

Your Project Calculations



Project Name: MTSOLAR_FCB7DC7727K

S3D Model Link:

[https://platform.skyciv.com/structural?](https://platform.skyciv.com/structural?preload_name=MTSOLAR_FCB7DC7727K&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023)

[preload_name=MTSOLAR_FCB7DC7727K&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023](https://platform.skyciv.com/structural?preload_name=MTSOLAR_FCB7DC7727K&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023)

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=o6QvnrUr3MMbpRjiMAOhfcJtNNmvehrKE1hnB0d6ZGuAGHTq7WIhXmHSVYRD1Ebe

Array Specification

Product:	Beam
Unique ID:	2P-17-8TOP-SD-57-L-4Hx6W-D8AK
Duty Classification:	SD
Module Width:	44.65 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	6
Total Number of Modules:	24
Desired Tilt Angle:	60
Front Edge Clearance:	4
Total Array Height at Tilt:	16.96 ft
Total Frame Length:	34.00 ft
Frame Weight:	1518 lbs
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	34.40 ft
Rail Length:	180.60 in
Rail Spacing:	2.82 ft
Rail Check:	Not Checked

Support Specifications

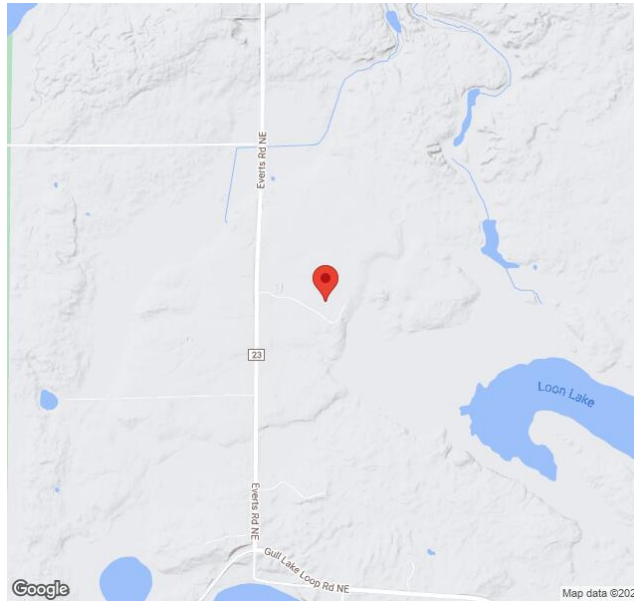
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	10.52 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 9.25 ft Pile 2: 9.25 ft
Foundation Volume:	4.843 y ³
Foundation Result:	PASSED
Mount Twist:	1.420458 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	23404 Everts Rd NE, Tenstrike, MN 56683, USA
Wind Speed:	101 mph
Snow Load:	70 psf
Design Uplift Pressure:	0.023204 ksf
Design Downforce Pressure:	-0.023204 ksf
Design Snow Pressure:	0.007697 ksf



Design Disclaimer

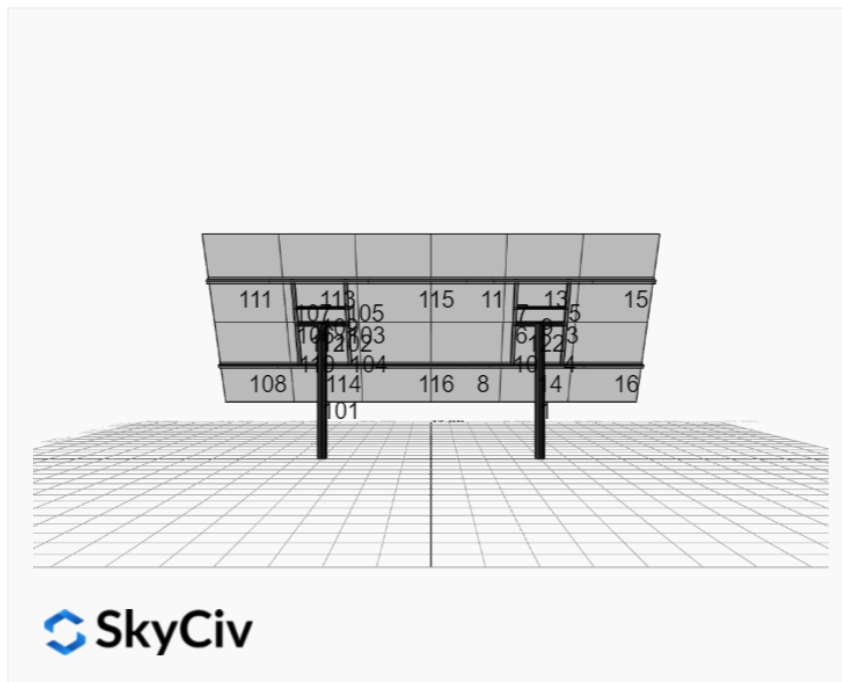
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

