DOTec Corp.

Customized Engineering Solutions

STRUCTURAL CALCULATIONS

OF

753 OLIVE BRANCH RD – FUQUAY-VARINA, NC

PROJECT NO.:	#B2508027
CLIENT:	Americana Outdoors

BILLING ADDRESS:

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COUNTRY:	United States

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REVISION HISTORY

#	DATE	DESCRIPTION	PREPARED BY	CHECKED BY
01	8/7/2025	First Submittal	CLE	MKY
02				
03				

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118 N 2nd St., St. Charles, MO 63301. Phone: (636) 724-9872

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1 PROJECT OVERVIEW

Figure-1: 3D view of the structure

2 IINTRODUCTION

The purpose of this report is to present the structural analysis and design check for the canopy as detailed in the design drawings, under expected loads and combinations to ensure that this structure is safe under all load combinations according to the standard.

3 STANDARDS AND CODES

- The adopted standards in this document are listed as follows:

Code	Description
ACI 318-14	Building Code Requirements for Structural Concrete
ASCE 7-16	American Society of civil engineering
IBC 2018	International Building Code
AISC 360-16	Specification for Structural Steel Buildings
AISI S100-16	North American Specification for the Design of Cold-formed Steel Structural Members
AWS D1.1	Structural Welding Code – Steel, 2015
AWS D1.4	Structural Welding Code - Reinforcing Steel Including Metal Inserts and Connections in Reinforced Concrete Construction, 2015
ADM 2015	Specification for Aluminum Structures
TMS 402-16	Building Code Requirements for Masonry Structures

4 MATERIAL PROPERTIES

- The following table lists the mechanical properties of structural steel and connecting bolts used:

Reinforced Concrete	
Concrete f'c	Building Code Requirements for Structural Concrete
Reinforcing Steel fy	American Society of civil engineering

Structural Steel	
Rolled shapes and plates	- ASTM A572, Fy = 50 ksi, UNO
Wide flange shapes	- ASTM A992, Fy=50 ksi
HSS sections	- ASTM A500, Gr C, Fy=50 ksi
Anchor Rods:	ASTM F1554 Grade 105
Welding Electrode:	E70XX
Connection bolts:	ASTM A3125 Grade A325/A490 with ASTM A959 direct tension indicator washers

	or ASTM A3125 Grade F1852/F2280 twist-off-type tension control bolts, as required (standard holes.

Structural Aluminum	
Aluminum Extrusion	ALUMINUM ALLOY 6061 – T6
Aluminum Bracing	ALUMINUM ALLOY 6061 – T6

5 Serviceability Criteria

1. Foundation Settlement Limit: 1” total, ½” differential settlement, maximum
2. Beam/Girder/Frame Deflection Limit: L/240 For Live Load, L/180 for Dead + Live load
3. Masonry Veneer Vertical Deflection: L/600
4. Overall Structure Drift Limit: H/480 based on a 10-year MRI wind speed of 72 mph.

6 STRUCTURAL ANALYSIS

The analysis of the steel structure was conducted using RISA-3D structural analysis software to determine the internal straining actions in structural members and reactions of the structures on the foundations whenever applicable.

7 LOAD CALCULATION

118 N 2nd St., St. Charles, MO 63301. Phone: (636) 724-9872

PROJECT DATA

BUILDING		
Roof Type:	Hip	
Roof Width:	18	ft
Roof Length:	24	ft
Eave Height:	8	ft
Roof Pitch:	4	: 12
REFERENCE CODES		
International Building Code:	IBC 2015	
American Society of Civi Engineers:	ASCE 7-10	
Aluminum Association:	Aluminum Design Manual 2015 (ADM)	
American Institute of Steel Construction:	AISC 360-10	
American Welding Society:	AWS D1.1: 2010	
CLASSIFICATION		
Risk Category:	II	
Occupancy:	U	(non-separated use)
Construction Type:	II-B	
Floor Area:	432	sq. ft.
Occupant Load:	7 sq. ft./occupant =	61 occupants/shelter

DESIGN LOADS & CRITERIA

DEAD & LIVE LOADS		
Roof Dead Load:	5	psf
Collateral Dead Load:	0	psf
Roof Live Load:	20	psf
WIND		
Wind Importance Factor:	1.00	
Basic Wind Speed:	116	mph (3-second gust)
Topography:	Homogeneous	(assumed)
Surface Roughness:	C	(assumed)
Wind Exposure:	C	
Enclosure Classification:	Open	
SNOW		
Snow Importance Factor:	1.00	
Ground Snow Load:	15	psf
SEISMIC		
Seismic Importance Factor:	1.00	
Soil Site Class:	D	(assumed)
Spectral Response Acceleration, 0.2 Second, S_s =	0.178	
Spectral Response Acceleration, 1.0 Second, S_1 =	0.085	
Spectral Response Coefficient, S_{ds} =	0.19	
Spectral Response Coefficient, S_{d1} =	0.135	
Seismic Design Category:	C	
Transverse Seismic Force Resisting System:	Steel System Not Specifically Detailed for Seismic Resistance	
Longitudinal Seismic Force Resisting System:	Steel System Not Specifically Detailed for Seismic Resistance	
Transverse Response Modification Coefficient, R_T =	3.00	
Longitudinal Response Modification Coefficient, R_L =	3.00	

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MATERIAL PROPERTIES

Component	Material	F_{tu} (ksi)	F_{ty} (ksi)	F_{cy} (ksi)	F_{su} (ksi)	E (ksi)	Reference
Steel Tubing	ASTM A500-B	58	46	46	58	29000	
Steel Plate	ASTM A36	58	36	36	58	29000	
Roof Decking	Aluminum 3004 H36	35	28	25	20	10100	ADM, Table 3.3-1
Alum. Extrusion (Gutter)	Aluminum 6105 T5	38	35	35	24	10100	ADM, Table 3.3-1
Alum. Extrusion (Ridge Cap)	Aluminum 6105 T5	38	35	35	24	10100	ADM, Table 3.3-1

FRAME ANALYSIS

The 3D space frame was analyzed using RISA 3D structural engineering software. Code checks were performed by the computer analysis to verify adequate member sizes. See attached sheets for RISA input and output.

WIND LOADS (Main Wind Force Resisting System)

DESIGN PARAMETERS				
Design Procedure:		Directional Procedure		ASCE 7-10, 27.4.1
Risk Category:		II		ASCE 7-10, Table 1.5-1
Importance Factor:	I_w =	1.00		ASCE 7-10, Table 1.5-2
Basic Wind Speed:	V =	116	mph	
Wind Directionality Factor:	K_d =	0.85		ASCE 7-10, Table 26.6-1
Exposure Category:		C		ASCE 7-10, 26.7.3
Topographic Factor:	K_{zt} =	1.00	(assumed)	ASCE 7-10, 26.8.2
Gust Effect Factor:	G =	0.85		ASCE 7-10, 26.9.1
Enclosure Classification:		Open		ASCE 7-10, 26.10
Internal Pressure Coefficient:	G C_{pi} =	0.00		ASCE 7-10, Table 26.11-1
Roof Type:		Pitched Free Roofs		
Roof Width:		18	ft	
Roof Length:		24	ft	
Roof Eave Height:	h_e =	8	ft	
Roof Angle:	θ =	18	degrees	
Roof Peak Height:	h_p =	11	ft	
Roof Mean Height:	h =	9.5	ft	
Velocity Pressure Exposure Coefficient:	K_h =	0.85		ASCE 7-10, Table 27.3-1
Ground Elevation Factor:	K_e =	1.00		(not applicable)
Velocity Pressure:	q_h =	24.888		ASCE 7-10, Eqn. 27.3-1 0.00256 Kh Kzt Kd Ke V^2 Iw
Wind Blockage:		Clear Wind Flow		(Less than 50% obstructed below roof)

Determine net pressure coefficent, using linear interpolation for values not shown in reference table.
For longitudinal wind, say horizontal distance from windward edge is less than *h* for entire roof area (conservative).

Net Pressure Coefficients, C_N, for MWFRS			
Wind Direction, γ	*Case A*	*Case B*	
Normal to Ridge, γ = 0°			ASCE 7-10, Fig. 27.4-5
Windward:	1.1	0.02	
Leeward:	-0.2	-0.98	
Normal to Ridge, γ = 180°			ASCE 7-10, Fig. 27.4-5
Windward:	1.1	0.02	
Leeward:	-0.2	-0.98	
Parallel to Ridge, γ = 90°	-0.8	0.8	ASCE 7-10, Fig. 27.4-7

Calculate design wind pressures using equation: p = qh G CN ASCE 7-10, Eqn. 27.4-3

Design Wind Pressures for MWFRS		
Wind Direction, γ	*Case A*	*Case B*
Normal to Ridge, γ = 0°		
Windward:	23.3	0.4
Leeward:	-4.2	-20.7
Normal to Ridge, γ = 180°		
Windward:	23.3	0.4
Leeward:	-4.2	-20.7
Parallel to Ridge, γ = 90°	-16.9	16.9

Pressures are shown in pounds per square foot. Positive pressure acts toward roof surface; negative pressure acts away from roof surface.

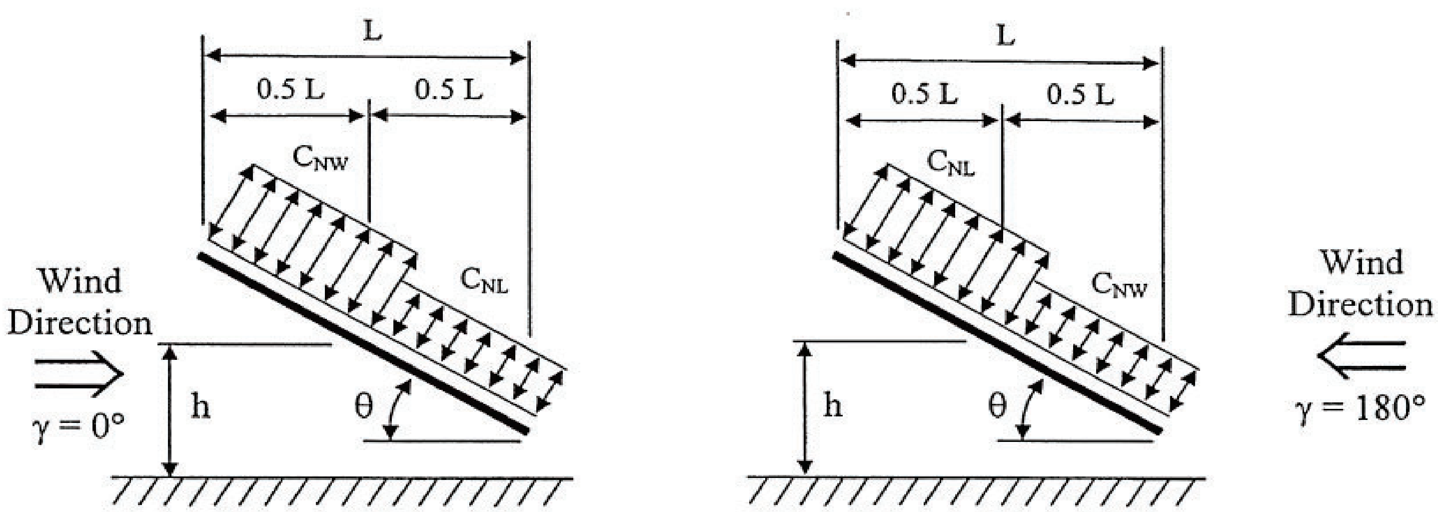
Say longitudinal (parallel to ridge) wind pressure on roof surface is covered by the calculated transverse (normal to ridge) wind pressures. Determine lateral wind load on posts by applying longitudinal wind pressure to the area of the structure projected on a plane normal to the wind direction and converting to a reaction which shall be applied to the top of each post. Assume each post takes equal force. For gable roofs, assume a filled gable.

Longitudinal Projected Area:	$A_{fL} \approx$	27	ft^2	
Total Number of Posts:	N =	4		
	$\boxed{R_x =}$	$\boxed{115}$	$\boxed{\text{\# per post}}$	16.9 psf × 27 ft^2 / 4

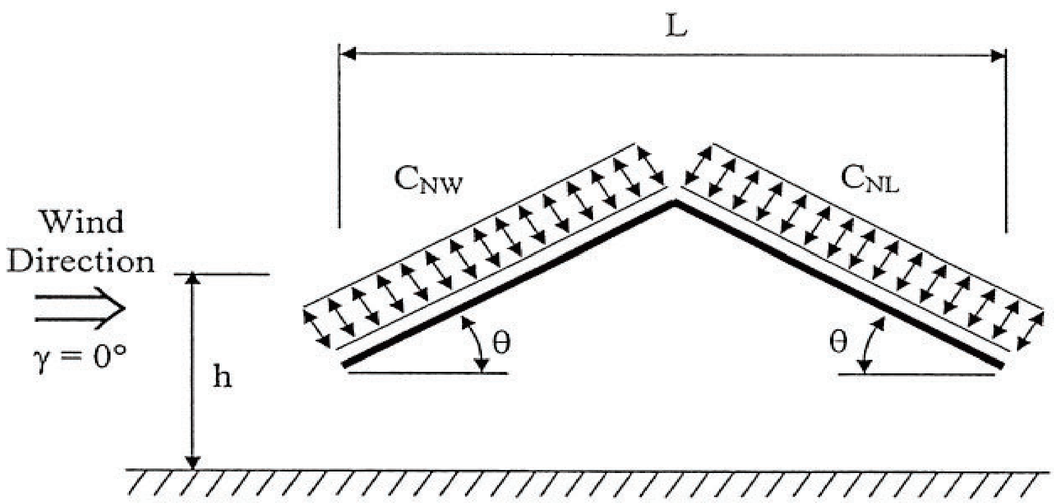
MINIMUM DESIGN WIND LOADS

The design wind force for open buildings shall not be less than the force p_{min} multiplied by the area A_f, which shall be the area of the structure projected on a plane normal to the wind direction. For gable roofs, assume a filled gable.

Minimum Wind Load:	p_{min} =	16	psf	ASCE 7-10, 27.4.7
Transverse Projected Area:	$A_{fT} \approx$	44	ft^2	



Monoslope Free Roofs



Pitched Free Roofs

WIND LOADS (Components & Cladding)

DESIGN PARAMETERS

Building Type:	Open Buildings			ASCE 7-10, 30.8	
Risk Category:		II		ASCE 7-10, Table 1.5-1	
Importance Factor:	I_w =	1.00		ASCE 7-10, Table 1.5-2	
Basic Wind Speed:	V =	116	mph		
Wind Directionality Factor:	K_d =	0.85		ASCE 7-10, Table 26.6-1	
Exposure Category:		C		ASCE 7-10, 26.7.3	
Topographic Factor:	K_{zt} =	1.00		ASCE 7-10, 26.8.2	
Gust Effect Factor:	G =	0.85		ASCE 7-10, 26.9.1	
Roof Type:		Pitched Free Roofs			
Max. Span of Roof Deck:	L_{pan} =	9	ft		
Roof Panel Width:	w_{pan} =	2	ft		
Roof Eave Height:	h_e =	8	ft		
Roof Angle:	θ =	18	degrees		
Roof Peak Height:	h_p =	9.5	ft		
Roof Mean Height:	h =	8.8	ft		
Velocity Pressure Exposure Coefficient:	K_z =	0.85		ASCE 7-10, Table 30.3-1	
Ground Elevation Factor:	K_e =	1.00		(not applicape)	
Velocity Pressure:	q_z =	24.888		ASCE 7-10, Eqn. 30.3-1	0.00256 Kz Kzt Kd Ke V^2 Iw
Wind Blockage:	Clear Wind Flow			(Less than 50% obstructed below roof)	
Effective Wind Area:	A_e =	216	ft^2	ASCE 7-10, 26.2	L_{pan} w_r
Actual Tributary Area:	A_t =	18	ft^2		L_{pan} w_{pan}
Design Wind Area:	A =	216	ft^2		greater of A_e and A_t
Width of Pressure Coefficient Zone:	a =	3	ft		greater of 0.1 L_{pan} and 3 ft
	a^2 =	9	ft^2		
	4 a^2 =	36	ft^2		

Determine net pressure coefficent, using linear interpolation for values not shown in reference table.

Net Pressure Coefficients, C_N, for C&C						
Design Wind Area	Zone 3		Zone 2		Zone 1	
	Case A	*Case B*	*Case A*	*Case B*	*Case A*	*Case B*
$A \le a^2$	2.28	-2.12	1.76	-1.64	1.14	-1.06
$a^2 < A \le 4\,a^2$	1.76	-1.64	1.76	-1.64	1.14	-1.06
$A > 4\,a^2$	1.14	-1.06	1.14	-1.06	1.14	-1.06

ASCE 7-10, Table 30.8-2

For simplicity, use Zone 3 pressures for all components (conservative).

Calculate design wind pressures using equation: p = qz G CN ASCE 7-10, Eqn. 30.8-1

Design Wind Pressures for C&C	
Zone 3	
Case A	*Case B*
24.1	-22.4

Pressures are shown in pounds per square foot. Positive pressure acts toward roof surface; negative pressure acts away from roof surface.

MINIMUM DESIGN WIND LOADS

The design wind pressure for components and cladding shall not be less than a net pressure p_{min} acting in either direction normal to the surface.

p_{min} = 16 psf ASCE 7-10, 30.2.2

SNOW LOADS

Risk Category:		II		ASCE 7-10, Table 1.5-1	
Importance Factor:	I_s =	1.00		ASCE 7-10, Table 1.5-2	
Ground Snow Load:	p_g =	15	psf		
Exposure Factor:	C_e =	1.00		ASCE 7-10, Table 7-2	
Thermal Factor:	C_t =	1.20		ASCE 7-10, Table 7-3	
Flat Roof Snow Load:	p_f =	12.6	psf	ASCE 7-10, Eqn. 7.3-1	$0.7\ C_e\ C_t\ I_s\ p_g$
Roof Angle:	θ =	18	degrees		
Slope Factor:	C_s =	1		ASCE 7-10, Fig. 7-2	
Sloped Roof Snow Load:	p_s =	12.6	psf	ASCE 7-10, Eqn. 7.4-1	$C_s\ p_f$
Minimum Snow Load for Low-Slope Roofs:	p_m =	15	psf	ASCE 7-10, 7.3.4	$p_g \leq 20 \rightarrow p_m = I_s\ p_g$ $p_g > 20 \rightarrow p_m = 20\ I_s$
Roof angle greater than or equal to 15 degrees, therefore minimum need not be applied.					
Design Use Snow Load:	p_u =	12.6	psf		

UNBALANCED ROOF SNOW LOADS FOR HIP AND GABLE ROOFS

Determine according to ASCE 7-10, section 7.6.1.

Roof Angle is greater than 1/2 on 12 slope and less than 7 on 12 slope, therefore unbalanced snow loads must be applied.

Case A, Leeward Side:	p_{AL} =	15	psf		
Case A, Windward Side:	p_{AW} =	0	psf		
Horizontal Eave to Ridge Distance:	W =	9	ft		For W < 20, use W = 20
Drift Height:	h_d =	1.11	ft	ASCE 7-10, Fig. 7-9	$0.43 \sqrt[3]{(W)} \sqrt[4]{(p_g + 10)} - 1.5$
Snow Density:	γ =	15.95	pcf	ASCE 7-10, Eqn. 7.7-1	$0.13\ p_g + 14 \leq 30$
Roof Slope (run for a rise of 1):	S =	3.1			1 ÷ tanθ
Case B, Leeward Side:	p_{BL} =	12.6	psf		
Case B, Windward Side:	p_{BW} =	3.78	psf		
Case B, Surcharge Magnitude:	p_{BS} =	10.06	psf	Leeward side, starting at the ridge	$h_d\ \gamma \div \sqrt{S}$
Case B, Surcharge Extent:	d_{BS} =	4.95	ft	Horizontal distance from ridge	$8\ h_d \sqrt{(S)} \div 3$

W ≤ 20 → Case A; W > 20 → Case B   Therefore, Case A governs.

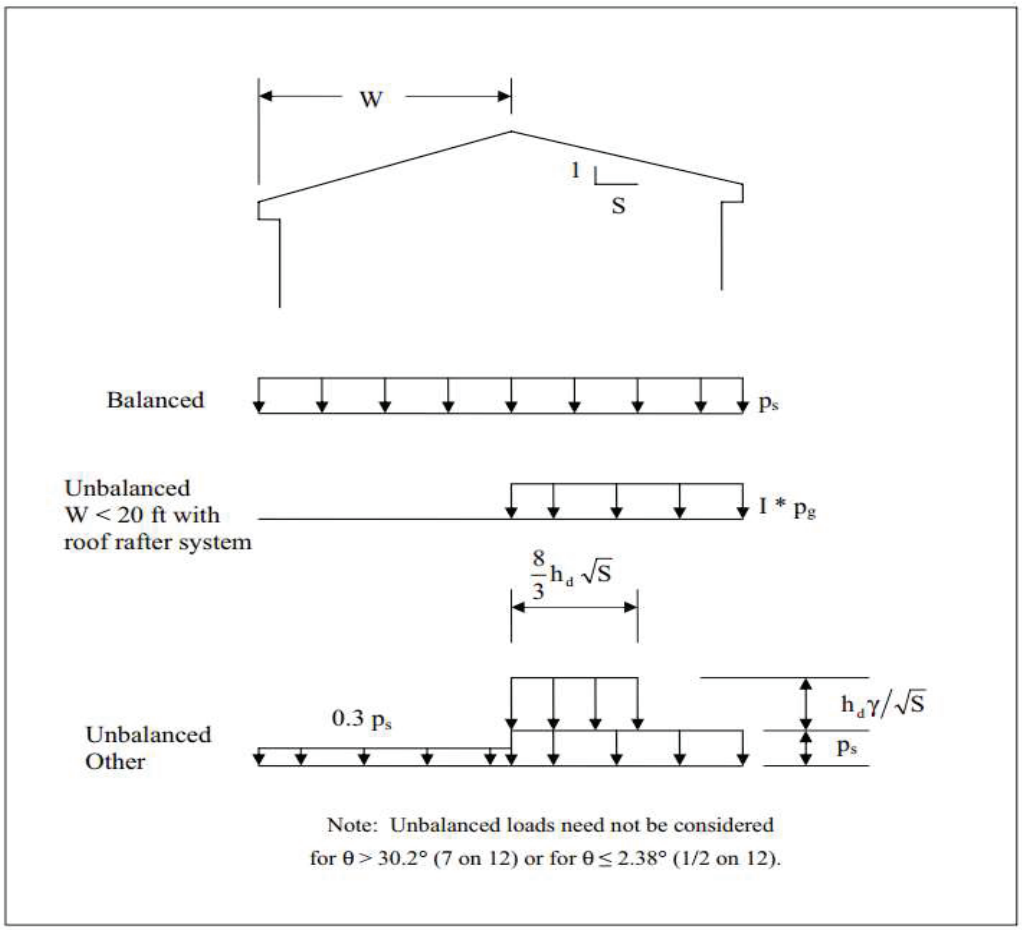


FIGURE 7-5 Balanced and Unbalanced Snow Loads for Hip and Gable Roofs.

