

Your Project Calculations



Project Name: MTSOLAR_262JCCIA84CF7

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_262JCCIA84CF7&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=5Vpl6gxyXK0z53Du3hL5AYIJADscEIKX4v888F61BpaJ5VuGz7dpY82WK3cYVjgH

Array Specification

Product:	Beam
Unique ID:	3P-17-8TOP-HD-12-L-4Hx6W-EJ4L
Duty Classification:	HD
Module Width:	44.60 in
Module Length:	89.70in
Number of Rows:	4
Number of Columns:	6
Total Number of Modules:	24
Desired Tilt Angle:	47
Front Edge Clearance:	5
Total Array Height at Tilt:	15.93 ft
Total Frame Length:	43.50 ft
Frame Weight:	2412 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	45.35 ft
Rail Length:	180.40 in
Rail Spacing:	3.74 ft
Rail Check:	Not Checked

Support Specifications

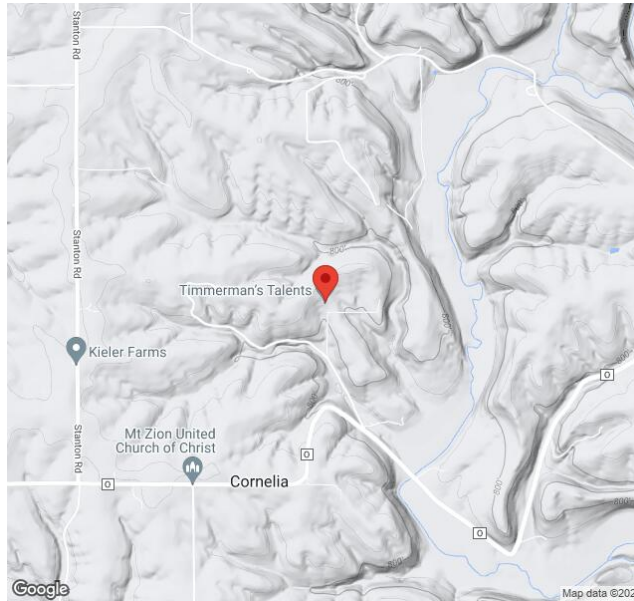
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	10.50 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.25 ft Pile 3: 5.75 ft
Foundation Volume:	10.519 y ³
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	5381 Beagle Ln, Platteville, WI 53818, USA
Wind Speed:	100 mph
Snow Load:	30 psf
Design Uplift Pressure:	0.019947 ksf
Design Downforce Pressure:	-0.019947 ksf
Design Snow Pressure:	0.005175 ksf



Design Disclaimer

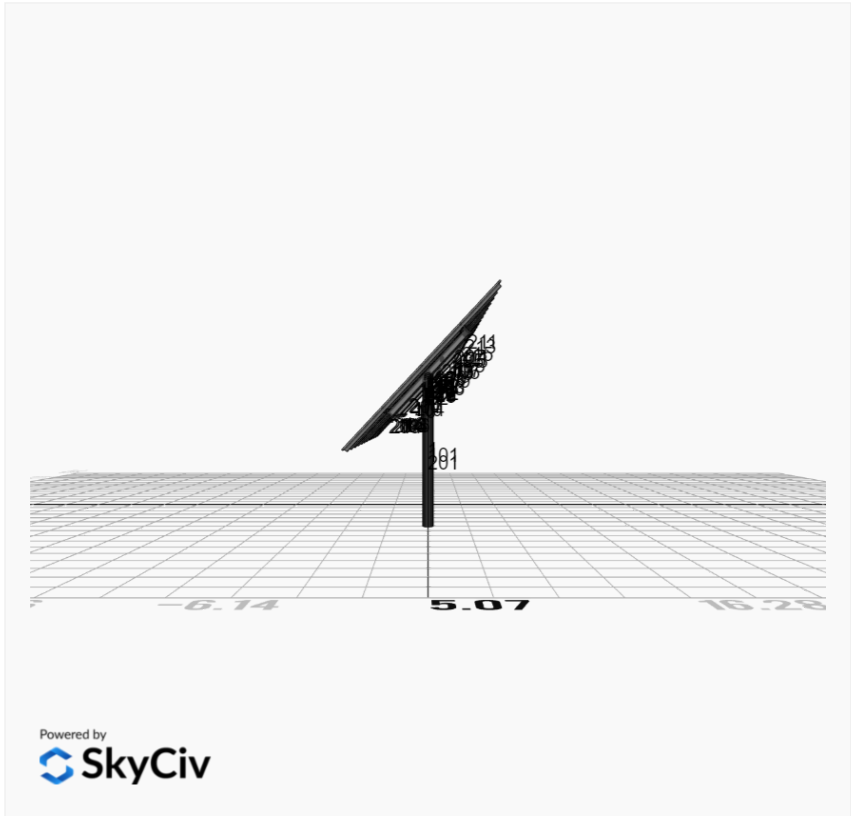
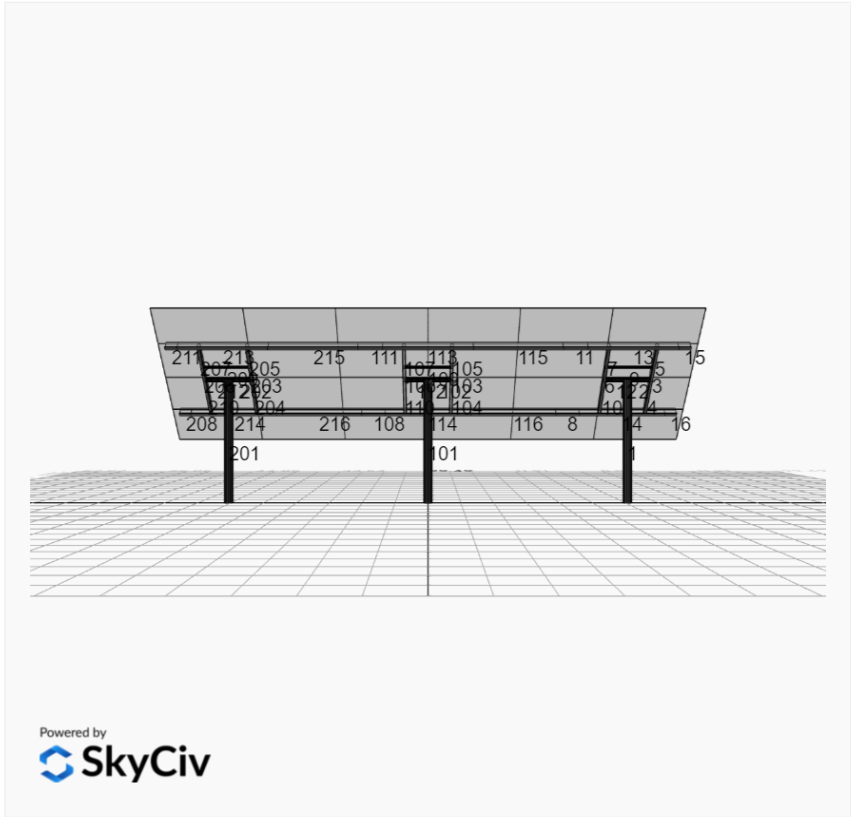
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

