

Your Project Calculations



Project Name: W12116 5x6 Revb CU

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=W12116%205x6%20Revb%20CU&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/10_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=6dBBHvgLzkVAZVaWfcVRKHIAcfWmepTrkFnweU0BY8Dvrvv54TC0F2jJjNs0fLIL

Array Specification

Product:	Beam
Unique ID:	2P-22.5-6TOP-HD-57-L-5Hx6W-2F1D
Duty Classification:	HD
Module Width:	42.00 in
Module Length:	80.00in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Desired Tilt Angle:	5
Front Edge Clearance:	12
Total Array Height at Tilt:	13.53 ft
Total Frame Length:	39.50 ft
Frame Weight:	1704 lbs
Array Dimensions N/S:	17.71 ft
Array Dimensions E/W:	40.50 ft
Rail Length:	212.50 in
Rail Spacing:	3.38 ft
Rail Check:	PASS (44% utilized)

Support Specifications

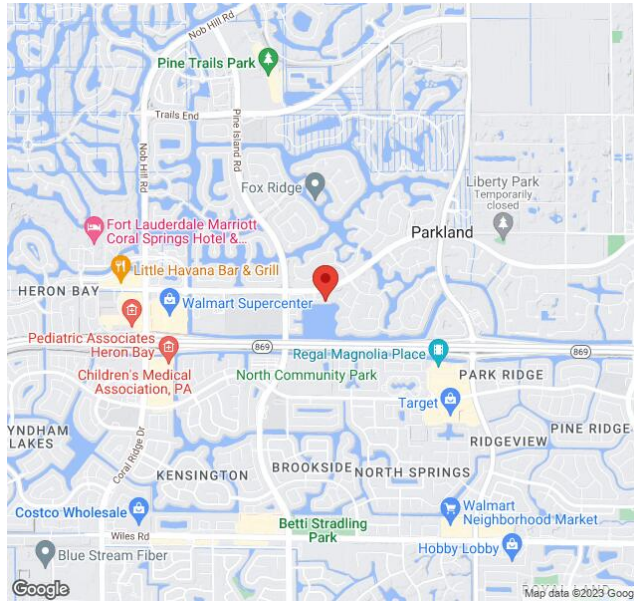
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.77 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 5.75 ft
Foundation Volume:	10.648 y ³
Foundation Result:	PASSED
Mount Twist:	0.064856 kip

Site Info

Risk Category:	II
Exposure:	B
Soil Classification:	sand
Site Location:	10250 NW 60th Pl, Parkland, FL 33076, USA
Wind Speed:	170 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

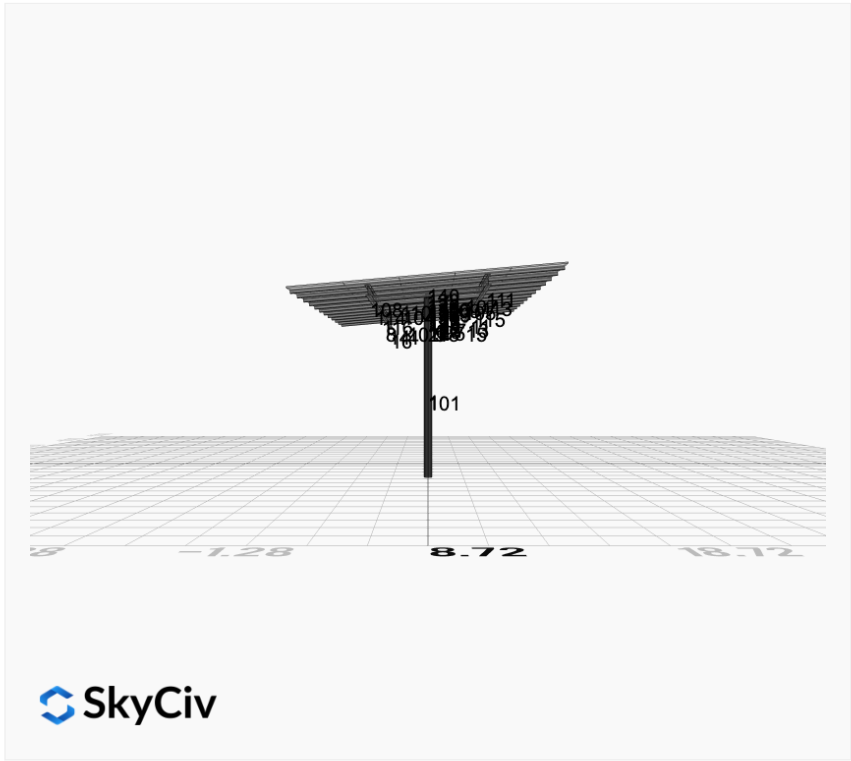
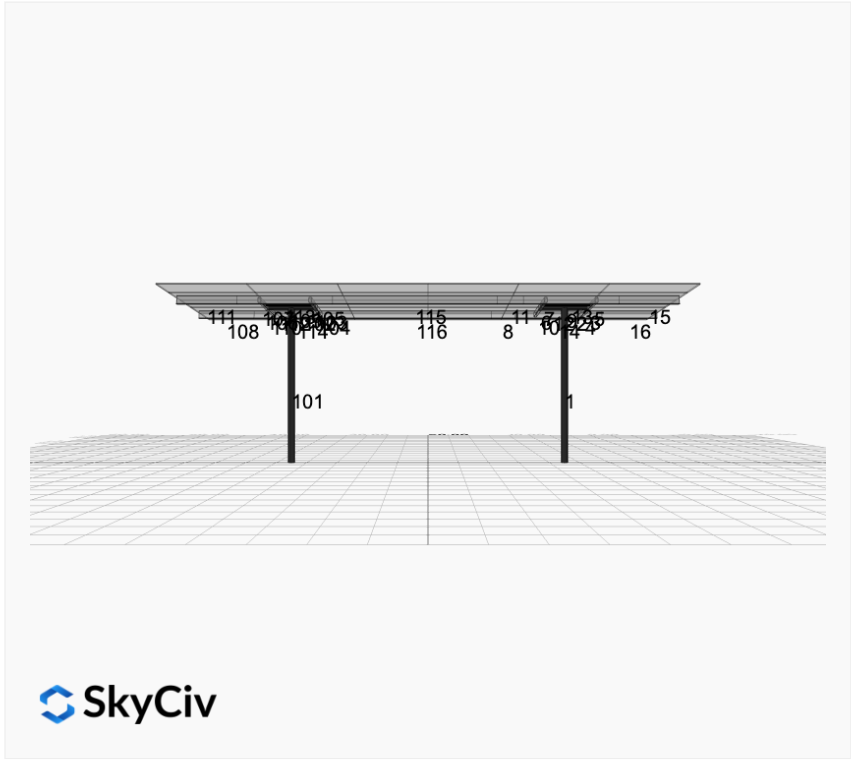
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

