

Your Project Calculations



Project Name: Walz

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Walz&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=VtDMsFWenC20lkKkA4WODMIXYBQjseZ6llkym7JLhYazE5GyXcxeRESAatwOZz0z

Array Specification

Product:	Beam
Unique ID:	2P-17-6TOP-SD-45-L-3Hx5W-96FK
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	79.00in
Number of Rows:	3
Number of Columns:	5
Total Number of Modules:	15
Desired Tilt Angle:	65
Front Edge Clearance:	3
Total Array Height at Tilt:	12.12 ft
Total Frame Length:	32.00 ft
Frame Weight:	1168 lbs
Array Dimensions N/S:	10.13 ft
Array Dimensions E/W:	33.33 ft
Rail Length:	121.50 in
Rail Spacing:	3.29 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	7.59 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.25 ft
Foundation Volume:	6.222 y ³
Foundation Result:	PASSED
Mount Twist:	0.209366 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Rowley Rd, Caledonia, WI 53901, USA
Wind Speed:	100 mph
Snow Load:	30 psf
Design Uplift Pressure:	0.023463 ksf
Design Downforce Pressure:	-0.023463 ksf
Design Snow Pressure:	0.001649 ksf



Design Disclaimer

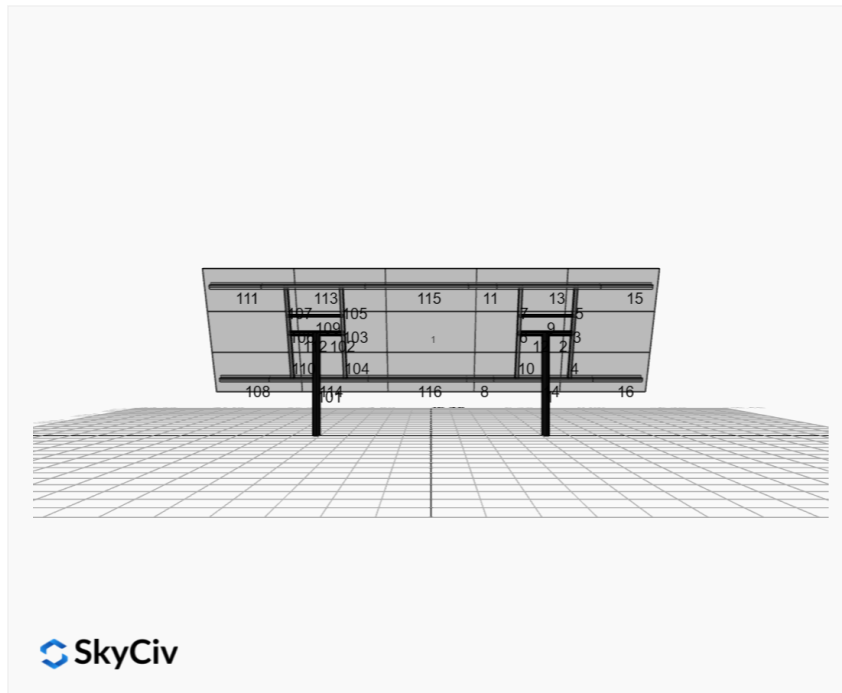
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