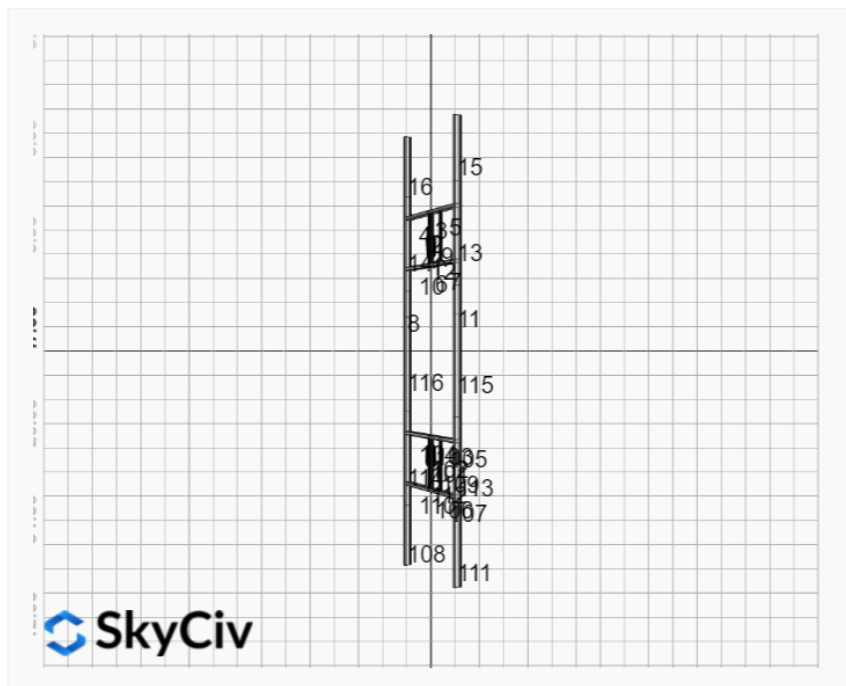
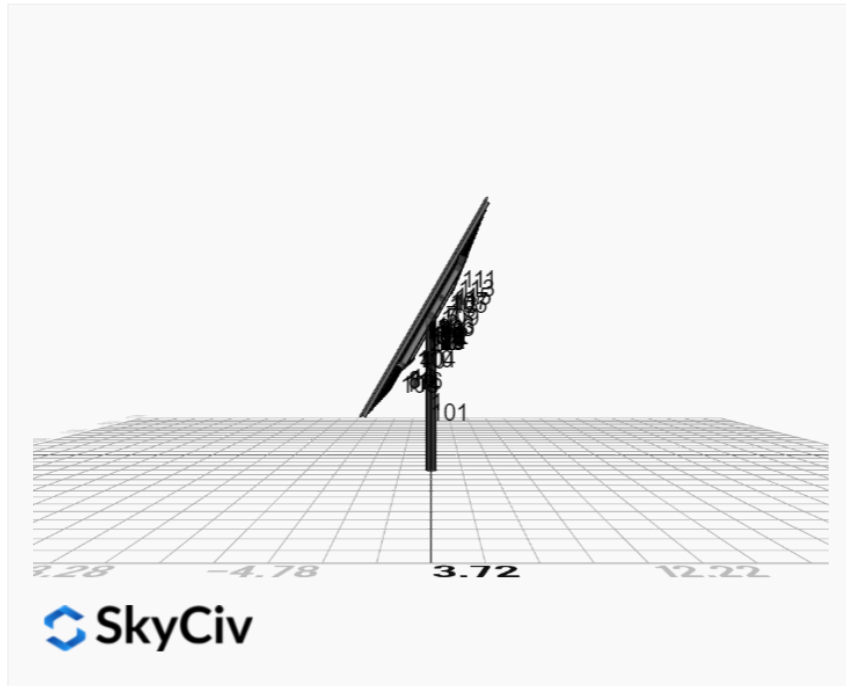
AutoDesigner Input

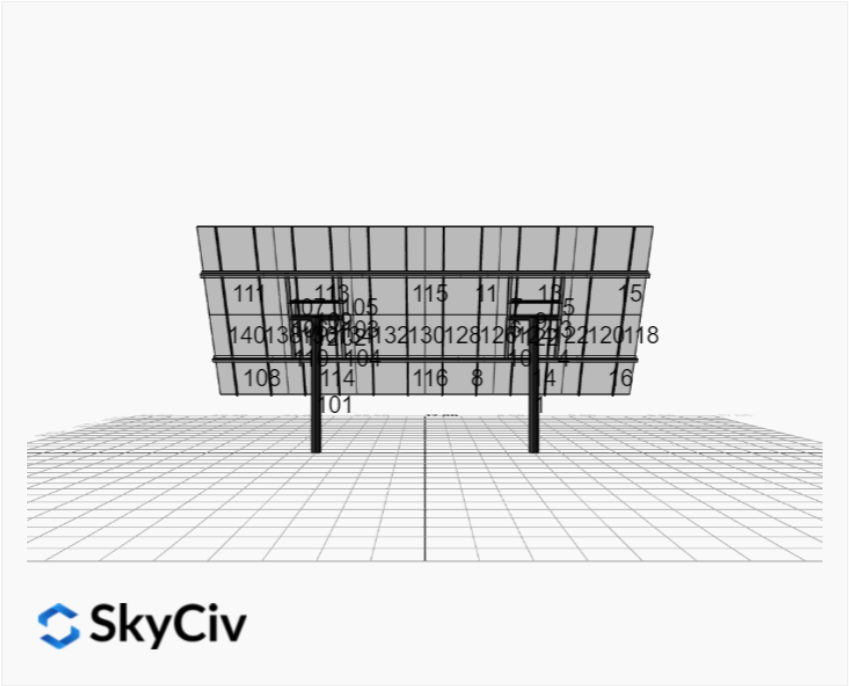
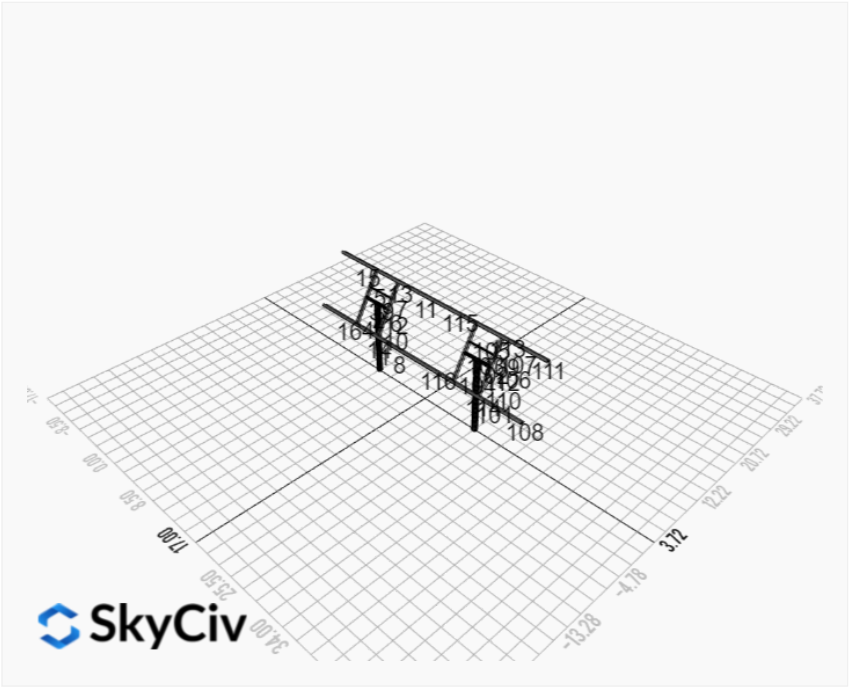
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Design Notes:

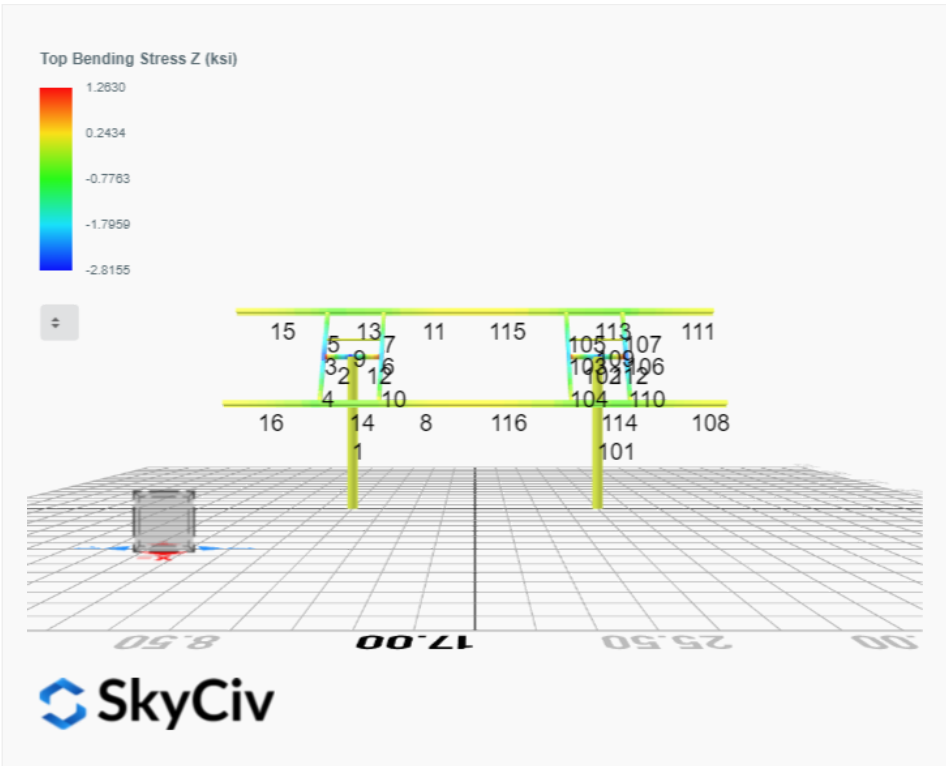
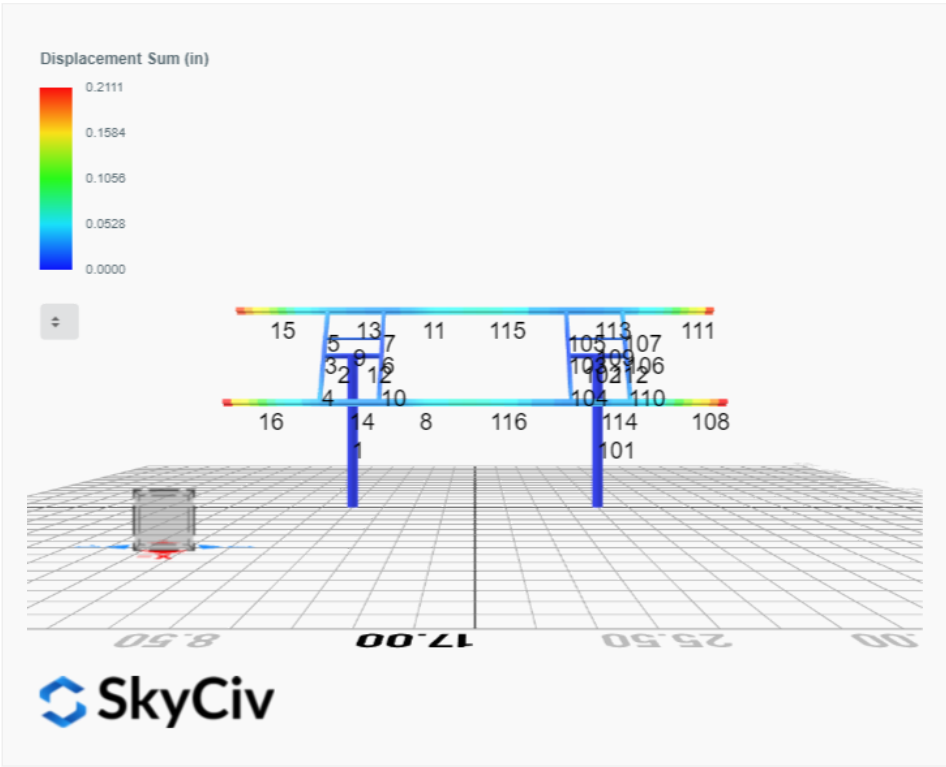
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