SEISMIC LOADS

DESIGN PARAMETERS					
Basis of Design:		ASCE 7-10, Chapter 12 (Building Structures)			
Risk Category:		II		ASCE 7-10, Table 1.5-1	
Seismic Importance Factor:	I_e =	1.00		ASCE 7-10, Table 1.5-2	
Soil Site Class:		D	(assumed)		
Spectral Response Acceleration, 0.2 Second:	S_s =	0.178			
Spectral Response Acceleration, 1.0 Second:	S_1 =	0.085			
Short-Period Site Coefficient:	F_a =	1.6			
Long-Period Site Coefficient:	F_v =	2.4			
Short-Period MCE_R:	S_{MS} =	0.285			$F_a S_S$
1s-Period MCE_R:	S_{M1} =	0.204			$F_v S_1$
Spectral Response Coefficient:	S_{DS} =	0.19			$2/3\ S_{MS}$
Spectral Response Coefficient:	S_{D1} =	0.136			$2/3\ S_{M1}$
Seismic Design Category:		C			
Design Procedure:		Equivalent Lateral Force			
Design Dead Load:	D =	5	psf		
Structure Floor Area:	A =	432	ft^2		
Effective Seismic Weight:	W =	2783	#	ASCE 7-10, 12.7.2	
Total Number of Posts:	N =	4			

TRANSVERSE SEISMIC					
Transverse Seismic Force Resisting System:		Steel System Not Specifically Detailed for Seismic Resistance			
Transverse Response Modification Coefficient:	R_T =	3.00			
Seismic Response Coefficient:	C_s =	0.063		ASCE 7-10, Eqn. 12.8-2	$S_{DS} / (R_T / I_e)$
Minimum C_s, Case A:	C_s =	0.01		ASCE 7-10, Eqn. 12.8-5	$0.044\ S_{DS}\ I_e \geq 0.01$
Minimum C_s, Case B:	C_s =	0.014		ASCE 7-10, Eqn. 12.8-6	$0.5\ S_1 / (R_T / I_e)$
	$S_1 < 0.6$, therefore Case B does not apply.				
	C_s =	0.063			
Seismic Base Shear:	V =	175.33	#	ASCE 7-10, Eqn. 12.8-1	C_s W
Redundancy Factor:	p =	1		ASCE 7-10, 12.3.4	
Horizontal Seismic Load Effect:	E_h =	175.3	#	ASCE 7-10, Eqn. 12.4-3	p V
Vertical Seismic Load Effect:	E_v =	105.8	#	ASCE 7-10, Eqn. 12.4-4	$0.2\ S_{DS}$ W
Overstrength Required:	TRUE				
Overstrength Factor:	Ω_o =	3		ASCE 7-10, Table 12.2-1	
Seismic Overstrength Load:	E_{mh} =	526	#	ASCE 7-10, Eqn. 12.4-7	$V\ \Omega_o$
Seismic Load Effect:	E_z =	631.8	#	ASCE 7-10, Eqn. 12.4-1/5	$E = \max(E_h, E_{mh}) + E_v$
Lateral seismic force shall be resisted by the posts. Assume each post takes equal force.					
Lateral Seismic Force:	F_{pz} =	158	# per post		E_z / (Number of Posts)

LONGITUDINAL SEISMIC

Longitudinal Seismic Force Resisting System:	Steel System Not Specifically Detailed for Seismic Resistance				
Longitudinal Response Modification Coefficient:	R_L =	3.00			
Seismic Response Coefficient:	C_s =	0.063		ASCE 7-10, Eqn. 12.8-2	S_{DS} / (R_L / I_e)
Minimum C_s, Case A:	C_s =	0.01		ASCE 7-10, Eqn. 12.8-5	0.044 S_{DS} I_e ≥ 0.01
Minimum C_s, Case B:	C_s =	0.014		ASCE 7-10, Eqn. 12.8-6	0.5 S_1 / (R_L / I_e)
	S_1 < 0.6, therefore Case B does not apply.				
	C_s =	0.063			
Seismic Base Shear:	V =	175.33	#	ASCE 7-10, Eqn. 12.8-1	C_s W
Redundancy Factor:	p =	1		ASCE 7-10, 12.3.4	
Horizontal Seismic Load Effect:	E_h =	175.3	#	ASCE 7-10, Eqn. 12.4-3	p V
Vertical Seismic Load Effect:	E_v =	105.8	#	ASCE 7-10, Eqn. 12.4-4	0.2 S_{DS} W
Overstrength Required:	TRUE				
Overstrength Factor:	Ω_o =	3		ASCE 7-10, Table 12.2-1	
Seismic Overstrength Load:	E_{mh} =	526	#	ASCE 7-10, Eqn. 12.4-7	V Ω_o
Seismic Load Effect:	E_x =	631.8	#	ASCE 7-10, Eqn. 12.4-1/5	E = max(E_h, E_{mh}) + E_v
Lateral seismic force shall be resisted by the posts. Assume each post takes equal force.					
Lateral Seismic Force:	F_{px} =	158	# per post		E_x / (Number of Posts)

LOAD COMBINATIONS

Determine maximum "inward" and "outward" pressures on roof decking using Allowable Stress Design per ASCE 7, section 2.4.1. Wind pressure values shall be taken as determined in Components & Cladding. For those structures where snow drift loads are required, see Exception 1 in the referenced code. In combinations 4 and 6, Snow Load shall be taken as the design snow load p_u and shall exclude drift surcharge load. In combination 3, Snow Load shall be taken as the total snow load including drift surcharge.

Dead Load:	$D =$	5	psf	
Live Load:	$L =$	0	psf	
Roof Live Load:	$L_r =$	20	psf	
Design Use Snow Load:	$S_U =$	12.6	psf	
Snow Load + Drift Case A:	$S_A =$	#N/A	psf	
Snow Load + Drift Case B:	$S_B =$	#N/A	psf	
Rain Load:	$R =$	5	psf	
Wind Load Case A:	$W_A =$	24.1	psf	
Wind Load Case B:	$W_B =$	-22.4	psf	
Earthquake Load:	$E =$	0	psf	(By inspection, seismic does not control and is therefore excluded.)

Maximum Load Combination ("inward"):

$\Sigma_{max} =$ D + .75 L + .75 (.6 W) + .75 Lr

= 30.8 psf (BLC #6a)

Minimum Load Combination ("outward"):

$\Sigma_{min} =$ .6 D + .6 W

= -10.4 psf (BLC #7)

ROOF DECKING

PROPERTIES

Panel Type:	W-rib (see drawing)			
Width:	w_{pan} =	2	ft	
Thickness:	t_{pan} =	0.038	in	
Section Modulus (top):	S_{top} =	0.85	in^3	
Section Modulus (bottom):	S_{bot} =	1.1	in^3	
Material:	Aluminum 3004 H36			
Yield Strength:	F_{ty} =	28	ksi	
Allowable Strength:	F_a =	16.8	ksi	
Max. Single Span:	L_{pan} =	9	ft	

DESIGN PROCEDURE

Analyze the roof deck as a single span, simple beam with uniformly distributed load. Use a tributary area with width equal to the panel width and length equal to the maximum span of the roof deck.

INWARD BENDING

Pressure:	Σ_{max} =	30.845	psf		
Linear Loading:	W^+ =	61.7	plf	$\Sigma_{max}\, w_{pan}$	
Maximum Moment:	M^+ =	624.7	ft-lb	$1/8\ W^+ L^2_{pan}$	
Required Strength:	f_b^+ =	8.9	ksi	$(12\ M^+) \div (1000\ S_{min})$	
Check:	PASS	53.0%	capacity		8.9 ksi < 16.8 ksi

OUTWARD BENDING

Pressure:	Σ_{min} =	10.44	psf		
Linear Loading:	W^- =	20.9	plf	$\Sigma_{min}\, w_{pan}$	
Maximum Moment:	M^- =	211.6	ft-lb	$1/8\ W^- L^2_{pan}$	
Required Strength:	f_b^- =	3	ksi	$(12\ M^-) \div (1000\ S_{min})$	
Check:	PASS	17.9%	capacity		3 ksi < 16.8 ksi

ATTACHMENT OF DECK TO GUTTER

FASTENER DATA

Fastener data is from ICC ESR-2196. Strengths are tested allowable capacities. Referenced tensile pull-out load is from 0.036 thick steel member. See below for calculated values used in design.

Fastener:	HILTI S-MD 8-18 X 1/2" or 3/4" HWH #2 Point Self Drilling				
Nominal Diameter:	$D =$	0.164	in		
Nominal Head Diameter:	$D_{ws} =$	0.335	in		
Number of Threads:	$n =$	18	threads per inch		
	$2/n =$	0.111			
Tensile Pull-Out Load:	$P_{not}/\Omega =$	75	# (ref.)		
Screw Tensile Strength:	$P_{nt}/\Omega =$	335	#		
Screw Shear Strength:	$P_{ns}/\Omega =$	390	#		
Applied Factor of Safety:	$\Omega = n_s =$	3.0			
Design Reference:	Aluminum Design Manual 2015 (ADM), Section J.5 Tapping Screw Connections				
MEMBER IN CONTACT WITH THE SCREW HEAD					
Component:	Roof Decking				
Tension Ultimate:	$F_{tu1} =$	35	ksi		
Tension Yield:	$F_{ty1} =$	28	ksi		
Member Thickness:	$t_1 =$	0.038	in		
Location Coefficient:	$C =$	1.0			
MEMBER NOT IN CONTACT WITH THE SCREW HEAD					
Component:	Alum. Extrusion (Gutter)				
Tension Ultimate:	$F_{tu2} =$	38	ksi		
Tension Yield:	$F_{ty2} =$	35	ksi		
Member Thickness:	$t_2 = t_c =$	0.062	in		
J.5.4.1 SCREW PULL-OUT	Use part 2, for spaced threads				
tc < 2/n, therefore use sub-part a.					
Thickness Coefficient:	$K_s =$	1.01	for 0.038 in. ≤ tc < 0.080 in.		
Governs → sub-part a. :	$P_{not} =$	0.359	kips	ADM, Eqn. J.5-1	$K_s D t_c F_{ty2}$
sub-part b. :	$P_{not} =$	0.108	kips	ADM, Eqn. J.5-5	$1.2 D F_{ty2} (4/n - t_c) + 3.26 D F_{tu2} (t_c - 2/n)$
sub-part c. :	$P_{not} =$	0.63	kips	ADM, Eqn. J.5-6	$1.63 D t_c F_{tu2}$
J.5.4.2 SCREW PULL-OVER					
Nominal Pull-Over Strength:	$P_{nov} =$	0.227	kips	ADM, Eqn. J.5-8	C t1 Ftu1 (Dws - D)
J.5.4 SCREW TENSION					
The allowable tension force on a screw is the least of:					
1) P_{not}/n_s		0.12	kips		
2) P_{nov}/n_s		0.076	kips	←Controls	
3) $P_{nt}/(1.25\ n_s)$		0.268	kips		
Therefore, allowable screw tension =		76	#		

DESIGN PROCEDURE

Analyze the attachment by using a tributary area with width equal to the roof deck panel width and length equal to half of the maximum span of the roof deck. See section "Roof Decking" for values. Determine minimum screw quantity per roof deck panel attachment to resist maximum uplift.

Uplift Pressure:	$p_{up} =$	94	#/panel	$w_{pan}\ 0.5\ L_{pan}\ \lvert\Sigma_{min}\rvert$
Required Screw Quantity:		2	screw(s) per panel width	2 × 76# = 152# > 94#

ATTACHMENT OF DECK TO RIDGE CAP

FASTENER DATA

Fastener data is from ICC ESR-2196. Strengths are tested allowable capacities. Referenced tensile pull-out load is from 0.036 thick steel member. See below for calculated values used in design.

Fastener:	HILTI S-MD 8-18 X 1/2" or 3/4" HWH #2 Point Self Drilling				
Nominal Diameter:	$D =$	0.164	in		
Nominal Head Diameter:	$D_{ws} =$	0.335	in		
Number of Threads:	$n =$	18	threads per inch		
	$2/n =$	0.111			
Tensile Pull-Out Load:	$P_{not}/\Omega =$	75	# (ref.)		
Screw Tensile Strength:	$P_{nt}/\Omega =$	335	#		
Screw Shear Strength:	$P_{ns}/\Omega =$	390	#		
Applied Factor of Safety:	$\Omega = n_s =$	3.0			
Design Reference:	Aluminum Design Manual 2015 (ADM), Section J.5 Tapping Screw Connections				
MEMBER IN CONTACT WITH THE SCREW HEAD					
Component:	Roof Decking				
Tension Ultimate:	$F_{tu1} =$	35	ksi		
Tension Yield:	$F_{ty1} =$	28	ksi		
Member Thickness:	$t_1 =$	0.038	in		
Location Coefficient:	$C =$	1.0			
MEMBER NOT IN CONTACT WITH THE SCREW HEAD					
Component:	Alum. Extrusion (Ridge Cap)				
Tension Ultimate:	$F_{tu2} =$	38	ksi		
Tension Yield:	$F_{ty2} =$	35	ksi		
Member Thickness:	$t_2 = t_c =$	0.094	in		
J.5.4.1 SCREW PULL-OUT	Use part 2, for spaced threads				
tc < 2/n, therefore use sub-part a.					
Thickness Coefficient:	$K_s =$	1.20	for 0.080 in. ≤ tc < 2/n		
Governs → sub-part a. :	$P_{not} =$	0.647	kips	ADM, Eqn. J.5-1	$K_s\ D\ t_c\ F_{ty2}$
sub-part b. :	$P_{not} =$	0.538	kips	ADM, Eqn. J.5-5	$1.2\ D\ F_{ty2}\ (4/n - t_c) + 3.26\ D\ F_{tu2}\ (t_c - 2/n)$
sub-part c. :	$P_{not} =$	0.955	kips	ADM, Eqn. J.5-6	$1.63\ D\ t_c\ F_{tu2}$
J.5.4.2 SCREW PULL-OVER					
Nominal Pull-Over Strength:	$P_{nov} =$	0.227	kips	ADM, Eqn. J.5-8	C t1 Ftu1 (Dws - D)
J.5.4 SCREW TENSION					
The allowable tension force on a screw is the least of:					
1)	P_{not}/n_s	0.216	kips		
2)	P_{nov}/n_s	0.076	kips	←Controls	
3)	$P_{nt}/(1.25\ n_s)$	0.268	kips		
Therefore, allowable screw tension =		76	#		

DESIGN PROCEDURE

Analyze the attachment by using a tributary area with width equal to the roof deck panel width and length equal to half of the maximum span of the roof deck. See section "Roof Decking" for values. Determine minimum screw quantity per roof deck panel attachment to resist maximum uplift.

Uplift Pressure:	$p_{up} =$	94	#/panel	$w_{pan}\ 0.5\ L_{pan}\ \lvert\Sigma_{min}\rvert$
Required Screw Quantity:		2	screw(s) per panel width	2 × 76# = 152# > 94#

ATTACHMENT OF DECK TO STEEL BEAM

FASTENER DATA

Fastener data is from ICC ESR-3528. Strengths are tested allowable capacities. Referenced tensile pull-out load is from 0.188 thick steel member (F_u = 58 ksi). See below for calculated values used in design.

Fastener:	StrongPoint H5 12-24 X 1-1/4" or 2" HWH #5 Point Self Drilling				
Nominal Diameter:	D =	0.216	in		
Nominal Head Diameter:	D_{ws} =	0.415	in		
Number of Threads:	n =	24	threads per inch		
	$2/n$ =	0.083			
Tensile Pull-Out Load:	P_{not}/Ω =	1265	# (ref.)		
Screw Tensile Strength:	P_{nt}/Ω =	1385	#		
Screw Shear Strength:	P_{ns}/Ω =	849	#		
Applied Factor of Safety:	$\Omega = n_s$ =	3.0			
Design Reference 1:	Aluminum Design Manual 2015 (ADM), Section J.5 Tapping Screw Connections				
Design Reference 2:	AISI S100-12, Section E4 Screw Connections				
MEMBER IN CONTACT WITH THE SCREW HEAD					
Component:	Roof Decking				
Tension Ultimate:	F_{tu1} =	35	ksi		
Tension Yield:	F_{ty1} =	28	ksi		
Member Thickness:	t_1 =	0.038	in		
Location Coefficient:	C =	1.0			
MEMBER NOT IN CONTACT WITH THE SCREW HEAD					
Component:	Steel Tubing				
Tension Ultimate:	F_{tu2} =	58	ksi		
Tension Yield:	F_{ty2} =	46	ksi		
Member Thickness:	$t_2 = t_c$ =	0.125	in (engineer's discretion: use as minimum for calculation)		
AISI E4.4.1 SCREW PULL-OUT					
Nominal Pull-Out Strength:	P_{not} =	1.331	kips	AISI, Eqn. E4.4.1-1	$0.85\ t_c\ D\ F_{tu2}$
ADM J.5.4.2 SCREW PULL-OVER					
Nominal Pull-Over Strength:	P_{nov} =	0.265	kips	ADM, Eqn. J.5-8	C t1 Ftu1 (Dws - D)
SCREW TENSION					
The allowable tension force on a screw is the least of:					
1)	P_{not}/n_s	0.444	kips		
2)	P_{nov}/n_s	0.088	kips	←Controls	
3)	$P_{nt}/(1.25\ n_s)$	1.108	kips		
Therefore, allowable screw tension =		88	#		

DESIGN PROCEDURE

Analyze the attachment by using a tributary area with width equal to the roof deck panel width and length equal to half of the maximum span of the roof deck. See section ''Roof Decking'' for values. Determine minimum screw quantity per roof deck panel attachment to resist maximum uplift.

Uplift Pressure:	p_{up} =	94	#/panel	0.5 wpan Lpan |Σmin|
Required Screw Quantity:		2	screw(s) per panel width	2 × 88# = 176# > 94#

POST TO RAFTER CONNECTION

CONNECTION FORCES					
Max. Moment at Connection:	M_{max} =	2.143	k-ft	(from RISA results)	
Max. Shear at Connection:	V_{max} =	0.483	kip	(from RISA results)	
Max. Axial Tension:	A_{Tmax} =	4.438	kip	(from RISA results)	
Moment Used in Calculation:	M_u =	2.17	k-ft	(engineer's discretion)	
	M_u =	26040	lb-in		
Shear Used in Calculation:	V_u =	490	#	(engineer's discretion)	
Tension Used in Calculation:	A_{Tu} =	4485	#	(engineer's discretion)	
CONNECTION GEOMETRY					
Post Depth:	d_{tube} =	6	in		
Post Width:	b_{post} =	6	in		
Post Thickness:	t_{tube} =	0.1875	in		
Plate Depth:	d_p =	5.25	in		
Bolt Vertical Spacing:	C_s =	3.5	in		
Bolt Horizontal Spacing:	R_s =	3.5	in		
Distance to Row 1:	d_1 =	1.1875	in		
Distance to Row 2:	d_2 =	4.6875	in		
Max. Distance to Edge of Plate:	d_3 =	1.0625	in		
CONNECTION BOLTS					
Use ASTM F3125 grade A325 structural bolts.					
Nominal Bolt Diameter:	D_b =	0.625	in		
Nominal Bolt Area:	A_b =	0.307	in^2		
Nominal Tensile Strength:	F_{nt} =	90	ksi		
Allowable Tensile Stress:	T_{ab} =	13.815	kip		$0.5\ F_{nt}\ A_b$
Nominal Shear Strength:	F_{nv} =	54	ksi		
Allowable Shear Stress:	V_{ab} =	8.289	kip		$0.5\ F_{nv}\ A_b$
Max. Bolt Tension:	T_{Bmax} =	3.899	kip		$[M_u \div (2\ \mathit{bolts})\ (d_2)] + [A_{Tu} \div (4\ \mathit{bolts})]$
Check Tension:	PASS	28.2%	capacity		
Max. Bolt Shear:	V_{Bmax} =	0.123	kip		$V_u \div (4\ \mathit{bolts})$
Check Shear:	PASS	1.5%	capacity		
PLATE THICKNESS					
Plate Yield Strength:	F_y =	36000	psi		
Deteremine required thickness from nominal moment using equation $M_n = F_y Z$ where $Z = b\ t^2 \div 4$ and $b = 0.5\ b_{post}$					
Nominal Moment:	M_n =	4143	lb-in		$T_{Bmax}\ d_3$
Safety Factor:	Ω_p =	1.67			
Required Thickness:	t_{min} =	0.506	in		$t^2_{min} = (4\ M_n\ \Omega_p) \div (F_y\ b)$
Use 5/8'' thick plate.					

STEEL TUBE

Check post side walls for flexural buckling due to concentrated compression load transferred from connection plate.

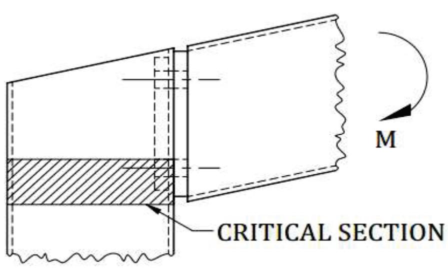
Assumptions made:

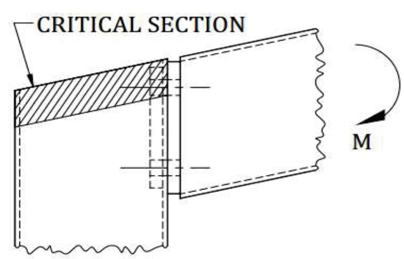
1) Connection will rotate about bottom edge of plate.
2) All compression will act at bottom edge of plate.
3) Compressive force will equal the moment divided by the plate depth.
4) Total compression load will be equally distributed to both side walls.
5) A 2'' wide strip of side wall (''critical section'') will resist compression load.