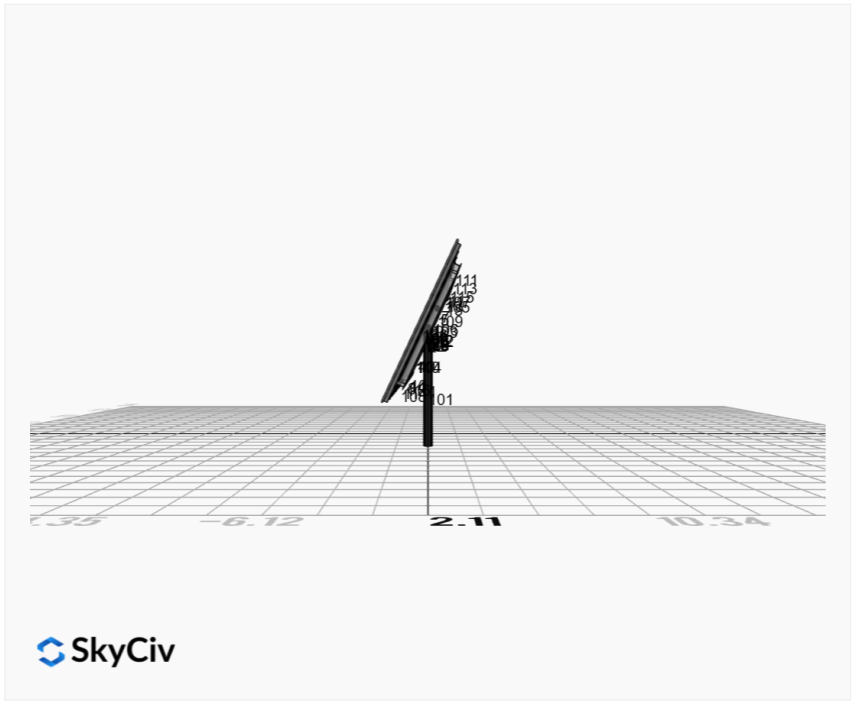
AutoDesigner Input

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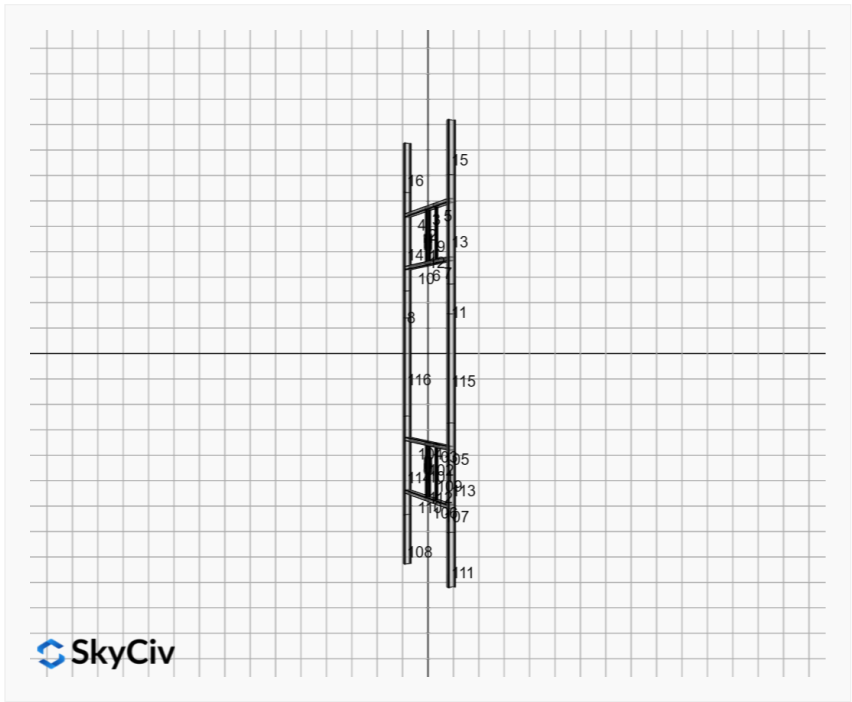
Design Notes:

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

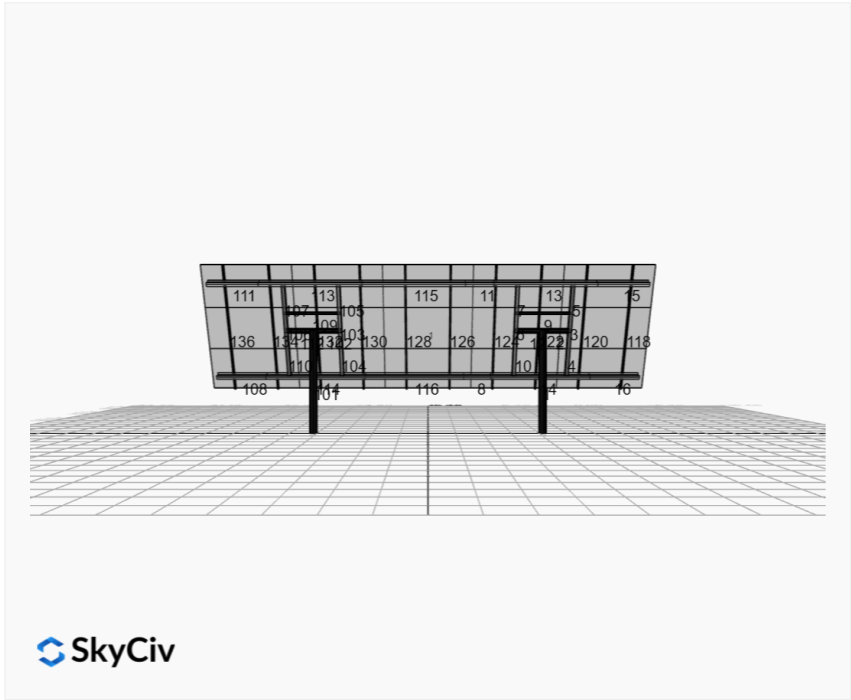
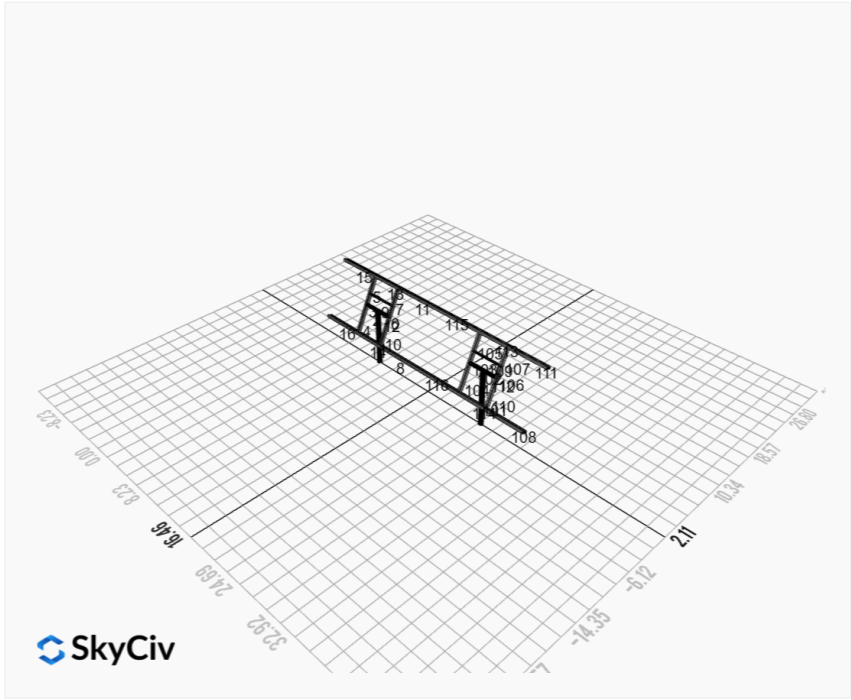