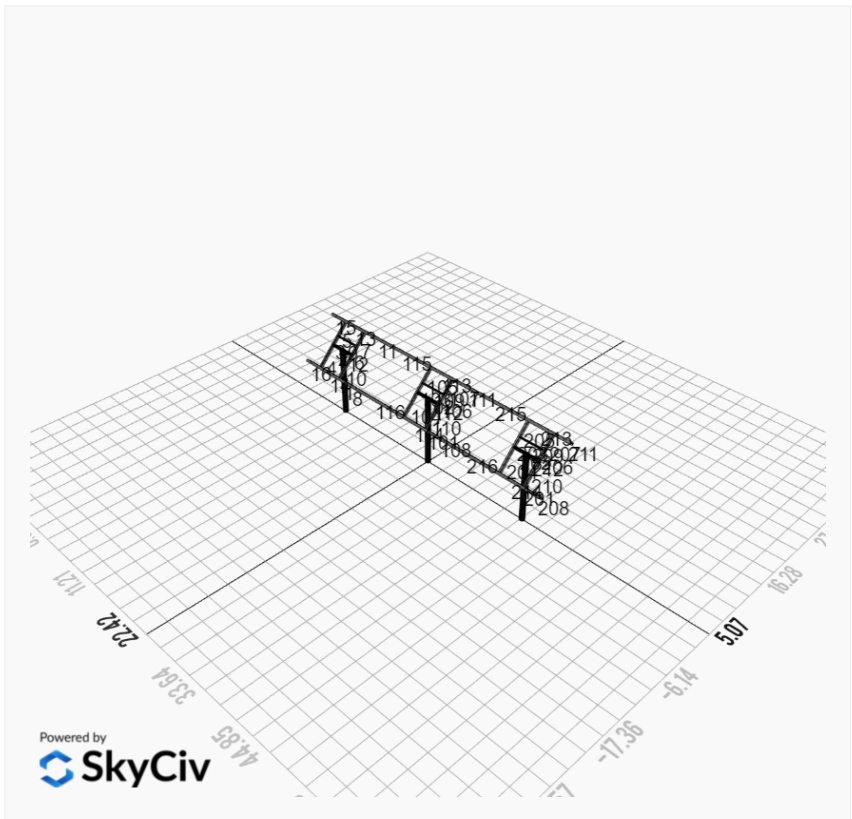
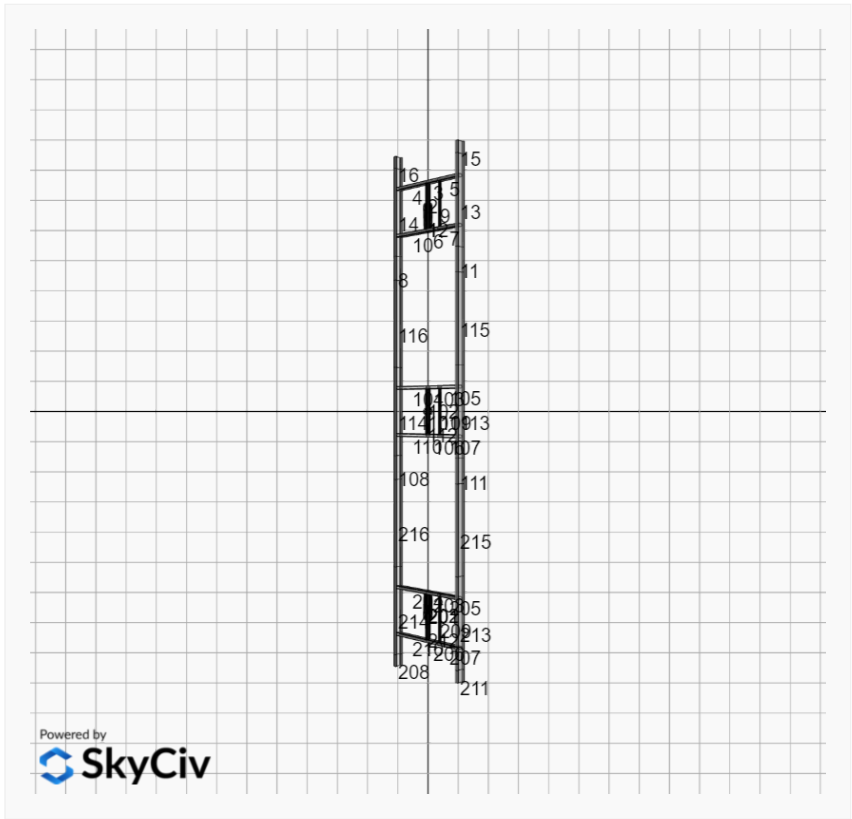
AutoDesigner Input

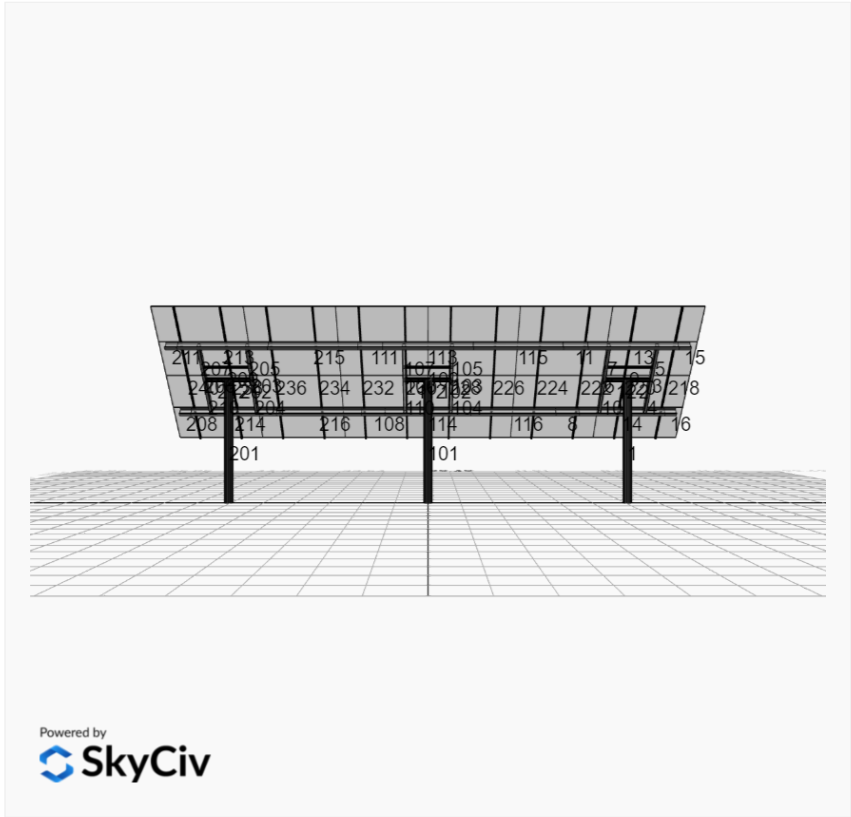
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Design Notes:

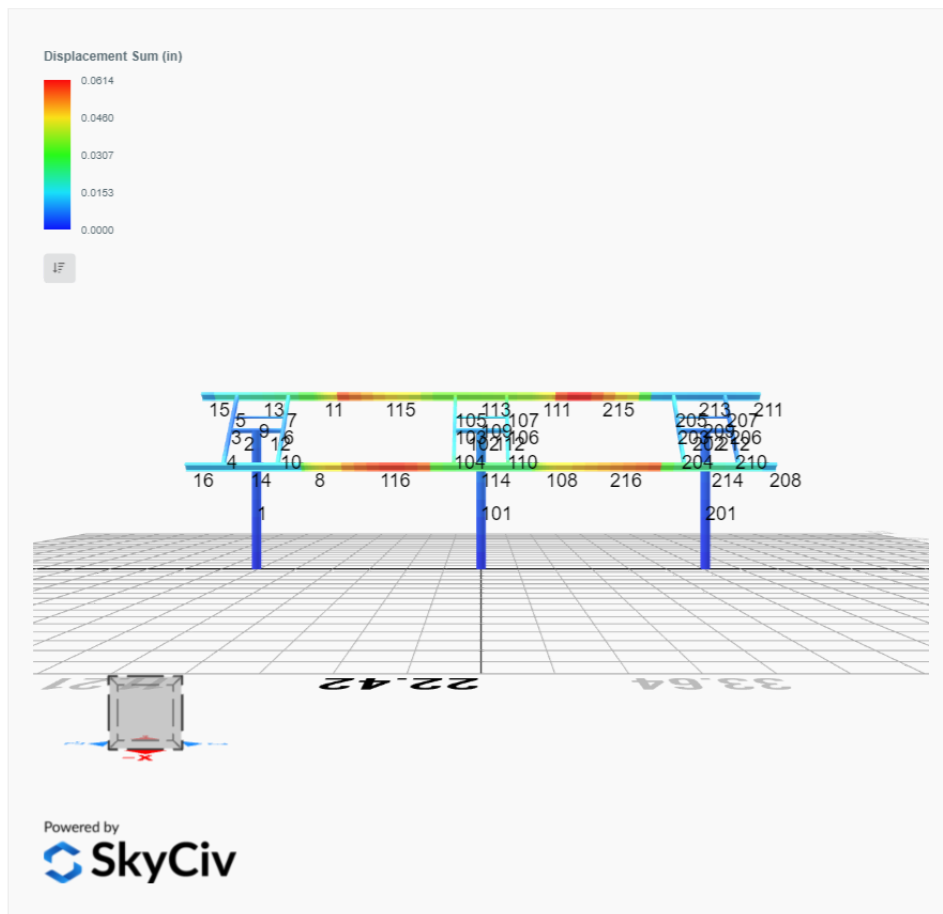
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

