Your Project Calculations

Project Name: W12185 - REVISION CU-5

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=W12185%20-%20REVISION%20CU-5&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/12_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=S4WyEL0apzQmkTzmrWQvaM7ShpBdK7rYYCRZzE3JgIPDQijPMIhaddLiV1cxDkh4

Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-HD-57-L-5Hx10W-BDAH
Duty Classification:	HD
Module Width:	40.90 in
Module Length:	69.13in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	17
Front Edge Clearance:	10
Total Array Height at Tilt:	15.01 ft
Total Frame Length:	56.50 ft
Frame Weight:	2487 lbs
Array Dimensions N/S:	17.25 ft
Array Dimensions E/W:	58.44 ft
Rail Length:	207.00 in
Rail Spacing:	2.88 ft
Rail Check:	Not Checked

Support Specifications

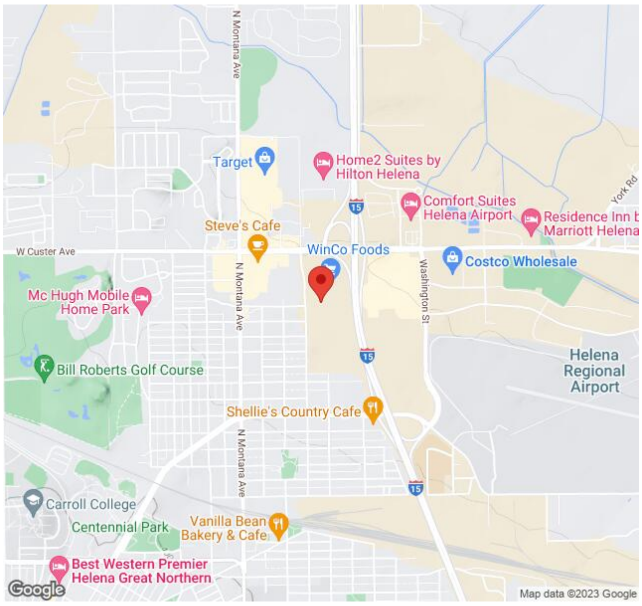
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.52 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 5.75 ft Pile 3: 5.75 ft
Foundation Volume:	10.222 y^3
Foundation Result:	PASSED
Mount Twist:	0.062108 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sandy_gravel
Site Location:	2845 N Sanders St, Helena, MT 59601, USA
Wind Speed:	106 mph
Snow Load:	30 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.017484 ksf



