

Your Project Calculations



Project Name: Stone

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Stone&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2024

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=YdFRC3VXSNzUGPLLMRICD5ffRfNwdRLOUvxsjiveZslkjres0vjxBg8DMr5ZIVl

Array Specification

Product:	Beam
Unique ID:	2P-15-6TOP-SD-45-L-4Hx4W-196K
Duty Classification:	SD
Module Width:	43.63 in
Module Length:	92.36in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	16.20 ft
Total Frame Length:	30.00 ft
Frame Weight:	1247 lbs
Array Dimensions N/S:	14.71 ft
Array Dimensions E/W:	31.12 ft
Rail Length:	176.50 in
Rail Spacing:	3.85 ft
Rail Check:	Not Checked

Support Specifications

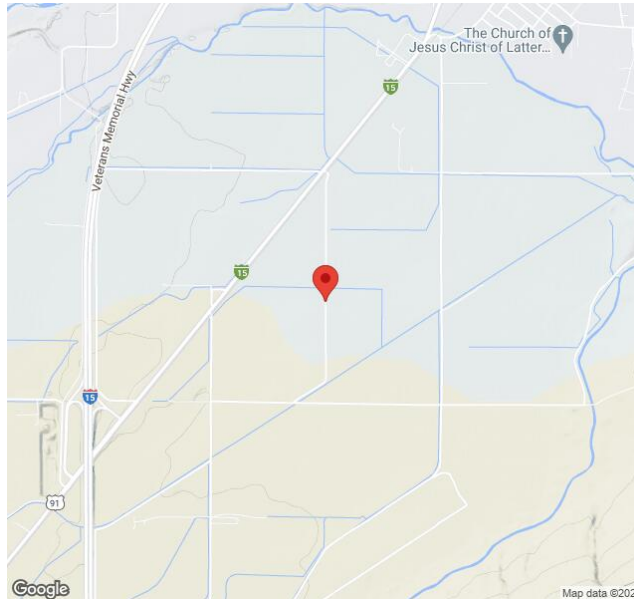
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.63 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 5.75 ft
Foundation Volume:	6.815 y ³
Foundation Result:	PASSED
Mount Twist:	0.441487 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Stone Ln, Idaho 83221, USA
Wind Speed:	95 mph
Snow Load:	78 psf
Design Uplift Pressure:	0.016247 ksf
Design Downforce Pressure:	-0.016247 ksf
Design Snow Pressure:	0.017154 ksf



Design Disclaimer

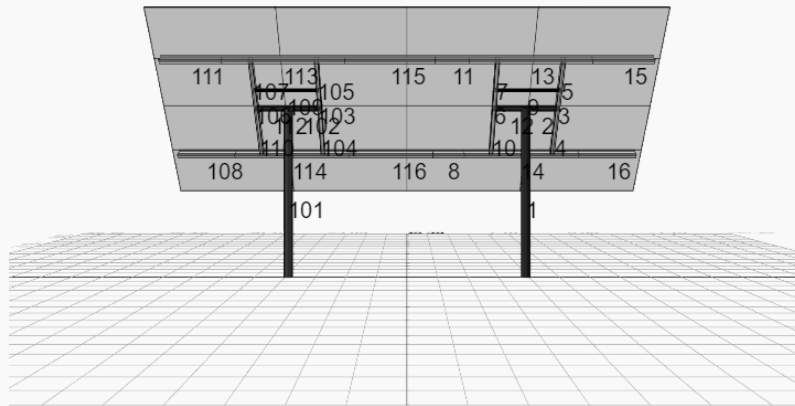
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

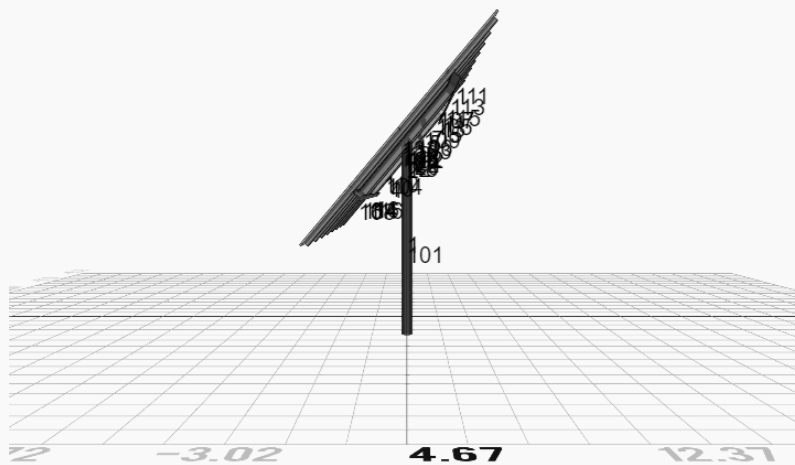
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Design Notes:

