		Sheet	1
		Date	9/26/2023
		Job No.	230813
FOR	Perry Builders	Eng	CF
DESCRIPTION	49' x 94' 6-Col Canopy		

STRUCTURAL CALCULATIONS

FOR

Perry Builders
49' x 94' 6-Col Canopy

LOCATED AT

1402 S. Meridian
Puyallup, WA

REVISION 0
Dated 9/26/2023
Original Calculations

CALCULATIONS PREPARED BY

BHB Consulting Engineers
2766 S. Main
Salt Lake City, UT 84115

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

DARREN D. TRUCHOT
STATE OF WASHINGTON
40683
REGISTERED
PROFESSIONAL ENGINEER
10-04-2023

PRCNC20240010

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			Date	9/26/2023
			Job No.	230813
	FOR	Perry Builders	Eng	CF
	DESCRIPTION	49' x 94' 6-Col Canopy		

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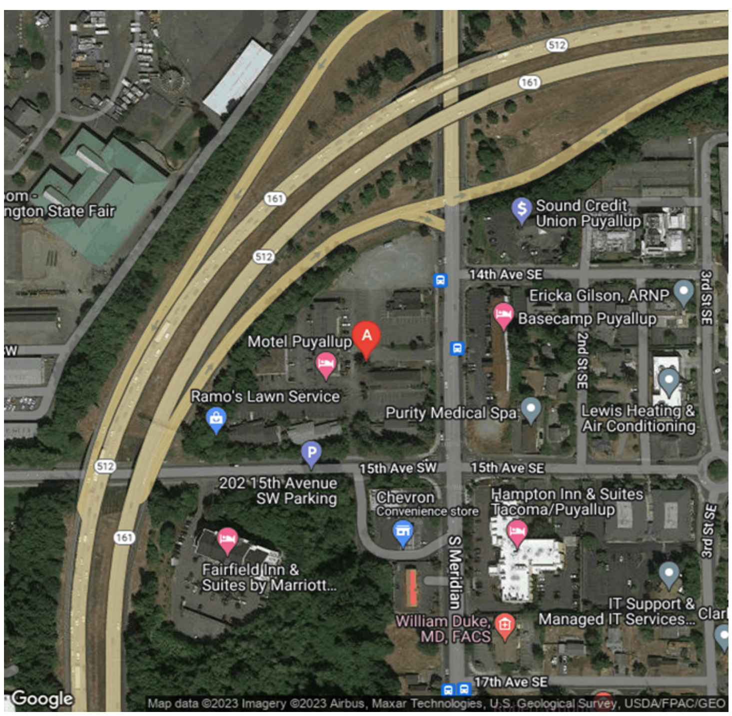
Calculation Sheets Submitted: See calcs beginning w/ sheet

Project Location 1402 S. Meridian

Puyallup, WA



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		By: CF

Design Criteria Per 2018 IBC

ATC Hazards by Location

Search Information

Coordinates:	47.178137, -122.2943517
Elevation:	46 ft
Timestamp:	2023-09-22T20:48:50.202Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	1.268	MCE_R ground motion (period=0.2s)
S_1	0.437	MCE_R ground motion (period=1.0s)
S_{MS}	1.268	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.846	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

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	49' x 94' 6-Col Canopy	Date: 26-Sep
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Design Criteria Per 2018 IBC

General

Structure Type:	**Steel**
Location:	**Puyallup, WA**
Design Codes/References:	**ACSE 7-16, 2018 IBC**

Load Combinations Per 1605.2

Design Methodology? **LRFD**

1.4(D+F)	*(Equation 16-1)*
1.2(D+F+T) + 1.6(L+H) + 0.5(Lr or S or R)	*(Equation 16-2)*
1.2(D+F) + 1.6(Lr or S or R) + (f1L or 0.5W)	*(Equation 16-3)*
1.2(D+F) + W + f1L + 1.6H + 0.5(Lr or S or R)	*(Equation 16-4)*
1.2(D+F) + 1.0E + f1L + f2S	*(Equation 16-5)*
0.9D + W + 1.6H	*(Equation 16-6)*
0.9(D+F) + 1.0E + 1.6H	*(Equation 16-7)*
.	.
.	.

Special Seismic Load Cases With Overstrength Factor - Used Where Applicable

$(1.2 + 0.2\ S_{DS})D + \Omega o Q_E + L + 0.2S$	*(ASCE 12.4.3.2)*
$(0.9 - 0.2\ S_{DS})D + \Omega o Q_E + 1.6H$	*(ASCE 12.4.3.2)*
- OR -	
$(1.0 + 0.14 S_{DS})D + H + F + 0.7\Omega o Q_E$	*(ASCE 12.4.3.2)*
$(1.0 + 0.105 S_{DS})D + H + F + 0.525\Omega o Q_E + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$	*(ASCE 12.4.3.2)*
$(0.6 - 0.14 S_{DS})D + 0.7\Omega o Q_E + H$	*(ASCE 12.4.3.2)*

Structural Design Gravity Loads

Roof Load:	Flat Roof, D =	**5**	psf (3 psf formed deck, 2 psf misc.)
	Lr =	**20**	psf
	Roof Snow =	**21**	psf (Cold Roof - 0.7*25*1.2)
Fascia	Dw =	**5**	psf = 15 lb/ft

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Seismic Criteria Per ASCE 7-16, 2018 IBC

Seismic on Primary System - Equivalent Lateral Force Procedure:

Cantilever Column System — Building Type:	**26**	*ASCE Table 12.2-1*
Lateral System = Z - Steel Oridinary Cantilevered Column System		
Site classification =	**D**	*Is E but Use Higher D -IBC Table 1613.5.2*
Spectral acc. for short periods, S_S =	**1.27**	*USGS Value*
Spectral acceleration for one second periods, S_1 =	**0.44**	*USGS Value*
Site coefficient with short periods, F_a =	1.20	*ASCE Tbl 11.4-1*
Site coefficient with one second period, F_v =	1.86	*ASCE Tbl 11.4-2*
Max spect. response accel. for short periods, $S_{MS} = F_aS_s$ =	1.52	*ASCE Eq 11.4-1*
Max spect. response accel. for 1 sec periods, $S_{M1} = F_vS_1$ =	0.81	*ASCE Eq 11.4-2*
Design spectral response accel parameter, $S_{DS} = {}^2/_3\, S_{MS}$ =	1.014	*ASCE Eq 11.4-3*
Design spectral response accel parameter, $S_{D1} = {}^2/_3\, S_{M1}$ =	0.543	*ASCE Eq 11.4-4*
Overstrength factor, Ωo = — **Flexible diaphram **N**	1.25	*ASCE Tbl 12.2.1*
Occupancy Category =	**II**	*ASCE Tbl 1-1*
Seismic design category based on short periods =	D	*ASCE Tbl 11.6-1*
Seismic design category based on one second periods =	D	*ASCE Tbl 11.6-2*

*If building is regular structure,five stories or less and period is less than 0.5 sec then Ss max = 1.5,
See - ASCE 12.8.1.3

**May reduce Ωo by 0.5 if flexible diaphram.
See Table 12.2.1 foot note

Seismic Base Shear:

Occupancy importance factor, I_E =	1.00	*ASCE Table 1.5-2*
Response modification factor, R =	1.25	*ASCE Table 12.2-1*
Deflection Amplification Factor, C_D =	1.25	*ASCE Table 12.2-1*
Period coefficeint, C_T & Variable x = **Steel Cantilever Column**	0.020	0.75 *ASCE Table 12.8-2*
Height of building from base, h_n =	**19.0**	*ft* Height Limit = 35 ft
Approx fundamental period, $T_a = C_T(H_n)^X$ =	0.18	*ASCE Eq 12.8-7*
Seismic response coefficient used, C_S =	0.81	*Max of #3, #4 & (min of #1 & #2)*
1. Calculated seis response coefficient, $C_S = (S_{DS}I_E)/R$ =	0.81	*ASCE Eq 12.8-2*
2. Max seis response coefficient required, $(S_{DI}I_E)/(RT)$ =	2.39	*ASCE Eq 12.8-3*
3. Minimum seis response coefficient = $0.044S_{DS}I_E$ =	0.04	*ASCE Eq 12.8-5*
4. Minimum seis response coefficient = $0.5S_1I_E/R$ =	0.00	*ASCE Eq 12.8-6*

Seismic Horizontal Base Shear, Vh = C_SW = **0.812** ***W {Strength}***

Seismic Vertical Base Shear, Vv = 0.2Sds = **0.203** ***W {Strength}***

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Project: 49' x 94' 6-Col Canopy	Sheet: 6
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Wind Criteria Per ASCE 7-16, 2018 IBC

Wind on Open Buildings:

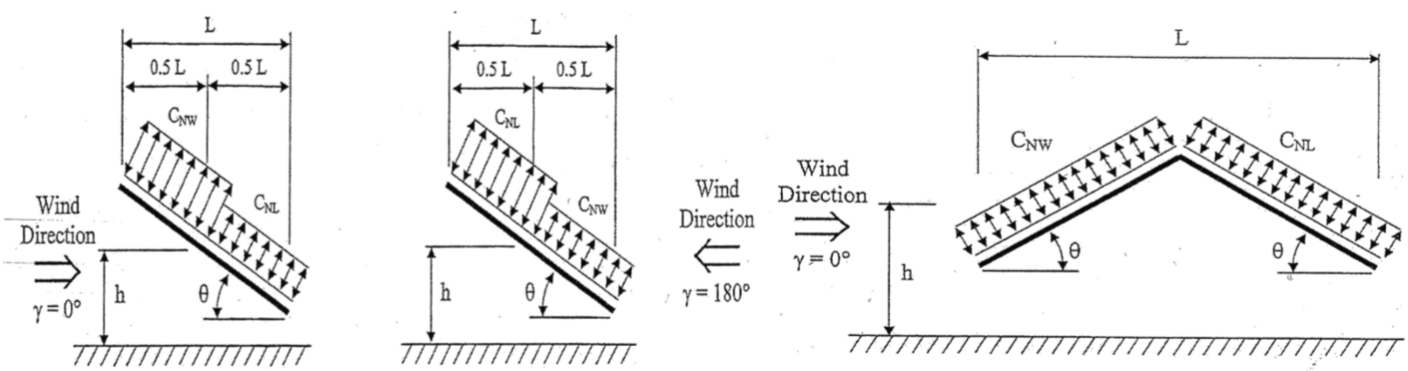
Exposure Category	= Urban, suburban, wooded areas.	B	*ASCE 26.7.3*
Mean Roof Height, h =		19.0	*ft (Max)*
Basic wind speed, V =		110.00	*mph, ASCE Fig. 26.5-1A or 26.5-1B*
Wind directionality factor, Kd =		0.85	*ASCE 26.6, ASCE Table 26.6-1*
Velocity pressure coefficient, Kz =		0.70	*ASCE 27.3, ASCE Table 27.3-1*
Topographical factor, Kzt =		1.00	*ASCE 26.8, ASCE 26.8-1*
Gust-effect factor, G =		0.85	*ASCE 26.9-1*
Roof Angle, ϕ =		0.00	*deg*
Wind flow is =	Free Flowing	F	*deg*

Net pressure coefficient, Cn:

					Net pressure, p
Wind Direction, γ = 0 deg	Load Case A	C_{NW} =	1.20	*ASCE Fig. 27.4-4/5*	p = 18.82 psf
		C_{NL} =	0.30	*ASCE Fig. 27.4-4/5*	p = 4.70 psf
	Load Case B	C_{NW} =	-1.10	*ASCE Fig. 27.4-4/5*	p = -17.25 psf
		C_{NL} =	-0.10	*ASCE Fig. 27.4-4/5*	p = -1.57 psf
Wind Direction, γ = 180 deg	Load Case A	C_{NW} =	1.20	*ASCE Fig. 27.4-4/5*	p = 18.82 psf
		C_{NL} =	0.30	*ASCE Fig. 27.4-4/5*	p = 4.70 psf
	Load Case B	C_{NW} =	-1.10	*ASCE Fig. 27.4-4/5*	p = -17.25 psf
		C_{NL} =	-0.10	*ASCE Fig. 27.4-4/5*	p = -1.57 psf

Velocity pressure, qz = $0.00256K_zK_{zt}K_dV^2I$ = 18.45 ***PSF, ASCE 27.3-1***

Net Up/Down Pressure, p = q_hGC_N =	**Max Down**	18.82	***psf, ASCE 27.4.3***
	Max Up	-17.25	***psf, ASCE 27.4-3***
Net Lat Pressure, $P_p = q_hGC_N$ =	**Windward Fascia =**	27.67	***psf, ASCE 27.4.5***
	Leeward Fascia =	18.45	***psf, ASCE 27.4.5***
	Total Load =	46.12	***psf, ASCE 27.4.5***



