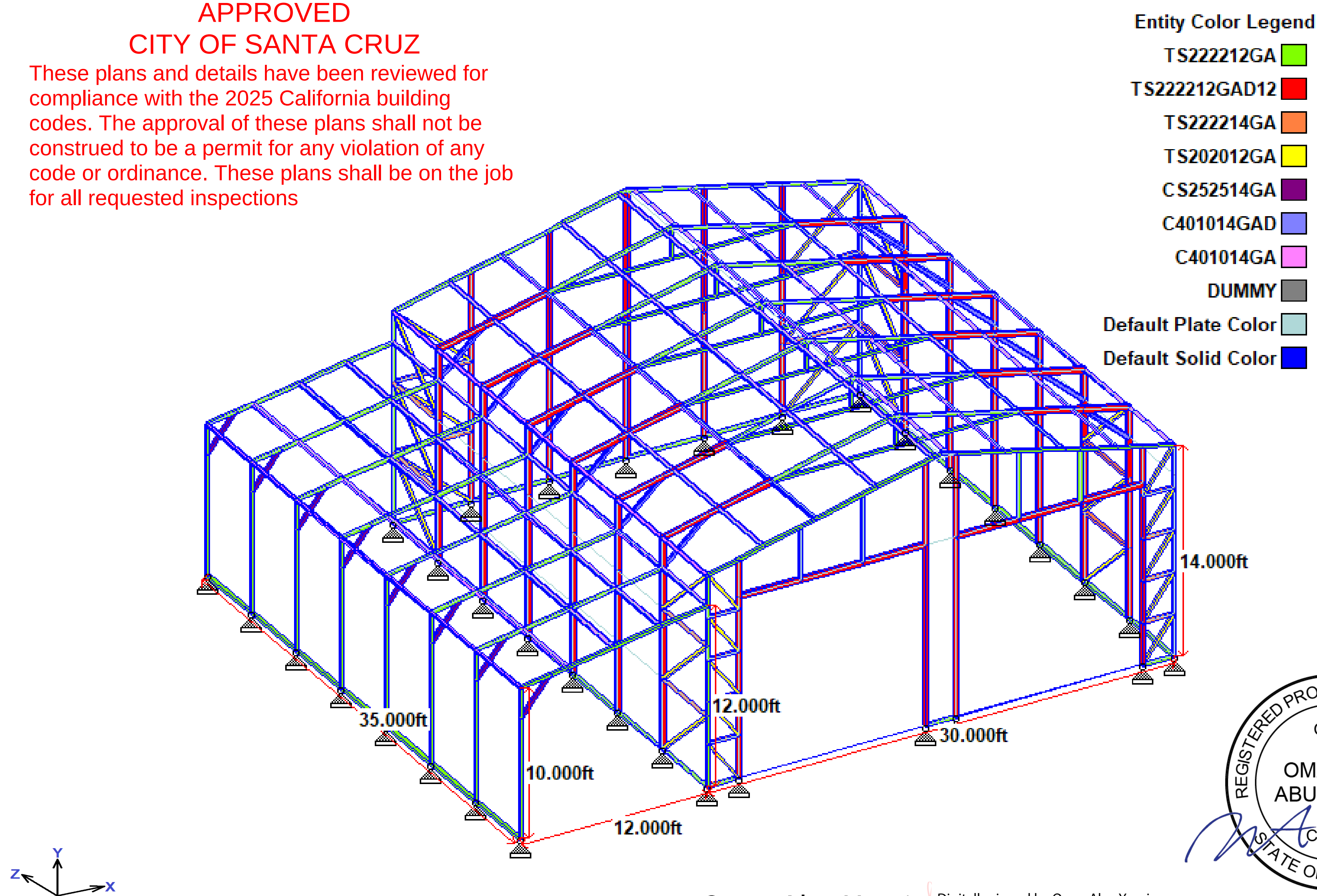
A&A ENGINEERING
CIVIL • STRUCTURAL
Tel: 419-292-1983
6036 Renaissance Place
Toledo, Ohio 43623

APPROVED
CITY OF SANTA CRUZ
These plans and details have been reviewed for compliance with the 2025 California building codes. The approval of these plans shall not be construed to be a permit for any violation of any code or ordinance. These plans shall be on the job for all requested inspections



REGISTERED PROFESSIONAL ENGINEER
CIVIL
OMAR A. ABU-YASEIN
C73389
STATE OF CALIFORNIA

Omar Abu-Yasein Digitally signed by Omar Abu-Yasein
Date: 2025.06.18 08:23:03 -04'00'

Expires: 12/31/2026
Signed: JUN 17 2025

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I Design Summary

Structure Information			Main Bldg		Left Lean-To 1	-	-	-	
Roof Type:				Gable	Monoslope				
Attachment Type Continuous (Shared) Roof, Dropped (Separted) Roof or Isolated Building:				-	Dropped Roof				
Attaches to OR is next to which building first:				-	Main Bldg				
Frame Width (Transverse):	W_b =			30.0	12.0				ft
Base Rail Length (Longitudinal):	L_b =			35.0	35.0				ft
Building Offset from Front to Adjacent Building:	F_o =			-	-				ft
Building Offset from Rear to Adjacent Building:	B_o =			-	-				ft
Eave or Leg Height:	SIDE WALL	LEFT	RIGHT	LEFT	RIGHT				
	H_B =	14.0	14.0	10.0	12.0				ft
Roof Pitch (R_p / 12):	R_p =		3.0		2.0				/ 12
Height of Roof Section:	H_R =		3.75		2.00				ft
Max Structure height:	H_T =		18.13		12.00				ft
Average Structure height:	H_a =		15.88		11.00				ft
Roof Diaphragm Tributary height :	$H_D = (H_R + H_B / 2)$ =		10.75		7.00				ft
Individual Building Square Footage:			1,050		420				sqft
Total Square Footage:			1,470						sqft
No. of Gravity Frames:			8		8				-

Design Criteria	
Prevailing Code:	CBC 2022
Risk Category:	IV
ASCE Standard followed:	7-16

Load Combinations: Allowable Strength Design (ASD)

Strength:

a. $D + (L_r \text{ or } S)$

b. $D + 0.75\ (L_r \text{ or } S) + 0.75\ (0.6W)$

c. $D + 0.75\ (L_r \text{ or } S) + 0.525E_V + 0.525E_H$

d. $D + 0.6W$

e. $D + 0.7E_V + 0.7\ E_H$

f. $0.6D + 0.6W$

g. $0.6D - 0.7E_V + 0.7\ E_H$

Service:

$D + 0.5L_r + W_a$

W_a is based on 10-year M.R.I. wind speed.

II Gravity Load Summary

Dead Loads		**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**		
Individual Self *Framing* Weight (auto-calc from F.E.M.):	w_{sf} =	5.648	2.259				kip	
Roof Collateral Dead Load (super imposed):	d_c =	2.00	2.00				psf	Ref Cover Sheet of Plans
Total Dead Load - pressure (per Structure):	$D = (w_{sf}/\text{Area}) + d_c + d_1 + d_2 + d_3$ =	7.379	7.379				psf	
Total Dead Load - force / Weight (per Structure):	wt = D * Area =	7.748	3.099				kip	
Total Weight (All Structures):	Σwt =	**10.847**					kip	

Live Loads				
Roof Live Load:	L_r =	20.0	psf	

Snow Loads				
Design (Rated) Ground Snow Load:	P_g =	0.0	psf	
Importance Factor:	I_s =	1.2	-	ASCE 1.5-2
Thermal Factor:	C_t =	1.2	-	ASCE 7.3-2
Exposure Factor:	C_e =	1.0	-	ASCE 7.3-1
Slope Factor:	C_s =	1.0	-	ASCE 7.4-1
Flat Roof Snow Load *(ASCE Eq 7.3-1)* :	$P_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot P_g$ =	0.0	psf	
Sloped Roof Load *(ASCE Eq 7.4-1)* :	$P_s = C_s * P_f$ =	0.0	psf	
Design Snow Load:	**$S = \max(P_f, P_s)$ =**	**0.0**	psf	
Governing Vertical Load:	***Since Lr > S , Roof Live Load Controls***			

III Wind Based Enclosure Summary

Wind design parameters & base shear

Design Wind Speed:	V =	103	mph	Refer ASCE Hazards report
10 Year MRI Wind Speed:	V_{MRI} =	63	mph	Refer ASCE Hazards report
Exposure:		C		
Procedure used:		MWFRS, Directional Procedure		
Calculation methodolgy:		Building of all heights		

Building Enclosure parameters			**Main Bldg**		**Left Lean-To 1**		-		-		-	
Building Side Walls		LEFT	RIGHT	-	LEFT	-	-	-	-	-	-	
Wind Exposed Total Wall Length (Segment 1):		35.00	35.00		0.00							ft
Wind Exposed Total Wall Height (Segment 1):		14.00	14.00		0.00							ft
Segment 2 -Total Wall Enclosure Length:		-	-		-							ft
Segment 2 - Total Wall Enclosure Height:		-	-		-							ft
Total Enclosed Wall Area:	A_{WALL_X} =	490.0	490.0		0.0							sqft
Gross Wall Area:	A_g =	490.0	490.0		420.0							sqft
Open Area:	A_o =	0.0	0.0		420.0							sqft
Wall Diaphragm Tributary:	A_{WALL_DX} =	245.0	245.0		-							sqft
Roof Type:			Gable		Monoslope							
Side of Building Roof :		LEFT	RIGHT	-	LEFT	-	-	-	-	-	-	
Gross Roof Area:	A_{ROOF} =	544.5	544.5		425.8							sqft
Diaphragm Tributary (Roof comp.):	A_{ROOF_DX} =	544.5	544.5		425.8							sqft
Diaphragm Tributary (Roof comp.):	A_{ROOF_DZ} =	544.5	544.5		425.8							sqft
Building End Walls		FRONT	BACK	FRONT	BACK	-	-	-	-	-	-	
Gable - Enclosure:		Enclosed	Enclosed	Open	Open							ft
Segment 1 -Total Wall Enclosure Length:		30.00	30.00	0.00	0.00							ft
Segment 1 - Total Wall Enclosure Height:		14.00	14.00	0.00	0.00							ft
Segment 2 -Total Wall Enclosure Length:		-	-	-	-							ft
Segment 2 - Total Wall Enclosure Height:		-	-	-	-							ft
Total Enclosed Wall Area:	A_{WALL_Z} =	481.9	481.9	0.0	0.0							sqft
Gross Wall Area:	A_g =	481.9	481.9	132.0	132.0							sqft
Open Area:	$A_o = A_g - A_{WALL\_Z}$ =	0.0	0.0	-132.0	-132.0							sqft
Wall Diaphragm Tributary:	A_{WALL_DZ} =	271.9	271.9	-	-							sqft

		Main Bldg	**Left Lean-To 1**	-	-	-	
ASCE Enclosure Classification:		Enclosed	Open				ASCE Table 26.13-1

III Transverse X - Direction, Wind Roof Diaphgram & Wind Base Shear Calculations

LEFT MOST WALL & ROOF PRESSURES (LWALL) (LROOF)		Main Bldg		Left Lean-To 1		-	-	-		
Pressure Coefficients:		+Gcpi	-Gcpi	-	-					
Windward Wall pressure from X direction Wind	P_{WW} =	10.38	17.45	-	-				psf	Refer Wind Tables
Windward Diaphragm Shear (Wall comp.):	$WW_{WALL,D} = P_{WW} \times A_{WALL\_DX}$ =	2,543.9	4,275.4	-	-				lbf	
Windward Structure Shear (Wall comp.):	$WW_{WALL} = P_{WW} \times A_{WALL\_X}$ =	5,087.7	8,550.8	-	-				lbf	
Open Wind Cases:		-	-	Case A	Case B					
Open - Clear Flow External Pressures:	Pext =	-	-	-1.99	-4.48				psf	
Open - Obstructed Flow External Presures:	Pext =	-	-	-3.00	-5.28				psf	
Windward Roof pressure (Horz comp) case 1 :	Pwr1 =	-4.09	-2.38	-	-				psf	
Windward Roof pressure (Horz comp) case 2 :	Pwr2 =	-1.63	0.09	-	-				psf	
Windward Diaphragm Shear (Roof comp.) case 1:	$WW_{ROOF1} = P_{WR1}, P_{EXT} \times A_{ROOF\_DX}$ =	-2,227.0	-1,293.7	-1,279.3	-2,249.8				lbf	
Windward Diaphragm Shear (Roof comp.) case 2:	$WW_{ROOF2} = P_{WR2}, P_{EXT} \times A_{ROOF\_DX}$ =	-885.1	48.2	-1,279.3	-2,249.8					
Total Diaphragm Shear (Wall + Roof):	$A = WW_{WALL,D} + \max(WW_{ROOF1}, WW_{ROOF2})$=	1,658.7	4,323.6	-1,279.3	-2,249.8				lbf	
Total Structure Base Shear (Wall + Roof):	$C = WW_{WALL} + \max(WW_{ROOF1}, WW_{ROOF2})$=	4,203	8,599	-1,279	-2,250				lbf	

RIGHT MOST WALL & ROOF PRESSURES (RWALL) (RROOF)		Main Bldg		Left Lean-To 1		-	-	-		
Pressure Coefficients:		+Gcpi	-Gcpi	-	-					
Leeward Wall pressure from X direction Wind	P_{LW} =	12.34	5.27	-	-				psf	Refer Wind Tables
Leeward Diaphragm Shear (Wall comp.):	$LW_{WALL} = P_{LW} \times A_{WALL\_DX}$ =	3,022.4	1,290.8	-	-				lbf	
Leeward Structure Shear (Wall comp.):	$LW_{WALL} = P_{LW} \times A_{WALL\_X}$ =	6,044.8	2,581.7	-	-				lbf	
Open Wind Cases:		-	-	-	-					
Open - Clear Flow External Pressures:	Pext =	-	-	-	-				psf	
Open - Obstructed Flow External Presures:	Pext =	-	-	-	-				psf	
Leeward Roof pressure (Horz comp) :	Plr =	3.02	1.31	-	-				psf	
Leeward Diaphragm Shear (Roof comp.):	$LW_{ROOF} = P_{LR}, P_{EXT} \times A_{ROOF\_DX}$ =	1,645.3	712.0	-	-				lbf	
Total Diaphragm Shear (Wall + Roof):	$B = LW_{WALL} + LW_{ROOF}$ =	4,667.7	2,002.8	-	-				lbf	
Total Structure Base Shear (Wall + Roof):	$D = LW_{WALL} + LW_{ROOF}$ =	7,690.1	3,293.6	-	-				lbf	

SIDEWALL PRESSURES (LWALL & RWALL)		Main Bldg		Left Lean-To 1		-	-	-		
Side Wall (LWALL) pressure from Z direction Wind:	P_{SWL} =	-15.87	-8.80	-	-				psf	
Diaphragm Shear (Wall comp.):	$SWL_{WALL,D} = P_{SWL} \times A_{WALL\_DX}$ =	-3,887.9	-2,156.3	-	-				lbf	
Structure Shear (Wall comp.):	$SWL_{WALL} = P_{SWL} \times A_{WALL\_X}$ =	-7,775.7	-4,312.6	-	-				lbf	
Side Wall (RWALL) pressure from Z direction Wind:	P_{SWR} =	15.87	8.80	-	-				psf	
Diaphragm Shear (Wall comp.):	$SWR_{WALL,D} = P_{SWR} \times A_{WALL\_DX}$ =	3,887.9	2,156.3	-	-				lbf	
Structure Shear (Wall comp.):	$SWR_{WALL} = P_{SWR} \times A_{WALL\_X}$ =	7,775.7	4,312.6	-	-				lbf	
Total Diaphragm Shear (Wall):	$E = SWL_{WALL,D} + SWR_{WALL,D}$ =	0.0	0.0	0.0	0.0				lbf	
Total Structure Shear (Wall):	$F = SWL_{WALL} + SWR_{WALL}$ =	0.0	0.0	0.0	0.0				lbf	

Wind	Individual Shear Load (for Diapghram level):	A + B , E =	6.326	6.326	1.279	2.250	kip
	Design Shear Load (for Diapghram level):	**F_{DWXU} = max (A + B , E) =**		**6.326**		**2.250**	kip
	Individual Base Shear Load:	C + D , F =	11.893	11.893	1.279	2.250	kip
	Design Invidual Base Shear Load:	V_{WX} = max (C + D , F) =		11.893		2.250	kip
	Total Structure Base Shear:	**ΣV_{WXU} =**		**14.143**			kip

III Longitudinal Z - Direction, Wind Roof Diaphgram & Wind Base Shear Calculations

WINDWARD END WALL (FWALL) & ROOF		Main Bldg		Left Lean-To 1		-	-	-		
Pressure Coefficients:		+Gcpi	-Gcpi	-	-					
Wall Windward pressure from Z direction Wind:	P_{WW} =	10.77	17.83	-	-				psf	Refer Wind Tables
Windward Diaphragm Shear (Wall comp.):	$WW_{WALL,D} = P_{WW} \times A_{WALL\_DZ}$ =	2,927.1	4,848.6	-	-				lbf	
Windward Structure Shear (Wall comp.):	$WW_{WALL} = P_{WW} \times A_{WALL\_Z}$ =	5,188.0	8,593.7	-	-				lbf	
Open Wind Cases:		-	-	Case A	Case B					
Open - Clear Flow External Pressures:	Pext =	-	-	-2.34	2.34				psf	
Open - Obstructed Flow External Presures:	Pext =	-	-	-3.51	1.46				psf	
Roof Windward pressure (Horz comp):	Pwr =	-3.69	-1.98	-	-				psf	
Windward Diaphragm Shear (Roof comp.):	$WW_{ROOF} = P_{WR}, P_{EXT} \times A_{ROOF\_DZ}$ =	-2,009.2	-1,075.8	-1,496.0	997.3				lbf	
Total Diaphragm Shear (Wall + Roof):	$A = WW_{WALL,D} + WW_{ROOF}$ =	917.9	3,772.7	-1,496.0	997.3				lbf	
Total Structure Base Shear (Wall + Roof):	$C = WW_{WALL} + WW_{ROOF}$ =	3,178.9	7,517.9	-1,496.0	997.3				lbf	

LEEWARD END WALL (BWALL) & ROOF		Main Bldg		Left Lean-To 1		-	-	-		
Pressure Coefficients:		+Gcpi	-Gcpi	-	-					
Wall Leeward pressure from Z direction Wind:	P_{LW} =	11.76	4.69	-	-				psf	Refer Wind Tables
Windward Diaphragm Shear (Wall comp.):	$LW_{WALL,D} = P_{LW} \times A_{WALL\_DZ}$ =	3,196.5	1,275.0	-	-				lbf	
Windward Structure Shear (Wall comp.):	$LW_{WALL} = PL_{W} \times A_{WALL\_Z}$ =	5,665.5	2,259.8	-	-				lbf	
Open Wind Cases:		-	-	Case A	Case B					
Open - Clear Flow External Pressures:	Pext =	-	-	2.34	-2.34				ft	
Open - Obstructed Flow External Presures:	Pext =	-	-	-2.34	-1.46				lbf	
Roof Windward pressure (Horz comp):	Plr =	3.69	1.98	-	-				lbf	
Windward Diaphragm Shear (Roof comp.):	$LW_{ROOF} = P_{LR}, P_{EXT} \times A_{ROOF\_DZ}$ =	2,009.2	1,075.8	997.3	-997.3				psf	
Total Diaphragm Shear (Wall + Roof):	$B = LW_{WALL,D} + LW_{ROOF}$ =	5,205.6	2,350.8	997.3	-997.3				ft	
Total Structure Base Shear (Wall + Roof):	$D = LW_{WALL} + LW_{ROOF}$ =	7,674.6	3,335.6	997.3	-997.3				lbf	

SIDEWALL PRESSURES (FWALL & BWALL)		Main Bldg		Left Lean-To 1		-	-	-	
Side Wall (FWALL) pressure from X direction Wind:	P_{SWF} =	15.86	8.79	-	-				psf
Diaphragm Shear (Wall comp.):	$SWF_{WALL,D} = P_{SWF} \times A_{WALL\_DZ}$ =	4,311.2	2,389.7	-	-				lbf
Structure Shear (Wall comp.):	$SWF_{WALL} = P_{SWF} \times A_{WALL\_Z}$ =	7,641.2	4,235.6	-	-				lbf
Side Wall (BWALL) pressure from X direction Wind:	P_{SWB} =	-15.86	-8.79	-	-				psf
Diaphragm Shear (Wall comp.):	$SWB_{WALL,D} = P_{SWB} \times A_{WALL\_DZ}$ =	-4,311.2	-2,389.7	-	-				lbf
Structure Shear (Wall comp.):	$SWB_{WALL} = P_{SWB} \times A_{WALL\_Z}$ =	-7,641.2	-4,235.6	-	-				lbf
Total Diaphragm Shear (Wall):	$E = SWF_{WALL,D} + SWB_{WALL,D}$ =	0.0	0.0	0.0	0.0				lbf
Total Structure Shear (Wall):	$F = SWF_{WALL} + SWB_{WALL}$ =	0.0	0.0	0.0	0.0				lbf

Wind	Individual Shear Load (for Diapghram level):	A + B , E =	6.124	6.124	0.499	0.000	kip
	Design Shear Load (for Diapghram level):	**F_{DWZU} = max (A + B , E) =**		**6.124**		**0.499**	kip
	Individual Base Shear Load:	C + D , F =	10.853	10.853	0.499	0.000	kip
	Design Invidual Base Shear Load:	V_{WZ} = max (C + D , F) =		10.853		0.499	kip
	Total Structure Base Shear:	**ΣV_{WZU} =**		**11.352**			kip

IV Seismic Design Parameters

Equivalent Lateral Force (ELF) Procedure - Seismic design parameters

Seismic Design Category :	SDC =	D	ASCE 7 Sec 11.4.8
Site Class :		D	ASCE 7 Table 1613.5.2
Spectral Response accel. at short periods :	**S_s =**	1.609	ASCE Figure 22-1
Spectral response accel. at short periods :	**S_1 =**	0.611	ASCE Figure 22-2
Site Coefficient :	F_a =	1.000	ASCE Table 11.4-1
Long-Period Transition periods :	T_L =	12	ASCE Figure 22-14~17
Importance Factor:	I_e =	1.5	ASCE Eq 11.4-3
Long-Period Site Coefficient:	F_v =	1.700	ASCE 7 Table 11.4-2
MCER Spectral Response at 1s :	$S_{M1} = F_v * S_1$ =	1.560	ASCE Eq 11.4-1
MCER Spectral Response at short periods :	$S_{MS} = F_a * S_S$ =	1.609	ASCE Eq 11.4-2
Adjusted spectral response acceleration:	**S_{DS} =**	**1.073**	ASCE Eq 11.4-3
Design spectral response acceleration :	**S_{D1} =**	**1.040**	ASCE Eq 11.4-4
Max S_{DS} :	if $S_{DS} > 1$, $S_{DS}' = \max(1.0, 70\%\ S_{DS})$ =	1.000	ASCE 7 Sec 12.8.1.3

Redundancy factor Calculations:

Transverse Dir	Total No. of LRFS frames / systems:	N_{LFRS} =	16		(refer Sections below)
	Reduced Story Strength after removal of 1 LFRS (per ASCE 12.3.4.2 (a)) =		93.8 % > 33 %		ASCE Table 12.3-3
	Redundancy factor:	**ρ_T =**	**1.0**	*(ASCE 12.3.4.2 Clause (a) - applies)*	ASCE 7 Sec 12.3.4.2
Longitudinal Dir	Total No. of LRFS frames / systems:	N_{LFRS} =	4		(refer Sections below)
	Reduced Story Strength after removal of 1 LFRS (per ASCE 12.3.4.2 (a)) =		75.0 % > 33 %		ASCE Table 12.3-3
	Redundancy factor:	**ρ_L =**	**1.0**	*(ASCE 12.3.4.2 Clause (a) - applies)*	ASCE 7 Sec 12.3.4.2

V Effective Seismic Weight & Seismic Load Summary

Effective Seismic Weight		**Main Bldg**	**Left Lean-To 1**	-	-	-		
Structure Weight:	wt =	7.748	3.099				kip	
20% of Snow Load if 'Flat Roof Snow' is over 30 psf:	s = 20% of S =	Not Req	Not Req				kip	ASCE 7 12.7.2 (4)
Structure Weight Tributary to Roof Level	$w_1 = wt * (H_D / H_T) + s$ =	4.595	1.808				kip	
Structure Weight Tributary to Roof Level Diaphragm:	$w_{p1} = w_1$ - parallel walls) =	4.595	1.808				kip	(assumed conserv.)
Structure Weight Tributary to Second Level	$w_2 = (wt + s)* (H_D / H_T)$ =	-	-				kip	
Structure Weight Tributary to Second Level Diaphragm:	$w_{p2} = w_2$ - parallel walls) =	-	-				kip	(assumed conserv.)
Effective Seismic Weight:	**$W_s = (w_1 + w_2)$ =**	**4.595**	**1.808**				kip	ASCE 7 Section 12.7.2

Governing Vertical Seismic Effects (E_V)			**Main Bldg**	**Left Lean-To 1**	-	-	-		
Vertical Seismic Force:		$E_V = 0.2\ S_{DS} * W_S$ =	0.919	0.362				kip	ASCE Sec 12.4-4a
Seismic	**Total Vertical (Y) Structure Base Shear Load:**	ΣE_V =	**1.281**					kip	

V Seismic Load Summary CONT'D : Longitudinal Z - Direction

Z-Dir, Longitudinal Equivalent Lateral Force (ELF) Procedure Design Coefficients		**Main Bldg**			**Left Lean-To 1**		**-**		**-**		**-**	
Building Wall / Frame location:	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	-	LEFT	-	-	-	-	-	-	
Seismic Force Resisting System(s) implemented:	S.F.R.S.	Steel S.W.	Steel S.W.		N.D.F.S.							ASCE 7 Table 12.2-1
Response Modification Factor (for LRFS design - ASCE 12.2.3.3 ex):	$R =$	7.00	7.00		3.00							ASCE 7 Table 12.2-1
Overstrength co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	$\Omega =$	2.50	2.50		2.50							ASCE 7 Table 12.2-1
Deflection Amplification co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	$C_d =$	4.50	4.50		3.00							ASCE 7 Table 12.2-1

Z-Dir, Longitudinal ELF Procedure Diaphragm Design Coefficients & Base Shear (E_{HZ})			**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**		
Response Modification Factor (for Diaphragm design - ASCE 12.2.3.3 ex):		$R' = \min(R) =$	7.00	3.00					ASCE 7 Table 12.2-1
Governing Seismic Force-Resisting System(s) :		S.F.R.S.	Steel S.W.	N.D.F.S.	-	-	-		ASCE 7 Table 12.2-1
Overstrength co-efficient (for Diaphragm design - ASCE 12.2.3.3 ex):		$\Omega =$	2.50	2.50					ASCE 7 Table 12.2-1
Deflection Amplification co-efficient (for Diaphragm design - ASCE 12.2.3.3 ex):		$C_d =$	4.50	3.00					ASCE 7 Table 12.2-1
Approximate fundamental period coefficient 1:		$C_t =$	0.020	0.020					ASCE Table 12.8-2
Approximate fundamental period coefficient 2:		$x =$	0.750	0.750					ASCE Table 12.8-2
Approximate fundamental period:		$T_a = C_t * h_n^x =$	0.159	0.121					ASCE Eq 12.8-7
Seismic Response Co-efficient:	Base C_s value	$C_{s,1} = S_{DS}' / (R / I_e) =$	0.214	0.500					ASCE Eq 12.8-2
	Max C_s Value	$C_{s,max} = S_{D1} / T * (R / I_e) =$	0.772	0.251					ASCE Eq 12.8-3
	Min. Case 1	$C_{s,min1} = 0.044\ S_{DS}'\ I_e =$	0.066	0.066					ASCE Eq 12.8-5
	Min. Case 2	$C_{s,min2} = 0.5\ S_1 / (R / I_e) =$	0.065	0.153					ASCE Eq 12.8-6
	Design Value	$C_s = \min(C_{s,1}, C_{s,max})\ \&\ \max(C_{s,min1}, C_{s,min2}, 0.01) =$	0.214	0.251					ASCE Sec 12.8.1.1
	Total Effective Seismic Weight Considered:	$W_s = w_1 + w_2 =$	4.595	1.808				kip	ASCE 7 Section 12.7.2
Seismic	**Base Shear:**	$V_s = C_s * W_s =$	**0.985**	**0.454**				**kip**	ASCE 7 Eq 12.8-1
	Exponent Related to Structure period:	$k =$	1.00	1.00					ASCE Sec 12.8.3
	Diaphragm Tributary Height at Roof:	$h^k_1 =$	10.75	7.00				ft	ASCE Sec 12.8.3
	Diaphragm Tributary Height at Second Floor:	$h^k_2 =$	-	-				ft	ASCE Sec 12.8.1.1
	Vertical Distritbution Factor	$C_{v1} = w_1 * h^k_1 / (w_1 * h^k_1 + w_2 * h^k_2) =$	1.00	1.00					ASCE 7 Eq 12.8-12
	Lateral (Z-dir) Structure Force:	$F_{1,Z} = C_{v1} * V_s =$	0.985	0.454				kip	ASCE 7 Eq 12.8-11
	Total Lateral (Z) Structure Base Shear Load:	$\Sigma E_Z =$	1.439					**kip**	

SFRS Legend / Notes:

N.D.F.S. = Steel Systems not specifically detailed for Seismic Resistance (Item H)
O.C.B.F = Steel Ordinary Concentrically Braced Frames (Item B-3)
Steel S.W. = Light-frame Walls Sheathed with Steel Sheets (Item B-23)
O.M.F = Steel Ordinary Moment Frame (Item C-4)
- Indicates all lateral loads of attached structure are transferred to, and resisted by Main Bldg

V Seismic Load Summary CONT'D : Transverse X - Direction

X-Dir, Transverse L.F.R.S. Design Coefficients		**Main Bldg**		**Left Lean-To 1**			**-**		**-**		**-**	
Building Wall / Frame location: (X-DIR) (TRANSVERSE) (PERIMETER)		FRONT	BACK	FRONT	BACK							
Seismic Force-Resisting System(s) implemented:	S.F.R.S.	Steel S.W.	Steel S.W.	O.M.F	O.M.F	-	-	-	-	-	-	ASCE 7 Table 12.2-1
Response Modification Factor (for LRFS design - ASCE 12.2.3.3 ex):	R_1 =	7.00	7.00	3.50	3.50							ASCE 7 Table 12.2-1
Overstrength co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	Ω_1 =	2.50	2.50	3.00	3.00							ASCE 7 Table 12.2-1
Deflection Amplification co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	C_{d1} =	4.50	4.50	3.00	3.00							ASCE 7 Table 12.2-1
Building Wall / Frame location: (X-DIR) (TRANSVERSE) (INTERIOR)		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-	
Seismic Force-Resisting System(s) implemented:	S.F.R.S.	-	O.M.F	-	O.M.F	-	-	-	-	-	-	ASCE 7 Table 12.2-1
Response Modification Factor (for LRFS design - ASCE 12.2.3.3 ex):	R_2 =		3.50		3.50							ASCE 7 Table 12.2-1
Overstrength co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	Ω_2 =		3.00		3.00							ASCE 7 Table 12.2-1
Deflection Amplification co-efficient (for LRFS design - ASCE 12.2.3.3 ex):	C_{d2} =		3.00		3.00							ASCE 7 Table 12.2-1
Number of Internal Wall / Frames with LFRS elements:	n_X =		6		6							

X-Dir, Transverse Diaphragm Design Coefficients & Base Shear (E_{HX})			**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**		
	Response Modification Factor (for Diaphragm design - ASCE 12.2.3.3 ex):	$R = \min(R_1, R_2)$ =	3.50	3.50					ASCE 7 Table 12.2-1
	Governing Seismic Force-Resisting System(s) :	S.F.R.S.	O.M.F	O.M.F	-	-	-		ASCE 7 Table 12.2-1
	Overstrength co-efficient (for Diaphragm design - ASCE 12.2.3.3 ex):	Ω =	3.00	3.00					ASCE 7 Table 12.2-1
	Deflection Amplification co-efficient (for Diaphragm design - ASCE 12.2.3.3 ex):	C_d =	3.00	3.00					ASCE 7 Table 12.2-1
	Approximate fundamental period coefficient 1:	C_t =	0.028	0.028					ASCE Table 12.8-2
	Approximate fundamental period coefficient 2:	x =	0.800	0.800					ASCE Table 12.8-2
	Approximate fundamental period:	$T_a = C_t * h_n^x$ =	0.256	0.191					ASCE Eq 12.8-7
Seismic Response Co-efficient:	Base C_s value	$C_{s,1} = S_{DS}' / (R / I_e)$ =	0.429	0.429					ASCE Eq 12.8-2
	Max C_s Value	$C_{s,max} = S_{D1} / T * (R / I_e)$ =	0.620	0.463					ASCE Eq 12.8-3
	Min. Case 1	$C_{s,min1} = 0.044\ S_{DS}'\ I_e$ =	0.066	0.066					ASCE Eq 12.8-5
	Min. Case 2	$C_{s,min2} = 0.5\ S_1 / (R / I_e)$ =	0.131	0.131					ASCE Eq 12.8-6
	Design Value	$C_s = \min(C_{s,1}, C_{s,max})$ & $\max(C_{s,min1}, C_{s,min2}, 0.01)$ =	0.429	0.429					ASCE Sec 12.8.1.1
	Total Effective Seismic Weight Considered:	$W_s = w_1 + w_2$ =	4.595	1.808				kip	ASCE 7 Section 12.7.2
Seismic	**Base Shear:**	$V_s = C_s * W_s$ =	**1.969**	**0.775**				**kip**	ASCE 7 Eq 12.8-1
	Exponent Related to Structure period:	k =	1.00	1.00					ASCE Sec 12.8.3
	Diaphragm Tributary Height at Roof:	h^k_1 =	10.75	7.00				ft	ASCE Sec 12.8.3
	Diaphragm Tributary Height at Second Floor:	h^k_2 =	-	-				ft	ASCE Sec 12.8.1.1
	Vertical Distritbution Factor	$C_{v1} = w_1 * h^k_1 / (w_1 * h^k_1 + w_2 * h^k_2)$ =	1.00	1.00					ASCE 7 Eq 12.8-12
	Lateral (X-dir) Structure Force:	$F_{1,X} = C_{v1} * V_s$ =	**1.969**	**0.775**				kip	ASCE 7 Eq 12.8-11
	Total Lateral (X) Structure Base Shear Load:	ΣE_X =	**2.744**					**kip**	

SFRS Legend / Notes:

N.D.F.S. = Steel Systems not specifically detailed for Seismic Resistance (Item H)
O.C.B.F = Steel Ordinary Concentrically Braced Frames (Item B-3)
Steel S.W. = Light-frame Walls Sheathed with Steel Sheets (Item B-23)
O.M.F = Steel Ordinary Moment Frame (Item C-4)
- Indicates L.F.R.S. is part of, or is attached to, and is resisted by S.F.R.S. of Main Bldg

