

Your Project Calculations



Project Name: DaleHowell-RevA

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=DaleHowell-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=2k2LAXUjw7PmRwvaK8IX162Tt7mU44wcWdfmogjkTMXrGHCMZ0jYtHf5nmW0wYT

Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-XD-24-L-4Hx8W-4CD3
Duty Classification:	XD
Module Width:	41.14 in
Module Length:	76.37in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Desired Tilt Angle:	15
Front Edge Clearance:	6
Total Array Height at Tilt:	9.57 ft
Total Frame Length:	51.00 ft
Frame Weight:	2710 lbs
Array Dimensions N/S:	13.88 ft
Array Dimensions E/W:	51.58 ft
Rail Length:	166.56 in
Rail Spacing:	3.18 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	7.80 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft Pile 2: 6.75 ft Pile 3: 6.50 ft
Foundation Volume:	11.704 y ³
Foundation Result:	PASSED
Mount Twist:	0.499756 kip

Site Info

Risk Category:	I
Exposure:	D
Soil Classification:	sand
Site Location:	3179 Vista Diego Rd, Jamul, CA 91935, USA
Wind Speed:	90 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

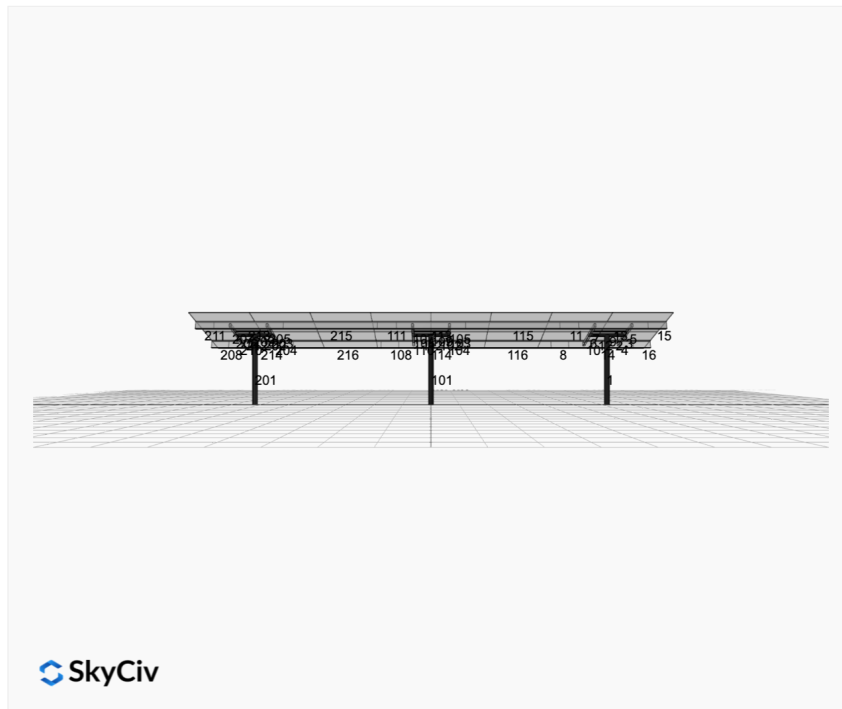
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

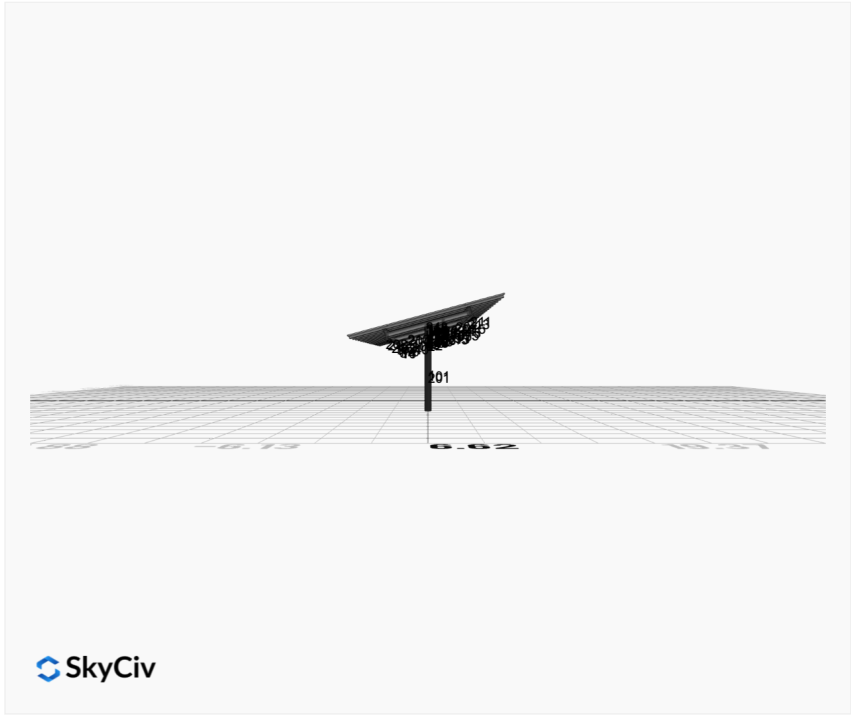
AutoDesigner Input

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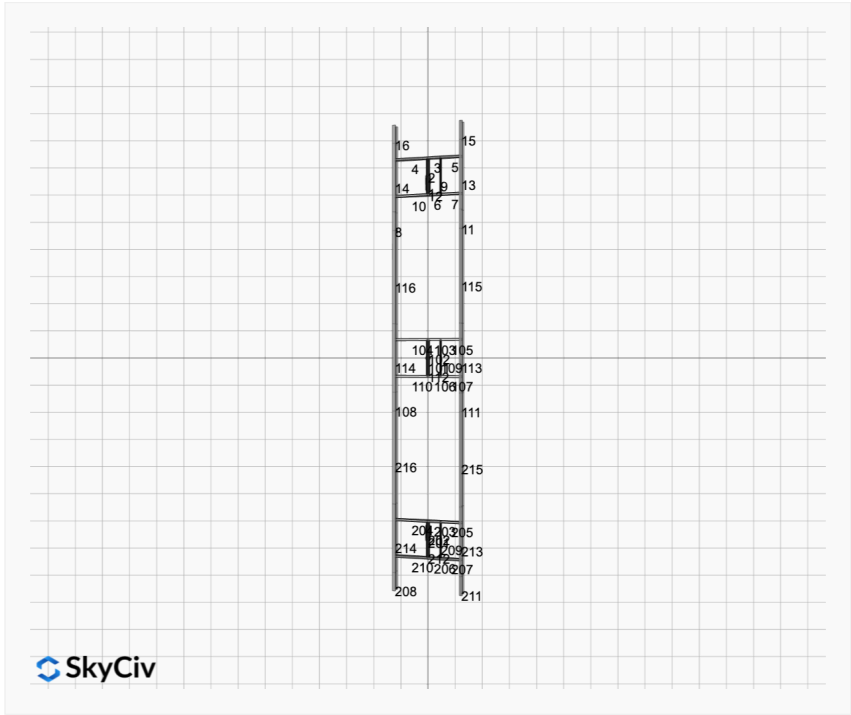
Design Notes:

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

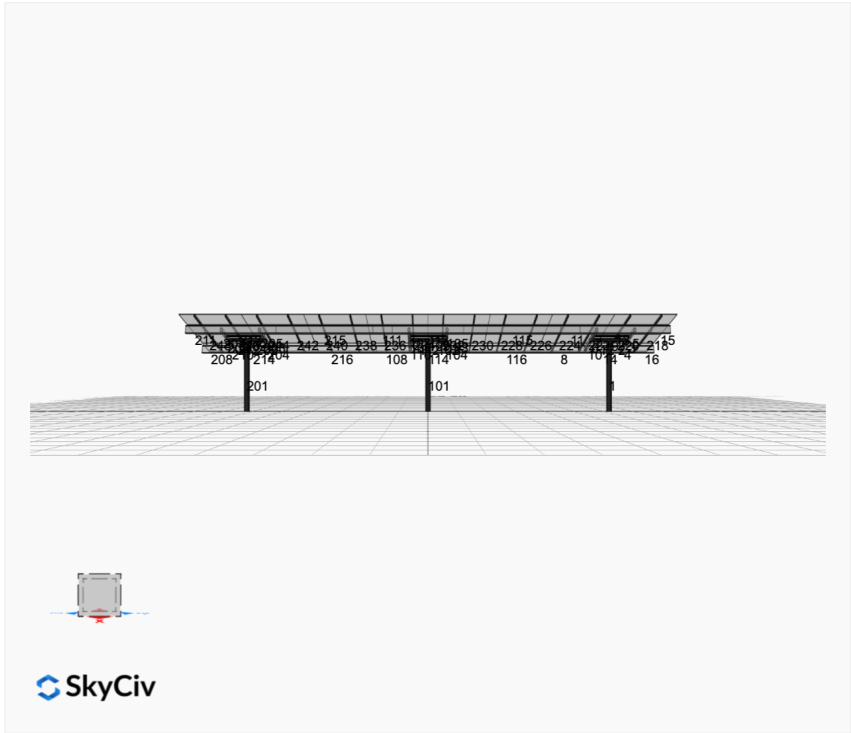
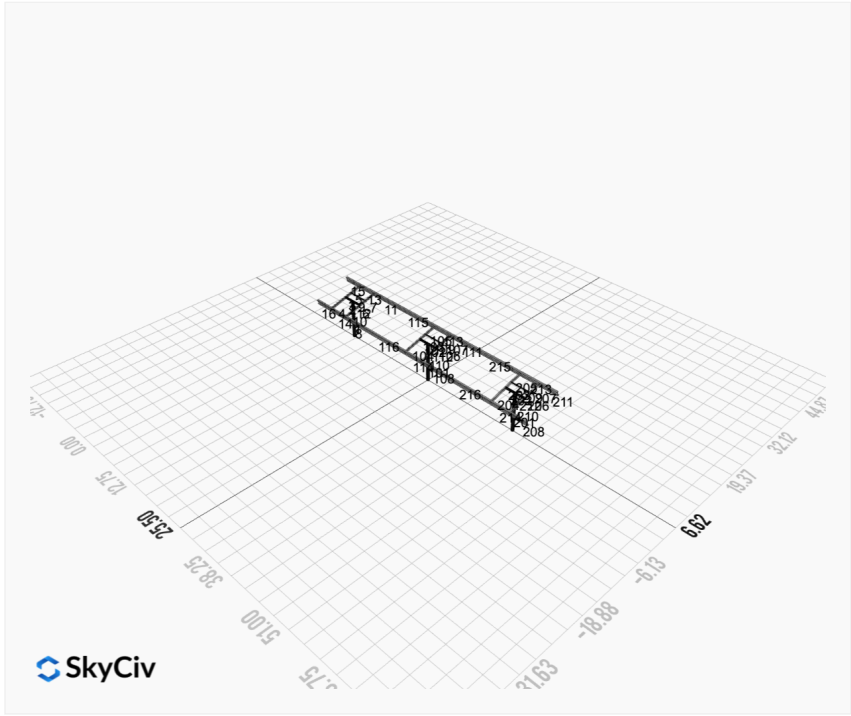




