

Your Project Calculations



Project Name: Steve Ballen

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Steve%20Ballen&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=JWYqDjmCXWalRmIK6BRVi3W47NI4Ha8r3gHZYknANqLigMemiQ1IzNYQpxom8RJW

Array Specification

Product:	Beam
Unique ID:	2P-19.75-6TOP-SD-45-L-3Hx6W-HL1A
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	69.00in
Number of Rows:	3
Number of Columns:	6
Total Number of Modules:	18
Desired Tilt Angle:	65
Front Edge Clearance:	3
Total Array Height at Tilt:	12.12 ft
Total Frame Length:	34.75 ft
Frame Weight:	1556 lbs
Array Dimensions N/S:	10.13 ft
Array Dimensions E/W:	35.00 ft
Rail Length:	121.50 in
Rail Spacing:	2.88 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	7.59 ft
Number of Poles:	2
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.75 ft Pile 2: 4.75 ft
Foundation Volume:	5.630 y ³
Foundation Result:	PASSED
Mount Twist:	0.313884 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Levin Dr, Wisconsin 54459, USA
Wind Speed:	100 mph
Snow Load:	60 psf
Design Uplift Pressure:	0.015686 ksf
Design Downforce Pressure:	-0.015686 ksf
Design Snow Pressure:	0.003299 ksf



Design Disclaimer

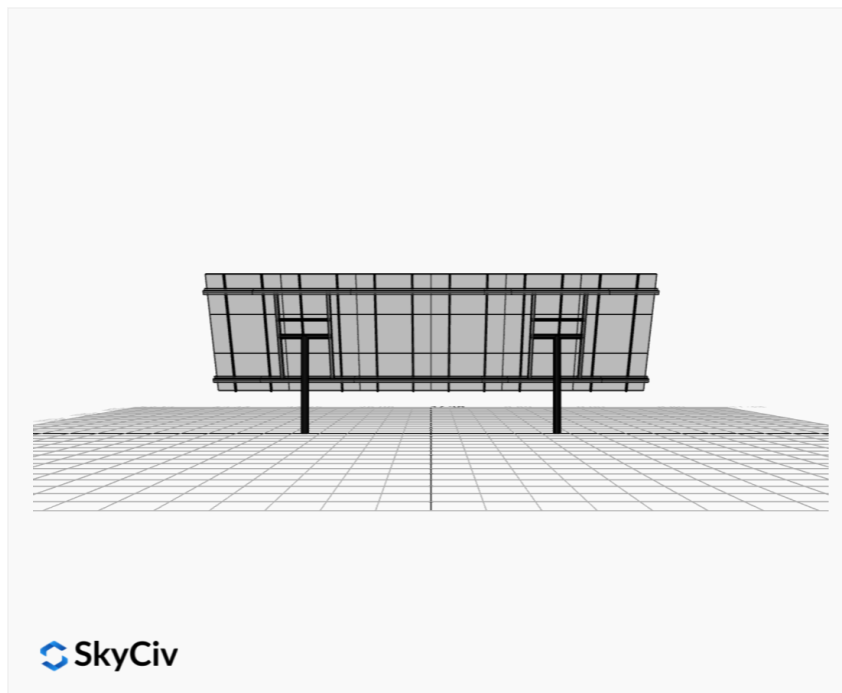
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

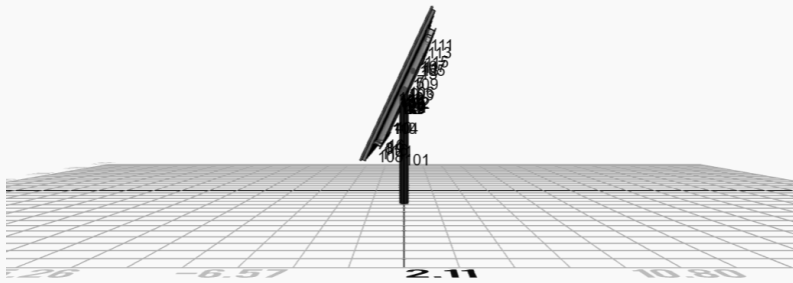
AutoDesigner Input

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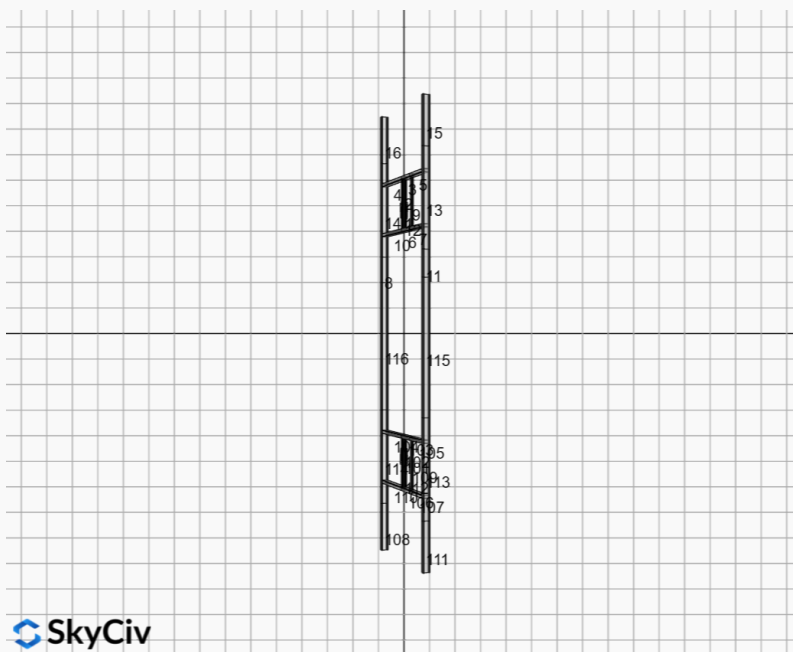
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

