

Your Project Calculations



Project Name: Elk Creek SPS 1 4x5 C2

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Elk%20Creek%20SPS%201%204x5%20C2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=pPvNGghC5513aoqsiZQ2gF0P8WrBOHI0LIINmAgexcp67kFRnr6rNthvXOazquX

Array Specification

Product:	Beam
Unique ID:	2P-22.5-10TOP-HD-45-L-4Hx5W-DK64
Duty Classification:	HD
Module Width:	44.60 in
Module Length:	89.20in
Number of Rows:	4
Number of Columns:	5
Total Number of Modules:	20
Desired Tilt Angle:	65
Front Edge Clearance:	15
Total Array Height at Tilt:	28.55 ft
Total Frame Length:	37.50 ft
Frame Weight:	2947 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	37.58 ft
Rail Length:	180.40 in
Rail Spacing:	3.76 ft
Rail Check:	PASS (58% utilized)

Support Specifications

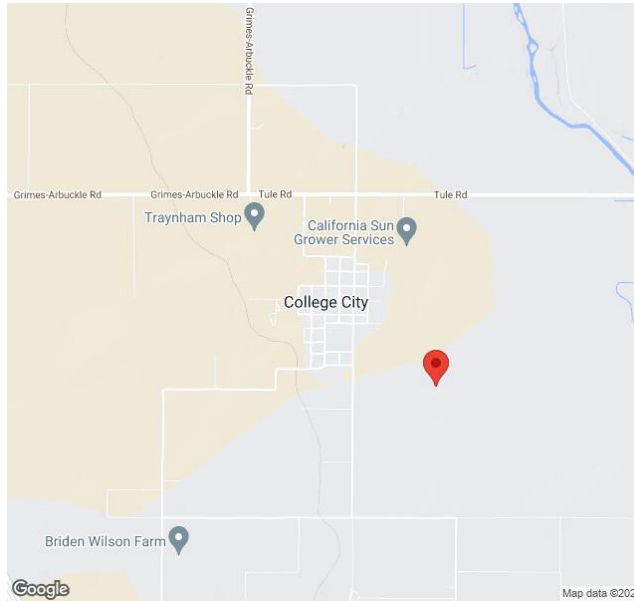
Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	21.81 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 9.00 ft Pile 2: 9.00 ft
Foundation Volume:	10.667 y ³
Foundation Result:	PASSED
Mount Twist:	1.628420 kip

Site Info

Risk Category:	III
Exposure:	B
Soil Classification:	sand
Site Location:	2222+22, College City, CA 95912, USA
Wind Speed:	100 mph
Snow Load:	176 psf
Design Uplift Pressure:	0.020059 ksf
Design Downforce Pressure:	-0.020059 ksf
Design Snow Pressure:	0.013306 ksf



Design Disclaimer

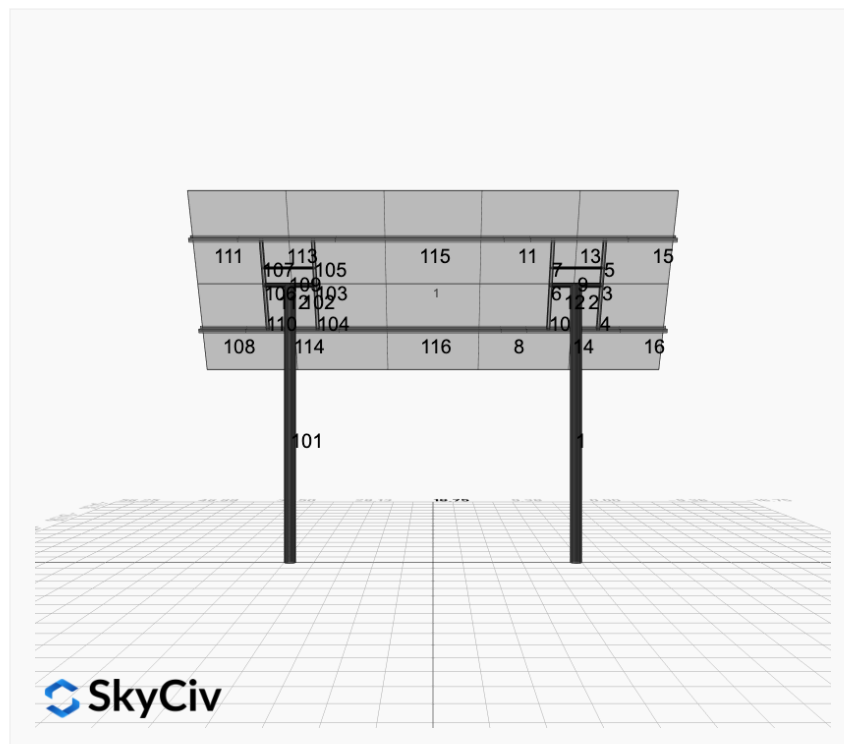
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