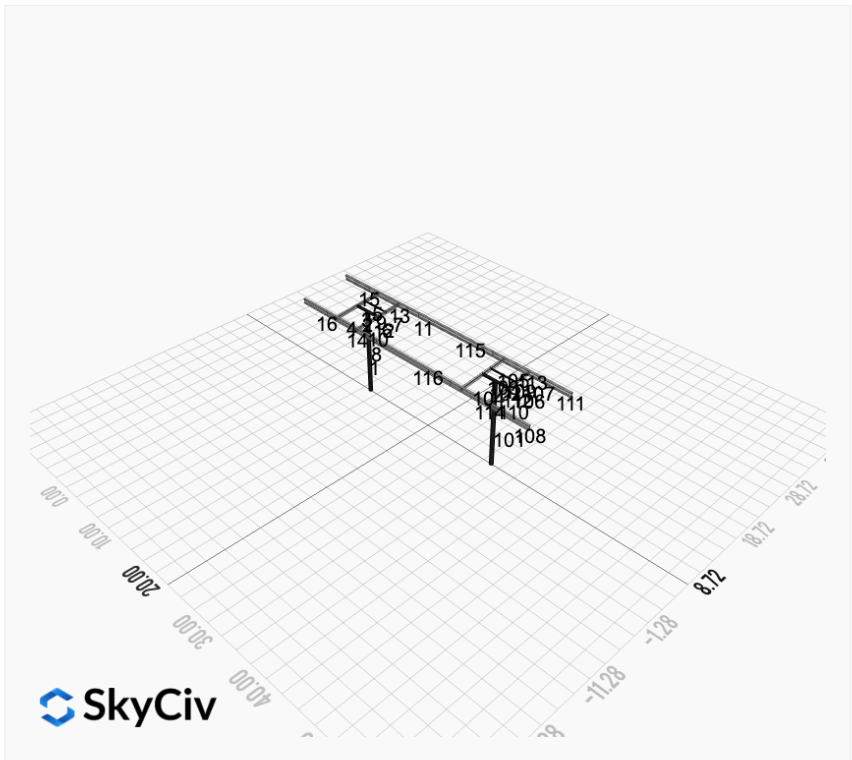
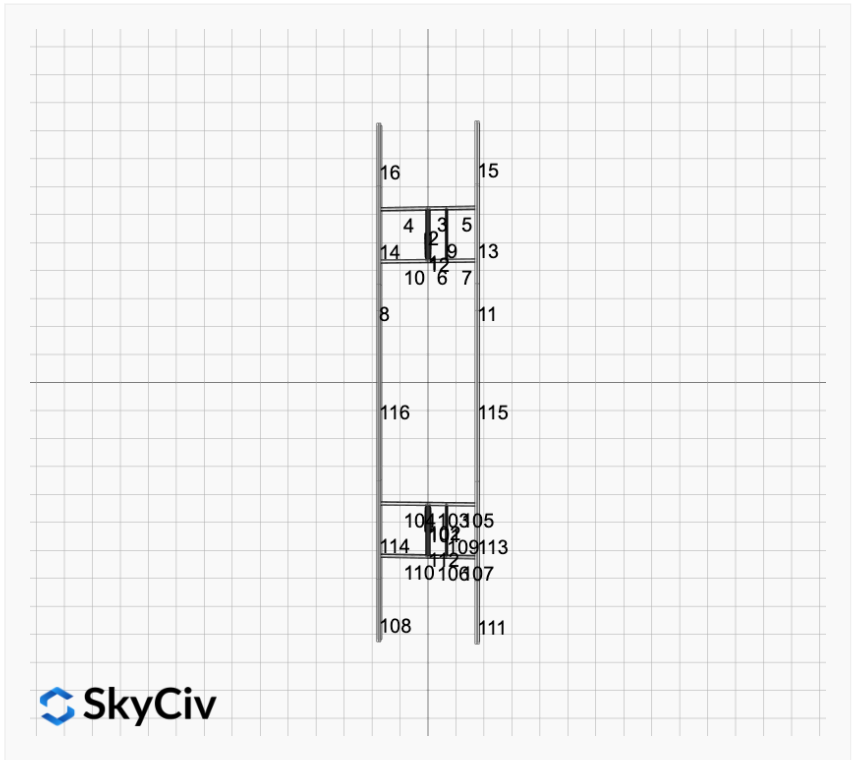
AutoDesigner Input

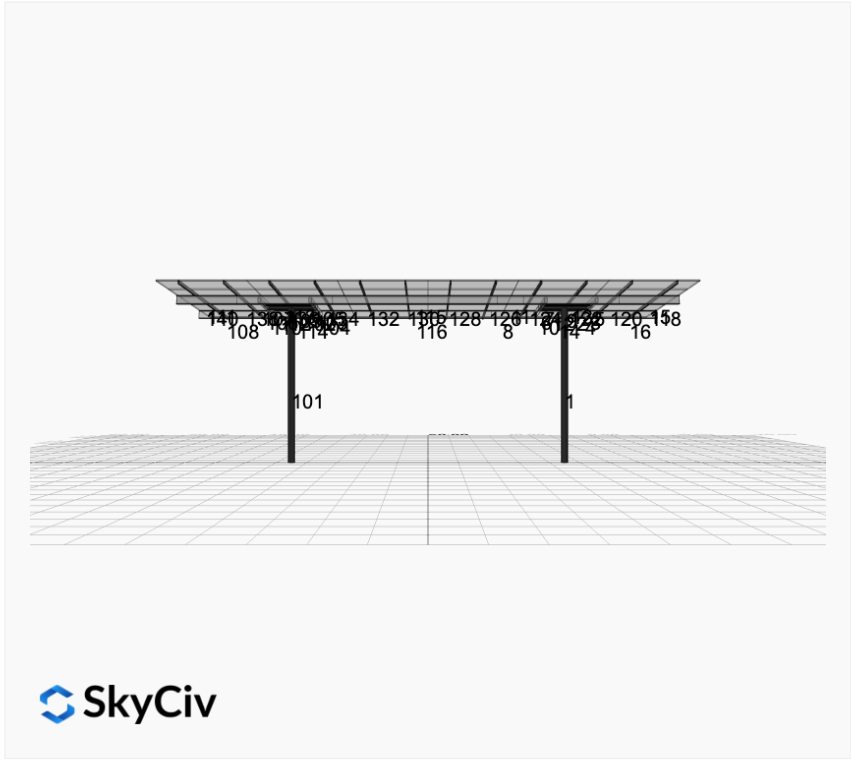
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Design Notes:

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

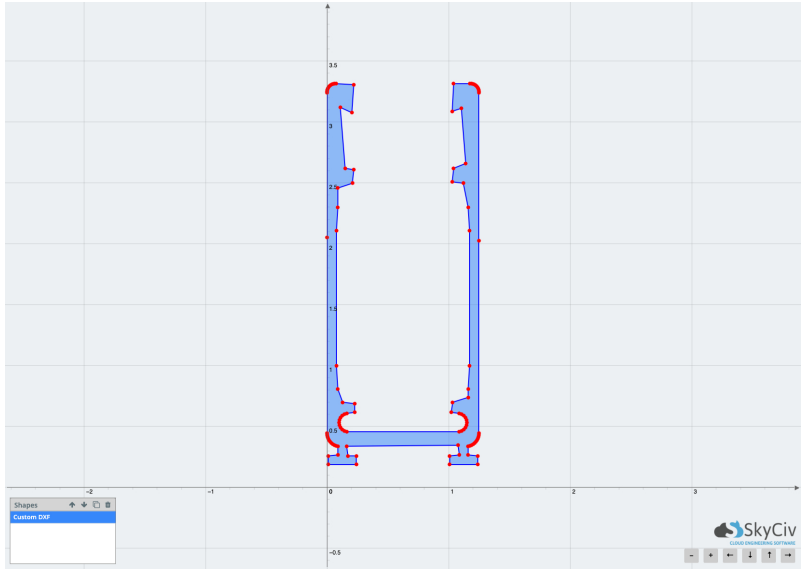






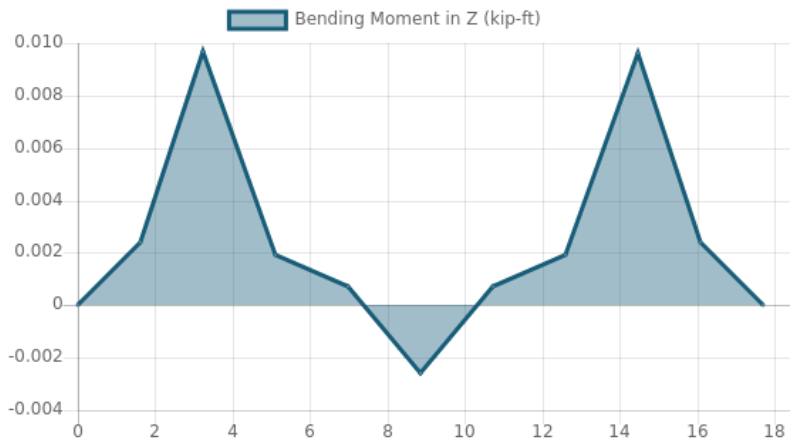
Rail Design Check

Rail Length: 17.708333333333332 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.375 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Wind uplift Case A (X): 0.0000 kip/ft
Wind uplift Case A (Y): 0.0587 kip/ft
Wind uplift Case A: 0.0000 kip/ft
Wind uplift Case B: 0.0000 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1347 kip/ft

