Your Project Calculations

Project Name: MTSOLAR_3BC9H21G0B846

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MTSOLAR_3BC9H21G0B846&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/3_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=nmtJjuWwTuqQJJvBkFvZuwYZYtzOh7x6lj4KlRbRuUCN686fFPDFAFSPa4WTjdBj

Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-SD-45-L-4Hx7W-7CD3
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	92.00in
Number of Rows:	4
Number of Columns:	7
Total Number of Modules:	28
Desired Tilt Angle:	60
Front Edge Clearance:	5
Total Array Height at Tilt:	16.62 ft
Total Frame Length:	54.50 ft
Frame Weight:	2369 lbs
Array Dimensions N/S:	13.50 ft
Array Dimensions E/W:	54.25 ft
Rail Length:	162.00 in
Rail Spacing:	3.83 ft
Rail Check:	Not Checked

Support Specifications

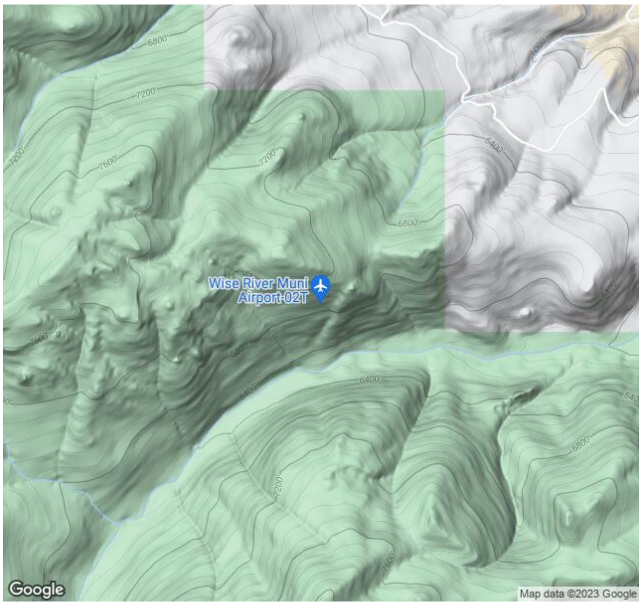
Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	10.85 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.50 ft Pile 3: 6.25 ft
Foundation Volume:	11.259 y^3
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Private Property near, Wise River, MT 59762, USA
Wind Speed:	99 mph
Snow Load:	27 psf
Design Uplift Pressure:	0.019233 ksf
Design Downforce Pressure:	-0.019233 ksf
Design Snow Pressure:	0.001485 ksf



Design Disclaimer

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

AutoDesigner Input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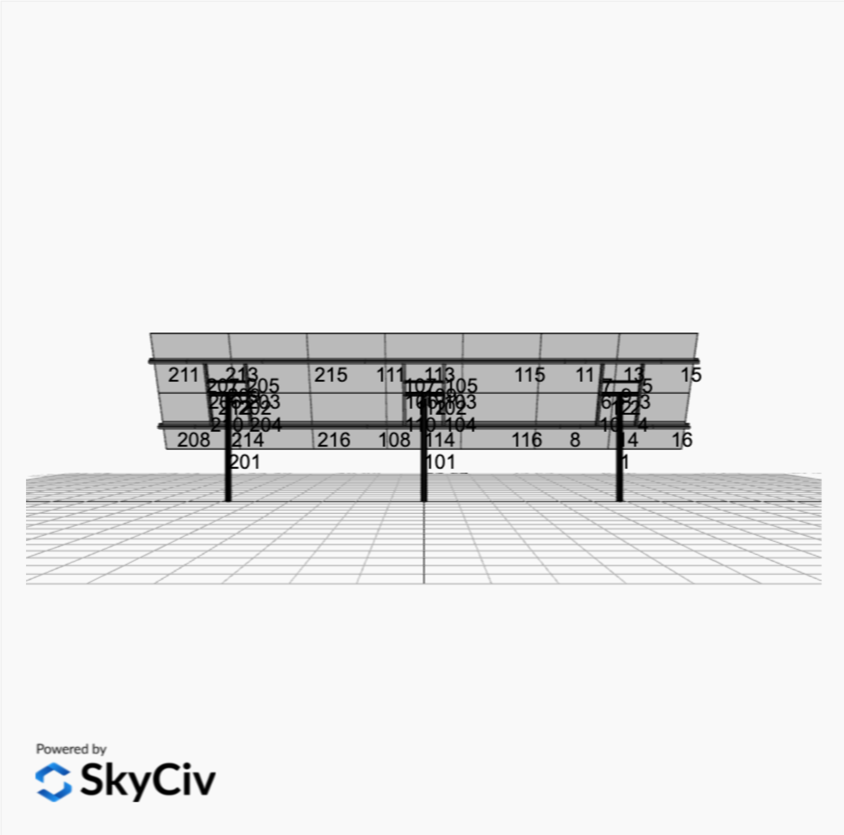
Design Notes:

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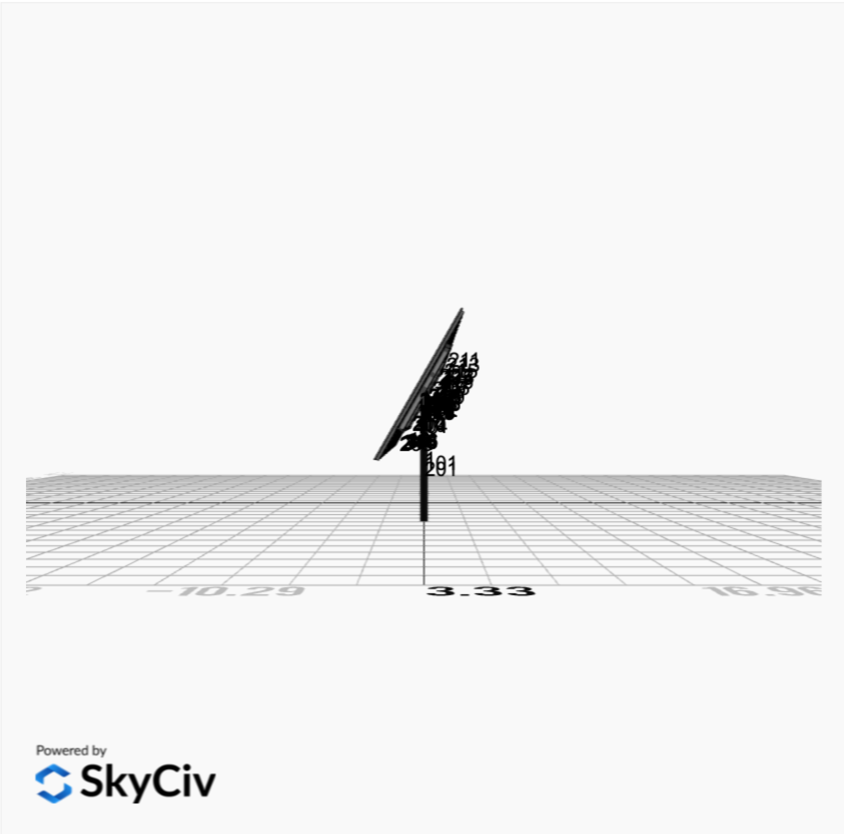
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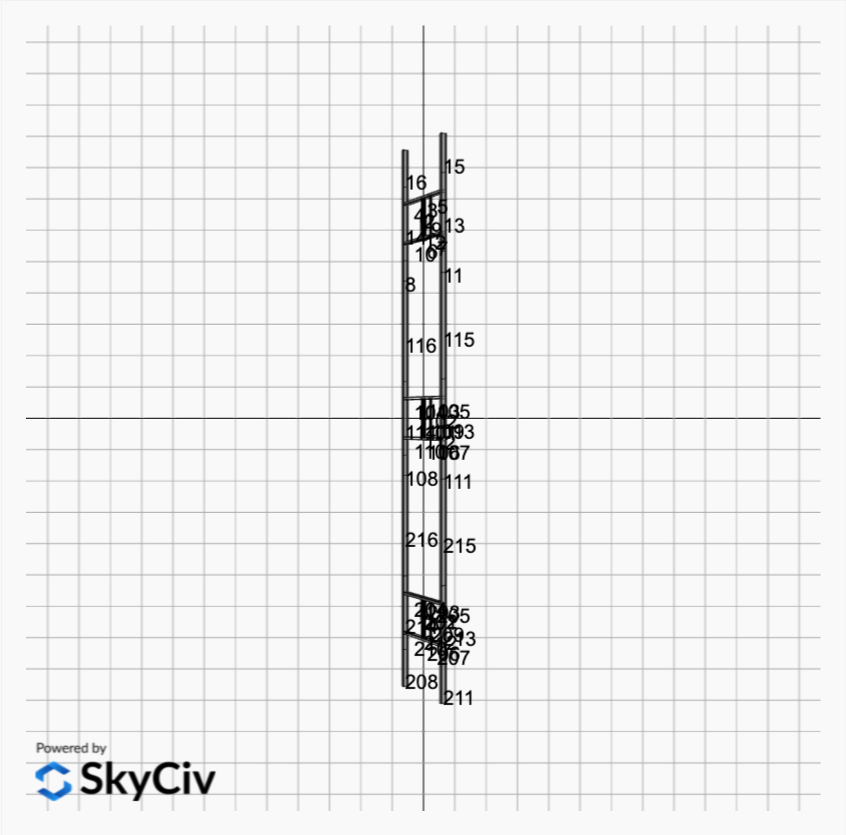
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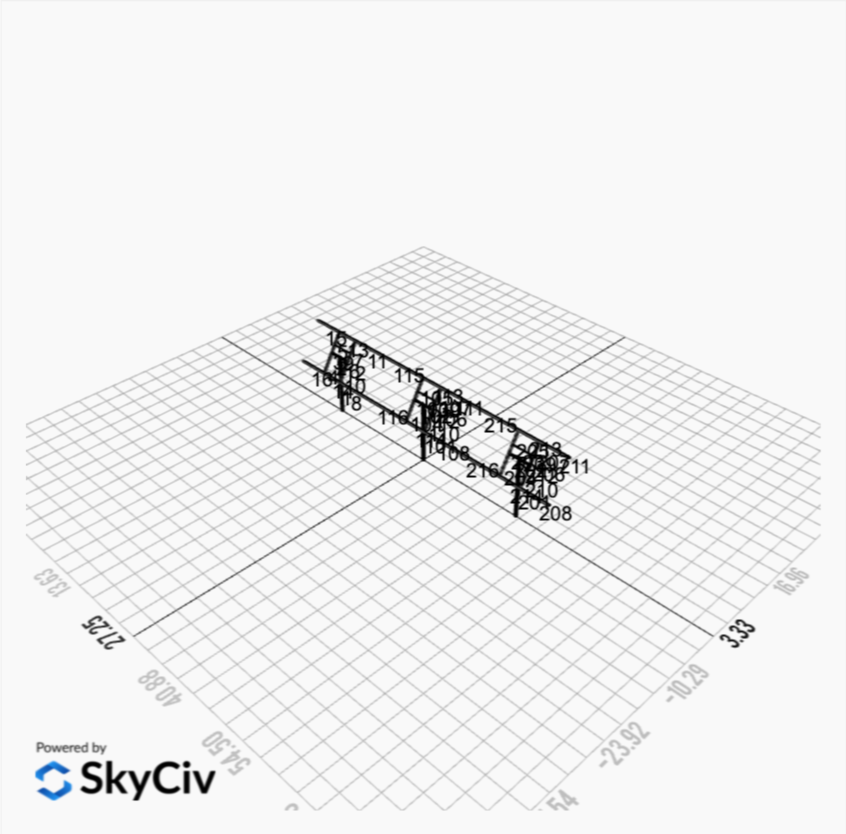
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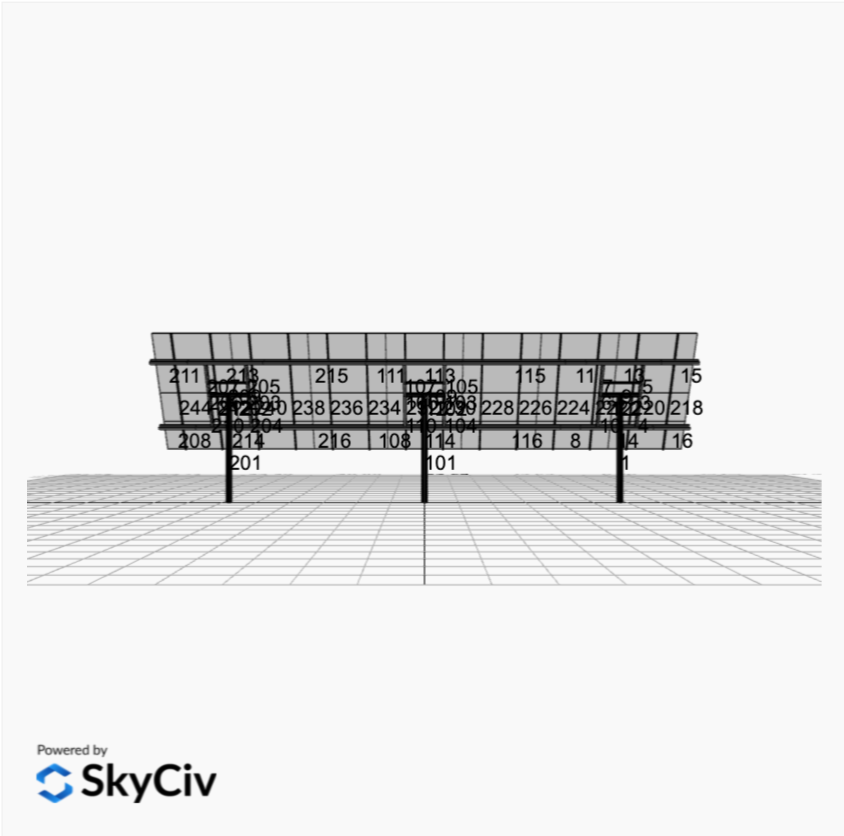
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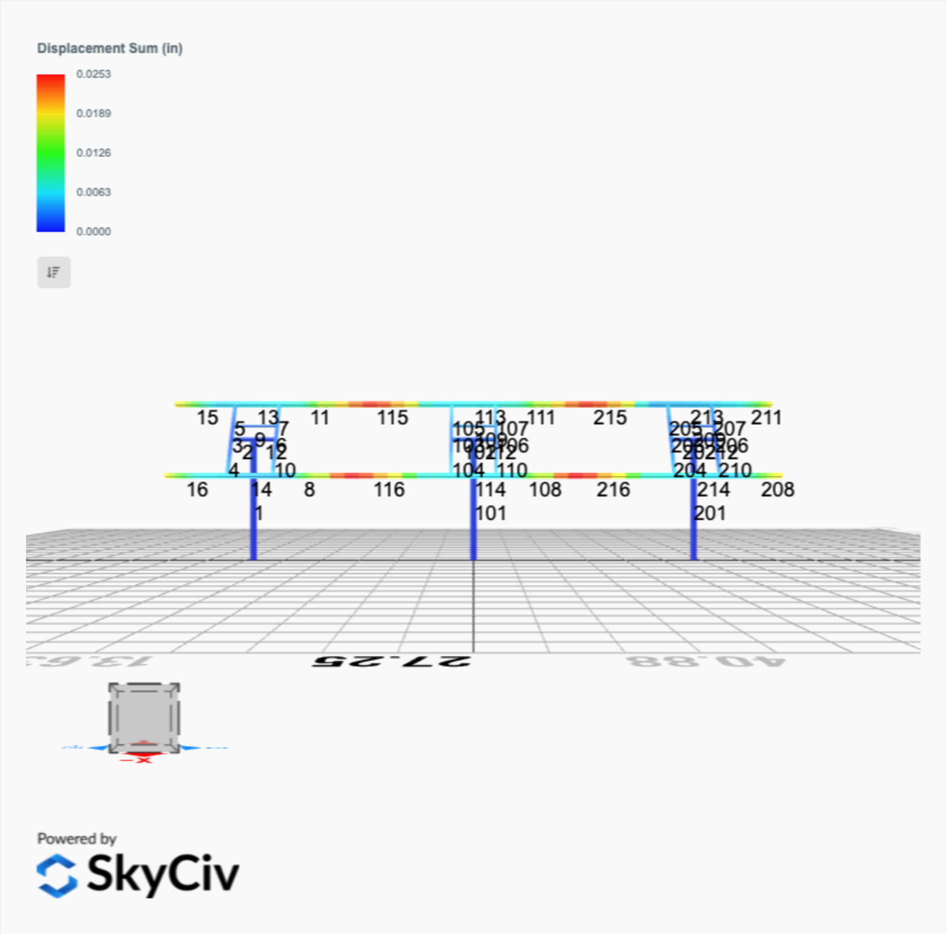
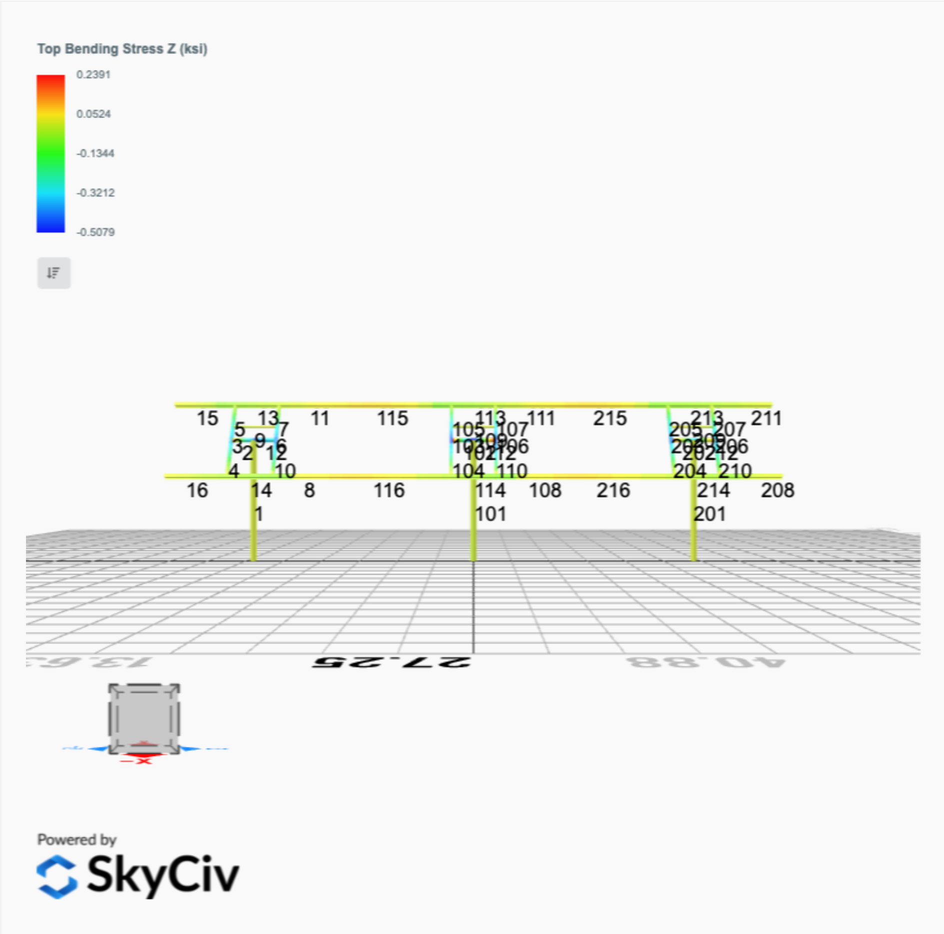
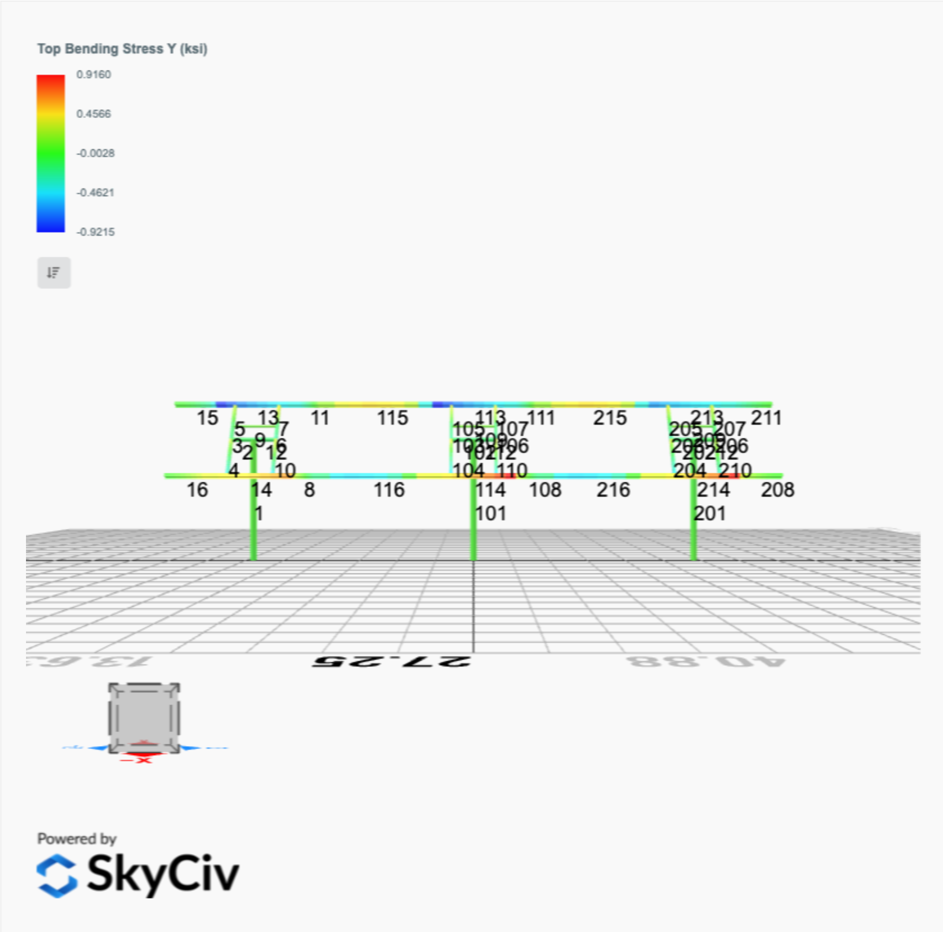
Powered by SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
0.2391
0.0524
-0.1344
-0.3212
-0.5079
15
13
11
115
113
111
215
213
211
16
14
8
116
114
108
216
214
208
1
101
201
27.25
40.88
Powered by
SkyCiv