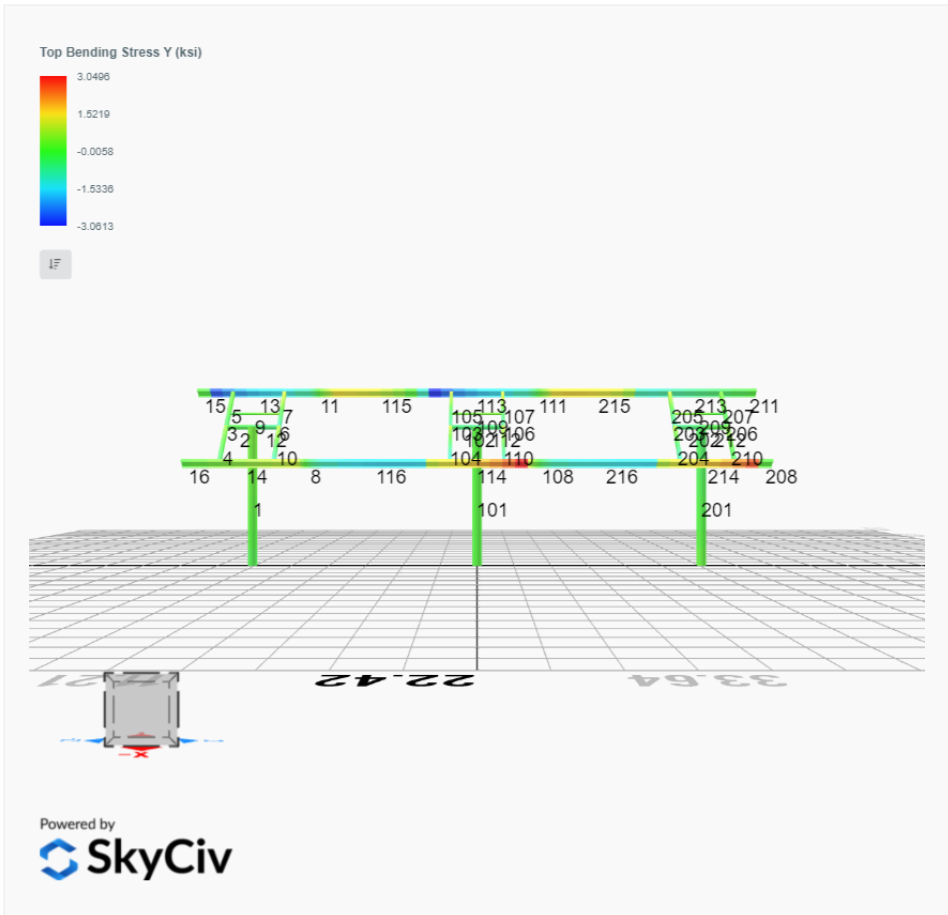
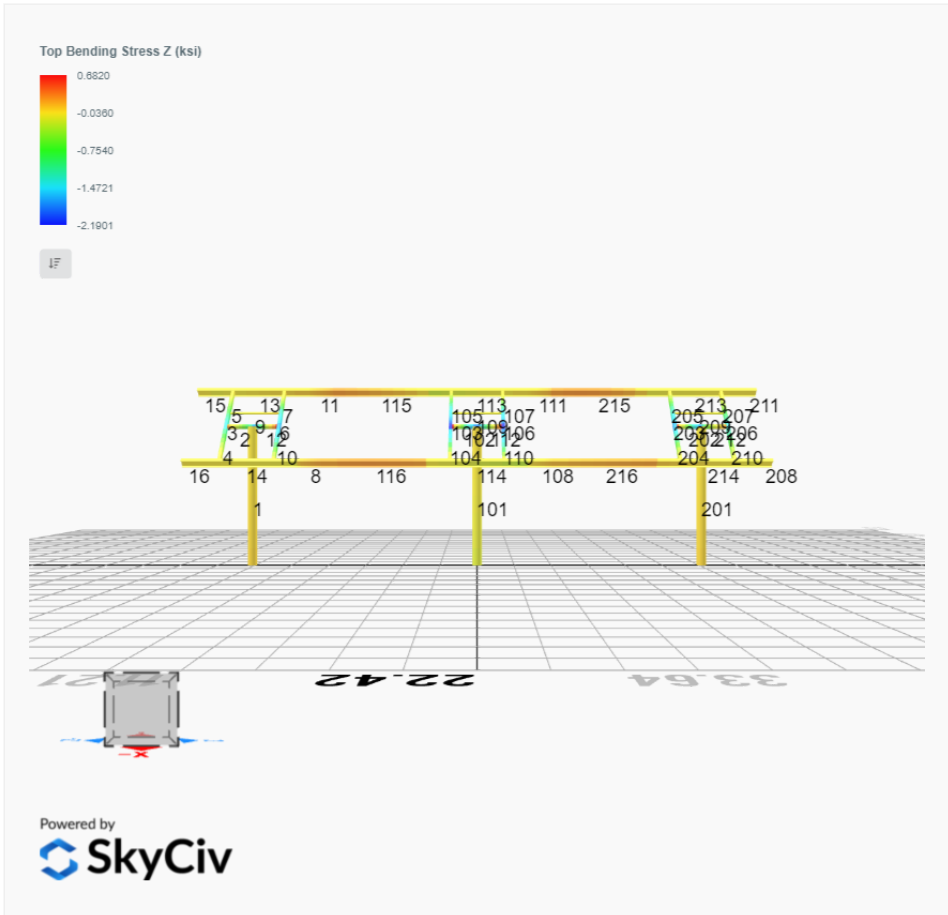


