Project Details

Project Name: UK Agrivoltaics Preliminary design - 4x5 - V1Jb

Location: Boston, MA, USA

Unique ID: 2P-17-6TOP-HD-45-L-4Hx5W-4KDG

Dealer: ______________________________

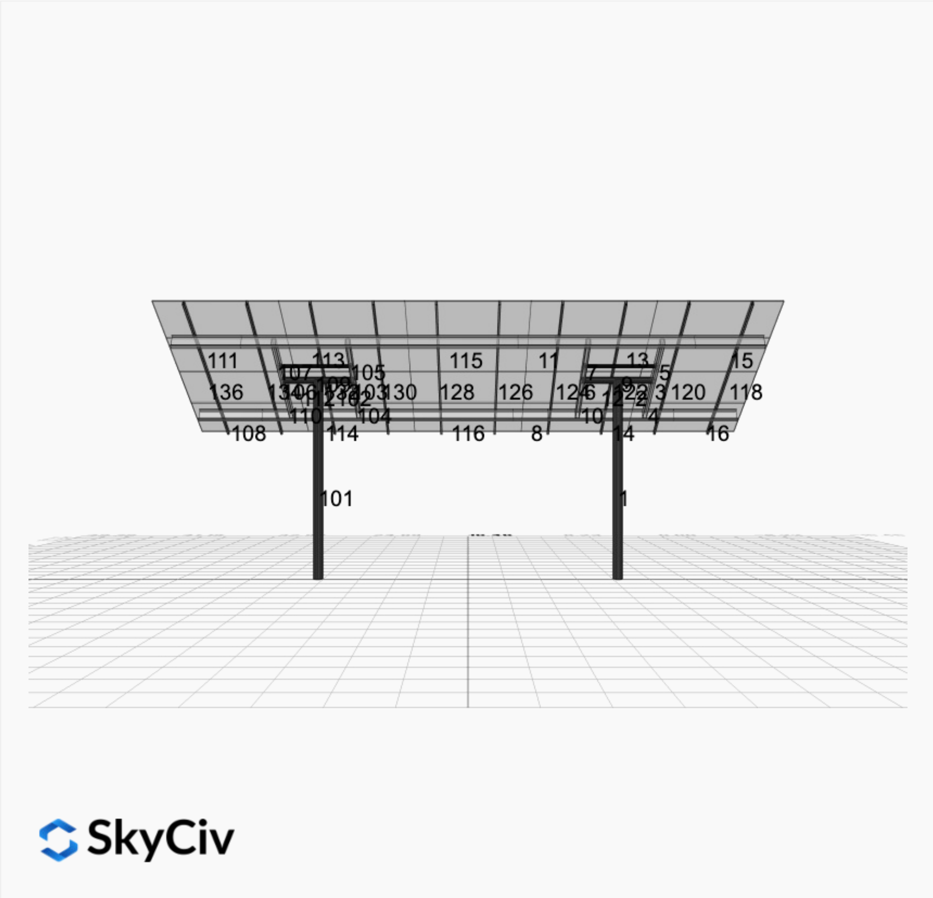
Date: Mon Dec 30 2024

Number of Modules: 20

Number of Poles: 2

Date Sold:

____



Array Dimensions N/S	13.17 ft
Array Dimensions E/W	32.92 ft
Winter Tilt Angle	30
Front Edge Clearance	8 ft

MT Solar Bill of Materials (2P-17-6TOP-HD-45-L-4Hx5W-4KDG)

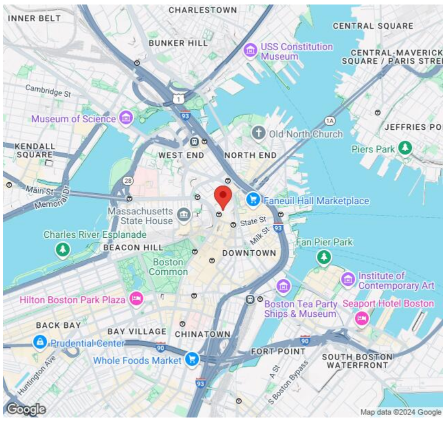
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-HD	H-Frame Assembly-HD	2
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-57	57IN HD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	5

Rail Bill of Materials

Part	Qty
Rails (156in)	10
Rail Attachment	20
Module Mid Clamp	30

Part	Qty
Module End Clamp	20
Ground Lug	5

Site Details:



Site Address: Boston, MA, USA

Array Specification

Duty Classification:	HD
Module Width:	39.00 in
Module Length:	78.00in
Number of Rows:	4
Number of Columns:	5
Total Number of Modules:	20
Winter Tilt Angle:	30
Front Edge Clearance:	8
Total Array Height at Tilt:	14.58 ft
Total Frame Length:	32.00 ft
Frame Weight:	2085 lbs
Array Dimensions N/S:	13.17 ft
Array Dimensions E/W:	32.92 ft
Rail Length:	158.00 in
Rail Spacing:	3.29 ft

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	11.29 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 6.75 ft
Foundation Volume:	8.000 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Boston, MA, USA
Wind Speed:	110 mph
Snow Load:	40 psf

Design Disclaimer

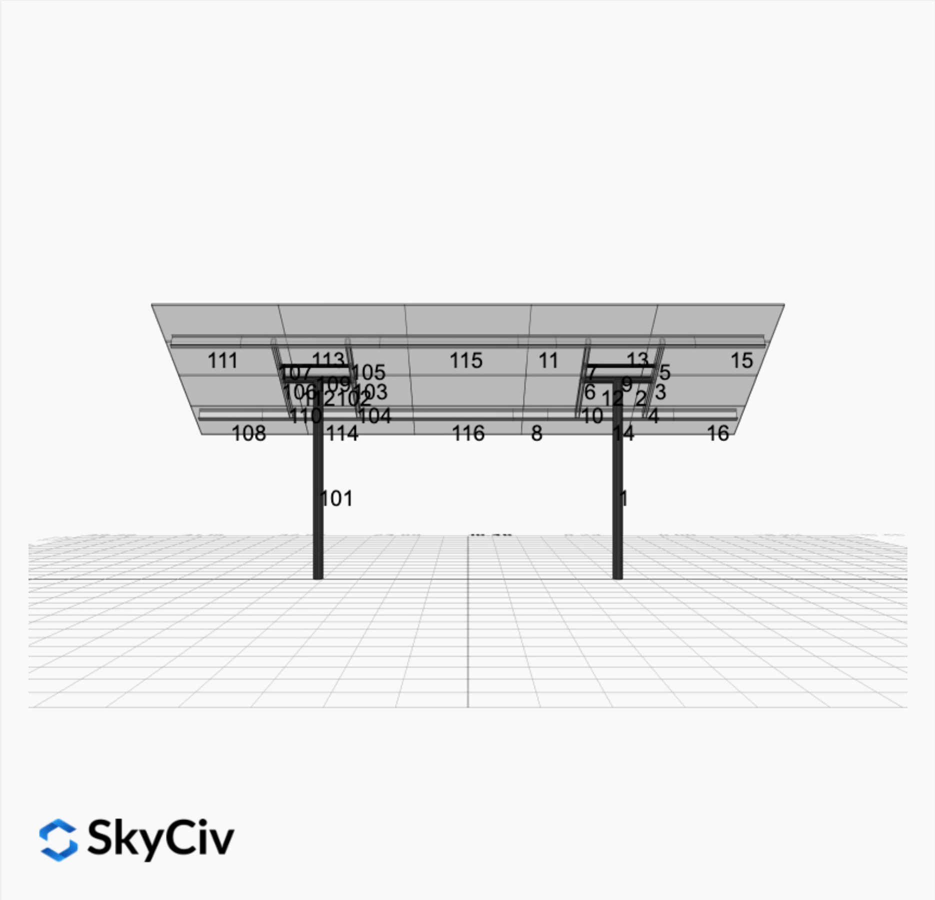
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