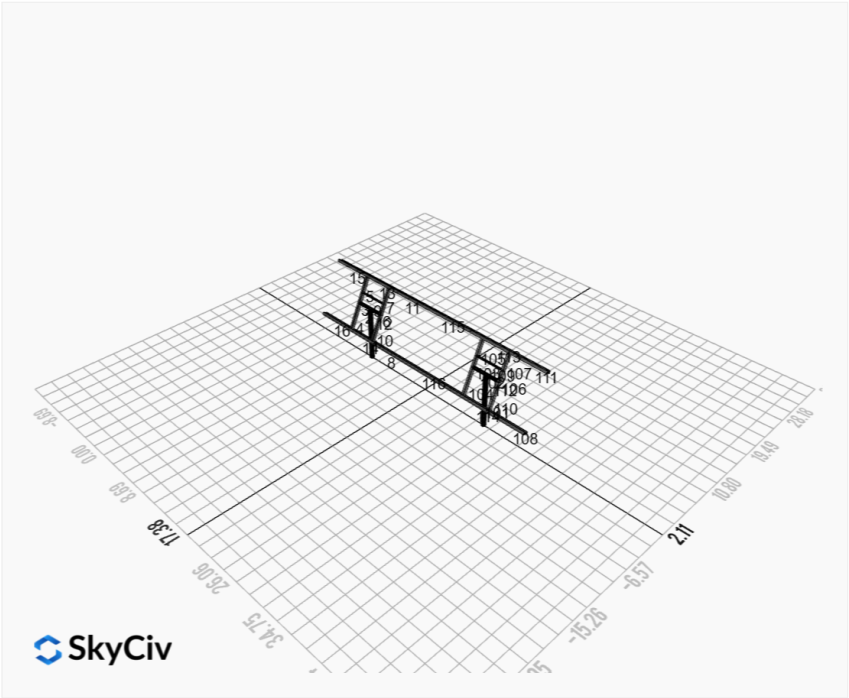




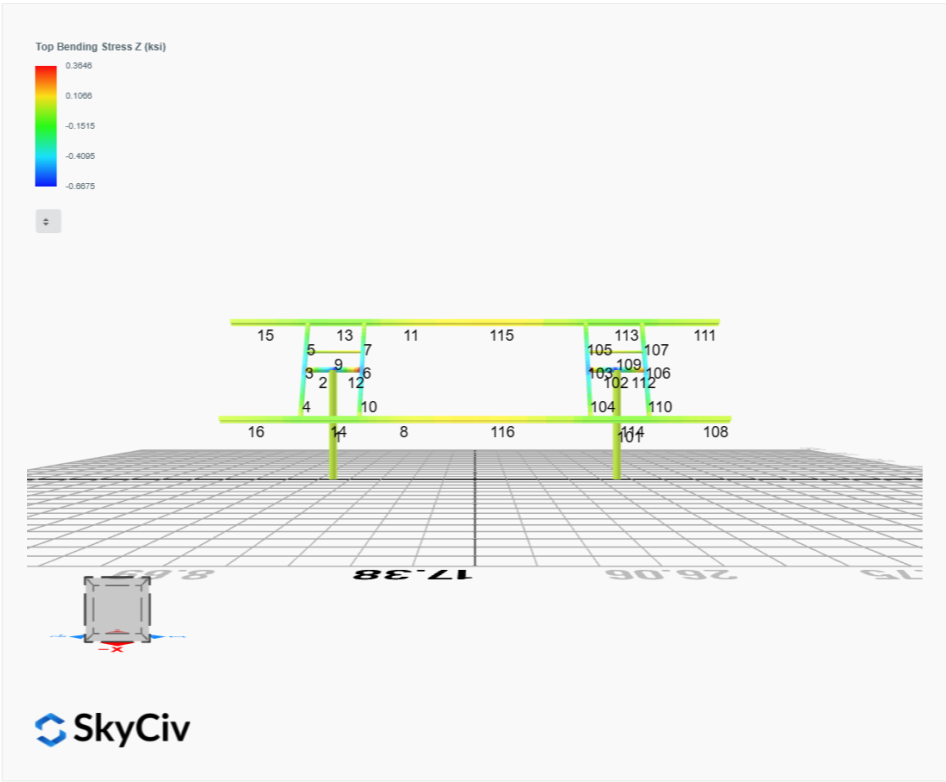
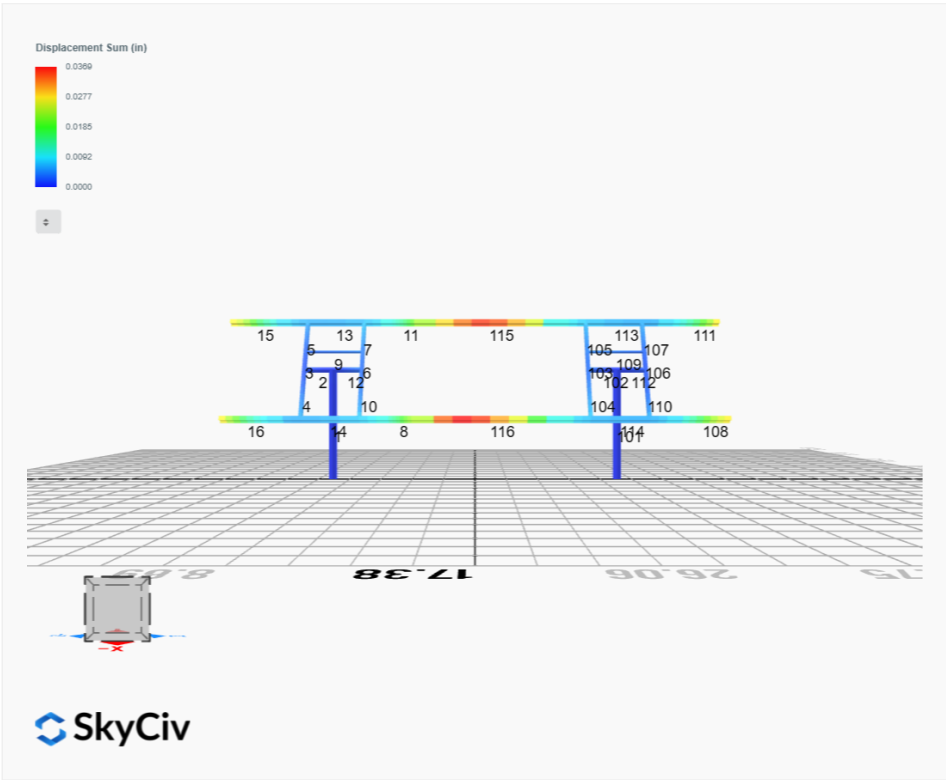