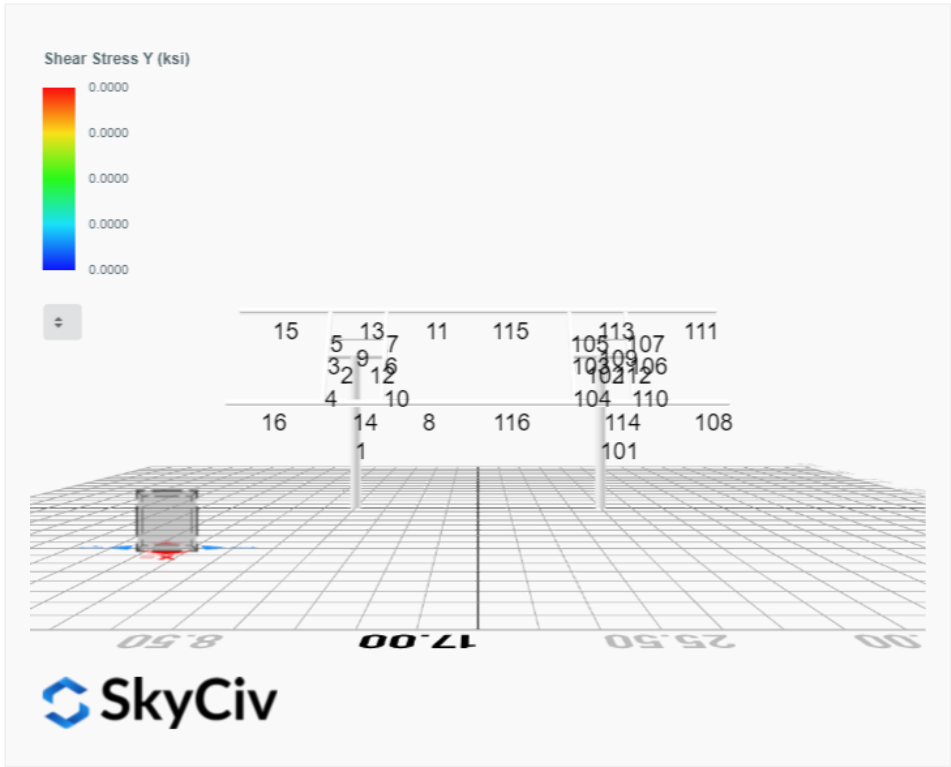
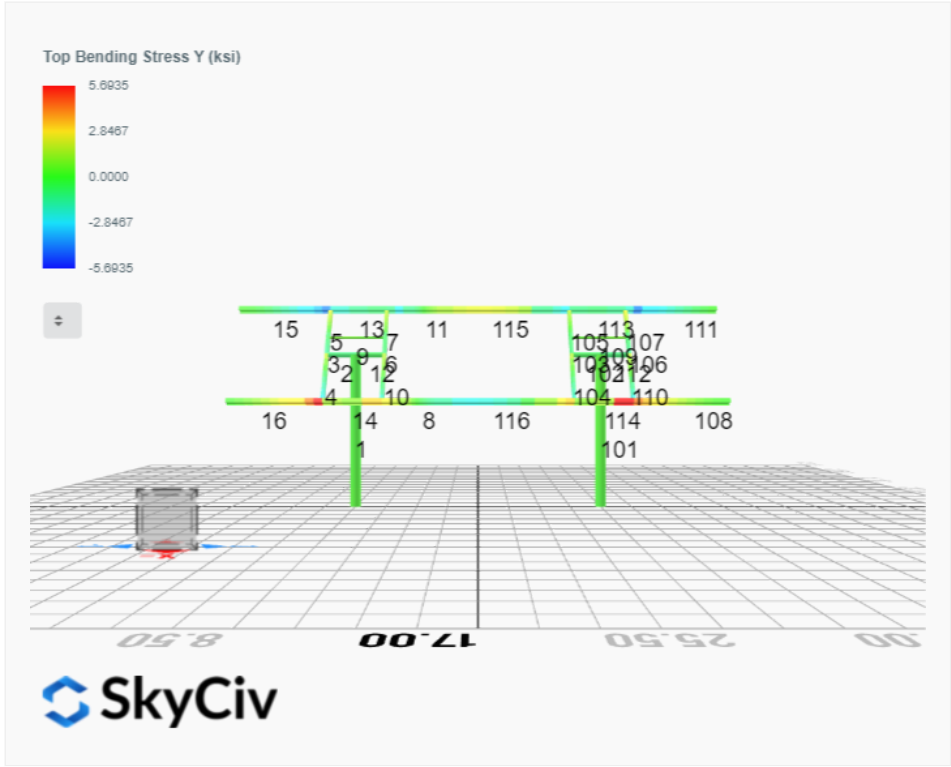


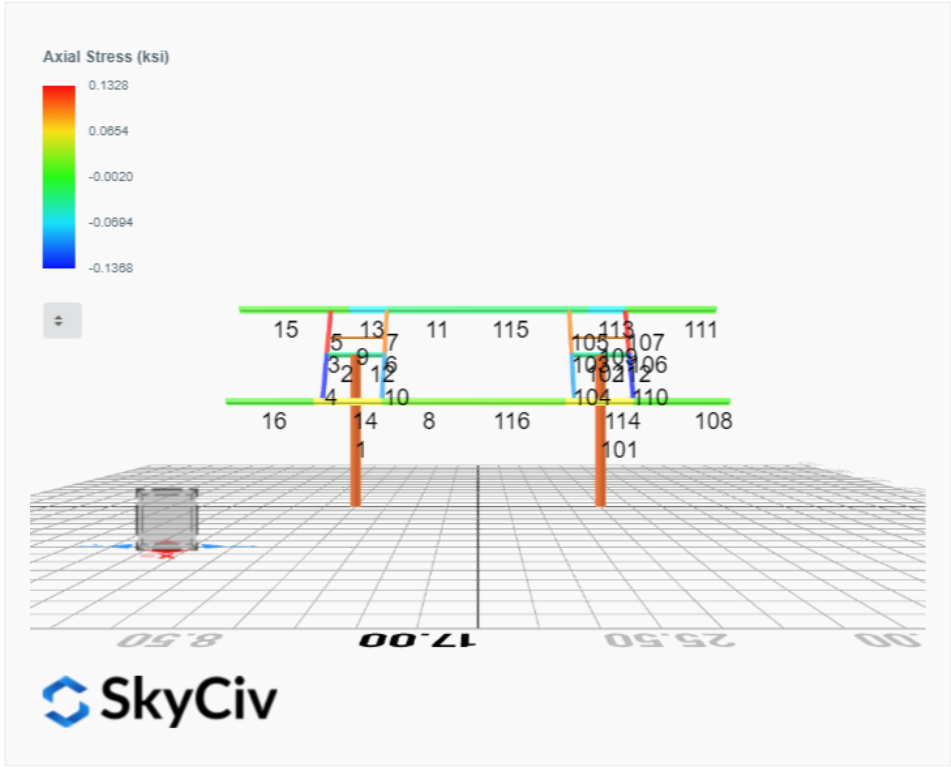




FEM Results (Envelope Worst Case for each member)







