 ft
+0.60D	1.600	0.315	0.1737	PASS	0.0 ft	0.001458	PASS	13.50 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
D Only	-0.023	0.023	-0.023	0.023	7.613				

Wood Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: C1

Maximum Reactions

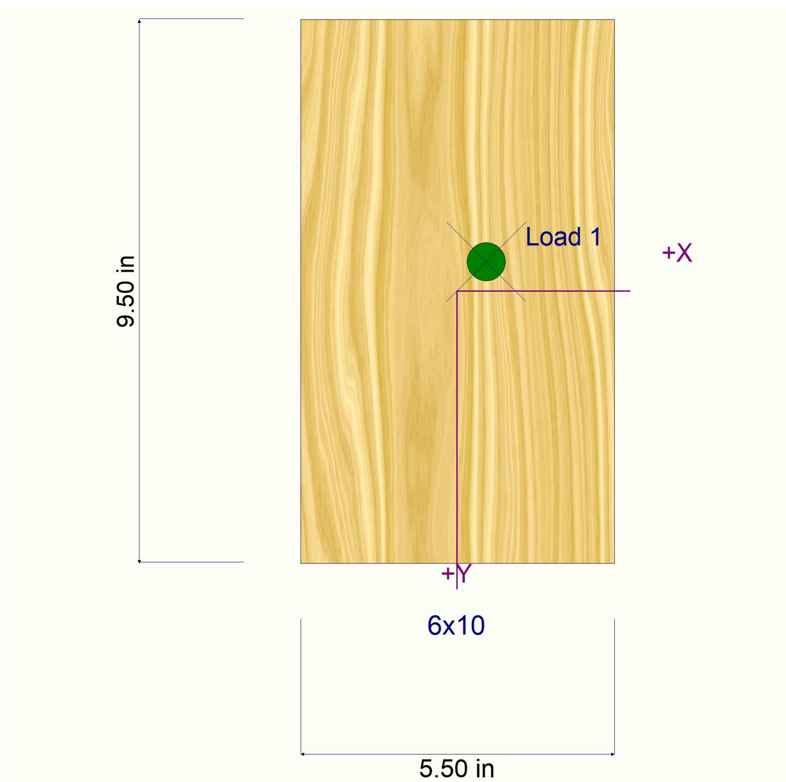
Note: Only non-zero reactions are listed.

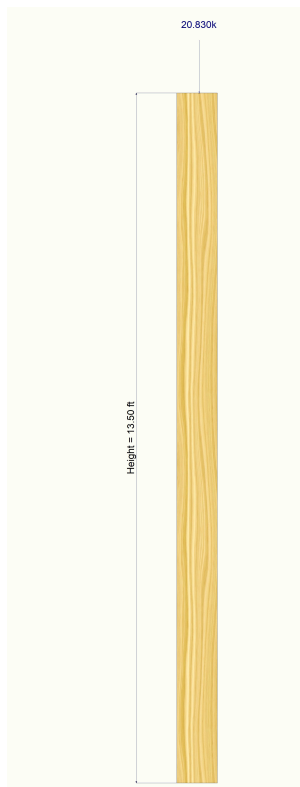
Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments k-ft @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+Lr	-0.041	0.041	-0.041	0.041	13.553				
+D+S	-0.046	0.046	-0.046	0.046	15.043				
+D+0.750Lr	-0.037	0.037	-0.037	0.037	12.068				
+D+0.750S	-0.040	0.040	-0.040	0.040	13.185				
+0.60D	-0.014	0.014	-0.014	0.014	4.568				
Lr Only	-0.018	0.018	-0.018	0.018	5.940				
S Only	-0.023	0.023	-0.023	0.023	7.430				

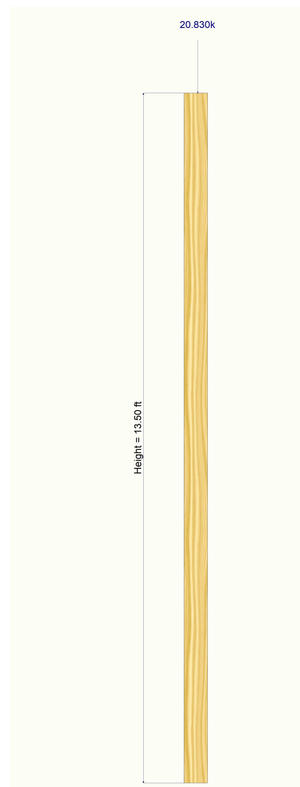
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	-0.0301 in	7.883 ft	-0.010 in	7.883 ft
+D+Lr	-0.0540 in	7.883 ft	-0.018 in	7.883 ft
+D+S	-0.0600 in	7.883 ft	-0.020 in	7.883 ft
+D+0.750Lr	-0.0480 in	7.883 ft	-0.016 in	7.883 ft
+D+0.750S	-0.0525 in	7.883 ft	-0.018 in	7.883 ft
+0.60D	-0.0180 in	7.883 ft	-0.006 in	7.883 ft
Lr Only	-0.0239 in	7.883 ft	-0.008 in	7.883 ft
S Only	-0.0299 in	7.883 ft	-0.010 in	7.883 ft

Sketches







Wood Column

DESCRIPTION: C2

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		13.5 ft	
(Used for non-slender calculations)			
Wood Species	Douglas Fir-Larch		
Wood Grade	No.1		
Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.210 pcf
Fc - Perp	625.0 psi		

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

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

Allow Stress Modification Factors

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

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 13.5 ft
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 13.5 ft,

Applied Loads

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

Column self weight included : 88.510 lbs * Dead Load Factor
AXIAL LOADS . . .
B2 Reaction: Axial Load at 13.50 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 4.460, Lr = 3.420, S = 4.280 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.8527 : 1**
	Load Combination	+D+S
	Governing NDS Forumla	Comp + Mxx + Myy, NDS Eq. 3.9-
	Location of max.above base	13.409 ft
	At maximum location values are .	
	Applied Axial	8.829 k
	Applied Mx	-0.3617 k-ft
	Applied My	-0.3617 k-ft
	Fc : Allowable	480.546 psi
PASS	Maximum Shear Stress Ratio =	**0.006842 : 1**
	Load Combination	+D+S
	Location of max.above base	13.50 ft
	Applied Design Shear	2.006 psi
	Allowable Shear	195.50 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.02698 k	Bottom along Y-Y	0.02698 k
Top along X-X	0.02698 k	Bottom along X-X	0.02698 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.06082 in at 7.883 ft above base
for load combination : +D+S
Along X-X -0.06082 in at 7.883 ft above base
for load combination : +D+S

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.507	0.3298	PASS	0.0 ft	0.004461	PASS	13.50 ft
+D+Lr	1.250	0.390	0.6537	PASS	13.409 ft	0.005675	PASS	13.50 ft
+D+S	1.150	0.418	0.8527	PASS	13.409 ft	0.006842	PASS	13.50 ft
+D+0.750Lr	1.250	0.390	0.5262	PASS	13.409 ft	0.005060	PASS	13.50 ft
+D+0.750S	1.150	0.418	0.6582	PASS	13.409 ft	0.006004	PASS	13.50 ft
+0.60D	1.600	0.315	0.1792	PASS	0.0 ft	0.001506	PASS	13.50 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction (k) @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only	-0.014	0.014	-0.014	0.014	4.549				

Project Title:	Taco Time NW
Engineer:	JMB
Project ID:	23514
Project Descr:	

Wood Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: C2

Maximum Reactions

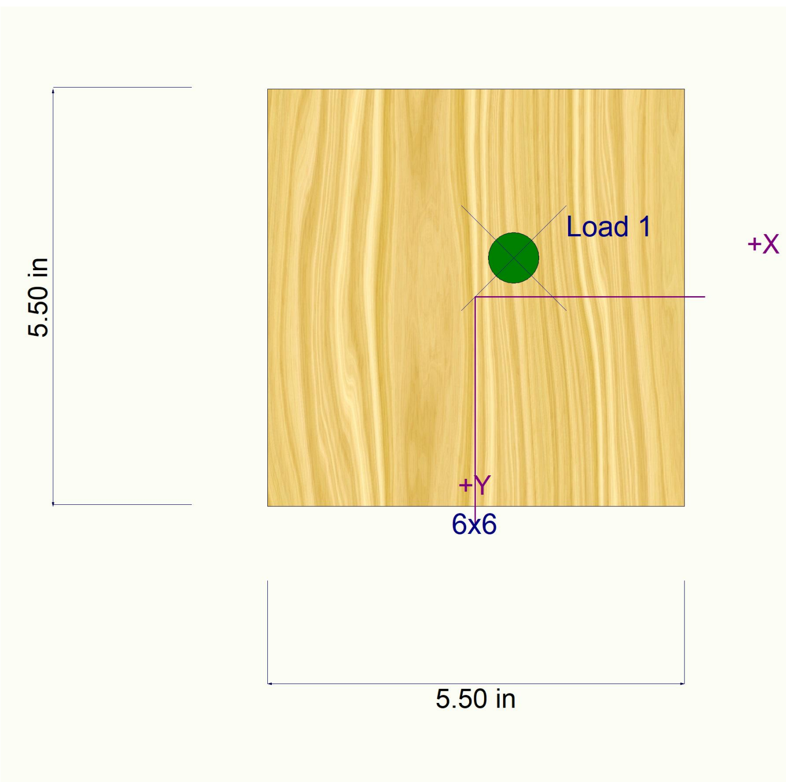
Note: Only non-zero reactions are listed.

