

Your Project Calculations

Project Name: SIL-400-HC-Plus 16-panel

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=SIL-400-HC-Plus%2016-panel&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=7HrTFgzBhdasphpBmzucJ7ZfRuSYeiCzedO0oDbnC3BrfNGzYDGUfS6gPTX2ltuc



Array Specification

Product:	Beam
Unique ID:	2P-15-6TOP-SD-12-L-4Hx4W-LJ00
Duty Classification:	SD
Module Width:	40.80 in
Module Length:	75.30in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	45
Front Edge Clearance:	5
Total Array Height at Tilt:	14.68 ft
Total Frame Length:	24.50 ft
Frame Weight:	1424 lbs
Array Dimensions N/S:	13.77 ft
Array Dimensions E/W:	25.43 ft
Rail Length:	165.20 in
Rail Spacing:	3.14 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.87 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.50 ft Pile 2: 5.50 ft
Foundation Volume:	6.519 y ³
Foundation Result:	PASSED
Mount Twist:	0.496521 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	25519 Wind Song Valley Rd, Custer, SD 57730, USA
Wind Speed:	105 mph
Snow Load:	30 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.008247 ksf



Design Disclaimer

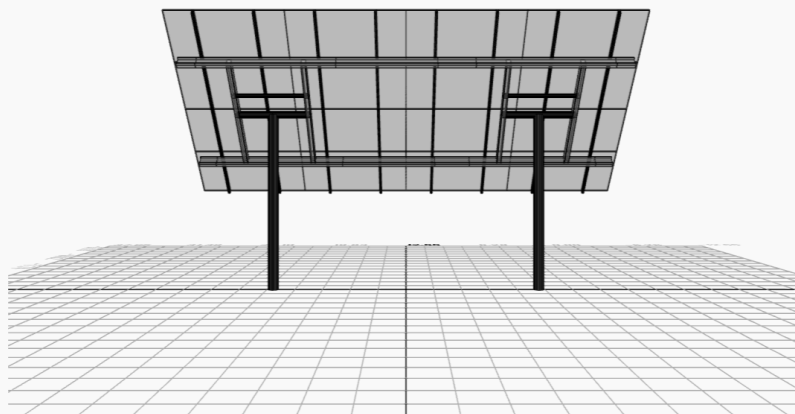
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