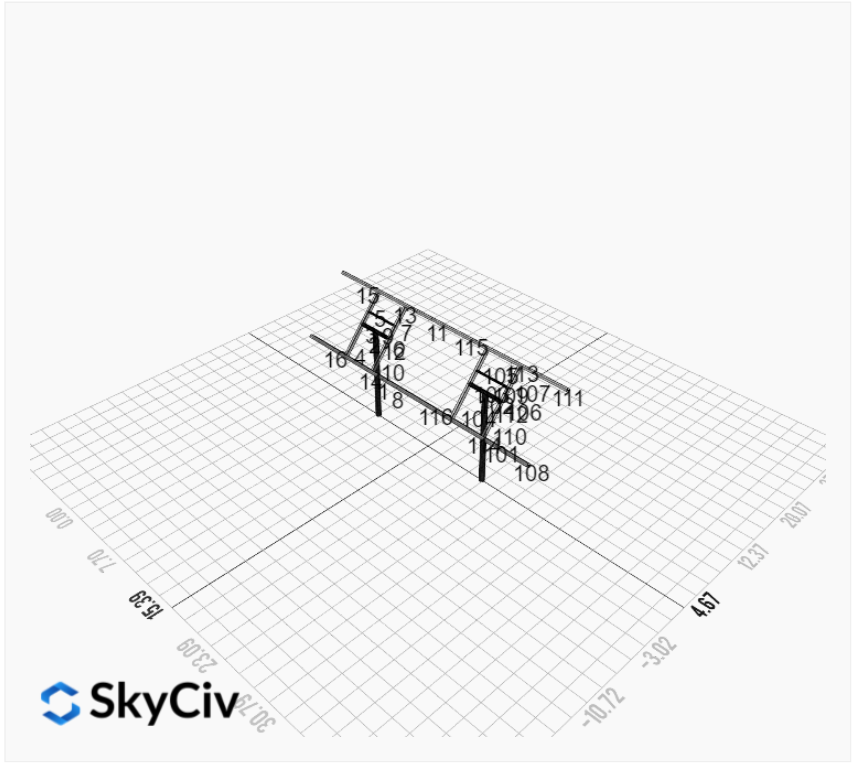
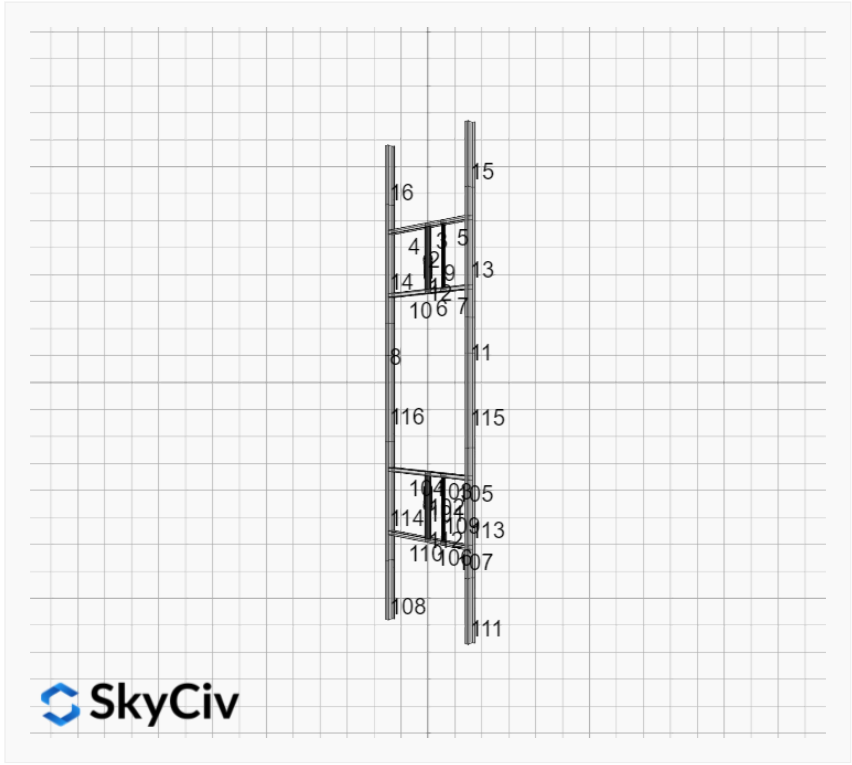
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