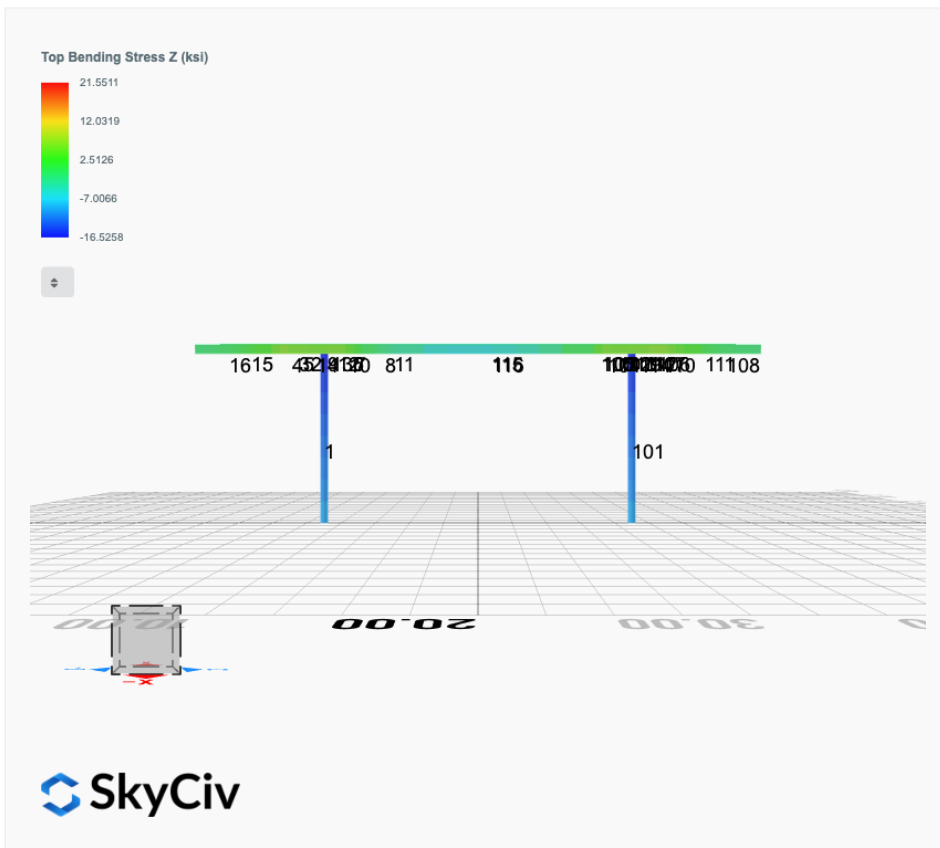
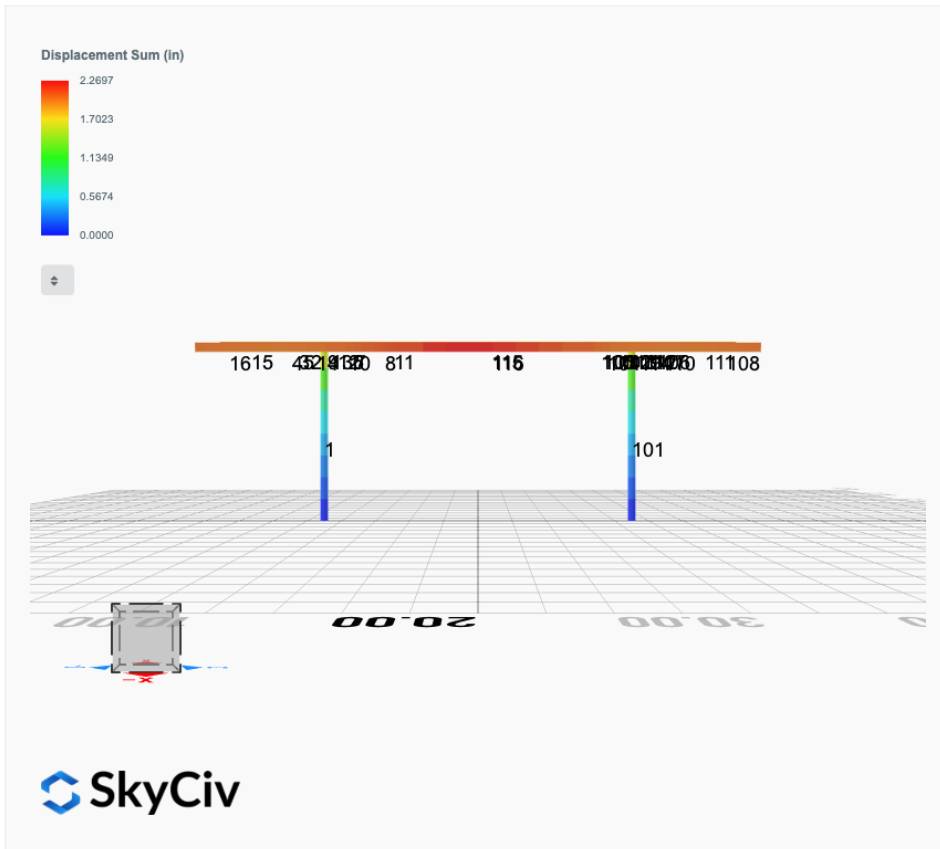


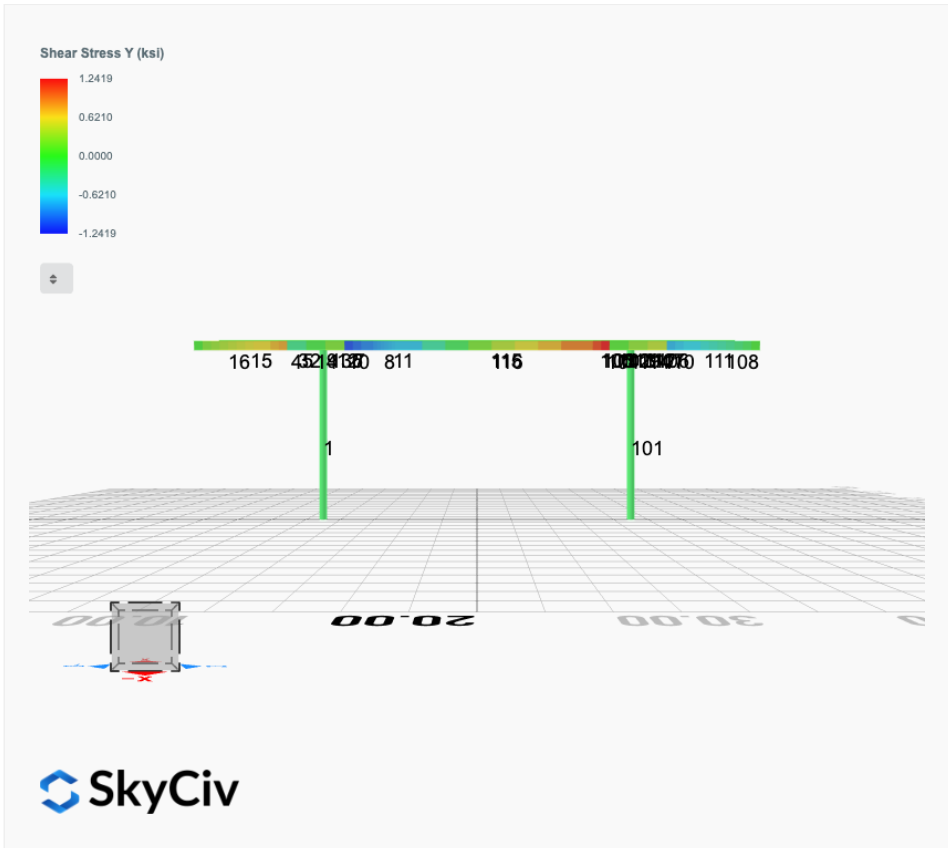
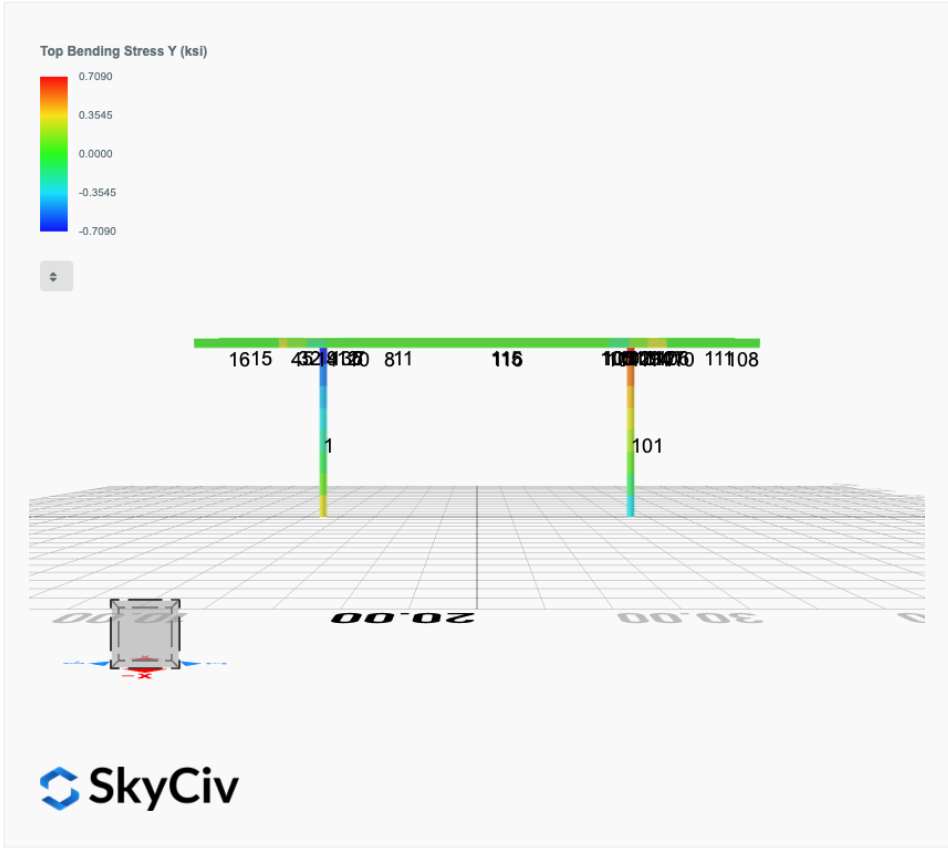
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	15.06412717	0.437	PASS
Material Yield	34.5	15.06412717	0.437	PASS
Material Strength	37	15.06412717	0.407	PASS