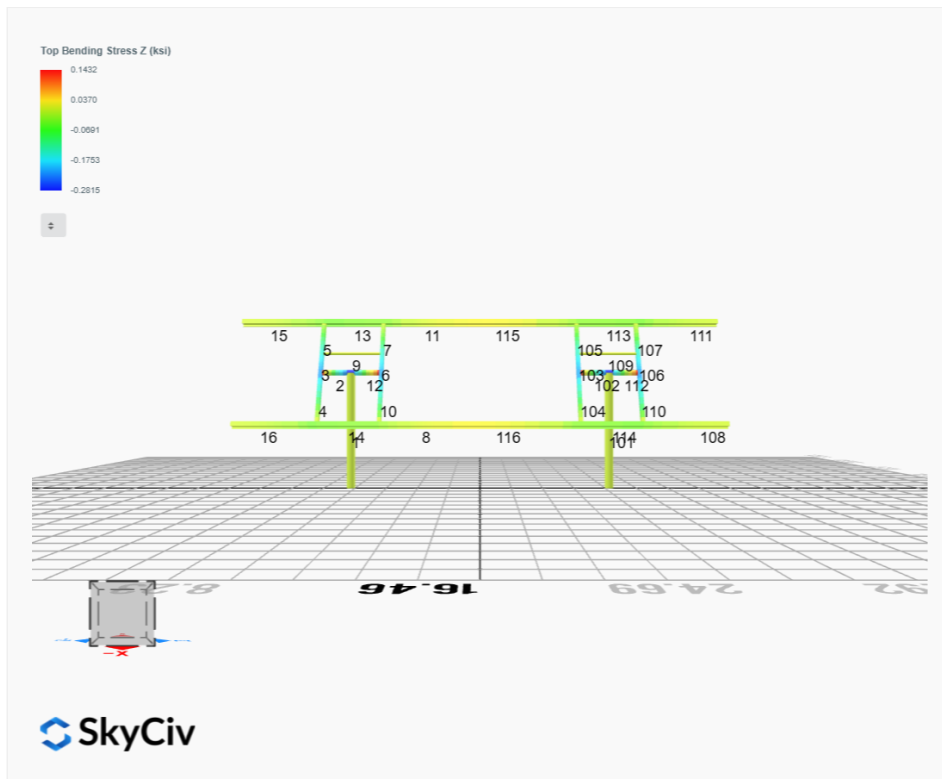
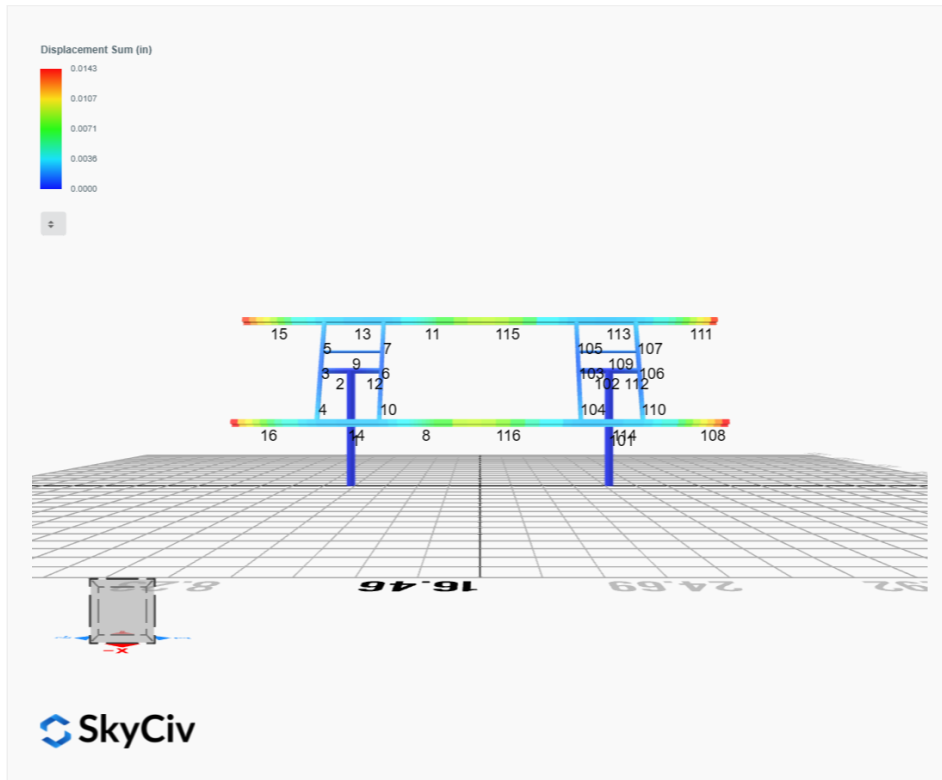
 SkyCiv