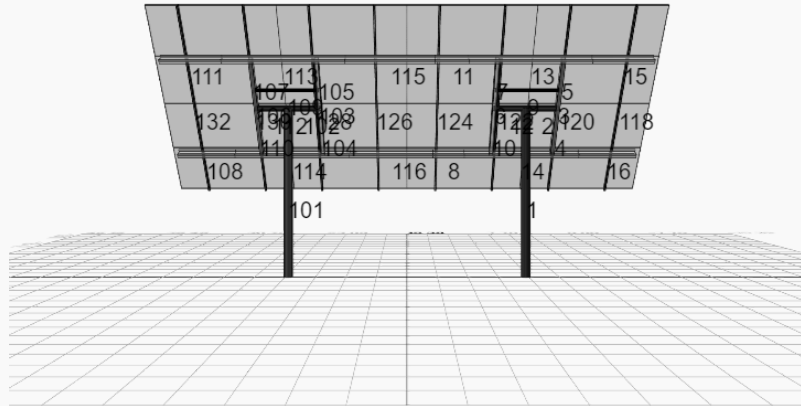


 SkyCiv

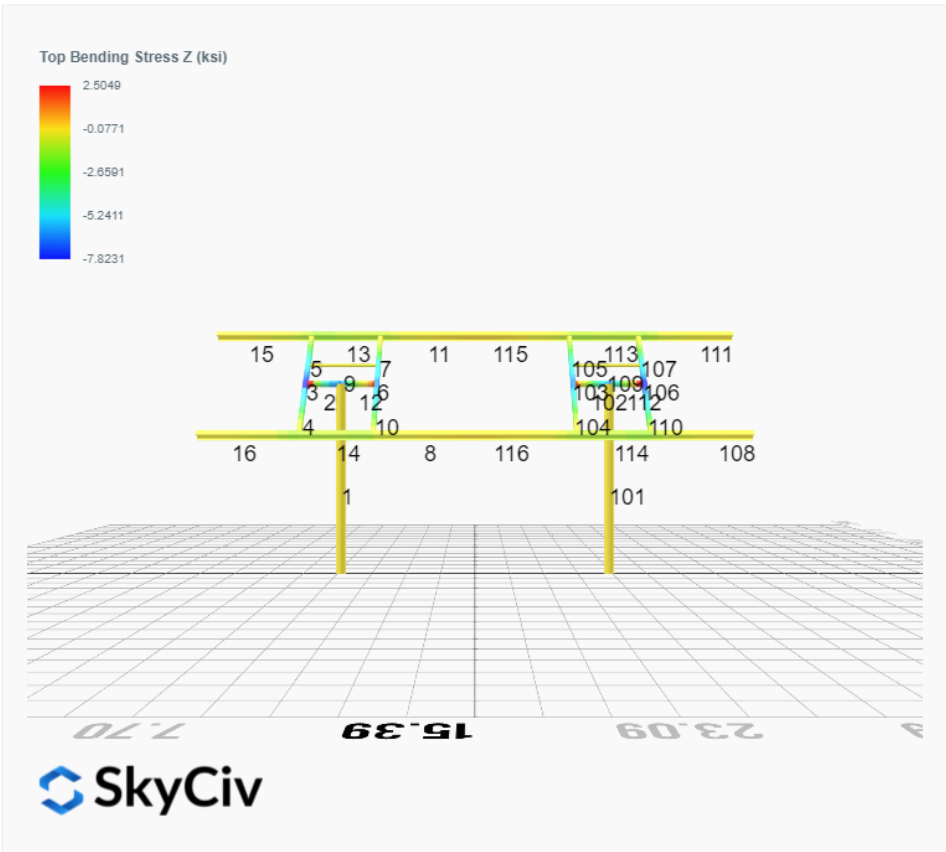
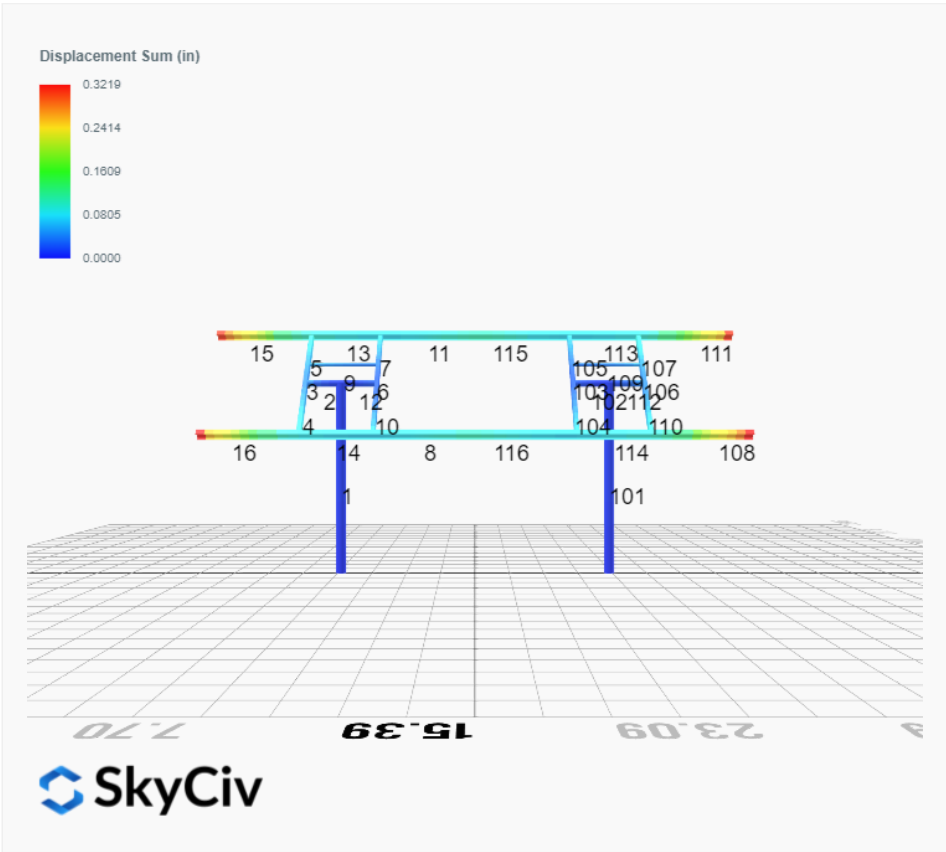


 SkyCiv

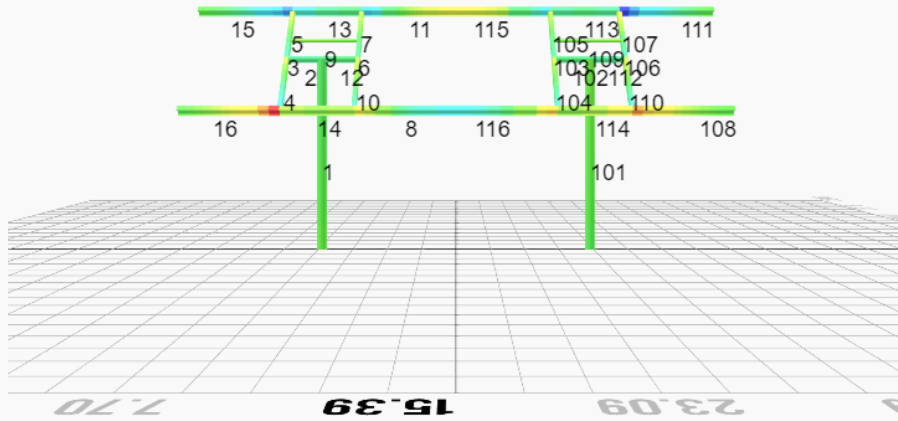
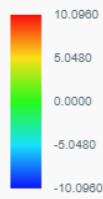




FEM Results (Envelope Worst Case for each member)

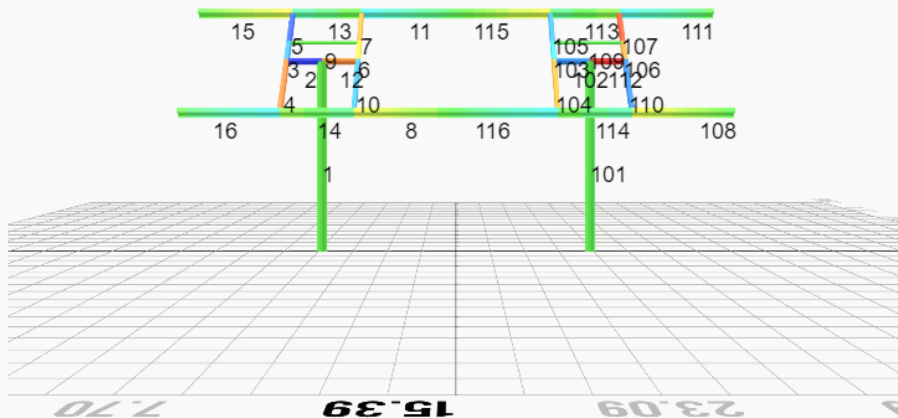


Top Bending Stress Y (ksi)



