PRCNC20231424

PCS Structural Solutions

FULL SIZED LEDGIBLE COLOR REPORT IS REQUIRED TO BE PROVIDED BY THE PERMITTEE ON SITE FOR ALL INSPECTIONS

Seattle 1011 Western Avenue, Suite 810 | Seattle, WA 98104 | 206.292.5076
Tacoma 1250 Pacific Avenue, Suite 701 | Tacoma, WA 98402 | 253.383.2797
Portland 101 SW Main Street, Suite 280 | Portland, OR 97204 | 503.232.3746
www.pcs-structural.com

STRUCTURAL CALCULATIONS

FOR

ARCO AMPM PUYALLUP C-STORE
1402 S MERIDIAN AVE
PUYALLUP, WASHINGTON

PREPARED BY
PCS STRUCTURAL *SOLUTIONS*

City of Puyallup
Building
ACCEPTED
JMontgomery
03/11/2024
9:25:57 AM

FEBRUARY 7, 2024
23-703

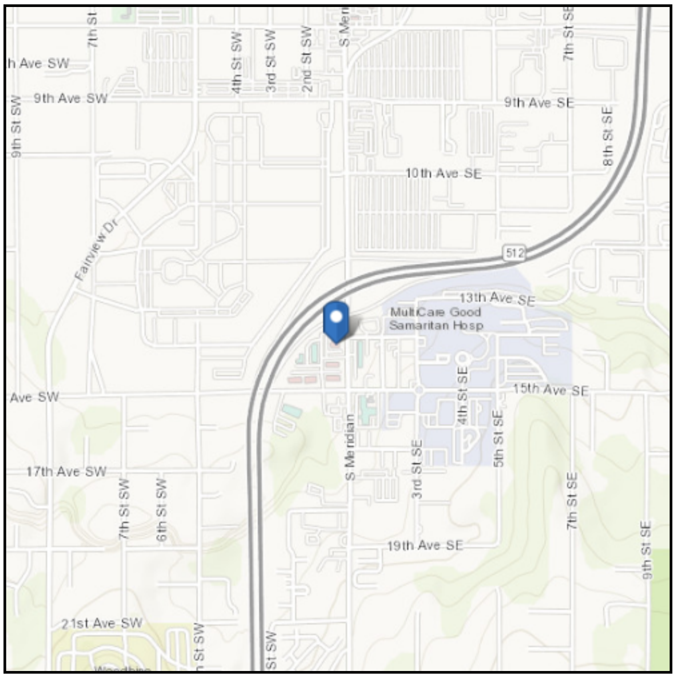
PRCNC20231424

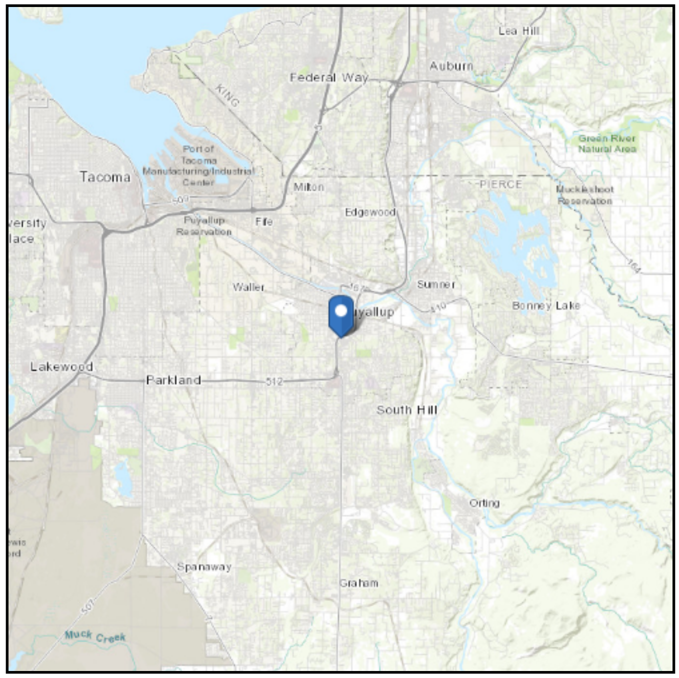
ASCE 7 Hazards Report

Address:
1402 S Meridian
Puyallup, Washington
98371

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: F - Site Response Analysis

Latitude: 47.178514
Longitude: -122.293971
Elevation: 47.922924404268436 ft (NAVD 88)





Wind

Results:

Wind Speed	97 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:	Tue Sep 19 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.

PRCNC20231424

Snow

Results:

Ground Snow Load, p_g : 18 lb/ft^2

Mapped Elevation: 47.9 ft

Data Source:

Date Accessed: Tue Sep 19 2023

Statutory requirements of the Authority Having Jurisdiction are not included.

Snow load values are mapped to a 0.5 mile resolution. This resolution can create a mismatch between the mapped elevation and the site-specific elevation in topographically complex areas. Engineers should consult the local authority having jurisdiction in locations where the reported ‘elevation’ and ‘mapped elevation’ differ significantly from each other.

PRCNC20231424

PCS Structural Solutions

Project:	ARCO Puyallup	Job Number:	23-703
Sheet:	of	Name:	BRT
Originating Office:	Portland	Date:	9/19/2023

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		

WIND DESIGN CRITERIA

BASIC WIND SPEED (V) =	97 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:	0-15 FT	(See ASCE 7-16 Section 26.2 - Definitions)
ELEVATION:	48 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):	0.25:12	(Enter vertical rise in 12 horizontal units) θ (degrees): 1.19

SEISMIC DESIGN CRITERIA

SITE CLASS:	F	(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):	2.5	(Per ASCE 7-16 Table 12.2-1)

From geotech report

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

LATITUDE:	47.179	S_S =	1.268	F_a =	1.000
LONGITUDE:	-122.294	S_1 =	0.437	F_v =	1.863

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: YES

SEE SOILS REPORT BY KRAZAN AND ASSOCIATES, INC. DATED MAY 6, 2022

BEARING:	1500 PSF		
ACTIVE:	35 PCF	MINIMUM FOOTING DIMENSIONS:	
PASSIVE:	300 PCF	CONTINUOUS:	1'-4"
COEFFICIENT OF FRICTION:	0.35	SPREAD:	1'-6"
		FROST DEPTH:	1'-6"
PILE TYPE:	8" STL PIPE		
VERTICAL CAPACITY :	25 KIPS	LATERAL CAPACITY:	N/A
UPLIFT CAPACITY:	N/A	SIZE:	N/A

PRCNC20231424

Project:	ARCO Puyallup	Job Number:	23-703
Sheet:	of	Name:	BRT
Originating Office:	Portland	Date:	09/19/23

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	-	-

PRCNC20231424

PCS Structural Solutions

Project: ARCO Puyallup | Job Number: 23-703

Sheet: ____ of ____ | Name: BRT

Originating Office: Portland | Date: 09/19/23

DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	97 MPH	MEAN ROOF HEIGHT:	15 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):	0.3:12
		θ (degrees):	1.19

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	N/A	N/A	N/A	2.1
	0.50	N/A	N/A	N/A	
	≥1.0	N/A	N/A	N/A	
Normal to Ridge for θ < 10° and Parallel to Ridge for All θ	h/L:	Horizontal Distance from Windward Edge	External Pressures (q*(GC_p)): Positive Pressure	Negative Pressure	Internal Pressures (±q_i*(GC_{pi})) All Roofs
	≤0.5	0 to h	-1.8	-8.9	2.1
		h to 2h		-5.0	
		>2h		-3.0	
	≥1.0	0 to h/2	-1.8	-12.9	
		>h/2		-6.9	

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	7.9	0-1	-5.0	-6.9	2.1
20	1.00	8.6	2	-3.0		
25	1.00	9.2	≥4	-2.0		
30	1.00	9.7				
40	1.00	10.6				
50	1.00	11.3				
60	1.00	11.8				
70	1.00	12.4				
80	1.00	12.9				
90	1.00	13.3				
100	1.00	13.8				
120	1.00	14.5				
140	1.00	15.1				
160	1.00	15.7				
180	1.00	16.3				
200	1.00	16.7				
250	1.00	17.8				
300	1.00	18.8				
350	1.00	19.6				
400	1.00	20.4				
450	1.00	21.1				
500	1.00	21.7				

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 = 11.6 PSF

PCS Structural Solutions

Project: ARCO Puyallup | Job Number: 23-703

Sheet: ________ of ________ | Name: BRT

Originating Office: Portland | Date: 09/19/23

DESIGN CRITERIA - WIND

BASIC WIND SPEED (V):	97 MPH	MEAN ROOF HEIGHT:	15 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):	0.3:12
		θ (degrees):	1.19

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'	1	2	3	N/A	N/A
10 SF	16.0	-16.0	-21.9	-28.9	-39.4	N/A	N/A
20 SF	16.0	-16.0	-20.5	-27.0	-35.7	N/A	N/A
50 SF	16.0	-16.0	-18.5	-24.6	-30.7	N/A	N/A
100 SF	16.0	-16.0	-17.1	-22.7	-27.0	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'	1	2	3	N/A	N/A
10 SF	16.0	16.0	-16.0	-18.4	-19.8	-19.8	-26.8	-37.3	N/A	N/A
20 SF	16.0	16.0	-16.0	-17.2	-19.5	-19.5	-24.3	-32.9	N/A	N/A
50 SF	16.0	16.0	-16.0	-16.0	-19.0	-19.0	-21.0	-27.2	N/A	N/A
100 SF	16.0	16.0	-16.0	-16.0	-18.6	-18.6	-18.6	-22.9	N/A	N/A
500 SF	16.0	16.0	-16.0	-16.0	-17.8	-17.8	-16.0	-16.0	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

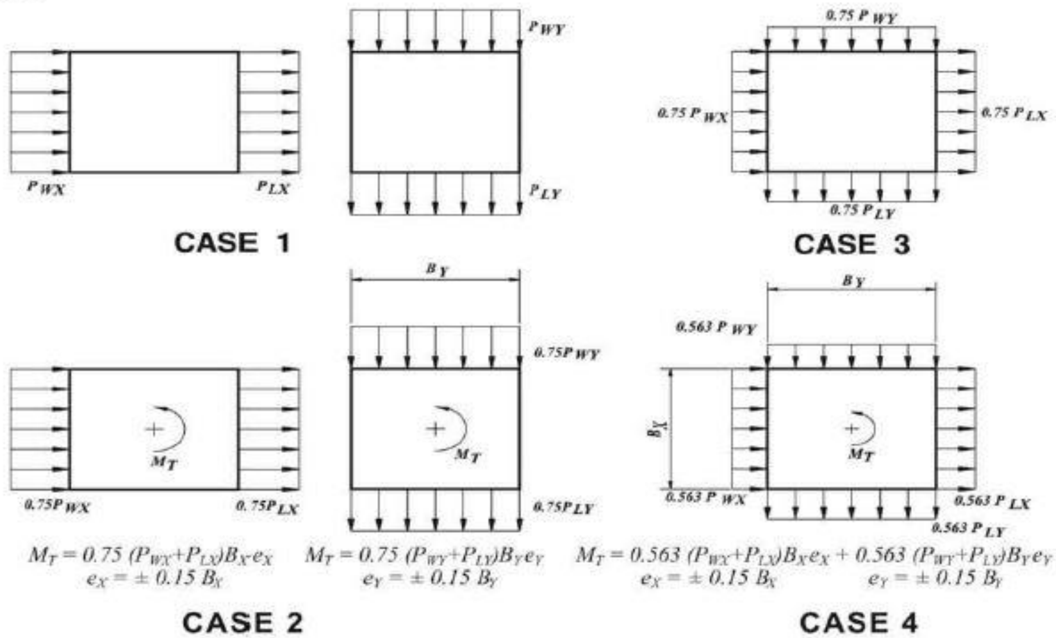
K_{ht} = 1.00

q_h = 11.6 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

PRCNC20231424

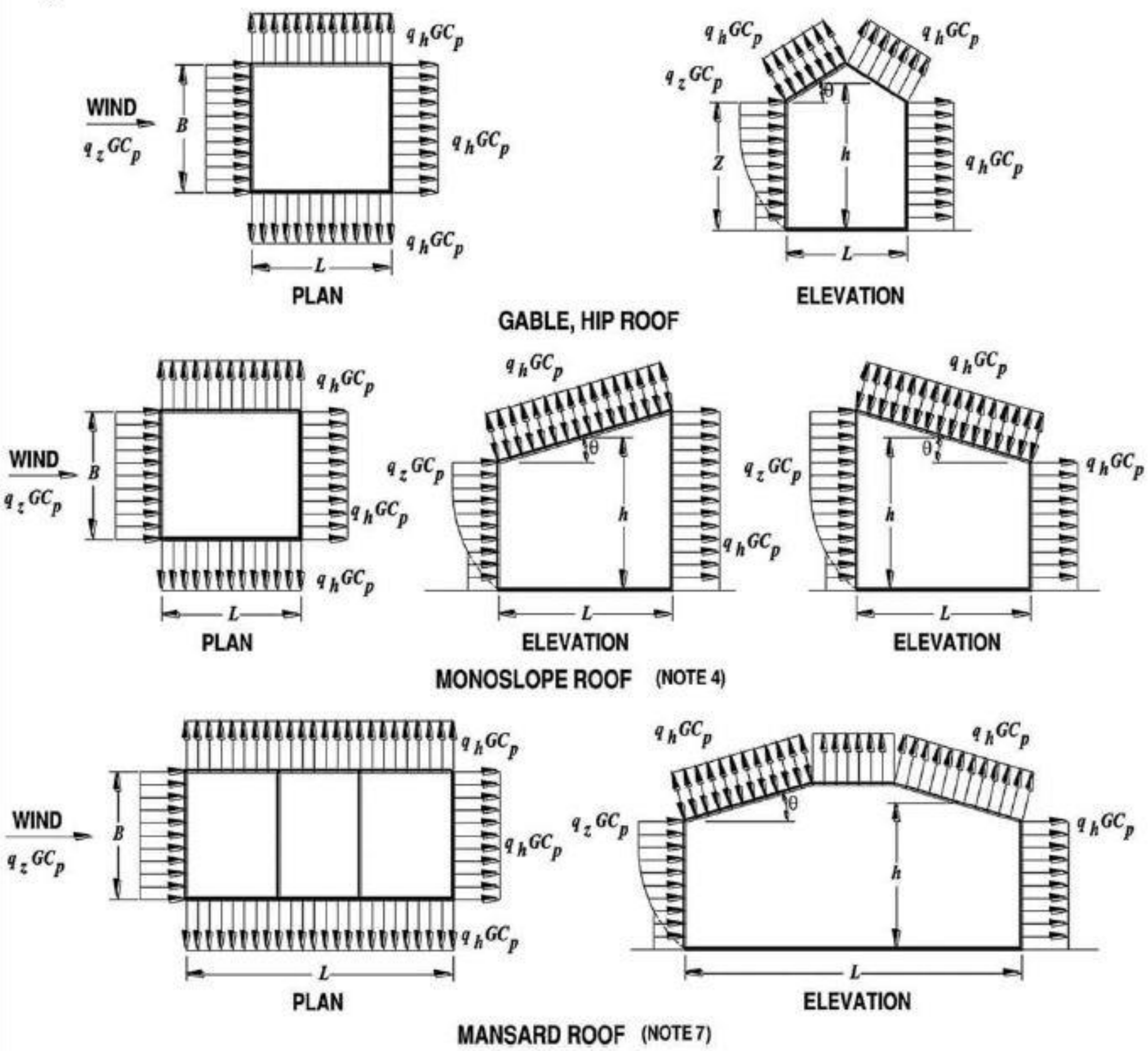
PCS Structural Solutions

Project:	ARCO Puyallup	Job Number:	23-703
Sheet:	of	Name:	BRT
Originating Office:	Portland	Date:	09/19/23

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

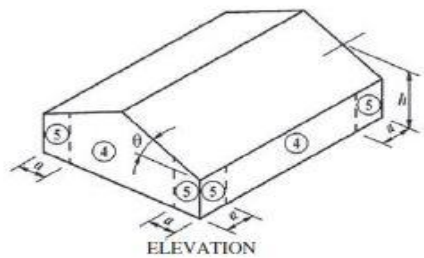
B = Horizontal dimension of building, in ft (m), measured normal to wind direction.
L = Horizontal dimension of building, in ft (m), measured parallel to wind direction.
h = Mean roof height, in ft (m), except that eave height shall be used for $\theta \leq 10$ degrees.
z = Height above ground, in ft (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

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

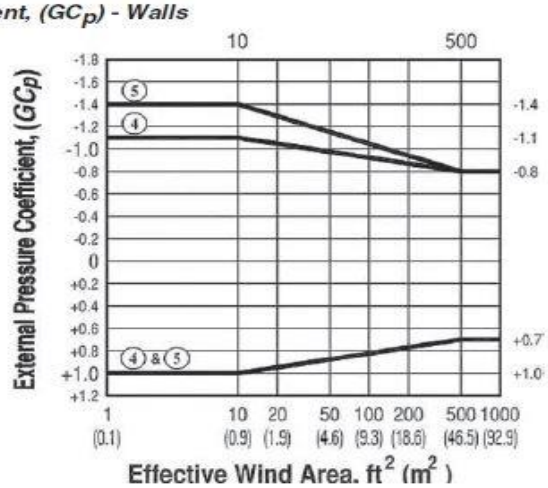
a = 10% of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

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 ft^2 (m^2).
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 [$h \le 60$ ft ($h \le 18.3$ m)]: External Pressure Coefficients, (GC_p), for Enclosed and Partially Enclosed Buildings—Walls

PRCNC20231424

DESIGN CRITERIA - SEISMIC

ASCE 7-16 SECTION 12.8 - EQUIVALENT LATERAL FORCE PROCEDURE

RISK CATEGORY:	II	LATITUDE:	47.179
SITE CLASS:	F	LONGITUDE:	-122.294
IMPORTANCE FACTOR (I_E):	1	S_S =	1.268
STRUCTURAL SYSTEM (R):	6.5	S_1 =	0.437
OVERSTRENGTH FACTOR (Ω_o):	2.5	F_a =	1.000
		F_v =	1.863

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

$S_{MS} = F_a{*}S_S =$ 1.268 $S_{M1} = F_v{*}S_1 =$ 0.814

Section 11.4.5 - Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3{*}S_{MS} =$ 0.845 $S_{D1} = 2/3{*}S_{M1} =$ 0.543

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 =	15 FT
x =	0.75
$T_a = C_t{*}h_n^{\,x}$ =	0.152

ASCE 7-16 SECTION 12.8.1.1 - SEISMIC RESPONSE COEFFICIENT

GENERAL EQUATION:	$C_S = S_{DS}/(R/I) =$	0.130	<--CONTROLS	EQ. 12.8-2
MAXIMUM:	CS = SD1/(T*(R/I)) =	0.548		EQ. 12.8-3
MINIMUM:	$C_S = 0.044{*}S_{DS}{*}I > 0.01 =$	0.037		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

$V = C_S{*}W =$ **0.130*W**

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

PRCNC20231424

LATERAL

Lateral

Seismic

Roof weight

Area = 85 ft (40.75 ft) + 1.25 ft (19.66 ft) = 3488 ft^2

weight = 3488 ft^2 (20 psf) = 69.8 k

wall weight N-S

$$\left[\frac{21.83 ft^2}{2(15.25 ft)}(2)(85 ft)\right](10 psf) = 26.6\ k$$

Wall weight E-W

$$\left[\frac{21.83 ft^2}{2(15.25 ft)}(2)(40.8 ft)\right](10 psf) = 12.8\ k$$

Total N-S

69.8 kips + 26.6 kips = 96.4 k

Total E-W

69.8 kips + 12.8 k = 82.6 k

Base Shear

N-S: 0.13 (96.4 kips) = **12.6 k**

E-W: 0.13 (82.6 kips) = **10.8 k**

Wind

$q_h = 0.00256\, k_z k_{zt} k_d k_e V^2 = 18.4$ psf

$k_{zt} = 1.0$ $V = 97$ mph
$k_z = 0.90$
$k_d = 0.85$
$k_e = 1.0$

$G = 0.85$ $GC_{pi} = +/-0.18$

$L/B = \frac{40.8}{85\text{ ft}} = 0.47$ $L/B = \frac{85}{40.8} = 2.09$

$P = q\,GC_p + q_i(GC_{pi})$

C_p: → windward → 0.8
→ leeward → -0.5
→ sidewal → -0.7

} N-S

$\therefore$ $P_{windward} = 18.4psf[(0.85)(0.8) + 0.18] = 15.8$ psf
$P_{leeward} = 18.4psf[(0.85)(-0.5) + 0.18] = -4.5$ psf
$P_{sidewall} = 18.4psf[(0.85)(-0.7) + 0.18] = -7.6$ psf

E-W

C_p: → windward → 0.8
→ leeward → -0.3
→ sidewal → -0.7
} E-W

$P_{windward} = 18.4[(0.85)(0.8) + 0.18] = 15.8$ psf
$P_{leeward} = 18.4[(0.85)(-0.3) + 0.18] = -1.4$ psf
$P_{sidewall} = 18.4[(0.85)(-0.7) + 0.18] = -7.6$ psf

Parapets

$P_p = q_p(GC_{pn})$ (27.3-3)

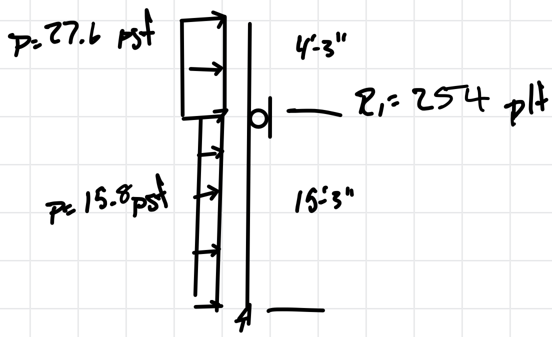
$GC_{pn} = 1.5$ windward parapet
$GC_{pn} = 1.0$ leeward parapet

$P_p = 18.4(1.5) = 27.6$ psf (windward)
$= 18.4(-1.0) = -18.4$ psf (leeward)

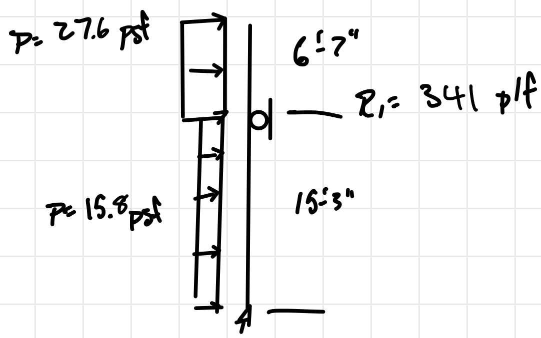
Wind

∴ N-S wind loading (windward)

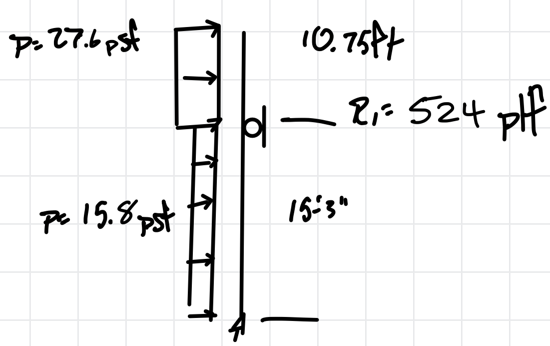
Typical Section



Mid parapet



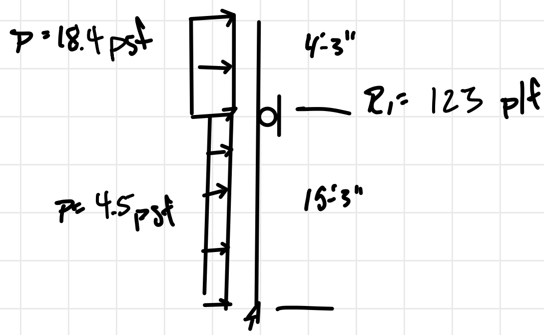
High parapet



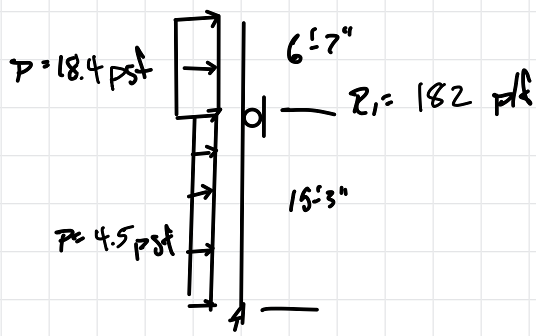
PCS Structural Solutions

leeward loading

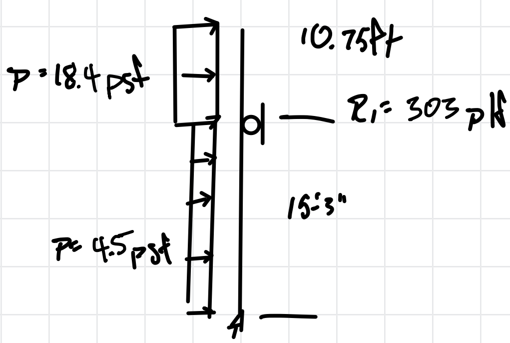
Typical Section



Mid parapet



High parapet



PRCNC20231424

PCS Structural Solutions

Project: ____________ Job No: ____________
Subject: ____________ Sheet ____________ Name: ____________
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ____________

Total N-S Base Shear

$V = 254\ plf(17.4\ ft)(2) + 123\ plf(85\ ft) + 341\ plf(15.3\ ft)(2) + 524\ plf(19.7\ ft)$

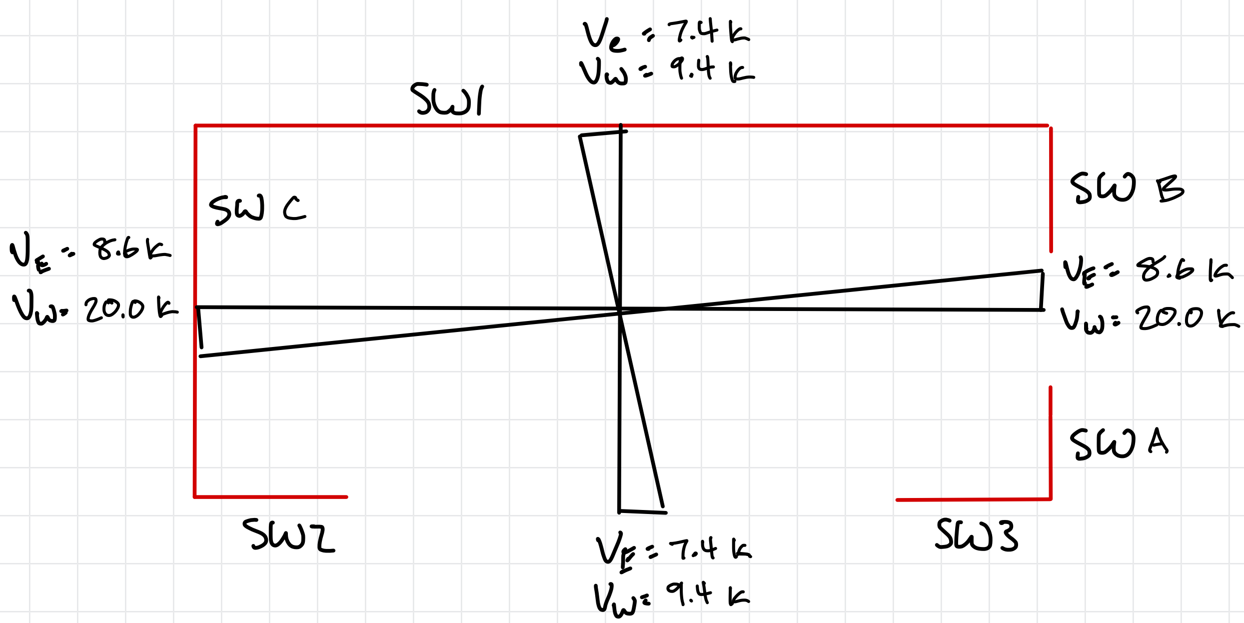
$= \underline{40.1\ kips} \rightarrow$ controls

Total E-W Base Shear

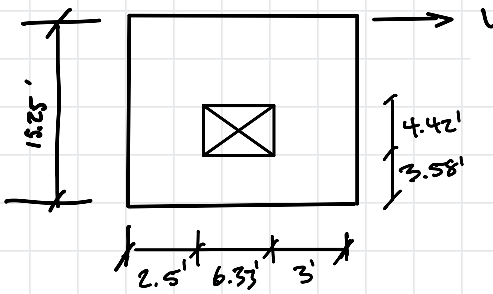
$V = (254\ plf + 123\ plf)(16\ ft) + (341 + 182\ plf)(40.3 - 16\ ft)$

$= \underline{18.7\ kips} \rightarrow$ controls

Forces shown are in ultimate



SW A & B



$V_W = 20.0\ k/2 = 10.0k$

FORCE TRANSFER AROUND OPENING

$V = 10.0(0.6) = 6.0k$

SEE SPREADSHEET

$v = 1091$ PLF → USE DBL SIDED 15/32" w/ 8d @ 4" O.C.

