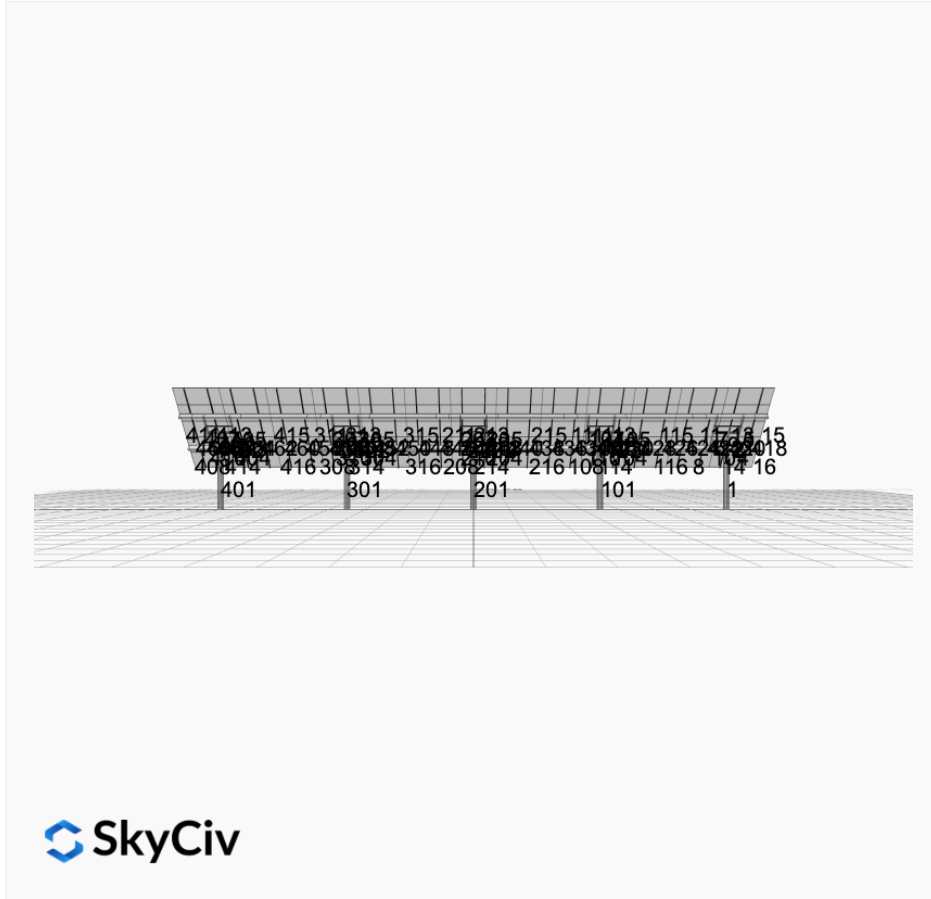


Project Details



Project Name: BT SIL530 40d 6h 60inSquare - V1Jb **Date:** Wed Apr 30 2025
Location: 31842 Frontage Rd, Bozeman, MT 59715, USA **Number of Modules:** 65
Unique ID: 5P-19.75-10TOP-XD-24-L-5Hx13W-022F **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	90.57 ft
Winter Tilt Angle	40
Front Edge Clearance	6 ft

MT Solar Bill of Materials (5P-19.75-10TOP-XD-24-L-5Hx13W-022F)

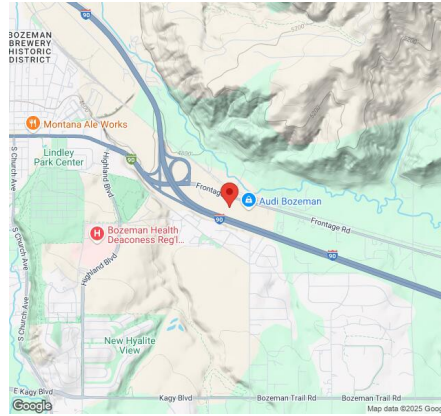
Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	5
MTS-HF-XD	H-Frame Assembly-XD	5
MTS-XD-Wing-24	24IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	8
MTS-XD-Splice-57	57IN XD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (226in)	26
Rail Attachment	104

Part	Qty
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:	XD
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:	40
Front Edge Clearance:	6
Total Array Height at Tilt:	18.08 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:	10in Pipe Sch 40
Pole Length above Grade:	12.04 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 8.00 ft Pile 3: 8.00 ft Pile 4: 8.00 ft Pile 5: 7.50 ft
Foundation Volume:	36.111 y ³

Site Info

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

Snow Load:

51 psf

Design Disclaimer

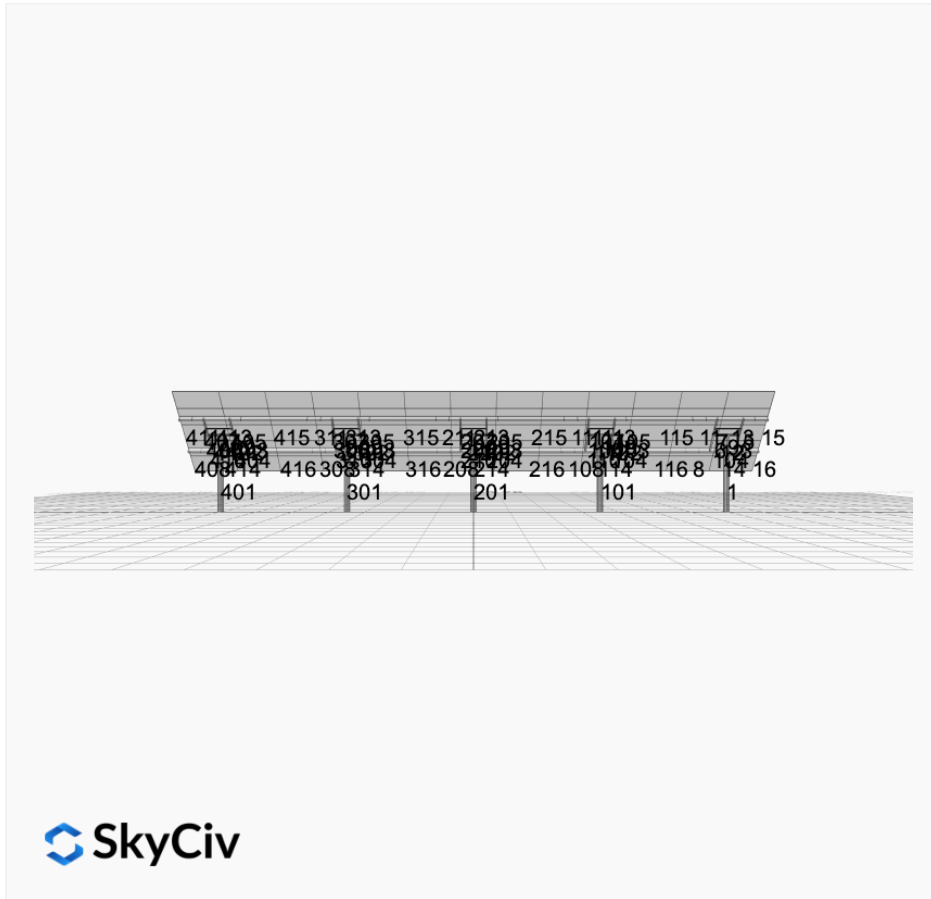
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

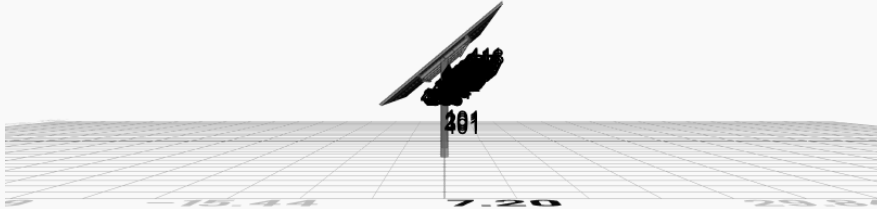
AutoDesigner Input

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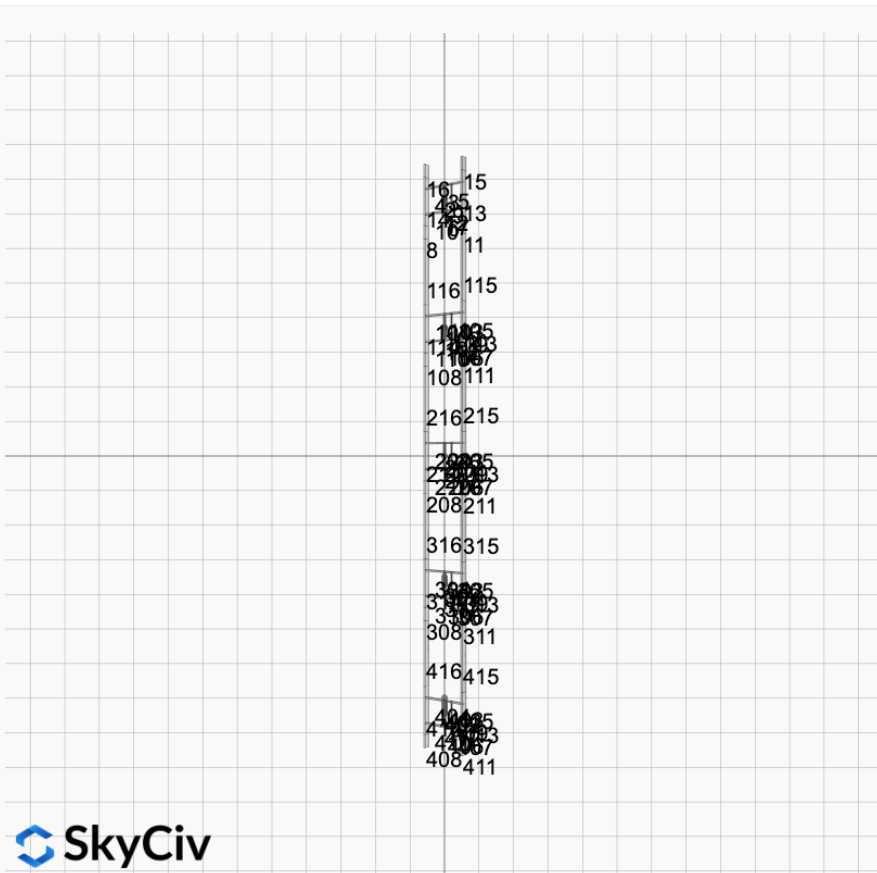
Design Notes:

- 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)

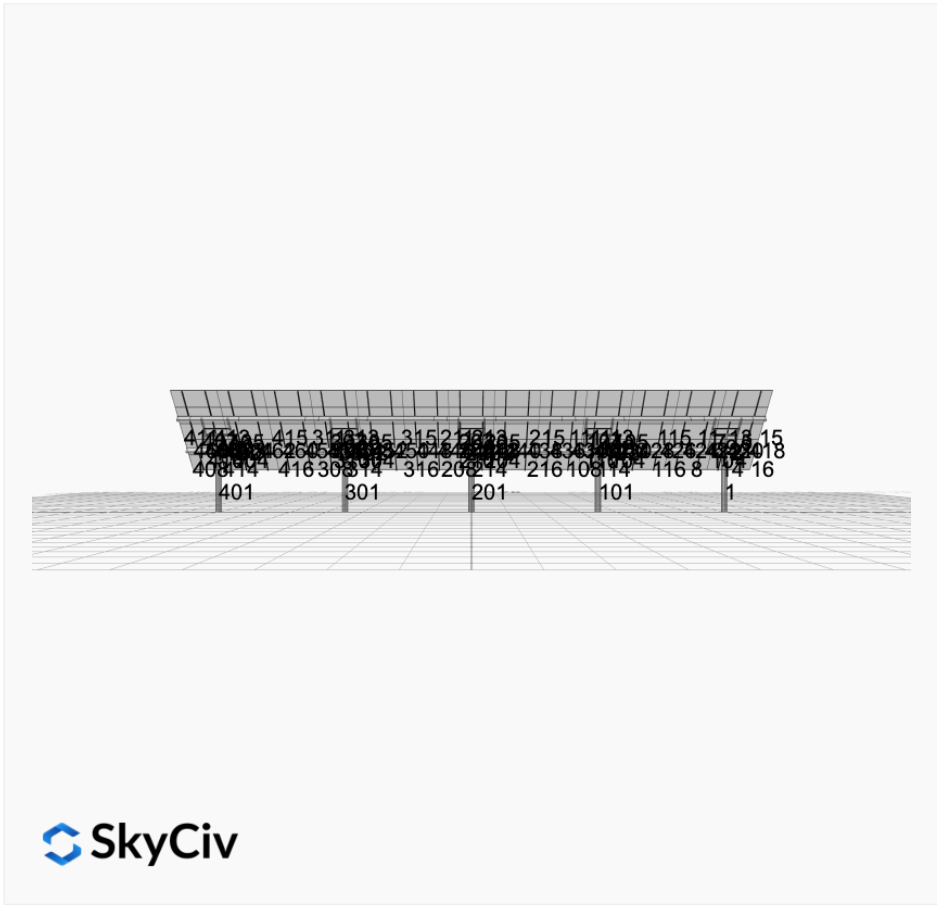
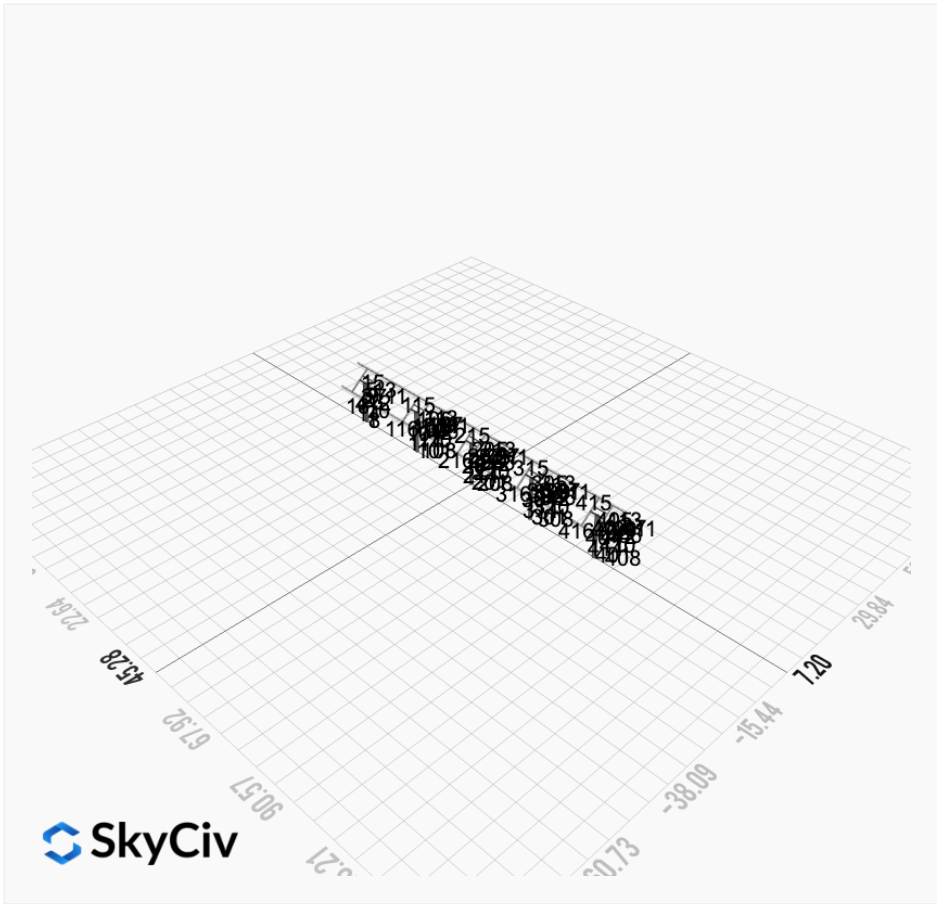




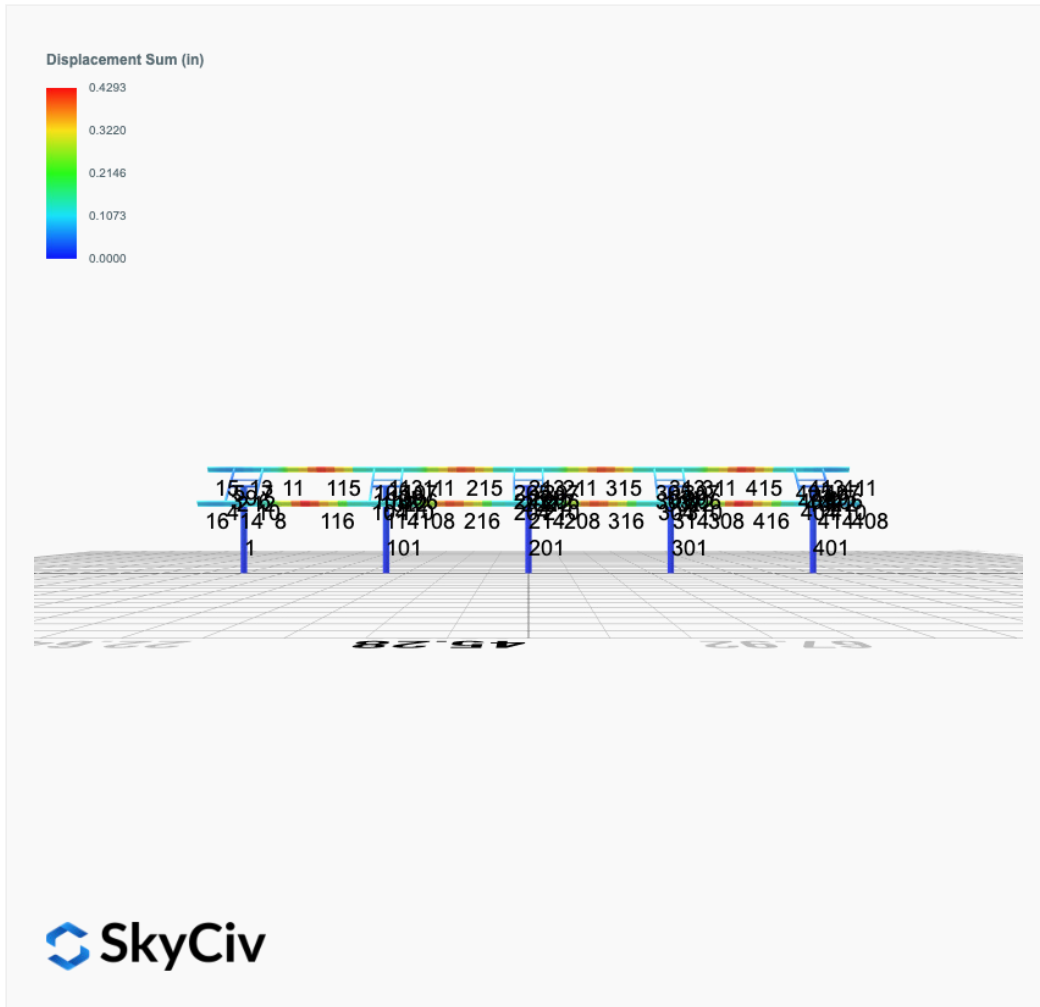
 SkyCiv



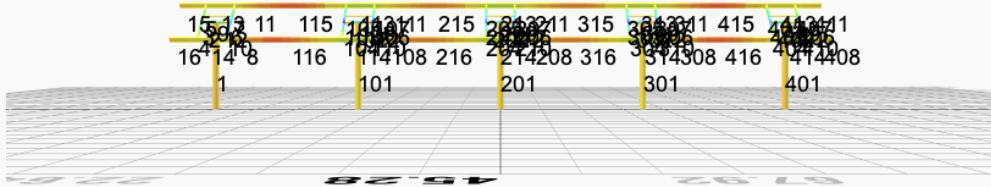
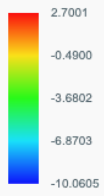
 SkyCiv