VI Roof Diaphragm Forces : Transverse X - Direction

Diaphragm Type (Member):			**Flexible (Corrugated Steel Roof Deck)**										-	ASCE 7 Sec 12.3.1.1
Diaphgram Calculations				**Main Bldg**		**Left Lean-To 1**		**-**		**-**		**-**		
Transverse Dir - Diaphragm Span:		$L_b =$		35.00		35.00							ft	
Roof Sub-Diaphragm Supports:		$n_X =$		8		8								
Wind	Weighted Base Shear Load (for Diapghram level):	$F_{DWXU} =$		6.326		2.250							kip	Refer Wind Base Shear Calc
	ASD Diaphragm Weighted Base Load	$F_{DWX} = 0.6 * F_{DWXU} =$		3.796		1.350							kip	
	ASD Diaphragm Linear Load	$U_{DWX} = F_{DWX} / L_b =$		0.108		0.039							klf	
	Roof Sub-Diaphragm Moment:	$M_{DWX} = \alpha * U_{DWX} * (L_b / n_x - 1)^2 =$		0.290		0.103							kip-ft	
	Direction / Location	**(X-DIR) (TRANSVERSE) (PERIMETER)**	**FRONT**	**BACK**	**FRONT**	**BACK**								
	Roof Sub-Diaphragm End Reaction:	$R_{DWXE} = \alpha * U_{DWZ} * (L_b / n_x - 1) =$	0.214	0.214	0.076	0.076							kip	
	Direction / Location	**(X-DIR) (TRANSVERSE) (INTERIOR)**	**-**	**TYP FRAME**	**-**	**TYP FRAME**	**-**	**-**	**-**	**-**	**-**	**-**		
	Interior Roof Sub-Diaphragm Reaction:	$R_{DWXE} = b * U_{DWZ} * (L_b / n_x - 1) =$		0.615		0.219							kip	
Seismic	Lateral (X-dir) Structure Force:	$F_{1,X} = C_{v1} * V_s =$		1.969		0.775							kip	ASCE 7 Eq 12.8-1
	Diaphragm Tributary Load at Roof level:	$F_{D1,SXU} = (F_{1,X} / w_1) * w_{p1} =$		1.969		0.775							kip	ASCE 7 Eq 12.10.1
	Diaphragm Tributary Load at 2nd Level:	$F_{D1+2,SXU} = (F_{1,X} + F_{2,X} / w_1) * w_{p2} =$		-		-							kip	ASCE 7 Eq 12.10.1
	Diaphragm Min Load:	$F_{SXU,MIN} = 0.2 * S_{DS} * I_e * w_{p1} =$		1.379		0.542							kip	ASCE 7 Eq 12.10-2
	Diaphragm Max Load:	$F_{SXU,MAX} = 0.4 * S_{DS} * I_e * w_{p1} =$		2.757		1.085							kip	ASCE 7 Eq 12.10-3
	Diaphragm Design Load:	$F_{DSXU} = \min(F_{SXU,MAX}, (F_{D1,SXU}, F_{SXU,MIN}) =$		1.969		0.775							kip	ASCE 7 Sec 12.10.1.1
	ASD Diaphragm Load:	$F_{DSX} = 0.7 * F_{DSXU} =$		1.379		0.542							kip	
	ASD Diaphragm Linear Load	$U_{DSX} = F_{DSX} / L_b =$		0.039		0.015							klf	(along chord)
	Roof Sub-Diaphragm Moment:	$M_{DSX} = \alpha * U_{DSX} * (L_b / n_x - 1)^2 =$		0.105		0.041							kip-ft	(chord moment)
	Direction / Location	**(X-DIR) (TRANSVERSE) (PERIMETER)**	**FRONT**	**BACK**	**FRONT**	**BACK**								
	Roof Sub-Diaphragm End Reaction:	$R_{DSXE} = \alpha * U_{DSX} * (L_b / n_x - 1) =$	0.078	0.078	0.031	0.031							kip	
	Direction / Location	**(X-DIR) (TRANSVERSE) (INTERIOR)**	**-**	**TYP FRAME**	**-**	**TYP FRAME**	**-**	**-**	**-**	**-**	**-**	**-**		
	Interior Roof Sub-Diaphragm Reaction:	$R_{DSXI} = b * U_{DWX} * (L_b / n_x - 1) =$		0.223		0.088							kip	

VII Roof Diaphragm Forces : Longitudinal Z - Direction

Diaphgram Calculations				**Main Bldg**		**Left Lean-To 1**	**-**	**-**	**-**		
Longitudinal Diaphragm Span:		W_b =		30.00		12.00				ft	
Roof Sub-Diaphragm Supports:		n_Z =		2		2					
Wind	Weighted Base Shear Load (for Diapghram level):	F_{DWZU} =		6.124		0.499				kip	Refer Wind Base Shear Calc
	ASD Diaphragm Weighted Base Load	$F_{DWZ} = 0.6 * F_{DWZU}$ =		3.674		0.299				kip	
	ASD Diaphragm Linear Load	$U_{DWZ} = F_{DWZ} / W_b$ =		0.122		0.025				klf	(along chord)
	Roof Sub-Diaphragm Moment:	$M_{DWZ} = \alpha * U_{DWZ} * (W_b / n_Z - 1)^2$ =		13.778		**0.449**				kip-ft	(chord moment)
	Direction / Location	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	RIGHT	LEFT					
	Roof Sub-Diaphragm End Reaction:	$R_{DWXE} = \alpha * U_{DWZ} * (W_b / n_x - 1)$ =	1.837	1.837	0.150	0.150				kip	
Seismic	Lateral (Z-dir) Structure Force:	$F_{1,Z} = C_{v1} * V_s$ =		0.985		0.454				kip	ASCE 7 Eq 12.8-1
	Diaphragm Tributary Load at Roof level:	$F_{D1,SZU} = (F_{1,Z} / w_1) * w_{p1}$ =		0.985		0.454				kip	ASCE 7 Eq 12.10.1
	Diaphragm Tributary Load at 2nd Level:	$F_{D1+2,SZU} = (F_{1,Z} + F_{2,Z} / w_1) * w_{p2}$ =		-		-				kip	ASCE 7 Eq 12.10.1
	Diaphragm Min Load:	$F_{SZU,MIN} = 0.2 * S_{DS} * I_e * w_{p1}$ =		1.379		0.542				kip	ASCE 7 Eq 12.10-2
	Diaphragm Max Load:	$F_{SZU,MAX} = 0.4 * S_{DS} * I_e * w_{p1}$ =		2.757		1.085				kip	ASCE 7 Eq 12.10-3
	Diaphragm Design Load:	$F_{DSZU} = \min (F_{SZU,MAX}, (F_{D1,SZU}, F_{SZU,MIN})$ =		1.379		0.542				kip	ASCE 7 Sec 12.10.1.1
	ASD Diaphragm Load:	$F_{DSZ} = 0.7 * F_{DSZU}$ =		0.965		0.380				kip	
	ASD Diaphragm Linear Load	$U_{DSZ} = F_{DSZ} / W_b$ =		0.032		0.032				klf	(along chord)
	Roof Sub-Diaphragm Moment:	$M_{DSZ} = \alpha * U_{DSWZ} * (W_b / n_Z - 1)^2$ =		3.619		0.569				kip-ft	(chord moment)
	Direction / Location	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	RIGHT	LEFT					
	Roof Sub-Diaphragm End Reaction:	$R_{DSZE} = \alpha * U_{DSZ} * (W_b / n_x - 1)$ =	0.483	0.483	0.190	0.190				kip	

VIII Governing Chord Design Forces

X-Dir, Transverse Applied Force				Main Bldg	Left Lean-To 1	-	-	-	
	Chord Location:	(Z-DIR) (LONGITUDINAL) (PERIMETER)		LEFT / RIGHT	LEFT / RIGHT				
	Transverse Dir - Diaphragm Depth:		W_b =	30.00	12.00				ft
Wind	ASD Diaphragm Linear Load		U_{DWX} =	0.108	0.039				klf
	Roof Sub-Diaphragm Moment:		M_{DWX} =	0.290	0.103				kip-ft
	Chord Axial Force:		$CH_{DWZ} = M_{DWX} / W_b$ =	0.010	0.009				kip
Seismic	ASD Diaphragm Linear Load		U_{DSX} =	0.039	0.015				klf
	Roof Sub-Diaphragm Moment:		M_{DSX} =	0.105	0.041				kip-ft
	Chord Axial Force:		$CH_{DSZ} = M_{DSX} / W_b$ =	0.004	0.003				kip

Z-Dir, Longitudinal Applied Force				Main Bldg	Left Lean-To 1	-	-	-	
	Chord Location:	(X-DIR) (TRANSVERSE) (PERIMETER)		FRONT / BACK	FRONT / BACK				
	Longitudinal Diaphragm Depth:		L_b =	35.00	35.00				ft
Wind	ASD Diaphragm Linear Load		U_{DWZ} =	0.122	0.025				klf
	Roof Sub-Diaphragm Moment:		M_{DWZ} =	13.778	0.449				kip-ft
	Chord Axial Force:		$CH_{DWX} = M_{DWZ} / L_b$ =	0.394	0.013				kip
Seismic	ASD Diaphragm Linear Load		U_{DSZ} =	0.032	0.032				klf
	Roof Sub-Diaphragm Moment:		M_{DSZ} =	3.619	0.569				kip-ft
	Chord Axial Force:		$CH_{DSX} = M_{DSZ} / L_b$ =	0.103	0.016				kip

IX Governing Collector & LFRS Design Forces : Transverse X - Direction

X-Dir, Transverse Applied Force			**Main Bldg**		**Left Lean-To 1**			**-**		**-**		**-**		
	Collector & LFRS Location:	(X-DIR) (TRANSVERSE) (PERIMETER)	FRONT	BACK	FRONT	BACK								
	Transverse Dir - Diaphragm Depth:	W_b =	30.00	30.00	12.00	12.00							ft	
	LFRS system at Exterior Wall	S.F.R.S. =	Steel S.W.	Steel S.W.	O.M.F	O.M.F	-	-	-	-	-	-	-	
	No. of LRFS:	N_{LFRS} =	1	1	1	1							-	
	No. of Collectors:	N_{CL} =	1	1	-	-							-	
	Average Length of Collector(s):	L_{CL} =	24.00	0.00	-	-							ft	
Wind	Roof Sub-Diaphragm End Reaction:	R_{DWXE} =	0.214	0.214	0.076	0.076							kip	(calculated earlier)
	Roof Sub-Diaphragm Linear Drag:	$V_{DWXE} = R_{DWXE} / W_b$ =	0.007	0.007	0.006	0.006							klf	
	Transferred out to **Perimeter Frames** of Attached Structure	R_{DWXT1} =	-	-	-	-							kip	
	Transferred out to **Interior frames** of Attached Structure	R_{DWXT2} =	-	-	-	-							kip	
	Reaction(s) **Received** from Attached Structure(s):	$R_{DWXA} = \Sigma R_{DWXT1}$ =	-	-	-	-							kip	
LFRS-1	Design Force resisted:	$F_{LFRS} = (\% \, r) * R_{DWXP}$ =	0.214	0.214	0.076	0.076							kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-0.043	-0.214	0.000	0.000							kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS}$ =	0.171	0.000	0.076	0.076							kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% \, r) * R_{DWXA}$ =	-	-	-	-							kip	
	Total Forces Resisted by LFRS 1:	$\Sigma F_{LFRS\text{-}W}$ =	**0.214**	**0.214**	**0.076**	**0.076**							**kip**	
COLL-1	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-0.171	0.000	-	-							kip	
LFRS-2	Design Force resisted:	$F_{LFRS} = (\% \, r) * R_{DWXP}$ =	-	-	-	-							kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% \, r) * R_{DWXA}$ =	-	-	-	-							kip	
	Total Forces Resisted by LFRS 2:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**	
COLL-2	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip	
LFRS-3	Design Force resisted:	$F_{LFRS} = (\% \, r) * R_{DWXP}$ =	-	-	-	-							kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% \, r) * R_{DWXA}$ =	-	-	-	-							kip	
	Total Forces Resisted by LFRS 3:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**	
COLL-3	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip	
LFRS-4	Design Force resisted:	$F_{LFRS} = (\% \, r) * R_{DWXP}$ =	-	-	-	-							kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% \, r) * R_{DWXA}$ =	-	-	-	-							kip	
	Total Forces Resisted by LFRS 4:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**	
COLL-4	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip	
LFRS-5	Design Force resisted:	$F_{LFRS} = (\% \, r) * R_{DWXP}$ =	-	-	-	-							kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% \, r) * R_{DWXA}$ =	-	-	-	-							kip	
	Total Forces Resisted by LFRS 5:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**	
All COLL	**Axial Design Forces**	$CL_{DWXP1} = \max(N_{CL}, D_{CL})$ =	**0.171**	**0.000**	**-**	**-**							**kip**	

IX Governing Collector & LFRS Design Forces : Transverse X - Direction CONT'D

X-Dir, Transverse Applied Force			Main Bldg		Left Lean-To 1		-	-	-		
	Collector & LFRS Location:	(X-DIR) (TRANSVERSE) (PERIMETER)	FRONT	BACK	FRONT	BACK					
Seismic	Roof Sub-Diaphragm End Reaction:	R_{DSXE} =	0.078	0.078	0.031	0.031				kip	(calculated earlier)
	Perimeter Roof Sub-Diaphragm Linear Drag:	$V_{DSXP} = R_{DSXP} / W_b$ =	0.003	0.003	0.003	0.003				klf	
	Transferred out to **Perimeter Frames** of Attached Structure	R_{DSXT1} =	-	-	-	-				kip	
	Transferred out to **Interior frames** of Attached Structure	R_{DSXT2} =	-	-	-	-				kip	
	Reaction(s) **Received** from Attached Structure(s):	$R_{DSXA} = \Sigma R_{DXST1}$ =	-	-	-	-				kip	
LFRS-1	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	0.078	0.078	0.031	0.031				kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-0.016	-0.078	0.000	0.000				kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS}$ =	0.062	0.000	0.031	0.031				kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip	
	Total Forces Resisted by LFRS 1:	$\Sigma F_{LFRS\text{-}W}$ =	**0.078**	**0.078**	**0.031**	**0.031**				**kip**	
COLL-1	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-0.062	0.000	-	-				kip	
LFRS-2	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip	
	Total Forces Resisted by LFRS 2:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**				**kip**	
COLL-2	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip	
LFRS-3	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip	
	Total Forces Resisted by LFRS 3:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**				**kip**	
COLL-3	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip	
LFRS-4	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip	
	Total Forces Resisted by LFRS 4:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**				**kip**	
COLL-4	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip	
LFRS-5	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip	
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip	
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip	
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip	
	Total Forces Resisted by LFRS 5:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**				**kip**	
All COLL	**Axial Design Forces**	$CL_{DWXP1} = \max(N_{CL}, D_{CL})$ =	**0.062**	**0.000**	**-**	**-**				**kip**	

IX Governing Collector & LFRS Design Forces : Transverse X - Direction CONT'D

X-Dir, Transverse Applied Force			**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**	
	Collector & LFRS Location:	(X-DIR) (TRANSVERSE) (INTERIOR)	TYP FRAME	TYP FRAME	-	-	-	
	Transverse Dir - Diaphragm Depth:	W_b =	30.00	12.00				ft
	LFRS system at Interior:	S.F.R.S. =	O.M.F	O.M.F	-	-	-	-
	No. of LRFS:	N_{LFRS} =	6	6				-
	No. of Collectors:	N_{CL} =	-	-				-
	Average Length of Collector(s):	L_{CL} =	-	-				ft
Wind	Interior Roof Sub-Diaphragm Reaction:	R_{DWXE} =	0.615	0.219				kip
	Reaction(s) **Received** from Attached Structure(s):	$R_{DWXA} = \Sigma R_{DWXT2}$ =	-	-				kip
	Interior Roof Sub-Diaphragm Reaction:	$R_{DWXI} = R_{DWXE} + R_{DWXA}$ =	0.615	0.219				kip
	Interior Roof Sub-Diaphragm Linear Drag:	$V_{DWXI} = R_{DWXI} / W_b$ =	0.020	0.018				klf
LFRS-1	Design Force resisted:	F_{LFRS} = (% r) * R_{DWXP} =	**0.615**	**0.219**				kip
Seismic	Interior Roof Sub-Diaphragm Reaction:	R_{DSXI} =	0.223	0.088				kip
	Reaction(s) **Received** from Attached Structure(s):	$R_{DSXA} = \Sigma R_{DXST2}$ =	-	-				kip
	Interior Roof Sub-Diaphragm Reaction:	$R_{DWXI} = R_{DWXE} + R_{DWXA}$ =	0.223	0.088				kip
	Interior Roof Sub-Diaphragm Linear Drag:	$V_{DSXI} = R_{DSXI} / W_b$ =	0.007	0.007				klf
LFRS-1	Design Force resisted:	F_{LFRS} = (% r) * R_{DWXP} =	**0.223**	**0.088**				kip

X Governing Collector & LFRS Design Forces : Longitudinal Z - Direction

Z-Dir, Longitudinal Applied Force				**Main Bldg**		**Left Lean-To 1**		**-**		**-**		**-**	
	Collector & LFRS Location:	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	RIGHT	LEFT							
	Transverse Dir - Diaphragm Depth:	L_b =	35.00	35.00	35.00	35.00							ft
	LFRS system at Exterior Wall	S.F.R.S. =	Steel S.W.	Steel S.W.	(shared)	N.D.F.S.	-	-	-	-	-	-	-
	No. of LRFS:	N_{LFRS} =	1	1	1	1							-
	No. of Collectors:	N_{CL} =	1	1	1	1							-
	Average Length of Collector(s):	L_{CL} =	0.00	3.00	0.00	4.38							ft
Wind	Roof Sub-Diaphragm End Reaction:	R_{DWZE} =	1.837	1.837	0.150	0.150							kip
	Roof Sub-Diaphragm Linear Drag:	$V_{DWZ} = R_{DWZ} / L_b$ =	0.052	0.052	0.004	0.004							klf
	Transferred out to **Shared Wall** of Attached Structure	R_{DWZT} =	-	-	**-0.150**	-							kip
	Reaction(s) **Received** from Attached Structure(s):	R_{DWZA} =	**+0.150**	-	-	-							kip
LFRS-1	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	1.837	1.837	0.150	0.150							kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-1.837	-1.680	-0.150	-0.150							kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS}$ =	0.000	0.157	0.000	0.000							kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	**+0.150**	-	-	-							kip
	Total Forces Resisted by LFRS 1:	$\Sigma F_{LFRS\text{-}W}$ =	**1.987**	**1.837**	**0.150**	**0.150**							**kip**
COLL-1	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-0.157	-	-0.019							kip
LFRS-2	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-							kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-							kip
	Total Forces Resisted by LFRS 2:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**
COLL-2	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip
LFRS-3	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-							kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-							kip
	Total Forces Resisted by LFRS 3:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**
COLL-3	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip
LFRS-4	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-							kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-							kip
	Total Forces Resisted by LFRS 4:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**
COLL-4	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-							kip
LFRS-5	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-							kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-							kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-							kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-							kip
	Total Forces Resisted by LFRS 5:	$\Sigma F_{LFRS\text{-}W}$ =	**-**	**-**	**-**	**-**							**kip**
All COLL	**Axial Design Forces**	$CL_{DWXP1} = \max(N_{CL}, D_{CL})$ =	**-**	**0.157**	**-**	**0.019**							**kip**

X Governing Collector & LFRS Design Forces : Longitudinal Z - Direction CONT'D

Z-Dir, Longitudinal Applied Force			Main Bldg		Left Lean-To 1		-	-	-	
	Collector & LFRS Location:	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	RIGHT	LEFT				
Seismic	Roof Sub-Diaphragm End Reaction:	R_{DSXE} =	0.483	0.483	0.190	0.190				kip
	Roof Sub-Diaphragm Linear Drag:	$V_{DSZ} = R_{DSZ} / L_b$ =	0.014	0.014	0.005	0.005				klf
	Transferred out to **Shared Wall** of Attached Structure	R_{DSZT} =	-	-	**-0.190**	-				kip
	Reaction(s) **Received** from Attached Structure(s):	R_{DSZA} =	**+0.190**	-	-	-				kip
LFRS-1	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	0.483	0.483	0.190	0.190				kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-0.483	-0.441	-0.190	-0.190				kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS}$ =	0.000	0.041	0.000	0.000				kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	**+0.190**	-	-	-				kip
	Total Forces Resisted by LFRS 1:	$\Sigma F_{LFRS\text{-}W}$ =	**0.672**	**0.483**	**0.190**	**0.190**				**kip**
COLL-1	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-0.041	-	-0.024				kip
LFRS-2	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip
	Total Forces Resisted by LFRS 2:	$\Sigma F_{LFRS\text{-}W}$ =	-	-	-	-				**kip**
COLL-2	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip
LFRS-3	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip
	Total Forces Resisted by LFRS 3:	$\Sigma F_{LFRS\text{-}W}$ =	-	-	-	-				**kip**
COLL-3	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip
LFRS-4	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip
	Total Forces Resisted by LFRS 4:	$\Sigma F_{LFRS\text{-}W}$ =	-	-	-	-				**kip**
COLL-4	Diaphragm Force developed:	$D_{CL} = V_{DWXP} * L_{CL}$ =	-	-	-	-				kip
LFRS-5	Design Force resisted:	$F_{LFRS} = (\% r) * R_{DWXP}$ =	-	-	-	-				kip
	Diaphragm Force developed over LFRS:	$D_{LFRS} = V_{DWXP} * L_{LFRS}$ =	-	-	-	-				kip
	Net Force (+T or -C) sent to Collector :	$R_{LFRS} = F_{LFRS} + D_{LFRS} + N_{CL}$ =	-	-	-	-				kip
	Additional Design Force resisted by LFRS:	$F_{LFRSA} = (\% r) * R_{DWXA}$ =	-	-	-	-				kip
	Total Forces Resisted by LFRS 5:	$\Sigma F_{LFRS\text{-}W}$ =	-	-	-	-				**kip**
All COLL	**Axial Design Forces**	$CL_{DWXP1} = \max(N_{CL}, D_{CL})$ =	-	**0.041**	-	**0.024**				**kip**

XI Roof Sheathing Diaphgram Checks

Roof Sheathing Specification		**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**		
Roof Sheathing Gauge:		29	29	-	-	-	GA	
Roof Sheathing Type:		Strata Rib - ¾" Rib	Strata Rib - ¾" Rib	-	-	-		Ref UES ER 0550
All Roof Sheathing Orientation:		Vertical	Vertical	-	-	-		
Max. Panel Attachment Spacing:	A =	48	42	-	-	-	in	Ref UES ER 0550
Side Lap / Stitch Screw spacing:	B =	1	1	-	-	-		Ref UES ER 0550
Allowable Shear Diaphragm Capacity:	$R_{ROOF,D}$ =	126.00	142.00				plf	UES ER 0550 Table 13.6

Roof Sheathing Shear Check			**Main Bldg**	**Left Lean-To 1**	**-**	**-**	**-**	
	Attachment Location	(X-DIR) (TRANSVERSE) (PERIMETER)	FRONT / BACK	FRONT / BACK				
Wind	Sub-Diaphragm Linear Shear Force	$V_{DWXP} = R_{DWXP} / W_b$ =	7.12	6.33				plf
Seismic	Sub-Diaphragm Linear Shear Force	$V_{DSXP} = \rho * R_{DSXP} / W_b$ =	2.59	2.54				plf
Design	Sub-Diaphragm Linear Shear Force	$V_{DX,MAX}$ = max (V_{DWXP}, V_{DSXP}) =	7.12	6.33				plf
Check:		$R_{ROOF,D} > V_{DX,MAX}$ =	126.00 > 7.12	142.00 > 6.33				plf
			OK	**OK**				

	Attachment Location	(X-DIR) (TRANSVERSE) (INTERIOR)	- TYP FRAME	- TYP FRAME	-	-	-	-	-	-	
Wind	Sub-Diaphragm Linear Shear Force	$V_{DWXI} = R_{DWXI} / W_b$ =	20.50	18.22							plf
Seismic	Sub-Diaphragm Linear Shear Force	$V_{DSXI} = \rho * R_{DSXI} / W_b$ =	7.44	7.32							plf
Design	Sub-Diaphragm Linear Shear Force	$V_{DX,MAX}$ = max (V_{DWXP}, V_{DSXP}) =	20.50	18.22							plf
Check:		$R_{ROOF,D} > V_{DX,MAX}$ =	126 >20.50	142 >18.22							plf
			OK	**OK**							

	Attachment Location	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT / RIGHT	LEFT / RIGHT	
Wind	Sub-Diaphragm Linear Shear Force	$V_{DWZP} = R_{DWZP} / L_b$ =	52.49	4.27	plf
Seismic	Sub-Diaphragm Linear Shear Force	$V_{DSZP} = \rho * R_{DSZP} / L_b$ =	13.79	5.42	plf
Design	Sub-Diaphragm Linear Shear Force	$V_{DZ,MAX}$ = max (V_{DWZ}, V_{DSZ}) =	52.49	5.42	plf
Check:		$R_{ROOF,D} > V_{DZ,MAX}$ =	126.00 > 52.49	142.00 > 5.42	plf
			OK	**OK**	

XII Framing Members Material Properties

Framing Member Properties		All Tubing		All Channels	
Type and Grade:		ASTM A500 Grade C		ASTM A653 (HSLA) Grade 50	
Yield Strength:	F_y =	50	ksi	50	ksi
Ultimate Strength:	F_u =	62	ksi	65	ksi

Sheathing Steel Properties			
Type and Grade:		ASTM A792 SS Grade 80	
Yield Strength:	F_y =	60	ksi
Ultimate Strength:	F_u =	61.5	ksi

Material Properties			
Modulus of Elasticity:	E =	29,000	ksi
Density:	γ =	490	lbs/ft^3

Framing Members Section Properties:

Section ID	Description	Depth / Width	X-Area	Gauge/ Thickness	Intertia X-X / Y-Y
		in	in^2	in	in4
TS222212GA	2.25" SQ x 12GA Tube	2.25 / 2.25	0.933	12 GA / 0.109	0.715 / 0.715
TS222212GAD12	(2) 2.25" SQ x 12GA Tubes	4.5 / 2.25	1.867	12 GA / 0.109	3.791 / 1.278
TS222214GA	2.25" x 2.25" x 14GA Tube	2.25 / 2.25	0.719	14 GA / 0.083	0.564 / 0.564
TS202012GA	2" SQ x 12GA Tube	2 / 2	0.824	12 GA / 0.109	0.493 / 0.493
CS252514GA	2.5" SQ x 14GA C-Channel	2.5 / 2.5	0.609	14 GA / 0.083	0.694 / 0.409
C401014GAD	(2) 4" x 1" x 14GA HAT-Channel - Back-to-back	1 / 4	1.188	14 GA / 0.083	1.929 / 4.098
C401014GA	4" x 1" x 14GA HAT-Channel	1 / 4	0.594	14 GA / 0.083	0.9645 / 2.049
-					
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XIII Self Drilling Screw, Fastener Capacities

Type 1 : Fastener Common Parameters		**14GA - 14GA**	**12GA - 14GA**	**14GA - 12GA**	**12GA - 12GA**	**18GA - 14GA**	
Fastener ESR:		ESR 2196	ESR 2196	ESR 2196	ESR 2196	ESR 2196	
Fastener type:		#12-14 x 3/4"	#12-14 x 3/4"	#12-14 x 3/4"	#12-14 x 3/4"	#12-14 x 3/4"	
Fastener Brand:		Hilti SD screws	Hilti SD screws	Hilti SD screws	Hilti SD screws	Hilti SD screws	
Allowable Fastener Steel Tensile Capacity (per fastener):	F_{TSA} =	775.00	775.00	775.00	775.00	775.00	lbf
Allowable Fastener Steel Shear Capacity (per fastener):	F_{VSA} =	625.00	625.00	625.00	625.00	625.00	lbf

Per Fastener - Connection Capacity			**14GA - 14GA**		**12GA - 14GA**		**14GA - 12GA**		**12GA - 12GA**		**18GA - 14GA**	
Ultimate Strength of Steel members:	F_{UM} =		65		65		65		65		61.5	ksi
Gauge of member in contact with screw head:	$t_{min,1}$ =		14		12		14		12		18	GA
Gauge of member not in contact with screw head:	$t_{min,2}$ =		14		14		12		12		14	GA
Allowable Pull-over Capacity (per fastener):	$F'_{P,OVR} = (F_{UM} / F_{UA}) * F_{P,OVR}$ =	65 / 45 x	700.00	65 / 45 x	840.00	65 / 45 x	980.00	65 / 45 x	980.00	61.5 / 45 x	700.00	lbf
Allowable Pull-out Capacity (per fastener):	$F'_{P,OUT} = (F_{UM} / F_{UA}) * F_{P,OUT}$ =	65 / 45 x	207.00	65 / 45 x	248.00	65 / 45 x	289.00	65 / 45 x	289.00	61.5 / 45 x	207.00	lbf
Allowable Shear Bearing Capacity (per fastener):	$F'_{V,BRG} = (F_{UM} / F_{UA}) * F_{V,BRG}$ =	65 / 45 x	600.00	65 / 45 x	787.00	65 / 45 x	657.00	65 / 45 x	920.00	61.5 / 45 x	420.00	lbf
Controlling Tensile Capacity (per fastener):	$\mathbf{F_{TA}}$ = min (F_{TSA}, $F'_{P,OUT}$, $F'_{O,OVR}$) =		**299.00**		**358.22**		**417.44**		**417.44**		**282.90**	lbf
Controlling Shear Capacity (per fastener):	$\mathbf{F_{VA}}$ = min (F_{VSA}, $F'_{V,BRG}$) =		**625.00**		**625.00**		**625.00**		**625.00**		**574.00**	lbf

Type 2 : Fastener Common Parameters		**14GA - 14GA**	**12GA - 14GA**	**14GA - 12GA**	**12GA - 12GA**	**18GA - 14GA**	
Fastener ESR:		ESR 2196	ESR 2196	ESR 2196	ESR 2196	ESR 2196	
Fastener type:		#12-24 x 1 1/4"	#12-24 x 1 1/4"	#12-24 x 1 1/4"	#12-24 x 1 1/4"	#12-24 x 1 1/4"	
Fastener Brand:		Hilti SD screws	Hilti SD screws	Hilti SD screws	Hilti SD screws	Hilti SD screws	
Allowable Fastener Steel Tensile Capacity (per fastener):	F_{TSA} =	1300.00	1300.00	1300.00	1300.00	1300.00	lbf
Allowable Fastener Steel Shear Capacity (per fastener):	F_{VSA} =	760.00	760.00	760.00	760.00	760.00	lbf

Per Fastener - Connection Capacity			**14GA - 14GA**		**12GA - 14GA**		**14GA - 12GA**		**12GA - 12GA**		**18GA - 14GA**	
Ultimate Strength of Steel members:	F_{UM} =		65		65		65		65		65	ksi
Gauge of member in contact with screw head:	$t_{min,1}$ =		14		12		14		12		18	GA
Gauge of member not in contact with screw head:	$t_{min,2}$ =		14		14		12		12		14	GA
Allowable Pull-over Capacity (per fastener):	$F'_{P,OVR} = (F_{UM} / F_{UA}) * F_{P,OVR}$ =	65 / 45 x	700.00	65 / 45 x	840.00	65 / 45 x	980.00	65 / 45 x	980.00	65 / 45 x	700.00	lbf
Allowable Pull-out Capacity (per fastener):	$F'_{P,OUT} = (F_{UM} / F_{UA}) * F_{P,OUT}$ =	65 / 45 x	207.00	65 / 45 x	248.00	65 / 45 x	289.00	65 / 45 x	289.00	65 / 45 x	207.00	lbf
Allowable Shear Bearing Capacity (per fastener):	$F'_{V,BRG} = (F_{UM} / F_{UA}) * F_{V,BRG}$ =	65 / 45 x	600.00	65 / 45 x	787.00	65 / 45 x	657.00	65 / 45 x	920.00	65 / 45 x	420.00	lbf
Controlling Tensile Capacity (per fastener):	$\mathbf{F_{TA}}$ = min (F_{TSA}, $F'_{P,OUT}$, $F'_{O,OVR}$) =		**299.00**		**358.22**		**417.44**		**417.44**		**299.00**	lbf
Controlling Shear Capacity (per fastener):	$\mathbf{F_{VA}}$ = min (F_{VSA}, $F'_{V,BRG}$) =		**760.00**		**760.00**		**760.00**		**760.00**		**606.67**	lbf

XIV Common Member Types and Gauges

		Main Bldg		Left Lean-To 1		-		-		-			
Building Wall / Frame location:	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	-	LEFT	-	-	-	-	-	-		
Side Wall Column Post Member Type:		TS222212GAD 12	TS222212GAD 12	-	TS222212GA							-	Refer Section properties table
Side Wall Column Post Member Gauge:		12	12	-	12							GA	
Sleeve Gauge:		12	12	-	12							GA	

		Main Bldg		Left Lean-To 1		-		-		-			
Building Wall / Frame location:	(X-DIR) (TRANSVERSE) (PERIMETER)	FRONT	BACK	FRONT	BACK								
Roof Member Type:		TS222212GA	TS222212GA	TS222212GA	TS222212GA							-	Refer Section properties table
Roof Member Gauge:		12	12	12	12							GA	
End Wall Column Post Member Type:		TS222212GAD 12	TS222212GAD 12	-	-							-	Refer Section properties table
End Wall Column Post Member Gauge:		12	12	-	-							GA	
Sleeve Gauge:		-	-	-	-							GA	

		Main Bldg		Left Lean-To 1		-		-		-			
Building Wall / Frame location:	(X-DIR) (TRANSVERSE) (INTERIOR)	-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Roof Member Type:		-	TS222212GA	-	TS222212GA							-	Refer Section properties table
Roof Member Gauge:		-	12	-	12							GA	
Column Post Member Type:		-	TS222212GAD 12	-	TS222212GA							-	Refer Section properties table
Column Post Member Gauge:		-	12	-	12							GA	
Sleeve Gauge:		-	12	-	12							GA	

XV Shear Wall Checks

Common Parameters			**Main Bldg**		**-**		**-**		**-**		**-**		
Wall Sheathing Gauge:			29		-		-		-		-	GA	
Wall Sheathing Type:			Strata Rib - ¾" Rib		-		-		-		-		

Shear Wall diaphragm check			**Main Bldg**		**-**		**-**		**-**		**-**		
Shear Wall Location	(X-DIR) (TRANSVERSE) (PERIMETER)	FRONT	BACK										
Max. Panel Attachment Spacing:	A =	24	60	-	-	-	-	-	-	-	-	in	
Side Lap / Stitch Screw spacing:	B =	1	1	-	-	-	-	-	-	-	-	-	
Total Length of Shear Walls:	$L_{SHEAR,W}$ =	6.00	30.00									ft	
Design Wall Shear:	$R_{DXP} = \max(R_{DWXP}, \rho * R_{DSXP})$ =	0.214	0.214									kip	
Design Shear force per linear foot:	$R_{DXP} / L_{SHEAR,W}$ =	35.6	7.12									plf	
Allowable Diaphragm Capacity:	$R_{W,X}$ =	218.00	103.00									plf	UES ER 0550 Table 13.6
Check:	$R_{W,X} = R_{DXP} / L_{SHEAR,W}$ =	**OK**	**OK**									plf	