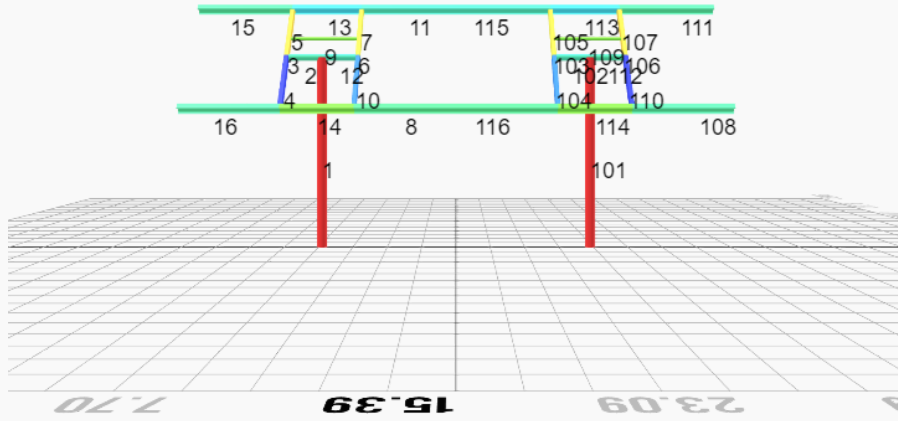
 SkyCiv

Shear Stress Y (ksi)



 SkyCiv

Axial Stress (ksi)



 SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 2. D + L	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 3. D + (S or Lr or R)	0.0000	4.0919	-0.0738	-0.2432	0.1659	0.0264
ULS: 3. D + (S or Lr or R)	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.4837	-0.0618	-0.2037	0.1390	0.0237
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 5b. D + 0.7E	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.4837	-0.0618	-0.2037	0.1390	0.0237
ULS: 8. 0.6D + 0.7E	0.0000	0.9955	-0.0155	-0.0511	0.0349	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7091	3.0933	-0.0824	-0.2678	0.2494	18.4841
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7091	0.2251	0.0307	0.0969	-0.1333	-17.8750
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2818	4.5593	-0.1042	-0.3407	0.2823	13.8752
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.4837	-0.0618	-0.2037	0.1390	0.0237
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2818	2.4081	-0.0194	-0.0671	-0.0047	-13.3942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.4837	-0.0618	-0.2037	0.1390	0.0237
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2818	2.7348	-0.0683	-0.2222	0.2016	13.8670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2818	0.5836	0.0166	0.0514	-0.0854	-13.4024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6592	-0.0259	-0.0852	0.0582	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7091	2.4296	-0.0721	-0.2337	0.2261	18.4779
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.9955	-0.0155	-0.0511	0.0349	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7091	-0.4386	0.0411	0.1310	-0.1566	-17.8812
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.9955	-0.0155	-0.0511	0.0349	0.0093

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.0785
Shear X	-2.8485
Shear Z	-0.1549
Moment X	-0.5088
Moment Y (Twist)	0.4412
Moment Z	31.5348

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	4.5593
Shear X	-1.7091
Shear Z	-0.1042
Moment X	-0.3407
Moment Y (Twist)	0.2823
Moment Z	18.4841

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	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 2. D + L	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 3. D + (S or Lr or R)	-0.0000	4.0919	0.0738	0.2432	-0.1659	0.0264
ULS: 3. D + (S or Lr or R)	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.4837	0.0618	0.2037	-0.1390	0.0237
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 5b. D + 0.7E	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.4837	0.0618	0.2037	-0.1390	0.0237
ULS: 8. 0.6D + 0.7E	-0.0000	0.9955	0.0155	0.0511	-0.0349	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7091	3.0933	0.0824	0.2678	-0.2494	18.4842
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7091	0.2251	-0.0307	-0.0969	0.1333	-17.8750
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2818	4.5593	0.1042	0.3407	-0.2823	13.8752
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	3.4837	0.0618	0.2037	-0.1390	0.0237
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2818	2.4081	0.0194	0.0671	0.0047	-13.3942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	3.4837	0.0618	0.2037	-0.1390	0.0237
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2818	2.7348	0.0683	0.2222	-0.2016	13.8670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2818	0.5836	-0.0166	-0.0514	0.0854	-13.4024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.6592	0.0259	0.0852	-0.0582	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7091	2.4296	0.0721	0.2337	-0.2261	18.4779
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.9955	0.0155	0.0511	-0.0349	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7091	-0.4386	-0.0411	-0.1310	0.1566	-17.8812
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.9955	0.0155	0.0511	-0.0349	0.0093

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.0785
Shear X	-2.8485
Shear Z	0.1549
Moment X	0.5088
Moment Y (Twist)	0.4415
Moment Z	31.5354

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	4.5593
Shear X	-1.7091
Shear Z	0.1042
Moment X	0.3407
Moment Y (Twist)	0.2823
Moment Z	18.4842

Project Details

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

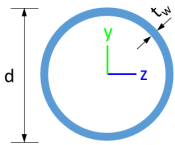
User Name: sales@mtsolar.us
Unit System: imperial



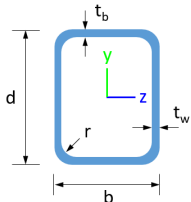
Design Input Information

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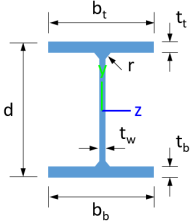
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