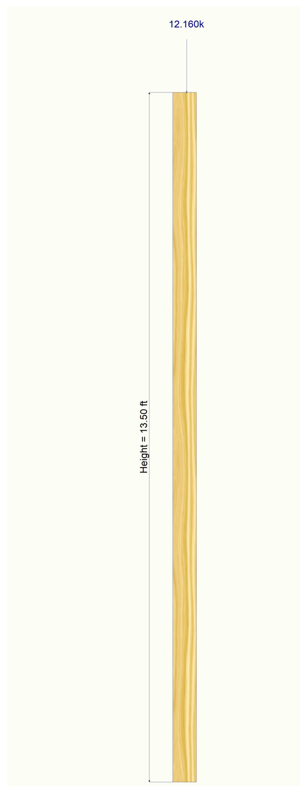
Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments k-ft @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+Lr	-0.024	0.024	-0.024	0.024	7.969				
+D+S	-0.027	0.027	-0.027	0.027	8.829				
+D+0.750Lr	-0.022	0.022	-0.022	0.022	7.114				
+D+0.750S	-0.024	0.024	-0.024	0.024	7.759				
+0.60D	-0.008	0.008	-0.008	0.008	2.729				
Lr Only	-0.011	0.011	-0.011	0.011	3.420				
S Only	-0.013	0.013	-0.013	0.013	4.280				

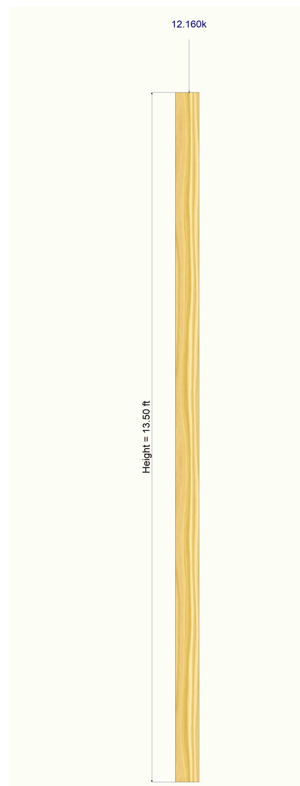
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	-0.0310 in	7.883 ft	-0.031 in	7.883 ft
+D+Lr	-0.0548 in	7.883 ft	-0.055 in	7.883 ft
+D+S	-0.0608 in	7.883 ft	-0.061 in	7.883 ft
+D+0.750Lr	-0.0489 in	7.883 ft	-0.049 in	7.883 ft
+D+0.750S	-0.0534 in	7.883 ft	-0.053 in	7.883 ft
+0.60D	-0.0186 in	7.883 ft	-0.019 in	7.883 ft
Lr Only	-0.0238 in	7.883 ft	-0.024 in	7.883 ft
S Only	-0.0298 in	7.883 ft	-0.030 in	7.883 ft

Sketches







Project Title: Taco Time NW
Engineer: JMB
Project ID: 23514
Project Descr:

Wood Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.05.01 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: C3

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		12 ft	

(Used for non-slender calculations)

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

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

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

Allow Stress Modification Factors

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

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for buckling ABOUT Y-Y Axis = 12 ft, K
Y-Y (depth) axis : Unbraced Length for buckling ABOUT X-X Axis = 12 ft, K

Applied Loads

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

Column self weight included : 78.675 lbs * Dead Load Factor
AXIAL LOADS . . .
B2 Reaction: Axial Load at 12.0 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 2.90, Lr = 2.40, S = 3.0 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.3417** : 1
	Load Combination	+D+S
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are .	
	Applied Axial	5.979 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	578.43 psi
PASS	Maximum Shear Stress Ratio =	**0.005196** : 1
	Load Combination	+D+S
	Location of max.above base	12.0 ft
	Applied Design Shear	1.524 psi
	Allowable Shear	195.50 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.02049 k	Bottom along Y-Y	0.02049 k
Top along X-X	0.02049 k	Bottom along X-X	0.02049 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y -0.03244 in at 7.007 ft above base
for load combination : +D+S
Along X-X -0.03244 in at 7.007 ft above base
for load combination : +D+S

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
D Only	0.900	0.596	0.1834	PASS	0.0 ft	0.003263	PASS	12.0 ft
+D+Lr	1.250	0.472	0.3014	PASS	0.0 ft	0.004294	PASS	12.0 ft
+D+S	1.150	0.503	0.3417	PASS	0.0 ft	0.005196	PASS	12.0 ft
+D+0.750Lr	1.250	0.472	0.2677	PASS	0.0 ft	0.003808	PASS	12.0 ft
+D+0.750S	1.150	0.503	0.2988	PASS	0.0 ft	0.004536	PASS	12.0 ft
+0.60D	1.600	0.386	0.09563	PASS	0.0 ft	0.001101	PASS	12.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	@ Top	Y-Y Axis Reaction @ Base	@ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	@ Top	Mx - End Moments @ Base	@ Top
D Only	-0.010	0.010	-0.010	0.010	2.979				

Wood Column

DESCRIPTION: C3

Maximum Reactions

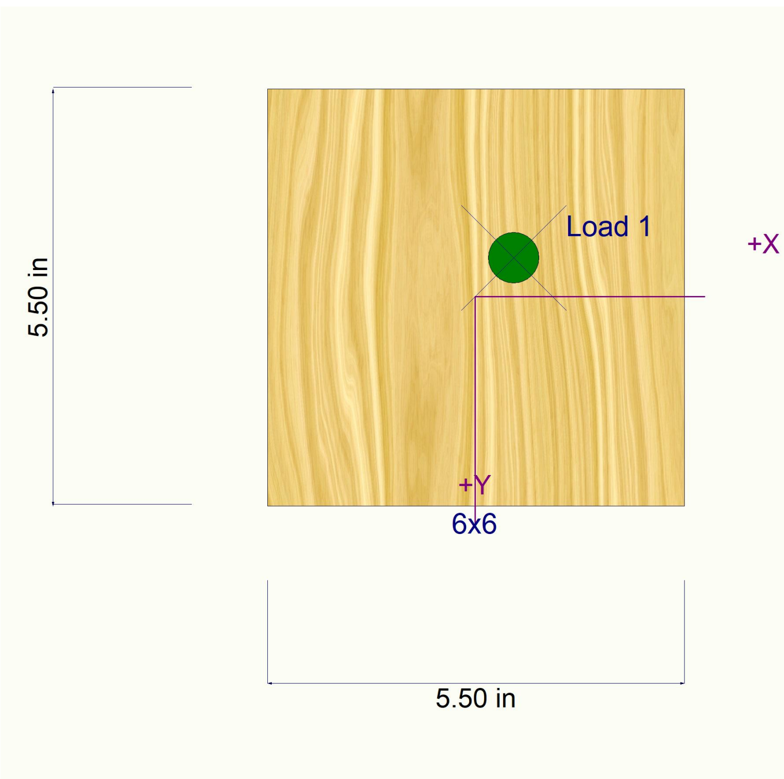
Note: Only non-zero reactions are listed.

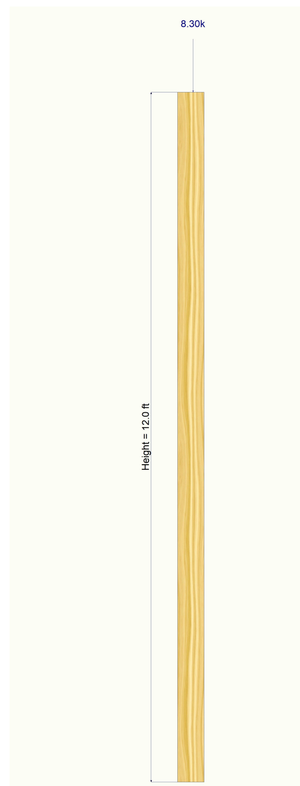
Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+Lr	-0.018	0.018	-0.018	0.018	5.379				
+D+S	-0.020	0.020	-0.020	0.020	5.979				
+D+0.750Lr	-0.016	0.016	-0.016	0.016	4.779				
+D+0.750S	-0.018	0.018	-0.018	0.018	5.229				
+0.60D	-0.006	0.006	-0.006	0.006	1.787				
Lr Only	-0.008	0.008	-0.008	0.008	2.400				
S Only	-0.010	0.010	-0.010	0.010	3.000				

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	-0.0159 in	7.007 ft	-0.016 in	7.007 ft
+D+Lr	-0.0291 in	7.007 ft	-0.029 in	7.007 ft
+D+S	-0.0324 in	7.007 ft	-0.032 in	7.007 ft
+D+0.750Lr	-0.0258 in	7.007 ft	-0.026 in	7.007 ft
+D+0.750S	-0.0283 in	7.007 ft	-0.028 in	7.007 ft
+0.60D	-0.0096 in	7.007 ft	-0.010 in	7.007 ft
Lr Only	-0.0132 in	7.007 ft	-0.013 in	7.007 ft
S Only	-0.0165 in	7.007 ft	-0.016 in	7.007 ft

Sketches





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Project: Taco Time NW — Job Number: 23-514

Sheet: ____ of ____ — Name: JMB

Originating Office: Tacoma — Date: 07-19-23

STUD WALL DESIGN - W1

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b =	900 PSI	W_{WIND} =	19.9 PSF
F_c =	1350 PSI	$W_{SEISMIC}$ =	5.0 PSF
$F_{c\perp}$ =	405 PSI		
E =	1.50E+06 PSI		

APPLIED LOADS:

P_{DEAD} =	400	LBS
P_{LIVE} =		LBS
P_{SNOW} =	375	LBS
P_{WIND} =		LBS
$P_{SEISMIC}$ =		LBS

STUD SIZE: (1) 2x6

A_X =	8.25 IN^2
S_X =	7.56 IN^3
I_X =	20.80 IN^4
$C_{F(BENDING)}$ =	1.3 (NDS 4.3.6)
F_{cE} =	518.7 PSI
C_b =	1.25 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT =	13.5 FT		
SPACING =	16 IN		
ECCENTRICITY =	1 IN		
$C_{F(COMPRESSION)}$ =	1.10 (NDS 4.3.6)		
		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.35	0.319	474	1346	506
2 & 3	1.15	1708	0.30	0.282	481	1547	506
4 & 5	1.60	2376	0.22	0.207	493	2527	506
6 & 7	1.60	2376	0.22	0.207	493	2153	506

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	400	48	152	33	173	274
2	775	94	152	65	192	305
3	681	83	152	57	187	297
4	681	83	272	57	307	488
5	400	48	363	33	384	609
6	400	48	106	33	127	202
7	681	83	80	57	116	184