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Lateral Forces On Parapets as a Component Per 2018 IBC, ASCE 7-16

Wind on Parapet / Fascia As a Component Per ASCE 7-16, 30.8:

Basic wind speed, V =	110	*mph, ASCE Fig. 26.5-1A or 26.5-1B*		
Wind directionality factor, Kd =	0.85	*ASCE 26.6, ASCE Table 26.6-1*		
Exposure Category = Urban, suburban, wooded areas.	B	*ASCE 26.7.3*		
Velocity pressure coefficient, Kz =	0.70	*ASCE 27.3, ASCE Table 27.3-1*		
Topographical factor, Kzt =	1.00	*ASCE 26.8, ASCE 26.8-1*		
Velocity pressure, qz = $0.00256K_zK_{zt}K_dV^2I$ =	18.45	*psf, ASCE 27.3-1*		
Effective Trib Parapet Area =	12.00	ft^2		
Consider parapet at corner =	Yes			
External Pressure Coefficient, GCp =	-1.37	*ASCE Fig. 30.4-1*	-1.40	*ASCE Fig. 30.4-1*
	0.99	*ASCE Fig. 30.4-1*		
Internal pressure coefficient = Gcpi =	0.18	*ASCE Fig. 26.11-1*		
Design wind pressure = p = q_p(GCp-GCpi) =	**47.3**	***psf (Strength), ASCE 30.9-1***		

Wind on Deck as a Component Per ASCE 7-16, 30.8.2:

Gust-effect factor, G =	0.85	*ASCE 26.9-1*		
Roof Angle, ϕ =	0.00	*deg*		
Wind flow is = Free Flowing	F	*Fig. 6-19A thru 6-19D*		
Effective Trib Deck Area =	9.48	*ft^2 (Min)*		
Mean roof height = h =	19.00	*ft*		
Least horizontal dimension of canopy roof =	49.00	*ft*		
Dimension a defined by figure 30.8-1 = a =	4.90	*ft, Fig 6-19*	a^2 = 24.01	$4a^2$ = 96.04

Net pressure coefficient, Cn:				
Zone #3	C_{N-TOP} = 2.40	*ASCE Fig. 30.8-1*	p = 37.6	psf
	C_{N-BOT} = -3.30	*ASCE Fig. 30.8-1*	p = -51.7	psf
Zone #2	C_{N-TOP} = 1.80	*ASCE Fig. 30.8-1*	p = 28.2	psf
	C_{N-BOT} = -1.70	*ASCE Fig. 30.8-1*	p = -26.7	psf
Zone #1	C_{N-TOP} = 1.20	*ASCE Fig. 30.8-1*	p = 18.8	psf
	C_{N-BOT} = -1.10	*ASCE Fig. 30.8-1*	p = -17.2	psf

Net Up/Down Pressure in Zone #1 and #2, p = q_hGC_N = **28.2** ***psf (Strength), ASCE 6.5.13.3***

Seismic on Parapet / Fascia As a Component Per ASCE 7-16, 13.3.1:

Occupancy importance factor, I_p =	1.00	*ASCE Table 1.5-2*
Response modification factor, R =	2.50	*ASCE Table 13.5-1*
Component amplification factor, Ap =	2.50	*ASCE Table 13.5-1*
Design spectral response accel parameter, $S_{DS} = {}^2/_3 S_{MS}$ =	1.01	*ASCE Eq 11.4-4*
Parapet wall weight =	5.00	*psf*
Seismic Design Forsce, Fp = $0.4a_pS_{ds}W_pI_p(1+2^z/_h)(1/R_p)$ =	**1.22**	***Wp (Strength), ASCE 13.3-1***
	6.09	***psf (Strength), ASCE 13.3-1***

WIND CONTROLS FASCIA DESIGN

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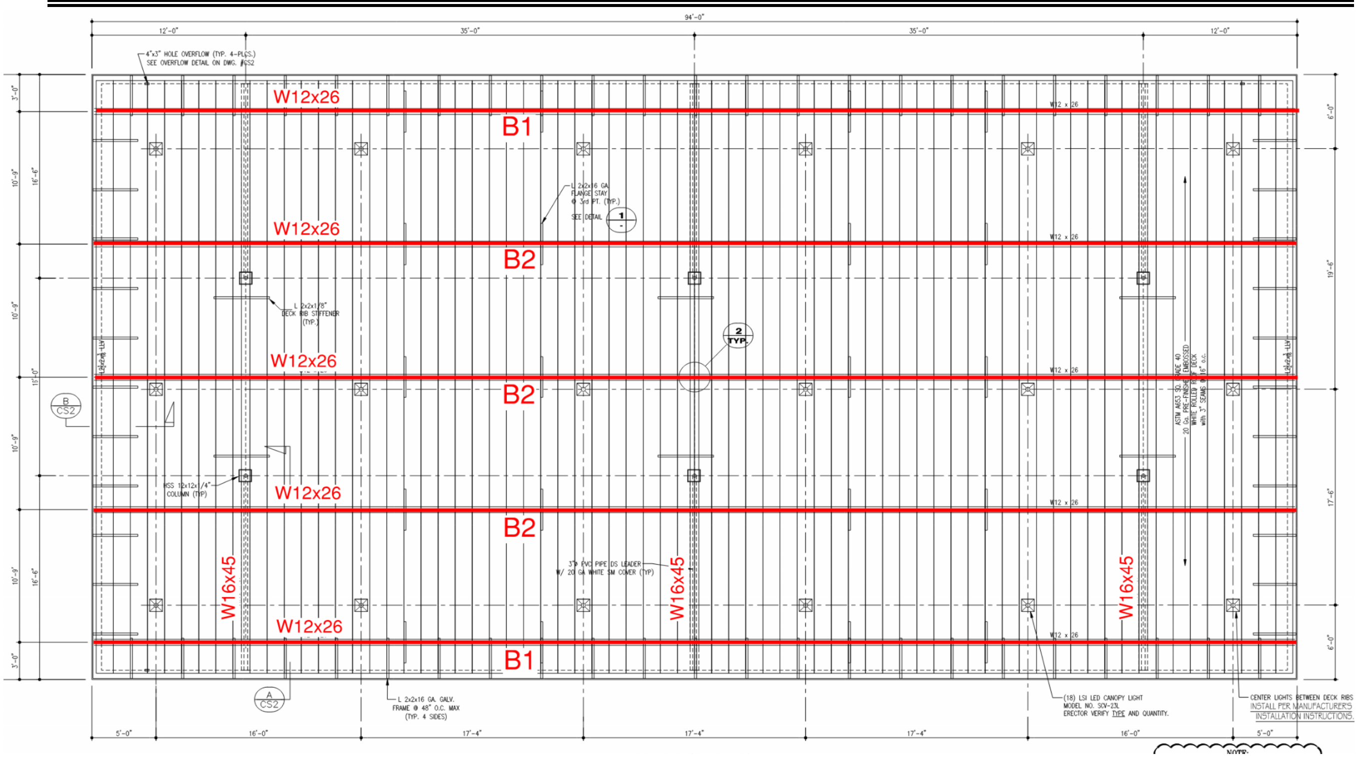
DESCRIPTION: 49' x 94' 6-Col Canopy		

Canopy Member Loads



	Beam B1		Beam B2	
Deck Cant Dim A (ft) =	3.00		0.00	
Deck Span Dim B1 (ft) =	10.75		10.75	
Deck Span Dim B2 (ft) =	0.00		10.75	
Trib Deck (ft) =	8.79		10.75	
Dead Load (psf) =	5		5	
Fascia Dead Load (plf) =	15		15	
Roof Live Load (psf) =	20		20	
Snow Load (psf) =	21		21	
Lateral Wind Load (psf) =	46		46	
Dist Dead Load (plf) =	63		54	
Dist Roof Live Load (plf) =	176		215	
Snow Load (plf) =	185		226	
Fascia Dead Load at End (lb) =	132		161	
Canopy Width (ft) =	49		49	
Number of Beams =	5		5	
Seismic Fph (Strength) =	0.812		0.812	
Seismic Fpv (Strength) =	0.203		0.203	
Seis Horiz (plf) (2nd=Snow) =	44.6	0	44.6	0
Seis Vert (plf) (2nd=Snow) =	13	0	10.9	0
Seis Horiz at End (lb) =	119		119	
Seis Vert at End (lb) =	27		33	
Fascia Ht (ft) =	3.00		3.00	
Wind Horiz (plf) =	28		28	

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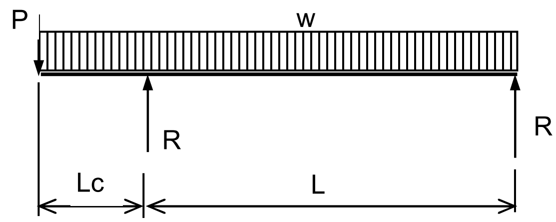
Check Roof Deck



wDL =	5 psf
wLL or wSL =	21 psf
PDL =	15 plf
PWL =	80 plf
Wind uplift load (enter negative number) =	11.3 psf
Deck Wt =	2.91 plf
Tributary Loading Strip =	1.33 ft
L =	10.75 ft
Lc =	3.00 ft
w_{DL} =	6.65 plf
w_{LL} =	27.93 plf
w_{DL+LL} =	34.6 plf
$w_{WIND\ UPLIFT}$ =	15.0 plf
$w_{SELF\ WEIGHT\ OF\ DECK}$ =	2.9 plf

Positive Bending

Max. allowable moment for positive bending (DL+LL):
Per AISI Cold-Formed Spec. Sect. C3.1.1:
Procedure I - Based on Initiation of Yielding

Nominal Moment (M_{np}) = $S_{ep}F_y$ = 14.492 in-k
Allow Moment (M_{ap}) = M_{np}/Ω_f = 8.7 in-k
Where Ω_f = 1.67 (Factor of Safety)

Actual positive bending moment:
$m_{ap} = (w_{DL+LL})(L^2)/8$ = 6.0 in-k

Since $m_{ap} < M_{ap}$, 20 Deck is OK for Positive Bending

Negative Bending At Center

Max Allow Moment for negative bending (Wind Uplift):
Per AISI Cold-Formed Spec. Sect. C3.1.1:
Procedure I - Based on Initiation of Yielding

Nominal Moment (M_{nn}) = $S_{en}F_y$ = 10.2 in-k
Allow Moment (M_{an}) = (M_{nn}/Ω_f) = 6.1 in-k
Where Ω_f = 1.67 (Factor of Safety)

Actual negative bending moment
$m_{an} = (w_{WL} - w_{WT})(L^2)/8+(PDL)(Lc)+(wDL)(Lc^2)/2$ = 2.9 in-k

Negative Bending At Ends

Actual negative bending moment (DL+WL)
m_{an} = (P)(Lc)+(w)(Lc^2)/2 = 2.8 k*in
Allow Moment = 6.1 in-k

Actual negative bending moment (DL+LL)
m_{an} = (P)(Lc)+(w)(Lc^2)/2 = 2.6 k*in
Allow Moment = 6.1 in-k

Since $m_{an} < M_{an}$, 20 Deck is OK for Negative Bending

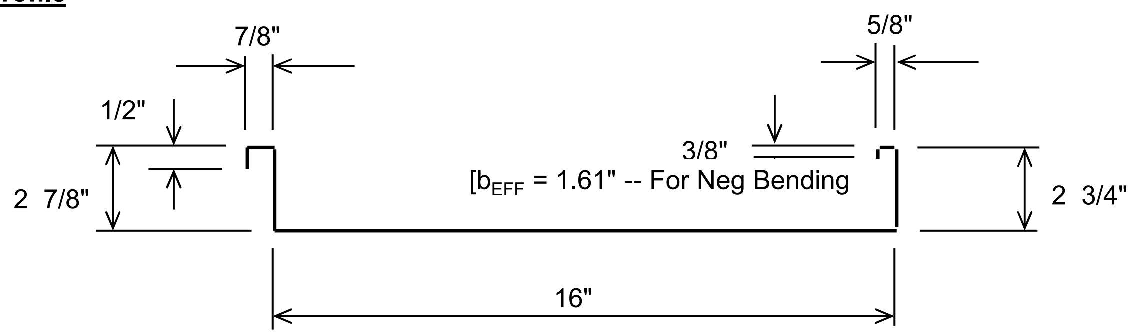
Selected Roof Deck Section:

(See following pages for calculated properties)

Gauge of Deck Used = **20**
S_{ep} = 0.362 in^3
S_{en} = 0.254 in^3

Material = A446 Gr.D
F_y = 40 ksi
E = 29000 ksi

Deck Profile



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20 Gauge Deck -- Positive Bending