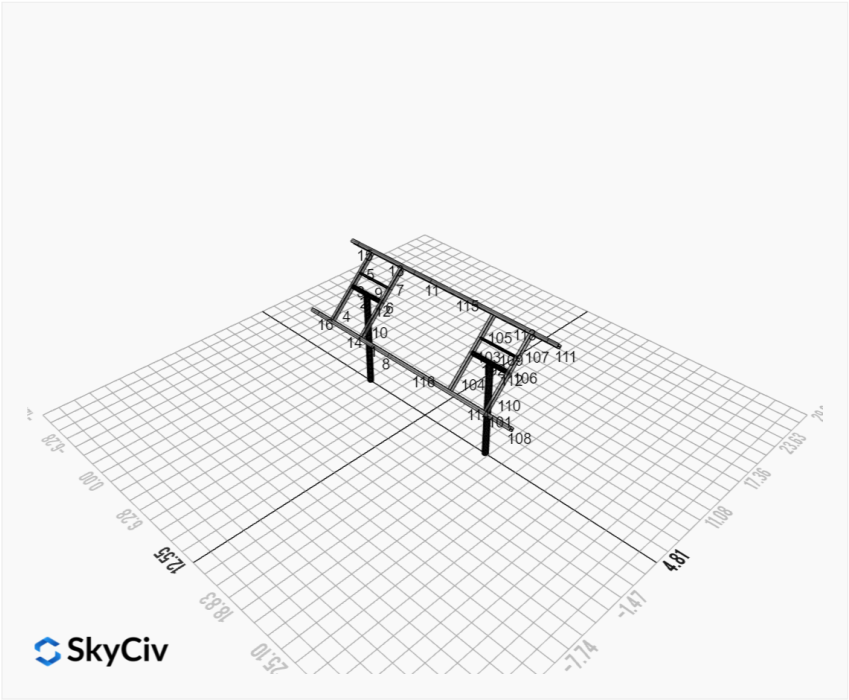
AutoDesigner Input

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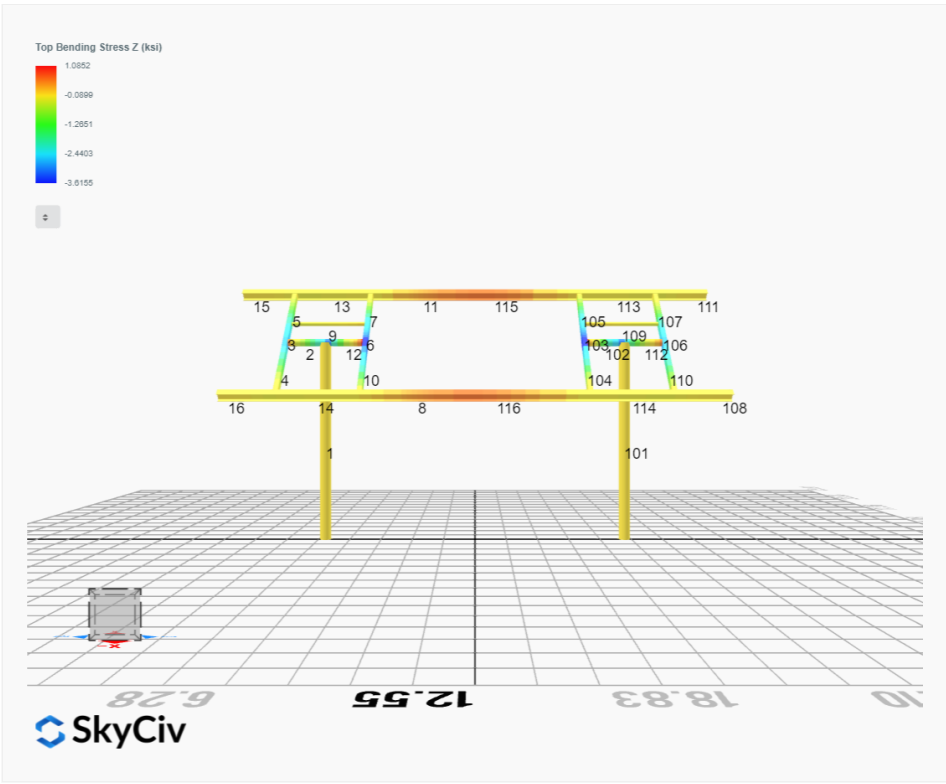
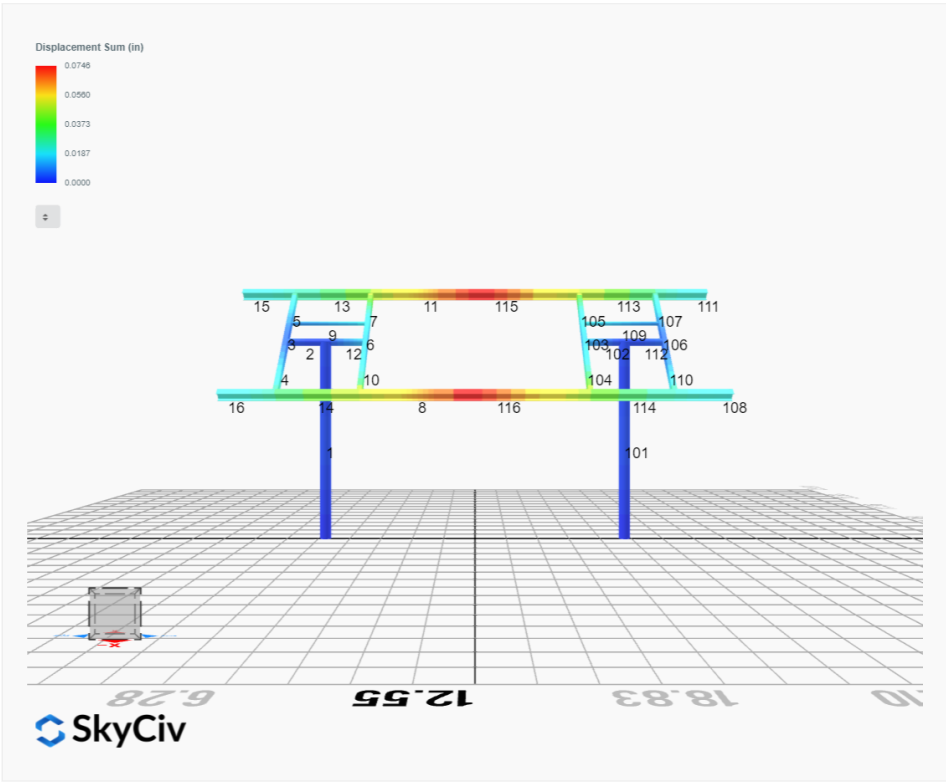
Design Notes:

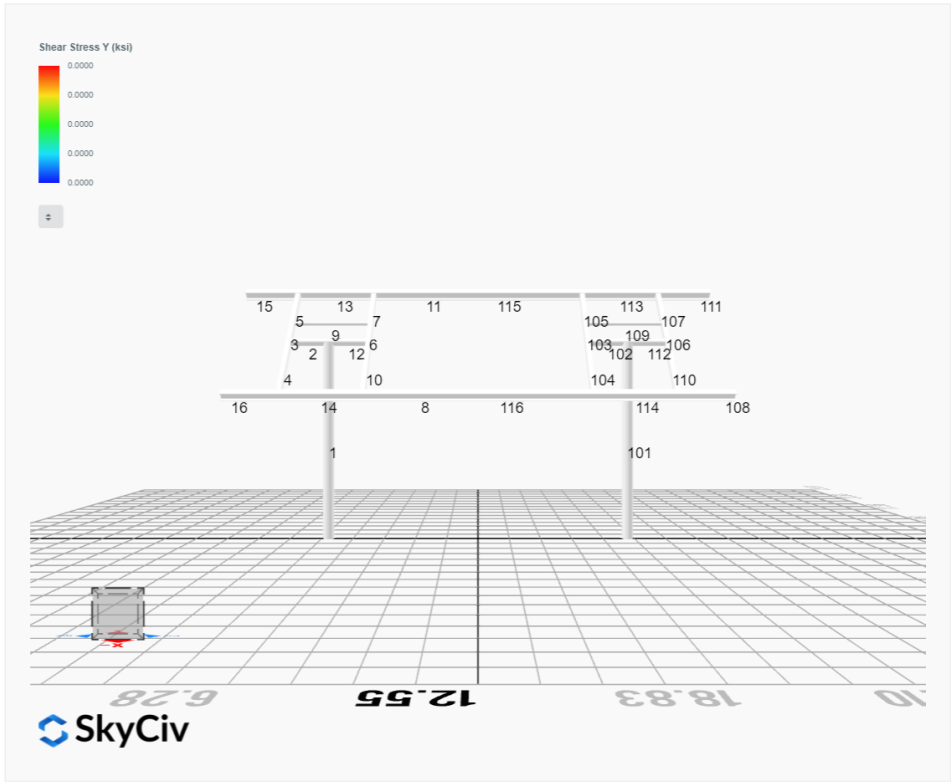
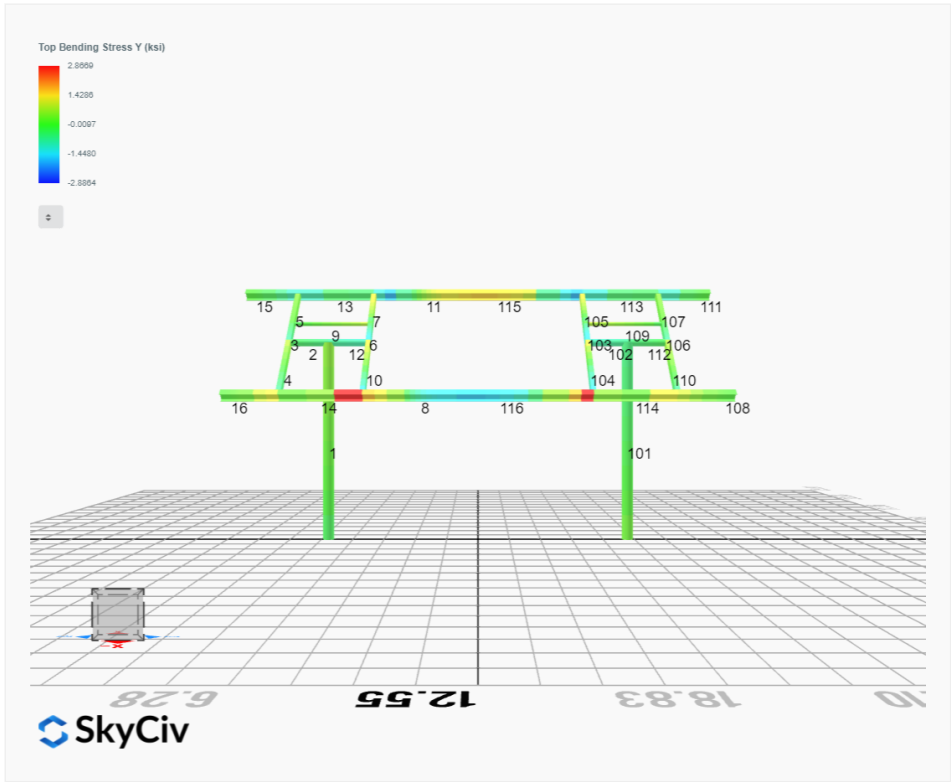
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

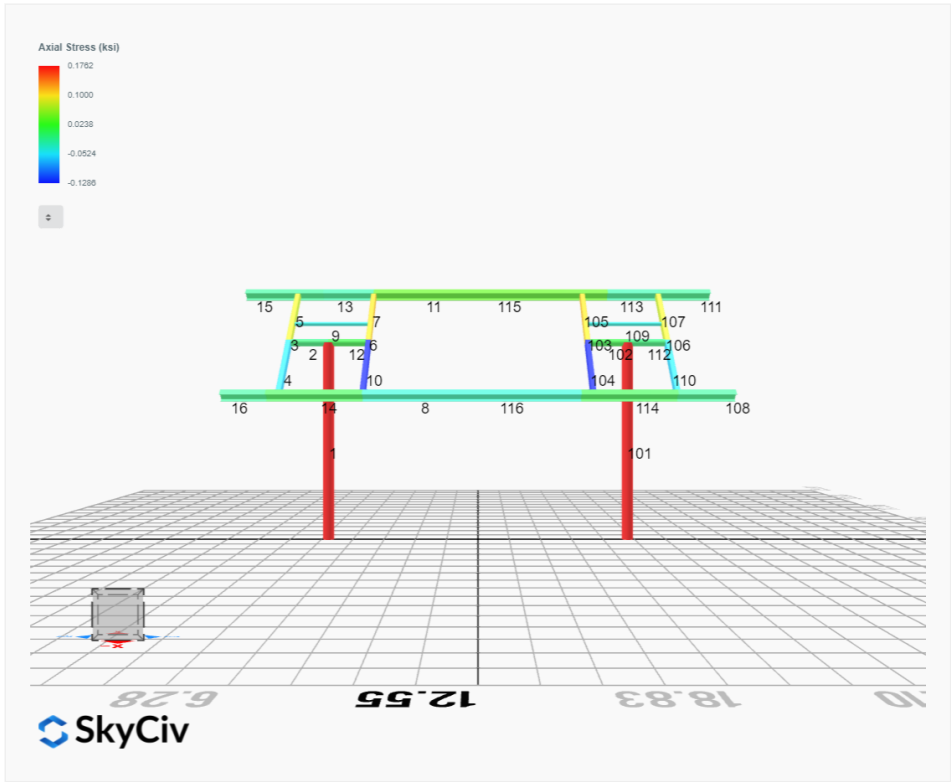




FEM Results (Envelope Worst Case for each member)