Section Dimensions								
								

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

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

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

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

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	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	22.33	22.33	10.63	-	300	200	1	
2	4	2.00	1.30	2.00	-	300	200	1	
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18	300	200	1	
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
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9	1	2.60	2.60	4.00	-	300	200	1	
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
11	18	1.33	1.33	2.05	2.36,2.36,2.36,2.36,2.36,2.36,2.35,2.36,2.32,2.36,2.35,2.36,2.33,2.36,2.35,2.36,2.39,2.36,2.35,2.36,2.32,2.36,2.34,2.36,2.34,2.36	300	200	1	
12	4	1.30	1.30	2.00	-	300	200	1	
13	18	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.11,1.10,1.11,1.10,1.11	300	200	1	
14	18	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.11,1.10,1.10,1.10,1.10,1.10,1.10	300	200	1	
15	18	7.88	7.88	3.75	2.33,2.33	300	200	1	
16	18	7.88	7.88	3.75	2.33,2.33	300	200	1	
101	7	22.33	22.33	10.63	-	300	200	1	
102	4	1.30	1.30	2.00	-	300	200	1	
103	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18	300	200	1	
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
106	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
107	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	

115	120.00	95.07	23.36	6.45	30.09	45.74
116	120.60	95.07	23.36	6.45	30.09	45.74

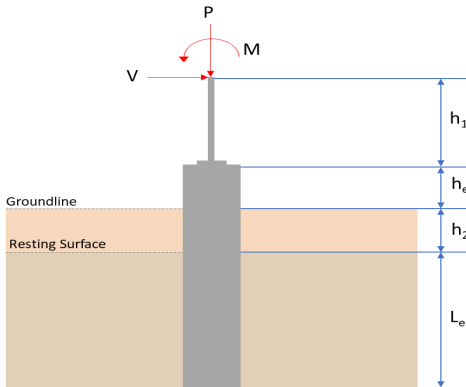
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.080	0.746	0.027	0.038	0.002	0.789	#13	0.597	Not Required	Pass
2	0.001	0.359	0.199	0.087	0.035	0.502	#13	0.079	Not Required	Pass
3	0.016	0.513	0.192	0.052	0.029	0.701	#21	0.044	Not Required	Pass
4	0.015	0.510	0.295	0.051	0.037	0.696	#21	0.078	Not Required	Pass
5	0.016	0.318	0.307	0.051	0.043	0.373	#21	0.073	Not Required	Pass
6	0.014	0.448	0.135	0.044	0.017	0.575	#21	0.044	Not Required	Pass
7	0.013	0.278	0.208	0.045	0.030	0.323	#21	0.073	Not Required	Pass
8	0.001	0.033	0.093	0.023	0.011	0.104	#21	0.088	Not Required	Pass
9	0.005	0.036	0.063	0.001	0.002	0.093	#13	0.198	Not Required	Pass
10	0.012	0.445	0.228	0.045	0.031	0.634	#21	0.078	Not Required	Pass
11	0.002	0.033	0.094	0.023	0.011	0.106	#21	0.059	Not Required	Pass
12	0.002	0.289	0.172	0.073	0.032	0.407	#13	0.052	Not Required	Pass
13	0.006	0.147	0.327	0.034	0.017	0.467	#21	0.177	Not Required	Pass
14	0.007	0.150	0.327	0.034	0.017	0.467	#21	0.177	Not Required	Pass
15	0.000	0.055	0.152	0.023	0.011	0.206	#21	Not Required	Not Required	Pass
16	0.000	0.055	0.152	0.023	0.011	0.206	#21	Not Required	Not Required	Pass
101	0.080	0.746	0.027	0.038	0.002	0.789	#13	0.597	Not Required	Pass
102	0.002	0.289	0.172	0.073	0.032	0.407	#13	0.052	Not Required	Pass
103	0.014	0.448	0.135	0.044	0.017	0.575	#21	0.044	Not Required	Pass
104	0.012	0.445	0.228	0.045	0.031	0.634	#21	0.078	Not Required	Pass
105	0.013	0.278	0.208	0.045	0.030	0.323	#21	0.073	Not Required	Pass
106	0.016	0.513	0.192	0.052	0.029	0.701	#21	0.044	Not Required	Pass
107	0.016	0.318	0.307	0.051	0.043	0.373	#21	0.073	Not Required	Pass
108	0.000	0.055	0.152	0.023	0.011	0.206	#21	Not Required	Not Required	Pass
109	0.005	0.036	0.063	0.001	0.002	0.093	#13	0.198	Not Required	Pass
110	0.015	0.510	0.295	0.051	0.037	0.696	#21	0.078	Not Required	Pass
111	0.000	0.055	0.152	0.023	0.011	0.206	#21	Not Required	Not Required	Pass
112	0.001	0.359	0.199	0.087	0.035	0.502	#13	0.079	Not Required	Pass
113	0.006	0.147	0.327	0.034	0.017	0.467	#21	0.177	Not Required	Pass
114	0.007	0.150	0.327	0.034	0.017	0.467	#21	0.265	Not Required	Pass
115	0.002	0.033	0.124	0.023	0.011	0.147	#21	0.156	Not Required	Pass
116	0.002	0.033	0.124	0.023	0.011	0.145	#21	0.235	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS																										
	<div><div>SkyCiv Foundation Design</div><div>Pile Foundation</div><div>Design Information :</div><div>Design code : IBC 2021 (International Building Code)</div><div>Unit System : Imperial</div></div>																											
	<div><div>Pile Input</div><div></div><div><div>Geometry</div><div>Pile shape: rectangular</div><div>b = 48 in - Pile width</div><div>D = 48 in - Pile depth</div><div>L = 5.75 ft - Total pile length</div><div>h1 = 0 ft - Lateral load height from the top of the pile,</div><div>h2 = 0 ft - Depth to resisting surface</div><div>he = 0 ft - Length of pile above the ground</div></div><div><div>Tabulation of Soil Parameters</div><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></div><div><div>Tabulation of Loads</div><table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>4.559</td><td>7.078</td></tr><tr><td>Vx (kip)</td><td>-1.709</td><td>-2.849</td></tr><tr><td>Vz (kip)</td><td>-0.104</td><td>-0.155</td></tr><tr><td>Mx (kipft)</td><td>-0.341</td><td>-0.509</td></tr><tr><td>Mz (kipft)</td><td>18.484</td><td>31.535</td></tr></table></div><div><div>Material Properties</div><div>f'ck = 2.5 ksi - Concrete strength,</div></div></div>	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)	4.559	7.078	Vx (kip)	-1.709	-2.849	Vz (kip)	-0.104	-0.155	Mx (kipft)	-0.341	-0.509	Mz (kipft)	18.484	31.535	
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	<div><div>Required depth to resist lateral loads (ASD)</div><div>H - Point of application of the lateral load</div><div><div><div><div><div>$H = h_1 + h_2 + h_e$</div><div>$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$</div><div>$H = 0 \text{ ft}$</div></div></div></div></div><div><div>Considering x-direction:</div><div>Ho - Lateral force per length of pile,</div><div><div><div><div>$H_o = \frac{V_x}{1.57 D}$</div><div>$H_o = \frac{(-1.709 \text{ kip})}{1.57 \times (48 \text{ in})}$</div><div>$H_o = -0.27213 \text{ kip/ft}$</div></div></div></div><div><div>Mo - Moment per length of pile,</div><div><div><div><div>$M_o = \frac{M_z + (V_x H)}{1.57 D}$</div></div></div></div></div></div></div>																											