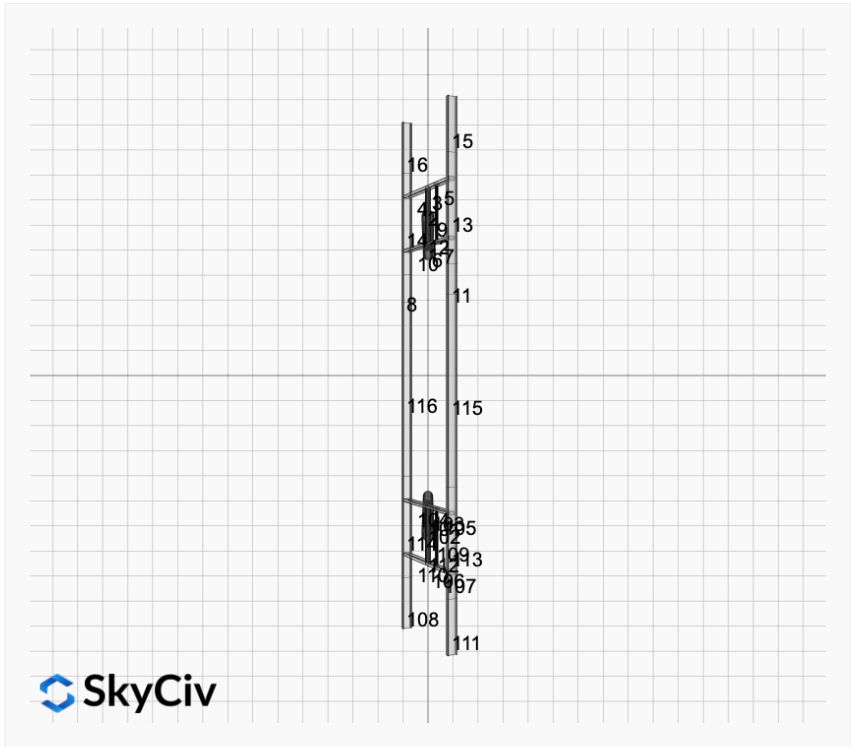
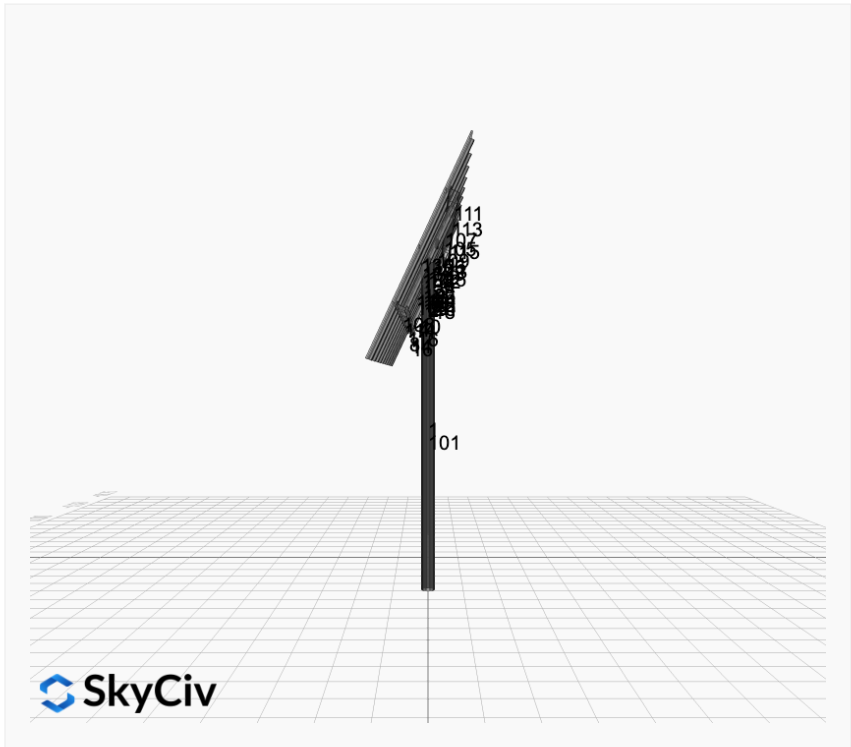
AutoDesigner Input

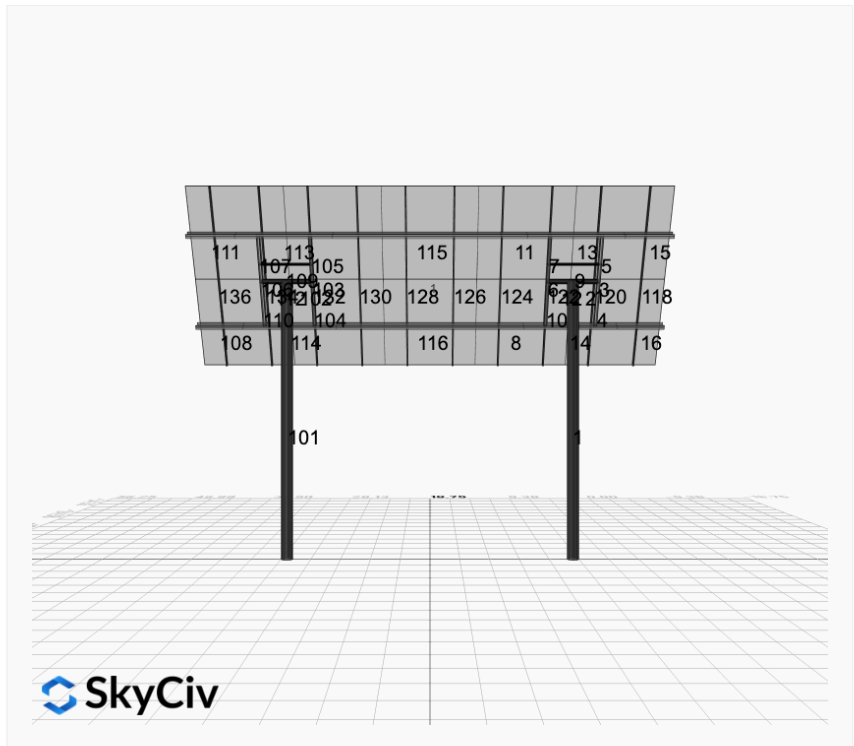
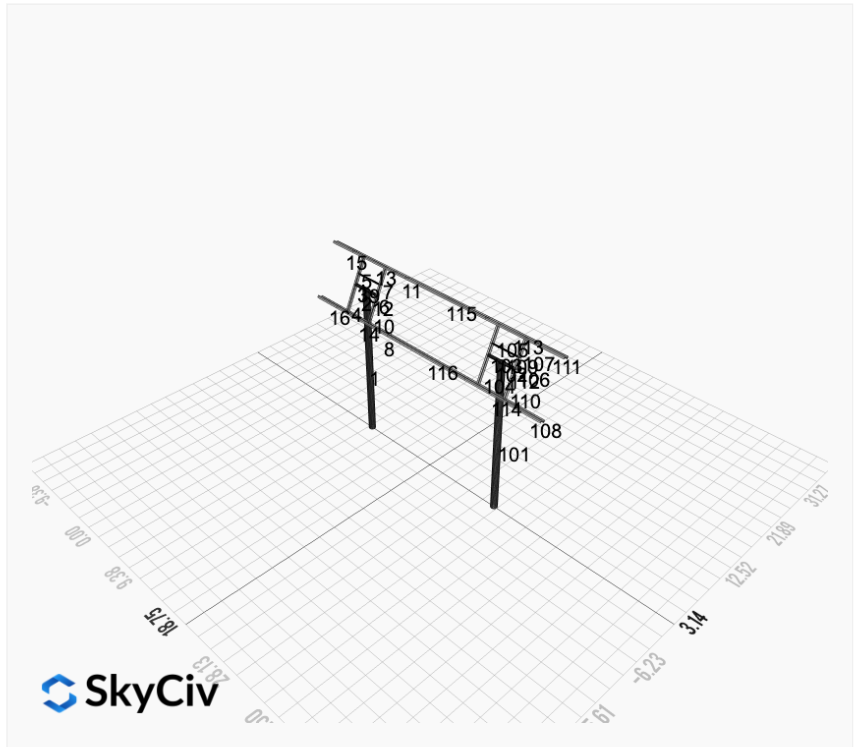
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Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent







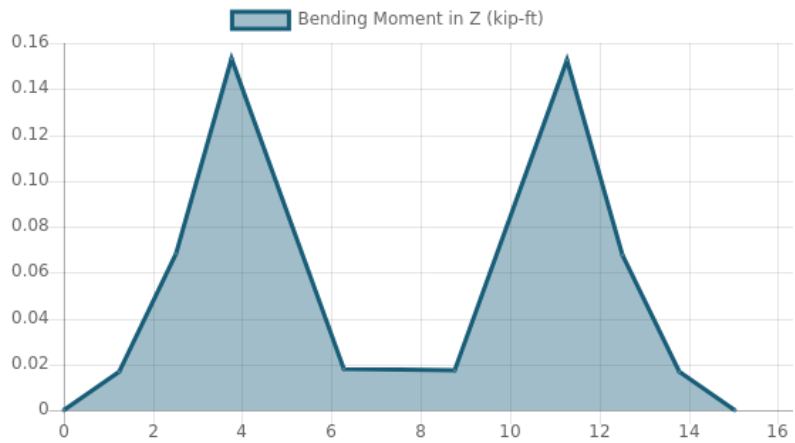
Rail Design Check

Rail Length: 15.033333333333333 ft
Additional Restraints Required: None
Tributary Width: 3.758333333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0211 kip/ft
Snow (Y): -0.0453 kip/ft
Wind uplift Case A: 0.0754 kip/ft
Wind downforce Case A: 0.0754 kip/ft
Dead (Panel load) (X): 0.0072 kip/ft
Dead (Panel load) (Y): -0.0154 kip/ft

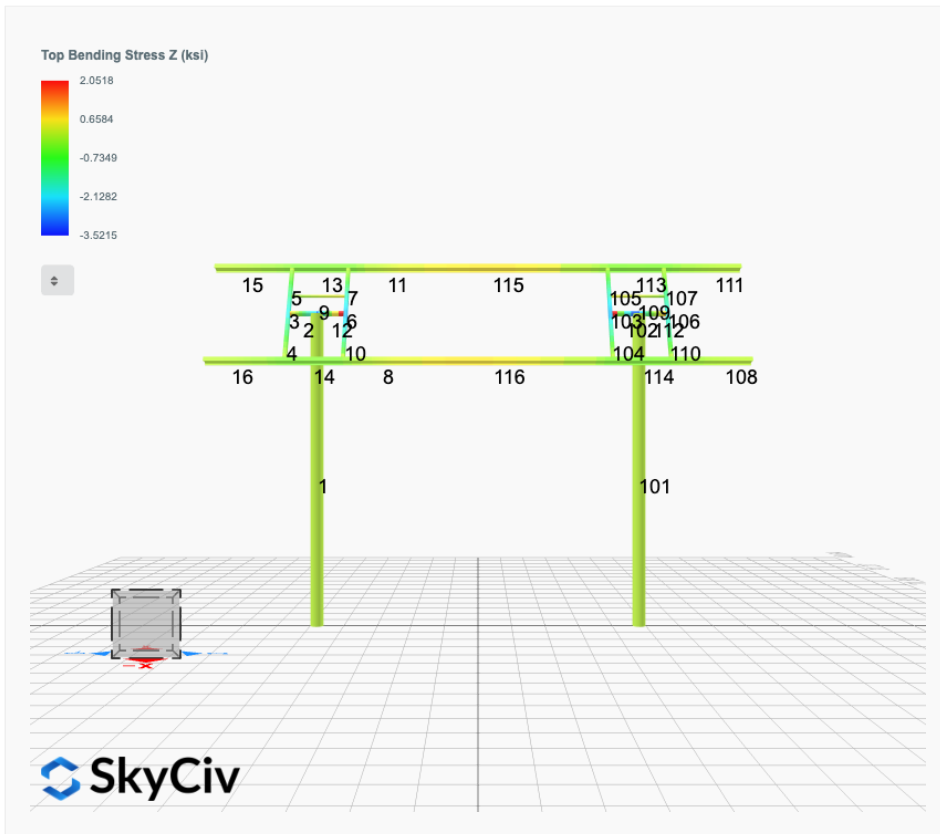
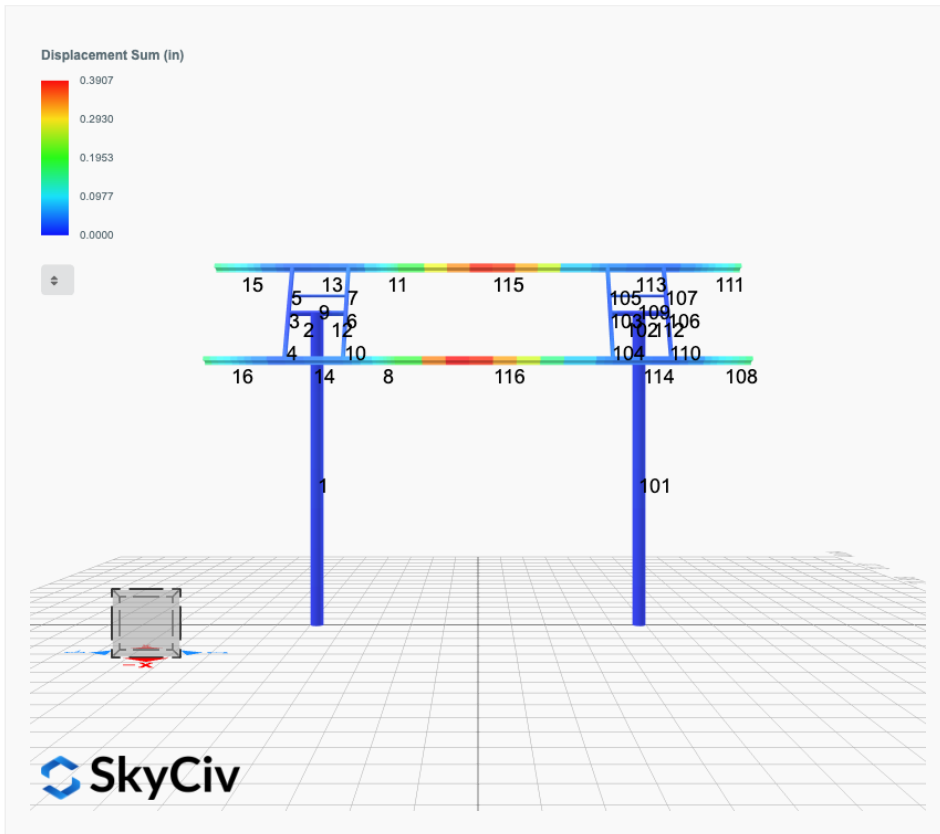