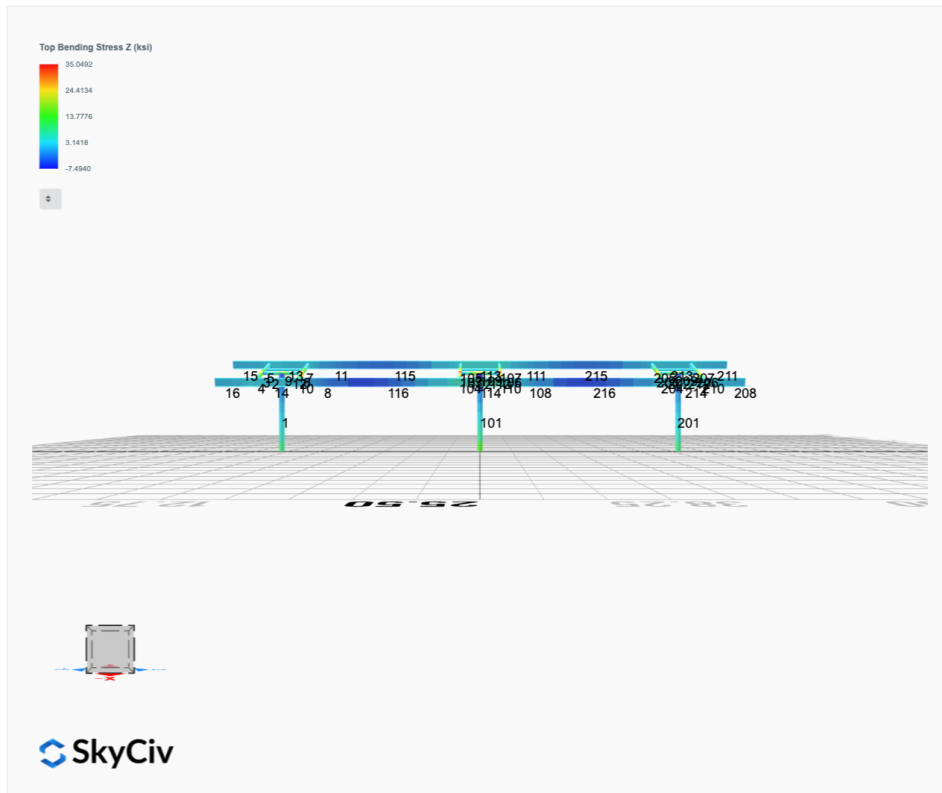
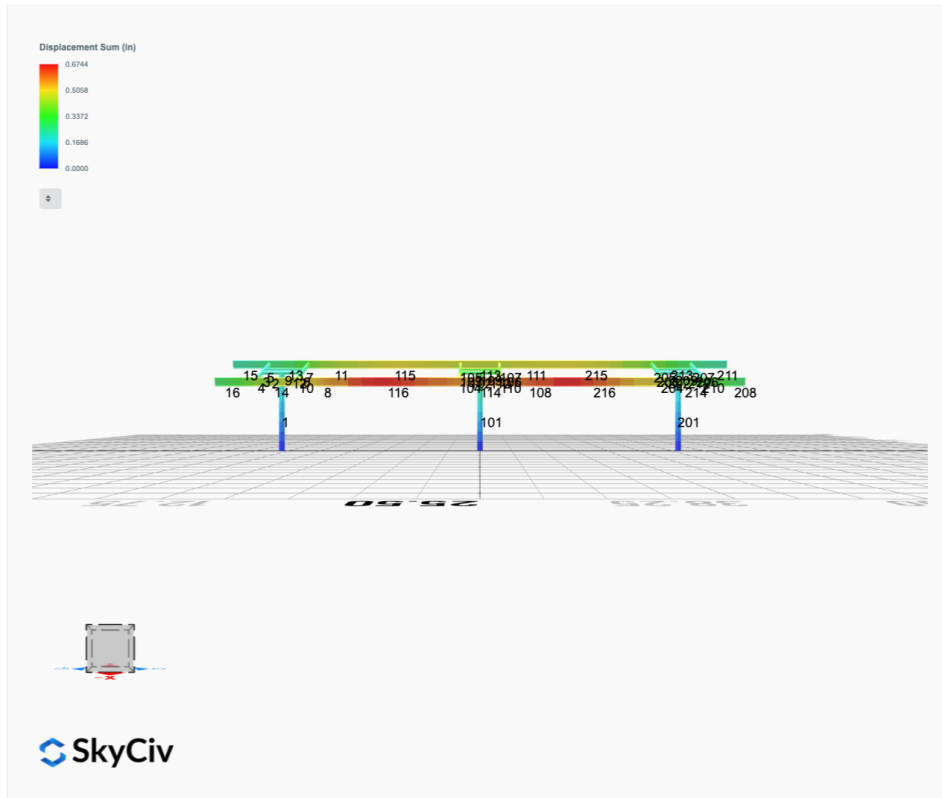
SkyCiv

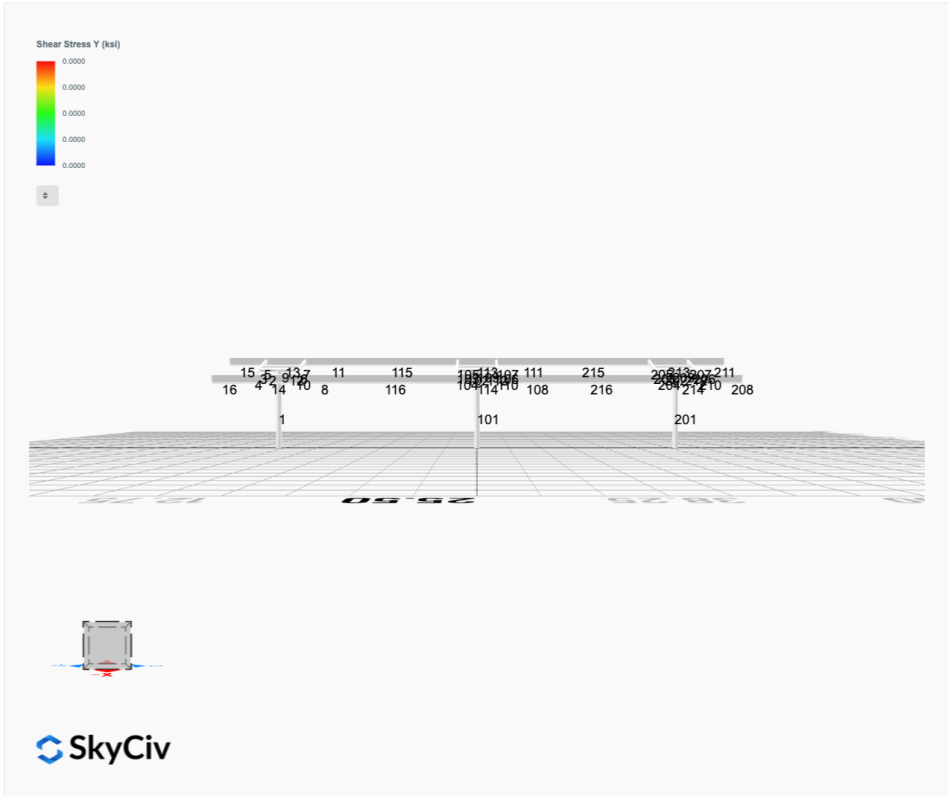
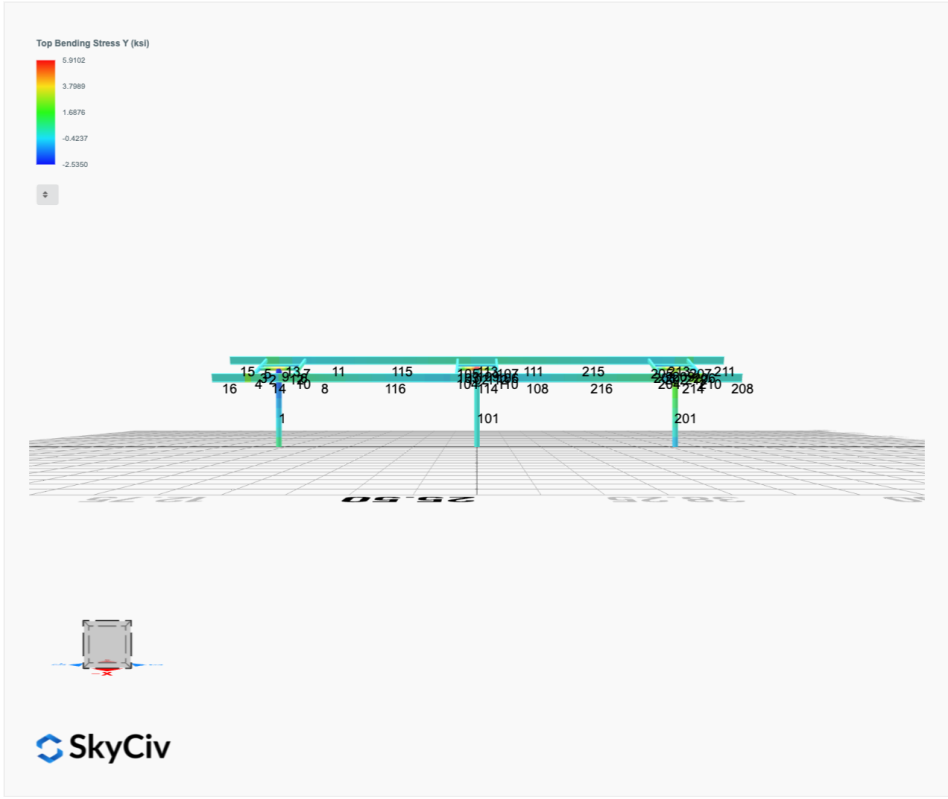