 SkyCiv

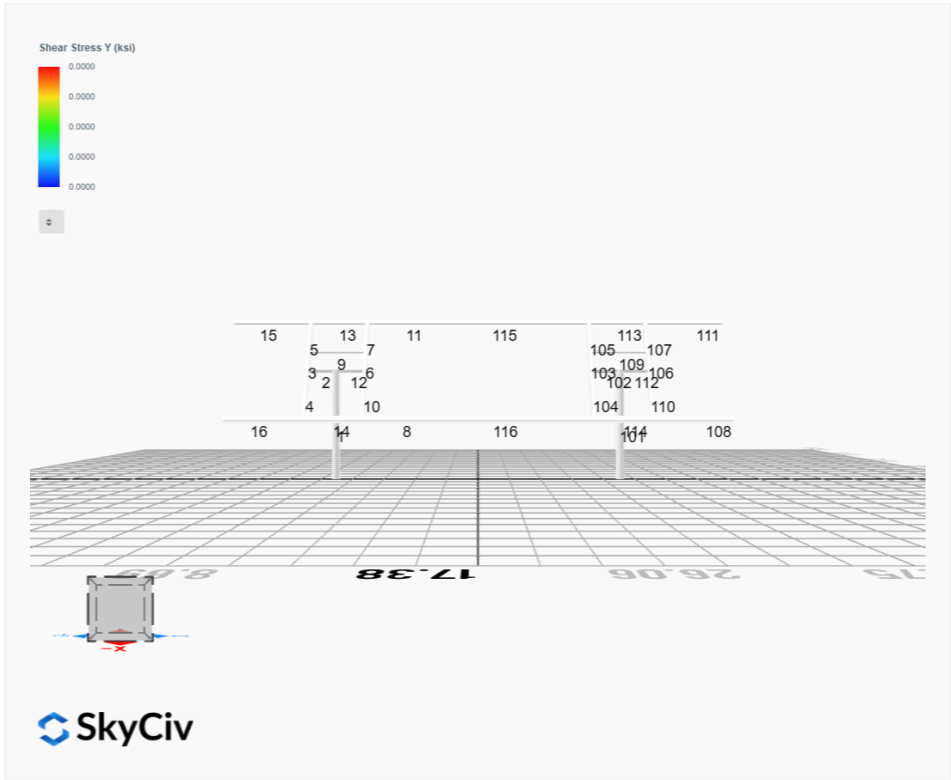
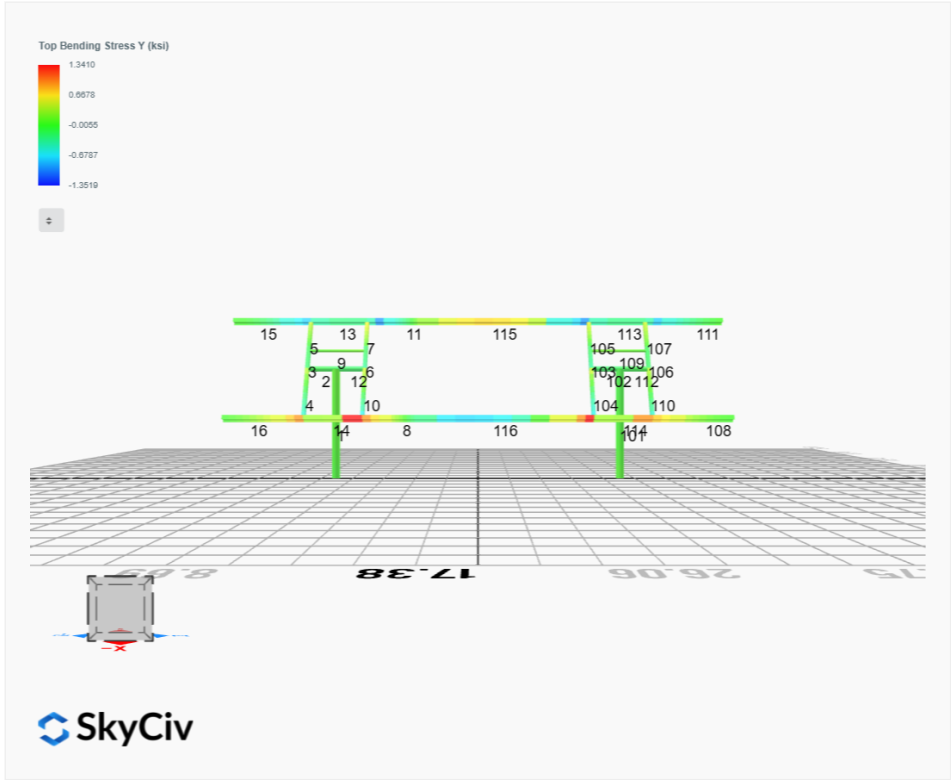