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.10	0.20	0.10	0.24	0.09	0.18	L/892
2	0.20	0.20	0.19	0.28	0.18	0.20	L/801
3	0.17	0.19	0.16	0.26	0.16	0.20	L/822
4*	0.17	0.19	0.16	0.26	0.16	0.17	L/966
5*	0.10	0.24	0.10	0.28	0.09	0.21	L/775
6	0.10	0.09	0.10	0.11	0.09	0.13	L/1212
7	0.17	0.09	0.16	0.13	0.16	0.12	L/1328
MAX. --->	0.20	0.24	0.19	0.28	0.18	0.21	L/775
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:

C_{fu} =	1.15 (NDS 4.3.7)
F_v =	150 PSI

ALLOWABLE STRESSES:

F_v' =	173 PSI
F_b' =	1547 PSI

STUD REACTIONS (OUT - OF - PLANE)
179 LB

DBL TOP PLATE PROPERTIES:

A_X =	16.50 IN^2
S_X =	4.13 IN^3
I_X =	3.09 IN^4

APPLIED STRESSES:

f_v =	47 PSI	<--- O.K.
f_b =	752 PSI	<--- O.K.
Δ_{MAX} =	0.014 IN	

Project: Taco Time NW | Job Number: 23-514

Sheet: ______ of ______ | Name: JMB

Originating Office: Tacoma | Date: 07-19-23

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STUD WALL DESIGN - W2

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b =	900	PSI	
F_c =	1350	PSI	
$F_{c\perp}$ =	405	PSI	
E =	1.50E+06	PSI	

APPLIED LOADS:

W_{WIND} =	19.9 PSF	P_{DEAD} =	480	LBS
$W_{SEISMIC}$ =	5.0 PSF	P_{LIVE} =		LBS
		P_{SNOW} =	600	LBS
		P_{WIND} =		LBS
		$P_{SEISMIC}$ =		LBS

STUD SIZE: (2) 2x6

A_X =	16.50	IN^2	
S_X =	15.13	IN^3	
I_X =	41.59	IN^4	
$C_{F(BENDING)}$ =	1.3	(NDS 4.3.6)	
F_{cE} =	186.7	PSI	
C_b =	1.13	(NDS 3.10.4)	

MISCELLANEOUS:

HEIGHT =	22.5	FT	
SPACING =	12	IN	
ECCENTRICITY =	1	IN	
$C_{F(COMPRESSION)}$ =	1.10	(NDS 4.3.6)	
		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.	CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 2	DEAD + SNOW + 5 PSF LAT.	CASE 5	DEAD + 0.60WIND
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.	CASE 6	DEAD + 0.7SEISMIC
		CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.13	0.122	182	1346	456
2 & 3	1.15	1708	0.11	0.107	182	1547	456
4 & 5	1.60	2376	0.08	0.077	184	2527	456
6 & 7	1.60	2376	0.08	0.077	184	2153	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT.\ LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	480	29	316	40	341	271
2	1080	65	316	90	373	296
3	930	56	316	78	365	289
4	930	56	567	78	615	488
5	480	29	756	40	781	619
6	480	29	221	40	246	196
7	930	56	168	78	216	171

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.16	0.20	0.06	0.26	0.16	0.50	L/541
2	0.36	0.19	0.14	0.42	0.35	0.54	L/496
3	0.31	0.19	0.12	0.36	0.30	0.53	L/507
4*	0.31	0.19	0.12	0.37	0.30	0.47	L/580
5*	0.16	0.25	0.06	0.32	0.16	0.59	L/457
6	0.16	0.09	0.06	0.13	0.16	0.36	L/750
7	0.31	0.08	0.12	0.21	0.30	0.32	L/855
MAX. --->	0.36	0.25	0.14	0.42	0.35	0.59	L/457
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:

C_{fu} =	1.15	(NDS 4.3.7)	
F_v =	150	PSI	

ALLOWABLE STRESSES:

F_v' =	173	PSI
F_b' =	1547	PSI

STUD REACTIONS (OUT - OF - PLANE)
224 LB

DBL TOP PLATE PROPERTIES:

A_X =	16.50	IN^2
S_X =	4.13	IN^3
I_X =	3.09	IN^4

APPLIED STRESSES:

f_v =	65	PSI	<--- O.K.
f_b =	785	PSI	<--- O.K.
Δ_{MAX} =	0.008	IN	

Project: Taco Time NW | Job Number: 23-514

Sheet: ______ of ______ | Name: JMB

Originating Office: Tacoma | Date: 07-19-23

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STUD WALL DESIGN - W3

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b = 900 PSI
F_c = 1350 PSI
$F_{c\perp}$ = 405 PSI
E = 1.50E+06 PSI

APPLIED LOADS:
W_{WIND} = 19.9 PSF
$W_{SEISMIC}$ = 5.0 PSF

P_{DEAD} = 480 LBS
P_{LIVE} = LBS
P_{SNOW} = 600 LBS
P_{WIND} = LBS
$P_{SEISMIC}$ = LBS

STUD SIZE: (1) 2x6

A_X = 8.25 IN^2
S_X = 7.56 IN^3
I_X = 20.80 IN^4
$C_{F(BENDING)}$ = 1.3 (NDS 4.3.6)
F_{cE} = 308.7 PSI
C_b = 1.25 (NDS 3.10.4)

MISCELLANEOUS:
HEIGHT = 17.5 FT
SPACING = 12 IN
ECCENTRICITY = 1 IN
$C_{F(COMPRESSION)}$ = 1.10 (NDS 4.3.6)

		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.21	0.198	294	1346	506
2 & 3	1.15	1708	0.18	0.173	296	1547	506
4 & 5	1.60	2376	0.13	0.126	300	2527	506
6 & 7	1.60	2376	0.13	0.126	300	2153	506

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	480	58	191	40	216	343
2	1080	131	191	90	248	393
3	930	113	191	78	240	381
4	930	113	343	78	391	621
5	480	58	457	40	482	765
6	480	58	134	40	159	252
7	930	113	101	78	150	238

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.20	0.26	0.11	0.35	0.19	0.38	L/549
2	0.44	0.25	0.26	0.64	0.42	0.44	L/480
3	0.38	0.25	0.22	0.53	0.37	0.42	L/495
4*	0.38	0.25	0.22	0.53	0.37	0.36	L/586
5*	0.19	0.30	0.11	0.41	0.19	0.44	L/475
6	0.19	0.12	0.11	0.18	0.19	0.28	L/747
7	0.38	0.11	0.22	0.32	0.37	0.26	L/793
MAX. --->	0.44	0.30	0.26	0.64	0.42	0.44	L/475
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:
C_{fu} = 1.15 (NDS 4.3.7)
F_V = 150 PSI

ALLOWABLE STRESSES:
F_V' = 173 PSI
F_b' = 1547 PSI

STUD REACTIONS (OUT - OF - PLANE)
174 LB

DBL TOP PLATE PROPERTIES:
A_X = 16.50 IN^2
S_X = 4.13 IN^3
I_X = 3.09 IN^4

APPLIED STRESSES:
f_v = 65 PSI <--- O.K.
f_b = 785 PSI <--- O.K.
Δ_{MAX} = 0.008 IN

Structural Solutions

Project: Taco Time NW | Job Number: 23-514

Sheet: ____ of ____ | Name: JMB

Originating Office: Tacoma | Date: 07-19-23

STUD WALL DESIGN - W4

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b = 900 PSI
F_c = 1350 PSI
$F_{c\perp}$ = 405 PSI
E = 1.50E+06 PSI

APPLIED LOADS:
W_{WIND} = 19.9 PSF
$W_{SEISMIC}$ = 5.0 PSF

P_{DEAD} = 200 LBS
P_{LIVE} = LBS
P_{SNOW} = 250 LBS
P_{WIND} = LBS
$P_{SEISMIC}$ = LBS

STUD SIZE: (2) 2x6

A_X = 16.50 IN^2
S_X = 15.13 IN^3
I_X = 41.59 IN^4
$C_{F(BENDING)}$ = 1.3 (NDS 4.3.6)
F_{cE} = 186.7 PSI
C_b = 1.13 (NDS 3.10.4)

MISCELLANEOUS:
HEIGHT = 22.5 FT
SPACING = 12 IN
ECCENTRICITY = 1 IN
$C_{F(COMPRESSION)}$ = 1.10 (NDS 4.3.6)

		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.13	0.122	182	1346	456
2 & 3	1.15	1708	0.11	0.107	182	1547	456
4 & 5	1.60	2376	0.08	0.077	184	2527	456
6 & 7	1.60	2376	0.08	0.077	184	2153	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	200	12	316	17	327	259
2	450	27	316	38	340	270
3	388	23	316	32	337	267
4	388	23	567	32	587	466
5	200	12	756	17	766	608
6	200	12	221	17	232	184
7	388	23	168	32	188	149

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.07	0.19	0.03	0.21	0.06	0.48	L/566
2	0.15	0.17	0.06	0.23	0.15	0.50	L/544
3	0.13	0.17	0.05	0.21	0.13	0.49	L/549
4*	0.13	0.18	0.05	0.23	0.13	0.44	L/607
5*	0.07	0.24	0.03	0.26	0.06	0.58	L/465
6	0.07	0.09	0.03	0.10	0.06	0.34	L/797
7	0.13	0.07	0.05	0.10	0.13	0.27	L/984
MAX. --->	0.15	0.24	0.06	0.26	0.15	0.58	L/465
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:
C_{fu} = 1.15 (NDS 4.3.7)
F_V = 150 PSI

ALLOWABLE STRESSES:
F_V' = 173 PSI
F_b' = 1547 PSI

STUD REACTIONS (OUT - OF - PLANE)
224 LB

DBL TOP PLATE PROPERTIES:
A_X = 16.50 IN^2
S_X = 4.13 IN^3
I_X = 3.09 IN^4

APPLIED STRESSES:
f_v = 27 PSI <--- O.K.
f_b = 327 PSI <--- O.K.
Δ_{MAX} = 0.003 IN

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Project: Taco Time NW | Job Number: 23-514

Sheet: ____ of ____ | Name: JMB

Originating Office: Tacoma | Date: 07-19-23

STUD WALL DESIGN - W5

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b = 900 PSI

F_c = 1350 PSI

$F_{c\perp}$ = 405 PSI

E = 1.50E+06 PSI

APPLIED LOADS:

W_{WIND} = 0.0 PSF

$W_{SEISMIC}$ = 5.0 PSF

P_{DEAD} = 780 LBS

P_{LIVE} = LBS

P_{SNOW} = 975 LBS

P_{WIND} = LBS

$P_{SEISMIC}$ = LBS

STUD SIZE: (1) 2x6

A_X = 8.25 IN^2

S_X = 7.56 IN^3

I_X = 20.80 IN^4

$C_{F(BENDING)}$ = 1.3 (NDS 4.3.6)

F_{cE} = 449.6 PSI

C_b = 1.25 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 14.5 FT

SPACING = 12 IN

ECCENTRICITY = 1 IN

$C_{F(COMPRESSION)}$ = 1.10 (NDS 4.3.6)

		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.30	0.281	417	1346	506
2 & 3	1.15	1708	0.26	0.247	422	1547	506
4 & 5	1.60	2376	0.19	0.181	431	2527	506
6 & 7	1.60	2376	0.19	0.181	431	2153	506

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT.\ LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	780	95	131	65	172	273
2	1755	213	131	146	223	354
3	1511	183	131	126	210	333
4	1511	183	0	126	79	125
5	780	95	0	65	41	64
6	780	95	92	65	133	210
7	1511	183	70	126	148	235

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.23	0.20	0.19	0.31	0.21	0.21	L/834
2	0.50	0.23	0.42	0.69	0.47	0.27	L/644
3	0.43	0.22	0.36	0.55	0.41	0.25	L/683
4*	0.43	0.05	0.36	0.26	0.41	0.05	L/3514
5*	0.22	0.03	0.19	0.08	0.21	0.03	L/6809
6	0.22	0.10	0.19	0.17	0.21	0.16	L/1082
7	0.43	0.11	0.36	0.37	0.41	0.18	L/967
MAX. --->	0.50	0.23	0.42	0.69	0.47	0.27	L/644
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:

C_{fu} = 1.15 (NDS 4.3.7)

F_v = 150 PSI

ALLOWABLE STRESSES:

F_v' = 173 PSI

F_b' = 1547 PSI

STUD REACTIONS (OUT - OF - PLANE)
36 LB

DBL TOP PLATE PROPERTIES:

A_X = 16.50 IN^2

S_X = 4.13 IN^3

I_X = 3.09 IN^4

APPLIED STRESSES:

f_v = 106 PSI <--- O.K.

f_b = 1276 PSI <--- O.K.

Δ_{MAX} = 0.014 IN

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Project: Taco Time NW | Job Number: 23-514

Sheet: ____ of ____ | Name: JMB

Originating Office: Tacoma | Date: 07-19-23

STUD WALL DESIGN - W6

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b = 900 PSI
F_c = 1350 PSI
$F_{c\perp}$ = 405 PSI
E = 1.50E+06 PSI

APPLIED LOADS:

W_{WIND} = 0.0 PSF
$W_{SEISMIC}$ = 5.0 PSF

P_{DEAD} = 200 LBS
P_{LIVE} = LBS
P_{SNOW} = 250 LBS
P_{WIND} = LBS
$P_{SEISMIC}$ = LBS

STUD SIZE: (2) 2x6

A_X = 16.50 IN^2
S_X = 15.13 IN^3
I_X = 41.59 IN^4
$C_{F(BENDING)}$ = 1.3 (NDS 4.3.6)
F_{cE} = 186.7 PSI
C_b = 1.13 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 22.5 FT
SPACING = 12 IN
ECCENTRICITY = 1 IN
$C_{F(COMPRESSION)}$ = 1.10 (NDS 4.3.6)

		APPLY?	
$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.15	YES	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.13	0.122	182	1346	456
2 & 3	1.15	1708	0.11	0.107	182	1547	456
4 & 5	1.60	2376	0.08	0.077	184	2527	456
6 & 7	1.60	2376	0.08	0.077	184	2153	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	200	12	316	17	327	259
2	450	27	316	38	340	270
3	388	23	316	32	337	267
4	388	23	0	32	20	16
5	200	12	0	17	10	8
6	200	12	221	17	232	184
7	388	23	168	32	188	149

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.07	0.19	0.03	0.21	0.06	0.48	L/566
2	0.15	0.17	0.06	0.23	0.15	0.50	L/544
3	0.13	0.17	0.05	0.21	0.13	0.49	L/549
4*	0.13	0.01	0.05	0.02	0.13	0.02	L/17665
5*	0.07	0.00	0.03	0.01	0.06	0.01	L/34226
6	0.07	0.09	0.03	0.10	0.06	0.34	L/797
7	0.13	0.07	0.05	0.10	0.13	0.27	L/984
MAX. --->	0.15	0.19	0.06	0.23	0.15	0.50	L/544
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:

C_{fu} = 1.15 (NDS 4.3.7)
F_V = 150 PSI

ALLOWABLE STRESSES:

F_V' = 173 PSI
F_b' = 1547 PSI

STUD REACTIONS (OUT - OF - PLANE)
56 LB

DBL TOP PLATE PROPERTIES:

A_X = 16.50 IN^2
S_X = 4.13 IN^3
I_X = 3.09 IN^4

APPLIED STRESSES:

f_v = 27 PSI <--- O.K.
f_b = 327 PSI <--- O.K.
Δ_{MAX} = 0.003 IN

Project:	Taco Time NW	Job Number:	23-514
Sheet:	of	Name:	JMB
Originating Office:	Tacoma	Date:	07-19-23

PCS Structural Solutions

STUD WALL DESIGN - W1 Jamb

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b =	900	PSI	**APPLIED LOADS:**		P_{DEAD} =	100 LBS
F_c =	1350	PSI	W_{WIND} =	18.0 PSF	P_{LIVE} =	LBS
$F_{c\perp}$=	405	PSI	$W_{SEISMIC}$ =	5.0 PSF	P_{SNOW} =	100 LBS
E =	1.50E+06	PSI			P_{WIND} =	LBS
					$P_{SEISMIC}$ =	LBS

STUD SIZE: (2) 2x6

A_X =	16.50	IN^2	**MISCELLANEOUS:**	HEIGHT =	13.5	FT
S_X =	15.13	IN^3		SPACING =	56	IN
I_X =	41.59	IN^4		ECCENTRICITY =	1	IN
$C_{F(BENDING)}$ =	1.3	(NDS 4.3.6)		$C_{F(COMPRESSION)}$ =	1.10	(NDS 4.3.6)
					APPLY?	
F_{cE} =	518.7	PSI	$C_{SYS(BENDING)}$ =	1.00	NO	(SDPWS T3.1.1.1)
C_b =	1.13	(NDS 3.10.4)	$C_{r(BENDING)}$ =	1.00	NO	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.	CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 2	DEAD + SNOW + 5 PSF LAT.	CASE 5	DEAD + 0.60WIND
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.	CASE 6	DEAD + 0.75SEISMIC
		CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.35	0.319	474	1170	456
2 & 3	1.15	1708	0.30	0.282	481	1346	456
4 & 5	1.60	2376	0.22	0.207	493	1872	456
6 & 7	1.60	2376	0.22	0.207	493	1872	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	100	6	532	8	537	426
2	200	12	532	17	542	430
3	175	11	532	15	541	429
4	175	11	861	15	870	690
5	100	6	1148	8	1153	915
6	100	6	372	8	377	299
7	175	11	282	15	291	231

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.01	0.36	0.01	0.37	0.01	0.28	L/574
2	0.03	0.32	0.03	0.33	0.02	0.28	L/568
3	0.02	0.32	0.02	0.33	0.02	0.28	L/570
4*	0.02	0.37	0.02	0.38	0.02	0.32	L/506
5*	0.01	0.49	0.01	0.49	0.01	0.42	L/382
6	0.01	0.16	0.01	0.16	0.01	0.20	L/817
7	0.02	0.12	0.02	0.13	0.02	0.15	L/1059
MAX. --->	0.03	0.49	0.03	0.49	0.02	0.42	L/382
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:			**ALLOWABLE STRESSES:**			STUD REACTIONS (OUT - OF - PLANE)
C_{fu} =	1.15	(NDS 4.3.7)	F_v' =	173	PSI	
F_v =	150	PSI	F_b' =	1547	PSI	567 LB

DBL TOP PLATE PROPERTIES:			**APPLIED STRESSES:**			
A_X =	16.50	IN^2	f_v =	12	PSI	<--- O.K.
S_X =	4.13	IN^3	f_b =	679	PSI	<--- O.K.
I_X =	3.09	IN^4	Δ_{MAX} =	0.158	IN	

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Project: Taco Time NW — Job Number: 23-514

Sheet: ____ of ____ — Name: JMB

Originating Office: Tacoma — Date: 07-19-23

STUD WALL DESIGN - W4 Jamb (5'-0" Opening)

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#1

F_b = 900 PSI
F_c = 1350 PSI
$F_{c\perp}$ = 405 PSI
E = 1.50E+06 PSI

APPLIED LOADS:

W_{WIND} = 18.0 PSF
$W_{SEISMIC}$ = 5.0 PSF

P_{DEAD} = 100 LBS
P_{LIVE} = LBS
P_{SNOW} = 100 LBS
P_{WIND} = LBS
$P_{SEISMIC}$ = LBS

STUD SIZE: (3) 2x6

A_X = 24.75 IN^2
S_X = 22.69 IN^3
I_X = 62.39 IN^4
$C_{F(BENDING)}$ = 1.3 (NDS 4.3.6)
F_{cE} = 261.9 PSI
C_b = 1.08 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 19 FT
SPACING = 36 IN
ECCENTRICITY = 1 IN
$C_{F(COMPRESSION)}$ = 1.10 (NDS 4.3.6)

		APPLY?	
$C_{SYS(BENDING)}$ =	1.00	NO	(SDPWS T3.1.1.1)
$C_{r(BENDING)}$ =	1.00	NO	(NDS 4.3.9)

LOAD CASES - IBC 1605.3.1

CASE 1	DEAD + LIVE + 5 PSF LAT.
CASE 2	DEAD + SNOW + 5 PSF LAT.
CASE 3	DEAD + 0.75LIVE + 0.75SNOW + 5 PSF LAT.
CASE 4	DEAD + 0.45WIND + 0.75LIVE + 0.75SNOW
CASE 5	DEAD + 0.60WIND
CASE 6	DEAD + 0.7SEISMIC
CASE 7	DEAD + 0.53SEISMIC + 0.75LIVE + 0.75SNOW