Compressive Force:	F_C =	4960	#	$M_u \div d_p$
Force Per Side Wall:	F_{Cs} =	2480	#/wall	$F_C \div 2$
Critical Section Area:	$A_{crit(1)}$ =	0.375	in^2	2 ttube
Load on Critical Section:	f_a =	6.61	ksi	$F_{Cs} \div 1000\ A_{crit(1)}$
Effective Length: Factor:	K =	0.65		(critical section is fixed each end)
Radius of Gyration:	r_{tube} =	0.054	in	$t_{tube} \div \sqrt{12}$
Slenderness Ratio:	λ_{tube} =	72.2		$K\ d_{tube} \div r_{tube}$
Control Determination:	$4.71 \sqrt{[E \div F_y]}$ =	118.3		≥ 72.2 therefore inelastic buckling controls.
Elastic Buckling Stress:	F_e =	54.9	ksi	$(\pi^2 E) \div \lambda^2$
Safety Factor:	Ω_c =	1.67		
Critical Stress:	F_{cr} =	32.39	ksi	Inelastic $\rightarrow (0.658^{[Fy/Fe]})\ F_y$ Elastic $\rightarrow 0.877\ F_e$
Allowable Stress:	F_a =	19.4	ksi	$F_{cr} \div \Omega_c$
Check:	PASS	34.1%	capacity	19.4 ksi > 6.61 ksi

Check post side walls for critical section under tension, similar to critical section under compression. Assume a 2.9375'' strip of side wall will resist tension load transferred from backing plate.

Critical Section Area:	$A_{crit(2)}$ =	0.551	in^2	2.9375 ttube
Tensile Force:	f_b	3.899	kip	T_{Bmax}
Allowable Tension:	F_b =	15.18	kip	$(F_y\ A_{crit(2)}) \div 1.67$
Check:	PASS	25.7%	capacity	15.18 kip > 3.899 kip





WELDS

Weld Width:	b_w =	5.625	in	
Weld Depth:	d_w =	5.75	in	
Weld Area:	A_w =	22.75	in	
Weld Section:	S_w =	43.36	in^2	$b_w\ d_w + (d_w^2 \div 3)$
Welding Electrode:	F_{EXX} =	70	ksi	
Trial Weld Size:	t_w =	3/16	in	
Safety Factor:	Ω_w =	2		
Allowable Weld Stress:	F_w =	2.78	k/in	$(0.6\ F_{EXX}\ t_w\ 0.707) \div \Omega_w$
Max. Stress on Weld:	f_w =	0.8	k/in	$f_w^2 = [(M_u \div S_w) + (A_{Tu} \div A_w)]^2 + [V_u \div A_w]^2$
Check:	PASS	28.8%	capacity	

RAFTER TO TIE BLOCK CONNECTION

CONNECTION BOLTS

Use ASTM F3125 grade A325 structural bolts.

Nominal Bolt Diameter:	D_b =	0.625	in		
Nominal Bolt Area:	A_b =	0.307	in^2		
Nominal Tensile Strength:	F_{nt} =	90	ksi		
Allowable Tensile Stress:	T_{ab} =	13.815	kip		$0.5\ F_{nt}\ A_b$
Nominal Shear Strength:	F_{nv} =	54	ksi		
Allowable Shear Stress:	V_{ab} =	8.289	kip		$0.5\ F_{nv}\ A_b$

FORCE ON BOLTS

Max. Moment at Connection:	M_{max} =	1.104	k-ft	(from RISA results)	
Moment Used in Calculation:	M_u =	1.12	k-ft	(engineer's discretion)	
	M_u =	13440	lb-in		
Max. Shear at Connection:	V_{max} =	0.918	kip	(from RISA results)	
Shear Used in Calculation:	V_u =	0.93	kip	(engineer's discretion)	
Max. Axial Stress in Tube:	A_{max} =	4.196	kip	(from RISA results)	
Stress Used in Calculation:	A_u =	4.24	kip	(engineer's discretion)	
Bolt Hole Diameter:	d_h =	0.8125	in		
Distance to Row 1:	d_1 =	1.125	in		
Distance to Row 2:	d_2 =	4.625	in		
Tension per Bolt:	T_b =	1453	#/bolt		$[M_u \div (2\text{ bolts})\ (d_2)] + [A_u \div (4\text{ bolts})]$
Check:	PASS	10.5%	capacity		1.453 kip < 13.815 kip
Shear per Bolt:	V_b =	0.233	kip/bolt		$V_u \div (4\text{ bolts})$
Check:	PASS	2.8%	capacity		0.233 kip < 8.289 kip

PLATE THICKNESS

Yield Strength:	F_y =	36000	psi		
Distance to Side of Tube:	d_3 =	1.25	in		
Assumed Resisting Width:	b =	5.75	in	(use total plate length)	
Max. Moment in Plate:	M_p =	3632.5	lb-in		$2\ T_b\ d_3$

Deteremine required thickness from nominal moment using equation $M_n = F_y Z$ where $Z = b\ t^2 \div 4$

Required Plastic Modulus:	Zreq =	0.169	in^3		$M_p\ \Omega_p \div F_y$
Safety Factor:	Ω_p =	1.67			
Required Thickness:	t_{min} =	0.343	in		$t^2_{min} = (4\ Z_{req}) \div b$

Use 3/8'' thickness min.

WELDS

Weld Width:	b_w =	5.625	in		
Weld Depth:	d_w =	5.75	in		
Weld Section:	S_w =	43.36	in^2		$b_w\ d_w + (d_w^2 \div 3)$
Welding Electrode:	F_{EXX} =	70	ksi		
Trial Weld Size:	t_w =	3/16	in		
Safety Factor:	Ω_w =	2			
Allowable Weld Stress:	R_{aw} =	2.78	k/in		$(0.6\ F_{EXX}\ t_w\ 0.707) \div \Omega_w$
Allowable Weld Moment:	M_{aw} =	120.54	k-in		$R_{aw}\ S_w$
Check:	PASS	11.1%	capacity		120.54 k-in > 13.44 k-in

RAFTER ACCESS HOLES

Check rafter section at 4" diameter access holes.

Member Depth:	d =	6	in	
Member Width:	b =	6	in	
Member Thickness:	t =	0.1875	in	
Section Area at Hole:	A =	3.609	in^2	
Section Modulus at Hole:	Z =	6.077	in^3	
Max Stress at Hole:	f_a =	4.421	ksi	$(M_u \div Z) + (A_{Tu} \div A)$
Safety Factor:	Ω =	1.67		
Allowable Stress:	F_a =	27.54	ksi	$F_y \div \Omega$
Check:	PASS	16.1%	capacity	

Check localized failure in compression element. Say compression element is 1" x 1" single L section.

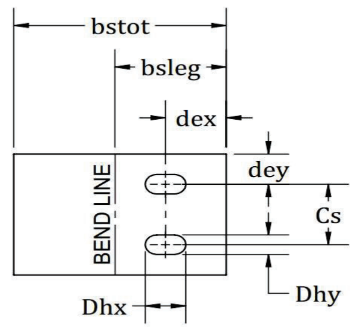
Distance to Extreme Fiber:	y =	0.682	in	
Element Moment of Inertia:	I =	0.03	in^4	
Element Area:	A_c =	0.34	in^2	
Element Radius of Gyration:	r =	0.297	in	
Effective Length Factor:	k =	0.65		
Element Length:	L =	4	in	
Slenderness Ratio:	λ =	8.754		
Control Determination:	$4.71\sqrt{[E \div F_y]}$ =	118.26		≥ 8.754 therefore inelastic buckling controls.
Elastic Buckling Stress:	F_e =	3734.9	ksi	AISC 360-10, Eqn. E3-4
Critical Stress:	F_{cr} =	45.76	ksi	
Allowable Stress:	F_b =	27.4	ksi	
Check:	PASS	16.1%	capacity	

Therefore hole in rafter is **OK**.

EDGE BEAM CONNECTION

GEOMETRY AND PROPERTIES

Tube Width:	b_t =	3	in
Tube Height:	h_t =	6	in
Tube Thickness:	t_t =	0.1875	in
Tube Ultimate Strength:	F_{ut} =	58	ksi
Tube Yield Strength:	F_{yt} =	46	ksi
Cap Plate Width:	b_c =	2.625	in
Cap Plate Height:	h_c =	5.625	in
Stem Plate Total Width:	b_{stot} =	5.25	in
Stem Plate Bent Leg Width:	b_{sleg} =	2.75	in
Stem Plate Height:	h_s =	3.5	in
Number of Bolts:	N_b =	2	
Bolt Spacing:	C_s =	1.75	in
Min. Edge Dist. X-axis:	d_{ex} =	1.5	in
Min. Edge Dist. Y-axis:	d_{ey} =	0.875	in
Hole Width:	D_{hx} =	1	in
Hole Height:	D_{hy} =	0.5625	in
Roof Slope Angle:	θ_r =	18	degrees
Plate Bend Angle:	θ_p =	45	degrees
Plate Ultimate Strength:	F_{up} =	58	ksi
Plate Yield Strength:	F_{yp} =	36	ksi



CHECK GOEMETRY RESTRICTIONS

Minimum Bolt Spacing:	PASS	AISC 360, J3.3
Minimum Edge Distance:	PASS	AISC 360, J3.4
Maximum Bolt Spacing:	PASS	AISC 360, J3.5a
Maximum Edge Distance:	PASS	AISC 360, J3.5

CONNECTION LOADS

Tension:	T =	2.915	kip
Compression:	C =	0.094	kip
Shear:	V =	1.317	kip
Max. Bolt Tension:	T_b =	0.664	kip
Max. Bolt Shear:	V_b =	1.331	kip

CHECK CONNECTION BOLTS

Use ASTM F3125 grade A325 structural bolts.

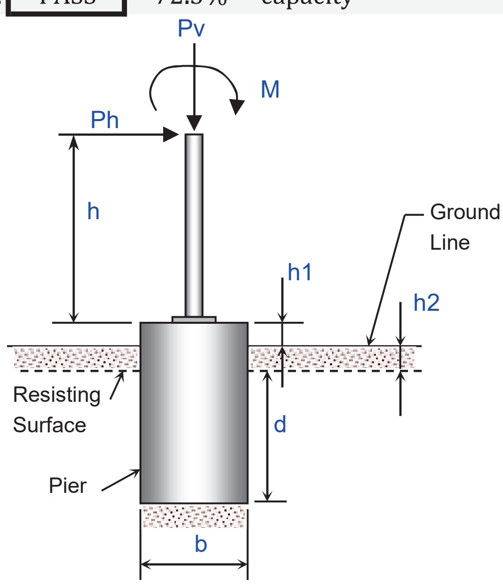
Nominal Tensile Strength:	F_{nt} =	90	ksi	AISC 360, Table J3.2	
Nominal Shear Strength:	F_{nv} =	54	ksi		
Safety Factor:	Ω_b =	2		AISC 360, J3.6	
Bolt Diameter:	d_b =	0.5	in		
Bolt Area:	A_b =	0.196	in^2		
Allowable Tensile Strength:	R_{nt} =	8.82	kip	AISC 360, Eqn. J3-1	$F_{nt} A_b / \Omega_b$
Allowable Shear Strength:	R_{nv} =	5.292	kip		$F_{nv} A_b / \Omega_b$
Check:	**PASS**	25.2%	capacity		

Check combined tension and shear per AISC 360 section J3.7					
Required Shear Strength:	f_{rv} =	6.8	ksi		V_b / A_b
Modified Nominal Tension:	F'_{nt} =	90	ksi	AISC 360, Eqn. J3-3b	$1.3\ F_{nt} - (\Omega_b\ F_{nt} / F_{nv})\ f_{rv} \le F_{nt}$
Allowable Tensile Strength:	R'_{nt} =	8.82	kip		$F'_{nt}\ A_b / \Omega_b$
Required Tensile Strength:	f_{rt} =	3.4	ksi		T_b / A_b
Modified Nominal Shear:	F'_{nv} =	54	ksi	AISC 360, Eqn. J3-3b	$1.3\ F_{nv} - (\Omega_b\ F_{nv} / F_{nt})\ f_{rt} \le F_{nv}$
Allowable Shear Strength:	R'_{nv} =	5.29	kip		$F'_{nv}\ A_b / \Omega_b$
Check:	**PASS**	25.2%	capacity		
STEM PLATE AXIAL YIELD					
Safety Factor:	Ω_1 =	1.67			
Required Yield Strength:	R_{n1} =	4.86805	kip		$T\ \Omega_1$
Required Plate Thickness:	t_1 =	0.039	in	AISC 360, Eqn. J4-1	$R_{n1} / F_{yp}\ h_s$
STEM PLATE TENSION RUPTURE					
Safety Factor:	Ω_2 =	2			
Required Rupture Strength:	R_{n2} =	5.83	kip		$T\ \Omega_2$
Required Effective Area:	A_e =	0.101	in^2	AISC 360, Eqn. J4-2	R_{n2} / F_{up}
Reqd. Thickness, Gross Area:	t_{2g} =	0.034	in		$A_e / 0.85\ h_s$
Reqd. Thickness, Net Area:	t_{2n} =	0.043	in		$A_e / (h_s - N_b\ D_{hy})$
STEM PLATE SHEAR YIELD					
Safety Factor:	Ω_3 =	1.5			
Required Yield Strength:	R_{n3} =	1.9755	kip		$V\ \Omega_3$
Required Plate Thickness:	t_3 =	0.034	in	AISC 360, Eqn. J4-3	$R_{n3} / 0.6\ F_{yp}\ b_{sleg}$
STEM PLATE SHEAR RUPTURE					
Safety Factor:	Ω_4 =	2			
Required Rupture Strength:	R_{n4} =	2.634	kip		$V\ \Omega_4$
Required Plate Thickness:	t_4 =	0.044	in	AISC 360, Eqn. J4-4	$R_{n4} / 0.6\ F_{up}\ (b_{sleg} - N_b\ D_{hx})$
STEM PLATE BLOCK SHEAR					
Safety Factor:	Ω_5 =	2			
Required Shear Strength:	R_{n5} =	2.662	kip		$V_b\ \Omega_5$
Uniform Stress Factor:	U_{bs} =	1			
Reqd. Thickness, Gross Area:	t_{5g} =	0.039	in	AISC 360, Eqn. J4-5	$R_{n5} / (0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt})$
Reqd. Thickness, Net Area:	t_{5n} =	0.04	in	AISC 360, Eqn. J4-5	$R_{n5} / (0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt})$
STEM PLATE BENDING DUE TO PRYING					
Plate End Reaction:	R_{n6} =	0.974	kip		
Moment at Plate End:	M_6 =	1.461	k-in		$R_{n6}\ d_{ex}$
Safety Factor:	Ω_6 =	1.67			
Required Section:	Z_{req} =	0.068	in^3		$M_6\ \Omega_6 / F_{yp}$
Section Width:	b =	2.375	in		
Required Thickness:	t_6 =	0.339	in		$\sqrt{(4\ Z_{req} / b)}$
Stem Plate Critical Limit State:	Bending Due to Prying				Therefore use 3/8'' thick plate min.
CAP PLATE SHEAR YIELD					
Safety Factor:	Ω_7 =	1.5			
Required Yield Strength:	R_{n7} =	1.9755	kip		$V\ \Omega_7$
Required Plate Thickness:	t_7 =	0.035	in	AISC 360, Eqn. J4-3	$R_{n7} / 0.6\ F_{yp}\ b_c$

CAP PLATE SHEAR RUPTURE					
Safety Factor:	Ω_8 =	2			
Required Rupture Strength:	R_{n8} =	2.634	kip		$V \Omega_8$
Required Plate Thickness:	t_8 =	0.029	in	AISC 360, Eqn. J4-4	$R_{n8} / 0.6 F_{up} b_c$
CAP PLATE MOMENT					
Cap Plate Horizontal Span:	b_h =	2.625	in		
Moment at Cap Plate:	M_9 =	1.913	k-in		$T b_h / 4$
Required Section:	Z_{req} =	0.089	in^3		$M_9 \Omega_6 / F_{yp}$
Cap Plate Vertical Span:	b_v =	5.625	in		
Required Thickness:	t_9 =	0.252	in		$\sqrt{(4 Z_{req} / b_v)}$
Cap Plate Critical Limit State:		Moment			Therefore use 3/8'' thick plate min.
LOCAL YIELDING OF HSS WALL					
Assume HSS tube is not fully engaged (conservative). Therefore, two walls contribute to available strength.					
Nominal Strength:	R_{nt} =	38.813	kip	AISC 360, Eqn. K1-14a	$2 t_t F_{yt} (5 t_c + t_s)$
Safety Factor:	Ω_t =	1.5			
Allowable Strength:	R_{at} =	25.875	kip		R_{nt} / Ω_t
Check:	PASS	11.3%	capacity		
WELD - STEM PLATE TO CAP PLATE					
Nominal Electrode Strength:	F_{EXX} =	70	ksi		
Trial Weld Size:	w_t =	3/16	in		
Effective Weld Throat:	t_w =	0.133	in	AISC 360, J2.2a	
Effective Weld Length:	l_{we} =	7	in		$2 h_s$
Effective Area of Weld:	A_{we} =	0.931	in^2		$t_w l_{we}$
Nominal Weld Stress:	F_{nw} =	42	ksi	AISC 360, Table J2.5	$0.6 F_{EXX}$
Safety Factor:	Ω_w =	2			
Allowable Weld Strength:	R_{aw} =	19.551	kip	AISC 360, Eqn. J2-3	$F_{nw} A_{we} / \Omega_w$
Check Tension:	PASS	14.9%	capacity		
Check Shear:	PASS	6.7%	capacity		
WELD - CAP PLATE TO HSS					
Weld Area:	A_w =	16.5	in		$2 (b_c + h_c)$
Weld Section X-Axis:	S_{wx} =	25.313	in3		$b_c h_c + h_c^2 / 3$
Weld Section Y-Axis:	S_{wy} =	17.063	in3		$h_c b_c + b_c^2 / 3$
Weld Torsional Constant:	J_w =	93.586	in3		$(b_c + h_c)^3 / 6$
Moment Vertical Component:	M_y =	0	k-in		
Moment Horiz. Component:	M_x =	2.577	k-in		
Shear Vertical Component:	V_y =	1.253	kip		
Shear Horizontal Component:	V_x =	0.407	kip		
Max. Pressure:	P_{max} =	2.915	kip		
Torque:	T_n =	1.329	k-in		
Shear in X-Axis:	f_{vx} =	0.065	k/in		
Shear in Y-Axis:	f_{vy} =	0.095	k/in		
Axial Force:	f_a =	0.278	k/in		
Resultant Force:	f_{rw} =	0.301	k/in		$\sqrt{(f_{vx}^2 + f_{vy}^2 + f_a^2)}$
Trial Weld Size:	w_t =	3/16	in		
Allowable Force:	F_{aw} =	2.784	k/in		$(0.6 F_{EXX} w_t 0.707) / \Omega_w$
Check:	PASS	10.8%	capacity		

POLE FOUNDATION ANALYSIS

PIER LOADING			(see RISA output)		
Axial Load:	P_v =	3.416	kip		
Uplift:	P_u =	0.250	kip		
Lateral Load:	P_h =	0.223	kip		
Externally Applied Moment:	M =	0.000	k-ft		
PIER DATA					
Pier Dimension:	b =	2	ft (diameter of round footing or diagonal dimension of square footing)		
Pier Height Above Soil:	h_1 =	0.000	ft		
Concrete Strength:	f'_c =	3.500	ksi		
Unit Weight of Concrete:	w_c =	150	pcf		
SOIL DATA					
Depth to Resisting Surface:	h_2 =	0.500	ft		
Presumptive Vertical Pressure:	f_V =	1500	psf	IBC 2015 Table 1806.2	
Presumptive Lateral Pressure:	f_L =	100	psf/ft	IBC 2015 Table 1806.2	
PIER EMBEDMENT - NONCONSTRAINED					
Design Lateral Load:	P_e =	0.223	kip		P_h
Allowable Lateral Pressure:	S_1 =	93.3	psf		$f_L\ d \div 3$
	A =	2.8			$2.34\ P_e \div S_1\ b$
Depth of Pier Embedment:	d =	2.8	ft	IBC 2015 Eqn. 18-1	$0.5\ A\ \{1 + [1 + (4.36\ h \div A)]^{1/2}\}$
Total Length of Pier:	L =	3.3	ft		$d + h_1 + h_2$
PIER EMBEDMENT - CONSTRAINED					
Total Applied Moment:	M_e =	0.00	k-ft		
Allowable Lateral Pressure:	S_3 =	34.9	psf		$f_L\ d$
Depth of Pier Embedment:	d =	0.35	ft	IBC 2015 Eqn. 18-2	$(4.25\ M_e \div S_3\ b)^{1/2}$
Total Length of Pier:	L =	0.85	ft		$d + h_1 + h_2$
PIER END BEARING PRESSURE					
Pier Base Area:	A_f =	3.142	ft^2		$\pi\ b^2 \div 4$
Pier Weight:	w_f =	1555.3	#		$A_f\ L\ w_c$
Design Vertical Load:	P_t =	3416	#		P_v
Vertical Bearing Pressure:	f_t =	1087.2	psf		$P_t \div A_f$
Check:	PASS	72.5%	capacity		1087.2 psf < 1500 psf



LATERAL SLIDING RESISTANCE

Assume class 5 soil material.					
Cohesion Value:	C =	130	psf	IBC 2015 Table 1806.2	
Contact Area:	A_c =	3.142	ft^2		
Cohesion Force:	C_F =	408.5	#		$C\ A_c$
Design Weight of Structure:	w_s =	2160	#		
Weight per Pier:	w_p =	1555.3	#		
Design Cohesion Force:	C_U =	408.5	#	IBC 2015 1806.3.2	*lesser of* C_F *and* $0.5\ w_p$
Sliding Resistance Ratio:	P_h / C_U =	0.55	< 1.0	**PASS**	

PIER SHEAR AND MOMENT

	H_o =	0.112	kip/ft	$P_h \div b$
	M_o =	0.056	k-ft/ft	$(M + P_h (h + h_1 + h_2)) \div b$
	E =	0.5	ft	$M_o \div H_o$
Distance to Pivot Point:	a =	2.051	ft	$d (4 E \div d + 3) \div (6 E \div d + 4)$
Maximum Shear:	V_{max} =	0.222	kip	
Maximum Moment:	M_{max} =	0.256	k-ft	