Design Disclaimer

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

AutoDesigner Input

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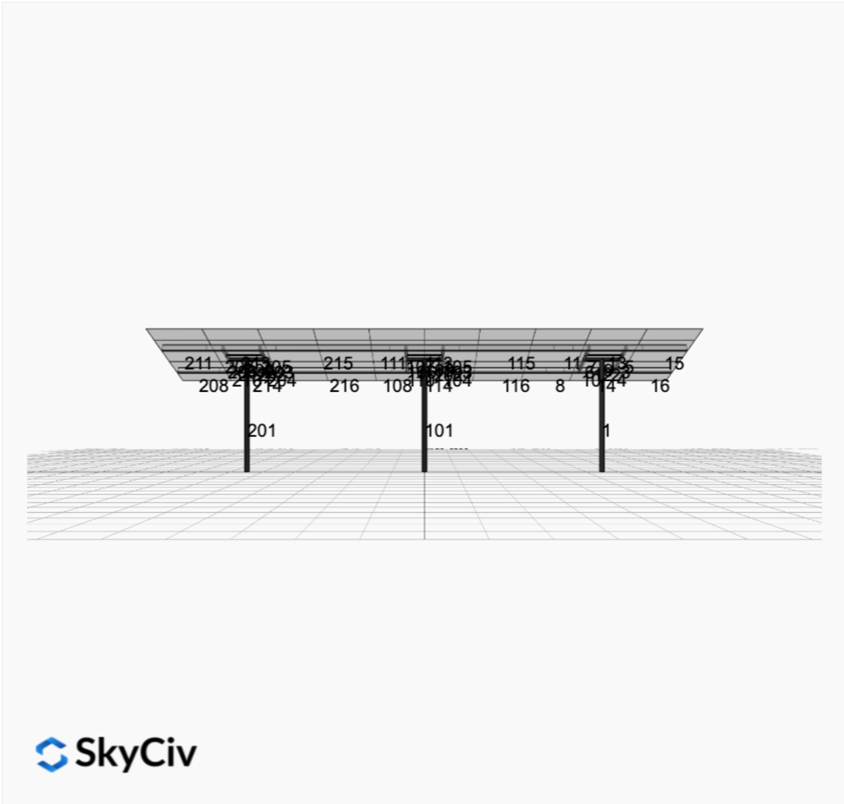
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation 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)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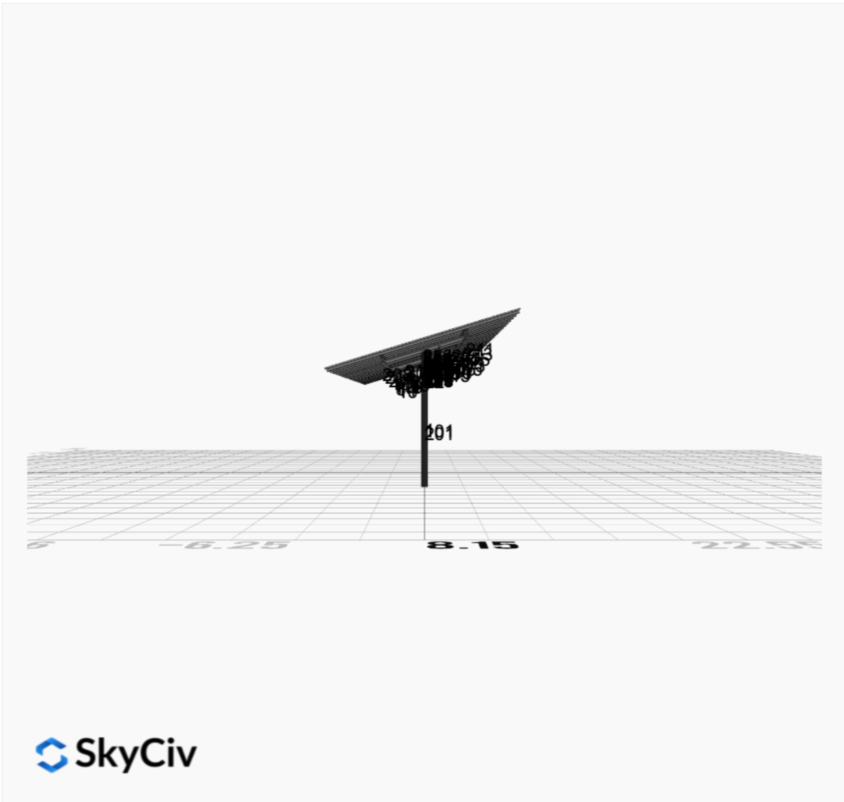
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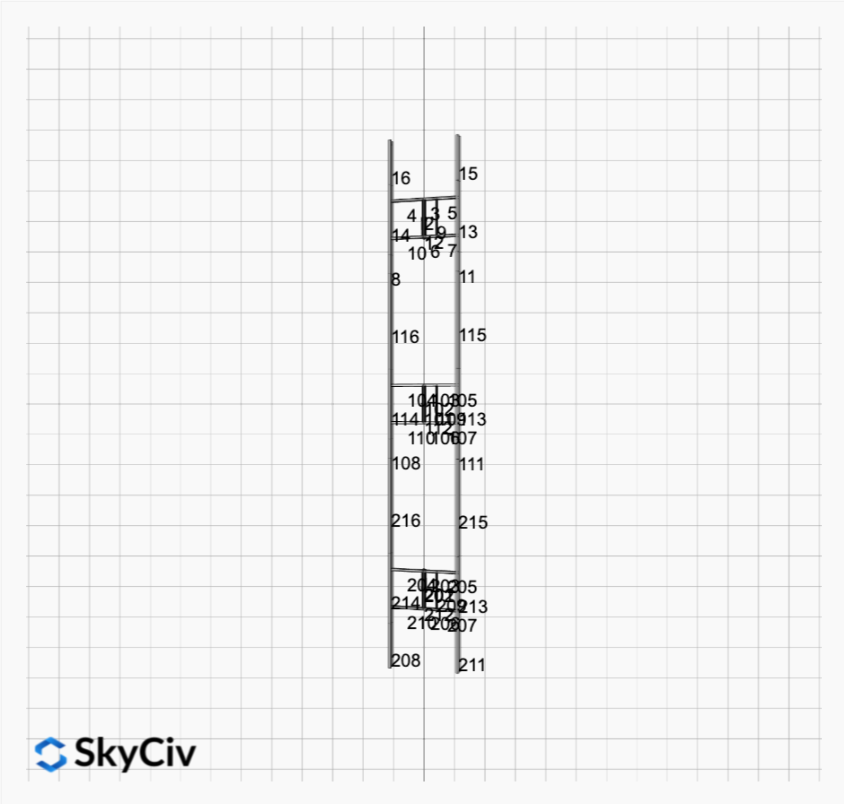
8.15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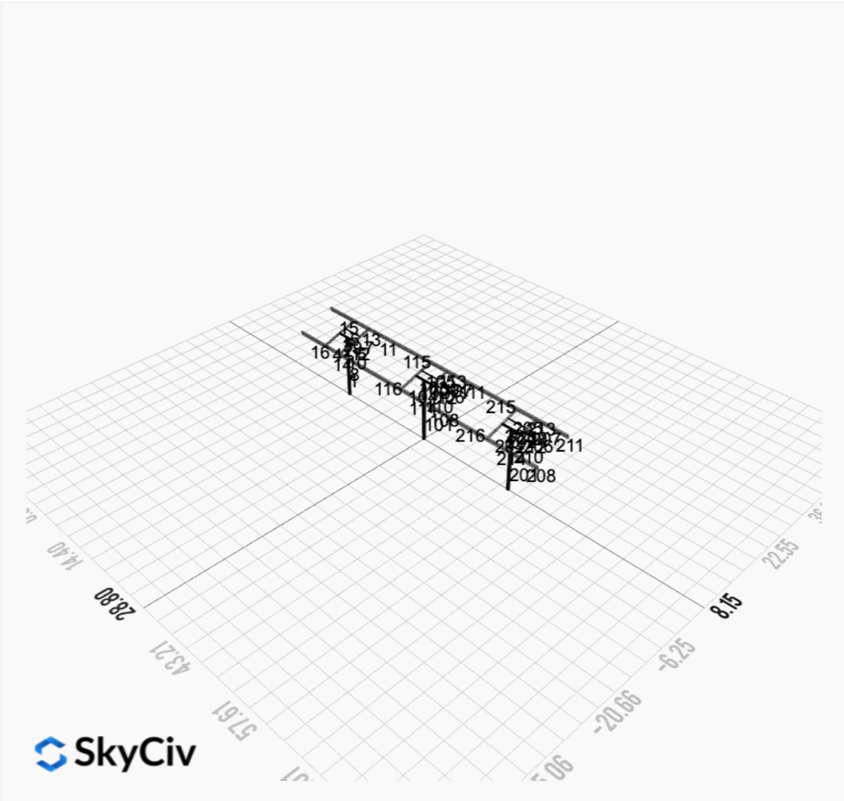
SkyCiv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8.15
-6.25
-20.66
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116
215
216
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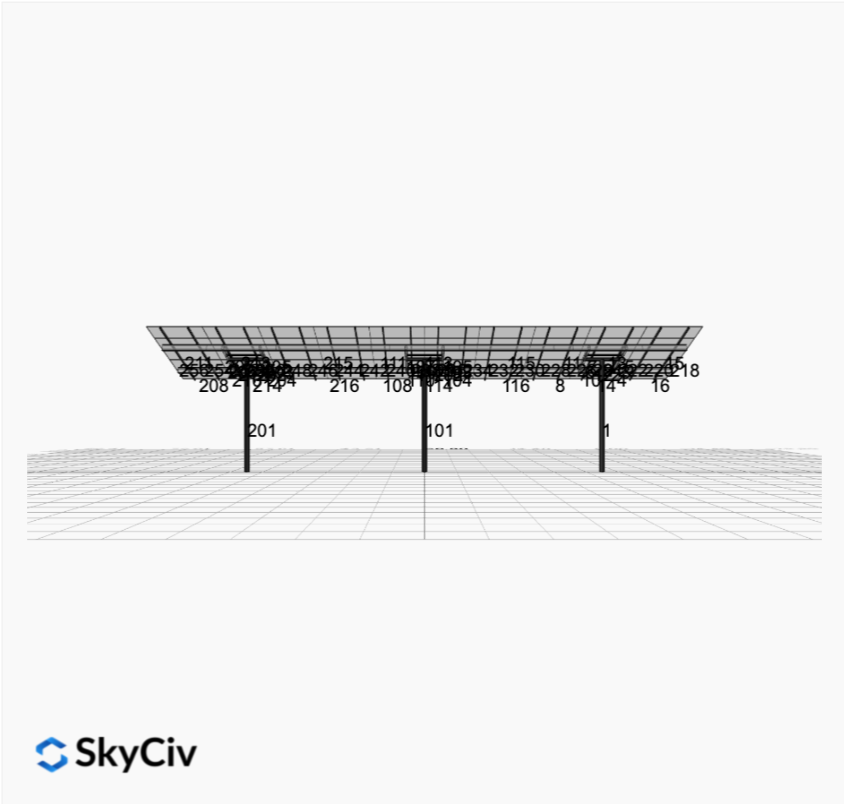
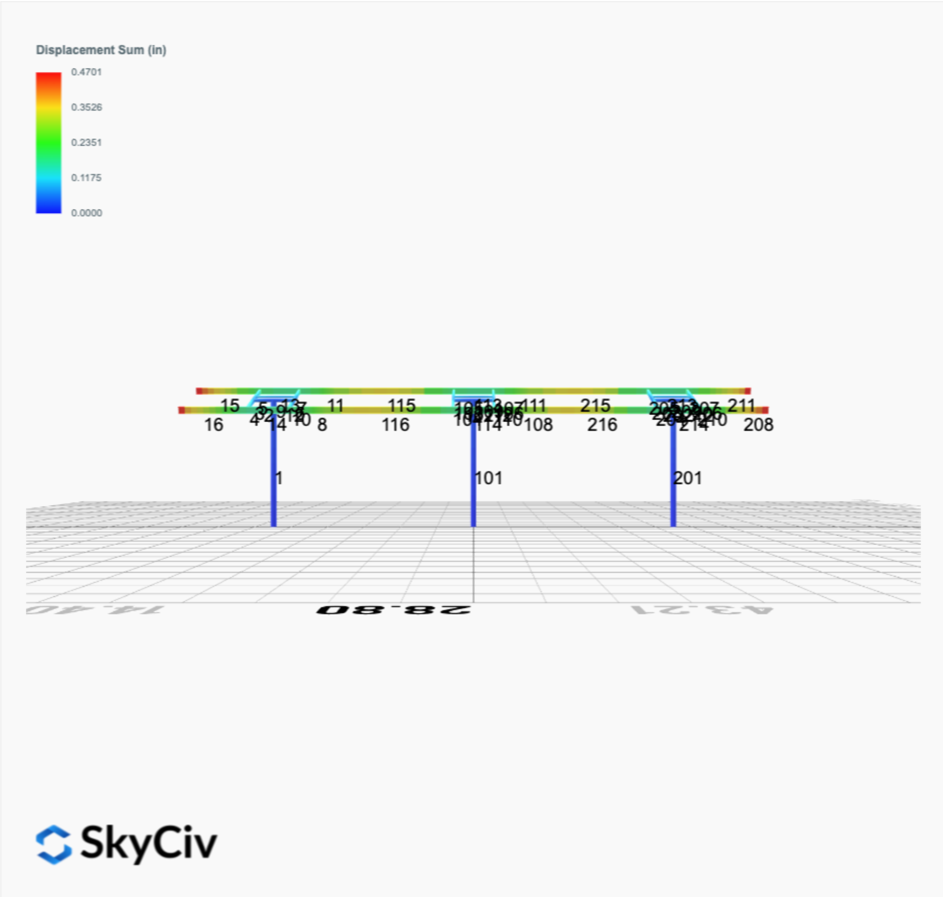
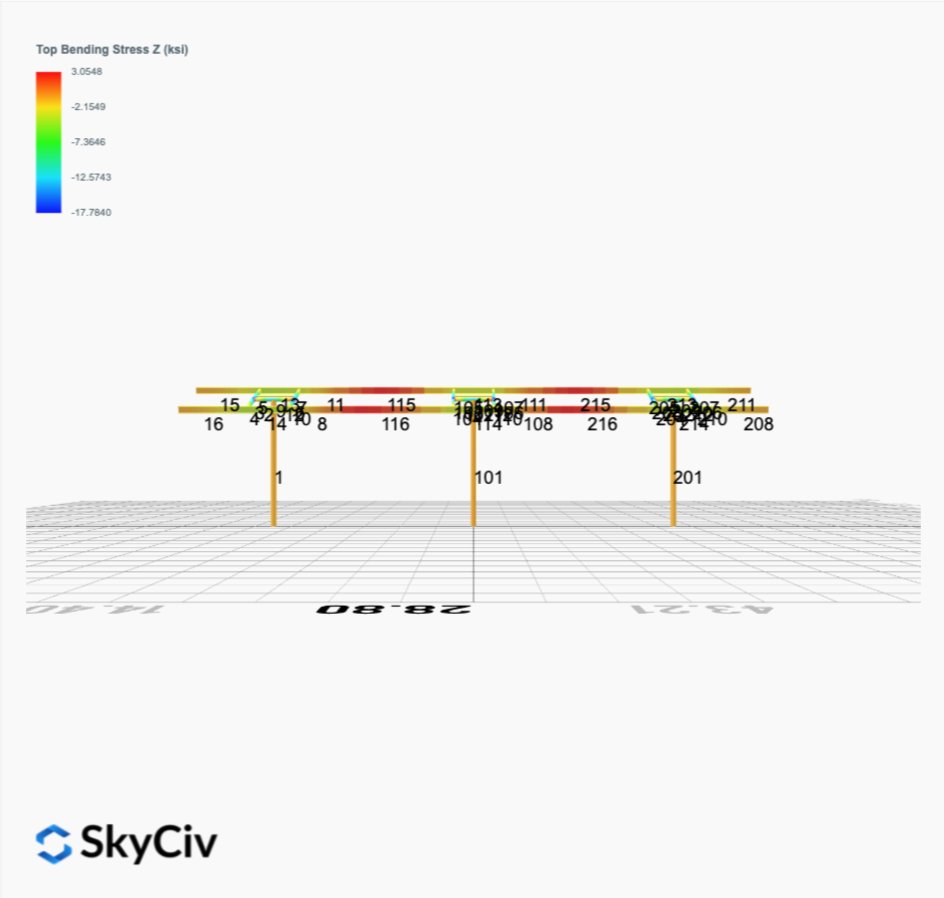
SkyCiv


