

Your Project Calculations



Project Name: W12185 -11ft- CU-

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=W12185%20-11ft-%20CU-&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=u8YBGFXVBteqtQ370hQYfY4fy1eRBTj5Hj10F6VReFZoZvcRQR0Kg3V0PgPK1wRQ

Array Specification

Product:	Beam
Unique ID:	3P-19.75-8TOP-XD-57-L-5Hx10W-21DE
Duty Classification:	XD
Module Width:	40.90 in
Module Length:	69.13in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	17
Front Edge Clearance:	11
Total Array Height at Tilt:	16.01 ft
Total Frame Length:	56.50 ft
Frame Weight:	3333 lbs
Array Dimensions N/S:	17.25 ft
Array Dimensions E/W:	58.44 ft
Rail Length:	207.00 in
Rail Spacing:	2.88 ft
Rail Check:	Not Checked

Support Specifications

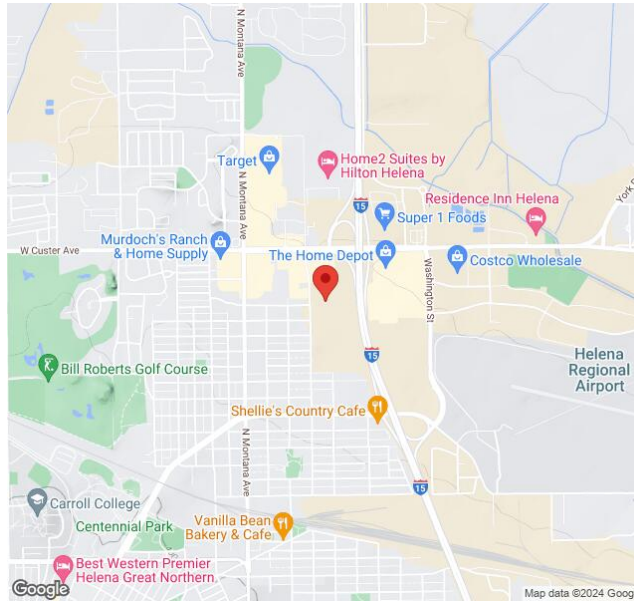
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	13.52 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.00 ft Pile 3: 5.75 ft
Foundation Volume:	10.370 y ³
Foundation Result:	PASSED
Mount Twist:	0.091704 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sandy_gravel
Site Location:	2845 N Sanders St, Helena, MT 59601, USA
Wind Speed:	106 mph
Snow Load:	30 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.017484 ksf



Design Disclaimer

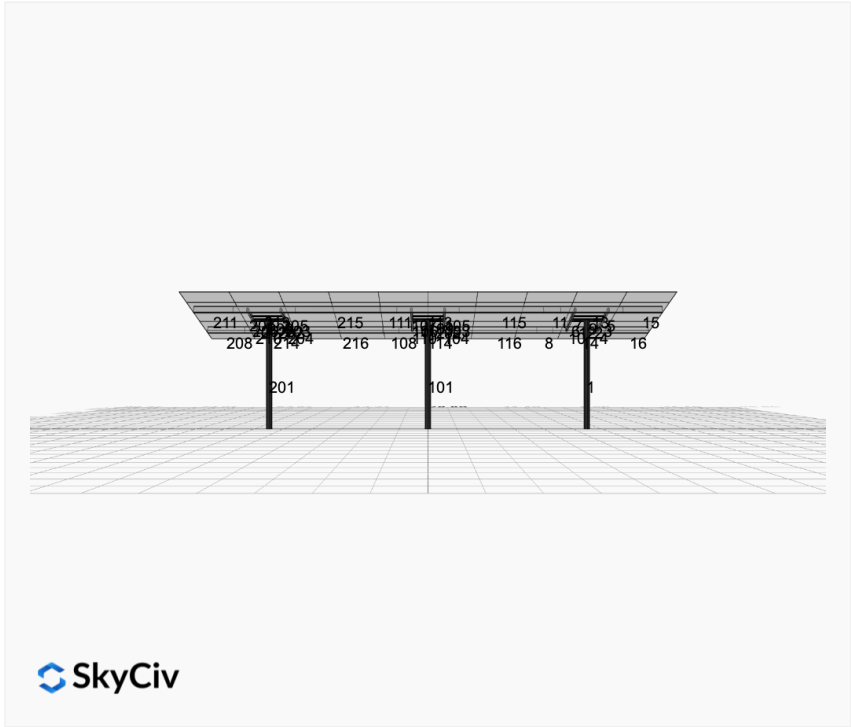
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

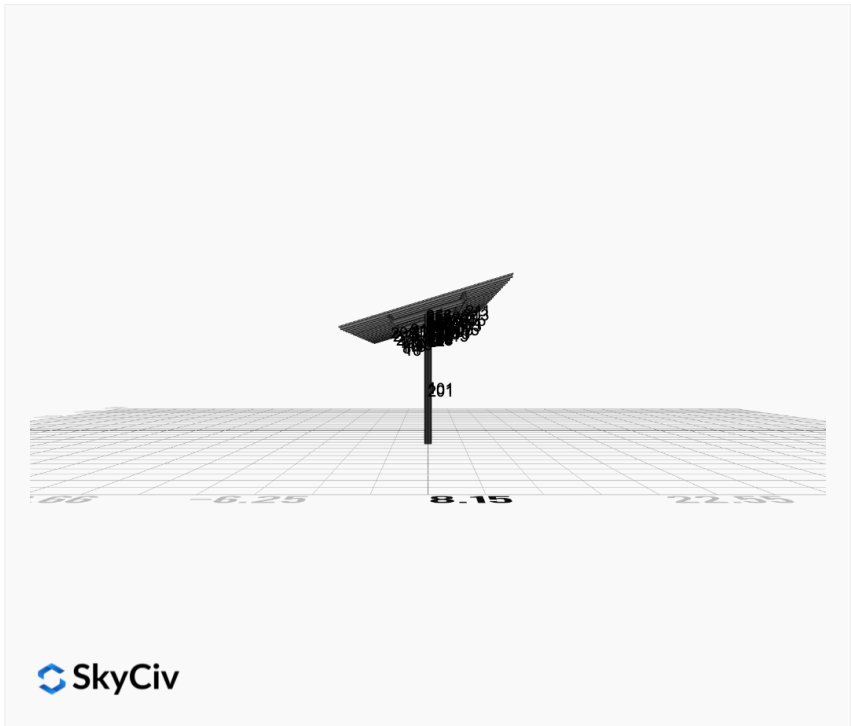
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Design Notes:

