

Project Details



Project Name: BT SIL530

Date: Thu Jun 05 2025

Location: 31842 Frontage Rd, Bozeman, MT 59715, USA

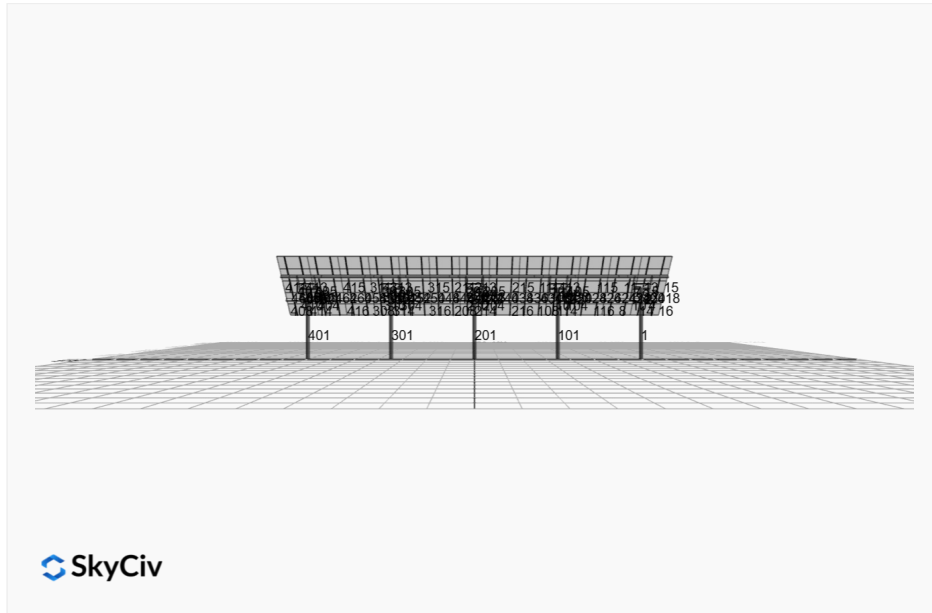
Number of Modules: 65

Unique ID: 5P-19.75-8TOP-HD-24-L-5Hx13W-3I2B

Number of Poles: 5

Dealer: _____

Date Sold: _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	90.57 ft
Winter Tilt Angle	46
Front Edge Clearance	10 ft

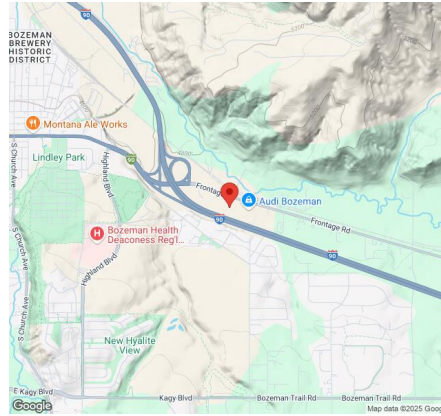
MT Solar Bill of Materials (5P-19.75-8TOP-HD-24-L-5Hx13W-3I2B)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (226in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: 31842 Frontage Rd, Bozeman, MT 59715, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	46
Front Edge Clearance:	10
Total Array Height at Tilt:	23.52 ft
Total Frame Length:	90.50 ft
Module Info/Notes:	sil-530-xm
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	90.57 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	16.76 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.50 ft Pile 3: 8.50 ft Pile 4: 8.50 ft Pile 5: 8.00 ft
Foundation Volume:	24.593 y ³

Site Info

Risk Category:	II
Exposure:	C
Soil Classification:	sand
Site Location:	31842 Frontage Rd, Bozeman, MT 59715, USA
Wind Speed:	115 mph

Snow Load:

51 psf

Design Disclaimer

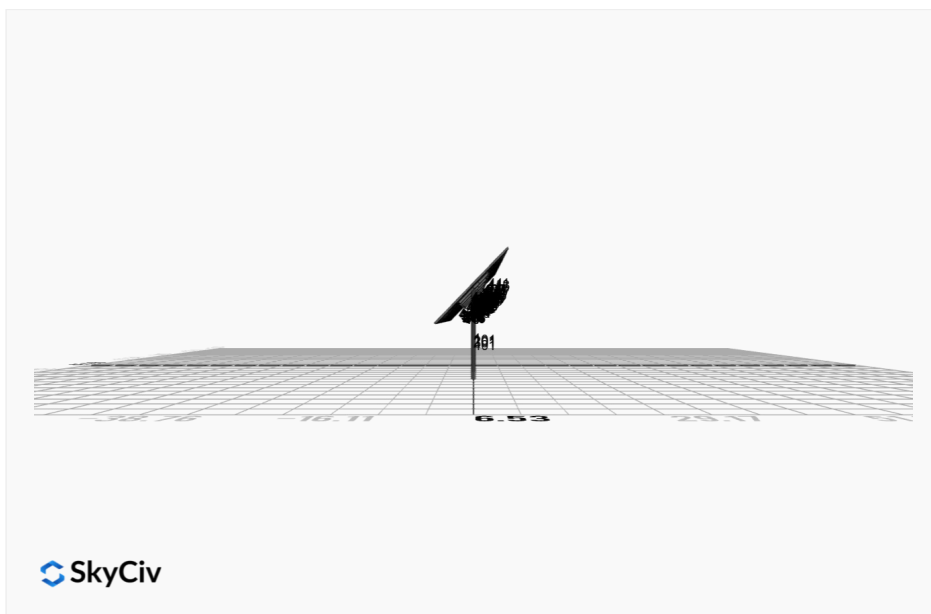
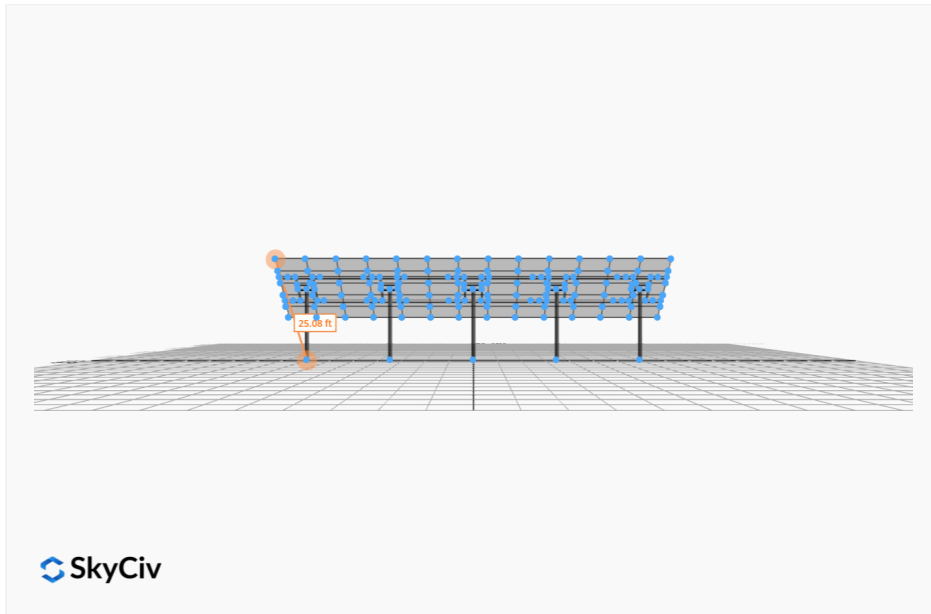
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