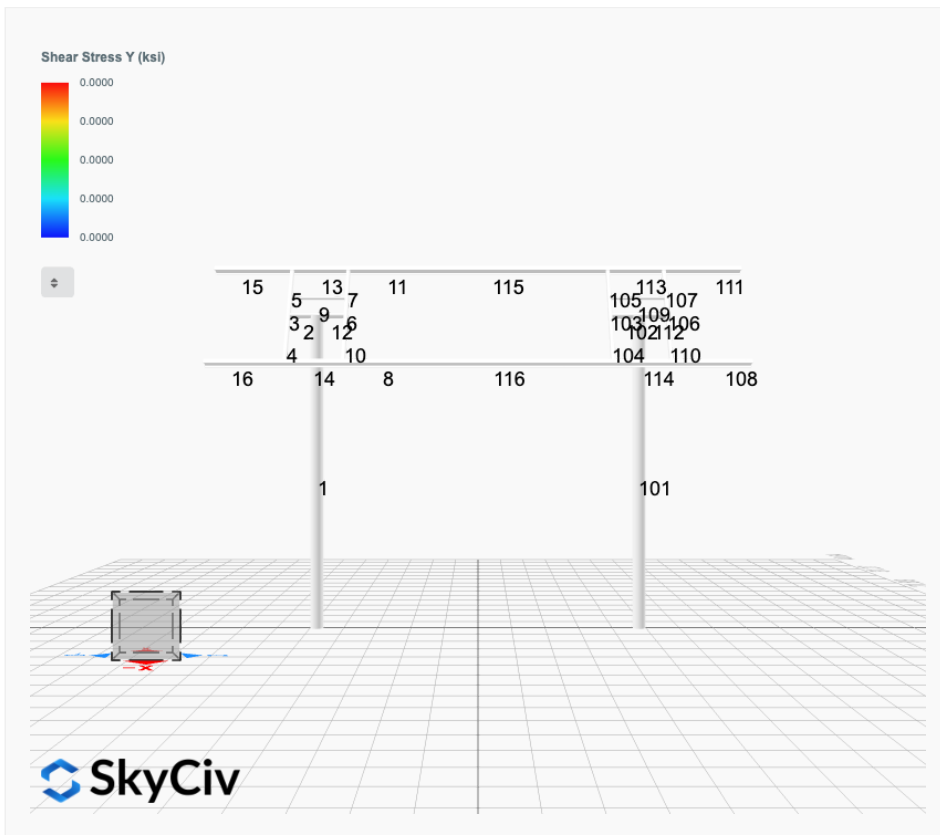
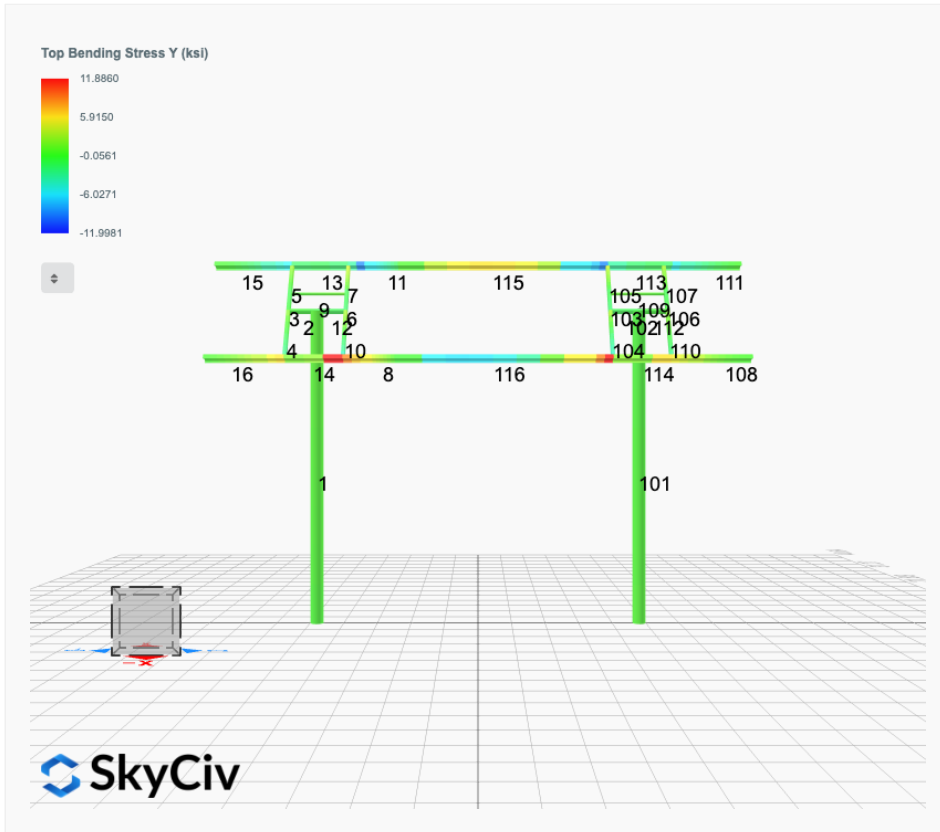
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	19.88697759	0.576	PASS
Material Yield	34.5	19.88697759	0.576	PASS
Material Strength	37	19.88697759	0.537	PASS