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	1.3793	0.0415	0.1262	-0.0357	0.0161
ULS: 2. D + L	0.0000	1.3793	0.0415	0.1262	-0.0357	0.0161
ULS: 3. D + (S or Lr or R)	0.0000	2.3627	0.0807	0.2456	-0.0696	0.0179
ULS: 3. D + (S or Lr or R)	0.0000	1.3793	0.0415	0.1262	-0.0357	0.0161
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1169	0.0709	0.2158	-0.0611	0.0175
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.3793	0.0415	0.1262	-0.0357	0.0161
ULS: 5b. D + 0.7E	0.0000	1.3793	0.0415	0.1262	-0.0357	0.0161
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.1169	0.0709	0.2158	-0.0611	0.0175
ULS: 8. 0.6D + 0.7E	0.0000	0.8276	0.0249	0.0757	-0.0214	0.0097
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6879	3.0672	0.1366	0.4047	-0.2985	17.5303
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6879	3.0672	0.1366	0.4047	-0.2985	17.5303
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2210	0.1582	-0.0271	-0.0741	0.1543	-11.5193
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0774	0.3019	-0.0196	-0.0520	0.1333	-13.5077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2659	3.3828	0.1423	0.4246	-0.2583	13.1531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2659	3.3828	0.1423	0.4246	-0.2583	13.1531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9158	1.2011	0.0194	0.0655	0.0813	-8.6341
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8080	1.3088	0.0251	0.0821	0.0656	-10.1254
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2659	2.6452	0.1129	0.3351	-0.2328	13.1517
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2659	2.6452	0.1129	0.3351	-0.2328	13.1517
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9158	0.4635	-0.0100	-0.0240	0.1068	-8.6355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8080	0.5712	-0.0043	-0.0074	0.0910	-10.1268
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6879	2.5155	0.1200	0.3542	-0.2843	17.5238
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6879	2.5155	0.1200	0.3542	-0.2843	17.5238
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2210	-0.3935	-0.0437	-0.1246	0.1685	-11.5258
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0774	-0.2498	-0.0361	-0.1025	0.1475	-13.5142

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	4.9600
Shear X	-2.8132
Shear Z	0.2277
Moment X	0.6754
Moment Y (Twist)	0.4968
Moment Z	29.6138

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	3.3828
Shear X	-1.6879
Shear Z	0.1423
Moment X	0.4246
Moment Y (Twist)	0.2985
Moment Z	17.5303

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	1.3793	-0.0415	-0.1262	0.0357	0.0161
ULS: 2. D + L	-0.0000	1.3793	-0.0415	-0.1262	0.0357	0.0161
ULS: 3. D + (S or Lr or R)	-0.0000	2.3627	-0.0807	-0.2456	0.0696	0.0179
ULS: 3. D + (S or Lr or R)	-0.0000	1.3793	-0.0415	-0.1262	0.0357	0.0161
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.1169	-0.0709	-0.2158	0.0611	0.0175
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.3793	-0.0415	-0.1262	0.0357	0.0161
ULS: 5b. D + 0.7E	-0.0000	1.3793	-0.0415	-0.1262	0.0357	0.0161

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.1169	-0.0709	-0.2158	0.0611	0.0175
ULS: 8. 0.6D + 0.7E	-0.0000	0.8276	-0.0249	-0.0757	0.0214	0.0097
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6879	3.0672	-0.1366	-0.4047	0.2986	17.5303
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6879	3.0672	-0.1366	-0.4047	0.2986	17.5303
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2210	0.1582	0.0271	0.0741	-0.1543	-11.5193
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0774	0.3019	0.0196	0.0520	-0.1332	-13.5077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2659	3.3828	-0.1423	-0.4246	0.2583	13.1531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2659	3.3828	-0.1423	-0.4246	0.2583	13.1531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9158	1.2011	-0.0194	-0.0655	-0.0813	-8.6341
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8080	1.3088	-0.0251	-0.0821	-0.0656	-10.1254
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2659	2.6452	-0.1129	-0.3351	0.2328	13.1518
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2659	2.6452	-0.1129	-0.3351	0.2328	13.1518
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9158	0.4635	0.0100	0.0241	-0.1068	-8.6355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8080	0.5712	0.0043	0.0074	-0.0910	-10.1268
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6879	2.5155	-0.1200	-0.3542	0.2843	17.5239
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6879	2.5155	-0.1200	-0.3542	0.2843	17.5239
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2210	-0.3935	0.0437	0.1246	-0.1685	-11.5258
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0774	-0.2498	0.0361	0.1025	-0.1475	-13.5142

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	4.9600
Shear X	-2.8132
Shear Z	-0.2277
Moment X	-0.6754
Moment Y (Twist)	0.4965
Moment Z	29.6144

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	3.3828
Shear X	-1.6879
Shear Z	-0.1423
Moment X	-0.4246
Moment Y (Twist)	0.2986
Moment Z	17.5303

