

Your Project Calculations



Project Name: Siakotos Pole mount

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Siakotos%20Pole%20mount&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2024

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=AAAE8XklSAqiKSTDg0VzXYpbqYEXdleUfAcz6rqfE01807Yu38IFwjAn3iiNKYgl

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-XD-57-L-4Hx3W-STRUTS-GIK7
Duty Classification:	XD
Module Width:	44.60 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	50
Front Edge Clearance:	4
Total Array Height at Tilt:	15.45 ft
Total Frame Length:	17.00 ft
Frame Weight:	936 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	17.20 ft
Rail Length:	180.40 in
Rail Spacing:	2.82 ft
Rail Check:	Not Checked

Support Specifications

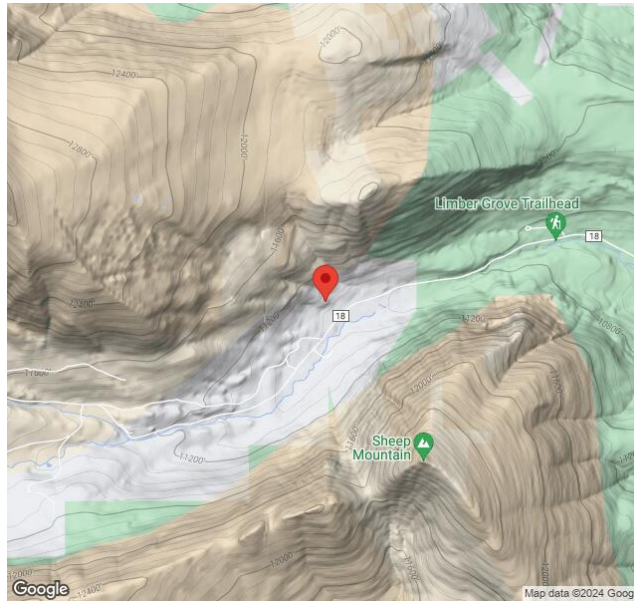
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.76 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft
Foundation Volume:	3.407 y ³
Foundation Result:	PASSED
Mount Twist:	0.001124 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	8500 Co Rd 18, Fairplay, CO 80440, USA
Wind Speed:	111 mph
Snow Load:	277 psf
Design Uplift Pressure:	0.015831 ksf
Design Downforce Pressure:	-0.015831 ksf
Design Snow Pressure:	0.060920 ksf



Design Disclaimer

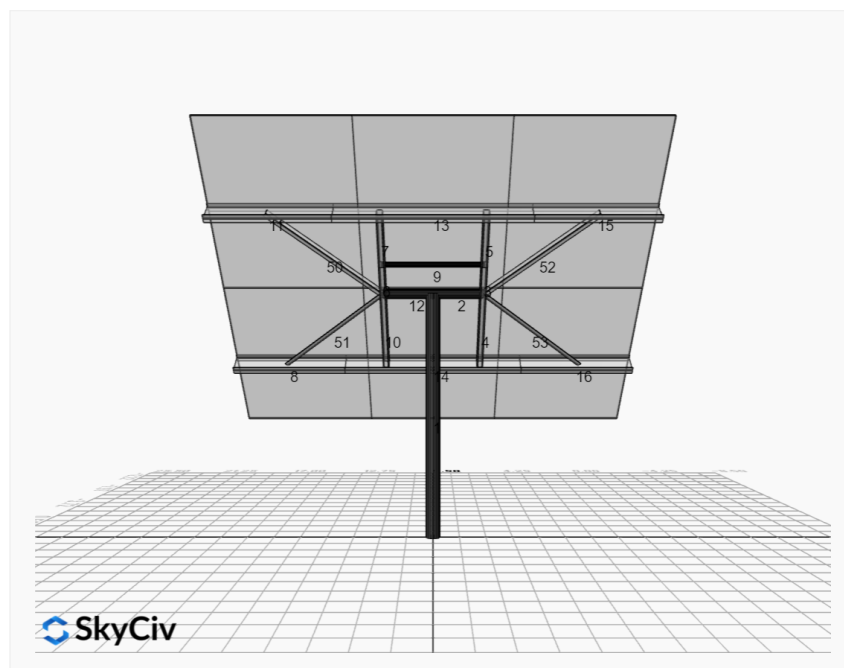
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

