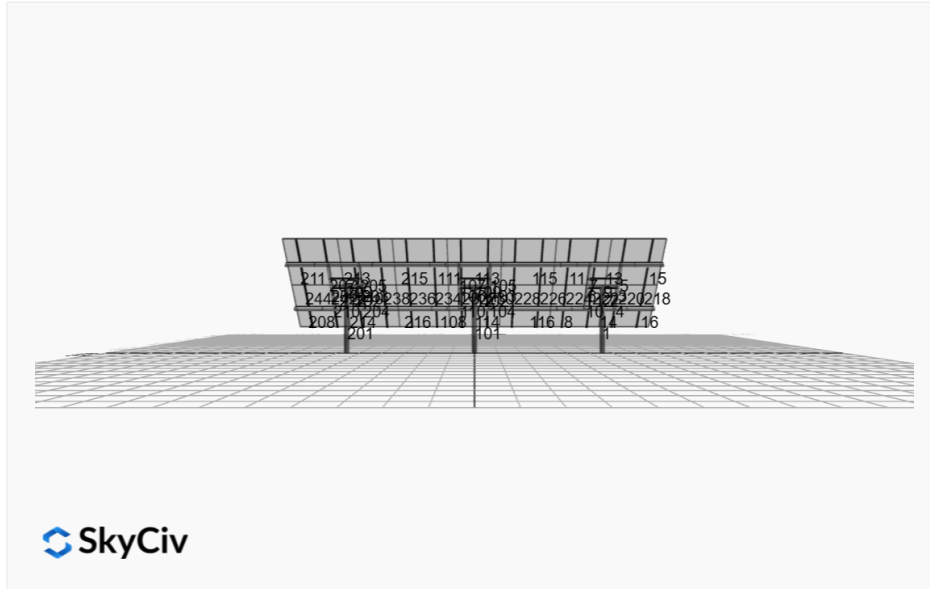


Project Details



Project Name: Walter Thomas **Date:** Wed Jul 16 2025
Location: 105 Reed Rd, Alton, NH 03810, USA **Number of Modules:** 28
Unique ID: 3P-17-6TOP-SD-45-L-4Hx7W-0828 **Number of Poles:** 3
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	48.77 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

MT Solar Bill of Materials (3P-17-6TOP-SD-45-L-4Hx7W-0828)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	3
MTS-HF-SD	H-Frame Assembly-SD	3
MTS-SD-Wing-45	45IN SD Wing	4
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	7

Rail Bill of Materials

Part	Qty
Rails (180in)	14
Rail Attachment	28
Module Mid Clamp	42
Module End Clamp	28
Ground Lug	7

Site Details:



Site Address: 105 Reed Rd, Alton, NH 03810, USA

Array Specification

Duty Classification:	SD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	4
Number of Columns:	7
Total Number of Modules:	28
Winter Tilt Angle:	50
Front Edge Clearance:	3
Total Array Height at Tilt:	14.52 ft
Total Frame Length:	49.00 ft
Module Info/Notes:	Silfab 530
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	48.77 ft
Rail Length:	180.40 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.76 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.25 ft Pile 3: 5.25 ft
Foundation Volume:	9.333 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	105 Reed Rd, Alton, NH 03810, USA
Wind Speed:	105 mph
Snow Load:	90 psf