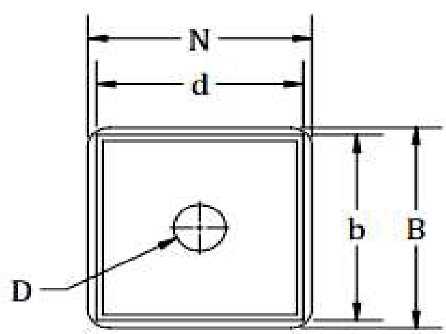
PIER PLAIN CONCRETE STRESSES

Axial Compressive Stresses:	f_a =	9.14	psi	$(P_v + A_f (h_1 + h_2 + a/2)\ 0.15) \div A_f$
Flexural Stress:	f_b =	2.26	psi	$M_{max} \div (\pi b^3 \div 32)$
Combined Compression Stress:	f_c =	11.4	psi	$f_b + f_a$
Allowable Stress:	F_c =	1022.7	psi	$0.4675\ f'_c \div 1.6$
Check:	PASS	1.1%	capacity	
Combined Tension Stress:	f_t =	-6.88	psi	$f_b - f_a$
Allowable Stress:	F_t =	101.68	psi	$2.75\ (f'_c)^{1/2} \div 1.6$
Check:	PASS	0.0%	capacity	
Shear Stress:	f_v =	0.49	psi	$V_{max} \div (\pi b^2 \div 4)$
Allowable Stress:	F_v =	49.3	psi	$4/3\ (f'_c)^{1/2} \div 1.6$
Check:	PASS	1.0%	capacity	

BASE PLATE DESIGN

Design Reference: AISC Design Guide 01: Base Plate and Anchor Rod Design

LOADING				(see RISA output)
Axial Compressive Load:	P_u =	3.416	kip	
Uplift:	P_w =	0.250	kip	
Strong Axis Shear:	V_{uz} =	0.223	kip	
Weak Axis Shear:	V_{uy} =	0.135	kip	
Strong Axis Moment:	M_{uz} =	0.000	k-ft	
Weak Axis Moment:	M_{uy} =	0.000	k-ft	
COMPONENT PROPERTIES				
Column Ultimate Strength:	F_{uC} =	58	ksi	
Column Depth:	d =	6	in	
Column Width:	b =	6	in	
Column Nominal Thickness:	t_{nC} =	0.1875	in	
Column Design Thicknss:	t_{desC} =	0.174	in	AISC 360-16, B4.2
Plate Yield Strength:	F_{yP} =	36	ksi	
Base Plate Length:	N =	6.5	in	
Base Plate Width:	B =	6.5	in	
Anchor Hole Diameter:	D =	1.25	in	
Edge Distance Strong Axis:	ED_z =	3.25	in	
Edge Distance Weak Axis:	ED_y =	3.25	in	
Concrete Support Length:	L =	24	in	
Concrete Support Width:	W =	24	in	
Concrete Compressive Strength:	f'_c =	3.5	ksi	
Anchor Bolt Type:	ASTM F1554 Grade 36			
Anchor Bolt Diameter:	d_A =	0.75	in	
Anchor Bolt Strength:	F_{uA} =	58	ksi	



CONCRETE BEARING

Base Plate Area:	A_1 =	42.25	in^2		
Effective Conc. Support Length:	L' =	24	in		
Effective Conc. Support Width:	W' =	24	in		
Effective Concrete Support Area:	A_2 =	576	in^2		
Bearing Stress Increase Factor:	α =	2		AISC DG1, 3.1.1	Min. of 2, $(A_2 / A_1)^{1/2}$
Nominal Bearing Stress:	R_n =	5.95	ksi		$0.85\ f'_c\ \alpha$
Safety Factor:	Ω_1 =	2.31		ACI 318-14, Table 22.8.3.2	
Allowable Bearing Stress:	R_a =	2.576	ksi		R_n / Ω_1
Equivalent Eccentricity:	e =	0	in	AISC DG1, Eqn. 3.3.6	M_{uz} / P_u
Bearing Length:	Y_z =	6.5	in	AISC DG1, Eqn. 3.3.8	N - 2 e
Strong Axis Req. Bearing Stress:	f_{pz} =	0.081	ksi		
Strong Axis Unity Check:		**0.031**	**PASS**		
Equivalent Eccentricity:	e =	0	in	AISC DG1, Eqn. 3.3.6	M_{uy} / P_u
Bearing Length:	Y_y =	6.5	in	AISC DG1, Eqn. 3.3.8	B - 2 e
Weak Axis Req. Bearing Stress:	f_{py} =	0.081	ksi		
Weak Axis Unity Check:		**0.031**	**PASS**		

CONCENTRIC COMPRESSIVE AXIAL LOADS

Concrete Bearing Area:	A_3 =	288	in^2		
Design Case:		2		AISC DG1, 3.1.4	
Safety Factor:	Ω_2 =	2.5		AISC DG1, 3.1.1	
Required Base Plate Area:	A_{1req} =	1.435	in^2		$\Omega_2\ P_u / 2\ (0.85\ f'_c)$
Nominal Bearing Strength:	P_p =	295.75	kip		$f'c\ 2\ A_1$
Case 3 Criterion:		N/A			Condition: $P_u \leq (P_p / \Omega_2)$
Bearing Interface Based on N:	m =	0.4	in	AISC DG1, 3.1.3	(N - 0.95 d) / 2
Bearing Interface Based on B:	n =	0.4	in	AISC DG1, 3.1.3	(B - 0.95 b) / 2
Maximum Bearing Interface:	l =	0.4	in		Max. of m, n
Required Plate Thickness:	t_{min} =	0.042	in		$l\ [(2\ P_u\ \Omega_2) / (F_{yP}\ B\ N)]^{1/2}$

Use 1/4" thick plate minimum.

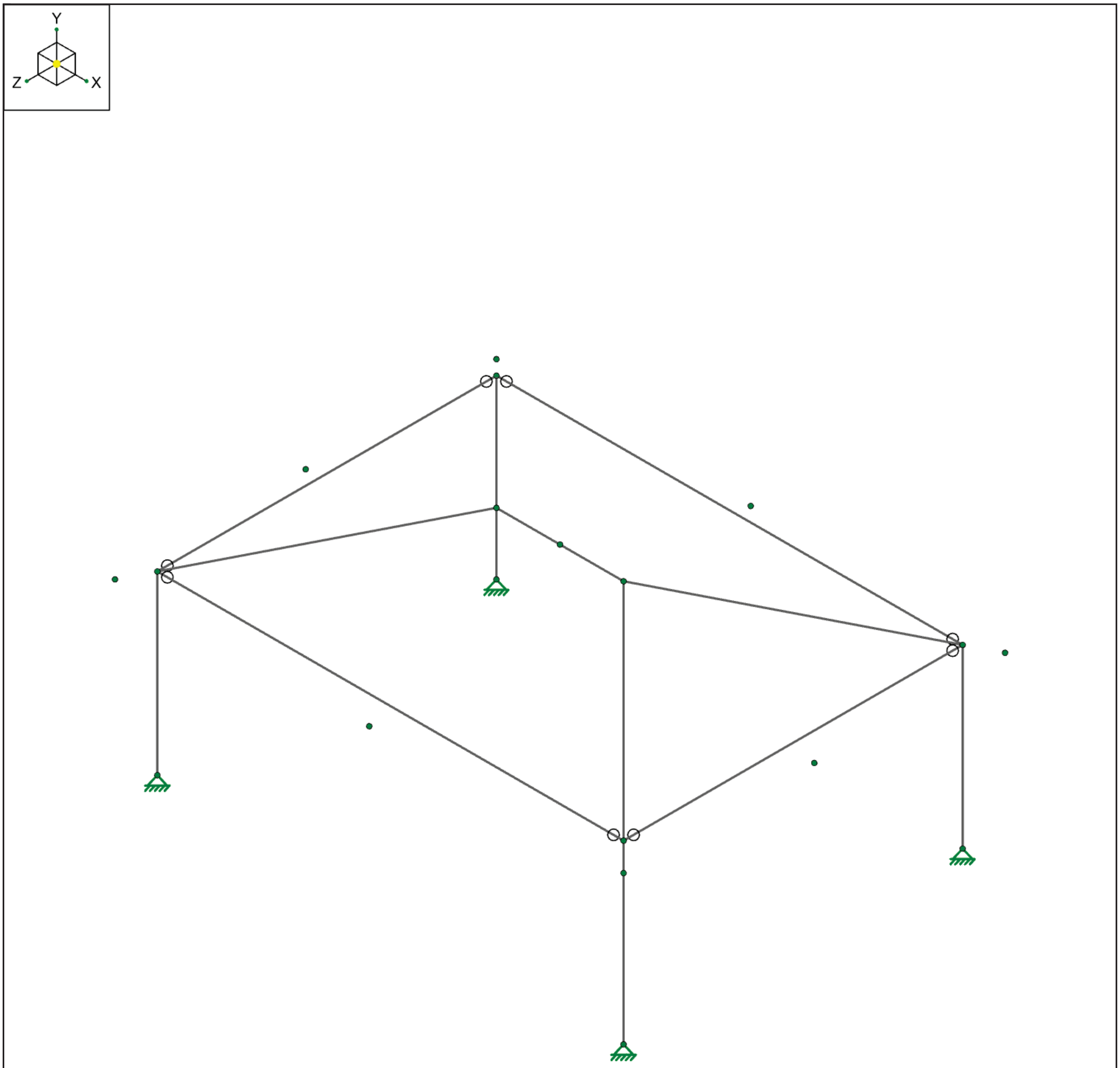
PLATE FLEXURAL YIELDING (COMPRESSION) (STRONG AXIS)

Nominal Bending Stress:	M_n =	0.563	kip-ft		$F_{yP}\ (t_P)^2 / 4$
Safety Factor:	Ω_3 =	1.67		AISC DG1, 3.3.13	
Bend Capacity per Unit Width:	M_a =	0.028	kip-ft/in		M_n / Ω_3
Bend Moment per Unit Width:	M_{pl1} =	0.001	kip-ft/in	AISC DG1, 3.3.2	
Side Bending Adjustment Factor:	β =	1			
Side Bending per Unit Width:	M_{pl2} =	0.001	kip-ft/in		
Reqd. Moment per Unit Width:	M_{pl} =	0.001	kip-ft/in	AISC DG1, 3.1.2	
Unity Check:		**0.040**	**PASS**		

PLATE FLEXURAL YIELDING (COMPRESSION) (WEAK AXIS)

Bend Moment per Unit Width:	M_{pl1} =	0.001	kip-ft/in	AISC DG1, 3.3.2	
Side Bending Adjustment Factor:	β =	1			
Side Bending per Unit Width:	M_{pl2} =	0.001	kip-ft/in		
Reqd. Moment per Unit Width:	M_{pl} =	0.001	kip-ft/in	AISC DG1, 3.1.2	
Unity Check:		**0.040**	**PASS**		

PLATE FLEXURAL YIELDING (TENSION) (STRONG AXIS)					
Effective Width of Plate Section:	b_e =	0.4	in		
Nominal Bending Stress:	M_n =	0.225	kip-ft		
Bend Capacity per Unit Width:	M_a =	0.011	kip-ft/in		M_n / Ω_3
Moment Arm for Anchor Bolt:	x =	0.337	in	AISC DG1, Eqn. 3.4.6	
Tension Moment at Anchor Bolt:	M_{pl} =	0.007	kip-ft/in		
Unity Check:		**0.640**	**PASS**		
ANCHOR BOLT SHEAR					
Resultant Shear Force:	V_u =	0.261	kip		
Nominal Shear Stress:	F_{nv} =	26.1	ksi	AISC 360-16, Table J3.2	
Bolt Cross Sectional Area:	A_b =	0.442	in^2		
Safety Factor:	Ω_4 =	2		AISC 360-16, J3-1	
Nominal Shear Stress	R_n =	11.536	kip		
Bolt Shear Rupture Strength:	R_a =	5.768	kip		R_n / Ω_4
Unity Check:		**0.050**	**PASS**		
ANCHOR BOLT TENSION (STRONG AXIS)					
Note: Prying effects are ignored.					
Tension at Anchor Bolt:	T_c =	0.250	kip		Use total uplift reaction
Safety Factor:	Ω_5 =	2		AISC 360-16, J3-2	
Required Tensile Stress:	f_{rt} =	0.566	ksi		
Nominal Tensile Stress:	F_{nt} =	43.5	ksi	AISC 360-16, Table J3.2	
Check User Note Limit:		TRUE	Bolt Tensile Check is not required.		Condition: $[f_{rt} / (F_{nt} / \Omega_5)] \leq 0.3$
Required Shear Stress:	f_{rv} =	0.59	ksi		
Nominal Tensile Stress:	R_n =	19.227	kip		
Bolt Tensile Strength:	R_a =	9.614	kip		R_n / Ω_5
Unity Check:		**0.030**	**PASS**		
ANCHOR BOLT BEARING ON BASE PLATE					
Angle Between Force and Z-Axis:	θ =	58.81	degrees		
Anchor Hole Center to Edge:	d_c =	0.625	in		
Minimum Clear Distance:	L_c =	3.174	in		
Bolt Shear Strength:	$R_{n\text{-}bolt}$ =	11.536	kip		
Nominal Bolt Bearing Stress:	R_n =	11.536	kip		
Safety Factor:	Ω_6 =	2		AISC 360-16, J3.10	
Bolt Bearing Strength:	R_a =	5.768	kip		R_n / Ω_6
Unity Check:		**0.050**	**PASS**		
WELDS					
Welding Electrode:	F_{EXX} =	70	ksi		
Allowable Electrode Strength:	F_w =	42	ksi		$0.60\ F_{EXX}$
Effective Weld Length:	l_w =	24	in		2 (b + d)
Trial Weld Size:	t_w =	3/16	in		
Weld Area:	A_w =	3.182	in^2		$l_w (t_w / \sqrt{2})$
Safety Factor:	Ω_w =	2			
Allowable Weld Stress:	R_{aw} =	66.822	kip		$F_w A_w / \Omega_w$
Unity Check:		**0.000**	**PASS**		



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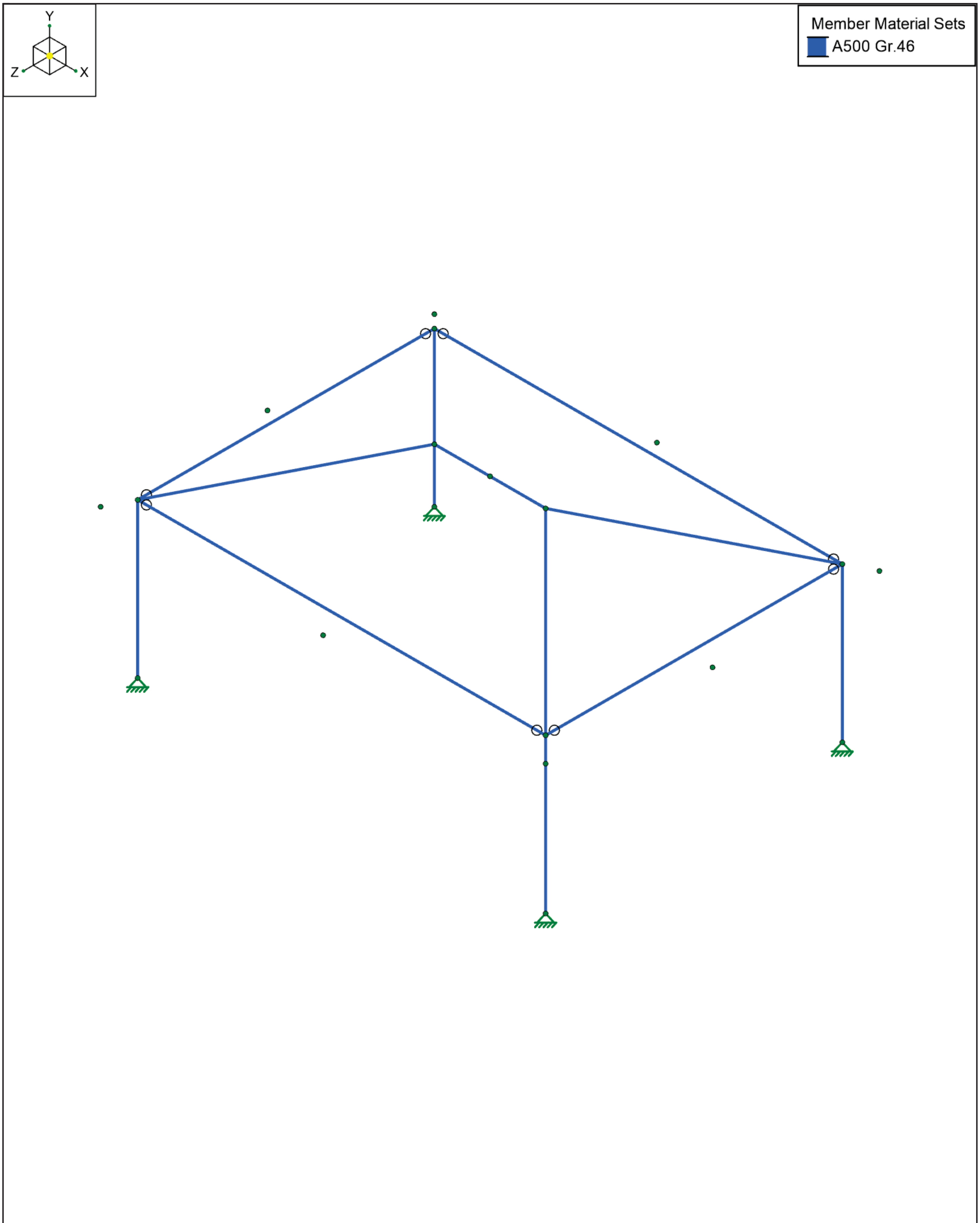
	225498	Overall - Isometric View	753 Olive Branch Rd 225498...



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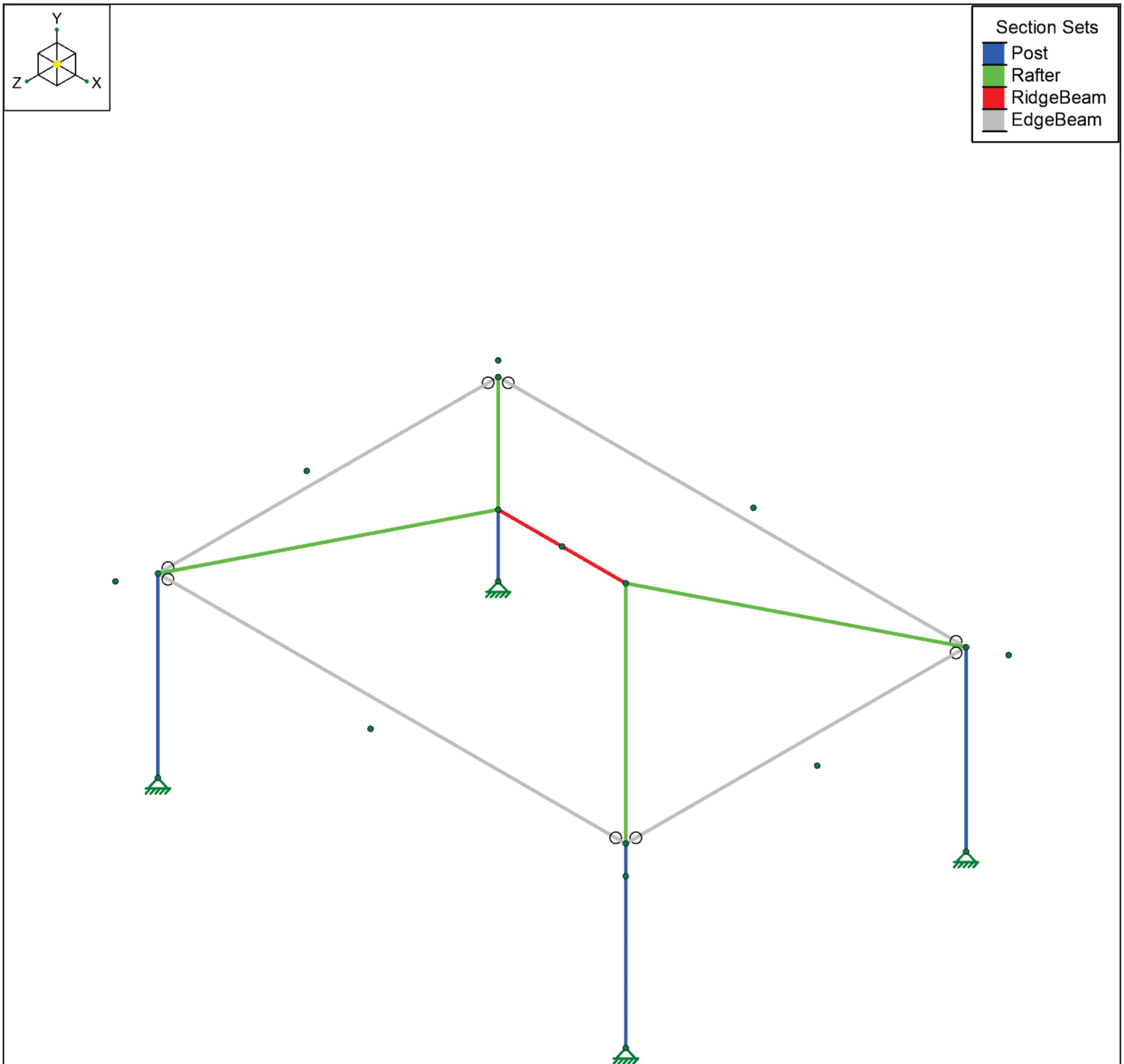