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	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 2. D + L	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 3. D + (S or Lr or R)	0.0000	2.9589	-0.0526	-0.1512	0.1997	0.0145
ULS: 3. D + (S or Lr or R)	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7128	-0.0475	-0.1364	0.1801	0.0141
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 5b. D + 0.7E	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.7128	-0.0475	-0.1364	0.1801	0.0141
ULS: 8. 0.6D + 0.7E	0.0000	1.1846	-0.0192	-0.0552	0.0729	0.0078
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1211	3.7762	-0.1295	-0.3525	0.8637	33.0889
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1211	0.1723	0.0654	0.1679	-0.6211	-32.5647
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3409	4.0643	-0.1206	-0.3318	0.7367	24.8211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.7128	-0.0475	-0.1364	0.1801	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3409	1.3613	0.0256	0.0586	-0.3768	-24.4191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.7128	-0.0475	-0.1364	0.1801	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3409	3.3257	-0.1052	-0.2874	0.6781	24.8199
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3409	0.6228	0.0410	0.1029	-0.4354	-24.4203
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.9743	-0.0320	-0.0920	0.1215	0.0129
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1211	2.9865	-0.1167	-0.3157	0.8150	33.0837
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.1846	-0.0192	-0.0552	0.0729	0.0078
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1211	-0.6174	0.0782	0.2047	-0.6697	-32.5699
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.1846	-0.0192	-0.0552	0.0729	0.0078

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	5.8648
Shear X	-5.2019
Shear Z	-0.2113
Moment X	-0.5746
Moment Y (Twist)	1.4204
Moment Z	55.6014

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	4.0643
Shear X	-3.1211
Shear Z	-0.1295
Moment X	-0.3525
Moment Y (Twist)	0.8637
Moment Z	33.0889

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	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 2. D + L	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 3. D + (S or Lr or R)	-0.0000	2.9589	0.0526	0.1512	-0.1997	0.0145
ULS: 3. D + (S or Lr or R)	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.7128	0.0475	0.1364	-0.1801	0.0141
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 5b. D + 0.7E	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.7128	0.0475	0.1364	-0.1801	0.0141
ULS: 8. 0.6D + 0.7E	-0.0000	1.1846	0.0192	0.0552	-0.0729	0.0078
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1211	3.7762	0.1295	0.3526	-0.8636	33.0889
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1211	0.1723	-0.0654	-0.1679	0.6211	-32.5647
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3409	4.0643	0.1206	0.3318	-0.7367	24.8211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.7128	0.0475	0.1364	-0.1801	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3409	1.3613	-0.0256	-0.0585	0.3768	-24.4191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.7128	0.0475	0.1364	-0.1801	0.0141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3409	3.3257	0.1052	0.2874	-0.6781	24.8199
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3409	0.6228	-0.0410	-0.1029	0.4354	-24.4203
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.9743	0.0320	0.0920	-0.1215	0.0129
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1211	2.9865	0.1167	0.3158	-0.8150	33.0837
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.1846	0.0192	0.0552	-0.0729	0.0078
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1211	-0.6174	-0.0782	-0.2047	0.6697	-32.5699
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.1846	0.0192	0.0552	-0.0729	0.0078

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	5.8648
Shear X	-5.2019
Shear Z	0.2113
Moment X	0.5747
Moment Y (Twist)	1.4205
Moment Z	55.6021

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	4.0643
Shear X	-3.1211
Shear Z	0.1295
Moment X	0.3526
Moment Y (Twist)	0.8636
Moment Z	33.0889

Project Details

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

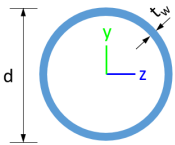
User Name: sales@mtsolar.us
Unit System: imperial



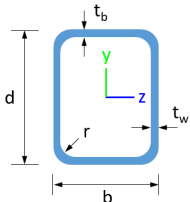
Design Input Information

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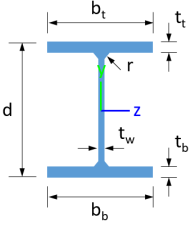
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

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					
9	8in Pipe Sch 40	8.63	0.32					

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

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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

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									
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1	9	22.09	22.09	10.52	-	300	200	1	
2	4	1.30	1.30	2.00	-	300	200	1	
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1	
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1	
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.66,1.67,1.67	300	200	1	
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.17,1.18,1.18	300	200	1	
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
8	18	1.33	1.33	2.05	2.21,2.20,2.21,2.20,2.20,2.21,2.12,2.20,2.09,2.20,2.12,2.21,2.10,2.21,2.14,2.20,2.01,2.20,2.13,2.21,2.07,2.21,2.12,2.21,2.10,2.21	300	200	1	
9	1	2.60	2.60	4.00	-	300	200	1	
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
11	18	1.33	1.33	2.05	2.23,2.23,2.23,2.23,2.23,2.23,2.15,2.23,2.11,2.23,2.15,2.23,2.12,2.23,2.17,2.23,2.03,2.23,2.16,2.23,2.10,2.23,2.15,2.23,2.12,2.23	300	200	1	
12	4	1.30	1.30	2.00	-	300	200	1	
13	18	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11	300	200	1	
14	18	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.11,1.11	300	200	1	
15	18	9.97	9.97	4.75	2.33,2.33	300	200	1	
16	18	9.97	9.97	4.75	2.33,2.33	300	200	1	
101	9	22.09	22.09	10.52	-	300	200	1	
102	4	1.30	1.30	2.00	-	300	200	1	
103	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.17,1.18,1.18,1.17,1.18,1.18	300	200	1	
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
106	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1	
107	15	1.52	1.52	2.33	1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67	300	200	1	