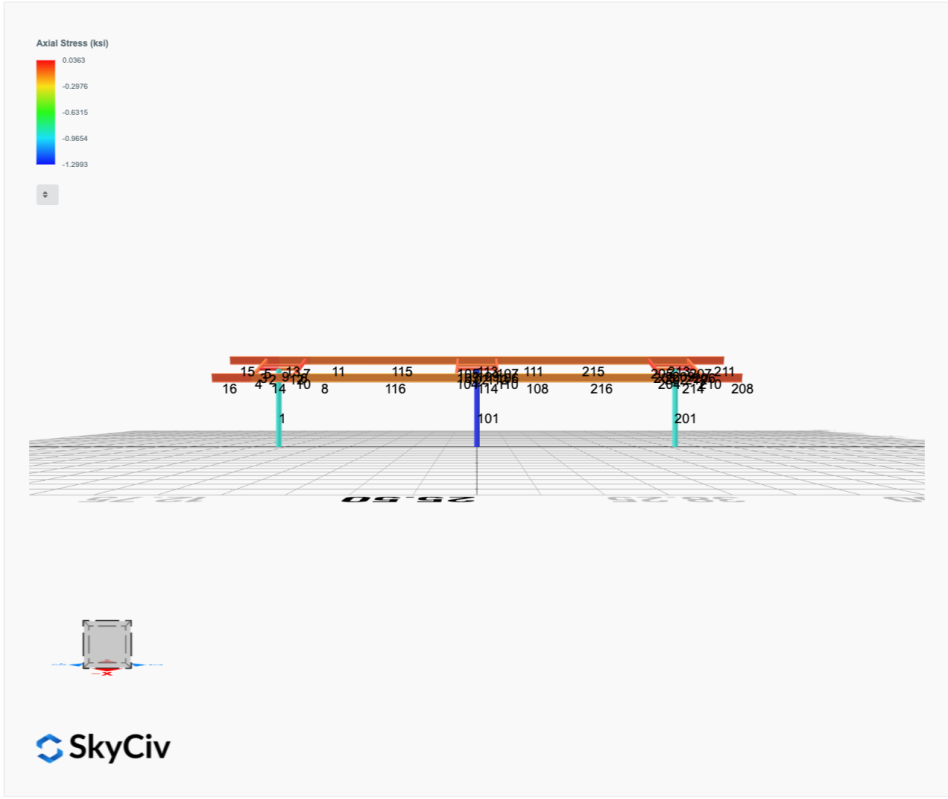
SkyCiv



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	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 2. D + L	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 3. D + (S or Lr or R)	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 3. D + (S or Lr or R)	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 5b. D + 0.7E	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0166	1.8259	0.0767	0.1615	-0.0167	-0.0779
ULS: 8. 0.6D + 0.7E	0.0100	1.0955	0.0460	0.0969	-0.0100	-0.0467
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6182	7.8236	0.4541	0.9347	-0.3039	15.9328
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6182	7.8236	0.4541	0.9347	-0.3039	15.9328
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2427	-2.7115	-0.1921	-0.3883	0.1828	-6.3270
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1185	-2.1304	-0.2004	-0.4030	0.2073	-24.4729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2095	6.3242	0.3597	0.7414	-0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2095	6.3242	0.3597	0.7414	-0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9361	-1.5772	-0.1249	-0.2509	0.1330	-4.7647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8430	-1.1413	-0.1311	-0.2619	0.1513	-18.3741
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2095	6.3242	0.3597	0.7414	-0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2095	6.3242	0.3597	0.7414	-0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9361	-1.5772	-0.1249	-0.2509	0.1330	-4.7647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8430	-1.1413	-0.1311	-0.2619	0.1513	-18.3741
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6248	7.0933	0.4234	0.8701	-0.2972	15.9640
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6248	7.0933	0.4234	0.8701	-0.2972	15.9640
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2360	-3.4419	-0.2227	-0.4530	0.1895	-6.2959
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1119	-2.8607	-0.2310	-0.4676	0.2140	-24.4417

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.1878
Shear X	-2.7246
Shear Z	0.7226
Moment X	1.4864
Moment Y (Twist)	0.4996
Moment Z	41.0648

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.8236
Shear X	-1.6248
Shear Z	0.4541
Moment X	0.9347
Moment Y (Twist)	0.3039
Moment Z	24.4729