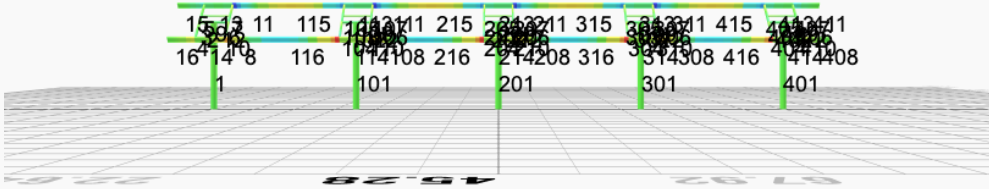
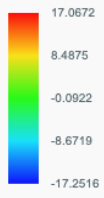
FEM Results (Envelope Worst Case for each member)



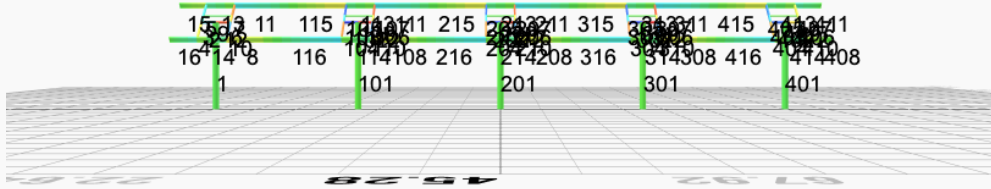
Top Bending Stress Z (ksi)



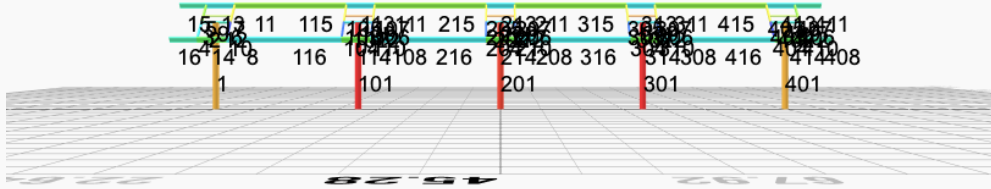
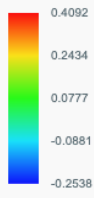
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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	0.0308	2.4051	0.0734	0.2405	-0.0543	-0.3300
ULS: 2. D + L	0.0308	2.4051	0.0734	0.2405	-0.0543	-0.3300
ULS: 3. D + (S or Lr or R)	0.1002	6.0997	0.2389	0.7847	-0.1778	-1.1196
ULS: 3. D + (S or Lr or R)	0.0308	2.4051	0.0734	0.2405	-0.0543	-0.3300
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0829	5.1760	0.1975	0.6486	-0.1469	-0.9222
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0308	2.4051	0.0734	0.2405	-0.0543	-0.3300
ULS: 5b. D + 0.7E	0.0308	2.4051	0.0734	0.2405	-0.0543	-0.3300
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0829	5.1760	0.1975	0.6486	-0.1469	-0.9222
ULS: 8. 0.6D + 0.7E	0.0185	1.4430	0.0440	0.1443	-0.0326	-0.1980
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2615	7.4578	0.3990	1.2172	-1.6395	53.2741
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.2615	7.4578	0.3990	1.2172	-1.6395	53.2741
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4451	-1.6170	-0.1783	-0.5144	1.1726	-40.8776
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.9566	-1.0262	-0.1632	-0.4679	1.1115	-45.1378
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1364	8.9656	0.4417	1.3811	-1.3358	39.2809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1364	8.9656	0.4417	1.3811	-1.3358	39.2809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6435	2.1595	0.0087	0.0825	0.7733	-31.3329
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2772	2.6025	0.0201	0.1173	0.7275	-34.5280
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1884	6.1946	0.3176	0.9730	-1.2432	39.8731
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1884	6.1946	0.3176	0.9730	-1.2432	39.8731
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5915	-0.6115	-0.1154	-0.3257	0.8659	-30.7407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2252	-0.1684	-0.1041	-0.2908	0.8200	-33.9358
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2738	6.4958	0.3696	1.1210	-1.6178	53.4061
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.2738	6.4958	0.3696	1.1210	-1.6178	53.4061
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4327	-2.5790	-0.2077	-0.6106	1.1943	-40.7456
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.9443	-1.9883	-0.1926	-0.5641	1.1332	-45.0058

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	13.1561
Shear X	-7.1539
Shear Z	0.7165
Moment X	2.1980
Moment Y (Twist)	2.7802
Moment Z	89.3933

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	8.9656
Shear X	-4.2738
Shear Z	0.4417
Moment X	1.3811
Moment Y (Twist)	1.6395
Moment Z	53.4061

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	-0.0299	2.9298	-0.0045	-0.0168	0.0237	0.3599
ULS: 2. D + L	-0.0299	2.9298	-0.0045	-0.0168	0.0237	0.3599
ULS: 3. D + (S or Lr or R)	-0.0970	7.8021	-0.0147	-0.0544	0.0770	1.1327
ULS: 3. D + (S or Lr or R)	-0.0299	2.9298	-0.0045	-0.0168	0.0237	0.3599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0802	6.5840	-0.0122	-0.0450	0.0637	0.9395

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0299	2.9298	-0.0045	-0.0168	0.0237	0.3599
ULS: 5b. D + 0.7E	-0.0299	2.9298	-0.0045	-0.0168	0.0237	0.3599
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0802	6.5840	-0.0122	-0.0450	0.0637	0.9395
ULS: 8. 0.6D + 0.7E	-0.0179	1.7579	-0.0027	-0.0101	0.0142	0.2160
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.6359	9.6713	0.0087	0.0089	-0.1173	69.8630
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.6359	9.6713	0.0087	0.0089	-0.1173	69.8630
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4464	-2.4507	-0.0111	-0.0258	0.1132	-52.0535
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7387	-1.6134	-0.0231	-0.0622	0.1782	-56.7677
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2847	11.6402	-0.0022	-0.0257	-0.0421	53.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2847	11.6402	-0.0022	-0.0257	-0.0421	53.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2770	2.5487	-0.0171	-0.0517	0.1308	-38.3706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7462	3.1766	-0.0261	-0.0790	0.1796	-41.9062
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2344	7.9859	0.0054	0.0025	-0.0820	52.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2344	7.9859	0.0054	0.0025	-0.0820	52.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3273	-1.1055	-0.0095	-0.0235	0.0908	-38.9501
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7966	-0.4776	-0.0184	-0.0508	0.1396	-42.4858
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.6239	8.4994	0.0105	0.0156	-0.1268	69.7190
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.6239	8.4994	0.0105	0.0156	-0.1268	69.7190
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4583	-3.6226	-0.0093	-0.0190	0.1037	-52.1975
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7507	-2.7853	-0.0212	-0.0555	0.1688	-56.9117

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	17.1850
Shear X	-9.4095
Shear Z	-0.0428
Moment X	-0.1229
Moment Y (Twist)	0.3207
Moment Z	117.8061

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	11.6402
Shear X	-5.6359
Shear Z	-0.0261
Moment X	-0.0790
Moment Y (Twist)	0.1796
Moment Z	69.8630

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	-0.0020	2.8907	0.0000	-0.0000	0.0000	0.0774
ULS: 2. D + L	-0.0020	2.8907	0.0000	-0.0000	0.0000	0.0774
ULS: 3. D + (S or Lr or R)	-0.0064	7.6753	0.0000	-0.0001	0.0001	0.2143
ULS: 3. D + (S or Lr or R)	-0.0020	2.8907	0.0000	-0.0000	0.0000	0.0774
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0053	6.4791	0.0000	-0.0000	0.0001	0.1801
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0020	2.8907	0.0000	-0.0000	0.0000	0.0774
ULS: 5b. D + 0.7E	-0.0020	2.8907	0.0000	-0.0000	0.0000	0.0774
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0053	6.4791	0.0000	-0.0000	0.0001	0.1801
ULS: 8. 0.6D + 0.7E	-0.0012	1.7344	0.0000	-0.0000	0.0000	0.0464
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.5853	9.5489	0.0000	-0.0000	0.0000	69.8858
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.5853	9.5489	0.0000	-0.0000	0.0000	69.8858
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4448	-2.4108	0.0000	-0.0000	0.0000	-52.4204
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7838	-1.6280	0.0000	-0.0000	0.0000	-57.7179

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	-4.1928	11.4728	0.0000	-0.0000	0.0001	52.5364
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.1928	11.4728	0.0000	-0.0000	0.0001	52.5364
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3298	2.5030	0.0000	-0.0000	0.0001	-39.1933
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8340	3.0901	0.0000	-0.0000	0.0001	-43.1664
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.1895	7.8844	0.0000	-0.0000	0.0000	52.4337
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.1895	7.8844	0.0000	-0.0000	0.0000	52.4337
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3331	-1.0854	0.0000	-0.0000	0.0000	-39.2960
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8374	-0.4983	0.0000	-0.0000	0.0000	-43.2691
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.5845	8.3926	0.0000	-0.0000	0.0000	69.8548
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.5845	8.3926	0.0000	-0.0000	0.0000	69.8548
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4456	-3.5671	0.0000	-0.0000	0.0000	-52.4514
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7846	-2.7843	0.0000	-0.0000	0.0000	-57.7489