Design Disclaimer

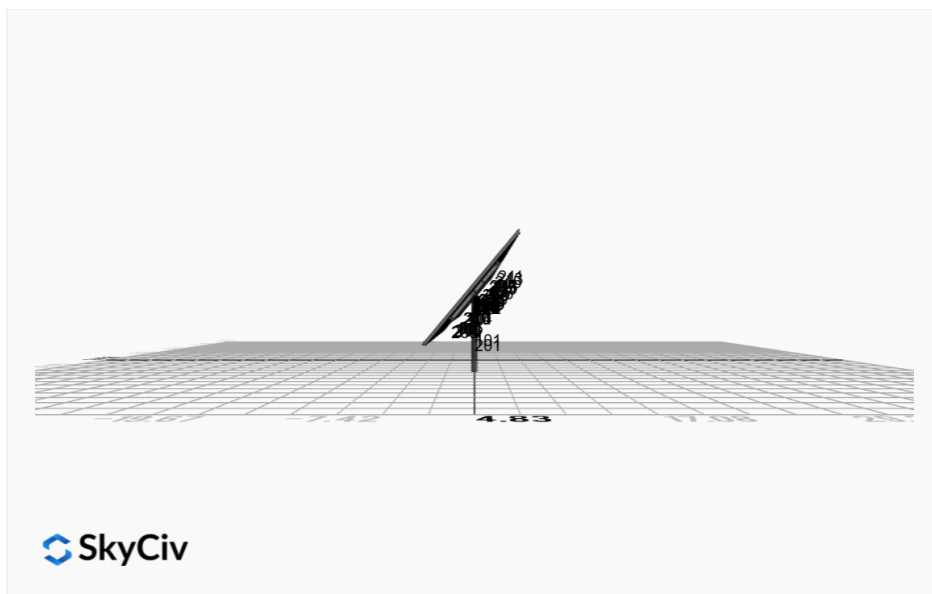
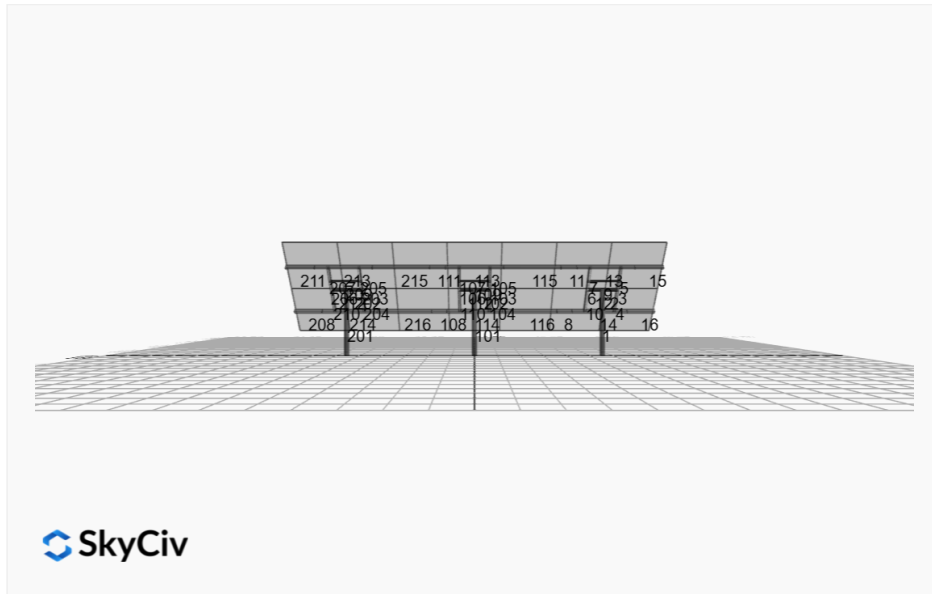
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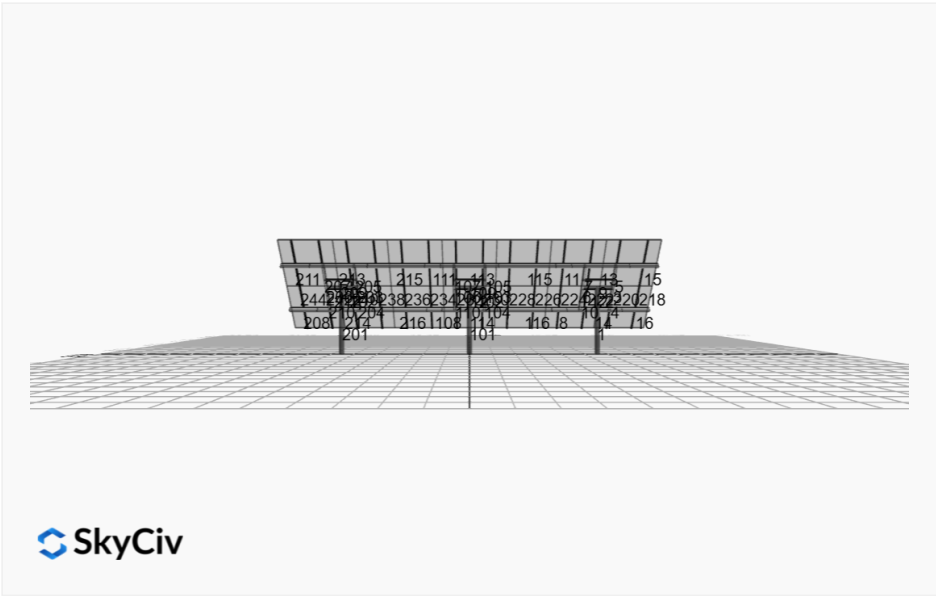
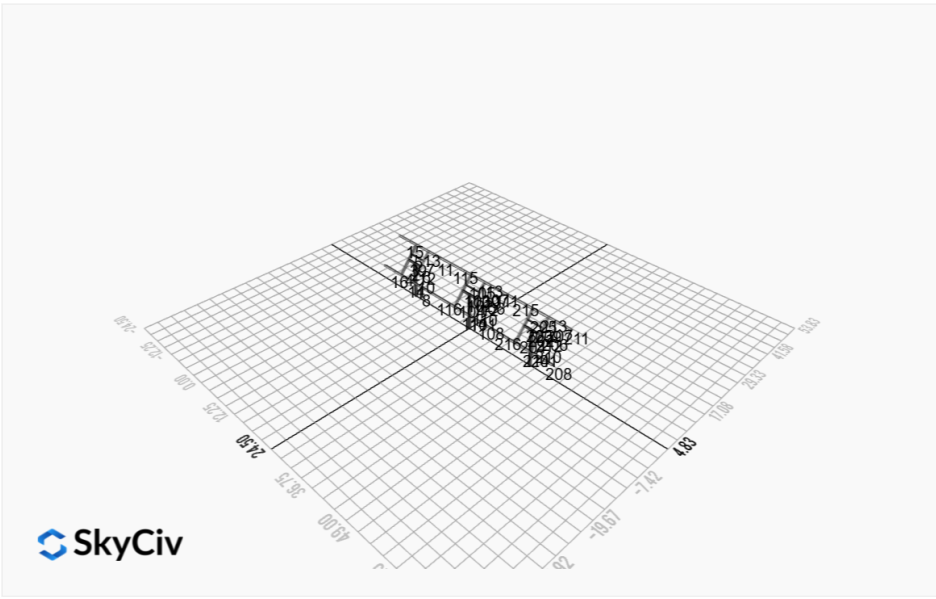
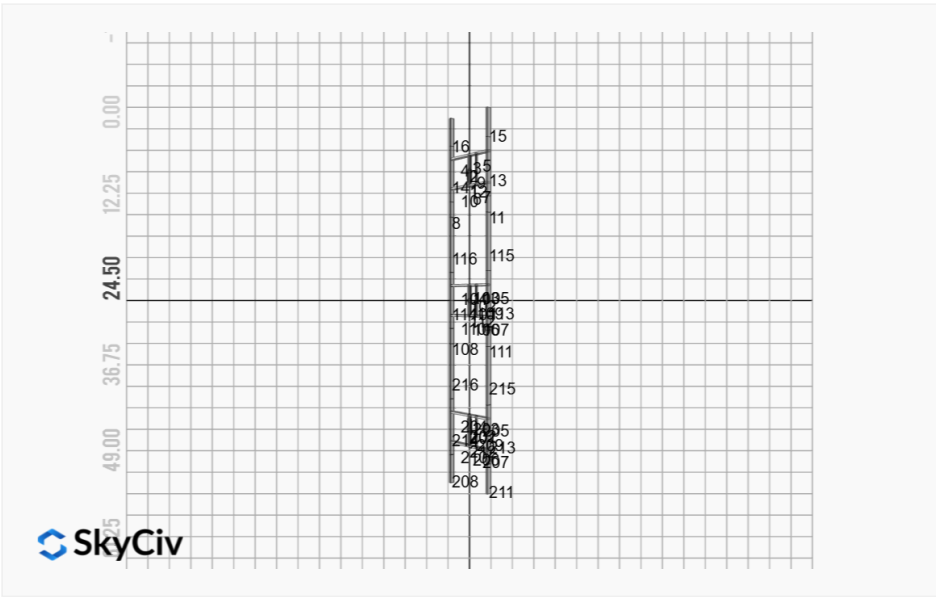
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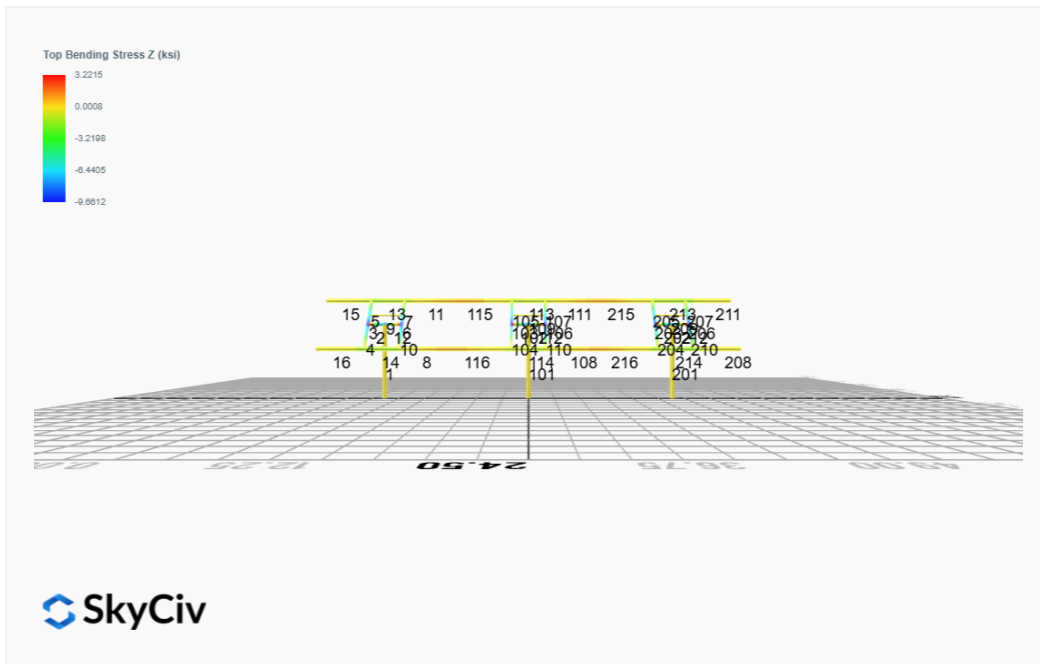
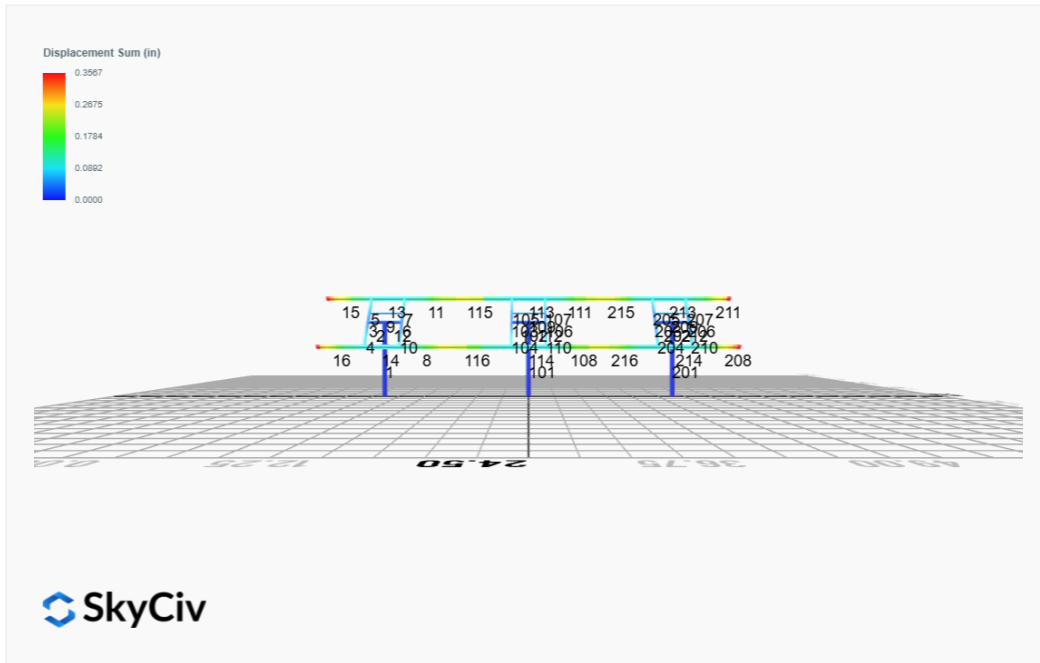
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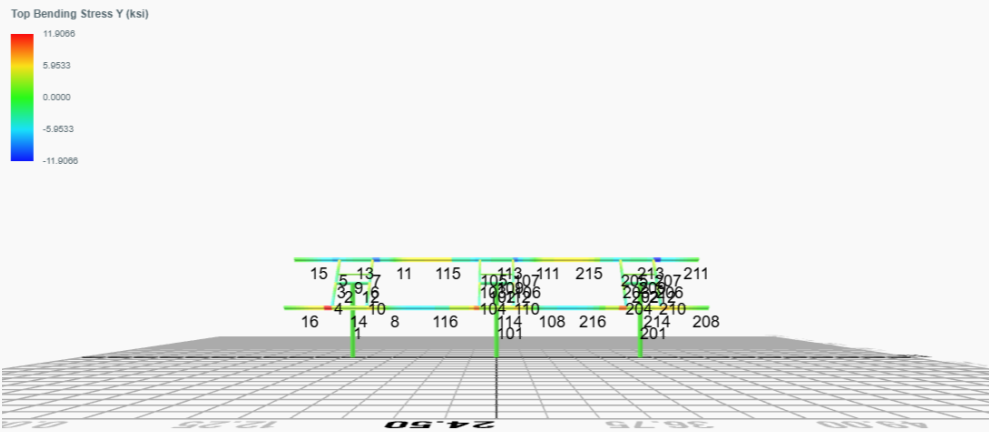
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