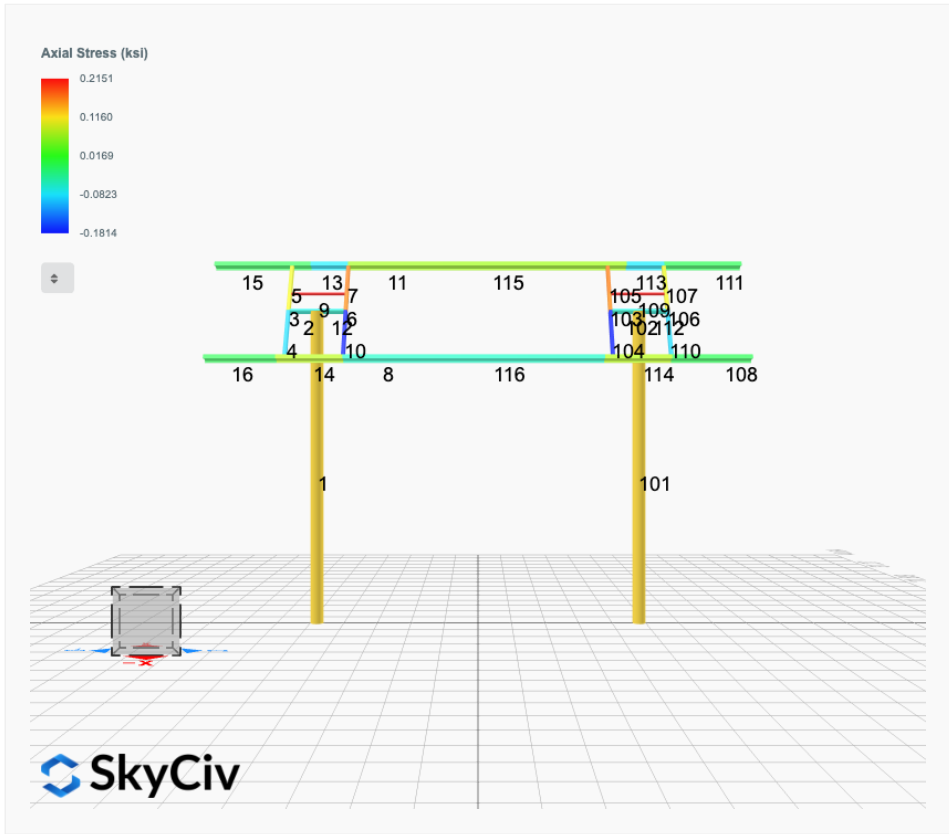
Member 1, ULS: 1. 1.4D



FEM Results (Envelope Worst Case for each member)







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	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 2. D + L	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 3. D + (S or Lr or R)	0.0000	4.3384	0.0637	0.4565	-0.1666	0.0166
ULS: 3. D + (S or Lr or R)	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.9421	0.0559	0.4007	-0.1461	0.0160
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 5b. D + 0.7E	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.9421	0.0559	0.4007	-0.1461	0.0160
ULS: 8. 0.6D + 0.7E	0.0000	1.6520	0.0195	0.1399	-0.0510	0.0084
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.0815	4.1903	0.0891	0.6225	-0.9818	68.0293
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0815	1.3164	-0.0241	-0.1555	0.8133	-66.4233
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3111	5.0198	0.0983	0.6927	-0.8188	51.0275
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.9421	0.0559	0.4007	-0.1461	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3111	2.8644	0.0134	0.1092	0.5275	-49.8120
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.9421	0.0559	0.4007	-0.1461	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3111	3.8311	0.0750	0.5251	-0.7576	51.0255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3111	1.6757	-0.0099	-0.0583	0.5887	-49.8139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.7534	0.0325	0.2331	-0.0849	0.0140
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.0815	3.0890	0.0761	0.5292	-0.9479	68.0237
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.6520	0.0195	0.1399	-0.0510	0.0084
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0815	0.2151	-0.0371	-0.2487	0.8473	-66.4289
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.6520	0.0195	0.1399	-0.0510	0.0084

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	7.0376
Shear X	-5.1359
Shear Z	0.1485
Moment X	1.0394
Moment Y (Twist)	1.6290
Moment Z	115.7962

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	5.0198
Shear X	-3.0815
Shear Z	0.0983
Moment X	0.6927
Moment Y (Twist)	0.9818
Moment Z	68.0293

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	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 2. D + L	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 3. D + (S or Lr or R)	-0.0000	4.3384	-0.0637	-0.4566	0.1666	0.0167
ULS: 3. D + (S or Lr or R)	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.9421	-0.0559	-0.4008	0.1462	0.0160
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 5b. D + 0.7E	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.9421	-0.0559	-0.4008	0.1462	0.0160
ULS: 8. 0.6D + 0.7E	-0.0000	1.6520	-0.0195	-0.1399	0.0510	0.0084
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.0815	4.1903	-0.0891	-0.6225	0.9819	68.0294
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0815	1.3164	0.0241	0.1554	-0.8133	-66.4233
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3111	5.0198	-0.0983	-0.6928	0.8189	51.0275
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	3.9421	-0.0559	-0.4008	0.1462	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3111	2.8644	-0.0134	-0.1093	-0.5275	-49.8120
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	3.9421	-0.0559	-0.4008	0.1462	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3111	3.8311	-0.0750	-0.5252	0.7576	51.0255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3111	1.6757	0.0099	0.0583	-0.5887	-49.8139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.7534	-0.0325	-0.2332	0.0850	0.0141
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.0815	3.0890	-0.0761	-0.5292	0.9479	68.0237
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.6520	-0.0195	-0.1399	0.0510	0.0084
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0815	0.2151	0.0371	0.2487	-0.8473	-66.4289
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.6520	-0.0195	-0.1399	0.0510	0.0084