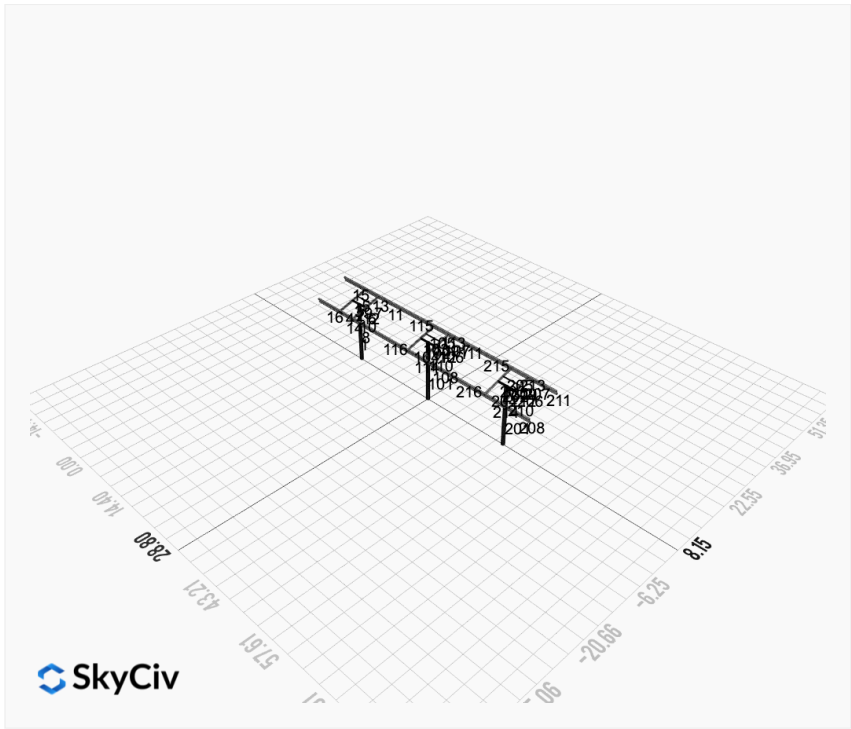
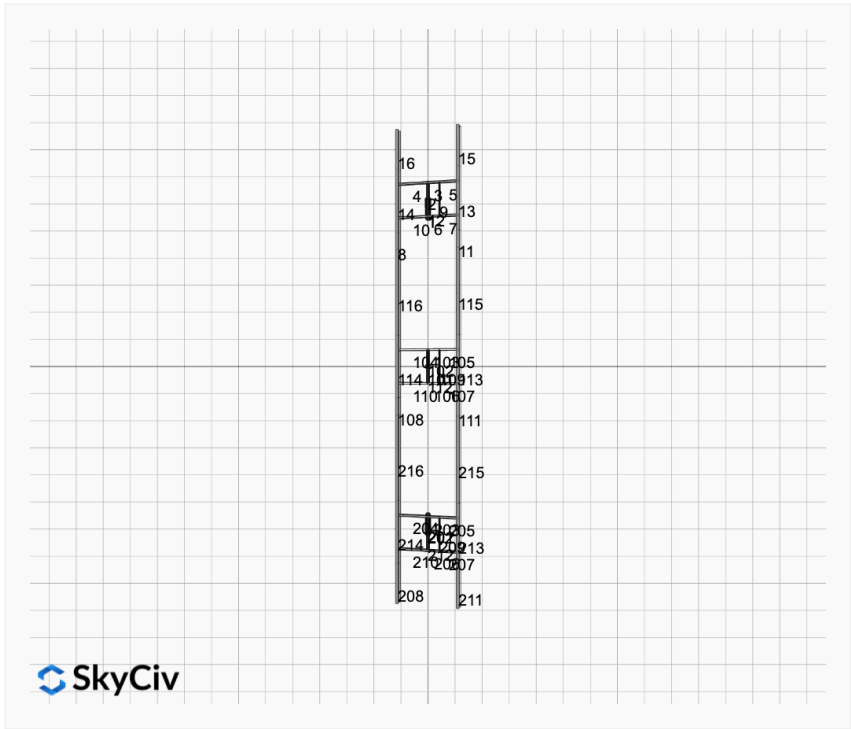
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