208
216
108
116
16
201
101

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
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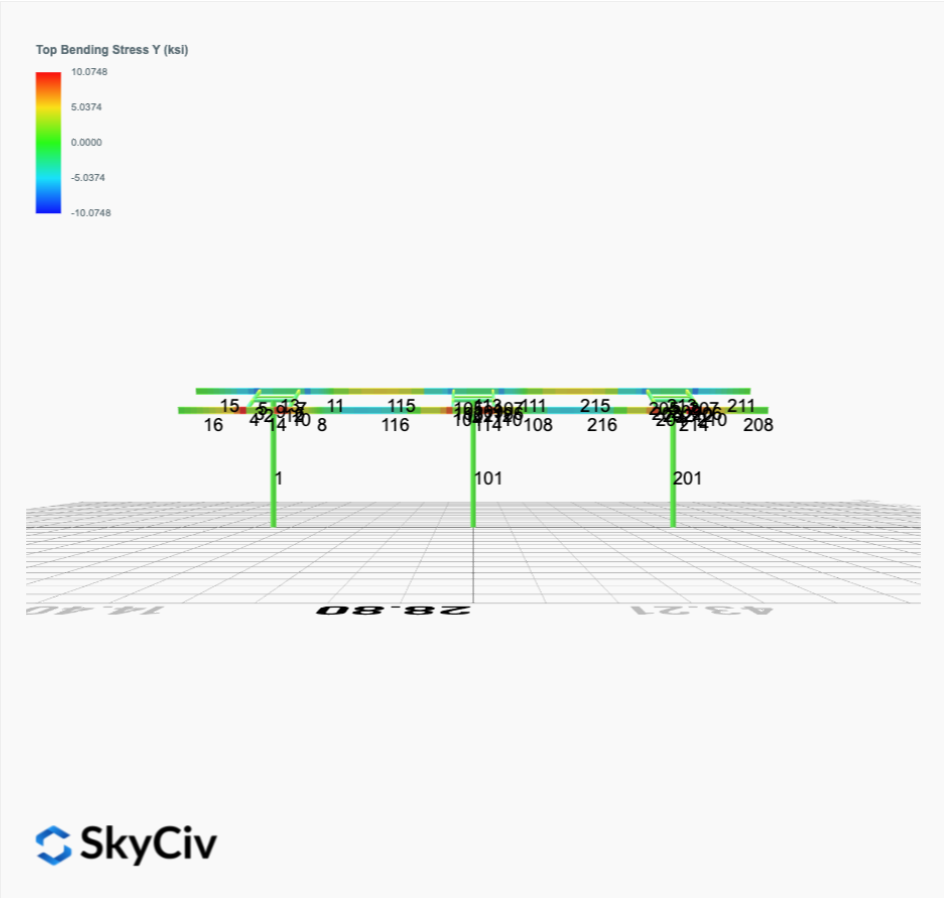
-2.1549
-7.3646
-12.5743
-17.7840
SkyCiv


Top Bending Stress Y (ksi)
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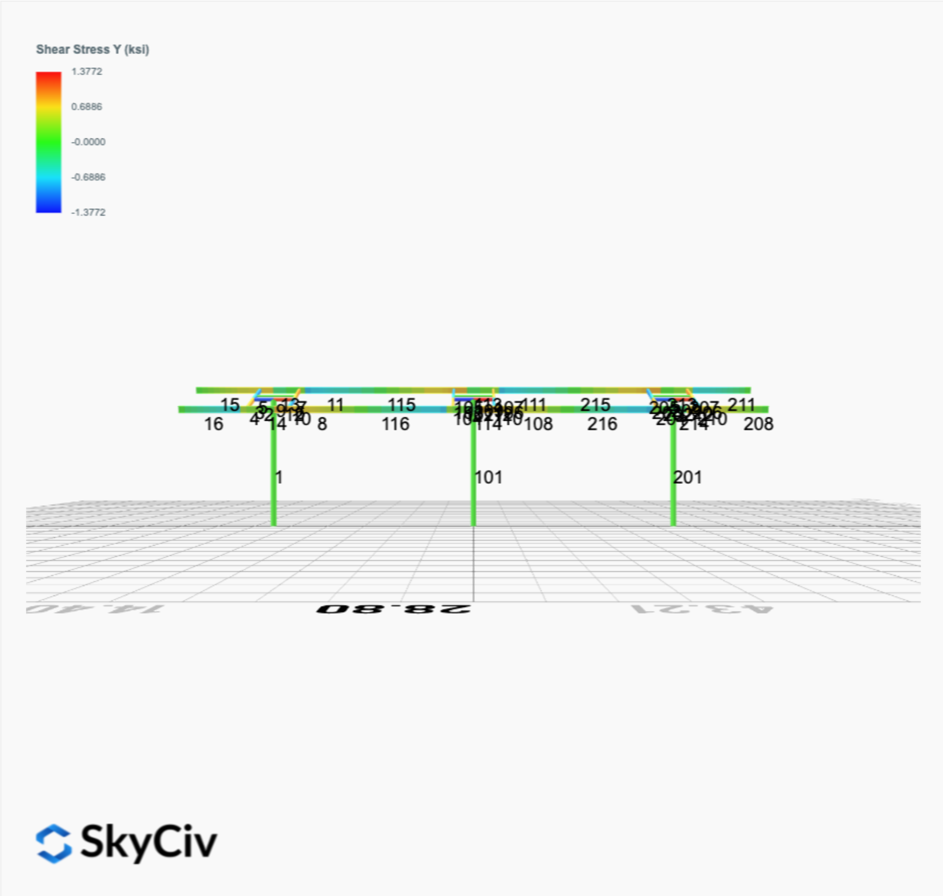
-5.0374
-10.0748
SkyCiv