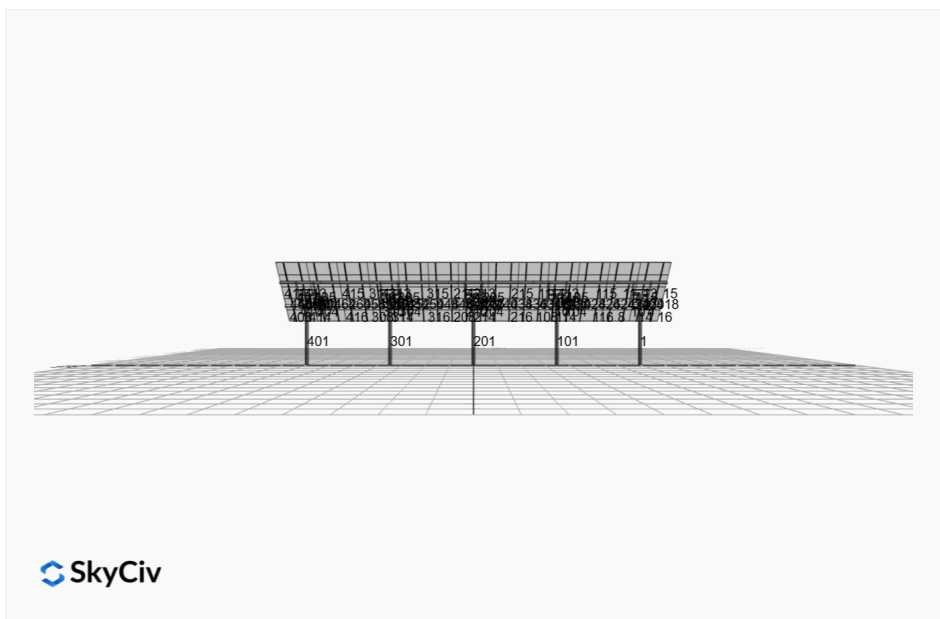
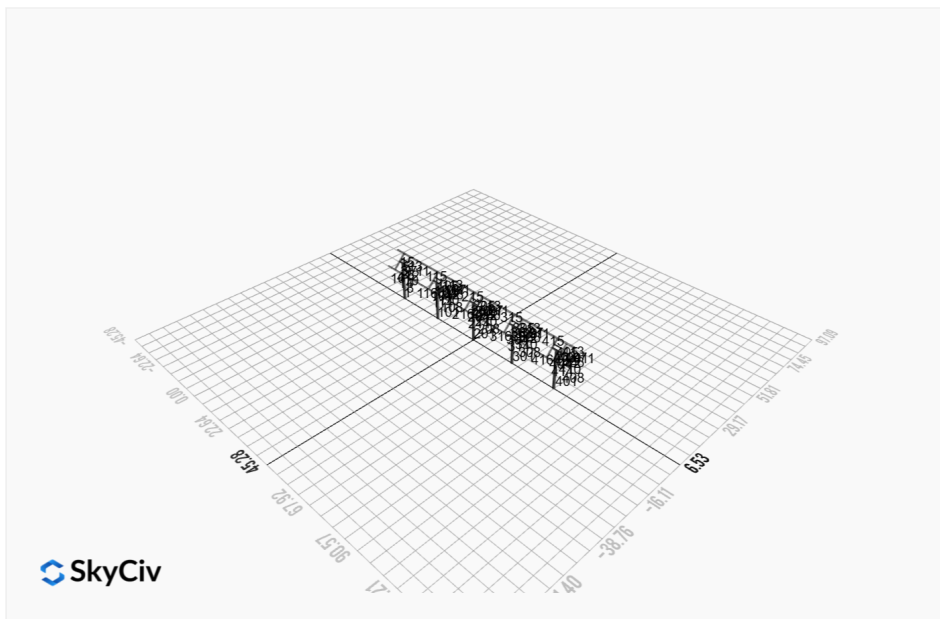
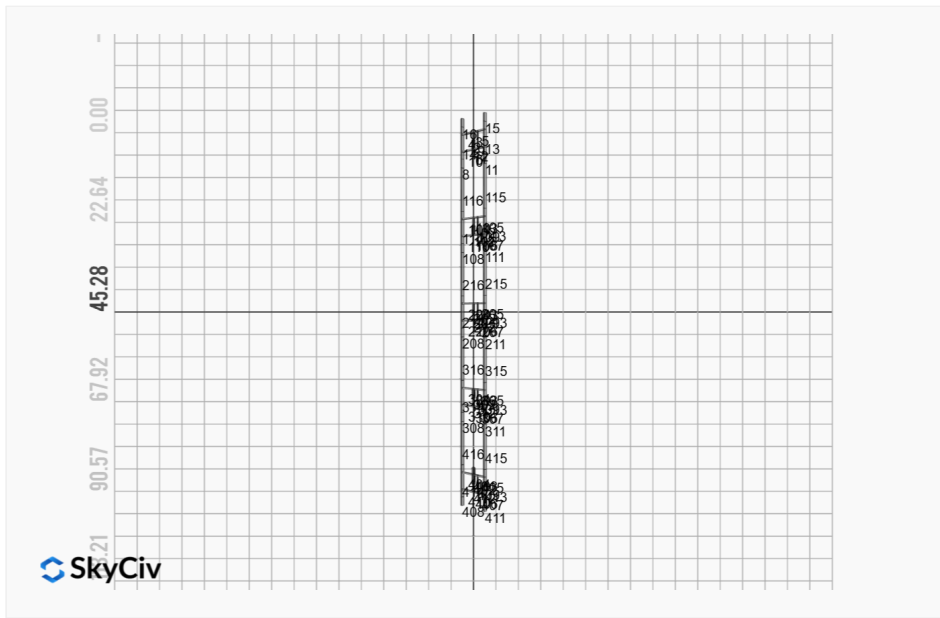
AutoDesigner Input

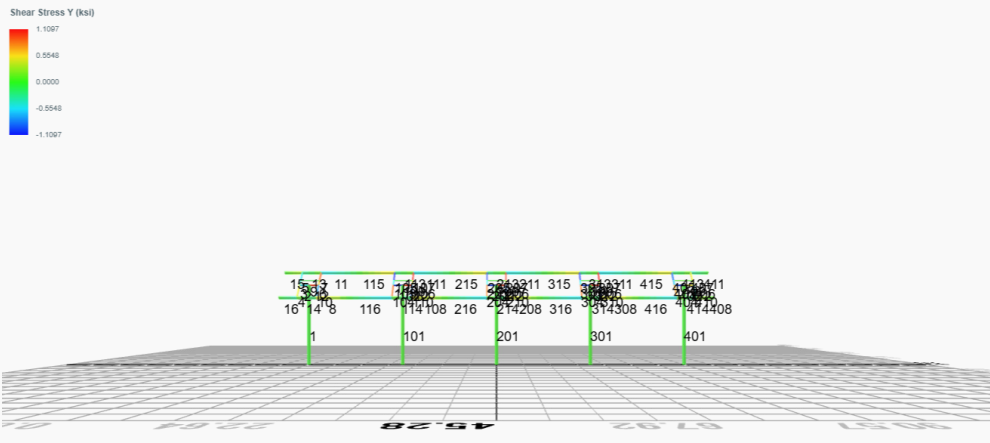
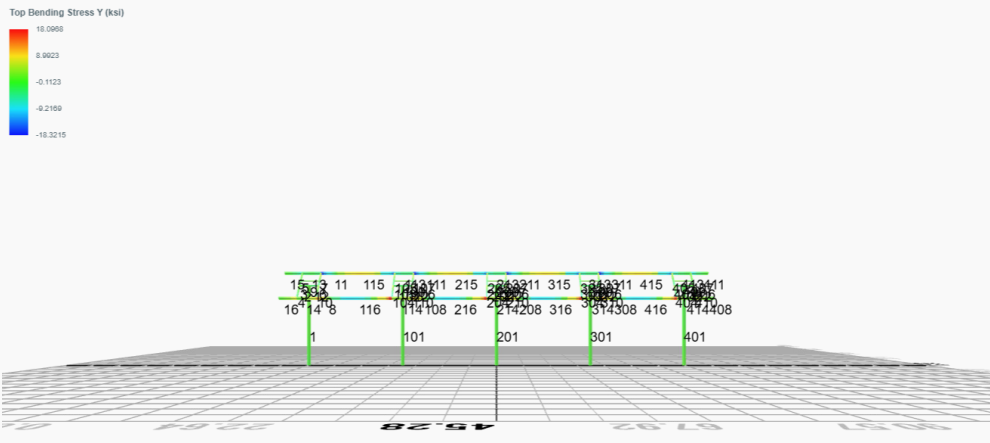
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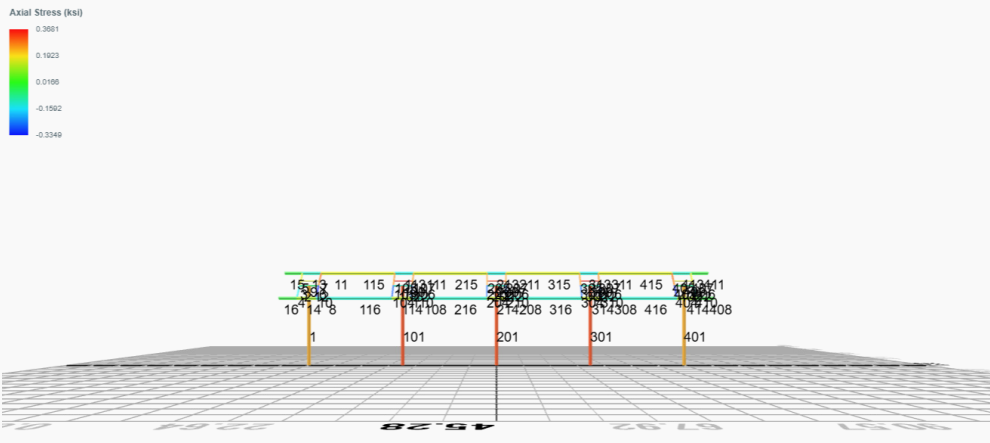
Design Notes:

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









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	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 2. D + L	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 3. D + (S or Lr or R)	0.0720	5.9064	0.1885	1.0049	-0.3261	-1.1173
ULS: 3. D + (S or Lr or R)	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0598	5.0637	0.1564	0.8341	-0.2706	-0.9249
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 5b. D + 0.7E	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0598	5.0637	0.1564	0.8341	-0.2706	-0.9249
ULS: 8. 0.6D + 0.7E	0.0139	1.5214	0.0362	0.1929	-0.0625	-0.2085
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9028	5.2625	0.2877	1.4911	-1.5091	50.0136
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9371	-0.1856	-0.1574	-0.7955	1.2414	-48.4982
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1347	7.1089	0.3269	1.7112	-1.3243	36.8459
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0598	5.0637	0.1564	0.8341	-0.2706	-0.9249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2453	3.0228	-0.0069	-0.0037	0.7385	-37.0379
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0598	5.0637	0.1564	0.8341	-0.2706	-0.9249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1713	4.5807	0.2309	1.1987	-1.1578	37.4234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2086	0.4947	-0.1030	-0.5162	0.9050	-36.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0232	2.5356	0.0604	0.3216	-0.1041	-0.3474
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9121	4.2482	0.2635	1.3625	-1.4674	50.1526
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0139	1.5214	0.0362	0.1929	-0.0625	-0.2085
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9278	-1.1999	-0.1816	-0.9241	1.2831	-48.3592
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0139	1.5214	0.0362	0.1929	-0.0625	-0.2085

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	10.7110
Shear X	-4.9176
Shear Z	0.5229
Moment X	2.7193
Moment Y (Twist)	2.6244
Moment Z	85.2254

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	7.1089
Shear X	-2.9371
Shear Z	0.3269
Moment X	1.7112
Moment Y (Twist)	1.5091
Moment Z	50.1526

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	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 2. D + L	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 3. D + (S or Lr or R)	-0.0615	7.4136	-0.0068	-0.0390	0.0389	0.9891
ULS: 3. D + (S or Lr or R)	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0510	6.3151	-0.0057	-0.0324	0.0324	0.8230

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 5b. D + 0.7E	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0510	6.3151	-0.0057	-0.0324	0.0324	0.8230
ULS: 8. 0.6D + 0.7E	-0.0119	1.8119	-0.0013	-0.0076	0.0076	0.1948
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6188	6.5779	0.0453	0.2268	-0.3737	61.9179
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5900	-0.5446	-0.0426	-0.2148	0.3497	-58.3462
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7503	8.9837	0.0300	0.1471	-0.2574	47.0180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0510	6.3151	-0.0057	-0.0324	0.0324	0.8230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6563	3.6418	-0.0360	-0.1840	0.2852	-43.1801
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0510	6.3151	-0.0057	-0.0324	0.0324	0.8230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7191	5.6884	0.0334	0.1669	-0.2771	46.5196
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6875	0.3465	-0.0325	-0.1643	0.2655	-43.6785
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0198	3.0199	-0.0022	-0.0127	0.0127	0.3247
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6109	5.3700	0.0462	0.2318	-0.3788	61.7881
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0119	1.8119	-0.0013	-0.0076	0.0076	0.1948
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5979	-1.7526	-0.0417	-0.2098	0.3447	-58.4760
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0119	1.8119	-0.0013	-0.0076	0.0076	0.1948

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	13.6144
Shear X	-6.0329
Shear Z	0.0803
Moment X	0.4012
Moment Y (Twist)	0.6578
Moment Z	106.1106

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	8.9837
Shear X	-3.6188
Shear Z	0.0462
Moment X	0.2318
Moment Y (Twist)	0.3788
Moment Z	61.9179

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	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 2. D + L	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 3. D + (S or Lr or R)	-0.0212	7.3446	0.0000	-0.0001	0.0001	0.4881
ULS: 3. D + (S or Lr or R)	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0176	6.2578	0.0000	-0.0000	0.0001	0.4068
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 5b. D + 0.7E	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0176	6.2578	0.0000	-0.0000	0.0001	0.4068
ULS: 8. 0.6D + 0.7E	-0.0041	1.7986	0.0000	-0.0000	0.0000	0.0978
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6791	6.5765	0.0000	-0.0000	0.0000	63.4169
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.6682	-0.5794	0.0000	-0.0000	0.0000	-59.9122
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629

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	-2.7718	8.9420	0.0000	-0.0000	0.0001	47.8473
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0176	6.2578	0.0000	-0.0000	0.0001	0.4068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7387	3.5750	0.0000	-0.0000	0.0001	-44.6496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0176	6.2578	0.0000	-0.0000	0.0001	0.4068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7610	5.6818	0.0000	-0.0000	0.0000	47.6034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7495	0.3148	0.0000	-0.0000	0.0000	-44.8935
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0068	2.9977	0.0000	-0.0000	0.0000	0.1629
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6764	5.3774	0.0000	-0.0000	0.0000	63.3517
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0041	1.7986	0.0000	-0.0000	0.0000	0.0978
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6709	-1.7785	0.0000	-0.0000	0.0000	-59.9774
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0041	1.7986	0.0000	-0.0000	0.0000	0.0978

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	13.5382
Shear X	-6.1347
Shear Z	0.0000
Moment X	-0.0006
Moment Y (Twist)	0.0005
Moment Z	108.7414

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	8.9420
Shear X	-3.6791
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	63.4169

Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 2. D + L	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 3. D + (S or Lr or R)	-0.0615	7.4136	0.0069	0.0389	-0.0388	0.9891
ULS: 3. D + (S or Lr or R)	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0510	6.3151	0.0057	0.0324	-0.0323	0.8230
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 5b. D + 0.7E	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0510	6.3151	0.0057	0.0324	-0.0323	0.8230
ULS: 8. 0.6D + 0.7E	-0.0119	1.8119	0.0013	0.0076	-0.0076	0.1948
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6188	6.5779	-0.0453	-0.2268	0.3737	61.9179
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5900	-0.5446	0.0426	0.2148	-0.3497	-58.3462
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7503	8.9837	-0.0300	-0.1472	0.2575	47.0180
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0510	6.3151	0.0057	0.0324	-0.0323	0.8230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6563	3.6418	0.0360	0.1840	-0.2850	-43.1801
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0510	6.3151	0.0057	0.0324	-0.0323	0.8230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7191	5.6884	-0.0334	-0.1669	0.2771	46.5196
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6875	0.3465	0.0325	0.1643	-0.2655	-43.6784
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0198	3.0199	0.0022	0.0127	-0.0127	0.3247

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6109	5.3700	-0.0462	-0.2318	0.3788	61.7881
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0119	1.8119	0.0013	0.0076	-0.0076	0.1948
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5979	-1.7526	0.0417	0.2097	-0.3446	-58.4760
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0119	1.8119	0.0013	0.0076	-0.0076	0.1948

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	13.6144
Shear X	-6.0330
Shear Z	-0.0803
Moment X	-0.4020
Moment Y (Twist)	0.6582
Moment Z	106.1112

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	8.9837
Shear X	-3.6188
Shear Z	-0.0462
Moment X	-0.2318
Moment Y (Twist)	0.3788
Moment Z	61.9179

Reaction Forces for Foundation 5 (Node ID#401), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 2. D + L	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 3. D + (S or Lr or R)	0.0720	5.9064	-0.1885	-1.0051	0.3263	-1.1172
ULS: 3. D + (S or Lr or R)	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0598	5.0637	-0.1564	-0.8343	0.2707	-0.9248
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 5b. D + 0.7E	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0598	5.0637	-0.1564	-0.8343	0.2707	-0.9248
ULS: 8. 0.6D + 0.7E	0.0139	1.5214	-0.0362	-0.1930	0.0625	-0.2084
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9028	5.2625	-0.2877	-1.4911	1.5091	50.0136
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9371	-0.1856	0.1574	0.7955	-1.2414	-48.4982
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1347	7.1088	-0.3269	-1.7114	1.3244	36.8460
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0598	5.0637	-0.1564	-0.8343	0.2707	-0.9248
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2452	3.0228	0.0069	0.0035	-0.7384	-37.0378
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0598	5.0637	-0.1564	-0.8343	0.2707	-0.9248
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1713	4.5807	-0.2309	-1.1987	1.1578	37.4234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2086	0.4947	0.1030	0.5162	-0.9050	-36.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0232	2.5356	-0.0604	-0.3216	0.1041	-0.3474
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9121	4.2482	-0.2635	-1.3625	1.4674	50.1526
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0139	1.5214	-0.0362	-0.1930	0.0625	-0.2084
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9278	-1.1999	0.1816	0.9241	-1.2831	-48.3592
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0139	1.5214	-0.0362	-0.1930	0.0625	-0.2084

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.