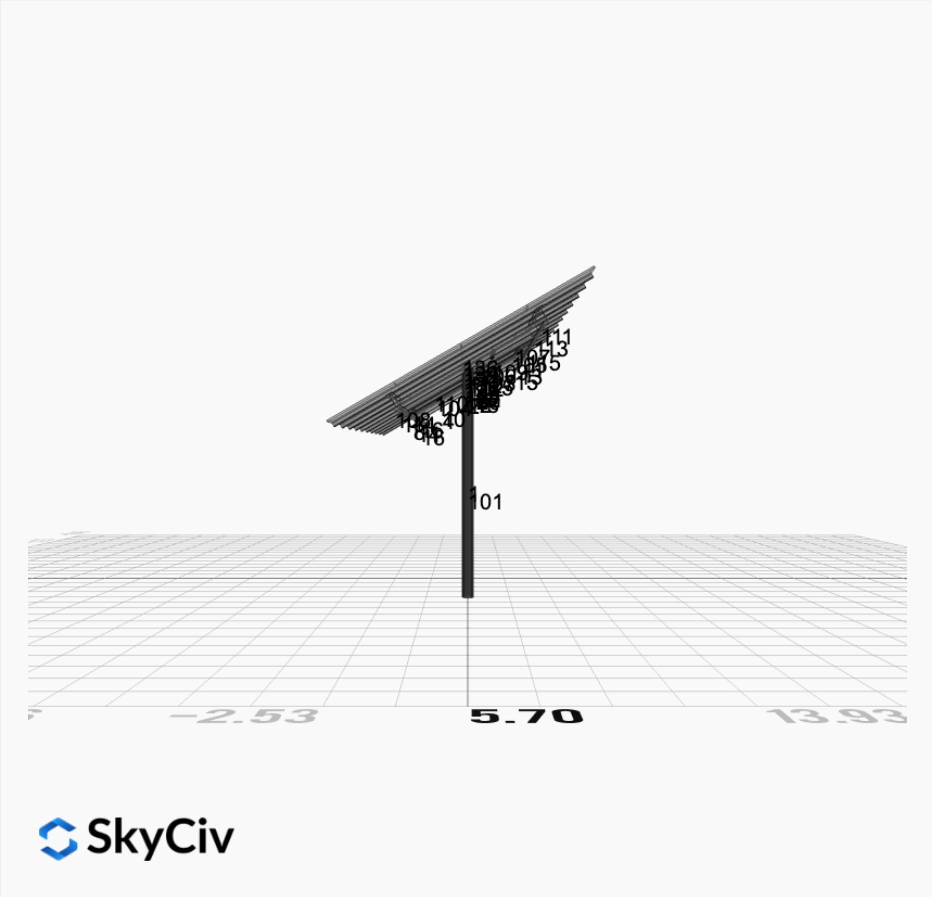
AutoDesigner Input


```
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"UK Agrivoltaics Preliminary design - 4x5 - V1Jb","site_address":"Boston, MA, USA","module_width":39,"module_length":78,"number_rows":"4","number_columns":5,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":30,"ground_clearance":8,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":true}
```

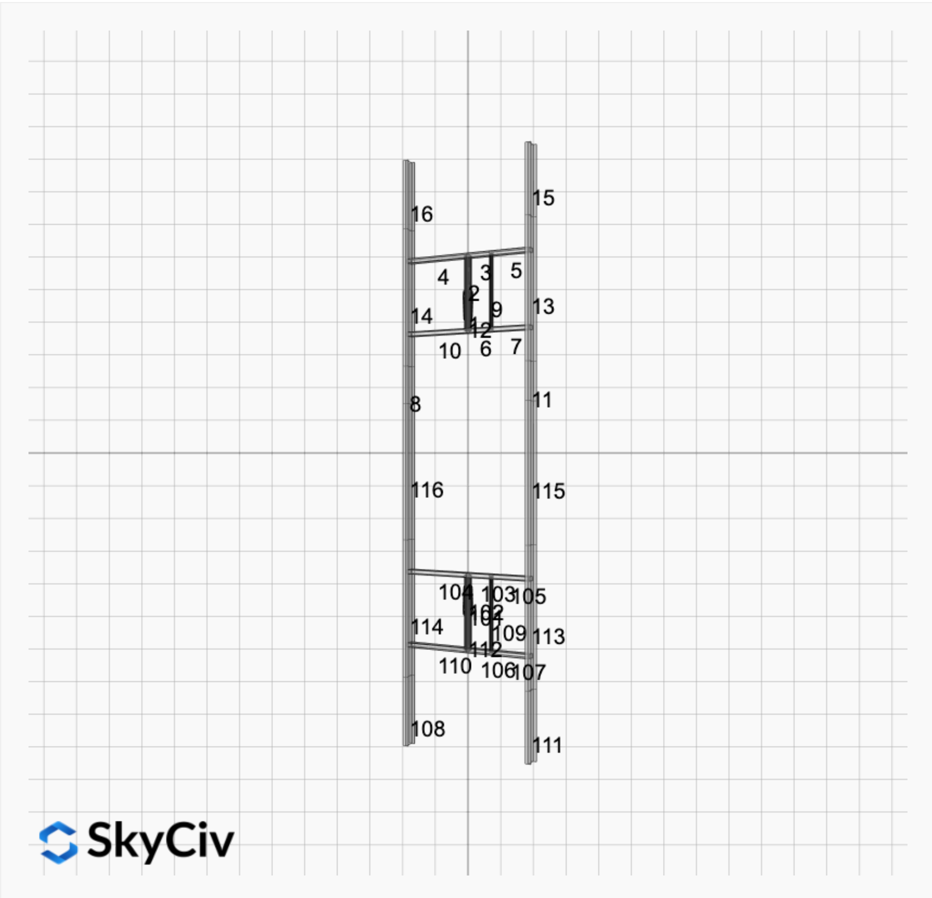

Design Notes:

- 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)

