

Your Project Calculations



Project Name: PR04243BretWoz-JB-RevA

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=PR04243BretWoz-JB-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=KZAJHfj05BscuiZKrYM758rAyPkj3sqHCNntS81CiFPTp0DSv8aG1VMMVm1rrVNk

Array Specification

Product:	Beam
Unique ID:	2P-15-6TOP-SD-45-L-3Hx5W-J20C
Duty Classification:	SD
Module Width:	40.60 in
Module Length:	73.70in
Number of Rows:	3
Number of Columns:	5
Total Number of Modules:	15
Desired Tilt Angle:	25
Front Edge Clearance:	3
Total Array Height at Tilt:	7.32 ft
Total Frame Length:	30.00 ft
Frame Weight:	1040 lbs
Array Dimensions N/S:	10.28 ft
Array Dimensions E/W:	31.13 ft
Rail Length:	123.30 in
Rail Spacing:	3.07 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	5.17 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 6.75 ft
Foundation Volume:	3.534 y ³
Foundation Result:	PASSED
Mount Twist:	0.440476 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1072 Pilgrim Pass, Victor, NY 14564, USA
Wind Speed:	102 mph
Snow Load:	40 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.019793 ksf



Design Disclaimer

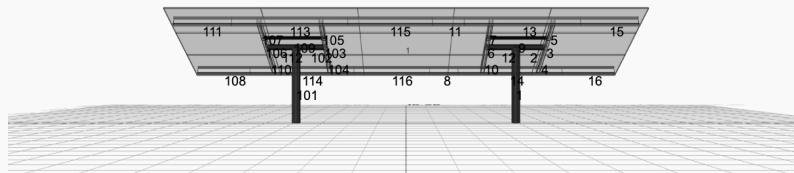
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

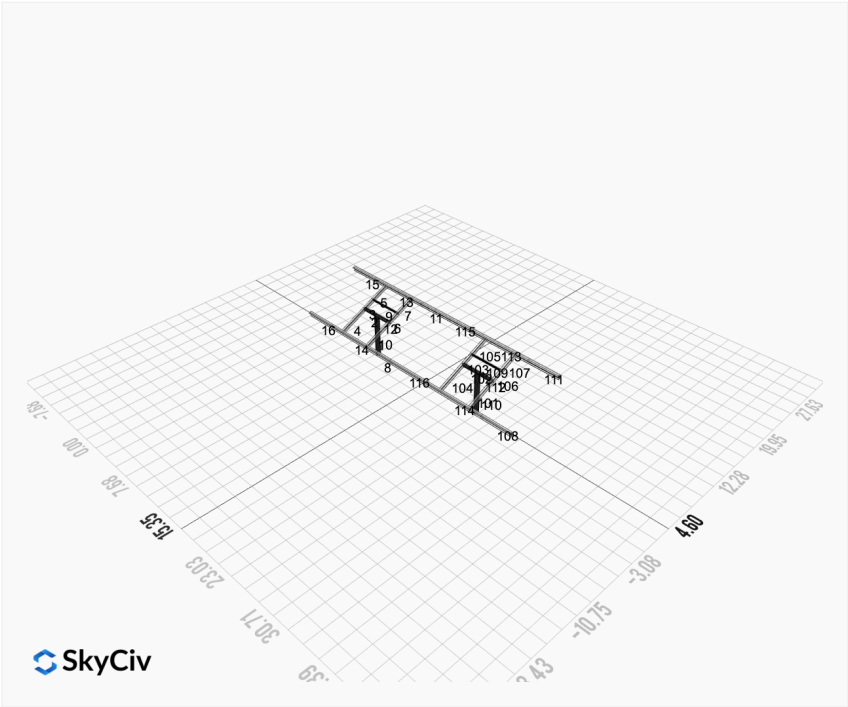
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Design Notes:

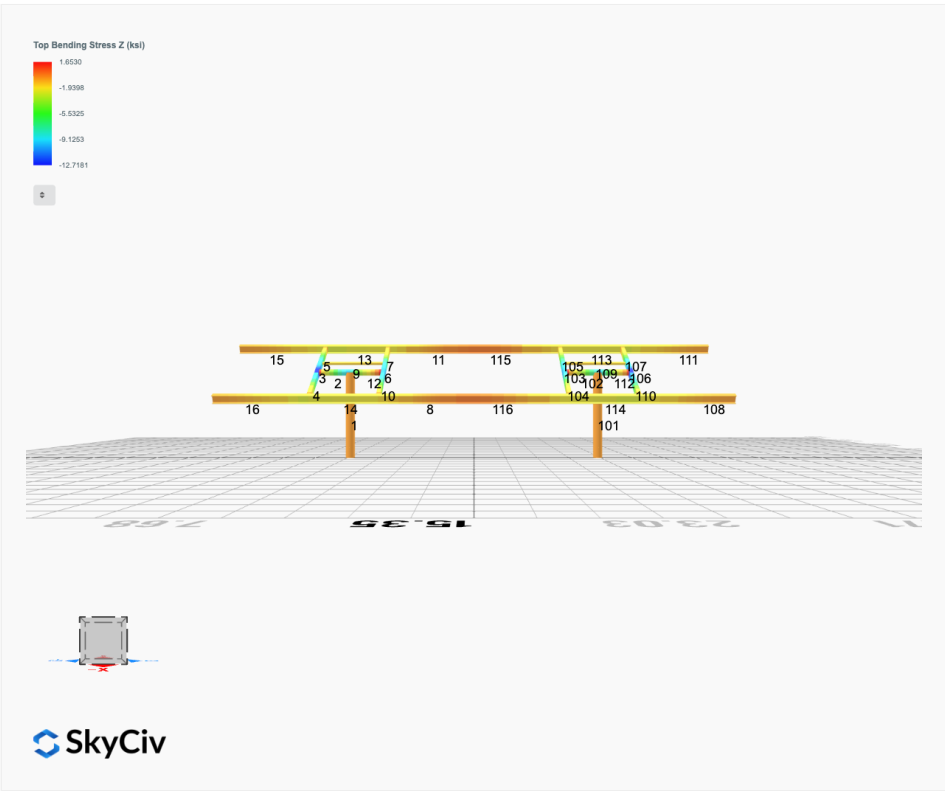
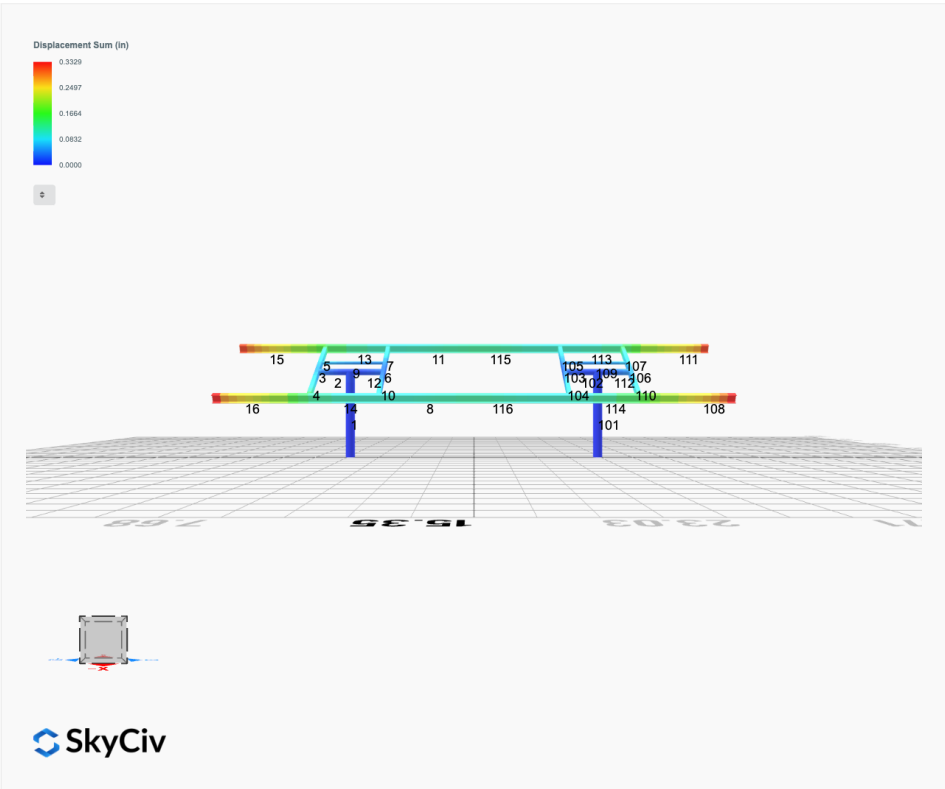
- AISC Deflection checks are set to L/1 due to structure design intent



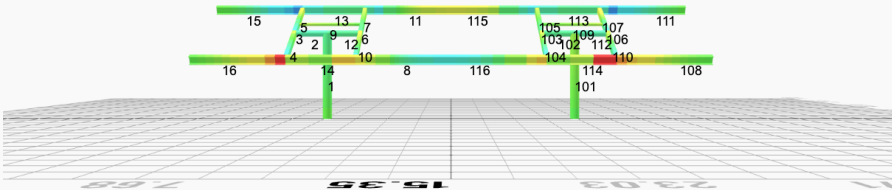




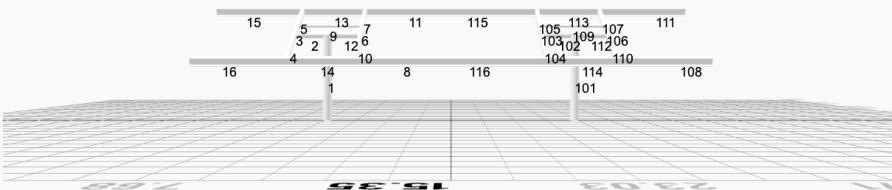
FEM Results (Envelope Worst Case for each member)