$v_{ALLOW} = \frac{1205 \text{ PLF}}{2}(2)\left[1-(0.5-0.43)\right] = 1120$ PLF

STRAP FORCE = 2470 LB → USE CS14

UPLIFT = 7735 LB

HD = $7735 - 0.6(10 \text{ PSF})(19.5')(12')/2 = 7033$ LB

→ USE HDU8 w/ (3) STUD POST & SSTB 28

PRCNC20231424

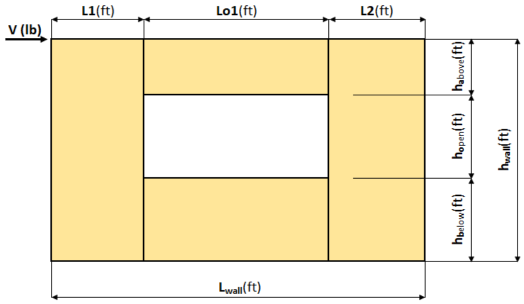
Force Transfer Around Openings Calculator

ONE OPENING

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

Project Information

Code:		**Date:**	9/22/2023
Designer:	BRT		
Client:			
Project:	Arco Puyallup		
Wall Line:	A & B		



Shear Wall Calculation Variables

V	6000 lbf		Opening 1		Adj. Factor Method =	2bs/h
L1	3.00 ft	ha1	7.25 ft		Wall Pier Aspect Ratio	Adj. Factor
L2	2.50 ft	ho1	4.42 ft		P1=ho1/L1= 1.47	N/A
h_{wall}	15.25 ft	hb1	3.58 ft		P2=ho2/L2= 1.77	N/A
L_{wall}	11.83 ft	Lo1	6.33 ft			

1. Hold-down forces: H = Vh$_{wall}$/L$_{wall}$ 7735 lbf

2. Unit shear above + below opening

First opening: va1 = vb1 = H/(ha1+hb1) = 714 plf

3. Total boundary force above + below openings

First opening: O1 = va1 x (Lo1) = 4521 lbf

4. Corner forces

F1 = O1(L1)/(L1+L2) = 2466 lbf

F2 = O1(L2)/(L1+L2) = 2055 lbf

5. Tributary length of openings

T1 = (L1*Lo1)/(L1+L2) = 3.45 ft

T2 = (L2*Lo1)/(L1+L2) = 2.88 ft

6. Unit shear beside opening

V1 = (V/L)(L1+T1)/L1 = 1091 plf

V2 = (V/L)(T2+L2)/L2 = 1091 plf

Check V1*L1+V2*L2=V? 6000 lbf **OK**

7. Resistance to corner forces

R1 = V1*L1 = 3273 lbf

R2 = V2*L2 = 2727 lbf

8. Difference corner force + resistance

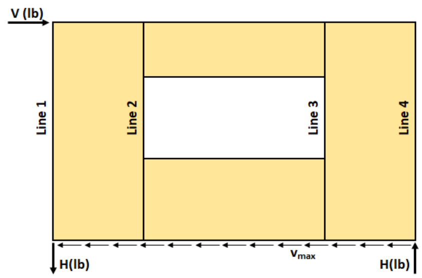
R1-F1 = 807 lbf

R2-F2 = 672 lbf

9. Unit shear in corner zones

vc1 = (R1-F1)/L1 = 269 plf

vc2 = (R2-F2)/L2 = 269 plf



Check Summary of Shear Values for One Opening

Line 1: vc1(ha1+hb1)+V1(ho1)=H?		2913	4822	7735 lbf
Line 2: va1(ha1+hb1)-vc1(ha1+hb1)-V1(ho1)=0?	7735	2913	4822	0
Line 3: vc2(ha1+hb1)+V2(ho1)=H?		2913	4822	7735 lbf

Design Summary

Req. Sheathing Capacity	1091 plf	**4-Term Deflection**		**3-Term Deflection**	
Req. Strap Force	2466 lbf	**4-Term Story Drift %**		**3-Term Story Drift %**	
Req. HD Force (H)	7735 lbf		See Page 2		See Page 3
Req. Shear Wall Anchorage Force (v_{max})	507 plf				

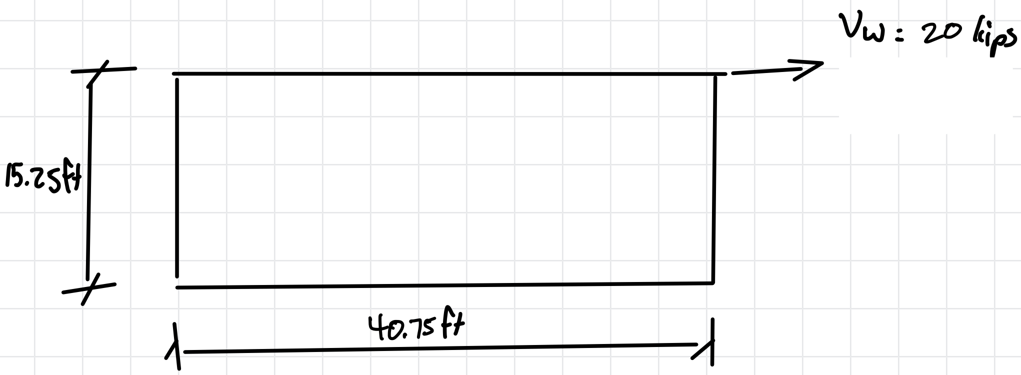
APA Disclaimer

PRCNC20231424

PCS Structural *Solutions*

Project: ______ Job No: ______
Subject: ______ Sheet ______ Name: ______
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

SW C



$V_W = \dfrac{20 \text{ kips}(0.6)}{40.75 \text{ ft}} = 294 \text{ plf}$

USE 15/32" w/ 8d @ 6" O.C.

$\dfrac{785}{2}\left(1-(0.5-0.43)\right) = 365.1 \text{ plf}$

$\dfrac{560}{2}\left(1-(0.5-0.43)\right) = 260.4 \text{ plf}$ ✓ GOOD

$M_{OVT} = 20 \text{ kips}(0.6)(15.25\text{ft}) = 183 \text{ kip ft}$

$M_{RES} = 19.5\text{ft}(10\text{psf})(40.75\text{ ft})^2/2\,(0.6) = 97.1 \text{ kip ft}$

$HD = \dfrac{183 \text{ kip}' - 97.1 \text{ kip}'}{40.75 \text{ ft}} = 2.1 \text{ kip}$ → USE HDU4 w/ SSTB16

PRCNC20231424

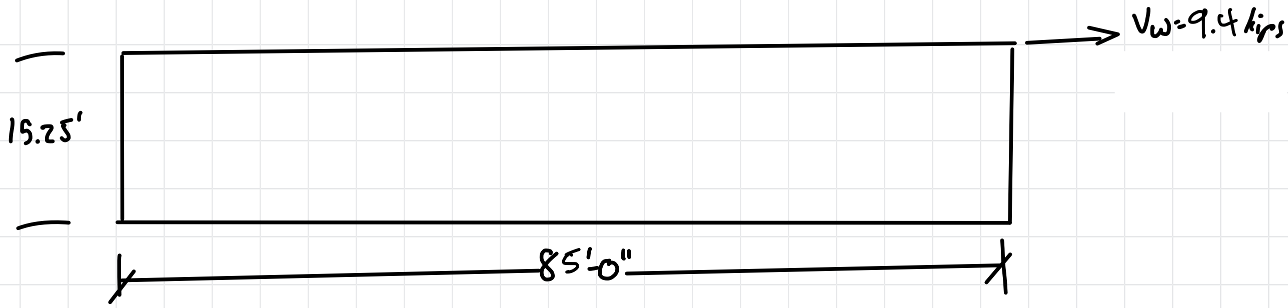
PCS Structural Solutions

Project: ________ Job No: ________

Subject: ________ Sheet ________ Name: ________

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ________

SW1



$$V_w = \frac{9.4\text{ kips}(0.6)}{85\text{ ft}} = 66\text{ plf} \quad \checkmark \text{ GOOD}$$

$$M_{OT} = 9.4\text{ kips}(0.6)(15.25\text{ft}) = 86.0\text{ kip}\cdot\text{ft}$$

$$M_{RES} = \left[10\text{psf}(19.5\text{ft}) + 20\text{psf}(20\text{ft})\right](85\text{ft})^2/2(0.6) = 1290\text{ kip}\cdot\text{ft} \quad \checkmark \text{ GOOD}$$

$$M_{OT} = 7.4\text{ kips}(0.7)(15.25\text{ft}) = 79\text{ kip}\cdot\text{ft} \quad \checkmark \text{ GOOD By inspection}$$

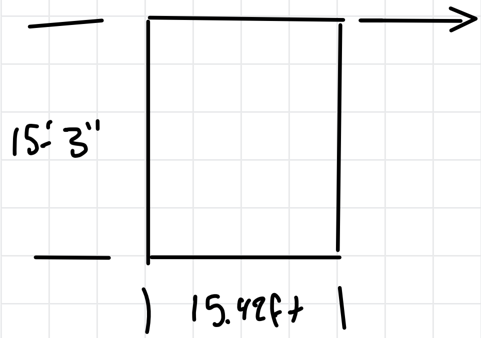
PRCNC20231424

PCS Structural Solutions

Project: ______ Job No: ______
Subject: ______ Sheet ______ Name: ______
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

SW2 & SW3

$V_w = 9.4k/2 = 4.7k$



$V_w = \frac{4.7 \text{ kip}(0.6)}{15.42 \text{ ft}} = 183 \text{ plf}$ } USE 15/32" w/ 8d @ 6" O.C.

$M_{OUT} = 4.7 \text{ kip}(15.25 \text{ ft})(0.6) = 43 \text{ kip-ft}$

$M_{RES} = [19.5 \text{ ft}(10 \text{ psf}) + 20 \text{ ft}(20 \text{ psf})](15.42 \text{ ft})^2/2 = 42.4 \text{ kip-ft}$ ✓ OK

$M_{OUT} = 3.7 \text{ kip}(15.25 \text{ ft})(0.7) = 39.5 \text{ kip-ft}$

$M_{RES}\ (19.5 \text{ ft}(10 \text{ psf}) + 20 \text{ ft}(20 \text{ psf})](15.42 \text{ ft})^2/2\ (0.6 - 0.14(1.16)) = 31.0 \text{ kip-ft}$

Provide HDU2

FOUNDATION

PILES:

TOTAL GRAVITY LOAD PER FOOT UNDER BRG WALLS:

$$w_D = 20\,psf\left(40.75ft/2\right) + 10\,psf\left(26ft\right) + 150\,pcf\left(1.5ft\right)\left(1.75ft\right) = 1.1\ klf$$

$$w_S = 25\,psf\left(40.75ft/2\right) = 0.6\,klf$$

$$w_{D+S} = 1.7\ klf$$

8" ϕ PILE CAPACITY = 25^k, $25^k / 1.7\,klf = 14.7\,ft/\text{PILE}$ $\therefore$ $\boxed{\text{SPACE PILES @ 14ft O.C. MAX}}$

EXTERIOR GRADE BEAMS:

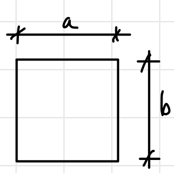
SEE ENERCALC FOR MOMENT + DEFLECTION CALCS...

$V_u = \underline{\underline{19.7^k}}$ ← FROM ENERCALC

$$\phi V_n = 2\lambda\sqrt{f'_c}\,b_w\,d = 2\left(1.0\right)\sqrt{3000psi}\left(18in\right)\left(12in\right) = \underline{\underline{23.7^k}} > 19.7^k \ \checkmark ok$$

INTERIOR GRADE BEAMS TIES:

[ACI 318-14, § 18.13.3.2]



$a + b \geq \left(14ft \times 12in/ft\right)/20 = 8.4in$

$\leq 18in$

TIE SPACING < 12"/2 = 6" ← CONTROLS

OR < 12"

$\therefore$ a = 12", b = 18" ✓ok

$T_u / C_u = 40.1^k / 8$ GRADE BEAMS $= 5^k$ $\therefore$ AXIAL TENSION / COMPRESSION CAPACITY OF GRADE BEAM ✓ok BY INSPECTION

↑ TOTAL BASE SHEAR FROM WIND

[IBC 2018, §1809.13]

$$T_u / C_u = \underbrace{0.845}_{S_{DS}}\ \underbrace{\left(25^k\right)}_{\text{PILE CAPACITY}} = \underline{\underline{21.2^k}} \quad \text{OR} \quad 0.25\left(25^k\right) = 6.25^k$$

$$\phi T_n = \left(0.9\right) 60ksi\left(6\ \text{BARS}\right)\left(0.44in^2\right) = \underline{\underline{143^k}} > 21.2^k \ \checkmark ok$$

$$\phi P_n = 0.75\left(0.85\right)\left[\left(12in \times 18in\right) - \left(6 \times 0.44in^2\right)\right]\left(3ksi\right) = \underline{\underline{408^k}} > 21.2^k \ \checkmark ok$$

PRCNC20231424

Structural Solutions

Project: ARCO PUYALLUP
Job No: 23-707
Subject: FOUNDATION
Sheet
Name: BRT
Originating Office: ☐ Seattle ☐ Tacoma ☒ Portland
Date: 9-21-23

FOUNDATION

GLOBAL SLIDING:

$V_{max} = 40.1^k$

(GRADE BMS ONLY, CONS.) $\longrightarrow D_{RES} = (1.5ft)(1.5ft)(250ft)(0.15kcf) + (1ft \times 1.5ft)(410ft)(.15kcf) = 176.6^k$

$F_s = 176.6^k \overset{\mu}{(0.35)} = \underline{\underline{61.8^k}} > 40.1^k$ ✓ OK

$F_s = \frac{61.8^k}{40.1^k} = \underline{\underline{1.54}} > 1.5$ ✓ OK

PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: 23703 Enercalc BRT.ec6

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

DESCRIPTION: Exterior Concrete Grade Beams

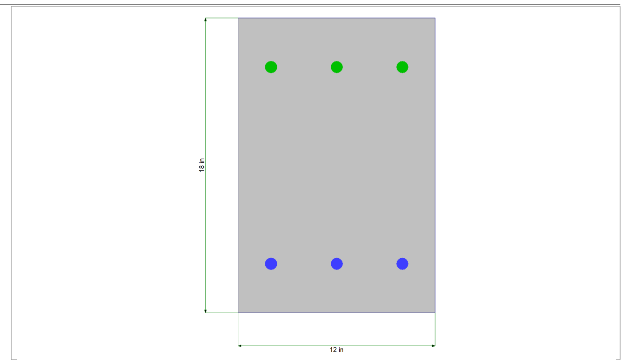
CODE REFERENCES

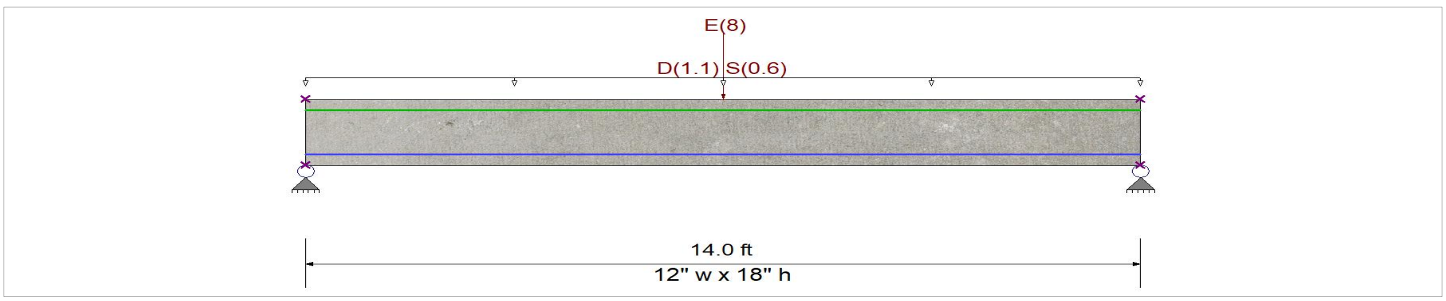
Calculations per ACI 318-19, IBC 2021, ASCE 7-16
Load Combination Set : IBC 2018

General Information

f'c =	3.0 ksi	ϕ Phi Values Flexure :	0.90
fr = $f'c^{1/2}$ * 7.50 =	410.792 psi	Shear :	0.750
ψ Density =	145.0 pcf	β_1 =	0.850
λ LtWt Factor =	1.0		
Elastic Modulus =	3,122.0 ksi	Fy - Stirrups	40.0 ksi
fy - Main Rebar =	60.0 ksi	E - Stirrups =	29,000.0 ksi
E - Main Rebar =	29,000.0 ksi	Stirrup Bar Size #	3
		Number of Resisting Legs Per Stirrup =	2

Seismic Design Category = A





Cross Section & Reinforcing Details

Rectangular Section, Width = 12.0 in, Height = 18.0 in
Span #1 Reinforcing....
3-#6 at 3.0 in from Bottom, from 0.0 to 14.0 ft in this span 3-#6 at 3.0 in from Top, from 0.0 to 14.0 ft in this span

Point Load : E = 8.0 k @ 7.0 ft, (From S/W Comp. Force)
Uniform Load : D = 1.10, S = 0.60 k/ft, Tributary Width = 1.0 ft, (Building Gravity Loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.867** : 1	
Section used for this span	**Typical Section**	
Mu : Applied	70.579	k-ft
Mn * Phi : Allowable	81.391	k-ft
Location of maximum on span	6.987	ft
Span # where maximum occurs	Span # 1	

Maximum Deflection

Max Downward Transient Deflection	0.090 in	Ratio =	1865	>=360.0	E Only
Max Upward Transient Deflection	0.000 in	Ratio =	0	<360.0	E Only
Max Downward Total Deflection	0.292 in	Ratio =	575	>=240.0	Span: 1 : +D+0.750S+0.5250E
Max Upward Total Deflection	0.000 in	Ratio =	0	<240.0	Span: 1 : +D+0.750S+0.5250E

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	12.950	12.950
Max Upward from Load Combinations	12.950	12.950
Max Upward from Load Cases	7.700	7.700
D Only	7.700	7.700
+D+S	11.900	11.900
+D+0.750S	10.850	10.850
+D+0.70E	10.500	10.500
+D+0.750S+0.5250E	12.950	12.950

Concrete Beam

Project File: 23703 Enercalc BRT.ec6

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

DESCRIPTION: Exterior Concrete Grade Beams

Vertical Reactions

Support notation : Far left is #1

Load Combination	Support 1	Support 2
+0.60D	4.620	4.620
+0.60D+0.70E	7.420	7.420
S Only	4.200	4.200
E Only	4.000	4.000

Shear Stirrup Requirements

Between 0.00 to 5.05 ft, Phi*lambda*sqrt(f'c)*bw*d < Vu <= Phi*Vc, Req'd Vs = Min per 9.6.3.1, use #3 stirrups spaced at 7.000 in
Between 5.07 to 8.93 ft, Vu <= Phi*lambda*sqrt(f'c)*bw*d, Req'd Vs = Not Reqd per 9.3.6.1, Stirrups are not required.
Between 8.95 to 13.97 ft, Phi*Vc < Vu, Req'd Vs = 1.391, use #3 stirrups spaced at 7.000 in

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu Actual	(k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+0.70S+E	1	0.00	15.00	16.18	16.18	0.00	1.00	14.79	Phi*Vc < Vu	1.391	28.9	7.5
+1.20D+0.70S+E	1	0.15	15.00	15.91	15.91	2.46	1.00	14.79	Phi*Vc < Vu	1.125	28.9	7.5
+1.20D+0.70S+E	1	0.31	15.00	15.65	15.65	4.87	1.00	14.79	Phi*Vc < Vu	0.8590	28.9	7.5
+1.20D+0.70S+E	1	0.46	15.00	15.38	15.38	7.24	1.00	14.79	Phi*Vc < Vu	0.5928	28.9	7.5
+1.20D+0.70S+E	1	0.61	15.00	15.12	15.12	9.58	1.00	14.79	Phi*Vc < Vu	0.3266	28.9	7.5
+1.20D+0.70S+E	1	0.77	15.00	14.85	14.85	11.87	1.00	14.79	Phi*Vc < Vu	0.06034	28.9	7.5
+1.20D+0.70S+E	1	0.92	15.00	14.58	14.58	14.12	1.00	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.07	15.00	14.32	14.32	16.33	1.00	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.22	15.00	14.05	14.05	18.50	0.95	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.38	15.00	13.78	13.78	20.63	0.84	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.53	15.00	13.52	13.52	22.72	0.74	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.68	15.00	13.25	13.25	24.77	0.67	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.84	15.00	12.99	12.99	26.77	0.61	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	1.99	15.00	12.72	12.72	28.74	0.55	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.14	15.00	12.45	12.45	30.67	0.51	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.30	15.00	12.19	12.19	32.55	0.47	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.45	15.00	11.92	11.92	34.40	0.43	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.60	15.00	11.65	11.65	36.20	0.40	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.75	15.00	11.39	11.39	37.96	0.37	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	2.91	15.00	11.12	11.12	39.68	0.35	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.06	15.00	10.86	10.86	41.37	0.33	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.21	15.00	10.59	10.59	43.01	0.31	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.37	15.00	10.32	10.32	44.61	0.29	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.52	15.00	10.06	10.06	46.17	0.27	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.67	15.00	9.79	9.79	47.68	0.26	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.83	15.00	9.52	9.52	49.16	0.24	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	3.98	15.00	9.26	9.26	50.60	0.23	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.13	15.00	8.99	8.99	51.99	0.22	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.28	15.00	8.73	8.73	53.35	0.20	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.44	15.00	8.46	8.46	54.66	0.19	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.59	15.00	8.19	8.19	55.94	0.18	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.74	15.00	7.93	7.93	57.17	0.17	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	4.90	15.00	7.66	7.66	58.36	0.16	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	5.05	15.00	7.39	7.39	59.52	0.16	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	5.20	15.00	7.13	7.13	60.63	0.15	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	5.36	15.00	6.86	6.86	61.70	0.14	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	5.51	15.00	6.60	6.60	62.73	0.13	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	5.66	15.00	6.33	6.33	63.72	0.12	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	5.81	15.00	6.06	6.06	64.66	0.12	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	5.97	15.00	5.80	5.80	65.57	0.11	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.12	15.00	5.53	5.53	66.44	0.10	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.27	15.00	5.26	5.26	67.26	0.10	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.43	15.00	5.00	5.00	68.05	0.09	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.58	15.00	4.73	4.73	68.79	0.09	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.73	15.00	4.47	4.47	69.50	0.08	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	6.89	15.00	4.20	4.20	70.16	0.07	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.04	15.00	-4.07	4.07	70.48	0.07	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0

PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:

Concrete Beam

Project File: 23703 Enercalc BRT.ec6

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

DESCRIPTION: Exterior Concrete Grade Beams

Detailed Shear Information

Load Combination	Span Number	Distance (ft)	'd' (in)	Vu (k) Actual	Vu (k) Design	Mu (k-ft)	d*Vu/Mu	Phi*Vc (k)	Comment	Phi*Vs (k)	Phi*Vn (k)	Spacing (in) Req'd
+1.20D+0.70S+E	1	7.19	15.00	-4.33	4.33	69.83	0.08	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.34	15.00	-4.60	4.60	69.15	0.08	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.50	15.00	-4.87	4.87	68.43	0.09	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.65	15.00	-5.13	5.13	67.66	0.09	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.80	15.00	-5.40	5.40	66.86	0.10	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	7.96	15.00	-5.66	5.66	66.01	0.11	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.11	15.00	-5.93	5.93	65.12	0.11	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.26	15.00	-6.20	6.20	64.19	0.12	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.42	15.00	-6.46	6.46	63.23	0.13	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.57	15.00	-6.73	6.73	62.22	0.14	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.72	15.00	-7.00	7.00	61.17	0.14	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	8.87	15.00	-7.26	7.26	60.08	0.15	10.28	Vu <= Phi*lambd	ot Reqd pe	10.3	0.0
+1.20D+0.70S+E	1	9.03	15.00	-7.53	7.53	58.94	0.16	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.18	15.00	-7.79	7.79	57.77	0.17	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.33	15.00	-8.06	8.06	56.56	0.18	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.49	15.00	-8.33	8.33	55.31	0.19	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.64	15.00	-8.59	8.59	54.01	0.20	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.79	15.00	-8.86	8.86	52.68	0.21	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	9.95	15.00	-9.12	9.12	51.30	0.22	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.10	15.00	-9.39	9.39	49.88	0.24	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.25	15.00	-9.66	9.66	48.43	0.25	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.40	15.00	-9.92	9.92	46.93	0.26	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.56	15.00	-10.19	10.19	45.39	0.28	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.71	15.00	-10.46	10.46	43.81	0.30	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	10.86	15.00	-10.72	10.72	42.19	0.32	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.02	15.00	-10.99	10.99	40.53	0.34	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.17	15.00	-11.25	11.25	38.83	0.36	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.32	15.00	-11.52	11.52	37.09	0.39	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.48	15.00	-11.79	11.79	35.30	0.42	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.63	15.00	-12.05	12.05	33.48	0.45	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.78	15.00	-12.32	12.32	31.61	0.49	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	11.93	15.00	-12.59	12.59	29.71	0.53	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.09	15.00	-12.85	12.85	27.76	0.58	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.24	15.00	-13.12	13.12	25.78	0.64	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.39	15.00	-13.38	13.38	23.75	0.70	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.55	15.00	-13.65	13.65	21.68	0.79	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.70	15.00	-13.92	13.92	19.57	0.89	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	12.85	15.00	-14.18	14.18	17.42	1.00	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	13.01	15.00	-14.45	14.45	15.23	1.00	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	13.16	15.00	-14.72	14.72	13.00	1.00	14.79	Phi*lambda*sqrt	in per 9.6.	28.9	7.5
+1.20D+0.70S+E	1	13.31	15.00	-14.98	14.98	10.73	1.00	14.79	Phi*Vc < Vu	0.1935	28.9	7.5
+1.20D+0.70S+E	1	13.46	15.00	-15.25	15.25	8.42	1.00	14.79	Phi*Vc < Vu	0.4597	28.9	7.5
+1.20D+0.70S+E	1	13.62	15.00	-15.51	15.51	6.06	1.00	14.79	Phi*Vc < Vu	0.7259	28.9	7.5
+1.20D+0.70S+E	1	13.77	15.00	-15.78	15.78	3.67	1.00	14.79	Phi*Vc < Vu	0.9921	28.9	7.5
+1.20D+0.70S+E	1	13.92	15.00	-16.05	16.05	1.23	1.00	14.79	Phi*Vc < Vu	1.258	28.9	7.5

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	14.000	70.58	81.39	0.87
+1.40D					
Span # 1	1	14.000	37.73	81.39	0.46
+1.20D					
Span # 1	1	14.000	32.34	81.39	0.40
+1.20D+0.50S					
Span # 1	1	14.000	39.69	81.39	0.49
+1.20D+1.60S					
Span # 1	1	14.000	55.86	81.39	0.69
+1.20D+0.70S+E					

PRCNC20231424

Concrete Beam

Project File: 23703 Enercalc BRT.ec6

DESCRIPTION: Exterior Concrete Grade Beams

Load Combination Segment	Span #	Location (ft) along Beam	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 1	1	14.000	70.58	81.39	0.87
+0.90D					
Span # 1	1	14.000	24.25	81.39	0.30
+0.90D+E					
Span # 1	1	14.000	52.20	81.39	0.64

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl (in)	Location in Span (ft)	Load Combination	Max. "+" Defl (in)	Location in Span (ft)
+D+0.750S+0.5250E	1	0.2918	7.000		0.0000	0.000

Check Chord Forces

$$M_a = \frac{20\text{ kips}(85\text{ ft})}{4} = 425\text{ kip}\cdot\text{ft}(0.6) = 255\text{ kip}\cdot\text{ft}$$

$$T/C = \frac{255\text{ kip}\cdot\text{ft}}{40.75\text{ ft}} = \frac{6.3\text{ kip}}{(1.6)(144\#)} = 27.3 \rightarrow (28)\ 16d \text{ NAILS @ TOP PL}$$

$$f_t = \frac{6.3\text{ kips}}{1.5(5.5\text{in})} = 767\text{psi} = f_c$$

$$F_T' = 575\text{psi}\underset{C_D}{(1.6)}\underset{C_F}{(1.3)} = 1196\text{psi} \checkmark \text{ Good}$$

$$F_c' = 1350\text{psi}(1.6)(1.1)(1.0) = 2376\text{psi} \checkmark \text{ Good}$$

VERTICAL

PRCNC20231424

PCS Structural Solutions

Project: ______ Job No: ______

Subject: ______ Sheet ______ Name: ______

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

Snow load

$P_g = 18\ psf$

$P_f = 0.7(1.0)(1.1)(1.0)(25\ psf) = 11.6\ psf$

$P_m = 1.0(18\ psf) = 18\ psf$

Check end trusses for snow drift

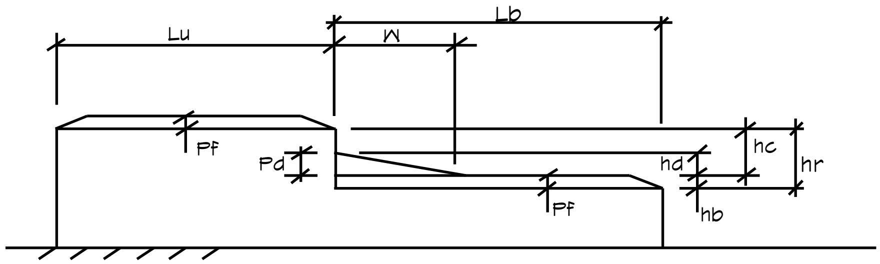
PRCNC20231424

PCS Structural Solutions

Project:	Arco Puyallup	Job Number:	23-703
Sheet:	of	Name:	BRT
Originating Office:	Portland	Date:	09/20/23

SNOW DRIFT

IBC2018/ASCE7-16



CRITERIA

GROUND SNOW LOAD, Pg =	18	PSF	Per ASCE 7-16 Figure 7.2.1 & Table 7.2
EXPOSURE FACTOR, C_e =	1.0		Per ASCE 7-16 Table 7.3-1
THERMAL FACTOR, C_t =	1.1		Per ASCE 7-16 Table 7.3-2
IMPORTANCE FACTOR, I_s =	1.0		Per ASCE 7-16 Table 1.5-2
ROOF SLOPE FACTOR, C_s =	1.0		Per ASCE 7-16 Figure 7.4-1

BALANCED SNOW LOAD

ROOF SNOW LOAD (P_s) =	13.9	PSF	$P_s = 0.7C_eC_tC_sI_sP_g$
SNOW DENSITY (γ) =	16.3	PCF	$\gamma = 0.13P_g + 14 \leq 30$ PCF
BALANCED SNOW LOAD (h_b) =	0.85	FT	$h_b = P_s/\gamma$

NOTE: SEE FOLLOWING PAGE FOR UNBALANCED SNOW LOAD

DRIFTING & SLIDING SNOW LOAD (in addition to balanced snow load)

	CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6
L_u =	0 FT	0 FT				
L_b =	85 FT	41 FT				
h_r =	4.5 FT	10.0 FT				
h_c =	3.65 FT	9.15 FT				
Leeward (h_d) =	1.18 FT	1.18 FT				
Windward (h_d) =	2.14 FT	1.42 FT				
Maximum?	2.14 FT	1.42 FT				
$h_c < h_d$?	NO	NO				
Maximum (h_d)=	2.14 FT	1.42 FT				
Drift W =	8.55 FT	5.69 FT				
W > L_b?	NO	NO				
$P_{d(MAX)}$ =	34.9 PSF	23.2 PSF				
IF W > L_b, $P_{d(TRUNCATED)}$ =	N/A	N/A				
Check Sliding?	NO	NO				
$P_{SLIDING}$ (on 1st 15ft) =	N/A	N/A				