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	7.0375
Shear X	-5.1359
Shear Z	-0.1485
Moment X	-1.0400
Moment Y (Twist)	1.6284
Moment Z	115.7972

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	5.0198
Shear X	-3.0815
Shear Z	-0.0983
Moment X	-0.6928
Moment Y (Twist)	0.9819
Moment Z	68.0294

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial



Design Input Information

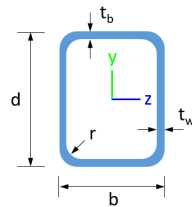
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
11	10in Pipe Sch 40	10.75	0.36				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	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)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38

115	133.20	46.28	11.92	6.12	40.24	43.62
116	133.20	46.28	11.92	6.12	40.24	43.62

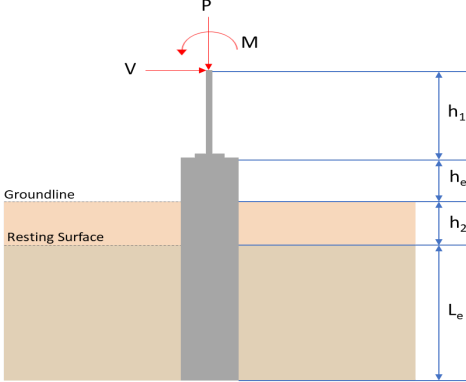
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.059	0.784	0.015	0.032	0.001	0.818	#13	0.748	Not Required	Pass
2	0.004	0.143	0.185	0.039	0.039	0.322	#13	0.035	Not Required	Pass
3	0.007	0.361	0.041	0.035	0.006	0.380	#13	0.045	Not Required	Pass
4	0.007	0.359	0.123	0.036	0.026	0.423	#13	0.080	Not Required	Pass
5	0.007	0.225	0.123	0.036	0.032	0.248	#13	0.074	Not Required	Pass
6	0.012	0.470	0.079	0.047	0.016	0.523	#13	0.045	Not Required	Pass
7	0.012	0.291	0.214	0.046	0.053	0.322	#13	0.074	Not Required	Pass
8	0.003	0.077	0.228	0.034	0.019	0.231	#23	0.095	Not Required	Pass
9	0.015	0.040	0.062	0.003	0.002	0.082	#13	0.204	Not Required	Pass
10	0.012	0.468	0.206	0.046	0.043	0.539	#13	0.080	Not Required	Pass
11	0.004	0.076	0.234	0.034	0.019	0.238	#23	0.095	Not Required	Pass
12	0.003	0.236	0.259	0.062	0.049	0.496	#13	0.054	Not Required	Pass
13	0.006	0.129	0.497	0.042	0.023	0.572	#21	0.286	Not Required	Pass
14	0.006	0.130	0.489	0.042	0.023	0.562	#21	0.190	Not Required	Pass
15	0.000	0.039	0.126	0.017	0.009	0.153	#21	Not Required	Not Required	Pass
16	0.000	0.039	0.126	0.017	0.009	0.153	#21	Not Required	Not Required	Pass
101	0.059	0.784	0.015	0.032	0.001	0.818	#13	0.748	Not Required	Pass
102	0.003	0.236	0.259	0.062	0.049	0.496	#13	0.054	Not Required	Pass
103	0.012	0.470	0.079	0.047	0.016	0.523	#13	0.045	Not Required	Pass
104	0.012	0.468	0.206	0.046	0.043	0.539	#13	0.080	Not Required	Pass
105	0.012	0.291	0.214	0.046	0.053	0.322	#13	0.074	Not Required	Pass
106	0.007	0.361	0.041	0.035	0.006	0.381	#13	0.045	Not Required	Pass
107	0.007	0.225	0.123	0.036	0.032	0.248	#13	0.074	Not Required	Pass
108	0.000	0.039	0.126	0.017	0.009	0.153	#21	Not Required	Not Required	Pass
109	0.015	0.040	0.062	0.003	0.002	0.082	#13	0.204	Not Required	Pass
110	0.007	0.359	0.123	0.036	0.026	0.423	#13	0.080	Not Required	Pass
111	0.000	0.039	0.126	0.017	0.009	0.153	#21	Not Required	Not Required	Pass
112	0.004	0.143	0.185	0.039	0.039	0.322	#13	0.035	Not Required	Pass
113	0.006	0.129	0.497	0.042	0.023	0.572	#21	0.190	Not Required	Pass
114	0.006	0.130	0.489	0.042	0.023	0.562	#21	0.286	Not Required	Pass
115	0.011	0.430	0.272	0.034	0.019	0.598	#13	0.601	Not Required	Pass
116	0.003	0.432	0.268	0.034	0.019	0.598	#13	0.601	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 _r	Length between braced points

L	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																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 9$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="675 1288 936 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.020</td> <td>7.038</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.082</td> <td>-5.136</td> </tr> <tr> <td>V_z (kip)</td> <td>0.098</td> <td>0.149</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.693</td> <td>1.039</td> </tr> <tr> <td>M_z (kipft)</td> <td>68.029</td> <td>115.796</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	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)	5.020	7.038	V_x (kip)	-3.082	-5.136	V_z (kip)	0.098	0.149	M_x (kipft)	0.693	1.039	M_z (kipft)	68.029	115.796	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.082 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.49076 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(68.029 \text{ kipft}) + ((-3.082 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.833 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.5089 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.098 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.015605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.693 \text{ kipft}) + ((0.098 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11035 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2176 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = \text{MAX}[(8.5089 \text{ ft}), (2.2176 \text{ ft})]$$

$$L_{e,req} = 8.509 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

$$\text{Ratio} = \frac{L_{e,req}}{L_e}$$

$$\text{Ratio} = \frac{(8.509 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.94544$$