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	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 2. D + L	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 3. D + (S or Lr or R)	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 3. D + (S or Lr or R)	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 5b. D + 0.7E	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0332	2.4016	-0.0000	0.0000	0.0000	0.2518
ULS: 8. 0.6D + 0.7E	-0.0199	1.4409	-0.0000	0.0000	0.0000	0.1511
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2832	11.0050	0.0000	-0.0000	0.0000	21.2031
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.2832	11.0050	0.0000	-0.0000	0.0000	21.2031
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7019	-4.1505	-0.0000	0.0000	0.0000	-8.5144
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.3792	-3.1819	-0.0000	0.0000	0.0000	-29.5732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7207	8.8542	0.0000	-0.0000	0.0000	15.9653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.7207	8.8542	0.0000	-0.0000	0.0000	15.9653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2681	-2.5125	-0.0000	0.0000	0.0000	-6.3228
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0261	-1.7860	-0.0000	0.0000	0.0000	-22.1169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7207	8.8542	0.0000	-0.0000	0.0000	15.9653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.7207	8.8542	0.0000	-0.0000	0.0000	15.9653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2681	-2.5125	-0.0000	0.0000	0.0000	-6.3228
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0261	-1.7860	-0.0000	0.0000	0.0000	-22.1169
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2699	10.0444	0.0000	-0.0000	0.0000	21.1024
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.2699	10.0444	0.0000	-0.0000	0.0000	21.1024
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7152	-5.1111	0.0000	-0.0000	0.0000	-8.6151
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3925	-4.1425	-0.0000	0.0000	0.0000	-29.6739

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	17.2202
Shear X	-3.7886
Shear Z	0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	49.8481

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	11.0050
Shear X	-2.2832
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	29.6739

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	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 2. D + L	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 3. D + (S or Lr or R)	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 3. D + (S or Lr or R)	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 5b. D + 0.7E	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0166	1.8259	-0.0767	-0.1615	0.0167	-0.0779
ULS: 8. 0.6D + 0.7E	0.0100	1.0955	-0.0460	-0.0969	0.0100	-0.0467
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6182	7.8236	-0.4541	-0.9347	0.3039	15.9328
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6182	7.8236	-0.4541	-0.9347	0.3039	15.9328
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2427	-2.7115	0.1921	0.3883	-0.1828	-6.3270
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1185	-2.1304	0.2004	0.4030	-0.2073	-24.4729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2095	6.3242	-0.3597	-0.7414	0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2095	6.3242	-0.3597	-0.7414	0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9361	-1.5772	0.1249	0.2509	-0.1329	-4.7647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8430	-1.1413	0.1311	0.2619	-0.1513	-18.3741

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 downforce Case A only	-1.2095	6.3242	-0.3597	-0.7414	0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2095	6.3242	-0.3597	-0.7414	0.2321	11.9302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9361	-1.5772	0.1249	0.2509	-0.1329	-4.7647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8430	-1.1413	0.1311	0.2619	-0.1513	-18.3741
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6248	7.0933	-0.4234	-0.8701	0.2972	15.9640
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6248	7.0933	-0.4234	-0.8701	0.2972	15.9640
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2360	-3.4419	0.2227	0.4530	-0.1895	-6.2959
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1119	-2.8607	0.2310	0.4676	-0.2140	-24.4417

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.1878
Shear X	-2.7246
Shear Z	-0.7226
Moment X	-1.4863
Moment Y (Twist)	0.4998
Moment Z	41.0656

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.8236
Shear X	-1.6248
Shear Z	-0.4541
Moment X	-0.9347
Moment Y (Twist)	0.3039
Moment Z	24.4729

Project Details

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



User Name: sales@mtsolar.us
 Project Name: DaleHowell-RevA
 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)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
8	6in Pipe Sch 80	6.63	0.43				

ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	

ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24

Member Design Capacity

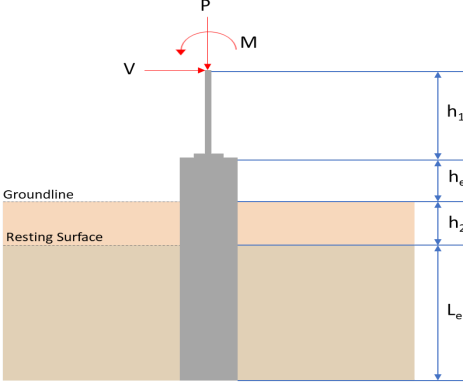
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	210.54	62.23	62.23	113.47	113.47
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	229.64	27.16	27.16	75.30	75.30
13	159.30	116.35	36.98	6.46	56.26	44.91
14	159.30	116.35	37.59	6.46	56.26	44.91
15	159.30	113.66	46.90	6.46	56.26	44.91
16	159.30	113.66	46.90	6.46	56.26	44.91
101	378.22	210.54	62.23	62.23	113.47	113.47
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.48	6.46	56.26	44.91
114	159.30	116.35	30.87	6.46	56.26	44.91
115	159.30	75.13	21.18	6.46	56.26	44.91
116	159.30	75.13	20.41	6.46	56.26	44.91
201	378.22	210.54	62.23	62.23	113.47	113.47
202	251.01	229.64	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	113.66	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	113.66	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	36.98	6.46	56.26	44.91
214	159.30	116.35	37.59	6.46	56.26	44.91
215	159.30	75.13	20.80	6.46	56.26	44.91
216	159.30	75.13	20.99	6.46	56.26	44.91