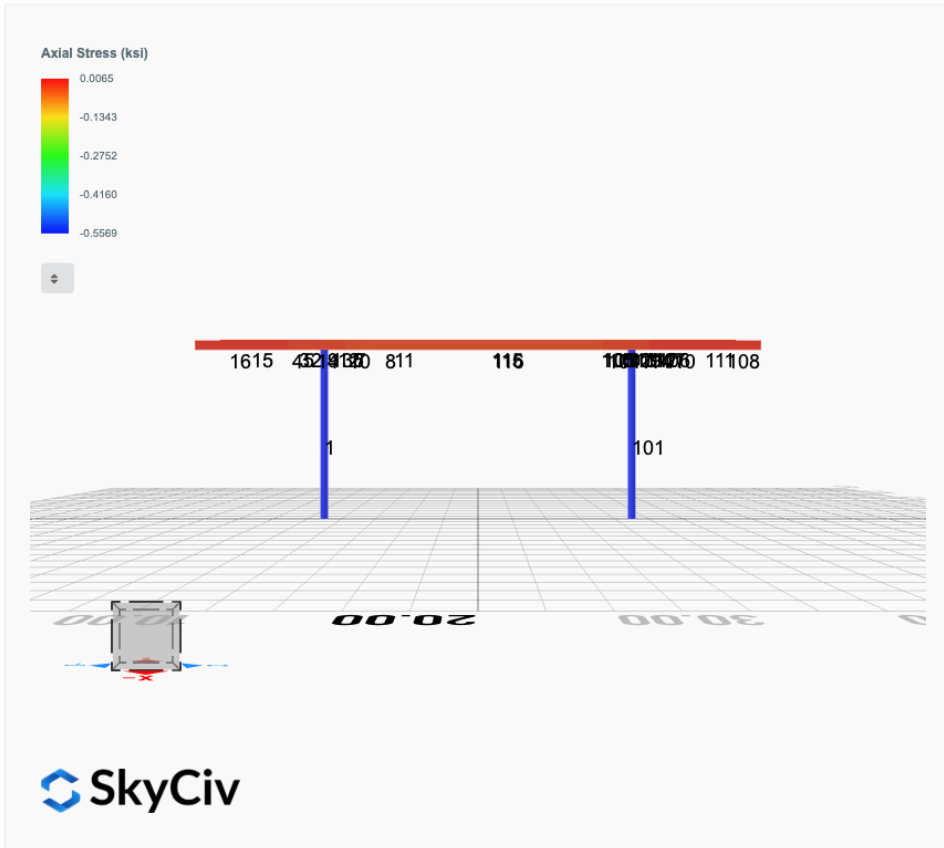
Member 1, ULS: 1. 1.4D



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	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 2. D + L	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 3. D + (S or Lr or R)	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 3. D + (S or Lr or R)	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 5b. D + 0.7E	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.4666	0.0376	0.1547	-0.0050	0.0294
ULS: 8. 0.6D + 0.7E	0.0000	1.4799	0.0226	0.0928	-0.0030	0.0176
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6046	9.3775	0.1686	0.6965	-0.0403	10.2123
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6046	9.3775	0.1686	0.6965	-0.0403	10.2123
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1632	0.6017	0.0030	0.0133	0.0043	4.7229
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3839	-1.9213	-0.0455	-0.1836	0.0174	-18.7301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4535	7.6498	0.1359	0.5611	-0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4535	7.6498	0.1359	0.5611	-0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1224	1.0679	0.0117	0.0487	0.0019	3.5495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2879	-0.8244	-0.0247	-0.0990	0.0118	-14.0403
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4535	7.6498	0.1359	0.5611	-0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4535	7.6498	0.1359	0.5611	-0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1224	1.0679	0.0117	0.0487	0.0019	3.5495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2879	-0.8244	-0.0247	-0.0990	0.0118	-14.0403
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6046	8.3909	0.1536	0.6347	-0.0382	10.2005
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6046	8.3909	0.1536	0.6347	-0.0382	10.2005
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1632	-0.3849	-0.0120	-0.0486	0.0063	4.7111
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3839	-2.9080	-0.0606	-0.2454	0.0194	-18.7419

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	14.4781
Shear X	-1.0077
Shear Z	0.2641
Moment X	1.0955
Moment Y (Twist)	0.0649
Moment Z	32.4131

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	9.3775
Shear X	-0.6046
Shear Z	0.1686
Moment X	0.6965
Moment Y (Twist)	0.0403
Moment Z	18.7419