Status: **PASS**
Ratio: **0.950**

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

$$q = \frac{P_c}{A}$$

$$q = \frac{(5.02 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31375 \text{ kip/ft}^2$$

$$q = 0.31375 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

$$\text{Ratio} = \frac{q}{q_a}$$

$$\text{Ratio} = \frac{(0.31375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15687$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

$$L/D = \frac{L}{D}$$

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.49076 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 10.833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.833 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (10.833 \text{ kipft/ft})) + (4 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.1603 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (10.833 \text{ kipft/ft})) + (3 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (10.833 \text{ kipft/ft})) + (2 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.35403 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (10.833 \text{ kipft/ft})) + ((-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2777 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1603 \text{ ft})}{2}$$

$$p_a = 0.46202 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.35403 \text{ kip/ft}^2)}{(0.46202 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76626$$

p_a - Allowable lateral soil pressure at depth L_e ,

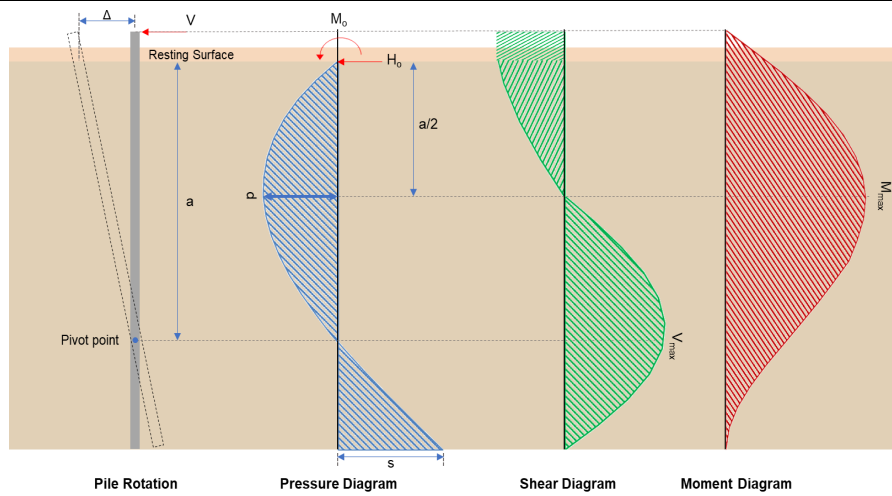
Status: **PASS**
Ratio: **0.770**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$ $p_s = 1.35 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.2777 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94641$	<p>Status: PASS Ratio: 0.950</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.015605 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11035 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11035 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.015605 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.11035 \text{ kipft/ft})) + (4 \times (0.015605 \text{ kip/ft}) \times (9 \text{ ft}))}$ $a = 6.3443 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.11035 \text{ kipft/ft})) + (3 \times (0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.11035 \text{ kipft/ft})) + (2 \times (0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]}$ $p = 0.011262 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.11035 \text{ kipft/ft})) + ((0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$ $s = 0.026752 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.3443 \text{ ft})}{2}$ $p_a = 0.47582 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.011262 \text{ kip/ft}^2)}{(0.47582 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.023669$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$ $p_s = 1.35 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.020</p>

$$Ratio = \frac{(0.026752 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.019816$$

Status: **PASS**
Ratio: **0.020**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.136 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.81783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(115.8 \text{ kipft}) + ((-5.136 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 18.439 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(18.439 \text{ kipft/ft})}{(-0.81783 \text{ kip/ft})}$$

$$E = 22.546 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.439 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.81783 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (18.439 \text{ kipft/ft})) + (4 \times (-0.81783 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.1576 \text{ ft}$$

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

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

$$V_{max} = ((-0.81783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.667 \text{ kip}$$

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

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

$$M_{max} = ((-0.81783 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(22.546 \text{ ft})}{(9 \text{ ft})} + \frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 72.317 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.149 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.023726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.039 \text{ kipft}) + ((0.149 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16545 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16545 \text{ kipft/ft})}{(0.023726 \text{ kip/ft})}$$

$$E = 6.9732 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16545 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.023726 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.16545 \text{ kipft/ft})) + (4 \times (0.023726 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.3469 \text{ ft}$$

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

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

$$V_{max} = ((0.023726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6.9732 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.3469 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (6.9732 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.3469 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19296 \text{ kip}$$

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

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

$$M_{max} = ((0.023726 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(6.9732 \text{ ft})}{(9 \text{ ft})} + \frac{(6.3469 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (6.9732 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.3469 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (6.9732 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.3469 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.79167 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

$$A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$$

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.038 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.362 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

$$A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$$

$$A_{min} = \text{Max} [(-84.362 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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

$$\text{Ratio} = \frac{A_{min}}{A_{st}}$$

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

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 No. 10: 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)**

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

$$\text{Ratio} = \frac{P}{\phi P_N}$$

$$\text{Ratio} = \frac{(7.038 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0026308$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.038 \text{ kip} \rightarrow 7038 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7038 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.42 \text{ kip}$$

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

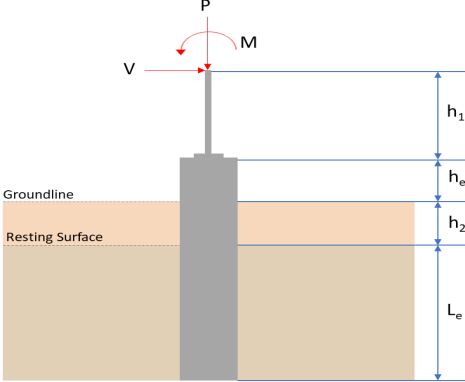
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.42 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.42 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.42 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.71 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 16.667 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(16.667 \text{ kip})}{(110.71 \text{ kip})}$ $\text{Ratio} = 0.15055$ <p>Considering z-direction:</p> <p>$V_{max} = 0.19296 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.19296 \text{ kip})}{(110.71 \text{ kip})}$ $\text{Ratio} = 0.001743$	<p>Status: PASS Ratio: 0.150</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 72.317 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(72.317 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.28973$	<p>Status: PASS Ratio: 0.290</p>
	<p>Considering z-direction: $M_{max} = 0.79167 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.79167 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0031718$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 9$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="675 1285 936 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.020</td> <td>7.038</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.082</td> <td>-5.136</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.098</td> <td>-0.149</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.693</td> <td>-1.040</td> </tr> <tr> <td>M_z (kipft)</td> <td>68.029</td> <td>115.797</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_n) (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)	5.020	7.038	V_x (kip)	-3.082	-5.136	V_z (kip)	-0.098	-0.149	M_x (kipft)	-0.693	-1.040	M_z (kipft)	68.029	115.797	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.082 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.49076 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(68.029 \text{ kipft}) + ((-3.082 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.833 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.5089 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.098 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.015605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.693 \text{ kipft}) + ((-0.098 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11035 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.916 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = \text{MAX}[(8.5089 \text{ ft}), (1.916 \text{ ft})]$$

$$L_{e,req} = 8.509 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

$$\text{Ratio} = \frac{L_{e,req}}{L_e}$$

$$\text{Ratio} = \frac{(8.509 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.94544$$