Project Details

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

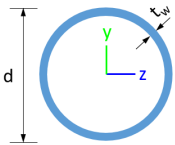
User Name: sales@mtsolar.us
Unit System: imperial



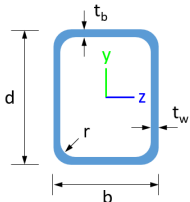
Design Input Information

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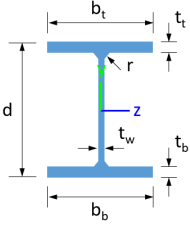
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

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	20.7 2	20.7 2	9.8 7	-	3 0 0	2 0 0	1	
2	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1	
3	15	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.19,2.31,1.18,1.18,1.15,1.16,1.18,1.18,1.17,1.17	3 0 0	2 0 0	1	
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5	15	1.52	1.52	2.3 3	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.69,1.13,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1	
6	15	0.92	0.92	1.4 2	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.27,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1	
7	15	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.80,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1	
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12	4	4.20	1.30	2.0 0	-	3 0 0	2 0 0	1	
13	18	4.88	4.00	7.5 0	1.53,1.54,1.53,1.55,1.53,1.53,1.67,1.67,1.83,1.85,1.67,1.67,1.78,1.81,1.63,1.63,1.37,1.34,1.65,1.65,2.03,1.96,1.68,1.68,1.75,1.78	3 0 0	2 0 0	1	
14	18	4.88	4.00	7.5 0	1.49,1.50,1.49,1.52,1.50,1.49,1.63,1.63,1.80,2.04,1.63,1.63,1.75,1.63,1.59,1.59,1.26,1.51,1.61,1.61,1.97,1.96,1.64,1.64,1.73,1.59	3 0 0	2 0 0	1	
15	18	2.10	2.10	1.0 0	2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1	
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101	7	20.7 2	20.7 2	9.8 7	-	3 0 0	2 0 0	1	
102	4	4.20	1.30	2.0 0	-	3 0 0	2 0 0	1	
103	15	0.92	0.92	1.4 2	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.27,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1	
104	15	2.44	2.44	3.7 5	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.67,1.66,2.42,1.67,1.67,1.66,1.62,1.67,1.67,1.70,1.68,1.67,1.67,1.64,1.72,1.67,1.67,1.66,1.64	3 0 0	2 0 0	1	
105	15	1.52	1.52	2.3 3	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.80,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1	
106	15	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.19,2.31,1.18,1.18,1.15,1.16,1.18,1.18,1.17,1.17	3 0 0	2 0 0	1	
107	15	1.52	1.52	2.3 3	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.69,1.13,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	3 0 0	2 0 0	1	

115	120.00	102.07	21.16	6.45	30.09	45.74
116	120.60	102.67	21.16	6.45	30.09	45.74