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	10.7109
Shear X	-4.9175
Shear Z	-0.5229
Moment X	-2.7200
Moment Y (Twist)	2.6248
Moment Z	85.2268

Result	Value (kip, kip-ft)
Axial	7.1088
Shear X	-2.9371
Shear Z	-0.3269
Moment X	-1.7114
Moment Y (Twist)	1.5091
Moment Z	50.1526

Project Details

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

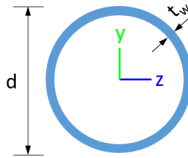


Design Input Information

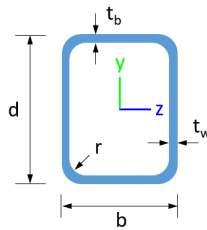
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

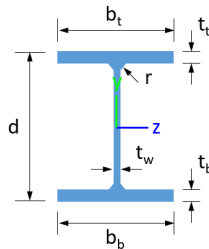
Section Dimensions



ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



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



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

Section Properties

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

113	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.11,1.06,1.20,1.06,1.12,1.06,1.15,1.06,1.09,1.06,1.07,1.06,1.11,1.06,1.21,1.06,1.12,1.06,1.14,1.06	300	200	1
114	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.08,1.05,1.12,1.05,1.09,1.05,1.10,1.05,1.07,1.05,1.06,1.05,1.08,1.05,1.12,1.05,1.09,1.05,1.10,1.05	300	200	1
115	19	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.10,1.14,1.08,1.14,1.09,1.14,1.08,1.14,1.12,1.14,1.32,1.14,1.10,1.14,1.08,1.14,1.09,1.14,1.08,1.14	300	200	1
116	19	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.13,1.15,1.12,1.15,1.13,1.15,1.13,1.15,1.14,1.15,1.20,1.15,1.13,1.15,1.12,1.15,1.13,1.15,1.13,1.15	300	200	1
201	10	35.19	35.19	16.76	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
208	19	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.08,2.09,2.07,2.09,2.08,2.09,2.07,2.09,2.09,2.09,2.09,2.37,2.09,2.08,2.09,2.07,2.09,2.08,2.09,2.09,2.08,2.09,2.09	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
211	19	1.33	1.33	2.05	2.10,2.10,2.10,2.11,2.10,2.10,1.96,2.10,1.64,2.10,1.90,2.10,1.77,2.10,2.07,2.11,1.81,2.11,2.00,2.10,1.62,2.10,1.89,2.10,1.80,2.10	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04	300	200	1
214	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04	300	200	1
215	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.14,1.17,1.12,1.17,1.14,1.17,1.13,1.17,1.15,1.17,1.44,1.17,1.14,1.17,1.12,1.17,1.14,1.17,1.13,1.17	300	200	1
216	19	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.16,1.16,1.18,1.16,1.16,1.16,1.15,1.16,1.15,1.16	300	200	1
301	10	35.19	35.19	16.76	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
304	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
305	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
306	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19	300	200	1
307	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
308	19	1.33	1.33	2.05	1.96,1.97,1.96,1.97,1.97,1.96,1.70,1.97,1.55,1.97,1.68,1.96,1.61,1.96,1.80,1.97,2.35,1.97,1.72,1.96,1.53,1.96,1.67,1.96,1.63,1.96	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
311	19	1.33	1.33	2.05	1.77,1.77,1.77,1.76,1.77,1.77,1.42,1.77,1.27,1.77,1.39,1.77,1.33,1.77,1.53,1.76,1.98,1.76,1.43,1.77,1.26,1.77,1.38,1.77,1.34,1.77	300	200	1
312	5	1.30	1.30	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.11,1.06,1.20,1.06,1.12,1.06,1.15,1.06,1.09,1.06,1.07,1.06,1.11,1.06,1.21,1.06,1.12,1.06,1.14,1.06	300	200	1
314	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.08,1.05,1.12,1.05,1.09,1.05,1.10,1.05,1.07,1.05,1.06,1.05,1.08,1.05,1.12,1.05,1.09,1.05,1.10,1.05	300	200	1

104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	24.31	6.12	40.24	43.62
114	133.20	85.85	24.16	6.12	40.24	43.62
115	133.20	69.16	16.66	6.12	40.24	43.62
116	133.20	69.16	17.30	6.12	40.24	43.62
201	574.32	133.90	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.56	6.12	40.24	43.62
214	133.20	85.85	23.70	6.12	40.24	43.62
215	133.20	69.16	17.38	6.12	40.24	43.62
216	133.20	69.16	17.80	6.12	40.24	43.62
301	574.32	133.90	123.94	123.94	172.30	172.30
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.31	6.12	40.24	43.62
314	133.20	85.85	24.16	6.12	40.24	43.62
315	133.20	69.16	17.07	6.12	40.24	43.62
316	133.20	69.16	17.47	6.12	40.24	43.62
401	574.32	133.90	123.94	123.94	172.30	172.30
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28

407	110.10	114.23	13.79	11.10	42.00	23.20
408	133.20	102.39	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	102.39	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	25.40	6.12	40.24	43.62
414	133.20	85.85	24.80	6.12	40.24	43.62
415	133.20	69.16	16.82	6.12	40.24	43.62
416	133.20	69.16	16.77	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.080	0.688	0.049	0.029	0.003	0.744	#13	0.734	Not Required	Pass
2	0.003	0.221	0.155	0.059	0.033	0.330	#13	0.035	Not Required	Pass
3	0.009	0.417	0.038	0.039	0.007	0.443	#13	0.045	Not Required	Pass
4	0.008	0.420	0.104	0.042	0.026	0.501	#13	0.080	Not Required	Pass
5	0.008	0.259	0.082	0.041	0.021	0.266	#21	0.074	Not Required	Pass
6	0.015	0.707	0.132	0.073	0.036	0.776	#13	0.045	Not Required	Pass
7	0.016	0.439	0.247	0.070	0.060	0.459	#13	0.074	Not Required	Pass
8	0.006	0.122	0.229	0.042	0.025	0.241	#23	0.095	Not Required	Pass
9	0.010	0.072	0.095	0.005	0.005	0.152	#13	0.204	Not Required	Pass
10	0.017	0.662	0.240	0.066	0.052	0.719	#21	0.080	Not Required	Pass
11	0.007	0.107	0.236	0.046	0.024	0.256	#23	0.095	Not Required	Pass
12	0.002	0.468	0.274	0.106	0.054	0.724	#13	0.053	Not Required	Pass
13	0.011	0.129	0.586	0.059	0.031	0.619	#21	0.286	Not Required	Pass
14	0.007	0.109	0.576	0.055	0.031	0.591	#23	0.190	Not Required	Pass
15	0.000	0.018	0.056	0.015	0.008	0.073	#21	Not Required	Not Required	Pass
16	0.000	0.018	0.056	0.015	0.008	0.073	#21	Not Required	Not Required	Pass
101	0.102	0.856	0.008	0.035	0.000	0.903	#13	0.734	Not Required	Pass
102	0.005	0.438	0.260	0.107	0.047	0.642	#13	0.035	Not Required	Pass
103	0.015	0.668	0.094	0.066	0.014	0.742	#21	0.045	Not Required	Pass
104	0.015	0.702	0.258	0.070	0.055	0.839	#21	0.080	Not Required	Pass
105	0.016	0.414	0.266	0.066	0.067	0.465	#21	0.074	Not Required	Pass
106	0.015	0.732	0.094	0.073	0.015	0.770	#13	0.045	Not Required	Pass
107	0.015	0.455	0.247	0.072	0.062	0.490	#13	0.074	Not Required	Pass
108	0.008	0.063	0.215	0.045	0.024	0.239	#21	0.095	Not Required	Pass
109	0.021	0.060	0.059	0.002	0.000	0.121	#13	0.204	Not Required	Pass
110	0.015	0.730	0.237	0.073	0.051	0.826	#21	0.080	Not Required	Pass
111	0.008	0.081	0.224	0.044	0.024	0.246	#23	0.095	Not Required	Pass
112	0.005	0.456	0.290	0.107	0.055	0.714	#13	0.035	Not Required	Pass
113	0.011	0.171	0.616	0.057	0.032	0.750	#21	0.286	Not Required	Pass
114	0.013	0.211	0.610	0.061	0.032	0.778	#21	0.286	Not Required	Pass
115	0.013	0.357	0.315	0.044	0.025	0.639	#21	0.473	Not Required	Pass
116	0.007	0.341	0.313	0.048	0.025	0.621	#21	0.473	Not Required	Pass
201	0.101	0.877	0.000	0.036	0.000	0.921	#13	0.734	Not Required	Pass
202	0.004	0.446	0.280	0.106	0.052	0.685	#13	0.035	Not Required	Pass
203	0.015	0.716	0.091	0.071	0.014	0.765	#13	0.045	Not Required	Pass
204	0.015	0.708	0.235	0.070	0.050	0.816	#21	0.080	Not Required	Pass
205	0.015	0.445	0.245	0.071	0.062	0.477	#13	0.074	Not Required	Pass