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.058	0.660	0.066	0.024	0.006	0.678	#16	0.448	Not Required	Pass
2	0.001	0.300	0.081	0.068	0.016	0.382	#13	0.054	Not Required	Pass
3	0.001	0.537	0.016	0.053	0.003	0.554	#13	0.046	Not Required	Pass
4	0.001	0.440	0.039	0.044	0.008	0.479	#13	0.122	Not Required	Pass
5	0.001	0.333	0.012	0.053	0.004	0.345	#13	0.076	Not Required	Pass
6	0.002	0.698	0.030	0.071	0.007	0.729	#13	0.046	Not Required	Pass
7	0.002	0.433	0.024	0.069	0.006	0.447	#13	0.076	Not Required	Pass
8	0.003	0.112	0.050	0.033	0.003	0.138	#13	0.102	Not Required	Pass
9	0.002	0.077	0.037	0.003	0.002	0.115	#13	0.206	Not Required	Pass
10	0.002	0.558	0.042	0.056	0.010	0.589	#13	0.082	Not Required	Pass
11	0.002	0.133	0.045	0.045	0.003	0.152	#13	0.102	Not Required	Pass
12	0.002	0.453	0.099	0.090	0.020	0.552	#13	0.174	Not Required	Pass
13	0.003	0.113	0.080	0.058	0.003	0.165	#16	0.306	Not Required	Pass
14	0.004	0.089	0.085	0.044	0.004	0.130	#13	0.204	Not Required	Pass
15	0.000	0.019	0.005	0.016	0.001	0.023	#13	Not Required	Not Required	Pass
16	0.000	0.016	0.005	0.013	0.001	0.020	#13	Not Required	Not Required	Pass
101	0.082	0.801	0.000	0.033	0.000	0.810	#32	0.448	Not Required	Pass
102	0.001	0.543	0.130	0.113	0.025	0.673	#13	0.054	Not Required	Pass
103	0.001	0.851	0.009	0.085	0.003	0.857	#13	0.046	Not Required	Pass
104	0.001	0.744	0.027	0.074	0.006	0.761	#13	0.082	Not Required	Pass
105	0.001	0.528	0.027	0.085	0.007	0.536	#13	0.076	Not Required	Pass
106	0.001	0.851	0.009	0.085	0.003	0.857	#13	0.046	Not Required	Pass
107	0.001	0.528	0.027	0.085	0.007	0.536	#13	0.076	Not Required	Pass
108	0.003	0.051	0.041	0.046	0.002	0.093	#13	0.102	Not Required	Pass
109	0.003	0.079	0.022	0.001	0.000	0.103	#13	0.206	Not Required	Pass
110	0.001	0.744	0.027	0.074	0.006	0.761	#13	0.082	Not Required	Pass
111	0.002	0.116	0.039	0.052	0.002	0.141	#32	0.102	Not Required	Pass
112	0.001	0.543	0.130	0.113	0.025	0.673	#13	0.054	Not Required	Pass
113	0.003	0.234	0.071	0.065	0.003	0.276	#13	0.306	Not Required	Pass
114	0.005	0.269	0.075	0.058	0.003	0.315	#13	0.306	Not Required	Pass
115	0.004	0.409	0.039	0.052	0.002	0.445	#32	0.507	Not Required	Pass
116	0.006	0.307	0.041	0.046	0.002	0.331	#13	0.507	Not Required	Pass
201	0.058	0.660	0.066	0.024	0.006	0.678	#16	0.448	Not Required	Pass
202	0.002	0.453	0.099	0.090	0.020	0.552	#13	0.174	Not Required	Pass
203	0.002	0.698	0.030	0.071	0.007	0.729	#13	0.046	Not Required	Pass
204	0.002	0.558	0.042	0.056	0.010	0.589	#13	0.082	Not Required	Pass
205	0.002	0.433	0.024	0.069	0.006	0.447	#13	0.076	Not Required	Pass
206	0.001	0.537	0.016	0.053	0.003	0.554	#13	0.046	Not Required	Pass
207	0.001	0.333	0.012	0.053	0.004	0.345	#13	0.076	Not Required	Pass
208	0.000	0.016	0.005	0.013	0.001	0.020	#13	Not Required	Not Required	Pass
209	0.002	0.077	0.037	0.003	0.002	0.115	#13	0.206	Not Required	Pass
210	0.001	0.440	0.039	0.044	0.008	0.479	#13	0.122	Not Required	Pass
211	0.000	0.019	0.005	0.016	0.001	0.023	#13	Not Required	Not Required	Pass
212	0.001	0.300	0.081	0.068	0.016	0.382	#13	0.054	Not Required	Pass
213	0.003	0.113	0.080	0.058	0.003	0.165	#16	0.204	Not Required	Pass
214	0.004	0.089	0.085	0.044	0.004	0.130	#13	0.306	Not Required	Pass
215	0.004	0.434	0.045	0.045	0.003	0.455	#13	0.507	Not Required	Pass
216	0.006	0.331	0.050	0.033	0.003	0.363	#13	0.507	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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.5$ 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 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.824</td> <td>12.188</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.625</td> <td>-2.725</td> </tr> <tr> <td>V_z (kip)</td> <td>0.454</td> <td>0.723</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.935</td> <td>1.486</td> </tr> <tr> <td>M_z (kipft)</td> <td>24.473</td> <td>41.065</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 3$ 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)	7.824	12.188	V_x (kip)	-1.625	-2.725	V_z (kip)	0.454	0.723	M_x (kipft)	0.935	1.486	M_z (kipft)	24.473	41.065	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.625 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25876 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24.473 \text{ kipft}) + ((-1.625 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.897 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.021 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.454 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.072293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.935 \text{ kipft}) + ((0.454 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14889 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.9048 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.021 \text{ ft}), (2.9048 \text{ ft})]$$

$$L_{e,req} = 6.021 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.021 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.92631$$

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

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

$$q = \frac{(7.824 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.489 \text{ kip/ft}^2$$

$$q = 0.409 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.489 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2445$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25876 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.897 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.897 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.897 \text{ kipft/ft})) + (4 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4544 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.897 \text{ kipft/ft})) + (3 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.897 \text{ kipft/ft})) + (2 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.23692 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.897 \text{ kipft/ft})) + ((-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.86798 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4544 \text{ ft})}{2}$$

$$p_a = 0.33408 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23692 \text{ kip/ft}^2)}{(0.33408 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.70917$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.710**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.86798 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.89024$	<p>Status: PASS Ratio: 0.890</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.072293 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.14889 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.14889 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.072293 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.14889 \text{ kipft/ft})) + (4 \times (0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.7005 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.14889 \text{ kipft/ft})) + (3 \times (0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.14889 \text{ kipft/ft})) + (2 \times (0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.051483 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.14889 \text{ kipft/ft})) + ((0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.10902 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.7005 \text{ ft})}{2}$ $p_a = 0.35254 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.051483 \text{ kip/ft}^2)}{(0.35254 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.14604$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.150</p>

$$Ratio = \frac{(0.10902 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.11181$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.725 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43392 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.065 \text{ kipft}) + ((-2.725 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.539 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.539 \text{ kipft/ft})}{(-0.43392 \text{ kip/ft})}$$

$$E = 15.07 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.539 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.43392 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.539 \text{ kipft/ft})) + (4 \times (-0.43392 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4543 \text{ ft}$$