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	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 2. D + L	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 3. D + (S or Lr or R)	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 3. D + (S or Lr or R)	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 5b. D + 0.7E	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.4666	-0.0376	-0.1547	0.0051	0.0294
ULS: 8. 0.6D + 0.7E	-0.0000	1.4799	-0.0226	-0.0928	0.0030	0.0176
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6046	9.3775	-0.1686	-0.6965	0.0403	10.2123
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6046	9.3775	-0.1686	-0.6965	0.0403	10.2123
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1632	0.6017	-0.0030	-0.0133	-0.0043	4.7229
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3839	-1.9213	0.0455	0.1836	-0.0174	-18.7301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4535	7.6498	-0.1359	-0.5611	0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4535	7.6498	-0.1359	-0.5611	0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1224	1.0679	-0.0117	-0.0487	-0.0019	3.5495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2879	-0.8244	0.0247	0.0990	-0.0118	-14.0403
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4535	7.6498	-0.1359	-0.5611	0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4535	7.6498	-0.1359	-0.5611	0.0315	7.6666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1224	1.0679	-0.0117	-0.0487	-0.0019	3.5495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2879	-0.8244	0.0247	0.0990	-0.0118	-14.0403
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6046	8.3909	-0.1536	-0.6347	0.0382	10.2005
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6046	8.3909	-0.1536	-0.6347	0.0382	10.2005
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1632	-0.3849	0.0120	0.0486	-0.0063	4.7111
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3839	-2.9080	0.0606	0.2454	-0.0194	-18.7419

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	14.4781
Shear X	-1.0077
Shear Z	-0.2641
Moment X	-1.0956
Moment Y (Twist)	0.0649
Moment Z	32.4134

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	9.3775
Shear X	-0.6046
Shear Z	-0.1686
Moment X	-0.6965
Moment Y (Twist)	0.0403
Moment Z	18.7419

Project Details

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

User Name: sales@mtsolar.us
 Project Name: W12116 5x6 Revb CU
 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				
7	6in Pipe Sch 40	6.63	0.28				

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

115	133.20	46.28	12.28	6.12	40.24	43.62
116	133.20	46.28	12.27	6.12	40.24	43.62

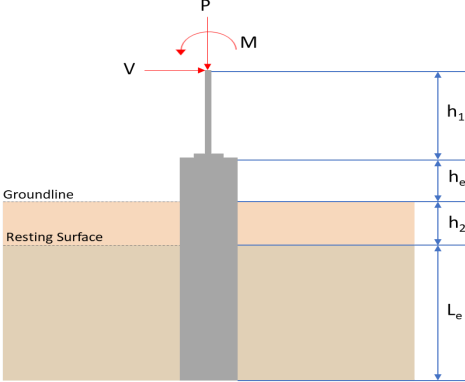
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.236	0.766	0.054	0.013	0.004	0.784	#16	0.717	Not Required	Pass
2	0.001	0.525	0.043	0.112	0.008	0.568	#13	0.053	Not Required	Pass
3	0.001	0.817	0.009	0.082	0.003	0.827	#13	0.045	Not Required	Pass
4	0.001	0.753	0.020	0.076	0.005	0.772	#13	0.120	Not Required	Pass
5	0.001	0.507	0.010	0.082	0.003	0.511	#13	0.074	Not Required	Pass
6	0.001	0.919	0.014	0.093	0.003	0.934	#13	0.045	Not Required	Pass
7	0.001	0.570	0.017	0.092	0.005	0.579	#13	0.074	Not Required	Pass
8	0.001	0.095	0.013	0.063	0.001	0.097	#13	0.095	Not Required	Pass
9	0.001	0.111	0.019	0.002	0.001	0.130	#13	0.204	Not Required	Pass
10	0.001	0.851	0.014	0.086	0.003	0.859	#13	0.080	Not Required	Pass
11	0.001	0.101	0.013	0.069	0.001	0.102	#13	0.095	Not Required	Pass
12	0.000	0.628	0.046	0.127	0.009	0.674	#13	0.053	Not Required	Pass
13	0.001	0.337	0.028	0.084	0.001	0.352	#13	0.286	Not Required	Pass
14	0.001	0.318	0.028	0.078	0.001	0.332	#13	0.190	Not Required	Pass
15	0.000	0.126	0.011	0.043	0.001	0.136	#13	Not Required	Not Required	Pass
16	0.000	0.117	0.011	0.040	0.001	0.126	#13	Not Required	Not Required	Pass
101	0.236	0.766	0.054	0.013	0.004	0.784	#16	0.717	Not Required	Pass
102	0.000	0.628	0.046	0.127	0.009	0.674	#13	0.053	Not Required	Pass
103	0.001	0.919	0.014	0.093	0.003	0.934	#13	0.045	Not Required	Pass
104	0.001	0.851	0.014	0.086	0.003	0.859	#13	0.080	Not Required	Pass
105	0.001	0.570	0.017	0.092	0.005	0.579	#13	0.074	Not Required	Pass
106	0.001	0.817	0.009	0.082	0.003	0.827	#13	0.045	Not Required	Pass
107	0.001	0.507	0.010	0.082	0.003	0.511	#13	0.074	Not Required	Pass
108	0.000	0.117	0.011	0.040	0.001	0.126	#13	Not Required	Not Required	Pass
109	0.001	0.111	0.019	0.002	0.001	0.130	#13	0.204	Not Required	Pass
110	0.001	0.753	0.020	0.076	0.005	0.772	#13	0.120	Not Required	Pass
111	0.000	0.126	0.011	0.043	0.001	0.136	#13	Not Required	Not Required	Pass
112	0.001	0.525	0.043	0.112	0.008	0.568	#13	0.053	Not Required	Pass
113	0.001	0.337	0.028	0.084	0.001	0.352	#13	0.190	Not Required	Pass
114	0.001	0.318	0.028	0.078	0.001	0.332	#13	0.286	Not Required	Pass
115	0.003	0.710	0.015	0.069	0.001	0.723	#13	0.601	Not Required	Pass
116	0.003	0.661	0.016	0.063	0.001	0.677	#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 _n	Buckling modification factor (from all load combinations)