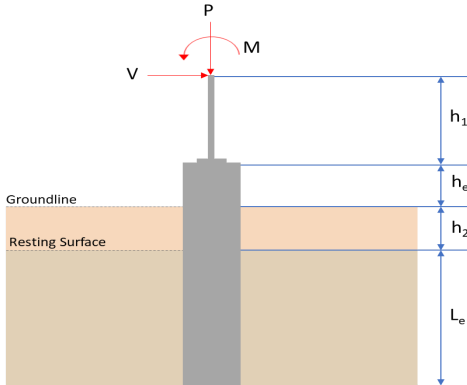
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.048	0.700	0.037	0.037	0.003	0.740	#13	0.554	Not Required	Pass
2	0.001	0.192	0.151	0.047	0.032	0.343	#13	0.052	Not Required	Pass
3	0.006	0.426	0.059	0.042	0.010	0.457	#13	0.044	Not Required	Pass
4	0.005	0.383	0.071	0.039	0.009	0.450	#13	0.078	Not Required	Pass
5	0.005	0.264	0.026	0.043	0.002	0.274	#13	0.073	Not Required	Pass
6	0.008	0.522	0.134	0.053	0.025	0.615	#13	0.044	Not Required	Pass
7	0.009	0.324	0.120	0.052	0.017	0.342	#13	0.073	Not Required	Pass
8	0.001	0.105	0.054	0.025	0.007	0.139	#13	0.088	Not Required	Pass
9	0.003	0.031	0.048	0.002	0.002	0.079	#13	0.132	Not Required	Pass
10	0.009	0.474	0.115	0.048	0.016	0.505	#13	0.078	Not Required	Pass
11	0.002	0.113	0.054	0.028	0.007	0.145	#13	0.088	Not Required	Pass
12	0.002	0.289	0.181	0.064	0.035	0.471	#13	0.167	Not Required	Pass
13	0.002	0.060	0.121	0.041	0.010	0.136	#21	0.265	Not Required	Pass
14	0.002	0.056	0.119	0.037	0.010	0.131	#21	0.177	Not Required	Pass
15	0.000	0.005	0.006	0.007	0.002	0.010	#21	Not Required	Not Required	Pass
16	0.000	0.004	0.006	0.007	0.002	0.010	#21	Not Required	Not Required	Pass
101	0.048	0.700	0.037	0.037	0.003	0.740	#13	0.554	Not Required	Pass
102	0.002	0.289	0.181	0.064	0.035	0.471	#13	0.167	Not Required	Pass
103	0.008	0.522	0.134	0.053	0.025	0.615	#13	0.044	Not Required	Pass
104	0.009	0.474	0.115	0.048	0.016	0.505	#13	0.078	Not Required	Pass
105	0.009	0.324	0.120	0.052	0.017	0.342	#13	0.073	Not Required	Pass
106	0.006	0.426	0.059	0.042	0.010	0.457	#13	0.044	Not Required	Pass
107	0.005	0.264	0.026	0.043	0.002	0.274	#13	0.073	Not Required	Pass
108	0.000	0.004	0.006	0.007	0.002	0.010	#21	Not Required	Not Required	Pass
109	0.003	0.031	0.048	0.002	0.002	0.079	#13	0.132	Not Required	Pass
110	0.005	0.383	0.071	0.039	0.009	0.450	#13	0.078	Not Required	Pass
111	0.000	0.005	0.006	0.007	0.002	0.010	#21	Not Required	Not Required	Pass
112	0.001	0.192	0.151	0.047	0.032	0.343	#13	0.052	Not Required	Pass
113	0.002	0.060	0.121	0.041	0.010	0.136	#21	0.177	Not Required	Pass
114	0.002	0.056	0.119	0.037	0.010	0.131	#21	0.265	Not Required	Pass
115	0.002	0.136	0.072	0.028	0.007	0.179	#13	0.235	Not Required	Pass
116	0.001	0.125	0.072	0.025	0.007	0.171	#13	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: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 5.5 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>3.383</td><td>4.960</td></tr><tr><td>Vx (kip)</td><td>-1.688</td><td>-2.813</td></tr><tr><td>Vz (kip)</td><td>0.142</td><td>0.228</td></tr><tr><td>Mx (kipft)</td><td>0.425</td><td>0.675</td></tr><tr><td>Mz (kipft)</td><td>17.530</td><td>29.614</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)	3.383	4.960	Vx (kip)	-1.688	-2.813	Vz (kip)	0.142	0.228	Mx (kipft)	0.425	0.675	Mz (kipft)	17.530	29.614	
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Mz (kipft)	17.530	29.614																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.688 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26879 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(17.53 \text{ kipft}) + ((-1.688 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.7914 \text{ kipft/ft}$ 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: $L_{e,x} = 5.1881 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.022611 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.425 \text{ kipft}) + ((0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.067675 \text{ kipft/ft}$ Required depth of embedment in earth: $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,z} = 2.0119 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.1881 \text{ ft}), (2.0119 \text{ ft})]$ $L_{e,req} = 5.188 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_c - h_2$ $L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.5 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.188 \text{ ft})}{(5.5 \text{ ft})}$ $Ratio = 0.94327$	Status: PASS Ratio: 0.940
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(3.383 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.21144 \text{ kips/ft}^2$	

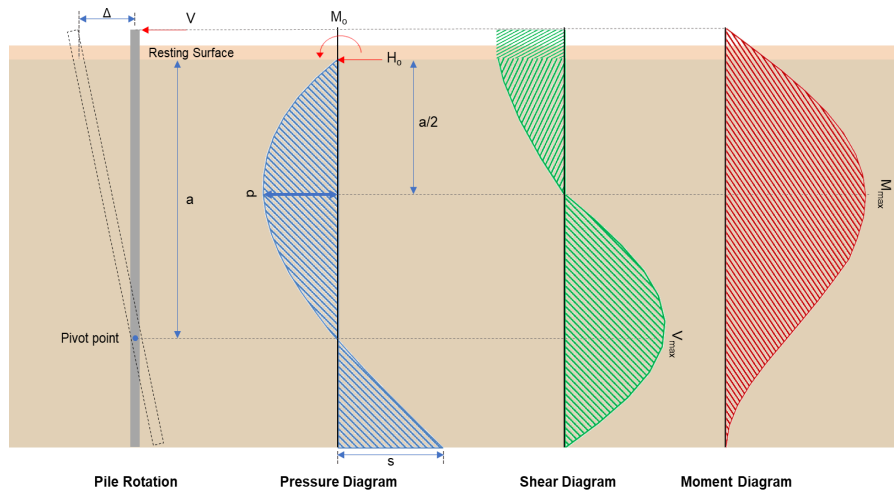
	$q = 0.21144 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.21144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.10572$	Status: PASS Ratio: 0.110
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.375$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.26879 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 2.7914 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (2.7914 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7914 \text{ kipft/ft})) + (4 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.7863 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (2.7914 \text{ kipft/ft})) + (3 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^3 [(3 \times (2.7914 \text{ kipft/ft})) + (2 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$ $p = 0.20732 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (2.7914 \text{ kipft/ft})) + ((-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$ $s = 0.81411 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.7863 \text{ ft})}{2}$ $p_a = 0.28397 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.20732 \text{ kip/ft}^2)}{(0.28397 \text{ kip/ft}^2)}$ $Ratio = 0.73008$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.730