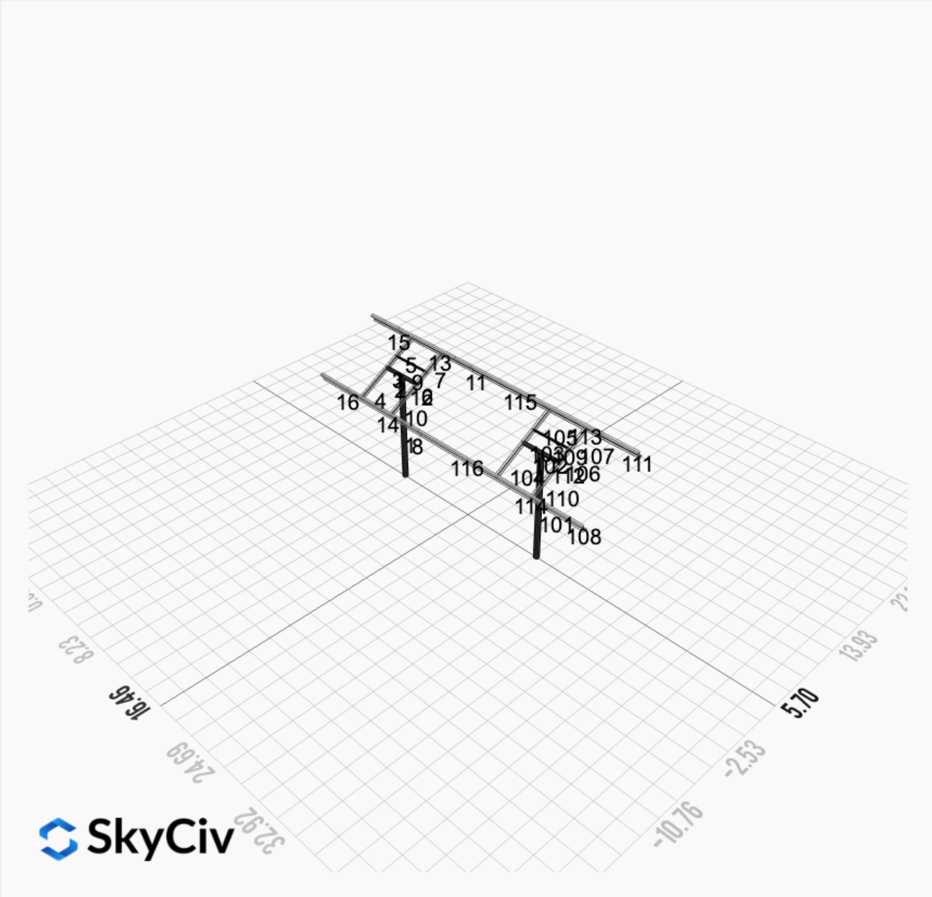


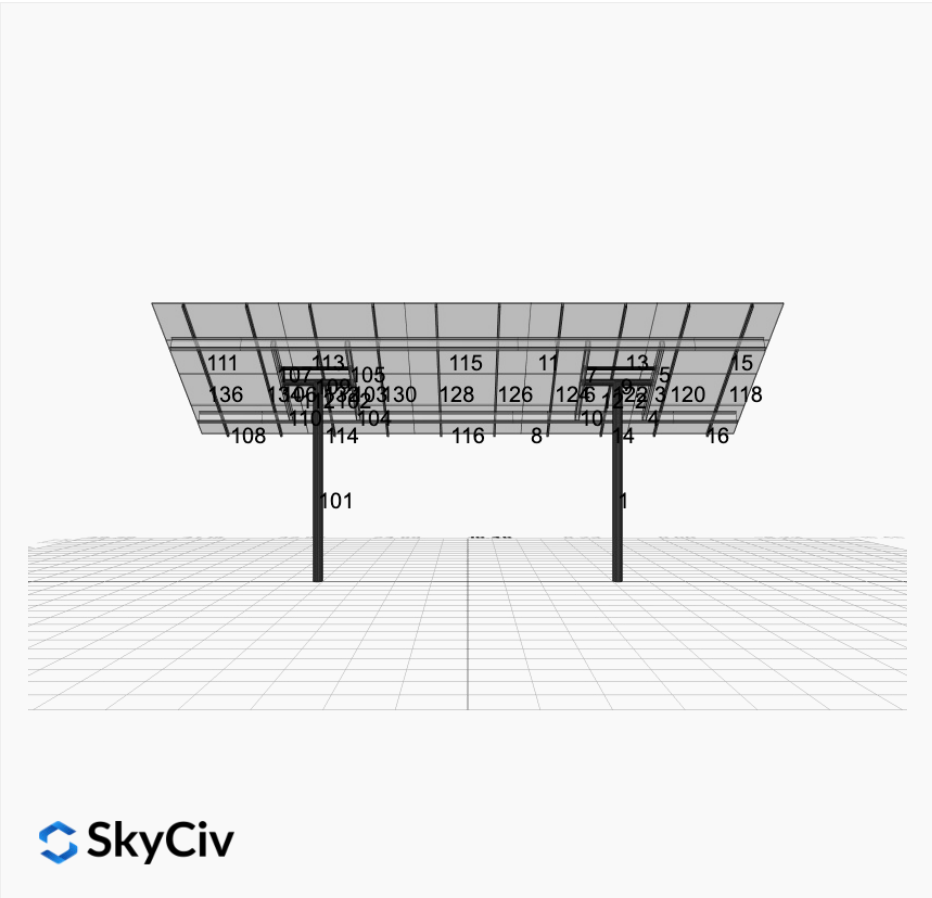


101
-2.53
5.70
13.93
SkyCiv



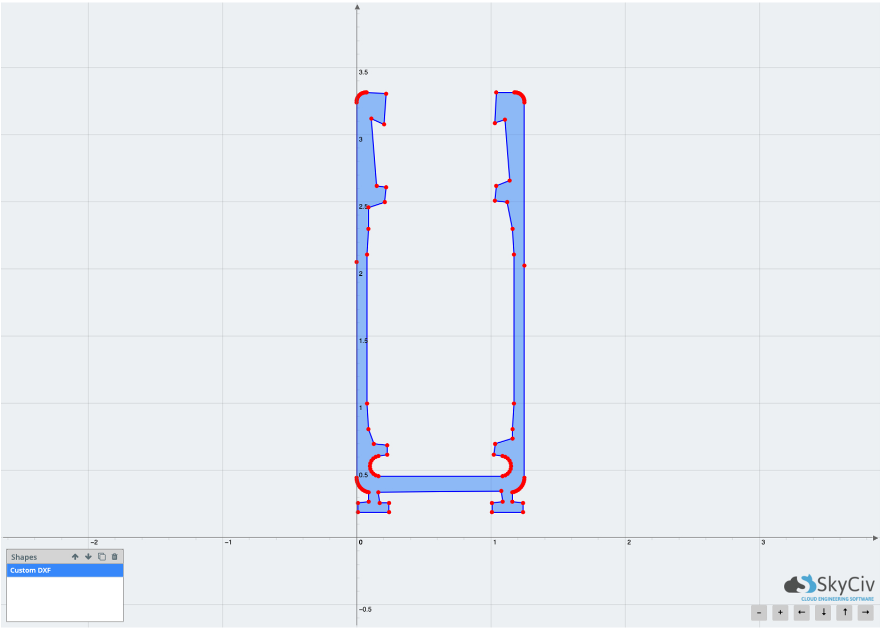
16
15
4
3
5
14
13
10
6
7
8
11
116
115
114
113
110
108
111
SkyCiv


8.23
16.46
24.69
32.92
5.70
-2.53
-10.76
13.93

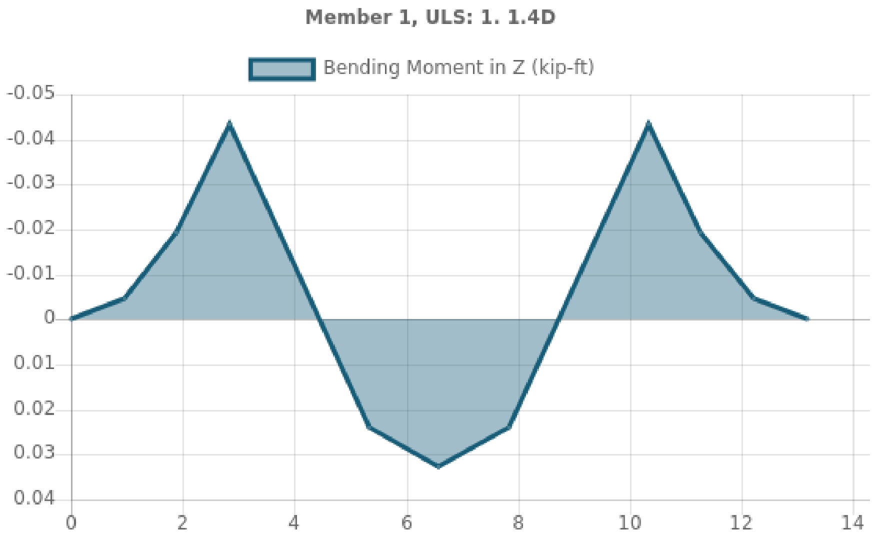


Rail Design Check

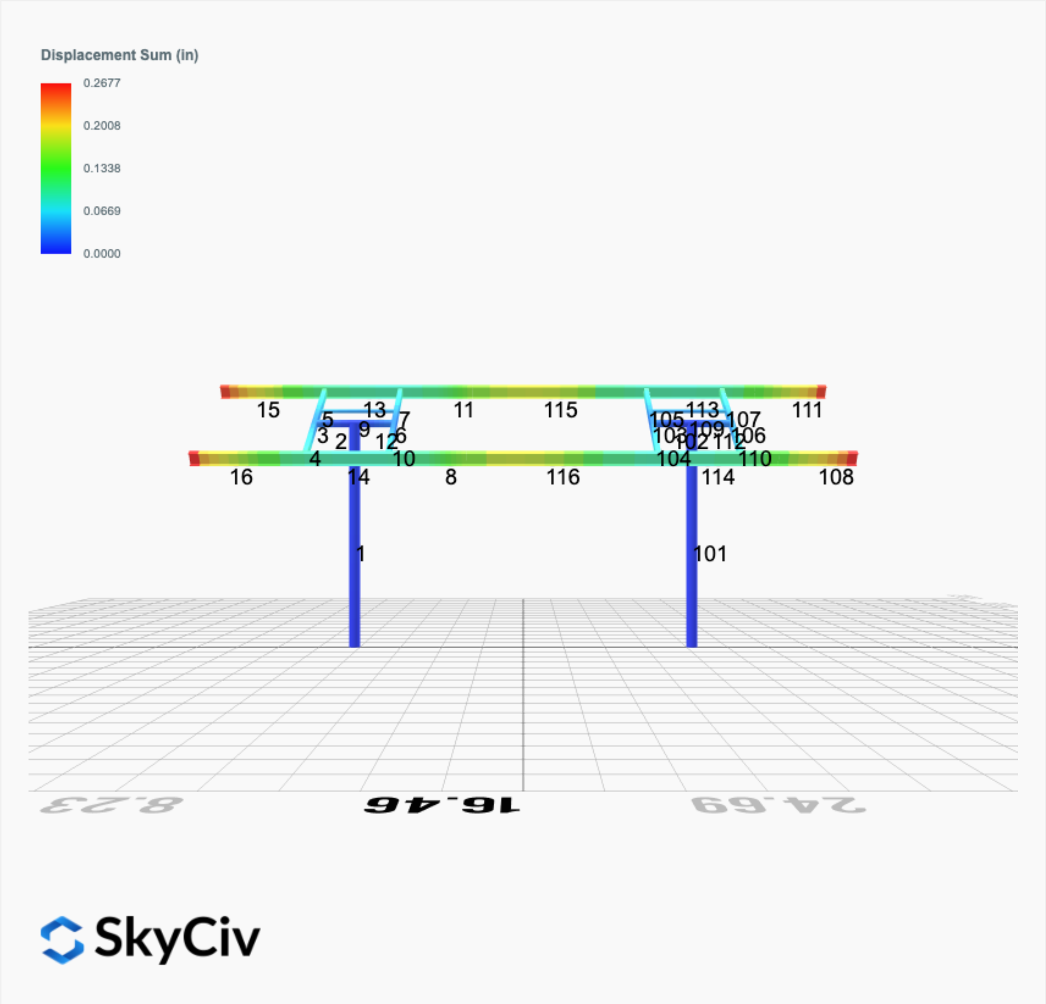
Rail Length: 13.166666666666666 ft
Additional Restraints Required: None
Tributary Width: 3.2916666666666665 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0502 kip/ft
Snow (Y): -0.0290 kip/ft
Wind uplift Case A: 0.1124 kip/ft
Wind uplift Case A: 0.1124 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1561 kip/ft