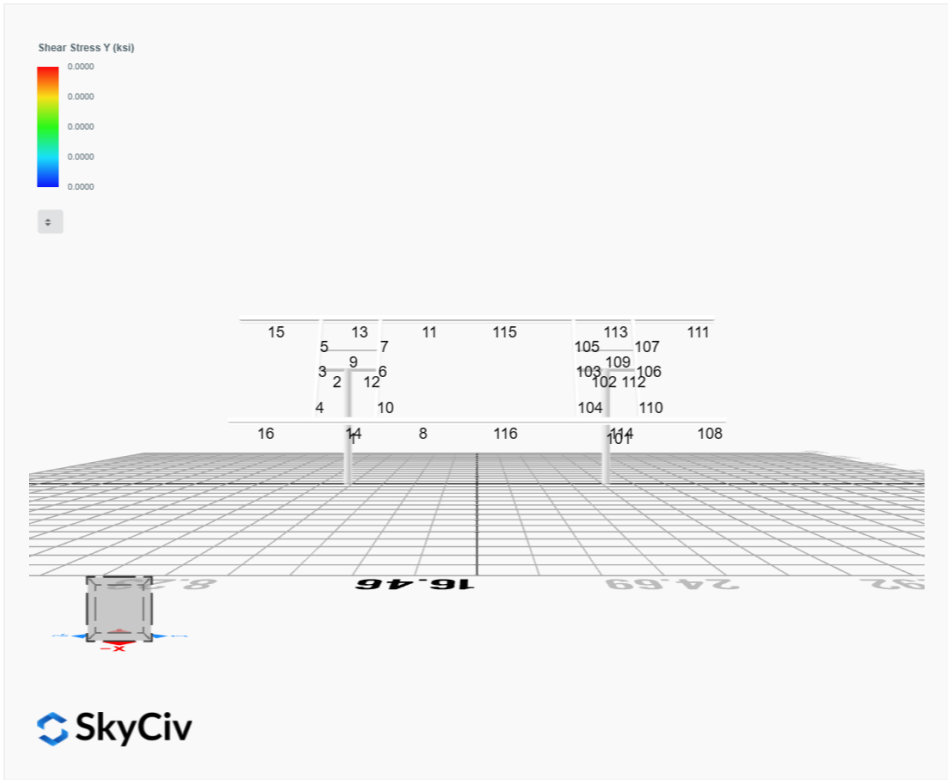
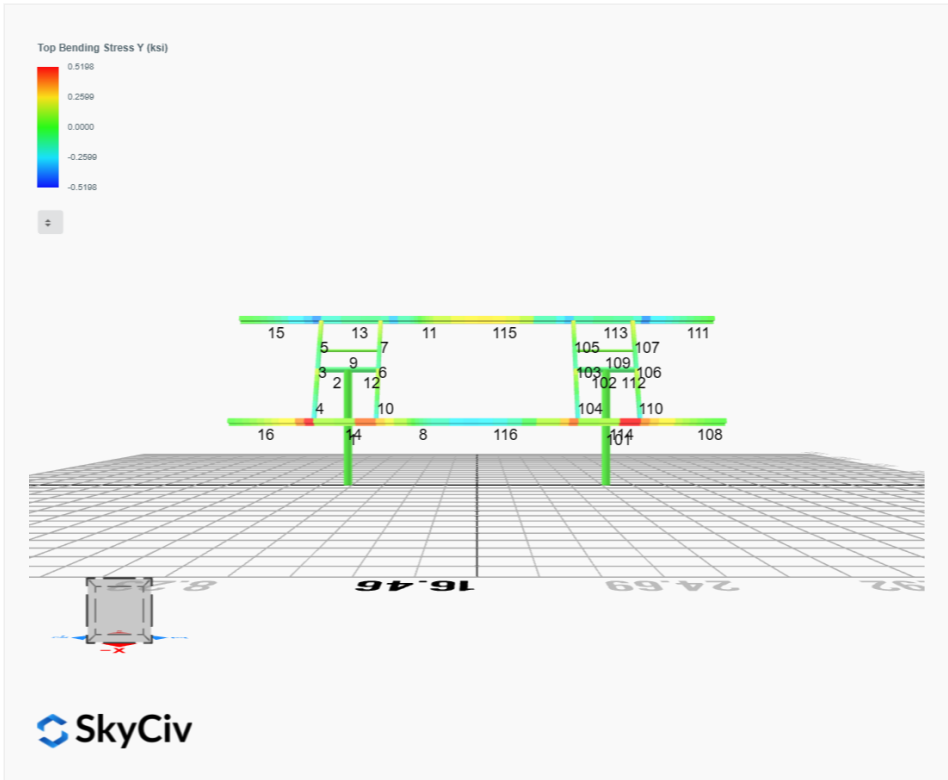