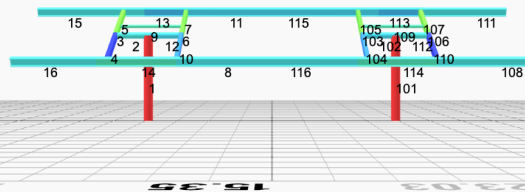
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



 SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.3096	-0.0510	-0.0283	0.0510	0.0198
ULS: 2. D + L	0.0000	1.3096	-0.0510	-0.0283	0.0510	0.0198
ULS: 3. D + (S or Lr or R)	-0.0000	4.0744	-0.1831	-0.1014	0.1826	0.0306
ULS: 3. D + (S or Lr or R)	0.0000	1.3096	-0.0510	-0.0283	0.0510	0.0198
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.3832	-0.1501	-0.0832	0.1497	0.0279
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.3096	-0.0510	-0.0283	0.0510	0.0198
ULS: 5b. D + 0.7E	0.0000	1.3096	-0.0510	-0.0283	0.0510	0.0198
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.3832	-0.1501	-0.0832	0.1497	0.0279
ULS: 8. 0.6D + 0.7E	0.0000	0.7858	-0.0306	-0.0170	0.0306	0.0119
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.2062	3.8963	-0.1942	-0.0988	0.2389	6.4931
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.2062	3.8963	-0.1942	-0.0988	0.2389	6.4931
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0554	-0.9538	0.0736	0.0332	-0.1132	-5.2164
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9047	-0.6305	0.0567	0.0248	-0.0908	-10.5148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9047	5.3233	-0.2575	-0.1360	0.2907	4.8829
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9047	5.3233	-0.2575	-0.1360	0.2907	4.8829
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7916	1.6857	-0.0566	-0.0371	0.0266	-3.8993
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.6785	1.9282	-0.0693	-0.0433	0.0434	-7.8730
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9047	3.2496	-0.1584	-0.0812	0.1919	4.8748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9047	3.2496	-0.1584	-0.0812	0.1919	4.8748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7916	-0.3880	0.0424	0.0178	-0.0721	-3.9074
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.6785	-0.1454	0.0297	0.0115	-0.0554	-7.8811
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.2062	3.3725	-0.1738	-0.0875	0.2185	6.4852
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.2062	3.3725	-0.1738	-0.0875	0.2185	6.4852
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0554	-1.4776	0.0940	0.0445	-0.1335	-5.2243
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9047	-1.1543	0.0771	0.0361	-0.1112	-10.5227

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	8.1508
Shear X	-2.0103
Shear Z	-0.3932
Moment X	-0.2100
Moment Y (Twist)	0.4405
Moment Z	17.6441

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	5.3233
Shear X	-1.2062
Shear Z	-0.2575
Moment X	-0.1360
Moment Y (Twist)	0.2907
Moment Z	10.5227

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.3096	0.0510	0.0283	-0.0510	0.0198
ULS: 2. D + L	-0.0000	1.3096	0.0510	0.0283	-0.0510	0.0198
ULS: 3. D + (S or Lr or R)	0.0000	4.0744	0.1831	0.1015	-0.1826	0.0306
ULS: 3. D + (S or Lr or R)	-0.0000	1.3096	0.0510	0.0283	-0.0510	0.0198
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.3832	0.1501	0.0832	-0.1497	0.0279
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.3096	0.0510	0.0283	-0.0510	0.0198
ULS: 5b. D + 0.7E	-0.0000	1.3096	0.0510	0.0283	-0.0510	0.0198