ALLOWABLE STRESSES - Cd PER NDS T2.3.2, Cp PER NDS 3.7.1, ASSUME Cm, Ct, Ci, CL = 1.0

CASE	C_D	F_c*	F_{cE}/F_c*	C_P	F_c'	F_b'	$F_{c\perp}$
1	1.00	1485	0.18	0.169	252	1170	439
2 & 3	1.15	1708	0.15	0.148	253	1346	439
4 & 5	1.60	2376	0.11	0.108	256	1872	439
6 & 7	1.60	2376	0.11	0.108	256	1872	439

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	100	4	677	8	682	361
2	200	8	677	17	687	364
3	175	7	677	15	686	363
4	175	7	1097	15	1106	585
5	100	4	1462	8	1467	776
6	100	4	474	8	479	253
7	175	7	359	15	368	195

DESIGN CHECKS - COMBINED STRESS CHECK PER NDS EQN 3.9-3

CASE	f_c/F_c'	f_b/F_b'	$f_c/F_{c\perp}$	Combined	f_c/F_{cE}	Deflection	L/?
1	0.02	0.31	0.01	0.31	0.02	0.47	L/481
2	0.03	0.27	0.02	0.28	0.03	0.48	L/478
3	0.03	0.27	0.02	0.28	0.03	0.48	L/479
4*	0.03	0.31	0.02	0.32	0.03	0.54	L/424
5*	0.02	0.41	0.01	0.42	0.02	0.71	L/320
6	0.02	0.14	0.01	0.14	0.02	0.33	L/686
7	0.03	0.10	0.02	0.11	0.03	0.26	L/893
MAX. --->	0.03	0.41	0.02	0.42	0.03	0.71	L/320
	O.K.	O.K.	O.K.	O.K.	O.K.		

* Deflections reduced by 0.42 per IBC Table 1604.3 footnote f. Increase deflection by 1.4 for jambs supporting glass.

PLATE BENDING - *ALIGN STUDS WITH JOISTS WHERE POSSIBLE*

MISCELLANEOUS:

C_{fu} = 1.15 (NDS 4.3.7)
F_V = 150 PSI

ALLOWABLE STRESSES:

F_V' = 173 PSI
F_b' = 1547 PSI

STUD REACTIONS (OUT - OF - PLANE)

513 LB

DBL TOP PLATE PROPERTIES:

A_X = 16.50 IN^2
S_X = 4.13 IN^3
I_X = 3.09 IN^4

APPLIED STRESSES:

f_v = 12 PSI <--- O.K.
f_b = 436 PSI <--- O.K.
Δ_{MAX} = 0.042 IN

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Project: Taco Time NW — Job No: 23-514
Subject: Footings — Sheet 1/1 — Name: JMB
Originating Office: ☐ Seattle ☒ Tacoma ☐ Portland — Date: 2023-06-20

Typical Continuous Footings

p_{allow} = 1500 psf (assumed) per Design Criteria

2'-0" wide ⇒ w_{allow} = 2' × 1.5 ksf = 3 klf

1'-6" wide ⇒ w_{allow} = 1.5' × 1.5 ksf = 2.25 klf

Typical Spread Footings

2'-0" ☐ ⇒ P_{allow} = 2' × 2' × 1.5 ksf = 6 K

2'-6" ☐ ⇒ P_{allow} = 9.4 K

3'-0" ☐ " " = 13.5 K

3'-6" ☐ " " = 18.4 K

4'-0" ☐ " " = 24 K

If needed

4'-6" ☐ " " = 30.4 K

* Compare to Enercalc Reactions

1'-4" DIA -> Pallow = 2.1 K

1'-6" DIA -> Pallow = 2.7 K

2'-0" DIA -> Pallow = 4.7 K

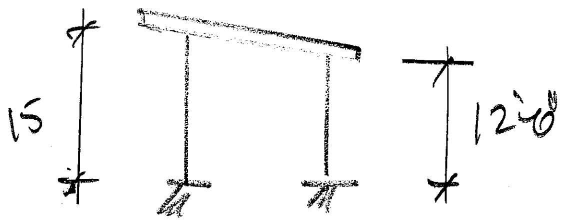
2'-6" DIA -> Pallow = 7.4 K

PCS Structural Solutions

Project: TACO TIME NW | Job No: 23-514
Subject: | Sheet | Name: LAH
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland | Date: 8/30/23

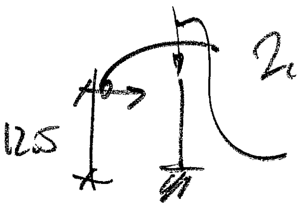
Review canopy at trash enclosure

$R = 2.5$
$\Omega = 1.25$



$V = 0.45\,[15\,psf \times 14 \times 30] = 2.83^k$

$2.83^k \times \underbrace{1.25}_{\Omega} \times \underbrace{1.3}_{torsion}$

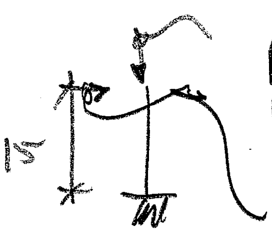


$P = \left.\begin{matrix} D = 15 \\ S = 25 \end{matrix}\right\} 7'(15) = \dfrac{1.6^k}{2.6^k}$

$\dfrac{P_u}{\phi P_{all}} = \dfrac{6.1^k}{73} = .08 < .15$, okay

$\Delta = 1.838" \times 2.5 = 4.6"$

$\therefore$ HSS 6x6x1/2 No good] Allowable $= .025 \times 12.5 \times 12 = 3.75"$



$P_D = 1.1^k$
$P_S = 2.6^k$

Try HSS 7x7x1/2
$\Delta_{ie} = 1.1\,(2.5) = 2.75"$
okay

$V = \dfrac{2.83(.25)}{2}\,1.25(1.3)$

$\Delta = .38" \rightarrow \Delta_{ie} = .38(2.5) = .95"$ okay

$\therefore$ HSS 7x7x1/2

Building Code Information

Governing Code : IBC 2018, ASCE 7-16, CBC 2019, AISC 360-16, NDS 2018, ACI 318-14, TMS 402-16

City Jurisdiction :

Contact Name :

Alternate Contact :

Building Official :

Address : , ,

Phone : Fax : eMail :

Notes :

Project Information

Project Title : Taco Time NW

Description :

I.D. : 23514

Address : , Puyallup, WA

Project Leader : JMB

Phone : Fax : eMail :

Project Notes

Steel Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.08.01 PCS STRUCTURAL SOLUTIONS

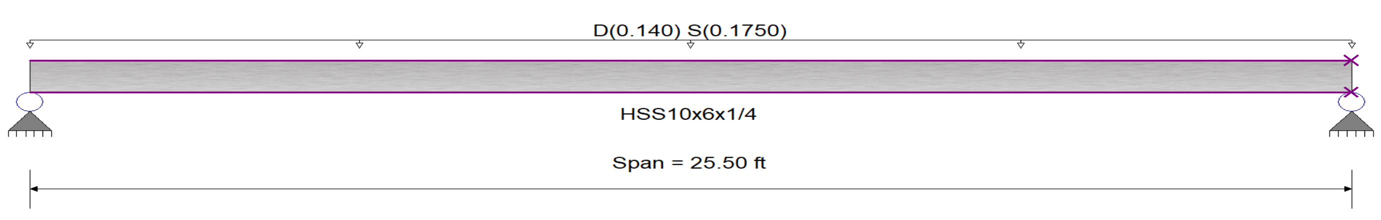
DESCRIPTION: Trash Enclosure Roof Beam

CODE REFERENCES

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.020, S = 0.0250 ksf, Tributary Width = 7.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.470** : 1	Maximum Shear Stress Ratio =	**0.056** : 1
Section used for this span	**HSS10x6x1/4**	Section used for this span	**HSS10x6x1/4**
Ma : Applied	27.702 k-ft	Va : Applied	4.345 k
Mn / Omega : Allowable	58.882 k-ft	Vn/Omega : Allowable	77.861 k
Load Combination	+D+S	Load Combination	+D+S
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.594 in	Ratio =	514	>=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	1.159 in	Ratio =	264	>=180	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0	<180	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
D Only													
Dsgn. L = 25.50 ft	1	0.229	0.027	13.48		13.48	98.33	58.88	1.00	1.00	2.11	130.03	77.86
+D+S													
Dsgn. L = 25.50 ft	1	0.470	0.056	27.70		27.70	98.33	58.88	1.00	1.00	4.35	130.03	77.86
+D+0.750S													
Dsgn. L = 25.50 ft	1	0.410	0.049	24.15		24.15	98.33	58.88	1.00	1.00	3.79	130.03	77.86
+0.60D													
Dsgn. L = 25.50 ft	1	0.137	0.016	8.09		8.09	98.33	58.88	1.00	1.00	1.27	130.03	77.86

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	1.1591	12.823		0.0000	0.000

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	4.345	4.345
Max Upward from Load Combinations	4.345	4.345
Max Upward from Load Cases	2.231	2.231
D Only	2.114	2.114
+D+S	4.345	4.345
+D+0.750S	3.788	3.788
+0.60D	1.269	1.269

Steel Beam

DESCRIPTION: Trash Enclosure Roof Beam

Vertical Reactions

Support notation : Far left is #: Values in KIPS

Load Combination	Support 1	Support 2
S Only	2.231	2.231

Steel Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.08.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: Trash Enclosure Steel Column (12.5')

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS7x7x1/2**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 12.50 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 12.50 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 12.50 ft, K = 2.1

Applied Loads

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

Column self weight included : 525.63 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 12.50 ft, D = 1.60, S = 2.60 k
BENDING LOADS . . .
Lat. Point Load at 12.50 ft creating Mx-x, E = 2.30 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.3566**	: 1
Load Combination	+1.402D+0.70S+1.250E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Pu	4.80	k
0.9 * Pn	182.679	k
Mu-x	-35.938	k-ft
0.9 * Mn-x :	104.625	k-ft
Mu-y	0.0	k-ft
0.9 * Mn-y :	104.625	k-ft
PASS Maximum Shear Stress Rati	**0.02451**	: 1
Load Combination	+1.402D+0.70S+1.250E	
Location of max.above base	0.0	ft
At maximum location values are . . .		
Vu : Applied	2.875	k
Vn * Phi : Allowable	117.285	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	2.30 k

Maximum Load Deflections . . .