Shear Stress Y (ksi)
1.3772
0.6886
-0.0000
-0.6886
-1.3772
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11
115
215
211
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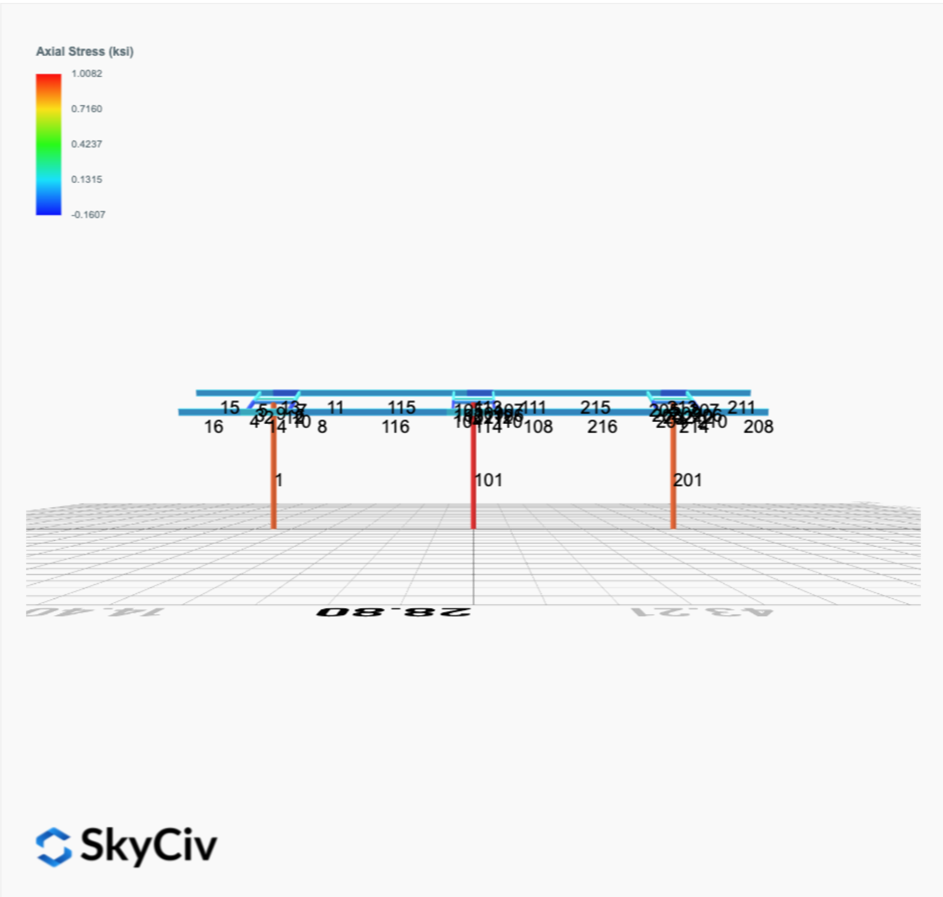
101
201
28.80
SkyCiv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11
115
215
211
16
8
116
108
216
208
1
101
201
28.80
SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0026	2.3387	-0.0049	-0.0192	0.0101	0.0547
ULS: 2. D + L	-0.0026	2.3387	-0.0049	-0.0192	0.0101	0.0547
ULS: 3. D + (S or Lr or R)	-0.0096	7.6731	-0.0185	-0.0723	0.0386	0.1488
ULS: 3. D + (S or Lr or R)	-0.0026	2.3387	-0.0049	-0.0192	0.0101	0.0547
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0079	6.3395	-0.0151	-0.0590	0.0315	0.1253
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0026	2.3387	-0.0049	-0.0192	0.0101	0.0547
ULS: 5b. D + 0.7E	-0.0026	2.3387	-0.0049	-0.0192	0.0101	0.0547
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0079	6.3395	-0.0151	-0.0590	0.0315	0.1253
ULS: 8. 0.6D + 0.7E	-0.0015	1.4032	-0.0029	-0.0115	0.0061	0.0328
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3750	6.7966	-0.0101	-0.0403	0.0112	20.1425
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3750	6.7966	-0.0101	-0.0403	0.0112	20.1425
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0884	-1.2139	0.0011	0.0049	0.0067	-11.2336
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9497	-0.7425	-0.0037	-0.0139	0.0128	-21.8152
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0372	9.6829	-0.0190	-0.0749	0.0322	15.1911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0372	9.6829	-0.0190	-0.0749	0.0322	15.1911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8103	3.6750	-0.0106	-0.0410	0.0289	-8.3410
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7063	4.0286	-0.0142	-0.0551	0.0335	-16.2772
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0319	5.6821	-0.0088	-0.0350	0.0109	15.1205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0319	5.6821	-0.0088	-0.0350	0.0109	15.1205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8157	-0.3257	-0.0004	-0.0011	0.0076	-8.4115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7116	0.0278	-0.0040	-0.0152	0.0122	-16.3477
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3739	5.8611	-0.0081	-0.0327	0.0071	20.1206
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3739	5.8611	-0.0081	-0.0327	0.0071	20.1206
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0894	-2.1494	0.0031	0.0125	0.0026	-11.2555
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9507	-1.6780	-0.0017	-0.0062	0.0088	-21.8371

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.0570
Shear X	-2.2957
Shear Z	-0.0316
Moment X	-0.1254
Moment Y (Twist)	0.0622
Moment Z	38.6848

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	9.6829
Shear X	-1.3750
Shear Z	-0.0190
Moment X	-0.0749
Moment Y (Twist)	0.0386
Moment Z	21.8371

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	0.0051	2.4419	-0.0000	0.0000	0.0000	-0.0212
ULS: 2. D + L	0.0051	2.4419	-0.0000	0.0000	0.0000	-0.0212
ULS: 3. D + (S or Lr or R)	0.0193	8.0693	0.0000	0.0000	-0.0000	-0.1412
ULS: 3. D + (S or Lr or R)	0.0051	2.4419	-0.0000	0.0000	0.0000	-0.0212
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0157	6.6624	0.0000	0.0000	-0.0000	-0.1112
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0051	2.4419	-0.0000	0.0000	0.0000	-0.0212
ULS: 5b. D + 0.7E	0.0051	2.4419	-0.0000	0.0000	0.0000	-0.0212

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0157	6.6624	0.0000	0.0000	-0.0000	-0.1112
ULS: 8. 0.6D + 0.7E	0.0031	1.4652	-0.0000	0.0000	0.0000	-0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4029	7.1094	-0.0000	0.0000	0.0000	20.5375
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4029	7.1094	-0.0000	0.0000	0.0000	20.5375
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1346	-1.2839	-0.0000	0.0000	0.0000	-11.6144
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9687	-0.7766	-0.0000	0.0000	0.0000	-22.2508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0403	10.1630	0.0000	0.0000	-0.0000	15.3078
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0403	10.1630	0.0000	0.0000	-0.0000	15.3078
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8628	3.8681	0.0000	0.0000	-0.0000	-8.8061
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7384	4.2485	0.0000	0.0000	-0.0000	-16.7834
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0509	5.9425	-0.0000	0.0000	0.0000	15.3978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0509	5.9425	-0.0000	0.0000	0.0000	15.3978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8522	-0.3524	-0.0000	0.0000	0.0000	-8.7161
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7278	0.0280	-0.0000	0.0000	0.0000	-16.6934
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4049	6.1326	-0.0000	0.0000	0.0000	20.5460
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4049	6.1326	-0.0000	0.0000	0.0000	20.5460
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1325	-2.2606	-0.0000	0.0000	0.0000	-11.6060
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9666	-1.7534	-0.0000	0.0000	0.0000	-22.2423

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.8221
Shear X	-2.3467
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	39.5372

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	10.1630
Shear X	-1.4049
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	22.2508

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	-0.0026	2.3387	0.0049	0.0192	-0.0101	0.0547
ULS: 2. D + L	-0.0026	2.3387	0.0049	0.0192	-0.0101	0.0547
ULS: 3. D + (S or Lr or R)	-0.0096	7.6731	0.0185	0.0723	-0.0386	0.1488
ULS: 3. D + (S or Lr or R)	-0.0026	2.3387	0.0049	0.0192	-0.0101	0.0547
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0079	6.3395	0.0151	0.0590	-0.0315	0.1253
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0026	2.3387	0.0049	0.0192	-0.0101	0.0547
ULS: 5b. D + 0.7E	-0.0026	2.3387	0.0049	0.0192	-0.0101	0.0547
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0079	6.3395	0.0151	0.0590	-0.0315	0.1253
ULS: 8. 0.6D + 0.7E	-0.0015	1.4032	0.0029	0.0115	-0.0061	0.0328
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3750	6.7966	0.0101	0.0403	-0.0112	20.1425
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3750	6.7966	0.0101	0.0403	-0.0112	20.1425
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0884	-1.2139	-0.0011	-0.0049	-0.0067	-11.2336
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9497	-0.7425	0.0037	0.0139	-0.0128	-21.8152
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0372	9.6829	0.0190	0.0749	-0.0322	15.1911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0372	9.6829	0.0190	0.0749	-0.0322	15.1911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8103	3.6750	0.0106	0.0410	-0.0289	-8.3410
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7063	4.0286	0.0142	0.0551	-0.0335	-16.2772

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0319	5.6821	0.0088	0.0350	-0.0109	15.1205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0319	5.6821	0.0088	0.0350	-0.0109	15.1205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8157	-0.3257	0.0004	0.0011	-0.0076	-8.4115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7116	0.0278	0.0040	0.0152	-0.0122	-16.3477
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3739	5.8611	0.0081	0.0327	-0.0071	20.1206
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3739	5.8611	0.0081	0.0327	-0.0071	20.1206
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0894	-2.1494	-0.0031	-0.0125	-0.0026	-11.2555
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9507	-1.6780	0.0017	0.0062	-0.0088	-21.8371

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.0570
Shear X	-2.2957
Shear Z	0.0316
Moment X	0.1255
Moment Y (Twist)	0.0621
Moment Z	38.6858

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	9.6829
Shear X	-1.3750
Shear Z	0.0190
Moment X	0.0749
Moment Y (Twist)	0.0386
Moment Z	21.8371

Project Details

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

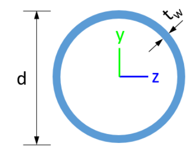
User Name: sales@mtsolar.us
Project Name: W12185 - REVISION CU-5
Unit System: imperial

Design Input Information

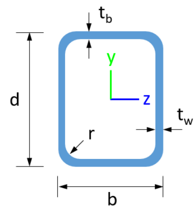
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