Shear Wall diaphragm check			**-**		**-**		**-**		**-**		**-**		
Shear Wall Location	(X-DIR) (TRANSVERSE) (INTERIOR)												
Max. Panel Attachment Spacing:	A =	-	-	-	-	-	-	-	-	-	-	in	
Side Lap / Stitch Screw spacing:	B =	-	-	-	-	-	-	-	-	-	-	-	
Total Length of Shear Walls:	$L_{SHEAR,W}$ =											ft	
Design Wall Shear:	$R_{DXI} = \max(R_{DWXI}, \rho * R_{DSXI})$ =											kip	
Design Shear force per linear foot:	$R_{DXI} / L_{SHEAR,W}$ =											plf	
Allowable Diaphragm Capacity:	$R_{W,X}$ =											plf	UES ER 0550 Table 13.6
Check:	$R_{W,X} = R_{DXI} / L_{SHEAR,W}$ =											plf	

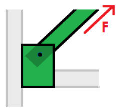
Shear Wall diaphragm check			**Main Bldg**		**-**		**-**		**-**		**-**		
Shear Wall Location	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT										
Max. Panel Attachment Spacing:	A =	60	60	-	-	-	-	-	-	-	-	in	
Side Lap / Stitch Screw spacing:	B =	1	1	-	-	-	-	-	-	-	-	-	
Total Length of Shear Walls:	$L_{SHEAR,W}$ =	35.00	32.00									ft	
Design Wall Shear:	$R_{DZ} = \max(R_{DWZ}, \rho * R_{DSZ})$ =	1.987	1.837									kip	
Design Shear force per linear foot:	$R_{DZ} / L_{SHEAR,W}$ =	56.8	57.4									plf	
Allowable Diaphragm Capacity:	$R_{W,Z}$ =	103.00	103.00									plf	UES ER 0550 Table 13.6
Check:	$R_{W,X} = R_{DZ} / L_{SHEAR,W}$ =	**OK**	**OK**									plf	

XVI Diagonal (Z) Brace Checks

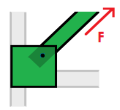
Common Connection Design Parameters		**Main Bldg**	-	-	-	-	
Diagonal Brace Tube Gauge:	12	12					GA
Horizontal Brace Tube Gauge:	14	14					GA
Plate / Angle Gauge:	14	14					GA

1 - Diagonal Brace to Plate / Angle Connection Check

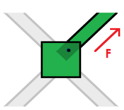
			Main Bldg		
	WALL	L / R	F/ B		
Max. Axial Force in Diagonal Brace:	P_{CON} =	0.747	0.648	kip	



Fastener thru Angle Capacity	WALL	L / R	F/ B		
No. of Angle Clips each connection:	n_{CL} =	1	1	-	
No. of Fasteners between Angle and Brace	$n_{F,V,CL}$ =	4	4	-	
Allowable Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	625.0	625.0	lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	$R_{FVA1} = n_{CL} * n_{F,V,CL} * F_{VA}$ =	2.500	2.500	kip	
Fastener Tensile Check:	$R_{FVA1} > P_{CON}$ =	**OK**	**OK**		



Fastener thru Plate Capacity	WALL	L / R	F/ B		
No. of Plates at each connection:	n_{FCL} =	-	-	-	
No. of Fasteners between Plate and Brace:	$n_{F,V,FCL}$ =	-	-	-	
Allowable Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	$R_{FVA2} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-	kip	
Fastener Shear Check:	$R_{FVA2} > P_{con}$ =	-	-		
Combined Connection Fastener Capacity:	$R_{FTA1} + R_{FVA2}$ =	**2.500**	**2.500**	kip	
Combined Connection Fastener Check:	$R_{FVA1} + R_{FVA2} > P_{con}$ =	**OK**	**OK**		

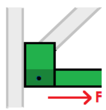


Fastener thru Plate Capacity	WALL	L / R	F/ B		
No. of Plates at each connection:	n_{FCL} =	-	-	-	
No. of Fasteners between Plate and Brace:	$n_{F,V,FCL}$ =	-	-	-	
Allowable Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	$R_{FVA2} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-	kip	
Fastener Shear Check:	$R_{FVA2} > P_{con}$ =	-	-		

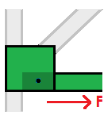
XVI Diagonal (Z) Brace Checks CONT'D

2 - Horizontal Brace to Plate / Angle Connection Check

		Main Bldg			
	WALL	L / R	F / B		
Max. Axial Force in Horizontal Brace:	P_{CON} =	0.484	1.222	kip	



Fastener thru Angle Capacity	WALL	L / R	F / B		
No. of Angle Clips each connection:	n_{CL} =	1	1		
No. of Fasteners between Angle and Brace	$n_{F,T,CL}$ =	4	4		
Allowable Shear Capacity (per Screw):	F_{VA} = min (F_{VSA}, $F_{V,BRG}$) =	625.0	625.0	lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	R_{FVA1} = n_{CL} * $n_{F,V,CL}$ * F_{VA} =	2.500	2.500	kip	
Fastener Tensile Check:	R_{FVA} > P_{con} =	**OK**	**OK**		

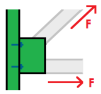


Fastener thru Plate Capacity	WALL	L / R	F / B		
No. of Plates at each connection:	n_{FCL} =	-	-		
No. of Fasteners between Plate and Brace:	$n_{F,V,FCL}$ =	-	-		
Allowable Shear Capacity (per Screw):	F_{VA} = min (F_{VSA}, $F_{V,BRG}$) =	-	-	lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	R_{FVA2} = n_{FCL} * $n_{F,V,FCL}$ * F_{VA} =	-	-	kip	
Fastener Shear Check:	R_{FVA} > P_{con} =	-	-		

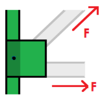
Combined Connection Fastener Capacity:	R_{FTA1} + R_{FVA2} =	**2.500**	**2.500**	kip
Combined Connection Fastener Check:	R_{FTA1} + R_{FVA2} > P_{con} =	**OK**	**OK**	

3 - Column to Plate / Angle Connection Check

		WALL	L / R	F / B		
			Main Bldg			
Design	X Component of Brace Force:	$XF_{BR,MAX}$ =	0.546	0.474	kip	
Design	Y Component of Brace Force:	$YF_{BR,MAX}$ =	0.510	0.442	kip	
Design	X Component of Strut Force:	$F_{ST,MAX}$ =	0.484	1.222	kip	



	WALL	L / R	F / B		
Fastener thru Angle Capacity					
No. of Angle Clips each connection:	n_{CL} =	1	1	-	
No. of Fasteners between Angle and Column	$n_{F,T,CL}$ =	8	8	-	
Allowable fastener Tensile Capacity (per Screw):	$F_{TA} = \min(F_{TSA}, F'_{P,OUT}, F'_{O,OVR})$ =	417.4	417.4	lbf	Refer SDS Calc sec above
Total Allowable fastener Tensile Capacity:	$R_{FTA} = n_{CL} * n_{F,T,CL} * F_{TA}$ =	**3.340**	**3.340**	kip	
Fastener Tensile Check:	$R_{FTA} > \max(XF_{BR,MAX}, F_{ST,MAX})$ =	**OK**	**OK**		
Allowable fastener Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	625.0	625.0	lbf	
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	**3.340**	**3.340**	kip	
Fastener Shear Check:	$R_{FVA} > YF_{BR,MAX}$ =	**OK**	**OK**		



	WALL	L / R	F / B		
Fastener thru Plate Capacity				-	
No. of Plates at each connection:	n_{FCL} =	-	-	-	
No. of Fasteners between Plate and Column:	$n_{F,V,FCL}$ =	-	-	-	
Allowable fastener Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	lbf	Refer SDS Calc sec above
Total Allowable fastener Shear Capacity:	$R_{FVA1} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-	kip	
Fastener Shear Check X Direction:	$R_{FVA1} > \max(XF_{BR,MAX}, F_{ST,MAX})$ =	-	-		
Fastener Shear Check Y Direction:	$R_{FVA2} > YF_{BR,MAX}$ =	-	-		

XVII Story Drift Checks : Seismic

Story Drift Caculations		Main Bldg	Left Lean-To 1	-	-	-		
Roof Sheathing Flexibility:	F =	23.20	22.00				e-6 in/lbs	UES ER 0550 Table 12.6
Roof Sheathing Stiffness factor:	G' = (1000 / F) =	43.10	45.45				k/ in	
Story height:	H_b =	168.00	144.00				in	
Allowable story drift:	$\Delta_A = 0.020 * H_b$ =	**3.36**	**2.88**				in	ASCE 7 Table 12.12.1

Direction	X, Transverse	Main Bldg	Left Lean-To 1	-	-	-		
Unfactored Seismic - Diaphragm Deflection:	$\Delta_1 = (w * L_b^2) / (8 W_b * G')$ =	0.0000	0.0000				in	
Unfactored Seismic - Structure Deflection:	Δ_2 =	0.019	0.018				in	From FEA output
Total Elastic Story Drift:	$\Delta_E = (\Delta_1 + \Delta_2) * C_d / I_e$ =	0.0380	0.0360				in	ASCE 7 eq 12.8-15
Check	$\Delta_A > \Delta_E$ =	**OK**	**OK**				in	

Direction	Z, Longitudinal	Main Bldg	Left Lean-To 1	-	-	-		
Unfactored Seismic - Diaphragm Deflection:	$\Delta_1 = (w * W_b^2) / (8 L_b * G')$ =	0.0000	0.0000				in	
Unfactored Seismic - Structure Deflection:	Δ_2 =	0.002	0.004				in	From FEA output
Total Elastic Story Drift:	$\Delta_E = (\Delta_1 + \Delta_2) * C_d / I_e$ =	0.0060	0.0080				in	ASCE 7 eq 12.8-15
Check	$\Delta_A > \Delta_E$ =	**OK**	**OK**				in	

XVIII Story Drift Checks : Wind (Based on Serviceability)

Direction	X, Transverse (Side Sway)		Main Bldg		Left Lean-To 1		
Allowable Deflection Limit (measured at Eave):	Δwxa = 0.0083 * HB =	**H / 120 =**	**1.400**	**H / 120 =**	**1.200**	in	
Maximum Structure deflection noted:		Δ_{WX} =	**0.300**		**0.334**	in	Refer Beam Disp Summary
Check:		Δwxa > Δ_{WX} =	**OK**		**OK**		

Direction	Z, Longitudinal		Main Bldg		Left Lean-To 1		
Allowable Deflection Limit (measured at Eave):	Δwxa = 0.0083 * HB =	**H / 120 =**	**1.400**	**H / 120 =**	**1.200**	in	
Maximum Structure deflection noted:		Δ_{WZ} =	**0.575**		**0.045**	in	Refer Beam Disp Summary
Check:		Δwza > Δ_{WZ} =	**OK**		**OK**		

XIX Gravity Deflection (Based on Serviceability)

Direction	Y, Vertical		Main Bldg		Left Lean-To 1		
Allowable Vertical Deflection:	Δ_{YA} =	**W / 180 =**	**2.000**	**W / 180 =**	**0.800**	in	
Maximum Structure deflection noted:		Δ_Y =	**0.514**		**0.648**	in	Refer Beam Disp Summary
Check:		$\Delta_{YA} > \Delta_Y$ =	**OK**		**OK**		

XX STAAD.Pro Finite Element Analysis (FEA) results summary:

Summary of Maximum Beam Forces by Property: **Total Structure**

Refer "Beam Max Forces by Section Property" table in SO section for values

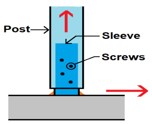
	Axial	Torsion	Shear	Bending
	Fx	Mx	Fy, Fz	My, Mz
Property Name	*kip*	*kip-in*	*kip*	*kip-in*
TS222212GA	6.158	6.199	2 , 1.05	6.032 , 19.532
TS222212GAD12	2.771	2.264	1.028 , 1.436	7.683 , 20.794
TS222214GA	1.222	0.610	0.058 , 0.089	2.582 , 1.042
TS202012GA	0.747	0.638	0.023 , 0.055	2.883 , 0.577
CS252514GA	1.013	0.000	0 , 0.002	0 , 0
C401014GAD	2.798	0.000	0 , 0.004	0 , 0
C401014GA	2.24	0.000	0.159 , 0.044	0.764 , 2.891
-				
-				
-				
-				
-				
-				

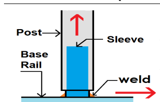
Note: All members have been checked per AISC 360 code using STAAD.Pro, and their utilization ratios (based on applicable Axial, Shear, Bending, Torsion, etc. checks), have been provided in the "Utilization Ratio" tables of the Staad Output (SO) section).

Summary of ASD Nodal Reactions		Main Bldg		Left Lean-To 1		-	-	-	
Nodes Located along	WALL	F / B	L / R	-	LEFT				
Fx Transverse Reaction (+ve or -ve)	$F_{X,max}$ =	**-1.436**	**-0.546**	**-**	**0.111**				kip
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.024	0.429	-	0.000				kip
Coresponding Fy Vertical Reaction (+ve or -ve)	F_Y =	2.110	4.232	-	0.781				kip
Fz Longitudinal Reaction (+ve or -ve)	$F_{Z,max}$ =	**-0.738**	**-0.777**	**-**	**-0.020**				kip
Coresponding Fx Transverse Reaction (+ve or -ve)	F_X =	-0.318	-0.297	-	0.051				kip
Coresponding Fy Vertical Reaction (+ve or -ve)	F_Y =	-1.572	-1.092	-	0.424				kip
Fy Bearing Reaction (+ve)	$F_{BRG,max}$ =	**2.990**	**4.232**	**-**	**0.792**				kip
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.403	0.429	-	-0.001				kip
Coresponding Fx Transverse Reaction (+ve or -ve)	F_X =	-1.216	-0.546	-	0.111				kip
Fy Uplift Max (-ve)	$F_{UP,max}$ =	**-2.369**	**-1.775**	**-**	**-0.482**				kip
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.511	-0.096	-	-0.001				kip
Coresponding Fx Transverse Reaction (+ve or -ve)	F_X =	-0.301	-0.325	-	-0.076				kip
Uplfit Resultant :	$R_{UP} = \sqrt{(F_{UP,MAX}^2 + F_Z^2 + F_X^2)}$ =	**2.442**	**1.807**	**-**	**0.488**				kip
Max Uplift Resultant (noted)	$R_{UP,MAX} = \sqrt{(F_{UP}^2 + F_Z^2 + F_X^2)}$ =	**2.442**	**1.807**	**-**	**0.488**				kip

XXI Base Connection Checks

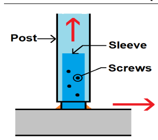
Post-to-Base Rail & Side Wall Post-to-Roof Beam Connection

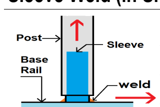




		Main Bldg		Left Lean-To 1		-		-		-			
Design Forces at:	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	-	LEFT	-	-	-	-	-	-		
Combined Uplift + Shear:	$P_{CON} = R_{UP} =$	**1.807**	**1.373**	**-**	**0.488**							kip	
Sleeve Fastener (in Shear) Parameters	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	-	LEFT	-	-	-	-	-	-		
Sleeve Gauge:		12	12	-	12							GA	
No. of Sleeves at each connection:	$n_{SL} =$	2	2	-	1							-	
Min. No. of Fasteners in each Sleeve:	$n_{FV} =$	4	4	-	4							-	
Allowable Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG}) =$	625.0	625.0	-	625.0							lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	$R_{FVA} = n_{SL} * n_{FV} * F_{VA} =$	5.000	5.000	-	2.500							kip	
Fastener Shear Check:	$R_{FVA} > P_{CON} =$	**OK**	**OK**	**-**	**OK**								
Sleeve Weld (in Shear) Parameters	(Z-DIR) (LONGITUDINAL) (PERIMETER)	LEFT	RIGHT	-	LEFT	-	-	-	-	-	-		
Length of Weld in Shear (include all faces):	$L_{weld} =$	4.00	4.00	-	4.00							in	
Weld thickness:	$t_{weld} =$	1/8	1/8	-	1/8							in	
Weld Strength:	$F_{nw} =$	50	50	-	50							ksi	
Weld loading angle:	$\theta =$	0	0	-	0							deg	Assumed conservatively
Total weld Shear per AISC:	$P_{wa1} = 0.60 * F_{nv} * (\sqrt{2}/2) * t_{weld} * L_{weld} * (1 + 0.5\sin^{1.5}\theta) =$	10.607	10.607	-	10.607							kip	AISC Eq 8-1 & J2-5
Total weld Shear per AISI:	$P_{wa2} = F_{nv} * L_{weld} * t_{weld} =$	25.000	25.000	-	25.000							kip	AISI E2.4-3
Allowable Weld Capacity:	$R_{WA} = \min(P_{wa1}/2.00), (P_{wa2}/2.35) =$	5.303	5.303	-	5.303							kip	
Weld Shear Check:	$R_{WA} > P_{CON} =$	**OK**	**OK**	**-**	**OK**								

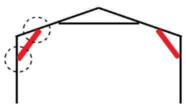
Post-to-Base Rail Connection

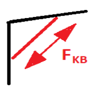




		Main Bldg		Left Lean-To 1		-		-		-			
Design Forces at:		-	CORNER 1	-	-								
Combined Uplift + Shear:	$P_{CON} = \sqrt{(P^2 + \sqrt{(V_X^2 + V_Z^2)})} =$	**-**	**1.807**	**-**	**-**							kip	
Sleeve Fastener (in Shear) Parameters		-	CORNER 1	-	-								
Sleeve Gauge:		-	12	-	-							GA	
No. of Sleeves at each connection:	$n_{SL} =$	-	1	-	-							-	
Min. No. of Fasteners in each Sleeve:	$n_{FV} =$	-	4	-	-							-	
Allowable Shear Capacity (per Screw):	$F_{VA} = \min(F_{VSA}, F_{V,BRG}) =$	-	625.0	-	-							lbf	Refer SDS Calc sec above
Total Allowable Shear Capacity:	$R_{FVA} = n_{SL} * n_{FV} * F_{VA} =$	-	2.500	-	-							kip	
Fastener Shear Check:	$R_{FVA} > P_{CON} =$	**-**	**OK**	**-**	**-**								
Sleeve Weld (in Shear) Parameters		-	CORNER 1	-	-								
Length of Weld in Shear (include all faces):	$L_{weld} =$	-	4.00	-	-							in	
Weld thickness:	$t_{weld} =$	-	1/8	-	-							in	
Weld Strength:	$F_{nw} =$	-	50	-	-							ksi	
Weld loading angle:	$\theta =$	-	0	-	-							deg	Assumed conservatively
Total weld Shear per AISC:	$P_{wa1} = 0.60 * F_{nv} * (\sqrt{2}/2) * t_{weld} * L_{weld} * (1 + 0.5\sin^{1.5}\theta) =$	-	10.607	-	-							kip	AISC Eq 8-1 & J2-5
Total weld Shear per AISI:	$P_{wa2} = F_{nv} * L_{weld} * t_{weld} =$	-	25.000	-	-							kip	AISI E2.4-3
Allowable Weld Capacity:	$R_{WA} = \min(P_{wa1}/2.00), (P_{wa2}/2.35) =$	-	5.303	-	-							kip	
Weld Shear Check:	$R_{WA} > P_{CON} =$	**-**	**OK**	**-**	**-**								

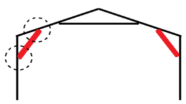
XXII Frame Connection Checks

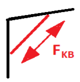




Knee Brace-to-Column Post connection			Main Bldg		Left Lean-To 1		-		-		-		
Connection located at:		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Max. Axial Forces at each Knee Brace element:	P_{KB} =	-	2.798	-	1.013							kip	From Max Forces Table
Knee Brace member:		-	C401014GAD	-	CS252514GA							-	
Knee Brace Length:	L_{KB} =	-	4.00	-	3.00							ft	
No. of Knee Braces at each connection:	n_{KB} =	-	2	-	1							-	
Knee-Brace Member Axial Design		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Allowable Knee Brace Axial Capacity:	$R_A = n_{KB} * R_{KB}$ =	-	4.580	-	6.362							kip	
Maximum Knee Brace Axial Force:	$F_{KB} = P_{KB}$ =	-	**2.798**	-	**1.013**							kip	
Knee Brace Axial Capacity Check:	$R_A > P_{KB}$ =	-	**OK**	-	**OK**								
Knee Brace Fastener Parameters		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Knee Brace Gauge:		-	14	-	14							GA	
Min. No. of Fasteners at End of Each Knee Brace:	n_{FV} =	-	3	-	2							-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	625.0	-	625.0							lbf	Refer SDS Calc Sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{KB} * n_{FV} * F_{VA}$ =	-	3.750	-	1.250							kip	
Knee Brace Fastener Shear Check:	$R_{FVA} > P_{KB}$ =	-	**OK**	-	**OK**								
Flat Clip (in Shear) Parameters		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Plate Gauge:		-	-	-	-							GA	
No. of Plates at each connection:	n_{FCL} =	-	-	-	-							-	
No. of Fasteners in each Plate:	$n_{F,V,FCL}$ =	-	-	-	-							-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	-	-							lbf	Refer SDS Calc sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-	-	-							kip	
Fastener at Plate Shear Check:	$R_{FVA} > P_{KB}$ =	-	-	-	-								
Flare V-Groove Weld Parameters		-	TYP FRAME	-	TYP FRAME	-	-	-	-	-	-		
Length of Weld (each face):	L_w =	-	-	-	-							in	
No. of Welded faces	n_w =	-	-	-	-							-	
Weld thickness / throat:	t_{wt} =	-	-	-	-							in	
Thickness of Welded Member:	t =	-	-	-	-							in	
Strength of Welded Member:	F_u =	-	-	-	-							ksi	AISC Eq 8-1 & J2-5
Weld Strength (E70XX)	F_{xx} =	-	-	-	-							ksi	
Effective Weld thickness (for Flare V-Grove Welds):	$t_w = \min(t_{wf}\text{-}d_1, t_{wf}\text{-}d_2)$ =	-	-	-	-							in	
A - Longitudinal Weld (Shear)													
Component Force (along length of weld):	$PL_{PB} = P_{PB} / n_w * \sqrt{2}$ =	-	-	-	-							**kip**	
If t ≤ t_w < 2t ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.75 * t * L_w * F_u / 2.80$ =	-	-	-	-							kip	AISI J2.6-3
If t_w > 2t ASD AISI Weld Capacity:	$P_{n2} / \Omega = 1.5 * t * L_w * F_u / 2.80$ =	-	-	-	-							kip	AISI J2.6-2
If t > 0.1in ASD AISI Weld Capacity:	$P_{n3} / \Omega = 0.75 * t_w * L_w * F_{xx} / 2.55$ =	-	-	-	-							kip	AISI J2.6-4
ASD AISC Weld Capacity:	$P_{n4} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_w\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	-	-	-							kip	AISC Eq 8-1 & J2-5
Longitudinal Weld Check:	$\min(P_{n1}/\Omega\ P_{n2}/\Omega\ P_{n3}/\Omega\ P_{n4}/\Omega) > PL_{PB}$ =	-	-	-	-								
B - Transverse Weld (Tensile)													
Component Force (perpicular to weld):	$PT_{PB} = P_{PB} / n_w * \sqrt{2}$ =	-	-	-	-							**kip**	
ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.833 * t * L_w * F_u / 2.55$ =	-	-	-	-							kip	AISI J2.6-1
ASD AISC Weld Capacity:	$P_{n2} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_w\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	-	-	-							kip	AISC Eq 8-1 & J2-5
Transverse Weld Check:	$\min(P_{n1}/\Omega, P_{n2}/\Omega) > PT_{PB}$ =	-	-	-	-								

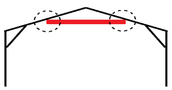
XXII Frame Connection Checks CONT'D





Knee Brace-to-Column Post connection		**Main Bldg**		**Left Lean-To 1**		**-**	**-**	**-**		
Connection located at:		-	-	FRONT	BACK					
Max. Axial Forces at Knee Brace element:	P_{KB} =	-	-	0.520	0.516				kip	From Max Forces Table
Knee Brace member:		-	-	CS252514GA	CS252514GA				-	
Knee Brace Length:	L_{KB} =	-	-	3.00	3.00				ft	
No. of Knee Braces at each connection:	n_{KB} =	-	-	1	1				-	
Knee-Brace Member Axial Design		-	-	FRONT	BACK					
Allowable Knee Brace Axial Capacity:	$R_A = n_{KB} * R_{KB}$ =	-	-	6.362	6.362				kip	
Maximum Knee Brace Axial Force:	$F_{KB} = P_{KB}$ =	-	-	0.520	0.516				kip	
Knee Brace Axial Capacity Check:	$R_A > P_{KB}$ =	**-**	**-**	**OK**	**OK**					
Knee Brace Fastener (in Shear) Parameters		-	-	FRONT	BACK					
Knee Brace Gauge:		-	-	14	14				GA	
No. of Fasteners at End of Each Knee Brace:	n_{FV} =	-	-	2	2				-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	625.0	625.0				lbf	Refer SDS Calc Sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{KB} * n_{FV} * F_{VA}$ =	-	-	1.250	1.250				kip	
Knee Brace Fastener Shear Check:	$R_{FVA} > P_{KB}$ =	**-**	**-**	**OK**	**OK**					
Flat Clip (in Shear) Parameters		-	-	FRONT	BACK					
Plate Gauge:		-	-	-	-				GA	
No. of Plates at each connection:	n_{FCL} =	-	-	-	-				-	
No. of Fasteners in each Plate:	$n_{F,V,FCL}$ =	-	-	-	-				-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-	-	-				lbf	Refer SDS Calc sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-	-	-				kip	
Fastener at Plate Shear Check:	$R_{FVA} > P_{KB}$ =	**-**	**-**	**-**	**-**					
Flare V-Groove Weld Parameters		-	-	FRONT	BACK					
Length of Weld (each face):	L_w =	-	-	-	-				in	
No. of Welded faces	n_w =	-	-	-	-				-	
Weld thickness / throat:	t_{wt} =	-	-	-	-				in	
Thickness of Welded Member:	t =	-	-	-	-				in	
Strength of Welded Member:	F_u =	-	-	-	-				ksi	
Weld Strength (E70XX)	F_{xx} =	-	-	-	-				ksi	
Effective Weld thickness (for Flare V-Grove Welds):	$t_w = \min(t_{wf}\text{-}d_1, t_{wf}\text{-}d_2)$ =	-	-	-	-				in	AISI J2.6-7
A - Longitudinal Weld (Shear)										
Component Force (along length of weld):	$PL_{PB} = P_{PB} / n_w * \sqrt{2}$ =	**-**	**-**	**-**	**-**				**kip**	
If $t \le t_w < 2t$ ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.75 * t * L_w * F_u / 2.80$ =	-	-	-	-				kip	AISI J2.6-3
If $t_w > 2t$ ASD AISI Weld Capacity:	$P_{n2} / \Omega = 1.5 * t * L_w * F_u / 2.80$ =	-	-	-	-				kip	AISI J2.6-2
If t > 0.1in ASD AISI Weld Capacity:	$P_{n3} / \Omega = 0.75 * t_w * L_w * F_{xx} / 2.55$ =	-	-	-	-				kip	AISI J2.6-4
ASD AISC Weld Capacity:	$P_{n4} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_w\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	-	-	-				kip	AISC Eq 8-1 & J2-5
Longitudinal Weld Check:	$\min(P_{n1}/\Omega\ P_{n2}/\Omega\ P_{n3}/\Omega\ P_{n4}/\Omega) > PL_{PB}$ =	**-**	**-**	**-**	**-**					
B - Transverse Weld (Tensile)										
Component Force (perpicular to weld):	$PT_{PB} = P_{PB} / n_w * \sqrt{2}$ =	**-**	**-**	**-**	**-**				**kip**	
ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.833 * t * L_w * F_u / 2.55$ =	-	-	-	-				kip	AISI J2.6-1
ASD AISC Weld Capacity:	$P_{n2} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_w\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	-	-	-				kip	AISC Eq 8-1 & J2-5
Transverse Weld Check:	$\min(P_{n1}/\Omega, P_{n2}/\Omega) > PT_{PB}$ =	**-**	**-**	**-**	**-**					

XXII Frame Connection Checks CONT'D

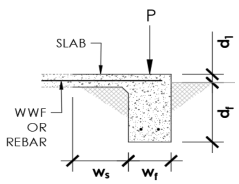


Peak Brace-to-Column Post connection			**Main Bldg**		-		-		-		-		
Connection located at:		-	TYP FRAME	-	-	-	-	-	-	-	-		
Max. Axial Force at each Brace element:	P_{PB} =	-	2.181									kip	From Max Forces Table
No. of Brace members at each connection:	n_{PB} =	-	1									-	
Peak Brace Fastener (in Shear) Parameters		-	TYP FRAME	-	-	-	-	-	-	-	-		
Roof Member Gauge:		-	12									GA	
Peak Brace Gauge:		-	12									GA	
No. of Fasteners at End of Each Peak Brace :	n_{FV} =	-	-									-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-									lbf	Refer SDS Calc Sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{PB} * n_{FV} * F_{VA}$ =	-	-									kip	
Peak Brace Fastener Shear Check:	$R_{FVA} > P_{PB}$ =	-	-										
Flat Clip (in Shear) Parameters		-	TYP FRAME	-	-	-	-	-	-	-	-		
Plate Gauge:		-	-									GA	
No. of Plates at each connection:	n_{FCL} =	-	-									-	
No. of Fasteners in each Plate:	$n_{F,V,FCL}$ =	-	-									-	
Allowable fastener Shear Capacity per Screw:	$F_{VA} = \min(F_{VSA}, F_{V,BRG})$ =	-	-									lbf	Refer SDS Calc sec above
Total Allowable fastener Shear Capacity:	$R_{FVA} = n_{FCL} * n_{F,V,FCL} * F_{VA}$ =	-	-									kip	
Fastener at Plate Shear Check:	$R_{FVA} > P_{PB}$ =	-	-										
Flare V-Groove Weld Parameters		-	TYP FRAME	-	-	-	-	-	-	-	-		
Length of Weld (each face):	L_w =	-	3.00									in	
Spacing of Welds (if applicable):	S_w=	-	24.00									in c.c	
No. of Welded faces	n_w =	-	2									-	
Weld thickness / throat:	t_{wt} =	-	1/8									in	
Thickness of Welded Member:	t =	-	0.075									in	
Strength of Welded Member:	F_u =	-	62									ksi	
Weld Strength (E70XX)	F_{xx} =	-	70									ksi	
Effective Weld thickness (for Flare V-Grove Welds):	$t_w = \min(t_{wf}-d_1, t_{wf}-d_2)$ =	-	0.08									in	AISI J2.6-7
A - Longitudinal Weld (Shear)													
Load Angle (along length of weld) :	$\theta = \tan^{-1}$ (rise / run) =	-	14.04									deg	
Component Force (along length of weld):	$PL_{PB} = P_{PB} / n_w * \cos\theta$ =	-	**1.058**									**kip**	
Effective Total Length of Weld in Shear:	L_{we} =	-	8.99									in	
If $t \le t_w < 2t$ ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.75 * t * L_{we} * F_u / 2.80$ =	-	11.201									kip	AISI J2.6-3
If $t_w > 2t$ ASD AISI Weld Capacity:	$P_{n2} / \Omega = 1.5 * t * L_{we} * F_u / 2.80$ =	-	Not Req									kip	AISI J2.6-2
If t > 0.1in ASD AISI Weld Capacity:	$P_{n3} / \Omega = 0.75 * t_w * L_{we} * F_{xx} / 2.55$ =	-	Not Req									kip	AISI J2.6-4
ASD AISC Weld Capacity:	$P_{n4} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_{we}\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	17.689									kip	AISC Eq 8-1 & J2-5
Longitudinal Weld Check:	$\min(P_{n1}/\Omega, P_{n2}/\Omega, P_{n3}/\Omega, P_{n4}/\Omega) > PL_{PB}$ =	-	**OK**										
B - Transverse Weld (Tensile)													
Component Force (perpicular to weld):	$PT_{PB} = P_{PB} / n_w * \sin\theta$ =	-	**0.264**									**kip**	
ASD AISI Weld Capacity:	$P_{n1} / \Omega = 0.833 * t * L_w * F_u / 2.55$ =	-	4.557									kip	AISI J2.6-1
ASD AISC Weld Capacity:	$P_{n2} / \Omega = 0.60\ F_{xx}\ (\sqrt{2}/2) * t_{WT} * L_w\ (1 + 0.5\sin^{1.5}\theta) / 2.00$ =	-	8.229									kip	AISC Eq 8-1 & J2-5
Transverse Weld Check:	$\min(P_{n1}/\Omega, P_{n2}/\Omega) > PT_{PB}$ =	-	**OK**										

I Foundation Reactions Summary

ASD Design Forces Summary:		Main Bldg		Left Lean-To 1	-	-	-		
Summary of Maximum Nodal Forces	WALL	F / B	L / R	- LEFT					
Fx Transverse Reaction (+ve or -ve)	$F_{X,max}$ =	**-1.436**	**-0.546**	**0.111**				**kip**	Ref. Reaction Summary
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.024	0.429	0.000				kip	
Coresponding Fy Vertical Reaction (+ve or -ve)	F_Y =	2.110	4.232	0.781				kip	
Fz Longitudinal Reaction (+ve or -ve)	$F_{Z,max}$ =	**-0.738**	**-0.777**	**-0.020**				**kip**	Ref. Reaction Summary
Coresponding Fx Longitudinal Reaction (+ve or -ve)	F_X =	-0.318	-0.297	0.051				kip	
Coresponding Fy Vertical Reaction (+ve or -ve)	F_Y =	-1.572	-1.092	0.424				kip	
Fy Bearing Reaction (+ve)	$F_{BRG,max}$ =	**2.990**	**4.232**	**0.792**				**kip**	Ref. Reaction Summary
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.403	0.429	-0.001				kip	
Coresponding Fx Longitudinal Reaction (+ve or -ve)	F_X =	-1.216	-0.546	0.111				kip	
Fy Uplift Max (-ve)	$F_{UP,max}$ =	**-2.369**	**-1.775**	**-0.482**				**kip**	Ref. Reaction Summary
Coresponding Fz Longitudinal Reaction (+ve or -ve)	F_Z =	-0.511	-0.096	-0.001				kip	
Coresponding Fx Longitudinal Reaction (+ve or -ve)	F_X =	-0.301	-0.325	-0.076				kip	
Summary of Total Forces along Entire Wall	WALL	F / B	L / R	- LEFT					
Total Fx Transverse Reaction (+ve or -ve)	max $F_{X,T}$ =	**-3.337**	**-2.156**	**0.764**				**kip**	
Coresponding Total Fz Longitudinal Reaction (+ve or -ve)	$F_{Z,T}$ =	1.800	-1.347	-0.008				kip	
Coresponding Total Fy Vertical Reaction (+ve or -ve)	$F_{Y,T}$ =	-0.077	5.227	5.556				kip	
Total Fz Longitudinal Reaction (+ve or -ve)	max $F_{Z,T}$ =	**-2.418**	**-1.985**	**-0.017**				**kip**	
Coresponding Total Fx Longitudinal Reaction (+ve or -ve)	$F_{X,T}$ =	-1.206	0.699	0.269				kip	
Coresponding Total Fy Vertical Reaction (+ve or -ve)	$F_{Y,T}$ =	0.467	2.537	2.192				kip	
Total Fy Bearing (+ve) Reaction	max $F_{BRG,T}$ =	**7.257**	**14.446**	**5.556**				**kip**	
Coresponding Total Fz Longitudinal Reaction (+ve or -ve)	$F_{Z,T}$ =	-0.066	-0.025	-0.008				kip	
Coresponding Total Fx Longitudinal Reaction (+ve or -ve)	$F_{X,T}$ =	0.023	0.499	0.764				kip	
Total Fy Uplift (-ve) Reaction	max $F_{UP,T}$ =	**-0.858**	**-7.432**	**-3.303**				**kip**	
Coresponding Total Fz Longitudinal Reaction (+ve or -ve)	$F_{Z,T}$ =	-1.282	-0.684	-0.007				kip	
Coresponding Total Fx Longitudinal Reaction (+ve or -ve)	$F_{X,T}$ =	0.412	-1.564	-0.519				kip	