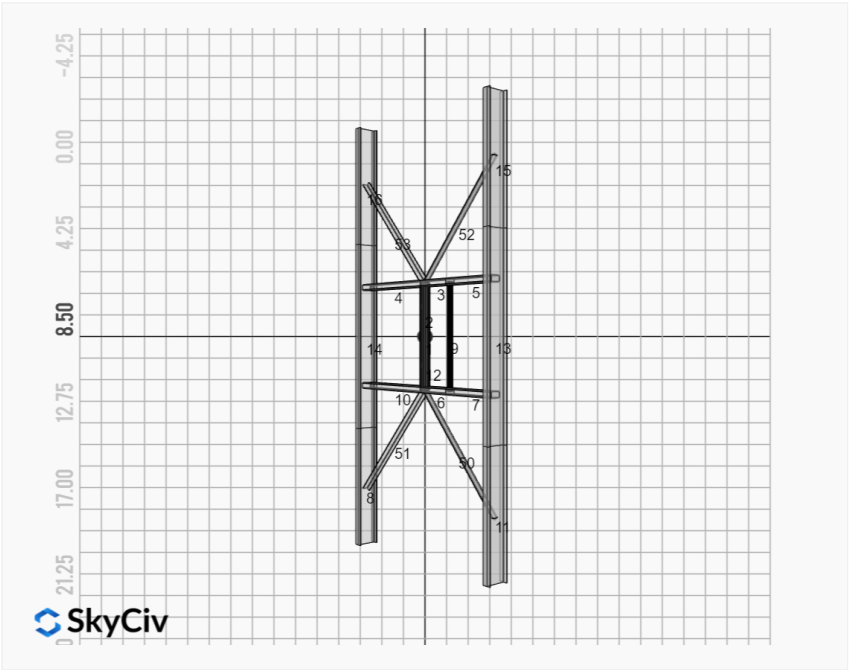
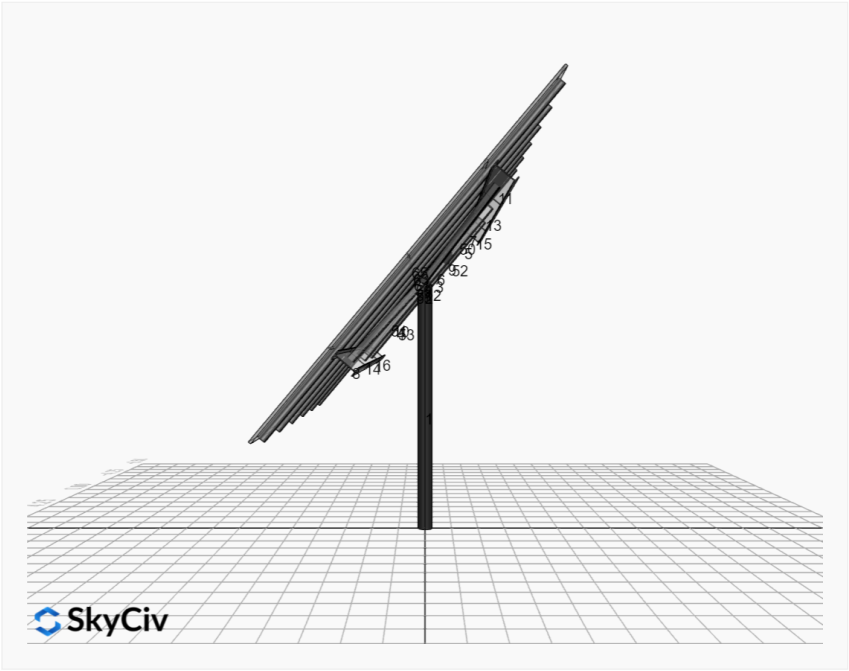
AutoDesigner Input