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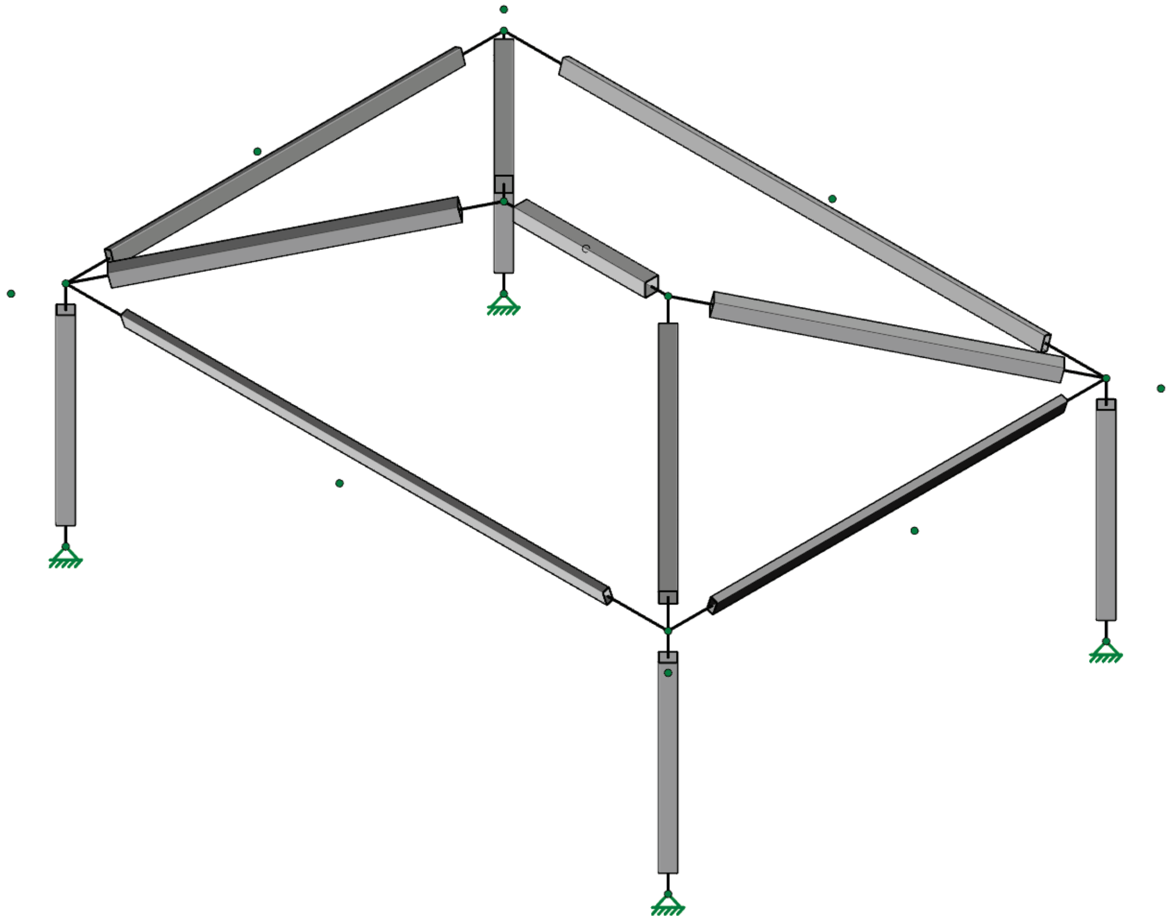

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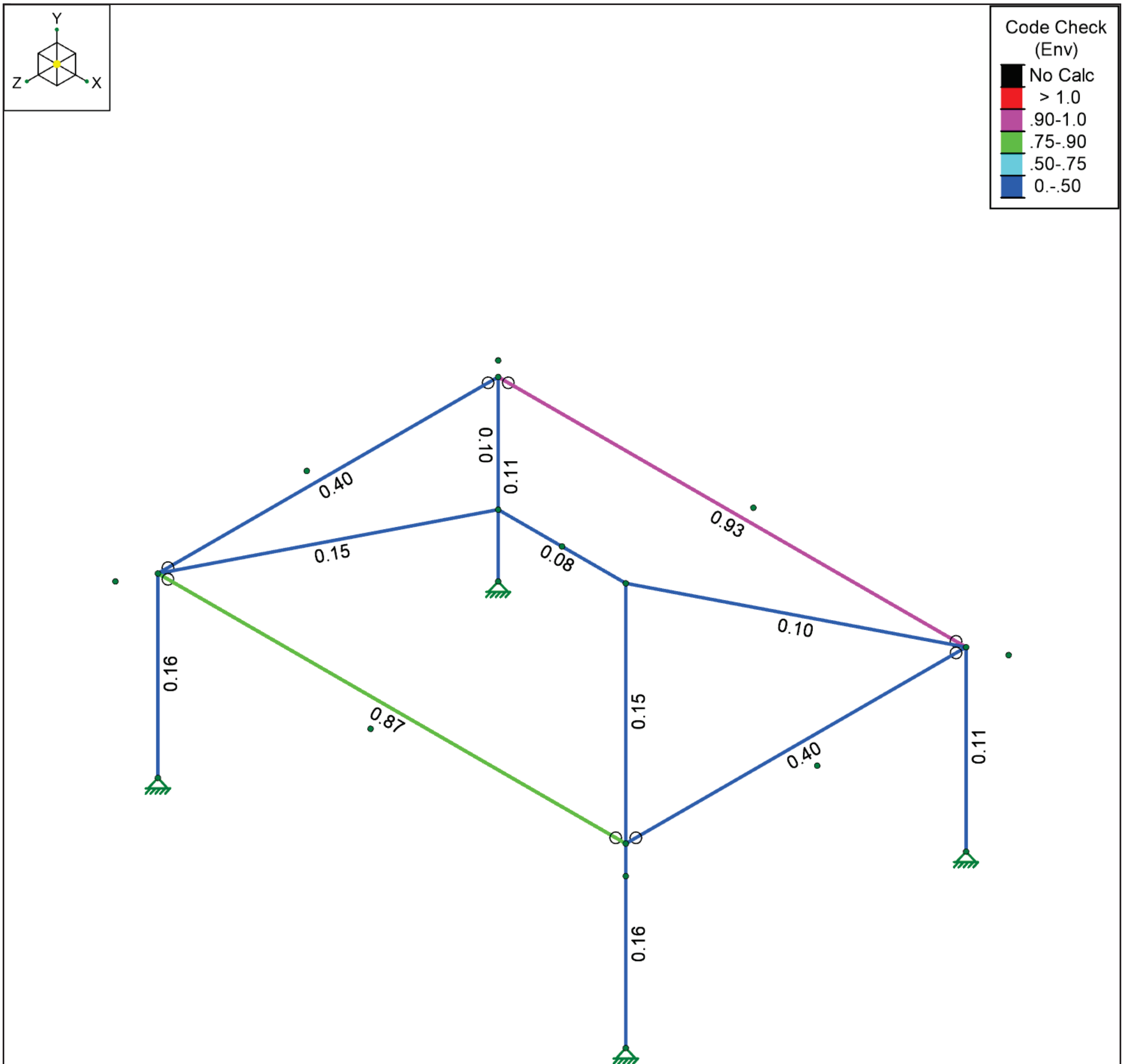

Member Code Checks Displayed (Enveloped)

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-5
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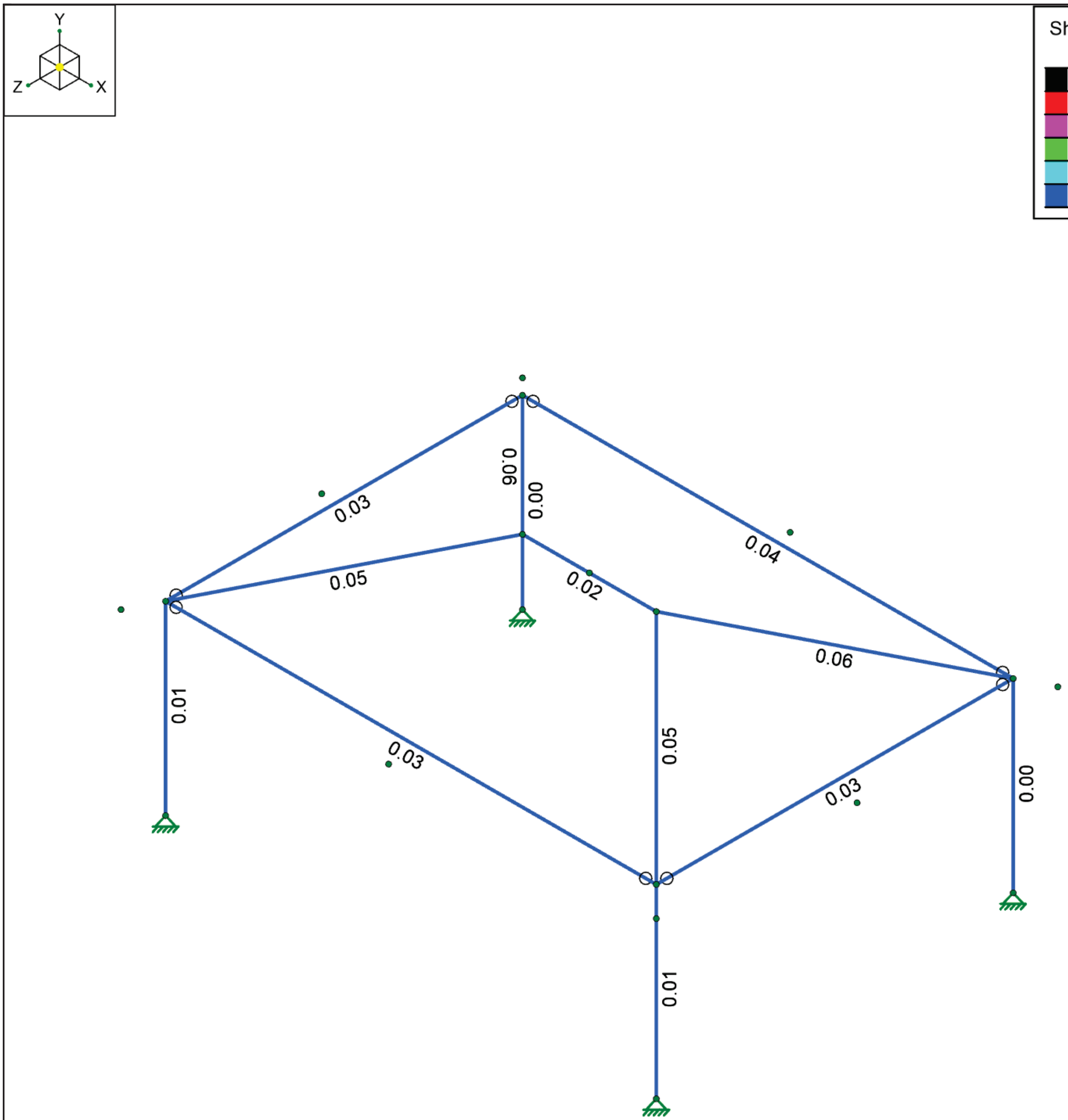
	225498	Overall - Combined Stress	753 Olive Branch Rd 225498…



Shear Check (Env)

- No Calc
- > 1.0
- .90-1.0
- .75-.90
- .50-.75
- 0.-.50

Member Shear Checks Displayed (Enveloped)

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-6
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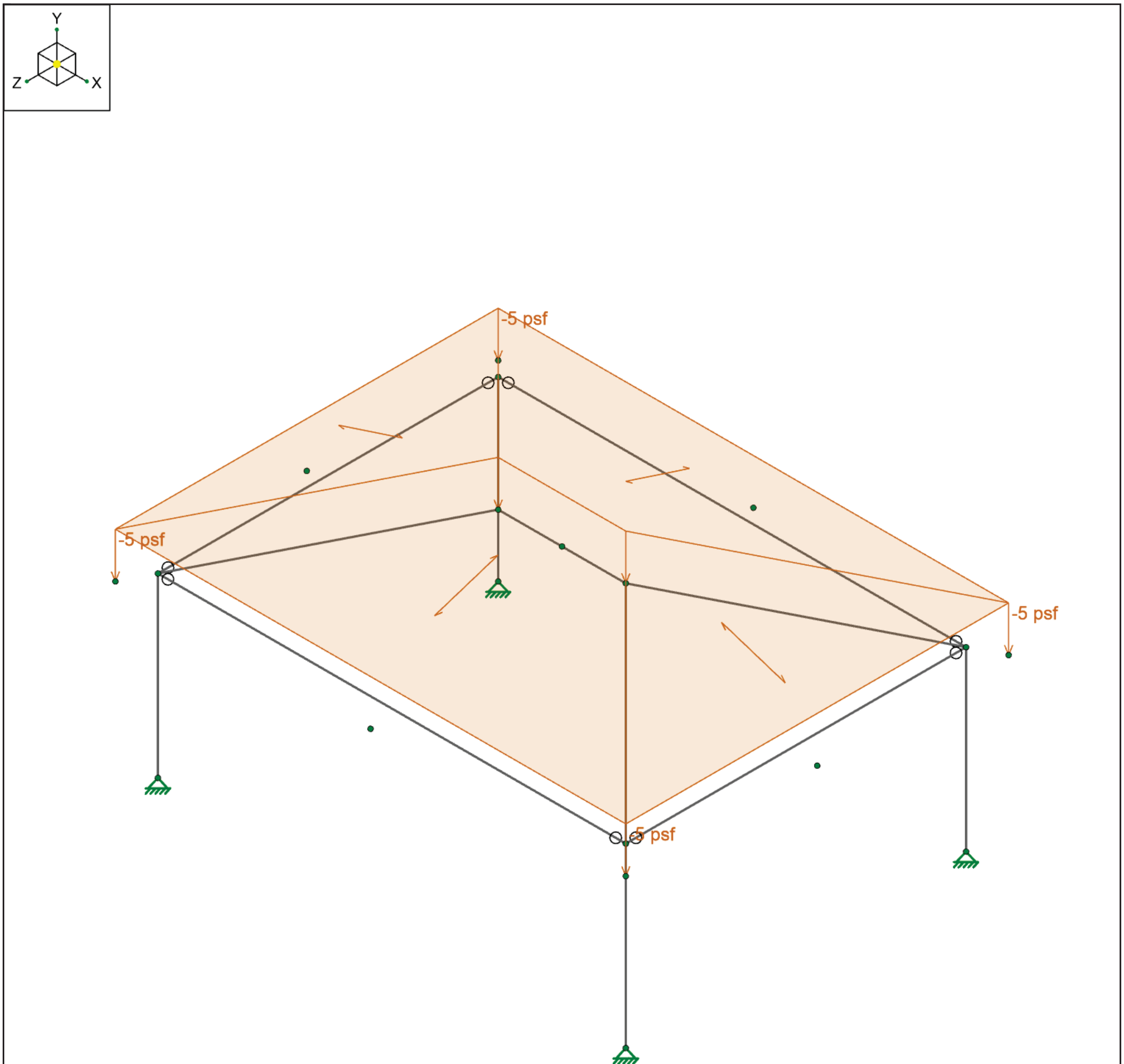

Loads: BLC 1, D

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-7
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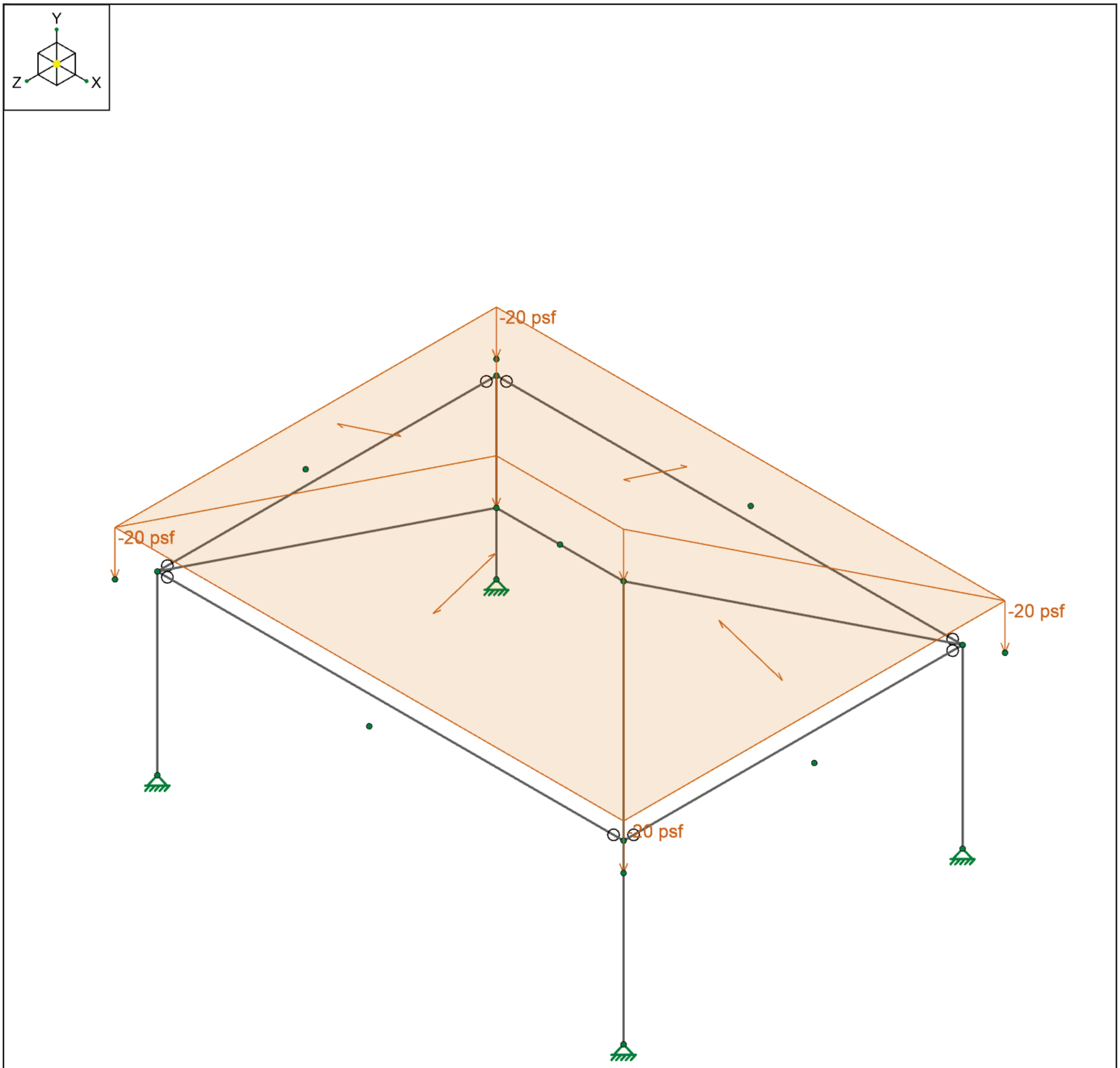

Loads: BLC 2, Lr

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-8
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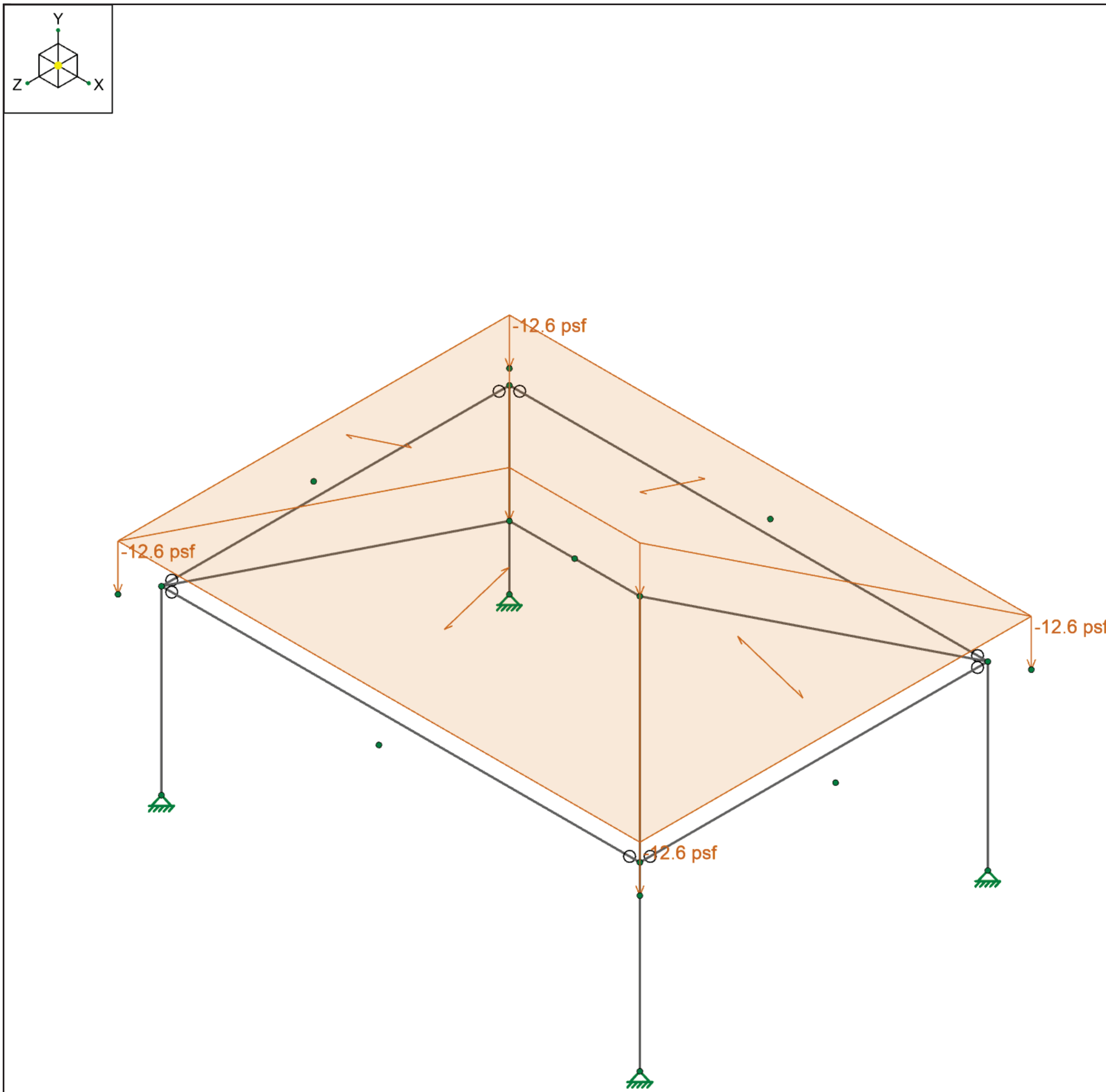