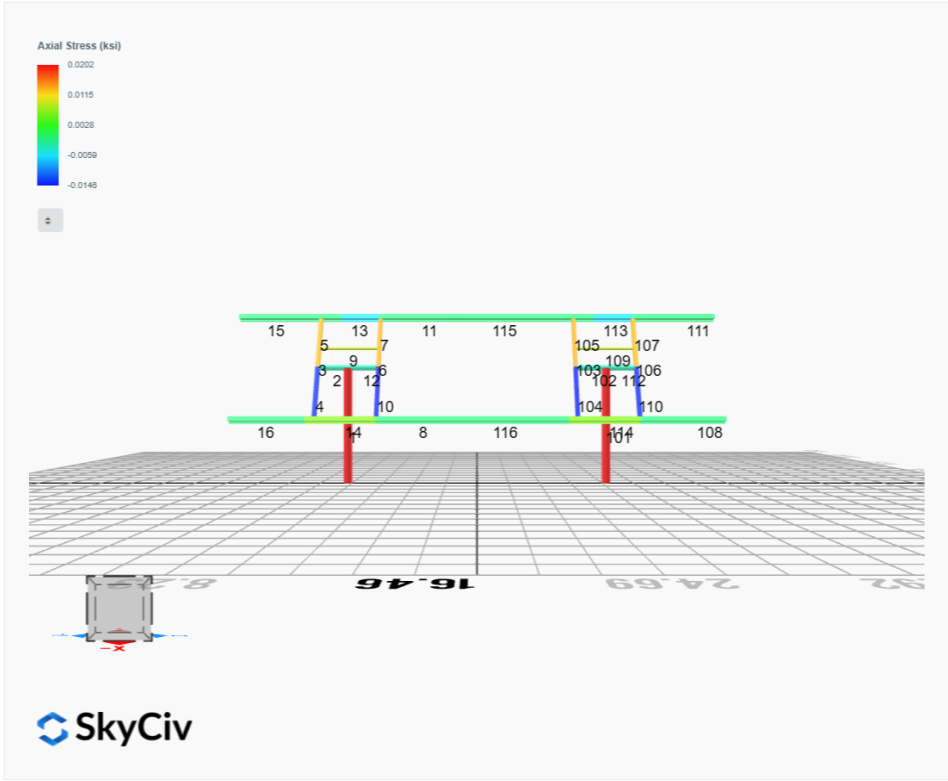
 SkyCiv



FEM Results (Envelope Worst Case for each member)







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	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 2. D + L	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 3. D + (S or Lr or R)	0.0000	1.5220	-0.0004	0.0005	0.0296	0.0099
ULS: 3. D + (S or Lr or R)	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4938	-0.0004	0.0004	0.0289	0.0099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 5b. D + 0.7E	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.4938	-0.0004	0.0004	0.0289	0.0099
ULS: 8. 0.6D + 0.7E	0.0000	0.8455	-0.0002	0.0002	0.0161	0.0059
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1531	2.4131	-0.0192	-0.0355	0.1322	16.4411
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1531	0.4051	0.0184	0.0363	-0.0785	-16.2362
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6148	2.2468	-0.0145	-0.0265	0.1079	12.3333
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.4938	-0.0004	0.0004	0.0289	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6148	0.7408	0.0137	0.0274	-0.0502	-12.1746
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.4938	-0.0004	0.0004	0.0289	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6148	2.1621	-0.0145	-0.0266	0.1059	12.3333
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6148	0.6561	0.0137	0.0273	-0.0522	-12.1746
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.4091	-0.0004	0.0004	0.0269	0.0099
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1531	1.8495	-0.0190	-0.0357	0.1215	16.4372
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.8455	-0.0002	0.0002	0.0161	0.0059
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1531	-0.1585	0.0186	0.0361	-0.0893	-16.2401
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.8455	-0.0002	0.0002	0.0161	0.0059

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	3.4207
Shear X	-3.5885
Shear Z	-0.0318
Moment X	0.0604
Moment Y (Twist)	0.2092
Moment Z	27.5829

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	2.4131
Shear X	-2.1531
Shear Z	-0.0192
Moment X	0.0363
Moment Y (Twist)	0.1322
Moment Z	16.4411