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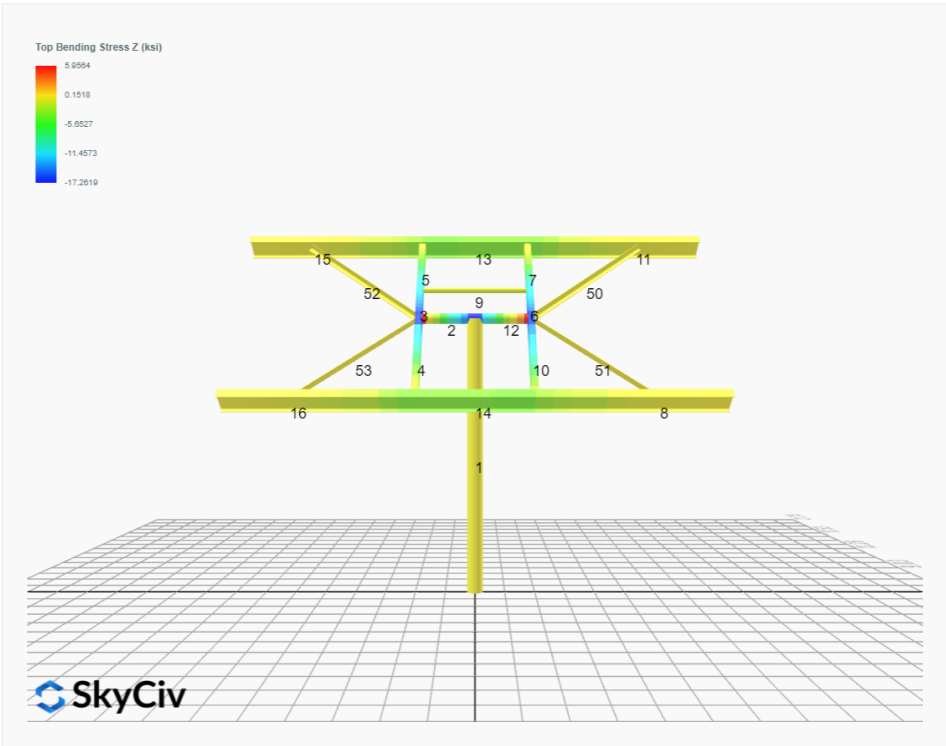
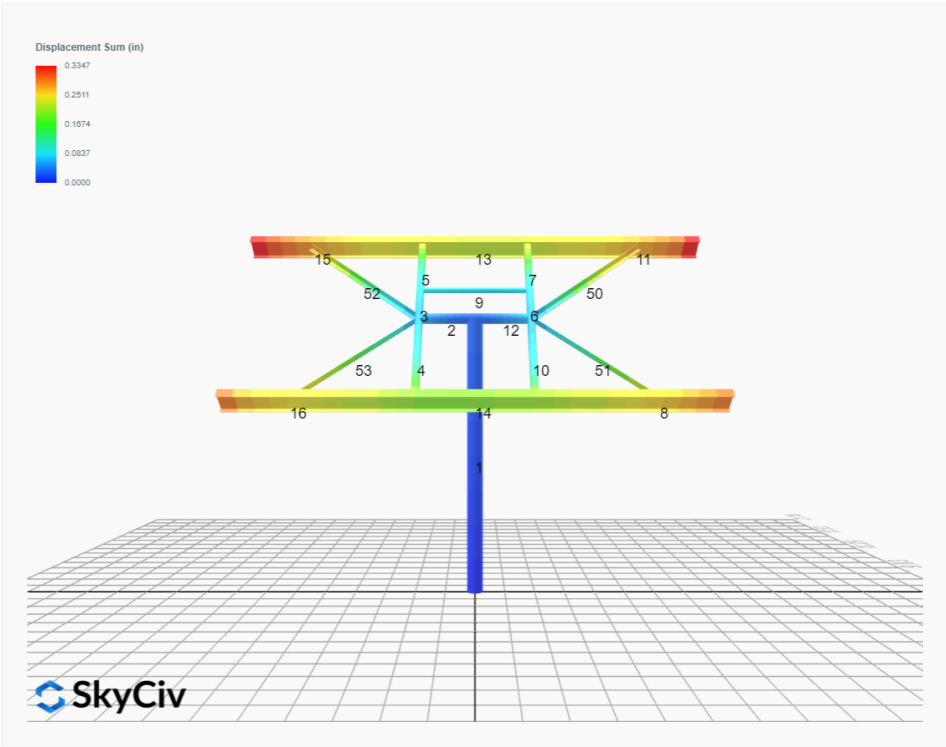
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigned are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only