 SkyCiv

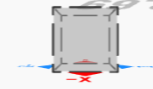
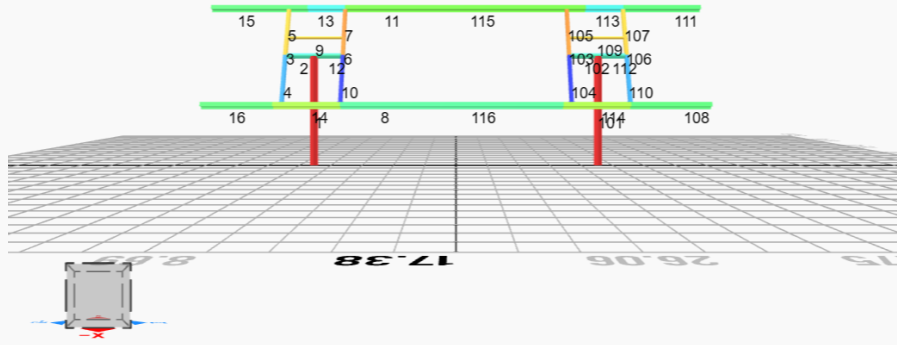


FEM Results (Envelope Worst Case for each member)





Axial Stress (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	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 2. D + L	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 3. D + (S or Lr or R)	0.0000	1.7432	0.0359	0.0779	-0.0230	0.0102
ULS: 3. D + (S or Lr or R)	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6819	0.0344	0.0746	-0.0221	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 5b. D + 0.7E	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.6819	0.0344	0.0746	-0.0221	0.0101
ULS: 8. 0.6D + 0.7E	0.0000	0.8988	0.0179	0.0388	-0.0115	0.0060
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5113	2.2027	0.0608	0.1237	-0.1929	11.5243
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5113	0.7932	-0.0011	0.0057	0.1546	-11.4129
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1335	2.2105	0.0576	0.1189	-0.1524	8.6458
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6819	0.0344	0.0746	-0.0221	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1335	1.1533	0.0112	0.0303	0.1082	-8.5571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6819	0.0344	0.0746	-0.0221	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1335	2.0265	0.0530	0.1090	-0.1494	8.6457
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1335	0.9694	0.0067	0.0205	0.1112	-8.5572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.4979	0.0298	0.0647	-0.0191	0.0101
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5113	1.6035	0.0488	0.0979	-0.1852	11.5203
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.8988	0.0179	0.0388	-0.0115	0.0060
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5113	0.1940	-0.0130	-0.0202	0.1622	-11.4169
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.8988	0.0179	0.0388	-0.0115	0.0060

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	3.0947
Shear X	-2.5189
Shear Z	0.0904
Moment X	0.1827
Moment Y (Twist)	0.3139
Moment Z	19.3466