Along Y-Y 1.103 in at 12.50ft above base
for load combination :E Only

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

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.016	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.000	PASS	0.00 ft
+1.20D	0.014	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.000	PASS	0.00 ft
+1.20D+0.50S	0.021	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.000	PASS	0.00 ft
+1.20D+1.60S	0.037	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.000	PASS	0.00 ft
+1.402D+0.70S+1.250E	0.357	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.025	PASS	0.00 ft
+0.90D	0.010	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.000	PASS	0.00 ft
+0.6980D+1.250E	0.348	PASS	0.00 ft	1.67	1.00	119.77	119.77	0.025	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	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	2.126								
+D+S	4.726								
+D+0.750S	4.076								
+D+0.70E	2.126			1.610		-20.125			
+D+0.750S+0.5250E	4.076			1.208		-15.094			
+0.60D	1.275								
+0.60D+0.70E	1.275			1.610		-20.125			

Steel Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.08.01 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: Trash Enclosure Steel Column (12.5')

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	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
S Only	2.600								
E Only				2.300		-28.750			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	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
Axial @ Base	Maximum	4.726								
"	Minimum				2.300		-28.750			
Reaction, X-X Axis Base	Maximum	2.126								
"	Minimum	2.126								
Reaction, Y-Y Axis Base	Maximum				2.300		-28.750			
"	Minimum	2.126								
Reaction, X-X Axis Top	Maximum	2.126								
"	Minimum	2.126								
Reaction, Y-Y Axis Top	Maximum	2.126								
"	Minimum	2.126								
Moment, X-X Axis Base	Maximum	2.126								
"	Minimum		-28.750		2.300		-28.750			
Moment, Y-Y Axis Base	Maximum	2.126								
"	Minimum	2.126								
Moment, X-X Axis Top	Maximum	2.126								
"	Minimum	2.126								
Moment, Y-Y Axis Top	Maximum	2.126								
"	Minimum	2.126								

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.772 in	12.500 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	0.579 in	12.500 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.772 in	12.500 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	1.092 in	12.416 ft

Steel Section Properties : HSS7x7x1/2

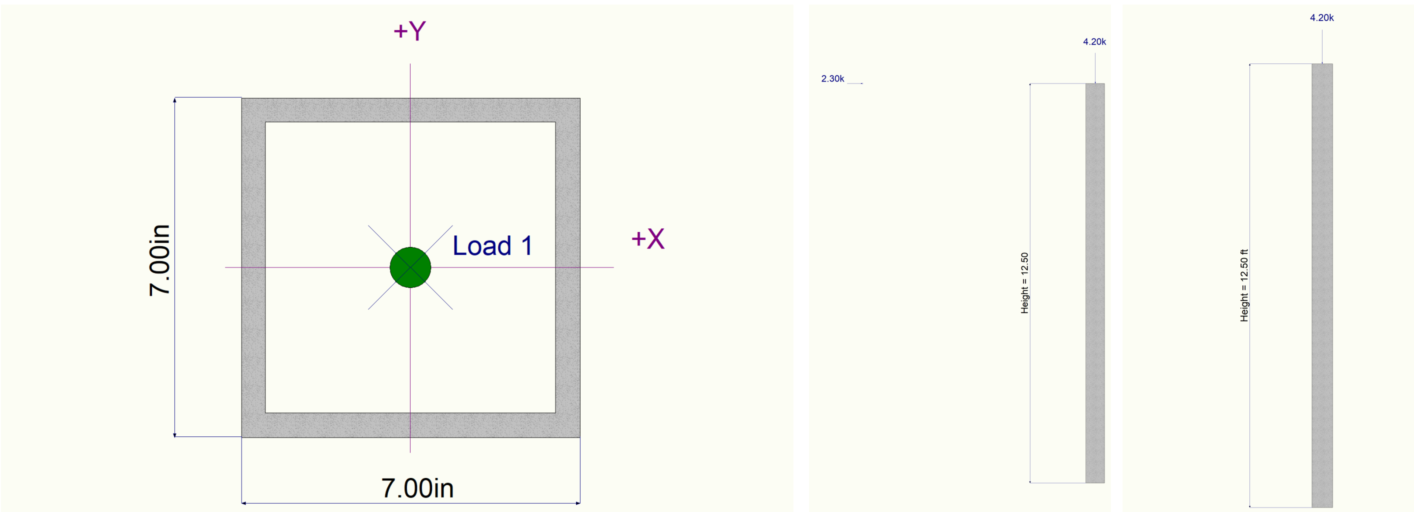
Depth	= 7.000 in	I xx	= 80.50 in^4	J	= 133.000 in^4
Design Thick	= 0.465 in	S xx	= 23.00 in^3		
Width	= 7.000 in	R xx	= 2.630 in		
Wall Thick	= 0.500 in	Zx	= 27.900 in^3		
Area	= 11.600 in^2	I yy	= 80.500 in^4	C	= 39.300 in^3
Weight	= 42.050 plf	S yy	= 23.000 in^3		
		R yy	= 2.630 in		
Ycg	= 0.000 in				

Steel Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Trash Enclosure Steel Column (12.5')

Sketches



Steel Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Trash Enclosure Steel Column (15')

Code References

Calculations per AISC 360-16, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Steel Section Name : **HSS7x7x1/2**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 50.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 15.0 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition :
Unbraced Length for buckling ABOUT X-X Axis = 15.0 ft, K = 2.1
Unbraced Length for buckling ABOUT Y-Y Axis = 15.0 ft, K = 2.1

Applied Loads

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

Column self weight included : 630.75 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 15.0 ft, D = 1.60, S = 2.60 k
BENDING LOADS . . .
Lat. Point Load at 12.50 ft creating Mx-x, E = 0.60 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.1091**	: 1
	Load Combination	+1.402D+0.70S+1.250E	
	Location of max.above base	0.0	ft
	At maximum location values are . . .		
	Pu	4.948	k
	0.9 * Pn	126.860	k
	Mu-x	-9.375	k-ft
	0.9 * Mn-x :	104.625	k-ft
	Mu-y	0.0	k-ft
	0.9 * Mn-y :	104.625	k-ft
PASS	Maximum Shear Stress Rati	**0.006395**	: 1
	Load Combination	+1.402D+0.70S+1.250E	
	Location of max.above base	0.0	ft
	At maximum location values are . . .		
	Vu : Applied	0.750	k
	Vn * Phi : Allowable	117.285	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	0.60 k

Maximum Load Deflections . . .

Along Y-Y 0.3741 in at 15.0ft above base
for load combination : E Only

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

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Cbx	Cby	KxLx/Ry	KyLy/Rx	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.025	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.000	PASS	0.00 ft
+1.20D	0.021	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.000	PASS	0.00 ft
+1.20D+0.50S	0.031	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.000	PASS	0.00 ft
+1.20D+1.60S	0.054	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.000	PASS	0.00 ft
+1.402D+0.70S+1.250E	0.109	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.006	PASS	0.00 ft
+0.90D	0.016	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.000	PASS	0.00 ft
+0.6980D+1.250E	0.096	PASS	0.00 ft	1.93	1.00	143.73	143.73	0.006	PASS	0.00 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments @ Top	My - End Moments @ Base	My - End Moments @ Top
D Only	2.231								
+D+S	4.831								
+D+0.750S	4.181								
+D+0.70E	2.231			0.420		-5.250			
+D+0.750S+0.5250E	4.181			0.315		-3.938			
+0.60D	1.338								
+0.60D+0.70E	1.338			0.420		-5.250			

Steel Column

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Trash Enclosure Steel Column (15')

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments @ Base	k-ft @ Top	My - End Moments @ Base	@ Top
S Only	2.600								
E Only				0.600		-7.500			

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction @ Base	k @ Top	Y-Y Axis Reaction @ Base	@ Top	Mx - End Moments @ Base	k-ft @ Top	My - End Moments @ Base	@ Top
Axial @ Base	Maximum	4.831								
"	Minimum				0.600		-7.500			
Reaction, X-X Axis Base	Maximum	2.231								
"	Minimum	2.231								
Reaction, Y-Y Axis Base	Maximum				0.600		-7.500			
"	Minimum	2.231								
Reaction, X-X Axis Top	Maximum	2.231								
"	Minimum	2.231								
Reaction, Y-Y Axis Top	Maximum	2.231								
"	Minimum	2.231								
Moment, X-X Axis Base	Maximum	2.231								
"	Minimum		-7.500		0.600		-7.500			
Moment, Y-Y Axis Base	Maximum	2.231								
"	Minimum	2.231								
Moment, X-X Axis Top	Maximum	2.231								
"	Minimum	2.231								
Moment, Y-Y Axis Top	Maximum	2.231								
"	Minimum	2.231								

Maximum Deflections for Load Combinations

Load Combination	Max. Deflection in X dir	Distance	Max. Deflection in Y dir	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750S	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.70E	0.0000 in	0.000 ft	0.262 in	15.000 ft
+D+0.750S+0.5250E	0.0000 in	0.000 ft	0.196 in	15.000 ft
+0.60D	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+0.70E	0.0000 in	0.000 ft	0.262 in	15.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.371 in	14.899 ft