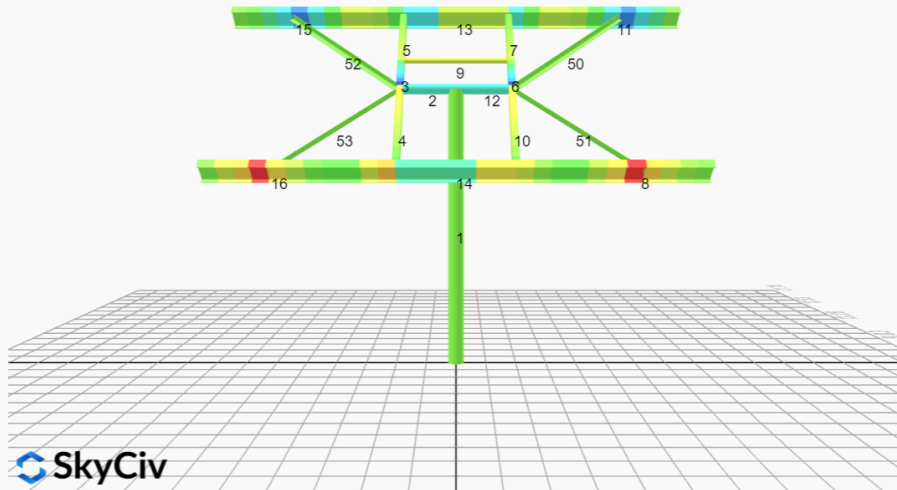




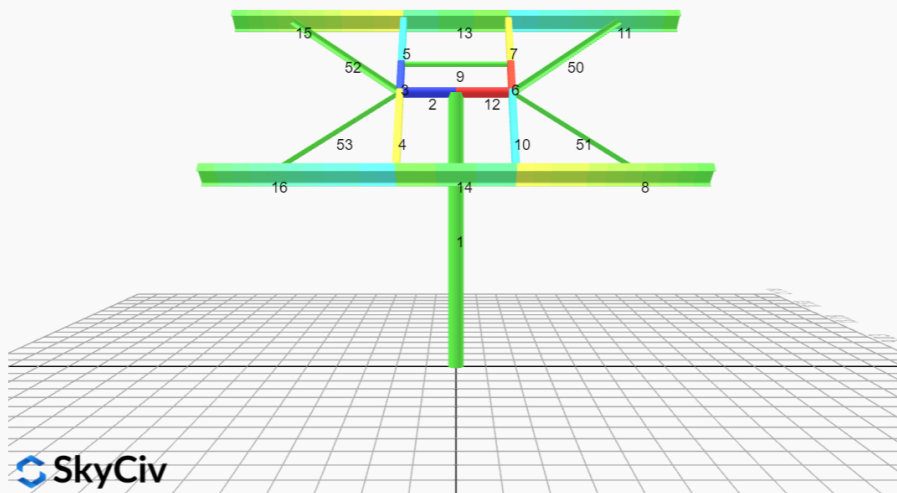
FEM Results (Envelope Worst Case for each member)