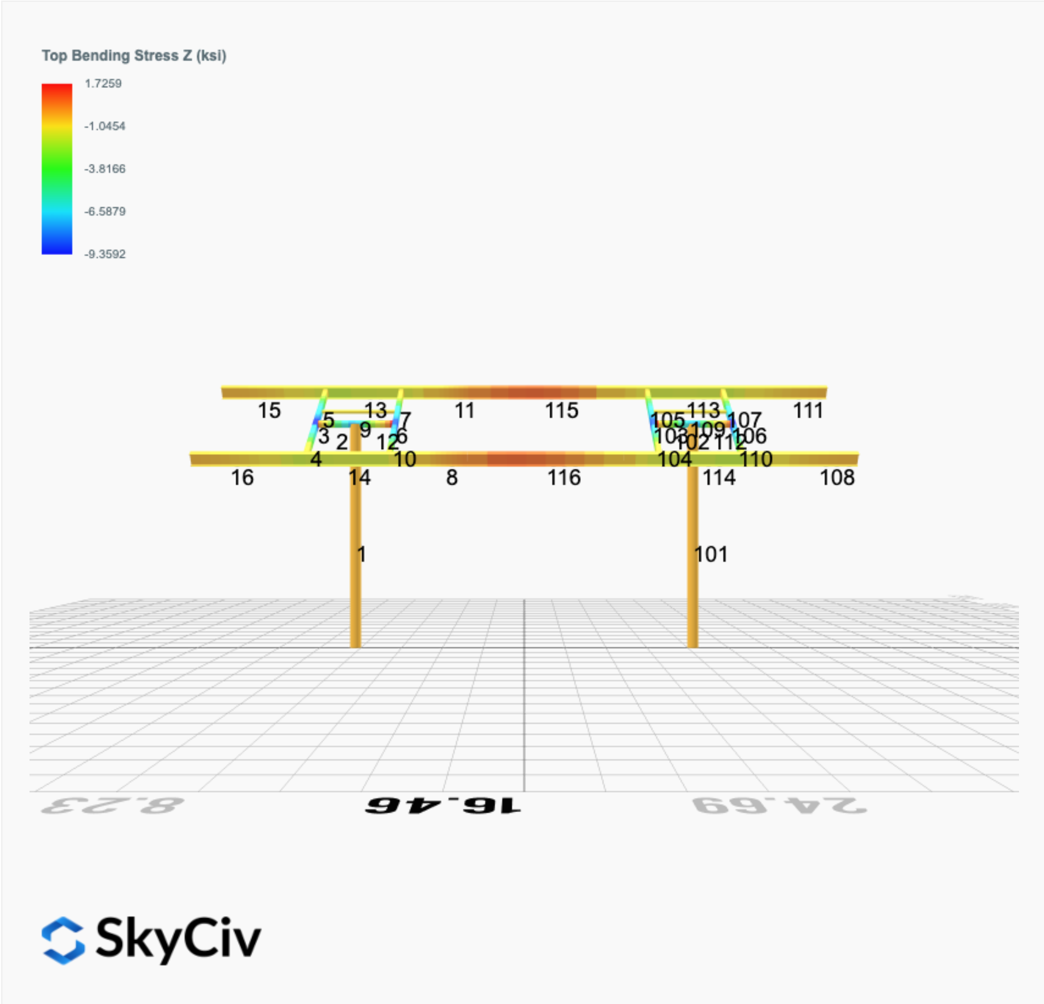


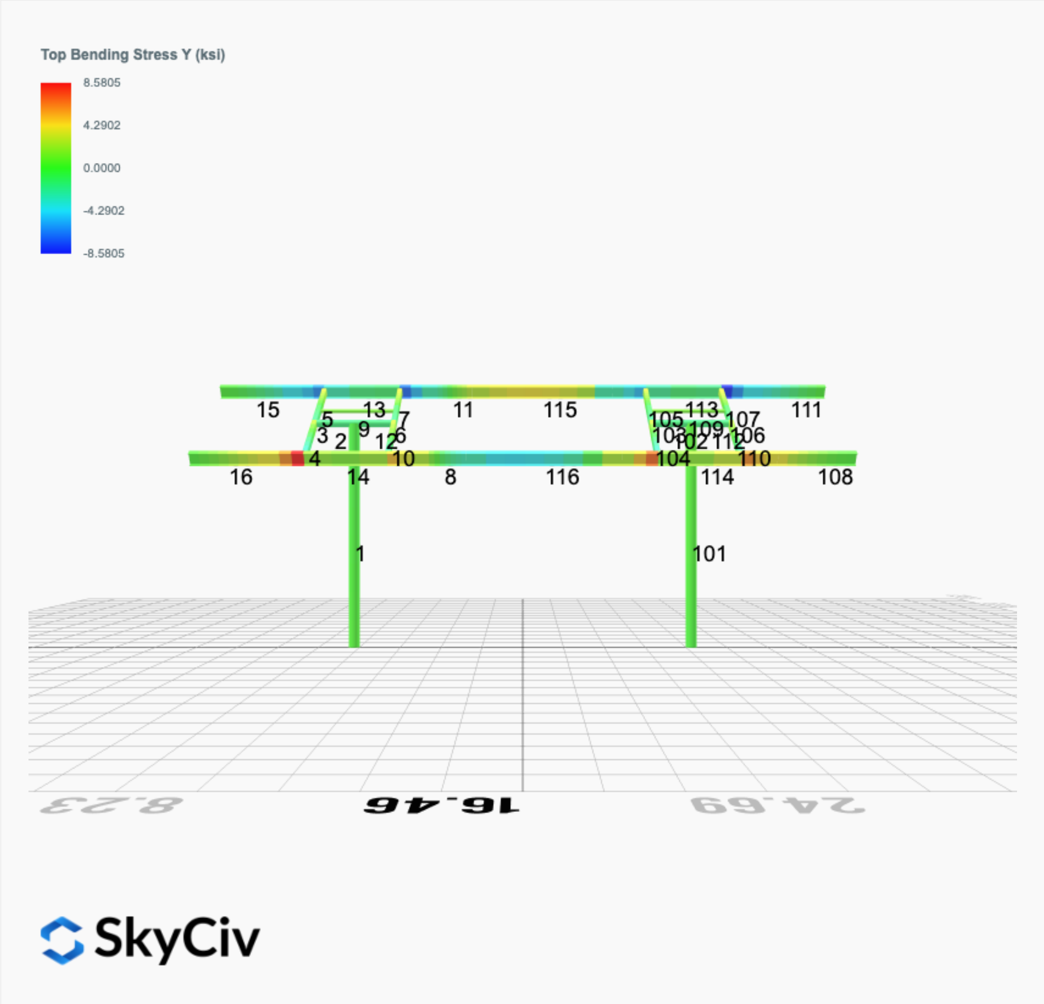
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	13.59131415	0.394	PASS
Material Yield	34.5	13.59131415	0.394	PASS
Material Strength	37	13.59131415	0.367	PASS