ASTM A446 Gr. D   Fb = 30,000 psi
Fy = 40,000 psi   t = 0.036 in

	Previous Area	b	d	Theta	a	h	AREA	Y	AY	AY^2	Io
1	-	16	0.036	0	0.000	0.036	0.576	0.018	0.01	0.0	0.00
2	1	0.072	2.339	0	0.036	2.375	0.168	1.206	0.20	0.2	0.08
3	2	0.144	0.339	0	2.375	2.714	0.049	2.545	0.12	0.3	0.00
4	3	0.697	0.036	0	2.714	2.750	0.025	2.732	0.07	0.2	0.00
5	4	0.072	0.089	0	2.750	2.839	0.006	2.795	0.02	0.1	0.00
6	5	0.875	0.036	0	2.839	2.875	0.032	2.857	0.09	0.3	0.00
7	6	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
8	7	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
9	8	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
10	9	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
					TOTAL AREA =		0.856 in^2		0.51		1.1

TOTAL DEPTH = 2.875 in
CENTROID (Y) = SUM(AY)/SUM(AREA) = 0.600 in
C1 = Y = 0.600 in
C2 = DEPTH - Ybar = 2.275 in
I(total)= [SUM(AY^2)+SUM(Io)]-(AREA)(Y)^2 = 0.82 in^4
Sx1 = I/C1 = 1.373 in^3
Sx2 = I/C2 = 0.362 in^3 ← comp. flg.
Radius of gyration (r) = (I/A)^1/2= 0.981 in

20 Gauge Deck -- Negative Bending

	Previous Area	b	d	Theta	a	h	AREA	Y	AY	AY^2	Io
1	-	1.61	0.036	0	0.000	0.036	0.058	0.018	0.00	0.0	0.00
2	1	0.072	2.339	0	0.036	2.375	0.168	1.206	0.20	0.2	0.08
3	2	0.144	0.339	0	2.375	2.714	0.049	2.545	0.12	0.3	0.00
4	3	0.697	0.036	0	2.714	2.750	0.025	2.732	0.07	0.2	0.00
5	4	0.072	0.089	0	2.750	2.839	0.006	2.795	0.02	0.1	0.00
6	5	0.875	0.036	0	2.839	2.875	0.032	2.857	0.09	0.3	0.00
7	6	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
8	7	0	0	0	2.875	2.875	0.000	2.875	0.00	0.0	0.00
					TOTAL AREA =		0.338 in^2		0.50		1.1

TOTAL DEPTH = 2.875 in
CENTROID (Y) = SUM(AY)/SUM(AREA) = 1.492 in
C1 = Y = 1.492 in
C2 = DEPTH - Ybar = 1.383 in
I(total)= [SUM(AY^2)+SUM(Io)]-(AREA)(Y)^2 = 0.38 in^4
Sx1 = I/C1 = 0.254 in^3 ← comp. flg.
Sx2 = I/C2 = 0.274 in^3
Radius of gyration (r) = (I/A)^1/2= 1.059 in

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DESCRIPTION	49' x 94' 6-Col Canopy		

Roof Deck Check (Cont)

Effective Uniformly Compressed Flange Width
Of Gauge Metal Section with an Edge Stiffener
According to Cold-Formed Steel Design Manual

Unstiffened Flange Width (w) = 16 in
Flange Thickness (t) = 0.0359 in
Web/Stiffener Depth (D) = 2.75 in

D/w = 0.172
w/t = 445.7

$S = 1.28*(E/f)^{1/2}$ = 34.47

Where: E = 29000000 psi
fy = 40000 psi

Since w/t > S and D/w <0.25, use k-factor calculated per Case IIIb:
$Ia = t^4\{[115(w/t)/S]+5\}$ = 0.0024784 in^4
$Is = tD^3/12$ = 0.0622173 in^4

k = min of a) 3.57(Is/Ia)^n+0.43 10.88
b) 4.00
Where n = 0.333
Therefore k = 4.00
l = $1.052/(k)^{1/2}(w/t)(f/E)^{1/2}$ = 8.706
r = (1-0.22/l)/l = 0.112

Since l > 0.673, effective width (b) = rw = 1.79 in

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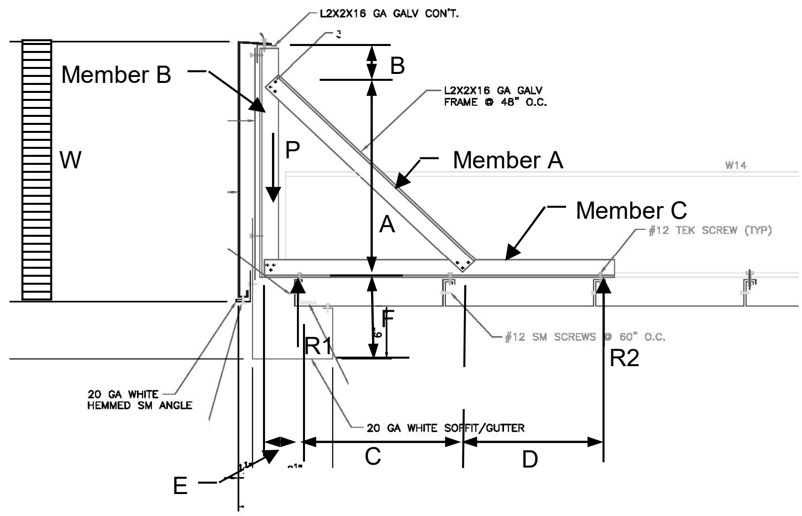
Check Fascia Members

Member Size = L 2x2x16ga
Dimension A = 25 in
Dimension B = 2 in
Dimension C = 19 in
Dimension D = 5 in
Dimension E = 5 in (Max)
Dimension F = 9 in
Frame Spacing = 48 in
Dead Load, P = 15 lb/ft
Wind Load, W = 28.4 psf (ASD)



Member A

Member Length = 34.7 in
Horiz Wind Load, Hw-A = 138 lb
Memb Angle From Horiz = 46.17 deg
Axial Wind Load, Pw-A = 199 lb

Check Connection Mem A to B/C

Screw Size = #12
# of Screws = 2
Max shear per screw = 100 lb
Alw shear per screw = 829 lb - OK

STEEL CONNECTING MATERIAL		GAGE THICKNESS (in)		20 0.0359	20 0.0359	18 0.0478	18 0.0478	16 0.0598	16 0.0598	14 0.0747	14 0.0747	12 0.1046	12 0.1046
		F_y (ksi)		33		33		50		50		50	
Size No.	Nominal Screw Diameter (in)	Minimum Head O.D. (in)	Point Type[3]	Shear	Tension	Shear	Tension	Shear	Tension	Shear	Tension	Shear	Tension
#6	0.138	0.302	SD #2	168	81	251	138	423	231	452	220	436	320
#8	0.164	0.322	SD #2,3	222	93	330	143	458	302	533	352	534	363
#10	0.190	0.384	SD #2,3	225	83	344	155	629	230	624	337	660	575
#12	0.216	0.398	SD #3	232	123	326	141	622	294	829	379	884	497
1/4 inch	0.250	0.480	SD #3	242	120	351	137	689	219	920	386	1,102	591

Member B

Axial Dead Load, Pdl-B = 60 lb
Wind Load, WL-B = 56.8 lb/ft
Wind Load Moment, Mwl-B = 355.0 lb*in
Axial Wind Load, Pwl-B = 143.8 lb
Total Axial Load, P-B = 204 lb
Horiz Shear at End, Vh = 59 lb

Check Connection Mem B to C

Result Shear, Vtot = 212 lb
Screw Size = #12
# of Screws = 2
Max shear per screw = 106 lb
Alw shear per screw = 829 lb - OK

SEE NEXT PAGES FOR MEMBER CHECKS

Member C

End Dead Load , Pdl-C = 60 lb
End Wind Load , Pwl-C = 144 lb
Moment Due To DL, Mdl-C = 300 lb*in
Moment Due to WL, Mwl-C = 719 lb*in
Total Moment, Mtot-C = 1019 lb*in

Check Connection Mem C To Roof Deck

Max Tension at R2, Ttot = 42 lb
Max Down at R1, Ptot = 204 lb
Screw Size = #12
# of Screws in tension = 1
Max tension per screw = 42 lb
Alw tension per screw = 123 lb - OK

PRCNC20240010

			Sheet	13
			Date	9/26/2023
			Job No.	230813
	FOR	Perry Builders	Eng	CF
	DESCRIPTION	49' x 94' 6-Col Canopy		

Check Fascia Angle Member A

Max Allow Length = 3.00 ft   Length Act = 2.89 ft
Max Allow Axial, P = 0.30 k   Axial Act = 0.20 k

Full Section Properties

Area	0.22045 in^2	Wt.	0.00074952 k/ft	Flat	3.8948 in
Ix	0.14464 in^4	rx	0.8100 in	Ixy	0.00000 in^4
Sx(t)	0.10228 in^3	y(t)	1.4142 in	α	0.000 deg
Sx(b)	0.10228 in^3	y(b)	1.4142 in		
Zx	0.15551 in^3	Height	2.8284 in		
Iy	0.03476 in^4	ry	0.3971 in	x_o	-0.7031 in
Sy(l)	0.05057 in^3	x(l)	0.6874 in	y_o	0.0000 in
Sy(r)	0.04909 in^3	x(r)	0.7082 in	jx	1.4098 in
Zy	0.07582 in^3	Width	1.3956 in	jy	0.0000 in
I_1	0.14464 in^4	r_1	0.8100 in	Cw	0.00000037 in^6
I_2	0.03476 in^4	r_2	0.3971 in	J	0.00023540 in^4
Ic	0.17941 in^4	rc	0.9021 in		
I_o	0.28838 in^4	r_o	1.1437 in		

Member Check - AISI S100-16/S1-18, US, ASD

Material Type: A653 SS Grade 50/1, Fy=50 ksi
Design Parameters:

Lx	3.000 ft	Ly	3.000 ft	Lt	3.000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000
Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kφ	0 k		
Red. Factor, R: 0		Lm	20.000 ft		

Loads:	P (k)	Mx (k-in)	Vy (k)	My (k-in)	Vx (k)
Entered	0.3000	0.0000	0.0000	0.0000	0.0000
Applied	0.3000	0.0000	0.0000	0.0115	0.0000
Strength	0.8417	1.3699	2.7893	0.5592	2.7893

Interaction Equations

Eq. H1.2-1	(P, Mx, My)	0.356 + 0.000 + 0.021 = 0.377 <= 1.0
Eq. H2-1	(Mx, Vy)	Sqrt(0.000 + 0.000)= 0.000 <= 1.0
Eq. H2-1	(My, Vx)	Sqrt(0.000 + 0.000)= 0.010 <= 1.0

		Sheet	14
		Date	9/26/2023
		Job No.	230813
FOR	Perry Builders	Eng	CF
DESCRIPTION	49' x 94' 6-Col Canopy		

Check Fascia Member B

Max Allow Length =	2.10 ft	Act =	2.08 ft
Max Allow Axial, P =	0.30 k	Act =	0.21 k
Max Allow Shear, V =	0.10 k	Act =	0.06 k
Max Allow Moment, M (Max) =	0.40 k*in	Act =	0.36 k*in

Member Check - AISI S100-16/S1-18, US, ASD

```
Material Type: A653 SS Grade 50/1, Fy=50 ksi
Design Parameters:
Lx         2.100 ft     Ly         2.100 ft     Lt          2.100 ft
Kx        1.0000        Ky        1.0000        Kt         1.0000
Cbx       1.0000        Cby       1.0000        ex         0.0000 in
Cmx       1.0000        Cmy       1.0000        ey         0.0000 in
Braced Flange: None     kφ             0 k
Red. Factor, R: 0       Lm        20.000 ft

Loads:          P        Mx         Vy        My         Vx
               (k)     (k-in)      (k)      (k-in)      (k)
Entered      0.3000    0.4000    0.1000    0.0000    0.0000
Applied      0.3000    0.4029    0.1000    0.0078    0.0000
Strength     0.8490    1.5014    2.7893    0.5621    2.7893

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0.353 + 0.268 + 0.014 = 0.636 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0.059 + 0.001)= 0.245 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0.000 + 0.000)= 0.007 <= 1.0
```

Check Fascia Member C

Max Allow Length =	0.50 ft	Act =	0.42 ft
Max Allow Axial, P =	0.10 k	Act =	0.06 k
Max Allow Shear, V =	0.30 k	Act =	0.21 k
Max Allow Moment, M (Max) =	1.10 k*in	Act =	1.05 k*in