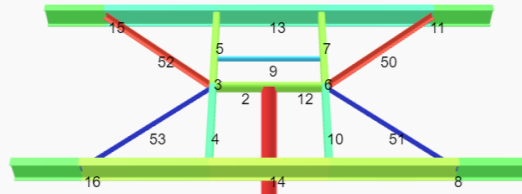
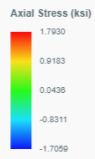


Top Bending Stress Y (ksi)



Shear Stress Y (ksi)





Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 2. D + L	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 3. D + (S or Lr or R)	0.0000	12.1582	0.0000	-0.0000	-0.0000	0.1569
ULS: 3. D + (S or Lr or R)	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	9.6563	0.0000	-0.0000	-0.0000	0.1234
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 5b. D + 0.7E	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	9.6563	0.0000	-0.0000	-0.0000	0.1234
ULS: 8. 0.6D + 0.7E	0.0000	1.2903	0.0000	-0.0000	-0.0000	0.0138
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8815	3.7293	0.0000	-0.0000	-0.0000	18.6578
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8815	0.5718	0.0000	-0.0000	-0.0000	-18.0705
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4111	10.8403	0.0000	-0.0000	-0.0000	14.0995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	9.6563	0.0000	-0.0000	-0.0000	0.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4111	8.4722	0.0000	-0.0000	-0.0000	-13.4468
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	9.6563	0.0000	-0.0000	-0.0000	0.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4111	3.3346	0.0000	-0.0000	-0.0000	13.9991
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4111	0.9665	0.0000	-0.0000	-0.0000	-13.5472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.1506	0.0000	-0.0000	-0.0000	0.0230
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8815	2.8691	0.0000	-0.0000	-0.0000	18.6486
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.2903	0.0000	-0.0000	-0.0000	0.0138
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8815	-0.2884	0.0000	-0.0000	-0.0000	-18.0797
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.2903	0.0000	-0.0000	-0.0000	0.0138

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	19.9085
Shear X	-3.1358
Shear Z	0.0000
Moment X	-0.0016
Moment Y (Twist)	0.0011
Moment Z	32.6364

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	12.1582
Shear X	-1.8815
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	18.6578

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States

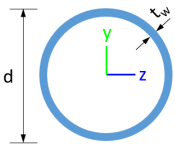
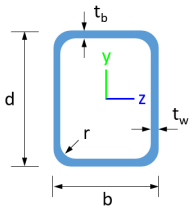
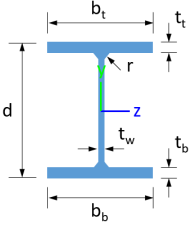
User Name: sales@mtsolar.us
 Project Name: Siakotos Pole mount
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

Section Dimensions								
								
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
7	6in Pipe Sch 40	6.63	0.28					
								
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		
								
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D
1	7	20.49	20.49	9.76	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.22,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1
4	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.77,1.67,1.67,1.69,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.69,1.62,1.69,1.67,1.69,1.66,1.69	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.73,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.64,1.68,1.67,1.68,1.66,1.68	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.22,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.73,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.64,1.68,1.67,1.68,1.66,1.68	300	200	1
8	20	9.97	9.97	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.33,2.32,2.16,2.32,2.32,2.34,2.31,2.34,2.32,2.32,2.31,2.32,2.33,2.34,2.30,2.34,2.32,2.34,2.32,2.34	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.77,1.67,1.67,1.69,1.66,1.69,1.67,1.67,1.67,1.67,1.68,1.69,1.62,1.69,1.67,1.69,1.66,1.69	300	200	1
11	20	9.97	9.97	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.33,2.32,2.15,2.32,2.33,2.34,2.32,2.34,2.33,2.32,2.31,2.32,2.33,2.34,2.30,2.34,2.32,2.34,2.32,2.34	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
14	20	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
15	20	9.97	9.97	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.33,2.32,2.15,2.32,2.33,2.34,2.32,2.34,2.33,2.32,2.31,2.32,2.33,2.34,2.30,2.34,2.32,2.34,2.32,2.34	300	200	1
16	20	9.97	9.97	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.33,2.32,2.16,2.32,2.32,2.34,2.31,2.34,2.32,2.32,2.31,2.32,2.33,2.34,2.30,2.34,2.32,2.34,2.32,2.34	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	104.50	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	34.37	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53