PRCNC20231424

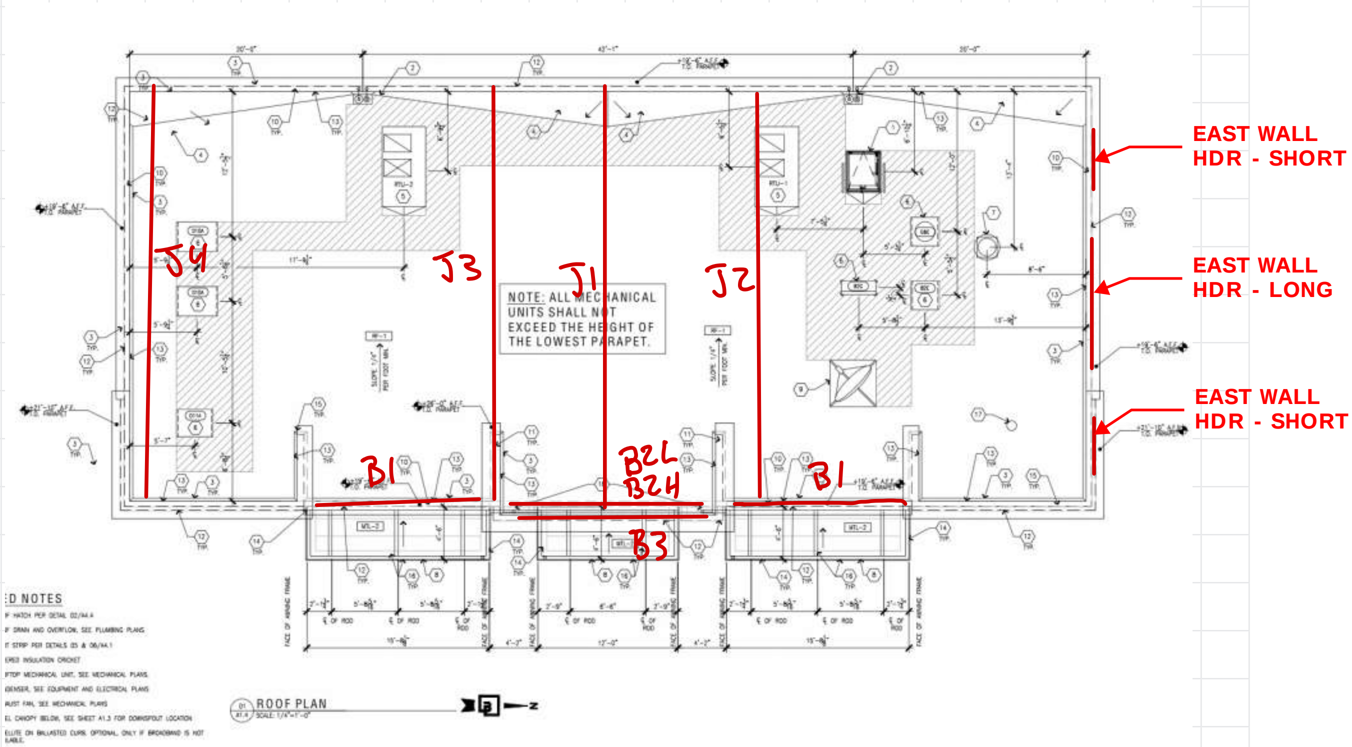
PCS Structural Solutions

Project: ______ Job No: ______

Subject: ______ Sheet ______ Name: ______

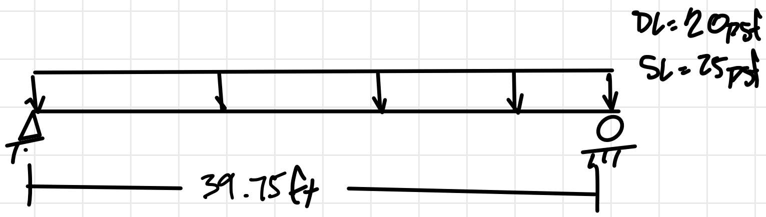
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

Roof Framing Map



J1

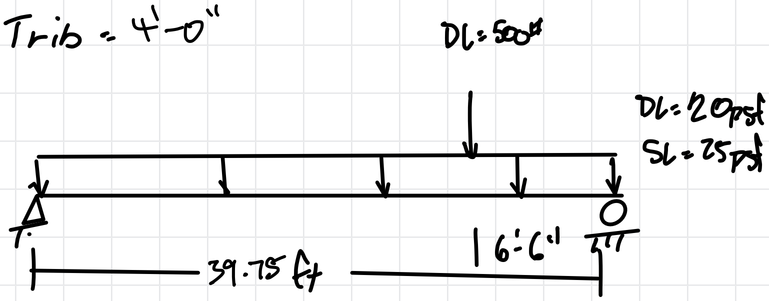
Trib = 4'-0"



USE Red - 5 32" — 41.9"

J2

Trib = 4'-0"



USE Red - 5 32" — 41.9"

PRCNC20231424

PCS Structural Solutions

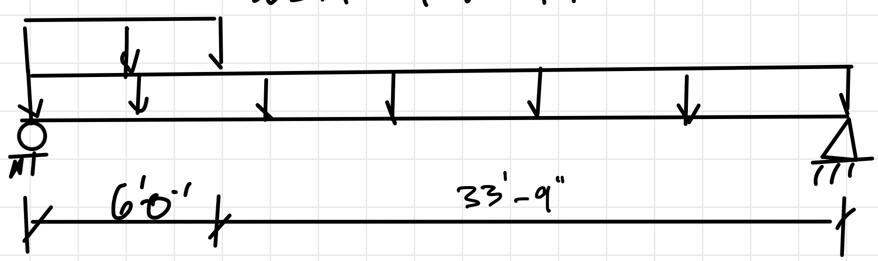
Project: ______ Job No: ______
Subject: ______ Sheet ______ Name: ______
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

J3

Trib = 4'-0"

w = 11ft(10psf) = 110plf

DL = 20psf
SL = 25psf



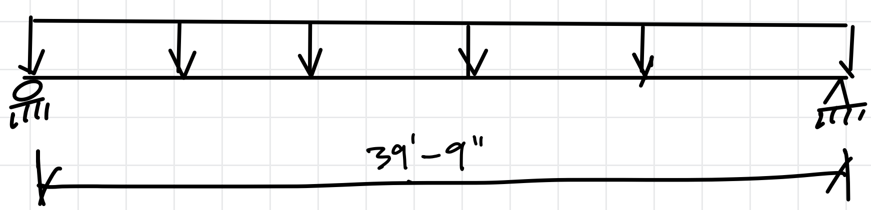
USE Red-S- 32"-41.9"

J4

Trib = (4'-0" + 2'-0")/2 = 3'-0"

DL = 20psf
SL = 43.8psf

conservative



USE Red-S 32-41.9 @ End

PRCNC20231424

RedSpec™ by RedBuilt™
v7.1.15

Project: ARCO PUYALLUP C-STORE
Location: PUYALLUP, WA
Folder:
Date: 2/7/24
Designer: BRT
Comment:

Type: J1

TAPERED 26-34.4" Red-S™ @ 48" 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	42%	0.472	1.117	L / 852	L / 360	PASS
Span Total	56%	0.944	1.675	L / 426	L / 240	PASS

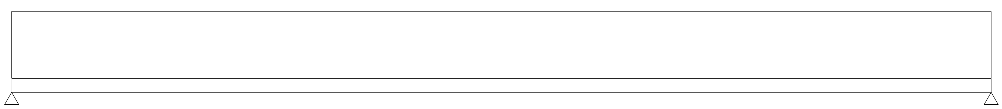
SUPPORTS

	Support 1	Support 2
Live Reaction (lb) (DOL%)	1363 (125)	1364 (125)
Dead Reaction (lb)	1363	1364
Total Reaction (lb) (DOL%)	2726 (125)	2727 (125)
Bearing	Top Chord	Top Chord
Support	Wall	Wall
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.25/12



APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Roof(125%)	20	20	0	48"	Roof Joist

NOTES

- Building code and design methodology: 2021 IBC ASD (US).
- No 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) = 160
- Pricing Index (plf) = 160

G:\2023 Jobs\23614 ARCO Walerga\Calcs\23-614 Roof Joists BRT.red

PRCNC20231424

RedSpec™ by RedBuilt™
v7.1.15

Project: ARCO PUYALLUP C-STORE
Location: PUYALLUP, WA
Folder:
Date: 2/7/24
Designer: BRT
Comment:

Type: J2

TAPERED 26-34.4" Red-S™ @ 48" 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	43%	0.476	1.117	L / 845	L / 360	PASS
Span Total	61%	1.029	1.675	L / 391	L / 240	PASS

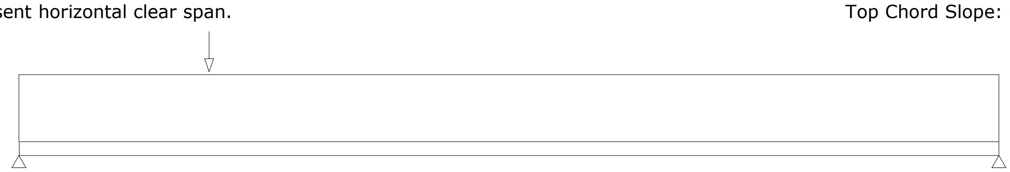
SUPPORTS

	Support 1	Support 2
Live Reaction (lb) (DOL%)	1363 (125)	1364 (125)
Dead Reaction (lb)	1766	1461
Total Reaction (lb) (DOL%)	3129 (125)	2824 (125)
Bearing	Top Chord	Top Chord
Support	Wall	Wall
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.25/12



33'- 6.0"

APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Roof(125%)	20	20	0	48"	Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Point	lb	Floor(100%)	0	500	6'-6.0"	Adds To	

NOTES

- Building code and design methodology: 2021 IBC ASD (US).
- No 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) = 175
- Pricing Index (plf) = 189

G:\2023 Jobs\23614 ARCO Walerga\Calcs\23-614 Roof Joists BRT.red

PRCNC20231424

RedSpec™ by RedBuilt™
v7.1.15

Project: ARCO PUYALLUP C-STORE
Location: PUYALLUP, WA
Folder:
Date: 2/7/24
Designer: BRT
Comment:

Type: J3

TAPERED 26-34.4" Red-S™ @ 48" 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	42%	0.474	1.117	L / 849	L / 360	PASS
Span Total	60%	1.010	1.675	L / 398	L / 240	PASS

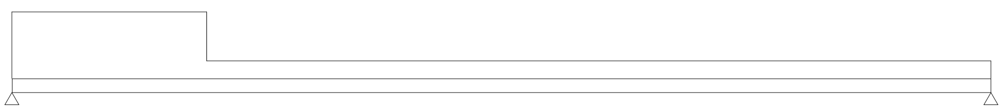
SUPPORTS

	Support 1	Support 2
Live Reaction (lb) (DOL%)	1363 (125)	1364 (125)
Dead Reaction (lb)	2023	1437
Total Reaction (lb) (DOL%)	3386 (125)	2800 (125)
Bearing	Top Chord	Top Chord
Support	Wall	Wall
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.25/12



33'- 6.0"

APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Roof(125%)	20	20	0	48"	Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Uniform	plf	Roof(125%)	0	110	0'-0.0" to 6'-8.0"	Adds To	

NOTES

- Building code and design methodology: 2021 IBC ASD (US).
- No 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) = 182
- Pricing Index (plf) = 182

G:\2023 Jobs\23614 ARCO Walerga\Calcs\23-614 Roof Joists BRT.red

PRCNC20231424

RedSpec™ by RedBuilt™
v7.1.15

Project: ARCO PUYALLUP C-STORE
Location: PUYALLUP, WA
Folder:
Date: 2/7/24
Designer: BRT
Comment:

Type: J4

TAPERED 26-34.4" Red-S™ @ 48" 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	43%	0.484	1.117	L / 830	L / 360	PASS
Span Total	64%	1.074	1.675	L / 374	L / 240	PASS

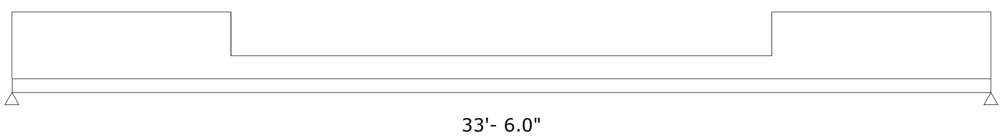
SUPPORTS

	Support 1	Support 2
Live Reaction (lb) (DOL%)	1363 (125)	1364 (125)
Dead Reaction (lb)	1944	1945
Total Reaction (lb) (DOL%)	3307 (125)	3309 (125)
Bearing	Top Chord	Top Chord
Support	Wall	Wall
Bearing Clip	(Red-S) S-Clip	(Red-S) S-Clip
Approx. Clip Height	3.5"	3.5"
Approx. Clip Width	5.5"	5.5"
Assumed Bearing Width	3.5"	3.5"

SPANS AND LOADS

Dimensions represent horizontal clear span.

Top Chord Slope: 0.25/12



APPLICATION LOADS

Type	Units	DOL	Live	Dead	Partition	Tributary	Member Type
Uniform	psf	Roof(125%)	20	20	0	48"	Roof Joist

ADDITIONAL LOADS

Type	Units	DOL	Live	Dead	Location from left	Application	Comment
Uniform	plf	Roof(125%)	0	77.5	0'-0.0" to 7'-6.0"	Adds To	
Uniform	plf	Roof(125%)	0	77.5	26'-0.0" to 33'-6.0"	Adds To	

NOTES

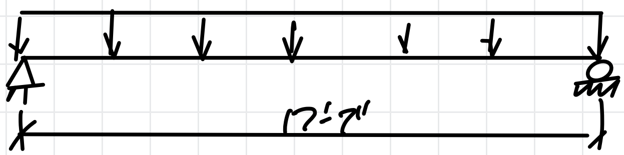
- Building code and design methodology: 2021 IBC ASD (US).
- No repetitive member increase applied in design.
- Beveled plate required at left support.
- Beveled plate required at right support.
- Truss design includes consideration for partial span application live load.
- Continuous lateral support required at top edge. Lateral support at bottom edge shall be per RedBuilt recommendations.
- Pricing Load (plf) = 194
- Pricing Index (plf) = 194

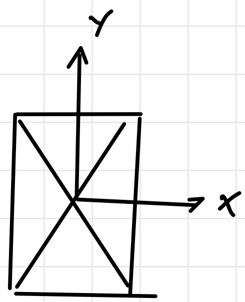
G:\2023 Jobs\23614 ARCO Walerga\Calcs\23-614 Roof Joists BRT.red

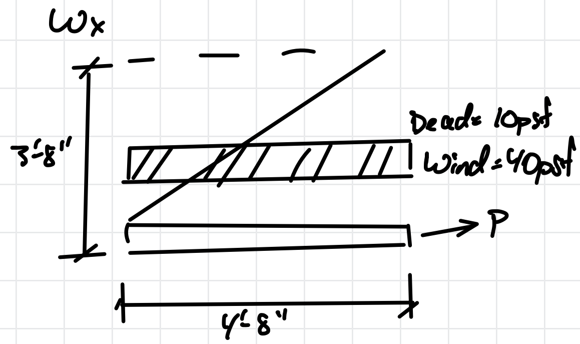
B1

w_Y = DL = 20psf(40.75ft)/2
+10ft(4'-6") = 453 #/ft

SL = 25psf(40.75ft)/2 = 509 #/ft







P_D = 30 #/ft

P_W = 120 #/ft

$$\frac{f_{bx}}{F'_{bx}} + \frac{f_{by}}{F'_{by}} = 0.72 + 0.152 = 0.87$$

✓ GOOD

USE 5 1/8 x 16.5 (SEE ATTACHED)

B2 High

Span = 12'-8"
w_D = 20psf(40.75 ft/2) = 408 plf
w_L = 25psf(40.75 ft/2) = 509 plf

USE 5 1/8x15 GL → SEE Attached

B2L

l = 12'-8"
weak axis loading from canopy
Same as B1
↳ w_D = 30 #/ft
w_w = -120 #/ft

↳ 5 1/8x12 GL Okay (See attached)

B3

l = 12'-8"
w_D = 10psf(13ft) = 130 #/ft

USE 5 1/8x12 GL BM (See attached)

PRCNC20231424

B4

Span = 16'-6"

DL = 20 PSF(2') = 40 PLF
SL = 43.8 PSF(2') = 87.6 PLF

conservative

→ SEE ENERCALC — 6x10 OK

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 | PCS STRUCTURAL SOLUTIONS | (c) ENERCALC INC 1983-2023

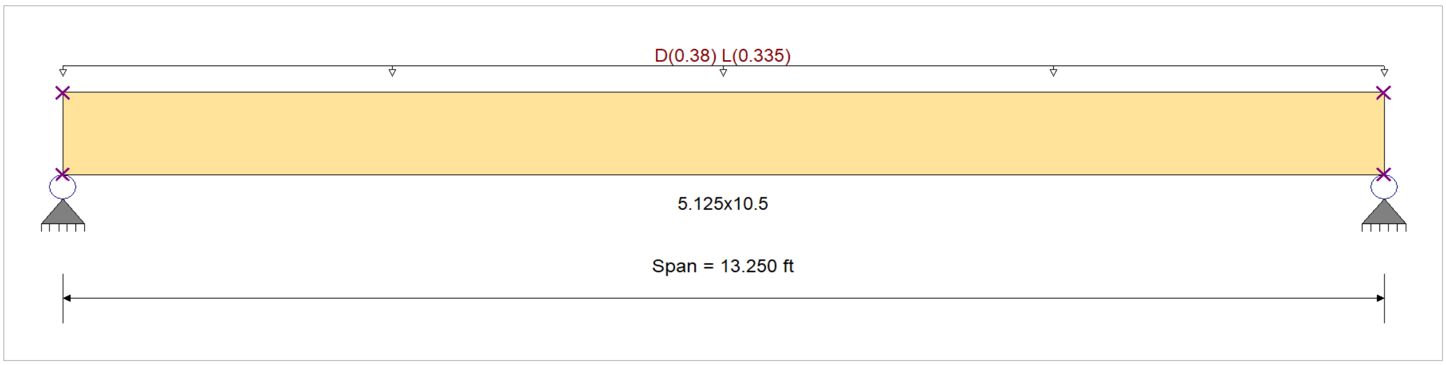
DESCRIPTION: B1 (Major Axis)

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	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 :	Completely Unbraced				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.380, L = 0.3350 , Tributary Width = 1.0 ft, (Roof Trib)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.843 : 1	Maximum Shear Stress Ratio	=	0.433 : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	1,999.44 psi	fv: Actual	=	114.69 psi
F'b	=	2,372.60 psi	F'v	=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.625 ft	Location of maximum on span	=	12.380 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.263 in	Ratio =	605 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.560 in	Ratio =	283 >=240	Span: 1 : +D+L
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 = 13.250 ft	**1**	0.497	0.256	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	8.34	1,062.6	2,138.2	2.19	61.0	238.5
+D+L					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.250 ft	**1**	0.843	0.433	1.00	1.00	1.00	0.99	1.000	1.00	1.00	1.00	15.69	1,999.4	2,372.6	4.11	114.7	265.0
+D+0.750L					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.250 ft	**1**	0.597	0.306	1.25	1.00	1.00	0.98	1.000	1.00	1.00	1.00	13.85	1,765.2	2,954.9	3.63	101.3	331.3
+0.60D					1.00	1.00	0.98	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.250 ft	**1**	0.170	0.086	1.60	1.00	1.00	0.98	1.000	1.00	1.00	1.00	5.00	637.6	3,760.4	1.31	36.6	424.0

PRCNC20231424

DESCRIPTION: B1 (Major Axis)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.5604	6.673		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	4.737	4.737
Max Upward from Load Combinations	4.737	4.737
Max Upward from Load Cases	2.518	2.518
D Only	2.518	2.518
+D+L	4.737	4.737
+D+0.750L	4.182	4.182
+0.60D	1.511	1.511
L Only	2.219	2.219

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS

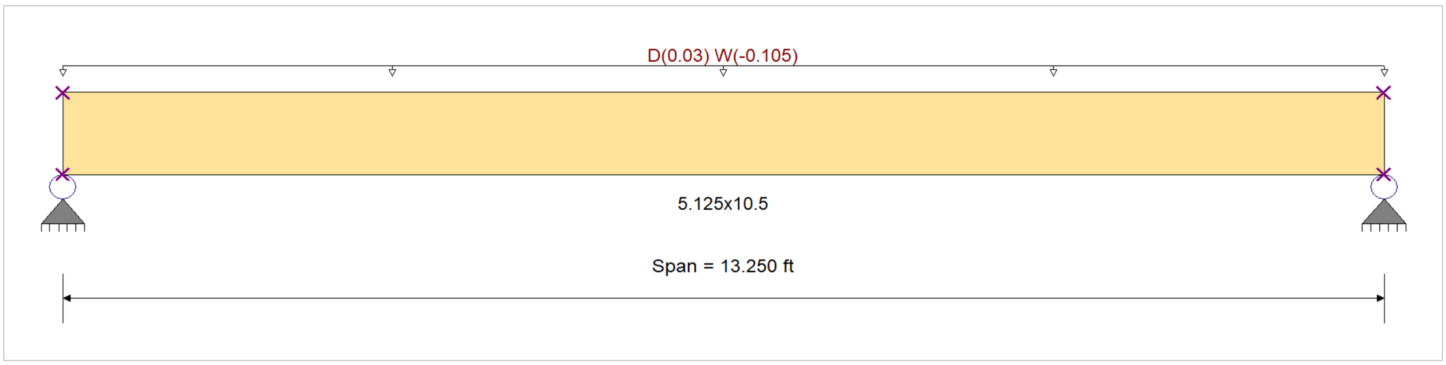
DESCRIPTION: B1 (Minor Axis)

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	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 :	Completely Unbraced				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.030, W = -0.1050 , Tributary Width = 1.0 ft, (Canopy Loads)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.087: 1	Maximum Shear Stress Ratio	=	0.020 : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	257.82 psi	fv: Actual	=	4.81 psi
F'b	=	2,960.00 psi	F'v	=	238.50 psi
Load Combination		+0.60D+0.60W	Load Combination		D Only
Location of maximum on span	=	6.625 ft	Location of maximum on span	=	12.380 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	-0.345 in	Ratio =	460 >=360	Span: 1 : W Only
Max Downward Total Deflection	0.099 in	Ratio =	1610 >=240	Span: 1 : D Only
Max Upward Total Deflection	-0.148 in	Ratio =	1073 >=240	Span: 1 : +0.60D+0.60W

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 = 13.250 ft	**1**	0.080	0.020	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.66	171.9	2,160.0	0.17	4.8	238.5
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.250 ft	**1**	0.064	0.012	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.72	189.1	2,960.0	0.19	5.3	424.0
+D+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 = 13.250 ft	**1**	0.033	0.007	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.38	98.8	2,960.0	0.10	2.8	424.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 13.250 ft	**1**	0.087	0.017	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.99	257.8	2,960.0	0.26	7.2	424.0
+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 = 13.250 ft	**1**	0.027	0.007	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.40	103.1	3,840.0	0.10	2.9	424.0

PRCNC20231424

DESCRIPTION: B1 (Minor Axis)

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.3455	6.673

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.199	0.199
Max Upward from Load Combinations	0.119	0.119
Max Upward from Load Cases	0.199	0.199
Max Downward from all Load Conditio	-0.696	-0.696
Max Downward from Load Combinations	-0.298	-0.298
Max Downward from Load Cases (Resis	-0.696	-0.696
D Only	0.199	0.199
+D+0.60W	-0.219	-0.219
+D+0.450W	-0.114	-0.114
+0.60D+0.60W	-0.298	-0.298
+0.60D	0.119	0.119
W Only	-0.696	-0.696

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: B2H

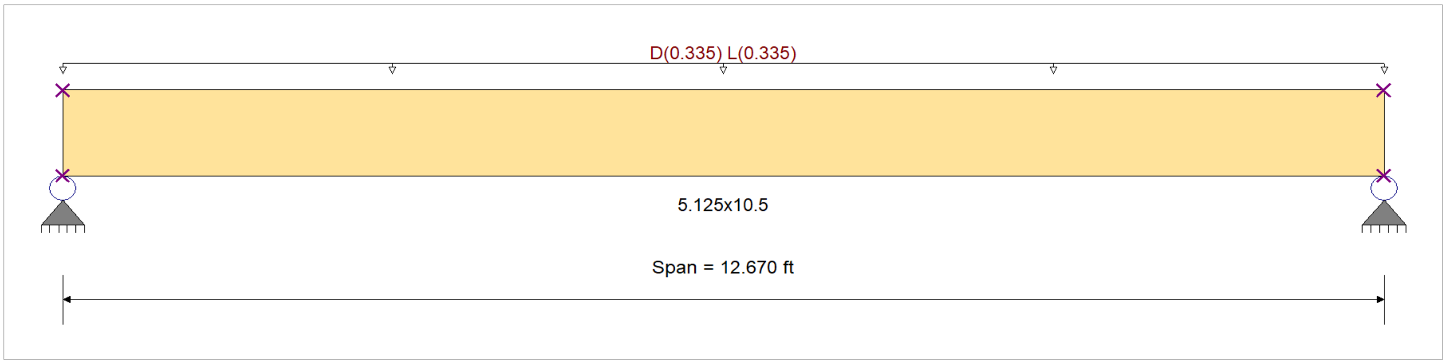
CODE REFERENCES

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

Load Combination Set : IBC 2018

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	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 :	Completely Unbraced				



Applied Loads

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

Beam self weight NOT internally calculated and added

Uniform Load : D = 0.3350, L = 0.3350 , Tributary Width = 1.0 ft, (Roof Trib)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.722** 1	Maximum Shear Stress Ratio	=	**0.388** : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	1,713.16 psi	fv: Actual	=	102.77 psi
F'b	=	2,374.03 psi	F'v	=	265.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	6.335 ft	Location of maximum on span	=	11.838 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.220 in	Ratio =	692 >=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.439 in	Ratio =	346 >=240	Span: 1 : +D+L
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 = 12.670 ft	**1**	0.400	0.215	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	6.72	856.6	2,139.4	1.84	51.4	238.5
+D+L					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.722	0.388	1.00	1.00	1.00	0.99	1.000	1.00	1.00	1.00	13.44	1,713.2	2,374.0	3.69	102.8	265.0
+D+0.750L					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.507	0.271	1.25	1.00	1.00	0.99	1.000	1.00	1.00	1.00	11.76	1,499.0	2,957.4	3.23	89.9	331.3
+0.60D					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.137	0.073	1.60	1.00	1.00	0.98	1.000	1.00	1.00	1.00	4.03	513.9	3,765.1	1.11	30.8	424.0

PRCNC20231424

DESCRIPTION: B2H

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4391	6.381		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	4.244	4.244
Max Upward from Load Combinations	4.244	4.244
Max Upward from Load Cases	2.122	2.122
D Only	2.122	2.122
+D+L	4.244	4.244
+D+0.750L	3.714	3.714
+0.60D	1.273	1.273
L Only	2.122	2.122

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

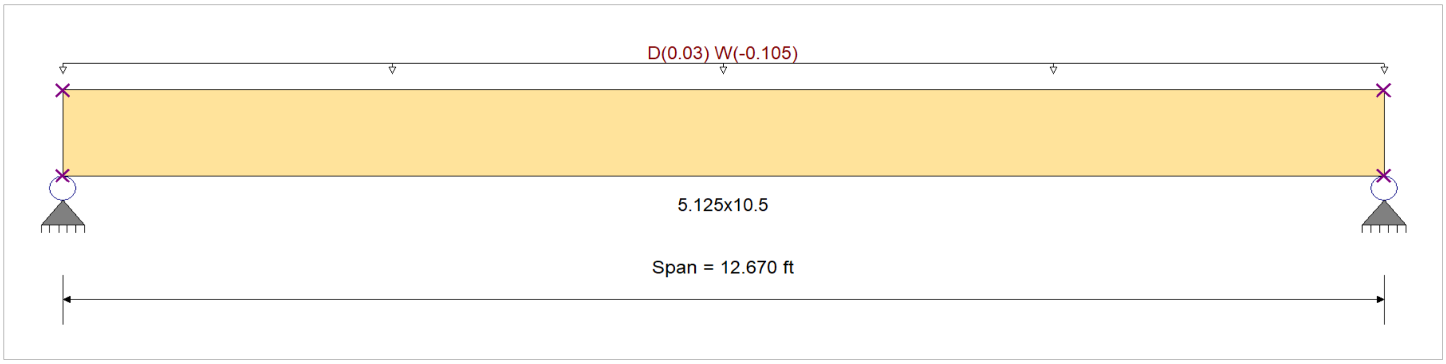
DESCRIPTION: B2L

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*		
Load Combination :	IBC 2018	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 :	Completely Unbraced					



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.030, W = -0.1050 , Tributary Width = 1.0 ft, (Canopy Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.080 : 1**	Maximum Shear Stress Ratio	=	**0.019 : 1**
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	235.74 psi	fv: Actual	=	4.60 psi
F'b	=	2,960.00 psi	F'v	=	238.50 psi
Load Combination		+0.60D+0.60W	Load Combination		D Only
Location of maximum on span	=	6.335 ft	Location of maximum on span	=	11.838 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Upward Transient Deflection	-0.289 in	Ratio =	526	>=360	Span: 1 : W Only
Max Downward Total Deflection	0.083 in	Ratio =	1842	>=240	Span: 1 : D Only
Max Upward Total Deflection	-0.124 in	Ratio =	1228	>=240	Span: 1 : +0.60D+0.60W

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 = 12.670 ft	**1**	0.073	0.019	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.60	157.2	2,160.0	0.17	4.6	238.5
+D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.058	0.012	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.66	172.9	2,960.0	0.18	5.1	424.0
+D+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 = 12.670 ft	**1**	0.031	0.006	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.35	90.4	2,960.0	0.09	2.6	424.0
+0.60D+0.60W					1.00	1.00	1.00	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.080	0.016	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.90	235.7	2,960.0	0.25	6.9	424.0
+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 = 12.670 ft	**1**	0.025	0.007	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.36	94.3	3,840.0	0.10	2.8	424.0