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	16.9610
Shear X	-9.3121
Shear Z	0.0000
Moment X	-0.0005
Moment Y (Twist)	0.0010
Moment Z	117.7197

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	11.4728
Shear X	-5.5853
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	69.8858

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	-0.0299	2.9298	0.0045	0.0168	-0.0237	0.3599
ULS: 2. D + L	-0.0299	2.9298	0.0045	0.0168	-0.0237	0.3599
ULS: 3. D + (S or Lr or R)	-0.0970	7.8021	0.0147	0.0543	-0.0767	1.1327
ULS: 3. D + (S or Lr or R)	-0.0299	2.9298	0.0045	0.0168	-0.0237	0.3599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0802	6.5840	0.0122	0.0449	-0.0635	0.9395
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0299	2.9298	0.0045	0.0168	-0.0237	0.3599
ULS: 5b. D + 0.7E	-0.0299	2.9298	0.0045	0.0168	-0.0237	0.3599
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0802	6.5840	0.0122	0.0449	-0.0635	0.9395
ULS: 8. 0.6D + 0.7E	-0.0179	1.7579	0.0027	0.0101	-0.0142	0.2160
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.6359	9.6713	-0.0087	-0.0089	0.1173	69.8630
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.6359	9.6713	-0.0087	-0.0089	0.1173	69.8630
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4464	-2.4507	0.0111	0.0257	-0.1132	-52.0535
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7387	-1.6134	0.0231	0.0622	-0.1782	-56.7677
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2847	11.6402	0.0022	0.0257	0.0423	53.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2847	11.6402	0.0022	0.0257	0.0423	53.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2770	2.5487	0.0171	0.0516	-0.1306	-38.3706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7462	3.1766	0.0261	0.0790	-0.1794	-41.9062
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2344	7.9859	-0.0054	-0.0025	0.0821	52.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2344	7.9859	-0.0054	-0.0025	0.0821	52.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3273	-1.1055	0.0095	0.0235	-0.0908	-38.9501
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7966	-0.4776	0.0184	0.0508	-0.1396	-42.4858

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.6239	8.4994	-0.0105	-0.0156	0.1268	69.7190
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.6239	8.4994	-0.0105	-0.0156	0.1268	69.7190
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4583	-3.6226	0.0093	0.0190	-0.1037	-52.1975
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7507	-2.7853	0.0212	0.0555	-0.1687	-56.9117

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	17.1850
Shear X	-9.4095
Shear Z	0.0428
Moment X	0.1234
Moment Y (Twist)	0.3206
Moment Z	117.8062

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	11.6402
Shear X	-5.6359
Shear Z	0.0261
Moment X	0.0790
Moment Y (Twist)	0.1794
Moment Z	69.8630

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	0.0308	2.4051	-0.0734	-0.2405	0.0544	-0.3300
ULS: 2. D + L	0.0308	2.4051	-0.0734	-0.2405	0.0544	-0.3300
ULS: 3. D + (S or Lr or R)	0.1002	6.0997	-0.2389	-0.7850	0.1780	-1.1194
ULS: 3. D + (S or Lr or R)	0.0308	2.4051	-0.0734	-0.2405	0.0544	-0.3300
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0828	5.1760	-0.1976	-0.6489	0.1471	-0.9221
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0308	2.4051	-0.0734	-0.2405	0.0544	-0.3300
ULS: 5b. D + 0.7E	0.0308	2.4051	-0.0734	-0.2405	0.0544	-0.3300
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0828	5.1760	-0.1976	-0.6489	0.1471	-0.9221
ULS: 8. 0.6D + 0.7E	0.0185	1.4430	-0.0440	-0.1443	0.0326	-0.1980
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2615	7.4578	-0.3990	-1.2172	1.6395	53.2741
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.2615	7.4578	-0.3990	-1.2172	1.6395	53.2741
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4451	-1.6170	0.1783	0.5143	-1.1726	-40.8776
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.9566	-1.0262	0.1632	0.4679	-1.1115	-45.1377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1364	8.9656	-0.4417	-1.3814	1.3359	39.2810
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1364	8.9656	-0.4417	-1.3814	1.3359	39.2810
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6435	2.1595	-0.0087	-0.0828	-0.7731	-31.3328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2772	2.6025	-0.0201	-0.1176	-0.7273	-34.5279
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1884	6.1946	-0.3176	-0.9730	1.2432	39.8731
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1884	6.1946	-0.3176	-0.9730	1.2432	39.8731
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5915	-0.6115	0.1154	0.3256	-0.8658	-30.7407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2252	-0.1684	0.1041	0.2908	-0.8200	-33.9358
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2738	6.4958	-0.3696	-1.1210	1.6178	53.4061
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.2738	6.4958	-0.3696	-1.1210	1.6178	53.4061
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4327	-2.5790	0.2077	0.6105	-1.1943	-40.7456
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.9443	-1.9883	0.1926	0.5641	-1.1332	-45.0057

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	13.1561
Shear X	-7.1539
Shear Z	-0.7165
Moment X	-2.1986
Moment Y (Twist)	2.7812
Moment Z	89.3944

Result	Value (kip, kip-ft)
Axial	8.9656
Shear X	-4.2738
Shear Z	-0.4417
Moment X	-1.3814
Moment Y (Twist)	1.6395
Moment Z	53.4061

Project Details

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



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions							
ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
11	10in Pipe Sch 40	10.75	0.36				

ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

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

113	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.09,1.10,1.08,1.08,1.08,1.10,1.07,1.07,1.03,1.06,1.08,1.08,1.09,1.11,1.08,1.08,1.08,1.09	300	200	1
114	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.08,1.06,1.06,1.07,1.06,1.06,1.06,1.04,1.05,1.06,1.06,1.07,1.09,1.06,1.06,1.07,1.06	300	200	1
115	20	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.12,1.12,1.11,1.11,1.12,1.12,1.12,1.11,1.13,1.13,1.25,2.26,1.12,1.12,1.11,1.11,1.12,1.12,1.12,1.11	300	200	1
116	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.35,1.15,1.15,1.14,1.17,1.15,1.15,1.18,1.15,1.15,1.15,1.14,1.53,1.15,1.15,1.15,1.17	300	200	1
201	11	25.28	25.28	12.04	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.63,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
204	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.56,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.15,1.67,1.67,1.66,1.66	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.80,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.63,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.83,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
208	20	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.10,1.57,2.09,2.09,2.10,2.07,2.09,2.09,2.07,2.09,2.09,2.09,2.10,1.40,2.09,2.09,2.10,2.07	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.56,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.15,1.67,1.67,1.66,1.66	300	200	1
211	20	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.08,2.08,2.07,2.08,2.08,2.08,2.07,2.08,2.09,2.09,2.29,1.01,2.08,2.08,2.07,2.08,2.08,2.08,2.08,2.08,2.08	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.01,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
214	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.01,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
215	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.16,1.16,1.15,1.16,1.16,1.16,1.20,1.46,1.16,1.16,1.15,1.15,1.16,1.16,1.16,1.15,1.16	300	200	1
216	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.11,1.16,1.16,1.16,1.15,1.16,1.16,1.15,1.16,1.16,1.16,1.11,1.10,1.16,1.16,1.16,1.15	300	200	1
301	11	25.28	25.28	12.04	-	300	200	1
302	6	1.30	1.30	2.00	-	300	200	1
303	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.27,1.18,1.18,1.17,1.18,1.18,1.18,1.18	300	200	1
304	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.59,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.67,1.66,1.37,1.67,1.67,1.66,1.66	300	200	1
305	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,2.05,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
306	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.23,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
307	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.71,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
308	20	1.33	1.33	2.05	2.00,2.00,2.00,2.01,2.00,2.00,1.89,1.89,1.81,2.28,1.88,1.88,1.84,2.08,1.93,1.93,2.11,1.98,1.89,1.89,1.81,2.16,1.88,1.88,1.85,2.08	300	200	1
309	3	2.60	2.60	4.00	-	300	200	1
310	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.56,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.14,1.67,1.67,1.66,1.66	300	200	1
311	20	1.33	1.33	2.05	1.85,1.85,1.85,1.84,1.85,1.85,1.58,1.58,1.46,1.46,1.56,1.56,1.52,1.50,1.66,1.66,2.25,1.14,1.59,1.59,1.46,1.46,1.56,1.56,1.53,1.51	300	200	1
312	6	1.30	1.30	2.00	-	300	200	1
313	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.09,1.10,1.08,1.08,1.08,1.10,1.07,1.07,1.03,1.06,1.08,1.08,1.09,1.11,1.08,1.08,1.08,1.09	300	200	1
314	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.08,1.06,1.06,1.07,1.06,1.06,1.06,1.04,1.05,1.06,1.06,1.07,1.09,1.06,1.06,1.07,1.06	300	200	1

104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.57	6.46	56.26	44.91
114	159.30	97.43	31.81	6.46	56.26	44.91
115	159.30	75.13	21.33	6.46	56.26	44.91
116	159.30	75.13	22.01	6.46	56.26	44.91
201	535.87	325.47	147.68	147.68	160.76	160.76
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	30.80	6.46	56.26	44.91
214	159.30	97.43	30.82	6.46	56.26	44.91
215	159.30	75.13	22.17	6.46	56.26	44.91
216	159.30	75.13	21.21	6.46	56.26	44.91
301	535.87	325.47	147.68	147.68	160.76	160.76
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	140.46	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	140.46	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	31.58	6.46	56.26	44.91
314	159.30	97.43	31.80	6.46	56.26	44.91
315	159.30	75.13	21.64	6.46	56.26	44.91
316	159.30	75.13	22.20	6.46	56.26	44.91
401	535.87	325.47	147.68	147.68	160.76	160.76
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95
407	151.65	149.10	20.17	14.14	54.12	28.95