115	120.00	89.27	23.36	6.45	30.09	45.74
116	120.60	89.27	23.36	6.45	30.09	45.74

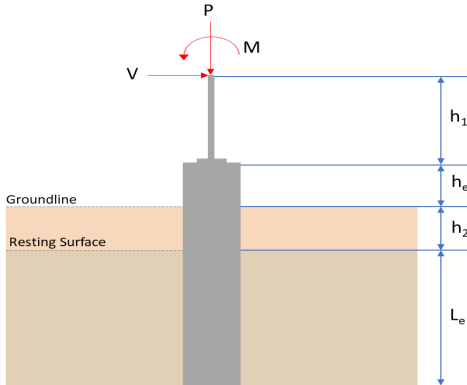
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.028	0.668	0.020	0.046	0.002	0.689	#13	0.451	Not Required	Pass
2	0.002	0.327	0.359	0.072	0.067	0.687	#13	0.034	Not Required	Pass
3	0.011	0.683	0.129	0.069	0.016	0.782	#13	0.044	Not Required	Pass
4	0.011	0.680	0.270	0.068	0.033	0.767	#13	0.078	Not Required	Pass
5	0.011	0.423	0.275	0.068	0.039	0.467	#13	0.073	Not Required	Pass
6	0.010	0.556	0.083	0.055	0.006	0.608	#13	0.044	Not Required	Pass
7	0.009	0.346	0.192	0.056	0.029	0.378	#13	0.073	Not Required	Pass
8	0.002	0.058	0.059	0.033	0.009	0.089	#13	0.088	Not Required	Pass
9	0.009	0.044	0.090	0.002	0.002	0.133	#13	0.198	Not Required	Pass
10	0.009	0.553	0.206	0.056	0.027	0.688	#13	0.078	Not Required	Pass
11	0.002	0.058	0.058	0.033	0.009	0.087	#13	0.059	Not Required	Pass
12	0.003	0.225	0.272	0.056	0.055	0.498	#13	0.034	Not Required	Pass
13	0.005	0.228	0.284	0.046	0.012	0.434	#21	0.177	Not Required	Pass
14	0.006	0.232	0.284	0.046	0.012	0.434	#21	0.177	Not Required	Pass
15	0.000	0.102	0.152	0.033	0.009	0.219	#21	Not Required	Not Required	Pass
16	0.000	0.102	0.152	0.033	0.009	0.219	#21	Not Required	Not Required	Pass
101	0.028	0.668	0.020	0.046	0.002	0.689	#13	0.451	Not Required	Pass
102	0.003	0.225	0.272	0.056	0.055	0.498	#13	0.034	Not Required	Pass
103	0.010	0.556	0.083	0.055	0.006	0.608	#13	0.044	Not Required	Pass
104	0.009	0.553	0.206	0.056	0.027	0.688	#13	0.078	Not Required	Pass
105	0.009	0.346	0.192	0.056	0.029	0.378	#13	0.073	Not Required	Pass
106	0.011	0.683	0.129	0.069	0.016	0.782	#13	0.044	Not Required	Pass
107	0.011	0.423	0.275	0.068	0.039	0.467	#13	0.073	Not Required	Pass
108	0.000	0.102	0.152	0.033	0.009	0.219	#21	Not Required	Not Required	Pass
109	0.009	0.044	0.090	0.002	0.002	0.133	#13	0.198	Not Required	Pass
110	0.011	0.680	0.270	0.068	0.033	0.767	#13	0.078	Not Required	Pass
111	0.000	0.102	0.152	0.033	0.009	0.219	#21	Not Required	Not Required	Pass
112	0.002	0.327	0.359	0.072	0.067	0.687	#13	0.034	Not Required	Pass
113	0.005	0.228	0.284	0.046	0.012	0.434	#21	0.177	Not Required	Pass
114	0.006	0.232	0.284	0.046	0.012	0.434	#21	0.265	Not Required	Pass
115	0.002	0.058	0.107	0.033	0.009	0.137	#21	0.214	Not Required	Pass
116	0.002	0.058	0.108	0.033	0.009	0.136	#21	0.321	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 = 9.25 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>4.064</td><td>5.865</td></tr><tr><td>Vx (kip)</td><td>-3.121</td><td>-5.202</td></tr><tr><td>Vz (kip)</td><td>-0.130</td><td>-0.211</td></tr><tr><td>Mx (kipft)</td><td>-0.353</td><td>-0.575</td></tr><tr><td>Mz (kipft)</td><td>33.089</td><td>55.601</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)	4.064	5.865	Vx (kip)	-3.121	-5.202	Vz (kip)	-0.130	-0.211	Mx (kipft)	-0.353	-0.575	Mz (kipft)	33.089	55.601	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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Mx (kipft)	-0.353	-0.575																										
Mz (kipft)	33.089	55.601																										
	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, $H_o = \frac{V_x}{D}$ $H_o = \frac{(-3.121 \text{ kip})}{(36 \text{ in})}$ $H_o = -1.0403 \text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_x + (V_x H)}{D}$																											

	$M_o = \frac{(33.089 \text{ kipft}) + ((-3.121 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 11.03 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 8.301 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(-0.13 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.043333 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.353 \text{ kipft}) + ((-0.13 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.11767 \text{ kipft/ft}$ 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: $L_{e,z} = 1.9112 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(8.301 \text{ ft}), (1.9112 \text{ ft})]$ $L_{e,req} = 8.301 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (9.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 9.25 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(8.301 \text{ ft})}{(9.25 \text{ ft})}$ $\text{Ratio} = 0.89741$	Status: PASS Ratio: 0.900
	End-bearing Capacity (ASD) A - Pile cros-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 $q = \frac{P_o}{A}$ $q = \frac{(4.064 \text{ kip})}{(7.0686 \text{ ft}^2)}$	

	$q = 0.57494 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.57494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.28747$	Status: PASS Ratio: 0.290
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(9.25 \text{ ft})}{(36 \text{ in})}$ $L/D = 3.0833$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -1.0403 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 11.03 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (11.03 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (11.03 \text{ kipft/ft})) + (4 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.4501 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (11.03 \text{ kipft/ft})) + (3 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (11.03 \text{ kipft/ft})) + (2 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$ $p = 0.23128 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (11.03 \text{ kipft/ft})) + ((-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$ $s = 1.3699 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.4501 \text{ ft})}{2}$ $p_a = 0.48376 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.23128 \text{ kip/ft}^2)}{(0.48376 \text{ kip/ft}^2)}$ $Ratio = 0.47809$	Status: PASS Ratio: 0.480