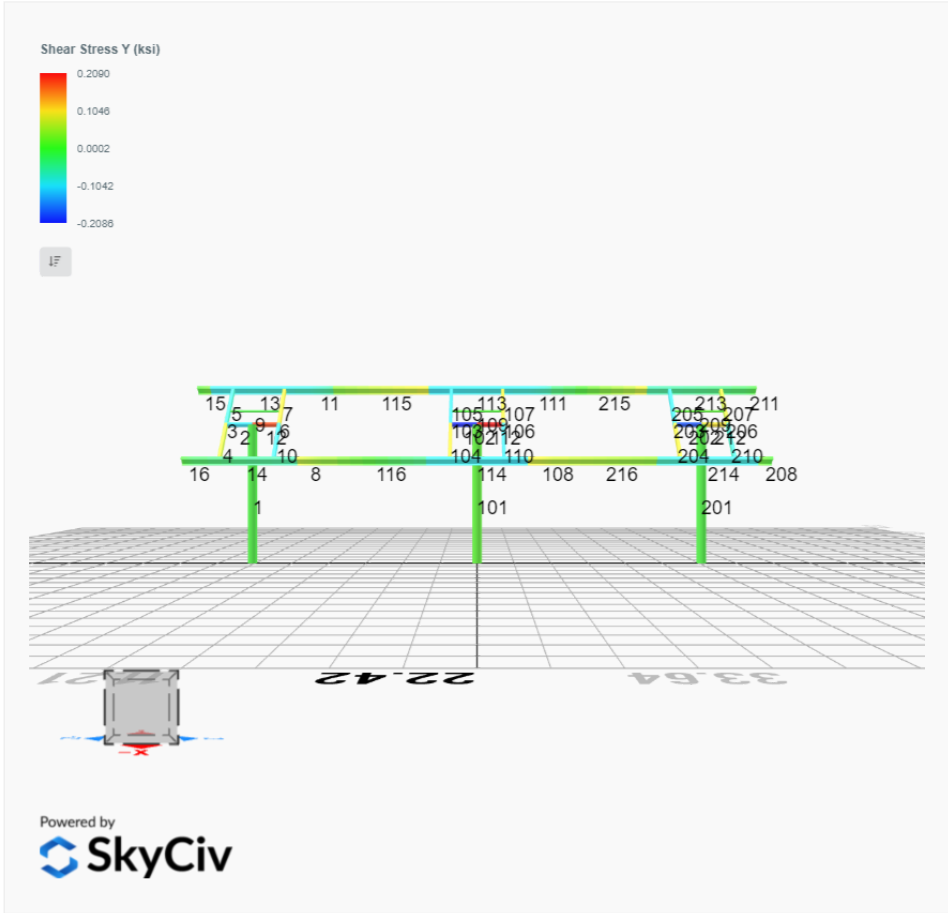


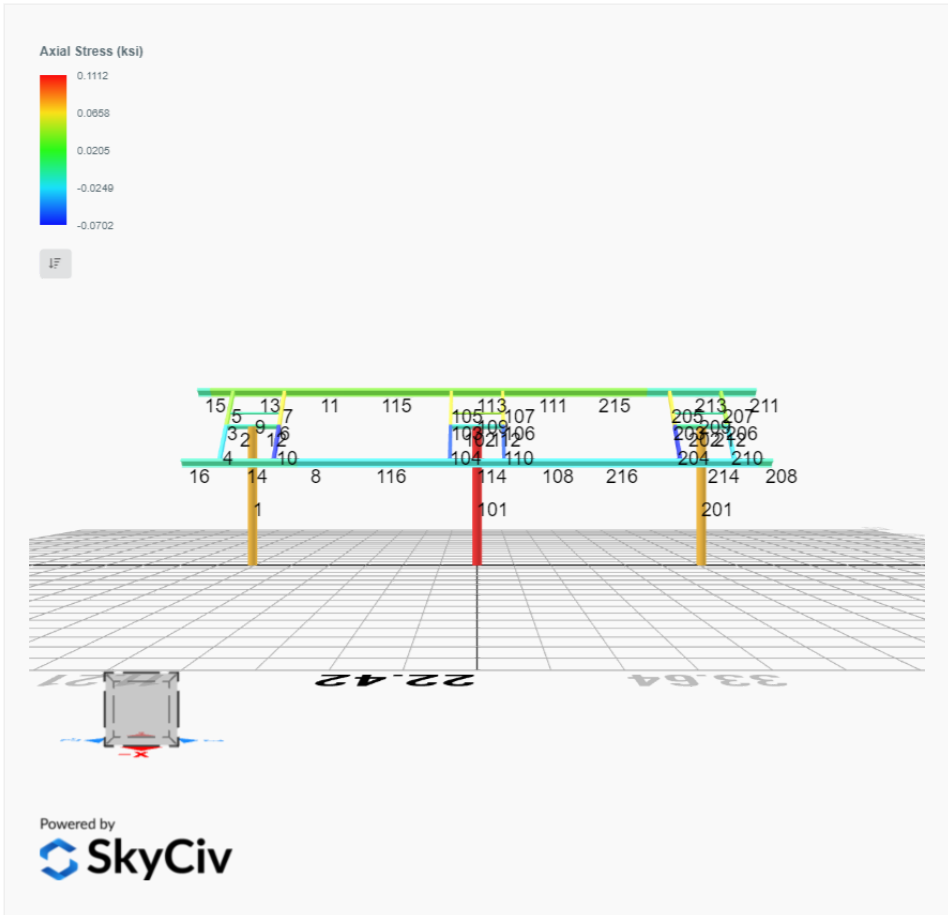


FEM Results (Envelope Worst Case for each member)









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	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 2. D + L	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 3. D + (S or Lr or R)	0.0360	2.3786	0.1045	0.3165	-0.1030	-0.3133
ULS: 3. D + (S or Lr or R)	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0327	2.2069	0.0949	0.2874	-0.0935	-0.2826
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 5b. D + 0.7E	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0327	2.2069	0.0949	0.2874	-0.0935	-0.2826
ULS: 8. 0.6D + 0.7E	0.0136	1.0150	0.0396	0.1199	-0.0390	-0.1143
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7791	3.3293	0.2251	0.6491	-0.7741	19.0866
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8223	0.0554	-0.0911	-0.2434	0.6358	-19.1722
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3187	3.4351	0.2142	0.6243	-0.6253	14.1753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0327	2.2069	0.0949	0.2874	-0.0935	-0.2826
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3824	0.9797	-0.0229	-0.0451	0.4321	-14.5188
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0327	2.2069	0.0949	0.2874	-0.0935	-0.2826
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3286	2.9199	0.1853	0.5368	-0.5968	14.2674
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3724	0.4645	-0.0518	-0.1326	0.4606	-14.4267
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0227	1.6917	0.0660	0.1999	-0.0649	-0.1905
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7882	2.6527	0.1987	0.5692	-0.7481	19.1628
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0136	1.0150	0.0396	0.1199	-0.0390	-0.1143
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8132	-0.6213	-0.1175	-0.3233	0.6618	-19.0960
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0136	1.0150	0.0396	0.1199	-0.0390	-0.1143

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
Axial	5.1037
Shear X	-3.0346
Shear Z	0.3649
Moment X	1.0509
Moment Z	32.1461

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
Axial	3.4351
Shear X	-1.8223
Shear Z	0.2251
Moment X	0.6491
Moment Z	19.1722

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	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 2. D + L	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 3. D + (S or Lr or R)	-0.0719	3.0491	0.0012	0.0054	-0.0039	0.6913
ULS: 3. D + (S or Lr or R)	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0652	2.8157	0.0011	0.0049	-0.0035	0.6294
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 5b. D + 0.7E	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0652	2.8157	0.0011	0.0049	-0.0035	0.6294

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 8. 0.6D + 0.7E	-0.0272	1.2692	0.0005	0.0020	-0.0015	0.2662
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4090	4.4045	0.0034	0.0131	-0.0102	25.0305
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3227	-0.1767	-0.0018	-0.0062	0.0052	-23.7411
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8379	4.5325	0.0031	0.0122	-0.0094	19.0695
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0652	2.8157	0.0011	0.0049	-0.0035	0.6294
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7108	1.0967	-0.0008	-0.0023	0.0022	-17.5092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0652	2.8157	0.0011	0.0049	-0.0035	0.6294
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8181	3.8322	0.0028	0.0107	-0.0083	18.8838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7307	0.3963	-0.0012	-0.0038	0.0033	-17.6949
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0454	2.1153	0.0008	0.0034	-0.0025	0.4437
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3908	3.5583	0.0031	0.0118	-0.0092	24.8531
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0272	1.2692	0.0005	0.0020	-0.0015	0.2662
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3408	-1.0228	-0.0021	-0.0075	0.0062	-23.9186
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0272	1.2692	0.0005	0.0020	-0.0015	0.2662

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
Axial	6.8189
Shear X	-4.0039
Shear Z	0.0056
Moment X	0.0214
Moment Z	41.9438

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
Axial	4.5325
Shear X	-2.4090
Shear Z	0.0034
Moment X	0.0131
Moment Z	25.0305