	$M_o = \frac{(18.484 \text{ kipft}) + ((-1.709 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.9433 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 5.3005 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(-0.104 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.016561 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.341 \text{ kipft}) + ((-0.104 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.054299 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,z} = 1.4299 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required:</p> <p>$L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.3005 \text{ ft}), (1.4299 \text{ ft})]$ $L_{e,req} = 5.301 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ <p>Ratio - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.301 \text{ ft})}{(5.75 \text{ ft})}$ $\text{Ratio} = 0.92191$	<p>Status: PASS Ratio: 0.920</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_o}{A}$ $q = \frac{(4.559 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.28494 \text{ kips/ft}^2$	

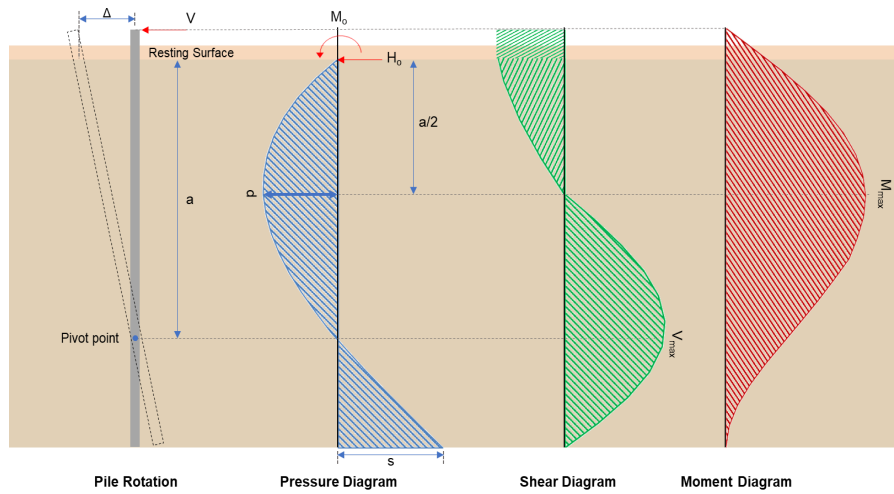
	<p>$q = 0.28494 \text{ kip/ft}^2$</p> <p>Check bearing capacity ratio:</p> <p>Ratio - Capacity</p> $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.28494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.14247$	Status: PASS Ratio: 0.140
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.4375$ <p>Since $L/D \leq 10$,</p> <p>Pile is short.</p> <p>Considering x-direction:</p> <p>$H_o = -0.27213 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 2.9433 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (2.9433 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.9433 \text{ kipft/ft})) + (4 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 3.9587 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (2.9433 \text{ kipft/ft})) + (3 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (2.9433 \text{ kipft/ft})) + (2 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.19941 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (2.9433 \text{ kipft/ft})) + ((-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.78431 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.9587 \text{ ft})}{2}$ $p_a = 0.2969 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.19941 \text{ kip/ft}^2)}{(0.2969 \text{ kip/ft}^2)}$ $Ratio = 0.67165$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p>	Status: PASS Ratio: 0.670

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.78431 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90934$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = -0.016561 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.054299 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.054299 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.016561 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.054299 \text{ kipft/ft})) + (4 \times (-0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0916 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.054299 \text{ kipft/ft})) + (3 \times (-0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (0.054299 \text{ kipft/ft})) + (2 \times (-0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = -0.0038606 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.054299 \text{ kipft/ft})) + ((-0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.0024274 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.0916 \text{ ft})}{2}$ $p_a = 0.30687 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0038606 \text{ kip/ft}^2)}{(0.30687 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.012581$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.010

$$Ratio = \frac{(0.0024274 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0028143$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.849 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45366 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.535 \text{ kipft}) + ((-2.849 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0215 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0215 \text{ kipft/ft})}{(-0.45366 \text{ kip/ft})}$$

$$E = 11.069 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0215 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.45366 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.0215 \text{ kipft/ft})) + (4 \times (-0.45366 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9566 \text{ ft}$$

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

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

$$V_{max} = ((-0.45366 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9566 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9566 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.3789 \text{ kip}$$

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

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

$$M_{max} = ((-0.45366 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(11.069 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.266 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.024682 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.509 \text{ kipft}) + ((-0.155 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.081051 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.081051 \text{ kipft/ft})}{(-0.024682 \text{ kip/ft})}$$

$$E = 3.2839 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.081051 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.024682 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.081051 \text{ kipft/ft})) + (4 \times (-0.024682 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0914 \text{ ft}$$