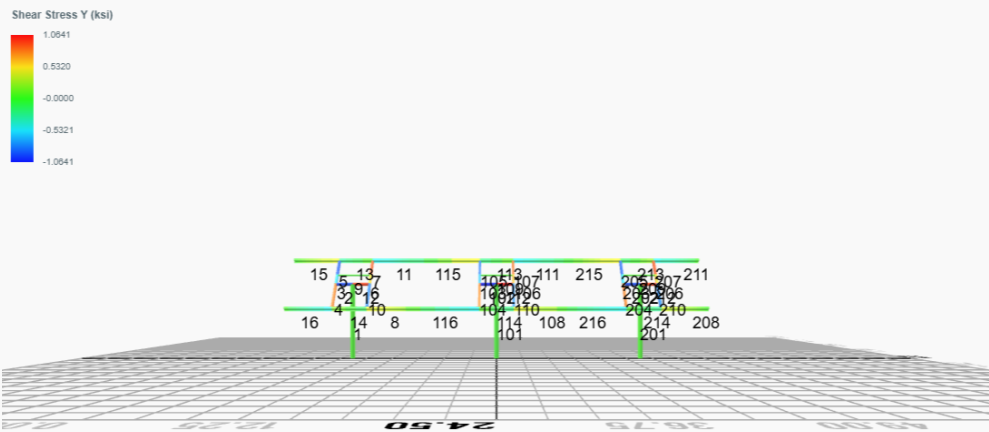


FEM Results (Envelope Worst Case for each member)