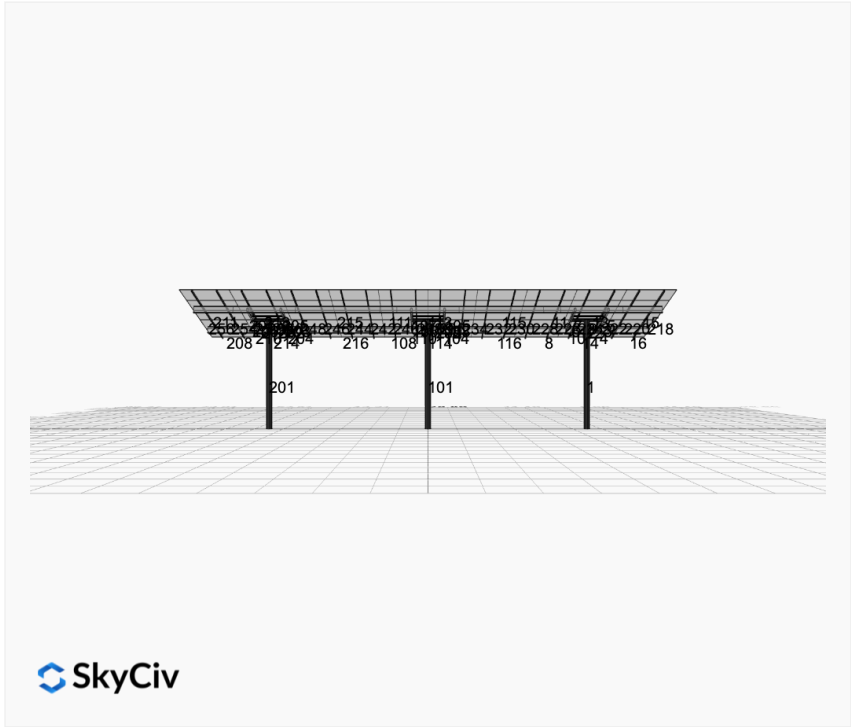


 SkyCiv

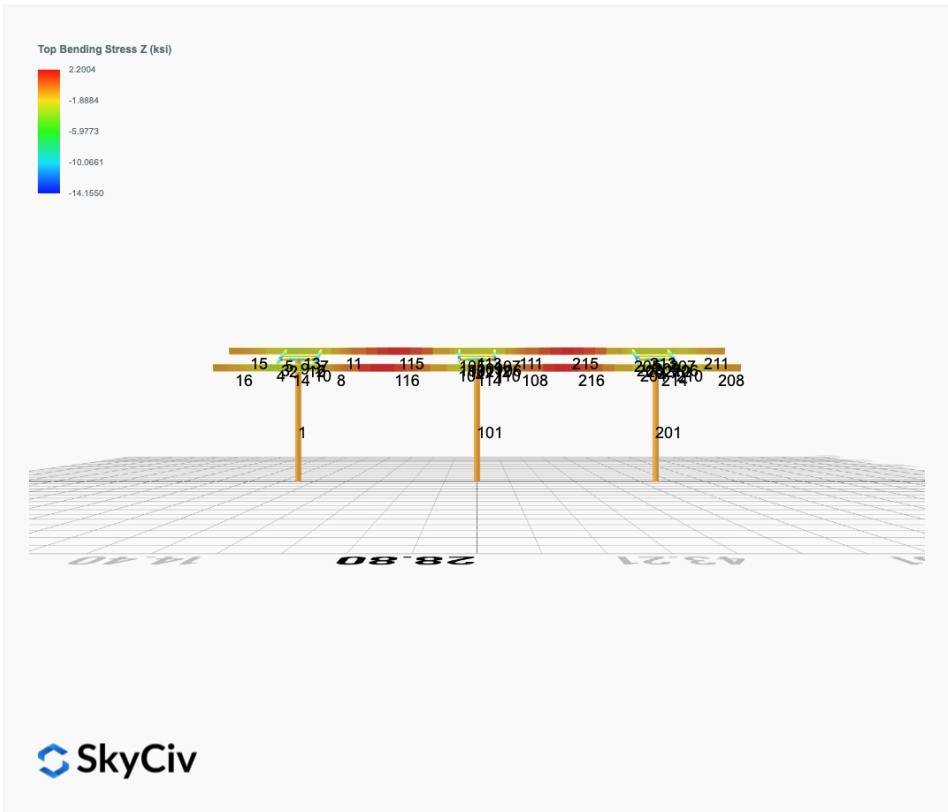
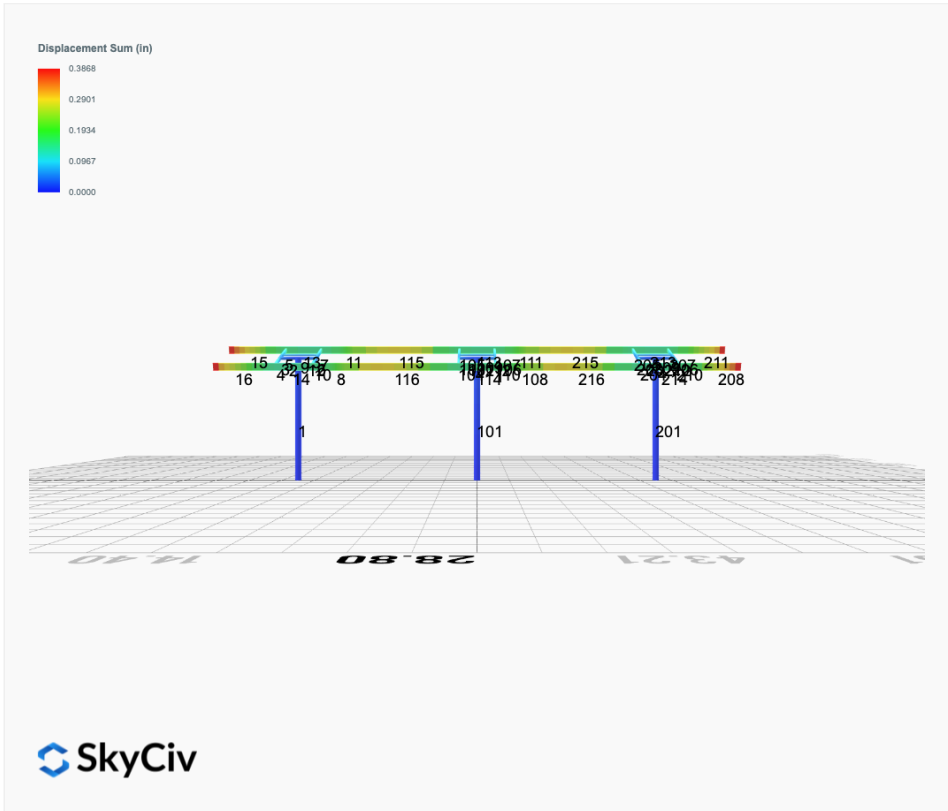


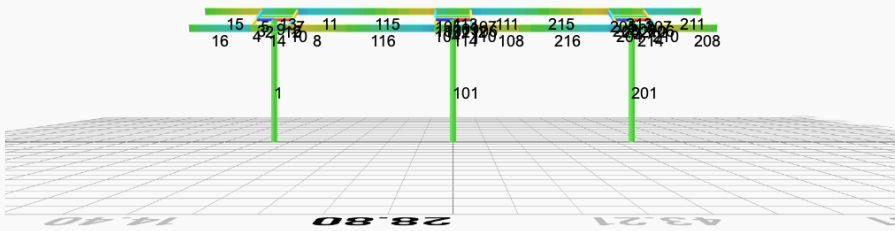
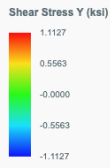
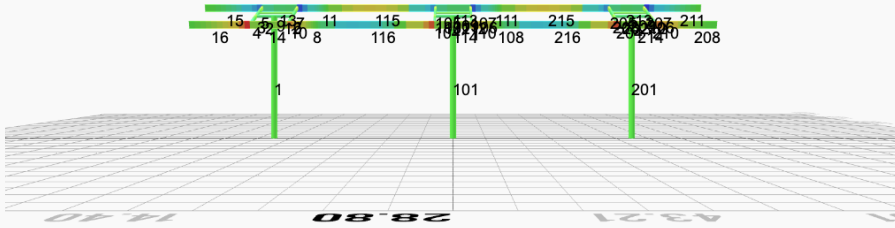
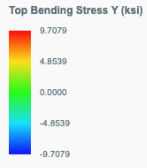
 SkyCiv