Top Bending Stress Y (ksi)
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0.4566
-0.0028
-0.4621
-0.9215
15
13
11
115
113
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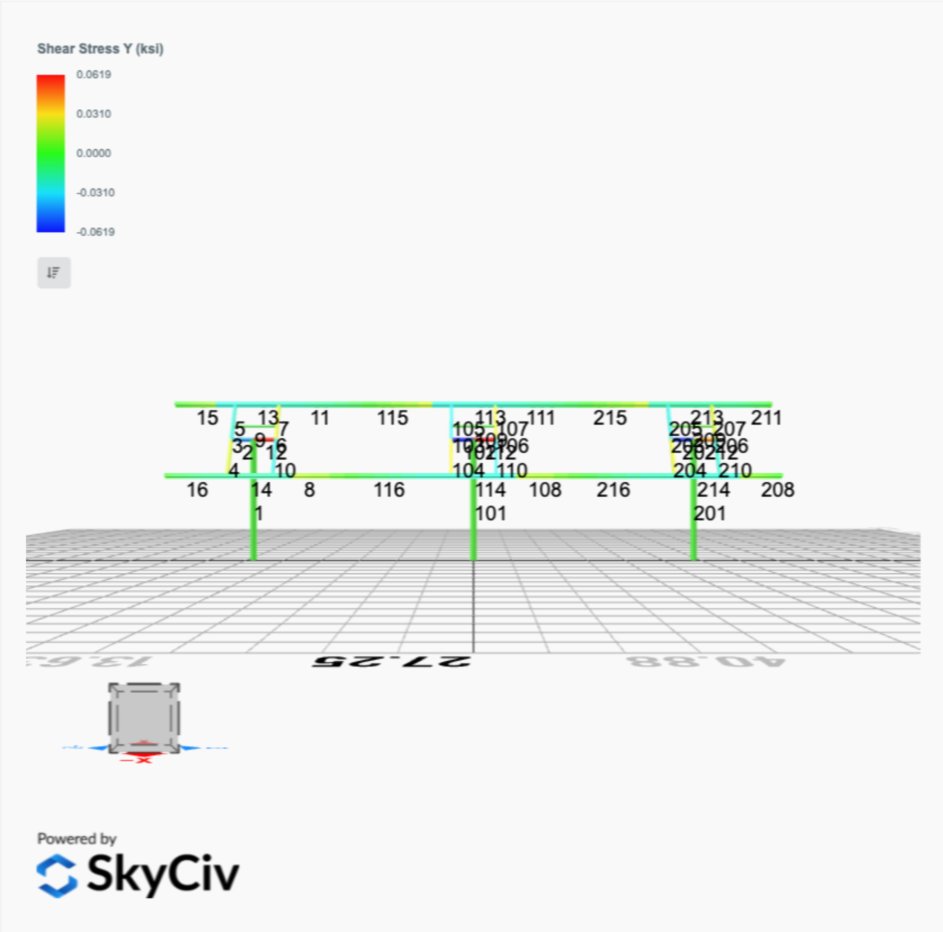
111
215
213
207
211
16
14
8
116
114
108
216
214
208
1
101
201
27.25
40.88
Powered by
SkyCiv