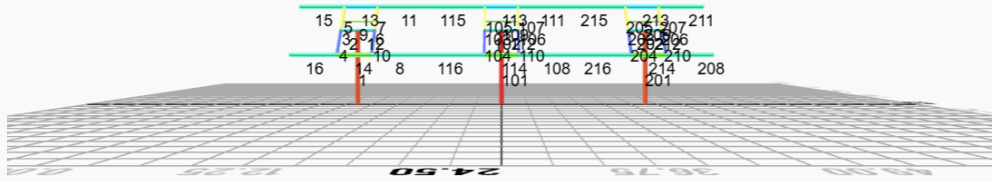




 SkyCiv



 SkyCiv



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 2. D + L	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 3. D + (S or Lr or R)	-0.0166	4.7788	-0.0143	-0.0320	0.0883	0.1571
ULS: 3. D + (S or Lr or R)	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0137	4.0107	-0.0119	-0.0265	0.0731	0.1315
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 5b. D + 0.7E	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0137	4.0107	-0.0119	-0.0265	0.0731	0.1315
ULS: 8. 0.6D + 0.7E	-0.0031	1.0239	-0.0027	-0.0061	0.0166	0.0327
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7205	3.1421	-0.0127	-0.0308	0.0506	15.2558
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7099	0.2710	0.0038	0.0108	0.0042	-14.8194
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3002	5.0874	-0.0180	-0.0420	0.0903	11.5324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0137	4.0107	-0.0119	-0.0265	0.0731	0.1315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2726	2.9340	-0.0056	-0.0108	0.0555	-11.0239
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0137	4.0107	-0.0119	-0.0265	0.0731	0.1315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2917	2.7832	-0.0107	-0.0257	0.0449	11.4555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2811	0.6298	0.0017	0.0056	0.0101	-11.1009
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0052	1.7065	-0.0045	-0.0102	0.0277	0.0545
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7184	2.4595	-0.0109	-0.0268	0.0395	15.2339
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0031	1.0239	-0.0027	-0.0061	0.0166	0.0327
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7119	-0.4116	0.0056	0.0149	-0.0069	-14.8412
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0031	1.0239	-0.0027	-0.0061	0.0166	0.0327

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	8.1599
Shear X	-2.8712
Shear Z	-0.0278
Moment X	-0.0643
Moment Y (Twist)	0.1486
Moment Z	25.9277

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	5.0874
Shear X	-1.7205
Shear Z	-0.0180
Moment X	-0.0420
Moment Y (Twist)	0.0903
Moment Z	15.2558

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	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 2. D + L	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 3. D + (S or Lr or R)	0.0330	5.0045	-0.0002	-0.0009	0.0003	-0.2128
ULS: 3. D + (S or Lr or R)	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0274	4.1976	-0.0002	-0.0008	0.0003	-0.1749

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 5b. D + 0.7E	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0274	4.1976	-0.0002	-0.0008	0.0003	-0.1749
ULS: 8. 0.6D + 0.7E	0.0062	1.0661	-0.0000	-0.0002	0.0001	-0.0368
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7379	3.2511	-0.0002	-0.0011	0.0003	15.3942
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7592	0.3022	0.0001	0.0005	-0.0001	-15.1833
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2838	5.3033	-0.0003	-0.0014	0.0004	11.4167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0274	4.1976	-0.0002	-0.0008	0.0003	-0.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3390	3.0917	-0.0000	-0.0002	0.0001	-11.5164
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0274	4.1976	-0.0002	-0.0008	0.0003	-0.1749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3008	2.8825	-0.0002	-0.0009	0.0002	11.5303
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3220	0.6709	0.0001	0.0003	-0.0000	-11.4028
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0104	1.7768	-0.0001	-0.0003	0.0001	-0.0614
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7421	2.5404	-0.0002	-0.0010	0.0002	15.4187
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0062	1.0661	-0.0000	-0.0002	0.0001	-0.0368
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7550	-0.4085	0.0001	0.0007	-0.0001	-15.1587
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0062	1.0661	-0.0000	-0.0002	0.0001	-0.0368

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.5249
Shear X	-2.9375
Shear Z	-0.0004
Moment X	-0.0021
Moment Y (Twist)	0.0007
Moment Z	26.0961

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.3033
Shear X	-1.7592
Shear Z	-0.0003
Moment X	-0.0014
Moment Y (Twist)	0.0004
Moment Z	15.4187

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	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 2. D + L	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 3. D + (S or Lr or R)	-0.0165	4.7787	0.0145	0.0315	-0.0902	0.1564
ULS: 3. D + (S or Lr or R)	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0137	4.0106	0.0120	0.0261	-0.0747	0.1308
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 5b. D + 0.7E	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0137	4.0106	0.0120	0.0261	-0.0747	0.1308
ULS: 8. 0.6D + 0.7E	-0.0031	1.0239	0.0027	0.0060	-0.0170	0.0326
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7204	3.1420	0.0129	0.0303	-0.0525	15.2549
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7098	0.2710	-0.0039	-0.0106	-0.0034	-14.8190
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3001	5.0873	0.0183	0.0414	-0.0929	11.5313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0137	4.0106	0.0120	0.0261	-0.0747	0.1308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2726	2.9340	0.0056	0.0106	-0.0561	-11.0241
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0137	4.0106	0.0120	0.0261	-0.0747	0.1308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2916	2.7831	0.0108	0.0253	-0.0465	11.4547
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2811	0.6299	-0.0018	-0.0055	-0.0097	-11.1007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0052	1.7065	0.0046	0.0100	-0.0283	0.0543
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7183	2.4594	0.0111	0.0263	-0.0412	15.2332
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0031	1.0239	0.0027	0.0060	-0.0170	0.0326
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7119	-0.4116	-0.0058	-0.0146	0.0079	-14.8407
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0031	1.0239	0.0027	0.0060	-0.0170	0.0326