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	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 2. D + L	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 3. D + (S or Lr or R)	-0.0000	1.5220	0.0004	-0.0005	-0.0296	0.0099
ULS: 3. D + (S or Lr or R)	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.4938	0.0004	-0.0004	-0.0289	0.0099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 5b. D + 0.7E	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.4938	0.0004	-0.0004	-0.0289	0.0099
ULS: 8. 0.6D + 0.7E	-0.0000	0.8455	0.0002	-0.0002	-0.0161	0.0059
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1531	2.4131	0.0192	0.0355	-0.1322	16.4411
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1531	0.4051	-0.0184	-0.0363	0.0785	-16.2361
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6148	2.2468	0.0145	0.0265	-0.1079	12.3333
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.4938	0.0004	-0.0004	-0.0289	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6148	0.7408	-0.0137	-0.0274	0.0502	-12.1746
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.4938	0.0004	-0.0004	-0.0289	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6148	2.1621	0.0145	0.0266	-0.1059	12.3333
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6148	0.6561	-0.0137	-0.0273	0.0522	-12.1746
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.4091	0.0004	-0.0004	-0.0269	0.0099
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1531	1.8495	0.0190	0.0357	-0.1215	16.4372
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.8455	0.0002	-0.0002	-0.0161	0.0059
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1531	-0.1585	-0.0186	-0.0361	0.0893	-16.2401
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.8455	0.0002	-0.0002	-0.0161	0.0059

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	3.4207
Shear X	-3.5885
Shear Z	0.0318
Moment X	-0.0604
Moment Y (Twist)	0.2094
Moment Z	27.5835

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	2.4131
Shear X	-2.1531
Shear Z	0.0192
Moment X	-0.0363
Moment Y (Twist)	0.1322
Moment Z	16.4411

Project Details

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



Design Input Information

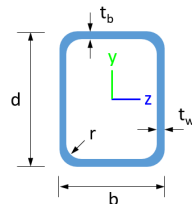
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties												
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L	S	T	L	S	L	D
1	7	15.94	15.94	7.59	-	3	0	0	2	0	0	1
2	4	1.30	1.30	2.00	-	3	0	0	2	0	0	1
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19	3	0	0	2	0	0	1
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	3	0	0	2	0	0	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19	3	0	0	2	0	0	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1
8	18	1.33	1.33	2.05	2.03,2.03,2.03,2.03,2.03,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03	3	0	0	2	0	0	1
9	1	2.60	2.60	4.00	-	3	0	0	2	0	0	1
10	15	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	3	0	0	2	0	0	1
11	18	1.33	1.33	2.05	2.03,2.03,2.03,2.03,2.03,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03,2.02,2.03	3	0	0	2	0	0	1
12	4	1.30	1.30	2.00	-	3	0	0	2	0	0	1
13	18	4.88	4.00	7.50	1.12,1.12	3	0	0	2	0	0	1
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15	18	7.88	7.88	3.75	2.33,2.33	3	0	0	2	0	0	1
16	18	7.88	7.88	3.75	2.33,2.33	3	0	0	2	0	0	1
101	7	15.94	15.94	7.59	-	3	0	0	2	0	0	1
102	4	1.30	1.30	2.00	-	3	0	0	2	0	0	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19	3	0	0	2	0	0	1
104	15	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	3	0	0	2	0	0	1
105	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1
106	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19	3	0	0	2	0	0	1
107	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1