10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	34.37	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.42	6.46	56.26	44.91
14	159.30	97.43	31.41	6.46	56.26	44.91
15	159.30	34.37	46.90	6.46	56.26	44.91
16	159.30	34.37	46.90	6.46	56.26	44.91

Design Ratio

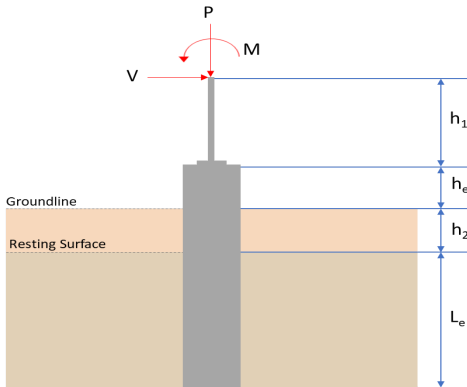
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.191	0.772	0.000	0.042	0.000	0.820	#13	0.548	Not Required	Pass
2	0.010	0.545	0.135	0.131	0.021	0.683	#21	0.054	Not Required	Pass
3	0.011	0.653	0.252	0.065	0.102	0.911	#21	0.046	Not Required	Pass
4	0.011	0.641	0.102	0.064	0.016	0.748	#21	0.082	Not Required	Pass
5	0.010	0.406	0.040	0.064	0.010	0.451	#21	0.076	Not Required	Pass
6	0.011	0.653	0.252	0.065	0.102	0.911	#21	0.046	Not Required	Pass
7	0.010	0.406	0.040	0.064	0.010	0.451	#21	0.076	Not Required	Pass
8	0.061	0.097	0.159	0.034	0.021	0.211	#21	0.763	Not Required	Pass
9	0.043	0.041	0.062	0.001	0.000	0.124	#21	0.137	Not Required	Pass
10	0.011	0.641	0.102	0.064	0.016	0.748	#21	0.082	Not Required	Pass
11	0.014	0.099	0.159	0.035	0.022	0.187	#21	0.508	Not Required	Pass
12	0.010	0.545	0.135	0.131	0.021	0.683	#21	0.054	Not Required	Pass
13	0.014	0.294	0.087	0.047	0.018	0.387	#21	0.204	Not Required	Pass
14	0.026	0.294	0.102	0.047	0.018	0.385	#21	0.306	Not Required	Pass
15	0.014	0.099	0.159	0.035	0.022	0.187	#21	0.508	Not Required	Pass
16	0.061	0.097	0.159	0.034	0.021	0.211	#21	0.508	Not Required	Pass

Definitions

Φ _t	Safety factor for tensile
Φ _c	Safety factor for compression
Φ _b	Safety factor for flexure
Φ _v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress
F _u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I _{yp}	Moment of inertia about the Y axes
I _{zp}	Moment of inertia about the Z axes
I _w	Warping constant
S _{yp}	Plastic section modulus about the Y axis
S _{zp}	Plastic section modulus about the Z axis
KL	Effective length
C _b	Buckling modification factor (from all load combinations)
L _b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P _n	Nominal axial strength (tension/compression)
M _n	Nominal flexural strength (about Z/Y axis)
V _n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M _z	Design ratio in case of bending about Z axis
M _y	Design ratio in case of bending about Y axis
V _y	Design ratio in case of shear along Y axis
V _z	Design ratio in case of shear along Z axis
(P,M _z ,M _y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided

ON
NG

Capacity is provided
Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 5.75 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>12.158</td><td>19.908</td></tr><tr><td>Vx (kip)</td><td>-1.881</td><td>-3.136</td></tr><tr><td>Vz (kip)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mx (kipft)</td><td>0.000</td><td>-0.002</td></tr><tr><td>Mz (kipft)</td><td>18.658</td><td>32.636</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	12.158	19.908	Vx (kip)	-1.881	-3.136	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	-0.002	Mz (kipft)	18.658	32.636	
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	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.881 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.29952 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(18.658 \text{ kipft}) + ((-1.881 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.971 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 5.2364 \text{ ft}$ - Required depth in x-direction, Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.2364 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.236 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.236 \text{ ft})}{(5.75 \text{ ft})}$ $Ratio = 0.91061$	Status: PASS Ratio: 0.910
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b \times D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_v}{A}$ $q = \frac{(12.158 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.75987 \text{ kip/ft}^2$ Check bearing capacity ratio: <i>Ratio</i> - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.75987 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.37994$	Status: PASS Ratio: 0.380
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.29952$ kip/ft - Lateral force per length of pile,

$M_o = 2.971$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

$$a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$$

$$a = \frac{(4 \times (2.971 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.29952 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.971 \text{ kipft/ft})) + (4 \times (-0.29952 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9669 \text{ ft}$$

p - Earth pressure against the pile at distance $a/2$ from resting surface,

$$p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{0.75 \times [(4 \times (2.971 \text{ kipft/ft})) + (3 \times (-0.29952 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (2.971 \text{ kipft/ft})) + (2 \times (-0.29952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18717 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{6 \times [(2 \times (2.971 \text{ kipft/ft})) + ((-0.29952 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.76578 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

p_a - Allowable lateral soil pressure at depth $a/2$,

$$p_a = R \frac{a}{2}$$

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9669 \text{ ft})}{2}$$

$$p_a = 0.29752 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

$$\text{Ratio} = \frac{p}{p_a}$$

$$\text{Ratio} = \frac{(0.18717 \text{ kip/ft}^2)}{(0.29752 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.62912$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

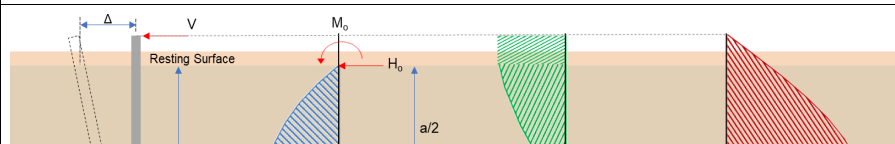
$$\text{Ratio} = \frac{s}{p_s}$$

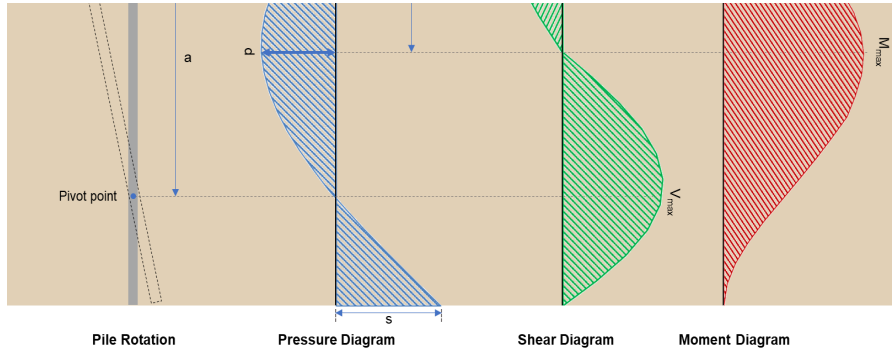
$$\text{Ratio} = \frac{(0.76578 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88786$$

Status: **PASS**
Ratio: **0.630**

Status: **PASS**
Ratio: **0.890**





Shear force and Bending moment (x-direction, LRFD)