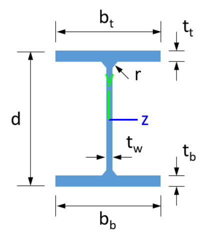
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
7	6in Pipe Sch 40	6.63	0.28					



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



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties

Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	26.30	26.30	12.52	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
4	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	300	200	1
8	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.11,2.11,2.11,2.33,2.11,2.11,2.12,2.33,2.12,2.12,2.12,2.12,2.11,2.11,2.12,2.35,2.11,2.11,2.12,2.29	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
11	19	1.33	1.33	2.05	2.11,2.11,2.11,2.11,2.11,2.12,2.12,2.26,2.31,2.12,2.12,2.12,2.23,2.12,2.12,2.11,2.10,2.12,2.12,2.20,2.30,2.12,2.12,2.12,2.21	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.08,1.12,1.09,1.09,1.09,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.08,1.12,1.09,1.09,1.09,1.10	300	200	1
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.15,1.09,1.09,1.09,1.36,1.09,1.09,1.09,1.10,1.09,1.09,1.10,1.15,1.09,1.09,1.09,1.59	300	200	1
15	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	7	26.30	26.30	12.52	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	30	20	1

				[illegible]	[illegible]	0	0	
108	19	1.33	1.33	2.05	2.06,2.06,2.06,2.06,2.06,2.07,2.07,2.07,2.04,2.07,2.07,2.06,1.84,2.06,2.06,2.06,2.06,2.07,2.07,2.07,2.02,2.07,2.07,2.06,1.74	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
111	19	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.06,2.06,1.34,1.74,2.05,2.05,1.92,1.89,2.06,2.06,2.08,2.13,2.06,2.06,1.50,1.77,2.04,2.04,1.97,1.92	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1
113	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.06,1.09,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.03,1.04,1.04,1.05,1.08,1.04,1.04,1.04,1.05	300	200	1
114	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.11,1.04,1.04,1.04,1.55,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.12,1.04,1.04,1.04,2.37	300	200	1
115	19	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.10,1.12,1.14,1.14,1.13,1.13,1.14,1.14,1.15,1.17,1.14,1.14,1.11,1.12,1.14,1.14,1.13,1.13	300	200	1
116	19	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.15,1.14,1.14,1.15,1.15,1.14,1.12,1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.13,1.15,1.15,1.14,1.12	300	200	1
201	7	26.30	26.30	12.52	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
204	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.17,1.18,1.18,1.17,1.18	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66	300	200	1
208	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.69	300	200	1
211	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.08,1.12,1.09,1.09,1.09,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.08,1.12,1.09,1.09,1.09,1.10	300	200	1
214	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.15,1.09,1.09,1.10,1.36,1.09,1.09,1.09,1.10,1.09,1.09,1.10,1.15,1.09,1.09,1.09,1.59	300	200	1
215	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.19,1.18,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.16,1.17,1.17,1.18,1.18,1.17,1.17,1.17,1.18	300	200	1
216	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.20,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.21	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	63.85	42.30	42.30	75.35	75.35
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	126.01	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.64	6.12	40.24	43.62
14	133.20	104.94	25.05	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62
101	251.16	63.85	42.30	42.30	75.35	75.35
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	104.94	23.67	6.12	40.24	43.62
114	133.20	104.94	23.79	6.12	40.24	43.62
115	133.20	69.16	16.97	6.12	40.24	43.62
116	133.20	69.16	17.37	6.12	40.24	43.62
201	251.16	63.85	42.30	42.30	75.35	75.35
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	32.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	32.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	104.94	24.63	6.12	40.24	43.62
214	133.20	104.94	25.05	6.12	40.24	43.62
215	133.20	69.16	17.97	6.12	40.24	43.62
216	133.20	69.16	18.09	6.12	40.24	43.62

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.236	0.915	0.006	0.030	0.000	0.955	#13	0.703	Not Required	Warn
2	0.003	0.595	0.113	0.125	0.020	0.686	#21	0.035	Not Required	Pass
3	0.007	0.880	0.036	0.088	0.003	0.920	#21	0.045	Not Required	Pass
4	0.007	0.838	0.118	0.084	0.024	0.913	#21	0.080	Not Required	Pass
5	0.007	0.546	0.124	0.087	0.031	0.577	#21	0.074	Not Required	Pass
6	0.007	0.870	0.041	0.087	0.004	0.914	#21	0.045	Not Required	Pass
7	0.007	0.541	0.129	0.086	0.033	0.576	#21	0.074	Not Required	Pass
8	0.001	0.063	0.114	0.058	0.012	0.164	#21	0.095	Not Required	Pass
9	0.011	0.094	0.038	0.001	0.000	0.132	#21	0.204	Not Required	Pass
10	0.007	0.828	0.127	0.083	0.027	0.915	#21	0.080	Not Required	Pass
11	0.000	0.066	0.116	0.061	0.012	0.169	#21	0.095	Not Required	Pass
12	0.003	0.583	0.112	0.124	0.019	0.671	#21	0.035	Not Required	Pass
13	0.005	0.349	0.295	0.078	0.016	0.625	#21	0.286	Not Required	Pass
14	0.005	0.341	0.295	0.075	0.016	0.611	#21	0.190	Not Required	Pass
15	0.000	0.135	0.157	0.046	0.009	0.292	#21	Not Required	Not Required	Pass
16	0.000	0.129	0.157	0.044	0.009	0.286	#21	Not Required	Not Required	Pass
101	0.248	0.935	0.000	0.031	0.000	0.977	#13	0.703	Not Required	Warn
102	0.002	0.613	0.114	0.131	0.020	0.703	#21	0.035	Not Required	Pass
103	0.007	0.919	0.045	0.092	0.007	0.968	#21	0.045	Not Required	Warn
104	0.007	0.876	0.116	0.088	0.025	0.958	#21	0.080	Not Required	Warn
105	0.007	0.571	0.121	0.091	0.030	0.602	#21	0.074	Not Required	Pass
106	0.007	0.919	0.045	0.092	0.007	0.968	#21	0.045	Not Required	Warn
107	0.007	0.571	0.121	0.091	0.030	0.602	#21	0.074	Not Required	Pass
108	0.001	0.076	0.106	0.056	0.012	0.146	#21	0.095	Not Required	Pass
109	0.009	0.088	0.035	0.001	0.000	0.127	#21	0.204	Not Required	Pass
110	0.007	0.876	0.116	0.088	0.025	0.958	#21	0.080	Not Required	Warn
111	0.000	0.080	0.108	0.058	0.012	0.149	#21	0.095	Not Required	Pass
112	0.002	0.613	0.114	0.131	0.020	0.703	#21	0.035	Not Required	Pass
113	0.005	0.281	0.279	0.075	0.015	0.534	#21	0.286	Not Required	Pass
114	0.005	0.276	0.277	0.072	0.015	0.522	#21	0.286	Not Required	Pass
115	0.001	0.316	0.155	0.058	0.012	0.472	#21	0.473	Not Required	Pass
116	0.001	0.305	0.156	0.056	0.012	0.460	#21	0.473	Not Required	Pass
201	0.236	0.915	0.006	0.030	0.000	0.955	#13	0.703	Not Required	Warn
202	0.003	0.583	0.112	0.124	0.019	0.671	#21	0.035	Not Required	Pass
203	0.007	0.870	0.041	0.087	0.004	0.914	#21	0.045	Not Required	Pass
204	0.007	0.828	0.127	0.083	0.027	0.915	#21	0.080	Not Required	Pass
205	0.007	0.541	0.129	0.086	0.033	0.576	#21	0.074	Not Required	Pass
206	0.007	0.880	0.036	0.088	0.003	0.920	#21	0.045	Not Required	Pass
207	0.007	0.546	0.124	0.087	0.031	0.577	#21	0.074	Not Required	Pass
208	0.000	0.129	0.157	0.044	0.009	0.286	#21	Not Required	Not Required	Pass
209	0.011	0.094	0.038	0.001	0.000	0.132	#21	0.204	Not Required	Pass
210	0.007	0.838	0.118	0.084	0.024	0.913	#21	0.080	Not Required	Pass
211	0.000	0.135	0.157	0.046	0.009	0.292	#21	Not Required	Not Required	Pass
212	0.003	0.595	0.113	0.125	0.020	0.686	#21	0.035	Not Required	Pass
213	0.005	0.349	0.295	0.078	0.016	0.625	#21	0.190	Not Required	Pass
214	0.005	0.341	0.295	0.075	0.016	0.611	#21	0.286	Not Required	Pass
215	0.001	0.312	0.155	0.061	0.012	0.466	#21	0.473	Not Required	Pass
216	0.001	0.299	0.156	0.058	0.012	0.455	#21	0.473	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