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	2.2105
Shear X	-1.5113
Shear Z	0.0608
Moment X	0.1237
Moment Y (Twist)	0.1929
Moment Z	11.5243

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 2. D + L	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 3. D + (S or Lr or R)	-0.0000	1.7432	-0.0359	-0.0779	0.0231	0.0102
ULS: 3. D + (S or Lr or R)	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.6819	-0.0344	-0.0746	0.0221	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 5b. D + 0.7E	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.6819	-0.0344	-0.0746	0.0221	0.0101
ULS: 8. 0.6D + 0.7E	-0.0000	0.8988	-0.0179	-0.0388	0.0115	0.0060
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5113	2.2027	-0.0608	-0.1237	0.1929	11.5243
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5113	0.7932	0.0011	-0.0057	-0.1546	-11.4129
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1335	2.2105	-0.0576	-0.1189	0.1524	8.6458
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.6819	-0.0344	-0.0746	0.0221	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1335	1.1533	-0.0112	-0.0303	-0.1082	-8.5571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.6819	-0.0344	-0.0746	0.0221	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1335	2.0265	-0.0530	-0.1090	0.1494	8.6458
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1335	0.9694	-0.0067	-0.0205	-0.1112	-8.5572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.4979	-0.0298	-0.0647	0.0191	0.0101
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5113	1.6035	-0.0488	-0.0979	0.1852	11.5203
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.8988	-0.0179	-0.0388	0.0115	0.0060
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5113	0.1940	0.0130	0.0201	-0.1622	-11.4169
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.8988	-0.0179	-0.0388	0.0115	0.0060

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	3.0947
Shear X	-2.5189
Shear Z	-0.0904
Moment X	-0.1828
Moment Y (Twist)	0.3139
Moment Z	19.3470

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	2.2105
Shear X	-1.5113
Shear Z	-0.0608
Moment X	-0.1237
Moment Y (Twist)	0.1929
Moment Z	11.5243

Project Details

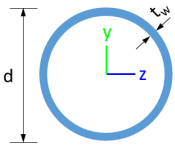
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 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial



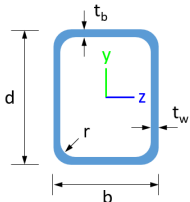
Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
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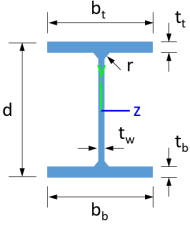
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

Section Dimensions								
								

ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
7	6in Pipe Sch 40	6.63	0.28					

								
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ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		

								
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ID	Name	d (in)	t_w (in)	b_f (in)	b_b (in)	t_f (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

115	120.00	66.63	15.30	6.45	30.09	45.74
116	120.60	68.63	15.30	6.45	30.09	45.74

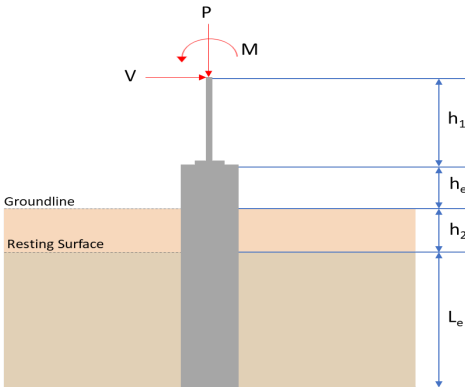
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.021	0.457	0.012	0.033	0.001	0.472	#13	0.426	Not Required	Pass
2	0.001	0.121	0.143	0.031	0.028	0.264	#13	0.034	Not Required	Pass
3	0.005	0.282	0.045	0.028	0.004	0.320	#13	0.044	Not Required	Pass
4	0.005	0.281	0.114	0.028	0.015	0.360	#13	0.078	Not Required	Pass
5	0.005	0.175	0.109	0.028	0.016	0.194	#13	0.073	Not Required	Pass
6	0.007	0.314	0.080	0.032	0.010	0.388	#13	0.044	Not Required	Pass
7	0.007	0.195	0.153	0.031	0.022	0.228	#13	0.073	Not Required	Pass
8	0.001	0.042	0.054	0.020	0.006	0.062	#13	0.088	Not Required	Pass
9	0.005	0.019	0.039	0.002	0.001	0.047	#13	0.198	Not Required	Pass
10	0.007	0.313	0.146	0.032	0.019	0.392	#13	0.078	Not Required	Pass
11	0.001	0.042	0.055	0.020	0.006	0.061	#13	0.088	Not Required	Pass
12	0.001	0.152	0.162	0.038	0.031	0.315	#13	0.034	Not Required	Pass
13	0.002	0.084	0.147	0.026	0.008	0.191	#13	0.265	Not Required	Pass
14	0.002	0.085	0.145	0.026	0.008	0.188	#13	0.177	Not Required	Pass
15	0.000	0.030	0.052	0.012	0.004	0.074	#13	Not Required	Not Required	Pass
16	0.000	0.030	0.052	0.012	0.004	0.074	#13	Not Required	Not Required	Pass
101	0.021	0.457	0.012	0.033	0.001	0.472	#13	0.426	Not Required	Pass
102	0.001	0.152	0.162	0.038	0.031	0.315	#13	0.034	Not Required	Pass
103	0.007	0.314	0.080	0.032	0.010	0.388	#13	0.044	Not Required	Pass
104	0.007	0.313	0.146	0.032	0.019	0.392	#13	0.078	Not Required	Pass
105	0.007	0.195	0.153	0.031	0.022	0.228	#13	0.073	Not Required	Pass
106	0.005	0.282	0.045	0.028	0.004	0.320	#13	0.044	Not Required	Pass
107	0.005	0.175	0.109	0.028	0.016	0.194	#13	0.073	Not Required	Pass
108	0.000	0.030	0.052	0.012	0.004	0.074	#13	Not Required	Not Required	Pass
109	0.005	0.019	0.039	0.002	0.001	0.047	#13	0.198	Not Required	Pass
110	0.005	0.281	0.114	0.028	0.015	0.360	#13	0.078	Not Required	Pass
111	0.000	0.030	0.052	0.012	0.004	0.074	#13	Not Required	Not Required	Pass
112	0.001	0.121	0.143	0.031	0.028	0.264	#13	0.034	Not Required	Pass
113	0.002	0.084	0.147	0.026	0.008	0.191	#13	0.177	Not Required	Pass
114	0.002	0.085	0.145	0.026	0.008	0.188	#13	0.265	Not Required	Pass
115	0.002	0.117	0.085	0.020	0.006	0.189	#13	0.439	Not Required	Pass
116	0.001	0.117	0.085	0.020	0.006	0.190	#13	0.439	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