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.3832	0.1501	0.0832	-0.1497	0.0279
ULS: 8. 0.6D + 0.7E	-0.0000	0.7858	0.0306	0.0170	-0.0306	0.0119
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.2062	3.8963	0.1942	0.0988	-0.2389	6.4931
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.2062	3.8963	0.1942	0.0988	-0.2389	6.4931
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0554	-0.9538	-0.0736	-0.0331	0.1132	-5.2164
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9047	-0.6305	-0.0567	-0.0248	0.0908	-10.5148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9047	5.3233	0.2575	0.1360	-0.2907	4.8829
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9047	5.3233	0.2575	0.1360	-0.2907	4.8829
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7916	1.6857	0.0566	0.0371	-0.0266	-3.8993
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.6785	1.9282	0.0693	0.0433	-0.0434	-7.8730
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9047	3.2496	0.1584	0.0812	-0.1919	4.8748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9047	3.2496	0.1584	0.0812	-0.1919	4.8748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7916	-0.3880	-0.0424	-0.0178	0.0721	-3.9074
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.6785	-0.1454	-0.0297	-0.0115	0.0554	-7.8811
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.2062	3.3725	0.1738	0.0875	-0.2185	6.4852
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.2062	3.3725	0.1738	0.0875	-0.2185	6.4852
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0554	-1.4776	-0.0940	-0.0445	0.1335	-5.2243
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9047	-1.1543	-0.0771	-0.0361	0.1112	-10.5227

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	8.1509
Shear X	-2.0104
Shear Z	0.3932
Moment X	0.2102
Moment Y (Twist)	0.4405
Moment Z	17.6446

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	5.3233
Shear X	-1.2062
Shear Z	0.2575
Moment X	0.1360
Moment Y (Twist)	0.2907
Moment Z	10.5227

Project Details

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

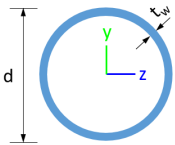
User Name: sales@mtsolar.us
Unit System: imperial



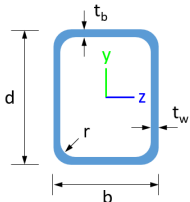
Design Input Information

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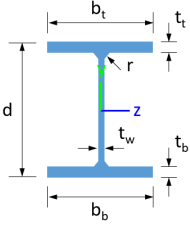
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

Section Dimensions							
							

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

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

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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D	
1	7	10.86	10.86	5.17	-	300	200	1	
2	4	1.30	1.30	2.00	-	300	200	1	
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18	300	200	1	
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.83,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.57	300	200	1	
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1	
6	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17	300	200	1	
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1	
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9	1	2.60	2.60	4.00	-	300	200	1	
10	15	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.73,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.89	300	200	1	
11	18	1.33	1.33	2.05	2.48,2.48,2.48,2.48,2.48,2.48,2.47,2.47,2.45,2.46,2.47,2.47,2.46,2.47,2.47,2.47,2.48,2.49,2.47,2.47,2.45,2.46,2.47,2.47,2.46,2.47	300	200	1	
12	4	1.30	1.30	2.00	-	300	200	1	
13	18	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.19,1.12,1.12,1.12,1.17,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.19,1.12,1.12,1.16	300	200	1	
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15	18	7.88	7.88	3.75	2.33,2.33	300	200	1	
16	18	7.88	7.88	3.75	2.33,2.33	300	200	1	
101	7	10.86	10.86	5.17	-	300	200	1	
102	4	1.30	1.30	2.00	-	300	200	1	
103	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17	300	200	1	
104	15	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.73,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.89	300	200	1	
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1	
106	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1	
107	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.67	300	200	1	

115	120.00	102.07	23.30	6.45	30.09	45.74
116	120.60	102.67	23.36	6.45	30.09	45.74