Member Check - AISI S100-16/S1-18, US, ASD

```
Material Type: A653 SS Grade 50/1, Fy=50 ksi
Design Parameters:
Lx         0.500 ft     Ly         0.500 ft     Lt          0.500 ft
Kx        1.0000        Ky        1.0000        Kt         1.0000
Cbx       1.0000        Cby       1.0000        ex         0.0000 in
Cmx       1.0000        Cmy       1.0000        ey         0.0000 in
Braced Flange: None     kφ             0 k
Red. Factor, R: 0       Lm        20.000 ft

Loads:          P        Mx         Vy        My         Vx
               (k)     (k-in)      (k)      (k-in)      (k)
Entered      0.1000    1.1000    0.3000    0.0000    0.0000
Applied      0.1000    1.1002    0.3000    0.0006    0.0000
Strength     0.8561    1.6631    2.7893    0.5654    2.7893

Interaction Equations
Eq. H1.2-1    (P, Mx, My)  0.117 + 0.661 + 0.001 = 0.779 <= 1.0
Eq. H2-1         (Mx, Vy)     Sqrt(0.438 + 0.012)= 0.670 <= 1.0
Eq. H2-1         (My, Vx)     Sqrt(0.000 + 0.000)= 0.001 <= 1.0
```

Project	Sheet: 15
49' x 94' 6-Col Canopy	Job #: 230813
	Date: 9/26/2023
	By: CF

Model - Nodes

	X	Y	Z
3	24	18	0
4	45.5	18	0
5	2.5	18	12
6	2.5	19	12
8	13.25	19	12
9	32	0	12
10	32	19	12
11	24	18	12
12	24	19	12
13	45.5	18	12
14	45.5	19	12
19	2.5	18	47
20	2.5	19	47
23	32	0	47
24	32	19	47
25	24	18	47
26	24	19	47
27	45.5	18	47
28	45.5	19	47
33	2.5	18	82
34	2.5	19	82
37	32	0	82
38	32	19	82
39	24	18	82
40	24	19	82
41	45.5	18	82
42	45.5	19	82
43	2.5	18	94
45	24	18	94
46	45.5	18	94
47	16	19	12
48	16	0	12
49	16	19	47
50	16	0	47
51	16	19	82
52	16	0	82
69	13.25	18	0
70	13.25	18	12
73	13.25	18	47
74	13.25	19	47
76	13.25	18	82
77	13.25	19	82
78	13.25	18	94
111	2.5	18	0

	X	Y	Z
112	34.75	18	0
113	34.75	18	12
114	34.75	19	12
116	34.75	18	47
117	34.75	19	47
119	34.75	18	82
120	34.75	19	82
121	34.75	18	94
122	2.5	18	35.333
123	2.5	18	23.667
124	2.5	18	70.333
125	2.5	18	58.667
126	13.25	18	23.667
127	13.25	18	35.333
128	24	18	70.333
129	24	18	58.667
130	13.25	18	70.333
131	13.25	18	58.667
132	34.75	18	23.667
133	34.75	18	35.333
134	24	18	35.333
135	24	18	23.667
136	34.75	18	58.667
137	34.75	18	70.333
138	45.5	18	70.333
139	45.5	18	58.667
140	45.5	18	35.333
141	45.5	18	23.667
142			
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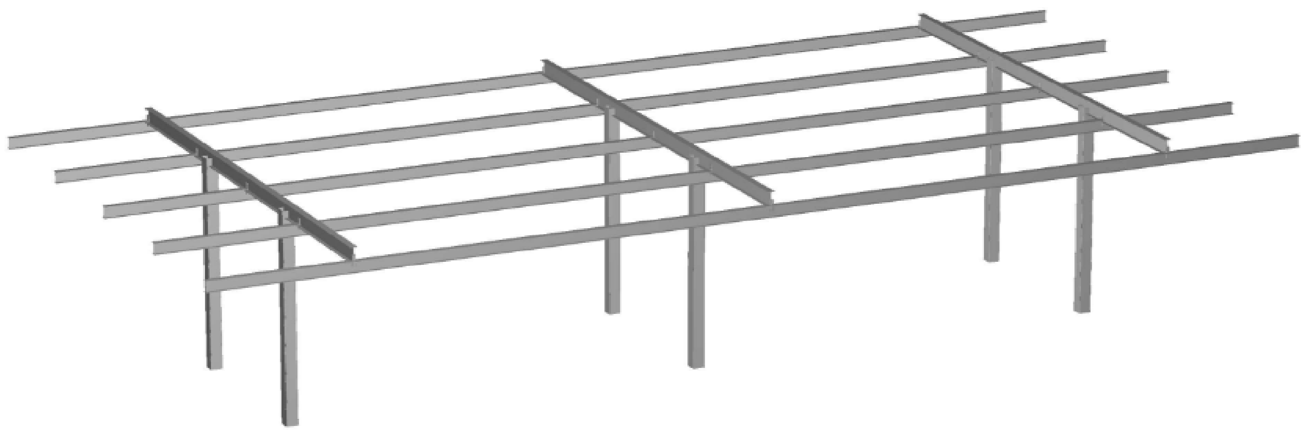
PRCNC20240010

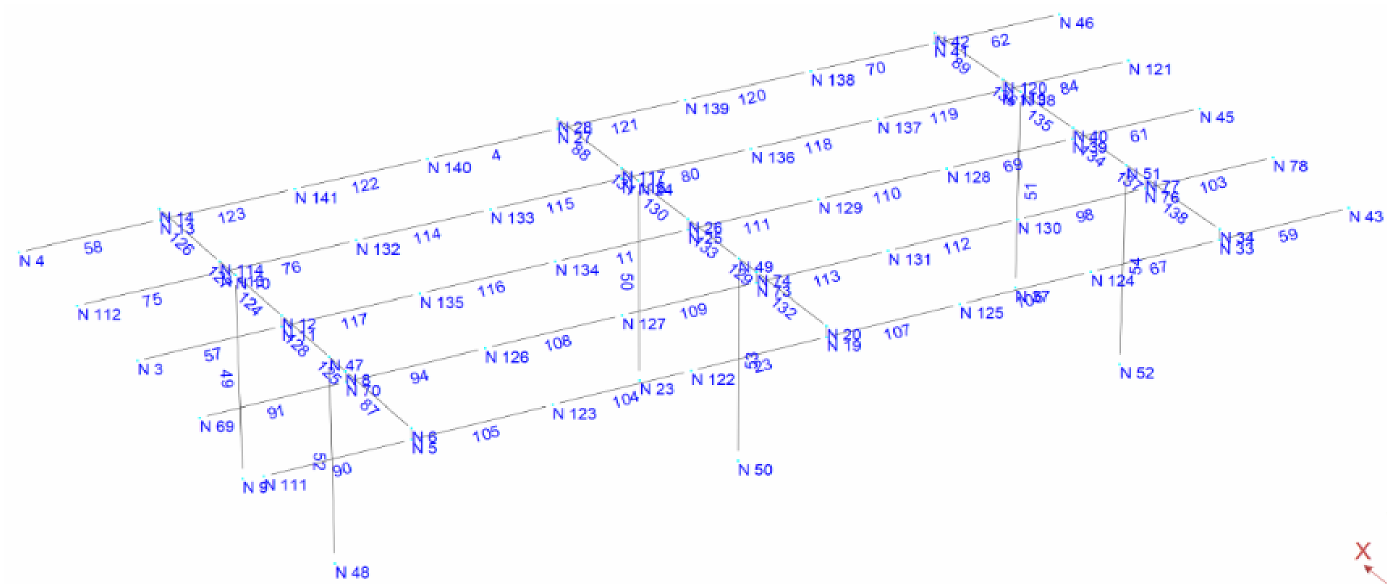
Project		
	49' x 94' 6-Col Canopy	Sheet: 16
		Job #: 230813
		Date: 9/26/2023
		By: CF

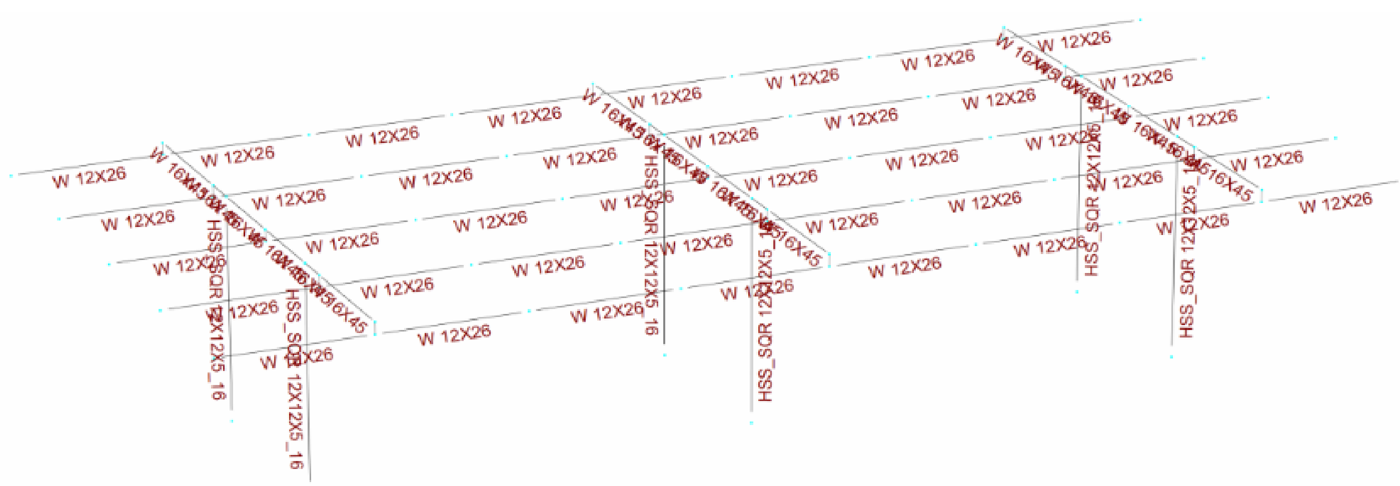
Model - Rendering



	Project		
	49' x 94' 6-Col Canopy	Sheet:	17
		Job #:	230813
		Date:	9/26/2023
		By:	CF

Nodes and Members





RAM Elements

Current Date: 9/26/2023 2:18 PM
Units system: English
File name: N:\Calculations__2023\230813 - 49'x94' (6) Col Canopy - Puyallup, WA\Calculations\230813 - 49'x94' (6) Col Puyallup, WA.retx

Geometry data

Nodes

Node	X [ft]	Y [ft]	Z [ft]	Rigid Floor
3	24.00	18.00	0.00	0
4	45.50	18.00	0.00	0
5	2.50	18.00	12.00	0
6	2.50	19.00	12.00	0
8	13.25	19.00	12.00	0
9	32.00	0.00	12.00	0
10	32.00	19.00	12.00	0
11	24.00	18.00	12.00	0
12	24.00	19.00	12.00	0
13	45.50	18.00	12.00	0
14	45.50	19.00	12.00	0
19	2.50	18.00	47.00	0
20	2.50	19.00	47.00	0
23	32.00	0.00	47.00	0
24	32.00	19.00	47.00	0
25	24.00	18.00	47.00	0
26	24.00	19.00	47.00	0
27	45.50	18.00	47.00	0
28	45.50	19.00	47.00	0
33	2.50	18.00	82.00	0
34	2.50	19.00	82.00	0
37	32.00	0.00	82.00	0
38	32.00	19.00	82.00	0
39	24.00	18.00	82.00	0
40	24.00	19.00	82.00	0
41	45.50	18.00	82.00	0
42	45.50	19.00	82.00	0
43	2.50	18.00	94.00	0
45	24.00	18.00	94.00	0
46	45.50	18.00	94.00	0
47	16.00	19.00	12.00	0
48	16.00	0.00	12.00	0
49	16.00	19.00	47.00	0
50	16.00	0.00	47.00	0
51	16.00	19.00	82.00	0
52	16.00	0.00	82.00	0
69	13.25	18.00	0.00	0
70	13.25	18.00	12.00	0
73	13.25	18.00	47.00	0
74	13.25	19.00	47.00	0
76	13.25	18.00	82.00	0
77	13.25	19.00	82.00	0
78	13.25	18.00	94.00	0
111	2.50	18.00	0.00	0
112	34.75	18.00	0.00	0
113	34.75	18.00	12.00	0
114	34.75	19.00	12.00	0
116	34.75	18.00	47.00	0
117	34.75	19.00	47.00	0