Shear Stress Y (ksi)
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0.0310
0.0000
-0.0310
-0.0619
15
13
11
115
113
111
215
213
211
16
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8
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114
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214
208
1
101
201
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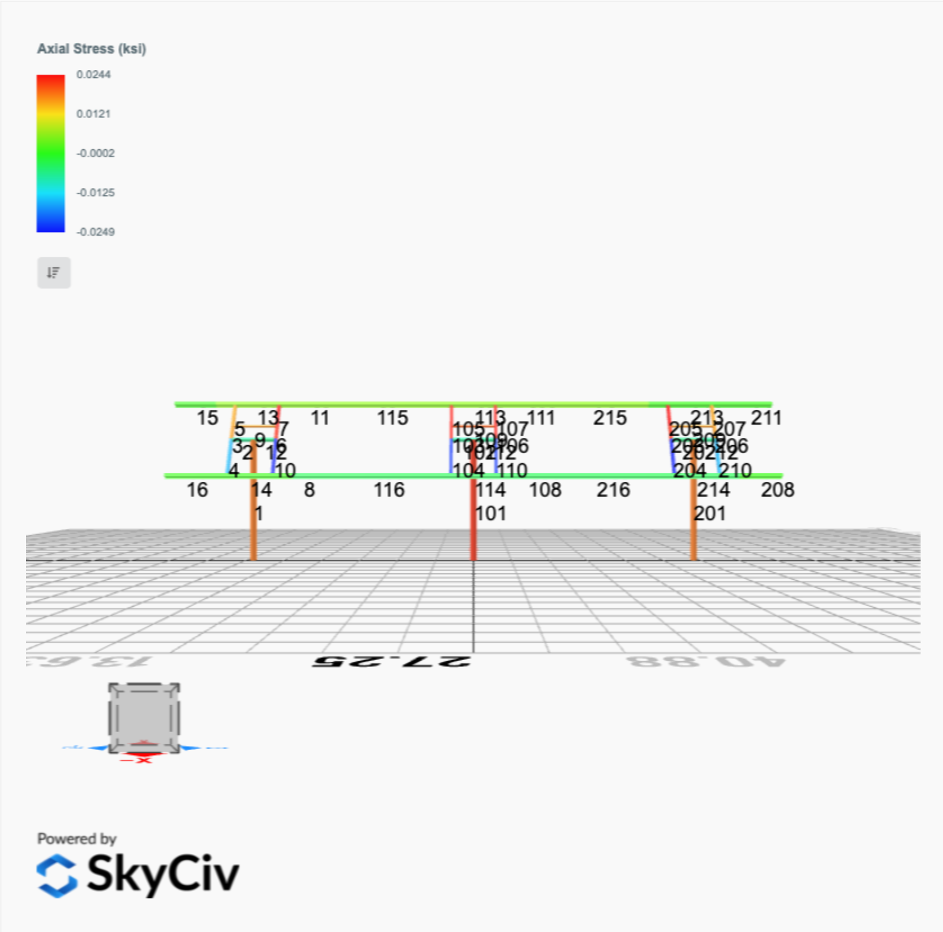
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-0.0002
-0.0125
-0.0249
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16 14 8 116 114 108 216 214 208
1
101
201
27.25
Powered by

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	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 2. D + L	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 3. D + (S or Lr or R)	0.0061	2.0124	0.0297	0.1005	-0.0236	-0.0465
ULS: 3. D + (S or Lr or R)	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0059	1.9690	0.0289	0.0976	-0.0229	-0.0449
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 5b. D + 0.7E	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0059	1.9690	0.0289	0.0976	-0.0229	-0.0449
ULS: 8. 0.6D + 0.7E	0.0032	1.1033	0.0158	0.0535	-0.0126	-0.0240
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3402	3.1775	0.0913	0.2932	-0.4225	25.7884
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3492	0.5007	-0.0372	-0.1103	0.3727	-25.2948
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7533	2.9730	0.0775	0.2506	-0.3241	19.3264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0059	1.9690	0.0289	0.0976	-0.0229	-0.0449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7638	0.9654	-0.0188	-0.0520	0.2723	-18.9860
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0059	1.9690	0.0289	0.0976	-0.0229	-0.0449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7538	2.8428	0.0751	0.2422	-0.3221	19.3313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7632	0.8352	-0.0213	-0.0604	0.2743	-18.9811
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0054	1.8388	0.0264	0.0892	-0.0209	-0.0399
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3423	2.4420	0.0807	0.2575	-0.4142	25.8044
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0032	1.1033	0.0158	0.0535	-0.0126	-0.0240
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3471	-0.2349	-0.0477	-0.1460	0.3811	-25.2788
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0032	1.1033	0.0158	0.0535	-0.0126	-0.0240

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
Axial	4.5249
Shear X	-3.9145
Shear Z	0.1424
Moment X	0.4557
Moment Z	43.5350

Worst Case Reactions ASD

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

Result	Value
Axial	3.1775
Shear X	-2.3492
Shear Z	0.0913
Moment X	0.2932
Moment Z	25.8044

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	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 2. D + L	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 3. D + (S or Lr or R)	-0.0121	2.2399	0.0000	-0.0000	0.0000	0.1312
ULS: 3. D + (S or Lr or R)	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0118	2.1901	0.0000	-0.0000	0.0000	0.1278
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 5b. D + 0.7E	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0118	2.1901	0.0000	-0.0000	0.0000	0.1278

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 8. 0.6D + 0.7E	-0.0065	1.2245	0.0000	-0.0000	0.0000	0.0707
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6389	3.5893	0.0000	-0.0000	0.0000	28.7548
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6209	0.4915	0.0000	-0.0000	0.0000	-27.8678
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9829	3.3515	0.0000	-0.0000	0.0000	21.6056
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0118	2.1901	0.0000	-0.0000	0.0000	0.1278
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9619	1.0281	0.0000	-0.0000	0.0000	-20.8614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0118	2.1901	0.0000	-0.0000	0.0000	0.1278
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9818	3.2022	0.0000	-0.0000	0.0000	21.5956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9629	0.8788	0.0000	-0.0000	0.0000	-20.8714
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0108	2.0409	0.0000	-0.0000	0.0000	0.1178
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6346	2.7730	0.0000	-0.0000	0.0000	28.7077
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0065	1.2245	0.0000	-0.0000	0.0000	0.0707
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6252	-0.3249	0.0000	-0.0000	0.0000	-27.9149
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0065	1.2245	0.0000	-0.0000	0.0000	0.0707

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
Axial	5.1286
Shear X	-4.3909
Shear Z	0.0000
Moment X	0.0001
Moment Z	48.4791

Worst Case Reactions ASD

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

Result	Value
Axial	3.5893
Shear X	-2.6389
Shear Z	0.0000
Moment X	-0.0000
Moment Z	28.7548

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	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 2. D + L	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 3. D + (S or Lr or R)	0.0061	2.0124	-0.0297	-0.1005	0.0236	-0.0465
ULS: 3. D + (S or Lr or R)	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0059	1.9690	-0.0289	-0.0977	0.0229	-0.0449
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 5b. D + 0.7E	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0059	1.9690	-0.0289	-0.0977	0.0229	-0.0449
ULS: 8. 0.6D + 0.7E	0.0032	1.1033	-0.0158	-0.0535	0.0126	-0.0240
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3402	3.1775	-0.0913	-0.2932	0.4225	25.7884
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3492	0.5007	0.0372	0.1103	-0.3727	-25.2947
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7533	2.9730	-0.0775	-0.2506	0.3241	19.3264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0059	1.9690	-0.0289	-0.0977	0.0229	-0.0449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7638	0.9654	0.0188	0.0520	-0.2723	-18.9860
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0059	1.9690	-0.0289	-0.0977	0.0229	-0.0449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7538	2.8428	-0.0751	-0.2422	0.3221	19.3313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7632	0.8352	0.0213	0.0604	-0.2743	-18.9810
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0054	1.8388	-0.0264	-0.0892	0.0209	-0.0399
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3423	2.4420	-0.0807	-0.2575	0.4142	25.8044
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0032	1.1033	-0.0158	-0.0535	0.0126	-0.0240
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3471	-0.2349	0.0477	0.1460	-0.3811	-25.2788
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0032	1.1033	-0.0158	-0.0535	0.0126	-0.0240

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
Axial	4.5249
Shear X	-3.9145
Shear Z	-0.1424
Moment X	-0.4558
Moment Z	43.5358

Worst Case Reactions ASD

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

Result	Value
Axial	3.1775
Shear X	-2.3492
Shear Z	-0.0913
Moment X	-0.2932
Moment Z	25.8044

Project Details

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

User Name: sales@mtsolar.us
Project Name: MTSOLAR_3BC9H21G0B846
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			
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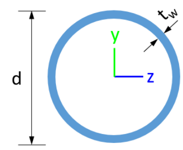
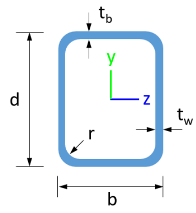
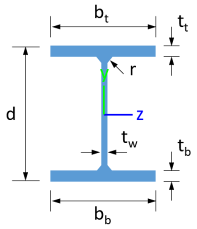
Section Dimensions



ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
8	6in Pipe Sch 80	6.63	0.43					