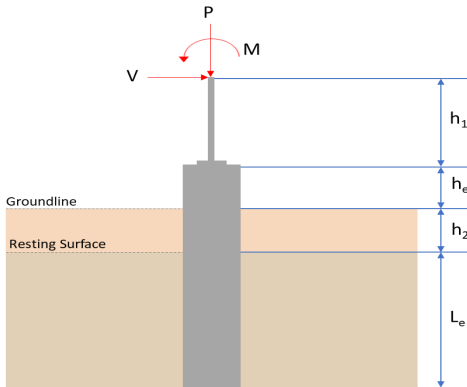
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.042	0.417	0.043	0.027	0.005	0.421	#32	0.290	Not Required	Pass
2	0.001	0.487	0.145	0.102	0.025	0.596	#21	0.052	Not Required	Pass
3	0.009	0.711	0.140	0.073	0.021	0.855	#21	0.044	Not Required	Pass
4	0.009	0.699	0.158	0.071	0.019	0.778	#21	0.078	Not Required	Pass
5	0.009	0.440	0.197	0.071	0.029	0.491	#21	0.073	Not Required	Pass
6	0.009	0.599	0.065	0.060	0.008	0.648	#21	0.044	Not Required	Pass
7	0.008	0.372	0.110	0.060	0.015	0.386	#21	0.073	Not Required	Pass
8	0.001	0.037	0.053	0.031	0.007	0.076	#21	0.088	Not Required	Pass
9	0.003	0.075	0.064	0.002	0.003	0.140	#21	0.198	Not Required	Pass
10	0.007	0.587	0.158	0.060	0.023	0.740	#21	0.078	Not Required	Pass
11	0.002	0.037	0.056	0.032	0.007	0.081	#21	0.088	Not Required	Pass
12	0.002	0.374	0.118	0.085	0.022	0.458	#21	0.052	Not Required	Pass
13	0.004	0.198	0.188	0.046	0.010	0.382	#21	0.265	Not Required	Pass
14	0.003	0.200	0.188	0.046	0.010	0.380	#21	0.177	Not Required	Pass
15	0.000	0.076	0.088	0.032	0.007	0.164	#21	Not Required	Not Required	Pass
16	0.000	0.075	0.088	0.031	0.007	0.163	#21	Not Required	Not Required	Pass
101	0.042	0.417	0.043	0.027	0.005	0.421	#32	0.290	Not Required	Pass
102	0.002	0.374	0.118	0.085	0.022	0.458	#21	0.052	Not Required	Pass
103	0.009	0.599	0.065	0.060	0.008	0.648	#21	0.044	Not Required	Pass
104	0.007	0.587	0.158	0.060	0.023	0.740	#21	0.078	Not Required	Pass
105	0.008	0.372	0.110	0.060	0.015	0.386	#21	0.073	Not Required	Pass
106	0.009	0.711	0.140	0.073	0.021	0.855	#21	0.044	Not Required	Pass
107	0.009	0.440	0.197	0.071	0.029	0.491	#21	0.073	Not Required	Pass
108	0.000	0.075	0.088	0.031	0.007	0.163	#21	Not Required	Not Required	Pass
109	0.003	0.075	0.064	0.002	0.003	0.140	#21	0.198	Not Required	Pass
110	0.009	0.699	0.158	0.071	0.019	0.778	#21	0.078	Not Required	Pass
111	0.000	0.076	0.088	0.032	0.007	0.164	#21	Not Required	Not Required	Pass
112	0.001	0.487	0.145	0.102	0.025	0.596	#21	0.052	Not Required	Pass
113	0.004	0.198	0.188	0.046	0.010	0.382	#21	0.177	Not Required	Pass
114	0.003	0.200	0.188	0.046	0.010	0.380	#21	0.265	Not Required	Pass
115	0.002	0.039	0.073	0.032	0.007	0.113	#21	0.235	Not Required	Pass
116	0.001	0.038	0.070	0.031	0.007	0.108	#21	0.235	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress
F _u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I _{yp}	Moment of inertia about the Y axes
I _{zp}	Moment of inertia about the Z axes
I _w	Warping constant
S _{yp}	Plastic section modulus about the Y axis
S _{zp}	Plastic section modulus about the Z axis
KL	Effective length
C _b	Buckling modification factor (from all load combinations)
L _b	Length between braced points

L_u	Length between brace points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: round D = 36 in - Pile diameter Le = 6.75 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>5.323</td><td>8.151</td></tr><tr><td>Vx (kip)</td><td>-1.206</td><td>-2.010</td></tr><tr><td>Vz (kip)</td><td>-0.258</td><td>-0.393</td></tr><tr><td>Mx (kipft)</td><td>-0.136</td><td>-0.210</td></tr><tr><td>Mz (kipft)</td><td>10.523</td><td>17.644</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.323	8.151	Vx (kip)	-1.206	-2.010	Vz (kip)	-0.258	-0.393	Mx (kipft)	-0.136	-0.210	Mz (kipft)	10.523	17.644	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mz (kipft)	10.523	17.644																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0\text{ ft}) + (0\text{ ft}) + (0\text{ ft})$ $H = 0\text{ ft}$ Considering x-direction: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.206\text{ kip})}{(36\text{ in})}$ $H_o = -0.402\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{D}$																											

	$M_o = \frac{(10.523 \text{ kipft}) + ((-1.206 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 3.5077 \text{ kipft/ft}$ 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: $L_{e,x} = 5.9823 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(-0.258 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.086 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.136 \text{ kipft}) + ((-0.258 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.045333 \text{ kipft/ft}$ Required depth of embedment in earth: $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,z} = 0.6662 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.9823 \text{ ft}), (0.6662 \text{ ft})]$ $L_{e,req} = 5.982 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.982 \text{ ft})}{(6.75 \text{ ft})}$ $\text{Ratio} = 0.88622$	Status: PASS Ratio: 0.890
	End-bearing Capacity (ASD) A - Pile cross-section area $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$ $A = 7.0686 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(5.323 \text{ kip})}{(7.0686 \text{ ft}^2)}$	