119	34.75	18.00	82.00	0
120	34.75	19.00	82.00	0
121	34.75	18.00	94.00	0
122	2.50	18.00	35.3333	0
123	2.50	18.00	23.6667	0
124	2.50	18.00	70.3333	0
125	2.50	18.00	58.6667	0
126	13.25	18.00	23.6667	0
127	13.25	18.00	35.3333	0
128	24.00	18.00	70.3333	0
129	24.00	18.00	58.6667	0
130	13.25	18.00	70.3333	0
131	13.25	18.00	58.6667	0
132	34.75	18.00	23.6667	0
133	34.75	18.00	35.3333	0
134	24.00	18.00	35.3333	0
135	24.00	18.00	23.6667	0
136	34.75	18.00	58.6667	0
137	34.75	18.00	70.3333	0
138	45.50	18.00	70.3333	0
139	45.50	18.00	58.6667	0
140	45.50	18.00	35.3333	0
141	45.50	18.00	23.6667	0

Restraints

Node	TX	TY	TZ	RX	RY	RZ
9	1	1	1	1	1	1
23	1	1	1	1	1	1
37	1	1	1	1	1	1
48	1	1	1	1	1	1
50	1	1	1	1	1	1
52	1	1	1	1	1	1

Members

Member	NJ	NK	Description	Section	Material	d0 [in]	dL [in]	Ig factor
4	27	140	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
11	25	134	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
23	19	122	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
49	10	9	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
50	24	23	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
51	38	37	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
52	47	48	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
53	49	50	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
54	51	52	Columns	HSS_SQR 12X12X5_16	A500 GrC rectangular	0.00	0.00	0.00
57	11	3	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
58	13	4	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
59	43	33	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
61	45	39	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
62	46	41	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
67	33	124	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
69	39	128	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00

70	41	138	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
75	112	113		W 12X26	A992 Gr50	0.00	0.00	0.00
76	113	132	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
80	116	136	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
84	121	119		W 12X26	A992 Gr50	0.00	0.00	0.00
87	6	8		W 16X45	A992 Gr50	0.00	0.00	0.00
88	28	117		W 16X45	A992 Gr50	0.00	0.00	0.00
89	42	120		W 16X45	A992 Gr50	0.00	0.00	0.00
90	5	111		W 12X26	A992 Gr50	0.00	0.00	0.00
91	69	70		W 12X26	A992 Gr50	0.00	0.00	0.00
94	70	126	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
98	76	130	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
103	76	78		W 12X26	A992 Gr50	0.00	0.00	0.00
104	122	123	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
105	123	5	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
106	124	125	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
107	125	19	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
108	126	127	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
109	127	73	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
110	128	129	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
111	129	25	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
112	130	131	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
113	131	73	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
114	132	133	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
115	133	116	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
116	134	135	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
117	135	11	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
118	136	137	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
119	137	119	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
120	138	139	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
121	139	27	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
122	140	141	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
123	141	13	Beams	W 12X26	A992 Gr50	0.00	0.00	0.00
124	12	10		W 16X45	A992 Gr50	0.00	0.00	0.00
125	8	47		W 16X45	A992 Gr50	0.00	0.00	0.00
126	114	14		W 16X45	A992 Gr50	0.00	0.00	0.00
127	10	114		W 16X45	A992 Gr50	0.00	0.00	0.00
128	47	12		W 16X45	A992 Gr50	0.00	0.00	0.00
129	49	74		W 16X45	A992 Gr50	0.00	0.00	0.00
130	24	26		W 16X45	A992 Gr50	0.00	0.00	0.00
131	117	24		W 16X45	A992 Gr50	0.00	0.00	0.00
132	74	20		W 16X45	A992 Gr50	0.00	0.00	0.00
133	26	49		W 16X45	A992 Gr50	0.00	0.00	0.00
134	40	51		W 16X45	A992 Gr50	0.00	0.00	0.00
135	38	40		W 16X45	A992 Gr50	0.00	0.00	0.00
136	120	38		W 16X45	A992 Gr50	0.00	0.00	0.00
137	51	77		W 16X45	A992 Gr50	0.00	0.00	0.00
138	77	34		W 16X45	A992 Gr50	0.00	0.00	0.00

Hinges

	Node-J				Node-K						
Member	**M33**	**M22**	**V3**	**V2**	**M33**	**M22**	**V3**	**V2**	**TOR**	**AXL**	**Axial rigidity**
4	1	0	0	0	0	0	0	0	0	0	Full
11	1	0	0	0	0	0	0	0	0	0	Full
23	1	0	0	0	0	0	0	0	0	0	Full
49	1	0	0	0	0	0	0	0	0	0	Full

50	1	0	0	0	0	0	0	0	0	0	Full
51	1	0	0	0	0	0	0	0	0	0	Full
52	1	0	0	0	0	0	0	0	0	0	Full
53	1	0	0	0	0	0	0	0	0	0	Full
54	1	0	0	0	0	0	0	0	0	0	Full
80	1	0	0	0	0	0	0	0	0	0	Full
107	0	0	0	0	1	0	0	0	0	0	Full
109	0	0	0	0	1	0	0	0	0	0	Full
111	0	0	0	0	1	0	0	0	0	0	Full
113	0	0	0	0	1	0	0	0	0	0	Full
115	0	0	0	0	1	0	0	0	0	0	Full
121	0	0	0	0	1	0	0	0	0	0	Full

Glossary

Cb22, Cb33	: Moment gradient coefficients
Cm22, Cm33	: Coefficients applied to bending term in interaction formula
d0	: Tapered member section depth at J end of member
DJX	: Rigid end offset distance measured from J node in axis X
DJY	: Rigid end offset distance measured from J node in axis Y
DJZ	: Rigid end offset distance measured from J node in axis Z
DKX	: Rigid end offset distance measured from K node in axis X
DKY	: Rigid end offset distance measured from K node in axis Y
DKZ	: Rigid end offset distance measured from K node in axis Z
dL	: Tapered member section depth at K end of member
Ig factor	: Inertia reduction factor (Effective Inertia/Gross Inertia) for reinforced concrete members
K22	: Effective length factor about axis 2
K33	: Effective length factor about axis 3
L22	: Member length for calculation of axial capacity
L33	: Member length for calculation of axial capacity
LB pos	: Lateral unbraced length of the compression flange in the positive side of local axis 2
LB neg	: Lateral unbraced length of the compression flange in the negative side of local axis 2
RX	: Rotation about X
RY	: Rotation about Y
RZ	: Rotation about Z
TO	: 1 = Tension only member 0 = Normal member
TX	: Translation in X
TY	: Translation in Y
TZ	: Translation in Z

RAM Elements

Current Date: 9/26/2023 2:19 PM
Units system: English
File name: N:\Calculations__2023\230813 - 49'x94' (6) Col Canopy - Puyallup, WA\Calculations\230813 - 49'x94' (6) Col Puyallup, WA.retx

Load data

Load Conditions

Condition	Description	Comb.	Category
DL	Dead Load	No	DL
RLL	Roof Live Load	No	LLR
S	Roof Snow Load	No	SNOW
Wx	Wind in X-Dir	No	WIND
SeisX	Seismic in X-Dir	No	EQ
SeisZ	Seismic in Z-Dir	No	EQ

Load on nodes

Condition	Node	FX [Kip]	FY [Kip]	FZ [Kip]	MX [Kip*ft]	MY [Kip*ft]	MZ [Kip*ft]
DL	3	0.00	-0.161	0.00	0.00	0.00	0.00
	4	0.00	-0.132	0.00	0.00	0.00	0.00
	43	0.00	-0.132	0.00	0.00	0.00	0.00
	45	0.00	-0.161	0.00	0.00	0.00	0.00
	46	0.00	-0.132	0.00	0.00	0.00	0.00
	69	0.00	-0.161	0.00	0.00	0.00	0.00
	78	0.00	-0.161	0.00	0.00	0.00	0.00
	111	0.00	-0.132	0.00	0.00	0.00	0.00
	112	0.00	-0.161	0.00	0.00	0.00	0.00
	121	0.00	-0.161	0.00	0.00	0.00	0.00
SeisX	3	-0.119	-0.033	0.00	0.00	0.00	0.00
	4	-0.119	-0.027	0.00	0.00	0.00	0.00
	43	-0.119	-0.027	0.00	0.00	0.00	0.00
	45	-0.119	-0.033	0.00	0.00	0.00	0.00
	46	-0.119	-0.027	0.00	0.00	0.00	0.00
	69	-0.119	-0.033	0.00	0.00	0.00	0.00
	78	-0.119	-0.033	0.00	0.00	0.00	0.00
	111	-0.119	-0.027	0.00	0.00	0.00	0.00
	112	-0.119	-0.033	0.00	0.00	0.00	0.00
	121	-0.119	-0.033	0.00	0.00	0.00	0.00
SeisZ	3	0.00	-0.033	0.119	0.00	0.00	0.00
	4	0.00	-0.027	0.119	0.00	0.00	0.00
	43	0.00	-0.027	0.119	0.00	0.00	0.00
	45	0.00	-0.033	0.119	0.00	0.00	0.00
	46	0.00	-0.027	0.119	0.00	0.00	0.00
	69	0.00	-0.033	0.119	0.00	0.00	0.00
	78	0.00	-0.033	0.119	0.00	0.00	0.00
	111	0.00	-0.027	0.119	0.00	0.00	0.00
	112	0.00	-0.033	0.119	0.00	0.00	0.00
	121	0.00	-0.033	0.119	0.00	0.00	0.00

Distributed force on members

Condition	Member	Dir1	Val1 [Kip/ft]	Val2 [Kip/ft]	Dist1 [ft]	%	Dist2 [ft]	%
DL	4	Y	-0.063	-0.063	0.00	No	11.6667	No
	11	Y	-0.054	-0.054	0.00	No	11.6667	No
	23	Y	-0.063	-0.063	0.00	No	11.6667	No
	57	Y	-0.054	-0.054	0.00	No	12.00	No
	58	Y	-0.063	-0.063	0.00	No	12.00	No
	59	Y	-0.063	-0.063	0.00	No	12.00	No
	61	Y	-0.054	-0.054	0.00	No	12.00	No
	62	Y	-0.063	-0.063	0.00	No	12.00	No
	67	Y	-0.063	-0.063	0.00	No	11.6667	No
	69	Y	-0.054	-0.054	0.00	No	11.6667	No
	70	Y	-0.063	-0.063	0.00	No	11.6667	No
	75	Y	-0.054	-0.054	0.00	No	12.00	No
	76	Y	-0.054	-0.054	0.00	No	11.6667	No
	80	Y	-0.054	-0.054	0.00	No	11.6667	No
	84	Y	-0.054	-0.054	0.00	No	12.00	No
	90	Y	-0.063	-0.063	0.00	No	12.00	No
	91	Y	-0.054	-0.054	0.00	No	12.00	No
	94	Y	-0.054	-0.054	0.00	No	11.6667	No
	98	Y	-0.054	-0.054	0.00	No	11.6667	No
	103	Y	-0.054	-0.054	0.00	No	12.00	No
	104	Y	-0.063	-0.063	0.00	No	11.6667	No
	105	Y	-0.063	-0.063	0.00	No	11.6667	No
	106	Y	-0.063	-0.063	0.00	No	11.6667	No
	107	Y	-0.063	-0.063	0.00	No	11.6667	No
	108	Y	-0.054	-0.054	0.00	No	11.6667	No
	109	Y	-0.054	-0.054	0.00	No	11.6667	No
	110	Y	-0.054	-0.054	0.00	No	11.6667	No
	111	Y	-0.054	-0.054	0.00	No	11.6667	No
	112	Y	-0.054	-0.054	0.00	No	11.6667	No
	113	Y	-0.054	-0.054	0.00	No	11.6667	No
	114	Y	-0.054	-0.054	0.00	No	11.6667	No
	115	Y	-0.054	-0.054	0.00	No	11.6667	No
	116	Y	-0.054	-0.054	0.00	No	11.6667	No
	117	Y	-0.054	-0.054	0.00	No	11.6667	No
	118	Y	-0.054	-0.054	0.00	No	11.6667	No
	119	Y	-0.054	-0.054	0.00	No	11.6667	No
	120	Y	-0.063	-0.063	0.00	No	11.6667	No
	121	Y	-0.063	-0.063	0.00	No	11.6667	No
	122	Y	-0.063	-0.063	0.00	No	11.6667	No
	123	Y	-0.063	-0.063	0.00	No	11.6667	No
RLL	4	Y	-0.176	-0.176	0.00	No	11.6667	No
	11	Y	-0.215	-0.215	0.00	No	11.6667	No
	23	Y	-0.176	-0.176	0.00	No	11.6667	No
	57	Y	-0.215	-0.215	0.00	No	12.00	No
	58	Y	-0.176	-0.176	0.00	No	12.00	No
	59	Y	-0.176	-0.176	0.00	No	12.00	No
	61	Y	-0.215	-0.215	0.00	No	12.00	No
	62	Y	-0.176	-0.176	0.00	No	12.00	No
	67	Y	-0.176	-0.176	0.00	No	11.6667	No
	69	Y	-0.215	-0.215	0.00	No	11.6667	No
	70	Y	-0.176	-0.176	0.00	No	11.6667	No
	75	Y	-0.215	-0.215	0.00	No	12.00	No
	76	Y	-0.215	-0.215	0.00	No	11.6667	No
	80	Y	-0.215	-0.215	0.00	No	11.6667	No
	84	Y	-0.215	-0.215	0.00	No	12.00	No
	90	Y	-0.176	-0.176	0.00	No	12.00	No
	91	Y	-0.215	-0.215	0.00	No	12.00	No
	94	Y	-0.215	-0.215	0.00	No	11.6667	No
	98	Y	-0.215	-0.215	0.00	No	11.6667	No
	103	Y	-0.215	-0.215	0.00	No	12.00	No
	104	Y	-0.176	-0.176	0.00	No	11.6667	No