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

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

$$V_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4543 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4543 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2684 \text{ kip}$$

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

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

$$M_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(15.07 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.844 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.723 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.11513 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.486 \text{ kipft}) + ((0.723 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23662 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23662 \text{ kipft/ft})}{(0.11513 \text{ kip/ft})}$$

$$E = 2.0553 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23662 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.11513 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.23662 \text{ kipft/ft})) + (4 \times (0.11513 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.7007 \text{ ft}$$

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

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

$$V_{max} = ((0.11513 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.7007 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.7007 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.56666 \text{ kip}$$

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

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

$$M_{max} = ((0.11513 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.0553 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5762 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.188 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.86 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.86 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.188 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0038286$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 12.188 \text{ kip} \rightarrow 12188 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(12188 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.42 \text{ kip}$$

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

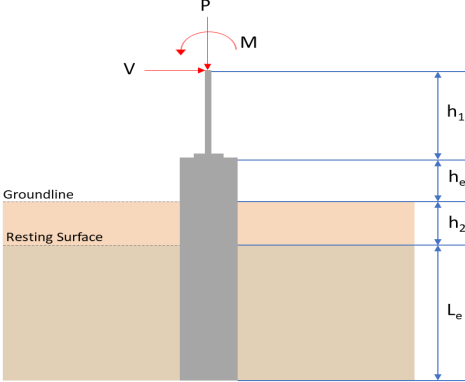
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.42 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.42 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.42 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.5 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.2684 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2684 \text{ kip})}{(118.5 \text{ kip})}$ $\text{Ratio} = 0.069773$ <p>Considering z-direction:</p> <p>$V_{max} = 0.56666 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.56666 \text{ kip})}{(118.5 \text{ kip})}$ $\text{Ratio} = 0.0047818$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<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: $M_{max} = 25.844\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.844\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.09452$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 1.5762\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5762\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0057647$	<p>Status: PASS Ratio: 0.010</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 $L = 6.5$ 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>7.824</td> <td>12.188</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.625</td> <td>-2.725</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.454</td> <td>-0.723</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.935</td> <td>-1.486</td> </tr> <tr> <td>M_z (kipft)</td> <td>24.473</td> <td>41.066</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)	7.824	12.188	V_x (kip)	-1.625	-2.725	V_z (kip)	-0.454	-0.723	M_x (kipft)	-0.935	-1.486	M_z (kipft)	24.473	41.066	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	-0.935	-1.486																										
M_z (kipft)	24.473	41.066																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.625 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25876 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24.473 \text{ kipft}) + ((-1.625 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.897 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.021 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.454 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.072293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.935 \text{ kipft}) + ((-0.454 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14889 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6708 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.021 \text{ ft}), (1.6708 \text{ ft})]$$

$$L_{e,req} = 6.021 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.021 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.92631$$

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

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

$$q = \frac{(7.824 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.489 \text{ kip/ft}^2$$

$$q = 0.409 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.489 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.2445$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25876 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.897 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.897 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.897 \text{ kipft/ft})) + (4 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4544 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.897 \text{ kipft/ft})) + (3 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.897 \text{ kipft/ft})) + (2 \times (-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.23692 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.897 \text{ kipft/ft})) + ((-0.25876 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.86798 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4544 \text{ ft})}{2}$$

$$p_a = 0.33408 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23692 \text{ kip/ft}^2)}{(0.33408 \text{ kip/ft}^2)}$$

$$Ratio = 0.70917$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.710**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.86798 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89024$$

Status: **PASS**
Ratio: **0.890**

Considering z-direction:

$H_o = -0.072293 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.14889 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14889 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.072293 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.14889 \text{ kipft/ft})) + (4 \times (-0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.7005 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.14889 \text{ kipft/ft})) + (3 \times (-0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.14889 \text{ kipft/ft})) + (2 \times (-0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = -0.023861 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.14889 \text{ kipft/ft})) + ((-0.072293 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = -0.024445 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7005 \text{ ft})}{2}$$

$$p_a = 0.35254 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.023861 \text{ kip/ft}^2)}{(0.35254 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.067683$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

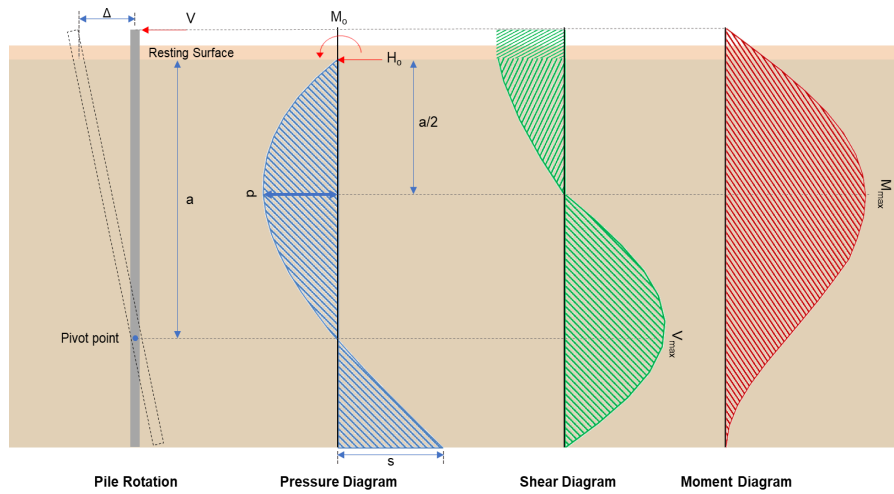
$$\text{Ratio} = \frac{s}{p_s}$$

Status: **PASS**
Ratio: **-0.070**

$$Ratio = \frac{(-0.024445 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = -0.025072$$

Status: **PASS**
Ratio: **-0.030**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.725 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43392 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.066 \text{ kipft}) + ((-2.725 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5392 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5392 \text{ kipft/ft})}{(-0.43392 \text{ kip/ft})}$$

$$E = 15.07 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5392 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.43392 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.5392 \text{ kipft/ft})) + (4 \times (-0.43392 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4543 \text{ ft}$$

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

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

$$V_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4543 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4543 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2685 \text{ kip}$$