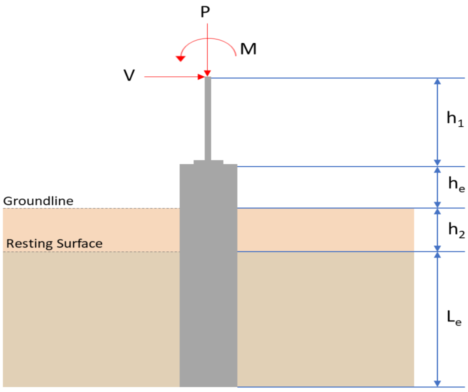
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 5.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sandy gravel and/or gravel	3000.000	200.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.683	15.057
V_x (kip)	-1.375	-2.296
V_z (kip)	-0.019	-0.032
M_x (kipft)	-0.075	-0.125
M_z (kipft)	21.837	38.685

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.375\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.21895\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.837\ \text{kipft}) + ((-1.375\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.4772\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.379\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.019\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.0030255\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075\ \text{kipft}) + ((-0.019\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.011943\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 0.84405\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.379\ \text{ft}), (0.84405\ \text{ft})]$$

$$L_{e,req} = 5.379\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 5.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.379\ \text{ft})}{(5.75\ \text{ft})}$$

$$Ratio = 0.93548$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.683\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.60519\ \text{kip/ft}^2$$

$q = 0.60519\ \mathrm{kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.60519\ \mathrm{kip/ft}^2)}{(3000\ \mathrm{psf})}$$

$$Ratio = 0.20173$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.21895\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.4772\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4772\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (-0.21895\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (3.4772\ \mathrm{kipft/ft})) + (4 \times (-0.21895\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 3.9265\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.4772\ \mathrm{kipft/ft})) + (3 \times (-0.21895\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]^{\ 2}}{(5.75\ \mathrm{ft})^2 \times [(3 \times (3.4772\ \mathrm{kipft/ft})) + (2 \times (-0.21895\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}$$

$$p = 0.29426\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.4772\ \mathrm{kipft/ft})) + ((-0.21895\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}{(5.75\ \mathrm{ft})^2}$$

$$s = 1.0336\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200\ \mathrm{psf/ft}) \times \frac{(3.9265\ \mathrm{ft})}{2}$$

$$p_a = 0.39265\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.29426\ \mathrm{kip/ft}^2)}{(0.39265\ \mathrm{kip/ft}^2)}$$

$$Ratio = 0.74943$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (200\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 1.15\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.0336\ \mathrm{kip/ft^2}\right)}{\left(1.15\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.89877$$

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

Considering z-direction:

$H_o = -0.0030255\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.011943\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{\left(4 \times (0.011943\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})\right) + \left(3 \times (-0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{\left(6 \times (0.011943\ \mathrm{kipft/ft})\right) + \left(4 \times (-0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})\right)}$$

$$a = 4.0694\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times \left[\left(4 \times (0.011943\ \mathrm{kipft/ft})\right) + \left(3 \times (-0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})\right)\right]^{\,2}}{(5.75\ \mathrm{ft})^2 \times \left[\left(3 \times (0.011943\ \mathrm{kipft/ft})\right) + \left(2 \times (-0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})\right)\right]}$$

$$p = 0.00042794\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times \left[\left(2 \times (0.011943\ \mathrm{kipft/ft})\right) + \left((-0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})\right)\right]}{(5.75\ \mathrm{ft})^2}$$

$$s = 0.0011776\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200\ \mathrm{psf/ft}) \times \frac{(4.0694\ \mathrm{ft})}{2}$$

$$p_a = 0.40694\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.00042794\ \mathrm{kip/ft^2}\right)}{\left(0.40694\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0010516$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (200\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 1.15\ \mathrm{kip/ft^2}$$

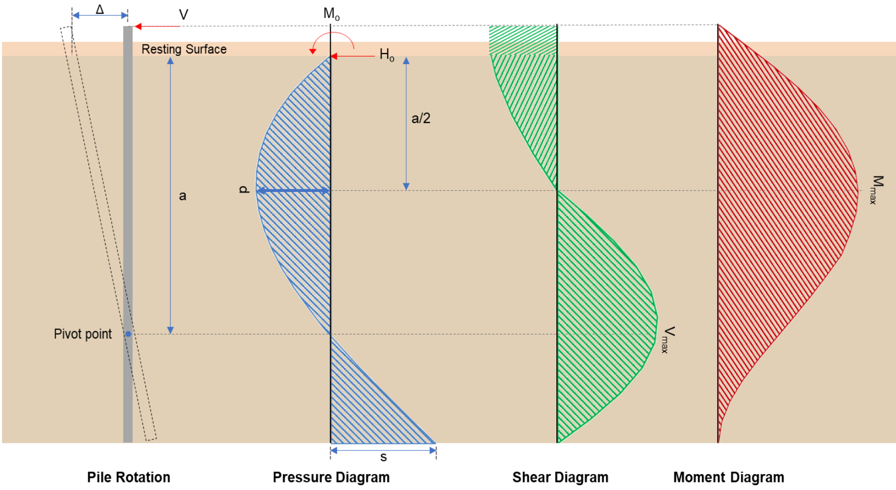
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0011776\ \mathrm{kip/ft^2})}{(1.15\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.001024$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.296\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.36561\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(38.685\ \mathrm{kipft}) + ((-2.296\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.16\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.16\ \mathrm{kipft/ft})}{(-0.36561\ \mathrm{kip/ft})}$$

$$E = 16.849\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.16\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (-0.36561\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (6.16\ \mathrm{kipft/ft})) + (4 \times (-0.36561\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 3.9221\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.36561\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.849\ \mathrm{ft})}{(5.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(3.9221\ \mathrm{ft})}{(5.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.849\ \mathrm{ft})}{(5.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(3.9221\ \mathrm{ft})}{(5.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.5542\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.36561\text{ kip/ft})\times(48\text{ in})\times(5.75\text{ ft}))\times\left[\left(\frac{(16.849\text{ ft})}{(5.75\text{ ft})}+\frac{(3.9221\text{ ft})}{2\times(5.75\text{ ft})}\right) - \left[\left(\frac{4\times(16.849\text{ ft})}{(5.75\text{ ft})}+3\right)\times\left(\frac{(3.9221\text{ ft})}{2\times(5.75\text{ ft})}\right)^3\right]+\left[\left(\frac{3\times(16.849\text{ ft})}{(5.75\text{ ft})}+2\right)\times\left(\frac{(3.9221\text{ ft})}{2\times(5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 23.825\text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.032\text{ kip})}{1.57\times(48\text{ in})}$$

$$H_o = -0.0050955\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.125\text{ kipft})+((-0.032\text{ kip})\times(0\text{ ft}))}{1.57\times(48\text{ in})}$$

$$M_o = 0.019904\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.019904\text{ kipft/ft})}{(-0.0050955\text{ kip/ft})}$$

$$E = 3.9062\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4\times(0.019904\text{ kipft/ft})\times(5.75\text{ ft}))+\left(3\times(-0.0050955\text{ kip/ft})\times(5.75\text{ ft})^2\right)}{(6\times(0.019904\text{ kipft/ft}))+(4\times(-0.0050955\text{ kip/ft})\times(5.75\text{ ft}))}$$

$$a = 4.0707\text{ ft}$$

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

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

$$V_{max} = ((-0.0050955\text{ kip/ft})\times(48\text{ in}))\times\left[1-\left[3\times\left(\frac{4\times(3.9062\text{ ft})}{(5.75\text{ ft})}+3\right)\times\left(\frac{(4.0707\text{ ft})}{(5.75\text{ ft})}\right)^2\right]+\left[4\times\left(\frac{3\times(3.9062\text{ ft})}{(5.75\text{ ft})}+2\right)\times\left(\frac{(4.0707\text{ ft})}{(5.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.038022\text{ kip}$$

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

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

$$M_{max} = ((-0.0050955\text{ kip/ft})\times(48\text{ in})\times(5.75\text{ ft}))\times\left[\left(\frac{(3.9062\text{ ft})}{(5.75\text{ ft})}+\frac{(4.0707\text{ ft})}{2\times(5.75\text{ ft})}\right) - \left[\left(\frac{4\times(3.9062\text{ ft})}{(5.75\text{ ft})}+3\right)\times\left(\frac{(4.0707\text{ ft})}{2\times(5.75\text{ ft})}\right)^3\right]+\left[\left(\frac{3\times(3.9062\text{ ft})}{(5.75\text{ ft})}+2\right)\times\left(\frac{(4.0707\text{ ft})}{2\times(5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.098814 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(15.057 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.096 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.096 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

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

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

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(15.057\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0056284$$	Status: **PASS** Ratio: **0.010**

	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\text{ in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,max} = 296.21\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 15.057\text{ kip} \rightarrow 15057\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(15057\text{ lbf})}{6 \times (2304\text{ in}^2)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.49\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.49\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.49\text{ kip}$$	

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.49\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.4\ \text{kip}$$

Considering x-direction:

$V_{max} = 8.5542\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(8.5542\ \text{kip})}{(111.4\ \text{kip})}$$

$$Ratio = 0.076787$$

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

Considering z-direction:

$V_{max} = 0.038022\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.038022\ \text{kip})}{(111.4\ \text{kip})}$$

$$Ratio = 0.0003413$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 23.825 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(23.825 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.095452$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $M_{max} = 0.098814 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.098814 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.00039589$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

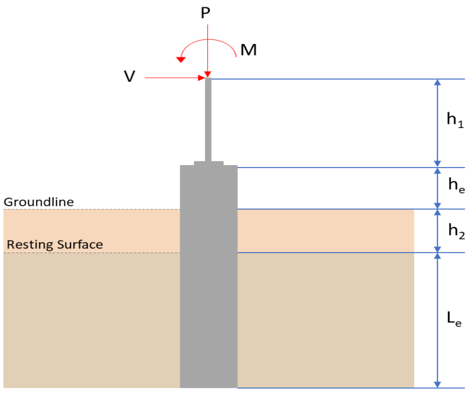
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 5.75\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sandy gravel and/or gravel	3000.000	200.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	10.163	15.822
V_x (kip)	-1.405	-2.347
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	22.251	39.537

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.405\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.22373\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

	$$M_o = \frac{(22.251\ \text{kipft}) + ((-1.405\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$ $$M_o = 3.5432\ \text{kipft/ft}$$ Required depth of embedment in earth: $$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$ Solving the cubic equation: $L_{e,x} = 5.4078\ \text{ft}$ - Required depth in x-direction, **Considering z-direction:** $L_{e,z} = 0\ \text{ft}$ - Required depth in z-direction, **Minimum embedded depth required:** $L_{e,req}$ - Depth of pile required, $$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$ $$L_{e,req} = MAX[(5.4078\ \text{ft}), (0\ \text{ft})]$$ $$L_{e,req} = 5.408\ \text{ft}$$ L_e - Actual embedded length of pile, $$L_e = L - h_e - h_2$$ $$L_e = (5.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$ $$L_e = 5.75\ \text{ft}$$ $Ratio$ - Embedded depth $$Ratio = \frac{L_{e,req}}{L_e}$$ $$Ratio = \frac{(5.408\ \text{ft})}{(5.75\ \text{ft})}$$ $$Ratio = 0.94052$$	Status: PASS Ratio: 0.940
	End-bearing Capacity (ASD) A - Pile cross-section area $$A = b\,D$$ $$A = (48\ \text{in}) \times (48\ \text{in})$$ $$A = 16\ \text{ft}^2$$ q - End-bearing pressure $$q = \frac{P_v}{A}$$ $$q = \frac{(10.163\ \text{kip})}{(16\ \text{ft}^2)}$$ $$q = 0.63519\ \text{kip/ft}^2$$ **Check bearing capacity ratio:** $Ratio$ - Capacity $$Ratio = \frac{q}{q_a}$$ $$Ratio = \frac{(0.63519\ \text{kip/ft}^2)}{(3000\ \text{psf})}$$ $$Ratio = 0.21173$$	Status: PASS Ratio: 0.210
Czerniak	**Lateral Soil Pressure (ASD):** L/D - Length to least lateral dimension ratio, $$L/D = \frac{L}{D}$$ $$L/D = \frac{(5.75\ \text{ft})}{(48\ \text{in})}$$	

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.22373\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.5432\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.5432\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.22373\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (3.5432\ \text{kipft/ft})) + (4 \times (-0.22373\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9267\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.5432\ \text{kipft/ft})) + (3 \times (-0.22373\ \text{kip/ft}) \times (5.75\ \text{ft}))]^2}{(5.75\ \text{ft})^2 \times [(3 \times (3.5432\ \text{kipft/ft})) + (2 \times (-0.22373\ \text{kip/ft}) \times (5.75\ \text{ft}))]}$$

$$p = 0.29948\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.5432\ \text{kipft/ft})) + ((-0.22373\ \text{kip/ft}) \times (5.75\ \text{ft}))]}{(5.75\ \text{ft})^2}$$

$$s = 1.0525\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200\ \text{psf/ft}) \times \frac{(3.9267\ \text{ft})}{2}$$

$$p_a = 0.39267\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.29948\ \text{kip/ft}^2\right)}{\left(0.39267\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.76268$$

Status: **PASS**
Ratio: **0.760**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (200\ \text{psf/ft}) \times (5.75\ \text{ft})$$

$$p_s = 1.15\ \text{kip/ft}^2$$

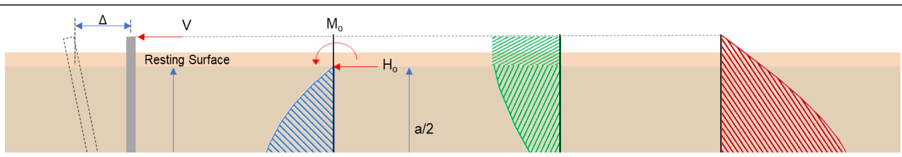
$Ratio$ - Lateral soil capacity

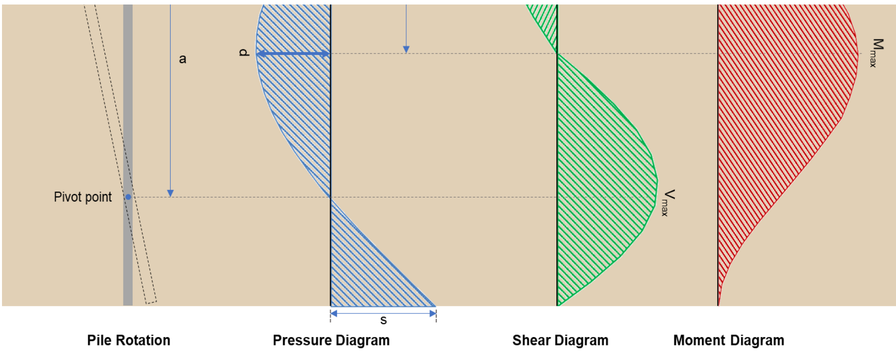
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.0525\ \text{kip/ft}^2\right)}{\left(1.15\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.91524$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.347\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.37373\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.537\text{ kipft}) + ((-2.347\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 6.2957\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.2957\text{ kipft/ft})}{(-0.37373\text{ kip/ft})}$$

$$E = 16.846\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2957\text{ kipft/ft}) \times (5.75\text{ ft})) + \left(3 \times (-0.37373\text{ kip/ft}) \times (5.75\text{ ft})^2\right)}{(6 \times (6.2957\text{ kipft/ft})) + (4 \times (-0.37373\text{ kip/ft}) \times (5.75\text{ ft}))}$$

$$a = 3.9222\text{ ft}$$

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

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

$$V_{max} = ((-0.37373\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.846\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(3.9222\text{ ft})}{(5.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.846\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(3.9222\text{ ft})}{(5.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 8.7427\text{ kip}$$

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

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

$$M_{max} = ((-0.37373\text{ kip/ft}) \times (48\text{ in}) \times (5.75\text{ ft})) \times \left[\left(\frac{(16.846\text{ ft})}{(5.75\text{ ft})} + \frac{(3.9222\text{ ft})}{2 \times (5.75\text{ ft})}\right) - \left[\left(\frac{4 \times (16.846\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(3.9222\text{ ft})}{2 \times (5.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.846\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(3.9222\text{ ft})}{2 \times (5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 24.35 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(15.822 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.07 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.07 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

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

	Ties: **#3(0.375 in) - 10 in**	
	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$	
	$Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(15.822\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0059144$$	Status: **PASS** Ratio: **0.010**
	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\text{ in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,max} = 296.21\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 15.822\text{ kip} \rightarrow 15822\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(15822\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 120.59\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$	
	V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (120.59\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 120.59\text{ kip}$$	

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) $$V_{s,a} = 8 \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$ $$V_{s,a} = 737.28 \text{ kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\, d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$ $$A_v = 0.11045 \text{ in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$ $$V_{s,b} = 50.894 \text{ kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$ $$V_s = 50.894 \text{ kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.59 \text{ kip}) + (50.894 \text{ kip}))$$ $$\phi V_n = 111.47 \text{ kip}$$ **Considering x-direction:** $V_{max} = 8.7427 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(8.7427 \text{ kip})}{(111.47 \text{ kip})}$$ $$Ratio = 0.078433$$	Status: **PASS** Ratio: **0.080**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$ $$S_m = 18432 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_c\, S_m$$	

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 24.35 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(24.35 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.097556$$