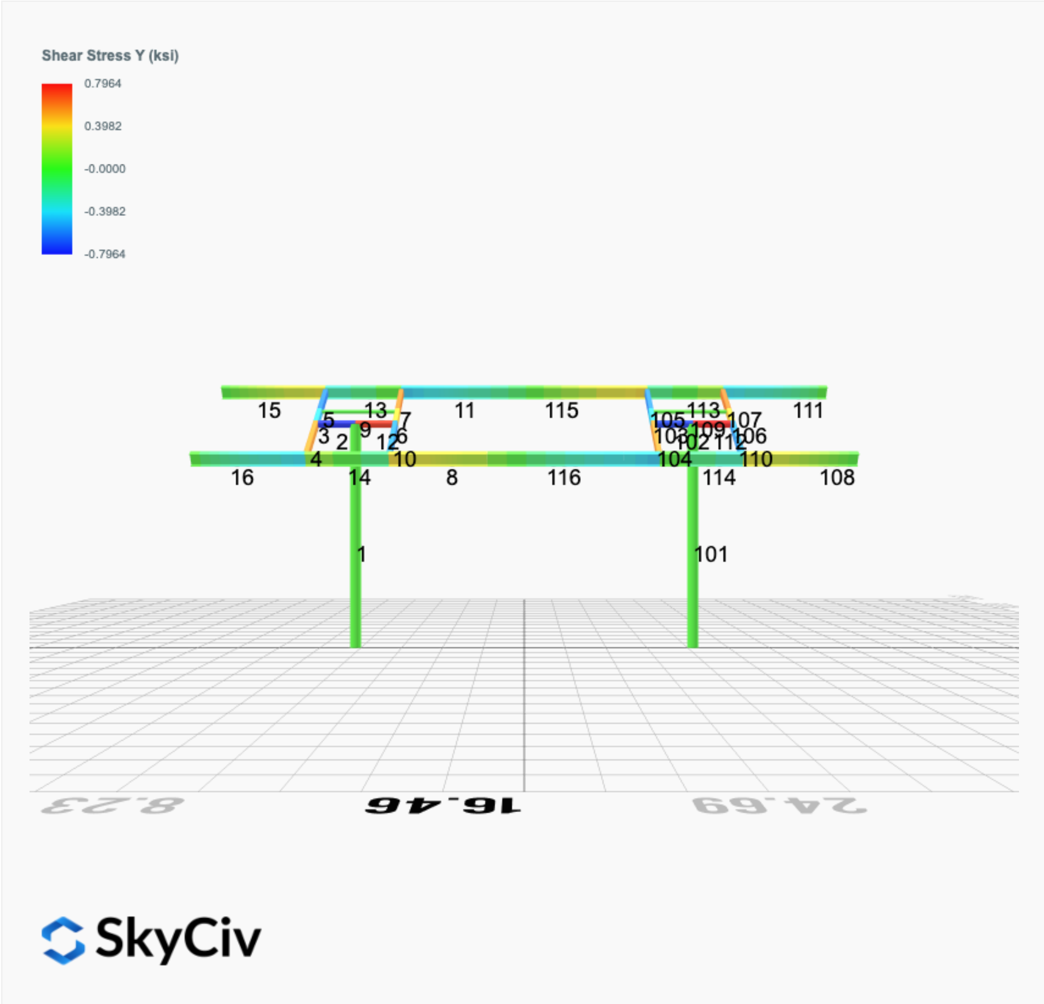


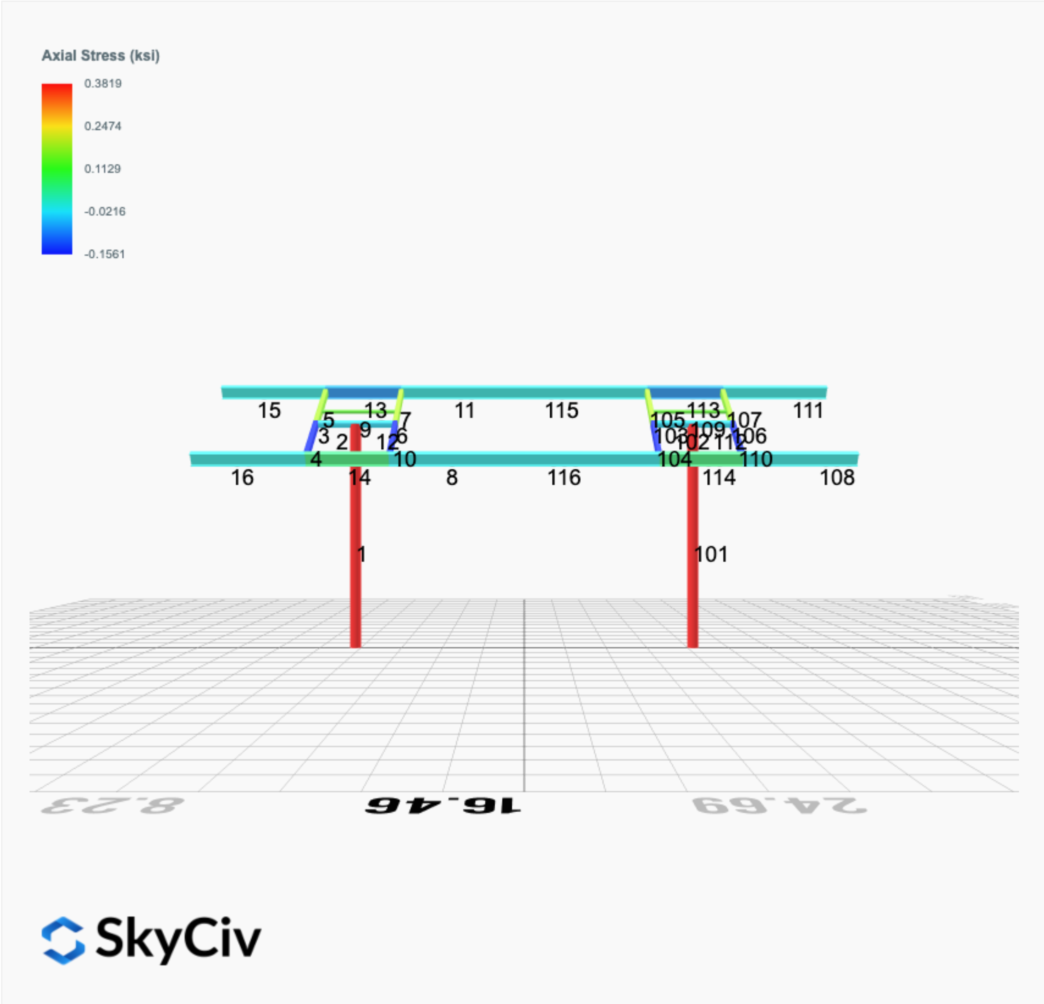
FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
1.7259
-1.0454
-3.8166
-6.5879
-9.3592
15
13
7
11
115
105
113
107
111
5
9
3
2
12
6
16
4
14
10
8
116
104
114
110
108
1
101
8.23
16.46
24.69
SkyCiv


Top Bending Stress Y (ksi)
8.5805
4.2902
0.0000
-4.2902
-8.5805
15
5
13
7
11
115
105
113
107
111
3
2
9
12
6
103
102
109
112
106
16
4
14
10
8
116
104
114
110
108
1
101
8.23
16.46
24.69
SkyCiv


Shear Stress Y (ksi)
0.7964
0.3982
-0.0000
-0.3982
-0.7964
15
13
11
115
113
111
16
14
8
116
114
108
1
101
8.23
16.46
24.69



Axial Stress (ksi)
0.3819
0.2474
0.1129
-0.0216
-0.1561
15
13
11
115
113
111
16
14
8
116
114
108
1
101
16.46
SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.8738	-0.0083	-0.0281	0.0228	0.0262
ULS: 2. D + L	0.0000	1.8738	-0.0083	-0.0281	0.0228	0.0262
ULS: 3. D + (S or Lr or R)	-0.0000	5.0838	-0.0277	-0.0938	0.0767	0.0379
ULS: 3. D + (S or Lr or R)	0.0000	1.8738	-0.0083	-0.0281	0.0228	0.0262
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.2813	-0.0229	-0.0774	0.0633	0.0349
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.8738	-0.0083	-0.0281	0.0228	0.0262
ULS: 5b. D + 0.7E	0.0000	1.8738	-0.0083	-0.0281	0.0228	0.0262
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.2813	-0.0229	-0.0774	0.0633	0.0349
ULS: 8. 0.6D + 0.7E	0.0000	1.1243	-0.0050	-0.0168	0.0137	0.0157
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5904	6.3606	-0.0488	-0.1682	0.0950	30.4726
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5904	6.3606	-0.0488	-0.1682	0.0950	30.4726
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2204	-1.9720	0.0263	0.0909	-0.0391	-24.2404
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8503	-1.3310	0.0207	0.0717	-0.0291	-29.1825
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9428	7.6464	-0.0533	-0.1825	0.1174	22.8698
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9428	7.6464	-0.0533	-0.1825	0.1174	22.8698
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6653	1.3969	0.0031	0.0118	0.0169	-18.1650
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3877	1.8777	-0.0011	-0.0026	0.0244	-21.8715
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9428	5.2389	-0.0387	-0.1332	0.0770	22.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9428	5.2389	-0.0387	-0.1332	0.0770	22.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6653	-1.0105	0.0177	0.0611	-0.0236	-18.1737
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3877	-0.5298	0.0135	0.0468	-0.0161	-21.8803
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5904	5.6111	-0.0455	-0.1570	0.0859	30.4622
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5904	5.6111	-0.0455	-0.1570	0.0859	30.4622
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2204	-2.7215	0.0296	0.1021	-0.0482	-24.2508
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8503	-2.0805	0.0241	0.0829	-0.0382	-29.1929