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

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

$$V_{max} = ((-0.024682 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0914 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0914 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.16542 \text{ kip}$$

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

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

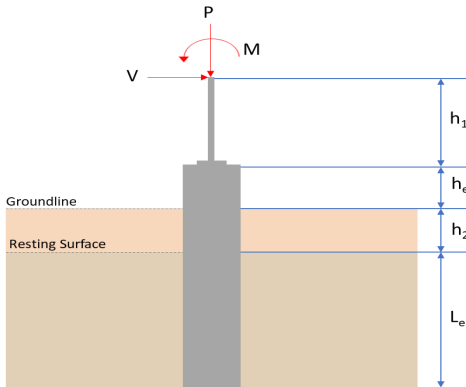
$$M_{max} = ((-0.024682 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(3.2839 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.42485 \text{ kipft}$	
		<p>Minimum Reinforcement Check (LRFD)</p> <p>Parameters:</p> <p>$f'_{ck} = 2.5 \text{ ksi}$ - 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,</p> <p>Longitudinal reinforcement:</p> <p>Required reinforcement due to axial load, $A_{st,required}$</p> <p>$A_{st,required}$</p> $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(7.078 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.361 \text{ in}^2$ <p>A_{min} - Governing minimum reinforcement area,</p> $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.361 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ <p>n_{rebar} - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <p>A_{st} - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ <p>25.2.3 s_{rebar} - Minimum spacing of reinforcement,</p> $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in)</p> <p>25.7.2.1 s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p>Main reinforcement: 14 - #5 (0.625 in)</p>	<p>Status: PASS Ratio: 0.970</p>

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(7.078 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0026458$	Status: PASS Ratio: 0.000
22.5.2.2	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	<p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	<p>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</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.078 \text{ kip} \rightarrow 7078 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7078 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.43 \text{ kip}$	
22.5.5.1.2	<p>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)</p> $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$ <p>V_c - Governing shear strength of concrete</p> $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (119.43 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.43 \text{ kip}$	

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p>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)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.43 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.71 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.3789 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.3789 \text{ kip})}{(110.71 \text{ kip})}$ $Ratio = 0.066651$ <p>Considering z-direction:</p> <p>$V_{max} = 0.16542 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.16542 \text{ kip})}{(110.71 \text{ kip})}$ $Ratio = 0.0014942$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	<p> $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ </p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p> $\phi M_{n,2}$ </p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p> Therefore, ϕM_n - Allowable flexural strength, </p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p> Considering x-direction: $M_{max} = 20.266 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity </p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(20.266 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.081193$	Status: PASS Ratio: 0.080
	<p> Considering z-direction: $M_{max} = 0.42485 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity </p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.42485 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0017021$	Status: PASS Ratio: 0.000

REFERENCES	CALCULATIONS	RESULTS																										
	<div><div>SkyCiv Foundation Design</div><div>Pile Foundation</div><div>Design Information :</div><div>Design code : IBC 2021 (International Building Code)</div><div>Unit System : Imperial</div></div>																											
	<div><div>Pile Input</div><div></div><div><div>Geometry</div><div>Pile shape: rectangular</div><div>b = 48 in - Pile width</div><div>D = 48 in - Pile depth</div><div>L = 5.75 ft - Total pile length</div><div>h1 = 0 ft - Lateral load height from the top of the pile,</div><div>h2 = 0 ft - Depth to resisting surface</div><div>he = 0 ft - Length of pile above the ground</div></div><div><div>Tabulation of Soil Parameters</div><table><thead><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr></thead><tbody><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></tbody></table></div><div><div>Tabulation of Loads</div><table><thead><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr></thead><tbody><tr><td>P (kip)</td><td>4.559</td><td>7.078</td></tr><tr><td>Vx (kip)</td><td>-1.709</td><td>-2.849</td></tr><tr><td>Vz (kip)</td><td>0.104</td><td>0.155</td></tr><tr><td>Mx (kipft)</td><td>0.341</td><td>0.509</td></tr><tr><td>Mz (kipft)</td><td>18.484</td><td>31.535</td></tr></tbody></table></div><div><div>Material Properties</div><div>f'ck = 2.5 ksi - Concrete strength,</div></div></div>	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)	4.559	7.078	Vx (kip)	-1.709	-2.849	Vz (kip)	0.104	0.155	Mx (kipft)	0.341	0.509	Mz (kipft)	18.484	31.535	
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	<div><div>Required depth to resist lateral loads (ASD)</div><div>H - Point of application of the lateral load</div><div><div><div><div><div>$H = h_1 + h_2 + h_e$</div><div>$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$</div><div>$H = 0 \text{ ft}$</div></div></div></div></div><div><div>Considering x-direction:</div><div>Ho - Lateral force per length of pile,</div><div><div><div><div>$H_o = \frac{V_x}{1.57 D}$</div><div>$H_o = \frac{(-1.709 \text{ kip})}{1.57 \times (48 \text{ in})}$</div><div>$H_o = -0.27213 \text{ kip/ft}$</div></div></div></div><div><div>Mo - Moment per length of pile,</div><div><div><div><div>$M_o = \frac{M_z + (V_x H)}{1.57 D}$</div></div></div></div></div></div></div>																											

	$M_o = \frac{(18.484 \text{ kipft}) + ((-1.709 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.9433 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 5.3005 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0.104 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0.016561 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.341 \text{ kipft}) + ((0.104 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.054299 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,z} = 1.8337 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required:</p> <p>$L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.3005 \text{ ft}), (1.8337 \text{ ft})]$ $L_{e,req} = 5.301 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ <p>Ratio - Embedded depth</p> $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.301 \text{ ft})}{(5.75 \text{ ft})}$ $Ratio = 0.92191$	<p>Status: PASS Ratio: 0.920</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_o}{A}$ $q = \frac{(4.559 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.28494 \text{ kips/ft}^2$	