206	0.015	0.716	0.091	0.071	0.014	0.765	#13	0.045	Not Required	Pass
207	0.015	0.445	0.245	0.071	0.062	0.477	#13	0.074	Not Required	Pass
208	0.008	0.059	0.215	0.045	0.024	0.251	#21	0.095	Not Required	Pass
209	0.019	0.046	0.060	0.001	0.000	0.109	#13	0.204	Not Required	Pass
210	0.015	0.708	0.235	0.070	0.050	0.816	#21	0.080	Not Required	Pass
211	0.008	0.072	0.221	0.046	0.024	0.254	#21	0.095	Not Required	Pass
212	0.004	0.446	0.279	0.106	0.052	0.685	#13	0.035	Not Required	Pass
213	0.011	0.209	0.566	0.059	0.031	0.754	#21	0.286	Not Required	Pass
214	0.014	0.229	0.559	0.058	0.031	0.750	#21	0.286	Not Required	Pass
215	0.014	0.281	0.316	0.046	0.024	0.559	#21	0.473	Not Required	Pass
216	0.010	0.250	0.312	0.045	0.024	0.536	#21	0.473	Not Required	Pass
301	0.102	0.856	0.008	0.035	0.000	0.903	#13	0.734	Not Required	Pass
302	0.005	0.456	0.290	0.107	0.055	0.714	#13	0.035	Not Required	Pass
303	0.015	0.732	0.094	0.073	0.015	0.770	#13	0.045	Not Required	Pass
304	0.015	0.730	0.237	0.073	0.051	0.826	#21	0.080	Not Required	Pass
305	0.015	0.455	0.247	0.072	0.062	0.490	#13	0.074	Not Required	Pass
306	0.015	0.668	0.094	0.066	0.014	0.742	#21	0.045	Not Required	Pass
307	0.016	0.414	0.266	0.066	0.067	0.465	#21	0.074	Not Required	Pass
308	0.006	0.091	0.255	0.048	0.025	0.272	#21	0.095	Not Required	Pass
309	0.021	0.060	0.059	0.002	0.000	0.121	#13	0.204	Not Required	Pass
310	0.015	0.702	0.258	0.070	0.055	0.839	#21	0.080	Not Required	Pass
311	0.007	0.114	0.260	0.044	0.025	0.269	#21	0.095	Not Required	Pass
312	0.005	0.438	0.260	0.107	0.047	0.642	#13	0.035	Not Required	Pass
313	0.011	0.171	0.616	0.057	0.032	0.750	#21	0.286	Not Required	Pass
314	0.013	0.211	0.610	0.061	0.032	0.778	#21	0.286	Not Required	Pass
315	0.014	0.285	0.316	0.044	0.024	0.562	#21	0.473	Not Required	Pass
316	0.010	0.252	0.312	0.045	0.024	0.537	#21	0.473	Not Required	Pass
401	0.080	0.688	0.049	0.029	0.003	0.744	#13	0.734	Not Required	Pass
402	0.002	0.468	0.274	0.106	0.054	0.724	#13	0.053	Not Required	Pass
403	0.015	0.707	0.132	0.073	0.036	0.776	#13	0.045	Not Required	Pass
404	0.017	0.662	0.240	0.066	0.052	0.719	#21	0.080	Not Required	Pass
405	0.016	0.439	0.247	0.070	0.060	0.459	#13	0.074	Not Required	Pass
406	0.009	0.417	0.038	0.039	0.007	0.443	#13	0.045	Not Required	Pass
407	0.008	0.259	0.082	0.041	0.021	0.266	#21	0.074	Not Required	Pass
408	0.000	0.018	0.056	0.015	0.008	0.073	#21	Not Required	Not Required	Pass
409	0.010	0.072	0.095	0.005	0.005	0.152	#13	0.204	Not Required	Pass
410	0.008	0.420	0.104	0.042	0.026	0.501	#13	0.080	Not Required	Pass
411	0.000	0.018	0.056	0.015	0.008	0.073	#21	Not Required	Not Required	Pass
412	0.003	0.221	0.155	0.059	0.033	0.330	#13	0.035	Not Required	Pass
413	0.011	0.130	0.586	0.059	0.031	0.619	#21	0.190	Not Required	Pass
414	0.007	0.109	0.577	0.055	0.031	0.591	#23	0.286	Not Required	Pass
415	0.013	0.354	0.316	0.046	0.024	0.639	#21	0.473	Not Required	Pass
416	0.007	0.353	0.309	0.042	0.025	0.629	#21	0.473	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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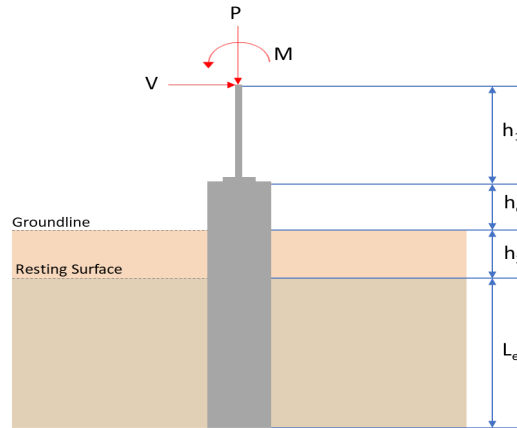
SkyCiv Foundation Design

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

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.109	10.711
V_x (kip)	-2.937	-4.918
V_z (kip)	0.327	0.523
M_x (kipft)	1.711	2.719
M_z (kipft)	50.153	85.225

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.937 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.46768 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(50.153 \text{ kipft}) + ((-2.937 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.9861 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5332 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.327 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.05207 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.711 \text{ kipft}) + ((0.327 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.27245 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.1642 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5332 \text{ ft}), (3.1642 \text{ ft})]$$

$$L_{e,req} = 7.533 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.533 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.94163$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.109 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22216$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.9861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9861 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (7.9861 \text{ kipft/ft})) + (4 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.492 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.9861 \text{ kipft/ft})) + (3 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (7.9861 \text{ kipft/ft})) + (2 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.30538 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.9861 \text{ kipft/ft})) + ((-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1466 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.492 \text{ ft})}{2}$$

$$p_a = 0.4119 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30538 \text{ kip/ft}^2)}{(0.4119 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74138$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1466 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95554$$

Status: **PASS**
Ratio: **0.740**

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

Considering z-direction:

$H_o = 0.05207 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.27245 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.27245 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.05207 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.27245 \text{ kipft/ft})) + (4 \times (0.05207 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.6699 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.27245 \text{ kipft/ft})) + (3 \times (0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 [(3 \times (0.27245 \text{ kipft/ft})) + (2 \times (0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.038861 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.27245 \text{ kipft/ft})) + ((0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.090137 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6699 \text{ ft})}{2}$$

$$p_a = 0.42524 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.038861 \text{ kip/ft}^2)}{(0.42524 \text{ kip/ft}^2)}$$

$$Ratio = 0.091386$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

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

Ratio - Lateral soil capacity

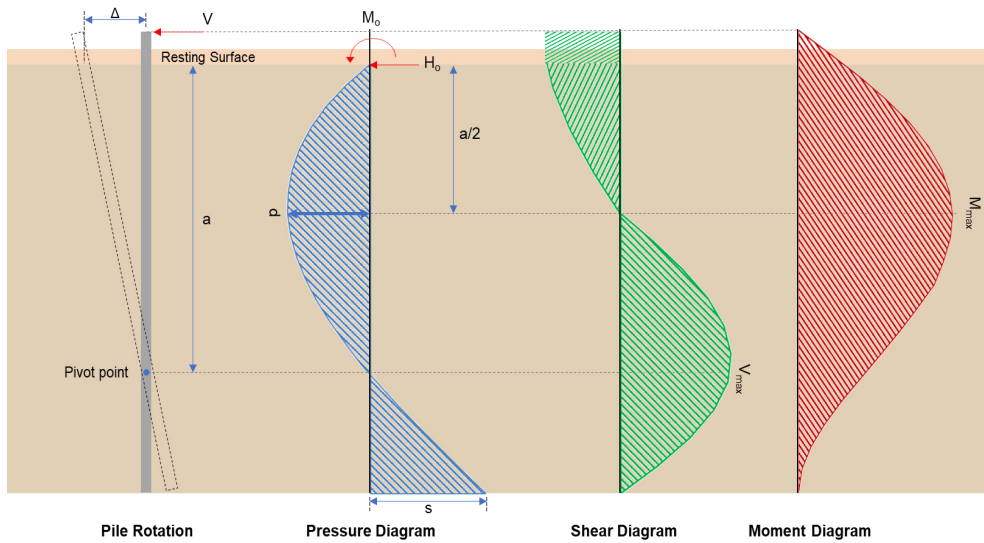
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.090137 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.075114$$