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

REFERENCES	CALCULATIONS	RESULTS

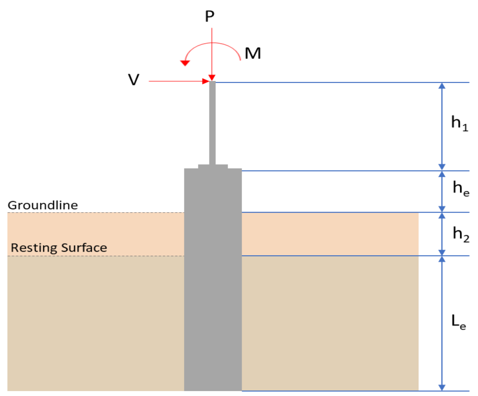
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 5.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sandy gravel and/or gravel	3000.000	200.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.683	15.057
V_x (kip)	-1.375	-2.296
V_z (kip)	0.019	0.032
M_x (kipft)	0.075	0.125
M_z (kipft)	21.837	38.686

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.375\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.21895\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.837 \text{ kipft}) + ((-1.375 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4772 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.379 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.019 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0030255 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075 \text{ kipft}) + ((0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011943 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 0.94553 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.379 \text{ ft}), (0.94553 \text{ ft})]$$

$$L_{e,req} = 5.379 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.379 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.93548$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.683 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.60519 \text{ kip/ft}^2$$

$q = 0.60519\ \text{kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.60519\ \text{kip/ft}^2)}{(3000\ \text{psf})}$$

$$Ratio = 0.20173$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.21895\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.4772\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4772\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.21895\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (3.4772\ \text{kipft/ft})) + (4 \times (-0.21895\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9265\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.4772\ \text{kipft/ft})) + (3 \times (-0.21895\ \text{kip/ft}) \times (5.75\ \text{ft}))]^2}{(5.75\ \text{ft})^2 \times [(3 \times (3.4772\ \text{kipft/ft})) + (2 \times (-0.21895\ \text{kip/ft}) \times (5.75\ \text{ft}))]}$$

$$p = 0.29426\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.4772\ \text{kipft/ft})) + ((-0.21895\ \text{kip/ft}) \times (5.75\ \text{ft}))]}{(5.75\ \text{ft})^2}$$

$$s = 1.0336\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200\ \text{psf/ft}) \times \frac{(3.9265\ \text{ft})}{2}$$

$$p_a = 0.39265\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.29426\ \text{kip/ft}^2)}{(0.39265\ \text{kip/ft}^2)}$$

$$Ratio = 0.74943$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (200\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 1.15\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(1.0336\ \mathrm{kip/ft^2})}{(1.15\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.89877$$

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

Considering z-direction:

$H_o = 0.0030255\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.011943\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011943\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (0.011943\ \mathrm{kipft/ft})) + (4 \times (0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 4.0694\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.011943\ \mathrm{kipft/ft})) + (3 \times (0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]^2}{(5.75\ \mathrm{ft})^2 \times [(3 \times (0.011943\ \mathrm{kipft/ft})) + (2 \times (0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}$$

$$p = 0.0032096\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.011943\ \mathrm{kipft/ft})) + ((0.0030255\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}{(5.75\ \mathrm{ft})^2}$$

$$s = 0.0074916\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200\ \mathrm{psf/ft}) \times \frac{(4.0694\ \mathrm{ft})}{2}$$

$$p_a = 0.40694\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0032096\ \mathrm{kip/ft^2})}{(0.40694\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.0078871$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (200\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 1.15\ \mathrm{kip/ft^2}$$

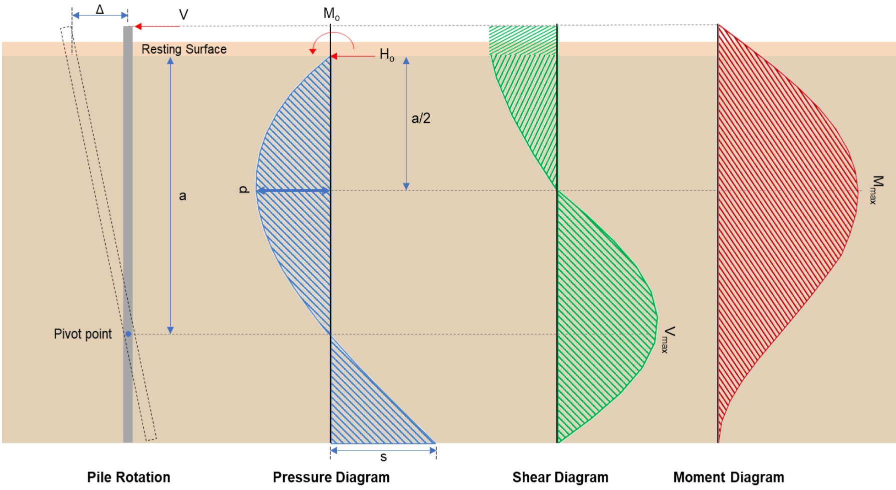
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0074916\ \text{kip/ft}^2)}{(1.15\ \text{kip/ft}^2)}$$

$$Ratio = 0.0065144$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.296\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.36561\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(38.686\ \text{kipft}) + ((-2.296\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 6.1602\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.1602\ \text{kipft/ft})}{(-0.36561\ \text{kip/ft})}$$

$$E = 16.849\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1602\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.36561\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (6.1602\ \text{kipft/ft})) + (4 \times (-0.36561\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9221\ \text{ft}$$

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

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

$$V_{max} = ((-0.36561\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.849\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9221\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.849\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9221\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.5544\ \text{kip}$$

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

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

$$M_{max} = ((-0.36561\ \text{kip/ft}) \times (48\ \text{in}) \times (5.75\ \text{ft})) \times \left[\left(\frac{(16.849\ \text{ft})}{(5.75\ \text{ft})} + \frac{(3.9221\ \text{ft})}{2 \times (5.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (16.849\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9221\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.849\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9221\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 23.825\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.032\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.0050955\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.125\ \text{kipft}) + ((0.032\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.019904\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.019904\ \text{kipft/ft})}{(0.0050955\ \text{kip/ft})}$$

$$E = 3.9062\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019904\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (0.0050955\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (0.019904\ \text{kipft/ft})) + (4 \times (0.0050955\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 4.0707\ \text{ft}$$

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

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

$$V_{max} = ((0.0050955\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.9062\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(4.0707\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.9062\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(4.0707\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.038022\ \text{kip}$$

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

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

$$M_{max} = ((0.0050955\ \text{kip/ft}) \times (48\ \text{in}) \times (5.75\ \text{ft})) \times \left[\left(\frac{(3.9062\ \text{ft})}{(5.75\ \text{ft})} + \frac{(4.0707\ \text{ft})}{2 \times (5.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.9062\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(4.0707\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.9062\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(4.0707\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.098814\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5\ \text{ksi}$ - Concrete strength,
$f_{yk} = 60\ \text{ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304\ \text{in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(15.057\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.096\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.096\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

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

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

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(2304\ \text{in}^2\right) - \left(4.2951\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(4.2951\ \text{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \text{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(15.057\ \text{kip})}{(2675.2\ \text{kip})}$$ $$Ratio = 0.0056284$$	Status: **PASS** Ratio: **0.010**
	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\ \text{in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\ \text{in})$$ $$d = 38.4\ \text{in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{c,max} = 296.21\ \text{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 15.057\ \text{kip} \rightarrow 15057\ \text{lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(15057\ \text{lbf})}{6 \times \left(2304\ \text{in}^2\right)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{c,a} = 120.49\ \text{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + (0.05 \times (2500\ \text{psi}))\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{c,b} = 348.89\ \text{kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\ \text{kip}), (120.49\ \text{kip}), (348.89\ \text{kip})\right]$$ $$V_c = 120.49\ \text{kip}$$	

	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\ b_w\ d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{s,a} = 737.28\ \text{kip}$$ A_v - Ties rebar area, $$A_v = \frac{\pi\ d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \text{in})^2}{4}$$ $$A_v = 0.11045\ \text{in}^2$$	
22.5.8.5.3	$V_{s,b}$ - Shear strength of steel (b) $$V_{s,b} = \frac{2\ A_v\ f_{ywk}\ d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$ $$V_{s,b} = 50.894\ \text{kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$ $$V_s = 50.894\ \text{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.49\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 111.4\ \text{kip}$$ **Considering x-direction:** $V_{max} = 8.5544\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(8.5544\ \text{kip})}{(111.4\ \text{kip})}$$ $$Ratio = 0.076789$$	Status: **PASS** Ratio: **0.080**
	Considering z-direction: $V_{max} = 0.038022\ \text{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.038022\ \text{kip})}{(111.4\ \text{kip})}$$ $$Ratio = 0.0003413$$	Status: **PASS** Ratio: **0.000**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\ D^2}{6}$$ $$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$ $$S_m = 18432\ \text{in}^3$$	

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(18432\ \text{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \text{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\ \text{kipft}), (2121.6\ \text{kipft})]$$ $$\phi M_n = 249.6\ \text{kipft}$$ **Considering x-direction:** $M_{max} = 23.825\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(23.825\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.095455$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $M_{max} = 0.098814\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.098814\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.00039589$$	Status: **PASS** Ratio: **0.000**