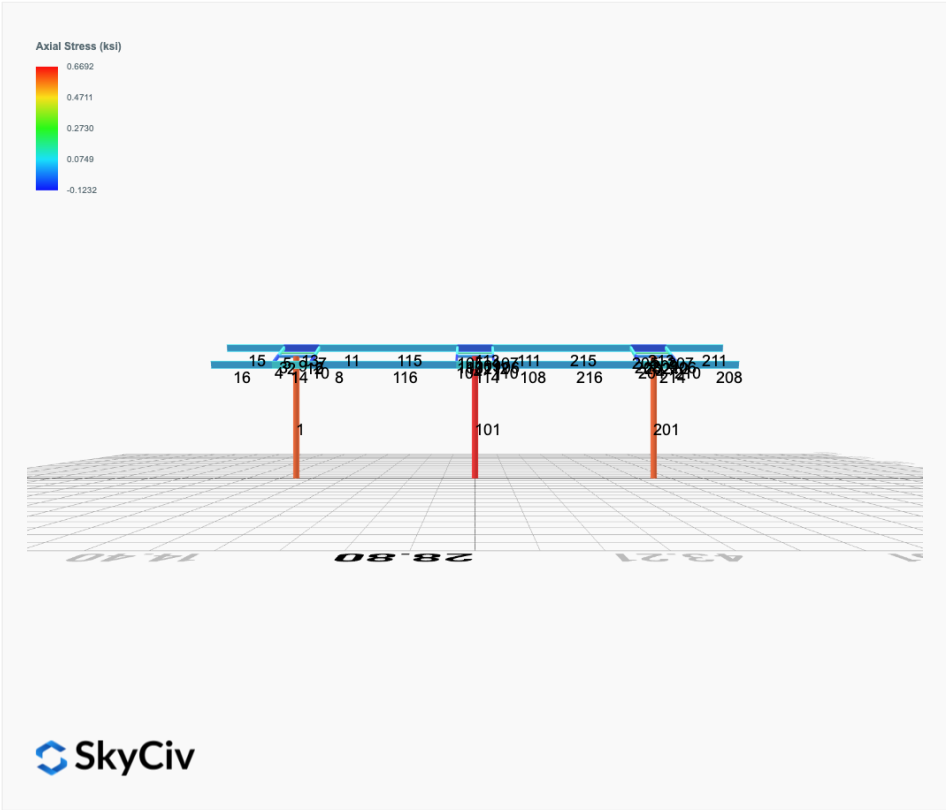




FEM Results (Envelope Worst Case for each member)







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	-0.0034	2.6204	-0.0039	-0.0154	0.0159	0.0708
ULS: 2. D + L	-0.0034	2.6204	-0.0039	-0.0154	0.0159	0.0708
ULS: 3. D + (S or Lr or R)	-0.0123	7.9578	-0.0141	-0.0557	0.0587	0.1914
ULS: 3. D + (S or Lr or R)	-0.0034	2.6204	-0.0039	-0.0154	0.0159	0.0708
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0100	6.6235	-0.0115	-0.0456	0.0480	0.1613
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0034	2.6204	-0.0039	-0.0154	0.0159	0.0708
ULS: 5b. D + 0.7E	-0.0034	2.6204	-0.0039	-0.0154	0.0159	0.0708
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0100	6.6235	-0.0115	-0.0456	0.0480	0.1613
ULS: 8. 0.6D + 0.7E	-0.0020	1.5723	-0.0023	-0.0092	0.0095	0.0425
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3731	7.0821	-0.0077	-0.0317	0.0172	20.7084
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3731	7.0821	-0.0077	-0.0317	0.0172	20.7084
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0871	-0.9356	0.0003	0.0023	0.0126	-12.5709
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9463	-0.4642	-0.0036	-0.0140	0.0195	-23.3830
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0374	9.9698	-0.0144	-0.0578	0.0490	15.6395
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0374	9.9698	-0.0144	-0.0578	0.0490	15.6395
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8078	3.9565	-0.0084	-0.0323	0.0455	-9.3200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7022	4.3100	-0.0113	-0.0445	0.0507	-17.4291
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0307	5.9667	-0.0067	-0.0276	0.0169	15.5490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0307	5.9667	-0.0067	-0.0276	0.0169	15.5490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8145	-0.0466	-0.0007	-0.0021	0.0134	-9.4105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7089	0.3069	-0.0037	-0.0143	0.0186	-17.5195
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3718	6.0340	-0.0062	-0.0255	0.0108	20.6801
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3718	6.0340	-0.0062	-0.0255	0.0108	20.6801
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0884	-1.9838	0.0019	0.0085	0.0063	-12.5992
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9477	-1.5124	-0.0021	-0.0078	0.0132	-23.4113

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	15.4027
Shear X	-2.2920
Shear Z	-0.0241
Moment X	-0.0966
Moment Y (Twist)	0.0918
Moment Z	40.1452

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	9.9698
Shear X	-1.3731
Shear Z	-0.0144
Moment X	-0.0578
Moment Y (Twist)	0.0587
Moment Z	23.4113

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	0.0067	2.7241	-0.0000	0.0000	0.0000	-0.0441
ULS: 2. D + L	0.0067	2.7241	-0.0000	0.0000	0.0000	-0.0441
ULS: 3. D + (S or Lr or R)	0.0246	8.3453	0.0000	0.0000	-0.0000	-0.2332
ULS: 3. D + (S or Lr or R)	0.0067	2.7241	-0.0000	0.0000	0.0000	-0.0441
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0201	6.9400	0.0000	0.0000	-0.0000	-0.1859
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0067	2.7241	-0.0000	0.0000	0.0000	-0.0441
ULS: 5b. D + 0.7E	0.0067	2.7241	-0.0000	0.0000	0.0000	-0.0441

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0201	6.9400	0.0000	0.0000	-0.0000	-0.1859
ULS: 8. 0.6D + 0.7E	0.0040	1.6345	-0.0000	0.0000	0.0000	-0.0265
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4065	7.3839	-0.0000	0.0000	0.0000	21.1776
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4065	7.3839	-0.0000	0.0000	0.0000	21.1776
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1372	-0.9949	-0.0000	0.0000	0.0000	-13.0774
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9754	-0.4875	-0.0000	0.0000	0.0000	-24.0156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0398	10.4348	0.0000	0.0000	-0.0000	15.7304
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0398	10.4348	0.0000	0.0000	-0.0000	15.7304
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8680	4.1508	0.0000	0.0000	-0.0000	-9.9609
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7466	4.5313	0.0000	0.0000	-0.0000	-18.1646
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0532	6.2189	-0.0000	0.0000	0.0000	15.8722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0532	6.2189	-0.0000	0.0000	0.0000	15.8722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8546	-0.0651	-0.0000	0.0000	0.0000	-9.8191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7332	0.3154	-0.0000	0.0000	0.0000	-18.0227
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4092	6.2943	-0.0000	0.0000	0.0000	21.1953
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4092	6.2943	-0.0000	0.0000	0.0000	21.1953
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1345	-2.0845	-0.0000	0.0000	0.0000	-13.0598
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9727	-1.5771	-0.0000	0.0000	0.0000	-23.9980