Status: **PASS**
Ratio: **0.950**

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

$$q = \frac{P_c}{A}$$

$$q = \frac{(5.02 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31375 \text{ kip/ft}^2$$

$$q = 0.31375 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

$$\text{Ratio} = \frac{q}{q_a}$$

$$\text{Ratio} = \frac{(0.31375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15687$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

$$L/D = \frac{L}{D}$$

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.49076 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 10.833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.833 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (10.833 \text{ kipft/ft})) + (4 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.1603 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (10.833 \text{ kipft/ft})) + (3 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (10.833 \text{ kipft/ft})) + (2 \times (-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.35403 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (10.833 \text{ kipft/ft})) + ((-0.49076 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2777 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1603 \text{ ft})}{2}$$

$$p_a = 0.46202 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.35403 \text{ kip/ft}^2)}{(0.46202 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76626$$

p_a - Allowable lateral soil pressure at depth L_e ,

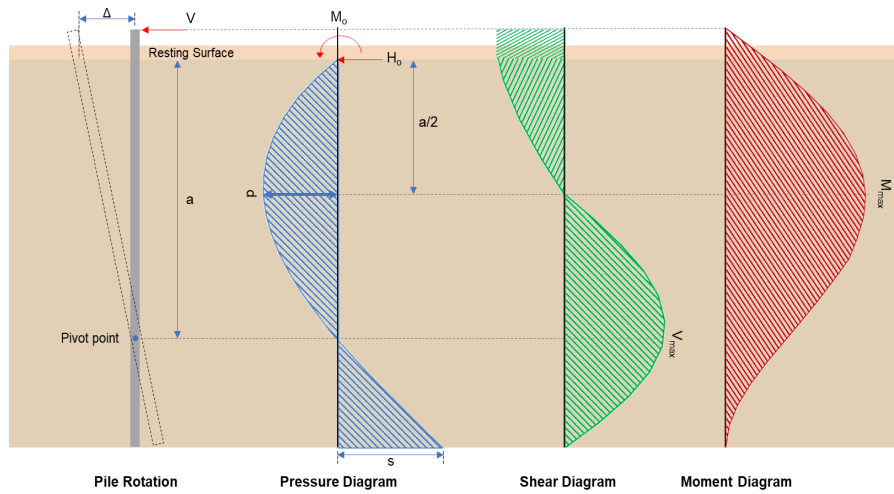
Status: **PASS**
Ratio: **0.770**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$ $p_s = 1.35 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.2777 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94641$	Status: PASS Ratio: 0.950
	<p>Considering z-direction:</p> <p>$H_o = -0.015605 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11035 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11035 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.015605 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.11035 \text{ kipft/ft})) + (4 \times (-0.015605 \text{ kip/ft}) \times (9 \text{ ft}))}$ $a = 6.3443 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.11035 \text{ kipft/ft})) + (3 \times (-0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.11035 \text{ kipft/ft})) + (2 \times (-0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]}$ $p = 0.00007431 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.11035 \text{ kipft/ft})) + ((-0.015605 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$ $s = 0.0059448 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.3443 \text{ ft})}{2}$ $p_a = 0.47582 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00007431 \text{ kip/ft}^2)}{(0.47582 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00015617$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$ $p_s = 1.35 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(0.0059448 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.0044036$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.136 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.81783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(115.8 \text{ kipft}) + ((-5.136 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 18.439 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(18.439 \text{ kipft/ft})}{(-0.81783 \text{ kip/ft})}$$

$$E = 22.546 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.439 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.81783 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (18.439 \text{ kipft/ft})) + (4 \times (-0.81783 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.1576 \text{ ft}$$

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

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

$$V_{max} = ((-0.81783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.667 \text{ kip}$$

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

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

$$M_{max} = ((-0.81783 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(22.546 \text{ ft})}{(9 \text{ ft})} + \frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.546 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 72.318 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.149 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.023726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.04 \text{ kipft}) + ((-0.149 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16561 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16561 \text{ kipft/ft})}{(-0.023726 \text{ kip/ft})}$$

$$E = 6.98 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16561 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.023726 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.16561 \text{ kipft/ft})) + (4 \times (-0.023726 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.3467 \text{ ft}$$

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

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

$$V_{max} = ((-0.023726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6.98 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.3467 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (6.98 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.3467 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19309 \text{ kip}$$

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

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

$$M_{max} = ((-0.023726 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(6.98 \text{ ft})}{(9 \text{ ft})} + \frac{(6.3467 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (6.98 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.3467 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (6.98 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.3467 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.79223 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

$$A_{st,required} = Min \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$$

$$A_{st,required} = Min \left[\frac{\frac{(7.038 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.362 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

$$A_{min} = Max [A_{st,required}, (0.0018 A_g)]$$

$$A_{min} = Max [(-84.362 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$Ratio = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3

s_{rebar} - 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. 10: Use #3(0.375 in)

25.7.2.1

s_{ties} - 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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

$$\text{Ratio} = \frac{P}{\phi P_N}$$

$$\text{Ratio} = \frac{(7.038 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0026308$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.038 \text{ kip} \rightarrow 7038 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7038 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.42 \text{ kip}$$

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

$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.42 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.42 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.42 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.71 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 16.667 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(16.667 \text{ kip})}{(110.71 \text{ kip})}$ $\text{Ratio} = 0.15055$ <p>Considering z-direction:</p> <p>$V_{max} = 0.19309 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.19309 \text{ kip})}{(110.71 \text{ kip})}$ $\text{Ratio} = 0.0017441$	<p>Status: PASS Ratio: 0.150</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 72.318 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(72.318 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.28973$	<p>Status: PASS Ratio: 0.290</p>
	<p>Considering z-direction: $M_{max} = 0.79223 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.79223 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.003174$	<p>Status: PASS Ratio: 0.000</p>