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.81411 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9868$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = 0.022611 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.067675 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.067675 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.022611 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.067675 \text{ kipft/ft})) + (4 \times (0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.919 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.067675 \text{ kipft/ft})) + (3 \times (0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^3 \times [(3 \times (0.067675 \text{ kipft/ft})) + (2 \times (0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$ $p = 0.022747 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.067675 \text{ kipft/ft})) + ((0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$ $s = 0.051513 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.919 \text{ ft})}{2}$ $p_a = 0.29393 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.022747 \text{ kip/ft}^2)}{(0.29393 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.07739$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.080

$$Ratio = \frac{(0.051513 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.06244$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.813 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44793 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.614 \text{ kipft}) + ((-2.813 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7156 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7156 \text{ kipft/ft})}{(-0.44793 \text{ kip/ft})}$$

$$E = 10.528 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7156 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.44793 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.7156 \text{ kipft/ft})) + (4 \times (-0.44793 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7851 \text{ ft}$$

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

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

$$V_{max} = ((-0.44793 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7851 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7851 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2511 \text{ kip}$$

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

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

$$M_{max} = ((-0.44793 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.528 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.044 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.228 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.036306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.675 \text{ kipft}) + ((0.228 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10748 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10748 \text{ kipft/ft})}{(0.036306 \text{ kip/ft})}$$

$$E = 2.9605 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10748 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.036306 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.10748 \text{ kipft/ft})) + (4 \times (0.036306 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9203 \text{ ft}$$

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

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

$$V_{max} = ((0.036306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9203 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9203 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23497 \text{ kip}$$

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

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

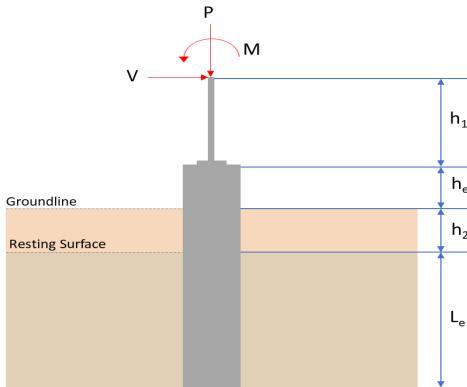
$$M_{max} = ((0.036306 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.9605 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.57486 \text{ kipft}$	
		Minimum Reinforcement Check (LRFD) Parameters: $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength, $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength, $\phi = 0.65$ - Reduction factor for axial strength, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(4.96 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.431 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.431 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(4.96 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0018541$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 4.96 \text{ kip} \rightarrow 4960 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(4960 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.15 \text{ kip}$	
22.5.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(296.21 \text{ kip}), (119.15 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.15 \text{ kip}$	

22.5.1.2 22.5.8.5.3 22.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ A_v - Ties rebar area, $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ $V_{s,b}$ - Shear strength of steel (b) $V_{s,b} = \frac{2 A_v f_{ysk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.15 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.53 \text{ kip}$ Considering x-direction: $V_{max} = 7.2511 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.2511 \text{ kip})}{(110.53 \text{ kip})}$ $Ratio = 0.065605$ Considering z-direction: $V_{max} = 0.23497 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.23497 \text{ kip})}{(110.53 \text{ kip})}$ $Ratio = 0.0021259$	Status: PASS Ratio: 0.070 Status: PASS Ratio: 0.000
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 19.044 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.044 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0763$	Status: PASS Ratio: 0.080
	Considering z-direction: $M_{max} = 0.57486 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.57486 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0023031$	Status: PASS Ratio: 0.000

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 5.5 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>3.383</td><td>4.960</td></tr><tr><td>Vx (kip)</td><td>-1.688</td><td>-2.813</td></tr><tr><td>Vz (kip)</td><td>-0.142</td><td>-0.228</td></tr><tr><td>Mx (kipft)</td><td>-0.425</td><td>-0.675</td></tr><tr><td>Mz (kipft)</td><td>17.530</td><td>29.614</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)	3.383	4.960	Vx (kip)	-1.688	-2.813	Vz (kip)	-0.142	-0.228	Mx (kipft)	-0.425	-0.675	Mz (kipft)	17.530	29.614	
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Mx (kipft)	-0.425	-0.675																										
Mz (kipft)	17.530	29.614																										
	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, $H_o = \frac{V_x}{1.57\text{ }D}$ $H_o = \frac{(-1.688\text{ kip})}{1.57 \times (48\text{ in})}$ $H_o = -0.26879\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x\text{ }H)}{1.57\text{ }D}$																											

	$M_o = \frac{(17.53 \text{ kipft}) + ((-1.688 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.7914 \text{ kipft/ft}$ 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: $L_{e,x} = 5.1881 \text{ ft}$ - Required depth in x-direction, Considering z-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.022611 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.425 \text{ kipft}) + ((-0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.067675 \text{ kipft/ft}$ 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: $L_{e,z} = 1.5006 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.1881 \text{ ft}), (1.5006 \text{ ft})]$ $L_{e,req} = 5.188 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.5 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.188 \text{ ft})}{(5.5 \text{ ft})}$ $\text{Ratio} = 0.94327$	Status: PASS Ratio: 0.940
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(3.383 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.21144 \text{ kips/ft}^2$	