Status: **PASS**
Ratio: **0.090**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.918 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.78312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(85.225 \text{ kipft}) + ((-4.918 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 13.571 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(13.571 \text{ kipft/ft})}{(-0.78312 \text{ kip/ft})}$$

$$E = 17.329 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.571 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (13.571 \text{ kipft/ft})) + (4 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(-0.78312 \text{ kip/ft}) \times (8 \text{ ft})}{(6 \times (13.571 \text{ kipft/ft})) + (4 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.4902 \text{ ft}$$

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

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

$$V_{max} = ((-0.78312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.329 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.4902 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.329 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.4902 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.077 \text{ kip}$$

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

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

$$M_{max} = ((-0.78312 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(17.329 \text{ ft})}{(8 \text{ ft})} + \frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.329 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.329 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 54.025 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.523 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.08328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.719 \text{ kipft}) + ((0.523 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.43296 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.43296 \text{ kipft/ft})}{(0.08328 \text{ kip/ft})}$$

$$E = 5.1989 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.43296 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.08328 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.43296 \text{ kipft/ft})) + (4 \times (0.08328 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.6709 \text{ ft}$$

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

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

$$V_{max} = ((0.08328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1989 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.6709 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.1989 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.6709 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.60417 \text{ kip}$$

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

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

$$M_{max} = ((0.08328 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(5.1989 \text{ ft})}{(8 \text{ ft})} + \frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.1989 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1989 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.1781 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.711 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.24 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.24 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.711 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0040038$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.711 \text{ kip} \rightarrow 10711 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10711 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.91 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.91 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.91 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.91 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.02 \text{ kip}$$

Considering x-direction:

V_{max} = 14.077 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.077 \text{ kip})}{(111.02 \text{ kip})}$$

$$Ratio = 0.12679$$

Status: **PASS**
Ratio: **0.130**

Considering z-direction:

$V_{max} = 0.60417 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.60417 \text{ kip})}{(111.02 \text{ kip})}$$

$$Ratio = 0.0054417$$

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

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:

$M_{max} = 54.025 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(54.025 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.21645$$

Status: **PASS**
Ratio: **0.220**

Considering z-direction:

$M_{max} = 2.1781 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.1781 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0087263$$

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

REFERENCES	CALCULATIONS	RESULTS
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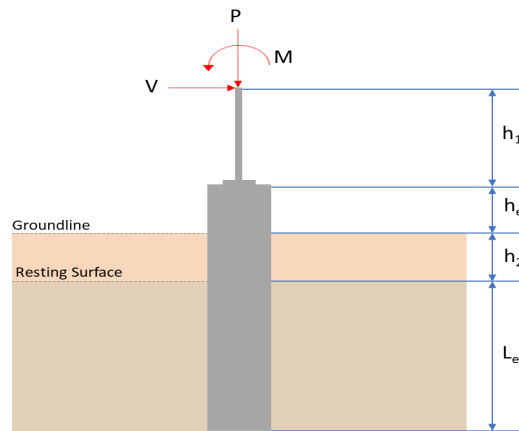
SkyCiv Foundation Design

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

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.109	10.711
V_x (kip)	-2.937	-4.918
V_z (kip)	-0.327	-0.523
M_x (kipft)	-1.711	-2.720
M_z (kipft)	50.153	85.227

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.937 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.46768 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(50.153 \text{ kipft}) + ((-2.937 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.9861 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5332 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.327 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.05207 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.711 \text{ kipft}) + ((-0.327 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.27245 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.423 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5332 \text{ ft}), (2.423 \text{ ft})]$$

$$L_{e,req} = 7.533 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.533 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.94163$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.109 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22216$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.9861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9861 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (7.9861 \text{ kipft/ft})) + (4 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.492 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.9861 \text{ kipft/ft})) + (3 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (7.9861 \text{ kipft/ft})) + (2 \times (-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.30538 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.9861 \text{ kipft/ft})) + ((-0.46768 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1466 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.492 \text{ ft})}{2}$$

$$p_a = 0.4119 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30538 \text{ kip/ft}^2)}{(0.4119 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74138$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1466 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95554$$

Status: **PASS**
Ratio: **0.740**

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

Considering z-direction:

$H_o = -0.05207 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.27245 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.27245 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.05207 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.27245 \text{ kipft/ft})) + (4 \times (-0.05207 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.6699 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.27245 \text{ kipft/ft})) + (3 \times (-0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.27245 \text{ kipft/ft})) + (2 \times (-0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.019 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.27245 \text{ kipft/ft})) + ((-0.05207 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.012032 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6699 \text{ ft})}{2}$$

$$p_a = 0.42524 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.019 \text{ kip/ft}^2)}{(0.42524 \text{ kip/ft}^2)}$$

$$Ratio = -0.044681$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

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

Ratio - Lateral soil capacity

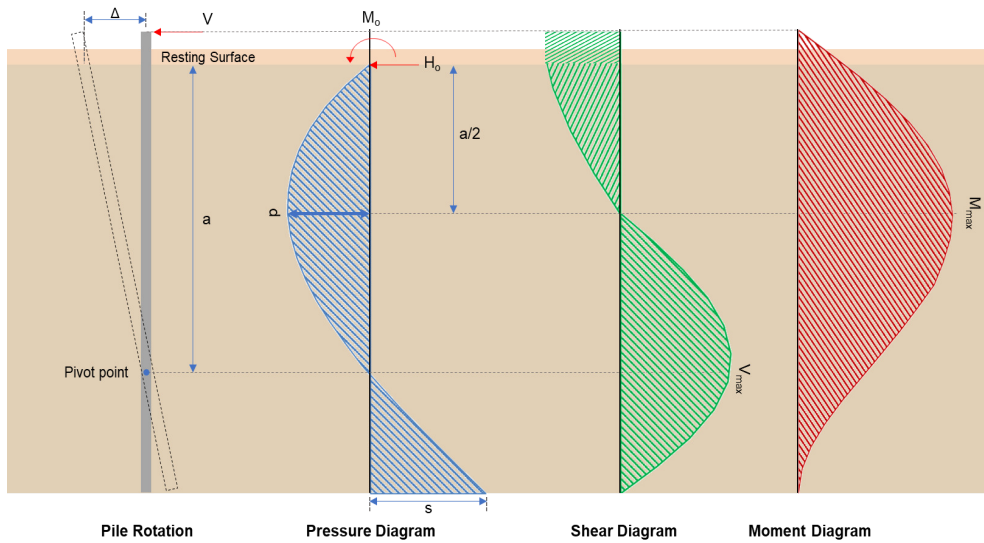
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.012032 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.010027$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.918 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.78312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(85.227 \text{ kipft}) + ((-4.918 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 13.571 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(13.571 \text{ kipft/ft})}{(-0.78312 \text{ kip/ft})}$$

$$E = 17.33 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.571 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (13.571 \text{ kipft/ft})) + (4 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (13.571 \text{ kipft/ft})) + (4 \times (-0.78312 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.4902 \text{ ft}$$

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

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

$$V_{max} = ((-0.78312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.33 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.4902 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.33 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.4902 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.077 \text{ kip}$$

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

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

$$M_{max} = ((-0.78312 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(17.33 \text{ ft})}{(8 \text{ ft})} + \frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.33 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.33 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.4902 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 54.026 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.523 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.08328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.72 \text{ kipft}) + ((-0.523 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.43312 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.43312 \text{ kipft/ft})}{(-0.08328 \text{ kip/ft})}$$

$$E = 5.2008 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.43312 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.08328 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.43312 \text{ kipft/ft})) + (4 \times (-0.08328 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.6709 \text{ ft}$$

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

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

$$V_{max} = ((-0.08328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2008 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.6709 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.2008 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.6709 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.60431 \text{ kip}$$

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

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

$$M_{max} = ((-0.08328 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(5.2008 \text{ ft})}{(8 \text{ ft})} + \frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.2008 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2008 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.6709 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.1786 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.711 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.24 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.24 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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,

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

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

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.711 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0040038$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.711 \text{ kip} \rightarrow 10711 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10711 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.91 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.91 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.91 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.91 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.02 \text{ kip}$$

Considering x-direction:

V_{max} = 14.077 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.077 \text{ kip})}{(111.02 \text{ kip})}$$

$$Ratio = 0.12679$$

Status: **PASS**
Ratio: **0.130**

Considering z-direction:

$V_{max} = 0.60431 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.60431 \text{ kip})}{(111.02 \text{ kip})}$$

$$Ratio = 0.005443$$

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

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:

$M_{max} = 54.026 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(54.026 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.21645$$

Status: **PASS**
Ratio: **0.220**

Considering z-direction:

$M_{max} = 2.1786 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.1786 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0087286$$