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



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

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31

8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	8	22.78	22.78	10.85	-	300	200	1
2	4	2.00	2.00	2.00	-	300	200	1
3	15	1.42	1.42	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18	300	200	1
4	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	2.33	2.33	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	1.42	1.42	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	2.33	2.33	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
8	18	2.05	2.05	2.05	1.75,1.75,1.75,1.75,1.75,1.75,1.70,1.75,1.69,1.75,1.70,1.75,1.69,1.75,1.71,1.75,1.66,1.75,1.71,1.75,1.67,1.75,1.70,1.75,1.69,1.75	300	200	1
9	1	4.00	4.00	4.00	-	300	200	1
10	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	2.05	2.05	2.05	1.81,1.81,1.81,1.81,1.81,1.81,1.90,1.81,1.93,1.81,1.90,1.81,1.93,1.81,1.88,1.81,1.98,1.81,1.88,1.81,1.97,1.81,1.90,1.81,1.92,1.81	300	200	1
12	4	0.58	0.58	0.58	-	300	200	1
13	18	1.75	1.75	1.75	1.57,1.57,1.57,1.57,1.57,1.57,1.54,1.57,1.54,1.57,1.54,1.57,1.54,1.57,1.55,1.57,1.52,1.57,1.55,1.57,1.53,1.57,1.54,1.57,1.54,1.57	300	200	1
14	18	1.75	1.75	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
15	18	3.75	3.75	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	18	3.75	3.75	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
17	18	4.00	4.00	4.00	1.16,1.16,1.16,1.16,1.16,1.16,1.14,1.16,1.14,1.16,1.14,1.16,1.14,1.16,1.15,1.16,1.13,1.16,1.15,1.16,1.14,1.16,1.14,1.16,1.14,1.16	300	200	1
18	18	1.75	1.75	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
19	18	4.00	4.00	4.00	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15	300	200	1
20	18	1.75	1.75	1.75	1.60,1.60,1.60,1.60,1.60,1.60,1.62,1.60,1.63,1.60,1.62,1.60,1.63,1.60,1.62,1.60,1.65,1.60,1.62,1.60,1.64,1.60,1.62,1.60,1.63,1.60	300	200	1
21	18	4.00	4.00	4.00	1.09,1.09,1.09,1.09,1.09,1.09,1.11,1.09,1.13,1.09,1.11,1.09,1.13,1.09,1.11,1.09,1.16,1.09,1.11,1.09,1.15,1.09,1.11,1.09,1.12,1.09	300	200	1
22	18	1.75	1.75	1.75	1.49,1.49,1.49,1.49,1.49,1.49,1.71,1.49,1.87,1.49,1.71,1.49,1.86,1.49,1.66,1.49,2.22,1.49,1.67,1.49,2.15,1.49,1.72,1.49,1.82,1.49	300	200	1
23	18	4.00	4.00	4.00	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.08,1.08,1.09,1.08,1.08,1.08,1.09,1.08,1.09,1.08,1.09,1.08	30	20	1

				[illegible]	[illegible]	0	0	
24	18	1.75	1.75	1.75	1.47,1.47,1.47,1.47,1.47,1.47,1.51,1.47,1.53,1.47,1.51,1.47,1.53,1.47,1.50,1.47,1.56,1.47,1.50,1.47,1.55,1.47,1.51,1.47,1.52,1.47	300	200	1
25	18	4.00	4.00	4.00	1.16,1.16,1.16,1.16,1.16,1.16,1.14,1.16,1.14,1.16,1.14,1.16,1.14,1.16,1.15,1.16,1.13,1.16,1.15,1.16,1.14,1.16,1.14,1.16,1.14,1.16	300	200	1
26	18	1.75	1.75	1.75	1.57,1.57,1.57,1.57,1.57,1.54,1.57,1.54,1.57,1.54,1.57,1.54,1.57,1.55,1.57,1.52,1.57,1.55,1.57,1.53,1.57,1.54,1.57,1.54,1.57	300	200	1
27	18	4.00	4.00	4.00	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15	300	200	1
28	18	1.75	1.75	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
29	4	1.42	1.42	1.42	-	300	200	1
30	4	0.58	0.58	0.58	-	300	200	1
101	8	22.78	22.78	10.85	-	300	200	1
102	4	2.00	2.00	2.00	-	300	200	1
103	15	1.42	1.42	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
104	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
105	15	2.33	2.33	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
106	15	1.42	1.42	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
107	15	2.33	2.33	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
108	18	2.05	2.05	2.05	2.08,2.08,2.08,2.08,2.08,2.08,2.06,2.08,1.97,2.08,2.06,2.08,1.97,2.08,2.06,2.08,1.84,2.08,2.06,2.08,1.88,2.08,2.06,2.08,1.99,2.08	300	200	1
109	1	4.00	4.00	4.00	-	300	200	1
110	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
111	18	2.05	2.05	2.05	2.07,2.07,2.07,2.07,2.07,2.07,1.59,2.07,1.49,2.07,1.59,2.07,1.49,2.07,1.65,2.07,1.37,2.07,1.64,2.07,1.41,2.07,1.58,2.07,1.51,2.07	300	200	1
112	4	2.00	2.00	2.00	-	300	200	1
113	18	1.75	1.75	1.75	1.49,1.49,1.49,1.49,1.49,1.49,1.71,1.49,1.87,1.49,1.71,1.49,1.86,1.49,1.66,1.49,2.22,1.49,1.67,1.49,2.16,1.49,1.72,1.49,1.82,1.49	300	200	1
114	18	1.75	1.75	1.75	1.47,1.47,1.47,1.47,1.47,1.47,1.51,1.47,1.53,1.47,1.51,1.47,1.53,1.47,1.50,1.47,1.56,1.47,1.50,1.47,1.55,1.47,1.51,1.47,1.52,1.47	300	200	1
115	18	10.20	10.20	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.11,1.15,1.10,1.15,1.11,1.15,1.10,1.15,1.12,1.15,1.09,1.15,1.12,1.15,1.09,1.15,1.11,1.15,1.10,1.15	300	200	1
116	18	10.20	10.20	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.16,1.14,1.16,1.15,1.16,1.14,1.16,1.15,1.16,1.14,1.16,1.15,1.16,1.14,1.16,1.15,1.16,1.14,1.16	300	200	1
201	8	22.78	22.78	10.85	-	300	200	1
202	4	1.42	1.42	1.42	-	300	200	1
203	15	1.42	1.42	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1

204	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
205	15	2.33	2.33	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
206	15	1.42	1.42	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18	300	200	1
207	15	2.33	2.33	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
208	18	3.75	3.75	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
209	1	4.00	4.00	4.00	-	300	200	1
210	15	3.75	3.75	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
211	18	3.75	3.75	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
212	4	2.00	2.00	2.00	-	300	200	1
213	18	1.75	1.75	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
214	18	1.75	1.75	1.75	1.60,1.60,1.60,1.60,1.60,1.60,1.62,1.60,1.63,1.60,1.62,1.60,1.63,1.60,1.62,1.60,1.65,1.60,1.62,1.60,1.64,1.60,1.62,1.60,1.63,1.60	300	200	1
215	18	10.20	10.20	10.20	1.13,1.13,1.13,1.13,1.13,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.15,1.13,1.14,1.13,1.15,1.13,1.14,1.13,1.14,1.13	300	200	1
216	18	10.20	10.20	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12	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	122.46	62.23	62.23	113.47	113.47
2	142.83	140.22	16.17	16.17	42.85	42.85
3	79.65	73.55	10.99	4.60	29.14	16.61
4	79.65	68.91	10.99	4.60	29.14	16.61
5	79.65	72.20	10.99	4.60	29.14	16.61
6	79.65	73.55	10.99	4.60	29.14	16.61
7	79.65	72.20	10.99	4.60	29.14	16.61
8	120.60	114.27	23.36	6.45	30.09	45.74
9	48.35	36.84	2.85	2.85	14.51	14.51
10	79.65	68.91	10.99	4.60	29.14	16.61
11	120.60	114.27	23.36	6.45	30.09	45.74
12	142.83	142.61	16.17	16.17	42.85	42.85
13	120.60	115.95	23.36	6.45	30.09	45.74
14	120.60	115.95	23.36	6.45	30.09	45.74
15	120.60	100.70	23.36	6.45	30.09	45.74
16	120.60	100.70	23.36	6.45	30.09	45.74
17	120.60	98.23	23.36	6.45	30.09	45.74
18	120.60	115.95	23.36	6.45	30.09	45.74
19	120.60	98.23	23.36	6.45	30.09	45.74
20	120.60	115.95	23.36	6.45	30.09	45.74
21	120.60	98.23	23.36	6.45	30.09	45.74
22	120.60	115.95	23.36	6.45	30.09	45.74