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.3316
Shear X	-4.3174
Shear Z	-0.0875
Moment X	-0.3016
Moment Y (Twist)	0.1748
Moment Z	51.9356

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	7.6464
Shear X	-2.5904
Shear Z	-0.0533
Moment X	-0.1825
Moment Y (Twist)	0.1174
Moment Z	30.4726

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.8738	0.0083	0.0281	-0.0228	0.0262
ULS: 2. D + L	-0.0000	1.8738	0.0083	0.0281	-0.0228	0.0262
ULS: 3. D + (S or Lr or R)	0.0000	5.0838	0.0277	0.0938	-0.0767	0.0379
ULS: 3. D + (S or Lr or R)	-0.0000	1.8738	0.0083	0.0281	-0.0228	0.0262
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.2813	0.0229	0.0774	-0.0633	0.0349

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.8738	0.0083	0.0281	-0.0228	0.0262
ULS: 5b. D + 0.7E	-0.0000	1.8738	0.0083	0.0281	-0.0228	0.0262
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.2813	0.0229	0.0774	-0.0633	0.0349
ULS: 8. 0.6D + 0.7E	-0.0000	1.1243	0.0050	0.0168	-0.0137	0.0157
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5904	6.3606	0.0488	0.1682	-0.0950	30.4726
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5904	6.3606	0.0488	0.1682	-0.0950	30.4726
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2204	-1.9720	-0.0263	-0.0909	0.0391	-24.2404
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8503	-1.3310	-0.0207	-0.0717	0.0291	-29.1825
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9428	7.6464	0.0533	0.1825	-0.1174	22.8698
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9428	7.6464	0.0533	0.1825	-0.1174	22.8698
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6653	1.3969	-0.0031	-0.0118	-0.0169	-18.1650
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3877	1.8777	0.0011	0.0026	-0.0244	-21.8715
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9428	5.2389	0.0387	0.1332	-0.0770	22.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9428	5.2389	0.0387	0.1332	-0.0770	22.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6653	-1.0105	-0.0177	-0.0611	0.0236	-18.1737
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3877	-0.5298	-0.0135	-0.0468	0.0161	-21.8803
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5904	5.6111	0.0455	0.1570	-0.0859	30.4622
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5904	5.6111	0.0455	0.1570	-0.0859	30.4622
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2204	-2.7215	-0.0296	-0.1021	0.0482	-24.2508
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8503	-2.0805	-0.0241	-0.0829	0.0382	-29.1929

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.3316
Shear X	-4.3174
Shear Z	0.0875
Moment X	0.3016
Moment Y (Twist)	0.1750
Moment Z	51.9364

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	7.6464
Shear X	-2.5904
Shear Z	0.0533
Moment X	0.1825
Moment Y (Twist)	0.1174
Moment Z	30.4726

Project Details

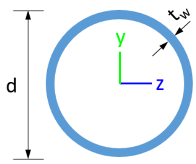
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

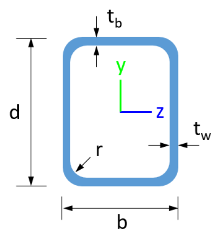
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

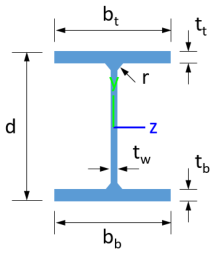
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
8	6in Pipe Sch 80	6.63	0.43					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	8	23.71	23.71	11.29	-	300	200	1
2	5	4.20	4.20	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.20,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.62,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.75,1.67,1.67,1.66,1.64	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.20,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
8	19	1.33	1.33	2.05	2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.03,2.02,2.02,2.02,1.95,2.02,2.02,2.03,2.02,2.02,2.02,2.02,2.03,2.02,2.02,2.02,1.97	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.71,1.67,1.67,1.66,1.61,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.75,1.67,1.67,1.66,1.64	300	200	1
11	19	1.33	1.33	2.05	2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.03,2.03,2.02,2.02,2.02,2.02,2.02,2.02,2.02,2.02	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.10,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12	300	200	1
14	19	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.12,3.03,1.12,1.12,1.12,1.10,1.12,1.12,1.11,1.12,1.12,1.12,1.12,2.82,1.12,1.12,1.12,1.10	300	200	1
15	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	8	23.71	23.71	11.29	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.20,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
104	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.71,1.67,1.67,1.66,1.61,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.75,1.67,1.67,1.66,1.64	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.20,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.70,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
108	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.62,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.75,1.67,1.67,1.66,1.64	300	200	1
111	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
112	5	4.20	4.20	2.00	-	300	200	1

113	19	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.10,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12	300	200	1
114	19	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.12,3.01,1.12,1.12,1.12,1.10,1.12,1.12,1.11,1.12,1.12,1.12,1.12,2.80,1.12,1.12,1.12,1.10	300	200	1
115	19	4.84	4.84	7.45	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	300	200	1
116	19	4.84	4.84	7.45	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.16,1.16,1.16,1.16	300	200	1

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	112.97	62.23	62.23	113.47	113.47
2	198.33	182.14	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	123.95	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	25.10	6.12	40.24	43.62
14	133.20	85.85	25.11	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	378.22	112.97	62.23	62.23	113.47	113.47
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	52.83	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	52.83	32.87	6.12	40.24	43.62
112	198.33	182.14	21.95	21.95	59.50	59.50
113	133.20	85.85	25.12	6.12	40.24	43.62
114	133.20	85.85	25.16	6.12	40.24	43.62
115	133.20	86.20	26.76	6.12	40.24	43.62
116	133.20	86.20	26.62	6.12	40.24	43.62

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.100	0.835	0.011	0.038	0.001	0.890	#13	0.648	Not Required	Pass
2	0.001	0.446	0.197	0.094	0.037	0.644	#13	0.114	Not Required	Pass
3	0.007	0.701	0.051	0.071	0.010	0.730	#13	0.045	Not Required	Pass
4	0.007	0.699	0.102	0.070	0.022	0.728	#13	0.080	Not Required	Pass

5	0.007	0.435	0.105	0.070	0.026	0.447	#13	0.074	Not Required	Pass
6	0.008	0.673	0.052	0.067	0.010	0.699	#13	0.045	Not Required	Pass
7	0.008	0.418	0.101	0.067	0.026	0.430	#13	0.074	Not Required	Pass
8	0.001	0.058	0.070	0.042	0.011	0.121	#21	0.095	Not Required	Pass
9	0.006	0.069	0.041	0.001	0.000	0.111	#13	0.204	Not Required	Pass
10	0.008	0.671	0.104	0.067	0.023	0.715	#13	0.080	Not Required	Pass
11	0.000	0.058	0.070	0.042	0.011	0.121	#21	0.095	Not Required	Pass
12	0.002	0.415	0.189	0.090	0.036	0.605	#13	0.035	Not Required	Pass
13	0.005	0.226	0.260	0.058	0.016	0.448	#21	0.286	Not Required	Pass
14	0.006	0.231	0.260	0.058	0.016	0.448	#21	0.190	Not Required	Pass
15	0.000	0.076	0.121	0.033	0.009	0.189	#21	Not Required	Not Required	Pass
16	0.000	0.076	0.121	0.033	0.009	0.189	#21	Not Required	Not Required	Pass
101	0.100	0.835	0.011	0.038	0.001	0.890	#13	0.648	Not Required	Pass
102	0.002	0.415	0.189	0.090	0.036	0.605	#13	0.035	Not Required	Pass
103	0.008	0.673	0.052	0.067	0.010	0.699	#13	0.045	Not Required	Pass
104	0.008	0.671	0.104	0.067	0.023	0.715	#13	0.080	Not Required	Pass
105	0.008	0.418	0.101	0.067	0.026	0.430	#13	0.074	Not Required	Pass
106	0.007	0.701	0.051	0.071	0.010	0.730	#13	0.045	Not Required	Pass
107	0.007	0.435	0.105	0.070	0.026	0.447	#13	0.074	Not Required	Pass
108	0.000	0.076	0.121	0.033	0.009	0.189	#21	Not Required	Not Required	Pass
109	0.006	0.069	0.041	0.001	0.000	0.111	#13	0.204	Not Required	Pass
110	0.007	0.699	0.102	0.070	0.022	0.728	#13	0.080	Not Required	Pass
111	0.000	0.076	0.121	0.033	0.009	0.189	#21	Not Required	Not Required	Pass
112	0.001	0.446	0.197	0.094	0.037	0.644	#13	0.114	Not Required	Pass
113	0.005	0.227	0.260	0.058	0.016	0.448	#21	0.190	Not Required	Pass
114	0.006	0.231	0.260	0.058	0.016	0.448	#21	0.286	Not Required	Pass
115	0.000	0.119	0.132	0.042	0.011	0.238	#21	0.346	Not Required	Pass
116	0.001	0.119	0.133	0.042	0.011	0.239	#21	0.346	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/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis

(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

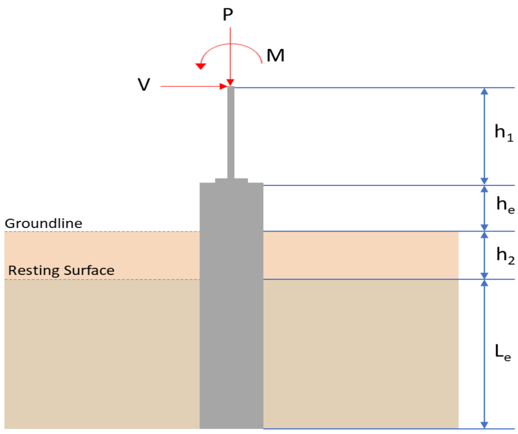
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 6.75$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.646	11.332
V_x (kip)	-2.590	-4.317
V_z (kip)	-0.053	-0.087
M_x (kipft)	-0.183	-0.302
M_z (kipft)	30.473	51.936

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \text{ft}) + (0\ \text{ft}) + (0\ \text{ft})$$

$$H = 0\ \text{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.59\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.41242\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.473 \text{ kipft}) + ((-2.59 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.8524 \text{ kipft/ft}$$

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:
$L_{e,x} = 6.1745 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.053 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0084395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.183 \text{ kipft}) + ((-0.053 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.02914 \text{ kipft/ft}$$

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:
$L_{e,z} = 1.1991 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.1745 \text{ ft}), (1.1991 \text{ ft})]$$

$$L_{e,req} = 6.175 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.175 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.91481$$

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

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

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

$$q = \frac{(7.646\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.47788\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.47788\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.23894$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.41242\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.8524\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8524\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.41242\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (4.8524\ \text{kipft/ft})) + (4 \times (-0.41242\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6556\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.8524\ \text{kipft/ft})) + (3 \times (-0.41242\ \text{kip/ft}) \times (6.75\ \text{ft}))]^{\ 2}}{(6.75\ \text{ft})^2 \times [(3 \times (4.8524\ \text{kipft/ft})) + (2 \times (-0.41242\ \text{kip/ft}) \times (6.75\ \text{ft}))]}$$

$$p = 0.22391\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.8524\ \text{kipft/ft})) + ((-0.41242\ \text{kip/ft}) \times (6.75\ \text{ft}))]}{(6.75\ \text{ft})^2}$$

$$s = 0.9114\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.6556\ \text{ft})}{2}$$

$$p_a = 0.34917\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22391\ \mathrm{kip/ft^2}\right)}{\left(0.34917\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.64126$$

Status: **PASS**
Ratio: **0.640**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.9114\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.90015$$

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.0084395\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.02914\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.02914\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.0084395\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.02914\ \mathrm{kipft/ft})) + (4 \times (-0.0084395\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8183\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.02914\ \mathrm{kipft/ft})) + (3 \times (-0.0084395\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^{\ 2}}{(6.75\ \mathrm{ft})^2 \times [(3 \times (0.02914\ \mathrm{kipft/ft})) + (2 \times (-0.0084395\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = -0.0018333\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.02914\ \mathrm{kipft/ft})) + ((-0.0084395\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.000173\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.8183\ \mathrm{ft})}{2}$$

$$p_a = 0.36137\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0018333\ \mathrm{kip/ft^2}\right)}{\left(0.36137\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.0050731$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

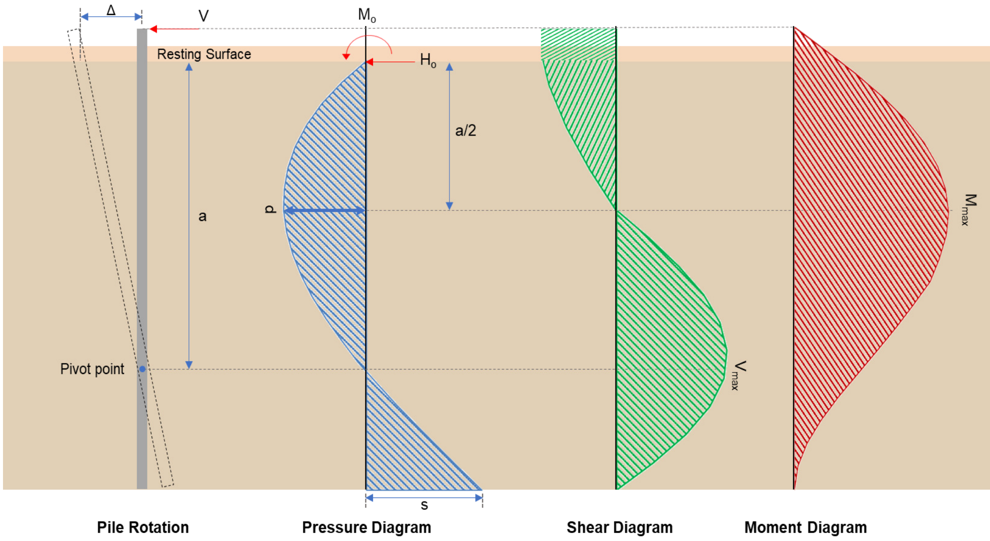
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.000173\ \mathrm{kip/ft^2})}{(1.0125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.00017086$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.317\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.68742\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(51.936\ \mathrm{kipft}) + ((-4.317\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.2701\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.2701\ \mathrm{kipft/ft})}{(-0.68742\ \mathrm{kip/ft})}$$

$$E = 12.031\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.2701\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.68742\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.2701\ \text{kipft/ft})) + (4 \times (-0.68742\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6531\ \text{ft}$$

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

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

$$V_{max} = ((-0.68742\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.031\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6531\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.031\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6531\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.486\ \text{kip}$$

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

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

$$M_{max} = ((-0.68742\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(12.031\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.6531\ \text{ft})}{2 \times (6.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (12.031\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6531\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.031\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6531\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 33.704\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.087\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.013854\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.302\ \text{kipft}) + ((-0.087\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.048089\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.048089\ \text{kipft/ft})}{(-0.013854\ \text{kip/ft})}$$

$$E = 3.4713\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.048089\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.013854\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (0.048089\ \text{kipft/ft})) + (4 \times (-0.013854\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.8175\ \text{ft}$$

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

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

$$V_{max} = ((-0.013854 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8175 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8175 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.087331 \text{ kip}$$

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

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

$$M_{max} = ((-0.013854 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.4713 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.26137 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.332 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.22 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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

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

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

$$Ratio = 0.96556$$

Status: PASS
Ratio: 0.970

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.332\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.004236$$

Status: PASS
Ratio: 0.000

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

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

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

$$\lambda_s = 0.64282$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.332 \text{ kip} \rightarrow 11332 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11332 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8\,\sqrt{f'_{ck}}\,b_w\,d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\,d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\,A_v\,f_{ywk}\,d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$

$$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\,\left(V_c + V_s\right)$$