113	120.00	89.27	20.50	6.45	30.09	45.74
116	120.60	89.27	20.50	6.45	30.09	45.74

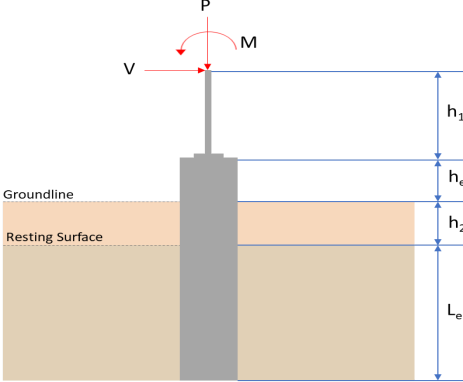
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.023	0.652	0.004	0.048	0.000	0.665	#13	0.426	Not Required	Pass
2	0.001	0.164	0.218	0.038	0.043	0.382	#13	0.034	Not Required	Pass
3	0.005	0.401	0.057	0.040	0.008	0.456	#13	0.044	Not Required	Pass
4	0.005	0.400	0.096	0.040	0.012	0.458	#13	0.078	Not Required	Pass
5	0.005	0.249	0.098	0.040	0.014	0.267	#13	0.073	Not Required	Pass
6	0.005	0.385	0.059	0.039	0.008	0.439	#13	0.044	Not Required	Pass
7	0.005	0.239	0.094	0.039	0.014	0.259	#13	0.073	Not Required	Pass
8	0.000	0.032	0.029	0.022	0.005	0.059	#13	0.088	Not Required	Pass
9	0.002	0.019	0.043	0.001	0.000	0.063	#13	0.198	Not Required	Pass
10	0.005	0.384	0.093	0.039	0.012	0.457	#13	0.078	Not Required	Pass
11	0.000	0.032	0.029	0.022	0.005	0.058	#13	0.088	Not Required	Pass
12	0.001	0.152	0.205	0.037	0.041	0.358	#13	0.034	Not Required	Pass
13	0.002	0.115	0.104	0.031	0.006	0.202	#13	0.265	Not Required	Pass
14	0.002	0.118	0.104	0.031	0.006	0.202	#13	0.177	Not Required	Pass
15	0.000	0.043	0.048	0.018	0.004	0.086	#13	Not Required	Not Required	Pass
16	0.000	0.043	0.048	0.018	0.004	0.086	#13	Not Required	Not Required	Pass
101	0.023	0.652	0.004	0.048	0.000	0.665	#13	0.426	Not Required	Pass
102	0.001	0.152	0.205	0.037	0.041	0.358	#13	0.034	Not Required	Pass
103	0.005	0.386	0.059	0.039	0.008	0.439	#13	0.044	Not Required	Pass
104	0.005	0.384	0.093	0.039	0.012	0.457	#13	0.078	Not Required	Pass
105	0.005	0.239	0.094	0.039	0.014	0.259	#13	0.073	Not Required	Pass
106	0.005	0.401	0.057	0.040	0.008	0.456	#13	0.044	Not Required	Pass
107	0.005	0.249	0.098	0.040	0.014	0.267	#13	0.073	Not Required	Pass
108	0.000	0.043	0.048	0.018	0.004	0.086	#13	Not Required	Not Required	Pass
109	0.002	0.019	0.043	0.001	0.000	0.063	#13	0.198	Not Required	Pass
110	0.005	0.400	0.096	0.040	0.012	0.458	#13	0.078	Not Required	Pass
111	0.000	0.043	0.048	0.018	0.004	0.086	#13	Not Required	Not Required	Pass
112	0.001	0.164	0.218	0.038	0.043	0.382	#13	0.034	Not Required	Pass
113	0.002	0.115	0.104	0.031	0.006	0.202	#13	0.177	Not Required	Pass
114	0.002	0.118	0.104	0.031	0.006	0.202	#13	0.265	Not Required	Pass
115	0.000	0.062	0.054	0.022	0.005	0.111	#13	0.321	Not Required	Pass
116	0.000	0.062	0.054	0.022	0.005	0.111	#13	0.321	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 _r	Length between braced points

L	Length between brace 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																										
	<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 Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <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="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>2.413</td> <td>3.421</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.153</td> <td>-3.588</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.019</td> <td>-0.032</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.036</td> <td>0.060</td> </tr> <tr> <td>M_z (kipft)</td> <td>16.441</td> <td>27.583</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ 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)	2.413	3.421	V_x (kip)	-2.153	-3.588	V_z (kip)	-0.019	-0.032	M_x (kipft)	0.036	0.060	M_z (kipft)	16.441	27.583	
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	<p>Required depth to resist lateral loads (ASD) 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: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.153 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.34283 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(16.441 \text{ kipft}) + ((-2.153 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.618 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.8013 \text{ ft}$ - 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,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((-0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0057325 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.69275 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8013 \text{ ft}), (0.69275 \text{ ft})]$$

$$L_{e,req} = 4.801 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.801 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.91448$$

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

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

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

$$q = \frac{(2.413 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.15081 \text{ kip/ft}^2$$

$$q = 0.15081 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.15081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.075406$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.34283 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.618 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.618 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.618 \text{ kipft/ft})) + (4 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6375 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.618 \text{ kipft/ft})) + (3 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (2.618 \text{ kipft/ft})) + (2 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.16457 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.618 \text{ kipft/ft})) + ((-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.748 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6375 \text{ ft})}{2}$$

$$p_a = 0.27281 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.16457 \text{ kip/ft}^2)}{(0.27281 \text{ kip/ft}^2)}$$