L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
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 $b = 60$ in - Pile width $D = 60$ in - Pile depth $L = 5.75$ 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_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="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.378</td> <td>14.478</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.605</td> <td>-1.008</td> </tr> <tr> <td>V_z (kip)</td> <td>0.169</td> <td>0.264</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.697</td> <td>1.096</td> </tr> <tr> <td>M_z (kipft)</td> <td>18.742</td> <td>32.413</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)	9.378	14.478	V_x (kip)	-0.605	-1.008	V_z (kip)	0.169	0.264	M_x (kipft)	0.697	1.096	M_z (kipft)	18.742	32.413	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.605 \text{ kip})}{1.57 \times (60 \text{ in})}$ $H_o = -0.07707 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(18.742 \text{ kipft}) + ((-0.605 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.3875 \text{ kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:

$L_{e,x} = 5.4915 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.169 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.021529 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.697 \text{ kipft}) + ((0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.08879 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1454 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4915 \text{ ft}), (2.1454 \text{ ft})]$$

$$L_{e,req} = 5.492 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.492 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95513$$

Status: **PASS**
Ratio: **0.960**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.378 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.37512 \text{ kip/ft}^2$$

$$q = 0.37512 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37512 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18756$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.15$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.07707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.3875 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3875 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.3875 \text{ kipft/ft})) + (4 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8861 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3875 \text{ kipft/ft})) + (3 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.3875 \text{ kipft/ft})) + (2 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24425 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.3875 \text{ kipft/ft})) + ((-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.78613 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8861 \text{ ft})}{2}$$

$$p_a = 0.29146 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24425 \text{ kip/ft}^2)}{(0.29146 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.83803$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.840**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.78613 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91145$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = 0.021529 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.08879 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.08879 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.021529 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.08879 \text{ kipft/ft})) + (4 \times (0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0642 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.08879 \text{ kipft/ft})) + (3 \times (0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.08879 \text{ kipft/ft})) + (2 \times (0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.023298 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.08879 \text{ kipft/ft})) + ((0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.054691 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.0642 \text{ ft})}{2}$ $p_a = 0.30481 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.023298 \text{ kip/ft}^2)}{(0.30481 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.076433$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.080

$$Ratio = \frac{(0.054691 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.06341$$

Status: **PASS**
Ratio: **0.060**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.008 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.12841 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.413 \text{ kipft}) + ((-1.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.129 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.129 \text{ kipft/ft})}{(-0.12841 \text{ kip/ft})}$$

$$E = 32.156 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.129 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.12841 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.129 \text{ kipft/ft})) + (4 \times (-0.12841 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8844 \text{ ft}$$

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

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

$$V_{max} = ((-0.12841 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8844 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8844 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7911 \text{ kip}$$

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

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

$$M_{max} = ((-0.12841 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(32.156 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.185 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.264 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.033631 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.096 \text{ kipft}) + ((0.264 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.13962 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.13962 \text{ kipft/ft})}{(0.033631 \text{ kip/ft})}$$

$$E = 4.1515 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13962 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.033631 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.13962 \text{ kipft/ft})) + (4 \times (0.033631 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0634 \text{ ft}$$

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

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

$$V_{max} = ((0.033631 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0634 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0634 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.32628 \text{ kip}$$

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

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

$$M_{max} = ((0.033631 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.1515 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85137 \text{ kipft}$$

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,
 $A_g = 3600 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.478 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.7 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.7 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

$$\text{Ratio} = 0.96007$$

Status: **PASS**
Ratio: **0.960**

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,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min} (D, b)]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min} ((60 \text{ in}), (60 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

Summary:

Main reinforcement: **22 - #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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$$

$$\phi P_N = 4181.1 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.478 \text{ kip})}{(4181.1 \text{ kip})}$$

$$\text{Ratio} = 0.0034627$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 60 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (60 \text{ in})$$

$$d = 48 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.58722$$

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

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.478 \text{ kip} \rightarrow 14478 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + \frac{(14478 \text{ lbf})}{6 \times (3600 \text{ in}^2)} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.05 \text{ kip}$$

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)

$$V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

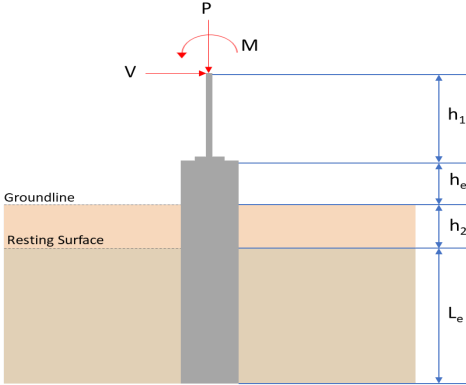
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(422.8 \text{ kip}), (171.05 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.05 \text{ kip}$$

<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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$ $V_{s,a} = 1152 \text{ kip}$ <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> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 63.617 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(1152 \text{ kip}), (63.617 \text{ kip})]$ $V_s = 63.617 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((171.05 \text{ kip}) + (63.617 \text{ kip}))$ $\phi V_n = 152.53 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 6.7911 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(6.7911 \text{ kip})}{(152.53 \text{ kip})}$ $\text{Ratio} = 0.044522$ <p>Considering z-direction:</p> <p>$V_{max} = 0.32628 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.32628 \text{ kip})}{(152.53 \text{ kip})}$ $\text{Ratio} = 0.0021391$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$ $S_m = 36000 \text{ in}^3$	

<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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 35999.999 \text{in}^3$ $\phi M_{n,1} = 487.500 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (36000 \text{in}^3)$ $\phi M_{n,2} = 4143.7 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(487.5 \text{kipft}), (4143.7 \text{kipft})]$ $\phi M_n = 487.5 \text{kipft}$ <p>Considering x-direction: $M_{max} = 19.185 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.185 \text{kipft})}{(487.5 \text{kipft})}$ $\text{Ratio} = 0.039355$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 0.85137 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.85137 \text{kipft})}{(487.5 \text{kipft})}$ $\text{Ratio} = 0.0017464$	<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 $b = 60$ in - Pile width $D = 60$ in - Pile depth $L = 5.75$ 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_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="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.378</td> <td>14.478</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.605</td> <td>-1.008</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.169</td> <td>-0.264</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.697</td> <td>-1.096</td> </tr> <tr> <td>M_z (kipft)</td> <td>18.742</td> <td>32.413</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)	9.378	14.478	V_x (kip)	-0.605	-1.008	V_z (kip)	-0.169	-0.264	M_x (kipft)	-0.697	-1.096	M_z (kipft)	18.742	32.413	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.605 \text{ kip})}{1.57 \times (60 \text{ in})}$ $H_o = -0.07707 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(18.742 \text{ kipft}) + ((-0.605 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.3875 \text{ kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:

$L_{e,x} = 5.4915 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.169 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.021529 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.697 \text{ kipft}) + ((-0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.08879 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6994 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4915 \text{ ft}), (1.6994 \text{ ft})]$$

$$L_{e,req} = 5.492 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.492 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95513$$

Status: **PASS**
Ratio: **0.960**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.378 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.37512 \text{ kip/ft}^2$$

$$q = 0.37512 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37512 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18756$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.15$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.07707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.3875 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3875 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.3875 \text{ kipft/ft})) + (4 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8861 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3875 \text{ kipft/ft})) + (3 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.3875 \text{ kipft/ft})) + (2 \times (-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24425 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.3875 \text{ kipft/ft})) + ((-0.07707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.78613 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8861 \text{ ft})}{2}$$

$$p_a = 0.29146 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24425 \text{ kip/ft}^2)}{(0.29146 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.83803$$

p_a - Allowable lateral soil pressure at depth L_e ,

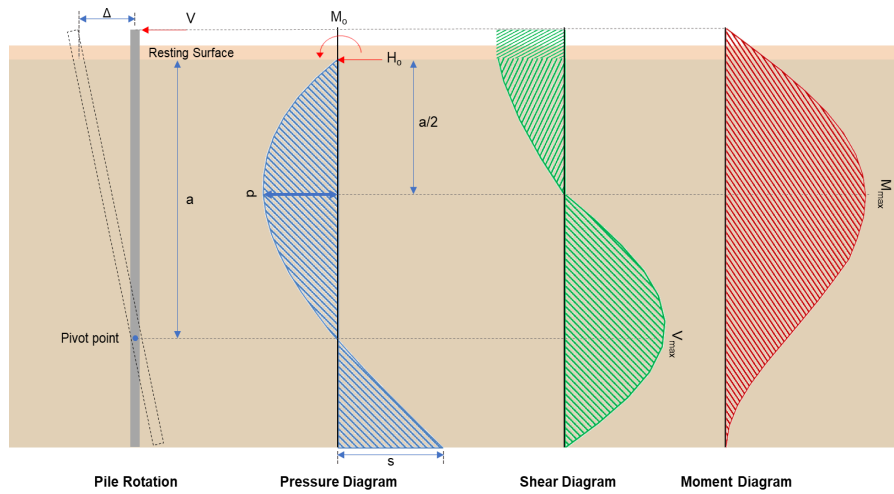
Status: **PASS**
Ratio: **0.840**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.78613 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91145$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = -0.021529 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.08879 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.08879 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.021529 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.08879 \text{ kipft/ft})) + (4 \times (-0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0642 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.08879 \text{ kipft/ft})) + (3 \times (-0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.08879 \text{ kipft/ft})) + (2 \times (-0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.00031723 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.08879 \text{ kipft/ft})) + ((-0.021529 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.0097615 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.0642 \text{ ft})}{2}$ $p_a = 0.30481 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00031723 \text{ kip/ft}^2)}{(0.30481 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0010408$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(0.0097615 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.011318$$

Status: **PASS**
Ratio: **0.010**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.008 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.12841 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.413 \text{ kipft}) + ((-1.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.129 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.129 \text{ kipft/ft})}{(-0.12841 \text{ kip/ft})}$$

$$E = 32.156 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.129 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.12841 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.129 \text{ kipft/ft})) + (4 \times (-0.12841 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8844 \text{ ft}$$

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

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

$$V_{max} = ((-0.12841 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8844 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8844 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7911 \text{ kip}$$

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

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

$$M_{max} = ((-0.12841 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(32.156 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (32.156 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8844 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.185 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.264 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.033631 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.096 \text{ kipft}) + ((-0.264 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.13962 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.13962 \text{ kipft/ft})}{(-0.033631 \text{ kip/ft})}$$

$$E = 4.1515 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13962 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.033631 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.13962 \text{ kipft/ft})) + (4 \times (-0.033631 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0634 \text{ ft}$$

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

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

$$V_{max} = ((-0.033631 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0634 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0634 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.32628 \text{ kip}$$

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

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

$$M_{max} = ((-0.033631 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.1515 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0634 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85137 \text{ kipft}$$

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,
 $A_g = 3600 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.478 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.7 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.7 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

$$\text{Ratio} = 0.96007$$

Status: **PASS**
Ratio: **0.960**

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,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min} (D, b)]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min} ((60 \text{ in}), (60 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

Summary:

Main reinforcement: **22 - #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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$$

$$\phi P_N = 4181.1 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.478 \text{ kip})}{(4181.1 \text{ kip})}$$

$$\text{Ratio} = 0.0034627$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 60 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (60 \text{ in})$$

$$d = 48 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.58722$$

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

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.478 \text{ kip} \rightarrow 14478 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + \frac{(14478 \text{ lbf})}{6 \times (3600 \text{ in}^2)} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.05 \text{ kip}$$

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)

$$V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(422.8 \text{ kip}), (171.05 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.05 \text{ kip}$$

<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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$ $V_{s,a} = 1152 \text{ kip}$ <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> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 63.617 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(1152 \text{ kip}), (63.617 \text{ kip})]$ $V_s = 63.617 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((171.05 \text{ kip}) + (63.617 \text{ kip}))$ $\phi V_n = 152.53 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 6.7911 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(6.7911 \text{ kip})}{(152.53 \text{ kip})}$ $\text{Ratio} = 0.044522$ <p>Considering z-direction:</p> <p>$V_{max} = 0.32628 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.32628 \text{ kip})}{(152.53 \text{ kip})}$ $\text{Ratio} = 0.0021391$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$ $S_m = 36000 \text{ in}^3$	

<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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 35999.999 \text{in}^3$ $\phi M_{n,1} = 487.500 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (36000 \text{in}^3)$ $\phi M_{n,2} = 4143.7 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(487.5 \text{kipft}), (4143.7 \text{kipft})]$ $\phi M_n = 487.5 \text{kipft}$ <p>Considering x-direction: $M_{max} = 19.185 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.185 \text{kipft})}{(487.5 \text{kipft})}$ $\text{Ratio} = 0.039355$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 0.85137 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.85137 \text{kipft})}{(487.5 \text{kipft})}$ $\text{Ratio} = 0.0017464$	<p>Status: PASS Ratio: 0.000</p>