23	120.60	98.23	23.36	6.45	30.09	45.74
24	120.60	115.95	23.36	6.45	30.09	45.74
25	120.60	98.23	23.36	6.45	30.09	45.74
26	120.60	115.95	23.36	6.45	30.09	45.74
27	120.60	98.23	23.36	6.45	30.09	45.74
28	120.60	115.95	23.36	6.45	30.09	45.74
29	142.83	141.51	16.17	16.17	42.85	42.85
30	142.83	142.61	16.17	16.17	42.85	42.85
101	378.22	122.46	62.23	62.23	113.47	113.47
102	142.83	140.22	16.17	16.17	42.85	42.85
103	79.65	73.55	10.99	4.60	29.14	16.61
104	79.65	68.91	10.99	4.60	29.14	16.61
105	79.65	72.20	10.99	4.60	29.14	16.61
106	79.65	73.55	10.99	4.60	29.14	16.61
107	79.65	72.20	10.99	4.60	29.14	16.61
108	120.60	114.27	23.36	6.45	30.09	45.74
109	48.35	36.84	2.85	2.85	14.51	14.51
110	79.65	68.91	10.99	4.60	29.14	16.61
111	120.60	114.27	23.36	6.45	30.09	45.74
112	142.83	140.22	16.17	16.17	42.85	42.85
113	120.60	115.95	23.36	6.45	30.09	45.74
114	120.60	115.95	23.36	6.45	30.09	45.74
115	120.60	33.17	14.89	6.45	30.09	45.74
116	120.60	33.17	15.57	6.45	30.09	45.74
201	378.22	122.46	62.23	62.23	113.47	113.47
202	142.83	141.51	16.17	16.17	42.85	42.85
203	79.65	73.55	10.99	4.60	29.14	16.61
204	79.65	68.91	10.99	4.60	29.14	16.61
205	79.65	72.20	10.99	4.60	29.14	16.61
206	79.65	73.55	10.99	4.60	29.14	16.61
207	79.65	72.20	10.99	4.60	29.14	16.61
208	120.60	100.70	23.36	6.45	30.09	45.74
209	48.35	36.84	2.85	2.85	14.51	14.51
210	79.65	68.91	10.99	4.60	29.14	16.61
211	120.60	100.70	23.36	6.45	30.09	45.74
212	142.83	140.22	16.17	16.17	42.85	42.85
213	120.60	115.95	23.36	6.45	30.09	45.74
214	120.60	115.95	23.36	6.45	30.09	45.74
215	120.60	33.17	15.44	6.45	30.09	45.74
216	120.60	33.17	15.30	6.45	30.09	45.74

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.037	0.700	0.017	0.034	0.001	0.725	#13	0.623	Not Required	Pass
2	0.001	0.172	0.211	0.043	0.043	0.384	#13	0.053	Not Required	Pass
3	0.006	0.429	0.046	0.042	0.005	0.473	#13	0.068	Not Required	Pass
4	0.005	0.428	0.120	0.043	0.016	0.525	#13	0.120	Not Required	Pass
5	0.005	0.266	0.111	0.043	0.016	0.285	#13	0.112	Not Required	Pass
6	0.007	0.511	0.079	0.052	0.010	0.586	#13	0.068	Not Required	Pass
7	0.007	0.318	0.160	0.051	0.023	0.350	#13	0.112	Not Required	Pass
8	0.002	0.059	0.059	0.031	0.007	0.089	#13	0.136	Not Required	Pass
9	0.006	0.036	0.059	0.002	0.001	0.088	#13	0.305	Not Required	Pass

10	0.007	0.490	0.163	0.049	0.021	0.554	#13	0.120	Not Required	Pass
11	0.001	0.056	0.060	0.032	0.007	0.083	#13	0.136	Not Required	Pass
12	0.001	0.240	0.255	0.054	0.050	0.494	#13	0.015	Not Required	Pass
13	0.001	0.099	0.153	0.041	0.008	0.219	#13	0.116	Not Required	Pass
14	0.000	0.100	0.113	0.028	0.006	0.202	#13	Not Required	Not Required	Pass
15	0.000	0.046	0.053	0.019	0.004	0.094	#13	Not Required	Not Required	Pass
16	0.000	0.046	0.053	0.019	0.004	0.094	#13	Not Required	Not Required	Pass
17	0.002	0.113	0.039	0.013	0.002	0.144	#13	0.177	Not Required	Pass
18	0.000	0.100	0.113	0.028	0.006	0.202	#13	Not Required	Not Required	Pass
19	0.003	0.110	0.049	0.013	0.003	0.146	#13	0.265	Not Required	Pass
20	0.002	0.087	0.152	0.040	0.008	0.204	#13	0.116	Not Required	Pass
21	0.002	0.090	0.042	0.010	0.002	0.131	#13	0.177	Not Required	Pass
22	0.001	0.080	0.155	0.039	0.008	0.228	#13	0.116	Not Required	Pass
23	0.004	0.114	0.046	0.010	0.002	0.159	#13	0.265	Not Required	Pass
24	0.002	0.104	0.155	0.041	0.008	0.251	#13	0.116	Not Required	Pass
25	0.002	0.113	0.039	0.013	0.002	0.144	#13	0.177	Not Required	Pass
26	0.001	0.099	0.153	0.041	0.008	0.219	#13	0.116	Not Required	Pass
27	0.003	0.110	0.049	0.013	0.003	0.146	#13	0.265	Not Required	Pass
28	0.000	0.100	0.113	0.028	0.006	0.202	#13	Not Required	Not Required	Pass
29	0.001	0.156	0.180	0.054	0.050	0.336	#13	0.038	Not Required	Pass
30	0.001	0.240	0.255	0.054	0.050	0.494	#13	0.015	Not Required	Pass
101	0.042	0.779	0.000	0.039	0.000	0.800	#13	0.623	Not Required	Pass
102	0.001	0.234	0.261	0.056	0.051	0.496	#13	0.053	Not Required	Pass
103	0.007	0.520	0.070	0.052	0.008	0.584	#13	0.068	Not Required	Pass
104	0.006	0.535	0.153	0.054	0.019	0.638	#13	0.120	Not Required	Pass
105	0.007	0.322	0.159	0.052	0.023	0.360	#13	0.112	Not Required	Pass
106	0.007	0.520	0.070	0.052	0.008	0.583	#13	0.068	Not Required	Pass
107	0.007	0.323	0.159	0.052	0.023	0.360	#13	0.112	Not Required	Pass
108	0.002	0.051	0.061	0.032	0.007	0.082	#13	0.136	Not Required	Pass
109	0.007	0.023	0.049	0.001	0.000	0.076	#13	0.305	Not Required	Pass
110	0.006	0.535	0.153	0.054	0.019	0.638	#13	0.120	Not Required	Pass
111	0.001	0.065	0.062	0.031	0.007	0.078	#13	0.136	Not Required	Pass
112	0.001	0.234	0.261	0.056	0.051	0.496	#13	0.053	Not Required	Pass
113	0.001	0.080	0.155	0.039	0.008	0.228	#13	0.116	Not Required	Pass
114	0.002	0.103	0.155	0.041	0.008	0.251	#13	0.116	Not Required	Pass
115	0.004	0.171	0.085	0.031	0.007	0.254	#13	0.675	Not Required	Pass
116	0.003	0.162	0.086	0.032	0.007	0.242	#13	0.675	Not Required	Pass
201	0.037	0.700	0.017	0.034	0.001	0.725	#13	0.623	Not Required	Pass
202	0.001	0.156	0.180	0.054	0.050	0.336	#13	0.038	Not Required	Pass
203	0.007	0.511	0.079	0.052	0.010	0.586	#13	0.068	Not Required	Pass
204	0.007	0.491	0.163	0.049	0.021	0.554	#13	0.120	Not Required	Pass
205	0.007	0.318	0.160	0.051	0.023	0.350	#13	0.112	Not Required	Pass
206	0.006	0.429	0.046	0.042	0.005	0.473	#13	0.068	Not Required	Pass
207	0.005	0.266	0.111	0.043	0.016	0.285	#13	0.112	Not Required	Pass
208	0.000	0.046	0.053	0.019	0.004	0.094	#13	Not Required	Not Required	Pass
209	0.006	0.036	0.059	0.002	0.001	0.088	#13	0.305	Not Required	Pass
210	0.005	0.428	0.120	0.043	0.016	0.525	#13	0.120	Not Required	Pass
211	0.000	0.046	0.053	0.019	0.004	0.094	#13	Not Required	Not Required	Pass
212	0.001	0.172	0.211	0.043	0.043	0.384	#13	0.053	Not Required	Pass
213	0.000	0.100	0.113	0.028	0.006	0.202	#13	Not Required	Not Required	Pass
214	0.002	0.087	0.152	0.040	0.008	0.204	#13	0.116	Not Required	Pass
215	0.004	0.175	0.086	0.032	0.007	0.250	#13	0.675	Not Required	Pass
216	0.003	0.167	0.086	0.031	0.007	0.245	#13	0.675	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

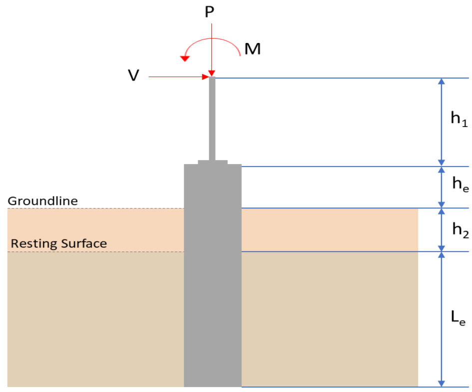
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 6.25\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - 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)	3.178	4.525
V_x (kip)	-2.349	-3.914
V_z (kip)	0.091	0.142
M_x (kipft)	0.293	0.456
M_z (kipft)	25.804	43.535