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	16.1454
Shear X	-2.3554
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	41.3404

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	10.4348
Shear X	-1.4092
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	24.0156

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	-0.0034	2.6204	0.0039	0.0154	-0.0159	0.0708
ULS: 2. D + L	-0.0034	2.6204	0.0039	0.0154	-0.0159	0.0708
ULS: 3. D + (S or Lr or R)	-0.0123	7.9578	0.0141	0.0557	-0.0587	0.1914
ULS: 3. D + (S or Lr or R)	-0.0034	2.6204	0.0039	0.0154	-0.0159	0.0708
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0100	6.6235	0.0115	0.0456	-0.0480	0.1613
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0034	2.6204	0.0039	0.0154	-0.0159	0.0708
ULS: 5b. D + 0.7E	-0.0034	2.6204	0.0039	0.0154	-0.0159	0.0708
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0100	6.6235	0.0115	0.0456	-0.0480	0.1613
ULS: 8. 0.6D + 0.7E	-0.0020	1.5723	0.0023	0.0092	-0.0095	0.0425
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3731	7.0821	0.0077	0.0317	-0.0172	20.7084
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3731	7.0821	0.0077	0.0317	-0.0172	20.7084
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0871	-0.9356	-0.0003	-0.0023	-0.0126	-12.5709
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9463	-0.4642	0.0036	0.0140	-0.0195	-23.3830
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0374	9.9698	0.0144	0.0578	-0.0490	15.6395
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0374	9.9698	0.0144	0.0578	-0.0490	15.6395
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8078	3.9565	0.0084	0.0323	-0.0455	-9.3200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7022	4.3100	0.0113	0.0445	-0.0507	-17.4291

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	-1.0307	5.9667	0.0067	0.0276	-0.0169	15.5490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0307	5.9667	0.0067	0.0276	-0.0169	15.5490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8145	-0.0466	0.0007	0.0021	-0.0134	-9.4105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7089	0.3069	0.0037	0.0143	-0.0186	-17.5195
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3718	6.0340	0.0062	0.0255	-0.0108	20.6801
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3718	6.0340	0.0062	0.0255	-0.0108	20.6801
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0884	-1.9838	-0.0019	-0.0085	-0.0063	-12.5992
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9477	-1.5124	0.0021	0.0078	-0.0132	-23.4113

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	15.4027
Shear X	-2.2920
Shear Z	0.0241
Moment X	0.0968
Moment Y (Twist)	0.0917
Moment Z	40.1459

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	9.9698
Shear X	-1.3731
Shear Z	0.0144
Moment X	0.0578
Moment Y (Twist)	0.0587
Moment Z	23.4113

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial



Design Input Information

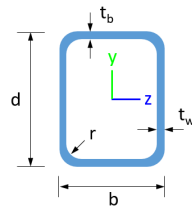
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

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	377.97	141.04	83.29	83.29	113.39	113.39
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	140.46	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	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	32.84	6.46	56.26	44.91
14	159.30	97.43	33.35	6.46	56.26	44.91
15	159.30	34.37	46.90	6.46	56.26	44.91
16	159.30	34.37	46.90	6.46	56.26	44.91
101	377.97	141.04	83.29	83.29	113.39	113.39
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	140.46	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	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.57	6.46	56.26	44.91
114	159.30	97.43	31.72	6.46	56.26	44.91
115	159.30	75.13	21.01	6.46	56.26	44.91
116	159.30	75.13	21.59	6.46	56.26	44.91
201	377.97	141.04	83.29	83.29	113.39	113.39
202	251.01	248.88	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	34.37	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	34.37	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	32.83	6.46	56.26	44.91
214	159.30	97.43	33.35	6.46	56.26	44.91
215	159.30	75.13	22.42	6.46	56.26	44.91
216	159.30	75.13	22.53	6.46	56.26	44.91