407	151.05	149.10	20.17	14.14	54.12	28.95
408	159.30	113.66	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	113.66	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	97.43	34.98	6.46	56.26	44.91
414	159.30	97.43	35.94	6.46	56.26	44.91
415	159.30	75.13	20.87	6.46	56.26	44.91
416	159.30	75.13	20.78	6.46	56.26	44.91

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.040	0.605	0.043	0.044	0.004	0.640	#13	0.413	Not Required	Pass
2	0.003	0.265	0.193	0.066	0.041	0.459	#13	0.054	Not Required	Pass
3	0.007	0.557	0.028	0.054	0.002	0.563	#13	0.046	Not Required	Pass
4	0.006	0.521	0.083	0.052	0.022	0.588	#13	0.082	Not Required	Pass
5	0.006	0.346	0.064	0.055	0.017	0.349	#13	0.076	Not Required	Pass
6	0.011	0.805	0.109	0.082	0.029	0.870	#13	0.046	Not Required	Pass
7	0.012	0.499	0.190	0.080	0.047	0.523	#13	0.076	Not Required	Pass
8	0.004	0.137	0.206	0.044	0.023	0.221	#24	0.102	Not Required	Pass
9	0.008	0.073	0.091	0.004	0.005	0.160	#13	0.206	Not Required	Pass
10	0.013	0.731	0.181	0.073	0.040	0.745	#13	0.082	Not Required	Pass
11	0.006	0.137	0.212	0.049	0.023	0.221	#6	0.102	Not Required	Pass
12	0.001	0.501	0.295	0.101	0.055	0.797	#13	0.054	Not Required	Pass
13	0.009	0.134	0.532	0.064	0.029	0.581	#21	0.306	Not Required	Pass
14	0.006	0.117	0.523	0.057	0.029	0.550	#24	0.204	Not Required	Pass
15	0.000	0.020	0.051	0.017	0.007	0.068	#21	Not Required	Not Required	Pass
16	0.000	0.019	0.051	0.016	0.007	0.067	#21	Not Required	Not Required	Pass
101	0.053	0.798	0.003	0.059	0.000	0.824	#13	0.413	Not Required	Pass
102	0.004	0.507	0.326	0.110	0.061	0.834	#13	0.036	Not Required	Pass
103	0.011	0.878	0.070	0.087	0.010	0.917	#13	0.046	Not Required	Pass
104	0.011	0.852	0.193	0.085	0.042	0.924	#13	0.082	Not Required	Pass
105	0.011	0.545	0.201	0.087	0.052	0.571	#13	0.076	Not Required	Pass
106	0.011	0.895	0.069	0.089	0.010	0.927	#13	0.046	Not Required	Pass
107	0.011	0.556	0.192	0.089	0.050	0.582	#13	0.076	Not Required	Pass
108	0.005	0.057	0.197	0.048	0.023	0.234	#21	0.102	Not Required	Pass
109	0.018	0.068	0.061	0.001	0.000	0.132	#13	0.206	Not Required	Pass
110	0.011	0.848	0.185	0.084	0.040	0.910	#13	0.082	Not Required	Pass
111	0.006	0.077	0.205	0.050	0.023	0.233	#21	0.102	Not Required	Pass
112	0.004	0.512	0.333	0.110	0.064	0.846	#13	0.036	Not Required	Pass
113	0.009	0.235	0.546	0.069	0.030	0.714	#21	0.306	Not Required	Pass
114	0.010	0.264	0.540	0.067	0.030	0.727	#21	0.306	Not Required	Pass
115	0.012	0.453	0.286	0.054	0.023	0.656	#21	0.507	Not Required	Pass
116	0.006	0.408	0.284	0.054	0.023	0.623	#21	0.507	Not Required	Pass
201	0.052	0.797	0.000	0.058	0.000	0.823	#13	0.413	Not Required	Pass
202	0.003	0.505	0.326	0.109	0.062	0.832	#13	0.036	Not Required	Pass
203	0.011	0.889	0.069	0.088	0.010	0.927	#13	0.046	Not Required	Pass
204	0.011	0.825	0.183	0.082	0.040	0.888	#13	0.082	Not Required	Pass
205	0.011	0.552	0.191	0.088	0.049	0.576	#13	0.076	Not Required	Pass

206	0.011	0.889	0.069	0.088	0.010	0.927	#13	0.046	Not Required	Pass
207	0.011	0.552	0.191	0.088	0.049	0.576	#13	0.076	Not Required	Pass
208	0.005	0.057	0.196	0.049	0.023	0.232	#21	0.102	Not Required	Pass
209	0.017	0.065	0.061	0.001	0.000	0.130	#13	0.206	Not Required	Pass
210	0.011	0.825	0.183	0.082	0.040	0.888	#13	0.082	Not Required	Pass
211	0.006	0.066	0.202	0.053	0.023	0.241	#21	0.102	Not Required	Pass
212	0.003	0.505	0.326	0.109	0.062	0.832	#13	0.036	Not Required	Pass
213	0.009	0.276	0.519	0.067	0.029	0.728	#21	0.306	Not Required	Pass
214	0.010	0.270	0.511	0.063	0.029	0.708	#21	0.306	Not Required	Pass
215	0.012	0.327	0.287	0.053	0.023	0.551	#21	0.507	Not Required	Pass
216	0.007	0.285	0.283	0.049	0.023	0.521	#21	0.507	Not Required	Pass
301	0.053	0.798	0.003	0.059	0.000	0.824	#13	0.413	Not Required	Pass
302	0.004	0.512	0.333	0.110	0.064	0.846	#13	0.036	Not Required	Pass
303	0.011	0.895	0.069	0.089	0.010	0.927	#13	0.046	Not Required	Pass
304	0.011	0.848	0.185	0.084	0.040	0.910	#13	0.082	Not Required	Pass
305	0.011	0.556	0.192	0.089	0.050	0.582	#13	0.076	Not Required	Pass
306	0.011	0.878	0.070	0.087	0.010	0.917	#13	0.046	Not Required	Pass
307	0.011	0.545	0.201	0.087	0.052	0.571	#13	0.076	Not Required	Pass
308	0.004	0.088	0.219	0.054	0.023	0.242	#21	0.102	Not Required	Pass
309	0.018	0.068	0.061	0.001	0.000	0.132	#13	0.206	Not Required	Pass
310	0.011	0.852	0.193	0.085	0.042	0.924	#13	0.082	Not Required	Pass
311	0.006	0.112	0.224	0.054	0.023	0.234	#21	0.102	Not Required	Pass
312	0.004	0.507	0.326	0.110	0.061	0.834	#13	0.036	Not Required	Pass
313	0.009	0.235	0.546	0.069	0.030	0.714	#21	0.306	Not Required	Pass
314	0.010	0.264	0.540	0.067	0.030	0.727	#21	0.306	Not Required	Pass
315	0.012	0.332	0.287	0.050	0.023	0.554	#21	0.507	Not Required	Pass
316	0.007	0.285	0.283	0.048	0.023	0.521	#21	0.507	Not Required	Pass
401	0.040	0.605	0.043	0.044	0.004	0.640	#13	0.413	Not Required	Pass
402	0.001	0.501	0.295	0.101	0.055	0.797	#13	0.054	Not Required	Pass
403	0.011	0.805	0.109	0.082	0.029	0.870	#13	0.046	Not Required	Pass
404	0.013	0.731	0.181	0.073	0.040	0.745	#13	0.082	Not Required	Pass
405	0.012	0.499	0.190	0.080	0.047	0.523	#13	0.076	Not Required	Pass
406	0.007	0.557	0.028	0.054	0.002	0.563	#13	0.046	Not Required	Pass
407	0.006	0.346	0.064	0.055	0.017	0.349	#13	0.076	Not Required	Pass
408	0.000	0.019	0.051	0.016	0.007	0.067	#21	Not Required	Not Required	Pass
409	0.008	0.073	0.091	0.004	0.005	0.160	#13	0.206	Not Required	Pass
410	0.006	0.521	0.083	0.052	0.022	0.588	#13	0.082	Not Required	Pass
411	0.000	0.020	0.051	0.017	0.007	0.068	#21	Not Required	Not Required	Pass
412	0.003	0.265	0.193	0.066	0.041	0.459	#13	0.054	Not Required	Pass
413	0.009	0.134	0.531	0.064	0.029	0.580	#21	0.204	Not Required	Pass
414	0.006	0.117	0.523	0.057	0.029	0.551	#24	0.306	Not Required	Pass
415	0.012	0.465	0.287	0.049	0.023	0.666	#21	0.507	Not Required	Pass
416	0.006	0.432	0.283	0.044	0.023	0.636	#21	0.507	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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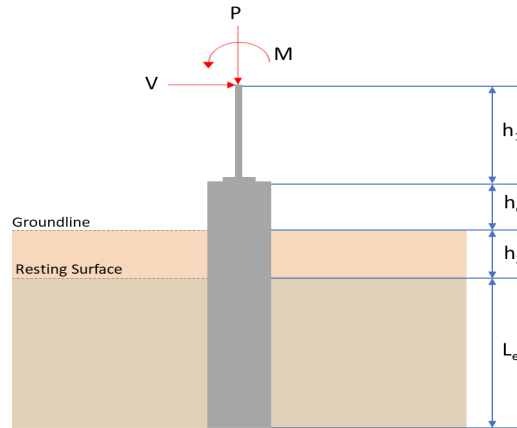
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 7.5$ 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)	8.966	13.156
V_x (kip)	-4.274	-7.154
V_z (kip)	0.442	0.717
M_x (kipft)	1.381	2.198
M_z (kipft)	53.406	89.393