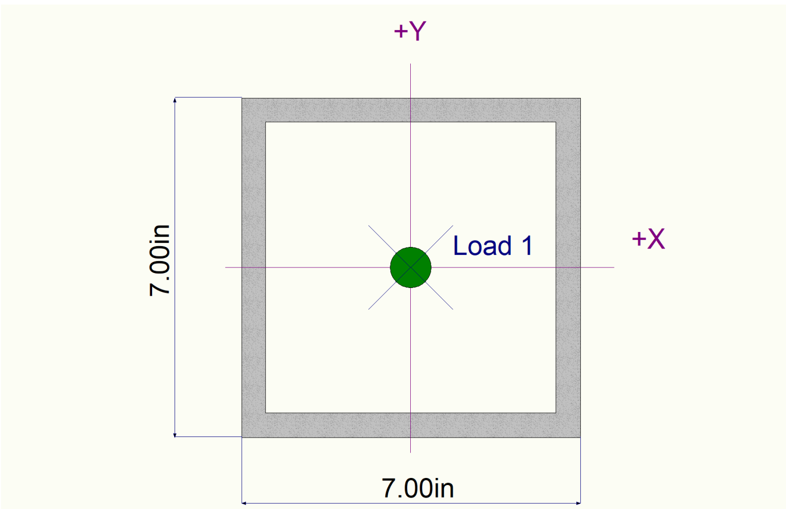
Steel Section Properties : HSS7x7x1/2

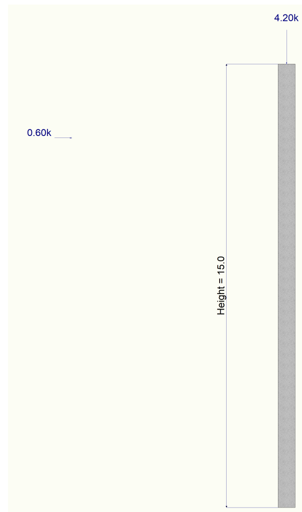
Depth	=	7.000 in	I xx	=	80.50 in^4	J	=	133.000 in^4
Design Thick	=	0.465 in	S xx	=	23.00 in^3			
Width	=	7.000 in	R xx	=	2.630 in			
Wall Thick	=	0.500 in	Zx	=	27.900 in^3			
Area	=	11.600 in^2	I yy	=	80.500 in^4	C	=	39.300 in^3
Weight	=	42.050 plf	S yy	=	23.000 in^3			
			R yy	=	2.630 in			
Ycg	=	0.000 in						

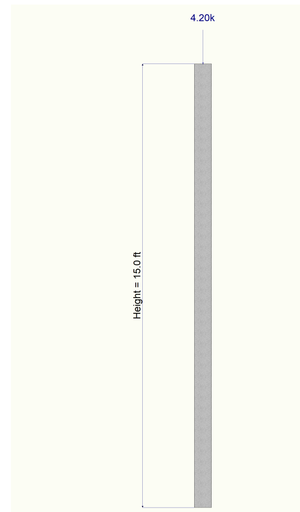
Steel Column

DESCRIPTION: Trash Enclosure Steel Column (15')

Sketches







General Footing

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Footing at 12.5 Tall Columns

Code References

Calculations per ACI 318-14, IBC 2018, CBC 2019, ASCE 7-16
Load Combinations Used : IBC 2021

General Information

Material Properties

f'c : Concrete 28 day strength	=	3.0	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.750	

Analysis Settings

Min Steel % Bending Reinf.	=	
Min Allow % Temp Reinf.	=	0.00180
Min. Overturning Safety Factor	=	1.0 : 1
Min. Sliding Safety Factor	=	1.0 : 1
Add Ftg Wt for Soil Pressure	:	Yes
Use ftg wt for stability, moments & shears	:	Yes
Add Pedestal Wt for Soil Pressure	:	No
Use Pedestal wt for stability, mom & shear	:	No

Soil Design Values

Allowable Soil Bearing	=	1.50	ksf
Soil Density	=	110.0	pcf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.350	

Increases based on footing Depth

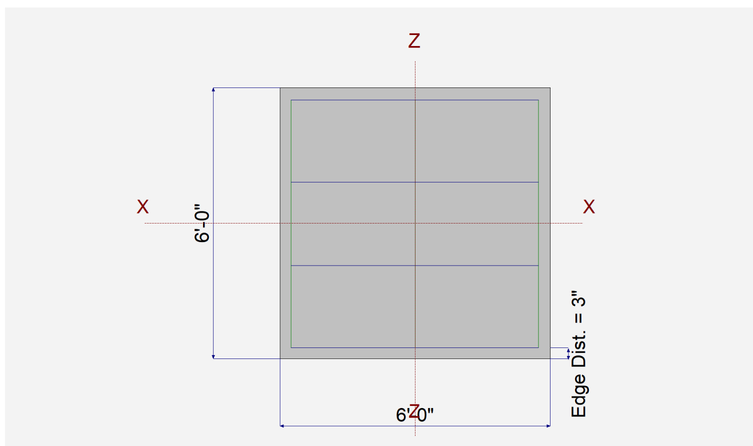
Footing base depth below soil surface	=	3.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	6.0	ft
Length parallel to Z-Z Axis	=	6.0	ft
Footing Thickness	=	18.0	in



Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in

Rebar Centerline to Edge of Concrete...

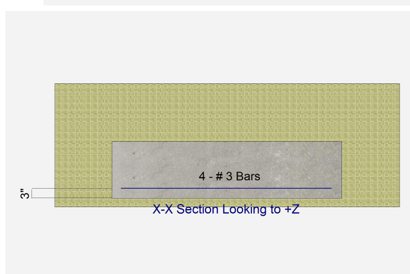
at Bottom of footing	=	3.0	in

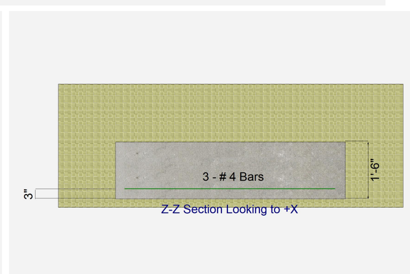
Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		4.0
Reinforcing Bar Size	=	#	3
Bars parallel to Z-Z Axis			
Number of Bars	=		3.0
Reinforcing Bar Size	=	#	4

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	n/a
# Bars required within zone	n/a
# Bars required on each side of zone	n/a





Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	58.0			2.20				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=						27.60		k-ft
V-x	=								k
V-z	=								k

Steel Base Plate

DESCRIPTION: Trash Enclosure Bearing Plate

Code Reference:

Calculations per AISC Design Guide # 1, IBC 2018, CBC 2019, ASCE 7-16, AISC 360-16

Load Combination Set : IBC 2021

General Information

Material Properties

AISC Design Method	Load Resistance Factor Design		Φ c : LRFD Resistance Factor	0.65
Steel Plate Fy	=	36 ksi		
Concrete Support f'c	=	3 ksi		
Assumed Bearing Area	Full Bearing		Nominal Bearing Fp per J8	2.550 ksi

Column & Plate

Column Properties

Steel Section HSS7x7x1/2

Depth	7 in	Area	11.6 in^2
Width	7 in	Ixx	80.5 in^4
Flange Thickness	0.465 in	Iyy	80.5 in^4
Web Thicknes	0 in		

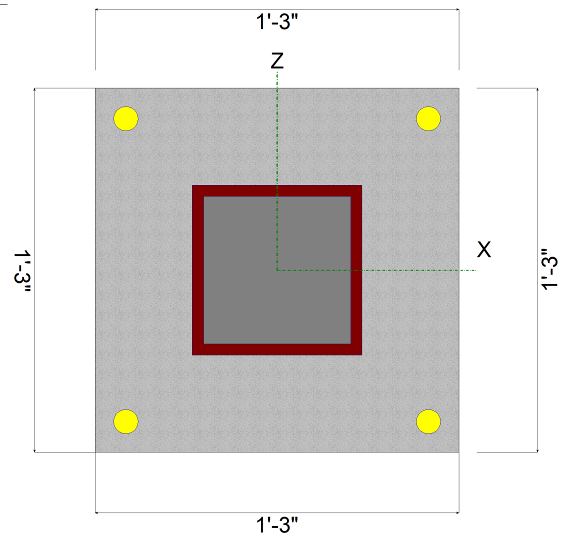


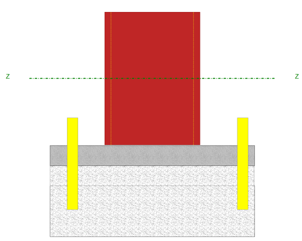
Plate Dimensions

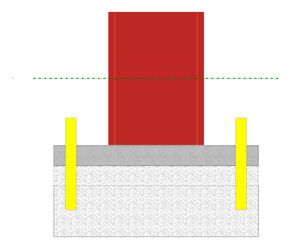
N : Length	15.0 in
B : Width	15.0 in
Thickness	1.50 in

Column assumed welded to base plate

Support Dimensions

Width along "X"	15.0 in
Length along "Z"	15.0 in



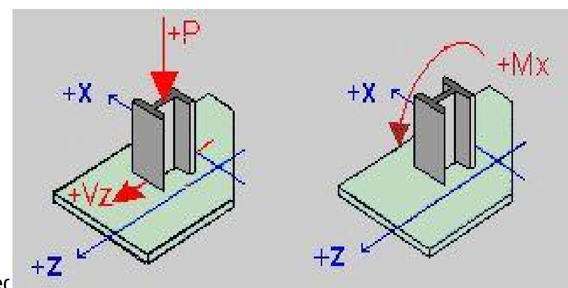


Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	2.20 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	k	k	k-ft
S : Snow	2.20 k	k	k-ft
W : Wind	k	k	k-ft
E : Earthquake	k	k	27.60 k-ft
H : Lateral Earth	k	k	k-ft

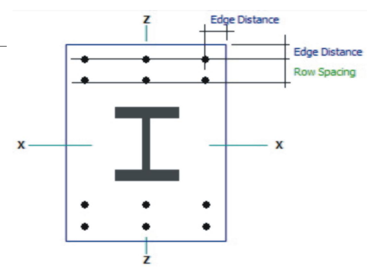
" P " = Gravity load, "+" sign is downwa "+" Moments create higher soil pressure at +Z ec

"+" Shears push plate towards +Z edge



Anchor Bolts

Anchor Bolt or Rod Description 1		
Max of Tension or Pullout Capacity......		k
Shear Capacity....................................		k
Edge distance : bolt to plate.................	1.25	in
Number of Bolts in each Row...............	2	
Number of Bolt Rows........................	1	



Steel Base Plate

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Trash Enclosure Bearing Plate

GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	**Load Resistance Factor Design**
Governing Load Combinat	**+1.20D+0.70S+E**
Governing Load Case Typ	**Axial + Moment, L/2 < Eccentricity, Tensio**
Governing STRESS RATI(	**1.0**
Design Plate Size	**1'-3" x 1'-3" x 1 -1/2"**

Pu : Axial	0.000	k
Mu : Moment	0.000	k-ft

Mu : Max. Moment	7.256	k-in
fb : Max. Bending Stress	19.348	ksi
Fb : Allowable : Fy * Phi	32.400	ksi
Bending Stress Rati	0.597	
	Bending Stress OK	
fu : Max. Plate Bearing Stress	1.658	ksi
Fp : Allowable :	1.658	ksi
Bearing Stress Ratic	1.000	
	Bearing Stress OK	

Load Comb. : +1.40D

Axial Load Only, No Moment

Loading

Pu : Axial	3.080	k
Design Plate Height	15.000	in
Design Plate Width	15.000	in

Will be different from entry if partial bearing used.