	$q = 0.75305 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.75305 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.37653$	Status: PASS Ratio: 0.380
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$ $L/D = 2.25$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.402 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.5077 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.5077 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (3.5077 \text{ kipft/ft})) + (4 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 4.6914 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (3.5077 \text{ kipft/ft})) + (3 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (3.5077 \text{ kipft/ft})) + (2 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = 0.17602 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (3.5077 \text{ kipft/ft})) + ((-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = 0.88987 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.6914 \text{ ft})}{2}$ $p_a = 0.35185 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.17602 \text{ kip/ft}^2)}{(0.35185 \text{ kip/ft}^2)}$ $Ratio = 0.50027$	Status: PASS Ratio: 0.500

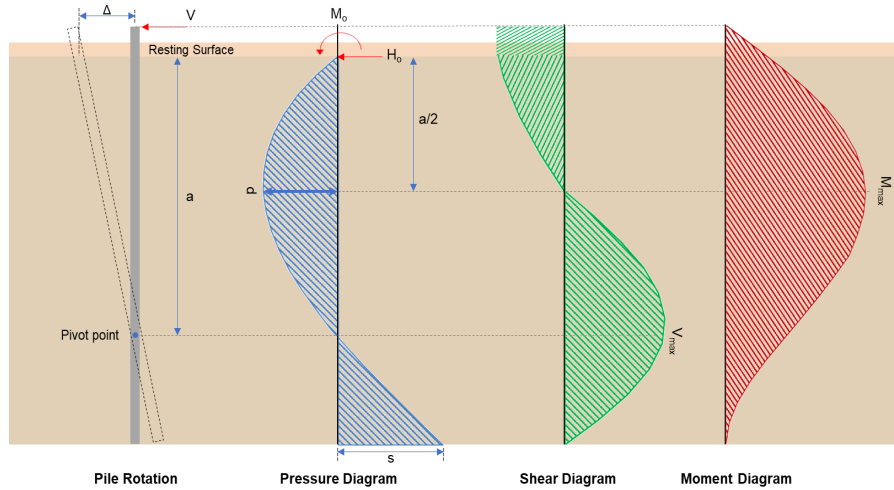
	p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.88987 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.87889$	Status: PASS Ratio: 0.880
	Considering z-direction: $H_o = -0.086 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.045333 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.045333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.086 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.045333 \text{ kipft/ft})) + (4 \times (-0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 5.0035 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (0.045333 \text{ kipft/ft})) + (3 \times (-0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.045333 \text{ kipft/ft})) + (2 \times (-0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = -0.061398 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (0.045333 \text{ kipft/ft})) + ((-0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = -0.10133 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0035 \text{ ft})}{2}$ $p_a = 0.37526 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.061398 \text{ kip/ft}^2)}{(0.37526 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.16361$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ Ratio - Lateral soil capacity	Status: PASS Ratio: -0.160

$$Ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(-0.10133 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = -0.10008$$

Status: **PASS**
Ratio: **-0.100**



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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-2.01 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.67 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.644 \text{ kipft}) + ((-2.01 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8813 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.8813 \text{ kipft/ft})}{(-0.67 \text{ kip/ft})}$$

$$E = 8.7781 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8813 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.67 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.8813 \text{ kipft/ft})) + (4 \times (-0.67 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6906 \text{ ft}$$

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

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

$$V_{max} = ((-0.67 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.7781 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6906 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (8.7781 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6906 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

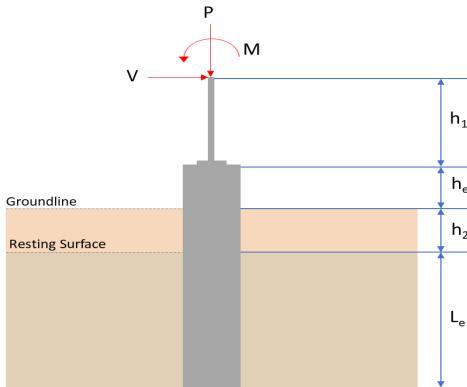
	$V_{max} = 5.9509 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-0.67 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(8.7781 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.7781 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.7781 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$ $M_{max} = 18.857 \text{ kipft}$	
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(-0.393 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.131 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.21 \text{ kipft}) + ((-0.393 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.07 \text{ kipft/ft}$ E - Distance from lateral load to resisting surface, $E = \frac{M_o}{H_o}$ $E = \frac{(0.07 \text{ kipft/ft})}{(-0.131 \text{ kip/ft})}$ $E = 0.53435 \text{ ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.07 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.131 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.07 \text{ kipft/ft})) + (4 \times (-0.131 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 5.0028 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = (H_o b) \left[1 - \left[3 \left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{L_e} \right)^2 \right] + \left[4 \left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{L_e} \right)^3 \right] \right]$ $V_{max} = ((-0.131 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (0.53435 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.0028 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (0.53435 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.0028 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$ $V_{max} = 0.323 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o b L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-0.131 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(0.53435 \text{ ft})}{(6.75 \text{ ft})} + \frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (0.53435 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (0.53435 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$	