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.1597
Shear X	-2.8710
Shear Z	0.0282
Moment X	0.0633
Moment Y (Twist)	0.1525
Moment Z	25.9266

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.0873
Shear X	-1.7204
Shear Z	0.0183
Moment X	0.0414
Moment Y (Twist)	0.0929
Moment Z	15.2549

Project Details

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 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial

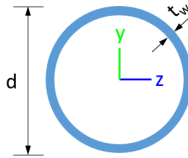


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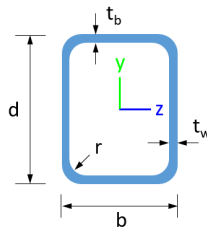
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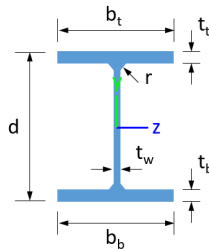
Section Dimensions



ID	Name	d (in)	t _w (in)				
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4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



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)
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101	251.10	123.93	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.89	10.99	6.26	29.14	16.61
104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.53	6.45	30.09	45.74
114	120.60	84.03	18.62	6.45	30.09	45.74
115	120.60	59.18	19.42	6.45	30.09	45.74
116	120.60	84.26	20.00	6.45	30.09	45.74
201	251.16	123.93	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	10.99	6.26	29.14	16.61
208	120.60	54.44	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.84	10.99	6.26	29.14	16.61
211	120.60	54.44	23.36	6.45	30.09	45.74
212	142.83	140.22	16.17	16.17	42.85	42.85
213	120.60	84.03	19.65	6.45	30.09	45.74
214	120.60	84.03	19.69	6.45	30.09	45.74
215	120.60	59.18	20.47	6.45	30.09	45.74
216	120.60	59.18	20.71	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	0.066	0.613	0.004	0.038	0.000	0.638	#13	0.491	Not Required	Pass
2	0.002	0.380	0.193	0.093	0.034	0.526	#21	0.053	Not Required	Pass
3	0.017	0.539	0.133	0.054	0.024	0.681	#21	0.044	Not Required	Pass
4	0.016	0.534	0.247	0.054	0.042	0.713	#21	0.078	Not Required	Pass
5	0.017	0.335	0.256	0.053	0.049	0.391	#21	0.073	Not Required	Pass
6	0.017	0.526	0.138	0.052	0.024	0.671	#21	0.044	Not Required	Pass
7	0.017	0.327	0.249	0.052	0.049	0.392	#21	0.073	Not Required	Pass
8	0.000	0.048	0.104	0.031	0.016	0.152	#21	0.088	Not Required	Pass
9	0.010	0.035	0.053	0.001	0.000	0.091	#21	0.198	Not Required	Pass
10	0.016	0.522	0.244	0.053	0.043	0.718	#21	0.078	Not Required	Pass
11	0.000	0.047	0.102	0.031	0.016	0.150	#21	0.088	Not Required	Pass
12	0.002	0.369	0.187	0.093	0.034	0.506	#21	0.034	Not Required	Pass
13	0.007	0.153	0.368	0.042	0.022	0.513	#21	0.265	Not Required	Pass
14	0.008	0.156	0.368	0.042	0.022	0.513	#21	0.177	Not Required	Pass
15	0.000	0.057	0.171	0.024	0.013	0.228	#21	Not Required	Not Required	Pass

16	0.000	0.057	0.171	0.024	0.013	0.228	#21	Not Required	Not Required	Pass
101	0.069	0.617	0.000	0.039	0.000	0.642	#13	0.491	Not Required	Pass
102	0.001	0.388	0.189	0.097	0.034	0.526	#21	0.034	Not Required	Pass
103	0.018	0.551	0.153	0.055	0.031	0.713	#21	0.044	Not Required	Pass
104	0.017	0.543	0.228	0.055	0.041	0.729	#21	0.078	Not Required	Pass
105	0.018	0.343	0.232	0.055	0.045	0.395	#21	0.073	Not Required	Pass
106	0.018	0.551	0.153	0.055	0.031	0.713	#21	0.044	Not Required	Pass
107	0.018	0.342	0.232	0.055	0.045	0.395	#21	0.073	Not Required	Pass
108	0.000	0.055	0.105	0.029	0.016	0.160	#21	0.088	Not Required	Pass
109	0.005	0.029	0.051	0.001	0.000	0.079	#21	0.198	Not Required	Pass
110	0.017	0.543	0.228	0.055	0.041	0.729	#21	0.078	Not Required	Pass
111	0.000	0.054	0.104	0.029	0.016	0.157	#21	0.088	Not Required	Pass
112	0.001	0.388	0.189	0.097	0.034	0.526	#21	0.034	Not Required	Pass
113	0.006	0.117	0.322	0.040	0.022	0.426	#21	0.265	Not Required	Pass
114	0.008	0.117	0.321	0.040	0.022	0.423	#21	0.265	Not Required	Pass
115	0.000	0.092	0.191	0.029	0.016	0.282	#21	0.493	Not Required	Pass
116	0.000	0.093	0.192	0.029	0.016	0.285	#21	0.321	Not Required	Pass
201	0.066	0.613	0.004	0.038	0.000	0.638	#13	0.491	Not Required	Pass
202	0.002	0.368	0.187	0.093	0.034	0.505	#21	0.034	Not Required	Pass
203	0.017	0.526	0.138	0.052	0.024	0.671	#21	0.044	Not Required	Pass
204	0.016	0.521	0.244	0.052	0.043	0.718	#21	0.078	Not Required	Pass
205	0.017	0.327	0.249	0.052	0.049	0.392	#21	0.073	Not Required	Pass
206	0.017	0.539	0.133	0.054	0.024	0.681	#21	0.044	Not Required	Pass
207	0.017	0.335	0.256	0.053	0.049	0.391	#21	0.073	Not Required	Pass
208	0.000	0.057	0.171	0.024	0.013	0.228	#21	Not Required	Not Required	Pass
209	0.010	0.035	0.053	0.001	0.000	0.091	#21	0.198	Not Required	Pass
210	0.016	0.534	0.247	0.054	0.042	0.714	#21	0.078	Not Required	Pass
211	0.000	0.057	0.171	0.024	0.013	0.228	#21	Not Required	Not Required	Pass
212	0.002	0.380	0.193	0.093	0.034	0.526	#21	0.053	Not Required	Pass
213	0.007	0.153	0.368	0.042	0.022	0.513	#21	0.177	Not Required	Pass
214	0.008	0.156	0.368	0.042	0.022	0.513	#21	0.265	Not Required	Pass
215	0.000	0.090	0.190	0.031	0.016	0.280	#21	0.493	Not Required	Pass
216	0.000	0.091	0.191	0.031	0.016	0.283	#21	0.493	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)