Loads: BLC 3, S balanced

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-9
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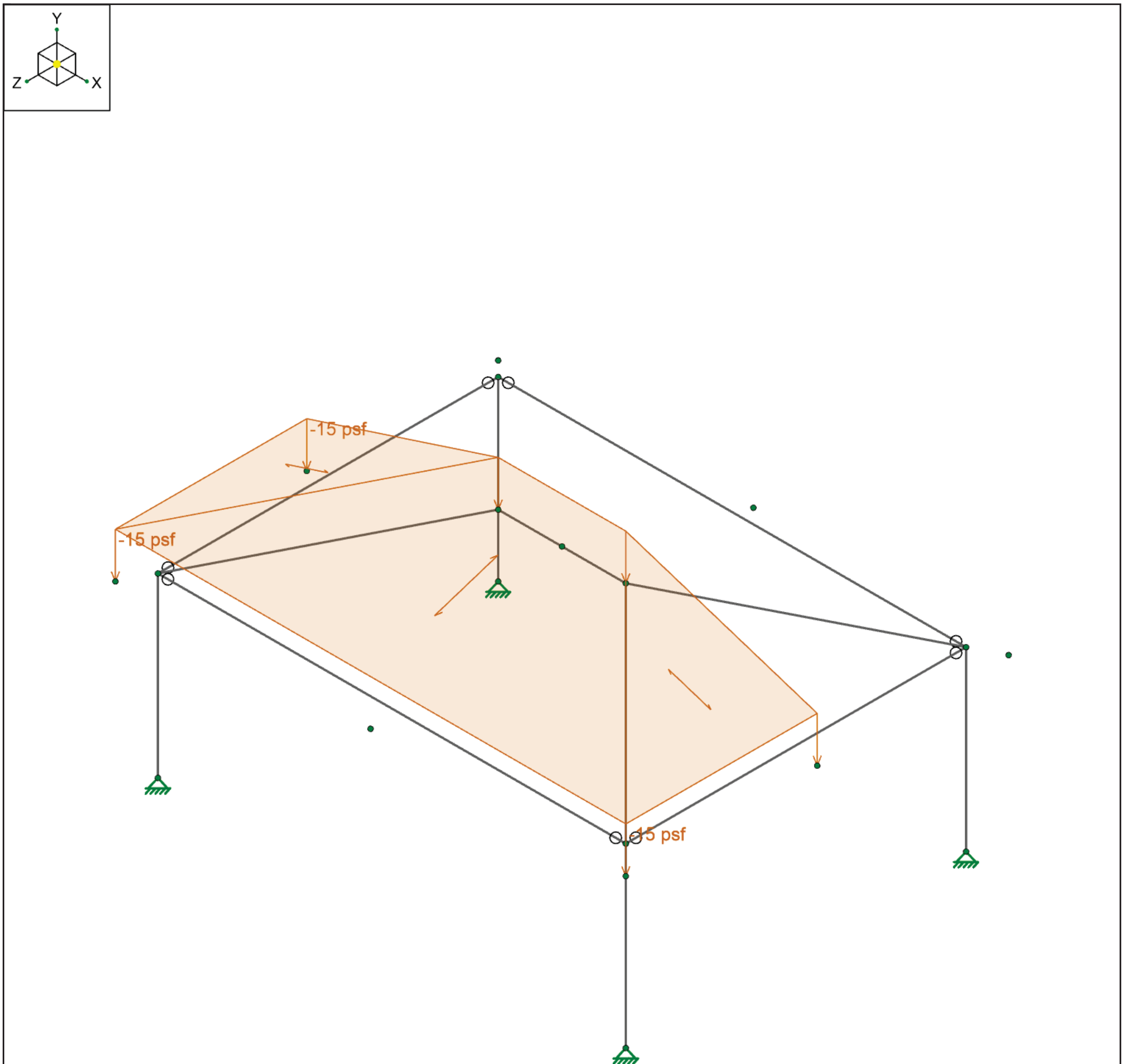

Loads: BLC 4, S unbalanced

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-10
	AO		Aug 07, 2025 at 01:34 PM
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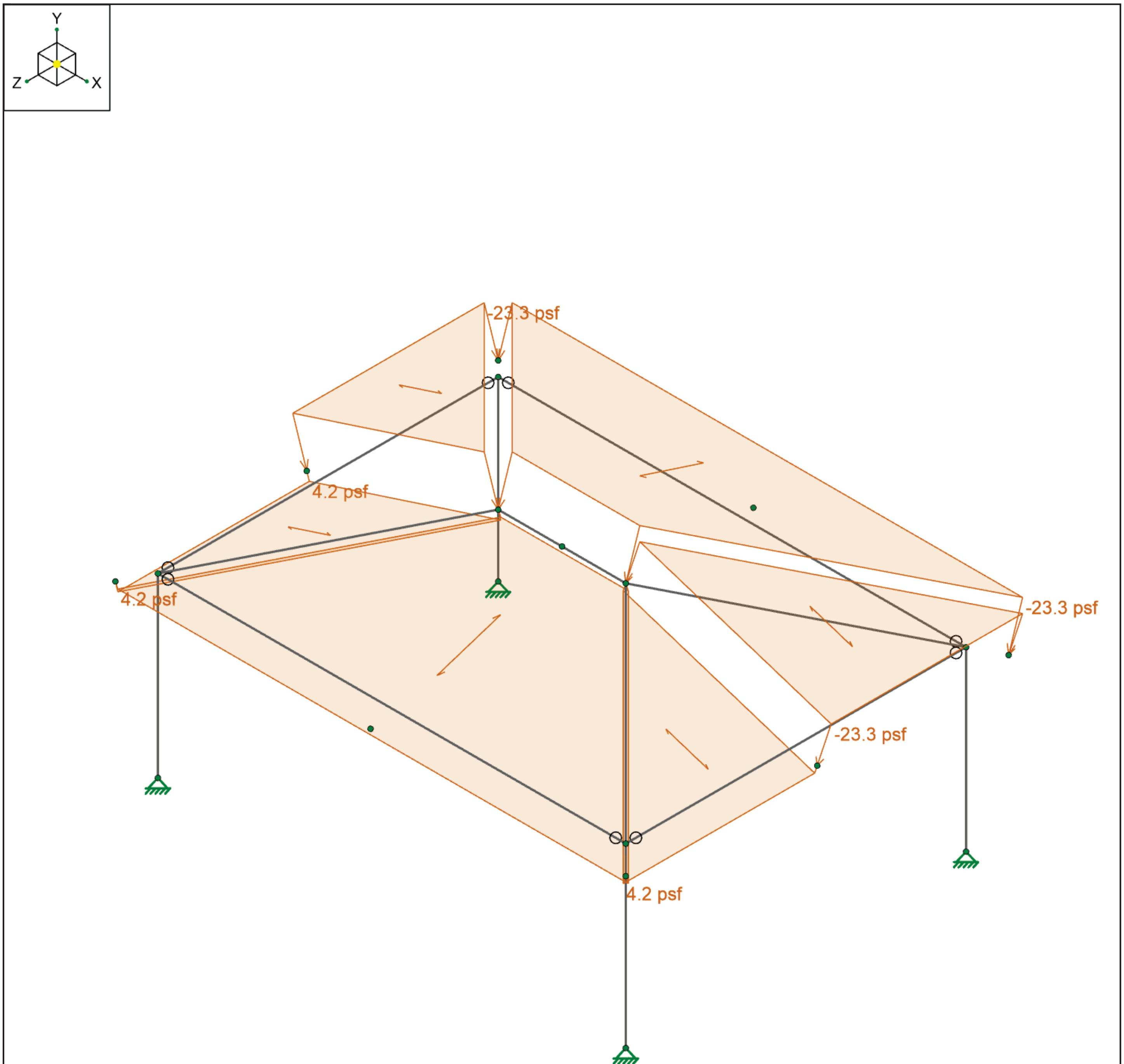
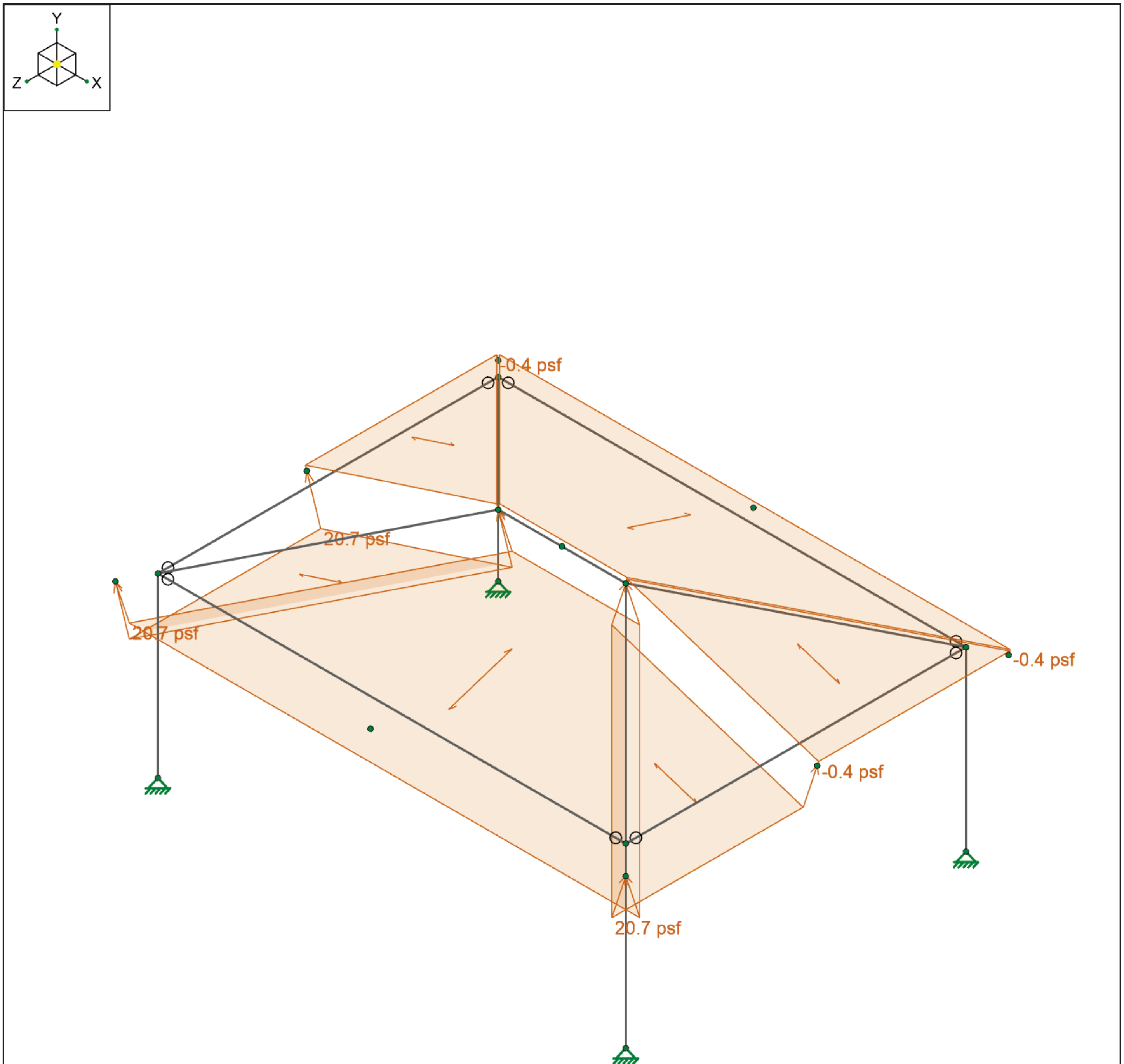


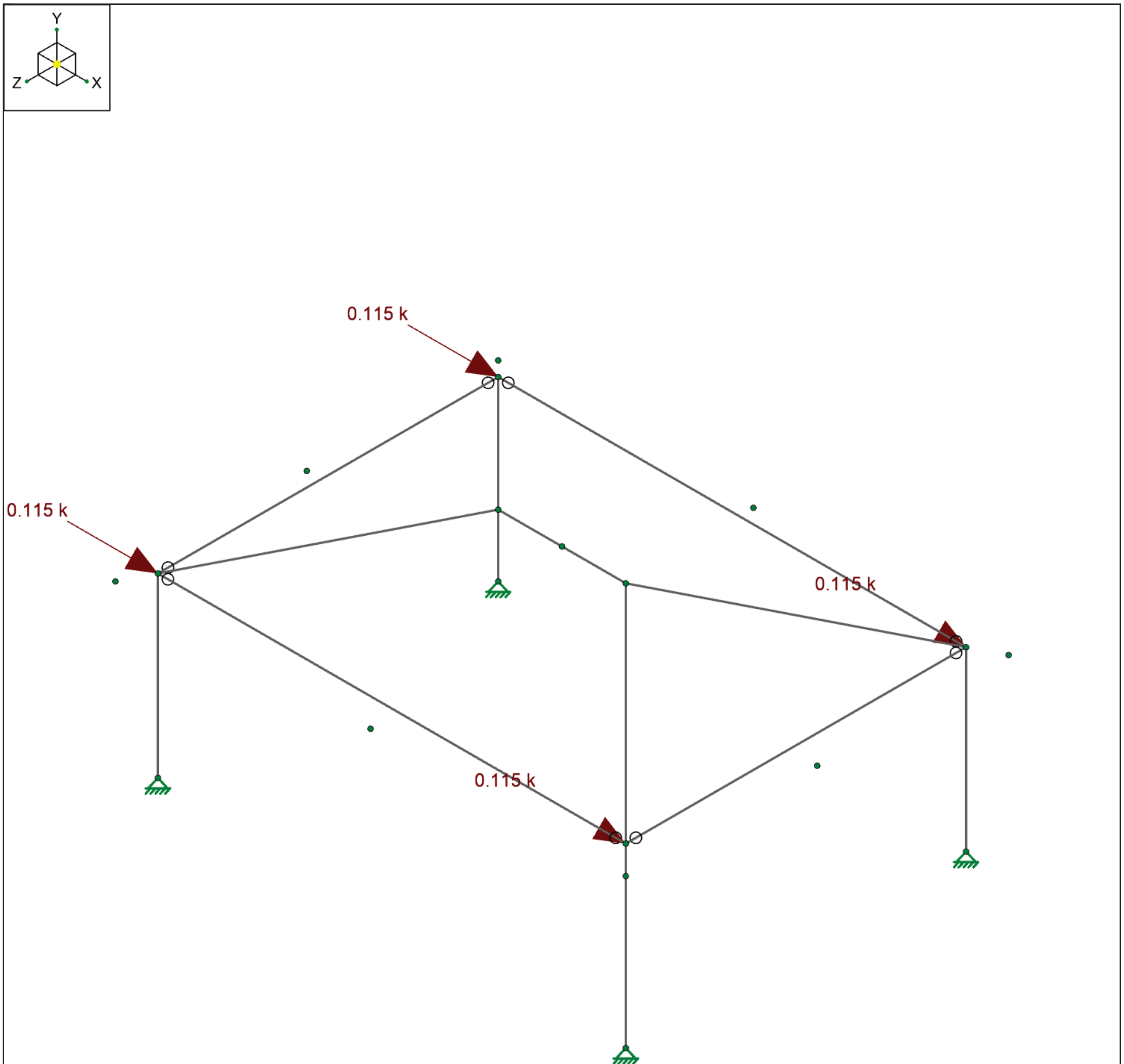
Loads: BLC 5, W Transverse case A

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-11
	AO		Aug 07, 2025 at 01:34 PM
	225498	Overall - Wind Loads (Transverse - Case A)	753 Olive Branch Rd 225498…



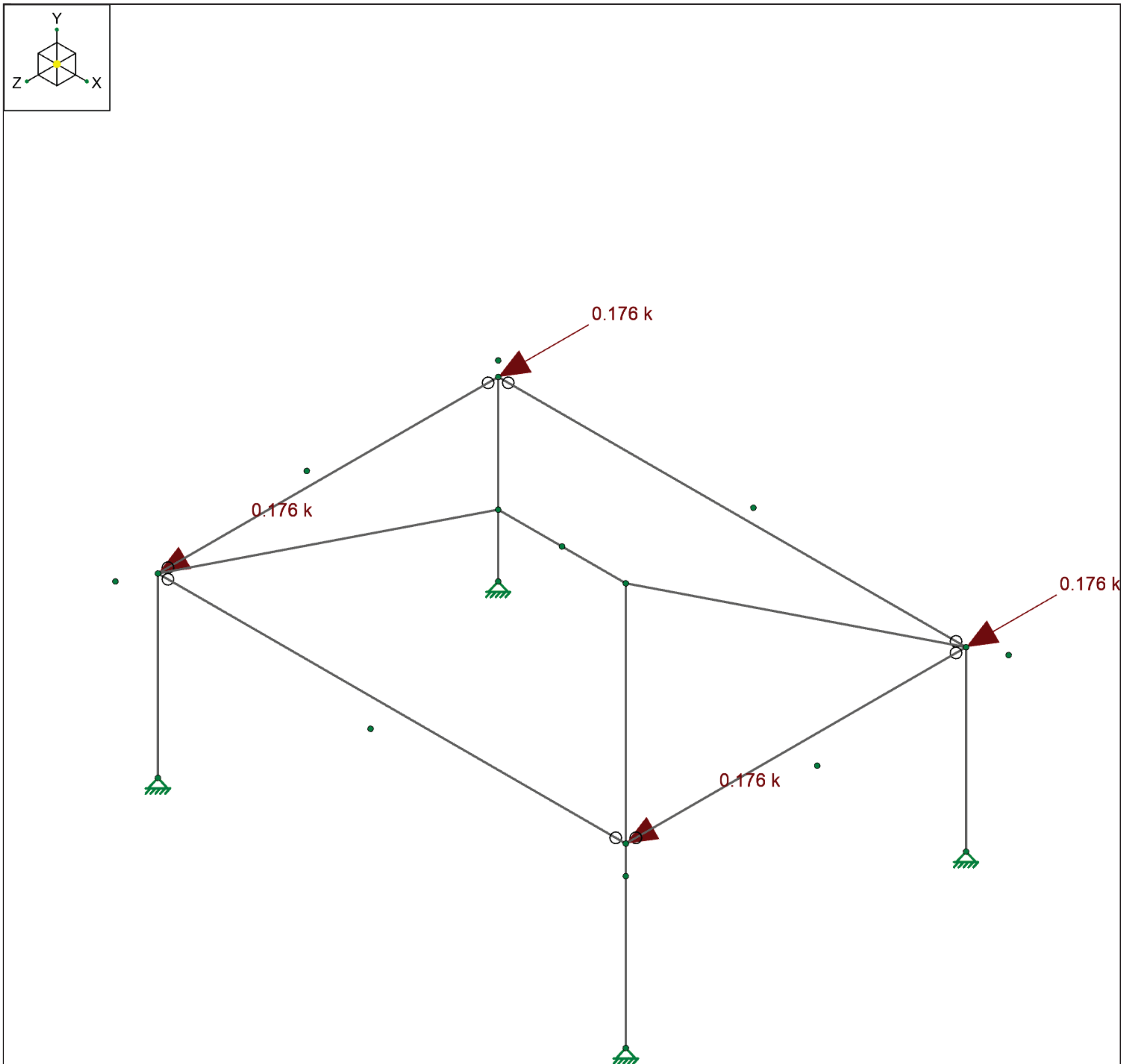
Loads: BLC 6, W Transverse case B

RISA	Americana Outdoors	Navajo Shelter 18' x 24'	SK-12
	AO		Aug 07, 2025 at 01:35 PM
	225498	Overall - Wind Loads (Transverse - Case B)	753 Olive Branch Rd 225498...



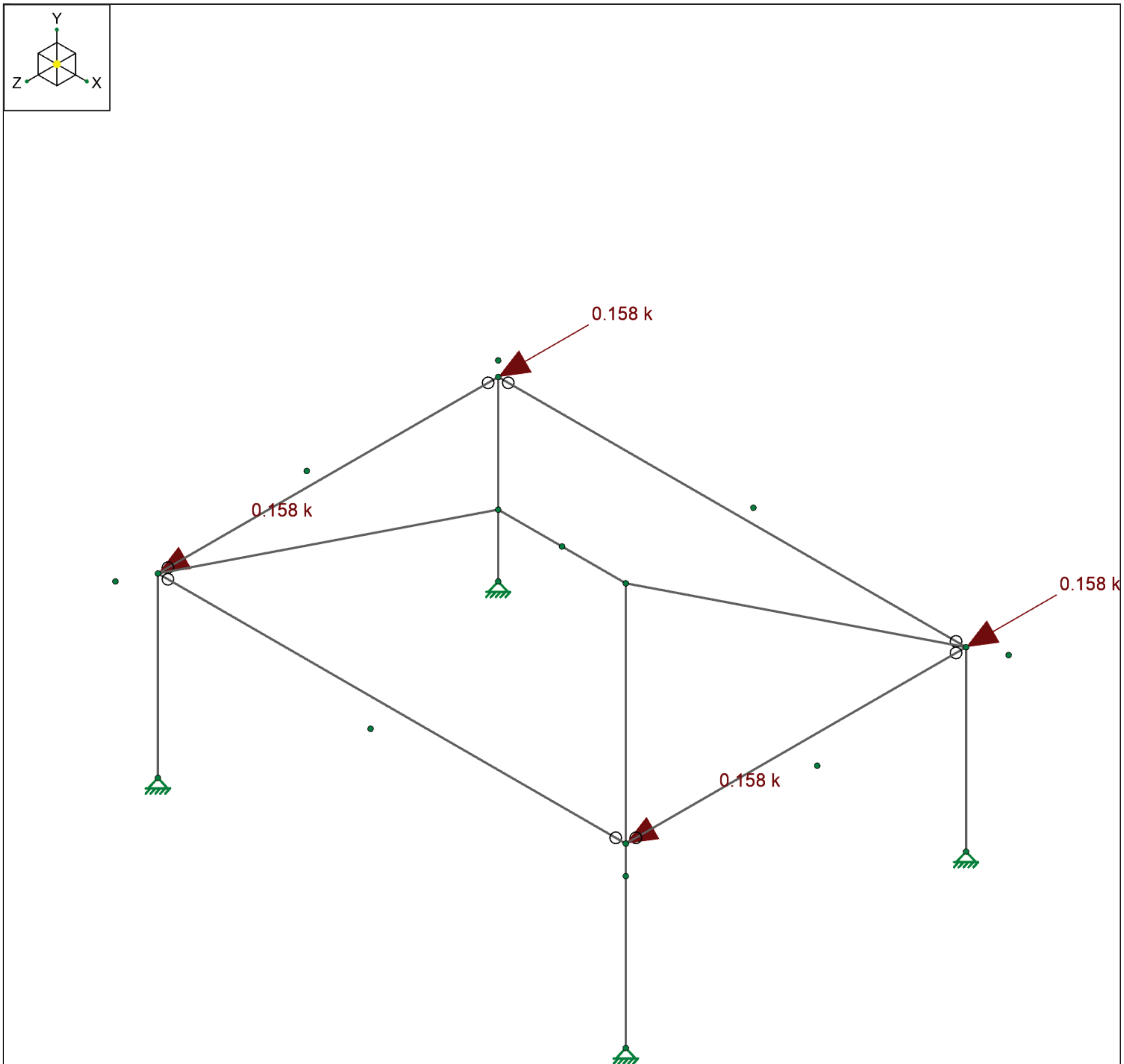
Loads: BLC 7, W Longitudinal

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-13
	AO		Aug 07, 2025 at 01:36 PM
	225498	Overall - Wind Loads (Longitudinal)	753 Olive Branch Rd 225498...