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 2. D + L	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 3. D + (S or Lr or R)	0.0359	2.3790	-0.1058	-0.3161	0.1090	-0.3120
ULS: 3. D + (S or Lr or R)	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0326	2.2072	-0.0960	-0.2870	0.0989	-0.2814
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 5b. D + 0.7E	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0326	2.2072	-0.0960	-0.2870	0.0989	-0.2814
ULS: 8. 0.6D + 0.7E	0.0136	1.0151	-0.0401	-0.1198	0.0412	-0.1138
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7797	3.3299	-0.2285	-0.6519	0.7938	19.0926
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8227	0.0552	0.0929	0.2467	-0.6479	-19.1764
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3192	3.4357	-0.2173	-0.6262	0.6427	14.1803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0326	2.2072	-0.0960	-0.2870	0.0989	-0.2814
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3826	0.9797	0.0237	0.0477	-0.4385	-14.5215
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0326	2.2072	-0.0960	-0.2870	0.0989	-0.2814
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3291	2.9204	-0.1881	-0.5388	0.6125	14.2721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3727	0.4644	0.0530	0.1351	-0.4687	-14.4298
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0226	1.6919	-0.0668	-0.1996	0.0687	-0.1897
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7887	2.6532	-0.2018	-0.5721	0.7663	19.1685
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0136	1.0151	-0.0401	-0.1198	0.0412	-0.1138
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8137	-0.6215	0.1196	0.3266	-0.6754	-19.1006
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0136	1.0151	-0.0401	-0.1198	0.0412	-0.1138

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
Axial	5.1047
Shear X	-3.0353
Shear Z	-0.3705
Moment X	-1.0556
Moment Z	32.1539

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
Axial	3.4357
Shear X	-1.8227
Shear Z	-0.2285
Moment X	-0.6519
Moment Z	19.1764

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
0.9	0.9	0.9	0.9

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

Section Dimensions								
ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x16	5.00	3.00	0.17	0.17	0.17		
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L	S	T
1	9	22.04	22.04	10.50	-	3	2	1
2	5	1.30	1.30	2.00	-	3	2	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.17,1.18,1.16,1.18,1.17,1.19,1.16,1.19,1.17,1.18,1.09,1.18,1.17,1.19,1.15,1.19,1.17,1.19,1.16,1.19	3	2	1
4	16	2.44	2.44	3.75	1.70,1.69,1.70,1.68,1.70,1.70,1.67,1.69,1.66,1.69,1.67,1.70,1.66,1.70,1.67,1.68,1.54,1.68,1.68,1.70,1.63,1.70,1.67,1.70,1.66,1.70	3	2	1
5	16	1.52	1.52	2.33	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.59,1.68,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	3	2	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.16,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.18,1.19	3	2	1
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.64,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	3	2	1
8	19	1.33	1.33	2.05	1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.21,1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.20,1.22,1.22,1.22,1.21,1.22,1.22,1.22,1.22,1.22	3	2	1
9	2	2.60	2.60	4.00	-	3	2	1
10	16	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.68,1.60,1.68,1.68,1.69,1.69,1.65,1.69,1.67,1.69,1.66,1.69	3	2	1
11	19	1.33	1.33	2.05	1.25,1.25,1.25,1.25,1.25,1.25,1.27,1.25,1.29,1.25,1.28,1.25,1.29,1.25,1.27,1.25,1.42,1.25,1.27,1.25,1.31,1.25,1.28,1.25,1.29,1.25	3	2	1
12	5	1.30	1.30	2.00	-	3	2	1
13	19	1.14	1.14	1.75	2.12,2.12,2.12,2.12,2.12,2.12,2.20,2.12,2.36,2.12,2.21,2.12,2.36,2.12,2.14,2.12,2.19,2.12,2.15,2.12,2.32,2.12,2.22,2.12,2.33,2.12	3	2	1
14	19	1.14	1.14	1.75	1.71,1.71	3	2	1
15	19	2.10	2.10	1.00	2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33	3	2	1
16	19	2.10	2.10	1.00	2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33	3	2	1
17	19	2.60	2.60	4.00	1.50,1.50,1.50,1.51,1.50,1.50,1.46,1.50,1.43,1.50,1.46,1.50,1.44,1.50,1.47,1.51,1.27,1.51,1.46,1.50,1.41,1.50,1.45,1.50,1.44,1.50	3	2	1
18	19	1.14	1.14	1.75	1.71,1.71	3	2	1
19	19	2.60	2.60	4.00	1.51,1.52,1.51,1.52,1.51,1.51,1.53,1.52,1.55,1.52,1.53,1.51,1.55,1.51,1.53,1.52,1.71,1.52,1.53,1.51,1.57,1.51,1.54,1.51,1.55,1.51	3	2	1
20	19	1.14	1.14	1.75	2.09,2.09,2.09,2.09,2.09,2.09,2.08,2.09,2.07,2.09,2.08,2.09,2.07,2.09,2.08,2.09,2.08,2.09,2.08,2.09,2.07,2.09,2.08,2.09,2.07,2.09	3	2	1
21	19	2.60	2.60	4.00	1.13,1.13	3	2	1
22	19	1.14	1.14	1.75	1.84,1.84,1.84,1.84,1.84,1.84,2.34,1.84,1.99,1.84,2.36,1.84,2.08,1.84,2.26,1.84,1.06,1.84,2.30,1.84,1.56,1.84,2.31,1.84,2.10,1.84	3	2	1
23	19	2.60	2.60	4.00	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.11,1.13,1.11,1.12,1.11,1.13,1.11,1.12,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11	3	2	1

19	133.20	118.19	32.87	6.12	40.24	43.62
20	133.20	126.79	32.87	6.12	40.24	43.62
21	133.20	118.19	31.22	6.12	40.24	43.62
22	133.20	126.79	32.87	6.12	40.24	43.62
23	133.20	118.19	32.87	6.12	40.24	43.62
24	133.20	126.79	32.87	6.12	40.24	43.62
25	133.20	118.19	32.87	6.12	40.24	43.62
26	133.20	126.79	32.87	6.12	40.24	43.62
27	133.20	118.19	32.87	6.12	40.24	43.62
28	133.20	126.79	32.87	6.12	40.24	43.62
29	133.20	119.56	32.87	6.12	40.24	43.62
30	133.20	107.55	29.00	6.12	40.24	43.62
31	133.20	119.56	32.16	6.12	40.24	43.62
32	133.20	126.97	32.87	6.12	40.24	43.62
101	377.97	208.92	83.29	83.29	113.39	113.39
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	126.79	32.87	6.12	40.24	43.62
114	133.20	126.79	32.87	6.12	40.24	43.62
115	133.20	119.56	31.85	6.12	40.24	43.62
116	133.20	119.56	31.85	6.12	40.24	43.62
201	377.97	208.92	83.29	83.29	113.39	113.39
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	121.82	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	121.82	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	126.79	32.87	6.12	40.24	43.62
214	133.20	126.79	32.87	6.12	40.24	43.62
215	133.20	107.55	29.54	6.12	40.24	43.62
216	133.20	126.97	32.87	6.12	40.24	43.62

Design Ratio

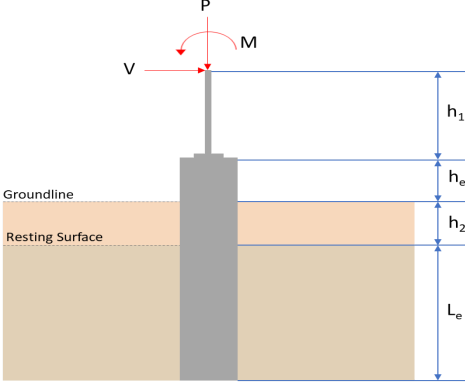
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.024	0.386	0.033	0.027	0.003	0.410	#13	0.450	Not Required	Pass
2	0.001	0.112	0.098	0.030	0.021	0.210	#13	0.053	Not Required	Pass
3	0.003	0.255	0.018	0.025	0.005	0.267	#13	0.045	Not Required	Pass
4	0.002	0.252	0.040	0.025	0.000	0.202	#13	0.080	Not Required	Pass