Material Properties

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

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

$$H_o = \frac{(-2.349\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.37404\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.804\ \text{kipft}) + ((-2.349\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.1089\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8277\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.091\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.01449\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.293\ \text{kipft}) + ((0.091\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.046656\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.7373\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8277\ \text{ft}), (1.7373\ \text{ft})]$$

$$L_{e,req} = 5.828\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.828\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.93248$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48\ \text{in}) \times (48\ \text{in})$$

$$A = 16\ \text{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(3.178\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.19862\ \text{kip/ft}^2$$

$q = 0.19863 \text{ kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.19863 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.099313$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.37404 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.1089 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1089 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.37404 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (4.1089 \text{ kipft/ft})) + (4 \times (-0.37404 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3099 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1089 \text{ kipft/ft})) + (3 \times (-0.37404 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.1089 \text{ kipft/ft})) + (2 \times (-0.37404 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22279 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.1089 \text{ kipft/ft})) + ((-0.37404 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.90318 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3099 \text{ ft})}{2}$$

$$p_a = 0.32324 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.22279 \text{ kip/ft}^2)}{(0.32324 \text{ kip/ft}^2)}$$

$$Ratio = 0.68923$$

Status: **PASS**
Ratio: **0.690**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.90318\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.96339$$

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

Considering z-direction:

$H_o = 0.01449\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.046656\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.046656\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.046656\ \mathrm{kipft/ft})) + (4 \times (0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4605\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.046656\ \mathrm{kipft/ft})) + (3 \times (0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.046656\ \mathrm{kipft/ft})) + (2 \times (0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.01256\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.046656\ \mathrm{kipft/ft})) + ((0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.028244\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4605\ \mathrm{ft})}{2}$$

$$p_a = 0.33454\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.01256\ \mathrm{kip/ft^2}\right)}{\left(0.33454\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.037546$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

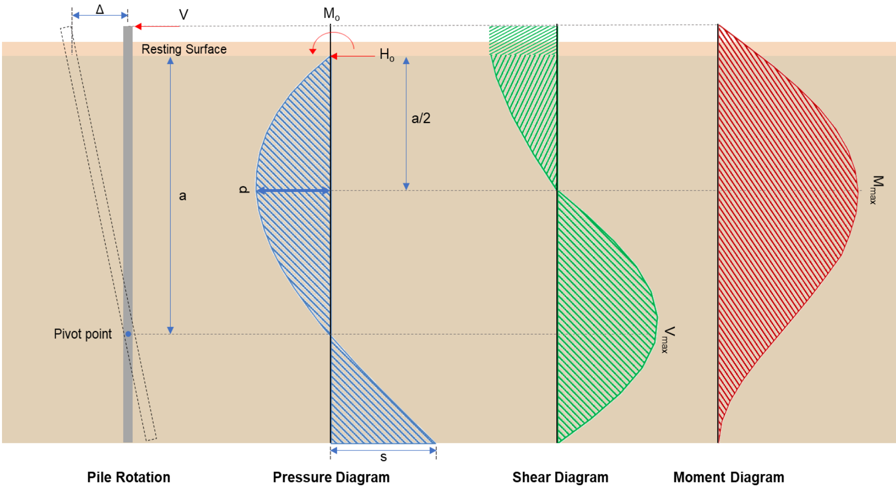
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.028244 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.030126$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.914 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.62325 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.535 \text{ kipft}) + ((-3.914 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.9323 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.9323 \text{ kipft/ft})}{(-0.62325 \text{ kip/ft})}$$

$$E = 11.123 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.9323 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.62325 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (6.9323 \text{ kipft/ft})) + (4 \times (-0.62325 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3086 \text{ ft}$$

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

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

$$V_{max} = ((-0.62325 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.3086 \text{ ft})}{(6.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.3086 \text{ ft})}{(6.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 9.4953 \text{ kip}$$

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

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

$$M_{max} = ((-0.62325 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(11.123 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.258 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.022611 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.456 \text{ kipft}) + ((0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.072611 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.072611 \text{ kipft/ft})}{(0.022611 \text{ kip/ft})}$$

$$E = 3.2113 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.072611 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (0.022611 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.072611 \text{ kipft/ft})) + (4 \times (0.022611 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4608 \text{ ft}$$

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

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

$$V_{max} = ((0.022611 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4608 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4608 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.14247 \text{ kip}$$

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

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

$$M_{max} = ((0.022611 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.2113 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.39477 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(4.525 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -102.11 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-102.11 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.525\ \text{kip})}{(3183.4\ \text{kip})}$$

$$Ratio = 0.0014214$$

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 $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$, $P = 4.525\ \text{kip} \rightarrow 4525\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \text{psi})} + \frac{(4525\ \text{lbf})}{6 \times \left(2304\ \text{in}^2\right)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 130.4\ \text{kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49\ \text{kip}), (130.4\ \text{kip}), (406.27\ \text{kip})\right]$$

$$V_c = 130.4\ \text{kip}$$

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{s,a} = 807.65\ \mathrm{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$ $$V_{s,b} = 50.894\ \mathrm{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(807.65\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((130.4\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 117.84\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 9.4953\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(9.4953\ \mathrm{kip})}{(117.84\ \mathrm{kip})}$$ $$Ratio = 0.080578$$	Status: **PASS** Ratio: **0.080**
	Considering z-direction: $V_{max} = 0.14247\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.14247\ \mathrm{kip})}{(117.84\ \mathrm{kip})}$$ $$Ratio = 0.001209$$	Status: **PASS** Ratio: **0.000**
	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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2545.9 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$ $$\phi M_n = 273.42 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 28.258 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(28.258 \text{ kipft})}{(273.42 \text{ kipft})}$$ $$Ratio = 0.10335$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $M_{max} = 0.39477 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.39477 \text{ kipft})}{(273.42 \text{ kipft})}$$ $$Ratio = 0.0014438$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

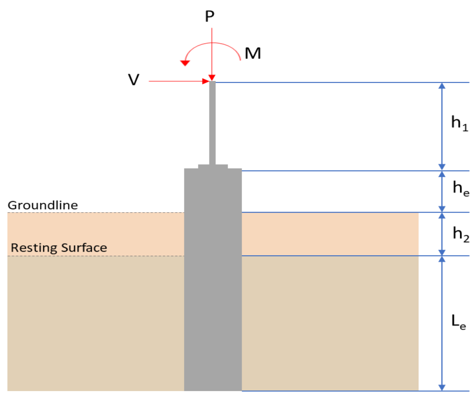
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 6.25\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - 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)	3.178	4.525
V_x (kip)	-2.349	-3.914
V_z (kip)	-0.091	-0.142
M_x (kipft)	-0.293	-0.456
M_z (kipft)	25.804	43.536

Material Properties

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

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

$$H_o = \frac{(-2.349\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.37404\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.804 \text{ kipft}) + ((-2.349 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1089 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.8277 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.091 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01449 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.293 \text{ kipft}) + ((-0.091 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.046656 \text{ kipft/ft}$$

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:
$L_{e,z} = 1.3654 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8277 \text{ ft}), (1.3654 \text{ ft})]$$

$$L_{e,req} = 5.828 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.828 \text{ ft})}{(6.25 \text{ ft})}$$

$$Ratio = 0.93248$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(3.178 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.19862 \text{ kip/ft}^2$$

$q = 0.19863\ \mathrm{kip/ft^2}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.19863\ \mathrm{kip/ft^2}\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.099313$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.37404\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.1089\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1089\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.37404\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (4.1089\ \mathrm{kipft/ft})) + (4 \times (-0.37404\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3099\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1089\ \mathrm{kipft/ft})) + (3 \times (-0.37404\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\,2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (4.1089\ \mathrm{kipft/ft})) + (2 \times (-0.37404\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.22279\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.1089\ \mathrm{kipft/ft})) + ((-0.37404\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.90318\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.3099\ \mathrm{ft})}{2}$$

$$p_a = 0.32324\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22279\ \mathrm{kip/ft^2}\right)}{\left(0.32324\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.68923$$

Status: **PASS**
Ratio: **0.690**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.90318\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.96339$$

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

Considering z-direction:

$H_o = -0.01449\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.046656\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.046656\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.046656\ \mathrm{kipft/ft})) + (4 \times (-0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4605\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.046656\ \mathrm{kipft/ft})) + (3 \times (-0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.046656\ \mathrm{kipft/ft})) + (2 \times (-0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.0033757\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.046656\ \mathrm{kipft/ft})) + ((-0.01449\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.00042191\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4605\ \mathrm{ft})}{2}$$

$$p_a = 0.33454\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0033757\ \mathrm{kip/ft^2}\right)}{\left(0.33454\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.010091$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