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

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

$$M_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(15.07 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.07 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4543 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.845 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.723 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11513 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.486 \text{ kipft}) + ((-0.723 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23662 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23662 \text{ kipft/ft})}{(-0.11513 \text{ kip/ft})}$$

$$E = 2.0553 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23662 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.11513 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.23662 \text{ kipft/ft})) + (4 \times (-0.11513 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.7007 \text{ ft}$$

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

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

$$V_{max} = ((-0.11513 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.7007 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.7007 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.56666 \text{ kip}$$

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

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

$$M_{max} = ((-0.11513 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.0553 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0553 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.7007 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5762 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.188 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.86 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.86 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.188 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0038286$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 12.188 \text{ kip} \rightarrow 12188 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(12188 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.42 \text{ kip}$$

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

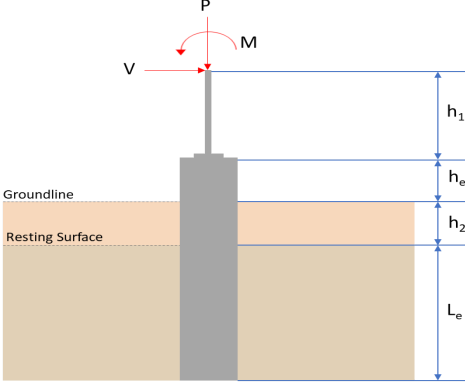
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.42 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.42 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.42 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.5 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.2685 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2685 \text{ kip})}{(118.5 \text{ kip})}$ $\text{Ratio} = 0.069775$ <p>Considering z-direction:</p> <p>$V_{max} = 0.56666 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.56666 \text{ kip})}{(118.5 \text{ kip})}$ $\text{Ratio} = 0.0047818$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<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: $M_{max} = 25.845\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.845\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.094522$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 1.5762\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5762\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0057647$	<p>Status: PASS Ratio: 0.010</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 $L = 6.75$ 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>11.005</td> <td>17.220</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.283</td> <td>-3.789</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>29.674</td> <td>49.848</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)	11.005	17.220	V_x (kip)	-2.283	-3.789	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	29.674	49.848	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.283 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.36354 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(29.674 \text{ kipft}) + ((-2.283 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.7252 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 6.2323 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.2323 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.232 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.232 \text{ ft})}{(6.75 \text{ ft})}$ $\text{Ratio} = 0.92326$	<p>Status: PASS Ratio: 0.920</p>
	<p>End-bearing Capacity (ASD) A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(11.005 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.68781 \text{ kip/ft}^2$ <p>Check bearing capacity ratio: <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.68781 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.34391$	<p>Status: PASS Ratio: 0.340</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.36354$ kip/ft - Lateral force per length of pile,

$M_o = 4.7252$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7252 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.36354 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.7252 \text{ kipft/ft})) + (4 \times (-0.36354 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6447 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.7252 \text{ kipft/ft})) + (3 \times (-0.36354 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.7252 \text{ kipft/ft})) + (2 \times (-0.36354 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.23649 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.7252 \text{ kipft/ft})) + ((-0.36354 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.92134 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6447 \text{ ft})}{2}$$

$$p_a = 0.34835 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23649 \text{ kip/ft}^2)}{(0.34835 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.6789$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

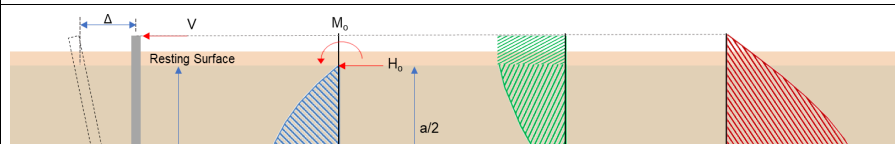
$$\text{Ratio} = \frac{s}{p_s}$$

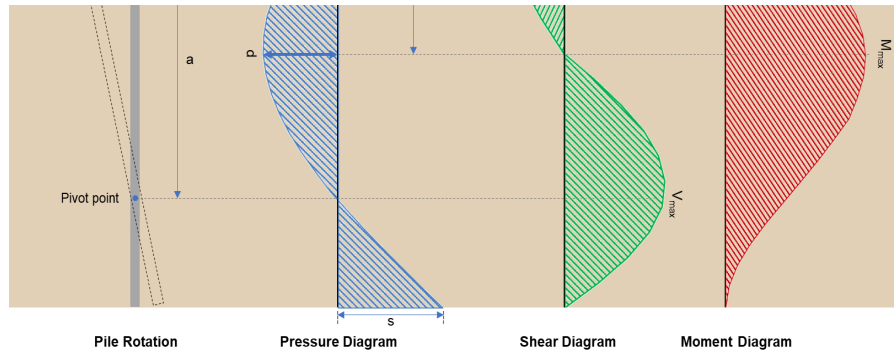
$$\text{Ratio} = \frac{(0.92134 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90997$$

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

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.789 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60334 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.848 \text{ kipft}) + ((-3.789 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.9376 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.9376 \text{ kipft/ft})}{(-0.60334 \text{ kip/ft})}$$

$$E = 13.156 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9376 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.60334 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (7.9376 \text{ kipft/ft})) + (4 \times (-0.60334 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6434 \text{ ft}$$

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

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

$$V_{max} = ((-0.60334 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.156 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6434 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.156 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6434 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.9163 \text{ kip}$$

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

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

$$M_{max} = ((-0.60334 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(13.156 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6434 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.156 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6434 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.156 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6434 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 31.986 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.22 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.69 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.69 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 \emptyset : 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)**

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

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

$$\text{Ratio} = \frac{(17.22 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0054093$$

Status: **PASS**
Ratio: **0.010****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 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

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 17.22 \text{ kip} \rightarrow 17220 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(17220 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 132.09 \text{ kip}$$

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

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (132.09 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 132.09 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - 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> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((132.09 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.94 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.9163 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.9163 \text{ kip})}{(118.94 \text{ kip})}$ $\text{Ratio} = 0.083372$	<p>Status: PASS Ratio: 0.080</p>
<p>14.5.2.1b</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>$\lambda = 1$ - 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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$$\phi M_{n,z} = \phi S_x F_y$$

$$\phi M_{n,z} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,z} = 2545.9 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

$$\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$

$$\phi M_n = 273.42 \text{ kipft}$$

Considering x-direction:

$M_{max} = 31.986 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{M_{max}}{\phi M_n}$$

$$\text{Ratio} = \frac{(31.986 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.11698$$

Status: **PASS**
Ratio: **0.120**