4	0.003	0.158	0.014	0.025	0.003	0.164	#13	0.074	Not Required	Pass
5	0.003	0.158	0.014	0.025	0.003	0.164	#13	0.074	Not Required	Pass
6	0.006	0.390	0.056	0.040	0.017	0.435	#13	0.045	Not Required	Pass
7	0.006	0.241	0.068	0.039	0.016	0.251	#13	0.074	Not Required	Pass
8	0.003	0.075	0.056	0.021	0.008	0.124	#13	0.095	Not Required	Pass
9	0.001	0.032	0.044	0.003	0.002	0.076	#13	0.204	Not Required	Pass
10	0.006	0.364	0.070	0.037	0.016	0.367	#13	0.080	Not Required	Pass
11	0.002	0.076	0.053	0.024	0.008	0.120	#13	0.095	Not Required	Pass
12	0.001	0.238	0.157	0.050	0.032	0.396	#13	0.053	Not Required	Pass
13	0.002	0.031	0.165	0.032	0.011	0.170	#23	0.081	Not Required	Pass
14	0.000	0.023	0.044	0.014	0.004	0.060	#21	Not Required	Not Required	Pass
15	0.000	0.003	0.006	0.005	0.002	0.008	#21	Not Required	Not Required	Pass
16	0.000	0.003	0.006	0.005	0.002	0.008	#21	Not Required	Not Required	Pass
17	0.001	0.041	0.041	0.012	0.004	0.058	#13	0.186	Not Required	Pass
18	0.000	0.023	0.044	0.014	0.004	0.060	#21	Not Required	Not Required	Pass
19	0.002	0.036	0.060	0.012	0.006	0.083	#13	0.186	Not Required	Pass
20	0.003	0.036	0.165	0.030	0.011	0.167	#23	0.081	Not Required	Pass
21	0.001	0.047	0.041	0.010	0.003	0.080	#21	0.186	Not Required	Pass
22	0.002	0.035	0.176	0.033	0.011	0.207	#21	0.081	Not Required	Pass
23	0.004	0.079	0.044	0.010	0.003	0.116	#13	0.186	Not Required	Pass
24	0.003	0.068	0.176	0.036	0.011	0.226	#21	0.081	Not Required	Pass
25	0.001	0.041	0.041	0.012	0.004	0.058	#13	0.186	Not Required	Pass
26	0.002	0.032	0.165	0.032	0.011	0.171	#23	0.081	Not Required	Pass
27	0.002	0.036	0.061	0.012	0.006	0.084	#13	0.186	Not Required	Pass
28	0.000	0.023	0.044	0.014	0.004	0.060	#21	Not Required	Not Required	Pass
29	0.003	0.081	0.085	0.027	0.008	0.137	#21	0.173	Not Required	Pass
30	0.003	0.100	0.090	0.021	0.008	0.165	#13	0.268	Not Required	Pass
31	0.003	0.100	0.090	0.013	0.005	0.162	#13	0.173	Not Required	Pass
32	0.002	0.093	0.086	0.014	0.005	0.147	#13	0.078	Not Required	Pass
101	0.033	0.504	0.000	0.035	0.000	0.520	#13	0.450	Not Required	Pass
102	0.001	0.238	0.175	0.054	0.034	0.413	#13	0.053	Not Required	Pass
103	0.006	0.416	0.041	0.042	0.010	0.445	#13	0.045	Not Required	Pass
104	0.006	0.441	0.073	0.044	0.016	0.485	#13	0.080	Not Required	Pass
105	0.006	0.257	0.074	0.042	0.018	0.272	#13	0.074	Not Required	Pass
106	0.006	0.418	0.041	0.042	0.010	0.447	#13	0.045	Not Required	Pass
107	0.006	0.258	0.074	0.042	0.018	0.274	#13	0.074	Not Required	Pass
108	0.003	0.054	0.056	0.027	0.008	0.077	#21	0.095	Not Required	Pass
109	0.004	0.026	0.032	0.001	0.000	0.060	#13	0.204	Not Required	Pass
110	0.006	0.443	0.072	0.045	0.016	0.486	#13	0.080	Not Required	Pass
111	0.002	0.074	0.057	0.024	0.008	0.093	#13	0.095	Not Required	Pass
112	0.001	0.239	0.176	0.054	0.034	0.416	#13	0.053	Not Required	Pass
113	0.002	0.035	0.176	0.033	0.011	0.208	#21	0.081	Not Required	Pass
114	0.003	0.067	0.176	0.036	0.011	0.225	#21	0.081	Not Required	Pass
115	0.003	0.094	0.086	0.024	0.008	0.148	#13	0.173	Not Required	Pass
116	0.003	0.091	0.090	0.010	0.005	0.157	#13	0.173	Not Required	Pass
201	0.024	0.386	0.034	0.027	0.003	0.410	#13	0.450	Not Required	Pass
202	0.001	0.239	0.158	0.050	0.032	0.398	#13	0.053	Not Required	Pass
203	0.006	0.391	0.056	0.041	0.017	0.436	#13	0.045	Not Required	Pass
204	0.006	0.365	0.070	0.037	0.016	0.367	#13	0.080	Not Required	Pass
205	0.006	0.242	0.068	0.039	0.016	0.252	#13	0.074	Not Required	Pass
206	0.003	0.254	0.018	0.025	0.005	0.265	#13	0.045	Not Required	Pass
207	0.003	0.157	0.014	0.025	0.003	0.163	#13	0.074	Not Required	Pass
208	0.000	0.003	0.006	0.005	0.002	0.008	#21	Not Required	Not Required	Pass
209	0.001	0.032	0.045	0.003	0.002	0.077	#13	0.204	Not Required	Pass

210	0.003	0.250	0.041	0.025	0.009	0.292	#13	0.080	Not Required	Pass
211	0.000	0.003	0.006	0.005	0.002	0.008	#21	Not Required	Not Required	Pass
212	0.001	0.110	0.098	0.030	0.021	0.208	#13	0.053	Not Required	Pass
213	0.000	0.023	0.044	0.014	0.004	0.060	#21	Not Required	Not Required	Pass
214	0.003	0.035	0.165	0.030	0.011	0.167	#23	0.081	Not Required	Pass
215	0.003	0.107	0.090	0.024	0.008	0.168	#13	0.268	Not Required	Pass
216	0.003	0.080	0.085	0.017	0.005	0.136	#21	0.078	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
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>3.435</td> <td>5.104</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.822</td> <td>-3.035</td> </tr> <tr> <td>V_z (kip)</td> <td>0.225</td> <td>0.365</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.649</td> <td>1.051</td> </tr> <tr> <td>M_z (kipft)</td> <td>19.172</td> <td>32.146</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	3.435	5.104	V_x (kip)	-1.822	-3.035	V_z (kip)	0.225	0.365	M_x (kipft)	0.649	1.051	M_z (kipft)	19.172	32.146	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	19.172	32.146																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.822 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.29013 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(19.172 \text{ kipft}) + ((-1.822 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0529 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3303 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.225 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.035828 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.649 \text{ kipft}) + ((0.225 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10334 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.3734 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.3303 \text{ ft}), (2.3734 \text{ ft})]$$

$$L_{e,req} = 5.33 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.33 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92696$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(3.435 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.21469 \text{ kip/ft}^2$$

$$q = 0.21469 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.21469 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.10734$$

Status: **PASS**
Ratio: **0.110**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.29013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.0529 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0529 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.29013 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.0529 \text{ kipft/ft})) + (4 \times (-0.29013 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9613 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0529 \text{ kipft/ft})) + (3 \times (-0.29013 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.0529 \text{ kipft/ft})) + (2 \times (-0.29013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20236 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.0529 \text{ kipft/ft})) + ((-0.29013 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.80529 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9613 \text{ ft})}{2}$$

$$p_a = 0.2971 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20236 \text{ kip/ft}^2)}{(0.2971 \text{ kip/ft}^2)}$$

$$Ratio = 0.68113$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.680**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.80529 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93367$$

Status: **PASS**
Ratio: **0.930**

Considering z-direction:

$H_o = 0.035828 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10334 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10334 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.035828 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.10334 \text{ kipft/ft})) + (4 \times (0.035828 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1068 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.10334 \text{ kipft/ft})) + (3 \times (0.035828 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.10334 \text{ kipft/ft})) + (2 \times (0.035828 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.033421 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.10334 \text{ kipft/ft})) + ((0.035828 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.074894 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1068 \text{ ft})}{2}$$

$$p_a = 0.30801 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.033421 \text{ kip/ft}^2)}{(0.30801 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.10851$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