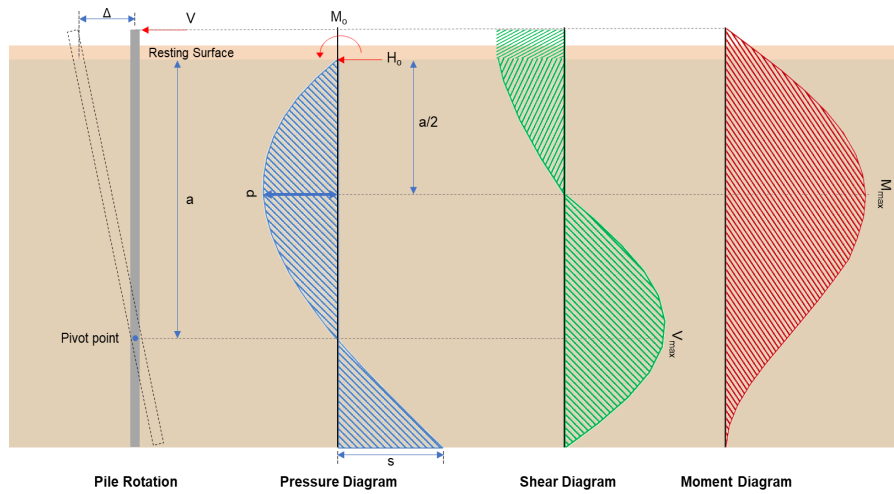
	$q = 0.28494 \text{ kip/ft}^2$	
	<p>Check bearing capacity ratio:</p> <p>Ratio - Capacity</p> $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.28494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.14247$	<p>Status: PASS Ratio: 0.140</p>
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$ $L/D = 1.4375$ <p>Since $L/D \leq 10$,</p> <p>Pile is short.</p> <p>Considering x-direction:</p> <p>$H_o = -0.27213 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 2.9433 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (2.9433 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.9433 \text{ kipft/ft})) + (4 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 3.9587 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (2.9433 \text{ kipft/ft})) + (3 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (2.9433 \text{ kipft/ft})) + (2 \times (-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.19941 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (2.9433 \text{ kipft/ft})) + ((-0.27213 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.78431 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.9587 \text{ ft})}{2}$ $p_a = 0.2969 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $Ratio = \frac{p}{p_a}$ $Ratio = \frac{(0.19941 \text{ kip/ft}^2)}{(0.2969 \text{ kip/ft}^2)}$ $Ratio = 0.67165$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p>	<p>Status: PASS Ratio: 0.670</p>

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.78431 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90934$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = 0.016561 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.054299 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.054299 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.016561 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.054299 \text{ kipft/ft})) + (4 \times (0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0916 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^3 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.054299 \text{ kipft/ft})) + (3 \times (0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (0.054299 \text{ kipft/ft})) + (2 \times (0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.016234 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.054299 \text{ kipft/ft})) + ((0.016561 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.036988 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.0916 \text{ ft})}{2}$ $p_a = 0.30687 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.016234 \text{ kip/ft}^2)}{(0.30687 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.052903$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.050

$$Ratio = \frac{(0.036988 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.042885$$

Status: **PASS**
Ratio: **0.040**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.849 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45366 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.535 \text{ kipft}) + ((-2.849 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0215 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0215 \text{ kipft/ft})}{(-0.45366 \text{ kip/ft})}$$

$$E = 11.069 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0215 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.45366 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.0215 \text{ kipft/ft})) + (4 \times (-0.45366 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9566 \text{ ft}$$

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

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

$$V_{max} = ((-0.45366 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9566 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9566 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.3789 \text{ kip}$$

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

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

$$M_{max} = ((-0.45366 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(11.069 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9566 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.266 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.024682 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.509 \text{ kipft}) + ((0.155 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.081051 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.081051 \text{ kipft/ft})}{(0.024682 \text{ kip/ft})}$$

$$E = 3.2839 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.081051 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.024682 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.081051 \text{ kipft/ft})) + (4 \times (0.024682 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0914 \text{ ft}$$

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

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

$$V_{max} = ((0.024682 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0914 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0914 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.16542 \text{ kip}$$

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

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

$$M_{max} = ((0.024682 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(3.2839 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2839 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0914 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 0.42485 \text{ kipft}$	
		<p>Minimum Reinforcement Check (LRFD)</p> <p>Parameters:</p> <p>$f'_{ck} = 2.5 \text{ ksi}$ - 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,</p> <p>Longitudinal reinforcement:</p> <p>Required reinforcement due to axial load, $A_{st,required}$</p> <p>$A_{st,required}$</p> $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(7.078 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.361 \text{ in}^2$ <p>A_{min} - Governing minimum reinforcement area,</p> $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.361 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ <p>n_{rebar} - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <p>A_{st} - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ <p>25.2.3 s_{rebar} - Minimum spacing of reinforcement,</p> $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in)</p> <p>25.7.2.1 s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p>Main reinforcement: 14 - #5 (0.625 in)</p>	<p>Status: PASS Ratio: 0.970</p>

		Ties: #3(0.375 in) - 10 in	
		Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength	$\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.078 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0026458$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.078 \text{ kip} \rightarrow 7078 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)	$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7078 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.43 \text{ kip}$	
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	$V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(296.21 \text{ kip}), (119.43 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.43 \text{ kip}$	

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p>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)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{tes}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.43 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.71 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.3789 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.3789 \text{ kip})}{(110.71 \text{ kip})}$ $Ratio = 0.066651$ <p>Considering z-direction:</p> <p>$V_{max} = 0.16542 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.16542 \text{ kip})}{(110.71 \text{ kip})}$ $Ratio = 0.0014942$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

14.5.2.1b	<p> $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ </p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p> $\phi M_{n,2}$ </p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p> Therefore, ϕM_n - Allowable flexural strength, </p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p> Considering x-direction: $M_{max} = 20.266 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity </p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(20.266 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.081193$	Status: PASS Ratio: 0.080
	<p> Considering z-direction: $M_{max} = 0.42485 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity </p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.42485 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0017021$	Status: PASS Ratio: 0.000