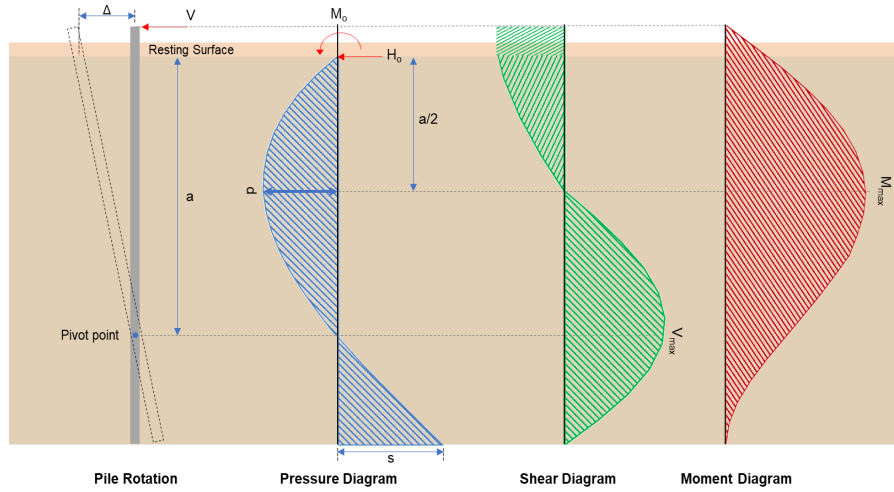
	p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$ $p_s = 1.3875 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.3699 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98731$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = -0.043333 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11767 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11767 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.043333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.11767 \text{ kipft/ft})) + (4 \times (-0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.7018 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (0.11767 \text{ kipft/ft})) + (3 \times (-0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (0.11767 \text{ kipft/ft})) + (2 \times (-0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$ $p = -0.016435 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (0.11767 \text{ kipft/ft})) + ((-0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$ $s = -0.01823 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.7018 \text{ ft})}{2}$ $p_a = 0.50264 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.016435 \text{ kip/ft}^2)}{(0.50264 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.032697$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$ $p_s = 1.3875 \text{ kip/ft}^2$ <i>Ratio</i> - Lateral soil capacity	Status: PASS Ratio: -0.030

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

$$Ratio = \frac{(-0.01823 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$Ratio = -0.013139$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.202 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.734 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.601 \text{ kipft}) + ((-5.202 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 18.534 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(18.534 \text{ kipft/ft})}{(-1.734 \text{ kip/ft})}$$

$$E = 10.688 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.534 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.734 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (18.534 \text{ kipft/ft})) + (4 \times (-1.734 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4487 \text{ ft}$$

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

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

$$V_{max} = ((-1.734 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.688 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4487 \text{ ft})}{(9.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (10.688 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4487 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

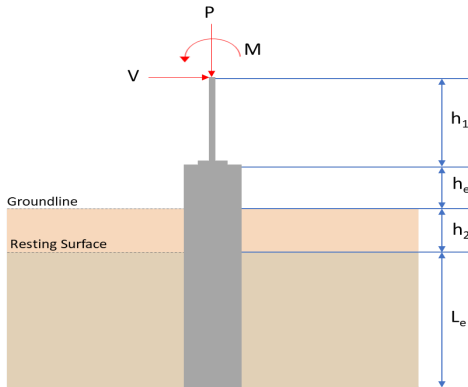
	$V_{max} = 14.069 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-1.734 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(10.688 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.688 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.688 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$ $M_{max} = 60.724 \text{ kipft}$	
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(-0.211 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.070333 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.575 \text{ kipft}) + ((-0.211 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.19167 \text{ kipft/ft}$ E - Distance from lateral load to resisting surface, $E = \frac{M_o}{H_o}$ $E = \frac{(0.19167 \text{ kipft/ft})}{(-0.070333 \text{ kip/ft})}$ $E = 2.7251 \text{ ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.19167 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.070333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.19167 \text{ kipft/ft})) + (4 \times (-0.070333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.7013 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = (H_o b) \left[1 - \left[3 \left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{L_e} \right)^2 \right] + \left[4 \left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{L_e} \right)^3 \right] \right]$ $V_{max} = ((-0.070333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.7013 \text{ ft})}{(9.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.7013 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$ $V_{max} = 0.25173 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o b L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-0.070333 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(2.7251 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$	

		$M_{max} = 0.99128 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(5.865 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.19 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.19 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ 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 $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(5.865 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0046774$	Status: PASS Ratio: 0.000
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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.865 \text{ kip} \rightarrow 5865 \text{ lbf}$. $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(5865 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 75.434 \text{ kip}$	
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 $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(186.09 \text{ kip}), (75.434 \text{ kip}), (204.04 \text{ kip})]$	

22.5.1.2	$V_c = 75.434 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ 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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.434 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.843 \text{ kip}$ Considering x-direction: $V_{max} = 14.069 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(14.069 \text{ kip})}{(73.843 \text{ kip})}$ $Ratio = 0.19052$ Considering z-direction: $V_{max} = 0.25173 \text{ kip}$ - Maximum shear force in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.25173 \text{ kip})}{(73.843 \text{ kip})}$ $Ratio = 0.003409$	Status: PASS Ratio: 0.190 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: $M_{max} = 60.724 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(60.724 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.979$	Status: PASS Ratio: 0.980
	Considering z-direction: $M_{max} = 0.99128 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.99128 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.015981$	Status: PASS Ratio: 0.020

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 L = 9.25 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>4.064</td><td>5.865</td></tr><tr><td>Vx (kip)</td><td>-3.121</td><td>-5.202</td></tr><tr><td>Vz (kip)</td><td>0.130</td><td>0.211</td></tr><tr><td>Mx (kipft)</td><td>0.353</td><td>0.575</td></tr><tr><td>Mz (kipft)</td><td>33.089</td><td>55.602</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)	4.064	5.865	Vx (kip)	-3.121	-5.202	Vz (kip)	0.130	0.211	Mx (kipft)	0.353	0.575	Mz (kipft)	33.089	55.602	
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Mz (kipft)	33.089	55.602																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_x}{D}$ $H_o = \frac{(-3.121 \text{ kip})}{(36 \text{ in})}$ $H_o = -1.0403 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_x H)}{D}$																											

	$M_o = \frac{(33.089 \text{ kipft}) + ((-3.121 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 11.03 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 8.301 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(0.13 \text{ kip})}{(36 \text{ in})}$ $H_o = 0.043333 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.353 \text{ kipft}) + ((0.13 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.11767 \text{ kipft/ft}$ 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: $L_{e,z} = 3.0018 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(8.301 \text{ ft}), (3.0018 \text{ ft})]$ $L_{e,req} = 8.301 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (9.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 9.25 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(8.301 \text{ ft})}{(9.25 \text{ ft})}$ $\text{Ratio} = 0.89741$	Status: PASS Ratio: 0.900
	End-bearing Capacity (ASD) A - Pile cros-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 $q = \frac{P_o}{A}$ $q = \frac{(4.064 \text{ kip})}{(7.0686 \text{ ft}^2)}$	