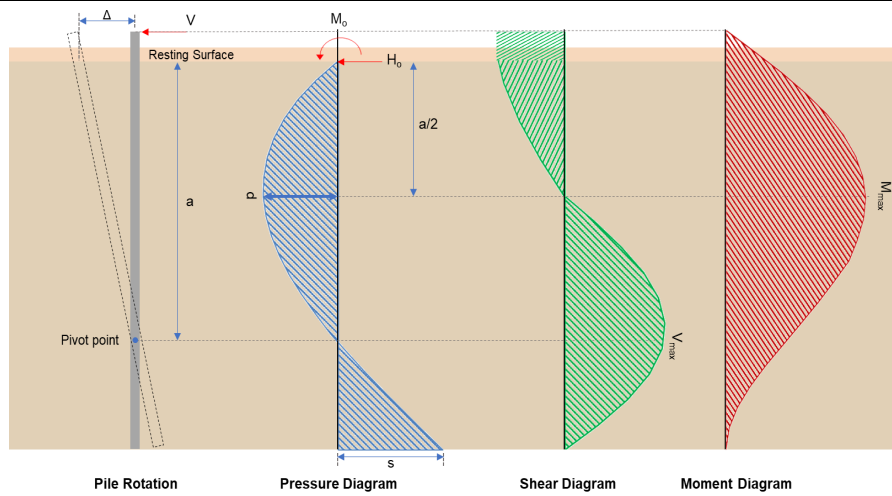
$$\text{Ratio} = \frac{s}{p_s}$$

Status: **PASS**
Ratio: **0.110**

$$Ratio = \frac{(0.074894 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.086834$$

Status: **PASS**
Ratio: **0.090**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.035 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.48328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.146 \text{ kipft}) + ((-3.035 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1188 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1188 \text{ kipft/ft})}{(-0.48328 \text{ kip/ft})}$$

$$E = 10.592 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1188 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.48328 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.1188 \text{ kipft/ft})) + (4 \times (-0.48328 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9607 \text{ ft}$$

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

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

$$V_{max} = ((-0.48328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.592 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.592 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.5765 \text{ kip}$$

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

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

$$M_{max} = ((-0.48328 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(10.592 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.592 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.592 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.772 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.365 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.058121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.051 \text{ kipft}) + ((0.365 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16736 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16736 \text{ kipft/ft})}{(0.058121 \text{ kip/ft})}$$

$$E = 2.8795 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16736 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.058121 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.16736 \text{ kipft/ft})) + (4 \times (0.058121 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.107 \text{ ft}$$

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

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

$$V_{max} = ((0.058121 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8795 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.107 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.8795 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.107 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.3609 \text{ kip}$$

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

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

$$M_{max} = ((0.058121 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.8795 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.107 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.8795 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.107 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.8795 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.107 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91836 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(5.104 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.1 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.1 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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 No. 10: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(5.104 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0016033$$

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

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 5.104 \text{ kip} \rightarrow 5104 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(5104 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.47 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

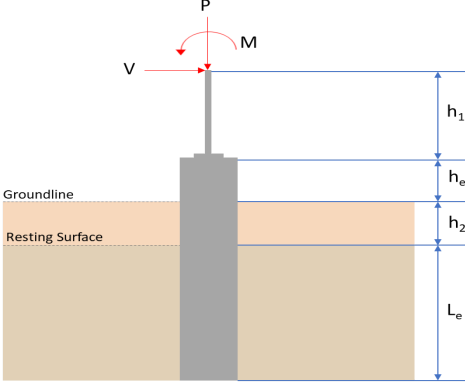
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.47 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.47 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.47 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 117.89 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.5765 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.5765 \text{ kip})}{(117.89 \text{ kip})}$ $\text{Ratio} = 0.064268$ <p>Considering z-direction:</p> <p>$V_{max} = 0.3609 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.3609 \text{ kip})}{(117.89 \text{ kip})}$ $\text{Ratio} = 0.0030614$	<p>Status: PASS Ratio: 0.060</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 20.772\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(20.772\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.075971$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 0.91836\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.91836\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0033587$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>3.436</td> <td>5.105</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.823</td> <td>-3.035</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.229</td> <td>-0.370</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.652</td> <td>-1.056</td> </tr> <tr> <td>M_z (kipft)</td> <td>19.176</td> <td>32.154</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	3.436	5.105	V_x (kip)	-1.823	-3.035	V_z (kip)	-0.229	-0.370	M_x (kipft)	-0.652	-1.056	M_z (kipft)	19.176	32.154	
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M_z (kipft)	19.176	32.154																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.823 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.29029 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(19.176 \text{ kipft}) + ((-1.823 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0535 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3303 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.229 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.036465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.652 \text{ kipft}) + ((-0.229 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10382 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6695 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.3303 \text{ ft}), (1.6695 \text{ ft})]$$

$$L_{e,req} = 5.33 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.33 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92696$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(3.436 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.21475 \text{ kip/ft}^2$$

$$q = 0.21475 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.21475 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.10737$$

Status: **PASS**
Ratio: **0.110**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.29029 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.0535 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0535 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.29029 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.0535 \text{ kipft/ft})) + (4 \times (-0.29029 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9613 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0535 \text{ kipft/ft})) + (3 \times (-0.29029 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.0535 \text{ kipft/ft})) + (2 \times (-0.29029 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20235 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.0535 \text{ kipft/ft})) + ((-0.29029 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.80536 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9613 \text{ ft})}{2}$$

$$p_a = 0.2971 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20235 \text{ kip/ft}^2)}{(0.2971 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.68107$$

p_a - Allowable lateral soil pressure at depth L_e ,

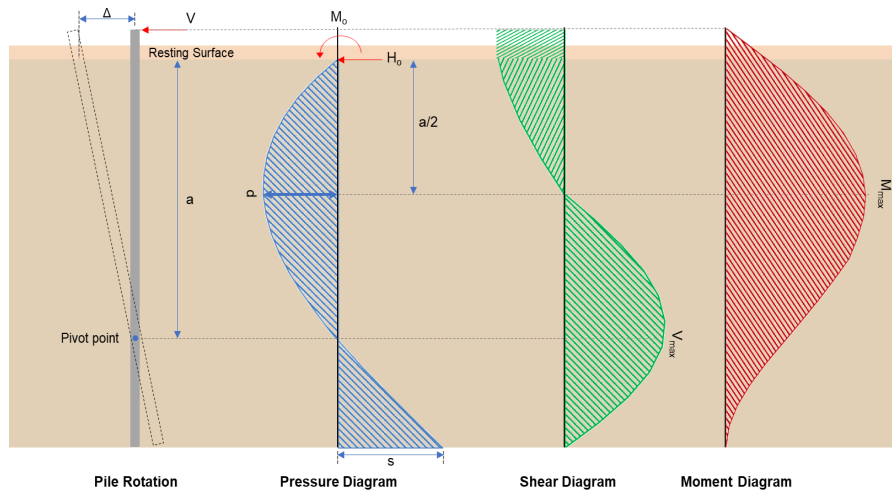
Status: **PASS**
Ratio: **0.680**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.80536 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93375$	Status: PASS Ratio: 0.930
	<p>Considering z-direction:</p> <p>$H_o = -0.036465 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.10382 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.10382 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.036465 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.10382 \text{ kipft/ft})) + (4 \times (-0.036465 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1083 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.10382 \text{ kipft/ft})) + (3 \times (-0.036465 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.10382 \text{ kipft/ft})) + (2 \times (-0.036465 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = -0.0096056 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.10382 \text{ kipft/ft})) + ((-0.036465 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = -0.00036844 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1083 \text{ ft})}{2}$ $p_a = 0.30812 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0096056 \text{ kip/ft}^2)}{(0.30812 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.031175$ <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: -0.030

$$\text{Ratio} = \frac{(-0.00036844 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.00042718$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.035 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.48328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.154 \text{ kipft}) + ((-3.035 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1201 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1201 \text{ kipft/ft})}{(-0.48328 \text{ kip/ft})}$$

$$E = 10.594 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1201 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.48328 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.1201 \text{ kipft/ft})) + (4 \times (-0.48328 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9606 \text{ ft}$$