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

REFERENCES	CALCULATIONS	RESULTS
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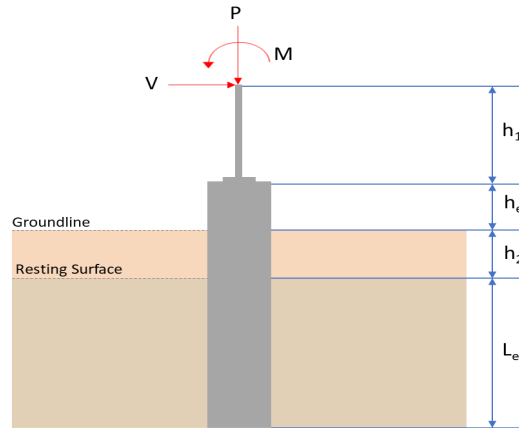
SkyCiv Foundation Design

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

$L = 8.5$ 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)	8.984	13.614
V_x (kip)	-3.619	-6.033
V_z (kip)	0.046	0.080
M_x (kipft)	0.232	0.401
M_z (kipft)	61.918	106.111

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,

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

$$H_o = \frac{(-3.619 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57627 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.918 \text{ kipft}) + ((-3.619 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.8596 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.0007 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.046 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0073248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.232 \text{ kipft}) + ((0.046 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036943 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5369 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.0007 \text{ ft}), (1.5369 \text{ ft})]$$

$$L_{e,req} = 8.001 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.001 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.94129$$

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

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

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

$$q = \frac{(8.984 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.5615 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.5615 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28075$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.57627 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.8596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.8596 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.8596 \text{ kipft/ft})) + (4 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8429 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.8596 \text{ kipft/ft})) + (3 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.8596 \text{ kipft/ft})) + (2 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.32127 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.8596 \text{ kipft/ft})) + ((-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2308 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8429 \text{ ft})}{2}$$

$$p_a = 0.43822 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32127 \text{ kip/ft}^2)}{(0.43822 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73312$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2308 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96533$$

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

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

Considering z-direction:

$H_o = 0.0073248 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.036943 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.036943 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.036943 \text{ kipft/ft})) + (4 \times (0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0414 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.036943 \text{ kipft/ft})) + (3 \times (0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.036943 \text{ kipft/ft})) + (2 \times (0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.0049368 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.036943 \text{ kipft/ft})) + ((0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.011306 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0414 \text{ ft})}{2}$$

$$p_a = 0.45311 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0049368 \text{ kip/ft}^2)}{(0.45311 \text{ kip/ft}^2)}$$

$$Ratio = 0.010895$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

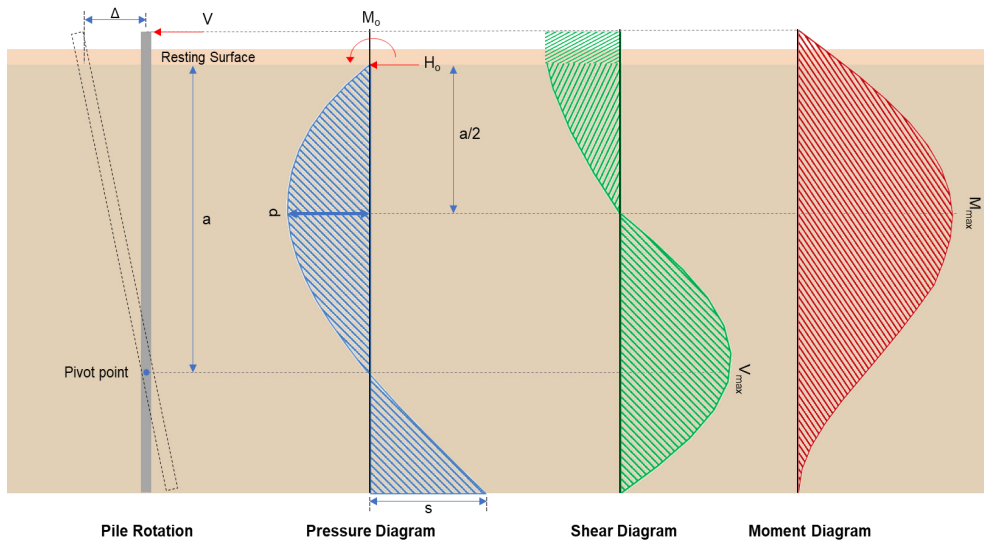
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.011306 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.0088677$$

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,

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

$$H_o = \frac{(-6.033 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(106.11 \text{ kipft}) + ((-6.033 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.897 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.897 \text{ kipft/ft})}{(-0.96067 \text{ kip/ft})}$$

$$E = 17.588 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.897 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.96067 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times 16.897) + (4 \times (-0.96067) \times 8.5)}$$

$$a = \frac{(-0.96067 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (16.897 \text{ kipft/ft})) + (4 \times (-0.96067 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8393 \text{ ft}$$

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

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

$$V_{max} = ((-0.96067 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8393 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8393 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.608 \text{ kip}$$

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

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

$$M_{max} = ((-0.96067 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(17.588 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8393 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8393 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8393 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 67.611 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.08 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.401 \text{ kipft}) + ((0.08 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.063854 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.063854 \text{ kipft/ft})}{(0.012739 \text{ kip/ft})}$$

$$E = 5.0125 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.063854 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.012739 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.063854 \text{ kipft/ft})) + (4 \times (0.012739 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0425 \text{ ft}$$

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

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

$$V_{max} = ((0.012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.0125 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0425 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.0125 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0425 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.087038 \text{ kip}$$

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

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

$$M_{max} = ((0.012739 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(5.0125 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0425 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.0125 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0425 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.0125 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0425 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.33119 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.614 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.144 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.144 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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;">$Ratio = \frac{(13.614 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.005089$</p>	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.614 \text{ kip} \rightarrow 13614 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13614 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.3 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.3 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.3 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.3 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

V_{max} = 16.608 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.608 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.14925$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.087038 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.087038 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.00078218$$

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:

$M_{max} = 67.611 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.611 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27088$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.33119 \text{ kipft}$ - Maximum moment in the z-direction,

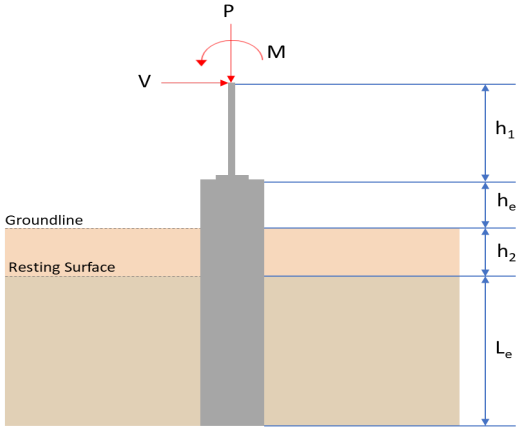
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33119 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0013269$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 8.5$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1086 1225 1187"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="652 1290 943 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.942</td> <td>13.538</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.679</td> <td>-6.135</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>-0.001</td> </tr> <tr> <td>M_z (kipft)</td> <td>63.417</td> <td>108.741</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength.</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	8.942	13.538	V_x (kip)	-3.679	-6.135	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	-0.001	M_z (kipft)	63.417	108.741	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	0.000	-0.001																										
M_z (kipft)	63.417	108.741																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.679 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.58583 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(63.417 \text{ kipft}) + ((-3.679 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.098 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.0641 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(8.0641 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 8.064 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.064 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.94871$$

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

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

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

$$q = \frac{(8.942 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55887 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55887 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27944$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58583 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 10.098 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.098 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.58583 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (10.098 \text{ kipft/ft})) + (4 \times (-0.58583 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8419 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (10.098 \text{ kipft/ft})) + (3 \times (-0.58583 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (10.098 \text{ kipft/ft})) + (2 \times (-0.58583 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.33074 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (10.098 \text{ kipft/ft})) + ((-0.58583 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2637 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8419 \text{ ft})}{2}$$

$$p_a = 0.43814 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.33074 \text{ kip/ft}^2)}{(0.43814 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75487$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