II Foundation Type : Concrete Turn-Down Footing + Slab on Grade



Foundation Design Parameters		**Main Bldg**		**Left Lean-To 1**		**-**	**-**	**-**	
Footing Location	WALL	F / B	L / R	-	LEFT				
Width of footing:	w_f =	12.00	12.00		12.00				in
Depth of footing (Below grade):	d_f =	15.00	15.00		15.00				in
Depth of footing (Above grade):	d_1 =	2.00	0.00		2.00				in
Thickness of concrete slab:	t_s =	4.00	4.00		4.00				in
Length of footing:	L_f =	30.00	35.00		35.00				ft
Density of concrete:	γ_c =	150	150		150				pcf
Allowable Soil Bearing Capacity:	s_b =	2,000	2,000		2,000				psf
Width of slab section used in uplift resistance:	w_s =	12.00	12.00		12.00				in

Uplift Resistance Check	WALL	F / B	L / R	-	LEFT	
Governing Total Uplift (-ve) Load:	$F_{UP,T}$ =	-0.858	-7.432		-3.303	kip
Total Linear force causing Uplift:	$F_{dem} = abs\ (F_{UP} / L_f)$ =	28.60	212.34		94.37	plf
Width of slab section added to uplift resistance (ref flexure check):	w_s =	1.00	1.00		1.00	ft
Linear Weight of footing:	$wt_f = \gamma_c \times w_f \times (d_f + d_1)$ =	212.50	187.50		212.50	plf
Linear Weight of attached slab:	$wt_s = \gamma_c \times w_s * t_s$ =	50.00	50.00		50.00	plf
Total Uplift Resistance:	$R_{dn} = wt_f + wt_s$ =	262.50	237.50		262.50	plf
Check:	$\mathbf{R_{dn} > F_{dem}}$ =	**OK**	**OK**		**OK**	

Bearing Resistance - Uniformly Loaded Grade Beam Check	WALL	F / B	L / R	-	LEFT	
Governing Total Bearing (+ve) Load:	$F_{BRG,T}$ =	7.257	14.446		5.556	kip
Total Linear force causing Bearing:	$F_{dem} = F_{BRG} / (L_f \times w_f)$ =	241.90	412.74		158.74	psf
Allowable soil bearing capacity:	s_b =	2,000	2,000		2,000	psf
Check:	$\mathbf{R_{brg} > F_{dem}}$ =	**OK**	**OK**		**OK**	

Sliding / Lateral Resistance	WALL	F / B	L / R	-	LEFT	
Governing Total Horizontal (+ve or -ve) Load:	$\max (F_{Z,T}, F_{X,T})$ =	3.337	2.156		0.764	kip
Total Linear force causing Sliding:	$F_U = F_{LAT} / L_f$ =	111.23	61.60		21.83	plf
Allowable soil lateral friction stress:	f_{r_ltr} =	130.00	130.00		130.00	psf
Friction area:	$A_{f\_ltr} = (L_f \times w_f) + (W_b - 2w_f)$	1,020.00	1,015.00		385.00	ft^2
Total Lateral Resistance:	$R_{\_LTR} = (f_{r\_ltr} \times A_{f\_ltr}) / L_f$ =	4,420.00	3,770.00		1,430.00	plf
Sliding Safety Factor	$\mathbf{F_U / R_{LTR}}$ =	**39.74 >1.50**	**61.20 >1.50**		**65.51 >1.50**	
		OK	**OK**		**OK**	

I Concrete Anchor Checks (Main Building)

Anchor Point Parameters			
Anchor ESR:	SIMPSON CARBON STEEL STONG-BOLT 2	(ESR 3037)	
Anchor ID:		2.2	
Anchorage Point:		2.25 in Tube	
Nominal Concrete Strength:	f'_c =	2,500	psi

Installation Parameters			
Nominal Concrete Embedment:	h_{nom} =	2.75	in
Min. Anchor hole depth :	h_{hole} =	3.00	in
Effective Embedment of Anchor:	$L_e = h_{ef}$ =	2.25	in
Critical Edge distance:	c_{ac} =	6.00	in

Anchor Parameters		*Expansion*	
Anchor Diameter:	d_a =	1/2	in
Anchor Length:	L =	7.00	in
└ Concrete Embedment Req.:	D. =	2.75	in
└ Anchor Hole Depth Req.:	H. =	3.00	in
└ Min. Edge Distance Req. (Side-Wall):	ED_1 =	4.00	in
└ Min. Edge Distance Req. (End-Wall):	ED_2 =	4.00	in
└ Min. Edge Distance Req. (Internal-Walls):	ED_3 =	4.00	in
└ Min. Spacing between anchors:	s_{min} =	2.75	in

Design Parameters					
Φ,tension steel	Φ_{TSA} =	0.75	Φ,shear steel	Φ_{VSA} =	0.65
Φ,tension breakout	Φ_{TCB} =	0.65	Φ,shear breakout	Φ_{VCB} =	0.70
Φ,pull out	Φ_{TP} =	0.65	Φ,shear pryout	Φ_{VCP} =	0.70
ACI 17.6.2.5 Breakout cracking factor:				Ψ_{cN} =	1.00
k_{CR} (cracked, ACI 17.6.2.2 or per ESR) =					17
$k_{UN,Cr}$ (uncracked, ACI 17.6.2.2 per ESR) =					24
k_{CP} (ACI 17.7.3.1 or per ESR) =					1.0
ASD to LRFD Conversion Factor (α) :				α = (0.9 / 0.6) x 38 % + (1 / 0.6) x 62 % =	1.60

Anchor Common Parameters				**Side Walls**			**End Walls**
Anchor location on plans:		-	-	-	-	Main Bldg	-	-	Main Bldg.
No. of Anchors per Wall / location :	n_{wall} =					8			4
Anchor layout / case :						Case 2			Case 2
No. of Anchors per Layout:	n =					2			2

Vertical Direction - Reactions		**Side Walls**	**End Walls**	
Anchor location on plans:		- - - Main Bldg	- - Main Bldg.	
Total ASD factored Uplift at Wall :	$\sum(N_{U,W}\ N_{U,E})$ =	7.432	0.858	kip
Uncracked vs Cracked condition (assumed):	**min** (ϕM_{NT} , ϕM_{NC}) > M_S =	*uncracked*	*uncracked*	(-)
a) Conc. Breakout (ACI 318 17.6.2):	$\Phi_{TCB} N_{cb} = \Phi_{TCB} (A_{Nc}/A_{Nco}) \Psi_{,edN} \Psi_{,cN} \Psi_{,cpN} N_b$ =	*2.084*	*2.084*	kip
└ **Min. Edge Distance Req. at Location:**	c_r =	**4.00**	**4.00**	in
└ Single Anchor breakout strength:	$N_b = k_c\ \lambda\ (f'c)^{0.5}\ (h_{ef})^{1.5}$ =	*4.050*	*4.050*	kip
└ ACI 17.6.2.4 Breakout edge effect factor:	Ψ_{edN} =	*1.00*	*1.00*	-
└ ACI 17.6.2.6 Breakout splitting factor:	Ψ_{cpN} =	*0.56*	*0.56*	-
b) Steel Tensile Resistance of Anchor (ACI 318 17.6.1):	$\Phi_{TSA}\ N_{sa}$ =	*9.075*	*9.075*	kip
c) Pull-Out Strength (ACI 318 17.6.3.1):	$\Phi_{TP}\ N_p\ \Psi_{cP}\ (f'c / 2500)^{0.5}$ =	*3.290*	*3.290*	kip
Total LRFD Tension Resistance :	ϕN_N =min [ΦN_{cb}, $n_t (\Phi N_{sa}$, $\Phi N_p)$] =	**2.084**	**2.084**	**kip**
Max LRFD Required Strength :	$N_U = \alpha \times \sum [N_{U,W}\ ,\ N_{U,E}] / n_{wall}$ =	**1.489**	**0.344**	**kip**
Check:	$\phi N_N > (N_U)$ =	**OK**	**OK**	

Horizontal Direction - Reactions						**Side Walls**			**End Walls**	
Anchor location on plans:		-	-	-	-	Main Bldg	-	-	Main Bldg.	
Total ASD factored Shear at Wall :	$\sum(V_{U,w}, V_{U,E})$ =					**2.156**			**3.337**	kip
Edge distance considered for Shear breakout:	**ca1 =**					*4.00*			*4.00*	*in*
Single Anchor breakout strength:	V_b = min [7 $(L_e/d_a)^{0.2}$ $(d_a)^{0.5}$ $(f'c)^{0.5}$ $(ca1)^{1.5}$, 9 $(f'c)^{0.5}$ $(ca1)^{1.5}$] =					*2.675*			*2.675*	*kip*
ACI 17.7.2.4 Breakout edge effect factor:	Ψ_{edV} =					*1.000*			*1.000*	-
ACI 17.7.2.6 Breakout thickness factor:	Ψ_{hV} =					*1.000*			*1.000*	-
ACI 17.7.2.5 Breakout cracking factor:	Ψ_{cV} =					*1.400*			*1.400*	-
Projected breakout area of single anchor: (ACI 17.7.2.1.3)	A_{Vco} =					*72.0*			*72.0*	*in^2*
Row Resistance (Only 1 Row available)	$V_{cb,a} = (A_{vc,a}/A_{Vco})\ \Psi_{,edV}\ \Psi_{,cV}\ \Psi_{,hV}\ V_b$ =					*4.603*			*4.603*	*kip*
Only 1st Row considered; half load on Row (ACI Case 1):	$V_{cb,1} = (A_{vc,1}/A_{Vco})\ \Psi_{,edV}\ \Psi_{,cV}\ \Psi_{,hV}\ V_b / (0.5)$ =					-			-	*kip*
Only Last Row considered; full load on Row (ACI Case 2):	$V_{cb,2} = (A_{vc,2}/A_{Vco})\ \Psi_{,edV}\ \Psi_{,cV}\ \Psi_{,hV}\ V_b$ =					-			-	*kip*
Only 1st Row considered; full load on Row (ACI Case 3):	$V_{cb,3} = (A_{vc,3}/A_{Vco})\ \Psi_{,edV}\ \Psi_{,cV}\ \Psi_{,hV}\ V_b$ =					-			-	*kip*
no. of Anchors in governing V row :	n_v =					*2*			*2*	
a) Conc. Breakout Resistance per Row (ACI 318 17.7.2):	Φ_{VCB} max [$V_{cb,a}$ $V_{cb,1}$ $V_{cb,2}$ $V_{cb,3}$] =					*3.222*			*3.222*	*kip*
b) Steel Shear Resistance of Anchor (ACI 318 17.2.1):	$n_v\ \Phi_{VSA}\ V_{sa}$ =					*9.406*			*9.406*	*kip*
c) Pry-Out resistance	$\Phi_{VCP} V_{cp} = \Phi_{VCP} k_{cp} N_{cb}$ =					*2.245*			*2.245*	*kip*
Total Shear Resistance :	ϕV_N = min [ΦV_{cb}, $n_v (\Phi V_{sa})$, ΦV_{cp}] =					**2.245**			**2.245**	**kip**
Max LRFD Required Strength :	$V_U = \alpha \times \sum [V_{U,W}, V_{U,E}] / n_{wall}$ =					**0.432**			**1.337**	**kip**
	$\phi V_N > (V_U)$ =					**OK**			**OK**	

A&A ENGINEERING
CIVIL • STRUCTURAL
Tel: 419-292-1983
6036 Renaissance Place
Toledo, Ohio 43623

Case 2

Anchor Edge distance X-dir	*(required)*	**ca1, x =**	4.00	in
Spacing between anchors X -dir		**s1, x =**	-	in
Spacing between anchors X -dir		**s2, x =**	-	in
Anchor Edge distance X-dir		ca2, x =	-	in
Spacing between anchors Y -dir		**s1, y =**	2.75	in
Spacing between anchors Y -dir		**s2, y =**	-	in
Spacing between anchors Y -dir		**s3, y =**	-	in
Foundation depth:		**ha =**	17.00	in
No of Anchors in Layout:		n =	2	
No of Anchors in Front V Row:		n_v =	2	
No of Anchors in Rear V Row:		n_v =	-	

Concrete Breakout in Tension area calculation

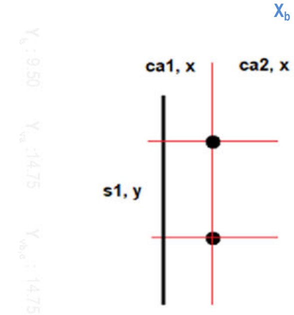
A Nco =	$(9 * hef)^2$ =	45.6	in^2
A Nc' =	(ca1x + s1x + s2x + ca2x) (ca1y + s1y + s2y + s3y + ca2y) =	64.1	in^2
A Nc =	min (n_t * A Nco , A Nc') =	64.1	in^2

Overall Layout Tensile Check (calcuated above): ***OK***

2 R x 1 C

Ψ,edN =	1.00
Ψ,cpN (cracked) =	1.00
Ψ,cpN (uncracked) =	0.56
Ψ,cP (cracked) =	1.00
Ψ,cP (uncracked) =	1.40
Ψ,edV =	1.00
Ψ,hV =	1.00
cax =	4.00
cbx =	-
ccx =	-
H Rows:	1

X_b : 6.75



d_v : 6.00

Concrete Breakout in Shear area calculation(s)

A Vco = 3 * (ca1x) * min(ha, 1.5ca1x)	72.00	in^2
A Vc,a Only 1 Row available	88.50	in^2
A Vc,1 Only 1st Row considered; half load on Row (ACI Case 1):	-	in^2
A Vc,2 Only Last Row considered; full load on Row (ACI Case 2):	-	in^2
A Vc,3 Only 1st Row considered; full load on Row (ACI Case 3):	-	in^2

Overall Layout Shear Check (calcuated above): ***OK***

II Concrete Anchor Checks (Lean-to)

Anchor Point Parameters

Anchor ESR:	SIMPSON CARBON STEEL STONG-BOLT 2	(ESR 3037)	
Anchor ID:		2.2	
Anchorage Point:		Angle	
Nominal Concrete Strength:	f'_c =	2,500	psi

Installation Parameters

Nominal Concrete Embedment:	h_{nom} =	2.75	in
Min. Anchor hole depth :	h_{hole} =	3.00	in
Effective Embedment of Anchor:	$L_e = h_{ef}$ =	2.25	in
Critical Edge distance:	c_{ac} =	6.00	in

Anchor Parameters *Expansion*

Anchor Diameter:	d_a =	1/2	in
Anchor Length:	L =	7.00	in
└ Concrete Embedment Req.:	D. =	2.75	in
└ Anchor Hole Depth Req.:	H. =	3.00	in
└ Min. Edge Distance Req. (Side-Wall):	ED_1 =	4.00	in
└ Min. Edge Distance Req. (End-Wall):	ED_2 =	4.00	in
└ Min. Edge Distance Req. (Internal-Walls):	ED_3 =	4.00	in
└ Min. Spacing between anchors:	s_{min} =	2.75	in

Design Parameters

Φ,tension steel	Φ_{TSA} =	0.75	Φ,shear steel	Φ_{VSA} =	0.65
Φ,tension breakout	Φ_{TCB} =	0.65	Φ,shear breakout	Φ_{VCB} =	0.70
Φ,pull out	Φ_{TP} =	0.65	Φ,shear pryout	Φ_{VCP} =	0.70
ACI 17.6.2.5 Breakout cracking factor:				Ψ_{cN} =	1.00
k_{CR} (cracked, ACI 17.6.2.2 or per ESR) =					17
$k_{UN,Cr}$ (uncracked, ACI 17.6.2.2 per ESR) =					24
k_{CP} (ACI 17.7.3.1 or per ESR) =					1.0
ASD to LRFD Conversion Factor (α) :				α = (0.9 / 0.6) x 38 % + (1 / 0.6) x 62 % =	1.60

Anchor Common Parameters		-	-	**Side Walls**	-	-	-	-
Anchor location on plans:		-	-	-	LeanTo 1	-	-	-	-
No. of Anchors per Wall / location :	n_{wall} =				8			
Anchor layout / case :					Case 3			
No. of Anchors per Layout:	n =				1			

Vertical Direction - Reactions				**Side Walls**				-	
Anchor location on plans:		-	-	LeanTo 1	-	-	-	-	
Total ASD factored Uplift at Wall :	$\sum(N_{U,W}\ N_{U,E})$ =			3.303					kip
Uncracked vs Cracked condition (assumed):	min (ϕM_{NT} , ϕM_{NC}) > M_S =			*uncracked*					(-)
a) Conc. Breakout (ACI 318 17.6.2):	$\Phi_{TCB}\ N_{cb} = \Phi_{TCB}\ (A_{Nc}/A_{Nco})\ \Psi_{,edN}\ \Psi_{,cN}\ \Psi_{,cpN}\ N_b$ =			*1.481*					kip
└ **Min. Edge Distance Req. at Location:**	c_r =			**4.00**					in
└ Single Anchor breakout strength:	$N_b = k_c\ \lambda\ (f'c)^{0.5}\ (h_{ef})^{1.5}$ =			*4.050*					kip
└ ACI 17.6.2.4 Breakout edge effect factor:	Ψ_{edN} =			*1.00*					-
└ ACI 17.6.2.6 Breakout splitting factor:	Ψ_{cpN} =			*0.56*					-
b) Steel Tensile Resistance of Anchor (ACI 318 17.6.1):	$\Phi_{TSA}\ N_{sa}$ =			*9.075*					kip
c) Pull-Out Strength (ACI 318 17.6.3.1):	$\Phi_{TP}\ N_p\ \Psi_{cP}\ (f'c / 2500)^{0.5}$ =			*3.290*					kip
Total LRFD Tension Resistance :	ϕN_N =min [ΦN_{cb}, n_t (ΦN_{sa}, ΦN_p)] =			**1.481**					**kip**
Max LRFD Required Strength :	$N_U = \alpha \times \sum [N_{U,W}\ , N_{U,E}] / n_{wall}$ =			**0.662**					**kip**
Check:	$\phi N_N > (N_U)$ =			**OK**					

Horizontal Direction - Reactions		**-**	**Side Walls**	**-**	
Anchor location on plans:		- -	- LeanTo 1 -	- - -	
Total ASD factored Shear at Wall :	$\sum(V_{U,w}, V_{U,E})$ =		0.764		kip
Edge distance considered for Shear breakout:	**ca1 =**		4.00		in
Single Anchor breakout strength:	$V_b = \min[\,7\,(L_e/d_a)^{0.2}\,(d_a)^{0.5}\,(f'c)^{0.5}\,(ca1)^{1.5}\,,\,9\,(f'c)^{0.5}\,(ca1)^{1.5}\,]$ =		2.675		kip
ACI 17.7.2.4 Breakout edge effect factor:	Ψ_{edV} =		1.000		-
ACI 17.7.2.6 Breakout thickness factor:	Ψ_{hV} =		1.000		-
ACI 17.7.2.5 Breakout cracking factor:	Ψ_{cV} =		1.400		-
Projected breakout area of single anchor: (ACI 17.7.2.1.3)	A_{Vco} =		72.0		in^2
Row Resistance (Only 1 Row available)	$V_{cb,a} = (A_{vc,a}/A_{Vco})\,\Psi_{,edV}\,\Psi_{,cV}\,\Psi_{,hV}\,V_b$ =		3.745		kip
Only 1st Row considered; half load on Row (ACI Case 1):	$V_{cb,1} = (A_{vc,1}/A_{Vco})\,\Psi_{,edV}\,\Psi_{,cV}\,\Psi_{,hV}\,V_b\,/\,(0.5)$ =		-		kip
Only Last Row considered; full load on Row (ACI Case 2):	$V_{cb,2} = (A_{vc,2}/A_{Vco})\,\Psi_{,edV}\,\Psi_{,cV}\,\Psi_{,hV}\,V_b$ =		-		kip
Only 1st Row considered; full load on Row (ACI Case 3):	$V_{cb,3} = (A_{vc,3}/A_{Vco})\,\Psi_{,edV}\,\Psi_{,cV}\,\Psi_{,hV}\,V_b$ =		-		kip
no. of Anchors in governing V row :	n_v =		1		
a) Conc. Breakout Resistance per Row (ACI 318 17.7.2):	$\Phi_{VCB}\max[\,V_{cb,a}\,V_{cb,1}\,V_{cb,2}\,V_{cb,3}\,]$ =		2.621		kip
b) Steel Shear Resistance of Anchor (ACI 318 17.2.1):	$n_v\,\Phi_{VSA}\,V_{sa}$ =		4.703		kip
c) Pry-Out resistance	$\Phi_{VCP}\,V_{cp} = \Phi_{VCP}\,k_{cp}\,N_{cb}$ =		1.595		kip
Total Shear Resistance :	$\phi\,V_N = \min[\,\Phi V_{cb},\,n_v\,(\Phi V_{sa}),\,\Phi V_{cp}\,]$ =		**1.595**		**kip**
Max LRFD Required Strength :	$V_U = \alpha \times \sum[V_{U,W}\,,\,V_{U,E}]\,/\,n_{wall}$ =		**0.153**		**kip**
	$\phi V_N > (V_U)$ =		**OK**		

Case 3

Anchor Edge distance X-dir	*(required)*	**ca1, x =**	4.00	in
Spacing between anchors X -dir		**s1, x =**	-	in
Spacing between anchors X -dir		**s2, x =**	-	in
Anchor Edge distance X-dir		ca2, x =	-	in
Spacing between anchors Y -dir		**s1, y =**	-	in
Spacing between anchors Y -dir		**s2, y =**	-	in
Spacing between anchors Y -dir		**s3, y =**	-	in
Foundation depth:		**ha =**	17.00	in
No of Anchors in Layout:		n =	1	
No of Anchors in Front V Row:		n_v =	1	
No of Anchors in Rear V Row:		n_v =	-	

Concrete Breakout in Tension area calculation

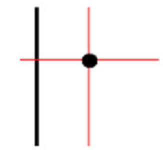
A Nco =	$(9 * hef)^2$ =	45.6	in^2
A Nc' =	(ca1x + s1x + s2x + ca2x) (ca1y + s1y + s2y + s3y + ca2y) =	45.6	in^2
A Nc =	min (n_t * A Nco , A Nc') =	45.6	in^2
Overall Layout Tensile Check (calcuated above):		***OK***	

1 R x 1 C

Ψ,edN =	1.00
Ψ,cpN (cracked) =	1.00
Ψ,cpN (uncracked) =	0.56
Ψ,cP (cracked) =	1.00
Ψ,cP (uncracked) =	1.40
Ψ,edV =	1.00
Ψ,hV =	1.00
cax =	4.00
cbx =	-
ccx =	-
H Rows:	1

X_b : 6.75

ca1, x ca2, x



d_v : 6.00

Concrete Breakout in Shear area calculation(s)

A Vco = 3 * (ca1x) * min(ha, 1.5ca1x)	72.00	in^2
A Vc,a Only 1 Row available	72.00	in^2
A Vc,1 Only 1st Row considered; half load on Row (ACI Case 1):	-	in^2
A Vc,2 Only Last Row considered; full load on Row (ACI Case 2):	-	in^2
A Vc,3 Only 1st Row considered; full load on Row (ACI Case 3):	-	in^2
Overall Layout Shear Check (calcuated above):	***OK***	

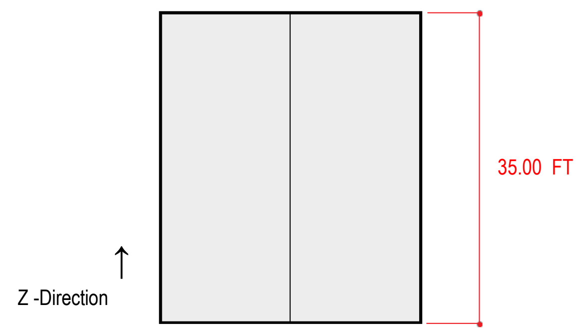
APPENDICES
SECTION

Main Building Wind Generation Report

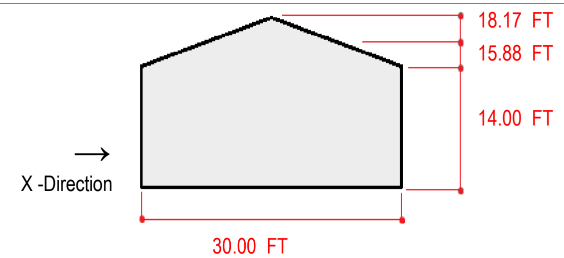
Structure Information		Main Building	
Roof Type:		Gable	
Frame Width (Transverse):		30.0	ft
Base Rail Length (Longitudinal)		35.0	ft
	LEFT	RIGHT	
Eave Height :	14.0	14.0	ft
Roof Pitch (R_p / 12):		3.0	/ 12
Max Structure Height:	H_T =	18.17	ft
Average Structure Height:	H_a =	15.88	ft
Building Enclosure:		Enclosed	

Wind Design Parameters			
Design Wind Speed:	V =	103	mph
Exposure:		C	-
Building Elevation above sea level:		65	ft
Procedure used:		Directional	
Topography:		Flat	
Topographic Factor	K_{zt} =	1.0	

Plan View



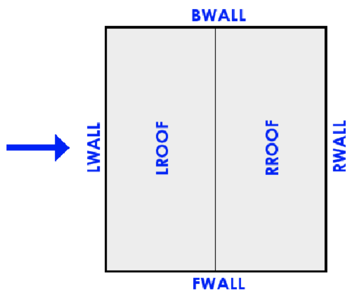
Section View



Main Building Wind Generation Report CONT'D

X Transverse Direction

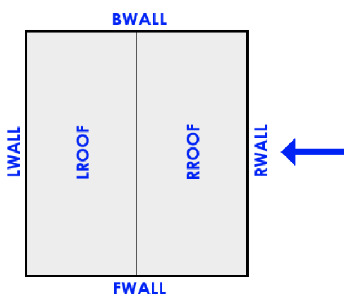
└ Wind Direction Left-to-Right



Group	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
LWALL	Windward Wall	14.00	19.40	0.897	0.80	0.18	15.52	10.38	17.45
BWALL	Side Wall	15.88	19.63	0.897	-0.70	0.18	-13.74	-15.86	-8.79
RWALL	Leeward Wall	15.88	19.63	0.897	-0.50	0.18	-9.82	-12.34	-5.27
FWALL	Side Wall	15.88	19.63	0.897	-0.70	0.18	-13.74	-15.86	-8.79
LROOF	Windward Roof	15.88	19.63	0.897	-0.76	0.18	-14.86	-16.86	-9.80
LROOF	Windward Roof	15.88	19.63	0.897	-0.18	0.18	-3.53	-6.70	0.36
RROOF	Leeward Roof	15.88	19.63	0.897	-0.51	0.18	-9.95	-12.46	-5.39

Per ASCE 7-16 Figure 27.3-1.

└ Wind Direction Right-to-Left



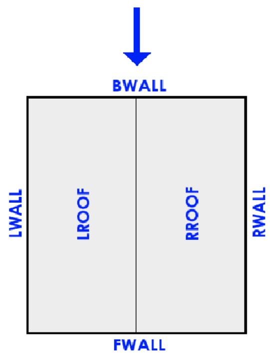
Group	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
LWALL	Leeward Wall	15.88	19.63	0.897	-0.50	0.18	-9.82	-12.34	-5.27
FWALL	Side Wall	15.88	19.63	0.897	-0.70	0.18	-13.74	-15.86	-8.79
RWALL	Windward Wall	14.00	19.40	0.897	0.80	0.18	15.52	10.38	17.45
BWALL	Side Wall	15.88	19.63	0.897	-0.70	0.18	-13.74	-15.86	-8.79
RROOF	Windward Roof	15.88	19.63	0.897	-0.76	0.18	-14.86	-16.86	-9.80
RROOF	Windward Roof	15.88	19.63	0.897	-0.18	0.18	-3.53	-6.70	0.36
LROOF	Leeward Roof	15.88	19.63	0.897	-0.51	0.18	-9.95	-12.46	-5.39

Per ASCE 7-16 Figure 27.3-1.

Main Building Wind Generation Report CONT'D

Z Longitudinal Direction

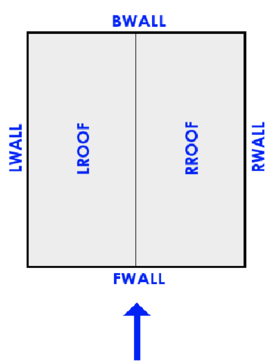
Wind Direction Back-to-Front



Group	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
LWALL	Side Wall	15.88	19.63	0.898	-0.70	0.18	-12.34	-15.87	-8.80
BWALL	Windward Wall	15.88	19.63	0.898	0.80	0.18	14.10	10.56	17.63
BWALL	Windward Wall	18.17	20.20	0.898	0.80	0.18	14.50	10.97	18.04
RWALL	Side Wall	15.88	19.63	0.898	-0.70	0.18	-12.34	-15.87	-8.80
FWALL	Leeward Wall	15.88	19.63	0.898	-0.47	0.18	-8.22	-11.76	-4.69
RROOF & LROOF	Roof	0-7.938	19.63	0.898	-0.90	0.18	-15.86	-19.39	-12.33
	Roof	7.938-15.88	19.63	0.898	-0.90	0.18	-15.86	-19.39	-12.33
	Roof	15.875-31.75	19.63	0.898	-0.50	0.18	-8.81	-12.34	-5.28
	Roof	31.75-35	19.63	0.898	-0.30	0.18	-5.29	-8.82	-1.75
	Roof	0-35	19.63	0.898	-0.18	0.18	-3.17	-6.71	0.36

Per ASCE 7-16 Figure 27.3-1.

Wind Direction Front-to-Back



Group	Surface	z (ft)	q (psf)	G	Cp	GCpi	Ext Pres (psf)	Net w/ +GCpi (psf)	Net w/ -GCpi (psf)
LWALL	Side Wall	15.88	19.63	0.90	-0.70	0.18	-12.34	-15.87	-8.80
BWALL	Leeward Wall	15.88	19.63	0.90	-0.47	0.18	-8.22	-11.76	-4.69
RWALL	Side Wall	15.88	19.63	0.90	-0.70	0.18	-12.34	-15.87	-8.80
FWALL	Windward Wall	15.88	19.63	0.90	0.80	0.18	14.10	10.56	17.63
FWALL	Windward Wall	18.17	20.20	0.90	0.80	0.18	14.50	10.97	18.04
	Roof	0-7.938	19.63	0.898	-0.90	0.18	-15.86	-19.39	-12.33
	Roof	7.938-15.88	19.63	0.898	-0.90	0.18	-15.86	-19.39	-12.33
RROOF & LROOF	Roof	15.875-31.75	19.63	0.898	-0.50	0.18	-8.81	-12.34	-5.28
	Roof	31.75-35	19.63	0.898	-0.30	0.18	-5.29	-8.82	-1.75
	Roof	0-35	19.63	0.898	-0.18	0.18	-3.17	-6.71	0.36

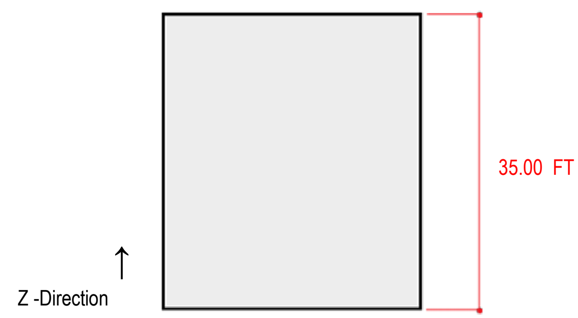
Per ASCE 7-16 Figure 27.3-1.

Left Lean-To 1 Wind Generation Report

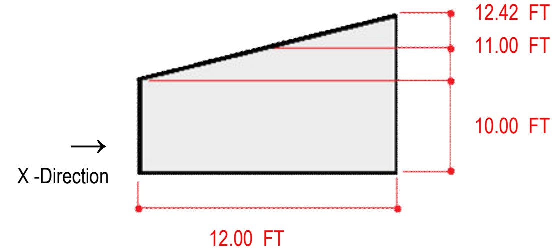
Structure Information		Left Lean-To 1	
Roof Type:		Monoslope	
Frame Width (Transverse):		12.0	ft
Base Rail Length (Longitudinal)		35.0	ft
	LEFT	RIGHT	
Eave Height :	10.0	12.0	ft
Roof Pitch (R_p / 12):		2	/ 12
Max Structure Height:	H_T =	12.42	ft
Average Structure Height:	H_a =	11.00	ft
Building Enclosure:		Open	

Wind Design Parameters			
Design Wind Speed:	V =	103	mph
Exposure:		C	-
Building Elevation above sea level:		65	ft
Procedure used:		Directional	
Topography:		Flat	
Topographic Factor	K_{zt} =	1.0	

Plan View



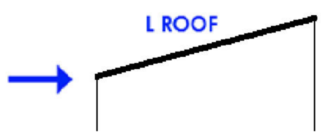
Section View



Left Lean-To 1 Wind Generation Report CONT'D

X Transverse Direction

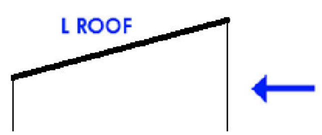
0 degrees



Group	Surface	z (ft)	q (psf)	G	Cn	Ext Pres (psf)	-	-
	Case A - Clear Flow							
LROOF	C_{NW}	11.00	0.85	19.40	-0.68	-12.08		
LROOF	C_{NL}	11.00	0.85	19.40	-1.08	-19.21		
	Case A - Obstructed							
LROOF	C_{NW}	11.00	0.85	19.40	-1.03	-18.28		
LROOF	C_{NL}	11.00	0.85	19.40	-1.50	-26.71		
	Case B - Clear Flow							
LROOF	C_{NW}	11.00	0.85	19.40	-1.53	-27.26		
LROOF	C_{NL}	11.00	0.85	19.40	0.00	0.00		
	Case B - Obstructed							
LROOF	C_{NW}	11.00	0.85	19.40	-1.80	-32.14		
LROOF	C_{NL}	11.00	0.85	19.40	-0.75	-13.32		

This is per ASCE 7-16 Figure 27.3-5 (0 degrees).