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,

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

$$H_o = \frac{(-4.274 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.54446 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.406 \text{ kipft}) + ((-4.274 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 6.8033 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8446 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.442 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.056306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.381 \text{ kipft}) + ((0.442 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.17592 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.8761 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.8446 \text{ ft}), (2.8761 \text{ ft})]$$

$$L_{e,req} = 6.845 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.845 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.91267$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.966 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.35864 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35864 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17932$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.54446 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8033 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8033 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (6.8033 \text{ kipft/ft})) + (4 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1786 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8033 \text{ kipft/ft})) + (3 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (6.8033 \text{ kipft/ft})) + (2 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.24383 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8033 \text{ kipft/ft})) + ((-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0158 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1786 \text{ ft})}{2}$$

$$p_a = 0.3884 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24383 \text{ kip/ft}^2)}{(0.3884 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.62778$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0158 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90294$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.056306 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.17592 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17592 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.056306 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.17592 \text{ kipft/ft})) + (4 \times (0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3846 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.17592 \text{ kipft/ft})) + (3 \times (0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.17592 \text{ kipft/ft})) + (2 \times (0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.037727 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.17592 \text{ kipft/ft})) + ((0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.082575 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3846 \text{ ft})}{2}$$

$$p_a = 0.40385 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.037727 \text{ kip/ft}^2)}{(0.40385 \text{ kip/ft}^2)}$$

$$Ratio = 0.09342$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

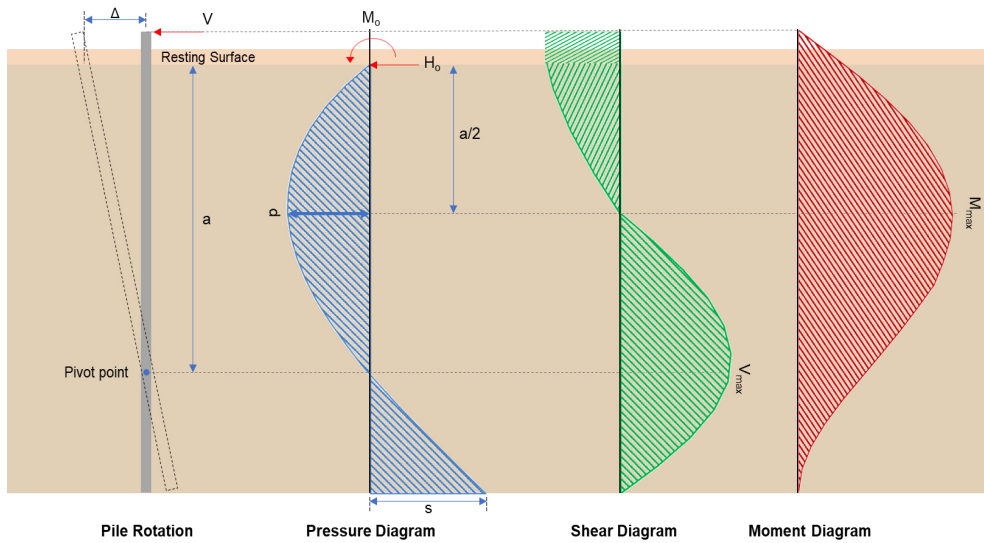
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.082575 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0734$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.154 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.91134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(89.393 \text{ kipft}) + ((-7.154 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 11.388 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.388 \text{ kipft/ft})}{(-0.91134 \text{ kip/ft})}$$

$$E = 12.496 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.388 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.388 \text{ kipft/ft})) + (4 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.91134 \text{ kip/ft}) \times (60 \text{ in}) + (4 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft}))}{(6 \times (11.388 \text{ kipft/ft})) + (4 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1786 \text{ ft}$$

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

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

$$V_{max} = ((-0.91134 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1786 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1786 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.439 \text{ kip}$$

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

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

$$M_{max} = ((-0.91134 \text{ kip/ft}) \times (60 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(12.496 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 58.544 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.717 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.091338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.198 \text{ kipft}) + ((0.717 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.28 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.28 \text{ kipft/ft})}{(0.091338 \text{ kip/ft})}$$

$$E = 3.0656 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.091338 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.28 \text{ kipft/ft})) + (4 \times (0.091338 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3875 \text{ ft}$$

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

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

$$V_{max} = ((0.091338 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.0656 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3875 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.0656 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3875 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.63553 \text{ kip}$$

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

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

$$M_{max} = ((0.091338 \text{ kip/ft}) \times (60 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.0656 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3875 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.0656 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3875 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.0656 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3875 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.0785 \text{ kipft}$$

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,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.156 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.74 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.74 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.156 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0031465$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.156 \text{ kip} \rightarrow 13156 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(13156 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.87 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.87 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.87 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.87 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.42 \text{ kip}$$

Considering x-direction:

V_{max} = 16.439 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.439 \text{ kip})}{(152.42 \text{ kip})}$$

$$Ratio = 0.10785$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$V_{max} = 0.63553 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.63553 \text{ kip})}{(152.42 \text{ kip})}$$

$$Ratio = 0.0041696$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 58.544 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(58.544 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.12009$$

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

Considering z-direction:

$M_{max} = 2.0785 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.0785 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.0042637$$

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

REFERENCES	CALCULATIONS	RESULTS
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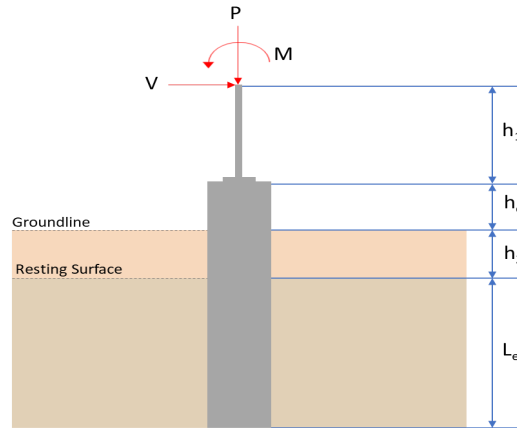
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 7.5$ 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)	8.966	13.156
V_x (kip)	-4.274	-7.154
V_z (kip)	-0.442	-0.716
M_x (kipft)	-1.381	-2.199
M_z (kipft)	53.406	89.394

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,

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

$$H_o = \frac{(-4.274 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.54446 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.406 \text{ kipft}) + ((-4.274 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 6.8033 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8446 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.442 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.056306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.381 \text{ kipft}) + ((-0.442 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.17592 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9548 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.8446 \text{ ft}), (1.9548 \text{ ft})]$$

$$L_{e,req} = 6.845 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.845 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.91267$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.966 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.35864 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35864 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17932$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.54446 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8033 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8033 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (6.8033 \text{ kipft/ft})) + (4 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1786 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8033 \text{ kipft/ft})) + (3 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (6.8033 \text{ kipft/ft})) + (2 \times (-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.24383 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8033 \text{ kipft/ft})) + ((-0.54446 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0158 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1786 \text{ ft})}{2}$$

$$p_a = 0.3884 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.24383 \text{ kip/ft}^2)}{(0.3884 \text{ kip/ft}^2)}$$

$$Ratio = 0.62778$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0158 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.90294$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.056306 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.17592 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17592 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.056306 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.17592 \text{ kipft/ft})) + (4 \times (-0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3846 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.17592 \text{ kipft/ft})) + (3 \times (-0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 [(3 \times (0.17592 \text{ kipft/ft})) + (2 \times (-0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.013349 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.17592 \text{ kipft/ft})) + ((-0.056306 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = -0.0075142 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3846 \text{ ft})}{2}$$

$$p_a = 0.40385 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.013349 \text{ kip/ft}^2)}{(0.40385 \text{ kip/ft}^2)}$$

$$Ratio = -0.033053$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

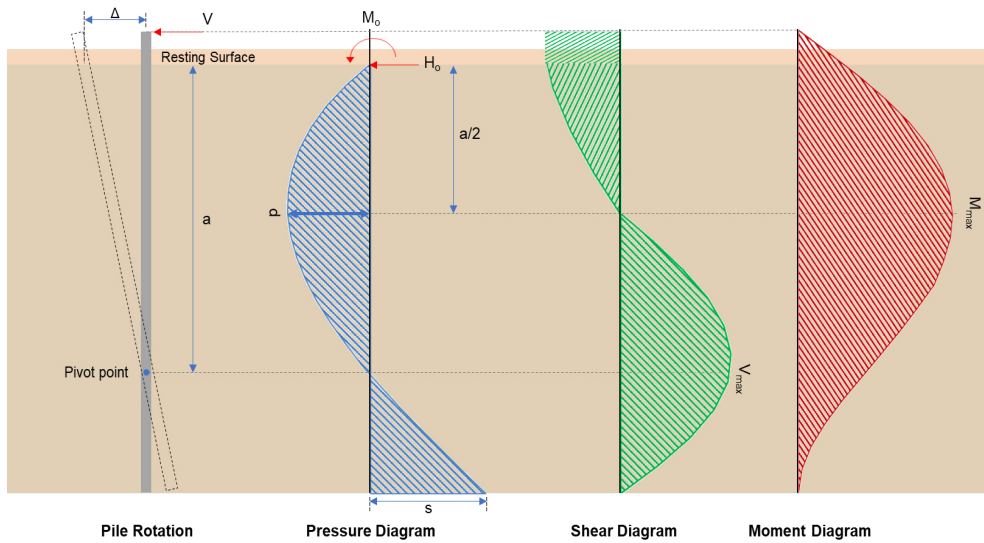
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0075142 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = -0.0066793$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.154 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.91134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(89.394 \text{ kipft}) + ((-7.154 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 11.388 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.388 \text{ kipft/ft})}{(-0.91134 \text{ kip/ft})}$$

$$E = 12.496 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.388 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.388 \text{ kipft/ft})) + (4 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft})}{(6 \times (11.388 \text{ kipft/ft})) + (4 \times (-0.91134 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1786 \text{ ft}$$

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

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

$$V_{max} = ((-0.91134 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1786 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1786 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.439 \text{ kip}$$