	$q = 0.57494 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.57494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.28747$	Status: PASS Ratio: 0.290
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(9.25 \text{ ft})}{(36 \text{ in})}$ $L/D = 3.0833$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -1.0403 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 11.03 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (11.03 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (11.03 \text{ kipft/ft})) + (4 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.4501 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (11.03 \text{ kipft/ft})) + (3 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^3 \times [(3 \times (11.03 \text{ kipft/ft})) + (2 \times (-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$ $p = 0.23128 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (11.03 \text{ kipft/ft})) + ((-1.0403 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$ $s = 1.3699 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.4501 \text{ ft})}{2}$ $p_a = 0.48376 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.23128 \text{ kip/ft}^2)}{(0.48376 \text{ kip/ft}^2)}$ $Ratio = 0.47809$	Status: PASS Ratio: 0.480

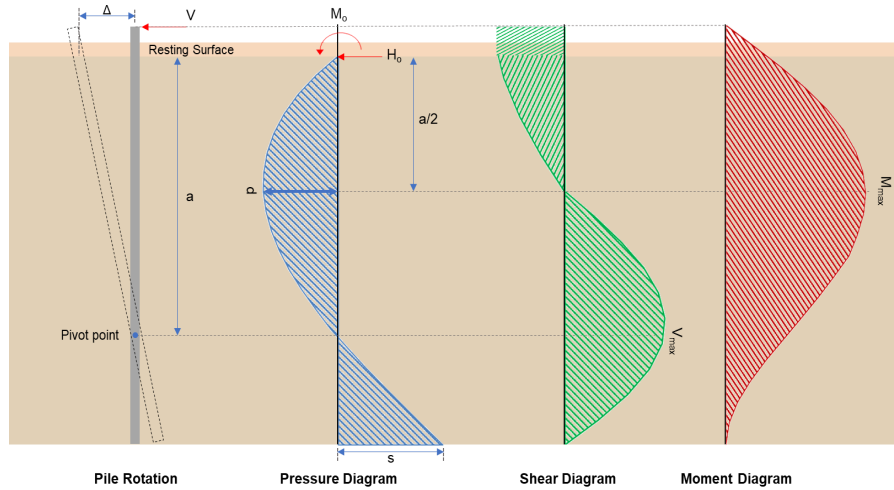
	p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$ $p_s = 1.3875 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.3699 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98731$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = 0.043333 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11767 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11767 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (0.043333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.11767 \text{ kipft/ft})) + (4 \times (0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.7018 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{1.178 \times [(4 \times (0.11767 \text{ kipft/ft})) + (3 \times (0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (0.11767 \text{ kipft/ft})) + (2 \times (0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$ $p = 0.03338 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{9.425 \times [(2 \times (0.11767 \text{ kipft/ft})) + ((0.043333 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$ $s = 0.070076 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(6.7018 \text{ ft})}{2}$ $p_a = 0.50264 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.03338 \text{ kip/ft}^2)}{(0.50264 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.066409$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$ $p_s = 1.3875 \text{ kip/ft}^2$ Ratio - Lateral soil capacity	Status: PASS Ratio: 0.070

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

$$Ratio = \frac{(0.070076 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$Ratio = 0.050505$$

Status: **PASS**
Ratio: **0.050**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.202 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.734 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.602 \text{ kipft}) + ((-5.202 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 18.534 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(18.534 \text{ kipft/ft})}{(-1.734 \text{ kip/ft})}$$

$$E = 10.689 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.534 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.734 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (18.534 \text{ kipft/ft})) + (4 \times (-1.734 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4487 \text{ ft}$$

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

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

$$V_{max} = ((-1.734 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.689 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4487 \text{ ft})}{(9.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (10.689 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4487 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 14.069 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-1.734 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(10.689 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.689 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.689 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4487 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$ $M_{max} = 60.724 \text{ kipft}$	
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{D}$ $H_o = \frac{(0.211 \text{ kip})}{(36 \text{ in})}$ $H_o = 0.070333 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{D}$ $M_o = \frac{(0.575 \text{ kipft}) + ((0.211 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 0.19167 \text{ kipft/ft}$ E - Distance from lateral load to resisting surface, $E = \frac{M_o}{H_o}$ $E = \frac{(0.19167 \text{ kipft/ft})}{(0.070333 \text{ kip/ft})}$ $E = 2.7251 \text{ ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.19167 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (0.070333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.19167 \text{ kipft/ft})) + (4 \times (0.070333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$ $a = 6.7013 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = (H_o b) \left[1 - \left[3 \left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{L_e} \right)^2 \right] + \left[4 \left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{L_e} \right)^3 \right] \right]$ $V_{max} = ((0.070333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.7013 \text{ ft})}{(9.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.7013 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$ $V_{max} = 0.25173 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o b L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((0.070333 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(2.7251 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.7251 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.7013 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$	

		$M_{max} = 0.99128 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(5.865 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.19 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.19 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ 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 $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(5.865 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0046774$	Status: PASS Ratio: 0.000
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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.865 \text{ kip} \rightarrow 5865 \text{ lbf}$. $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(5865 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 75.434 \text{ kip}$	
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 $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(186.09 \text{ kip}), (75.434 \text{ kip}), (204.04 \text{ kip})]$	

22.5.1.2 22.5.8.5.3 22.5.1.1	$V_c = 75.434 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a} - \text{Shear strength of steel (a)}$ $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ $V_{s,b}$ - 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 $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.434 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.843 \text{ kip}$ Considering x-direction: $V_{max} = 14.069 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(14.069 \text{ kip})}{(73.843 \text{ kip})}$ $Ratio = 0.19053$ Considering z-direction: $V_{max} = 0.25173 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.25173 \text{ kip})}{(73.843 \text{ kip})}$ $Ratio = 0.003409$ Status: PASS Ratio: 0.190	Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4500.473$	

	$S_m = 4580.4 \text{ in}^3$ $\lambda = 1$ - 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: $M_{max} = 60.724 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(60.724 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.979$	Status: PASS Ratio: 0.980
	Considering z-direction: $M_{max} = 0.99128 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.99128 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.015981$	Status: PASS Ratio: 0.020