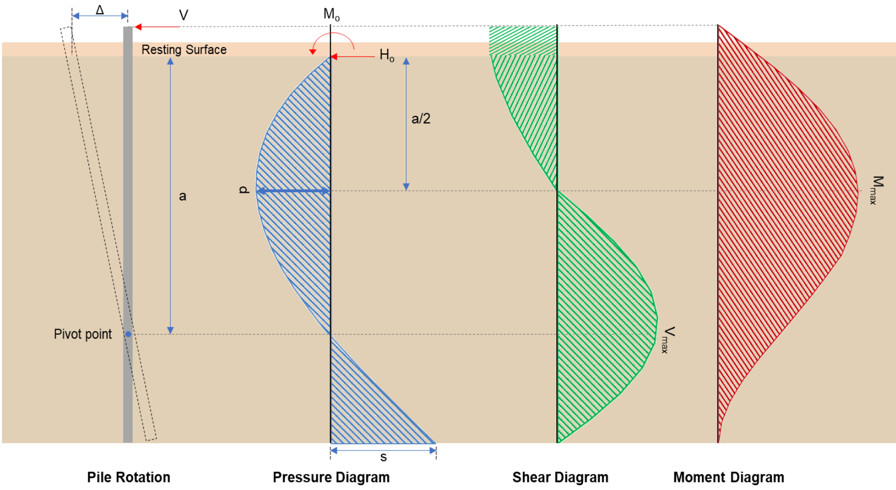
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.00042191\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.00045004$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.914\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.62325\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.536\ \mathrm{kipft}) + ((-3.914\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.9325\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.9325\ \mathrm{kipft/ft})}{(-0.62325\ \mathrm{kip/ft})}$$

$$E = 11.123\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.9325\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.62325\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.9325\ \mathrm{kipft/ft})) + (4 \times (-0.62325\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3086\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.62325\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.123\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3086\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.123\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3086\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.4955\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.62325 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(11.123 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.123 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3086 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.258 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.022611 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.456 \text{ kipft}) + ((-0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.072611 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.072611 \text{ kipft/ft})}{(-0.022611 \text{ kip/ft})}$$

$$E = 3.2113 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.072611 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.022611 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.072611 \text{ kipft/ft})) + (4 \times (-0.022611 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4608 \text{ ft}$$

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

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

$$V_{max} = ((-0.022611 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4608 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4608 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.14247 \text{ kip}$$

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

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

$$M_{max} = ((-0.022611 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.2113 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2113 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4608 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.39477 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(4.525 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -102.11 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-102.11 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.525 \text{ kip})}{(3183.4 \text{ kip})}$$

$$Ratio = 0.0014214$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 4.525 \text{ kip} \rightarrow 4525 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(4525 \text{ lbf})}{6 \times \left(2304 \text{ in}^2\right)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.4 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49 \text{ kip}), (130.4 \text{ kip}), (406.27 \text{ kip})\right]$$

$$V_c = 130.4 \text{ kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 807.65 \text{ kip}$$

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

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

$$V_s = MIN\left[(807.65 \text{ kip}), (50.894 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\ (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((130.4 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 117.84 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.4955 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.4955 \text{ kip})}{(117.84 \text{ kip})}$$

$$Ratio = 0.08058$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$V_{max} = 0.14247 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.14247 \text{ kip})}{(117.84 \text{ kip})}$$

$$Ratio = 0.001209$$

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

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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2545.9 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$ $$\phi M_n = 273.42 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 28.258 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(28.258 \text{ kipft})}{(273.42 \text{ kipft})}$$ $$Ratio = 0.10335$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $M_{max} = 0.39477 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.39477 \text{ kipft})}{(273.42 \text{ kipft})}$$ $$Ratio = 0.0014438$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

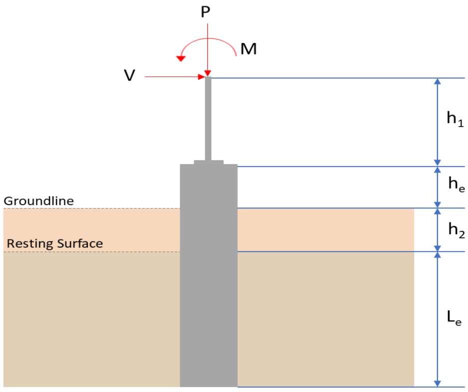
SkyCiv Foundation Design

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
$L = 6.5\ \mathrm{ft}$ - 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)	3.589	5.129
V_x (kip)	-2.639	-4.391
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	28.755	48.479

Material Properties

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

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

$$H_o = \frac{(-2.639\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42022\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.755 \text{ kipft}) + ((-2.639 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.5788 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.9927 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.9927 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.993 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.993 \text{ ft})}{(6.5 \text{ ft})}$$

$$Ratio = 0.922$$

Status: PASS
Ratio: 0.920

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(3.589 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.22431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.22431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.11216$$

Status: PASS
Ratio: 0.110

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.42022\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.5788\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.5788\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.42022\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (4.5788\ \mathrm{kipft/ft})) + (4 \times (-0.42022\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4875\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.5788\ \mathrm{kipft/ft})) + (3 \times (-0.42022\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^2}{(6.5\ \mathrm{ft})^2 \times [(3 \times (4.5788\ \mathrm{kipft/ft})) + (2 \times (-0.42022\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$

$$p = 0.21978\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.5788\ \mathrm{kipft/ft})) + ((-0.42022\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$

$$s = 0.9126\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4875\ \mathrm{ft})}{2}$$

$$p_a = 0.33656\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.21978\ \mathrm{kip/ft^2}\right)}{\left(0.33656\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65301$$

Status: **PASS**
Ratio: **0.650**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

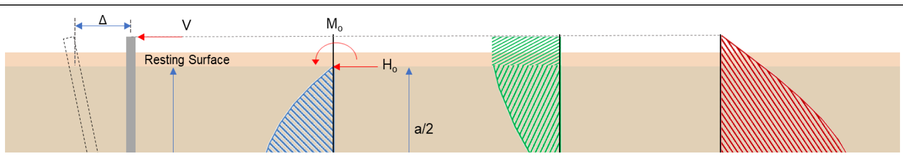
$Ratio$ - Lateral soil capacity

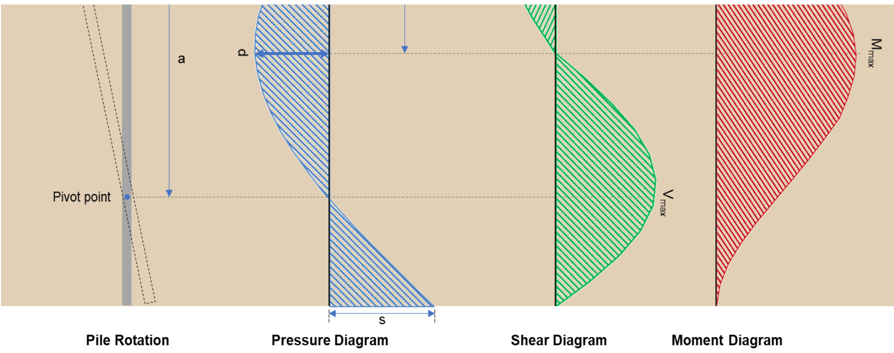
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.9126\ \mathrm{kip/ft^2}\right)}{\left(0.975\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.936$$

Status: **PASS**
Ratio: **0.940**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.391\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.6992\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(48.479\ \mathrm{kipft}) + ((-4.391\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 7.7196\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.7196\ \mathrm{kipft/ft})}{(-0.6992\ \mathrm{kip/ft})}$$

$$E = 11.041\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.7196\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.6992\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (7.7196\ \mathrm{kipft/ft})) + (4 \times (-0.6992\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.486\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.6992\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.041\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.486\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.041\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.486\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.251\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.6992\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.5\ \mathrm{ft})) \times \left[\left(\frac{(11.041\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + \frac{(4.486\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (11.041\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.486\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.041\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.486\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 31.664 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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 | $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(5.129 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -102.09 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-102.09 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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. 10ø: 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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(5.129\ \mathrm{kip})}{(3183.4\ \mathrm{kip})}$$

$$Ratio = 0.0016112$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

$$d = 0.80 \times (48\ \mathrm{in})$$

$$d = 38.4\ \mathrm{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $P = 5.129\ \mathrm{kip} \rightarrow 5129\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + \frac{(5129\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 130.48\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49\ \mathrm{kip}), (130.48\ \mathrm{kip}), (406.27\ \mathrm{kip})\right]$$

$$V_c = 130.48\ \mathrm{kip}$$

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, 22.5.1.2 $V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\,b_w\,d$$ $$V_{s,a} = 8 \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{s,a} = 807.65\ \mathrm{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\,d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$ 22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\,A_v\,f_{ywk}\,d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$ $$V_{s,b} = 50.894\ \mathrm{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(807.65\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$ 22.5.1.1 ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((130.48\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 117.89\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 10.251\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(10.251\ \mathrm{kip})}{(117.89\ \mathrm{kip})}$$ $$Ratio = 0.086949$$	Status: **PASS** Ratio: **0.090**
	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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$ 14.5.2.1b $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\,0.85\,f'_c\,S_m$$	

$$\phi M_{n,2} = \phi\ 0.85\ f_{ck}\ S_m$$

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

$$\phi M_{n,2} = 2545.9\ \mathrm{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 31.664\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(31.664\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$

$$Ratio = 0.11581$$

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