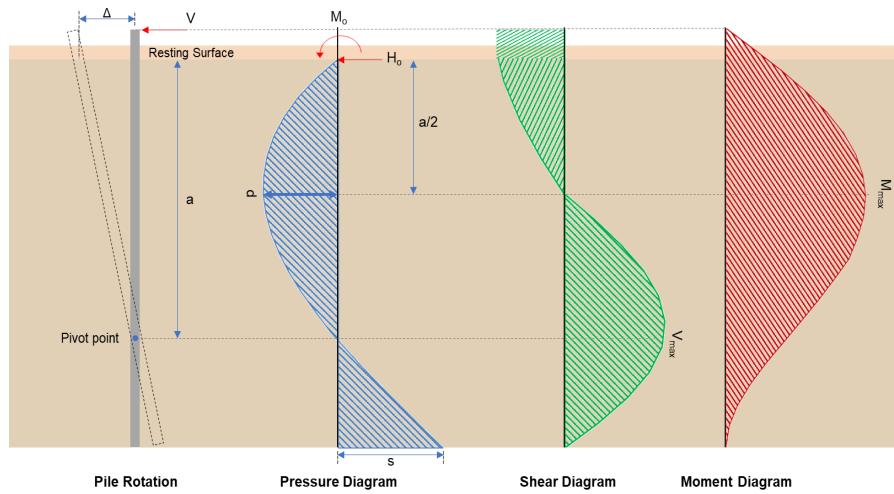
	$q = 0.21144 \text{ kip/ft}^2$	
	Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.21144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.10572$	Status: PASS Ratio: 0.110
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.375$ Since $L/D \leq 10$, Pile is short. Considering x-direction: $H_o = -0.26879 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 2.7914 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (2.7914 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7914 \text{ kipft/ft})) + (4 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.7863 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (2.7914 \text{ kipft/ft})) + (3 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^3 [(3 \times (2.7914 \text{ kipft/ft})) + (2 \times (-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$ $p = 0.20732 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (2.7914 \text{ kipft/ft})) + ((-0.26879 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$ $s = 0.81411 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.7863 \text{ ft})}{2}$ $p_a = 0.28397 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.20732 \text{ kip/ft}^2)}{(0.28397 \text{ kip/ft}^2)}$ $Ratio = 0.73008$ p_s - Allowable lateral soil pressure at depth L_e,	Status: PASS Ratio: 0.730

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.81411 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9868$	Status: PASS Ratio: 0.990
	Considering z-direction: $H_o = -0.022611 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.067675 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.067675 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.022611 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.067675 \text{ kipft/ft})) + (4 \times (-0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.919 \text{ ft}$ p - Earth pressure against the pile at distance $a/2$ from resting surface, $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.067675 \text{ kipft/ft})) + (3 \times (-0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^3 [(3 \times (0.067675 \text{ kipft/ft})) + (2 \times (-0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$ $p = -0.0056874 \text{ kip/ft}^2$ s - Earth pressure against the pile at distance L_e, $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.067675 \text{ kipft/ft})) + ((-0.022611 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$ $s = 0.0021793 \text{ kip/ft}^2$ Check lateral soil pressure capacity: p_a - Allowable lateral soil pressure at depth $a/2$, $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.919 \text{ ft})}{2}$ $p_a = 0.29393 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0056874 \text{ kip/ft}^2)}{(0.29393 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.01935$ p_s - Allowable lateral soil pressure at depth L_e, $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$ $p_s = 0.825 \text{ kip/ft}^2$ Ratio - Lateral soil capacity $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.020

$$Ratio = \frac{(0.0021793 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.0026416$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.813 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44793 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.614 \text{ kipft}) + ((-2.813 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7156 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7156 \text{ kipft/ft})}{(-0.44793 \text{ kip/ft})}$$

$$E = 10.528 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7156 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.44793 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.7156 \text{ kipft/ft})) + (4 \times (-0.44793 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7851 \text{ ft}$$

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

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

$$V_{max} = ((-0.44793 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7851 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7851 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2511 \text{ kip}$$

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

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

$$M_{max} = ((-0.44793 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.528 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.528 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7851 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.044 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.228 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.036306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.675 \text{ kipft}) + ((-0.228 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10748 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10748 \text{ kipft/ft})}{(-0.036306 \text{ kip/ft})}$$

$$E = 2.9605 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10748 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.036306 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.10748 \text{ kipft/ft})) + (4 \times (-0.036306 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9203 \text{ ft}$$

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

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

$$V_{max} = ((-0.036306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9203 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9203 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23497 \text{ kip}$$

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

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

$$M_{max} = ((-0.036306 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.9605 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9605 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9203 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.57486 \text{ kipft}$	
		Minimum Reinforcement Check (LRFD) Parameters: $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength, $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength, $\phi = 0.65$ - Reduction factor for axial strength, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(4.96 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.431 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.431 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

		Ties: #3(0.375 in) - 10 in	
		Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength	$\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(4.96 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0018541$	Status: PASS Ratio: 0.000
		Shear Strength (ACI 318-19, LRFD)	
		Parameters:	
22.5.2.2	$b_w = 48 \text{ in}$ - Effective width, d - Effective depth	$d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor	$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete	$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 4.96 \text{ kip} \rightarrow 4960 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)	$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(4960 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.15 \text{ kip}$	
22.5.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b)	$V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$	
	V_c - Governing shear strength of concrete	$V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(296.21 \text{ kip}), (119.15 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.15 \text{ kip}$	

14.5.2.1b	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 19.044 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.044 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0763$	Status: PASS Ratio: 0.080
	Considering z-direction: $M_{max} = 0.57486 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.57486 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0023031$	Status: PASS Ratio: 0.000