180 degrees



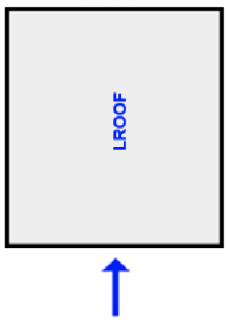
Group	Surface	z (ft)	q (psf)	G	Cn	Ext Pres (psf)	-	-
	Case A - Clear Flow							
LROOF	C_{NW}	11.00	0.85	19.40	1.00	17.89		
LROOF	C_{NL}	11.00	0.85	19.40	1.53	27.18		
	Case A - Obstructed							
LROOF	C_{NW}	11.00	0.85	19.40	-0.04	-0.77		
LROOF	C_{NL}	11.00	0.85	19.40	-1.17	-20.91		
	Case B - Clear Flow							
LROOF	C_{NW}	11.00	0.85	19.40	1.65	29.43		
LROOF	C_{NL}	11.00	0.85	19.40	0.38	6.74		
	Case B - Obstructed							
LROOF	C_{NW}	11.00	0.85	19.40	0.90	16.11		
LROOF	C_{NL}	11.00	0.85	19.40	-0.30	-5.34		

This is per ASCE 7-16 Figure 27.3-5 (180 degrees).

Left Lean-To 1 Wind Generation Report CONT'D

Z Longitudinal Direction

90 degrees



Group	h (ft)	z (ft)	q (psf)	G	Cn	Ext Pres (psf)	-	-
	Case A - Clear Flow							
LROOF	0.0	11.0	0.8	19.4	-0.8	-14.2		
	11.0	11.0	0.8	19.4	-0.6	-10.7		
	22.0	11.0	0.8	19.4	-0.3	-5.3		
	Case A - Obstructed							
LROOF	0.0	11.0	0.8	19.4	-1.2	-21.4		
	11.0	11.0	0.8	19.4	-0.9	-16.0		
	22.0	11.0	0.8	19.4	-0.6	-10.7		
	Case B - Clear Flow							
LROOF	0.0	11.0	0.8	19.4	0.8	14.2		
	11.0	11.0	0.8	19.4	0.5	8.9		
	22.0	11.0	0.8	19.4	0.3	5.3		
	Case B - Obstructed							
LROOF	0.0	11.0	0.8	19.4	0.5	8.9		
	11.0	11.0	0.8	19.4	0.5	8.9		
	22.0	11.0	0.8	19.4	0.3	5.3		

This is per ASCE 7-16 Figure 27.3-7 (90 degrees).

Job Information

	Engineer	Checked	Approved
Name:	AF	RS	
Date:	6/17/2025	6/17/2025	

Project ID	
Project Name	

Structure Type	SPACE FRAME

Number of Nodes	270	Highest Node	336
Number of Elements	492	Highest Beam	1308
Number of Plates	169	Highest Plate	1668

Number of Basic Load Cases	15
Number of Combination Load Cases	46

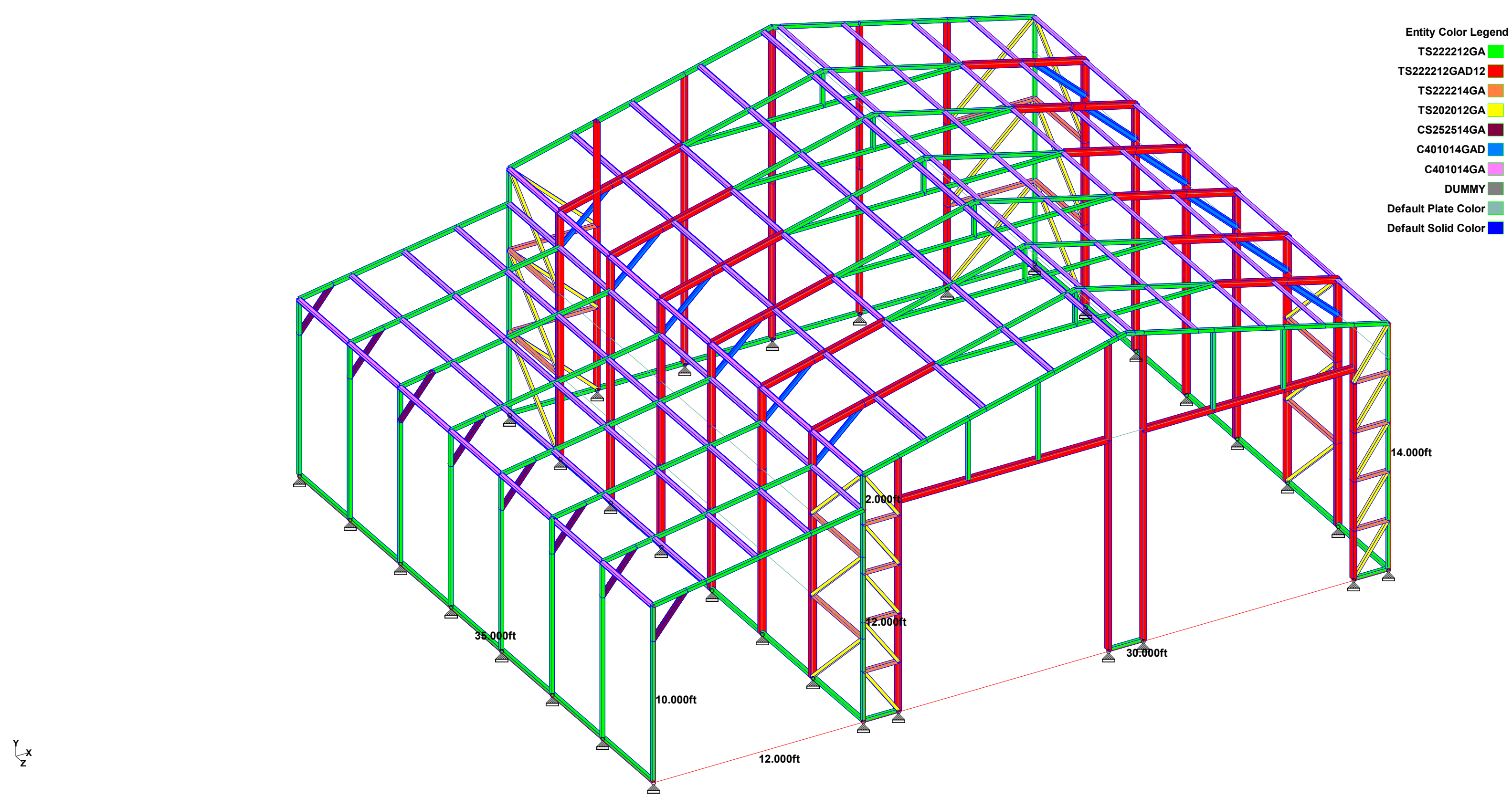
Included in this printout are data for:

View	FRAME

Included in this printout are results for load cases:

Type	L/C	Name
Combination	16	D+LR
Combination	17	D+0.6WXP1
Combination	18	D+0.6WXP2
Combination	19	D+0.6WXN1
Combination	20	D+0.6WXN2
Combination	21	D+0.6WZP1
Combination	22	D+0.6WZP2
Combination	23	D+0.6WZN1
Combination	24	D+0.6WZN2
Combination	25	D+0.75LR+0.45WXP1

A&A ENGINEERING CIVIL • STRUCTURAL 6036 Renaissance Place Tel: 419-292-1983 Software licensed to A & A Engineering CONNECTED User: Kevin O'Malley	Job No **232-24-1425**	Sheet No **SO 2**	Rev
	Part		
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std		Date/Time 17-Jun-2025 13:59



MAIN FRAME OVERVIEW

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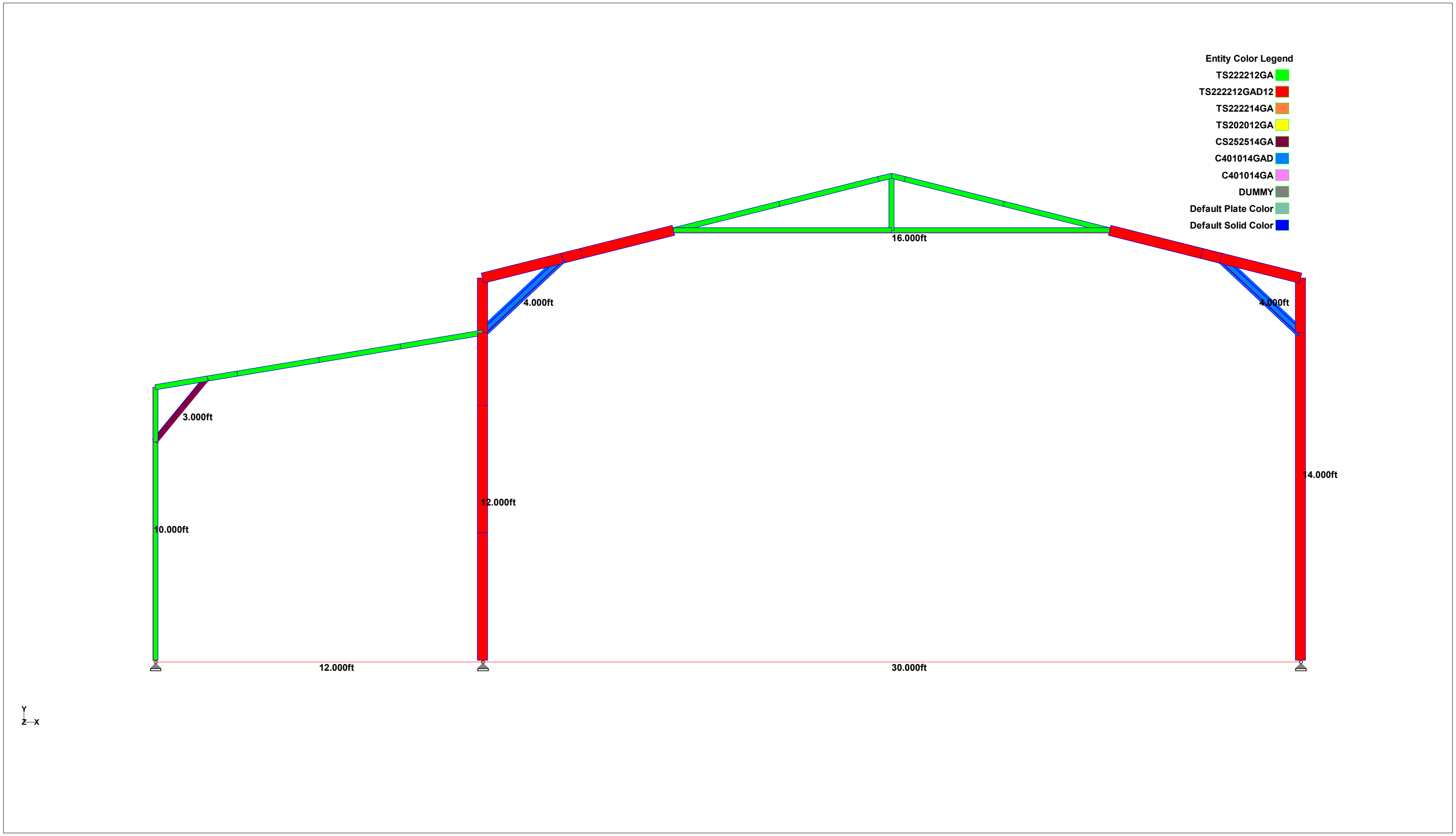
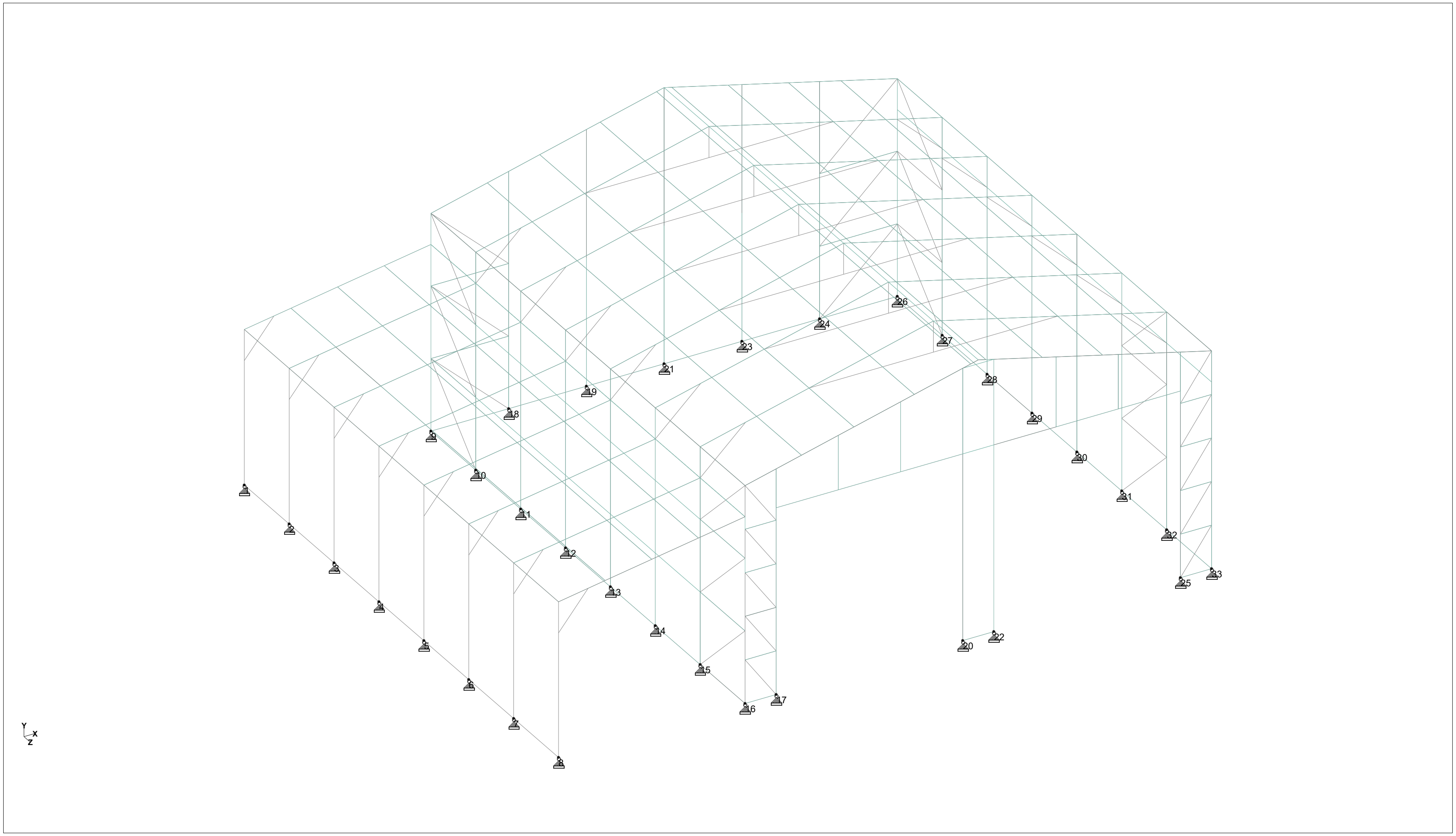
Job No 232-24-1425	Sheet No SO 3	Rev	
Part			
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	



MAIN FRAME

A&A ENGINEERING CIVIL • STRUCTURAL 6036 Renaissance Place Tel: 419-292-1983 Software licensed to A & A Engineering CONNECTED User: Kevin O'Malley	Job No 232-24-1425	Sheet No SO 4	Rev
	Part		
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	



SUPPORTS WITH NODE NUMBER

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
2	TS222212GA	0.933	0.715	0.715	1.070	STEEL
3	TS222212GAD12	1.867	1.430	3.793	1.070	STEEL
4	TS222214GA	0.719	0.564	0.564	0.845	STEEL
5	TS202012GA	0.824	0.493	0.493	0.737	STEEL
6	CS252514GA	0.609	0.409	0.694	0.001	STEEL
7	C401014GAD	0.902	1.196	0.411	0.003	STEEL
8	C401014GA	0.320	0.317	0.117	0.000	STEEL
9	DUMMY	0.020	0.000	0.000	0.000	STEEL

Plate Thickness

Prop	Node A (in)	Node B (in)	Node C (in)	Node D (in)	Material
1	0.014	0.014	0.014	0.014	STEEL

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6.5E -6
2	CONCRETE	3.15E+3	0.170	8.68e-05	5.5E -6
3	ALUMINUM	10E+3	0.330	9.8e-05	12.8E -6
4	STAINLESSSTEEL	28E+3	0.300	0.000	9.9E -6
5	STEEL_36_KSI	29E+3	0.300	0.000	6.5E -6
6	STEEL_50_KSI	29E+3	0.300	0.000	6.5E -6
7	STEEL_275_NMM2	29.7E+3	0.300	0.000	6.67E -6
8	STEEL_355_NMM2	29.7E+3	0.300	0.000	6.67E -6
9	Q235	29.9E+3	0.300	0.000	6.67E -6
10	Q345	29.9E+3	0.300	0.000	6.67E -6
11	Q355	29.9E+3	0.300	0.000	6.67E -6
12	Q390	29.9E+3	0.300	0.000	6.67E -6
13	Q420	29.9E+3	0.300	0.000	6.67E -6
14	Q460	29.9E+3	0.300	0.000	6.67E -6
15	TIMBER	1.5E+3	0.150	0.000	3E -6

Primary Load Cases

Number	Name	Type
1	DEAD	Dead
2	ROOFLIVE	Roof Live
3	XP1	Wind
4	XP2	Wind
5	XN1	Wind
6	XN2	Wind
7	ZP1	Wind
8	ZP2	Wind
9	ZN1	Wind
10	ZN2	Wind
11	EXP	None
12	EXN	None
13	EZP	None
14	EZN	None
15	EY	None

Combination Load Cases

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
16	D+LR	1	DEAD	1.00
		2	ROOFLIVE	1.00
17	D+0.6WXP1	1	DEAD	1.00
		3	XP1	0.60
18	D+0.6WXP2	1	DEAD	1.00
		4	XP2	0.60
19	D+0.6WXN1	1	DEAD	1.00
		5	XN1	0.60
20	D+0.6WXN2	1	DEAD	1.00
		6	XN2	0.60
21	D+0.6WZP1	1	DEAD	1.00
		7	ZP1	0.60
22	D+0.6WZP2	1	DEAD	1.00
		8	ZP2	0.60
23	D+0.6WZN1	1	DEAD	1.00
		9	ZN1	0.60
24	D+0.6WZN2	1	DEAD	1.00
		10	ZN2	0.60
25	D+0.75LR+0.45WXP1	1	DEAD	1.00
		2	ROOFLIVE	0.75
		3	XP1	0.45
26	D+0.75LR+0.45WXP2	1	DEAD	1.00
		2	ROOFLIVE	0.75
		4	XP2	0.45
27	D+0.75LR+0.45WXN1	1	DEAD	1.00

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
		2	ROOFLIVE	0.75
		5	XN1	0.45
28	D+0.75LR+0.45WXN2	1	DEAD	1.00
		2	ROOFLIVE	0.75
		6	XN2	0.45
29	D+0.75LR+0.45WZP1	1	DEAD	1.00
		2	ROOFLIVE	0.75
		7	ZP1	0.45
30	D+0.75LR+0.45WZP2	1	DEAD	1.00
		2	ROOFLIVE	0.75
		8	ZP2	0.45
31	D+0.75LR+0.45WZN1	1	DEAD	1.00
		2	ROOFLIVE	0.75
		9	ZN1	0.45
32	D+0.75LR+0.45WZN2	1	DEAD	1.00
		2	ROOFLIVE	0.75
		10	ZN2	0.45
33	0.6D+0.6WXP1	1	DEAD	0.60
		3	XP1	0.60
34	0.6D+0.6WXP2	1	DEAD	0.60
		4	XP2	0.60
35	0.6D+0.6WXN1	1	DEAD	0.60
		5	XN1	0.60
36	0.6D+0.6WXN2	1	DEAD	0.60
		6	XN2	0.60
37	0.6D+0.6WZP1	1	DEAD	0.60
		7	ZP1	0.60
38	0.6D+0.6WZP2	1	DEAD	0.60
		8	ZP2	0.60
39	0.6D+0.6WZN1	1	DEAD	0.60
		9	ZN1	0.60
40	0.6D+0.6WZN2	1	DEAD	0.60
		10	ZN2	0.60
41	D+0.7EY+0.7EXP	1	DEAD	1.00
		15	EY	0.70
		11	EXP	0.70
42	D+0.7EY+0.7EXN	1	DEAD	1.00
		15	EY	0.70
		12	EXN	0.70
43	D+0.7EY+0.7EZP	1	DEAD	1.00
		15	EY	0.70
		13	EZP	0.70
44	D+0.7EY+0.7EZN	1	DEAD	1.00
		15	EY	0.70
		14	EZN	0.70

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
45	0.6D-0.7EY+0.7EXP	1	DEAD	0.60
		15	EY	0.70
		11	EXP	0.70
46	0.6D-0.7EY+0.7EXN	1	DEAD	0.60
		15	EY	-0.70
		12	EXN	0.70
47	0.6D-0.7EY+0.7EZP	1	DEAD	0.60
		15	EY	-0.70
		13	EZP	0.70
48	0.6D-0.7EY+0.7EZN	1	DEAD	0.60
		15	EY	-0.70
		14	EZN	0.70
49	D+0.75LR+0.525EY+0.525EXP	1	DEAD	1.00
		2	ROOFLIVE	0.75
		15	EY	0.52
		11	EXP	0.52
50	D+0.75LR+0.525EY+0.525EXN	1	DEAD	1.00
		2	ROOFLIVE	0.75
		15	EY	0.52
		12	EXN	0.52
51	D+0.75LR+0.525EY+0.525EZP	1	DEAD	1.00
		2	ROOFLIVE	0.75
		15	EY	0.52
		13	EZP	0.52
52	D+0.75LR+0.525EY+0.525EZN	1	DEAD	1.00
		2	ROOFLIVE	0.75
		15	EY	0.52
		14	EZN	0.52
53	D+0.5LR+0.49 WXP1	1	DEAD	1.00
		2	ROOFLIVE	0.50
		3	XP1	0.49
54	D+0.5LR+0.49 WXP2	1	DEAD	1.00
		2	ROOFLIVE	0.50
		4	XP2	0.49
55	D+0.5LR+0.49 WXN1	1	DEAD	1.00
		2	ROOFLIVE	0.50
		5	XN1	0.49
56	D+0.5LR+0.49 WXN2	1	DEAD	1.00
		2	ROOFLIVE	0.50
		6	XN2	0.49
57	D+0.5LR+0.49 WZP1	1	DEAD	1.00
		2	ROOFLIVE	0.50
		7	ZP1	0.49
58	D+0.5LR+0.49 WZP2	1	DEAD	1.00
		2	ROOFLIVE	0.50

Combination Load Cases Cont...

Comb.	Combination L/C Name	Primary	Primary L/C Name	Factor
		8	ZP2	0.49
59	D+0.5LR+0.49 WZN1	1	DEAD	1.00
		2	ROOFLIVE	0.50
		9	ZN1	0.49
60	D+0.5LR+0.49 WZN2	1	DEAD	1.00
		2	ROOFLIVE	0.50
		10	ZN2	0.49
61	D+0.5LR	1	DEAD	1.00
		2	ROOFLIVE	0.50

Beam End Displacement Summary (Main Building)

Displacements shown in italic indicate the presence of an offset

	Beam	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	401	315	60:D+0.5LR+0.	**0.300**	0.000	0.010	0.301
Min X	301	151	58:D+0.5LR+0.	**-0.159**	-0.000	0.001	0.159
Max Y	338	199	53:D+0.5LR+0.	-0.034	**0.058**	0.001	0.067
Min Y	149	266	60:D+0.5LR+0.	-0.050	**-0.514**	0.002	0.516
Max Z	162	270	53:D+0.5LR+0.	0.021	-0.003	**0.575**	0.576
Min Z	358	335	54:D+0.5LR+0.	0.007	-0.001	**-0.361**	0.362
Max Rst	162	270	53:D+0.5LR+0.	0.021	-0.003	0.575	**0.576**

Beam End Displacement Summary (Lean-to)

Displacements shown in italic indicate the presence of an offset

	Beam	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	46	138	59:D+0.5LR+0.	**0.086**	-0.629	0.007	0.635
Min X	10	102	59:D+0.5LR+0.	**-0.334**	-0.002	0.005	0.334
Max Y	41	133	56:D+0.5LR+0.	-0.065	**0.092**	0.007	0.113
Min Y	45	137	59:D+0.5LR+0.	0.085	**-0.648**	0.007	0.654
Max Z	15	107	60:D+0.5LR+0.	-0.176	-0.001	**0.045**	0.182
Min Z	8	100	59:D+0.5LR+0.	-0.197	-0.001	**-0.029**	0.199
Max Rst	45	137	59:D+0.5LR+0.	0.085	-0.648	0.007	**0.654**

Beam Maximum Forces by Section Property

		Axial	Shear		Torsion	Bending	
Section		Max Fx (kip)	Max Fy (kip)	Max Fz (kip)	Max Mx (kip-in)	Max My (kip-in)	Max Mz (kip-in)
TS222212GA	Max +ve	2.367	1.347	1.050	6.199	6.032	19.532
	Max -ve	-6.158	-2.000	-1.049	-5.947	-4.868	-16.307
TS222212GAD12	Max +ve	2.277	1.028	0.485	2.264	7.683	20.794
	Max -ve	-2.771	-0.976	-1.436	-2.199	-6.100	-20.445
TS222214GA	Max +ve	0.484	0.053	0.058	0.544	2.582	1.042
	Max -ve	-1.222	-0.058	-0.089	-0.610	-2.527	-0.897
TS202012GA	Max +ve	0.648	0.023	0.055	0.638	2.883	0.577
	Max -ve	-0.747	-0.023	-0.046	-0.617	-1.741	-0.446
CS252514GA	Max +ve	1.013	0	0.002	0	0	0
	Max -ve	-0.694	0	-0.002	0	0	0
C401014GAD	Max +ve	2.798	0	0.004	0	0	0
	Max -ve	-1.207	0	-0.004	0	0	0
C401014GA	Max +ve	2.237	0.157	0.043	0.000	0.764	2.891
	Max -ve	-1.438	-0.159	-0.044	-0.000	-0.712	-2.714

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip-in)	MY (kip-in)	MZ (kip-in)
Max FX	20	16:D+LR	**1.416**	1.844	-0.017	0	0	0
Min FX	22	16:D+LR	**-1.436**	2.110	-0.024	0	0	0
Max FY	26	26:D+0.75LR+(	-0.546	**4.232**	0.429	0	0	0
Min FY	20	34:0.6D+0.6W	-0.301	**-2.369**	-0.511	0	0	0
Max FZ	33	33:0.6D+0.6W	-0.113	0.977	**0.446**	0	0	0
Min FZ	31	30:D+0.75LR+(	-0.297	-1.092	**-0.777**	0	0	0
Max MX	1	16:D+LR	0.049	0.393	0.017	**0**	0	0
Min MX	1	16:D+LR	0.049	0.393	0.017	**0**	0	0
Max MY	1	16:D+LR	0.049	0.393	0.017	0	**0**	0
Min MY	1	16:D+LR	0.049	0.393	0.017	0	**0**	0
Max MZ	1	16:D+LR	0.049	0.393	0.017	0	0	**0**
Min MZ	1	16:D+LR	0.049	0.393	0.017	0	0	**0**

Statics Check Results

L/C		FX (kip)	FY (kip)	FZ (kip)	MX (kip˘in)	MY (kip˘in)	MZ (kip˘in)
1:DEAD	Loads	-0.000	-10.847	0.000	-2.32E+3	-0.000	-1.42E+3
1:DEAD	Reactions	0.000	10.847	-0.000	2.32E+3	0.000	1.42E+3
	Difference	0.000	-0.000	-0.000	-0.000	0.000	-0.000
2:ROOFLIVE	Loads	0.000	-29.398	0.000	-6.17E+3	-0.001	-3.17E+3
2:ROOFLIVE	Reactions	-0.000	29.398	-0.000	6.17E+3	0.001	3.17E+3
	Difference	0.000	-0.000	-0.000	-0.000	0.000	-0.000
3:XP1	Loads	11.899	23.465	0.000	4.93E+3	-2.5E+3	979.910
3:XP1	Reactions	-11.899	-23.465	-0.000	-4.93E+3	2.5E+3	-979.910
	Difference	-0.000	-0.000	-0.000	0.000	-0.000	-0.000
4:XP2	Loads	13.754	21.280	0.000	4.47E+3	-2.89E+3	-49.908
4:XP2	Reactions	-13.754	-21.280	-0.000	-4.47E+3	2.89E+3	49.908
	Difference	-0.000	-0.000	-0.000	0.000	-0.000	-0.000
5:XN1	Loads	8.646	19.421	0.000	4.08E+3	-1.82E+3	-170.556
5:XN1	Reactions	-8.646	-19.421	-0.000	-4.08E+3	1.82E+3	170.556
	Difference	-0.000	-0.000	-0.000	0.000	-0.000	-0.000
6:XN2	Loads	9.642	16.139	0.000	3.39E+3	-2.02E+3	-1.01E+3
6:XN2	Reactions	-9.642	-16.139	-0.000	-3.39E+3	2.02E+3	1.01E+3
	Difference	-0.000	-0.000	-0.000	0.000	0.000	-0.000
7:ZP1	Loads	-0.996	21.957	10.730	5.64E+3	-1.72E+3	2.58E+3
7:ZP1	Reactions	0.996	-21.957	-10.730	-5.64E+3	1.72E+3	-2.58E+3
	Difference	-0.000	0.000	0.000	0.000	-0.000	-0.000
8:ZP2	Loads	-1.495	16.022	10.730	4.39E+3	-1.62E+3	819.293
8:ZP2	Reactions	1.495	-16.022	-10.730	-4.39E+3	1.62E+3	-819.293
	Difference	-0.000	-0.000	0.000	0.000	-0.000	-0.000
9:ZN1	Loads	0.996	2.568	10.725	1.56E+3	-2.14E+3	1.84E+3
9:ZN1	Reactions	-0.996	-2.568	-10.725	-1.56E+3	2.14E+3	-1.84E+3
	Difference	0.000	0.000	0.000	0.000	-0.000	0.000
10:ZN2	Loads	0.719	-4.117	10.725	157.669	-2.08E+3	100.370
10:ZN2	Reactions	-0.719	4.117	-10.725	-157.669	2.08E+3	-100.370
	Difference	0.000	-0.000	0.000	0.000	-0.000	-0.000
11:EXP	Loads	2.744	0	0	0	-576.242	-433.093
11:EXP	Reactions	-2.744	-0.000	0.000	0.000	576.242	433.093
	Difference	-0.000	-0.000	0.000	0.000	0.000	0.000
12:EXN	Loads	-2.744	0	0	0	576.242	433.093
12:EXN	Reactions	2.744	0.000	-0.000	-0.000	-576.242	-433.093
	Difference	0.000	0.000	-0.000	-0.000	-0.000	-0.000
13:EZP	Loads	0	0	1.439	225.408	-144.612	0
13:EZP	Reactions	-0.000	0.000	-1.439	-225.408	144.612	0.000
	Difference	-0.000	0.000	0.000	0.000	-0.000	0.000
14:EZN	Loads	0	0	-1.439	-225.408	144.612	0
14:EZN	Reactions	0.000	-0.000	1.439	225.408	-144.612	-0.000
	Difference	0.000	-0.000	-0.000	-0.000	0.000	-0.000
15:EY	Loads	0	-1.281	0	-269.010	0	-139.356
15:EY	Reactions	0.000	1.281	0.000	269.010	-0.000	139.356
	Difference	0.000	-0.000	0.000	-0.000	-0.000	-0.000

Statics Check Results Cont...