L_u	Length between brace 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
NG	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 $L = 4.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (q_a) (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>2.210</td><td>3.095</td></tr><tr><td>V_x (kip)</td><td>-1.511</td><td>-2.519</td></tr><tr><td>V_z (kip)</td><td>0.061</td><td>0.090</td></tr><tr><td>M_x (kipft)</td><td>0.124</td><td>0.183</td></tr><tr><td>M_z (kipft)</td><td>11.524</td><td>19.347</td></tr></table> Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (q_a) (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)	2.210	3.095	V_x (kip)	-1.511	-2.519	V_z (kip)	0.061	0.090	M_x (kipft)	0.124	0.183	M_z (kipft)	11.524	19.347	
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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}$$H_o = \frac{(-1.511 \text{ kip})}{1.57 \times (48 \text{ in})}$$H_o = -0.24061 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(11.524 \text{ kipft}) + ((-1.511 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.835 \text{ kipft/ft}$ 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: $L_{e,x} = 4.3736 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.061 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.0097134 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.124 \text{ kipft}) + ((0.061 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.019745 \text{ kipft/ft}$ Required depth of embedment in earth: $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,z} = 1.3305 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(4.3736 \text{ ft}), (1.3305 \text{ ft})]$ $L_{e,req} = 4.374 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 4.75 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(4.374 \text{ ft})}{(4.75 \text{ ft})}$ $Ratio = 0.92084$	Status: PASS Ratio: 0.920
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(2.21 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.138125 \text{ kips/ft}^2$	

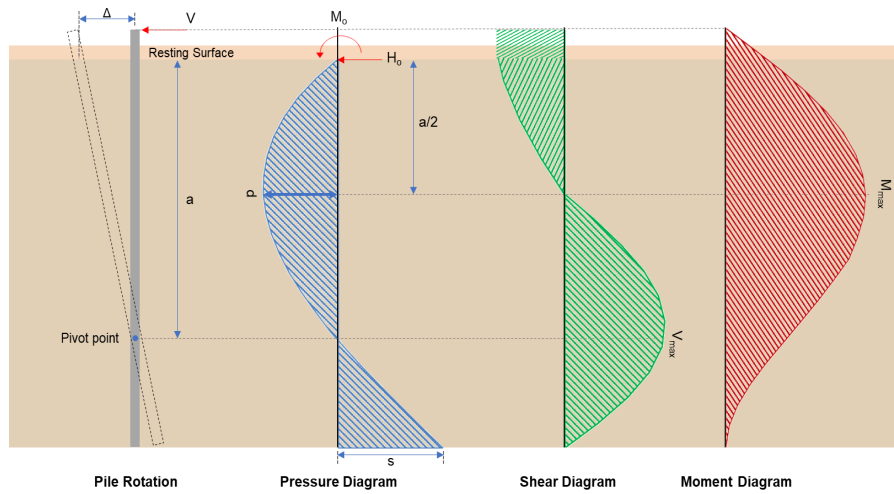
	$q = 0.13812 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.13812 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.069062$	Status: PASS Ratio: 0.070
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.1875$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.24061 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 1.835 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (1.835 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.835 \text{ kipft/ft})) + (4 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))}$ $a = 3.2828 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (1.835 \text{ kipft/ft})) + (3 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^3 [(3 \times (1.835 \text{ kipft/ft})) + (2 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$ $p = 0.15798 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (1.835 \text{ kipft/ft})) + ((-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$ $s = 0.67205 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.2828 \text{ ft})}{2}$ $p_a = 0.24621 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.15798 \text{ kip/ft}^2)}{(0.24621 \text{ kip/ft}^2)}$ $Ratio = 0.64164$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.640