	105	Y	-0.176	-0.176	0.00	No	11.6667	No
	106	Y	-0.176	-0.176	0.00	No	11.6667	No
	107	Y	-0.176	-0.176	0.00	No	11.6667	No
	108	Y	-0.215	-0.215	0.00	No	11.6667	No
	109	Y	-0.215	-0.215	0.00	No	11.6667	No
	110	Y	-0.215	-0.215	0.00	No	11.6667	No
	111	Y	-0.215	-0.215	0.00	No	11.6667	No
	112	Y	-0.215	-0.215	0.00	No	11.6667	No
	113	Y	-0.215	-0.215	0.00	No	11.6667	No
	114	Y	-0.215	-0.215	0.00	No	11.6667	No
	115	Y	-0.215	-0.215	0.00	No	11.6667	No
	116	Y	-0.215	-0.215	0.00	No	11.6667	No
	117	Y	-0.215	-0.215	0.00	No	11.6667	No
	118	Y	-0.215	-0.215	0.00	No	11.6667	No
	119	Y	-0.215	-0.215	0.00	No	11.6667	No
	120	Y	-0.176	-0.176	0.00	No	11.6667	No
	121	Y	-0.176	-0.176	0.00	No	11.6667	No
	122	Y	-0.176	-0.176	0.00	No	11.6667	No
	123	Y	-0.176	-0.176	0.00	No	11.6667	No
S	4	Y	-0.185	-0.185	0.00	No	11.6667	No
	11	Y	-0.226	-0.226	0.00	No	11.6667	No
	23	Y	-0.185	-0.185	0.00	No	11.6667	No
	57	Y	-0.226	-0.226	0.00	No	12.00	No
	58	Y	-0.185	-0.185	0.00	No	12.00	No
	59	Y	-0.185	-0.185	0.00	No	12.00	No
	61	Y	-0.226	-0.226	0.00	No	12.00	No
	62	Y	-0.185	-0.185	0.00	No	12.00	No
	67	Y	-0.185	-0.185	0.00	No	11.6667	No
	69	Y	-0.226	-0.226	0.00	No	11.6667	No
	70	Y	-0.185	-0.185	0.00	No	11.6667	No
	75	Y	-0.226	-0.226	0.00	No	12.00	No
	76	Y	-0.226	-0.226	0.00	No	11.6667	No
	80	Y	-0.226	-0.226	0.00	No	11.6667	No
	84	Y	-0.226	-0.226	0.00	No	12.00	No
	90	Y	-0.185	-0.185	0.00	No	12.00	No
	91	Y	-0.226	-0.226	0.00	No	12.00	No
	94	Y	-0.226	-0.226	0.00	No	11.6667	No
	98	Y	-0.226	-0.226	0.00	No	11.6667	No
	103	Y	-0.226	-0.226	0.00	No	12.00	No
	104	Y	-0.185	-0.185	0.00	No	11.6667	No
	105	Y	-0.185	-0.185	0.00	No	11.6667	No
	106	Y	-0.185	-0.185	0.00	No	11.6667	No
	107	Y	-0.185	-0.185	0.00	No	11.6667	No
	108	Y	-0.226	-0.226	0.00	No	11.6667	No
	109	Y	-0.226	-0.226	0.00	No	11.6667	No
	110	Y	-0.226	-0.226	0.00	No	11.6667	No
	111	Y	-0.226	-0.226	0.00	No	11.6667	No
	112	Y	-0.226	-0.226	0.00	No	11.6667	No
	113	Y	-0.226	-0.226	0.00	No	11.6667	No
	114	Y	-0.226	-0.226	0.00	No	11.6667	No
	115	Y	-0.226	-0.226	0.00	No	11.6667	No
	116	Y	-0.226	-0.226	0.00	No	11.6667	No
	117	Y	-0.226	-0.226	0.00	No	11.6667	No
	118	Y	-0.226	-0.226	0.00	No	11.6667	No
	119	Y	-0.226	-0.226	0.00	No	11.6667	No
	120	Y	-0.185	-0.185	0.00	No	11.6667	No
	121	Y	-0.185	-0.185	0.00	No	11.6667	No
	122	Y	-0.185	-0.185	0.00	No	11.6667	No
	123	Y	-0.185	-0.185	0.00	No	11.6667	No
Wx	4	X	-0.028	-0.028	0.00	No	11.6667	No
	11	X	-0.028	-0.028	0.00	No	11.6667	No
	23	X	-0.028	-0.028	0.00	No	11.6667	No

	57	X	-0.028	-0.028	0.00	No	12.00	No
	58	X	-0.028	-0.028	0.00	No	12.00	No
	59	X	-0.028	-0.028	0.00	No	12.00	No
	61	X	-0.028	-0.028	0.00	No	12.00	No
	62	X	-0.028	-0.028	0.00	No	12.00	No
	67	X	-0.028	-0.028	0.00	No	11.6667	No
	69	X	-0.028	-0.028	0.00	No	11.6667	No
	70	X	-0.028	-0.028	0.00	No	11.6667	No
	75	X	-0.028	-0.028	0.00	No	12.00	No
	76	X	-0.028	-0.028	0.00	No	11.6667	No
	80	X	-0.028	-0.028	0.00	No	11.6667	No
	84	X	-0.028	-0.028	0.00	No	12.00	No
	90	X	-0.028	-0.028	0.00	No	12.00	No
	91	X	-0.028	-0.028	0.00	No	12.00	No
	94	X	-0.028	-0.028	0.00	No	11.6667	No
	98	X	-0.028	-0.028	0.00	No	11.6667	No
	103	X	-0.028	-0.028	0.00	No	12.00	No
	104	X	-0.028	-0.028	0.00	No	11.6667	No
	105	X	-0.028	-0.028	0.00	No	11.6667	No
	106	X	-0.028	-0.028	0.00	No	11.6667	No
	107	X	-0.028	-0.028	0.00	No	11.6667	No
	108	X	-0.028	-0.028	0.00	No	11.6667	No
	109	X	-0.028	-0.028	0.00	No	11.6667	No
	110	X	-0.028	-0.028	0.00	No	11.6667	No
	111	X	-0.028	-0.028	0.00	No	11.6667	No
	112	X	-0.028	-0.028	0.00	No	11.6667	No
	113	X	-0.028	-0.028	0.00	No	11.6667	No
	114	X	-0.028	-0.028	0.00	No	11.6667	No
	115	X	-0.028	-0.028	0.00	No	11.6667	No
	116	X	-0.028	-0.028	0.00	No	11.6667	No
	117	X	-0.028	-0.028	0.00	No	11.6667	No
	118	X	-0.028	-0.028	0.00	No	11.6667	No
	119	X	-0.028	-0.028	0.00	No	11.6667	No
	120	X	-0.028	-0.028	0.00	No	11.6667	No
	121	X	-0.028	-0.028	0.00	No	11.6667	No
	122	X	-0.028	-0.028	0.00	No	11.6667	No
	123	X	-0.028	-0.028	0.00	No	11.6667	No
SeisX	4	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	11	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	23	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	57	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	58	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.013	-0.013	0.00	No	12.00	No
	59	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.013	-0.013	0.00	No	12.00	No
	61	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	62	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.013	-0.013	0.00	No	12.00	No
	67	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	69	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	70	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	75	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	76	X	-0.0446	-0.0446	0.00	No	11.6667	No

		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	80	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	84	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	90	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.013	-0.013	0.00	No	12.00	No
	91	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	94	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	98	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	103	X	-0.0446	-0.0446	0.00	No	12.00	No
		Y	-0.0109	-0.0109	0.00	No	12.00	No
	104	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	105	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	106	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	107	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	108	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	109	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	110	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	111	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	112	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	113	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	114	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	115	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	116	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	117	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	118	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	119	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.0109	-0.0109	0.00	No	11.6667	No
	120	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	121	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	122	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
	123	X	-0.0446	-0.0446	0.00	No	11.6667	No
		Y	-0.013	-0.013	0.00	No	11.6667	No
SeisZ	4	Y	-0.013	-0.013	0.00	No	11.6667	No
		Z	0.0446	0.0446	0.00	No	11.6667	No
	11	Y	-0.0109	-0.0109	0.00	No	11.6667	No
		Z	0.0446	0.0446	0.00	No	11.6667	No
	23	Y	-0.013	-0.013	0.00	No	11.6667	No
		Z	0.0446	0.0446	0.00	No	11.6667	No
	57	Y	-0.0109	-0.0109	0.00	No	12.00	No

	Z	0.0446	0.0446	0.00	No	12.00	No
58	Y	-0.013	-0.013	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
59	Y	-0.013	-0.013	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
61	Y	-0.0109	-0.0109	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
62	Y	-0.013	-0.013	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
67	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
69	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
70	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
75	Y	-0.0109	-0.0109	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
76	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
80	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
84	Y	-0.0109	-0.0109	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
90	Y	-0.013	-0.013	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
91	Y	-0.0109	-0.0109	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
94	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
98	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
103	Y	-0.0109	-0.0109	0.00	No	12.00	No
	Z	0.0446	0.0446	0.00	No	12.00	No
104	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
105	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
106	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
107	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
108	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
109	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
110	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
111	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
112	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
113	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
114	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
115	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
116	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
117	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
118	Y	-0.0109	-0.0109	0.00	No	11.6667	No

	Z	0.0446	0.0446	0.00	No	11.6667	No
119	Y	-0.0109	-0.0109	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
120	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
121	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
122	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No
123	Y	-0.013	-0.013	0.00	No	11.6667	No
	Z	0.0446	0.0446	0.00	No	11.6667	No

Self weight multipliers for load conditions

		Self weight multiplier			
Condition	**Description**	**Comb.**	**MultX**	**MultY**	**MultZ**
DL	Dead Load	No	0.00	-1.00	0.00
RLL	Roof Live Load	No	0.00	0.00	0.00
S	Roof Snow Load	No	0.00	0.00	0.00
Wx	Wind in X-Dir	No	0.00	0.00	0.00
SeisX	Seismic in X-Dir	No	0.812	-0.203	0.00
SeisZ	Seismic in Z-Dir	No	0.00	-0.203	0.812

Earthquake (Dynamic analysis only)

Condition	**a/g**	**Ang.** [Deg]	**Damp.** [%]
DL	0.00	0.00	0.00
RLL	0.00	0.00	0.00
S	0.00	0.00	0.00
Wx	0.00	0.00	0.00
SeisX	0.00	0.00	0.00
SeisZ	0.00	0.00	0.00

Glossary

Comb : Indicates if load condition is a load combination

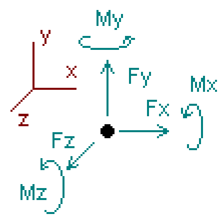
RAM Elements

Current Date: 9/25/2023 11:43 PM
Units system: English
File name: N:\Calculations__2023\230813 - 49'x94' (6) Col Canopy - Puyallup, WA\Calculations\230813 - 49'x94' (6) Col Puyallup, WA.retx