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

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

$$M_{max} = ((-0.91134 \text{ kip/ft}) \times (60 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(12.496 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.496 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1786 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 58.544 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.716 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.09121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.199 \text{ kipft}) + ((-0.716 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.28013 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.28013 \text{ kipft/ft})}{(-0.09121 \text{ kip/ft})}$$

$$E = 3.0712 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28013 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.09121 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.28013 \text{ kipft/ft})) + (4 \times (-0.09121 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3872 \text{ ft}$$

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

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

$$V_{max} = ((-0.09121 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.0712 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3872 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.0712 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3872 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.63525 \text{ kip}$$

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

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

$$M_{max} = ((-0.09121 \text{ kip/ft}) \times (60 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.0712 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3872 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.0712 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3872 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (3.0712 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3872 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 2.0779 \text{ kipft}$$

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,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.156 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.74 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.74 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.156 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0031465$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.156 \text{ kip} \rightarrow 13156 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(13156 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.87 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.87 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.87 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.87 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.42 \text{ kip}$$

Considering x-direction:

V_{max} = 16.439 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.439 \text{ kip})}{(152.42 \text{ kip})}$$

$$Ratio = 0.10785$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$V_{max} = 0.63525 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.63525 \text{ kip})}{(152.42 \text{ kip})}$$

$$Ratio = 0.0041678$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 58.544 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(58.544 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.12009$$

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

Considering z-direction:

$M_{max} = 2.0779 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(2.0779 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.0042623$$

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

REFERENCES	CALCULATIONS	RESULTS
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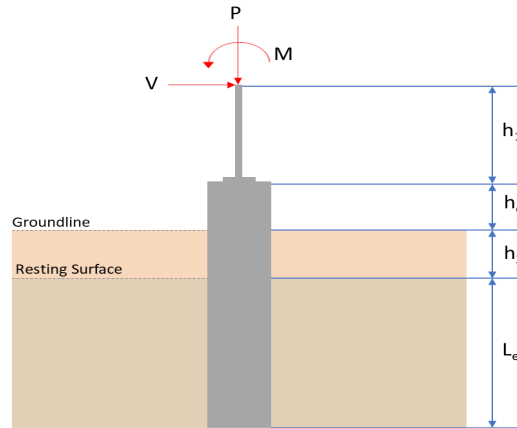
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ 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)	11.640	17.185
V_x (kip)	-5.636	-9.409
V_z (kip)	-0.026	-0.043
M_x (kipft)	-0.079	-0.123
M_z (kipft)	69.863	117.806

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,

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

$$H_o = \frac{(-5.636 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.71796 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.863 \text{ kipft}) + ((-5.636 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 8.8997 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3417 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.026 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0033121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.079 \text{ kipft}) + ((-0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.010064 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.85915 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3417 \text{ ft}), (0.85915 \text{ ft})]$$

$$L_{e,req} = 7.342 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(7.342 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.91775$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.64 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.4656 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4656 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2328$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.6$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.71796 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8997 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8997 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.8997 \text{ kipft/ft})) + (4 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5339 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.8997 \text{ kipft/ft})) + (3 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.8997 \text{ kipft/ft})) + (2 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.25991 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.8997 \text{ kipft/ft})) + ((-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1302 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5339 \text{ ft})}{2}$$

$$p_a = 0.41504 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.25991 \text{ kip/ft}^2)}{(0.41504 \text{ kip/ft}^2)}$$

$$Ratio = 0.62622$$

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

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

$$Ratio = \frac{(1.1302 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.94186$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.0033121 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010064 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0033121 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.010064 \text{ kipft/ft})) + (4 \times (-0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.758 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.010064 \text{ kipft/ft})) + (3 \times (-0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 [(3 \times (0.010064 \text{ kipft/ft})) + (2 \times (-0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.00079115 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.010064 \text{ kipft/ft})) + ((-0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = -0.00059713 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.758 \text{ ft})}{2}$$

$$p_a = 0.43185 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00079115 \text{ kip/ft}^2)}{(0.43185 \text{ kip/ft}^2)}$$

$$Ratio = -0.001832$$

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

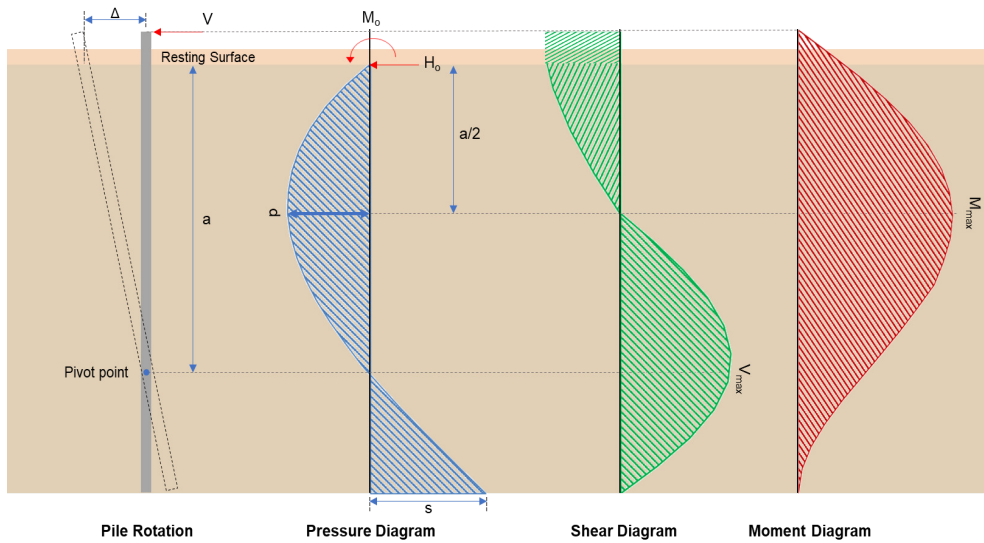
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00059713 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = -0.00049761$$

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,

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

$$H_o = \frac{(-9.409 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -1.1986 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(117.81 \text{ kipft}) + ((-9.409 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 15.007 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.007 \text{ kipft/ft})}{(-1.1986 \text{ kip/ft})}$$

$$E = 12.521 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.007 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.007 \text{ kipft/ft})) + (4 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.007 \text{ kipft/ft})) + (4 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5325 \text{ ft}$$

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

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

$$V_{max} = ((-1.1986 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5325 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5325 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 20.549 \text{ kip}$$

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

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

$$M_{max} = ((-1.1986 \text{ kip/ft}) \times (60 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.521 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 77.847 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.043 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0054777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((-0.043 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.015669 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015669 \text{ kipft/ft})}{(-0.0054777 \text{ kip/ft})}$$

$$E = 2.8605 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015669 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0054777 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.015669 \text{ kipft/ft})) + (4 \times (-0.0054777 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7673 \text{ ft}$$

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

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

$$V_{max} = ((-0.0054777 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7673 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7673 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.035672 \text{ kip}$$

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

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

$$M_{max} = ((-0.0054777 \text{ kip/ft}) \times (60 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.8605 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12323 \text{ kipft}$$

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,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.185 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.61 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.61 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(17.185 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0041101$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.185 \text{ kip} \rightarrow 17185 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(17185 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.41 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (171.41 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.41 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((171.41 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.77 \text{ kip}$$

Considering x-direction:

V_{max} = 20.549 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(20.549 \text{ kip})}{(152.77 \text{ kip})}$$

$$Ratio = 0.13451$$

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

Considering z-direction:

$V_{max} = 0.035672 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.035672 \text{ kip})}{(152.77 \text{ kip})}$$

$$Ratio = 0.0002335$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 77.847 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(77.847 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.15969$$

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

Considering z-direction:

$M_{max} = 0.12323 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.12323 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00025277$$

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

REFERENCES	CALCULATIONS	RESULTS
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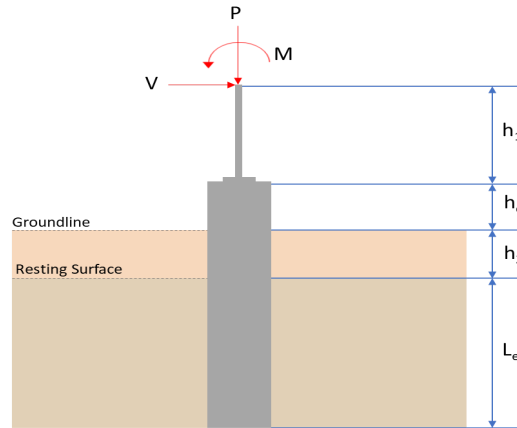
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ 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)	11.473	16.961
V_x (kip)	-5.585	-9.312
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	69.886	117.720

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,

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

$$H_o = \frac{(-5.585 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.71146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.886 \text{ kipft}) + ((-5.585 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 8.9027 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.3568 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(7.3568 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.357 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(7.357 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.91963$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.473 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.45892 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.45892 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22946$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.6$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.71146 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.9027 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.9027 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.71146 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.9027 \text{ kipft/ft})) + (4 \times (-0.71146 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5326 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.9027 \text{ kipft/ft})) + (3 \times (-0.71146 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.9027 \text{ kipft/ft})) + (2 \times (-0.71146 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.26273 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.9027 \text{ kipft/ft})) + ((-0.71146 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1357 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5326 \text{ ft})}{2}$$

$$p_a = 0.41494 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26273 \text{ kip/ft}^2)}{(0.41494 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63316$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