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$ $p_s = 0.7125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.67205 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94323$	Status: PASS Ratio: 0.940
	Considering z-direction: $H_o = 0.0097134 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.019745 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.019745 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.019745 \text{ kipft/ft})) + (4 \times (0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))}$ $a = 3.4077 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.019745 \text{ kipft/ft})) + (3 \times (0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^3 \times [(3 \times (0.019745 \text{ kipft/ft})) + (2 \times (0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$ $p = 0.010369 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.019745 \text{ kipft/ft})) + ((0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$ $s = 0.022771 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.4077 \text{ ft})}{2}$ $p_a = 0.25558 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.010369 \text{ kip/ft}^2)}{(0.25558 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.04057$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$ $p_s = 0.7125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.040

$$Ratio = \frac{(0.022771 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.031959$$

Status: **PASS**
Ratio: **0.030**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.519 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.40111 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.347 \text{ kipft}) + ((-2.519 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0807 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0807 \text{ kipft/ft})}{(-0.40111 \text{ kip/ft})}$$

$$E = 7.6804 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0807 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.40111 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (3.0807 \text{ kipft/ft})) + (4 \times (-0.40111 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2822 \text{ ft}$$

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

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

$$V_{max} = ((-0.40111 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2822 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2822 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.6486 \text{ kip}$$

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

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

$$M_{max} = ((-0.40111 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(7.6804 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.724 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57 b}$$

$$H_o = \frac{(0.09 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.014331 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{1.57 b}$$

$$M_o = \frac{(0.183 \text{ kipft}) + ((0.09 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.02914 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.02914 \text{ kipft/ft})}{(0.014331 \text{ kip/ft})}$$

$$E = 2.0333 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.02914 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.014331 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.02914 \text{ kipft/ft})) + (4 \times (0.014331 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.4077 \text{ ft}$$

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

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

$$V_{max} = ((0.014331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.4077 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.4077 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.081707 \text{ kip}$$

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

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

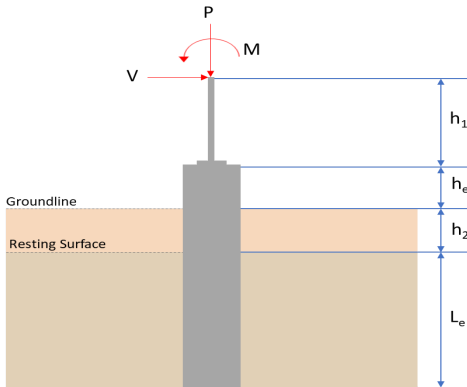
$$M_{max} = ((0.014331 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(2.0333 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.16982 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(3.095 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.493 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.493 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(3.095 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0011569$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - 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 $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 3.095 \text{ kip} \rightarrow 3095 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3095 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 118.9 \text{ kip}$	
22.5.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (118.9 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 118.9 \text{ kip}$	

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{ysk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((118.9 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.36 \text{ kip}$ Considering x-direction: $V_{max} = 5.6486 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(5.6486 \text{ kip})}{(110.36 \text{ kip})}$ $Ratio = 0.051182$ Considering z-direction: $V_{max} = 0.081707 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.081707 \text{ kip})}{(110.36 \text{ kip})}$ $Ratio = 0.00074034$	Status: PASS Ratio: 0.050 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 12.724 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.724 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.050978$	Status: PASS Ratio: 0.050
	Considering z-direction: $M_{max} = 0.16982 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.16982 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00068036$	Status: PASS Ratio: 0.000

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 = 4.75 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>2.210</td><td>3.095</td></tr><tr><td>Vx (kip)</td><td>-1.511</td><td>-2.519</td></tr><tr><td>Vz (kip)</td><td>-0.061</td><td>-0.090</td></tr><tr><td>Mx (kipft)</td><td>-0.124</td><td>-0.183</td></tr><tr><td>Mz (kipft)</td><td>11.524</td><td>19.347</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)	2.210	3.095	Vx (kip)	-1.511	-2.519	Vz (kip)	-0.061	-0.090	Mx (kipft)	-0.124	-0.183	Mz (kipft)	11.524	19.347	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mx (kipft)	-0.124	-0.183																										
Mz (kipft)	11.524	19.347																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.511 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24061 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(11.524 \text{ kipft}) + ((-1.511 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.835 \text{ kipft/ft}$ 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: $L_{e,x} = 4.3736 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.061 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.0097134 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.124 \text{ kipft}) + ((-0.061 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.019745 \text{ kipft/ft}$ Required depth of embedment in earth: $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,z} = 0.99927 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(4.3736 \text{ ft}), (0.99927 \text{ ft})]$ $L_{e,req} = 4.374 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 4.75 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(4.374 \text{ ft})}{(4.75 \text{ ft})}$ $Ratio = 0.92084$	Status: PASS Ratio: 0.920
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(2.21 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.138125 \text{ kips/ft}^2$	