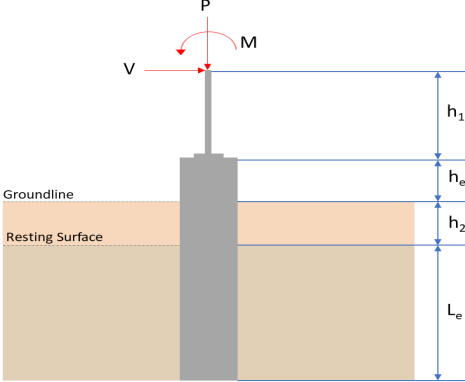
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.109	0.482	0.003	0.020	0.000	0.485	#16	0.580	Not Required	Pass
2	0.002	0.477	0.090	0.100	0.016	0.550	#21	0.036	Not Required	Pass
3	0.005	0.691	0.028	0.069	0.002	0.722	#21	0.046	Not Required	Pass
4	0.005	0.659	0.097	0.066	0.020	0.720	#21	0.082	Not Required	Pass
5	0.005	0.429	0.101	0.068	0.026	0.454	#21	0.076	Not Required	Pass
6	0.006	0.684	0.032	0.068	0.003	0.719	#21	0.046	Not Required	Pass
7	0.006	0.425	0.104	0.068	0.027	0.453	#21	0.076	Not Required	Pass
8	0.000	0.045	0.107	0.042	0.012	0.144	#21	0.068	Not Required	Pass
9	0.011	0.072	0.029	0.001	0.000	0.101	#21	0.206	Not Required	Pass
10	0.006	0.652	0.102	0.065	0.022	0.722	#21	0.082	Not Required	Pass
11	0.000	0.046	0.109	0.044	0.012	0.148	#21	0.102	Not Required	Pass
12	0.002	0.469	0.089	0.099	0.015	0.539	#21	0.036	Not Required	Pass
13	0.005	0.268	0.282	0.056	0.015	0.531	#21	0.306	Not Required	Pass
14	0.006	0.262	0.282	0.054	0.015	0.521	#21	0.204	Not Required	Pass
15	0.000	0.095	0.150	0.033	0.009	0.245	#21	Not Required	Not Required	Pass
16	0.000	0.091	0.150	0.032	0.009	0.241	#21	Not Required	Not Required	Pass
101	0.114	0.496	0.000	0.021	0.000	0.499	#16	0.580	Not Required	Pass
102	0.002	0.493	0.091	0.104	0.016	0.564	#21	0.036	Not Required	Pass
103	0.006	0.722	0.036	0.072	0.005	0.760	#21	0.046	Not Required	Pass
104	0.006	0.687	0.095	0.069	0.021	0.754	#21	0.082	Not Required	Pass
105	0.006	0.448	0.098	0.071	0.025	0.473	#21	0.076	Not Required	Pass
106	0.006	0.722	0.036	0.072	0.005	0.760	#21	0.046	Not Required	Pass
107	0.006	0.448	0.098	0.071	0.025	0.473	#21	0.076	Not Required	Pass
108	0.000	0.055	0.102	0.040	0.012	0.129	#21	0.068	Not Required	Pass
109	0.009	0.068	0.027	0.001	0.000	0.100	#21	0.206	Not Required	Pass
110	0.006	0.687	0.095	0.069	0.021	0.754	#21	0.082	Not Required	Pass
111	0.000	0.057	0.103	0.042	0.012	0.132	#21	0.102	Not Required	Pass
112	0.002	0.493	0.091	0.104	0.016	0.564	#21	0.036	Not Required	Pass
113	0.004	0.215	0.267	0.054	0.015	0.458	#21	0.306	Not Required	Pass
114	0.006	0.209	0.265	0.052	0.015	0.447	#21	0.204	Not Required	Pass
115	0.000	0.257	0.146	0.042	0.012	0.404	#21	0.507	Not Required	Pass
116	0.000	0.248	0.147	0.040	0.012	0.395	#21	0.338	Not Required	Pass
201	0.109	0.482	0.003	0.020	0.000	0.485	#16	0.580	Not Required	Pass
202	0.002	0.469	0.089	0.099	0.015	0.539	#21	0.036	Not Required	Pass
203	0.006	0.684	0.032	0.068	0.003	0.719	#21	0.046	Not Required	Pass
204	0.006	0.652	0.102	0.065	0.022	0.722	#21	0.082	Not Required	Pass
205	0.006	0.425	0.104	0.068	0.027	0.453	#21	0.076	Not Required	Pass
206	0.005	0.691	0.028	0.069	0.002	0.722	#21	0.046	Not Required	Pass
207	0.005	0.429	0.101	0.068	0.026	0.454	#21	0.076	Not Required	Pass
208	0.000	0.091	0.150	0.032	0.009	0.241	#21	Not Required	Not Required	Pass
209	0.011	0.072	0.029	0.001	0.000	0.101	#21	0.206	Not Required	Pass
210	0.005	0.659	0.097	0.066	0.020	0.720	#21	0.082	Not Required	Pass
211	0.000	0.095	0.150	0.033	0.009	0.245	#21	Not Required	Not Required	Pass
212	0.002	0.477	0.090	0.100	0.016	0.550	#21	0.036	Not Required	Pass
213	0.005	0.268	0.282	0.056	0.015	0.531	#21	0.204	Not Required	Pass
214	0.006	0.262	0.282	0.054	0.015	0.521	#21	0.204	Not Required	Pass
215	0.000	0.253	0.146	0.044	0.012	0.399	#21	0.507	Not Required	Pass
216	0.000	0.243	0.147	0.042	0.012	0.391	#21	0.338	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="515 1102 1094 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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</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>9.970</td> <td>15.403</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.373</td> <td>-2.292</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.014</td> <td>-0.024</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.058</td> <td>-0.097</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.411</td> <td>40.145</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	9.970	15.403	V_x (kip)	-1.373	-2.292	V_z (kip)	-0.014	-0.024	M_x (kipft)	-0.058	-0.097	M_z (kipft)	23.411	40.145	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.373 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.21863 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(23.411 \text{ kipft}) + ((-1.373 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.7279 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5315 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.058 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0092357 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.78082 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5315 \text{ ft}), (0.78082 \text{ ft})]$$

$$L_{e,req} = 5.532 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.532 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.96209$$

Status: **PASS**
Ratio: **0.960**

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

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

$$q = \frac{(9.97 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.623125 \text{ kip/ft}^2$$

$$q = 0.02010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62313 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.20771$$

Status: **PASS**
Ratio: **0.210**

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:

$H_o = -0.21863 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.7279 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.7279 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.7279 \text{ kipft/ft})) + (4 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9213 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.7279 \text{ kipft/ft})) + (3 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.7279 \text{ kipft/ft})) + (2 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.32473 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.7279 \text{ kipft/ft})) + ((-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 1.1249 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(3.9213 \text{ ft})}{2}$$

$$p_a = 0.39213 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32473 \text{ kip/ft}^2)}{(0.39213 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82811$$

p_a - Allowable lateral soil pressure at depth L_e ,

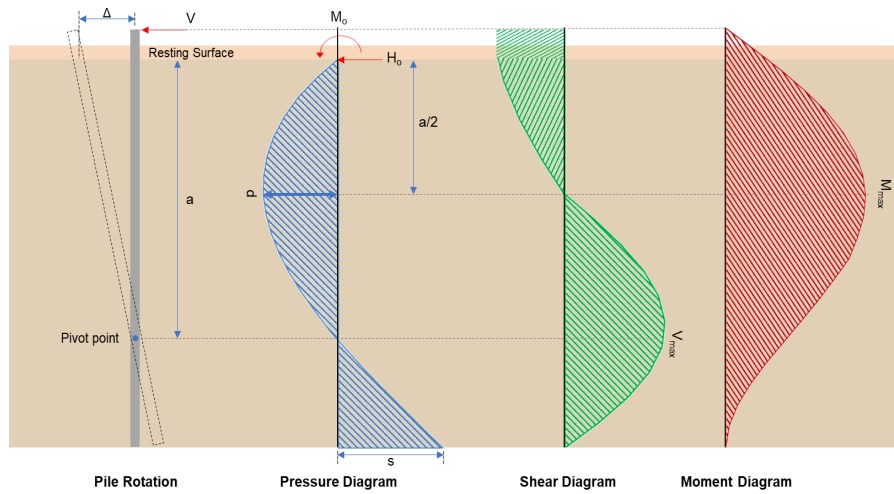
Status: **PASS**
Ratio: **0.830**

	$p_s = R L_e$ $p_s = (200 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 1.15 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.1249 \text{ kip/ft}^2)}{(1.15 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97816$	<p>Status: PASS Ratio: 0.980</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.0022293 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0092357 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0092357 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.0092357 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0636 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0092357 \text{ kipft/ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.0092357 \text{ kipft/ft})) + (2 \times (-0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.000025077 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0092357 \text{ kipft/ft})) + ((-0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.0010259 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (200 \text{ psf/ft}) \times \frac{(4.0636 \text{ ft})}{2}$ $p_a = 0.40636 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.000025077 \text{ kip/ft}^2)}{(0.40636 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00006171$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (200 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 1.15 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.000</p>

$$\text{Ratio} = \frac{(0.0010259 \text{ kip/ft}^2)}{(1.15 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00089204$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.292 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36497 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.145 \text{ kipft}) + ((-2.292 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3925 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3925 \text{ kipft/ft})}{(-0.36497 \text{ kip/ft})}$$

$$E = 17.515 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3925 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.36497 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (6.3925 \text{ kipft/ft})) + (4 \times (-0.36497 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9194 \text{ ft}$$