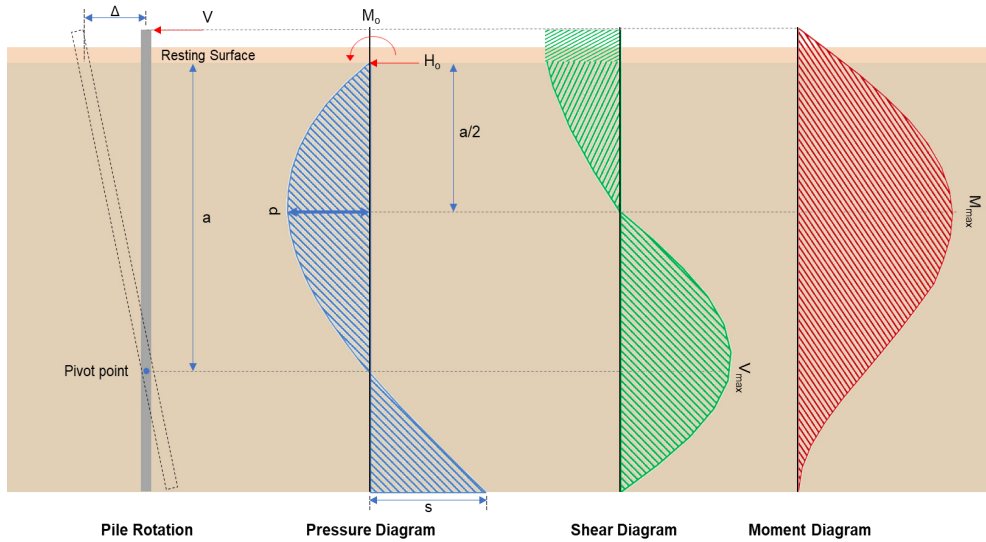
$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2637 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.750**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.135 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.97691 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(108.74 \text{ kipft}) + ((-6.135 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 17.315 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(17.315 \text{ kipft/ft})}{(-0.97691 \text{ kip/ft})}$$

$$E = 17.725 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (17.315 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.97691 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (17.315 \text{ kipft/ft})) + (4 \times (-0.97691 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8383 \text{ ft}$$

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

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

$$V_{max} = ((-0.97691 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.725 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8383 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.725 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8383 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11 \text{ kip}$$

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

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

$$M_{max} = ((-0.97691 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(17.725 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8383 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.725 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8383 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (17.725 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8383 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 69.225 \text{ kipft}$$

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,

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

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

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

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.6667 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(8.5 \text{ ft})} \right) \times \left(\frac{(5.6667 \text{ ft})}{(8.5 \text{ ft})} - 1 \right) \times \left(\frac{(5.6667 \text{ ft})}{(8.5 \text{ ft})} \right)^2$$

$$V_{max} = 0.00013322 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(5.6667 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 + \left(3 \times \frac{(5.6667 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\left(\frac{13.538 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.146 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.146 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

22.4.2.2 **Axial Compression Strength (ACI 318-19, LRFD)**

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

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

$$Ratio = \frac{(13.538 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0050606$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2

$$d = 0.80 D$$

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

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.538 \text{ kip} \rightarrow 13538 \text{ lbf}$,

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13538 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.29 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.29 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.29 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((120.29 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.27 \text{ kip}$$

Considering x-direction:

$V_{max} = 17 \text{ kip}$ - Maximum shear force in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(17 \text{ kip})}{(111.27 \text{ kip})}$$

$$\text{Ratio} = 0.15278$$

Status: **PASS**
Ratio: **0.150**

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 $\phi M_{n,2}$

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

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

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

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:

$M_{max} = 69.225 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(69.225 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.27735$$

Status: **PASS**
Ratio: **0.280**

Considering z-direction:

$M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 2.2683 \times 10^{-6}$$

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

REFERENCES	CALCULATIONS	RESULTS
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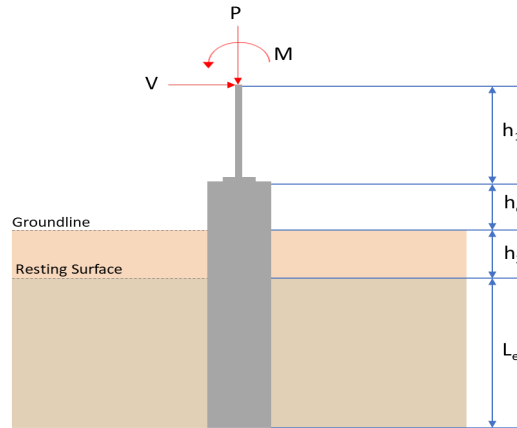
SkyCiv Foundation Design

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

$L = 8.5$ 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)	8.984	13.614
V_x (kip)	-3.619	-6.033
V_z (kip)	-0.046	-0.080
M_x (kipft)	-0.232	-0.402
M_z (kipft)	61.918	106.111

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,

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

$$H_o = \frac{(-3.619 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57627 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.918 \text{ kipft}) + ((-3.619 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.8596 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.0007 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.046 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0073248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.232 \text{ kipft}) + ((-0.046 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036943 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3332 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.0007 \text{ ft}), (1.3332 \text{ ft})]$$

$$L_{e,req} = 8.001 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.001 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.94129$$

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

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

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

$$q = \frac{(8.984 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.5615 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.5615 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28075$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.57627 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.8596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.8596 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.8596 \text{ kipft/ft})) + (4 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8429 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.8596 \text{ kipft/ft})) + (3 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.8596 \text{ kipft/ft})) + (2 \times (-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.32127 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.8596 \text{ kipft/ft})) + ((-0.57627 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2308 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8429 \text{ ft})}{2}$$

$$p_a = 0.43822 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.32127 \text{ kip/ft}^2)}{(0.43822 \text{ kip/ft}^2)}$$

$$Ratio = 0.73312$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2308 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.96533$$

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

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

Considering z-direction:

$H_o = -0.0073248 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.036943 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.036943 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.036943 \text{ kipft/ft})) + (4 \times (-0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0414 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.036943 \text{ kipft/ft})) + (3 \times (-0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.036943 \text{ kipft/ft})) + (2 \times (-0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.0011537 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.036943 \text{ kipft/ft})) + ((-0.0073248 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.00096533 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0414 \text{ ft})}{2}$$

$$p_a = 0.45311 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0011537 \text{ kip/ft}^2)}{(0.45311 \text{ kip/ft}^2)}$$

$$Ratio = -0.0025462$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

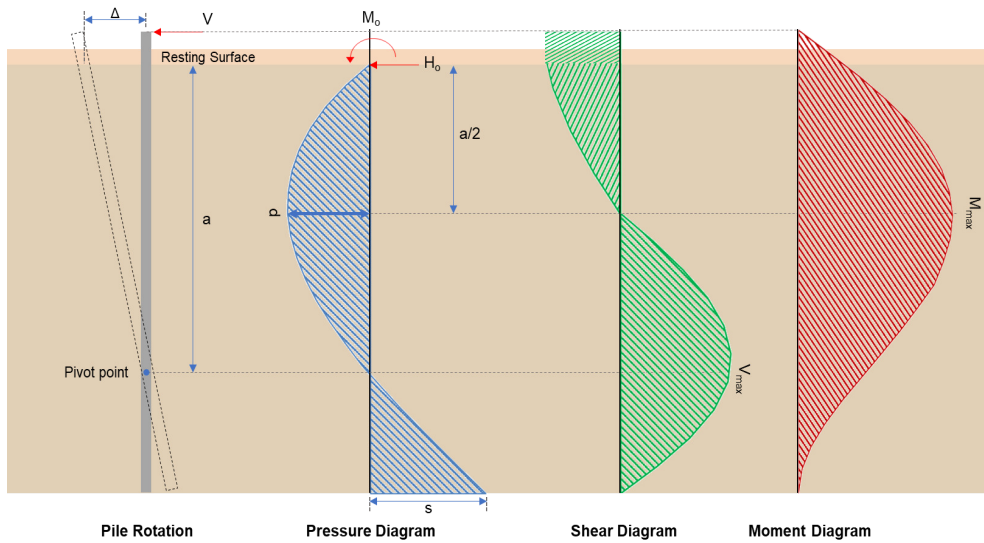
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00096533 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.00075712$$

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,

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

$$H_o = \frac{(-6.033 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(106.11 \text{ kipft}) + ((-6.033 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.897 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.897 \text{ kipft/ft})}{(-0.96067 \text{ kip/ft})}$$

$$E = 17.588 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.897 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.96067 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (16.897 \text{ kipft/ft})) + (4 \times (-0.96067 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = \frac{(-0.96067 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (16.897 \text{ kipft/ft})) + (4 \times (-0.96067 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8393 \text{ ft}$$

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

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

$$V_{max} = ((-0.96067 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8393 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8393 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.608 \text{ kip}$$

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

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

$$M_{max} = ((-0.96067 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(17.588 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8393 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8393 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (17.588 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8393 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 67.611 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.08 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.402 \text{ kipft}) + ((-0.08 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.064013 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.064013 \text{ kipft/ft})}{(-0.012739 \text{ kip/ft})}$$

$$E = 5.025 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.064013 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.012739 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.064013 \text{ kipft/ft})) + (4 \times (-0.012739 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0421 \text{ ft}$$

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

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

$$V_{max} = ((-0.012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.025 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0421 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.025 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0421 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.08717 \text{ kip}$$

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

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

$$M_{max} = ((-0.012739 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(5.025 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.025 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.025 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.33175 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.614 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.144 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.144 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.614 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.005089$	<p>Status: PASS Ratio: 0.010</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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.614 \text{ kip} \rightarrow 13614 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13614 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.3 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.3 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.3 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.3 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

V_{max} = 16.608 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.608 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.14925$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.08717 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.08717 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.00078336$$

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:

$M_{max} = 67.611 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.611 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27088$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.33175 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33175 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0013291$$

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