M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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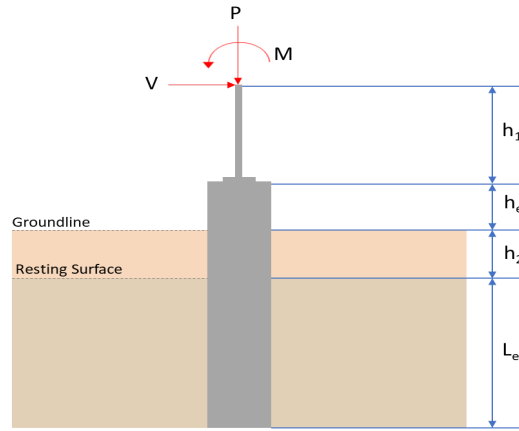
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.087	8.160
V_x (kip)	-1.721	-2.871
V_z (kip)	-0.018	-0.028
M_x (kipft)	-0.042	-0.064
M_z (kipft)	15.256	25.928

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.721 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27404 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.256 \text{ kipft}) + ((-1.721 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.4293 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8559 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.042 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0066879 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.7414 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8559 \text{ ft}), (0.7414 \text{ ft})]$$

$$L_{e,req} = 4.856 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.856 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.92495$$

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

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

$$q = \frac{(5.087 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31794 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31794 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15897$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.27404 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.4293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4293 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.27404 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.4293 \text{ kipft/ft})) + (4 \times (-0.27404 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6238 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.4293 \text{ kipft/ft})) + (3 \times (-0.27404 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.4293 \text{ kipft/ft})) + (2 \times (-0.27404 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.17997 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.4293 \text{ kipft/ft})) + ((-0.27404 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74446 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6238 \text{ ft})}{2}$$

$$p_a = 0.27179 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.17997 \text{ kip/ft}^2)}{(0.27179 \text{ kip/ft}^2)}$$

$$Ratio = 0.66218$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.74446 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.94535$$

Status: **PASS**
Ratio: **0.660**

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = -0.0028662 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0066879 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0066879 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0066879 \text{ kipft/ft})) + (4 \times (-0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7625 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0066879 \text{ kipft/ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.0066879 \text{ kipft/ft})) + (2 \times (-0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = -0.0009175 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0066879 \text{ kipft/ft})) + ((-0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = -0.00036397 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7625 \text{ ft})}{2}$$

$$p_a = 0.28219 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0009175 \text{ kip/ft}^2)}{(0.28219 \text{ kip/ft}^2)}$$

$$Ratio = -0.0032514$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

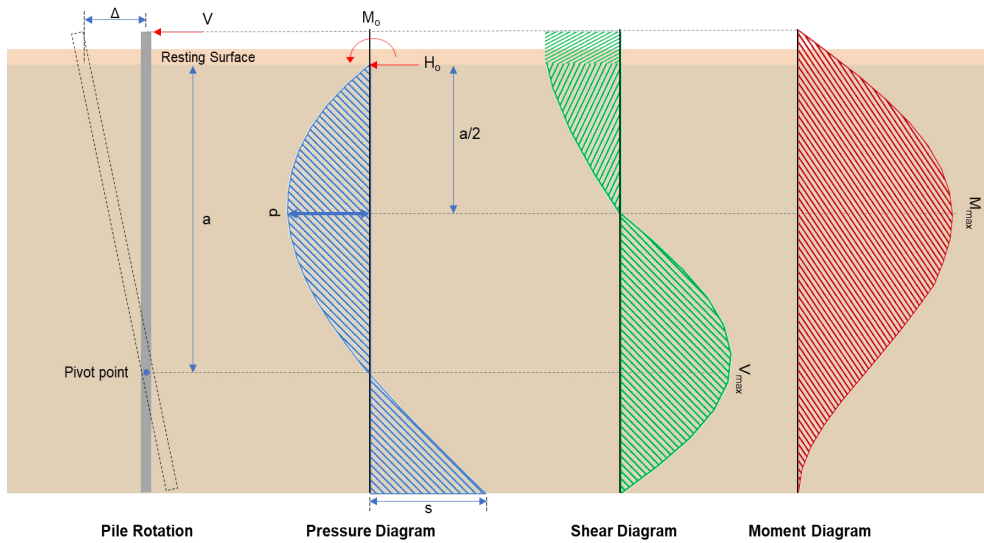
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00036397 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00046218$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.871 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45717 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.928 \text{ kipft}) + ((-2.871 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1287 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1287 \text{ kipft/ft})}{(-0.45717 \text{ kip/ft})}$$

$$E = 9.031 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1287 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.1287 \text{ kipft/ft})) + (4 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(6 \times (4.1287 \text{ kipft/ft})) + (4 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft}))}{}$$

$$a = 3.6222 \text{ ft}$$

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

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

$$V_{max} = ((-0.45717 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.031 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6222 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.031 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6222 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7723 \text{ kip}$$

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

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

$$M_{max} = ((-0.45717 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(9.031 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.031 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.031 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 16.906 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.028 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0044586 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((-0.028 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010191 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010191 \text{ kipft/ft})}{(-0.0044586 \text{ kip/ft})}$$

$$E = 2.2857 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010191 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0044586 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.010191 \text{ kipft/ft})) + (4 \times (-0.0044586 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7647 \text{ ft}$$

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

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

$$V_{max} = ((-0.0044586 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2857 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7647 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.2857 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7647 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.025647 \text{ kip}$$

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

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

$$M_{max} = ((-0.0044586 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.2857 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.2857 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2857 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.058988 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.16 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.325 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.325 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.16 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0030503$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.16 \text{ kip} \rightarrow 8160 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8160 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.7723 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.7723 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.06112$$

Status: **PASS**
Ratio: **0.060**

Considering z-direction:

$V_{max} = 0.025647 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.025647 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.00023147$$

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:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 16.906 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.906 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.067732$$

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

Considering z-direction:

$M_{max} = 0.058988 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.058988 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00023633$$

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

REFERENCES	CALCULATIONS	RESULTS
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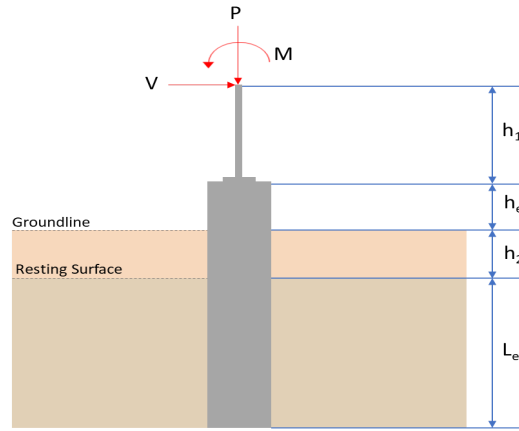
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.303	8.525
V_x (kip)	-1.759	-2.938
V_z (kip)	0.000	0.000
M_x (kipft)	-0.001	-0.002
M_z (kipft)	15.419	26.096

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.759 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.2801 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.419 \text{ kipft}) + ((-1.759 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.4553 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.8594 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00015924 \text{ kipft/ft}$$

L_e - Required depth of embedment in earth,

$$L_e = 2.29 \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.00015924 \text{ kipft/ft})}{(150 \text{ psf/ft})}}$$

$$L_e = 0.23361 \text{ ft}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8594 \text{ ft}), (0.23361 \text{ ft})]$$

$$L_{e,req} = 4.859 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.859 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.92552$$

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

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

$$q = \frac{(5.303 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33144 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16572$$

Status: **PASS**
Ratio: **0.170**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.2801 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.4553 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4553 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.2801 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.4553 \text{ kipft/ft})) + (4 \times (-0.2801 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6248 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.4553 \text{ kipft/ft})) + (3 \times (-0.2801 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.4553 \text{ kipft/ft})) + (2 \times (-0.2801 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.17996 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.4553 \text{ kipft/ft})) + ((-0.2801 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6248 \text{ ft})}{2}$$

$$p_a = 0.27186 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17996 \text{ kip/ft}^2)}{(0.27186 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66194$$

Status: **PASS**
Ratio: **0.660**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.74885 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95091$$

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = 0 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.00015924 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

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

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

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

$$p = \frac{0.75 \times [(4 \times (0.00015924 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.00015924 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.000023109 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.00015924 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.000069327 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5 \text{ ft})}{2}$$

$$p_a = 0.2625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000023109 \text{ kip/ft}^2)}{(0.2625 \text{ kip/ft}^2)}$$

$$Ratio = 0.000088034$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

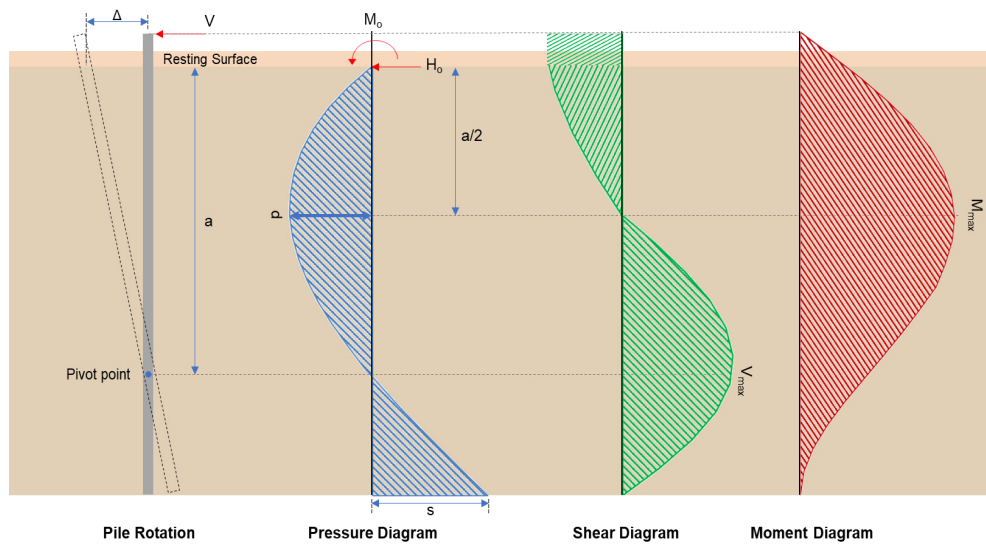
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.000069327 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.000088034$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.938 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.46783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.096 \text{ kipft}) + ((-2.938 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1554 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1554 \text{ kipft/ft})}{(-0.46783 \text{ kip/ft})}$$

$$E = 8.8822 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$= (6 M_o) + (4 H_o L_e)$$

$$a = \frac{(4 \times (4.1554 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.46783 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.1554 \text{ kipft/ft})) + (4 \times (-0.46783 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6237 \text{ ft}$$

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

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

$$V_{max} = ((-0.46783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8822 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6237 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8822 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6237 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.8365 \text{ kip}$$

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

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

$$M_{max} = ((-0.46783 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(8.8822 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6237 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8822 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6237 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8822 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6237 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 17.054 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00031847 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

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

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

$$V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(5.25 \text{ ft})} \right) \times \left(\frac{(3.5 \text{ ft})}{(5.25 \text{ ft})} - 1 \right) \times \left(\frac{(3.5 \text{ ft})}{(5.25 \text{ ft})} \right)^2$$

$$V_{max} = 0.00043137 \text{ kip}$$

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

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

$$M_{max} = (M_o) \left[1 - \left(\frac{4}{2 L_e} \right) + \left(\frac{3}{2 L_e} \right) \right]$$

$$M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(3.5 \text{ ft})^3}{2 \times (5.25 \text{ ft})} \right) + \left(3 \times \frac{(3.5 \text{ ft})^4}{2 \times (5.25 \text{ ft})} \right) \right]$$

$$M_{max} = 0.0011323 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{8.525 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.313 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.313 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

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

Summary:

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

Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.525 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0031867$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 $b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.525 \text{ kip} \rightarrow 8525 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8525 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.62 \text{ kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.62 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.62 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.62 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.84 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.8365 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.8365 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.061681$$