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

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

$$V_{max} = ((-0.36497 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9194 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9194 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.8396 \text{ kip}$$

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

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

$$M_{max} = ((-0.36497 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(17.515 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.515 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.515 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 24.647 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.097 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015446 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015446 \text{ kipft/ft})}{(-0.0038217 \text{ kip/ft})}$$

$$E = 4.0417 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015446 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.015446 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0666 \text{ ft}$$

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

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

$$V_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0666 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0666 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.029149 \text{ kip}$$

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

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

$$M_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.0417 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.075925 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.403 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.084 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.084 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.403 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0057577$$

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.403 \text{ kip} \rightarrow 15403 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(15403 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

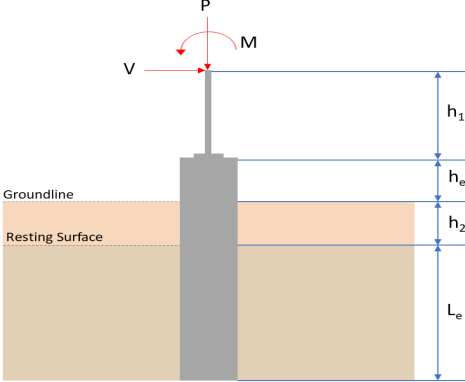
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.43 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.8396 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.8396 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.079327$ <p>Considering z-direction:</p> <p>$V_{max} = 0.029149 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.029149 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.00026159$	<p>Status: PASS Ratio: 0.080</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 24.647 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.647 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.098744$	<p>Status: PASS Ratio: 0.100</p>
	<p>Considering z-direction: $M_{max} = 0.075925 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.075925 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00030419$	<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="517 1102 1094 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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1460"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.970</td> <td>15.403</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.373</td> <td>-2.292</td> </tr> <tr> <td>V_z (kip)</td> <td>0.014</td> <td>0.024</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.058</td> <td>0.097</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.411</td> <td>40.146</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	9.970	15.403	V_x (kip)	-1.373	-2.292	V_z (kip)	0.014	0.024	M_x (kipft)	0.058	0.097	M_z (kipft)	23.411	40.146	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sandy gravel and/or gravel	3000.000	200.000																									
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V_z (kip)	0.014	0.024																										
M_x (kipft)	0.058	0.097																										
M_z (kipft)	23.411	40.146																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.373 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.21863 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(23.411 \text{ kipft}) + ((-1.373 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.7279 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5315 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.058 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0092357 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.86219 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5315 \text{ ft}), (0.86219 \text{ ft})]$$

$$L_{e,req} = 5.532 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.532 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.96209$$

Status: **PASS**
Ratio: **0.960**

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

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

$$q = \frac{(9.97 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.62319 \text{ kip/ft}^2$$

$$q = 0.02010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62313 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.20771$$

Status: **PASS**
Ratio: **0.210**

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:

$H_o = -0.21863 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.7279 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.7279 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.7279 \text{ kipft/ft})) + (4 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9213 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.7279 \text{ kipft/ft})) + (3 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.7279 \text{ kipft/ft})) + (2 \times (-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.32473 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.7279 \text{ kipft/ft})) + ((-0.21863 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 1.1249 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(3.9213 \text{ ft})}{2}$$

$$p_a = 0.39213 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32473 \text{ kip/ft}^2)}{(0.39213 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82811$$

p_a - Allowable lateral soil pressure at depth L_e ,

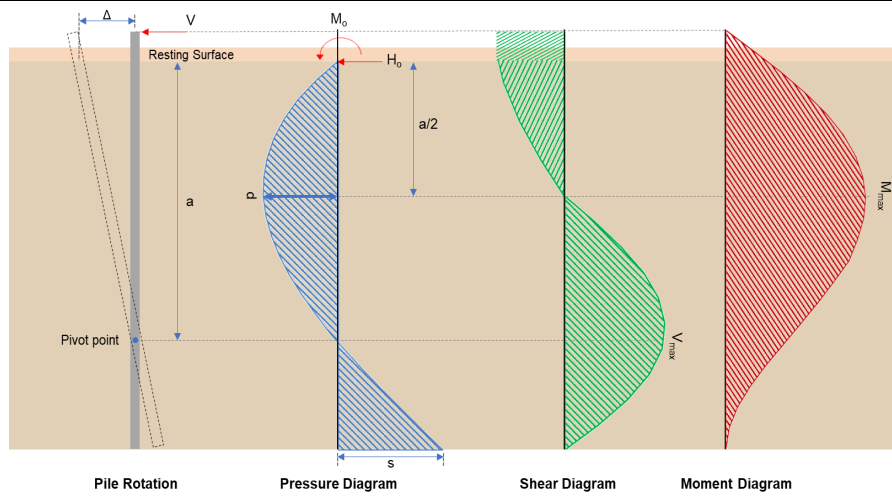
Status: **PASS**
Ratio: **0.830**

	$p_s = R L_e$ $p_s = (200 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 1.15 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.1249 \text{ kip/ft}^2)}{(1.15 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97816$	Status: PASS Ratio: 0.980
	<p>Considering z-direction:</p> <p>$H_o = 0.0022293 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0092357 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0092357 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.0092357 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.0636 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0092357 \text{ kipft/ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.0092357 \text{ kipft/ft})) + (2 \times (0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.0024175 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0092357 \text{ kipft/ft})) + ((0.0022293 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.0056783 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (200 \text{ psf/ft}) \times \frac{(4.0636 \text{ ft})}{2}$ $p_a = 0.40636 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0024175 \text{ kip/ft}^2)}{(0.40636 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0059491$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (200 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 1.15 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.010

$$Ratio = \frac{(0.0056783 \text{ kip/ft}^2)}{(1.15 \text{ kip/ft}^2)}$$

$$Ratio = 0.0049377$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.292 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36497 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.146 \text{ kipft}) + ((-2.292 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3927 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3927 \text{ kipft/ft})}{(-0.36497 \text{ kip/ft})}$$

$$E = 17.516 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3927 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.36497 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (6.3927 \text{ kipft/ft})) + (4 \times (-0.36497 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9194 \text{ ft}$$