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.486 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

	$$Ratio = \frac{(10.486\ \mathrm{kip})}{(111.08\ \mathrm{kip})}$$ $$Ratio = 0.0944$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $V_{max} = 0.087331\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.087331\ \mathrm{kip})}{(111.08\ \mathrm{kip})}$$ $$Ratio = 0.00078621$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f_{ck}'\ S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\ \mathrm{kipft}), (2121.6\ \mathrm{kipft})]$$ $$\phi M_n = 249.6\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 33.704\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(33.704\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.13503$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 0.26137\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(0.26137\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.0010471$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

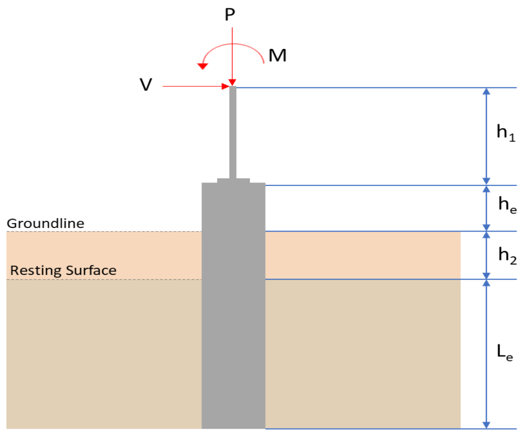
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.75\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.646	11.332
V_x (kip)	-2.590	-4.317
V_z (kip)	0.053	0.087
M_x (kipft)	0.183	0.302
M_z (kipft)	30.473	51.936

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.59\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.41242\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.473\ \text{kipft}) + ((-2.59\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.8524\ \text{kipft/ft}$$

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:
$L_{e,x} = 6.1745\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.053\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.0084395\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.183\ \text{kipft}) + ((0.053\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.02914\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.4529\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.1745\ \text{ft}), (1.4529\ \text{ft})]$$

$$L_{e,req} = 6.175\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.175\ \text{ft})}{(6.75\ \text{ft})}$$

$$Ratio = 0.91481$$

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

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

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

$$q = \frac{(7.646 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47788 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.47788 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.23894$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.41242 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.8524 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8524 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.41242 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (4.8524 \text{ kipft/ft})) + (4 \times (-0.41242 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6556 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.8524 \text{ kipft/ft})) + (3 \times (-0.41242 \text{ kip/ft}) \times (6.75 \text{ ft})]^{\;2}}{(6.75 \text{ ft})^2 \times [(3 \times (4.8524 \text{ kipft/ft})) + (2 \times (-0.41242 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.22391 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.8524 \text{ kipft/ft})) + ((-0.41242 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.9114 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6556 \text{ ft})}{2}$$

$$p_a = 0.34917 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22391 \text{ kip/ft}^2\right)}{\left(0.34917 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.64126$$

Status: **PASS**
Ratio: **0.640**

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

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

$$Ratio = \frac{\left(0.9114 \text{ kip/ft}^2\right)}{\left(1.0125 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.90015$$

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.0084395 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.02914 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.02914 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (0.0084395 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.02914 \text{ kipft/ft})) + (4 \times (0.0084395 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8183 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.02914 \text{ kipft/ft})) + (3 \times (0.0084395 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.02914 \text{ kipft/ft})) + (2 \times (0.0084395 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.0067554 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.02914 \text{ kipft/ft})) + ((0.0084395 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.015177 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8183 \text{ ft})}{2}$$

$$p_a = 0.36137 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0067554 \text{ kip/ft}^2\right)}{\left(0.36137 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.018694$$

Status: **PASS**
Ratio: **0.020**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

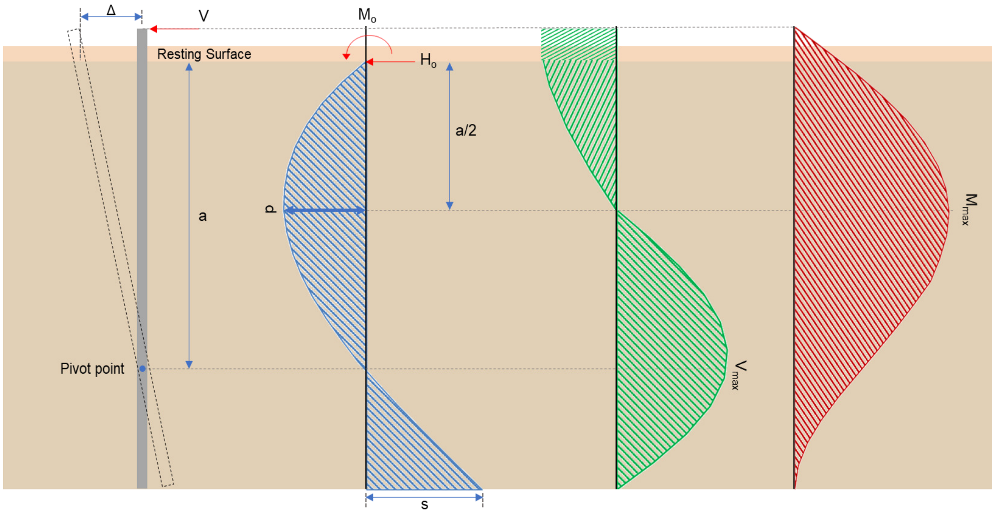
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.015177 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.014989$$

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



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.317 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68742 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(51.936 \text{ kipft}) + ((-4.317 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.2701 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.2701 \text{ kipft/ft})}{(-0.68742 \text{ kip/ft})}$$

$$E = 12.031 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.2701 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.68742 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.2701\ \mathrm{kipft/ft})) + (4 \times (-0.68742\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6531\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.68742\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.031\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6531\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.031\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6531\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.486\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.68742\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.75\ \mathrm{ft})) \times \left[\left(\frac{(12.031\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + \frac{(4.6531\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (12.031\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6531\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.031\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6531\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 33.704\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.087\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.013854\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.302\ \mathrm{kipft}) + ((0.087\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.048089\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.048089\ \mathrm{kipft/ft})}{(0.013854\ \mathrm{kip/ft})}$$

$$E = 3.4713\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.048089\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (0.013854\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.048089\ \mathrm{kipft/ft})) + (4 \times (0.013854\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8175\ \mathrm{ft}$$

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

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

$$V_{max} = ((0.013854 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8175 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8175 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.087331 \text{ kip}$$

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

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

$$M_{max} = ((0.013854 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.4713 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.4713 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8175 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.26137 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.332 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.22 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max[(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

	$$Ratio = 0.96556$$	Status: **PASS** Ratio: **0.970**
25.2.3	s_{rebar} - Minimum spacing of reinforcement, $$s_{rebar} = Max\left[1.5, \left(1.5\, d_{bar}\right)\right]$$ $$s_{rebar} = Max\left[1.5, \left(1.5 \times (0.625 \text{ in})\right)\right]$$ $$s_{rebar} = 1.5 \text{ in}$$ **Ties:**	
25.7.2.2 25.7.2.1	Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) s_{ties} - Maximum spacing of ties, $$s_{ties} = Min\left[\left(16\, d_{bar}\right), \left(48\, d_{ties}\right), Min\left(D, b\right)\right]$$ $$s_{ties} = Min\left[\left(16 \times (0.625 \text{ in})\right), \left(48 \times (0.375 \text{ in})\right), Min\left((48 \text{ in}), (48 \text{ in})\right)\right]$$ $$s_{ties} = 10 \text{ in}$$ **Summary:** Main reinforcement: **14 - #5 (0.625 in)** Ties: **#3(0.375 in) - 10 in**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5 \text{ ksi}) \times \left[\left(2304 \text{ in}^2\right) - \left(4.2951 \text{ in}^2\right)\right]\right) + \left((60 \text{ ksi}) \times \left(4.2951 \text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2 \text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(11.332 \text{ kip})}{(2675.2 \text{ kip})}$$ $$Ratio = 0.004236$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 22.5.5.1.3 22.5.5.1.1	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48 \text{ in})$$ $$d = 38.4 \text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ 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 $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$	

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.332 \text{ kip} \rightarrow 11332 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11332 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8\,\sqrt{f'_{ck}}\,b_w\,d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\,d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2\,A_v\,f_{ywk}\,d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$

$$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\,\left(V_c + V_s\right)$$

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.486 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

	$$Ratio = \frac{(10.486 \text{ kip})}{(111.08 \text{ kip})}$$ $$Ratio = 0.0944$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $V_{max} = 0.087331 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.087331 \text{ kip})}{(111.08 \text{ kip})}$$ $$Ratio = 0.00078621$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$ $$S_m = 18432 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 33.704 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(33.704 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.13503$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 0.26137 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(0.26137 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0010471$$	Status: **PASS** Ratio: **0.000**