H_o - Lateral force per length of pile,

$$H_o = \frac{V_x}{1.57 D}$$

$$H_o = \frac{(-3.136 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.49936 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_x H)}{1.57 D}$$

$$M_o = \frac{(32.636 \text{ kipft}) + ((-3.136 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1968 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

$$E = \frac{M_o}{H_o}$$

$$E = \frac{(5.1968 \text{ kipft/ft})}{(-0.49936 \text{ kip/ft})}$$

$$E = 10.407 \text{ ft}$$

a - Distance from resting surface to pivot point,

$$a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$$

$$a = \frac{(4 \times (5.1968 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.49936 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.1968 \text{ kipft/ft})) + (4 \times (-0.49936 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9623 \text{ ft}$$

V_{max} - Max shear force located at depth a ,

$$V_{max} = (H_o D) \left[1 - \left[3 \left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{L_e} \right)^2 \right] + \left[4 \left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{L_e} \right)^3 \right] \right]$$

$$V_{max} = ((-0.49936 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.407 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9623 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.407 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9623 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.7148 \text{ kip}$$

M_{max} - Max bending moment located at depth $a/2$,

$$M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$$

$$M_{max} = ((-0.49936 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(10.407 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9623 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.407 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9623 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.407 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9623 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 21.137 \text{ kipft}$	
		Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.00031847 \text{ kipft/ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 3.8333 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$ $V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(5.75 \text{ ft})} \right) \times \left(\frac{(3.8333 \text{ ft})}{(5.75 \text{ ft})} - 1 \right) \times \left(\frac{(3.8333 \text{ ft})}{(5.75 \text{ ft})} \right)^2$ $V_{max} = 0.00039386 \text{ kip}$ M_{max} - Max bending moment at depth $a/2$, $M_{max} = (M_o b) \left[1 - \left(4 \frac{a}{2 L_e} \right)^3 \right] + \left(3 \frac{a}{2 L_e} \right)^4$ $M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(3.8333 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left(3 \times \frac{(3.8333 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4$ $M_{max} = 0.0011323 \text{ kipft}$	
Table 22.4.2.1	22.4.2.2, 10.6.1.1	Minimum Reinforcement Check (LRFD) Parameters: $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength, $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength, $\phi = 0.65$ - Reduction factor for axial strength, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(19.908 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -83.935 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-83.935 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ $n_{s,min}$ - Required number of reinforcement	

	<i>n_{rebar}</i> - Required number of reinforcement bars, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <i>A_{st}</i> - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <i>Ratio</i> - Capacity $Ratio = \frac{A_{min}}{A_{st}}$ $Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $Ratio = 0.96556$	Status: PASS Ratio: 0.970
25.2.3	<i>s_{rebar}</i> - Minimum spacing of reinforcement, $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) 25.7.2.1 <i>s_{ties}</i> - Maximum spacing of ties, $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) <i>φP_N</i> - Allowable axial compressive strength $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <i>Ratio</i> - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(19.908 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0074417$	Status: PASS Ratio: 0.010
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: <i>b_w</i> = 48 in - Effective width, <i>d</i> - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$	

		$d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor	$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 19.908 \text{ kip} \rightarrow 19908 \text{ lbf}$, $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(19908 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 121.14 \text{ kip}$	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$	
	V_c - Governing shear strength of concrete	$V_c = MIN[V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN[(296.21 \text{ kip}), (121.14 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 121.14 \text{ kip}$	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$	
	A_v - Ties rebar area,	$A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b)	$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$	
	V_s - Governing shear strength of steel	$V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$	

22.5.1.1	$V_s = 50.894 \text{ kip}$ $V_s = 50.894 \text{ kip}$ $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((121.14 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.82 \text{ kip}$ Considering x-direction: $V_{max} = 7.7148 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.7148 \text{ kip})}{(111.82 \text{ kip})}$ $Ratio = 0.068992$	Status: PASS Ratio: 0.070
14.5.2.1b	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 21.137 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(21.137 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.084682$	Status: PASS Ratio: 0.080
	Considering z-direction: $M_{max} = 0.0011323 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$	

	ψ_{max}	
	$Ratio = \frac{(0.0011323 \text{ kipft})}{(249.6 \text{ kipft})}$	
	$Ratio = 4.5366 \times 10^{-6}$	
		Status: PASS Ratio: 0.000