		$M_{max} = 0.85724 \text{ kipft}$	
		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, $\alpha = 0.85$ - Alpha factor for axial strength, $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(8.151 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.119 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.119 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 6$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 1.8408 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $\text{Ratio} = 0.99533$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ Summary:	Status: PASS Ratio: 1.000

	Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.85 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$ $\phi P_N = 1253.9 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.151 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0065005$	Status: PASS Ratio: 0.010
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 36 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$	
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 $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.151 \text{ kip} \rightarrow 8151 \text{ lbf}$. $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8151 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 75.822 \text{ kip}$	
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) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,b} = 204.04 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(186.09 \text{ kip}), (75.822 \text{ kip}), (204.04 \text{ kip})]$	

	$V_c = 75.822 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, 22.5.1.2 $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.822 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.095 \text{ kip}$ Considering x-direction: $V_{max} = 5.9509 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(5.9509 \text{ kip})}{(74.095 \text{ kip})}$ $Ratio = 0.080315$ Considering z-direction: $V_{max} = 0.323 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.323 \text{ kip})}{(74.095 \text{ kip})}$ $Ratio = 0.0043592$	Status: PASS Ratio: 0.080 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

14.5.2.1b	$S_m = 4580.4 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ Considering x-direction: $M_{max} = 18.857 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(18.857 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.30402$	Status: PASS Ratio: 0.300
	Considering z-direction: $M_{max} = 0.85724 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.85724 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.01382$	Status: PASS Ratio: 0.010

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: round D = 36 in - Pile diameter Le = 6.75 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>5.323</td><td>8.151</td></tr><tr><td>Vx (kip)</td><td>-1.206</td><td>-2.010</td></tr><tr><td>Vz (kip)</td><td>0.258</td><td>0.393</td></tr><tr><td>Mx (kipft)</td><td>0.136</td><td>0.210</td></tr><tr><td>Mz (kipft)</td><td>10.523</td><td>17.645</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.323	8.151	Vx (kip)	-1.206	-2.010	Vz (kip)	0.258	0.393	Mx (kipft)	0.136	0.210	Mz (kipft)	10.523	17.645	
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Mx (kipft)	0.136	0.210																										
Mz (kipft)	10.523	17.645																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0\text{ ft}) + (0\text{ ft}) + (0\text{ ft})$ $H = 0\text{ ft}$ Considering x-direction: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.206\text{ kip})}{(36\text{ in})}$ $H_o = -0.402\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{D}$																											

	$M_o = \frac{(10.523 \text{ kipft}) + ((-1.206 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 3.5077 \text{ kipft/ft}$ 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: $L_{e,x} = 5.9823 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(0.258 \text{ kip})}{(36 \text{ in})}$ $H_o = 0.086 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.136 \text{ kipft}) + ((0.258 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.045333 \text{ kipft/ft}$ 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: $L_{e,z} = 3.149 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.9823 \text{ ft}), (3.149 \text{ ft})]$ $L_{e,req} = 5.982 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.982 \text{ ft})}{(6.75 \text{ ft})}$ $\text{Ratio} = 0.88622$	Status: PASS Ratio: 0.890
	End-bearing Capacity (ASD) A - Pile cross-section area $A = \pi \left(\frac{D}{2} \right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2} \right)^2$ $A = 7.0686 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(5.323 \text{ kip})}{(7.0686 \text{ ft}^2)}$	

	$q = 0.75305 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.75305 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.37653$	Status: PASS Ratio: 0.380
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$ $L/D = 2.25$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.402 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 3.5077 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (3.5077 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (3.5077 \text{ kipft/ft})) + (4 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 4.6914 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (3.5077 \text{ kipft/ft})) + (3 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (3.5077 \text{ kipft/ft})) + (2 \times (-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = 0.17602 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (3.5077 \text{ kipft/ft})) + ((-0.402 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = 0.88987 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.6914 \text{ ft})}{2}$ $p_a = 0.35185 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.17602 \text{ kip/ft}^2)}{(0.35185 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.50027$	Status: PASS Ratio: 0.500