Analysis result

Nodes

Reactions



Direction of positive forces and moments

Node	Forces [Kip] FX	FY	FZ	Moments [Kip*ft] MX	MY	MZ
Condition **DL=Dead Load**						
9	0.00000	8.93727	-0.02783	-0.57475	0.00012	0.00001
23	0.00000	8.03316	0.00000	0.00000	0.00000	-0.00002
37	0.00000	8.93727	0.02783	0.57475	-0.00012	0.00001
48	0.00000	8.93727	-0.02783	-0.57475	-0.00012	-0.00001
50	0.00000	8.03316	0.00000	0.00000	0.00000	0.00002
52	0.00000	8.93727	0.02783	0.57475	0.00012	-0.00001
SUM	0.00000	51.81539	0.00000	0.00000	0.00000	0.00000
Condition **RLL=Roof Live Load**						
9	0.00000	15.73943	-0.10968	-2.26124	0.00622	-0.00001
23	0.00000	15.38013	0.00000	0.00000	0.00000	0.00001
37	0.00000	15.73943	0.10968	2.26124	-0.00622	-0.00001
48	0.00000	15.73943	-0.10968	-2.26124	-0.00622	0.00001
50	0.00000	15.38013	0.00000	0.00000	0.00000	-0.00001
52	0.00000	15.73943	0.10968	2.26124	0.00622	0.00001
SUM	0.00000	93.71800	0.00000	0.00000	0.00000	0.00000
Condition **S=Roof Snow Load**						
9	0.00000	16.54456	-0.11529	-2.37694	0.00654	-0.00001
23	0.00000	16.16688	0.00000	0.00000	0.00000	0.00001
37	0.00000	16.54456	0.11529	2.37694	-0.00654	-0.00001
48	0.00000	16.54456	-0.11529	-2.37694	-0.00654	0.00001
50	0.00000	16.16688	0.00000	0.00000	0.00000	-0.00001
52	0.00000	16.54456	0.11529	2.37694	0.00654	0.00001
SUM	0.00000	98.51200	0.00000	0.00000	0.00000	0.00000

Condition **Wx=Wind in X-Dir**

9	2.04305	0.25045	0.00152	0.02961	-0.02972	-38.81791
23	2.49390	0.32159	0.00000	0.00000	0.00000	-47.38418
37	2.04305	0.25045	-0.00152	-0.02961	0.02972	-38.81791
48	2.04305	-0.25045	-0.00152	-0.02961	-0.02972	-38.81791
50	2.49390	-0.32159	0.00000	0.00000	0.00000	-47.38418
52	2.04305	-0.25045	0.00152	0.02961	0.02972	-38.81791
SUM	13.16000	0.00000	0.00000	0.00000	0.00000	-250.04000

Condition **SeisX=Seismic in X-Dir**

9	0.57237	2.08039	-0.00663	-0.13562	0.02045	-17.51628
23	0.49201	1.87526	0.00000	0.00000	0.00000	-15.98950
37	0.57237	2.08039	0.00663	0.13562	-0.02045	-17.51628
48	0.57237	1.55741	-0.00455	-0.09518	0.02023	-17.51629
50	0.49201	1.39257	0.00000	0.00000	0.00000	-15.98949
52	0.57237	1.55741	0.00455	0.09518	-0.02023	-17.51629
SUM	3.27350	10.54343	0.00000	0.00000	0.00000	-102.04412

Condition **SeisZ=Seismic in Z-Dir**

9	-0.00024	1.96215	-7.41614	-133.37046	18.99489	0.00464
23	0.00000	1.63392	-5.69415	-97.34596	9.80005	0.00000
37	0.00024	1.67565	-7.40497	-133.13966	18.99466	-0.00464
48	0.00024	1.96215	-7.41614	-133.37046	-18.99489	-0.00464
50	0.00000	1.63392	-5.69415	-97.34596	-9.80005	0.00000
52	-0.00024	1.67565	-7.40497	-133.13966	-18.99466	0.00464
SUM	0.00000	10.54343	-41.03050	-727.71216	0.00000	0.00000

RAM Elements

Current Date: 9/26/2023 2:22 PM
Units system: English
File name: N:\Calculations__2023\230813 - 49'x94' (6) Col Canopy - Puyallup, WA\Calculations\230813 - 49'x94' (6) Col Puyallup, WA.retx

Steel Code Check
Summary - Group by member

Load conditions to be included in design :

D1=1.4DL
D2=1.2DL+0.5RLL
D3=1.2DL+0.5S
D4=1.2DL+1.6RLL
D5=1.2DL+1.6S
D6=1.2DL+0.5Wx
D7=1.2DL+1.6RLL+0.5Wx
D8=1.2DL+1.6S+0.5Wx
D9=1.2DL+Wx
D10=1.2DL+Wx+0.5RLL
D11=1.2DL+Wx+0.5S
D12=1.2DL+0.2S
D13=1.2DL+SeisX
D14=1.2DL+SeisZ
D15=1.2DL+SeisX+0.2S
D16=1.2DL+SeisZ+0.2S
D17=0.9DL+Wx
D18=0.9DL+SeisX
D19=0.9DL+SeisZ

Description	Section	Member	Ctrl Eq.	Ratio	Status	Reference
	W 12X26	**75**	D8 at 100.00%	0.29	OK	
		84	D8 at 100.00%	0.29	OK	
		90	D8 at 0.00%	0.25	OK	
		91	D8 at 100.00%	**0.29**	**OK**	
		103	D8 at 0.00%	0.29	OK	
	W 16X45	**87**	D16 at 100.00%	0.62	OK	
		88	D8 at 100.00%	0.44	OK	
		89	D16 at 100.00%	0.61	OK	
		124	D8 at 100.00%	0.72	OK	
		125	D16 at 100.00%	0.91	OK	
		126	D16 at 0.00%	0.62	OK	
		127	D16 at 0.00%	**0.91**	**OK**	
		128	D5 at 0.00%	0.72	OK	
		129	D5 at 0.00%	0.68	OK	
		130	D8 at 0.00%	0.68	OK	
		131	D8 at 100.00%	0.68	OK	
		132	D5 at 0.00%	0.44	OK	
		133	D5 at 100.00%	0.68	OK	
		134	D5 at 100.00%	0.72	OK	
		135	D8 at 0.00%	0.72	OK	
		136	D16 at 100.00%	0.90	OK	
		137	D16 at 0.00%	0.90	OK	
		138	D16 at 0.00%	0.61	OK	
<u>**Beams**</u>	***W 12X26***	**4**	D5 at 100.00%	0.32	OK	
		11	D8 at 100.00%	0.38	OK	
		23	D8 at 100.00%	0.34	OK	

		57	D8 at 0.00%	0.29	OK
		58	D8 at 0.00%	0.25	OK
		59	D8 at 100.00%	0.25	OK
		61	D5 at 100.00%	0.25	OK
		62	D8 at 100.00%	0.25	OK
		67	D16 at 0.00%	0.30	OK
		69	D8 at 0.00%	0.30	OK
		70	D16 at 0.00%	0.30	OK
		76	D8 at 0.00%	0.30	OK
		80	D8 at 100.00%	0.38	OK
		94	D8 at 0.00%	0.30	OK
		98	D8 at 0.00%	0.30	OK
		104	D8 at 31.25%	0.46	OK
		105	D16 at 100.00%	0.33	OK
		106	D8 at 68.75%	0.46	OK
		107	D8 at 0.00%	0.34	OK
		108	D8 at 68.75%	0.51	OK
		109	D8 at 0.00%	0.38	OK
		110	D8 at 68.75%	**0.51**	**OK**
		111	D8 at 0.00%	0.38	OK
		112	D8 at 68.75%	0.51	OK
		113	D8 at 0.00%	0.38	OK
		114	D8 at 68.75%	0.51	OK
		115	D8 at 0.00%	0.38	OK
		116	D8 at 31.25%	0.51	OK
		117	D8 at 100.00%	0.30	OK
		118	D8 at 31.25%	0.51	OK
		119	D8 at 100.00%	0.30	OK
		120	D8 at 68.75%	0.43	OK
		121	D5 at 0.00%	0.32	OK
		122	D8 at 31.25%	0.43	OK
		123	D16 at 100.00%	0.33	OK
Columns	***HSS_SQR 12X12X5_16***	**49**	D16 at 100.00%	**0.76**	**OK**
		50	D16 at 100.00%	0.55	OK
		51	D16 at 100.00%	0.75	OK
		52	D16 at 100.00%	0.76	OK
		53	D16 at 100.00%	0.55	OK
		54	D16 at 100.00%	0.75	OK

		SHT 33
		DATE 9/26/2023
	FOR: Perry Builders	DES. BY CF
	DESCRIPTION: 49' x 94' 6-Col Canopy	BHB Nc 230813

Check Base Plate

Worse Case Wind Shear =	2494 lb (Strength)	Worse Case Seis Shear =	9270 lb
Worse Case Wind Moment =	47384 lb*ft (Strength)	Worse Case Seis Moment =	166713 lb*ft
Axial Dead Load =	8937 lb	Note 1.25 Overstrength in Seismic Numbers	
Worse Case Dead Load Moment =	575 lb*ft		
Axial Roof Live Load =	15739 lb		
Worse Case Live Load Moment =	2261 lb*ft		
Axial Snow Load =	16545 lb		
Worse Case Snow Load Moment =	23694 lb*ft (Strength)		

General Information

Material Properties

AISC Design Method Load Resistance Factor Design — Φ c : LRFD Resistance Factor 0.65

Steel Plate Fy = 36 ksi

Concrete Support f'c = 3 ksi

Assumed Bearing Area Full Bearing — Nominal Bearing Fp per J8 5.10 ksi

Column & Plate

Column Properties

Steel Section HSS12x12x5/16

Depth	12 in	Area	13.4 in^2
Width	12 in	Ixx	304 in^4
Flange Thickness	0.291 in	Iyy	304 in^4
Web Thicknes	0 in		

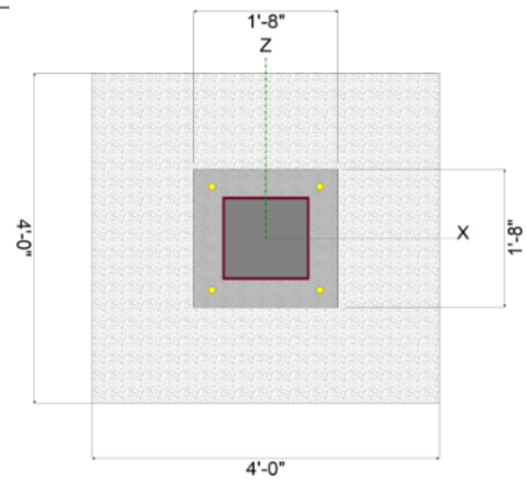
Plate Dimensions

N : Length	20.0 in
B : Width	20.0 in
Thickness	2.50 in

Column assumed welded to base plat

Support Dimensions

Width along "X"	48.0 in
Length along "Z'	48.0 in

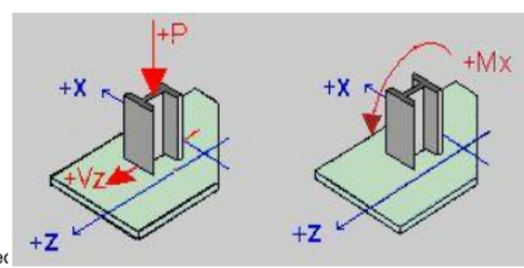


Applied Loads

	P-Y	V-Z	M-X
D : Dead Load	8.937 k	k	k-ft
L : Live	k	k	k-ft
Lr : Roof Live	15.739 k	k	k-ft
S : Snow	16.545 k	k	k-ft
W : Wind	k	2.494 k	47.384 k-ft
E : Earthquake	k	9.270 k	166.713 k-ft
H : Lateral Earth	k	k	k-ft

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

"+" Shears push plate towards +Z edge



GOVERNING DESIGN LOAD CASE SUMMARY

Plate Design Summary

Design Method	**Load Resistance Factor Design**
Governing Load Combinat	**+1.20D+0.50Lr+W**
Governing Load Case Typ	**Axial + Moment, L/2 < Eccentricity, Tensioı**
Governing STRESS RATI(	**1.0**
Design Plate Size	**1'-8" x 1'-8" x 2 -1/2"**
Pu : Axial	0.000 k
Mu : Moment	0.000 k-ft

Mu : Max. Moment	29.749 k-in
fb : Max. Bending Stress	28.559 ksi
Fb : Allowable : Fy * Phi	32.400 ksi
Bending Stress Rati	0.881
	Bending Stress OK
fu : Max. Plate Bearing Stress	3.315 ksi
Fp : Allowable :	3.315 ksi
Bearing Stress Ratio	1.000
	Bearing Stress OK

		SHT	34
		DATE	9/26/2023
	FOR: Monitor	DES. BY	CF
	DESCRIPTION: 25x36 2 Col Canopy	BHB Nc	230813

Check Weld With Rectangular Perimeter

Members:

Member #1: **HSS 12x12** Member #2: **Base Plate**

Weld Properties:

Weld dimension, b =	12	in	Weld section modulus = $bd+1/3d^2$, Sx =	192	in^2
Weld dimension, d =	12	in	Weld section modulus = $db+1/3b^2$, Sy =	192	in^2
Weld size, c =	0.313	in	Weld polar m. of inertia = $1/6\ (b+d)^3$, Jz =	2304	in^3
Weld yeild stress = fy =	70.00	ksi	Weld overall length, Lw =	48	in