L/C		FX (kip)	FY (kip)	FZ (kip)	MX (kip˘in)	MY (kip˘in)	MZ (kip˘in)
	Difference	0.000	-0.000	0.000	-0.000	-0.000	-0.000

Reaction Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip˘in)	MY (kip˘in)	MZ (kip˘in)
1	+ve	0.051	0.416	0.018	0	0	0
1	+ve	Load: 31	Load: 31	Load: 31	-	-	-
1	-ve	-0.032	-0.224	-0.015	0	0	0
1	-ve	Load: 36	Load: 36	Load: 36	-	-	-
2	+ve	0.111	0.781	0.000	0	0	0
2	+ve	Load: 31	Load: 31	Load: 39	-	-	-
2	-ve	-0.077	-0.474	-0.002	0	0	0
2	-ve	Load: 36	Load: 36	Load: 22	-	-	-
3	+ve	0.111	0.791	0.000	0	0	0
3	+ve	Load: 31	Load: 31	Load: 33	-	-	-
3	-ve	-0.076	-0.482	-0.002	0	0	0
3	-ve	Load: 36	Load: 36	Load: 30	-	-	-
4	+ve	0.109	0.785	0.001	0	0	0
4	+ve	Load: 31	Load: 31	Load: 33	-	-	-
4	-ve	-0.074	-0.475	-0.002	0	0	0
4	-ve	Load: 36	Load: 36	Load: 30	-	-	-
5	+ve	0.109	0.785	0.001	0	0	0
5	+ve	Load: 31	Load: 31	Load: 33	-	-	-
5	-ve	-0.074	-0.475	-0.002	0	0	0
5	-ve	Load: 36	Load: 36	Load: 30	-	-	-
6	+ve	0.111	0.792	0.001	0	0	0
6	+ve	Load: 31	Load: 31	Load: 33	-	-	-
6	-ve	-0.076	-0.481	-0.003	0	0	0
6	-ve	Load: 36	Load: 36	Load: 30	-	-	-
7	+ve	0.111	0.782	0.001	0	0	0
7	+ve	Load: 31	Load: 31	Load: 33	-	-	-
7	-ve	-0.077	-0.474	-0.003	0	0	0
7	-ve	Load: 36	Load: 36	Load: 30	-	-	-
8	+ve	0.051	0.424	0.013	0	0	0
8	+ve	Load: 31	Load: 31	Load: 36	-	-	-
8	-ve	-0.033	-0.218	-0.020	0	0	0
8	-ve	Load: 36	Load: 36	Load: 31	-	-	-
9	+ve	0.391	3.240	0.306	0	0	0
9	+ve	Load: 30	Load: 16	Load: 16	-	-	-
9	-ve	-0.325	-1.775	-0.219	0	0	0
9	-ve	Load: 34	Load: 34	Load: 37	-	-	-
10	+ve	0.285	2.275	0.272	0	0	0
10	+ve	Load: 22	Load: 16	Load: 33	-	-	-

Reaction Envelope Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip`in)	MY (kip`in)	MZ (kip`in)
10	+ve	Load: 22	Load: 16	Load: 33	-	-	-
10	-ve	-0.269	-1.068	-0.585	0	0	0
10	-ve	Load: 35	Load: 33	Load: 30	-	-	-
11	+ve	0.282	1.273	0.140	0	0	0
11	+ve	Load: 22	Load: 16	Load: 52	-	-	-
11	-ve	-0.265	-0.499	-0.274	0	0	0
11	-ve	Load: 35	Load: 33	Load: 37	-	-	-
12	+ve	0.265	1.124	0.120	0	0	0
12	+ve	Load: 38	Load: 16	Load: 52	-	-	-
12	-ve	-0.257	-0.526	-0.256	0	0	0
12	-ve	Load: 19	Load: 38	Load: 39	-	-	-
13	+ve	0.262	1.143	0.084	0	0	0
13	+ve	Load: 38	Load: 16	Load: 48	-	-	-
13	-ve	-0.256	-0.505	-0.262	0	0	0
13	-ve	Load: 19	Load: 38	Load: 23	-	-	-
14	+ve	0.271	1.272	0.081	0	0	0
14	+ve	Load: 38	Load: 16	Load: 48	-	-	-
14	-ve	-0.265	-0.569	-0.276	0	0	0
14	-ve	Load: 19	Load: 33	Load: 23	-	-	-
15	+ve	0.235	1.916	0.273	0	0	0
15	+ve	Load: 38	Load: 16	Load: 50	-	-	-
15	-ve	-0.256	-1.279	-0.737	0	0	0
15	-ve	Load: 19	Load: 33	Load: 34	-	-	-
16	+ve	0.237	2.563	0.290	0	0	0
16	+ve	Load: 30	Load: 30	Load: 33	-	-	-
16	-ve	-0.174	-1.570	-0.259	0	0	0
16	-ve	Load: 34	Load: 34	Load: 32	-	-	-
17	+ve	0.324	2.934	0.001	0	0	0
17	+ve	Load: 38	Load: 26	Load: 46	-	-	-
17	-ve	-0.492	-0.884	-0.657	0	0	0
17	-ve	Load: 17	Load: 38	Load: 17	-	-	-
18	+ve	1.043	0.883	0.337	0	0	0
18	+ve	Load: 30	Load: 16	Load: 33	-	-	-
18	-ve	-0.890	-0.615	-0.370	0	0	0
18	-ve	Load: 33	Load: 38	Load: 24	-	-	-
19	+ve	0.201	0.831	0.397	0	0	0
19	+ve	Load: 46	Load: 16	Load: 18	-	-	-
19	-ve	-0.600	-0.239	-0.450	0	0	0
19	-ve	Load: 18	Load: 33	Load: 39	-	-	-
20	+ve	1.416	2.094	0	0	0	0
20	+ve	Load: 16	Load: 50	-	-	-	-
20	-ve	-0.417	-2.369	-0.512	0	0	0
20	-ve	Load: 33	Load: 34	Load: 18	-	-	-
21	+ve	0.198	0.639	0.372	0	0	0

Reaction Envelope Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip`in)	MY (kip`in)	MZ (kip`in)
21	+ve	Load: 46	Load: 32	Load: 18	-	-	-
21	-ve	-0.594	-0.107	-0.421	0	0	0
21	-ve	Load: 18	Load: 33	Load: 39	-	-	-
22	+ve	0.022	2.990	0	0	0	0
22	+ve	Load: 33	Load: 26	-	-	-	-
22	-ve	-1.436	-0.434	-0.517	0	0	0
22	-ve	Load: 16	Load: 46	Load: 17	-	-	-
23	+ve	0.202	1.013	0.398	0	0	0
23	+ve	Load: 46	Load: 16	Load: 17	-	-	-
23	-ve	-0.638	-0.007	-0.449	0	0	0
23	-ve	Load: 18	Load: 37	Load: 39	-	-	-
24	+ve	0.167	1.632	0.305	0	0	0
24	+ve	Load: 46	Load: 16	Load: 34	-	-	-
24	-ve	-0.912	-0.394	-0.378	0	0	0
24	-ve	Load: 26	Load: 37	Load: 24	-	-	-
25	+ve	0.186	2.029	0	0	0	0
25	+ve	Load: 50	Load: 50	-	-	-	-
25	-ve	-0.394	-2.131	-0.738	0	0	0
25	-ve	Load: 34	Load: 34	Load: 17	-	-	-
26	+ve	0.048	4.232	0.429	0	0	0
26	+ve	Load: 46	Load: 26	Load: 26	-	-	-
26	-ve	-0.546	-0.876	-0.204	0	0	0
26	-ve	Load: 26	Load: 37	Load: 37	-	-	-
27	+ve	0	1.102	0.342	0	0	0
27	+ve	-	Load: 16	Load: 16	-	-	-
27	-ve	-0.267	-0.191	-0.448	0	0	0
27	-ve	Load: 22	Load: 37	Load: 39	-	-	-
28	+ve	0	0.180	0.282	0	0	0
28	+ve	-	Load: 42	Load: 26	-	-	-
28	-ve	-0.296	-0.350	-0.258	0	0	0
28	-ve	Load: 30	Load: 38	Load: 37	-	-	-
29	+ve	0	0.148	0.219	0	0	0
29	+ve	-	Load: 42	Load: 26	-	-	-
29	-ve	-0.293	-0.570	-0.254	0	0	0
29	-ve	Load: 30	Load: 30	Load: 37	-	-	-
30	+ve	0	0.144	0.173	0	0	0
30	+ve	-	Load: 42	Load: 34	-	-	-
30	-ve	-0.306	-0.607	-0.267	0	0	0
30	-ve	Load: 30	Load: 30	Load: 22	-	-	-
31	+ve	0	0.167	0.250	0	0	0
31	+ve	-	Load: 17	Load: 34	-	-	-
31	-ve	-0.297	-1.092	-0.777	0	0	0
31	-ve	Load: 30	Load: 30	Load: 30	-	-	-
32	+ve	0	1.876	0.215	0	0	0

Reaction Envelope Cont...

		Horizontal	Vertical	Horizontal	Moment		
Node	Env	FX (kip)	FY (kip)	FZ (kip)	MX (kip`in)	MY (kip`in)	MZ (kip`in)
32	+ve	-	Load: 16	Load: 34	-	-	-
32	-ve	-0.279	-0.119	-0.283	0	0	0
32	-ve	Load: 22	Load: 33	Load: 30	-	-	-
33	+ve	0.034	3.167	0.446	0	0	0
33	+ve	Load: 46	Load: 26	Load: 33	-	-	-
33	-ve	-0.225	-0.177	-0.365	0	0	0
33	-ve	Load: 26	Load: 46	Load: 16	-	-	-

Utilization Ratio

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
1	TS222212	TS222212	0.038	1.000	0.038	Eq. H3-1	31	0.933	0.715	0.715	1.070
2	TS222212	TS222212	0.010	1.000	0.010	Eq. H3-1	31	0.933	0.715	0.715	1.070
3	TS222212	TS222212	0.007	1.000	0.007	Eq. H1-1b	30	0.933	0.715	0.715	1.070
4	TS222212	TS222212	0.007	1.000	0.007	Eq. H1-1b	30	0.933	0.715	0.715	1.070
5	TS222212	TS222212	0.008	1.000	0.008	Eq. H1-1b	30	0.933	0.715	0.715	1.070
6	TS222212	TS222212	0.010	1.000	0.010	Eq. H3-1	31	0.933	0.715	0.715	1.070
7	TS222212	TS222212	0.039	1.000	0.039	Eq. H3-1	31	0.933	0.715	0.715	1.070
8	TS222212	TS222212	0.313	1.000	0.313	Eq. H1-1b	31	0.933	0.715	0.715	1.070
9	TS222212	TS222212	0.488	1.000	0.488	Eq. H1-1b	31	0.933	0.715	0.715	1.070
10	TS222212	TS222212	0.498	1.000	0.498	Eq. H1-1b	31	0.933	0.715	0.715	1.070
11	TS222212	TS222212	0.497	1.000	0.497	Eq. H1-1b	31	0.933	0.715	0.715	1.070
12	TS222212	TS222212	0.497	1.000	0.497	Eq. H1-1b	31	0.933	0.715	0.715	1.070
13	TS222212	TS222212	0.500	1.000	0.500	Eq. H1-1b	31	0.933	0.715	0.715	1.070
14	TS222212	TS222212	0.492	1.000	0.492	Eq. H1-1b	31	0.933	0.715	0.715	1.070
15	TS222212	TS222212	0.317	1.000	0.317	Eq. H1-1b	31	0.933	0.715	0.715	1.070
16	TS222212	TS222212	0.297	1.000	0.297	Eq. H1-1b	31	0.933	0.715	0.715	1.070
17	TS222212	TS222212	0.456	1.000	0.456	Eq. H1-1b	31	0.933	0.715	0.715	1.070
18	TS222212	TS222212	0.466	1.000	0.466	Eq. H1-1b	31	0.933	0.715	0.715	1.070
19	TS222212	TS222212	0.465	1.000	0.465	Eq. H1-1b	31	0.933	0.715	0.715	1.070
20	TS222212	TS222212	0.466	1.000	0.466	Eq. H1-1b	31	0.933	0.715	0.715	1.070
21	TS222212	TS222212	0.468	1.000	0.468	Eq. H1-1b	31	0.933	0.715	0.715	1.070
22	TS222212	TS222212	0.460	1.000	0.460	Eq. H1-1b	31	0.933	0.715	0.715	1.070
23	TS222212	TS222212	0.301	1.000	0.301	Eq. H1-1b	31	0.933	0.715	0.715	1.070
24	TS222212	TS222212	0.083	1.000	0.083	Eq. H3-1	31	0.933	0.715	0.715	1.070
25	TS222212	TS222212	0.108	1.000	0.108	Eq. H1-1b	31	0.933	0.715	0.715	1.070
26	TS222212	TS222212	0.109	1.000	0.109	Eq. H1-1b	31	0.933	0.715	0.715	1.070
27	TS222212	TS222212	0.107	1.000	0.107	Eq. H1-1b	31	0.933	0.715	0.715	1.070
28	TS222212	TS222212	0.106	1.000	0.106	Eq. H1-1b	31	0.933	0.715	0.715	1.070
29	TS222212	TS222212	0.108	1.000	0.108	Eq. H1-1b	31	0.933	0.715	0.715	1.070
30	TS222212	TS222212	0.108	1.000	0.108	Eq. H1-1b	31	0.933	0.715	0.715	1.070
31	TS222212	TS222212	0.084	1.000	0.084	Eq. H3-1	31	0.933	0.715	0.715	1.070
32	TS222212	TS222212	0.140	1.000	0.140	Eq. H1-1b	31	0.933	0.715	0.715	1.070

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
33	TS222212	TS222212	0.262	1.000	0.262	Eq. H1-1b	31	0.933	0.715	0.715	1.070
34	TS222212	TS222212	0.270	1.000	0.270	Eq. H1-1b	31	0.933	0.715	0.715	1.070
35	TS222212	TS222212	0.270	1.000	0.270	Eq. H1-1b	31	0.933	0.715	0.715	1.070
36	TS222212	TS222212	0.270	1.000	0.270	Eq. H1-1b	31	0.933	0.715	0.715	1.070
37	TS222212	TS222212	0.270	1.000	0.270	Eq. H1-1b	31	0.933	0.715	0.715	1.070
38	TS222212	TS222212	0.263	1.000	0.263	Eq. H1-1b	31	0.933	0.715	0.715	1.070
39	TS222212	TS222212	0.141	1.000	0.141	Eq. H1-1b	31	0.933	0.715	0.715	1.070
40	TS222212	TS222212	0.265	1.000	0.265	Eq. H1-1b	31	0.933	0.715	0.715	1.070
41	TS222212	TS222212	0.483	1.000	0.483	Eq. H1-1b	31	0.933	0.715	0.715	1.070
42	TS222212	TS222212	0.501	1.000	0.501	Eq. H1-1b	31	0.933	0.715	0.715	1.070
43	TS222212	TS222212	0.500	1.000	0.500	Eq. H1-1b	31	0.933	0.715	0.715	1.070
44	TS222212	TS222212	0.499	1.000	0.499	Eq. H1-1b	31	0.933	0.715	0.715	1.070
45	TS222212	TS222212	0.503	1.000	0.503	Eq. H1-1b	31	0.933	0.715	0.715	1.070
46	TS222212	TS222212	0.488	1.000	0.488	Eq. H1-1b	31	0.933	0.715	0.715	1.070
47	TS222212	TS222212	0.271	1.000	0.271	Eq. H1-1b	31	0.933	0.715	0.715	1.070
48	TS222212	TS222212	0.270	1.000	0.270	Eq. H1-1b	31	0.933	0.715	0.715	1.070
49	TS222212	TS222212	0.483	1.000	0.483	Eq. H1-1b	31	0.933	0.715	0.715	1.070
50	TS222212	TS222212	0.502	1.000	0.502	Eq. H1-1b	31	0.933	0.715	0.715	1.070
51	TS222212	TS222212	0.502	1.000	0.502	Eq. H1-1b	31	0.933	0.715	0.715	1.070
52	TS222212	TS222212	0.502	1.000	0.502	Eq. H1-1b	31	0.933	0.715	0.715	1.070
53	TS222212	TS222212	0.505	1.000	0.505	Eq. H1-1b	31	0.933	0.715	0.715	1.070
54	TS222212	TS222212	0.488	1.000	0.488	Eq. H1-1b	31	0.933	0.715	0.715	1.070
55	TS222212	TS222212	0.276	1.000	0.276	Eq. H1-1b	31	0.933	0.715	0.715	1.070
56	TS222212	TS222212	0.449	1.000	0.449	Eq. H1-1b	31	0.933	0.715	0.715	1.070
57	TS222212	TS222212	0.871	1.000	0.871	Eq. H1-1b	31	0.933	0.715	0.715	1.070
58	TS222212	TS222212	0.873	1.000	0.873	Eq. H1-1b	31	0.933	0.715	0.715	1.070
59	TS222212	TS222212	0.863	1.000	0.863	Eq. H1-1b	31	0.933	0.715	0.715	1.070
60	TS222212	TS222212	0.862	1.000	0.862	Eq. H1-1b	31	0.933	0.715	0.715	1.070
61	TS222212	TS222212	0.871	1.000	0.871	Eq. H1-1b	31	0.933	0.715	0.715	1.070
62	TS222212	TS222212	0.864	1.000	0.864	Eq. H1-1b	31	0.933	0.715	0.715	1.070
63	TS222212	TS222212	0.431	1.000	0.431	Eq. H1-1b	31	0.933	0.715	0.715	1.070
64	TS222212	TS222212	0.054	1.000	0.054	Eq. H1-1b	22	0.933	0.715	0.715	1.070
65	TS222212	TS222212	0.026	1.000	0.026	Eq. H1-1b	22	0.933	0.715	0.715	1.070
66	TS222212	TS222212	0.024	1.000	0.024	Eq. H1-1b	19	0.933	0.715	0.715	1.070
67	TS222212	TS222212	0.022	1.000	0.022	Eq. H1-1b	20	0.933	0.715	0.715	1.070
68	TS222212	TS222212	0.027	1.000	0.027	Eq. H1-1b	22	0.933	0.715	0.715	1.070
69	TS222212	TS222212	0.024	1.000	0.024	Eq. H1-1b	19	0.933	0.715	0.715	1.070
70	TS222212	TS222212	0.047	1.000	0.047	Eq. H1-1b	38	0.933	0.715	0.715	1.070
71	TS222212	TS222212	0.066	1.000	0.066	Sec. E1	30	0.933	0.715	0.715	1.070
72	TS222212	TS222212	0.090	1.000	0.090	Eq. H1-1b	16	0.933	0.715	0.715	1.070
73	TS222212	TS222212	0.054	1.000	0.054	Eq. H1-1b	30	0.933	0.715	0.715	1.070
74	TS222212	TS222212	0.106	1.000	0.106	Eq. H3-1	22	0.933	0.715	0.715	1.070
75	TS222212	TS222212	0.066	1.000	0.066	Sec. E1	16	0.933	0.715	0.715	1.070
76	TS222212	TS222212	0.046	1.000	0.046	Sec. E1	16	0.933	0.715	0.715	1.070
77	TS222212	TS222212	0.114	1.000	0.114	Eq. H3-1	22	0.933	0.715	0.715	1.070

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Job No 232-24-1425	Sheet No SO 17		Rev
Part			
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
78	TS222212	TS222212	0.088	1.000	0.088	Eq. H1-1b	33	0.933	0.715	0.715	1.070
79	TS222212	TS222212	0.567	1.000	0.567	Eq. H1-1b	30	0.933	0.715	0.715	1.070
80	TS222212	TS222212	0.558	1.000	0.558	Eq. H1-1b	30	0.933	0.715	0.715	1.070
81	TS222212	TS222212	0.740	1.000	0.740	Eq. H1-1b	30	0.933	0.715	0.715	1.070
82	TS222212	TS222212	0.522	1.000	0.522	Eq. H1-1b	16	0.933	0.715	0.715	1.070
83	TS222212	TS222212	0.267	1.000	0.267	Eq. H1-1b	33	0.933	0.715	0.715	1.070
84	TS222212	TS222212	0.083	1.000	0.083	Eq. H1-1b	16	0.933	0.715	0.715	1.070
85	TS222212	TS222212	0.069	1.000	0.069	Eq. H1-1b	32	0.933	0.715	0.715	1.070
86	TS222212	TS222212	0.107	1.000	0.107	Eq. H1-1b	24	0.933	0.715	0.715	1.070
87	TS222212	TS222212	0.120	1.000	0.120	Eq. H1-1b	33	0.933	0.715	0.715	1.070
88	TS222212	TS222212	0.172	1.000	0.172	Eq. H1-1b	16	0.933	0.715	0.715	1.070
89	TS222212	TS222212	0.110	1.000	0.110	Eq. H3-1	18	0.933	0.715	0.715	1.070
90	TS222212	TS222212	0.158	1.000	0.158	Eq. H1-1b	33	0.933	0.715	0.715	1.070
91	TS222212	TS222212	0.117	1.000	0.117	Eq. H1-1b	32	0.933	0.715	0.715	1.070
92	TS222212	TS222212	0.090	1.000	0.090	Eq. H3-1	33	0.933	0.715	0.715	1.070
93	TS222212	TS222212	0.079	1.000	0.079	Eq. H1-1b	24	0.933	0.715	0.715	1.070
94	TS222212	TS222212	0.169	1.000	0.169	Eq. H1-1b	32	0.933	0.715	0.715	1.070
95	TS222212	TS222212	0.122	1.000	0.122	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
96	TS222212	TS222212	0.224	1.000	0.224	Eq. H1-1b	16	0.933	0.715	0.715	1.070
97	TS222212	TS222212	0.252	1.000	0.252	Eq. H1-1b	16	0.933	0.715	0.715	1.070
98	TS222212	TS222212	0.246	1.000	0.246	Eq. H1-1b	16	0.933	0.715	0.715	1.070
99	TS222212	TS222212	0.245	1.000	0.245	Eq. H1-1b	16	0.933	0.715	0.715	1.070
100	TS222212	TS222212	0.248	1.000	0.248	Eq. H1-1b	16	0.933	0.715	0.715	1.070
101	TS222212	TS222212	0.226	1.000	0.226	Eq. H1-1b	16	0.933	0.715	0.715	1.070
102	TS222212	TS222212	0.152	1.000	0.152	Eq. H1-1b	33	0.933	0.715	0.715	1.070
103	TS222212	TS222212	0.129	1.000	0.129	Eq. H3-1	33	0.933	0.715	0.715	1.070
104	TS222212	TS222212	0.162	1.000	0.162	Eq. H3-1	32	0.933	0.715	0.715	1.070
105	TS222212	TS222212	0.190	1.000	0.190	Eq. H1-1b	16	0.933	0.715	0.715	1.070
106	TS222212	TS222212	0.184	1.000	0.184	Eq. H1-1b	16	0.933	0.715	0.715	1.070
107	TS222212	TS222212	0.165	1.000	0.165	Eq. H1-1b	16	0.933	0.715	0.715	1.070
108	TS222212	TS222212	0.160	1.000	0.160	Eq. H1-1b	16	0.933	0.715	0.715	1.070
109	TS222212	TS222212	0.166	1.000	0.166	Eq. H1-1b	16	0.933	0.715	0.715	1.070
110	TS222212	TS222212	0.187	1.000	0.187	Sec. E1	16	0.933	0.715	0.715	1.070
111	TS222212	TS222212	0.171	1.000	0.171	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
112	TS222212	TS222212	0.069	1.000	0.069	Eq. H1-1b	23	0.933	0.715	0.715	1.070
113	TS222212	TS222212	0.178	1.000	0.178	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
114	TS222212	TS222212	0.249	1.000	0.249	Eq. H1-1b	16	0.933	0.715	0.715	1.070
115	TS222212	TS222212	0.266	1.000	0.266	Eq. H1-1b	16	0.933	0.715	0.715	1.070
116	TS222212	TS222212	0.262	1.000	0.262	Eq. H1-1b	16	0.933	0.715	0.715	1.070
117	TS222212	TS222212	0.260	1.000	0.260	Eq. H1-1b	16	0.933	0.715	0.715	1.070
118	TS222212	TS222212	0.262	1.000	0.262	Eq. H1-1b	16	0.933	0.715	0.715	1.070
119	TS222212	TS222212	0.246	1.000	0.246	Eq. H1-1b	16	0.933	0.715	0.715	1.070
120	TS222212	TS222212	0.333	1.000	0.333	Eq. H3-1	33	0.933	0.715	0.715	1.070
121	TS222212	TS222212	0.341	1.000	0.341	Eq. H1-1b	32	0.933	0.715	0.715	1.070
122	TS222212	TS222212	0.466	1.000	0.466	Eq. H1-1b	16	0.933	0.715	0.715	1.070

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Job No 232-24-1425	Sheet No SO 18		Rev
Part			
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
123	TS222212	TS222212	0.433	1.000	0.433	Eq. H1-1b	16	0.933	0.715	0.715	1.070
124	TS222212	TS222212	0.406	1.000	0.406	Eq. H1-1b	16	0.933	0.715	0.715	1.070
125	TS222212	TS222212	0.398	1.000	0.398	Eq. H1-1b	16	0.933	0.715	0.715	1.070
126	TS222212	TS222212	0.417	1.000	0.417	Eq. H1-1b	16	0.933	0.715	0.715	1.070
127	TS222212	TS222212	0.436	1.000	0.436	Eq. H1-1b	16	0.933	0.715	0.715	1.070
128	TS222212	TS222212	0.335	1.000	0.335	Eq. H3-6	18	0.933	0.715	0.715	1.070
129	TS222212	TS222212	0.237	1.000	0.237	Eq. H1-1b	33	0.933	0.715	0.715	1.070
130	TS222212	TS222212	0.083	1.000	0.083	Eq. H1-1b	17	0.933	0.715	0.715	1.070
131	TS222212	TS222212	0.105	1.000	0.105	Eq. H1-1b	25	0.933	0.715	0.715	1.070
132	TS222212	TS222212	0.113	1.000	0.113	Eq. H1-1b	25	0.933	0.715	0.715	1.070
133	TS222212	TS222212	0.114	1.000	0.114	Eq. H1-1b	25	0.933	0.715	0.715	1.070
134	TS222212	TS222212	0.108	1.000	0.108	Eq. H1-1b	25	0.933	0.715	0.715	1.070
135	TS222212	TS222212	0.089	1.000	0.089	Eq. H1-1b	25	0.933	0.715	0.715	1.070
136	TS222212	TS222212	0.339	1.000	0.339	Eq. H1-1a	16	0.933	0.715	0.715	1.070
137	TS222212	TS222212	0.493	1.000	0.493	Eq. H1-1b	16	0.933	0.715	0.715	1.070
138	TS222212	TS222212	0.496	1.000	0.496	Eq. H1-1b	16	0.933	0.715	0.715	1.070
139	TS222212	TS222212	0.491	1.000	0.491	Eq. H1-1b	16	0.933	0.715	0.715	1.070
140	TS222212	TS222212	0.487	1.000	0.487	Eq. H1-1b	16	0.933	0.715	0.715	1.070
141	TS222212	TS222212	0.493	1.000	0.493	Eq. H1-1b	16	0.933	0.715	0.715	1.070
142	TS222212	TS222212	0.476	1.000	0.476	Eq. H1-1b	16	0.933	0.715	0.715	1.070
143	TS222212	TS222212	0.329	1.000	0.329	Eq. H3-6	18	0.933	0.715	0.715	1.070
144	TS222212	TS222212	0.347	1.000	0.347	Eq. H3-1	33	0.933	0.715	0.715	1.070
145	TS222212	TS222212	0.171	1.000	0.171	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
146	TS222212	TS222212	0.275	1.000	0.275	Eq. H1-1b	16	0.933	0.715	0.715	1.070
147	TS222212	TS222212	0.318	1.000	0.318	Eq. H1-1b	16	0.933	0.715	0.715	1.070
148	TS222212	TS222212	0.328	1.000	0.328	Eq. H1-1b	16	0.933	0.715	0.715	1.070
149	TS222212	TS222212	0.328	1.000	0.328	Eq. H1-1b	16	0.933	0.715	0.715	1.070
150	TS222212	TS222212	0.323	1.000	0.323	Eq. H1-1b	16	0.933	0.715	0.715	1.070
151	TS222212	TS222212	0.283	1.000	0.283	Eq. H1-1b	16	0.933	0.715	0.715	1.070
152	TS222212	TS222212	0.072	1.000	0.072	Eq. H1-1b	23	0.933	0.715	0.715	1.070
153	TS222212	TS222212	0.164	1.000	0.164	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
154	TS222212	TS222212	0.278	1.000	0.278	Eq. H1-1b	16	0.933	0.715	0.715	1.070
155	TS222212	TS222212	0.347	1.000	0.347	Eq. H1-1b	16	0.933	0.715	0.715	1.070
156	TS222212	TS222212	0.378	1.000	0.378	Eq. H1-1b	16	0.933	0.715	0.715	1.070
157	TS222212	TS222212	0.378	1.000	0.378	Eq. H1-1b	16	0.933	0.715	0.715	1.070
158	TS222212	TS222212	0.355	1.000	0.355	Eq. H1-1b	16	0.933	0.715	0.715	1.070
159	TS222212	TS222212	0.299	1.000	0.299	Eq. H1-1b	16	0.933	0.715	0.715	1.070
160	TS222212	TS222212	0.175	1.000	0.175	Eq. H3-1	32	0.933	0.715	0.715	1.070
161	TS222212	TS222212	0.122	1.000	0.122	Eq. H3-1	34	0.933	0.715	0.715	1.070
162	TS222212	TS222212	0.137	1.000	0.137	Eq. H1-1b	34	0.933	0.715	0.715	1.070
163	TS222212	TS222212	0.252	1.000	0.252	Eq. H1-1b	16	0.933	0.715	0.715	1.070
164	TS222212	TS222212	0.302	1.000	0.302	Eq. H1-1b	16	0.933	0.715	0.715	1.070
165	TS222212	TS222212	0.310	1.000	0.310	Eq. H1-1b	16	0.933	0.715	0.715	1.070
166	TS222212	TS222212	0.311	1.000	0.311	Eq. H1-1b	16	0.933	0.715	0.715	1.070
167	TS222212	TS222212	0.308	1.000	0.308	Eq. H1-1b	16	0.933	0.715	0.715	1.070

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Job No 232-24-1425	Sheet No SO 19		Rev
Part			
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
168	TS222212	TS222212	0.258	1.000	0.258	Eq. H1-1b	16	0.933	0.715	0.715	1.070
169	TS222212	TS222212	0.160	1.000	0.160	Eq. H1-1b	32	0.933	0.715	0.715	1.070
170	TS222212	TS222212	0.104	1.000	0.104	Eq. Sec. D2	16	0.933	0.715	0.715	1.070
171	TS222212	TS222212	0.075	1.000	0.075	Eq. H1-1b	24	0.933	0.715	0.715	1.070
172	TS222212	TS222212	0.095	1.000	0.095	Eq. H3-1	16	0.933	0.715	0.715	1.070
173	TS222212	TS222212	0.143	1.000	0.143	Eq. H1-1b	34	0.933	0.715	0.715	1.070
174	TS222212	TS222212	0.117	1.000	0.117	Eq. H1-1b	32	0.933	0.715	0.715	1.070
175	TS222212	TS222212	0.107	1.000	0.107	Eq. H3-1	18	0.933	0.715	0.715	1.070
176	TS222212	TS222212	0.161	1.000	0.161	Eq. H1-1b	32	0.933	0.715	0.715	1.070
177	TS222212	TS222212	0.137	1.000	0.137	Eq. H1-1b	26	0.933	0.715	0.715	1.070
178	TS222212	TS222212	0.104	1.000	0.104	Eq. H1-1b	24	0.933	0.715	0.715	1.070
179	TS222212	TS222212	0.074	1.000	0.074	Eq. H1-1b	16	0.933	0.715	0.715	1.070
180	TS222212	TS222212	0.284	1.000	0.284	Eq. H1-1b	18	0.933	0.715	0.715	1.070
181	TS222212	TS222212	0.101	1.000	0.101	Eq. H3-1	17	0.933	0.715	0.715	1.070
182	TS222212	TS222212	0.056	1.000	0.056	Eq. H1-1b	30	0.933	0.715	0.715	1.070
183	TS222212	TS222212	0.030	1.000	0.030	Eq. H3-1	16	0.933	0.715	0.715	1.070
184	TS222212	TS222212	0.023	1.000	0.023	Eq. H1-1b	17	0.933	0.715	0.715	1.070
185	TS222212	TS222212	0.023	1.000	0.023	Eq. H1-1b	22	0.933	0.715	0.715	1.070
186	TS222212	TS222212	0.021	1.000	0.021	Eq. H1-1b	21	0.933	0.715	0.715	1.070
187	TS222212	TS222212	0.027	1.000	0.027	Eq. H1-1b	21	0.933	0.715	0.715	1.070
188	TS222212	TS222212	0.059	1.000	0.059	Eq. H3-1	30	0.933	0.715	0.715	1.070
189	TS222212	TS222212	0.125	1.000	0.125	Eq. H1-1b	18	0.933	0.715	0.715	1.070
190	TS222212	TS222212	0.114	1.000	0.114	Sec. E1	26	0.933	0.715	0.715	1.070
191	TS222212	TS222212	0.063	1.000	0.063	Sec. E1	26	0.933	0.715	0.715	1.070
192	TS222212	TS222212	0.081	1.000	0.081	Eq. H1-1b	26	0.933	0.715	0.715	1.070
193	TS222212	TS222212	0.093	1.000	0.093	Eq. H1-1b	18	0.933	0.715	0.715	1.070
194	TS222212	TS222212	0.081	1.000	0.081	Eq. H1-1b	33	0.933	0.715	0.715	1.070
195	TS222212	TS222212	0.053	1.000	0.053	Eq. H1-1b	26	0.933	0.715	0.715	1.070
196	TS222212	TS222212	0.205	1.000	0.205	Eq. H1-1b	34	0.933	0.715	0.715	1.070
197	TS222212	TS222212	0.038	1.000	0.038	Eq. H1-1b	16	0.933	0.715	0.715	1.070
198	TS222212	TS222212	0.100	1.000	0.100	Eq. H3-1	17	0.933	0.715	0.715	1.070
300	TS222212	TS222212	0.121	1.000	0.121	Eq. H1-1b	38	1.867	3.793	1.430	1.070
301	TS222212	TS222212	0.137	1.000	0.137	Eq. H1-1b	38	1.867	3.793	1.430	1.070
302	TS222212	TS222212	0.135	1.000	0.135	Eq. H1-1b	38	1.867	3.793	1.430	1.070
303	TS222212	TS222212	0.132	1.000	0.132	Eq. H1-1b	38	1.867	3.793	1.430	1.070
304	TS222212	TS222212	0.127	1.000	0.127	Eq. H1-1b	38	1.867	3.793	1.430	1.070
305	TS222212	TS222212	0.105	1.000	0.105	Eq. H1-1b	35	1.867	3.793	1.430	1.070
306	TS222212	TS222212	0.122	1.000	0.122	Eq. H1-1b	38	1.867	3.793	1.430	1.070
307	TS222212	TS222212	0.142	1.000	0.142	Eq. H1-1b	38	1.867	3.793	1.430	1.070
308	TS222212	TS222212	0.139	1.000	0.139	Eq. H1-1b	38	1.867	3.793	1.430	1.070
309	TS222212	TS222212	0.135	1.000	0.135	Eq. H1-1b	38	1.867	3.793	1.430	1.070
310	TS222212	TS222212	0.130	1.000	0.130	Eq. H1-1b	38	1.867	3.793	1.430	1.070
311	TS222212	TS222212	0.100	1.000	0.100	Eq. H1-1b	35	1.867	3.793	1.430	1.070
312	TS222212	TS222212	0.214	1.000	0.214	Eq. H1-1b	20	1.867	3.793	1.430	1.070
313	TS222212	TS222212	0.208	1.000	0.208	Eq. H1-1b	20	1.867	3.793	1.430	1.070

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Job No 232-24-1425	Sheet No SO 20		Rev
Part			
Job Title CITY OF SANTA CRUZ	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
314	TS222212	TS222212	0.205	1.000	0.205	Eq. H1-1b	36	1.867	3.793	1.430	1.070
315	TS222212	TS222212	0.201	1.000	0.201	Eq. H1-1b	36	1.867	3.793	1.430	1.070
316	TS222212	TS222212	0.199	1.000	0.199	Eq. H1-1b	36	1.867	3.793	1.430	1.070
317	TS222212	TS222212	0.194	1.000	0.194	Eq. H1-1b	36	1.867	3.793	1.430	1.070
318	TS222212	TS222212	0.370	1.000	0.370	Eq. H1-1b	16	1.867	3.793	1.430	1.070
319	TS222212	TS222212	0.305	1.000	0.305	Eq. H1-1b	16	1.867	3.793	1.430	1.070
320	TS222212	TS222212	0.266	1.000	0.266	Eq. H1-1b	16	1.867	3.793	1.430	1.070
321	TS222212	TS222212	0.256	1.000	0.256	Eq. H1-1b	16	1.867	3.793	1.430	1.070
322	TS222212	TS222212	0.273	1.000	0.273	Eq. H1-1b	16	1.867	3.793	1.430	1.070
323	TS222212	TS222212	0.301	1.000	0.301	Eq. H1-1b	16	1.867	3.793	1.430	1.070
324	TS222212	TS222212	0.151	1.000	0.151	Eq. H1-1b	38	1.867	3.793	1.430	1.070
325	TS222212	TS222212	0.225	1.000	0.225	Eq. H1-1b	16	1.867	3.793	1.430	1.070
326	TS222212	TS222212	0.285	1.000	0.285	Eq. H1-1b	16	1.867	3.793	1.430	1.070
327	TS222212	TS222212	0.295	1.000	0.295	Eq. H1-1b	16	1.867	3.793	1.430	1.070
328	TS222212	TS222212	0.255	1.000	0.255	Eq. H1-1b	16	1.867	3.793	1.430	1.070
329	TS222212	TS222212	0.163	1.000	0.163	Eq. H1-1b	16	1.867	3.793	1.430	1.070
330	TS222212	TS222212	0.065	1.000	0.065	Eq. H1-1b	18	1.867	3.793	1.430	1.070
331	TS222212	TS222212	0.077	1.000	0.077	Eq. H1-1b	18	1.867	3.793	1.430	1.070
332	TS222212	TS222212	0.070	1.000	0.070	Eq. H1-1b	33	1.867	3.793	1.430	1.070
333	TS222212	TS222212	0.090	1.000	0.090	Eq. H1-1b	30	1.867	3.793	1.430	1.070
334	TS222212	TS222212	0.292	1.000	0.292	Eq. H1-1b	30	1.867	3.793	1.430	1.070
335	TS222212	TS222212	0.153	1.000	0.153	Eq. H1-1b	30	1.867	3.793	1.430	1.070
336	TS222212	TS222212	0.149	1.000	0.149	Eq. H1-1b	38	1.867	3.793	1.430	1.070
337	TS222212	TS222212	0.241	1.000	0.241	Eq. H1-1b	16	1.867	3.793	1.430	1.070
338	TS222212	TS222212	0.303	1.000	0.303	Eq. H1-1b	16	1.867	3.793	1.430	1.070
339	TS222212	TS222212	0.313	1.000	0.313	Eq. H1-1b	16	1.867	3.793	1.430	1.070
340	TS222212	TS222212	0.272	1.000	0.272	Eq. H1-1b	16	1.867	3.793	1.430	1.070
341	TS222212	TS222212	0.170	1.000	0.170	Eq. H1-1b	16	1.867	3.793	1.430	1.070
342	TS222212	TS222212	0.144	1.000	0.144	Eq. H1-1b	26	1.867	3.793	1.430	1.070
343	TS222212	TS222212	0.113	1.000	0.113	Eq. H1-1b	33	1.867	3.793	1.430	1.070
344	TS222212	TS222212	0.127	1.000	0.127	Eq. H1-1b	24	1.867	3.793	1.430	1.070
345	TS222212	TS222212	0.116	1.000	0.116	Eq. H1-1b	24	1.867	3.793	1.430	1.070
346	TS222212	TS222212	0.117	1.000	0.117	Eq. H1-1b	38	1.867	3.793	1.430	1.070
347	TS222212	TS222212	0.177	1.000	0.177	Eq. H1-1b	16	1.867	3.793	1.430	1.070
348	TS222212	TS222212	0.221	1.000	0.221	Eq. H1-1b	16	1.867	3.793	1.430	1.070
349	TS222212	TS222212	0.230	1.000	0.230	Eq. H1-1b	16	1.867	3.793	1.430	1.070
350	TS222212	TS222212	0.204	1.000	0.204	Eq. H1-1b	16	1.867	3.793	1.430	1.070
351	TS222212	TS222212	0.127	1.000	0.127	Eq. H1-1b	16	1.867	3.793	1.430	1.070
352	TS222212	TS222212	0.149	1.000	0.149	Eq. H1-1b	34	1.867	3.793	1.430	1.070
353	TS222212	TS222212	0.213	1.000	0.213	Eq. H1-1b	39	1.867	3.793	1.430	1.070
354	TS222212	TS222212	0.213	1.000	0.213	Eq. H1-1b	39	1.867	3.793	1.430	1.070
355	TS222212	TS222212	0.152	1.000	0.152	Eq. H1-1b	26	1.867	3.793	1.430	1.070
356	TS222212	TS222212	0.302	1.000	0.302	Eq. H1-1b	18	1.867	3.793	1.430	1.070
357	TS222212	TS222212	0.230	1.000	0.230	Eq. H1-1b	18	1.867	3.793	1.430	1.070
358	TS222212	TS222212	0.203	1.000	0.203	Eq. H1-1b	24	1.867	3.793	1.430	1.070