A1 : Plate Area	225.000	in^2
A2: Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.175	in
" n "	4.175	in
X	0.000	in^2
Lambda	0.000	
n'	0.000	in
n' * Lambda	0.000	in
L = max(m, n, n")	4.175	in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	0.014	ksi
Stress Ratio	**0.008**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.119	k-in on 1" strip
fb : Actual	0.212	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.007**	

Load Comb. : +1.20D

Axial Load Only, No Moment

Loading

Pu : Axial	2.640	k
Design Plate Height	15.000	in
Design Plate Width	15.000	in

Will be different from entry if partial bearing used.

A1 : Plate Area	225.000	in^2
A2: Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.175	in
" n "	4.175	in
X	0.000	in^2
Lambda	0.000	
n'	0.000	in
n' * Lambda	0.000	in
L = max(m, n, n")	4.175	in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	0.012	ksi
Stress Ratio	**0.007**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.102	k-in on 1" strip
fb : Actual	0.182	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.006**	

Steel Base Plate

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Trash Enclosure Bearing Plate

Load Comb. : +1.20D+0.50S

Axial Load Only, No Moment

Loading

Pu : Axial	3.740	k
Design Plate Height	15.000	in
Design Plate Width	15.000	in

Will be different from entry if partial bearing used.

A1 : Plate Area	225.000	in^2
A2: Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.175	in
" n "	4.175	in
X	0.000	in^2
Lambda	0.000	
n'	0.000	in
n' * Lambda	0.000	in
L = max(m, n, n")	4.175	in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	0.017	ksi
Stress Ratio	**0.010**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.145	k-in on 1" strip
fb : Actual	0.258	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.008**	

Load Comb. : +1.20D+1.60S

Axial Load Only, No Moment

Loading

Pu : Axial	6.160	k
Design Plate Height	15.000	in
Design Plate Width	15.000	in

Will be different from entry if partial bearing used.

A1 : Plate Area	225.000	in^2
A2: Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.175	in
" n "	4.175	in
X	0.000	in^2
Lambda	0.000	
n'	0.000	in
n' * Lambda	0.000	in
L = max(m, n, n")	4.175	in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	0.027	ksi
Stress Ratio	**0.017**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.239	k-in on 1" strip
fb : Actual	0.424	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.013**	

Load Comb. : +1.20D+0.70S+E

Axial Load + Moment, Ecc. > L/2

Loading

Pu : Axial	4.180	k
Mu : Moment	27.600	k-ft
Eccentricity	79.234	in
A1 : Plate Area	225.000	in^2
A2 : Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Calculate plate moment from bearing . . .

max(m, n)	4.700	in
"A" : Bearing Length	2.209	in
Mpl : Plate Moment	0.605	k-in

Calculate plate moment from bolt tension . . .

Tension per Bolt	11.639	k
Tension : Allowable	0.000	k
Stress Ratio	**0.000**	
Dist. from Bolt to Col. Edge	2.925	in
Effective Bolt Width for Bending	11.700	in
Plate Moment from Bolt Tension	5.819	k-in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	*(set equal to Fp)*	
Stress Ratio	**1.000**	

Plate Bending Stresses

Mmax	7.256	k-in on 1" strip
fb : Actual	19.348	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.597**	

Steel Base Plate

DESCRIPTION: Trash Enclosure Bearing Plate

Load Comb. : +0.90D

Axial Load Only, No Moment

Loading

Pu : Axial	1.980	k
Design Plate Height	15.000	in
Design Plate Width	15.000	in
Will be different from entry if partial bearing used.		
A1 : Plate Area	225.000	in^2
A2: Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Distance for Moment Calculation

" m "	4.175	in
" n "	4.175	in
X	0.000	in^2
Lambda	0.000	
n'	0.000	in
n' * Lambda	0.000	in
L = max(m, n, n")	4.175	in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	0.009	ksi
Stress Ratio	**0.005**	

Plate Bending Stresses

Mmax = Fu * L^2 / 2	0.077	k-in on 1" strip
fb : Actual	0.136	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.004**	

Load Comb. : +0.90D+E

Axial Load + Moment, Ecc. > L/2

Loading

Pu : Axial	1.980	k
Mu : Moment	27.600	k-ft
Eccentricity	167.273	in
A1 : Plate Area	225.000	in^2
A2 : Support Area	225.000	in^2
sqrt(A2/A1)	1.000	

Calculate plate moment from bearing . . .

max(m, n)	4.700	in
"A" : Bearing Length	2.119	in
Mpl : Plate Moment	0.584	k-in

Calculate plate moment from bolt tension . . .

Tension per Bolt	12.180	k
Tension : Allowable	0.000	k
Stress Ratio	**0.000**	
Dist. from Bolt to Col. Edge	2.925	in
Effective Bolt Width for Bending	11.700	in
Plate Moment from Bolt Tension	6.090	k-in

Bearing Stresses

Fp : Allowable	1.658	ksi
fu : Max. Bearing Pressure	*(set equal to Fp)*	
Stress Ratio	**1.000**	

Plate Bending Stresses

Mmax	7.013	k-in on 1" strip
fb : Actual	18.701	ksi
Fb : Allowable	32.400	ksi
Stress Ratio	**0.577**	

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

LIC# : KW-06014122, Build:20.23.08.01 PCS STRUCTURAL SOLUTIONS

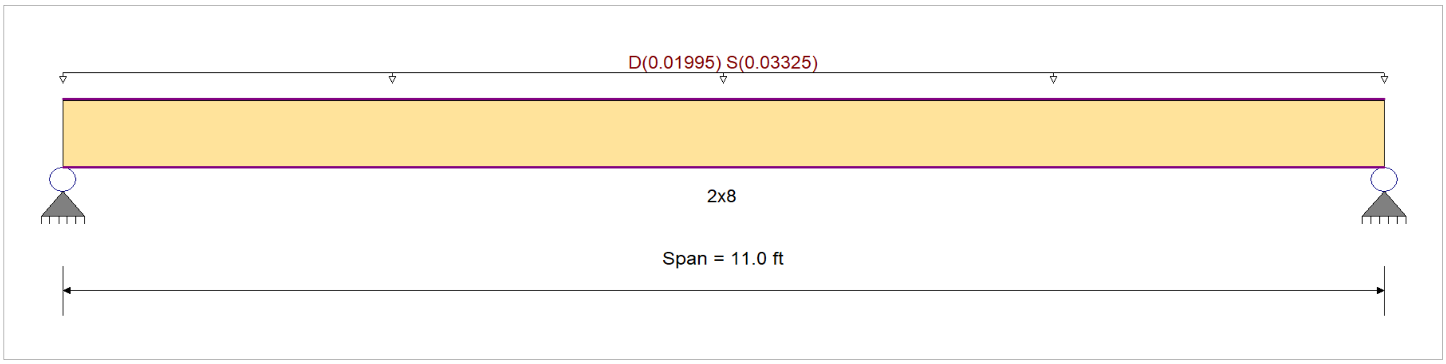
DESCRIPTION: Roof Joists

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : IBC 2021

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	900 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2021	Fb -	900 psi	Ebend- xx	1600 ksi
		Fc - Prll	1350 psi	Eminbend - xx	580 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.2	Fv	180 psi		
		Ft	575 psi	Density	31.21 pcf
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

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

Beam self weight calculated and added to loading
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 1.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.618** : 1	Maximum Shear Stress Ratio	=	**0.181** : 1
Section used for this span		**2x8**	Section used for this span		**2x8**
fb: Actual	=	767.36 psi	fv: Actual	=	37.53 psi
F'b	=	1,242.00 psi	F'v	=	207.00 psi
Load Combination		+D+S	Load Combination		+D+S
Location of maximum on span	=	5.500 ft	Location of maximum on span	=	10.398 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.145 in	Ratio =	913 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.242 in	Ratio =	546 >=240	Span: 1 : +D+S
Max Upward Total Deflection	0 in	Ratio =	0 <240	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios										Moment Values			Shear Values		
Segment Length	Span #	M	V	CD	CM	C_t	CLx	C_F	Cfu	C_i	C_r	M	fb	F'b	V	fv	F'v
D Only														0.0	0.00	0.0	0.0
Length = 11.0 ft	1	0.317	0.093	0.90	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.34	308.1	972.0	0.11	15.1	162.0
+D+S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 11.0 ft	1	0.618	0.181	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.84	767.4	1,242.0	0.27	37.5	207.0
+D+0.750S					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 11.0 ft	1	0.525	0.154	1.15	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.71	652.5	1,242.0	0.23	31.9	207.0
+0.60D					1.00	1.00	1.00	1.200	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 11.0 ft	1	0.107	0.031	1.60	1.00	1.00	1.00	1.200	1.00	1.00	1.00	0.20	184.9	1,728.0	0.07	9.0	288.0

Wood Beam

Project File: 23514 enercalc 2023-06-17 jmb.ec6

DESCRIPTION: Roof Joists

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.2415	5.540		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.306	0.306
Max Upward from Load Combinations	0.306	0.306
Max Upward from Load Cases	0.183	0.183
D Only	0.123	0.123
+D+S	0.306	0.306
+D+0.750S	0.260	0.260
+0.60D	0.074	0.074
S Only	0.183	0.183