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: B2L

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	W Only	-0.2888	6.381

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.190	0.190
Max Upward from Load Combinations	0.114	0.114
Max Upward from Load Cases	0.190	0.190
Max Downward from all Load Conditio	-0.665	-0.665
Max Downward from Load Combinations	-0.285	-0.285
Max Downward from Load Cases (Resis	-0.665	-0.665
D Only	0.190	0.190
+D+0.60W	-0.209	-0.209
+D+0.450W	-0.109	-0.109
+0.60D+0.60W	-0.285	-0.285
+0.60D	0.114	0.114
W Only	-0.665	-0.665

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

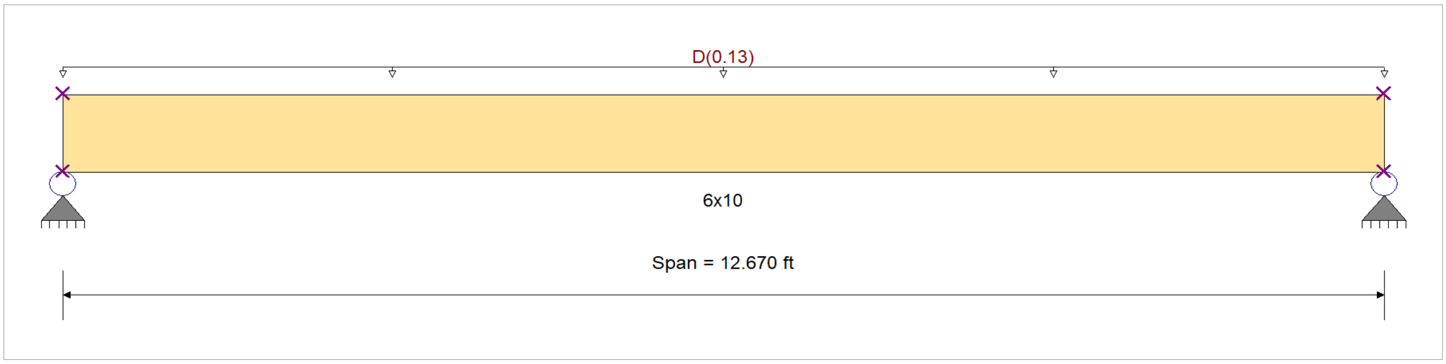
DESCRIPTION: B3

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1350 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1350 psi	Ebend- xx	1600 ksi
		Fc - Prll	925 psi	Eminbend - xx	580 ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625 psi		
Wood Grade :	No.1	Fv	170 psi		
		Ft	675 psi	Density	31.21 pcf
Beam Bracing :	Completely Unbraced				



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.130 , Tributary Width = 1.0 ft, (Wall Loading)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.314 : 1	Maximum Shear Stress Ratio	=	0.135 : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb: Actual	=	378.38 psi	fv: Actual	=	20.71 psi
F'b	=	1,204.21 psi	F'v	=	153.00 psi
Load Combination		D Only	Load Combination		D Only
Location of maximum on span	=	6.335 ft	Location of maximum on span	=	11.884 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.121 in	Ratio =	1260 >=240	Span: 1 : D Only
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 = 12.670 ft	**1**	0.314	0.135	0.90	1.00	1.00	0.99	1.000	1.00	1.00	1.00	2.61	378.4	1,204.2	0.72	20.7	153.0
+0.60D					1.00	1.00	0.99	1.000	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 12.670 ft	**1**	0.107	0.046	1.60	1.00	1.00	0.98	1.000	1.00	1.00	1.00	1.57	227.0	2,121.2	0.43	12.4	272.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D Only	1	0.1206	6.381		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	0.824	0.824

DESCRIPTION: B3

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from Load Combinations	0.494	0.494
Max Upward from Load Cases	0.824	0.824
D Only	0.824	0.824
+0.60D	0.494	0.494

PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 23703 Enercalc BRT.ec6

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

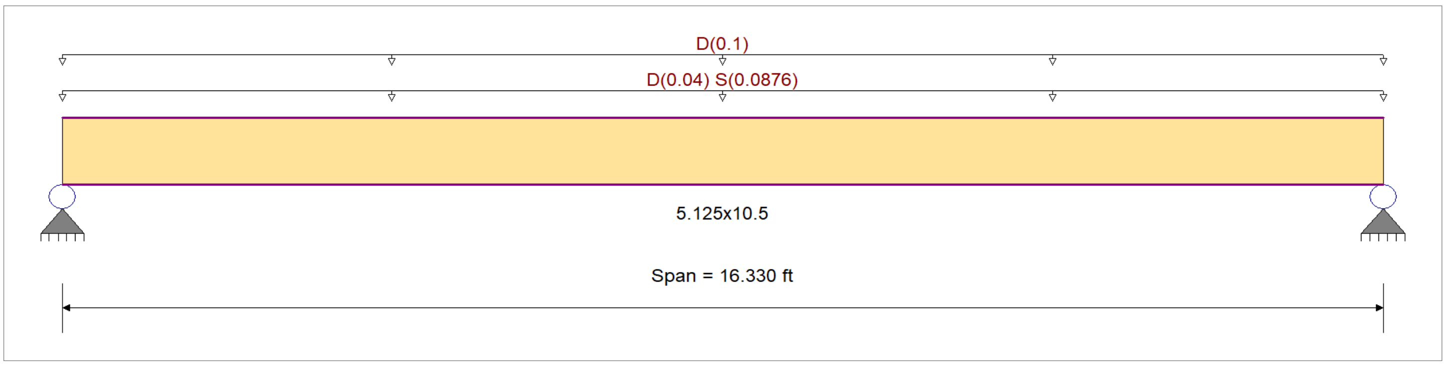
DESCRIPTION: East Wall Hdr. - long

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	2,400.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,850.0 psi	Ebend- xx	1,800.0ksi
		Fc - Prll	1,650.0 psi	Eminbend - xx	950.0ksi
Wood Species :	DF/DF	Fc - Perp	650.0 psi	Ebend- yy	1,600.0ksi
Wood Grade :	24F-V4	Fv	265.0 psi	Eminbend - yy	850.0ksi
		Ft	1,100.0 psi	Density	31.210pcf
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, S = 0.04380 ksf, Tributary Width = 2.0 ft, (From Roof)
Uniform Load : D = 0.10 , Tributary Width = 1.0 ft, (From Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.350** : 1	Maximum Shear Stress Ratio	=	**0.153** : 1
Section used for this span		**5.125x10.5**	Section used for this span		**5.125x10.5**
fb: Actual	=	966.75 psi	fv: Actual	=	46.51 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	=	8.165ft	Location of maximum on span	=	15.496 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.158 in	Ratio =	1236 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.412 in	Ratio =	476 >=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 = 16.330 ft	1	0.275	0.120	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	4.67	594.7	2,160.0	1.03	28.6	238.5
+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 = 16.330 ft	1	0.350	0.153	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	7.59	966.8	2,760.0	1.67	46.5	304.8
+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
Length = 16.330 ft	1	0.317	0.138	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	6.86	873.7	2,760.0	1.51	42.0	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 = 16.330 ft	1	0.093	0.040	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	2.80	356.8	3,840.0	0.62	17.2	424.0

PRCNC20231424

Wood Beam

DESCRIPTION: East Wall Hdr. - long

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.4116	8.225		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	1.858	1.858
Max Upward from Load Combinations	1.858	1.858
Max Upward from Load Cases	1.143	1.143
D Only	1.143	1.143
+D+S	1.858	1.858
+D+0.750S	1.680	1.680
+0.60D	0.686	0.686
S Only	0.715	0.715

PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:

Wood Beam

Project File: 23703 Enercalc BRT.ec6

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

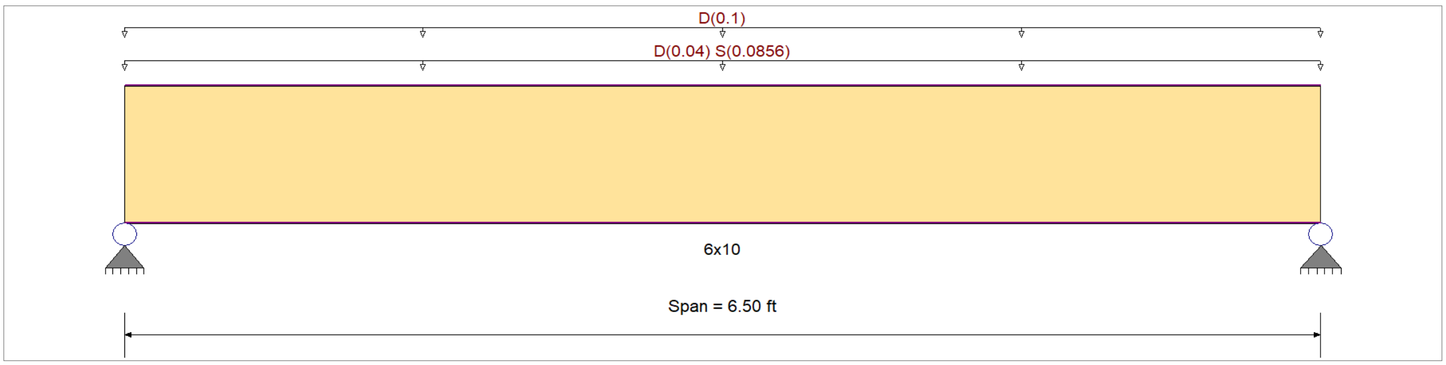
DESCRIPTION: East Wall Hdr. - short

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2018	Fb -	1,350.0 psi	Ebend- xx	1,600.0ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0ksi
Wood Species :	Douglas Fir-Larch	Fc - Perp	625.0 psi		
Wood Grade :	No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.210pcf
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, S = 0.04280 ksf, Tributary Width = 2.0 ft, (From Roof)
Uniform Load : D = 0.10 , Tributary Width = 1.0 ft, (From Wall)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.111**: 1	Maximum Shear Stress Ratio	=	**0.082** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb: Actual	=	172.82 psi	fv: Actual	=	15.98 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	=	3.250ft	Location of maximum on span	=	5.717 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.006 in	Ratio =	14181 >=360	Span: 1 : S Only
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.014 in	Ratio =	5381 >=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 = 6.50 ft	1	0.088	0.065	0.90	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.74	107.2	1,215.0	0.35	9.9	153.0
+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 = 6.50 ft	1	0.111	0.082	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.19	172.8	1,552.5	0.56	16.0	195.5
+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
Length = 6.50 ft	1	0.101	0.074	1.15	1.00	1.00	1.00	1.000	1.00	1.00	1.00	1.08	156.4	1,552.5	0.50	14.5	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 = 6.50 ft	1	0.030	0.022	1.60	1.00	1.00	1.00	1.000	1.00	1.00	1.00	0.44	64.3	2,160.0	0.21	5.9	272.0

PRCNC20231424

Wood Beam

Project File: 23703 Enercalc BRT.ec6

DESCRIPTION: East Wall Hdr. - short

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S	1	0.0145	3.274		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	0.733	0.733
Max Upward from Load Combinations	0.733	0.733
Max Upward from Load Cases	0.455	0.455
D Only	0.455	0.455
+D+S	0.733	0.733
+D+0.750S	0.664	0.664
+0.60D	0.273	0.273
S Only	0.278	0.278

PRCNC20231424

Out-of-plane Wall Framing

PRCNC20231424

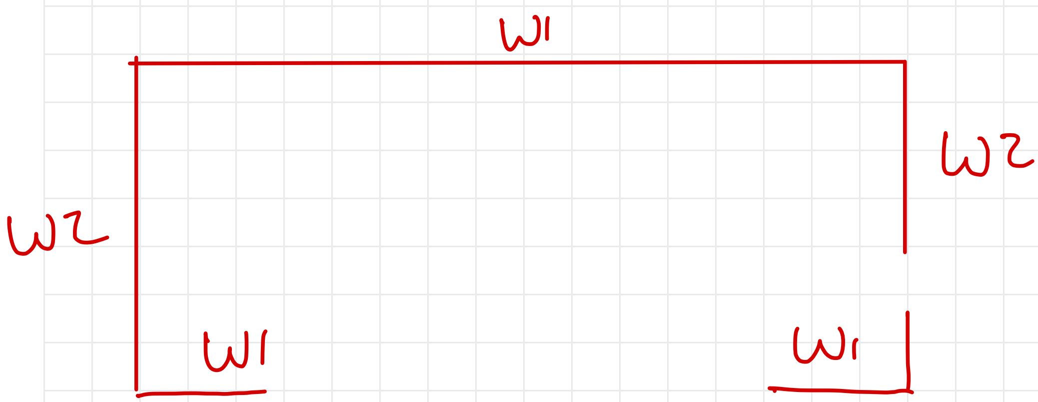
PCS Structural Solutions

Project: ______ Job No: ______

Subject: ______ Sheet ______ Name: ______

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

Wall framing



PLAN

W1

$h_t = 15'\text{-}6"$

$P_D = 20psf(40.8\ ft)/2\ (4ft) = 1630\ \#$

$P_S = 25psf$ " " $= 2038\ \#$

$P_w = 16psf\ (40.8\ ft)/2\ (4ft) = 1304\ \#$

$W_w = \ell\ ^2/_3 = 81 ft^2 \rightarrow 16psf$

USE (2) 2x6 studs @ TRUSS
(See attached)

W2

$h_t = 15'\text{-}6"$

$P_D = 20psf(2ft)(16^{in}/12_{in}) = 54^{\#}$
$P_S = 25psf(2ft)(16^{in}/12_{in}) = 67^{\#}$

$w_w = \ell^2/3 = 81 ft^2 \Rightarrow 16 PSF$

2x6 @ 16" o.c. OK - See attached

Check FDN

$DL = 20psf(20.4\ ft) + 21ft(10psf) + 150pcf(8in/12)(0.5ft) + 150pcf(1.5ft)(10in/12)$
$= 856\ plf$

$SL = 25psf(20.4\ ft) = 509\ plf$

Total = 1365 plf ⇒ 1'-6" wide fdn

PCS Structural Solutions

Project: Arco Puyallup | Job Number: 23-703
Sheet: ____ of ____ | Name: BRT
Originating Office: Portland | Date: 09/20/23

STUD WALL DESIGN - W1

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#2

F_b =	850 PSI	APPLIED LOADS:		P_{DEAD} =	1630 LBS
F_c =	1300 PSI	W_{WIND} =	16.0 PSF	P_{LIVE} =	0 LBS
$F_{c\perp}$ =	405 PSI	$W_{SEISMIC}$ =	6.0 PSF	P_{SNOW} =	2038 LBS
E =	1.30E+06 PSI			P_{WIND} =	1304 LBS
				$P_{SEISMIC}$ =	350 LBS

STUD SIZE: (2) 2x6

A_X =	16.50 IN^2	MISCELLANEOUS: HEIGHT =	15.5 FT		
S_X =	15.13 IN^3	SPACING =	16 IN		
I_X =	41.59 IN^4	ECCENTRICITY =	0.1 IN		
$C_{F(BENDING)}$ =	1.3 (NDS 4.3.6)	$C_{F(COMPRESSION)}$ =	1.10 (NDS 4.3.6)		
				APPLY?	
F_{cE} =	341.0 PSI	$C_{SYS(BENDING)}$ =	1.35	YES	(SDPWS T3.1.1.1)
C_b =	1.13 (NDS 3.10.4)	$C_{r(BENDING)}$ =	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	1430	0.24	0.225	322	1271	456
2 & 3	1.15	1645	0.21	0.198	325	1461	456
4 & 5	1.60	2288	0.15	0.144	330	2387	456
6 & 7	1.60	2288	0.15	0.144	330	2033	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT.\ LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	1630	99	200	14	209	166
2	3668	222	200	31	219	174
3	3159	191	200	26	217	172
4	3745	227	288	31	308	244
5	2412	146	384	20	397	315
6	1875	114	168	16	178	141
7	3344	203	127	28	145	115

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	0.31	0.13	0.22	0.28	0.29	0.17	L/1114
2	0.68	0.12	0.49	0.81	0.65	0.18	L/1060
3	0.59	0.12	0.42	0.62	0.56	0.17	L/1073
4*	0.69	0.10	0.50	0.78	0.67	0.13	L/1457
5*	0.44	0.13	0.32	0.43	0.43	0.16	L/1130
6	0.34	0.07	0.25	0.22	0.33	0.14	L/1307
7	0.61	0.06	0.44	0.52	0.59	0.12	L/1607
MAX. --->	0.69	0.13	0.50	0.81	0.67	0.18	L/1060
	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.

PRCNC20231424

Project:	Arco Puyallup	Job Number:	23-703
Sheet:	of	Name:	BRT
Originating Office:	Portland	Date:	09/20/23

PCS Structural Solutions

STUD WALL DESIGN - W2

2018 NDS/2018 IBC

WALL DATA

LUMBER TYPE: DF#2/HF#2

F_b = 850 PSI
F_c = 1300 PSI
$F_{c\perp}$ = 405 PSI
E = 1.30E+06 PSI

APPLIED LOADS:

W_{WIND} = 16.0 PSF
$W_{SEISMIC}$ = 6.0 PSF

P_{DEAD} = 54 LBS
P_{LIVE} = 0 LBS
P_{SNOW} = 67 LBS
P_{WIND} = 0 LBS
$P_{SEISMIC}$ = 0 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} = 341.0 PSI
C_b = 1.25 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 15.5 FT
SPACING = 16 IN
ECCENTRICITY = 0.1 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	1430	0.24	0.225	322	1271	506
2 & 3	1.15	1645	0.21	0.198	325	1461	506
4 & 5	1.60	2288	0.15	0.144	330	2387	506
6 & 7	1.60	2288	0.15	0.144	330	2033	506

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	54	7	200	0	200	318
2	121	15	200	1	201	319
3	104	13	200	1	201	319
4	104	13	288	1	289	458
5	54	7	384	0	385	610
6	54	7	168	0	168	267
7	104	13	127	1	128	203

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	0.25	0.01	0.26	0.02	0.32	L/580
2	0.05	0.22	0.03	0.23	0.04	0.32	L/579
3	0.04	0.22	0.02	0.23	0.04	0.32	L/579
4*	0.04	0.19	0.02	0.20	0.04	0.24	L/776
5*	0.02	0.26	0.01	0.26	0.02	0.32	L/583
6	0.02	0.13	0.01	0.13	0.02	0.27	L/690
7	0.04	0.10	0.02	0.11	0.04	0.20	L/909
MAX. --->	0.05	0.26	0.03	0.26	0.04	0.32	L/579
	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.

PRCNC20231424

Project: Arco Puyallup | Job Number: 23-703
Sheet: ____ of ____ | Name: BRT
Originating Office: Portland | Date: 09/25/23

PCS Structural Solutions

STUD WALL DESIGN - East Wall Brg Stud

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
$W_{SEISMIC}$ = 0.0 PSF

P_{DEAD} = 1150 LBS
P_{LIVE} = 0 LBS
P_{SNOW} = 720 LBS
P_{WIND} = 0 LBS
$P_{SEISMIC}$ = 0 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} = 393.5 PSI
C_b = 1.25 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 15.5 FT
SPACING = 0 IN
ECCENTRICITY = 0.1 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	0.26	0.249	369	1170	506
2 & 3	1.15	1708	0.23	0.218	373	1346	506
4 & 5	1.60	2376	0.17	0.160	379	1872	506
6 & 7	1.60	2376	0.17	0.160	379	1872	506

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT.\ LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	1150	139	0	10	6	10
2	1870	227	0	16	10	15
3	1690	205	0	14	9	14
4	1690	205	0	14	9	14
5	1150	139	0	10	6	10
6	1150	139	0	10	6	10
7	1690	205	0	14	9	14

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	0.38	0.01	0.28	0.16	0.35	0.01	L/22401
2	0.61	0.01	0.45	0.40	0.58	0.01	L/13776
3	0.55	0.01	0.40	0.32	0.52	0.01	L/15243
4*	0.54	0.01	0.40	0.31	0.52	0.01	L/21776
5*	0.37	0.01	0.28	0.14	0.35	0.01	L/32002
6	0.37	0.01	0.28	0.14	0.35	0.01	L/22401
7	0.54	0.01	0.40	0.31	0.52	0.01	L/15243
MAX. --->	0.61	0.01	0.45	0.40	0.58	0.01	L/13776
	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): 0 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 = 113 PSI <--- O.K.
f_b = 0 PSI <--- O.K.
Δ_{MAX} = 0.000 IN

PRCNC20231424

Project: Arco Puyallup | Job Number: 23-703

Sheet: ____ of ____ | Name: BRT

Originating Office: Portland | Date: 09/25/23

PCS Structural Solutions

STUD WALL DESIGN - East Wall Full-Ht (short)

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} = 16.0 PSF
$W_{SEISMIC}$ = 6.0 PSF

P_{DEAD} = 0 LBS
P_{LIVE} = 0 LBS
P_{SNOW} = 0 LBS
P_{WIND} = 0 LBS
$P_{SEISMIC}$ = 0 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} = 393.5 PSI
C_b = 1.13 (NDS 3.10.4)

MISCELLANEOUS:

HEIGHT = 15.5 FT
SPACING = 47 IN
ECCENTRICITY = 0.1 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	0.26	0.249	369	1170	456
2 & 3	1.15	1708	0.23	0.218	373	1346	456
4 & 5	1.60	2376	0.17	0.160	379	1872	456
6 & 7	1.60	2376	0.17	0.160	379	1872	456

APPLIED STRESSES - NDS CHAPTER 3 DESIGN EQUATIONS

CASE	$P_{APPLIED}$	f_c	$M_{LAT. LOAD}$	$M_{ECC.}$	M_{TOTAL}	f_b
1	0	0	588	0	588	467
2	0	0	588	0	588	467
3	0	0	588	0	588	467
4	0	0	847	0	847	672
5	0	0	1129	0	1129	896
6	0	0	494	0	494	392
7	0	0	374	0	374	297

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	0.00	0.40	0.00	0.40	0.00	0.41	L/456
2	0.00	0.35	0.00	0.35	0.00	0.41	L/456
3	0.00	0.35	0.00	0.35	0.00	0.41	L/456
4*	0.00	0.36	0.00	0.36	0.00	0.41	L/453
5*	0.00	0.48	0.00	0.48	0.00	0.55	L/339
6	0.00	0.21	0.00	0.21	0.00	0.34	L/543
7	0.00	0.16	0.00	0.16	0.00	0.26	L/717
MAX. --->	0.00	0.48	0.00	0.48	0.00	0.55	L/339
	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' = 150 PSI
F_b' = 1346 PSI

STUD REACTIONS (OUT - OF - PLANE)

486 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 = 0 PSI <--- O.K.
f_b = 0 PSI <--- O.K.
Δ_{MAX} = 0.000 IN

Parapets

$q_p = 0.00256 k_z k_{zt} k_d k_e V^2$

$k_{zt} = 0.94$
$k_{zt} = 1.0$
$k_d = 0.85$
$k_e = 1.0$
$V = 97 mph$

$q_p = 19.2 psf$

$P = q_p [GC_p \; GC_{pi}]$

$GC_p \Rightarrow$

WALL ZONE	FACTOR
4	0.8 & -0.9
5	0.8 & -1.05
ROOF ZONE	
2	-1.5

CASE A

$P = 19.2 psf (0.8 + 0.18)$
$+ 19.2 psf (-1.5 + 0.18)$
$= 44.3 psf$ → controls

CASE B

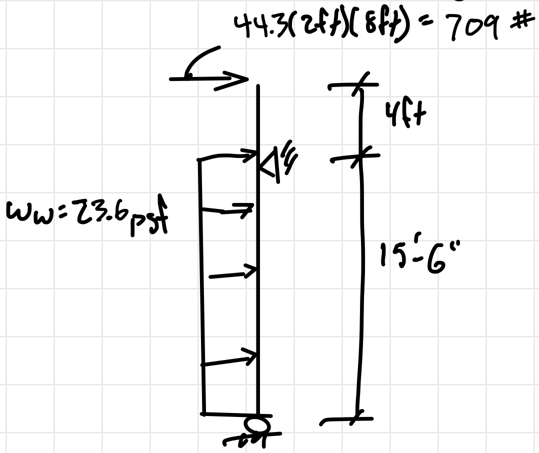
$P = 19.2 psf (0.8 + 0.18)$
$+ 19.2 psf (1.05 - 0.18) = 35.6 psf$

PRCNC20231424

PCS Structural Solutions

Project: ______ Job No: ______
Subject: ______ Sheet ______ Name: ______
Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

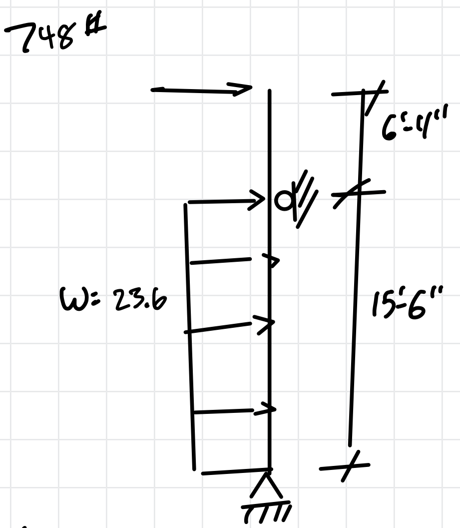
Typical wall framing



USE (3) 2x6 @ 8'0" O.C.

Δ= 2h/208

High parapet loading



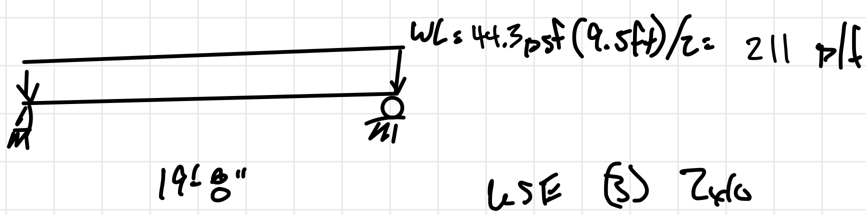
USE (4) 2x6 @ 5'4" O.C.