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
359	TS222212	TS222212	0.214	1.000	0.214	Eq. H1-1b	32	1.867	3.793	1.430	1.070
360	TS222212	TS222212	0.325	1.000	0.325	Eq. H1-1b	26	1.867	3.793	1.430	1.070
361	TS222212	TS222212	0.232	1.000	0.232	Eq. H1-1b	17	1.867	3.793	1.430	1.070
362	TS222212	TS222212	0.153	1.000	0.153	Eq. H1-1b	26	1.867	3.793	1.430	1.070
363	TS222212	TS222212	0.212	1.000	0.212	Eq. H1-1b	23	1.867	3.793	1.430	1.070
364	TS222212	TS222212	0.212	1.000	0.212	Eq. H1-1b	23	1.867	3.793	1.430	1.070
365	TS222212	TS222212	0.155	1.000	0.155	Eq. H1-1b	33	1.867	3.793	1.430	1.070
366	TS222212	TS222212	0.104	1.000	0.104	Eq. H1-1b	30	1.867	3.793	1.430	1.070
367	TS222212	TS222212	0.121	1.000	0.121	Eq. H1-1b	16	1.867	3.793	1.430	1.070
368	TS222212	TS222212	0.149	1.000	0.149	Eq. H1-1b	16	1.867	3.793	1.430	1.070
369	TS222212	TS222212	0.151	1.000	0.151	Eq. H1-1b	16	1.867	3.793	1.430	1.070
370	TS222212	TS222212	0.131	1.000	0.131	Eq. H1-1b	16	1.867	3.793	1.430	1.070
371	TS222212	TS222212	0.126	1.000	0.126	Eq. H1-1b	30	1.867	3.793	1.430	1.070
372	TS222212	TS222212	0.115	1.000	0.115	Eq. H1-1b	24	1.867	3.793	1.430	1.070
373	TS222212	TS222212	0.133	1.000	0.133	Eq. H1-1b	24	1.867	3.793	1.430	1.070
374	TS222212	TS222212	0.122	1.000	0.122	Eq. H1-1b	24	1.867	3.793	1.430	1.070
375	TS222212	TS222212	0.121	1.000	0.121	Eq. H1-1b	18	1.867	3.793	1.430	1.070
376	TS222212	TS222212	0.109	1.000	0.109	Eq. H1-1b	37	1.867	3.793	1.430	1.070
377	TS222212	TS222212	0.118	1.000	0.118	Eq. H1-1b	37	1.867	3.793	1.430	1.070
378	TS222212	TS222212	0.118	1.000	0.118	Eq. H1-1b	37	1.867	3.793	1.430	1.070
379	TS222212	TS222212	0.118	1.000	0.118	Eq. H1-1b	37	1.867	3.793	1.430	1.070
380	TS222212	TS222212	0.115	1.000	0.115	Eq. H1-1b	37	1.867	3.793	1.430	1.070
381	TS222212	TS222212	0.113	1.000	0.113	Eq. H1-1b	38	1.867	3.793	1.430	1.070
382	TS222212	TS222212	0.085	1.000	0.085	Eq. H1-1b	34	1.867	3.793	1.430	1.070
383	TS222212	TS222212	0.139	1.000	0.139	Eq. H1-1b	34	1.867	3.793	1.430	1.070
384	TS222212	TS222212	0.130	1.000	0.130	Eq. H1-1b	33	1.867	3.793	1.430	1.070
385	TS222212	TS222212	0.109	1.000	0.109	Eq. H1-1b	18	1.867	3.793	1.430	1.070
386	TS222212	TS222212	0.191	1.000	0.191	Eq. H1-1b	18	1.867	3.793	1.430	1.070
387	TS222212	TS222212	0.138	1.000	0.138	Eq. H1-1b	30	1.867	3.793	1.430	1.070
388	TS222212	TS222212	0.109	1.000	0.109	Eq. H1-1b	38	1.867	3.793	1.430	1.070
389	TS222212	TS222212	0.115	1.000	0.115	Eq. H1-1b	37	1.867	3.793	1.430	1.070
390	TS222212	TS222212	0.115	1.000	0.115	Eq. H1-1b	37	1.867	3.793	1.430	1.070
391	TS222212	TS222212	0.115	1.000	0.115	Eq. H1-1b	37	1.867	3.793	1.430	1.070
392	TS222212	TS222212	0.111	1.000	0.111	Eq. H1-1b	37	1.867	3.793	1.430	1.070
393	TS222212	TS222212	0.113	1.000	0.113	Eq. H1-1b	22	1.867	3.793	1.430	1.070
394	TS222212	TS222212	0.120	1.000	0.120	Eq. H1-1b	30	1.867	3.793	1.430	1.070
395	TS222212	TS222212	0.180	1.000	0.180	Eq. H1-1b	30	1.867	3.793	1.430	1.070
396	TS222212	TS222212	0.158	1.000	0.158	Eq. H1-1b	30	1.867	3.793	1.430	1.070
397	TS222212	TS222212	0.291	1.000	0.291	Eq. H1-1b	16	1.867	3.793	1.430	1.070
398	TS222212	TS222212	0.335	1.000	0.335	Eq. H1-1b	16	1.867	3.793	1.430	1.070
399	TS222212	TS222212	0.337	1.000	0.337	Eq. H1-1b	16	1.867	3.793	1.430	1.070
400	TS222212	TS222212	0.143	1.000	0.143	Eq. H1-1b	16	1.867	3.793	1.430	1.070
401	TS222212	TS222212	0.239	1.000	0.239	Eq. H1-1b	16	1.867	3.793	1.430	1.070
402	TS222212	TS222212	0.183	1.000	0.183	Eq. H1-1b	30	1.867	3.793	1.430	1.070
403	TS222212	TS222212	0.242	1.000	0.242	Eq. H1-1b	16	1.867	3.793	1.430	1.070

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
404	TS222212	TS222212	0.334	1.000	0.334	Eq. H1-1b	16	1.867	3.793	1.430	1.070
405	TS222212	TS222212	0.218	1.000	0.218	Eq. H1-1b	16	1.867	3.793	1.430	1.070
406	TS222212	TS222212	0.238	1.000	0.238	Eq. H1-1b	16	1.867	3.793	1.430	1.070
407	TS222212	TS222212	0.297	1.000	0.297	Eq. H1-1b	16	1.867	3.793	1.430	1.070
408	TS222212	TS222212	0.334	1.000	0.334	Eq. H1-1b	16	1.867	3.793	1.430	1.070
409	TS222212	TS222212	0.338	1.000	0.338	Eq. H1-1b	16	1.867	3.793	1.430	1.070
410	TS222212	TS222212	0.315	1.000	0.315	Eq. H1-1b	16	1.867	3.793	1.430	1.070
411	TS222212	TS222212	0.236	1.000	0.236	Eq. H1-1b	16	1.867	3.793	1.430	1.070
500	TS222214	TS222214	0.049	1.000	0.049	Eq. H1-1b	39	0.719	0.564	0.564	0.845
501	TS222214	TS222214	0.109	1.000	0.109	Eq. H1-1b	22	0.719	0.564	0.564	0.845
502	TS222214	TS222214	0.098	1.000	0.098	Eq. H1-1b	39	0.719	0.564	0.564	0.845
503	TS222214	TS222214	0.103	1.000	0.103	Eq. H1-1b	18	0.719	0.564	0.564	0.845
504	TS222214	TS222214	0.082	1.000	0.082	Eq. H1-1b	34	0.719	0.564	0.564	0.845
505	TS222214	TS222214	0.099	1.000	0.099	Eq. H1-1b	34	0.719	0.564	0.564	0.845
506	TS222214	TS222214	0.113	1.000	0.113	Eq. H1-1b	18	0.719	0.564	0.564	0.845
507	TS222214	TS222214	0.176	1.000	0.176	Eq. H1-1b	30	0.719	0.564	0.564	0.845
508	TS222214	TS222214	0.079	1.000	0.079	Eq. H1-1b	39	0.719	0.564	0.564	0.845
509	TS222214	TS222214	0.109	1.000	0.109	Eq. H1-1b	39	0.719	0.564	0.564	0.845
510	TS222214	TS222214	0.108	1.000	0.108	Eq. H1-1b	25	0.719	0.564	0.564	0.845
511	TS222214	TS222214	0.147	1.000	0.147	Eq. H1-1b	25	0.719	0.564	0.564	0.845
512	TS222214	TS222214	0.072	1.000	0.072	Eq. H1-1b	18	0.719	0.564	0.564	0.845
513	TS222214	TS222214	0.043	1.000	0.043	Eq. H1-1b	18	0.719	0.564	0.564	0.845
514	TS222214	TS222214	0.042	1.000	0.042	Eq. H1-1b	18	0.719	0.564	0.564	0.845
515	TS222214	TS222214	0.066	1.000	0.066	Eq. H1-1b	34	0.719	0.564	0.564	0.845
516	TS222214	TS222214	0.130	1.000	0.130	Eq. H1-1b	25	0.719	0.564	0.564	0.845
517	TS222214	TS222214	0.045	1.000	0.045	Eq. H1-1b	30	0.719	0.564	0.564	0.845
518	TS222214	TS222214	0.159	1.000	0.159	Eq. H1-1b	25	0.719	0.564	0.564	0.845
519	TS222214	TS222214	0.037	1.000	0.037	Eq. H1-1b	32	0.719	0.564	0.564	0.845
600	TS202012	TS202012	0.077	1.000	0.077	Eq. H1-1b	22	0.824	0.493	0.493	0.737
601	TS202012	TS202012	0.063	1.000	0.063	Eq. H1-1b	18	0.824	0.493	0.493	0.737
602	TS202012	TS202012	0.065	1.000	0.065	Eq. H1-1b	30	0.824	0.493	0.493	0.737
603	TS202012	TS202012	0.065	1.000	0.065	Eq. H1-1b	22	0.824	0.493	0.493	0.737
604	TS202012	TS202012	0.059	1.000	0.059	Eq. H1-1b	30	0.824	0.493	0.493	0.737
605	TS202012	TS202012	0.041	1.000	0.041	Eq. H1-1b	28	0.824	0.493	0.493	0.737
606	TS202012	TS202012	0.075	1.000	0.075	Eq. H1-1b	33	0.824	0.493	0.493	0.737
607	TS202012	TS202012	0.079	1.000	0.079	Eq. H1-1b	17	0.824	0.493	0.493	0.737
608	TS202012	TS202012	0.058	1.000	0.058	Eq. H1-1b	17	0.824	0.493	0.493	0.737
609	TS202012	TS202012	0.120	1.000	0.120	Eq. H1-1b	33	0.824	0.493	0.493	0.737
610	TS202012	TS202012	0.120	1.000	0.120	Eq. H1-1b	33	0.824	0.493	0.493	0.737
611	TS202012	TS202012	0.080	1.000	0.080	Eq. H1-1b	33	0.824	0.493	0.493	0.737
612	TS202012	TS202012	0.074	1.000	0.074	Eq. H1-1b	17	0.824	0.493	0.493	0.737
613	TS202012	TS202012	0.094	1.000	0.094	Eq. H1-1b	17	0.824	0.493	0.493	0.737
614	TS202012	TS202012	0.077	1.000	0.077	Eq. H1-1b	26	0.824	0.493	0.493	0.737
615	TS202012	TS202012	0.074	1.000	0.074	Eq. H1-1b	25	0.824	0.493	0.493	0.737
616	TS202012	TS202012	0.085	1.000	0.085	Eq. H1-1b	23	0.824	0.493	0.493	0.737

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Job Title CITY OF SANTA CRUZ	Job No 232-24-1425	Sheet No SO 23	Rev
	Part		
	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
617	TS202012	TS202012	0.060	1.000	0.060	Eq. H1-1b	18	0.824	0.493	0.493	0.737
618	TS202012	TS202012	0.065	1.000	0.065	Eq. H1-1b	18	0.824	0.493	0.493	0.737
619	TS202012	TS202012	0.072	1.000	0.072	Eq. H1-1b	18	0.824	0.493	0.493	0.737
620	TS202012	TS202012	0.076	1.000	0.076	Eq. H1-1b	34	0.824	0.493	0.493	0.737
621	TS202012	TS202012	0.170	1.000	0.170	Eq. H1-1b	17	0.824	0.493	0.493	0.737
622	TS202012	TS202012	0.079	1.000	0.079	Eq. H1-1b	25	0.824	0.493	0.493	0.737
623	TS202012	TS202012	0.053	1.000	0.053	Eq. H1-1b	30	0.824	0.493	0.493	0.737
624	TS202012	TS202012	0.077	1.000	0.077	Eq. H1-1b	25	0.824	0.493	0.493	0.737
625	TS202012	TS202012	0.061	1.000	0.061	Eq. H1-1b	30	0.824	0.493	0.493	0.737
626	TS202012	TS202012	0.088	1.000	0.088	Eq. H1-1b	32	0.824	0.493	0.493	0.737
627	TS202012	TS202012	0.080	1.000	0.080	Eq. H1-1b	16	0.824	0.493	0.493	0.737
700	CS252514	CS252514	0.087	1.000	0.087	Sec. E4	31	0.609	0.694	0.409	0.001
701	CS252514	CS252514	0.167	1.000	0.167	Sec. E4	31	0.609	0.694	0.409	0.001
702	CS252514	CS252514	0.169	1.000	0.169	Sec. E4	31	0.609	0.694	0.409	0.001
703	CS252514	CS252514	0.169	1.000	0.169	Sec. E4	31	0.609	0.694	0.409	0.001
704	CS252514	CS252514	0.169	1.000	0.169	Sec. E4	31	0.609	0.694	0.409	0.001
705	CS252514	CS252514	0.170	1.000	0.170	Sec. E4	31	0.609	0.694	0.409	0.001
706	CS252514	CS252514	0.168	1.000	0.168	Sec. E4	31	0.609	0.694	0.409	0.001
707	CS252514	CS252514	0.087	1.000	0.087	Sec. E4	31	0.609	0.694	0.409	0.001
800	C401014C	C401014C	0.107	1.000	0.107	Sec. E1	16	0.902	0.411	1.196	0.003
801	C401014C	C401014C	0.164	1.000	0.164	Sec. E1	16	0.902	0.411	1.196	0.003
802	C401014C	C401014C	0.193	1.000	0.193	Sec. E1	16	0.902	0.411	1.196	0.003
803	C401014C	C401014C	0.197	1.000	0.197	Sec. E1	16	0.902	0.411	1.196	0.003
804	C401014C	C401014C	0.175	1.000	0.175	Sec. E1	16	0.902	0.411	1.196	0.003
805	C401014C	C401014C	0.127	1.000	0.127	Sec. E1	16	0.902	0.411	1.196	0.003
806	C401014C	C401014C	0.074	1.000	0.074	Sec. E1	16	0.902	0.411	1.196	0.003
807	C401014C	C401014C	0.091	1.000	0.091	Sec. E1	16	0.902	0.411	1.196	0.003
808	C401014C	C401014C	0.107	1.000	0.107	Sec. E1	16	0.902	0.411	1.196	0.003
809	C401014C	C401014C	0.108	1.000	0.108	Sec. E1	16	0.902	0.411	1.196	0.003
810	C401014C	C401014C	0.100	1.000	0.100	Sec. E1	16	0.902	0.411	1.196	0.003
811	C401014C	C401014C	0.063	1.000	0.063	Sec. E1	16	0.902	0.411	1.196	0.003
900	C401014C	C401014C	0.255	1.000	0.255	Eq. H1-1b	31	0.320	0.117	0.317	0.000
901	C401014C	C401014C	0.197	1.000	0.197	Eq. H1-1b	16	0.320	0.117	0.317	0.000
902	C401014C	C401014C	0.208	1.000	0.208	Eq. H1-1b	16	0.320	0.117	0.317	0.000
903	C401014C	C401014C	0.226	1.000	0.226	Eq. H2-1	33	0.320	0.117	0.317	0.000
904	C401014C	C401014C	0.222	1.000	0.222	Eq. H2-1	33	0.320	0.117	0.317	0.000
905	C401014C	C401014C	0.212	1.000	0.212	Eq. H1-1b	16	0.320	0.117	0.317	0.000
906	C401014C	C401014C	0.244	1.000	0.244	Eq. H1-1b	31	0.320	0.117	0.317	0.000
907	C401014C	C401014C	0.309	1.000	0.309	Eq. H1-1b	31	0.320	0.117	0.317	0.000
908	C401014C	C401014C	0.376	1.000	0.376	Eq. H1-1b	31	0.320	0.117	0.317	0.000
909	C401014C	C401014C	0.382	1.000	0.382	Eq. H1-1b	31	0.320	0.117	0.317	0.000
910	C401014C	C401014C	0.382	1.000	0.382	Eq. H1-1b	31	0.320	0.117	0.317	0.000
911	C401014C	C401014C	0.384	1.000	0.384	Eq. H1-1b	31	0.320	0.117	0.317	0.000
912	C401014C	C401014C	0.373	1.000	0.373	Eq. H1-1b	31	0.320	0.117	0.317	0.000
913	C401014C	C401014C	0.304	1.000	0.304	Eq. H1-1b	31	0.320	0.117	0.317	0.000

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	Part		
	Ref		
	By AF	Date 6/17/2025	Chd RS
Client PACIFIC METAL BUILDINGS INC.	File 232-24-1425.std	Date/Time 17-Jun-2025 13:59	

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
914	C401014C	C401014C	0.331	1.000	0.331	Eq. H1-1b	31	0.320	0.117	0.317	0.000
915	C401014C	C401014C	0.373	1.000	0.373	Eq. H1-1b	31	0.320	0.117	0.317	0.000
916	C401014C	C401014C	0.385	1.000	0.385	Eq. H1-1b	31	0.320	0.117	0.317	0.000
917	C401014C	C401014C	0.381	1.000	0.381	Eq. H1-1b	31	0.320	0.117	0.317	0.000
918	C401014C	C401014C	0.386	1.000	0.386	Eq. H1-1b	31	0.320	0.117	0.317	0.000
919	C401014C	C401014C	0.373	1.000	0.373	Eq. H1-1b	31	0.320	0.117	0.317	0.000
920	C401014C	C401014C	0.334	1.000	0.334	Eq. H1-1b	31	0.320	0.117	0.317	0.000
921	C401014C	C401014C	0.341	1.000	0.341	Eq. H1-1b	31	0.320	0.117	0.317	0.000
922	C401014C	C401014C	0.381	1.000	0.381	Eq. H1-1b	16	0.320	0.117	0.317	0.000
923	C401014C	C401014C	0.394	1.000	0.394	Eq. H1-1b	16	0.320	0.117	0.317	0.000
924	C401014C	C401014C	0.383	1.000	0.383	Eq. H1-1b	31	0.320	0.117	0.317	0.000
925	C401014C	C401014C	0.393	1.000	0.393	Eq. H1-1b	31	0.320	0.117	0.317	0.000
926	C401014C	C401014C	0.380	1.000	0.380	Eq. H1-1b	16	0.320	0.117	0.317	0.000
927	C401014C	C401014C	0.334	1.000	0.334	Eq. H1-1b	31	0.320	0.117	0.317	0.000
928	C401014C	C401014C	0.215	1.000	0.215	Eq. H1-1b	16	0.320	0.117	0.317	0.000
929	C401014C	C401014C	0.238	1.000	0.238	Eq. H1-1b	16	0.320	0.117	0.317	0.000
930	C401014C	C401014C	0.231	1.000	0.231	Eq. H1-1b	16	0.320	0.117	0.317	0.000
931	C401014C	C401014C	0.210	1.000	0.210	Eq. H1-1b	16	0.320	0.117	0.317	0.000
932	C401014C	C401014C	0.228	1.000	0.228	Eq. H1-1b	16	0.320	0.117	0.317	0.000
933	C401014C	C401014C	0.242	1.000	0.242	Eq. H1-1b	16	0.320	0.117	0.317	0.000
934	C401014C	C401014C	0.230	1.000	0.230	Eq. H1-1b	16	0.320	0.117	0.317	0.000
935	C401014C	C401014C	0.281	1.000	0.281	Eq. H1-1b	16	0.320	0.117	0.317	0.000
936	C401014C	C401014C	0.286	1.000	0.286	Eq. H1-1b	16	0.320	0.117	0.317	0.000
937	C401014C	C401014C	0.274	1.000	0.274	Eq. H1-1b	16	0.320	0.117	0.317	0.000
938	C401014C	C401014C	0.256	1.000	0.256	Eq. H1-1b	16	0.320	0.117	0.317	0.000
939	C401014C	C401014C	0.273	1.000	0.273	Eq. H1-1b	16	0.320	0.117	0.317	0.000
940	C401014C	C401014C	0.283	1.000	0.283	Eq. H1-1b	16	0.320	0.117	0.317	0.000
941	C401014C	C401014C	0.277	1.000	0.277	Eq. H1-1b	16	0.320	0.117	0.317	0.000
942	C401014C	C401014C	0.457	1.000	0.457	Eq. H1-1b	16	0.320	0.117	0.317	0.000
943	C401014C	C401014C	0.418	1.000	0.418	Eq. H1-1b	16	0.320	0.117	0.317	0.000
944	C401014C	C401014C	0.439	1.000	0.439	Eq. H1-1b	16	0.320	0.117	0.317	0.000
945	C401014C	C401014C	0.431	1.000	0.431	Eq. H1-1b	16	0.320	0.117	0.317	0.000
946	C401014C	C401014C	0.441	1.000	0.441	Eq. H1-1b	16	0.320	0.117	0.317	0.000
947	C401014C	C401014C	0.445	1.000	0.445	Eq. H1-1b	16	0.320	0.117	0.317	0.000
948	C401014C	C401014C	0.397	1.000	0.397	Eq. H1-1b	16	0.320	0.117	0.317	0.000
949	C401014C	C401014C	0.589	1.000	0.589	Eq. H2-1	32	0.320	0.117	0.317	0.000
950	C401014C	C401014C	0.409	1.000	0.409	Eq. H2-1	16	0.320	0.117	0.317	0.000
951	C401014C	C401014C	0.468	1.000	0.468	Eq. H2-1	16	0.320	0.117	0.317	0.000
952	C401014C	C401014C	0.474	1.000	0.474	Eq. H2-1	16	0.320	0.117	0.317	0.000
953	C401014C	C401014C	0.475	1.000	0.475	Eq. H2-1	16	0.320	0.117	0.317	0.000
954	C401014C	C401014C	0.442	1.000	0.442	Eq. H2-1	16	0.320	0.117	0.317	0.000
955	C401014C	C401014C	0.554	1.000	0.554	Eq. H1-1b	33	0.320	0.117	0.317	0.000
956	C401014C	C401014C	0.669	1.000	0.669	Eq. H2-1	16	0.320	0.117	0.317	0.000
957	C401014C	C401014C	0.806	1.000	0.806	Eq. H2-1	32	0.320	0.117	0.317	0.000
958	C401014C	C401014C	0.569	1.000	0.569	Eq. H2-1	16	0.320	0.117	0.317	0.000

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
959	C401014C	C401014C	0.723	1.000	0.723	Eq. H2-1	16	0.320	0.117	0.317	0.000
960	C401014C	C401014C	0.744	1.000	0.744	Eq. H2-1	16	0.320	0.117	0.317	0.000
961	C401014C	C401014C	0.722	1.000	0.722	Eq. H2-1	16	0.320	0.117	0.317	0.000
962	C401014C	C401014C	0.585	1.000	0.585	Eq. H2-1	16	0.320	0.117	0.317	0.000
963	C401014C	C401014C	0.620	1.000	0.620	Eq. H2-1	16	0.320	0.117	0.317	0.000
964	C401014C	C401014C	0.705	1.000	0.705	Eq. H2-1	32	0.320	0.117	0.317	0.000
965	C401014C	C401014C	0.851	1.000	0.851	Eq. H2-1	16	0.320	0.117	0.317	0.000
966	C401014C	C401014C	0.952	1.000	0.952	Eq. H2-1	16	0.320	0.117	0.317	0.000
967	C401014C	C401014C	0.973	1.000	0.973	Eq. H2-1	16	0.320	0.117	0.317	0.000
968	C401014C	C401014C	0.937	1.000	0.937	Eq. H2-1	16	0.320	0.117	0.317	0.000
969	C401014C	C401014C	0.822	1.000	0.822	Eq. H2-1	16	0.320	0.117	0.317	0.000
970	C401014C	C401014C	0.725	1.000	0.725	Eq. H2-1	32	0.320	0.117	0.317	0.000
971	C401014C	C401014C	0.866	1.000	0.866	Eq. H2-1	16	0.320	0.117	0.317	0.000
972	C401014C	C401014C	0.957	1.000	0.957	Eq. H2-1	16	0.320	0.117	0.317	0.000
973	C401014C	C401014C	0.969	1.000	0.969	Eq. H2-1	16	0.320	0.117	0.317	0.000
974	C401014C	C401014C	0.942	1.000	0.942	Eq. H2-1	16	0.320	0.117	0.317	0.000
975	C401014C	C401014C	0.833	1.000	0.833	Eq. H2-1	16	0.320	0.117	0.317	0.000
976	C401014C	C401014C	0.648	1.000	0.648	Eq. H2-1	16	0.320	0.117	0.317	0.000
977	C401014C	C401014C	0.885	1.000	0.885	Eq. H2-1	32	0.320	0.117	0.317	0.000
978	C401014C	C401014C	0.606	1.000	0.606	Eq. H2-1	16	0.320	0.117	0.317	0.000
979	C401014C	C401014C	0.722	1.000	0.722	Eq. H2-1	16	0.320	0.117	0.317	0.000
980	C401014C	C401014C	0.717	1.000	0.717	Eq. H2-1	16	0.320	0.117	0.317	0.000
981	C401014C	C401014C	0.722	1.000	0.722	Eq. H2-1	16	0.320	0.117	0.317	0.000
982	C401014C	C401014C	0.593	1.000	0.593	Eq. H2-1	16	0.320	0.117	0.317	0.000
983	C401014C	C401014C	0.792	1.000	0.792	Eq. H2-1	16	0.320	0.117	0.317	0.000
984	C401014C	C401014C	0.698	1.000	0.698	Eq. H2-1	32	0.320	0.117	0.317	0.000
985	C401014C	C401014C	0.492	1.000	0.492	Eq. H1-1b	16	0.320	0.117	0.317	0.000
986	C401014C	C401014C	0.455	1.000	0.455	Eq. H2-1	16	0.320	0.117	0.317	0.000
987	C401014C	C401014C	0.416	1.000	0.416	Eq. H2-1	16	0.320	0.117	0.317	0.000
988	C401014C	C401014C	0.462	1.000	0.462	Eq. H2-1	16	0.320	0.117	0.317	0.000
989	C401014C	C401014C	0.460	1.000	0.460	Eq. H1-1b	16	0.320	0.117	0.317	0.000
990	C401014C	C401014C	0.644	1.000	0.644	Eq. H2-1	16	0.320	0.117	0.317	0.000
991	C401014C	C401014C	0.533	1.000	0.533	Eq. H1-1b	16	0.320	0.117	0.317	0.000
992	C401014C	C401014C	0.542	1.000	0.542	Eq. H1-1b	16	0.320	0.117	0.317	0.000
993	C401014C	C401014C	0.482	1.000	0.482	Eq. H1-1b	16	0.320	0.117	0.317	0.000
994	C401014C	C401014C	0.459	1.000	0.459	Eq. H1-1b	16	0.320	0.117	0.317	0.000
995	C401014C	C401014C	0.430	1.000	0.430	Eq. H1-1b	16	0.320	0.117	0.317	0.000
996	C401014C	C401014C	0.468	1.000	0.468	Eq. H1-1b	16	0.320	0.117	0.317	0.000
997	C401014C	C401014C	0.453	1.000	0.453	Eq. H1-1b	16	0.320	0.117	0.317	0.000
998	C401014C	C401014C	0.276	1.000	0.276	Eq. H1-1b	16	0.320	0.117	0.317	0.000
999	C401014C	C401014C	0.307	1.000	0.307	Eq. H1-1b	16	0.320	0.117	0.317	0.000
1000	C401014C	C401014C	0.303	1.000	0.303	Eq. H1-1b	16	0.320	0.117	0.317	0.000
1001	C401014C	C401014C	0.285	1.000	0.285	Eq. H1-1b	16	0.320	0.117	0.317	0.000
1002	C401014C	C401014C	0.300	1.000	0.300	Eq. H1-1b	16	0.320	0.117	0.317	0.000
1003	C401014C	C401014C	0.301	1.000	0.301	Eq. H1-1b	16	0.320	0.117	0.317	0.000

Utilization Ratio Cont...

Beam	Analysis Property	Design Property	Actual Ratio	Allowable Ratio	Ratio (Act./Allow.)	Clause	L/C	Ax (in^2)	Iz (in^4)	Iy (in^4)	Ix (in^4)
1004	C401014C	C401014C	0.295	1.000	0.295	Eq. H1-1b	16	0.320	0.117	0.317	0.000

Failed Members

There is no data of this type.

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ASC PROFILES LLC
2110 Enterprise Blvd.
West Sacramento, CA 95691
(800) 733-4955
www.ascprofiles.com

AEP SPAN AND ASC BUILDING PRODUCTS: SINGLE SKIN STEEL ROOF AND WALL PANELS WITH EXPOSED FASTENERS

CSI Sections:
07 61 00 Sheet Metal Roofing
07 64 00 Sheet Metal Wall Cladding

1.0 RECOGNITION

ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners have been evaluated for use as exterior roof and wall covering panels. The structural and fire-resistance properties of the panels have been evaluated for compliance with the following codes:

- 2021, 2018, and 2015 International Building Code® (IBC)
- 2021, 2018, and 2015 International Residential Code® (IRC)
- 2022 and 2019 California Building Code® (CBC) – Attached supplement
- 2022 and 2019 California Residential Code® (CRC) – Attached supplement
- 2023 and 2020 City of Los Angeles Building Code (LABC) –Attached supplement
- 2023 and 2020 City of Los Angeles Residential Code (LARC) –Attached supplement

2.0 LIMITATIONS

Use of the ASC Profiles' LLC Single Skin Steel Roof and Wall Panels and Fasteners recognized in this report is subject to the following limitations:

2.1 Metal panels used in roof applications shall be applied to a solid or closely fitted deck, except where the roof covering is specifically designed to be applied to spaced support members. The panel installation tables in this report provide applicable substrate limitations.

2.2 Calculations demonstrating compliance with this report shall be submitted to the building official for approval. The calculations shall be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed.

2.3 The manufacturer's recommended roof slopes are defined within the OVERVIEW portion of this report. The minimum roof panel slopes shall conform to IBC Section 1507.4.2 or IRC Section R905.10.2, or as stated in this report.

2.4 Roof panel flashing requirements, when applicable, shall comply with IBC Sections 1503.2 and 1503.3, or IRC Sections R903.2 and R903.3. Underlayment, including installation, shall comply with IBC Sections 1507.1 and 1507.4.5, or IRC Section R905.10.5, with consideration of applicable wind conditions.

2.5 Panels used on exterior walls shall be flashed in accordance with IBC Section 1405.4 or IRC Section R905.4.6 and be placed over a water-resistive barrier in accordance with IBC Sections 1402.2, 1403.2, and 1404.2, or IRC Section R703.1.

2.6 For modifications of panel installations, design of partial panels, panel penetrations, and other panel discontinuities shall consider effects on strength and stiffness and be the responsibility of the design professional in accordance with IBC Section 1604.4, using rational engineering mechanics or in accordance with the manufacturer's installation instructions as approved by the building official.

2.7 Where panels are used as vertical diaphragm shear resistance in walls (shear wall) of light-frame construction, the walls shall be classified as a "bearing wall system" or "building frame system" with "light-framed walls with shear panels of all other materials" subject to the conditions of this classification as defined in ASCE/SEI 7, Section 12.2.

2.8 Panel use as protection of glazed openings located in wind-borne debris regions is outside the scope of this report.

2.9 Product Performance:

2.9.1 Structural: The tables provided in this report specify the gross and effective section properties, inward (positive) uniform allowable loads, allowable reactions at supports, outward (negative) uniform allowable loads, and diaphragm shear capacities, *q* (plf) and flexibility factors, *F* (10-6 in/lbs) for each of the panels described in Section 4.1 of this report.

2.9.2 Roof Classification: Roof assemblies complying with the requirements of IBC Section 1505.2, Exception 2, or IRC Section R902.1, Exception 2, are considered Class A roof assemblies. For other conditions, roof assemblies shall be listed as Class A, B, or C in accordance with ASTM E108 or UL 790, by an approved listing agency or shall be considered non-classified roofing. ASC Profiles shall be contacted for information on the specific listed assemblies.