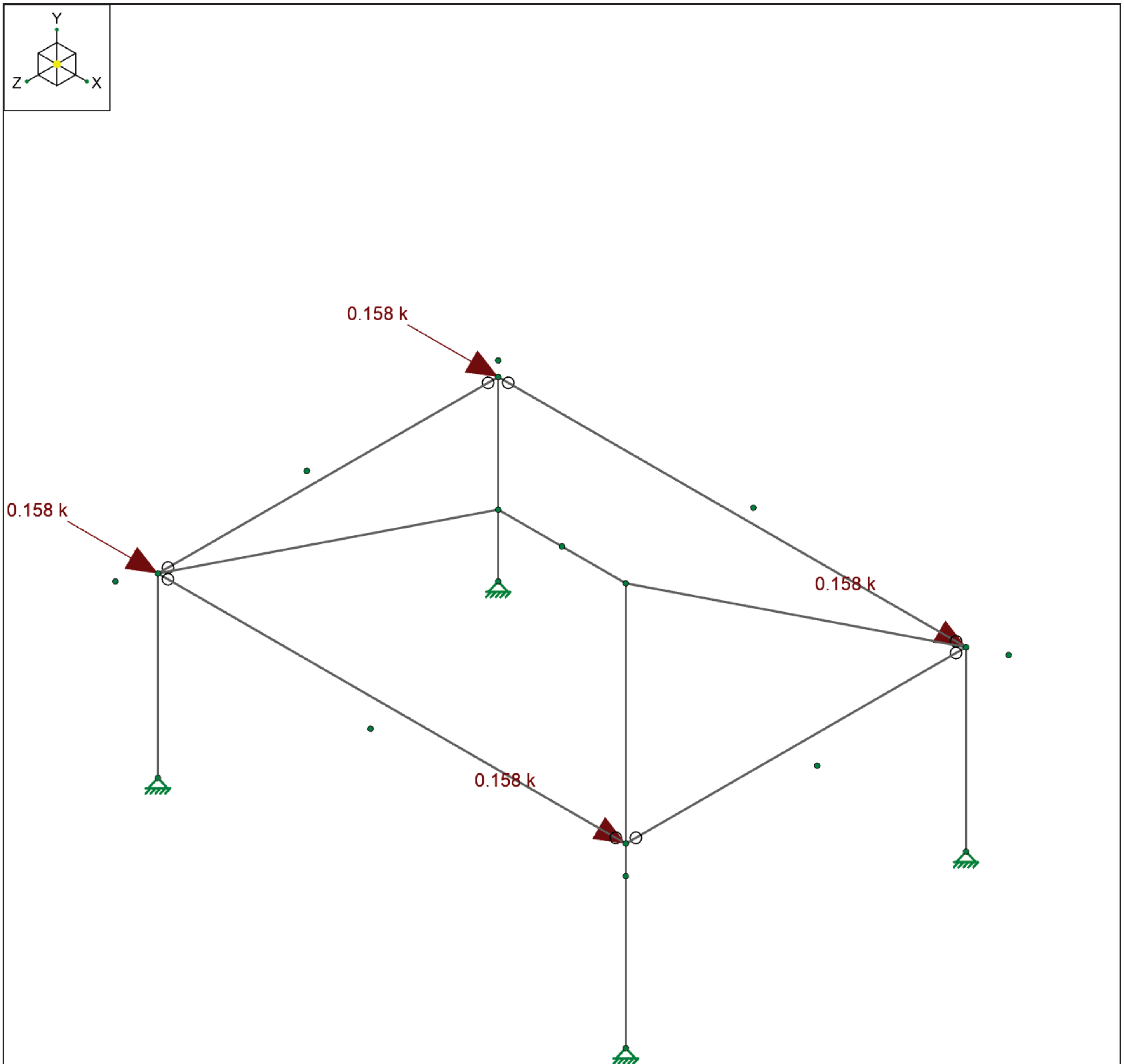
Loads: BLC 8, W Minimum

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-14
	AO		Aug 07, 2025 at 01:37 PM
	225498	Overall - Wind Loads (Minimum)	753 Olive Branch Rd 225498...



Loads: BLC 9, E Transverse

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-15
	AO		Aug 07, 2025 at 01:38 PM
	225498	Overall - Seismic Loads (Transverse)	753 Olive Branch Rd 225498...

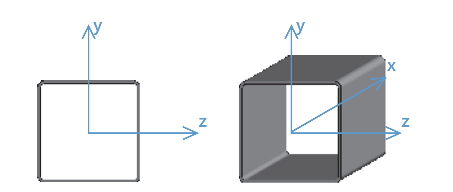


Loads: BLC 10, E Longitudinal

RISA A NEMETSCHEK COMPANY	Americana Outdoors	Navajo Shelter 18' x 24'	SK-16
	AO		Aug 07, 2025 at 01:39 PM
	225498	Overall - Seismic Loads (Longitudinal)	753 Olive Branch Rd 225498...

Detail Report: M2

Load Combination: Envelope **Code check:** 0.160 (LC 14)



Input Data

Shape:	HSS6X6X3	**I Node:**	N10
Member Type:	Column	**J Node:**	N14
Length (in):	99.899	**I Release:**	Fixed
Material Type:	Hot Rolled Steel	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	96	**J Offset:**	N/A
Design Code:	AISC 14th (360-10): ASD	**T/C Only:**	Both Way

Material Properties

Material:	A500 Gr.46	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

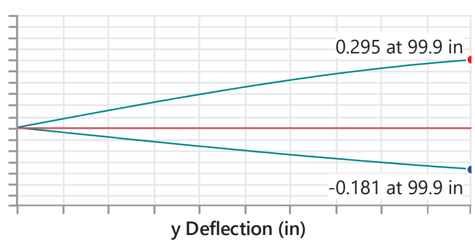
Shape Properties

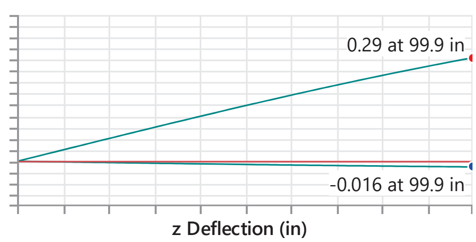
d (in):	6	**I_{yy} (in^4):**	22.3	**J (in^4):**	35
b_f (in):	6	**I_{zz} (in^4):**	22.3		
t (in):	0.174	**Area (in^2):**	3.98		

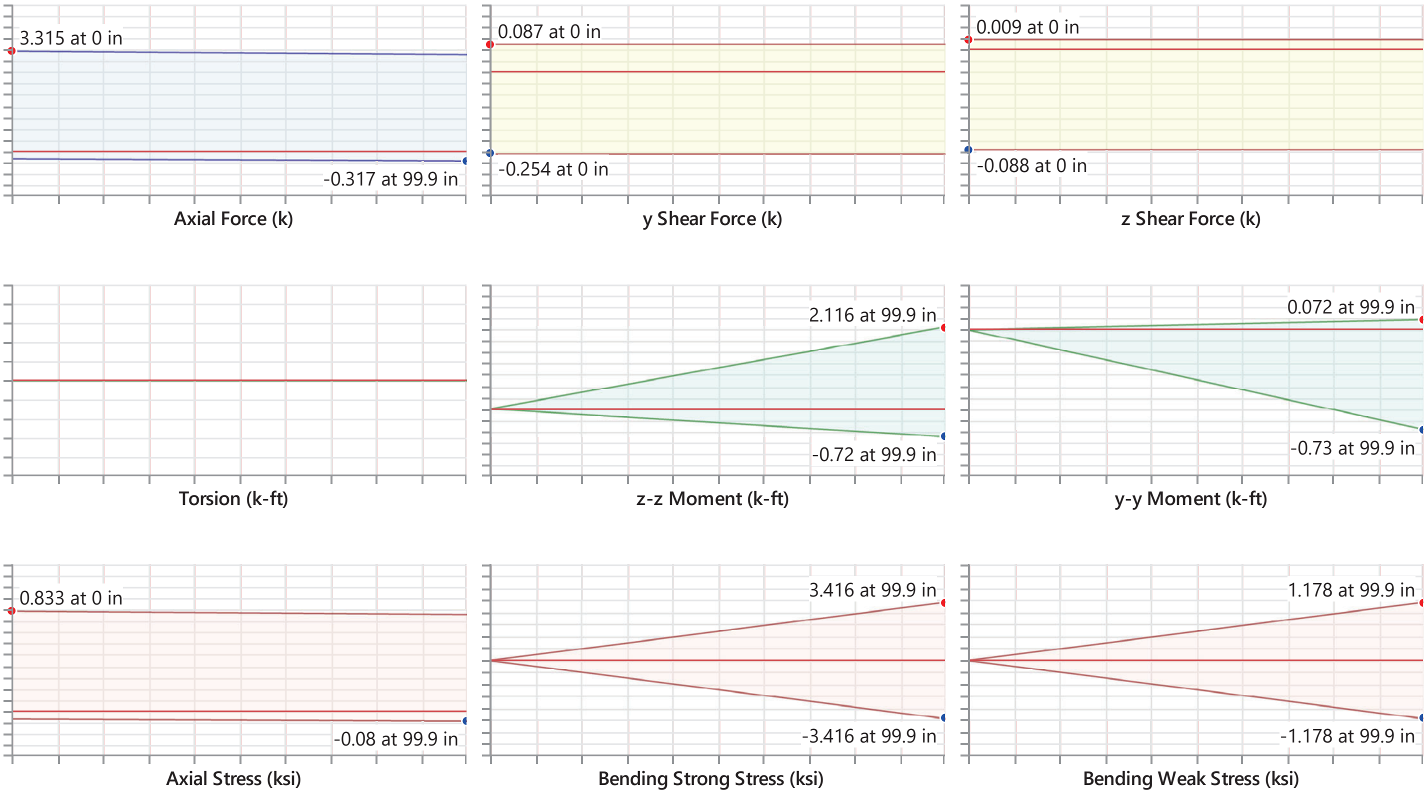
Design Properties

$L_{b\ y\text{-}y}$ (in):	99.899	**$K_{y\text{-}y}$:**	1	**Max Defl Ratio:**	L/1022
$L_{b\ z\text{-}z}$ (in):	99.899	**$K_{z\text{-}z}$:**	1	**Max Defl Location:**	99.899
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Span:**	N/A
$L_{comp\ bot}$ (in):	99.899	**z sway:**	No	**τ_b:**	1
L_{torque} (in):	99.899	**Function:**	Lateral		
C_b:	1.678	**Seismic DR:**	None		

Diagrams:





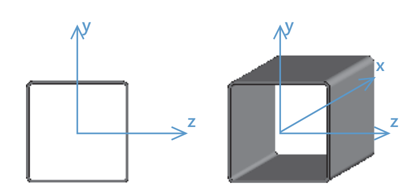


AISC 14th (360-10): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	14	-	-	-	-
Applied Loading - Shear + Torsion	14	-	-	-	-
Axial Tension Analysis	14	0 k	109.629 k	-	-
Axial Compression Analysis	14	2.9 k	97.25 k	-	-
Flexural Analysis (Strong Axis)	14	2.116 k-ft	18.501 k-ft	-	-
Flexural Analysis (Weak Axis)	-	0.577 k-ft	18.501 k-ft	-	-
Shear Analysis (Major Axis y)	14	0.254 k	31.506 k	**0.008**	**PASS**
Shear Analysis (Minor Axis z)	14	0.069 k	31.506 k	**0.002**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	14	-	-	**0.16**	**PASS**
Torsional Analysis	14	0 k-ft	16.24 k-ft	**0**	**PASS**

Detail Report: M6

Load Combination: Envelope **Code check:** 0.148 (LC 14)



Input Data

Shape:	HSS6X6X3	**I Node:**	N14
Member Type:	Beam	**J Node:**	N17
Length (in):	139.302	**I Release:**	Fixed
Material Type:	Hot Rolled Steel	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	96	**J Offset:**	N/A
Design Code:	AISC 14th (360-10): ASD	**T/C Only:**	Both Way

Material Properties

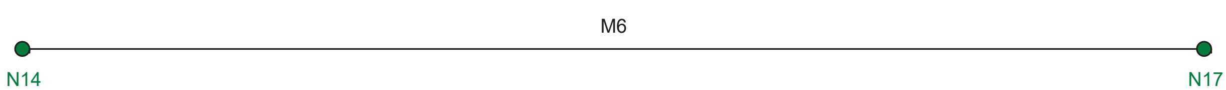
Material:	A500 Gr.46	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

Shape Properties

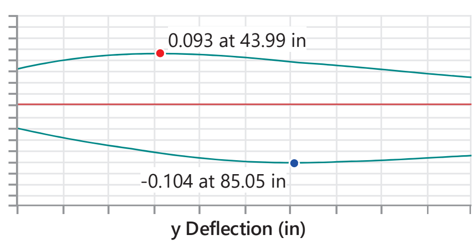
d (in):	6	**I_{yy} (in^4):**	22.3	**J (in^4):**	35
b_f (in):	6	**I_{zz} (in^4):**	22.3		
t (in):	0.174	**Area (in^2):**	3.98		

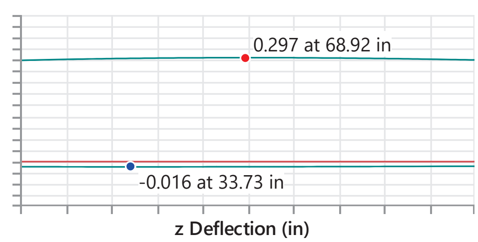
Design Properties

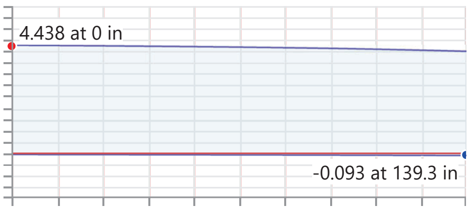
$L_{b\ y\text{-}y}$ (in):	139.302	**$K_{y\text{-}y}$:**	1	**Max Defl Ratio:**	L/1223
$L_{b\ z\text{-}z}$ (in):	139.302	**$K_{z\text{-}z}$:**	1	**Max Defl Location:**	139.302
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Span:**	1
$L_{comp\ bot}$ (in):	139.302	**z sway:**	No	**τ_b:**	1
L_{torque} (in):	139.302	**Function:**	Lateral		
C_b:	2.896	**Seismic DR:**	None		



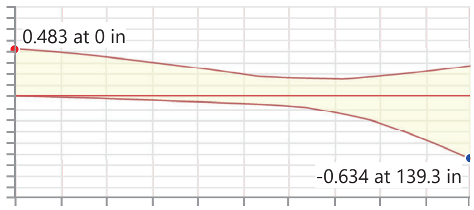
Diagrams:



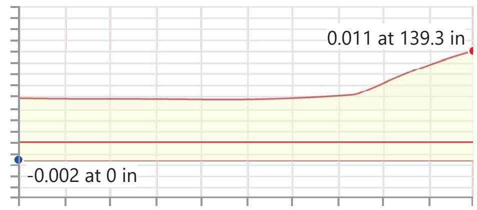




Axial Force (k)

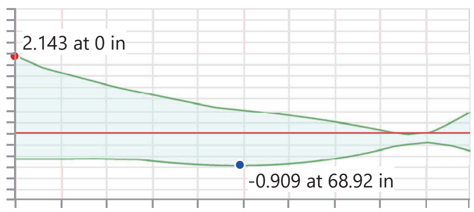


y Shear Force (k)

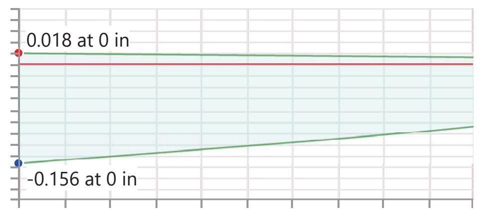


z Shear Force (k)

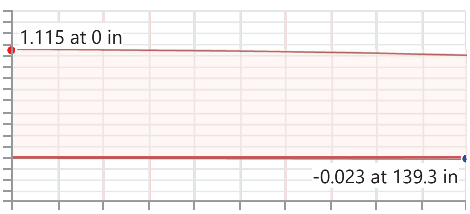
Torsion (k-ft)



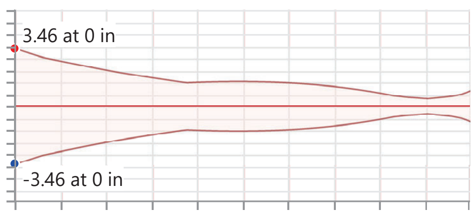
z-z Moment (k-ft)



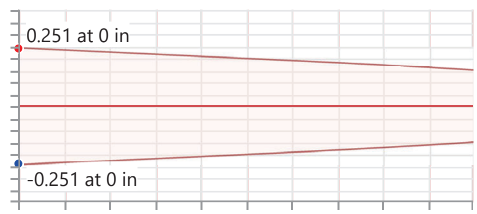
y-y Moment (k-ft)



Axial Stress (ksi)



Bending Strong Stress (ksi)



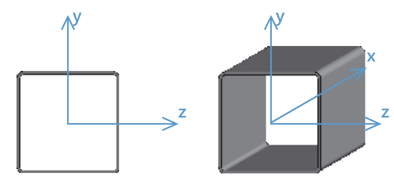
Bending Weak Stress (ksi)

AISC 14th (360-10): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	14	-	-	-	-
Applied Loading - Shear + Torsion	8	-	-	-	-
Axial Tension Analysis	14	0 k	109.629 k	-	-
Axial Compression Analysis	14	4.376 k	86.845 k	-	-
Flexural Analysis (Strong Axis)	14	2.143 k-ft	18.501 k-ft	-	-
Flexural Analysis (Weak Axis)	-	0.123 k-ft	18.501 k-ft	-	-
Shear Analysis (Major Axis y)	8	1.627 k	31.506 k	**0.052**	**PASS**
Shear Analysis (Minor Axis z)	8	1.318 k	31.506 k	**0.042**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	14	-	-	**0.148**	**PASS**
Torsional Analysis	14	0.535 k-ft	16.24 k-ft	**0.033**	**PASS**

Detail Report: M9

Load Combination: Envelope **Code check:** 0.076 (LC 14)



Input Data

Shape:	HSS6X6X3	**I Node:**	N17
Member Type:	Beam	**J Node:**	N19
Length (in):	72	**I Release:**	Fixed
Material Type:	Hot Rolled Steel	**J Release:**	Fixed
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	96	**J Offset:**	N/A
Design Code:	AISC 14th (360-10): ASD	**T/C Only:**	Both Way

Material Properties

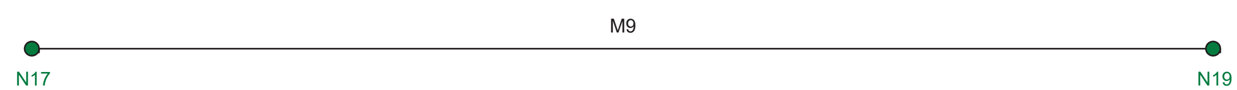
Material:	A500 Gr.46	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

Shape Properties

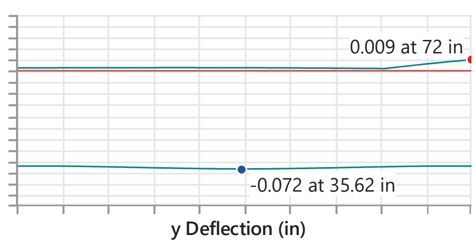
d (in):	6	**I_{yy} (in^4):**	22.3	**J (in^4):**	35
b_f (in):	6	**I_{zz} (in^4):**	22.3		
t (in):	0.174	**Area (in^2):**	3.98		

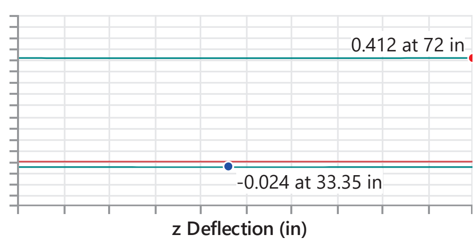
Design Properties

$L_{b\ y\text{-}y}$ (in):	72	**$K_{y\text{-}y}$:**	1	**Max Defl Ratio:**	L/10000
$L_{b\ z\text{-}z}$ (in):	72	**$K_{z\text{-}z}$:**	1	**Max Defl Location:**	0
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Span:**	N/A
$L_{comp\ bot}$ (in):	72	**z sway:**	No	**τ_b:**	1
L_{torque} (in):	72	**Function:**	Lateral		
C_b:	3	**Seismic DR:**	None		

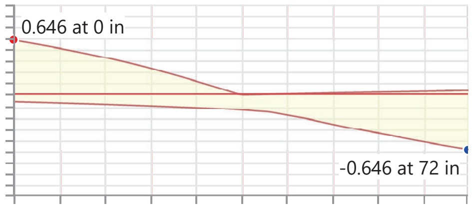


Diagrams:

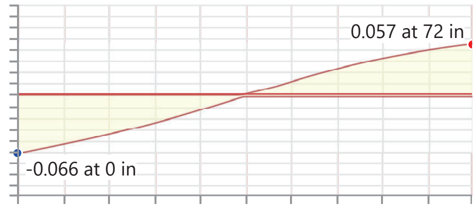




Axial Force (k)

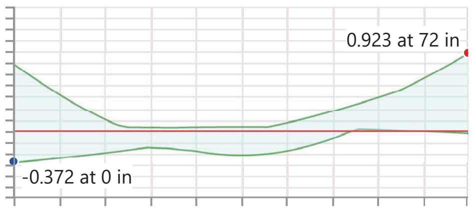


y Shear Force (k)