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

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

$$V_{max} = ((-0.36497 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.516 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9194 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.516 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9194 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.8398 \text{ kip}$$

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

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

$$M_{max} = ((-0.36497 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(17.516 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.516 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (17.516 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9194 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 24.647 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.097 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015446 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015446 \text{ kipft/ft})}{(0.0038217 \text{ kip/ft})}$$

$$E = 4.0417 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015446 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.015446 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0666 \text{ ft}$$

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

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

$$V_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0666 \text{ ft})}{(5.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0666 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.029149 \text{ kip}$$

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

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

$$M_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.0417 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (4.0417 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0666 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.075925 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.403 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.084 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.084 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.403 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0057577$$

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.403 \text{ kip} \rightarrow 15403 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(15403 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

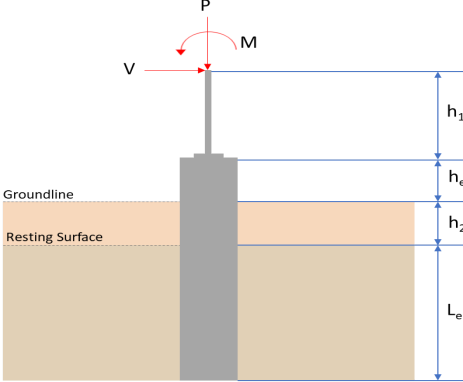
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.43 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.8398 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.8398 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.079329$ <p>Considering z-direction:</p> <p>$V_{max} = 0.029149 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.029149 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.00026159$	<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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 24.647 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.647 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.098746$	<p>Status: PASS Ratio: 0.100</p>
	<p>Considering z-direction: $M_{max} = 0.075925 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.075925 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00030419$	<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 $L = 6$ 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="517 1102 1093 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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</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>10.435</td> <td>16.145</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.409</td> <td>-2.355</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>24.016</td> <td>41.340</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	10.435	16.145	V_x (kip)	-1.409	-2.355	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	24.016	41.340	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	0.000	0.000																										
M_z (kipft)	24.016	41.340																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.409 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.22436 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(24.016 \text{ kipft}) + ((-1.409 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.8242 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 5.5739 \text{ ft}$ - 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> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.5739 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.574 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ <p>Ratio - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.574 \text{ ft})}{(6 \text{ ft})}$ $\text{Ratio} = 0.929$	<p>Status: PASS Ratio: 0.930</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> $q = \frac{P_v}{A}$ $q = \frac{(10.435 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.65219 \text{ kip/ft}^2$ <p>Check bearing capacity ratio: Ratio - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.65219 \text{ kip/ft}^2)}{(3000 \text{ psf})}$ $\text{Ratio} = 0.2174$	<p>Status: PASS Ratio: 0.220</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.22436$ kip/ft - Lateral force per length of pile,

$M_o = 3.8242$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8242 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.22436 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.8242 \text{ kipft/ft})) + (4 \times (-0.22436 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.095 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8242 \text{ kipft/ft})) + (3 \times (-0.22436 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.8242 \text{ kipft/ft})) + (2 \times (-0.22436 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.30074 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.8242 \text{ kipft/ft})) + ((-0.22436 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 1.0504 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.095 \text{ ft})}{2}$$

$$p_a = 0.4095 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30074 \text{ kip/ft}^2)}{(0.4095 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73441$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

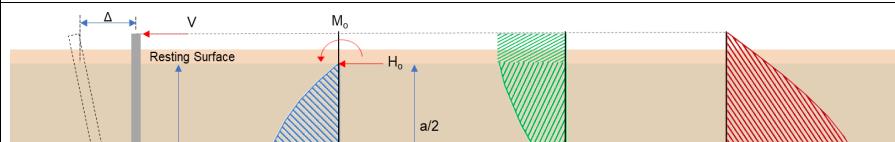
$$\text{Ratio} = \frac{s}{p_s}$$

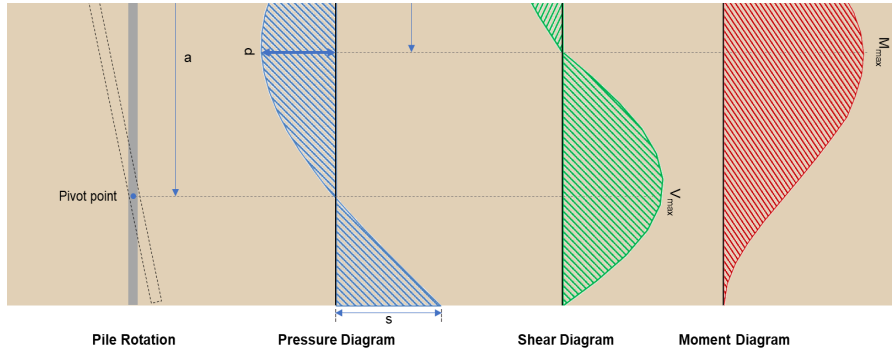
$$\text{Ratio} = \frac{(1.0504 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87531$$

Status: **PASS**
Ratio: **0.730**

Status: **PASS**
Ratio: **0.880**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.355 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.375 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.34 \text{ kipft}) + ((-2.355 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5828 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5828 \text{ kipft/ft})}{(-0.375 \text{ kip/ft})}$$

$$E = 17.554 \text{ ft}$$

a - Distance from resisting surface to pivot point,

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

$$a = \frac{(4 \times (6.5828 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.375 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.5828 \text{ kipft/ft})) + (4 \times (-0.375 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0928 \text{ ft}$$

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

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

$$V_{max} = ((-0.375 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.554 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0928 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.554 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0928 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.7619 \text{ kip}$$

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

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

$$M_{max} = ((-0.375 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(17.554 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0928 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.554 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0928 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.554 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0928 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.463 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(16.145 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.06 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.06 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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)**

Axial Compression Strength (ACI 318-19, LRFD)22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.145 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0060351$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.145 \text{ kip} \rightarrow 16145 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(16145 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.64 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.64 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.64 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.5 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.7619 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.7619 \text{ kip})}{(111.5 \text{ kip})}$ $\text{Ratio} = 0.078585$	<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:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$$\phi M_{n,2} = \phi F_y Z_{xk} S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,
 ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 25.463 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(25.463 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.10202$$

Status: **PASS**
Ratio: **0.100**