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2.9.3 Wall Assembly Fire-Resistance: Wall panels are limited to installations where non-fire-resistance-rated construction is permitted by the IBC or IRC. Wall panels may be permitted in fire-resistance-rated wall assemblies based on successful testing in accordance with the requirements prescribed in IBC Section 703.

2.9.4 Air and Water Infiltration: Air and water infiltration resistance is outside the scope of this report. Weather protection shall comply with Sections 2.4 and 2.5 of this report.

2.9.5 Hail Resistance: Hail resistance is outside the scope of this report.

2.9.6 Wind-blown Debris Resistance: Wind-blown debris resistance is outside the scope of this report.

2.10 The Single Skin Steel Roof and Wall Panels recognized in this report are produced by ASC Profiles LLC in Sacramento, California, and Salem, Oregon.

3.0 PRODUCT USE

General Design and Installation shall be in accordance with the referenced codes in Section 1.0 of this report, the information provided in this report, and ASC Profiles' product installation guides. Where conflicts occur, the more restrictive shall govern.

4.0 PRODUCT DESCRIPTION

4.1 Panels: The panels are available in various configurations as illustrated in the figures accompanying the tables in this report. Panels are either unpainted or are provided with a painted finish. Additional information on the panel configurations is provided in the OVERVIEW portion of this report.

The roof panels comply with the requirements for metal roof panels in Chapter 15 of the IBC and Section R905 of the IRC. The wall panels comply with the requirements for steel wall coverings in Chapter 14 of the IBC and Section R703 of the IRC.

4.2 Base Materials: All No. 18 and No. 20 gage panels are manufactured from sheet steel with G90 galvanized coatings conforming to ASTM A653 SS Grade 40.

All No. 22 and No. 24 gage panels are manufactured from AZ50 aluminum-zinc alloy coated steel sheet conforming to ASTM A792 SS Grade 50, or from G90 galvanized sheet per ASTM A653 SS Grade 50.

All No. 26 and No. 29 gage panels are manufactured from AZ50 aluminum-zinc alloy coated steel sheets conforming to ASTM A792 SS Grade 80. The panels are also available prepainted in accordance with ASTM A755.

4.3 Fasteners: The fasteners' size and type requirements are identified in the panel installation tables within this report. All fasteners shall be zinc-plated with an added corrosion-resistant coating, or of a 300 series stainless steel construction. Self-tapping metal-to-metal fasteners shall comply with ASTM C1513. Fasteners installed into treated wood shall be 300 series stainless steel or designed specifically for use with treated wood.

4.4 Substrates: ASC Profiles' roof and wall panels may be fastened to numerous substrates including, but not limited to, the following:

- Cold-formed steel in accordance with AISI S100
- Hot rolled steel in accordance with AISC 360
- Concrete in accordance with ACI 318
- Plywood and OSB in accordance with DOC PS-1 and DOC PS-2
- Dimensional lumber in accordance with ANSI/AWC NDS®

The panel installation tables within this report provide applicable substrate and fastening requirements. For other support conditions, structural calculations complying with the applicable code shall be submitted to the building official for approval.

5.0 IDENTIFICATION

A permanent label or a die-stamp label bearing the name and address of the manufacturers, the model number, and this evaluation report number (Evaluation Report ER-550) identifies the products listed in this report. The identification labels may also include one of the following IAPMO Uniform Evaluation Service Marks of Conformity:

IAPMO ES or UES

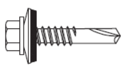
IAPMO UES ER-550

6.0 SUBSTANTIATING DATA

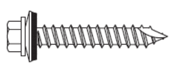
Data submitted in conformance with IAPMO UES Evaluation Criteria Single Skin Steel Roof and Wall Panels, EC-011, revised January 2022. All product testing is from laboratories in compliance with ISO/IEC 17025.

Fasteners:

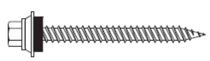
Below is a summary of the fastener sizes and types specified within this report. Fastener size and type shall be compatible with the material type, thickness, and grade of the supporting members. Section 4.3 of this report provides additional fastener requirements. Hex washer head (HWH) fasteners are shown below. However, alternative fastener heads are acceptable. Fasteners require sealing washers for weather-tight applications.



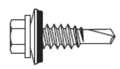
#12 metal-to-metal fastener (self-drilling point shown)



#14 metal-to-wood fastener (milled point shown)



#9 metal-to-wood fastener (dimensional lumber only; self-piercing point shown)



Side Lap Screw (#12 or #14 most common)

To assist in the evaluation of panel shear resistance, fastener connection shear strengths have been provided in Table B of this report for #12 fasteners into steel supports. These capacities are based on AISI S100, Section J4.3 for fastener shear strength and shear strengths limited by tilting and bearing. The capacities listed within Table B of this report are used to determine edge (perimeter) fastening requirements for shear resistance. The general notes and shear and flexibility tables of this report provide further information.

TABLE B

#12 Fastener Connection Shear Strengths *(lbs)*

Substrate Material / Grade	Substrate Thickness	Panel Gauge / Grade: 29ga (.0139") Gr80 ASD W/Ω	29ga (.0139") Gr80 LRFD ϕW	26ga (.0173") Gr80 ASD W/Ω	26ga (.0173") Gr80 LRFD ϕW	24ga (.0232") Gr50 ASD W/Ω	24ga (.0232") Gr50 LRFD ϕW	22ga (.0294") Gr50 ASD W/Ω	22ga (.0294") Gr50 LRFD ϕW	20ga (.0354") Gr40 ASD W/Ω	20ga (.0354") Gr40 LRFD ϕW	18ga (.0459") Gr40 ASD W/Ω	18ga (.0459") Gr40 LRFD ϕW
Steel (Gr 50 min.)	16ga *(.0590")* min.	222	*332*	276	*414*	293	*440*	371	*557*	378	*568*	491	*736*
	18ga *(.0459")*	222	*332*	276	*414*	293	*440*	371	*557*	378	*568*	416	*624*
	20ga *(.0354")*	222	*332*	276	*414*	286	*429*	294	*441*	282	*423*	282	*423*
	22ga *(.0294")*	219	*329*	242	*364*	227	*341*	213	*320*	213	*320*	213	*320*
Steel (Gr 33 min.)	16ga *(.0590")* min.	222	*332*	276	*414*	293	*440*	371	*557*	378	*568*	433	*650*
	18ga *(.0459")*	222	*332*	276	*414*	291	*437*	319	*479*	306	*459*	288	*432*
	20ga *(.0354")*	222	*332*	251	*377*	229	*344*	211	*316*	195	*293*	195	*293*
	22ga *(.0294")*	203	*304*	199	*298*	167	*251*	148	*221*	148	*221*	148	*221*

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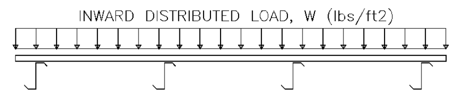
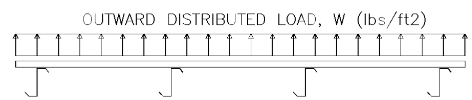
Positive (inward/ gravity) load capacity:



1. For uniform inward loading conditions, the appropriate support spacing is identified by referring to the inward load tables within this report. For conditions not defined by these tables, values are readily derived through standard engineering mechanics using the panel's section properties.
2. The ASD allowable (W/Ω) and LRFD factored (ϕW) panel strengths have been evaluated for bending, shear, and combined bending and shear.
3. The L/240, L/180, and L/120 values are based on allowable service load deflections.
4. Table values denoted by '- 'indicate that capacities are limited by allowable panel strength, not deflection. In those instances, the panel strength limits (W/Ω and ϕW) are reached prior to the deflection limits (L/180, etc.) being reached.

Negative (outward/ uplift) load capacity:



1. Common panel attachment combinations and associated outward load capacities are provided within this report. Panel system negative (outward) uniform load capacities are based on fastener pullout, pullover, fastener tension, panel outward bending strength, and IBC Section 1604.3.1 deflection limit of L/60 for formed metal sheet roofing and siding.
2. Using the project's defined outward wind loads, the appropriate *Panel System Negative (Outward) Uniform Load Capacity* table shall be reviewed to select a panel attachment that equals or exceeds the project wind load requirements. Not all possible fastener and substrate combinations are listed within the report. Alternative combinations are acceptable (i.e. attaching a panel assembly with fasteners into a concrete substrate), subject to the approval of the building official, provided an analysis is performed by a registered design professional based on the failure modes noted above.
3. Fastener capacities in steel are based on AISI S100. Fastener capacities in wood substrates are based on ANSI/AWC NDS with a combined adjustment factor of 1.6 (2.16 LRFD). The specific gravity of 0.50 was used for Plywood, OSB, and Lumber (DFL) substrates.
4. Although nominal fastener sizes are provided in the tables, the appropriate fastener thread and point type shall still be properly specified for the selected substrate.
5. The 1-inch minimum fastener penetration specified for the Douglas Fir-Larch values applies to the usable thread length and this minimum depth does not include the tapered portion of the fastener.
6. Negative load table capacities are based on a multi-span (triple-span) condition.

Shear and flexibility tables:

1. Shear capacity (lbs/lineal ft) and flexibility factors (10^{-6} in/lb) are presented in the tables. Product capacities are determined in accordance with the American Iron and Steel Institute's *North American Standard for the Design of Profiled Steel Diaphragm Panels* (AISI S310). Values are based on seismic governed loading conditions ($\Omega_d = 2.30$ and $\phi_d = 0.7$) and are therefore conservative for applications governed by wind loads ($\Omega_d = 2.00$ and $\phi_d = 0.8$).
2. Perimeter (edge) fastener spacing parallel to panel ribs has not been included in performance tables. Maximum spacing for edge fasteners parallel to panel flutes is defined as:

   Max. edge fastener spacing = the lesser of 1) shear capacity per fastener (lb) / shear demand (lb/ft); or 2) the interior side-seam fastener spacing

   The Fastener Connection Shear Strength performance table within this report assists with the needed fastener capacity in the above equation.
3. Tabulated shear capacities apply to both roof and wall diaphragms.
4. Tables apply to panels installed over steel supports. Evaluation of shear and flexibility over wood framing is outside the scope of this report.
5. Capacities are based on panel gauge, span, support fastener size and type, support attachment pattern, side lap connection fastener, and side lap fastener spacing. Capacities based on minimum #12 side lap and support screws.
6. Interpolation of diaphragm shear strength and flexibility factor between adjacent spans or side seam spacing is permitted provided the higher value does not exceed the lower value by more than 50 percent.
7. Flexibility Factors provided in charts are provided in an A +/- B(R) format.

   Where:

   A and B are constants provided in the shear tables.
   R = panel attachment spacing (panel span) ÷ panel length.

13.0 Strata Rib®

Figure 13.1 - Basic Dimensions and Panel Attachment (Strata Rib):

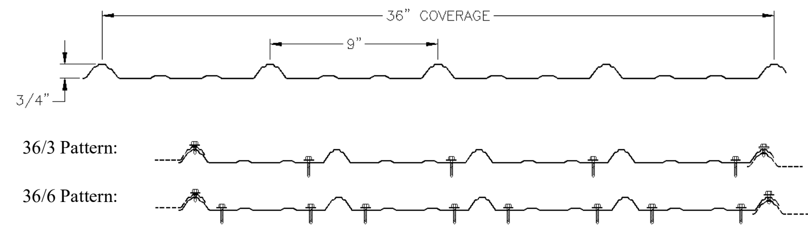


TABLE 13.1 - Section Properties (Strata Rib):

Gauge	Weight	Base Metal Thickness	Yield Strength	Tensile Strength	Gross Section Properties: Area	Moment of Inertia	Distance to N.A. from Bottom	Positive Section Modulus	Negative Section Modulus
	w psf	t in	Fy ksi	Fu ksi	A_g in²/ft	I_g in⁴/ft	y_b in	S_g+ in³/ft	S_g- in³/ft
29	0.65	0.0139	80	82	0.1914	0.0103	0.16	0.0170	0.0664
26	0.81	0.0173	80	82	0.2382	0.0130	0.16	0.0211	0.0817

Gauge	Effective Section Properties							Uniform Load Only	
	Area	Positive			Negative			$I_d = (2I_e+I_g)/3$	
		Moment of Inertia	Distance to N.A. from Bottom	Section Modulus	Moment of Inertia	Distance to N.A. from Bottom	Section Modulus		
	A_e/ft in²	I_e+ in⁴/ft	y_b in	S_e+ in³/ft	I_e- in⁴/ft	y_b in	S_e- in³/ft	I+ in⁴/ft	I- in⁴/ft
29	0.0285	0.0103	0.16	0.0170	0.0070	0.34	0.0144	0.0103	0.0081
26	0.0379	0.0130	0.16	0.0211	0.0090	0.32	0.0181	0.0130	0.0103

TABLE 13.2 - Inward (Positive) Uniform Allowable Loads (Strata Rib):

Gauge	Span	Limit Condition	Panel Span (Support Spacing) 16"	2' - 0"	2' - 6"	3' - 0"	3' - 6"	4' - 0"	4' - 6"	5' - 0"	6' - 0"
29	Single Span	ASD, W/Ω	230	102	65	45	33	25	20	16	11
		LRFD, ϕW	365	162	103	72	53	40	32	26	18
		L/240	-	85	43	25	16	11	7	5	3
		L/180	-	-	58	33	21	14	10	7	4
		L/120	-	-	-	-	32	21	15	11	6
	Double Span	ASD, W/Ω	185	84	54	38	28	21	17	13	9
		LRFD, ϕW	279	126	81	57	42	32	25	20	13
		L/240	-	-	-	-	-	-	-	-	8
		L/180	-	-	-	-	-	-	-	-	-
		L/180	-	-	-	-	-	-	-	-	-
	Triple Span	ASD, W/Ω	227	104	67	47	35	26	20	17	12
		LRFD, ϕW	342	157	101	70	52	39	31	26	18
		L/240	-	-	-	-	30	20	14	10	6
		L/180	-	-	-	-	-	-	19	14	8
		L/120	-	-	-	-	-	-	-	-	-
26	Single Span	ASD, W/Ω	285	126	81	56	41	32	25	20	14
		LRFD, ϕW	453	200	128	89	65	50	40	32	22
		L/240	-	107	55	32	20	13	9	7	4
		L/180	-	-	73	42	27	18	12	9	5
		L/120	-	-	-	-	40	27	19	14	8
	Double Span	ASD, W/Ω	233	105	68	47	34	27	21	17	12
		LRFD, ϕW	350	159	102	71	52	40	32	26	18
		L/240	-	-	-	-	-	-	-	16	10
		L/180	-	-	-	-	-	-	-	-	-
		L/120	-	-	-	-	-	-	-	-	-
	Triple Span	ASD, W/Ω	285	131	85	59	43	33	26	21	15
		LRFD, ϕW	429	197	128	89	65	50	40	32	23
		L/240	-	-	-	-	38	25	18	13	7
		L/180	-	-	-	-	-	-	24	17	10
		L/120	-	-	-	-	-	-	-	-	-

TABLE 13.3 - Allowable Reactions at Supports (Strata Rib):

Reactions at Supports based on Web Crippling							
Gauge	Condition	Bearing Length of Webs: ASD (Pn/Ω) *(lbs/ft width)* 1"	1.5"	2"	LRFD (ϕPn) *(lbs/ft width)* 1"	1.5"	2"
29	End	131	151	168	200	231	257
	Interior	187	212	233	278	315	346
26	End	196	224	249	299	343	380
	Interior	285	321	351	424	478	523

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TABLE 13.4 - Outward (Negative) Uniform Allowable Loads (No. 29 gauge Strata Rib):

Strata Rib, 29ga

Panel System Negative (Outward) Uniform Load Capacity, (lbs/ft^2) *Based on Triple-Span*

Substrate Material / Grade	Thick-ness	Fastener Nom. Size	Fastener Attach. Pattern	16" ASD W/Ω	16" LRFD ϕW	2' - 0" ASD W/Ω	2' - 0" LRFD ϕW	2' - 6" ASD W/Ω	2' - 6" LRFD ϕW	3' - 0" ASD W/Ω	3' - 0" LRFD ϕW	3' - 6" ASD W/Ω	3' - 6" LRFD ϕW	4' - 0" ASD W/Ω	4' - 0" LRFD ϕW	4' - 6" ASD W/Ω	4' - 6" LRFD ϕW	5' - 0" ASD W/Ω	5' - 0" LRFD ϕW	6' - 0" ASD W/Ω	6' - 0" LRFD ϕW
Steel (Gr 50 min.)	≥10ga *(.1350")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	285	*427*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	12ga *(.1050")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	285	*427*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	14ga *(.0700")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	278	*418*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	16ga *(.0590")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	235	*352*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	18ga *(.0459")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	183	*274*	122	*183*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	20ga *(.0354")*	#12	36/8	282	*422*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	141	*211*	94	*141*	75	*113*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	22ga *(.0294")*	#12	36/8	234	*351*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	117	*175*	78	*117*	62	*94*	52	*78*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
Steel (Gr 33 min.)	≥10ga *(.1350")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	285	*427*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	12ga *(.1050")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	285	*427*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	14ga *(.0700")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	193	*289*	127	*193*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	16ga *(.0590")*	#12	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	162	*244*	108	*162*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	18ga *(.0459")*	#12	36/8	253	*379*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	126	*190*	84	*126*	67	*101*	56	*84*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	20ga *(.0354")*	#12	36/8	195	*292*	127	*195*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	97	*146*	65	*97*	52	*78*	43	*65*	37	*56*	32	*49*	25	*40*	20	*32*	14	*19*
	22ga *(.0294")*	#12	36/8	162	*243*	108	*162*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#12	36/4	81	*121*	54	*81*	43	*65*	36	*54*	31	*46*	27	*40*	24	*36*	20	*32*	14	*19*
Plywood & OSB	15/32"	#14	36/8	259	*349*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#14	36/4	129	*175*	86	*116*	69	*93*	57	*78*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	19/32"	#14	36/8	286	*442*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#14	36/4	164	*221*	109	*147*	81	*118*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
	23/32"	#14	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#14	36/4	198	*268*	127	*178*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
Lumber (DFL)	1" min	#9	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#9	36/4	194	*262*	127	*174*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#14	36/8	286	*454*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*
		#14	36/4	265	*358*	127	*202*	81	*129*	57	*90*	42	*66*	32	*50*	25	*40*	20	*32*	14	*19*

Attachment Spacing, (ft-in)

TABLE 13.5 - Outward (Negative) Uniform Allowable Loads (No. 26 gauge Strata Rib):

Strata Rib, 26ga

Panel System Negative (Outward) Uniform Load Capacity, (lbs/ft²) *Based on Triple-Span*

Substrate Material / Grade	Substrate Thickness	Fastener Nom. Size	Fastener Attach. Pattern	16" ASD W/Ω	16" LRFD φW	2'-0" ASD W/Ω	2'-0" LRFD φW	2'-6" ASD W/Ω	2'-6" LRFD φW	3'-0" ASD W/Ω	3'-0" LRFD φW	3'-6" ASD W/Ω	3'-6" LRFD φW	4'-0" ASD W/Ω	4'-0" LRFD φW	4'-6" ASD W/Ω	4'-6" LRFD φW	5'-0" ASD W/Ω	5'-0" LRFD φW	6'-0" ASD W/Ω	6'-0" LRFD φW
Steel (Gr 50 min.)	≥10ga (.1350")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	355	*532*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	12ga (.1050")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	355	*532*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	14ga (.0700")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	278	*418*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	16ga (.0590")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	235	*352*	156	*235*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	18ga (.0459")	#12	36/8	355	*548*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	183	*274*	122	*183*	97	*146*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	20ga (.0354")	#12	36/8	282	*422*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	141	*211*	94	*141*	75	*113*	63	*94*	51	*80*	39	*63*	31	*49*	25	*40*	18	*24*
	22ga (.0294")	#12	36/8	234	*351*	156	*234*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	117	*175*	78	*117*	62	*94*	52	*78*	45	*67*	39	*58*	31	*49*	25	*40*	18	*24*
Steel (Gr 33 min.)	≥10ga (.1350")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	355	*532*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	12ga (.1050")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	289	*434*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	14ga (.0700")	#12	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	193	*289*	129	*193*	101	*154*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	16ga (.0590")	#12	36/8	325	*487*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	162	*244*	108	*162*	87	*130*	70	*108*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	18ga (.0459")	#12	36/8	253	*379*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	126	*190*	84	*126*	67	*101*	56	*84*	48	*72*	39	*63*	31	*49*	25	*40*	18	*24*
	20ga (.0354")	#12	36/8	195	*292*	130	*195*	101	*156*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	97	*146*	65	*97*	52	*78*	43	*65*	37	*56*	32	*49*	29	*43*	25	*39*	18	*24*
	22ga (.0294")	#12	36/8	162	*243*	108	*162*	86	*130*	70	*108*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#12	36/4	81	*121*	54	*81*	43	*65*	36	*54*	31	*46*	27	*40*	24	*36*	22	*32*	18	*24*
Plywood & OSB	15/32"	#14	36/8	259	*349*	158	*233*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#14	36/4	129	*175*	86	*116*	69	*93*	57	*78*	49	*67*	39	*58*	31	*49*	25	*40*	18	*24*
	19/32"	#14	36/8	328	*442*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#14	36/4	164	*221*	109	*147*	87	*118*	70	*98*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
	23/32"	#14	36/8	355	*535*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#14	36/4	198	*268*	132	*178*	101	*143*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
Lumber (DFL)	1" min	#9	36/8	355	*523*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#9	36/4	194	*262*	129	*174*	101	*140*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#14	36/8	355	*563*	158	*250*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*
		#14	36/4	265	*358*	158	*238*	101	*160*	70	*111*	51	*82*	39	*63*	31	*49*	25	*40*	18	*24*

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TABLE 13.6 - Shear and Flexibility (No. 29 gauge Strata Rib, 36/4):

With min. #12 SD HWH Screw into supports and min. #12 SD HWH Screw at side laps

Allowable Shear (q_a) (plf), Factored Shear (q_f) (plf), and Flexibility Factor (F) (10^{-6} in/lbs)

Support Fastener Pattern	Gage	Seam Attach. Spacing	Span →	16"		2' - 0"		2' - 6"		3' - 0"		3' - 6"		4' - 0"		4' - 6"		5' - 0"		6' - 0"	
36/4	29 ga	6"	q_a q_f	293	472	262	422	251	405	244	392	238	383	233	375	229	369	204	326	141	226
			F	22.3 -1.5R		23.1 -1.3R		23.5 -1.2R		23.8 -1.1R		24 -1R		24.1 -0.9R		24.3 -0.9R		24.4 -0.8R		24.5 -0.7R	
		12"	q_a q_f	265	427	206	331	204	328	178	286	180	291	162	261	166	268	152	245	141	226
			F	25.8 -3.1R		27.8 -3.1R		28.8 -3R		29.6 -2.9R		30.2 -2.8R		30.7 -2.7R		31.1 -2.5R		31.4 -2.4R		31.9 -2.2R	
		18"	q_a q_f	226	364	206	331	174	280	150	241	157	252	140	225	126	203	134	216	114	183
			F	27.9 -4.2R		30.9 -4.6R		32.5 -4.7R		33.8 -4.7R		34.9 -4.6R		35.8 -4.5R		36.5 -4.4R		37.2 -4.3R		38.2 -4R	
		24"	q_a q_f	226	364	168	270	174	280	150	241	131	211	116	187	126	203	115	185	96	155
			F	29.2 -5R		33.1 -5.8R		35.3 -6.1R		37.1 -6.2R		38.6 -6.2R		39.9 -6.2R		41 -6.2R		42 -6.1R		43.5 -5.8R	
		30"	q_a q_f	226	364	168	270	139	224	150	241	131	211	116	187	104	168	93	150	96	155
			F	30.2 -5.6R		34.8 -6.7R		37.5 -7.2R		39.7 -7.5R		41.6 -7.7R		43.3 -7.8R		44.7 -7.8R		46 -7.8R		48.1 -7.6R	
		36"	q_a q_f	226	364	168	270	139	224	118	190	131	211	116	187	104	168	93	150	77	124
			F	31 -6R		36 -7.5R		39.2 -8.2R		41.8 -8.6R		44.1 -9R		46.1 -9.2R		47.9 -9.3R		49.4 -9.4R		52.1 -9.3R	

Note: Presented shear capacities do not include parallel edge fastener limitations. See General Notes for details. 'R' = Panel attachment spacing (span) ÷ panel length.

TABLE 13.7 - Shear and Flexibility (No. 26 gauge Strata Rib, 36/4):

With min. #12 SD HWH Screw into supports and min. #12 SD HWH Screw at side laps

Allowable Shear (q_a) (plf), Factored Shear (q_f) (plf), and Flexibility Factor (F) (10^{-6} in/lbs)

Support Fastener Pattern	Gage	Seam Attach. Spacing	Span →	16"		2' - 0"		2' - 6"		3' - 0"		3' - 6"		4' - 0"		4' - 6"		5' - 0"		6' - 0"	
36/4	26 ga	6"	q_a q_f	374	602	338	545	326	526	318	511	311	501	306	493	302	486	285	456	198	317
			F	18.6 -1.3R		19.3 -1.2R		19.6 -1.1R		19.9 -1R		20 -0.9R		20.2 -0.8R		20.3 -0.8R		20.4 -0.7R		20.5 -0.6R	
		12"	q_a q_f	339	546	266	428	266	427	232	374	237	382	214	344	220	354	202	325	194	312
			F	21.7 -2.7R		23.4 -2.8R		24.4 -2.7R		25 -2.6R		25.6 -2.5R		26 -2.4R		26.4 -2.3R		26.7 -2.2R		27.2 -2R	
		18"	q_a q_f	288	464	266	428	225	363	195	313	206	331	184	296	166	268	178	286	151	243
			F	23.5 -3.7R		26.2 -4.1R		27.7 -4.2R		28.9 -4.2R		29.8 -4.1R		30.6 -4R		31.3 -3.9R		31.9 -3.8R		32.8 -3.6R	
		24"	q_a q_f	288	464	215	346	225	363	195	313	171	275	151	244	166	268	151	244	128	206
			F	24.8 -4.5R		28.2 -5.2R		30.2 -5.4R		31.8 -5.6R		33.2 -5.6R		34.3 -5.6R		35.3 -5.5R		36.2 -5.4R		37.6 -5.2R	
		30"	q_a q_f	288	464	215	346	178	287	195	313	171	275	151	244	136	219	123	198	128	206
			F	25.6 -5R		29.7 -6R		32.1 -6.5R		34.2 -6.7R		35.9 -6.9R		37.4 -7R		38.6 -7R		39.8 -7R		41.7 -6.8R	
		36"	q_a q_f	288	464	215	346	178	287	152	245	171	275	151	244	136	219	123	198	102	164
			F	26.3 -5.4R		30.9 -6.7R		33.7 -7.3R		36 -7.8R		38.1 -8R		39.9 -8.2R		41.5 -8.3R		42.9 -8.4R		45.2 -8.4R	

Note: Presented shear capacities do not include parallel edge fastener limitations. See General Notes for details. 'R' = Panel attachment spacing (span) ÷ panel length.

Note: Shear Tables 12.6 & 12.7 based on 36/4 attachment pattern. Values acceptable for use (conservative) for 36/8 pattern.

CALIFORNIA SUPPLEMENT

ASC PROFILES LLC
2110 Enterprise Blvd.
West Sacramento, CA 95691
(800) 360-2477
www.ascprofiles.com

AEP SPAN AND ASC BUILDING PRODUCTS: SINGLE SKIN STEEL ROOF AND WALL PANELS WITH EXPOSED FASTENERS

CSI Sections:
07 61 00 Sheet Metal Roofing
07 64 00 Sheet Metal Wall Cladding

1.0 RECOGNITION

ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners as evaluated and represented in IAPMO UES Evaluation Report ER-550 and with changes as noted in this supplement are satisfactory alternatives for use in buildings built under the following codes (and regulations):

- 2022 and 2019 California Building Code® (CBC)
- 2022 and 2019 California Residential Code® (CRC)

2.0 LIMITATIONS

Use of the ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners recognized in this report is subject to the following limitations:

2.1 ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners shall comply with the provisions applicable to the 2021 and 2018 IBC or 2021 and 2018 IRC in IAPMO UES ER-550.

2.2 Roof panels may be used in "new buildings located in any Fire Hazard Severity Zone or any Wildland-Urban Interface Fire Area designated by the enforcing agency constructed after the application date stipulated in CBC Section 701A.3.1 shall comply with the provisions" in accordance with Sections 701A.3 and 705A of the CBC and with the 2021 or 2018 IBC as presented in ER-550.

2.3 Pertaining to structures under the jurisdiction of DSA and HCAi (formerly OSHPD), designs for the transfer of anchorage forces into the diaphragm shall comply with CBC Section 1613A.5.1 of the 2022 and 2019 CBC.

2.4 This supplement expires concurrently with ER-550.

For additional information about this evaluation report please visit www.uniform-es.org or email us at info@uniform-es.org

EVALUATION REPORT Number: 550
Originally Issued: 01/31/2018 Revised: 11/27/2024 Valid Through: 01/31/2026

CITY OF LOS ANGELES SUPPLEMENT

ASC PROFILES LLC
2110 Enterprise Blvd.
West Sacramento, CA 95691
(800) 360-2477
www.ascprofiles.com

AEP SPAN AND ASC BUILDING PRODUCTS: SINGLE SKIN STEEL ROOF AND WALL PANELS WITH EXPOSED FASTENERS

CSI Sections:
07 61 00 Sheet Metal Roofing
07 64 00 Sheet Metal Wall Cladding

1.0 RECOGNITION

ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners as evaluated and represented in IAPMO UES Evaluation Report ER-550 and with changes as noted in this supplement are satisfactory alternatives for use in buildings built under the following codes (and regulations):

- 2023 and 2020 City of Los Angeles Building Code (LABC)
- 2023 and 2020 City of Los Angeles Residential Code (LARC)

2.0 LIMITATIONS

Roof panels recognized in this report supplement is subject to the following limitations:

2.1 ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners shall be installed and identified in accordance with this report, codes listed in Section 1.0 of this report, and the manufacturer's instructions. Where conflicts occur, the more restrictive shall govern.

2.2 ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners shall comply with the provisions applicable to the 2022 and 2019 CBC or CRC, as applicable, in the California supplement to IAPMO UES ER-550.

2.3 Prior to installation, calculations and details demonstrating compliance with this approval report and the 2023 and 2020 Los Angeles Building Code or 2023 and 2020 Los Angeles Residential Code, as applicable, shall be submitted to the structural plan check section for review and approval. The calculations and details shall be prepared stamped, and signed by a California registered design professional.

2.4 The design, installation, and inspection of the ASC Profiles' LLC Single Skin Steel Roof and Wall Panels with Exposed Fasteners shall be in accordance with LABC Chapters 15, 16, and 17 as applicable, due to local amendments to these chapters.

2.5 This supplement expires concurrently with ER-550.

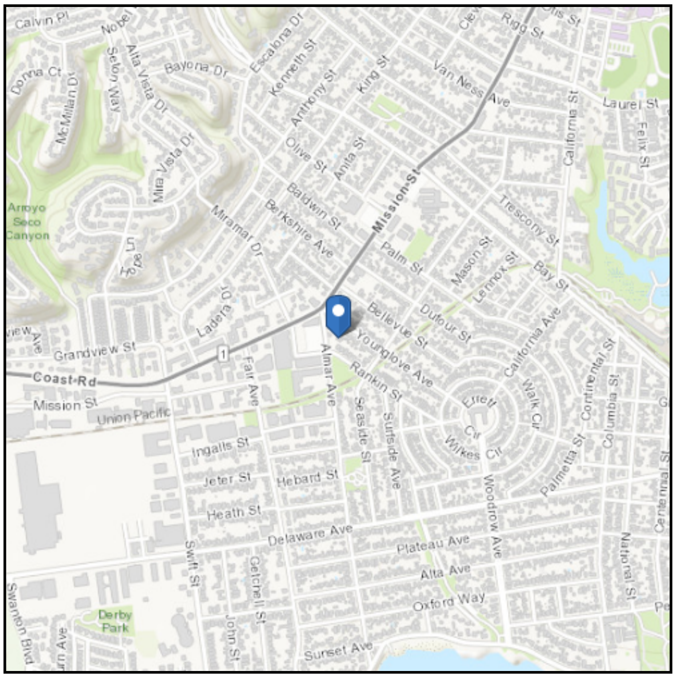
For additional information about this evaluation report please visit www.uniform-es.org or email us at info@uniform-es.org

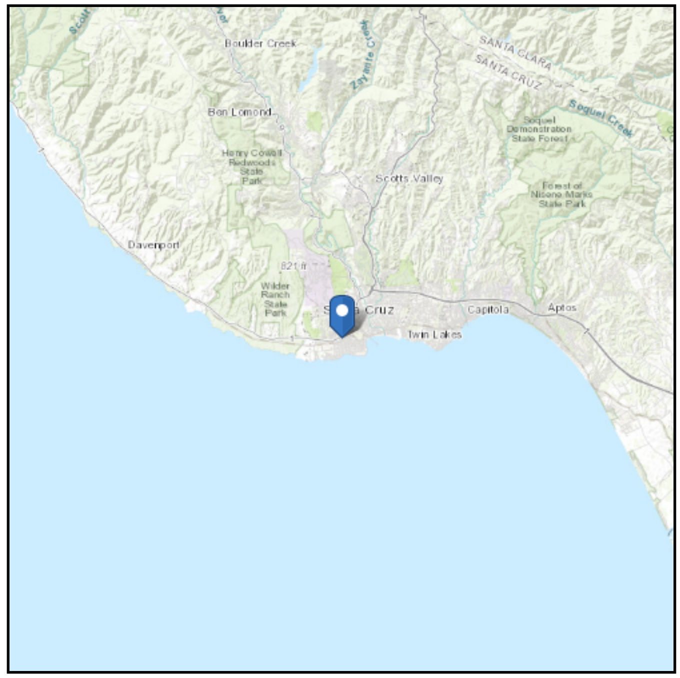
ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE Hazards Report

Address:
335 Younglove Ave
Santa Cruz, California
95060

Standard:	ASCE/SEI 7-16	**Latitude:**	36.961739
Risk Category:	IV	**Longitude:**	-122.043235
Soil Class:	D - Stiff Soil	**Elevation:**	64.67435823427755 ft (NAVD 88)





Wind

Results:

Wind Speed	103 Vmph
10-year MRI	63 Vmph
25-year MRI	70 Vmph
50-year MRI	74 Vmph
100-year MRI	78 Vmph

Data Source:	ASCE/SEI 7-16, Fig. 26.5-1D and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed:	Tue Jun 17 2025

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 1.6% probability of exceedance in 50 years (annual exceedance probability = 0.00033, MRI = 3,000 years).

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

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.609	S_{D1} :	N/A
S_1 :	0.611	T_L :	12
F_a :	1	PGA :	0.677
F_v :	N/A	PGA_M :	0.745
S_{MS} :	1.609	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1.5
S_{DS} :	1.073	C_v :	1.422

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

Data Accessed: Tue Jun 17 2025

Date Source: [USGS Seismic Design Maps]

Snow

Results:

Ground Snow Load, p_g :	0 lb/ft^2
Mapped Elevation:	64.7 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Tue Jun 17 2025

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

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.