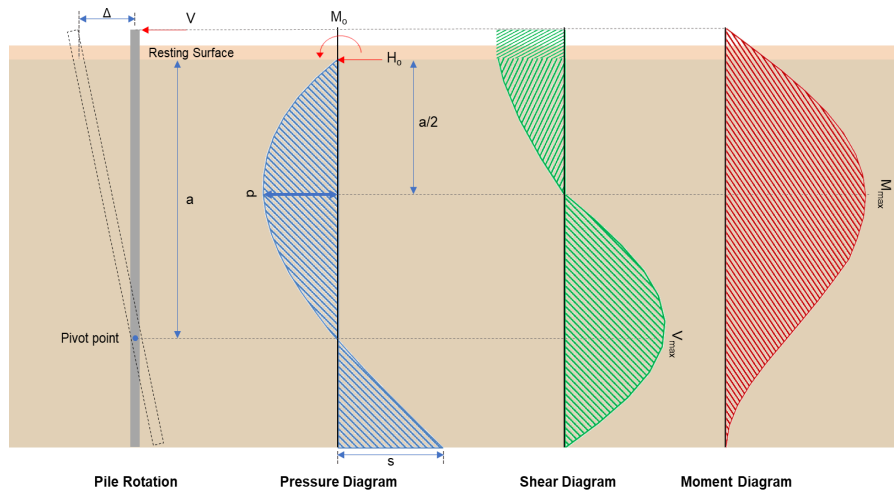
	$q = 0.13812 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.13812 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.069062$	Status: PASS Ratio: 0.070
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.1875$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.24061 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 1.835 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (1.835 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.835 \text{ kipft/ft})) + (4 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))}$ $a = 3.2828 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (1.835 \text{ kipft/ft})) + (3 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^3 [(3 \times (1.835 \text{ kipft/ft})) + (2 \times (-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$ $p = 0.15798 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (1.835 \text{ kipft/ft})) + ((-0.24061 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$ $s = 0.67205 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.2828 \text{ ft})}{2}$ $p_a = 0.24621 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.15798 \text{ kip/ft}^2)}{(0.24621 \text{ kip/ft}^2)}$ $Ratio = 0.64164$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.640

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$ $p_s = 0.7125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{s}{p_s}$ $Ratio = \frac{(0.67205 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$ $Ratio = 0.94323$	Status: PASS Ratio: 0.940
	Considering z-direction: $H_o = -0.0097134 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.019745 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.019745 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.019745 \text{ kipft/ft})) + (4 \times (-0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))}$ $a = 3.4077 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.019745 \text{ kipft/ft})) + (3 \times (-0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^3 \times [(3 \times (0.019745 \text{ kipft/ft})) + (2 \times (-0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$ $p = -0.0035538 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.019745 \text{ kipft/ft})) + ((-0.0097134 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$ $s = -0.0017679 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.4077 \text{ ft})}{2}$ $p_a = 0.25558 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(-0.0035538 \text{ kip/ft}^2)}{(0.25558 \text{ kip/ft}^2)}$ $Ratio = -0.013905$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$ $p_s = 0.7125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{s}{p_s}$	Status: PASS Ratio: -0.010

$$Ratio = \frac{(-0.0017679 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = -0.0024813$$

Status: **PASS**
Ratio: **0.000**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.519 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.40111 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(19.347 \text{ kipft}) + ((-2.519 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0807 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0807 \text{ kipft/ft})}{(-0.40111 \text{ kip/ft})}$$

$$E = 7.6804 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0807 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.40111 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (3.0807 \text{ kipft/ft})) + (4 \times (-0.40111 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2822 \text{ ft}$$

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

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

$$V_{max} = ((-0.40111 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2822 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2822 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.6486 \text{ kip}$$

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

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

$$M_{max} = ((-0.40111 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(7.6804 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.6804 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2822 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.724 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57 b}$$

$$H_o = \frac{(-0.09 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.014331 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z H)}{1.57 b}$$

$$M_o = \frac{(0.183 \text{ kipft}) + ((-0.09 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.02914 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.02914 \text{ kipft/ft})}{(-0.014331 \text{ kip/ft})}$$

$$E = 2.0333 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.02914 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.014331 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.02914 \text{ kipft/ft})) + (4 \times (-0.014331 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.4077 \text{ ft}$$

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

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

$$V_{max} = ((-0.014331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.4077 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.4077 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.081707 \text{ kip}$$

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

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

$$M_{max} = ((-0.014331 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(2.0333 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0333 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.4077 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.16982 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(3.095 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.493 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.493 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(3.095 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0011569$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - 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 $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 3.095 \text{ kip} \rightarrow 3095 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3095 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 118.9 \text{ kip}$	
22.5.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (118.9 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 118.9 \text{ kip}$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 12.724 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.724 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.050978$	Status: PASS Ratio: 0.050
	Considering z-direction: $M_{max} = 0.16982 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.16982 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00068036$	Status: PASS Ratio: 0.000