See attached

2h/180 ✓ GOOD

Check Entry

wall spans to top ℄



USE (3) 2x6

King studs @ front Windows

HT = 15'-6"

Trib = 16ft/2 + 16 in/12in/ft = 9'-4"
Eff area = 145ft^2

P_W = 22.6psf
W_W = 22.6psf(9.33ft) = 211 PLF

↳ USE (4) 2x6

$\frac{f_b}{F_b}$ = 0.74 Δ = 0.93" → L/200 < L/180 ✓ good

Check Jack studs @ Front windows

P_D = 20psf(20 ft)(16ft)/2 = 3200#

P_S = 25 psf(20 ft)(16ft)/2 = 4000#

USE (3) 2x6   See attached

PRCNC20231424

Wood Beam

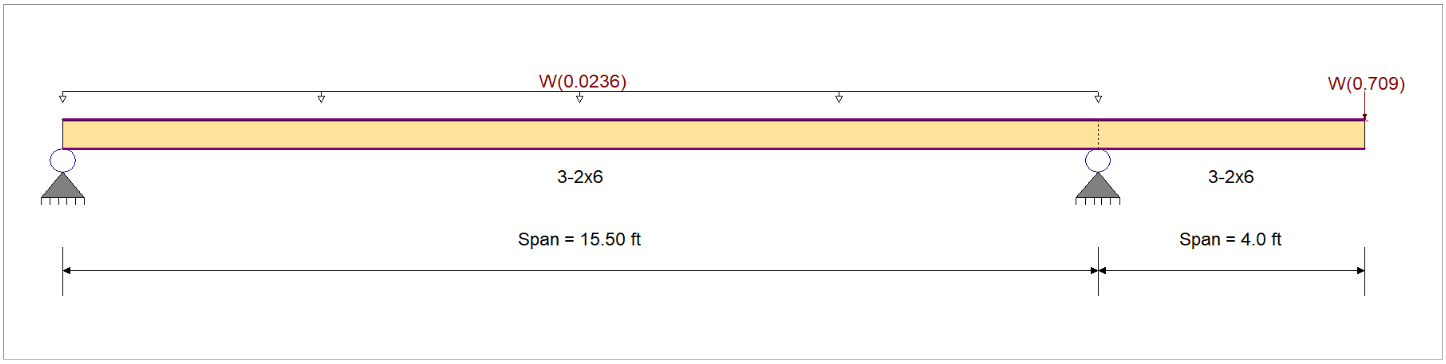
DESCRIPTION: TYPICAL PARAPET LOADING

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 +	975.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	975.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	550.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade :	No.1	Fv	150.0 psi		
		Ft	625.0 psi	Density	26.840 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 : W = 0.02360 , Tributary Width = 1.0 ft
Load for Span Number 2
Point Load : W = 0.7090 k @ 4.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.444** : 1	Maximum Shear Stress Ratio	=	**0.107** : 1
Section used for this span		**3-2x6**	Section used for this span		**3-2x6**
fb: Actual	=	900.02 psi	fv: Actual	=	25.78 psi
F'b	=	2,028.00 psi	F'v	=	240.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	15.500 ft	Location of maximum on span	=	15.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0	<180	n/a
Max Upward Transient Deflection	0 in	Ratio =	0	<180	n/a
Max Downward Total Deflection	0.458 in	Ratio =	208	>=180	Span: 2 : +0.420W
Max Upward Total Deflection	-0.209 in	Ratio =	888	>=180	Span: 1 : +0.420W

PRCNC20231424

Wood Beam

Project File: enercalc.ec6

LIC# : KW-06014122, Build:20.23.2.14 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2022

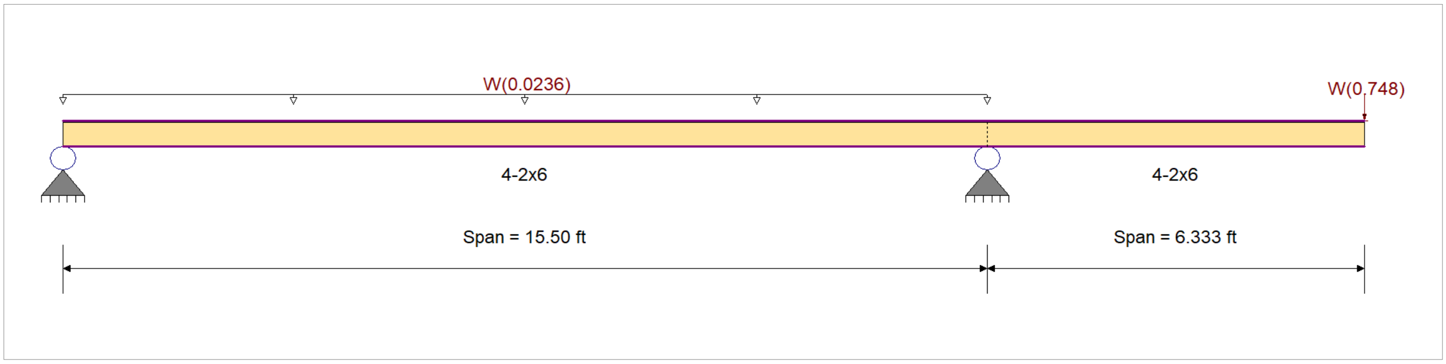
DESCRIPTION: HIGH PARAPET

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 +	975.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	975.0 psi	Ebend- xx	2,025.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	550.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade :	No.1	Fv	150.0 psi		
		Ft	625.0 psi	Density	26.840 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 : W = 0.02360 , Tributary Width = 1.0 ft
Load for Span Number 2
Point Load : W = 0.7480 k @ 6.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.556** : 1	Maximum Shear Stress Ratio	=	**0.085** : 1
Section used for this span		**4-2x6**	Section used for this span		**4-2x6**
fb: Actual	=	1,126.97 psi	fv: Actual	=	20.40 psi
F'b	=	2,028.00 psi	F'v	=	240.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	15.500 ft	Location of maximum on span	=	15.500 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <180	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <180	n/a
Max Downward Total Deflection	0.839 in	Ratio =	180 >=180	Span: 2 : +0.420W
Max Upward Total Deflection	-0.242 in	Ratio =	767 >=180	Span: 1 : +0.420W

PRCNC20231424

Wood Beam

Project File: enercalc.ec6

LIC# : KW-06014122, Build:20.23.2.14 PCS STRUCTURAL SOLUTIONS

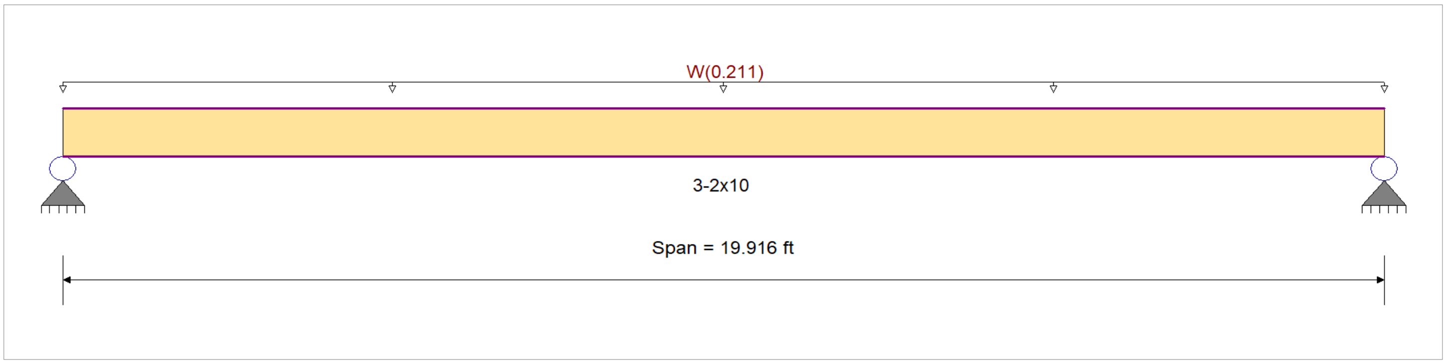
DESCRIPTION: ENTRY PARAPET

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 +	975.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-16	Fb -	975.0 psi	Ebend- xx	1,500.0 ksi
		Fc - Prll	1,350.0 psi	Eminbend - xx	550.0 ksi
Wood Species :	Hem-Fir	Fc - Perp	405.0 psi		
Wood Grade :	No.1	Fv	150.0 psi		
		Ft	625.0 psi	Density	26.840 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 : W = 0.2110 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.684**: 1	Maximum Shear Stress Ratio	=	**0.175** : 1
Section used for this span		**3-2x10**	Section used for this span		**3-2x10**
fb: Actual	=	1,173.77 psi	fv: Actual	=	42.11 psi
F'b	=	1,716.00 psi	F'v	=	240.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	9.958 ft	Location of maximum on span	=	19.189 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.709 in	Ratio =	337 >=180	Span: 1 : +0.420W
Max Upward Transient Deflection	0 in	Ratio =	0 <180	n/a
Max Downward Total Deflection	1.013 in	Ratio =	236 >=180	Span: 1 : +0.60W
Max Upward Total Deflection	0 in	Ratio =	0 <180	n/a

PRCNC20231424

Wood Beam

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS (c) ENERCALC INC 1983-2023

DESCRIPTION: King Studs @ Front Window

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : IBC 2018

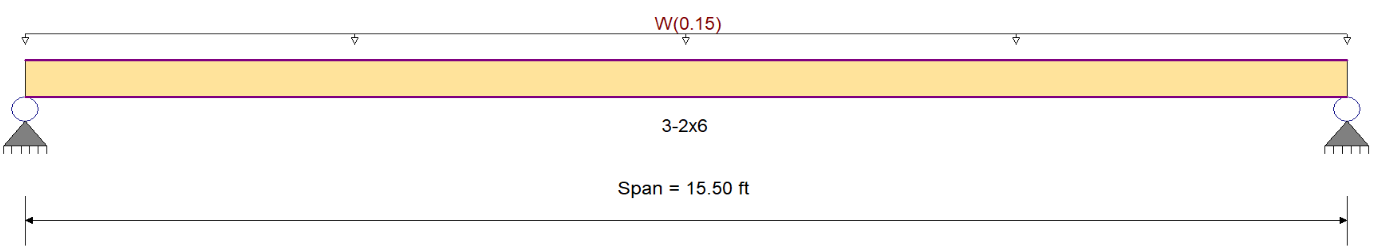
Wood Species : Hem-Fir
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb +	975	psi
Fb -	975	psi
Fc - Prll	1350	psi
Fc - Perp	405	psi
Fv	150	psi
Ft	625	psi

E : Modulus of Elasticity

Ebend- xx	1500	ksi
Eminbend - xx	550	ksi
Density	26.84	pcf



Applied Loads

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

Beam self weight NOT internally calculated and added
Uniform Load : W = 0.150 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.705** : 1	Maximum Shear Stress Ratio	=	**0.166** : 1
Section used for this span		**3-2x6**	Section used for this span		**3-2x6**
fb: Actual	=	1,429.59 psi	fv: Actual	=	39.80 psi
F'b	=	2,028.00 psi	F'v	=	240.00 psi
Load Combination		+0.60W	Load Combination		+0.60W
Location of maximum on span	=	7.750 ft	Location of maximum on span	=	15.047 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Upward Transient Deflection	0 in	Ratio =	0 <360	n/a
Max Downward Total Deflection	0.879 in	Ratio =	211 >=180	Span: 1 : +0.420W
Max Upward Total Deflection	0 in	Ratio =	0 <180	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
														0.0	0.00	0.0	0.0
Length = 15.50 ft	1			0.90	1.00	1.00	1.00	1.300	1.00	1.00	1.00			1,140.8	0.00	0.0	135.0
+0.60W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 15.50 ft	1	0.705	0.166	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	2.70	1,429.6	2,028.0	0.66	39.8	240.0
+0.450W					1.00	1.00	1.00	1.300	1.00	1.00	1.00			0.0	0.00	0.0	0.0
Length = 15.50 ft	1	0.529	0.124	1.60	1.00	1.00	1.00	1.300	1.00	1.00	1.00	2.03	1,072.2	2,028.0	0.49	29.9	240.0

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+0.420W	1	0.8794	7.807		0.0000	0.000

PRCNC20231424

DESCRIPTION: King Studs @ Front Window

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	1.163	1.163
Max Upward from Load Combinations	0.698	0.698
Max Upward from Load Cases	1.163	1.163
+0.60W	0.698	0.698
+0.450W	0.523	0.523
W Only	1.163	1.163

PRCNC20231424

Wood Column

Project File: 23-703 Enercalc BRT.ec6

DESCRIPTION: Jack Stud At Front Window

Code References

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

General Information

Analysis Method	Allowable Stress Design	Wood Section Name	**2-2x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	10 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Hem-Fir	Exact Width	**3.0** in
Wood Grade	No.1	Exact Depth	**5.50** in
		Area	16.50 in^2
		Ix	41.594 in^4
		Iy	12.375 in^4

Fb +	975 psi	Fv	150 psi
Fb -	975 psi	Ft	625 psi
Fc - Prll	1350 psi	Density	26.84 pcf
Fc - Perp	405 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1500	1500	1500 ksi
Minimum	550	550	

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compressioı	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Fact	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0
Use Cr : Repetitive ?	No

Column Buckling Condition:
Fully braced against buckling ABOUT X-X Axis
Fully braced against buckling ABOUT Y-Y Axis

Applied Loads

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

Column self weight included : 30.754 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, Xecc = 0.750 in, D = 2.295, L = 2.295 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio =	**0.3626 : 1**
Load Combination	+D+L
Governing NDS Forumla	Comp + Myy, NDS Eq. 3.9-3
Location of max.above base	9.933 ft
At maximum location values are .	
Applied Axial	4.621 k
Applied Mx	0.0 k-ft
Applied My	-0.2850 k-ft
Fc : Allowable	1,485.0 psi
PASS Maximum Shear Stress Ratio =	**0.01739 : 1**
Load Combination	+D+L
Location of max.above base	10.0 ft
Applied Design Shear	3.912 psi
Allowable Shear	150.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.02869 k	Bottom along X-X	0.02869 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X -0.1728 in at 5.839 ft above base
for load combination : +D+L

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	1.000	0.1928	PASS	9.933 ft	0.009659	PASS	10.0 ft
+D+L	1.000	1.000	0.3626	PASS	9.933 ft	0.01739	PASS	10.0 ft
+D+0.750L	1.250	1.000	0.2464	PASS	9.933 ft	0.01217	PASS	10.0 ft
+0.60D	1.600	1.000	0.06258	PASS	9.933 ft	0.003260	PASS	10.0 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	-0.014	0.014			2.326				
+D+L	-0.029	0.029			4.621				
+D+0.750L	-0.025	0.025			4.047				

PRCNC20231424

Wood Column

Project File: 23-703 Enercalc BRT.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS

DESCRIPTION: Jack Stud At Front Window

Maximum Reactions

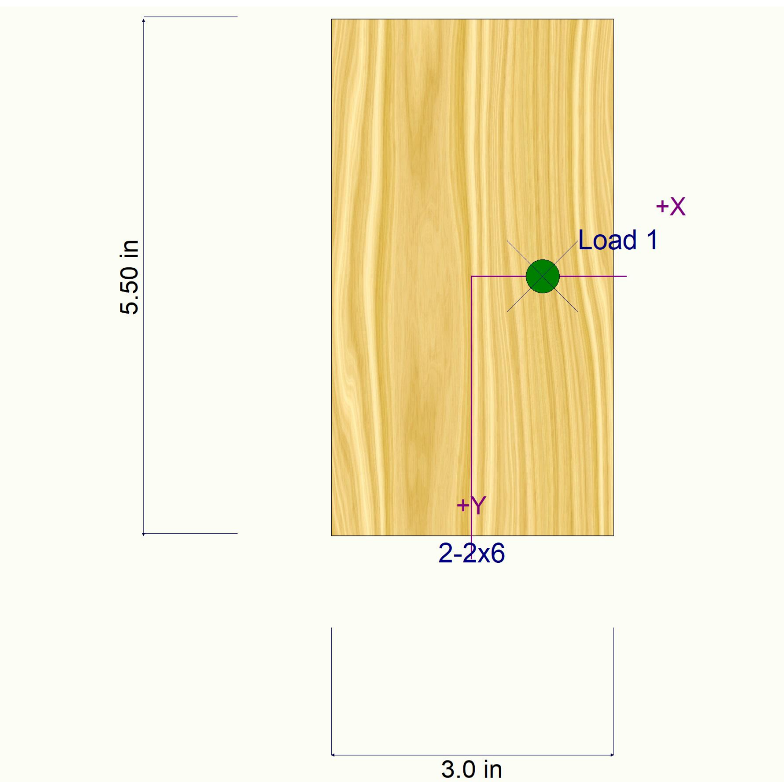
Note: Only non-zero reactions are listed.

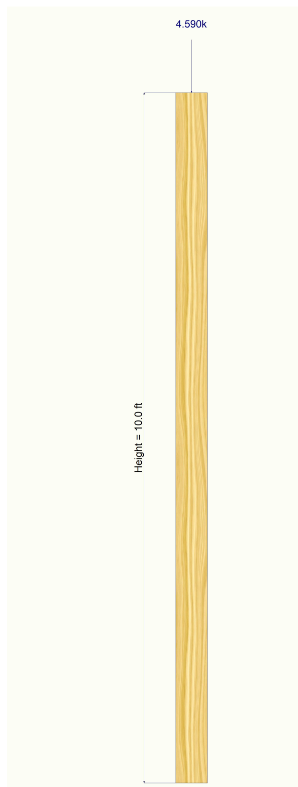
Load Combination	X-X Axis Reaction (k) @ Base	X-X Axis Reaction (k) @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments (k-ft) @ Base	My - End Moments (k-ft) @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+0.60D	-0.009	0.009			1.395				
L Only	-0.014	0.014			2.295				

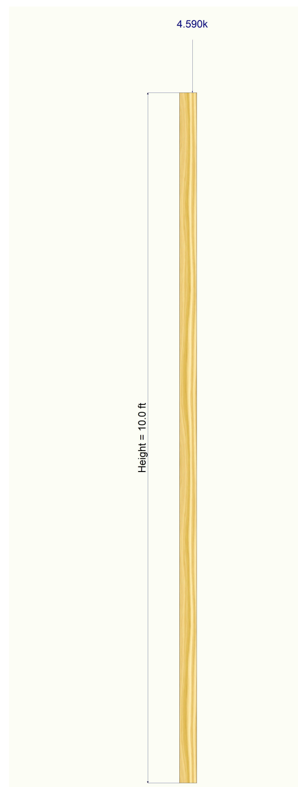
Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	-0.0864 in	5.839ft	0.000 in	0.000 ft
+D+L	-0.1728 in	5.839ft	0.000 in	0.000 ft
+D+0.750L	-0.1512 in	5.839ft	0.000 in	0.000 ft
+0.60D	-0.0518 in	5.839ft	0.000 in	0.000 ft
L Only	-0.0864 in	5.839ft	0.000 in	0.000 ft

Sketches

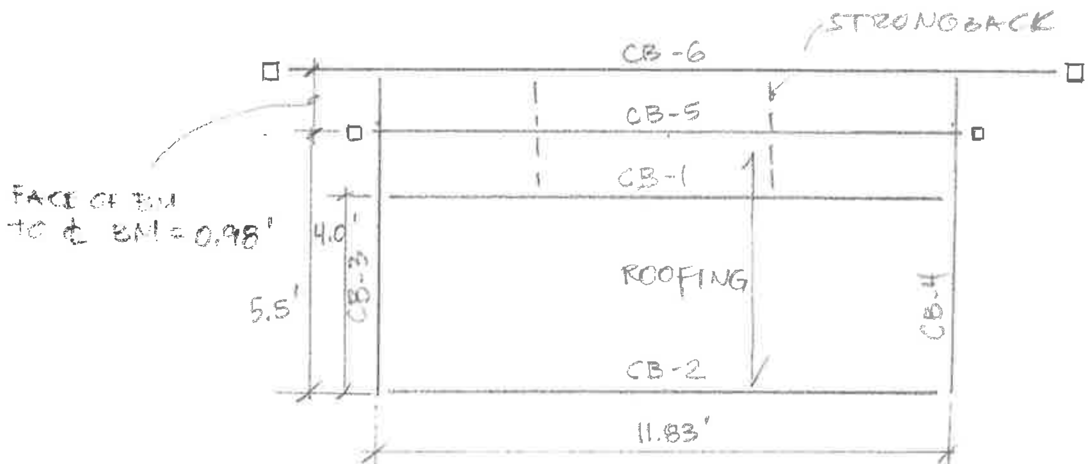






PRCNC20231424

MAIN CANOPY



snow load and wind loads are conservative for canopy design

DEAD LOAD = 10 PSF CONSERVATIVE - ACTUAL IS ~5 PSF

LIVE LOAD = 20 psf

SNOW LOAD: 25 PSF OR DRIFT

$\gamma = 0.13(25) + 14 = 17.25$ PCF

$P_f = 0.7\, C_e C_t I_s P_g$

$= 0.7(1.2)(1.2)(1.0)(25 \text{ psf})$

$= 25.2$ psf

$h_b = \frac{25.}{17.25} = 1.46'$

$h_d = 0.43 \sqrt[3]{l_u}\, \sqrt[4]{P_g + 10} - 1.5$

$= 0.43 \sqrt[3]{39}\, \sqrt[4]{25 + 10} - 1.5$

$= 2.02'$

$P_{MAX} = 17.25 \text{ psf}\,(1.45 + 2.02)$

$= 60$ psf

$W = 4 h_d = 4(2.02') = 8.0'$

~~OR DESIGN IS ALSO OK FOR 30 PSF UNIFORM SNOW IN EXPOSURE C CATEGORIES WHERE $C_e = 1.0$~~

$P_f =$

$= 0.7(1.0)(1.2)(1.0)(30 \text{ psf})$

$= 25.2$ psf

PRCNC20231424

WIND ON CANOPIES

ASCE 7-16
CH 30.11

$p = q_h G C_p$

$q_h = 0.00256 K_z K_{zt} K_d K_e V^2$

$K_z = 0.90$

$K_{zt} = 1.00$

$K_d = 0.85$

$K_e = 1.00$

$V = 110$ MPH

$= 0.00256(0.90)(1.00)(0.85)(1.00)(110)^2$

$= 23.7$ PSF

GC_p

$\frac{h_c}{h_e} = \frac{10'}{20'} = 0.5'$

$CC_p < +0.90$ FOR $A_{EFF} = 5'(11') = 55\ FT^2$

> -0.90

$= 23.7\ psf(\pm 0.90)$

$= \pm 21$ psf

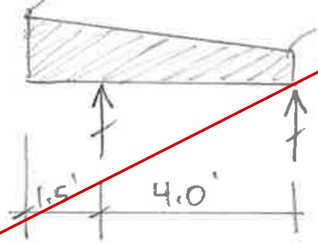
PRCNC20231424

ROOFING

D+S

10 psf + 60 psf = 70 psf

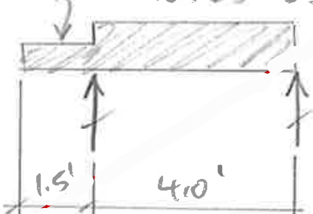
10 psf + $\left(25 + \frac{2.5}{8.0}(60-25)\right)$ = 49 psf



238 #/FT 89 #/FT

$10 + \frac{25}{2} = 22.5$ psf

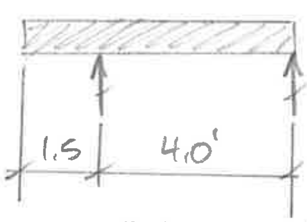
10 + 25 = 35 psf



110 #/FT 64 #/FT

0.6D + 0.6W

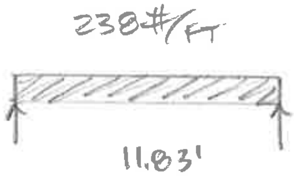
0.6(10 psf) − 0.6(22.8 psf) = −8 psf



−30 #/FT −14 #/FT

PRCNC20231424

CB-1

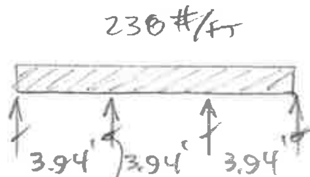


$$M = \frac{238(11.83)^2}{8} = 16{,}654\#' = 199{,}848\#''$$

$$I_{rqd} = \frac{5(238)(11.83)^4 1728}{384(29\times10^6)\frac{11.83(12)}{180}} = 4.6\,\text{in}^4$$

(2) 1200S200-68
(2) 1000S200-97

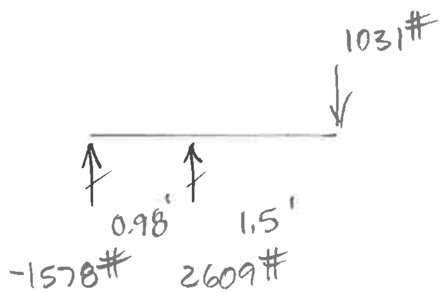
OR ADD STRONGBACKS TO REDUCE CB-1 SPAN AND USE 1000S250-54



$0.4(3.94)(238\#/\text{FT}) = 375\#$

$(0.6+0.5)(3.94')(238\#/\text{FT}) = 1031\#$

STRONG BACK



$$M = 1031\#(1.5') = 1547\#' = 18{,}564\#''$$

$$I_{rqd} = \frac{1031\#(1.5)^2(2.48)1728}{3(29\times10^6)\frac{(2)(1.5')(12)}{180}} = 0.6\,\text{in}^4$$

UNBRACED 18"

PER AISC CHART 11-2b

$KL_y = 2.0(18") = 36"$

600S162-54 OR BIGGER

USE 1000S250-54

$M_n = 48000\#''$

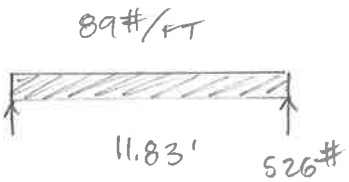
$$\frac{M_n}{\Omega} = \frac{48000\#''}{1.67} = 28{,}743\#''$$

PRCNC20231424

CB-2

$$M = \frac{89\#/FT\,(11.83)^2}{8} = 1557\#' = 18,683\#''$$

UNBRACED 142"



PER AISC CHART 11-26

10005250-54

PER AISI MANUAL CHART 11-26

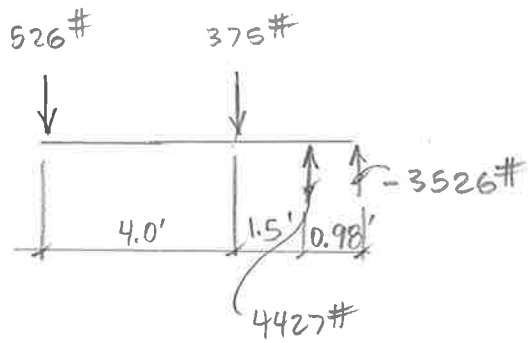
$M_n = 43,000\#''$

$$\frac{M_n}{\Omega_b} = \frac{43000\#''}{1.67} = 25,749\#''$$

OK

PRCNC20231424

CB-3 AND CB-4



$M = 526\#(5.5') + 375\#(1.5')$

$= 3456\#'$

$= 41{,}466\#''$

PER AISI MANUAL CHART 11-2b

$kL_y = 2(4')(12''/1') = 96''$

1000S250-68

$M_n = 102{,}000\#''$

$\frac{M_n}{\Omega_b} = \frac{102000\#''}{1.67} = 61{,}078\#''$

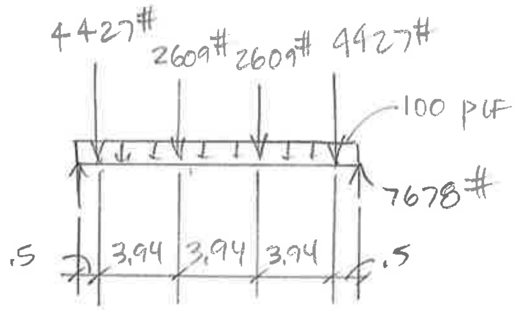
DocuSign Envelope ID: E969C03A-0A0E-40C3-8854-FEC7F26D90B6

PCS Structural Solutions

Project: Arco 3000 Prototype | Job No: 17712
Subject: | Sheet | Name: JJP
Originating Office: ☐ Seattle ☑ Tacoma ☐ Portland | Date: 4/5/18

PRCNC20231424

CB-5



NOTE:
SOFFIT AT ENTRY
REDUCES BEAM
LOAD - IT IS
NEGLECTED IN
ANALYSIS

$$A_{rqd} = \frac{1.5(7678)}{240(1.15)} = 41 \text{ IN}^2$$

$$M = 2609\#(4.44') + 4427\#(.5') + \frac{100(12.83)^2}{8}$$

$$= 15{,}855\#'$$

$$S_{rqd} = \frac{15855\#'(12''/1')}{2400 \text{ psi}(1.15)} = 69 \text{ IN}^3$$

$$\Delta = \frac{2069(4.44)(3(12.83)^2 - 4(4.44)^2)1728}{24(1.8\times10^6)I}$$

$$+ \frac{4427(0.5)(3(12.83)^2 - 4(0.5)^2)1728}{24(1.8\times10^6)I}$$

$$+ \frac{5(100)(12.83)^4 1728}{384(1.8\times10^6)I}$$

$$= \frac{152 + 44 + 34}{I}$$

$$= \frac{230}{I}$$

$$\Delta_{MAX} = \frac{12.83(12)}{240} = 0.64''$$

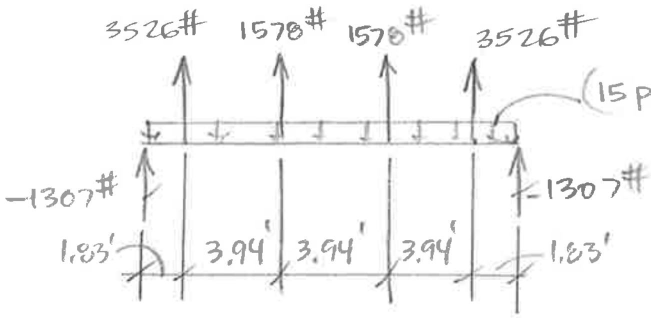
$$I_{rqd} = \frac{230}{0.64''} = 359 \text{ IN}^4$$

5 1/8 x 10 1/2 GL
USE 5 1/8 x 12 GL

PRCNC20231424

CB-6

D + S DRIFT ON CANOPY + $\frac{1}{2}$ SNOW ON ROOF



NOTE: SOFFIT AT ENTRY REDUCES LOADS - IT IS NEGLECTED IN THIS ANALYSIS

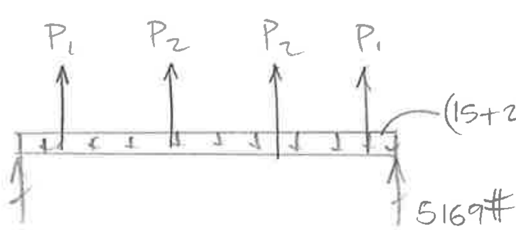
$$M = -1307\#(7.75') + 1578(1.97') + 3526(5.91') - \frac{490(7.75)^2}{2} =$$

$$= 897\#'$$

D + S_{ROOF} + NO SNOW ON CANOPY

$$P_1 \sim 3526\# \frac{10\text{ psf}}{90\text{ psf}} = 392\#$$

$$P_2 \sim 1578\# \frac{10}{90} = 175\#$$



$$M = 5169(7.75) + 175(1.97') + 392(5.91) - \frac{740(7.75)^2}{2}$$

$$= 20,498\#'$$

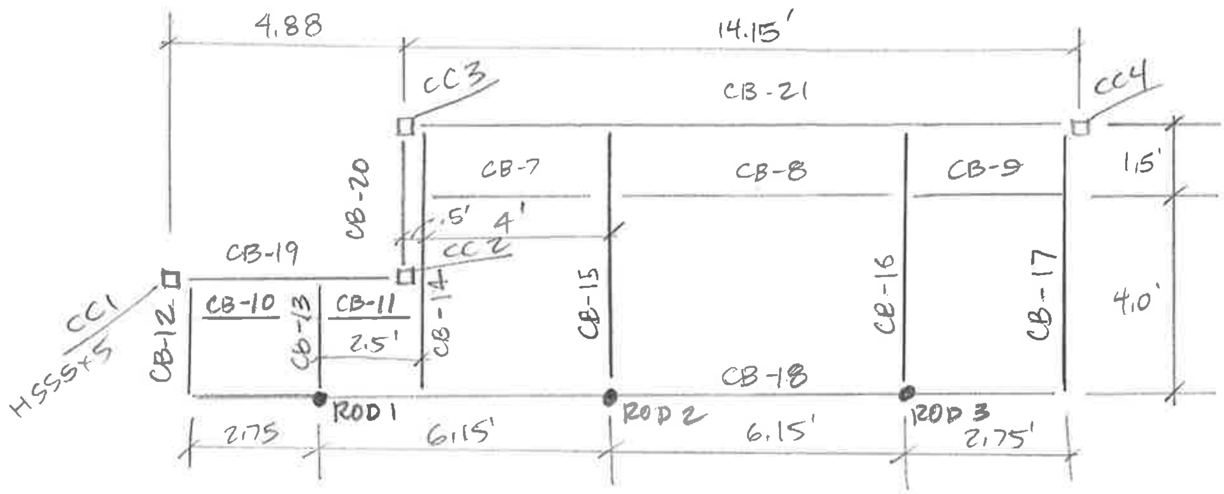
$$S_{rqd} = \frac{20498(12)}{2400(1.15)} = 89\text{ IN}^3$$

$$I_{rqd} = \frac{5(20498)(15.5)^2 1728}{48(1.8\times10^6)\frac{15.5(12)}{240}} = 635\text{ IN}^4$$

5 1/8 x 12 GL

PRCNC20231424

SIDE CANOPY



SEE MAIN CANOPY FOR:

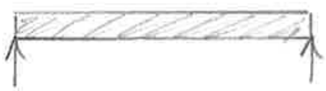
WIND UPLIFT

SNOW LOAD

PRCNC20231424

CB-7 TO CB-9

238 PLF (SEE MAIN CANOPY FOR D+S)



$$M = \frac{238\ \text{PLF}(6)^2}{8}$$

$$= 1071\ \#'$$

$$= 12{,}852\ \#''$$

$$KL = 1.0(6.0')(12''/1') = 72''$$

FROM AISI MANUAL CHART 11-2b

1000S 200-54

$$M_n = 73\ k''$$

$$\frac{M_n}{\Omega_b} = \frac{73\ k''}{1.67}$$

$$= 44\ k''$$

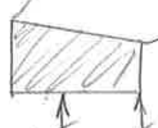
OK - USE 1000S 200-54

PRCNC20231424

CB-10 TO CB-11

SNOW DRIFT

10 psf D
60 psf S



10 psf D

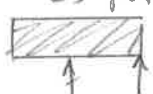
$25 + \frac{5.5}{8.0}(60-25) = 49$ psf S