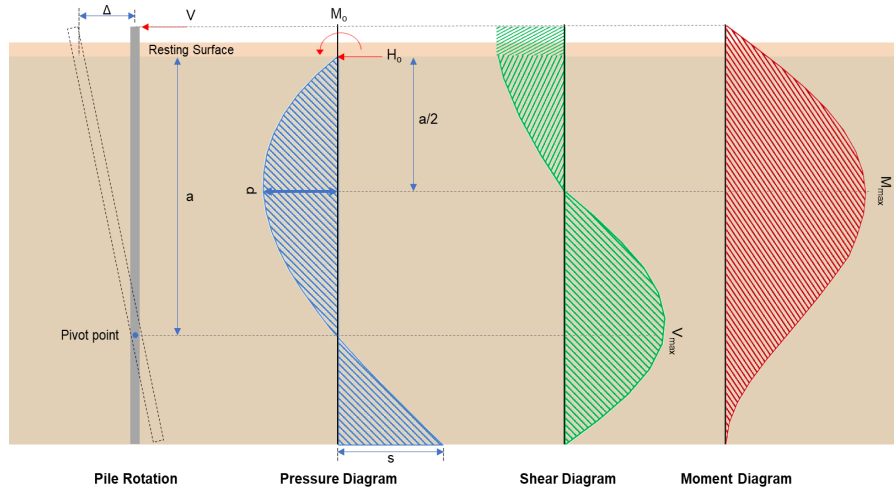
	p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.88987 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.87889$	Status: PASS Ratio: 0.880
	Considering z-direction: $H_o = 0.086 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.045333 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.045333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.086 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.045333 \text{ kipft/ft})) + (4 \times (0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 5.0035 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (0.045333 \text{ kipft/ft})) + (3 \times (0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.045333 \text{ kipft/ft})) + (2 \times (0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = 0.073702 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (0.045333 \text{ kipft/ft})) + ((0.086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = 0.13884 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0035 \text{ ft})}{2}$ $p_a = 0.37526 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.073702 \text{ kip/ft}^2)}{(0.37526 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.1964$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity	Status: PASS Ratio: 0.200

$$Ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.13884 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.13712$$

Status: **PASS**
Ratio: **0.140**



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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-2.01 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.67 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.645 \text{ kipft}) + ((-2.01 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8817 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.8817 \text{ kipft/ft})}{(-0.67 \text{ kip/ft})}$$

$$E = 8.7786 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8817 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.67 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.8817 \text{ kipft/ft})) + (4 \times (-0.67 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6906 \text{ ft}$$

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

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

$$V_{max} = ((-0.67 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.7786 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6906 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (8.7786 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6906 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 5.9512 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-0.67 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(8.7786 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.7786 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.7786 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6906 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$ $M_{max} = 18.858 \text{ kipft}$	
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(0.393 \text{ kip})}{(36 \text{ in})}$ $H_o = 0.0131 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.21 \text{ kipft}) + ((0.393 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.07 \text{ kipft/ft}$ E - Distance from lateral load to resisting surface, $E = \frac{M_o}{H_o}$ $E = \frac{(0.07 \text{ kipft/ft})}{(0.0131 \text{ kip/ft})}$ $E = 5.3435 \text{ ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.07 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.0131 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.07 \text{ kipft/ft})) + (4 \times (0.0131 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 5.0028 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = (H_o b) \left[1 - \left[3 \left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{L_e} \right)^2 \right] + \left[4 \left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{L_e} \right)^3 \right] \right]$ $V_{max} = ((0.0131 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.3435 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.0028 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.3435 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.0028 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$ $V_{max} = 0.323 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o b L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((0.0131 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(5.3435 \text{ ft})}{(6.75 \text{ ft})} + \frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.3435 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.3435 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.0028 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$	

		$M_{max} = 0.85724 \text{ kipft}$	
		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, $\alpha = 0.85$ - Alpha factor for axial strength, $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(8.151 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.119 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.119 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 6$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 1.8408 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $\text{Ratio} = 0.99533$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\emptyset$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ Summary:	Status: PASS Ratio: 1.000

	Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.85 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$ $\phi P_N = 1253.9 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.151 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0065005$	Status: PASS Ratio: 0.010
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 36 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$	
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 $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.151 \text{ kip} \rightarrow 8151 \text{ lbf}$. $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8151 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 75.822 \text{ kip}$	
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) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,b} = 204.04 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(186.09 \text{ kip}), (75.822 \text{ kip}), (204.04 \text{ kip})]$	

	$V_c = 75.822 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, 22.5.1.2 $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.822 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.095 \text{ kip}$ Considering x-direction: $V_{max} = 5.9512 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(5.9512 \text{ kip})}{(74.095 \text{ kip})}$ $Ratio = 0.080318$ Considering z-direction: $V_{max} = 0.323 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.323 \text{ kip})}{(74.095 \text{ kip})}$ $Ratio = 0.0043592$	Status: PASS Ratio: 0.080 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

	$S_m = 4580.4 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ Considering x-direction: $M_{max} = 18.858 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(18.858 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.30403$	Status: PASS Ratio: 0.300
	Considering z-direction: $M_{max} = 0.85724 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.85724 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.01382$	Status: PASS Ratio: 0.010