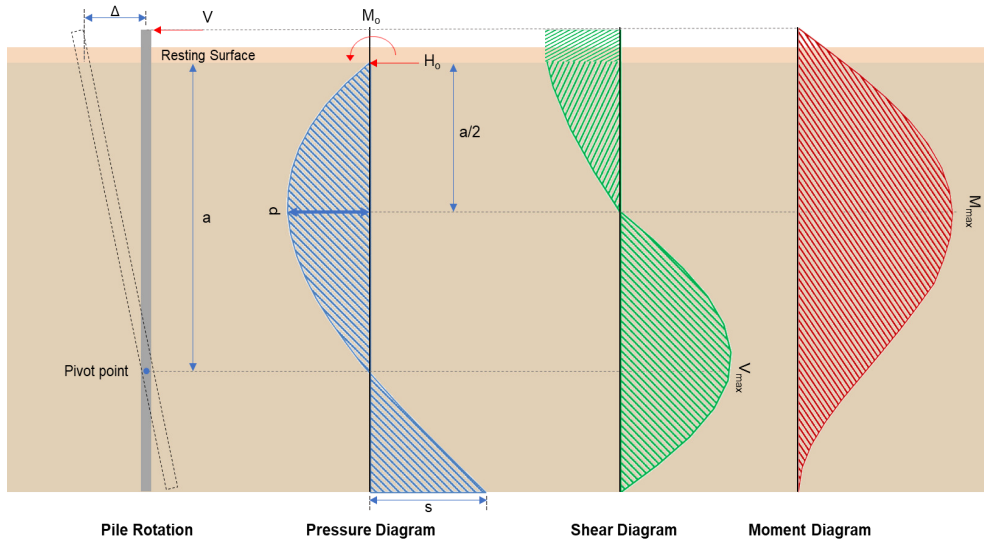
$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1357 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.630**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.312 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -1.1862 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(117.72 \text{ kipft}) + ((-9.312 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 14.996 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.996 \text{ kipft/ft})}{(-1.1862 \text{ kip/ft})}$$

$$E = 12.642 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.996 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.1862 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.996 \text{ kipft/ft})) + (4 \times (-1.1862 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5311 \text{ ft}$$

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

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

$$V_{max} = ((-1.1862 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.642 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5311 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.642 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5311 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 20.490 \text{ kip}$$

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

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

$$M_{max} = ((-1.1862 \text{ kip/ft}) \times (60 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.642 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5311 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.642 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5311 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (12.642 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5311 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 77.68 \text{ kipft}$$

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,
 $A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{16.961 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.62 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.62 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

$$\text{Ratio} = 0.96007$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

<p>25.7.2.2 25.7.2.1</p>	$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties: Since longitudinal reinforcement is \leq No. 10ϕ: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p>Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$ $\phi P_N = 4181.1 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(16.961 \text{ kip})}{(4181.1 \text{ kip})}$ $Ratio = 0.0040566$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters: $b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (60 \text{ in})$ $d = 48 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.58722$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$ $V_{c,max} = 422.8 \text{ kip}$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.961 \text{ kip} \rightarrow 16961 \text{ lbf}$,</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + \frac{(16961 \text{ lbf})}{6 \times (3600 \text{ in}^2)} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.38 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (171.38 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.38 \text{ kip}$$

22.5.1.2 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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((171.38 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.75 \text{ kip}$$

Considering x-direction:

$V_{max} = 20.496 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(20.496 \text{ kip})}{(152.75 \text{ kip})}$$

$$\text{Ratio} = 0.13418$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 77.68 \text{ kipft}$ - Maximum moment in the x-direction,

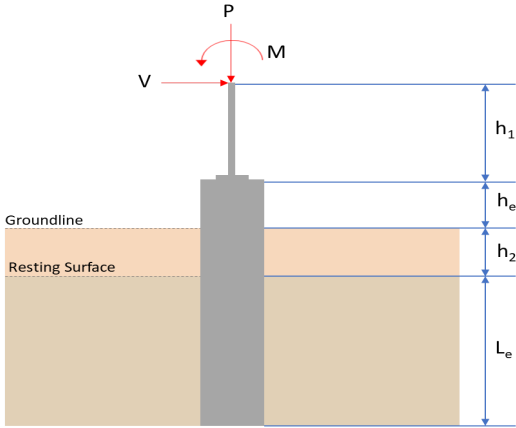
Ratio - Capacity

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

$$\text{Ratio} = \frac{(77.68 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.15934$$

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

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 $b = 60$ in - Pile width $D = 60$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1084 1225 1184"> <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>11.640</td> <td>17.185</td> </tr> <tr> <td>V_x (kip)</td> <td>-5.636</td> <td>-9.409</td> </tr> <tr> <td>V_z (kip)</td> <td>0.026</td> <td>0.043</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.079</td> <td>0.123</td> </tr> <tr> <td>M_z (kipft)</td> <td>69.863</td> <td>117.806</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)	11.640	17.185	V_x (kip)	-5.636	-9.409	V_z (kip)	0.026	0.043	M_x (kipft)	0.079	0.123	M_z (kipft)	69.863	117.806	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	69.863	117.806																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-5.636 \text{ kip})}{1.57 \times (60 \text{ in})}$ $H_o = -0.71796 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.863 \text{ kipft}) + ((-5.636 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 8.8997 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.3417 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0033121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.079 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.010064 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0012 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3417 \text{ ft}), (1.0012 \text{ ft})]$$

$$L_{e,req} = 7.342 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(7.342 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.91775$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.64 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.4656 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4656 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2328$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.6$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.71796 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8997 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8997 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.8997 \text{ kipft/ft})) + (4 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5339 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.8997 \text{ kipft/ft})) + (3 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.8997 \text{ kipft/ft})) + (2 \times (-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.25991 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.8997 \text{ kipft/ft})) + ((-0.71796 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1302 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5339 \text{ ft})}{2}$$

$$p_a = 0.41504 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.25991 \text{ kip/ft}^2)}{(0.41504 \text{ kip/ft}^2)}$$

$$Ratio = 0.62622$$

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

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

$$Ratio = \frac{(1.1302 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.94186$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.0033121 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010064 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0033121 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.010064 \text{ kipft/ft})) + (4 \times (0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.758 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010064 \text{ kipft/ft})) + (3 \times (0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.010064 \text{ kipft/ft})) + (2 \times (0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.00202 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010064 \text{ kipft/ft})) + ((0.0033121 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.004371 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.758 \text{ ft})}{2}$$

$$p_a = 0.43185 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00202 \text{ kip/ft}^2)}{(0.43185 \text{ kip/ft}^2)}$$

$$Ratio = 0.0046775$$

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

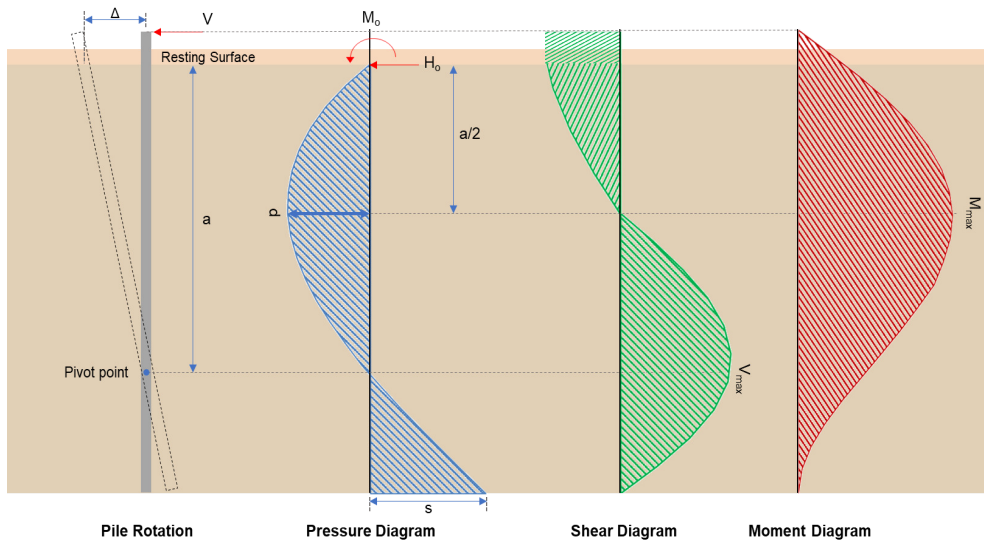
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.004371 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.0036425$$

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,

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

$$H_o = \frac{(-9.409 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -1.1986 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(117.81 \text{ kipft}) + ((-9.409 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 15.007 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.007 \text{ kipft/ft})}{(-1.1986 \text{ kip/ft})}$$

$$E = 12.521 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.007 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.007 \text{ kipft/ft})) + (4 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.007 \text{ kipft/ft})) + (4 \times (-1.1986 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5325 \text{ ft}$$

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

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

$$V_{max} = ((-1.1986 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5325 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5325 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 20.549 \text{ kip}$$

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

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

$$M_{max} = ((-1.1986 \text{ kip/ft}) \times (60 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.521 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.521 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5325 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 77.847 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.043 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0054777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((0.043 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.015669 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015669 \text{ kipft/ft})}{(0.0054777 \text{ kip/ft})}$$

$$E = 2.8605 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015669 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0054777 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.015669 \text{ kipft/ft})) + (4 \times (0.0054777 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7673 \text{ ft}$$

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

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

$$V_{max} = ((0.0054777 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7673 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7673 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.035672 \text{ kip}$$

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

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

$$M_{max} = ((0.0054777 \text{ kip/ft}) \times (60 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.8605 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.8605 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7673 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12323 \text{ kipft}$$

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,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{17.185 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.61 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.61 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(17.185 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0041101$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.185 \text{ kip} \rightarrow 17185 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(17185 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.41 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (171.41 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.41 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((171.41 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.77 \text{ kip}$$

Considering x-direction:

V_{max} = 20.549 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(20.549 \text{ kip})}{(152.77 \text{ kip})}$$

$$Ratio = 0.13451$$

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

Considering z-direction:

$V_{max} = 0.035672 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.035672 \text{ kip})}{(152.77 \text{ kip})}$$

$$Ratio = 0.0002335$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 77.847 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(77.847 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.15969$$

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

Considering z-direction:

$M_{max} = 0.12323 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.12323 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00025277$$

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