Loads:

Load in X direction, Vx =	6489	lb (Working)	Moment about X-X axis, Mx =	116699	lb*ft (Working)
Load in Y direction, Vy =	0	lb (Working)	Moment about Y-Y axis, My =	0	lb*ft (Working)
Load in Z direction, Vz =	0	lb (Working)	Moment about Z-Z axis, Mz =	0	lb*ft (Working)

Load on Welds:

Weld load due to Mx, W_{Mx} = 12*Mx / Sx =	7294	lb/in (Z axis)
Weld load due to My, W_{My} = 12*My / Sy =	0	lb/in (Z axis)
Weld load at A due to Mz, $W_{A\text{-}Mz}$ = 12*Mz*d/2 / Jz =	0	lb/in (X axis)
Weld load at B due to Mz, $W_{B\text{-}Mz}$ = 12*Mz*b/2 / Jz =	0	lb/in (Y axis)

Maximum load on weld at A = $\{(V_x/L_w+W_{A\text{-}Mz})^2 + (Vy/Lw)^2 + (Vz/Lw + W_{Mx} + W_{My})2\}^{1/2}$ = 7295 lb/in

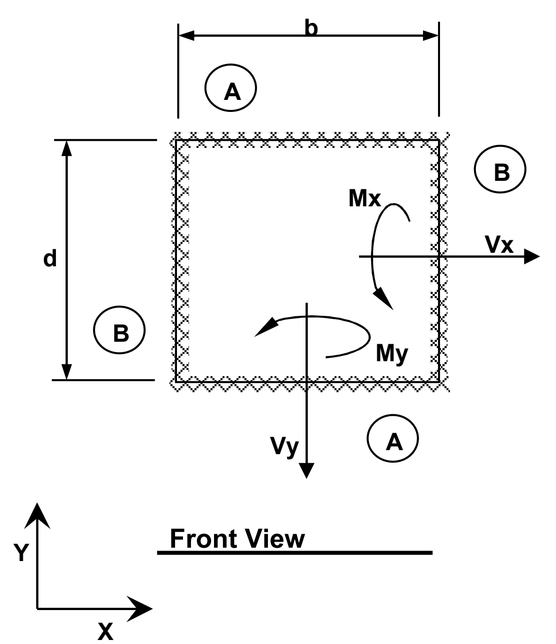
Maximum load on weld at B = $\{(V_x/L_w+W_{B\text{-}Mz})^2 + (Vy/Lw)^2 + (Vz/Lw + W_{Mx} + W_{My})2\}^{1/2}$ = 7295 lb/in

Check Welds:

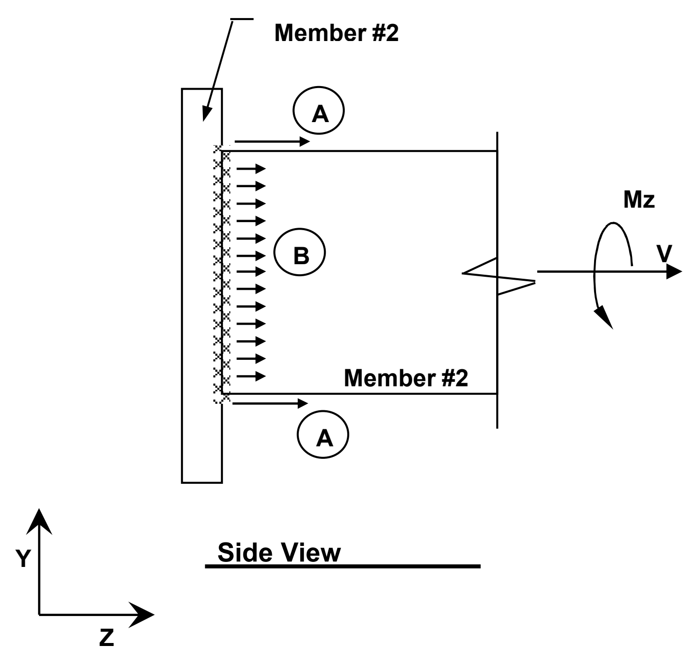
Maximum load in weld = Fact = 7295 lb/in

Allowable load in weld = 0.707*C*0.3*1000*fy = 4640 lb/in

FILLET NO GOOD - USE FULL PEN WELD



Front View



Side View

PRCNC20240010

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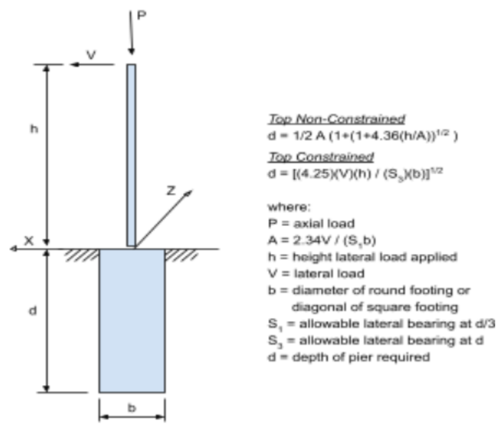
		Engineer:	CF

Code Cycle: 2018 IBC

Check Drilled Piers

Loc All Piers



Gravity Loads (Unfactored)

Dead	Live	Roof Live	Snow
kip	kip	kip	kip
8.94	0.00	15.74	16.55

Lateral Loads (Ultimate)

Wind x	Wind z	Height, h	Seis x	Seis z	Height, h
kip	kip	ft	kip	kip	ft
2.49	0.00	19.00	7.42	0.00	19.00

Drilled Pier Properties

Pier Round/Square?:	R	(R or S)	Tie Bar Yield, F'yt:	60.00	ksi	Use Default Soil?:	Y	(Y or N)	Core Area, Ach:	2,290.22		
Pier Diameter, b:	60.00	in	Sesimic Category:	D		1/2 in Defl OK?:		(Y or N)	Gross Area, Ag:	2,827.43		
Concrete Comp, f'c:	3.00	ksi	Top Constrained?:	Y	(Y or N)	Limit Lat Bearing?:	N	(Y or N)	Pier Diameter, b:	60.00		
Rebar Size:	7		Allow 1.33 Increase?:	N	(Y or N)							
Total # Rebar:	24		Use Overstrength?:	Y	(Y or N)							
Bar clear cover:	3.00	in	Overstrength Factor:	1.25								
Rebar Yield, F'y:	60.00	ksi	Soils Report?:	N	(Y or N)	Alw Lat Bearing:	100	pcf			100	pcf
Min Steel Ratio:	0.50	%				Alw End Bearing:	1500	psf			1500	psf

Total Loads on Piers

Worse Case Axial Loads on Pier - ASD		***Worse Case Loads on Pier - LRFD***			
Pu-asd:	25.48 k	Pu-lrfd:	37.20 k		
Vu-asd-wind-x:	1.50 k	Vu-lrfd-wind-x:	2.49 k	Mu-lrfd-wind-x:	47.39 k*ft
Vu-asd-seis-x:	5.79 k	Vu-lrfd-seis-x:	9.27 k	Mu-lrfd-seis-x:	176.13 k*ft
Vu-asd-wind-z:	0.00 k	Vu-lrfd-wind-z:	0.00 k	Mu-lrfd-wind-z:	0.00 k*ft
Vu-asd-seis-z:	0.00 k	Vu-lrfd-seis-z:	0.00 k	Mu-lrfd-seis-z:	0.00 k*ft

Pier Min Longitudinal Steel

Steel Area, Ast:	14.43 in^2	Percentage of Steel:	0.51 %

OK

Pier Moment and Shear

Pier width, b1:	58.90 in, equiv	Comp Strip, a:	2.88 in	Whit Eq Width:	0.30 in
Mom Cap, Phi*Mn:	1,252.06 k*ft	Shear Cap, Phi*Vn:	170.35 k	Whit Eq depth, d1:	40.00 in
Mom Act, Mu:	176.13 k*ft	Shear Act, Vu:	9.27 k		

OK

Check Sspot Footing

Pier End Area, Ap:	19.63 ft^2	Act Soil Pressure:	1,298 psf	Allow Soil Pressure:	1,500 psf

OK

Summary

Pier Depth:	**10.00 ft**	**Pier Vert Reinforcing: (24) # 7 Rebar**
Pier Diameter:	**60.00 in**	**Pier Ties: #4@ 12" o.c.**

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Check Anchor Bolts

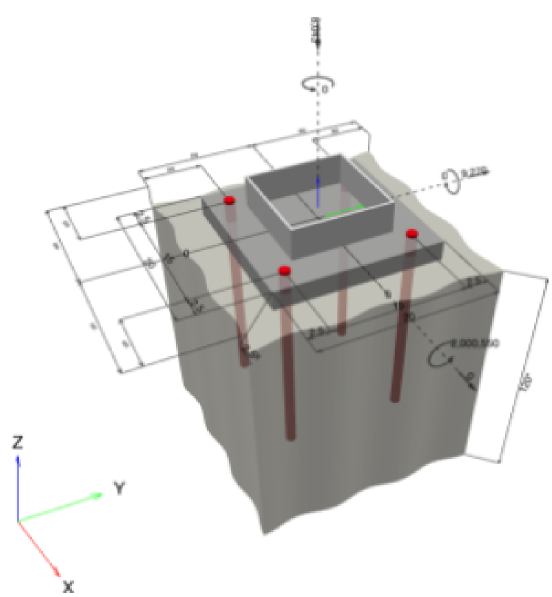
Worse Case Seismic Shear =	7416 lb (Strength)	=	9270 lb (W/ 1.25 Overstrength)
Worse Case Seismic Moment =	133370 lb*ft (Strength)	=	2000550 lb*in (W/ 1.25 Overstrength)
Axial Load =	8043 lb (0.9DL)		

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 105 1 1/4**
Item number:	not available
Effective embedment depth:	h_{ef} = 25.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 2.500 in.
Anchor plate[R] :	l_x x l_y x t = 20.000 in. x 20.000 in. x 2.500 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS12X12X.3125; (L x W x T) = 12.000 in. x 12.000 in. x 0.312 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 120.000 in.
Reinforcement:	tension: present, shear: present; anchor reinforcement: tension, shear edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d)) Shear load: yes (17.10.6.3 (c))

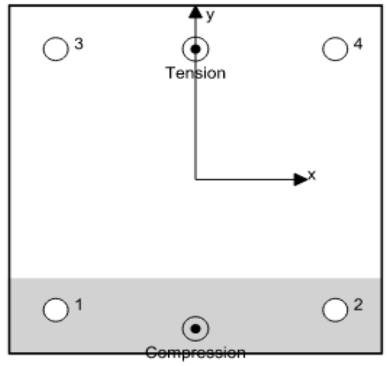
[R] - The anchor calculation is based on a rigid anchor plate assumption.

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



Check Anchor Bolts (Cont)

2 Load case/Resulting anchor forces



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	2,318	0	2,318
2	0	2,318	0	2,318
3	60,074	2,318	0	2,318
4	60,074	2,318	0	2,318

max. concrete compressive strain: 0.69 [‰]
max. concrete compressive stress: 3,002 [psi]
resulting tension force in (x/y)=(0.000/7.500): 120,149 [lb]
resulting compression force in (x/y)=(0.000/-8.576): 128,192 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	60,074	90,844	67	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**[1]	N/A	N/A	N/A	N/A
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)
[1] Tension Anchor Reinforcement has been selected!

3.1 Steel Strength

$N_{sa} = A_{se,N}\, f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.97	125,001

Calculations

N_{sa} [lb]
121,125

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
121,125	0.750	90,844	60,074

3.2 Pullout Strength

$N_{pN} = \psi_{c,p}\, N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8\, A_{brg}\, f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi\ N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	2.24	1.000	3,000

Calculations

N_p [lb]
53,688

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
53,688	0.700	0.750	1.000	28,186	60,074

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Check Anchor Bolts (Cont)

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	2,318	47,239	5	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	9,270	377,634	3	OK
Concrete edge failure in direction **[1]	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

[1] Shear Anchor Reinforcement has been selected!

4.1 Steel Strength

$V_{sa} = 0.6\, A_{se,V}\, f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)

$\phi\, V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.97	125,001

Calculations

V_{sa} [lb]
72,675

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\, V_{sa,eq}$ [lb]	V_{ua} [lb]
72,675	0.650	47,239	2,318

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Check Anchor Bolts (Cont)

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = 16\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{5/3}$ ACI 318-19 Eq. (17.6.2.2.3)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	25.000	0.000	0.000	∞

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	∞	16	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
8,100.00	5,625.00	1.000	1.000	1.000	1.000	187,318

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
539,477	0.700	1.000	1.000	377,634	9,270

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.67	0.049	1.000	60	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$