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

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

$$V_{max} = ((-0.48328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.594 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9606 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.594 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9606 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.5781 \text{ kip}$$

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

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

$$M_{max} = ((-0.48328 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(10.594 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9606 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.594 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9606 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.594 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9606 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.777 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.37 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.058917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.056 \text{ kipft}) + ((-0.37 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16815 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16815 \text{ kipft/ft})}{(-0.058917 \text{ kip/ft})}$$

$$E = 2.8541 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16815 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.058917 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.16815 \text{ kipft/ft})) + (4 \times (-0.058917 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.108 \text{ ft}$$

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

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

$$V_{max} = ((-0.058917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8541 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.108 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.8541 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.108 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.36403 \text{ kip}$$

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

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

$$M_{max} = ((-0.058917 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.8541 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.108 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.8541 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.108 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.8541 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.108 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.92572 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(5.105 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.1 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.1 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(5.105 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0016036$$

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

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 5.105 \text{ kip} \rightarrow 5105 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(5105 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.47 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

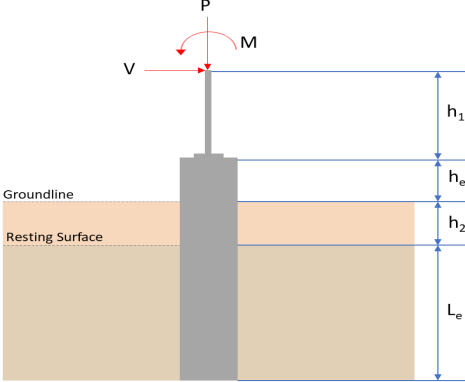
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.47 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.47 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.47 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 117.89 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.5781 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.5781 \text{ kip})}{(117.89 \text{ kip})}$ $\text{Ratio} = 0.064281$ <p>Considering z-direction:</p> <p>$V_{max} = 0.36403 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.36403 \text{ kip})}{(117.89 \text{ kip})}$ $\text{Ratio} = 0.0030878$	<p>Status: PASS Ratio: 0.060</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 20.777\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(20.777\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.075988$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 0.92572\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.92572\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0033857$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.533</td> <td>6.819</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.409</td> <td>-4.004</td> </tr> <tr> <td>V_z (kip)</td> <td>0.003</td> <td>0.006</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.013</td> <td>0.021</td> </tr> <tr> <td>M_z (kipft)</td> <td>25.031</td> <td>41.944</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.533	6.819	V_x (kip)	-2.409	-4.004	V_z (kip)	0.003	0.006	M_x (kipft)	0.013	0.021	M_z (kipft)	25.031	41.944	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.409 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.3836 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.031 \text{ kipft}) + ((-2.409 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.9858 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.7205 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.003 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.013 \text{ kipft}) + ((0.003 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0020701 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.56662 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[5.7205 \text{ ft}, 0.56662 \text{ ft}]$$

$$L_{e,req} = 5.721 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.721 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.91536$$

Status: **PASS**
Ratio: **0.920**

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

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

$$q = \frac{(4.533 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28331 \text{ kip/ft}^2$$

$$q = 0.28331 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28331 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14166$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.3836 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.9858 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9858 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.3836 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.9858 \text{ kipft/ft})) + (4 \times (-0.3836 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3157 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.9858 \text{ kipft/ft})) + (3 \times (-0.3836 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (3.9858 \text{ kipft/ft})) + (2 \times (-0.3836 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20527 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (3.9858 \text{ kipft/ft})) + ((-0.3836 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85619 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3157 \text{ ft})}{2}$$

$$p_a = 0.32368 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20527 \text{ kip/ft}^2)}{(0.32368 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63419$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.630**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.85619 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91327$$

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = 0.00047771 \text{ kip/ft}$ - Lateral force per length of pile.

$M_o = 0.0020701 \text{ kipft/ft}$ - Overturning moment per length of pile.

a - Distance from resting surface to pivot point.

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

$$a = \frac{(4 \times (0.0020701 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.00047771 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0020701 \text{ kipft/ft})) + (4 \times (0.00047771 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.422 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0020701 \text{ kipft/ft})) + (3 \times (0.00047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0020701 \text{ kipft/ft})) + (2 \times (0.00047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.00046831 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e .

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

$$s = \frac{6 \times [(2 \times (0.0020701 \text{ kipft/ft})) + ((0.00047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0010945 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.422 \text{ ft})}{2}$$

$$p_a = 0.33165 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00046831 \text{ kip/ft}^2)}{(0.33165 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0014121$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

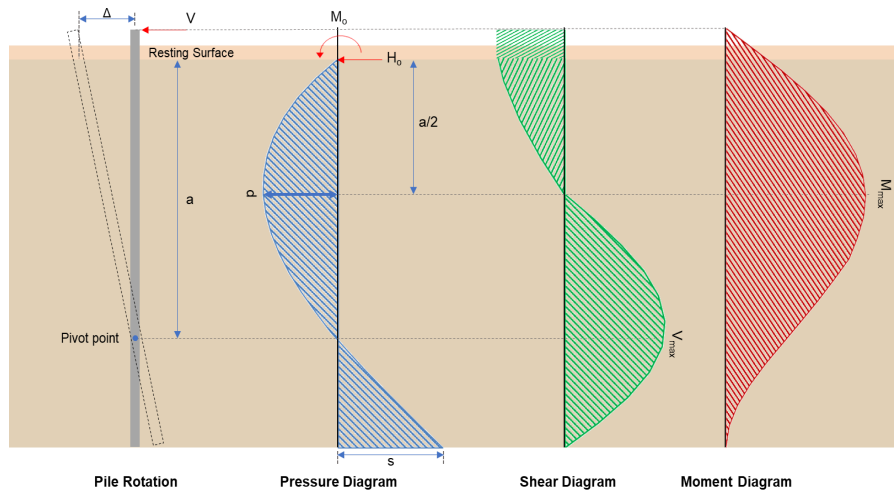
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0010945 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0011675$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.004 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.63758 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.944 \text{ kipft}) + ((-4.004 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.679 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.679 \text{ kipft/ft})}{(-0.63758 \text{ kip/ft})}$$

$$E = 10.476 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.679 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.63758 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.679 \text{ kipft/ft})) + (4 \times (-0.63758 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3149 \text{ ft}$$

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

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

$$V_{max} = ((-0.63758 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2457 \text{ kip}$$

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

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

$$M_{max} = ((-0.63758 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(10.476 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.446 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.006 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00095541 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.021 \text{ kipft}) + ((0.006 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0033439 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0033439 \text{ kipft/ft})}{(0.00095541 \text{ kip/ft})}$$

$$E = 3.5 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0033439 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.00095541 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0033439 \text{ kipft/ft})) + (4 \times (0.00095541 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4497 \text{ ft}$$

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

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

$$V_{max} = ((0.00095541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4497 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.5 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4497 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0063289 \text{ kip}$$

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

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

$$M_{max} = ((0.00095541 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.5 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4497 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.5 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4497 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4497 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.017644 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - 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,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.819 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.04 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.04 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

Status: **PASS**
Ratio: **0.970**

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

Ties:

25.7.2.2 Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.819 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.002142$$

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

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1 The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.819 \text{ kip} \rightarrow 6819 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(6819 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.7 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.7 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.7 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.7 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.04 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.2457 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.2457 \text{ kip})}{(118.04 \text{ kip})}$ $\text{Ratio} = 0.078328$ <p>Considering z-direction:</p> <p>$V_{max} = 0.0063289 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.0063289 \text{ kip})}{(118.04 \text{ kip})}$ $\text{Ratio} = 0.000053617$	<p>Status: PASS Ratio: 0.080</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 27.446\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(27.446\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.10038$	<p>Status: PASS Ratio: 0.100</p>
	<p>Considering z-direction: $M_{max} = 0.017644\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.017644\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00006453$	<p>Status: PASS Ratio: 0.000</p>