1' 1.5'

4 #/FT D
19 #/FT S

21 #/FT D
117 #/FT S

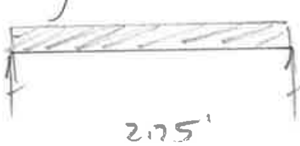
10 psf D
25 psf S



1' 1.5'

21 #/FT D 4 #/FT D
52 #/FT S 11 #/FT S

21 #/FT D
117 #/FT S



2.75'

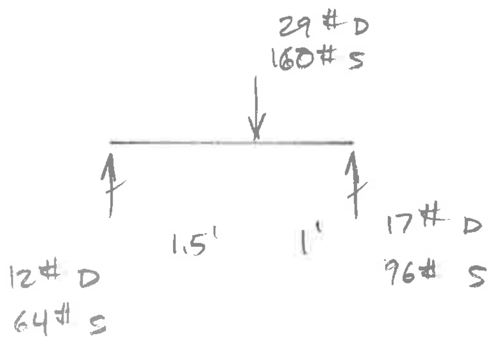
USE 1000S250-54

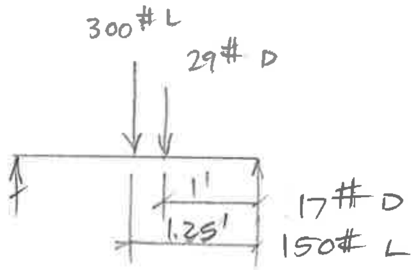
$M = \frac{(21+117)2.75^2}{8} = 130\#'$

29# D
160# S

PRCNC20231424

CB-12





$$M = 17(1.25) + 150(1.25) - 29(.25)$$

$$= 202\#'$$

$$= 2418\#''$$

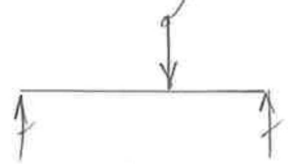
USE 1000S200-54

PRCNC20231424

CB-13

21#/FT (2.75 + 2.5)/2 = 55# D

117#/FT (2.75 + 2.5)/2 = 307# S



1.5'   1'

22# D
123# S

33# D
184# S

M = (22 + 123) 1.5 = 218#'

= 2610#"

kLy = 1.0 (1.5')(12"/1') = 18"

800S200-54

AISI MANUAL
CHART II-B

$M_n = 75^{k''}$

$$\frac{M_n}{\Omega_b} = \frac{75^{k''}}{1.67}$$

$= 45^{k''}$ OK

PRCNC20231424

CB-14

FROM CB11

$21\#/' \left(\frac{2.5}{2}\right) = 26\#$ D

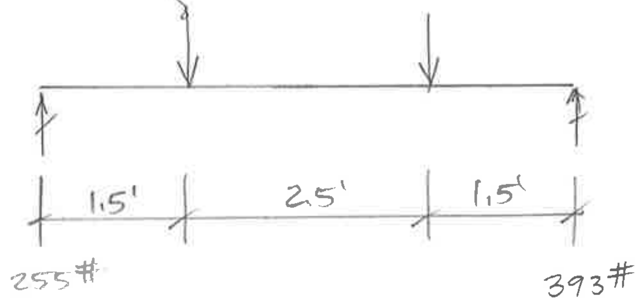
$117\#/' \left(\frac{2.5}{2}\right) = 146\#$ S

172#

FROM CB-7

238 PLF (2')

= 476#



$M = 393\#(1.5') = 590\#'$

$= 7074\#''$

AISI MANUAL
CHART II-B

800S200-54

$KL_y = 1.0(2.5)(12) = 30''$

$M_n = 75\,k''$

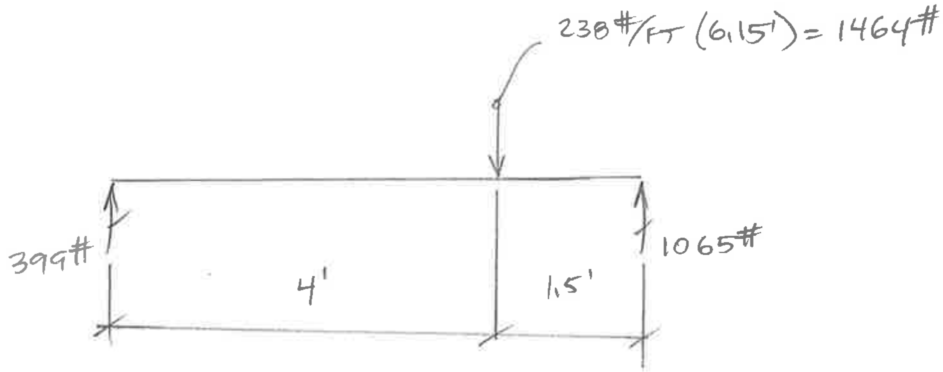
$\frac{M_n}{\Omega_b} = \frac{75\,k''}{1.67}$

$= 45\,k''$

USE 800S200-54

PRCNC20231424

CB-15



$$M = \frac{1464\#(4')(1.5')}{5.5'} = 1597\#'$$

$$= 19{,}165\#''$$

AISI MANUAL
CHART 11-B
1000S200-54

$kL_y = 1.(4')(12''/1) = 48''$

$M_n = 80k''$

$$\frac{M_n}{\Omega_b} = \frac{80k''}{1.67} = 48k'' \quad \text{OK}$$

ALSO CHECK w/ AXIAL LOAD FROM ROD - SEE FOLLOWING PAGES

$C = 1156\#$

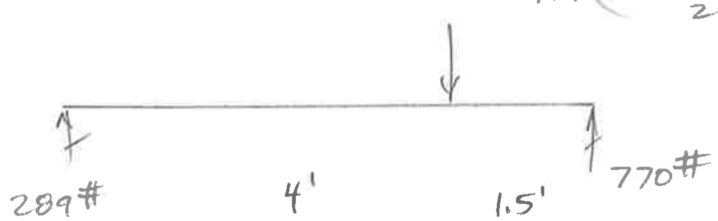
$$f_c = \frac{1156\#}{.839\ in^2} = 1378\ psi$$

$$\frac{Kl}{r} = \frac{1.0(4')(12''/1)}{.671} = 72 \quad \text{OK}$$

PRCNC20231424

CB-16

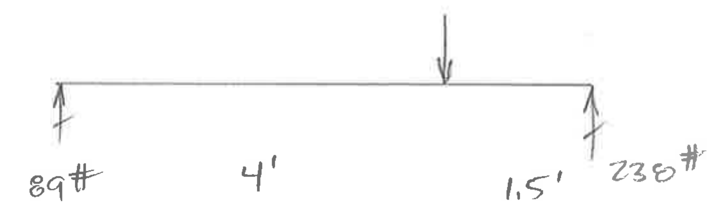
$238\#/\text{FT}\left(\frac{6.15'+2.75'}{2}\right) = 1059\#$



1000S 200-54

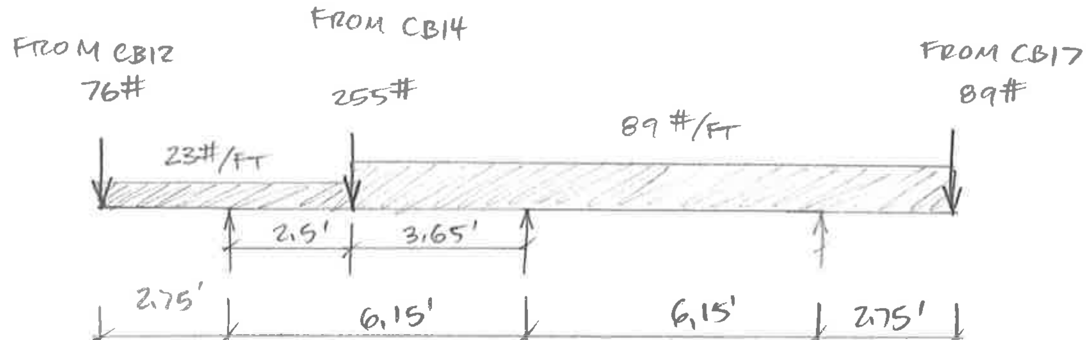
CB-17

$238\#/\text{FT}\ \frac{2.75'}{2} = 327\#$



PRCNC20231424

CB-18



	0	1.0	.5	.5	1.0	0
	209	-225	154	-281	281	-245
	87	-46	12			-337
		-34	293			
	0	9	-89	-89	301	0
		-45	5	151	-45	
	0	45	-78	-78	45	0
		-39	23	23	-39	
		39	-23	-23	39	
M	296	-296	297	-297	582	-582
R	139#	293#	344#	227#	320#	334#

$$M_- = \frac{89(2.75)^2}{2} + 89(2.75)$$
$$= 581\#'$$
$$= 6975\#''$$

AISI MANUAL
CHART II-B

1000S200-54 $\quad Kl/y = 1.0(2.75)(12''/') = 33''$

$M_n = 86\,k''$

$$\frac{M_n}{\Omega_b} = \frac{86\,k''}{1.67} = 51\,k''$$

1000S200-54

PRCNC20231424

SUPPORT ROD

ROD 1

D+S $(22^{\#} + 123^{\#}) + (139^{\#} + 293^{\#}) = 577^{\#}$

CB-13 CB-18

ROD 2

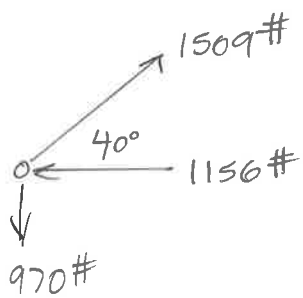
D+S $(399^{\#}) + (344^{\#} + 227^{\#}) = 970^{\#}$

CB-15 CB-18

ROD 3

D+S $(289^{\#}) + (320^{\#} + 334^{\#}) = 943^{\#}$

CB-16 CB-18



TRY 1"φ STD PIPE

$$f = \frac{1509^{\#}}{.494\ in^2} = 3055\ \text{PSI}$$

FOR WIND UPLIFT

$L = 6'$

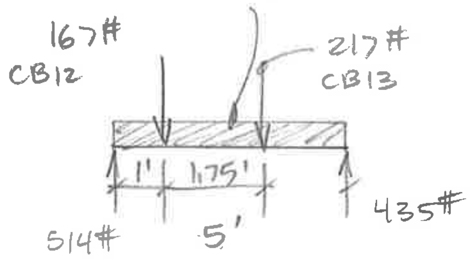
$$\frac{kl}{r} = \frac{1.0(6)(12)}{.421} = 171$$

OK

PRCNC20231424

CB-19 D+S

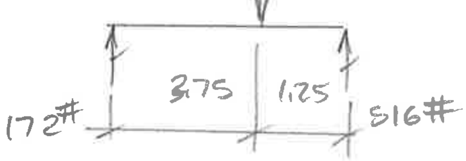
ROOF w/ SNOW DRIFT (15 PSF + 60 PSF) 1.5' = 113 PLF



VERT LOAD

$$M_x = 514(3.07') - \frac{113(3.07)^2}{2} - 167(2.07)$$

$$= 700 \#'$$



$$\frac{577\#}{\tan 40} = 688\# \text{ FROM ROD 1}$$

HORIZ LOAD

$$M_y = \frac{688\#(3.75')(1.25')}{5'}$$

$$= 645\#'$$

TRY 6x10

$$f_{bx} = \frac{700\#'(12''/')}{83 \, in^3} = 101 \text{ psi}$$

$$f_{by} = \frac{645\#'(12''/')}{\frac{9.5(5.5)^2}{6}} = 162 \text{ psi}$$

6x10
OK BY INSP

CB-20

6x10 BY INSP

PCS Structural Solutions

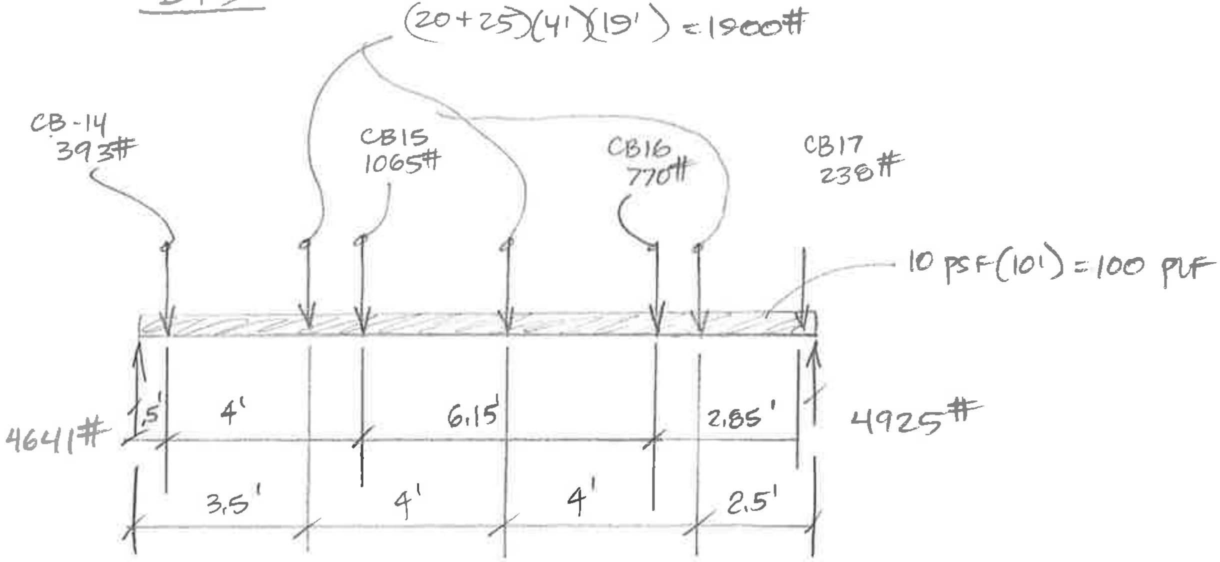
Project: ARCO PROTOTYPE Job No: 17712

Subject: Sheet Name: JJP

Originating Office: ☐ Seattle ☑ Tacoma ☐ Portland Date: 4/24/18

CB-21

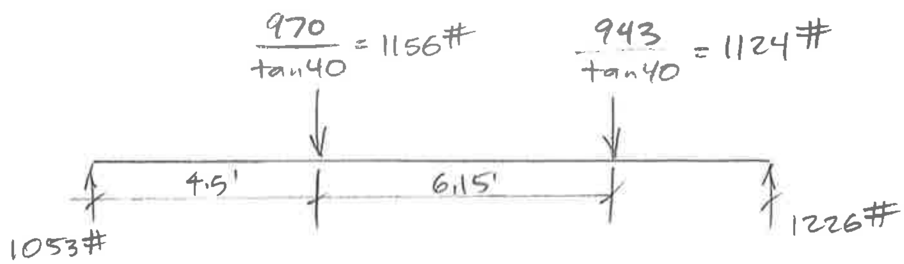
D+S



$V = 0$ AT $x = 7.5'$

$M_x = 4641\#(7.5') - 393(7.0) - 1900(4') - (1065)(3.5') - 100\frac{(7.5)^2}{2}$

$= 17{,}917\#'$



$M = 1053\#(4.5') = 4740\#'$

PRCNC20231424

TRY 5 1/8 x 16 1/2 GL

$$f_{bx} = \frac{17917 \#' (12''/1)}{232.5 \text{ IN}^3} = 925 \text{ psi}$$

$$C_L = \frac{1+F_{bE}/F_b^*}{1.9} - \sqrt{\left[\frac{1+F_{bE}/F_b^*}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

$$F_{bE} = \frac{1.20 E'_{min}}{R_B^2}$$

$$E'_{min} = 930,000 \text{ psi}$$

$$R_B = \sqrt{\frac{l_e d}{b^2}}$$

$$l_e = 2.06\, l_u$$

$$l_u = 14'$$

$$= 2.06(14')(12''/1) = 346''$$

$$= \sqrt{\frac{346(16.5)}{5.125^2}}$$

$$= 14.74$$

$$= \frac{1.20(930000)}{14.74^2}$$

$$= 5134 \text{ psi}$$

$$F_b^* = 2400 \text{ psi}(1.15) = 2760 \text{ psi}$$

$$F_{bE}/F_b^* = 5134/2760 = 1.86$$

$$= \left(\frac{1+1.86}{1.9}\right) - \sqrt{\left[\frac{1+1.86}{1.9}\right]^2 - \frac{1.86}{.95}}$$

$$= 1.51 - \sqrt{2.27-1.96} = 0.95$$

PRCNC20231424

$$F_{bx}' = 2400\,psi\,\underset{C_D}{(1.15)}\,\underset{C_L}{(0.95)} = 2622\,psi$$

$$\frac{f_{bx}}{F_{bx}'} = \frac{925\,psi}{2622\,psi} = 0.35$$

$$f_{by} = \frac{4740\#'\,(12''/1)}{\frac{16.5\,(5.125)^2}{6}} = 787\,psi$$

$$F_{by}' = 1450\,psi\,\underset{C_D}{(1.15)} = 1668\,psi$$

$$\frac{f_{by}}{F_{by}'} = \frac{787\,psi}{1668\,psi} = 0.47$$

$$\frac{f_{bx}}{F_{bx}'} + \frac{f_{by}}{F_{by}'} \le 1.0$$

$$0.35 + 0.47 \le 1.0$$

$$0.82 \le 1.0 \quad \text{OK}$$

$$\Delta_{VERT} \approx \frac{5(17917)(14)^2 1728}{48(1.8\times10^6)\frac{5.125(16.5)^3}{12}} = 0.18''$$

$$\Delta_{HORIZ} \approx \frac{5(4740)(14)^2 1728}{48(1.6\times10^6)\frac{16.5(5.125)^3}{12}} = 0.56'' = \frac{L}{300} \quad \text{OK}$$

PRCNC20231424

WIND LOADING

WIND UPLIFT ON CANOPY = 21 PSF

WIND DOWN ON CANOPY = 21 PSF

D + 0.6 W

CONSERVATIVELY ASSUME DEAD LOAD IS $\frac{20}{45}$ THAT SHOWN FOR D+S WITH DRIFT

$M_x \approx \frac{20}{45} 17917\#' = 7963\#'$ D

$M_y \approx \frac{20}{45} 4740\#' = 2107\#'$ D

FROM WIND

$M_x \approx \frac{0.6(21\ psf)}{20} 7963\#' = 5017\#'$ 0.6W

$M_y \approx \frac{0.6(21\ psf)}{20} 2107\#' = 1327\#'$ 0.6W

ALSO - POSITIVE PRESSURE

$M_y = \frac{0.6(23.9\ psf)(7')(14')^2}{8} = 2459\#'$ 0.6W

TOTAL

$M_x = 7963\#' + 5017\#' = 12980\#'$

$f_{bx} = \frac{12980\#'(12''/')}{233} = 668\ psi$

$M_y = 2107\#' + 1327\#' + 2459\#' = 5893\#'$

$f_{by} = \frac{5893\#'(12''/')}{16.5(5.125)^2/6} = 979\ psi$

PRCNC20231424

$$F_{bx}' = 2400 \text{ psi} \underset{C_D}{(1.6)} \underset{C_L}{(0.95)} = 3648 \text{ psi}$$

$$F_{by}' = 1450 \text{ psi} (1.6) = 2320 \text{ psi}$$

$$\frac{f_{bx}}{F_{bx}'} + \frac{f_{by}}{F_{by}'} = \frac{668}{3648} + \frac{979}{2320}$$

$$= 0.18 + 0.42$$

$$= 0.60 < 1.0 \quad \text{OK}$$

FOR 0.7 (0.6W)

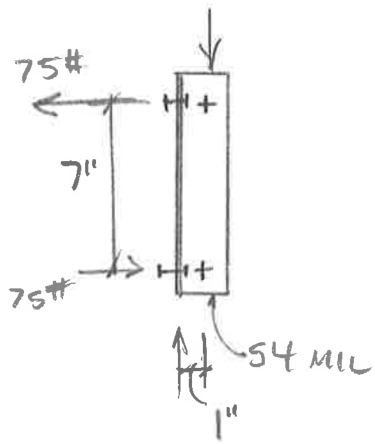
$$\Delta_{HORIZ} \approx \frac{5(2107 + 0.7(1327 + 2459))14^2 1728}{48(1.6\times10^6)\,\frac{16.5(5.125)^3}{12}} = 0.57 = \frac{L}{295} \quad \text{OK}$$

PRCNC20231424

BEAM-TO-BEAM CONNECTION

CB-2

$V_{MAX} = 526\#$



(2) 1/2" ϕ M.B.S.

$$f_{brg} = \frac{526\#}{0.5''(0.054'')(2)\text{BOLTS}}$$

$$= 9741 \text{ psi}$$

$$P_n = C m_f d t F_u$$

$C = 3.0$

$m_f = 1.0$

$d = 0.5$

$t = 0.054$

$F_u = 65 \text{ KSI}$

$$= 3.0(1.0)(0.5)(0.054)(65 \text{ KSI})$$

$$= 5.27 \text{ KIPS}$$

$$\frac{P_n}{\Omega} = \frac{5.27 \text{ KIPS}}{2.5} = 2.1 \text{ KIPS}$$

OK

SHEAR RUPTURE

$$V_n = 0.6 F_u A_{nv}$$

$$F_u = 65 \text{ KSI}$$

$$A_{nv} = 2 n t e_{net}$$

$$n = 2$$

$$t_1 = 0.054''$$

$$e_{net} = 1'' - \frac{(0.5 + 1/16)}{2} = 0.719''$$

$$= 2(2)(0.054)(0.719)$$

$$= 0.155 \text{ IN}^2$$

$$= 0.6(65 \text{ KSI})(0.155 \text{ IN}^2)$$

$$= 6.1 \text{ KIPS}$$

$$\frac{V_n}{\Omega} = \frac{6.1 \text{ KIPS}}{2.22} = 2.7 \text{ KIPS}$$ OK

TENSION RUPTURE

$T = 75^{\#}$

$T_n = F_u A_e$

$F_u = 65$ KSI

$A_e = U_{sl} A_{nt}$

$U_{sl} = 3.33 d/s \leq 1.0$

$d = 0.5''$

$s = \frac{2''}{1} =$

$= \frac{3.33(0.5)}{2} = 0.83$

$A_{nt} = A_g - n_b d_n t + (\Sigma s'^2 / 4g) t$

$A_g = 2(0.054) = 0.108$ IN^2

$n_b = 1$

$d_h = 1/2 + 1/16 = 0.563$

$t = 0.054$

$s' = 0$

$= 0.108 - (1)(0.563)(0.054)$

$= 0.078$ IN^2

$= 0.83(0.078$ IN$^2)$

$= 0.064$ IN^2

$= 65$ KSI$(0.064$ IN$^2)$

$= 4.19$ KIPS

$\frac{T_n}{\Omega} = \frac{4.19 \text{ KIPS}}{2.22} = 1.89$ KIPS OK

PRCNC20231424

BLOCK SHEAR RUPTURE

$R_n = 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$ OR $= 0.6 F_u A_{nv} + U_{bs} F_u A_{nt}$

$F_y = 50$ KSI

$A_{gv} = 9''(0.054) = 0.486 \text{ IN}^2$

$U_{bs} = 1.0$

$F_u = 65$ KSI

$A_{nt} = 0.078 \text{ IN}^2$

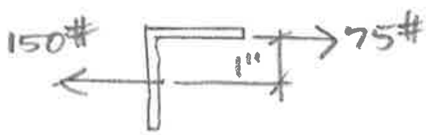
$A_{nv} = 0.155 \text{ IN}^2$

$= 0.6(50)(0.486) + 1.0(65)(0.078) = 19.7$ KIPS

OR $= 0.6(65)(0.155) + 1.0(65)(0.078) = 11.1$ KIPS

$\frac{R_n}{\Omega} = \frac{11.1 \text{ KIPS}}{2.22} = 5.0$ KIPS OK

ANGLE FLEXURE



$M = 75^{\#}(1'') = 75^{\#}{}''$

$l_{eff} = 2'' + 2'' = 4''$

$M_n = S_e F_y$

$S_e = \frac{4(0.054)^2}{6} = 0.0019 \text{ IN}^3$

$F_y = 50$ KSI

$= 0.0019 \text{ IN}^3 (50 \text{ KSI})$

$= 0.095 \text{ k}''$

$\frac{M_n}{\Omega_b} = \frac{0.095 \text{ k}''}{1.67} = 0.057 \text{ k}''$ N.G. → USE 68 MIL

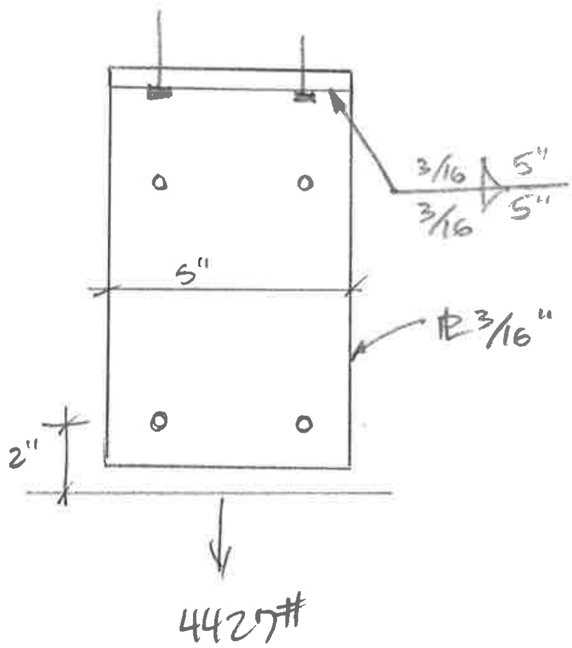
PRCNC20231424

CB3 TO BEAM

$V = 4427\#$

(4) BOLTS

$V = \frac{4427\#}{4} = 1107\#/\text{BOLT}$



TENSION RUPTURE

$T_n = F_u A_e$

$F_u = 65 \text{ KSI}$

$A_e = U_{sl} A_{nt}$

$U_{sl} = 1.0$

$A_{nt} = A_g - n_b d_h t + (\Sigma s'^2 / 4g) t$

$A_g = 0.054 \text{ IN} (5'') = 0.027 \text{ IN}^2$

$n_b = 2$

$d_h = 9/16'' = 0.563$

$t = 0.054''$

PRCNC20231424

$s' = 5''$

$g = 3''$

$A_{nt} = 0.270 - 2(0.563)(0.054) + \left(\frac{5^2}{4(3)}\right)0.054$

$= 0.270 - 0.0608 + 0.1125$

$= 0.322 \text{ IN}^2$

$A_e = 1.0(0.322 \text{ IN}^2)$

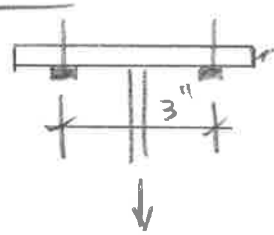
$= 0.322 \text{ IN}^2$

$T_n = 65 \text{ KSI}(0.322 \text{ IN}^2)$

$= 20.9 \text{ KIPS}$

$\frac{T_n}{\Omega} = \frac{20.9 \text{ KIPS}}{2.22} = 9.4 \text{ KIPS}$ OK

PLATE



4.4 kips

$M_{℄} = \frac{4.4 \text{ KIPS}(3)}{4} = 3.3 \text{ K}''$

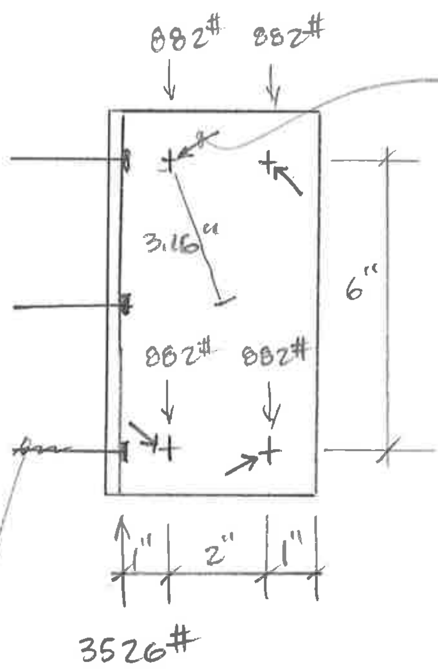
$S_{reqd} = \frac{3.3 \text{ K}''}{.75(36 \text{ KSI})} = 0.122 \text{ IN}^3$

$\frac{1}{2}'' \times 5''$ $S = \frac{(0.5)^2(5)}{6} = 0.21 \text{ IN}^3$ ✓

PRCNC20231424

CB-4 TO GLULAM BEAM

$V = 3526^{\#}$



$$\frac{3526^{\#}(2")}{4(3.16")} = 558^{2}$$

BOLT

$V_{MAX} = \sqrt{882^2 + 558^2}$

$= 1044^{\#}$

1/2" ∅ BOLT
~~68~~ MIL

$P_n = C m_f\, d\, t\, F_u$

$C = 3.0$

$m_f = 1.0$

$d = 0.5"$

$t = 0.068$

$F_u = 65$ KSI

$= 3.0(1.0)(0.5)(0.068)(65\text{ KSI})$

$= 6.6$ KIPS

$$\frac{P_n}{\Omega} = \frac{6.6\text{ KIPS}}{2.5} = 2.6\text{ KIPS}$$

OK

(6) 5/8" ∅ M.B.S.