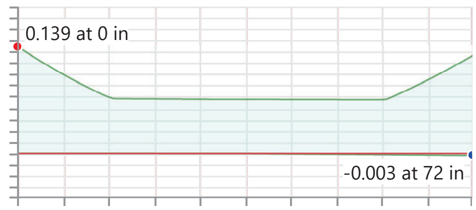


z Shear Force (k)

Torsion (k-ft)

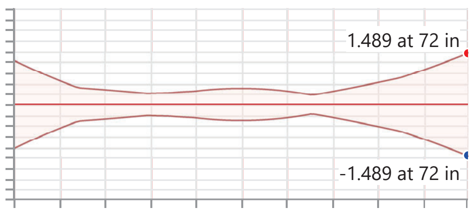


z-z Moment (k-ft)

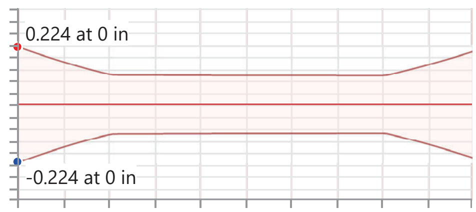


y-y Moment (k-ft)

Axial Stress (ksi)



Bending Strong Stress (ksi)



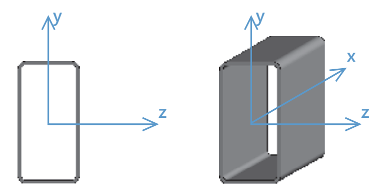
Bending Weak Stress (ksi)

AISC 14th (360-10): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	14	-	-	-	-
Applied Loading - Shear + Torsion	2	-	-	-	-
Axial Tension Analysis	14	0 k	109.629 k	-	-
Axial Compression Analysis	14	5.799 k	103.014 k	-	-
Flexural Analysis (Strong Axis)	14	0.784 k-ft	18.501 k-ft	-	-
Flexural Analysis (Weak Axis)	-	0.111 k-ft	18.501 k-ft	-	-
Shear Analysis (Major Axis y)	2	0.657 k	31.506 k	**0.021**	**PASS**
Shear Analysis (Minor Axis z)	2	0.012 k	31.506 k	**0.0003798**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	14	-	-	**0.076**	**PASS**
Torsional Analysis	14	0.009 k-ft	16.24 k-ft	**0.0005555**	**PASS**

Detail Report: M12

Load Combination: Envelope **Code check:** 0.930 (LC 14)



Input Data

Shape:	HSS6X3X3	**I Node:**	N13
Member Type:	Beam	**J Node:**	N15
Length (in):	264	**I Release:**	BenPIN
Material Type:	Hot Rolled Steel	**J Release:**	BenPIN
Design Rule:	Typical	**I Offset:**	N/A
Internal Sections:	96	**J Offset:**	N/A
Design Code:	AISC 14th (360-10): ASD	**T/C Only:**	Both Way

Material Properties

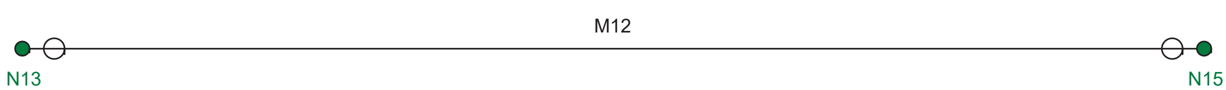
Material:	A500 Gr.46	**Therm. Coeff. (/1E5 F):**	0.65	**F_u (ksi):**	58
E (ksi):	29000	**Density (k/ft^3):**	0.49	**R_t:**	1.3
G (ksi):	11154	**F_y (ksi):**	46		
Nu:	0.3	**R_y:**	1.4		

Shape Properties

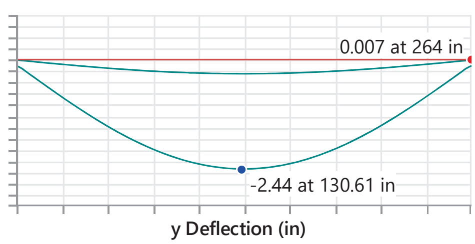
d (in):	6	**I_{yy} (in^4):**	4.55	**J (in^4):**	11.1
b_f (in):	3	**I_{zz} (in^4):**	13.4		
t (in):	0.174	**Area (in^2):**	2.93		

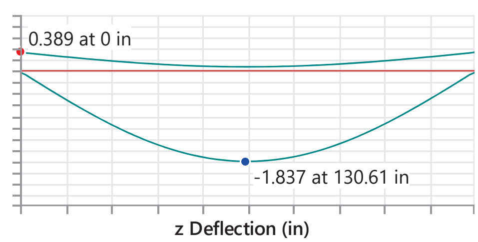
Design Properties

$L_{b\ y\text{-}y}$ (in):	264	**$K_{y\text{-}y}$:**	1	**Max Defl Ratio:**	L/113
$L_{b\ z\text{-}z}$ (in):	264	**$K_{z\text{-}z}$:**	1	**Max Defl Location:**	130.611
$L_{comp\ top}$:	Lbyy	**y sway:**	No	**Span:**	1
$L_{comp\ bot}$ (in):	264	**z sway:**	No	**τ_b:**	1
L_{torque} (in):	264	**Function:**	Lateral		
C_b:	1.163	**Seismic DR:**	None		

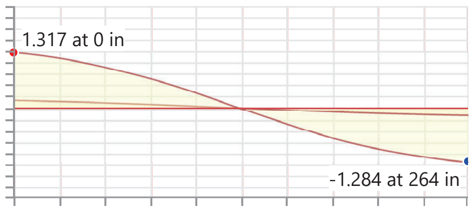


Diagrams:

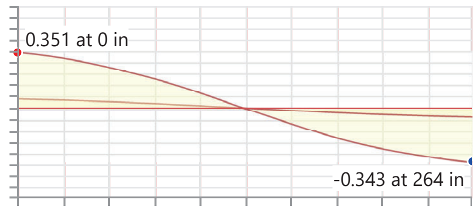




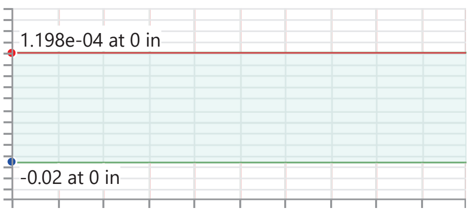
Axial Force (k)



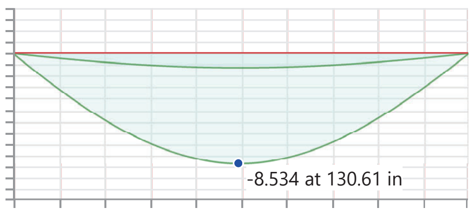
y Shear Force (k)



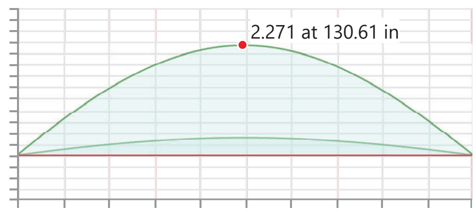
z Shear Force (k)



Torsion (k-ft)

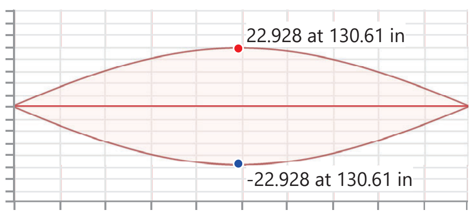


z-z Moment (k-ft)

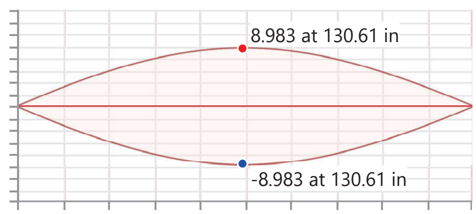


y-y Moment (k-ft)

Axial Stress (ksi)



Bending Strong Stress (ksi)



Bending Weak Stress (ksi)

AISC 14th (360-10): ASD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	14	-	-	-	-
Applied Loading - Shear + Torsion	14	-	-	-	-
Axial Tension Analysis	14	2.739 k	80.707 k	-	-
Axial Compression Analysis	14	0 k	9.813 k	-	-
Flexural Analysis (Strong Axis)	14	8.534 k-ft	12.831 k-ft	-	-
Flexural Analysis (Weak Axis)	-	1.854 k-ft	7.464 k-ft	-	-
Shear Analysis (Major Axis y)	14	1.319 k	31.506 k	**0.042**	**PASS**
Shear Analysis (Minor Axis z)	14	0.289 k	14.252 k	**0.02**	**PASS**
Bending & Axial Interaction Check (UC Bending Max)	14	-	-	**0.93**	**PASS**
Torsional Analysis	14	0.0005472 k-ft	7.863 k-ft	**6.959e-5**	**PASS**

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm. Coeff. [1e⁵°F⁻¹]	Density [k/ft³]	Yield [ksi]	Ry	Fu [ksi]	Rt
1	A36 Gr.36	29000	11154	0.3	0.65	0.49	36	1.5	58	1.2
2	A500 Gr.46	29000	11154	0.3	0.65	0.49	46	1.4	58	1.3

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rule	Area [in²]	Iyy [in⁴]	Izz [in⁴]	J [in⁴]
1	Post	HSS6X6X3	Column	Tube	A500 Gr.46	Typical	3.98	22.3	22.3	35
2	Rafter	HSS6X6X3	Beam	Tube	A500 Gr.46	Typical	3.98	22.3	22.3	35
3	RidgeBeam	HSS6X6X3	Beam	Tube	A500 Gr.46	Typical	3.98	22.3	22.3	35
4	EdgeBeam	HSS6X3X3	Beam	Tube	A500 Gr.46	Typical	2.93	4.55	13.4	11.1

Hot Rolled Steel Design Parameters

	Label	Shape	Length [in]	Lcomp top [in]	Channel Conn.	a [in]	Function
1	M1	Post	99.899	Lbyy	N/A	N/A	Lateral
2	M2	Post	99.899	Lbyy	N/A	N/A	Lateral
3	M3	Post	99.899	Lbyy	N/A	N/A	Lateral
4	M4	Post	99.899	Lbyy	N/A	N/A	Lateral
5	M5	Rafter	139.302	Lbyy	N/A	N/A	Lateral
6	M6	Rafter	139.302	Lbyy	N/A	N/A	Lateral
7	M7	Rafter	139.302	Lbyy	N/A	N/A	Lateral
8	M8	Rafter	139.302	Lbyy	N/A	N/A	Lateral
9	M9	RidgeBeam	72	Lbyy	N/A	N/A	Lateral
10	M10	EdgeBeam	192	Lbyy	N/A	N/A	Lateral
11	M11	EdgeBeam	264	Lbyy	N/A	N/A	Lateral
12	M12	EdgeBeam	264	Lbyy	N/A	N/A	Lateral
13	M13	EdgeBeam	192	Lbyy	N/A	N/A	Lateral

Node Loads and Enforced Displacements (BLC 7 : W Longitudinal)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/in, k*s²*in)]
1	N13	L	X	0.115
2	N14	L	X	0.115
3	N15	L	X	0.115
4	N16	L	X	0.115

Node Loads and Enforced Displacements (BLC 8 : W Minimum)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/in, k*s²*in)]
1	N13	L	Z	0.176
2	N14	L	Z	0.176
3	N15	L	Z	0.176
4	N16	L	Z	0.176

Node Loads and Enforced Displacements (BLC 9 : E Transverse)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/in, k*s²*in)]
1	N13	L	Z	0.158
2	N14	L	Z	0.158
3	N15	L	Z	0.158
4	N16	L	Z	0.158

Node Loads and Enforced Displacements (BLC 10 : E Longitudinal)

	Node Label	L, D, M	Direction	Magnitude [(k, k-ft), (in, rad), (k*s²/in, k*s²*in)]
1	N13	L	X	0.158
2	N14	L	X	0.158
3	N15	L	X	0.158
4	N16	L	X	0.158

Member Area Loads (BLC 1 : D)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N2	N17		Y	Perp to A-B	-5	-5	-5		Yes
2	N2	N4	N19	N17	Y	Perp to A-B	-5	-5	-5	-5	Yes
3	N4	N3	N19		Y	Perp to A-B	-5	-5	-5		Yes
4	N3	N1	N17	N19	Y	Perp to A-B	-5	-5	-5	-5	Yes

Member Area Loads (BLC 2 : Lr)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N2	N17		Y	Perp to A-B	-20	-20	-20		Yes
2	N2	N4	N19	N17	Y	Perp to A-B	-20	-20	-20	-20	Yes
3	N4	N3	N19		Y	Perp to A-B	-20	-20	-20		Yes
4	N3	N1	N17	N19	Y	Perp to A-B	-20	-20	-20	-20	Yes

Member Area Loads (BLC 3 : S balanced)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N2	N17		Y	Perp to A-B	-12.6	-12.6	-12.6		Yes
2	N2	N4	N19	N17	Y	Perp to A-B	-12.6	-12.6	-12.6	-12.6	Yes
3	N4	N3	N19		Y	Perp to A-B	-12.6	-12.6	-12.6		Yes
4	N3	N1	N17	N19	Y	Perp to A-B	-12.6	-12.6	-12.6	-12.6	Yes

Member Area Loads (BLC 4 : S unbalanced)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N5	N17		Y	Perp to A-B	0	0	0		Yes
2	N3	N1	N17	N19	Y	Perp to A-B	0	0	0	0	Yes
3	N6	N3	N19		Y	Perp to A-B	0	0	0		Yes
4	N5	N2	N17		Y	Perp to A-B	-15	-15	-15		Yes
5	N2	N4	N19	N17	Y	Perp to A-B	-15	-15	-15	-15	Yes
6	N4	N6	N19		Y	Perp to A-B	-15	-15	-15		Yes

Member Area Loads (BLC 5 : W Transverse case A)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N5	N17		Perp	Perp to A-B	-23.3	-23.3	-23.3		Yes
2	N3	N1	N17	N19	Perp	Perp to A-B	-23.3	-23.3	-23.3	-23.3	Yes
3	N6	N3	N19		Perp	Perp to A-B	-23.3	-23.3	-23.3		Yes
4	N5	N2	N17		Perp	Perp to A-B	4.2	4.2	4.2		Yes
5	N2	N4	N19	N17	Perp	Perp to A-B	4.2	4.2	4.2	4.2	Yes
6	N4	N6	N19		Perp	Perp to A-B	4.2	4.2	4.2		Yes

Member Area Loads (BLC 6 : W Transverse case B)

	Node A	Node B	Node C	Node D	Direction	Load Direction	A Magnitude [psf]	B Magnitude [psf]	C Magnitude [psf]	D Magnitude [psf]	Exclude Braces
1	N1	N5	N17		Perp	Perp to A-B	-0.4	-0.4	-0.4		Yes
2	N3	N1	N17	N19	Perp	Perp to A-B	-0.4	-0.4	-0.4	-0.4	Yes
3	N6	N3	N19		Perp	Perp to A-B	-0.4	-0.4	-0.4		Yes
4	N5	N2	N17		Perp	Perp to A-B	20.7	20.7	20.7		Yes
5	N2	N4	N19	N17	Perp	Perp to A-B	20.7	20.7	20.7	20.7	Yes
6	N4	N6	N19		Perp	Perp to A-B	20.7	20.7	20.7		Yes

Envelope Node Reactions

	Node Label		X [k]	LC	Y [k]	LC	Z [k]	LC	MX [k-ft]	LC	MY [k-ft]	LC	MZ [k-ft]	LC
1	N9	max	0.077	2	3.416	14	0.078	2	0	35	0	35	0	35
2		min	-0.096	35	0.19	31	-0.151	30	0	1	0	1	0	1
3	N10	max	0.13	14	3.315	2	0.024	35	0	35	0	35	0	35
4		min	-0.096	35	-0.25	31	-0.222	8	0	1	0	1	0	1
5	N11	max	0.049	31	3.41	14	0.086	16	0	35	0	35	0	35
6		min	-0.135	13	0.192	31	-0.148	30	0	1	0	1	0	1
7	N12	max	-0.024	1	3.322	2	-0.025	1	0	35	0	35	0	35
8		min	-0.134	13	-0.249	31	-0.223	8	0	1	0	1	0	1
9	Totals:	max	0	7	13.275	2	0	4						
10		min	-0.442	13	-0.117	31	-0.724	8						

Envelope Maximum Member Section Forces

	Member		Axial[k]	Loc[in]	LC	y Shear[k]	Loc[in]	LC	z Shear[k]	Loc[in]	LC	Torque[k-ft]	Loc[in]	LC	y-y Moment[k-ft]	Loc[in]	LC	z-z Moment[k-ft]	Loc[in]	LC
1	M1	max	3.416	0	14	0.141	99.899	30	0.054	99.899	13	0	99.899	35	0.448	99.899	13	0.917	99.899	2
2		min	0.122	99.899	31	-0.11	0	2	-0.087	0	8	0	0	1	-0.723	99.899	8	-1.17	99.899	30
3	M2	max	3.315	0	2	0.087	99.899	35	0.009	99.899	4	0	99.899	35	0.072	99.899	4	2.116	99.899	14
4		min	-0.317	99.899	31	-0.254	0	14	-0.088	0	8	0	0	1	-0.73	99.899	8	-0.72	99.899	35
5	M3	max	3.41	0	14	0.157	99.899	27	0.01	99.899	4	0	99.899	35	0.083	99.899	4	1.154	99.899	30
6		min	0.125	99.899	31	-0.139	0	30	-0.086	0	8	0	0	1	-0.714	99.899	8	-1.304	99.899	27
7	M4	max	3.322	0	2	0.253	99.899	14	0.052	99.899	13	0	99.899	35	0.434	99.899	13	0	0	35
8		min	-0.317	99.899	31	0.035	0	1	-0.089	0	8	0	0	1	-0.738	99.899	8	-2.106	99.899	14
9	M5	max	4.434	0	2	0.41	0	2	0.008	11.731	8	0.672	139.302	8	0.097	0	13	1.089	139.302	14
10		min	-0.303	139.302	31	-0.893	139.302	14	-0.004	0	35	-0.423	0	13	-0.154	0	8	-1.323	39.591	8
11	M6	max	4.438	0	2	0.483	0	14	0.011	139.302	31	0.678	139.302	8	0.018	0	4	2.143	0	14
12		min	-0.093	139.302	31	-0.634	139.302	2	-0.002	0	2	-0.077	0	4	-0.156	0	8	-0.909	68.918	29
13	M7	max	4.438	0	2	0.418	0	2	-0.001	139.302	1	0.086	139.302	4	0.152	0	8	1.315	0	27
14		min	-0.301	139.302	31	-0.918	139.302	14	-0.015	139.302	8	-0.663	0	8	-0.044	139.302	4	-1.35	42.524	8
15	M8	max	4.434	0	2	0.479	0	14	0.005	139.302	13	0.409	139.302	13	0.158	0	8	2.132	0	14
16		min	-0.091	139.302	31	-0.626	139.302	2	-0.005	11.731	31	-0.687	0	8	-0.094	0	13	-0.735	83.581	2
17	M9	max	5.981	35.621	2	0.646	0	2	0.057	72	30	0.011	35.621	22	0.139	0	8	0.923	72	16
18		min	-0.277	36.379	31	-0.646	72	2	-0.066	0	8	-0.006	0	31	-0.003	72	2	-0.372	0	35
19	M10	max	0.031	192	31	0.704	0	14	0.215	192	2	0.048	192	8	0	192	35	0.327	129.347	31
20		min	-2.915	0	2	-0.661	192	2	-0.211	0	2	0	0	32	-1.009	97.011	2	-3.107	97.011	2
21	M11	max	0.021	264	31	1.055	0	2	0.351	264	2	0.02	264	13	0	264	35	2.237	133.389	31
22		min	-2.913	0	2	-1.082	264	2	-0.343	0	2	-0.001	0	4	-2.271	133.389	2	-6.989	133.389	2
23	M12	max	0.094	264	31	1.317	0	14	0.351	0	2	0	264	4	2.271	130.611	2	0	264	35
24		min	-2.913	0	2	-1.284	264	14	-0.343	264	2	-0.02	0	13	0	0	1	-8.534	130.611	14
25	M13	max	0.03	192	31	0.716	0	14	0.215	0	2	0	192	2	1.009	94.989	2	0.309	129.347	31
26		min	-2.915	0	2	-0.65	192	2	-0.211	192	2	-0.047	0	8	0	0	1	-3.107	94.989	2

Envelope AISC 14TH (360-10): ASD Member Steel Code Checks

	Member	Shape	Code Check	Loc[in]	LC	Shear Check	Loc[in]	Dir	LC	Pnc/om [k]	Pnt/om [k]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	M1	HSS6X6X3	0.109	99.899	30	0.004	99.899	y	30	97.25	109.629	18.501	18.501	1.678	H1-1b
2	M2	HSS6X6X3	0.16	99.899	14	0.008	99.899	y	14	97.25	109.629	18.501	18.501	1.678	H1-1b
3	M3	HSS6X6X3	0.108	99.899	30	0.005	99.899	y	27	97.25	109.629	18.501	18.501	1.678	H1-1b
4	M4	HSS6X6X3	0.16	99.899	14	0.008	99.899	y	14	97.25	109.629	18.501	18.501	1.678	H1-1b
5	M5	HSS6X6X3	0.099	63.052	14	0.063	139.302	y	8	86.845	109.629	18.501	18.501	1.181	H1-1b
6	M6	HSS6X6X3	0.148	0	14	0.052	0	y	8	86.845	109.629	18.501	18.501	2.896	H1-1b
7	M7	HSS6X6X3	0.101	63.052	14	0.063	139.302	y	8	86.845	109.629	18.501	18.501	1.178	H1-1b
8	M8	HSS6X6X3	0.147	0	14	0.052	0	y	8	86.845	109.629	18.501	18.501	2.875	H1-1b
9	M9	HSS6X6X3	0.076	0	14	0.021	72	y	2	103.014	109.629	18.501	18.501	3	H1-1b
10	M10	HSS6X3X3	0.395	97.011	2	0.027	0	y	14	18.552	80.707	7.464	12.831	1.165	H1-1b
11	M11	HSS6X3X3	0.867	133.389	2	0.034	264	y	2	9.813	80.707	7.464	12.831	1.163	H1-1b
12	M12	HSS6X3X3	0.93	130.611	14	0.042	0	y	14	9.813	80.707	7.464	12.831	1.163	H1-1b
13	M13	HSS6X3X3	0.395	94.989	2	0.027	0	y	14	18.552	80.707	7.464	12.831	1.164	H1-1b