Status: **PASS**
Ratio: **0.060**

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$$\phi M_{n,1}$$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$$\phi M_{n,2}$$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 17.054 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.054 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.068325$$

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

Considering z-direction:

$M_{max} = 0.0011323 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.0011323 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 4.5366 \times 10^{-6}$$

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

REFERENCES	CALCULATIONS	RESULTS
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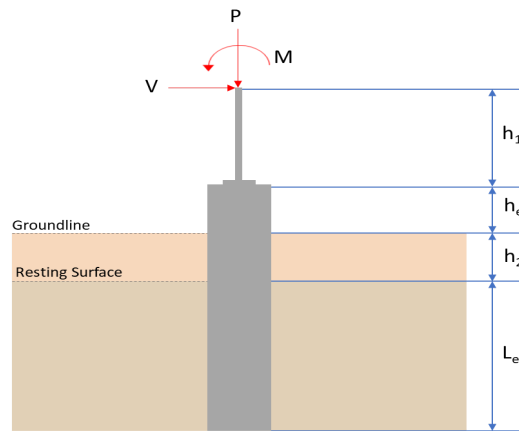
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.087	8.160
V_x (kip)	-1.720	-2.871
V_z (kip)	0.018	0.028
M_x (kipft)	0.041	0.063
M_z (kipft)	15.255	25.927

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.72 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27389 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.255 \text{ kipft}) + ((-1.72 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.4291 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.8563 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.041 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0065287 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.87648 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.8563 \text{ ft}), (0.87648 \text{ ft})]$$

$$L_{e,req} = 4.856 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.856 \text{ ft})}{(5.25 \text{ ft})}$$

$$Ratio = 0.92495$$

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

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

$$q = \frac{(5.087 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31794 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31794 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15897$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.27389 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.4291 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4291 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.27389 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.4291 \text{ kipft/ft})) + (4 \times (-0.27389 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6238 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.4291 \text{ kipft/ft})) + (3 \times (-0.27389 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.4291 \text{ kipft/ft})) + (2 \times (-0.27389 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.18005 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.4291 \text{ kipft/ft})) + ((-0.27389 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74457 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6238 \text{ ft})}{2}$$

$$p_a = 0.27178 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18005 \text{ kip/ft}^2)}{(0.27178 \text{ kip/ft}^2)}$$

$$Ratio = 0.66247$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.74457 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.94549$$

Status: **PASS**
Ratio: **0.660**

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = 0.0028662 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0065287 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0065287 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0065287 \text{ kipft/ft})) + (4 \times (0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.765 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0065287 \text{ kipft/ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.0065287 \text{ kipft/ft})) + (2 \times (0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.0027811 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0065287 \text{ kipft/ft})) + ((0.0028662 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.0061181 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.765 \text{ ft})}{2}$$

$$p_a = 0.28238 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0027811 \text{ kip/ft}^2)}{(0.28238 \text{ kip/ft}^2)}$$

$$Ratio = 0.0098488$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

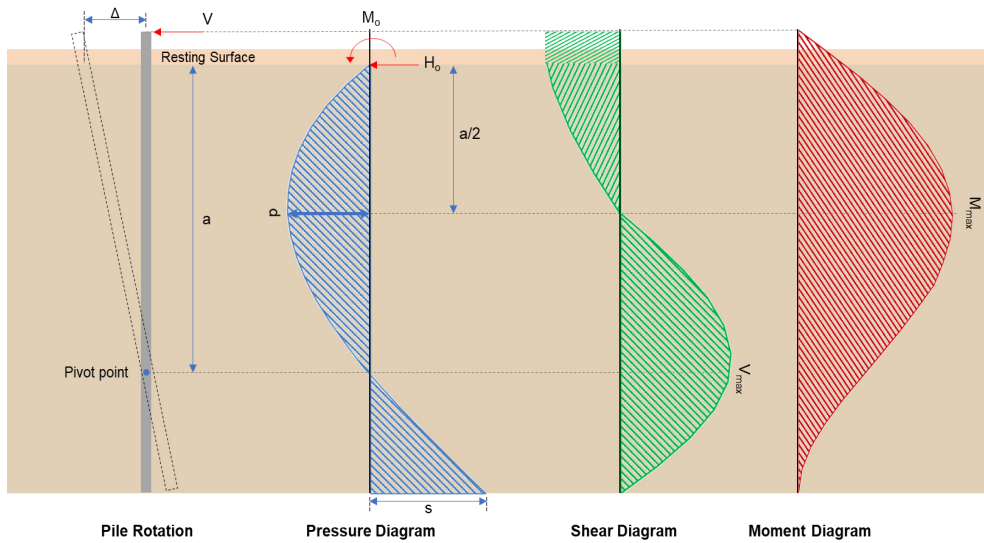
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0061181 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.007769$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.871 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45717 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.927 \text{ kipft}) + ((-2.871 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1285 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1285 \text{ kipft/ft})}{(-0.45717 \text{ kip/ft})}$$

$$E = 9.0307 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1285 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.1285 \text{ kipft/ft})) + (4 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(-0.45717 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.1285 \text{ kipft/ft})) + (4 \times (-0.45717 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6222 \text{ ft}$$

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

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

$$V_{max} = ((-0.45717 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.0307 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6222 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.0307 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6222 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.7721 \text{ kip}$$

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

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

$$M_{max} = ((-0.45717 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(9.0307 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.0307 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.0307 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6222 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 16.905 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.028 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0044586 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.063 \text{ kipft}) + ((0.028 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010032 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010032 \text{ kipft/ft})}{(0.0044586 \text{ kip/ft})}$$

$$E = 2.25 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010032 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0044586 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.010032 \text{ kipft/ft})) + (4 \times (0.0044586 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7663 \text{ ft}$$

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

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

$$V_{max} = ((0.0044586 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.25 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7663 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.25 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7663 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.025436 \text{ kip}$$

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

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

$$M_{max} = ((0.0044586 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.25 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7663 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.25 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7663 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.25 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7663 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.058434 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.16 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.325 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.325 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.16 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0030503$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.16 \text{ kip} \rightarrow 8160 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8160 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.7721 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.7721 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.061118$$

Status: **PASS**
Ratio: **0.060**

Considering z-direction:

$V_{max} = 0.025436 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.025436 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.00022956$$

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:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 16.905 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.905 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.067729$$

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

Considering z-direction:

$M_{max} = 0.058434 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.058434 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00023411$$

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