$V = \frac{3526^{\#}}{6} = 588^{\#}/\text{BOLT}$

$V' = 710^{\#}(1.15) = 817^{\#}/\text{BOLT}$

PRCNC20231424

PCS Structural Solutions

Project: ______ Job No: ______

Subject: ______ Sheet ______ Name: ______

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date: ______

Mechanical Anchorage

PRCNC20231424

PCS Structural Solutions

Project: Arco Puyallup | Job Number: 23-703

Sheet: ______ of ______ | Name: BRT

Originating Office: Portland | Date: 9/20/2023

MECHANICAL UNIT ANCHORAGE SPREADSHEET

UNIT NAME:	RTU 1	LOCATION:	PUYALLUP, WA
CODE:	IBC 2021, ASCE 7-16		
UNIT EXT. OR INT.?	EXTERNAL		

UNIT INFORMATION: / CURB INFORMATION:

OPERATING WEIGHT (W_p) =	900 lb		IS THERE A CURB?	YES	
			CURB WEIGHT, $W_{p\text{-}2}$ =	100 lb	
UNIT HEIGHT (h) =	48 in	4.0 ft	CURB HEIGHT (h) =	18 in	1.5 ft
UNIT WIDTH (w) =	46 in	3.8 ft	CURB WIDTH AT BASE (w) =	46 in	3.8 ft
UNIT LENGTH (l) =	84 in	7.0 ft	CURB LENGTH AT BASE (l) =	84 in	7.0 ft

C.O.G. (VERTICAL) =	32 in	2.7 ft	(2/3)*UNIT HEIGHT (ASSUMED)

TOTAL WEIGHT (W_{tot}) =	1000 lb		DESIGN WIDTH (w_{des}) =	3.8 ft
h_{wind} =	2.8 ft	(1/2)*(UNIT HT + CURB HT)	DESIGN LENGTH (l_{des}) =	7.0 ft
$h_{seismic}$ =	4.2 ft	CENTER OF GRAVITY + CURB HT		

BUILDING INFORMATION

ARE BUILDING DIMENSIONS KNOWN?	YES	BUILDING WIDTH (B) =	42.0 ft	B*h =	1290 sq ft
		BUILDING LENGTH (L) =	86.0 ft	B*L =	3612 sq ft
		AVERAGE ROOF HT. (h) =	15.0 ft		

DESIGN CRITERIA - GRAVITY

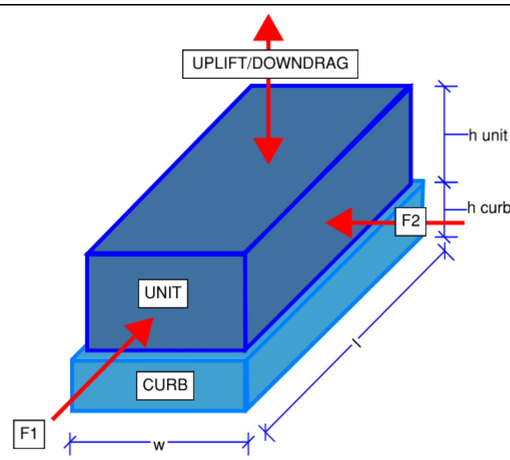
ROOF DEAD (DL) =	20 PSF
ROOF SNOW (SL) =	25 PSF
$P_{roof\ DL}$ =	537 lb
$P_{roof\ SL}$ =	671 lb

DESIGN CRITERIA - LATERAL (THE UNIT DOES NOT HAVE A SIGNIFICANT IMPACT TO THE BUILDING'S CAPACITY TO RESIST WIND OR SEISMIC FORCES)

SEISMIC (ASCE 7-16, CHAPTER 13):

COMPONENT TYPE: AIR-SIDE HVACR, FANS, AIR HANDLERS, A/C UNITS, CABINET HEATERS, AIR DISTRIBUTION BOXES, AND OTHER MECH. COMPONENTS CONSTRUCTED OF SHEET METAL FRAMING

S_{DS} =	0.845	
a_p =	2.5	(TABLE 13.6-1)
I_p =	1.5	(§ 13.1.3)
R_p =	6	(TABLE 13.6-1)
OVERSTRENGTH FACTOR (Ω_o) =	2.0	(TABLE 13.6-1)
ATTACHMENT OR BTM. OF CURB HT. (z) =	15.0 ft	
AVERAGE ROOF HT. (h) =	15.0 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/h]$ =	634 lb	(EQN. 13.3-1)
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	2028 lb	(EQN. 13.3-2)
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	380 lb	(EQN. 13.3-3)
HORIZ. EQ. DESIGN FORCE, F_p =	**634 lb**	
VERT. EQ. DESIGN FORCE, $F_{pv} = \pm 0.2*S_{DS}W_p$ =	**169 lb**	(§ 13.3.1.2)



WIND (ASCE7-16, CHAPTER 29 - DIRECTIONAL PROCEDURE):

WIND SPEED (V) =	97 MPH	
RISK CATEGORY:	II	(TABLE 1.5-2)
WIND EXPOSURE:	B	(§ 26.7.3)
TOPO. EFFECT (K_{zt}) =	1.0	(FIG. 26.8-1)
K_h =	0.57	(TABLE 26.10-1)
DIRECTIONALITY FACTOR, K_d =	0.85	(TABLE 26.6-1)
GROUND ELEV. FACTOR, K_e =	1.00	(TABLE 26.9-1)

		F_1	F_2	
	A_f =	21.0 sq ft	38.5 sq ft	
	A_r =	26.8 sq ft		
	$q_h = 0.00256*K_d*K_h*K_{zt}*K_e*V^2$ =	9.9 psf		(EQN. 26.10-1)
(§ 29.4.1)	GC_r =	1.90	1.90	
(§ 29.4.1)	GC_r =	1.50		
		F_1	F_2	
(EQN. 29.4-2)	$F_h = q_h*(GC_r)*A_f$ =	**396 lb**	**726 lb**	
(EQN. 29.4-3)	$F_v = q_h*(GC_r)*A_r$ =	**399 lb**		

PRCNC20231424

Project: Arco Puyallup
Job Number: 23-703
Sheet: of
Name: BRT
Originating Office: Portland
Date: 09/20/23

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	2641 lb-ft	2641 lb-ft
$M_{OT\ Wind}$ =	1090 lb-ft	1995 lb-ft
$M_{R\ SL}$ =	2348 lb-ft	1275 lb-ft
$M_{R\ DL\ Unit}$ =	3150 lb-ft	1710 lb-ft
$M_{R\ DL\ Curb}$ =	350 lb-ft	190 lb-ft

P_{SL} =	671 lb
$P_{DL\ Unit}$ =	900 lb
$P_{DL\ Curb}$ =	100 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	169 lb	169 lb	377 lb	695 lb	634 lb	634 lb
WIND:	399 lb	N/A	156 lb	525 lb	396 lb	726 lb
SNOW:	N/A	671 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	1000 lb	N/A	N/A	N/A	N/A

COMBINATIONS (ASD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.6DL-0.6WL	-1446 lb-ft	57 lb-ft
0.6DL-0.7EQ	-252 lb-ft	708 lb-ft
DL+SL	-5848 lb-ft	-3175 lb-ft
DL+0.6WL	-4154 lb-ft	-3097 lb-ft
DL+0.45WL+0.75SL	-5752 lb-ft	-3754 lb-ft
DL+0.7EQ	-5348 lb-ft	-3748 lb-ft
DL+0.525EQ+0.75SL	-6647 lb-ft	-4242 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$0.6*F_v$	240 lb	
$\pm 0.7*F_{pv}$	118 lb	118 lb
$0.6*F_v$	240 lb	
$0.45*F_v$	180 lb	
$\pm 0.7*F_{pv}$	118 lb	118 lb
$\pm 0.525*F_{pv}$	89 lb	89 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	N/A	135 lb	207 lb	N/A	-238 lb	-435 lb
$(0.6\text{-}0.14S_{DS})$DL-0.7EQ	23 lb	246 lb	95 lb	N/A	-444 lb	-444 lb
DL+SL	N/A	N/A	835 lb	835 lb	N/A	N/A
DL+0.6WL	N/A	N/A	593 lb	815 lb	238 lb	435 lb
DL+0.45WL+0.75SL	N/A	N/A	822 lb	988 lb	178 lb	327 lb
$(1+0.14S_{DS})$DL+0.7EQ	N/A	N/A	823 lb	1046 lb	444 lb	444 lb
$(1+0.105S_{DS})$DL+0.525EQ+0.75SL	N/A	N/A	994 lb	1161 lb	333 lb	333 lb

DESIGN LOADS (ASD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**23 lb**	**246 lb**	**994 lb**	**1161 lb**	**444 lb**	**444 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**1 SCREWS**			**1 SCREWS**	**1 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#14	12ga

PRCNC20231424

Project: Arco Puyallup　Job Number: 23-703

Sheet: of　Name: BRT

Originating Office: Portland　Date: 09/20/23

PCS Structural Solutions

MECHANICAL UNIT ANCHORAGE SPREADSHEET

MOMENTS AND REACTIONS (UNFACTORED)

	F_1	F_2
$M_{OT\ Seismic}$ =	2641 lb-ft	2641 lb-ft
$M_{OT\ Wind}$ =	1090 lb-ft	1995 lb-ft
$M_{R\ SL}$ =	2348 lb-ft	1275 lb-ft
$M_{R\ DL\ Unit}$ =	3150 lb-ft	1710 lb-ft
$M_{R\ DL\ Curb}$ =	350 lb-ft	190 lb-ft

P_{SL} =	671 lb
$P_{DL\ Unit}$ =	900 lb
$P_{DL\ Curb}$ =	100 lb

	VERTICAL FORCE		OVERTURNING T/C		HORIZ. SHEAR	
LOAD CASE	UPLIFT	DOWNWARD	T_1/C_1	T_2/C_2	V_1	V_2
SEISMIC:	169 lb	169 lb	377 lb	695 lb	634 lb	634 lb
WIND:	399 lb	N/A	156 lb	525 lb	396 lb	726 lb
SNOW:	N/A	671 lb	N/A	N/A	N/A	N/A
DEAD:	N/A	1000 lb	N/A	N/A	N/A	N/A

COMBINATIONS (LRFD)

	OVERTURNING MOMENT	
COMBINATION	OT_1	OT_2
0.9DL-1.0WL	-2060 lb-ft	285 lb-ft
0.9DL-1.0EQ	-509 lb-ft	931 lb-ft
1.2DL+1.6SL	-7957 lb-ft	-4319 lb-ft
1.2DL+1.0WL	-5290 lb-ft	-4275 lb-ft
1.2DL+1.0WL+0.5SL	-6464 lb-ft	-4913 lb-ft
1.2DL+1.0EQ+0.2SL	-7310 lb-ft	-5176 lb-ft

	VERTICAL FORCE	
COMBINATION	UPLIFT	DOWNWARD
$1.0*F_v$	399 lb	
$\pm$ $1.0*F_{pv}$	169 lb	169 lb
$1.0*F_v$	399 lb	
$1.0*F_v$	399 lb	
$\pm$ $1.0*F_{pv}$	169 lb	169 lb

	TENSION		COMPRESSION		HORIZ. SHEAR	
RESULTANT COMBINATION	T_1	T_2	C_1	C_2	V_1	V_2
0.9DL-1.0WL	N/A	275 lb	294 lb	N/A	-396 lb	-726 lb
$(0.9\text{-}0.2S_{DS})$DL-1.0EQ	12 lb	329 lb	157 lb	N/A	-634 lb	-634 lb
1.2DL+1.6SL	N/A	N/A	1137 lb	1137 lb	N/A	N/A
1.2DL+1.0WL	N/A	N/A	756 lb	1125 lb	396 lb	726 lb
1.2DL+1.0WL+0.5SL	N/A	N/A	923 lb	1293 lb	396 lb	726 lb
$(1.2+0.2S_{DS})$DL+1.0EQ+0.2SL	N/A	N/A	1129 lb	1446 lb	634 lb	634 lb

DESIGN LOADS (LRFD) AND ATTACHMENT DESIGN

	TENSION		COMPRESSION		HORIZ. SHEAR	
	T_1	T_2	C_1	C_2	V_1	V_2
FORCE:	**12 lb**	**329 lb**	**1137 lb**	**1446 lb**	**634 lb**	**726 lb**
# OF SCREWS/SIDE:	**1 SCREWS**	**2 SCREWS**			**1 SCREWS**	**1 SCREWS**

SCREW SIZE	CURB/UNIT THICKNESS
#10	16ga

PRCNC20231424

TRASH ENCLOSURE

PRCNC20231424

PCS Structural Solutions

Project: ARCO PUYALLUP

Job No: 23-703

Subject:

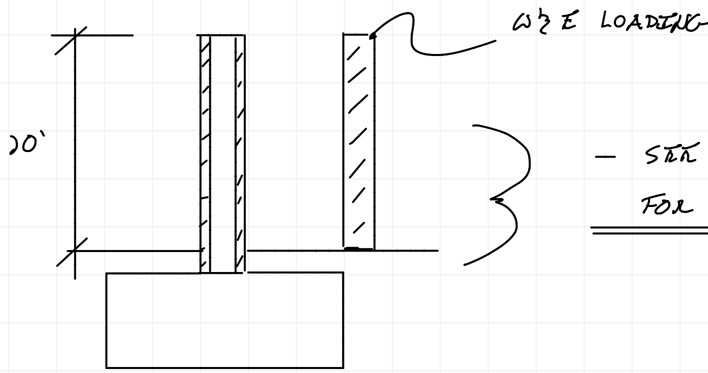
Sheet

Name: SJW

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland

Date:

TRASH ENCLOSURE



– SEE PROVIDE DESIGN SPREADSHEET FOR RESULTS

PRCNC20231424

Description

Trash Enclosure Walls

IBC 2018 / TMS 402-16

Design Criteria

Masonry Strength $f_m := 1900 \cdot psi$

Concrete Strength $f_c := 3000 \cdot psi$

Steel Strength $f_y := 60000 \cdot psi$

Wall Height $h := 10 \cdot ft$

Wall Weight $w := 85 \cdot psf$

Wall Thickness(nominal) $t_n := 8 \cdot in$

Wall Thickness (actual) $t_a = 7.63\ in$

Section Width (wall & footing) $b := 12 \cdot in$

Section Depth (wall) $d_{wall} := 3.75 \cdot in$

Gross Area (masonry) $A_g = 91.5\ in^2$

Modulus of Elasticity

$E_s := 29000000 \cdot psi$ $E_m = 1710000\ psi$ $n := \frac{E_s}{E_m} = 16.96$

Loads

SEISMIC (ASCE 7-16 Section 13.3)

$S_{DS} := .845$ Spectral Acceleration (ASCE 7-16 Section 11.4.5)

$I_p := 1.0$ Component Importance Factor (ASCE 7-16 Section 13.1.3)

$a_p := 2.5$ Component Amplification Factor (ASCE 7-16 Table 13.5-1)

$R_p := 3.0$ Component Response Modification Factor (ASCE 7-16 Table 13.5-1)

$z := 0 \cdot ft$ Height in structure of anchorage (cantilevered from footing)

$W_p := 85 \cdot psf$ Seismic Wall Weight (includes veneer, etc.)

ASCE 7-16 Equation 13.3-1, 13.3-2, 13.3-3:

$$F_p := \left(\frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_p}{\frac{R_p}{I_p}} \right) \cdot \left(1 + 2 \cdot \frac{z}{h} \right) = 23.9\ psf$$

WIND (ASCE 7-16 Section 29.3.1)

$K_d := .85$ Wind Directionality Factor (ASCE 7-16 Section 26.6 - Table 26.6-1)

$K_z := .85$ Velocity Pressure Exposure Coefficient (ASCE 7-16 Section 26.10.1 - Table 26.10-1)

$K_{zt} := 1.0$ Topographic Factor (ASCE 7-16 Section 26.8)

$V_{wind} := 97$ Wind Speed (mph)

ASCE 7-16 Equation 26.10-1:

$$q_z := \left(.00256 \cdot K_d \cdot K_z \cdot K_{zt} \cdot V_{wind}^2 \right) \cdot psf = 17.4\ psf$$

$G := .85$ Gust Effect Factor (ASCE 7-16 Section 26.11)

$C_f := 1.45$ Net Force Coefficient (ASCE 7-16 Fig. 29.3-1)

Shall not be less than: $F'_p := 0.3 \cdot S_{DS} \cdot I_p \cdot W_p$

$F'_p = 21.5\ psf$

ASCE 7-16 Equation 29.3-1:

Shall not exceed: $F''_p := 1.6 \cdot S_{DS} \cdot I_p \cdot W_p$

$F_{wind} := q_z \cdot G \cdot C_f = 21.4\ psf$

$F''_p = 114.9\ psf$

$F_{seismic} = 23.9\ psf$

CONTROLLING

$W_{latULT} = 23.9\ psf$ (Ultimate) $W_{latASD} = 16.8\ psf$ (Allowable)

Footing Design

$B := 4 \cdot ft$ (Footing width) $t_f := 15 \cdot in$ (Footing thickness) $p_{allow} := 1500 \cdot psf$ (Allowable soil bearing pressure)

$W_{ftg} := b \cdot t_f \cdot B \cdot 150 \cdot \frac{lb}{ft^3} = 750\ lb$ $W_{wall} := w \cdot h \cdot b = 850\ lb$ $W_{total} := W_{ftg} + W_{wall}$ $W_{total} = 1600\ lb$

(ASCE 7-16 Section 2.4.1 & 12.14.3.2): $W_{totalASD} := 0.6 \cdot W_{total} = 960\ lb$

$W_{total5} := \left(1.0 + 0.14 \cdot S_{DS}\right) \cdot W_{total} = 1789.28\ lb$ $W_{total8} := \left(0.6 - 0.14 \cdot S_{DS}\right) \cdot W_{total} = 770.72\ lb$

$M_{res8} := W_{total8} \cdot \frac{B}{2} = 1541\ lb \cdot ft$ $M_{ovtASD} := W_{latASD} \cdot b \cdot h \cdot \left(\left(\frac{h}{2}\right) + t_f\right) = 1047\ lb \cdot ft$

Overturning

$M_{res8} = 1541\ lb \cdot ft$

$M_{ovtASD} = 1047\ lb \cdot ft$

$Overturning_{check} =$ "Overturning okay"

Sliding (ASCE 7-16 Section 12.14.3.1 Exception 2)

$u := 0.35$ Friction Coefficient (Including FOS)

$FOS := 1.5$

$F_{slidingASD} := W_{latASD} \cdot b \cdot h = 168\ lb$

$F_{slidingASD} = 167.59\ lb$

$F_{res} := 0.6 \cdot W_{total} \cdot u \cdot FOS = 504\ lb$

$Sliding_{check} =$ "Sliding okay"

Soil Bearing (ASCE 7-16 Section 12.14.3.1 Exception 2)

$e := \frac{M_{ovtASD}}{W_{totalASD}} = 1.09\ ft$

$resultant =$ "In Outer Third"

$p_{soil.max} = 704\ psf$

$Bearing_{check} =$ "Bearing okay"

$Stability_{check} =$ "Stability okay"

Footing Rebar (ACI 318-14 Section 22.3)

Try #6 at 12" o.c (Transverse rebar)

$A_{s.footing} := 0.44 \cdot in^2$ (Area of steel in section width "b")

$d_{footing} := 11 \cdot in$ (Adjust based on footing thickness)

$$a_{footing} := \frac{A_{s.footing} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.86\ in$$

$$\phi M_{footing.nominal} := .9 \cdot A_{s.footing} \cdot f_y \cdot \left(d_{footing} - \frac{a_{footing}}{2}\right) = 20926\ lb \cdot ft$$

$M_{footingULT} = 1844\ lb \cdot ft$

$A_{s.footing.min} := 0.0018 \cdot t_f \cdot b = 0.32\ in^2$

$Footing_{check} =$ "Footing okay"

$Rebar_{check} =$ "Rebar meets minimum reinforcement requirement"

Footing Shear (ACI 318-14 Section 22.5.5)

$$\phi V_{footing.nominal} := 0.75 \cdot 2 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot b \cdot d_{footing} = 10845\ lb$$

$V_{footingULT} = 1656\ lb$

$Shear_{check} =$ "Shear okay"

Wall Design (TMS 402-16 Section 9.3.5.2)

Try #5 at 24" o.c

$A_{s.wall} := 0.155 \cdot in^2$ (Area of steel in section width "b")

$$a_{wall} := \frac{A_{s.wall} \cdot f_y}{0.8 \cdot f_m \cdot b} = 0.51\ in$$

$$\phi M_{wall.nominal} := .9 \cdot A_{s.wall} \cdot f_y \cdot \left(d_{wall} - \frac{a_{wall}}{2}\right) = 2438\ lb \cdot ft$$

$$M_{wallULT} := W_{latULT} \cdot b \cdot h \cdot \left(\frac{h}{2}\right) = 1197\ lb \cdot ft$$

$Wall_{check} =$ "Wall okay"

PRCNC20231424

LOT LIGHT FOUNDATION

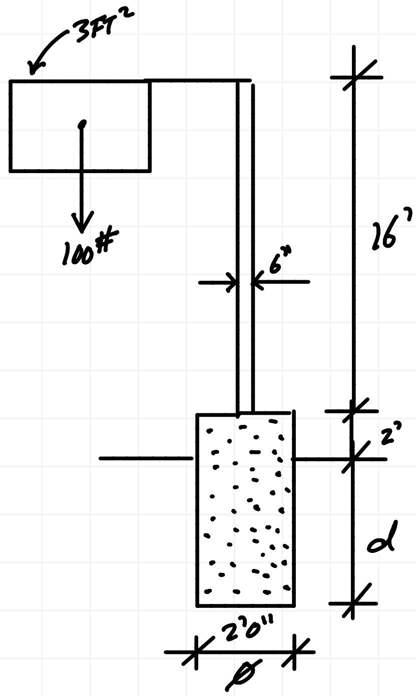
PCS Structural Solutions

Project: ARCO PUYALLUP Job No: 23-703

Subject: Sheet Name: SJW

Originating Office: ☐ Seattle ☐ Tacoma ☐ Portland Date:

LOT LIGHT FOUNDATION



WIND

$F = q_h\, G\, C_f\, A_s$ [7-16 / 29.3-1]

$q_h = 0.00256\, K_z K_{zt} K_d V^2$

$K_z = 0.90$
$K_{zt} = 1.0$
$K_d = 0.85$
$V = 97$ MPH

$q_h = 18.4$ PSF ; $G = 0.85$; $C_f = 1.8$

$F = 28.2$ PSF $* A_s$

$F = 28.2$ PSF$[3FT^2 + 0.5FT(16FT) + 2FT(2FT)]$
$= 423\#$

$M_D + 0.6W$
$= 100\#(2FT) + (28.2\text{ PSF})[3FT^2(17.5FT) + 0.5FT(16FT)(10FT) + 2FT(2FT)(1FT)]$
$= 200\#\text{-FT} + 17\text{PSF}[136.5FT^2] = 2{,}510\#\text{-FT}$

$h_{eff} = \dfrac{M}{F} = \dfrac{2{,}510\ \#\text{-FT}}{0.6(423\#)} = 9.9$ FT

FOOTING DEPTH

$S_1 = \sigma_{PASSIVE}(d/3) = 300\text{ PCF}(5FT/3) = 500$ PSF

$A = \dfrac{2.34P}{S_1 b} = \dfrac{2.34(423\#)}{500\text{ PSF}(2FT)(4/3)} = 0.74$ FT

$d = 0.5A[1 + (1 + \dfrac{4.36h}{A})^{1/2}]$

$d = 3'6"$ FT MIN.

PRCNC20231424

EV CHARGING STATION

PRCNC20231424

PCS Structural Solutions

Project: ARCO PUYALLUP

Job Number = 23-703
Date = 25-Sep-23
Name = SJW

Unit = EV CHARGE

Unit Information

Weight, W_p = 3800 lb
Height to Unit = 0.0 ft
Unit Width(w) = 3.3 ft
Unit Height (h) = 8.0 ft
Unit Length (ℓ) = 3.6 ft
Center of Gravity (Vertical) = 5.3 ft 2/3(Unit Height Assumed)

Curb Information

Is there a curb? NO
Weight, W_{p-2} = 0 lb
Curb Width at Base (w) = 0.0 ft
Curb Height (h) = 0.0 ft
Curb Length at Base (ℓ) = 0.0 ft

Total Weight, W_{tot} 3800 lb
h_{wind} 4.0 ft 1/2(Unit height + Curb Height)
$h_{seismic}$ 5.3 ft Center of Gravity + Curb Height
Design Width, w_{des} 3.3 ft
Design Length, $ℓ_{des}$ 3.6 ft

Building Information

Are building dimensions known? YES
Building Width (B) = 100.0 ft
Building Length (L) = 100.0 ft

Design Criteria = ASCE 7-16, IBC 2018

Gravity

Dead = 0 psf
Roof Snow = 0 psf
$P_{Roof\ DL}$ 0 lb
$P_{Roof\ SL}$ 0 lb

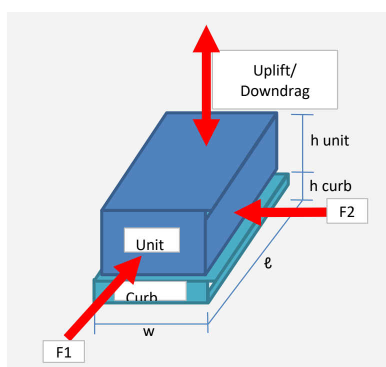
Lateral

The new unit does not have a significant impact to the building's capacity to resist wind or seismic forces

Seismic - ASCE 7-16 Chapter 13

S_{DS} =	0.85	
a_p =	1	Tbl. 13.6-1
I_p =	1	§ 13.1.3
R_p =	1.5	Tbl. 13.6-1
Height of Attachment or Bot of Curb (z) =	0.0 ft	
Total Building Height (H) =	0.1 ft	
$F_p = 0.4*S_{DS}*(I_p/R_p)*a_p*W_p*[1+2*z/H]$ =	861 lb	Eqn. 13.3-1
$F_{p\ max} = 1.6*S_{DS}*I_p*W_p$ =	5168 lb	Eqn. 13.3-2
$F_{p\ min} = 0.3*S_{DS}*I_p*W_p$ =	969 lb	Eqn. 13.3-3

Controlling F_p = **969 lb**
Vertical F_p = **646 lb**



Wind - ASCE 7-16 Chapter 29

Wind Lateral:

Wind Speed	97 mph
Risk Category	II
Wind Exposure	B
K_{zt}, Topographical Effect	1.00
K_e, Ground Elevation Factor	1.00

K_h (Table 29.3-1) 0.57

	F_1	F_2	
A_f =	26.7 sq ft	28.7 sq ft	
A_r =	11.9 sq ft		
$q_h = .00256*K_d*K_h*K_{zt}*K_e*V^2$ =	11.7 psf		Eqn. 26.10-1

Building B*h = 10 sq ft
Building B*L = 10000 sq ft

$(G*C_r)_1$	$(G*C_r)_2$	
1.00	1.00	for horizontal force - See § 29.4.1 - Worst case used for B*h or L*h
1.50		for vertical force - See § 29.4.1 - Maximum dimensions used for A_r

	F_1	F_2	
$F_h = q_h*(G*C_r)*A_f$ =	**311 lb**	**335 lb**	Eqn. 29.4-2
$F_v = q_h*(G*C_r)*A_r$ =	**209 lb**		Eqn. 29.4-3

Moments (Unfactored)

PRCNC20231424

PCS Structural Solutions

Job Number = 23-703
Date = 25-Sep-23
Name = SJW

Project: ARCO PUYALLUP

Unit = EV CHARGE

	F_1	F_2
$M_{OT\ Seismic}$ =	5168 lb-ft	5168 lb-ft
$M_{OT\ Wind}$ =	1245 lb-ft	1338 lb-ft
$M_{R\ SL}$ =	0 lb-ft	0 lb-ft
$M_{R\ DL\ Unit}$ =	6808 lb-ft	6333 lb-ft
$M_{R\ DL\ Curb}$ =	0 lb-ft	0 lb-ft

Combinations (ASD):

	Overturning		Vertical Force		Tension		Compression		Shear	
Combination	OT_1	OT_2	Uplift	Downdrag	T_1	T_2	C_1	C_2	V_1	V_2
0.6DL-0.6WL	-3338 lb-ft	-2997 lb-ft	125 lb	0 lb	0 lb	0 lb	932 lb	899 lb	-187 lb	-201 lb
0.6DL-0.7EQ	-467 lb-ft	-182 lb-ft	452 lb	452 lb	96 lb	171 lb	357 lb	281 lb	-678 lb	-678 lb
DL+SL	-6808 lb-ft	-6333 lb-ft	0 lb	0 lb	0 lb	0 lb	1900 lb	1900 lb	0 lb	0 lb
DL+0.6WL	-7555 lb-ft	-7136 lb-ft	125 lb	0 lb	0 lb	0 lb	2108 lb	2141 lb	187 lb	201 lb
DL+0.45WL+0.75SL	-7369 lb-ft	-6936 lb-ft	94 lb	0 lb	0 lb	0 lb	2056 lb	2081 lb	140 lb	151 lb
DL+0.7EQ	-10426 lb-ft	-9951 lb-ft	452 lb	452 lb	0 lb	0 lb	3136 lb	3211 lb	678 lb	678 lb
DL+0.525EQ+0.75SL	-9522 lb-ft	-9047 lb-ft	339 lb	339 lb	0 lb	0 lb	2827 lb	2884 lb	509 lb	509 lb

Design (ASD):

Tension		Compression		Shear	
T_1	T_2	C_1	C_2	V_1	V_2
96 lb	171 lb	3136 lb	3211 lb	678 lb	678 lb

Anchorage Design

$\Omega_0 := 1.5$ $\qquad S_{DS} := 0.85$

By inspection, Seismic will control, use Load Combo:

$0.9\ D - Ev + Emh$

From the Spreadsheet in the previous page :

$H := 969\ lbf \cdot \Omega_0 = \left(1.454 \cdot 10^{3}\right)\ lbf$

$V := 646\ lbf \cdot \Omega_0 = 969\ lbf$

$M_E := H \cdot 5.3\ ft \cdot \Omega_0 = \left(1.387 \cdot 10^{5}\right)\ lbf \cdot in$

$D := 3800\ lbf$

Vertical Load

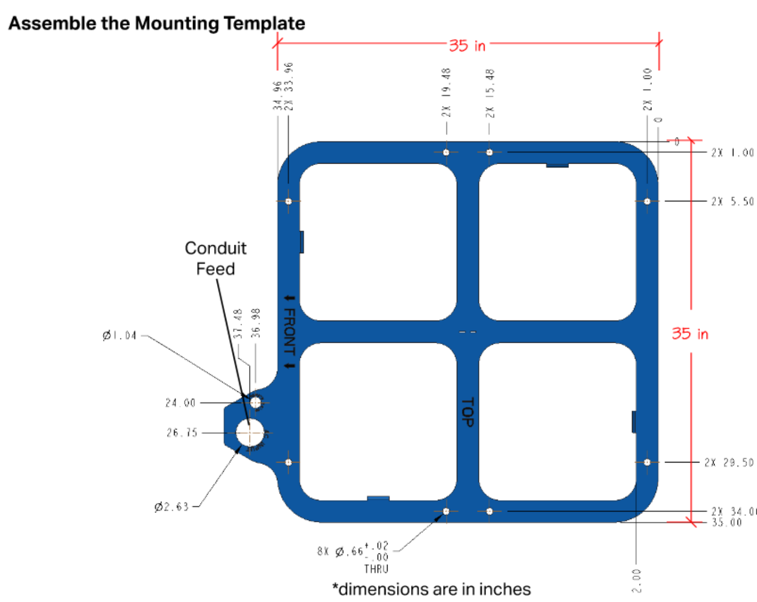
$0.9\ D - V = \left(2.451 \cdot 10^{3}\right)\ lbf$

Base Shear

$H = \left(1.454 \cdot 10^{3}\right)\ lbf$

Base Moment

$M_E = \left(1.387 \cdot 10^{5}\right)\ in \cdot lbf$



PCS Structural Solutions

Project: ARCO AMPM PUYALLUP
Subject: EV Charging Station
Originating Office: Portland

Job Number: 23703
Name: SJW
Date: 09/25/23

PRCNC20231424

Footing Design

$B := 4 \cdot ft$ (Footing width)
$L := 4\ ft$ (Footing length)

$t_f := 24 \cdot in$ (Footing thickness)

$p_{allow} := 1500 \cdot psf$ (Allowable soil bearing pressure, assumed)

$W_{ftg} := L \cdot t_f \cdot B \cdot 150 \cdot \dfrac{lbf}{ft^3} = \left(4.8 \cdot 10^3\right)\ lbf$

$W_{equipment} := D = \left(3.8 \cdot 10^3\right)\ lbf$

$W_{total} := W_{ftg} + W_{equipment}$

$W_{total} = \left(8.6 \cdot 10^3\right)\ lbf$

(ASCE 7-16 Section 2.4.1 & 12.14.3.2):

$W_{totalASD} := 0.6 \cdot W_{total} = \left(5.16 \cdot 10^3\right)\ lbf$

$W_{total5} := \left(1.0 + 0.14 \cdot S_{DS}\right) \cdot W_{total}$

$W_{total8} := \left(0.6 - 0.14 \cdot S_{DS}\right) \cdot W_{total} = \left(4.137 \cdot 10^3\right)\ lbf$

$M_{res8} := W_{total8} \cdot \dfrac{B}{2} = 8273\ lbf \cdot ft$

$M_{ovtASD} := \dfrac{M_E \cdot 0.7}{\Omega_0} = 5392\ lbf \cdot ft$

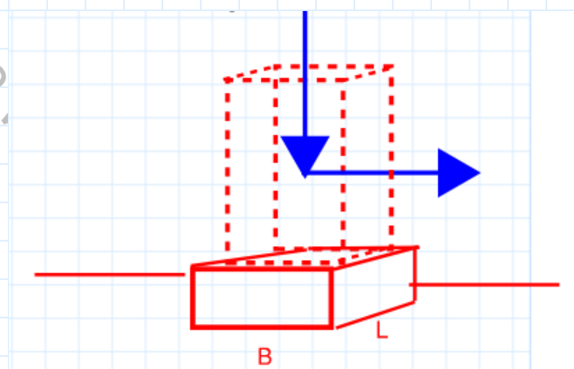
Soil Bearing

$e := \dfrac{M_{ovtASD}}{W_{totalASD}} = 1.045\ ft$

$resultant =$ "In Outer Third"

$p_{soil.max} = 901\ psf$

$Bearing_{check} =$ "Bearing okay"



Sliding

(ASCE 7-16 Section 12.14.3.1 Exception 2)

$u := 0.35$ Friction Coefficient (Including FOS)

$FOS := 1.5$

$F_{slidingASD} := H = 1454\ lbf$

$F_{res} := 0.6 \cdot W_{total} \cdot u \cdot FOS = 2709\ lbf$

$Sliding_{check} =$ "Sliding okay"

Overturning

$M_{res8} = 8273\ lbf \cdot ft$
$M_{ovtASD} = 5392\ lbf \cdot ft$

$Overturning_{check} =$ "Overturning okay"

$Stability_{check} =$ "Stability okay"

Reinforcement

By inspection, flexure strength of footing will not control, check temp&shrink only

(7) #5 bar, EACH WAY

$N_{bar} := 7$ $As := 0.31\ in^2$

$Rebar_{check} =$ "Rebar okay"

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 25, 2023	Date:	6/8/2023
Fastening point:			

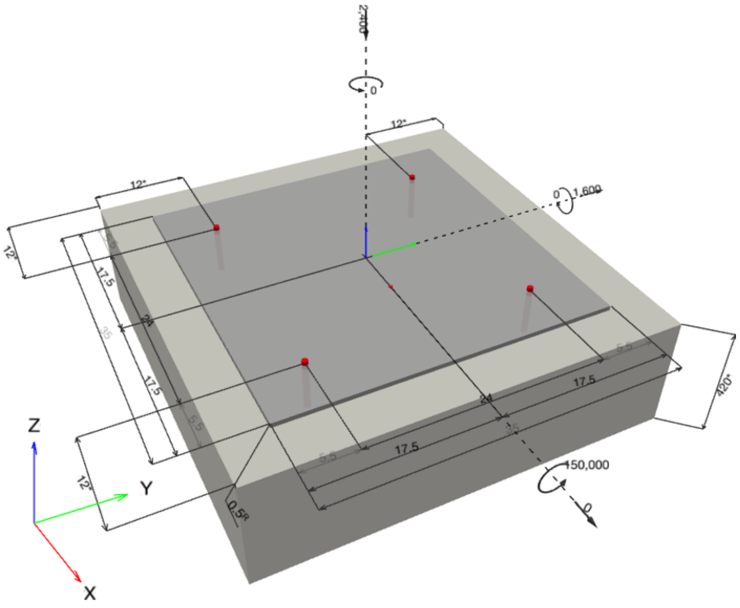
Specifier's comments:

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 55 5/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 4.724 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-14 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 35.000 in. x 35.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 420.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.2.3.4.3 (d))
	Shear load: yes (17.2.3.5.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



Hilti PROFIS Engineering 3.0.85

www.hilti.com

Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 25, 2023	Date:	6/8/2023
Fastening point:			

1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = -2{,}400$; $V_x = 0$; $V_y = 1{,}600$; $M_x = 150{,}000$; $M_y = 0$; $M_z = 0$;	yes	36

PRCNC20231424

2 Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb] Load	Capacity	Utilization β_N / β_V [%]	Status
Tension	Concrete Breakout Failure	4,181	11,889	36 / -	OK
Shear	Concrete edge failure in direction y+	1,600	15,861	- / 11	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.352	0.101	5/3	20	OK

3 Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 25, 2023	Date:	6/8/2023
Fastening point:			

4 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

PRCNC20231424

RETAINING WALL IS A
SEPARATE PERMIT
PRRWF2023331581

RETAINING WALL

PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:

Cantilevered Retaining Wall

Project File: Retaining Wall.ec6

LIC# : KW-06014122, Build:20.23.05.25 PCS STRUCTURAL SOLUTIONS

Code References

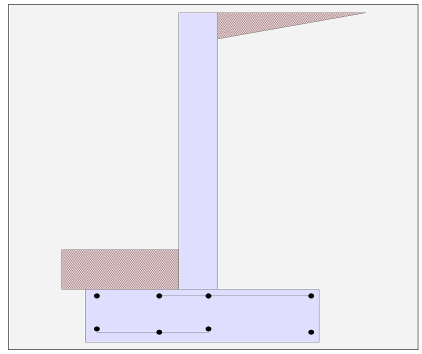
Calculations per IBC 2018 1807.3, CBC 2019, ASCE 7-16

Criteria

Retained Height	=	7.00 ft
Wall height above soil	=	0.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	12.00 in
Water table above bottom of footing	=	0.0 ft

Soil Data

Allow Soil Bearing	=	1,500.0 psf
Equivalent Fluid Pressure Method		
Active Heel Pressure	=	35.0 psf/ft
	=	
Passive Pressure	=	300.0 psf/ft
Soil Density, Heel	=	110.00 pcf
Soil Density, Toe	=	110.00 pcf
Footing||Soil Friction	=	0.350
Soil height to ignore for passive pressure	=	12.00 in



Surcharge Loads

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

Axial Load Applied to Stem

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

Lateral Load Applied to Stem

Lateral Load	=	30.0 #/ft
...Height to Top	=	7.00 ft
...Height to Bottom	=	0.10 ft
Load Type	=	Live Load (L) (Service Level)
Wind on Exposed Stem (Strength Level)	=	0.0 psf

Adjacent Footing Load

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

PRCNC20231424

Cantilevered Retaining Wall

Project File: Retaining Wall.ec6

Design Summary

Wall Stability Ratios

Overturning	=	2.59 OK
Sliding	=	~~1.39 Ratio < 1.5!~~
		- Passive & friction include F.S. > 1.5 - OK!
Global Stability	=	2.45
Total Bearing Load	=	3,763 lbs
...resultant ecc.	=	7.74 in

Eccentricity within middle third

Soil Pressure @ Toe	=	1,335 psf OK
Soil Pressure @ Heel	=	170 psf OK
Allowable	=	1,500 psf

Soil Pressure Less Than Allowable

ACI Factored @ Toe	=	1,869 psf
ACI Factored @ Heel	=	239 psf
Footing Shear @ Toe	=	8.3 psi OK
Footing Shear @ Heel	=	7.7 psi OK
Allowable	=	82.2 psi

Sliding Calcs

Lateral Sliding Force	=	1,422.3 lbs
less 100% Passive Force	-	666.7 lbs
less 100% Friction Force	= -	1,317.2 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 Stability	=	~~149.6 lbs NG~~

- Passive & friction include F.S. > 1.5 - OK!

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing

Load Factors

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

Stem Construction

		Bottom		
		Stem OK		
Design Height Above Ftg	ft =	0.00		
Wall Material Above "Ht"	=	Concrete		
Design Method	=	SD	SD	SD
Thickness	=	10.00		
Rebar Size	=	# 5		
Rebar Spacing	=	12.00		
Rebar Placed at	=	7.6875		
Design Data				
fb/FB + fa/Fa	=	**0.425**		
Total Force @ Section				
Service Level	lbs =			
Strength Level	lbs =	1,703.2		
Moment....Actual				
Service Level	ft-# =			
Strength Level	ft-# =	4,377.1		
Moment.....Allowable	=	10,298.8		
Shear.....Actual				
Service Level	psi =			
Strength Level	psi =	18.5		
Shear.....Allowable	psi =	82.2		
Anet (Masonry)	in2 =			
Wall Weight	psf =	125.0		
Rebar Depth 'd'	in =	7.69		
Masonry Data				
f'm	psi =			
Fs	psi =			
Solid Grouting	=			
Modular Ratio 'n'	=			
Equiv. Solid Thick.	=			
Masonry Block Type	=			
Masonry Design Method	=	ASD		
Concrete Data				
f'c	psi =	3,000.0		
Fy	psi =	60,000.0		

PRCNC20231424

Cantilevered Retaining Wall

Concrete Stem Rebar Area Details

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing	
As (based on applied moment) :	0.132 in2/ft		
(4/3) * As :	0.176 in2/ft	Min Stem T&S Reinf Area 1.680 in2	
200bd/fy : 200(12)(7.6875)/60000 :	0.3075 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.240 in2/ft	
0.0018bh : 0.0018(12)(10) :	0.216 in2/ft	Horizontal Reinforcing Options :	
	============	One layer of :	Two layers of :
Required Area :	0.216 in2/ft	#4@ 10.00 in	#4@ 20.00 in
Provided Area :	0.31 in2/ft	#5@ 15.50 in	#5@ 31.00 in
Maximum Area :	1.2497 in2/ft	#6@ 22.00 in	#6@ 44.00 in

Footing Data

Toe Width	=	2.00 ft
Heel Width	=	3.00
Total Footing Width	=	5.00
Footing Thickness	=	16.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 3,000 psi	Fy =	60,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top 2.00	@ Btm.=	3.00 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	1,869	239 psf	
Mu' : Upward	=	3,303	1,113 ft-#	
Mu' : Downward	=	744	2,732 ft-#	
Mu: Design	=	2,559 OK	1,619 ft-#	OK
phiMn	=	24,143	26,123 ft-#	
Actual 1-Way Shear	=	8.28	7.65 psi	
Allow 1-Way Shear	=	82.16	82.16 psi	
Toe Reinforcing	=	# 6 @ 12.00 in		
Heel Reinforcing	=	# 6 @ 12.00 in		
Key Reinforcing	=	None Spec'd		
Footing Torsion, Tu	=		0.00 ft-lbs	
Footing Allow. Torsion, phi Tu	=		0.00 ft-lbs	

If torsion exceeds allowable, provide supplemental design for footing torsion.

Other Acceptable Sizes & Spacings

Toe: #4@ 6.94 in, #5@ 10.76 in, #6@ 15.27 in, #7@ 20.83 in, #8@ 27.43 in, #9@ 34.72 in, #10@ 44.09 in

Heel: #4@ 6.94 in, #5@ 10.76 in, #6@ 15.27 in, #7@ 20.83 in, #8@ 27.43 in, #9@ 34.72 in, #10@ 44.09 in

Key: No key defined

Min footing T&S reinf Area	1.73	in2
Min footing T&S reinf Area per foot	0.35	in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 6.94 in	#4@ 13.89 in
#5@ 10.76 in	#5@ 21.53 in
#6@ 15.28 in	#6@ 30.56 in

Cantilevered Retaining Wall

Summary of Overturning & Resisting Forces & Moments

Item		.....OVERTURNING..... Force lbs	Distance ft	Moment ft-#
HL Act Pres (ab water tbl)		1,215.3	2.78	3,375.8
HL Act Pres (be water tbl)				
Hydrostatic Force				
Buoyant Force	=			
Surcharge over Heel	=			
Surcharge Over Toe	=			
Adjacent Footing Load	=			
Added Lateral Load	=	207.0	4.88	1,010.9
Load @ Stem Above Soil	=			
	=			
Total	=	1,422.3	**O.T.M.** =	4,386.6

Resisting/Overturning Ratio = **2.59**

Vertical Loads used for Soil Pressure = 3,763.3 lbs

Item		.....RESISTING..... Force lbs	Distance ft	Moment ft-#
Soil Over HL (ab. water tbl)		1,668.3	3.92	6,534.3
Soil Over HL (bel. water tbl)			3.92	6,534.3
Water Table				
Sloped Soil Over Heel	=			
Surcharge Over Heel	=			
Adjacent Footing Load	=			
Axial Dead Load on Stem	=			
* Axial Live Load on Stem	=			
Soil Over Toe	=	220.0	1.00	220.0
Surcharge Over Toe	=			
Stem Weight(s)	=	875.0	2.42	2,114.6
Earth @ Stem Transitions	=			
Footing Weight	=	1,000.0	2.50	2,500.0
Key Weight	=			
Vert. Component	=			
Total =		3,763.3 lbs	**R.M.=**	11,368.9

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

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Sliding Resistance.

Vertical component of active lateral soil pressure IS NOT considered in the calculation of Overturning Resistance.

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus	250.0	pci
Horizontal Defl @ Top of Wall (approximate only)	0.052	in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.

Rebar Lap & Embedment Lengths Information

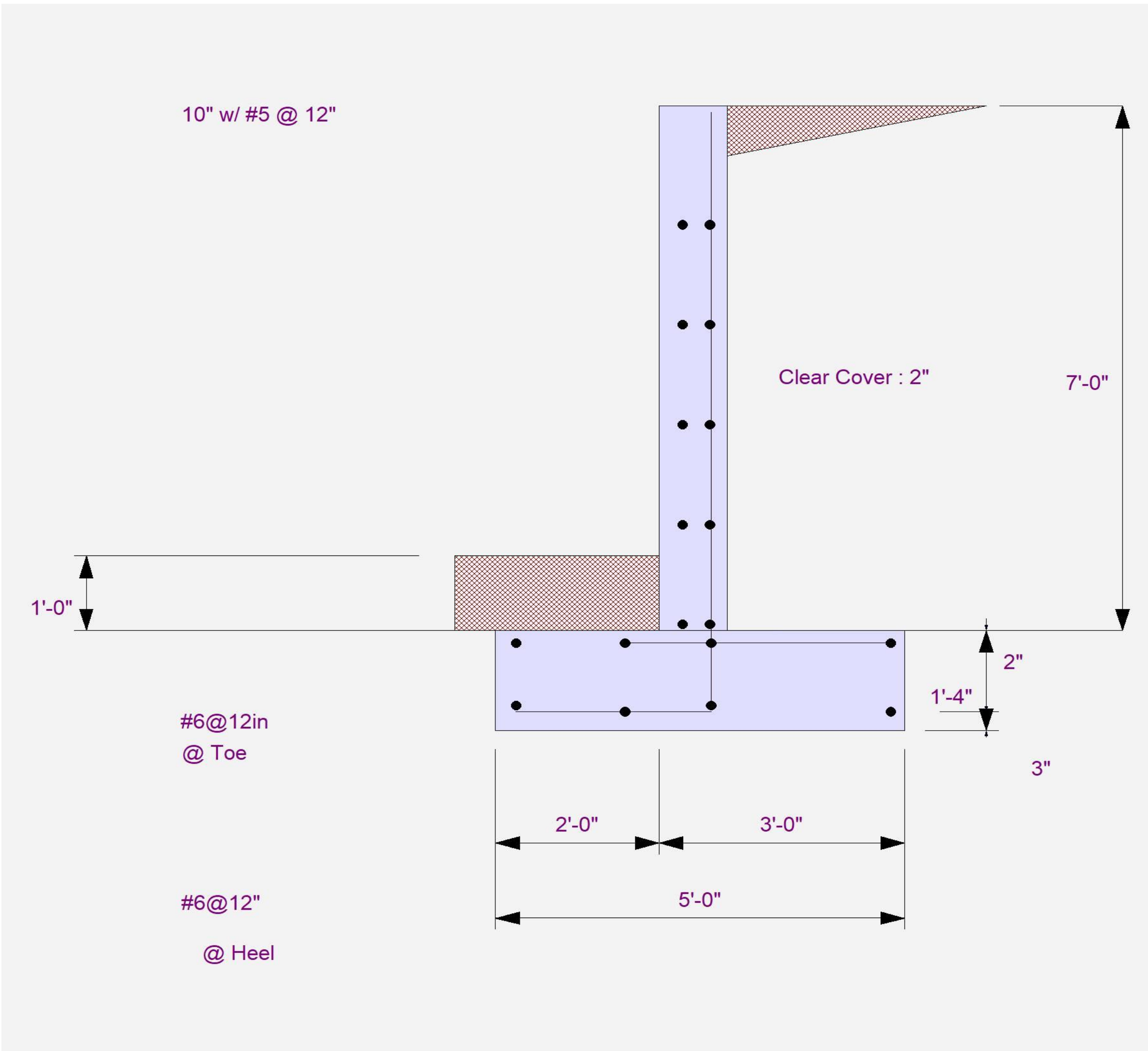
Stem Design Segment: Bottom

Stem Design Height: 0.00 ft above top of footing

Lap Splice length for #5 bar specified in this stem design segment (25.4.2.3a) =	21.36 in
Development length for #5 bar specified in this stem design segment =	16.43 in
Hooked embedment length into footing for #5 bar specified in this stem design segment =	9.59 in
As Provided =	0.3100 in2/ft
As Required =	0.2160 in2/ft

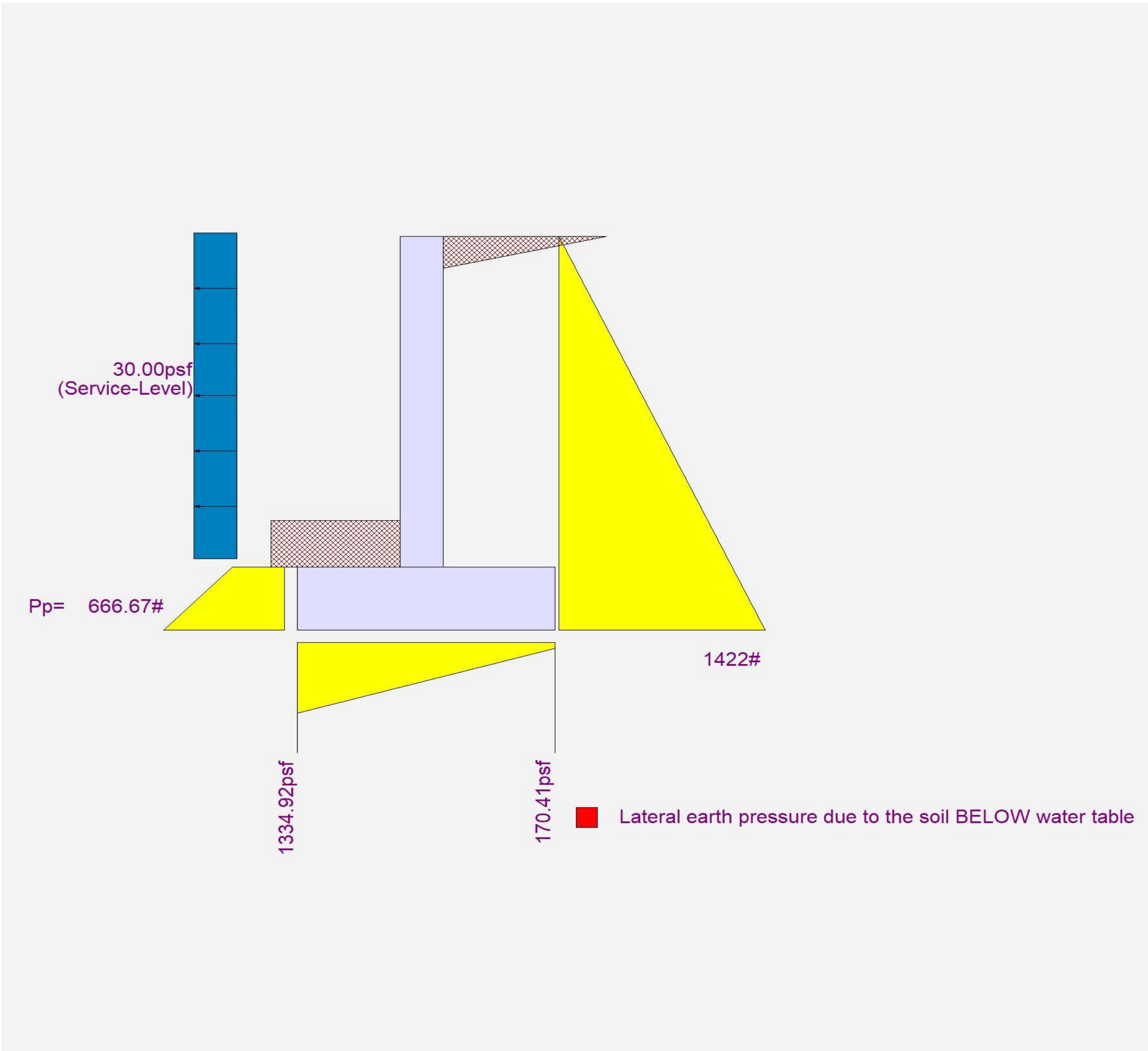
PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:



PRCNC20231424

Project Title:
Engineer:
Project ID:
Project Descr:



PRCNC20231424

22000 GALLON TANK

Underground Storage Tank Floatout and Anchorage Design

Description: Floatout and anchorage calculations for a 22,000 gallon below grade fuel tank and anchorage to a concrete slab.

Design Criteria

Design per PEI RP100 - Recommended Practices for Installation of Underground Liquid Storage Systems

Member Properties

Tank Diameter:	$D := 10.5\ ft$
Tank Length:	$L := 41.25\ ft$
Tank Weight:	$W_{tank} := 12873\ lbf$
Reflected Tank Area:	$A := D \cdot L = 433\ ft^2$
Tank Volume:	$V := 22000\ gal$
Tank Displacement:	$V = 2941\ ft^3$
Concrete Pad Overhang:	$b := 2\ ft$
Concrete Pad Thickness:	$t_c := 12\ in$
Buried Depth (Top of Tank to Bottom of Paving):	$d := 5\ ft$

Material Properties

Concrete Submerged Unit Weight:	$\gamma_{cs} := 150\ pcf - 62.4\ pcf = 87.6\ pcf$
Backfill Submerged Unit Weight:	$\gamma_{bs} := 120\ pcf - 62.4\ pcf = 57.6\ pcf$
Concrete Strength:	$f'_c := 4000\ psi$

Buoyancy

Concrete Pad Width:	$B_{conc} := (D + 2 \cdot b) = 14.5\ ft$
Concrete Pad Area:	$A_{conc} := (D + 2 \cdot b) \cdot (L + 2 \cdot b) = 656.125\ ft^2$
Concrete Pad Volume:	$V_{conc} := A_{conc} \cdot t_c = 656.125\ ft^3$
Concrete Pad Weight:	$W_{conc} := V_{conc} \cdot \gamma_{cs} = 57\ kip$
Backfill Volume:	$V_{fill} := \dfrac{\left(\dfrac{D}{2} + d\right)}{3} \cdot \left(A_{conc} + A + \sqrt{A_{conc} \cdot A}\right) - \left(\dfrac{V}{2}\right) = 4073\ ft^3$
Backfill Weight:	$W_{fill} := V_{fill} \cdot \gamma_{bs} = 235\ kip$

Buoyant Force: $F_{buoyant} := V \cdot 62.4\ pcf = 184\ kip$

Resisting Force: $F_{resisting} := W_{conc} + W_{fill} + W_{tank} = 305\ kip$

Factor of Safety: $FS := \dfrac{F_{resisting}}{F_{buoyant}} = 1.66$

Slab Design

Anchorage Spacing: $s := 6.83\ ft = 6.83\ ft$

Anchorage Force: $R_u := 1.6\ \dfrac{F_{buoyant}}{6} = 48.9\ kip$

Slab Uniform Load: $w_u := 1.6\ \dfrac{R_u}{D} = 7.5\ klf$

Slab Max Moment: $M_u := \dfrac{w_u \cdot D^2}{8} = 103\ kip \cdot ft$

Slab Rebar: $A_s := 0.31 \cdot in^2 \cdot \dfrac{B_{conc}}{12\ in} = 4.495\ in^2$

$$a := \frac{A_s \cdot 60\ ksi}{0.85 \cdot f'_c \cdot B_{conc}} = 0.456\ in$$

$$d := t_c - 3\ in - 0.75\ \frac{in}{2} = 8.625\ in$$

Slab Capacity: $\phi M_n := 0.9 \cdot A_s \cdot 60\ ksi \cdot \left(d - \dfrac{a}{2}\right) = 169.9\ kip \cdot ft$

Slab DCR: $\dfrac{M_u}{\phi M_n} = 60.5\%$

PRCNC20231424

25000 GALLON TANK

Underground Storage Tank Floatout and Anchorage Design

Description: Floatout and anchorage calculations for a 25,000 gallon below grade fuel tank and anchorage to a concrete slab.

Design Criteria

Design per PEI RP100 - Recommended Practices for Installation of Underground Liquid Storage Systems

Member Properties

Tank Diameter:	$D := 10.5\ ft$
Tank Length:	$L := 46\ ft$
Tank Weight:	$W_{tank} := 12250\ lbf$
Reflected Tank Area:	$A := D \cdot L = 483\ ft^2$
Tank Volume:	$V := 25000\ gal$
Tank Displacement:	$V = 3342\ ft^3$
Concrete Pad Overhang:	$b := 2\ ft$
Concrete Pad Thickness:	$t_c := 12\ in$
Buried Depth (Top of Tank to Bottom of Paving):	$d := 5\ ft$

Material Properties

Concrete Submerged Unit Weight:	$\gamma_{cs} := 150\ pcf - 62.4\ pcf = 87.6\ pcf$
Backfill Submerged Unit Weight:	$\gamma_{bs} := 120\ pcf - 62.4\ pcf = 57.6\ pcf$
Concrete Strength:	$f'_c := 4000\ psi$

Buoyancy

Concrete Pad Width:	$B_{conc} := (D + 2 \cdot b) = 14.5\ ft$
Concrete Pad Area:	$A_{conc} := (D + 2 \cdot b) \cdot (L + 2 \cdot b) = 725\ ft^2$
Concrete Pad Volume:	$V_{conc} := A_{conc} \cdot t_c = 725\ ft^3$
Concrete Pad Weight:	$W_{conc} := V_{conc} \cdot \gamma_{cs} = 64\ kip$
Backfill Volume:	$V_{fill} := \dfrac{\left(\dfrac{D}{2} + d\right)}{3} \cdot \left(A_{conc} + A + \sqrt{A_{conc} \cdot A}\right) - \left(\dfrac{V}{2}\right) = 4478\ ft^3$
Backfill Weight:	$W_{fill} := V_{fill} \cdot \gamma_{bs} = 258\ kip$

Buoyant Force: $F_{buoyant} := V \cdot 62.4\ pcf = 209\ kip$

Resisting Force: $F_{resisting} := W_{conc} + W_{fill} + W_{tank} = 334\ kip$

Factor of Safety: $FS := \dfrac{F_{resisting}}{F_{buoyant}} = 1.6$

Slab Design

Anchorage Spacing: $s := 5.5\ ft = 5.5\ ft$

Anchorage Force: $R_u := 1.6\ \dfrac{F_{buoyant}}{8} = 41.7\ kip$

Slab Uniform Load: $w_u := 1.6\ \dfrac{R_u}{D} = 6.4\ klf$

Slab Max Moment: $M_u := \dfrac{w_u \cdot D^2}{8} = 88\ kip \cdot ft$

Slab Rebar: $A_s := 0.31 \cdot in^2 \cdot \dfrac{B_{conc}}{12\ in} = 4.495\ in^2$

$$a := \frac{A_s \cdot 60\ ksi}{0.85 \cdot f'_c \cdot B_{conc}} = 0.456\ in$$

$$d := t_c - 3\ in - 0.75\ \frac{in}{2} = 8.625\ in$$

Slab Capacity: $\phi M_n := 0.9 \cdot A_s \cdot 60\ ksi \cdot \left(d - \dfrac{a}{2}\right) = 169.9\ kip \cdot ft$

Slab DCR: $\dfrac{M_u}{\phi M_n} = 51.6\%$