$$Ratio = 0.60322$$

p_a - Allowable lateral soil pressure at depth L_e ,

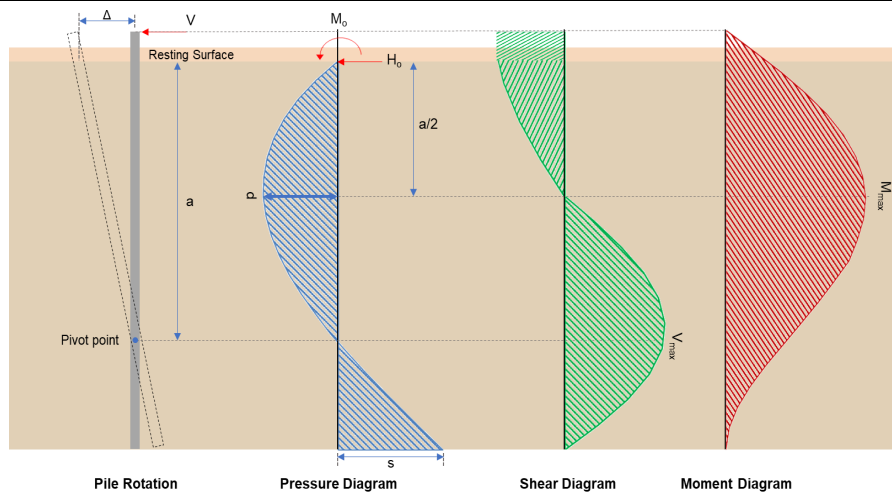
Status: **PASS**
Ratio: **0.600**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.748 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94984$	Status: PASS Ratio: 0.950
	<p>Considering z-direction:</p> <p>$H_o = -0.0030255 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0057325 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0057325 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0057325 \text{ kipft/ft})) + (4 \times (-0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.7838 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0057325 \text{ kipft/ft})) + (3 \times (-0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.0057325 \text{ kipft/ft})) + (2 \times (-0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = -0.0011414 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0057325 \text{ kipft/ft})) + ((-0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = -0.00096191 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.7838 \text{ ft})}{2}$ $p_a = 0.28379 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0011414 \text{ kip/ft}^2)}{(0.28379 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0040219$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(-0.00096191 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = -0.0012215$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.588 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.583 \text{ kipft}) + ((-3.588 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.3922 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3922 \text{ kipft/ft})}{(-0.57134 \text{ kip/ft})}$$

$$E = 7.6876 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3922 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.57134 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.3922 \text{ kipft/ft})) + (4 \times (-0.57134 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6369 \text{ ft}$$

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

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

$$V_{max} = ((-0.57134 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.6876 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6369 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.6876 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6369 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.4284 \text{ kip}$$

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

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

$$M_{max} = ((-0.57134 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(7.6876 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.6876 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.6876 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.413 \text{ kipft}$$

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,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((-0.032 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0095541 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0095541 \text{ kipft/ft})}{(-0.0050955 \text{ kip/ft})}$$

$$E = 1.875 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0095541 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0050955 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0095541 \text{ kipft/ft})) + (4 \times (-0.0050955 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7849 \text{ ft}$$

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

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

$$V_{max} = ((-0.0050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7849 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7849 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.026532 \text{ kip}$$

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

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

$$M_{max} = ((-0.0050955 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(1.875 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.060142 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(3.421 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.482 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.482 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(3.421 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0012788$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 3.421 \text{ kip} \rightarrow 3421 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3421 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 118.94 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

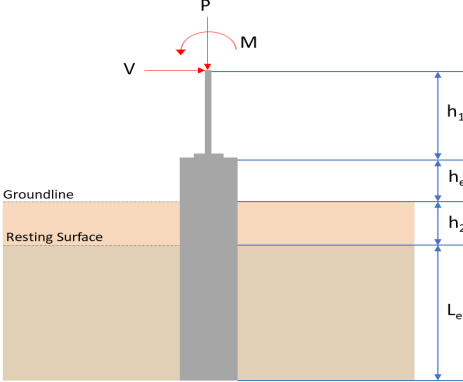
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (118.94 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 118.94 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ytik} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((118.94 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.39 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.4284 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.4284 \text{ kip})}{(110.39 \text{ kip})}$ $\text{Ratio} = 0.06729$ <p>Considering z-direction:</p> <p>$V_{max} = 0.026532 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.026532 \text{ kip})}{(110.39 \text{ kip})}$ $\text{Ratio} = 0.00024034$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 18.413 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(18.413 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.073768$	<p>Status: PASS Ratio: 0.070</p>
	<p>Considering z-direction: $M_{max} = 0.060142 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.060142 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00024095$	<p>Status: PASS Ratio: 0.000</p>

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 Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1192 1191"> <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="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>2.413</td> <td>3.421</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.153</td> <td>-3.588</td> </tr> <tr> <td>V_z (kip)</td> <td>0.019</td> <td>0.032</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.036</td> <td>-0.060</td> </tr> <tr> <td>M_z (kipft)</td> <td>16.441</td> <td>27.584</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ 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)	2.413	3.421	V_x (kip)	-2.153	-3.588	V_z (kip)	0.019	0.032	M_x (kipft)	-0.036	-0.060	M_z (kipft)	16.441	27.584	
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	<p>Required depth to resist lateral loads (ASD) 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: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.153 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.34283 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(16.441 \text{ kipft}) + ((-2.153 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.618 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.8013 \text{ ft}$ - 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,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0057325 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.8494 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8013 \text{ ft}), (0.8494 \text{ ft})]$$

$$L_{e,req} = 4.801 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.801 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.91448$$

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

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

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

$$q = \frac{(2.413 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.15081 \text{ kip/ft}^2$$

$$q = 0.15081 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.15081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.075406$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.34283 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.618 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.618 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.618 \text{ kipft/ft})) + (4 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6375 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.618 \text{ kipft/ft})) + (3 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (2.618 \text{ kipft/ft})) + (2 \times (-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.16457 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.618 \text{ kipft/ft})) + ((-0.34283 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.748 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6375 \text{ ft})}{2}$$

$$p_a = 0.27281 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16457 \text{ kip/ft}^2)}{(0.27281 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.60322$$

p_a - Allowable lateral soil pressure at depth L_e ,

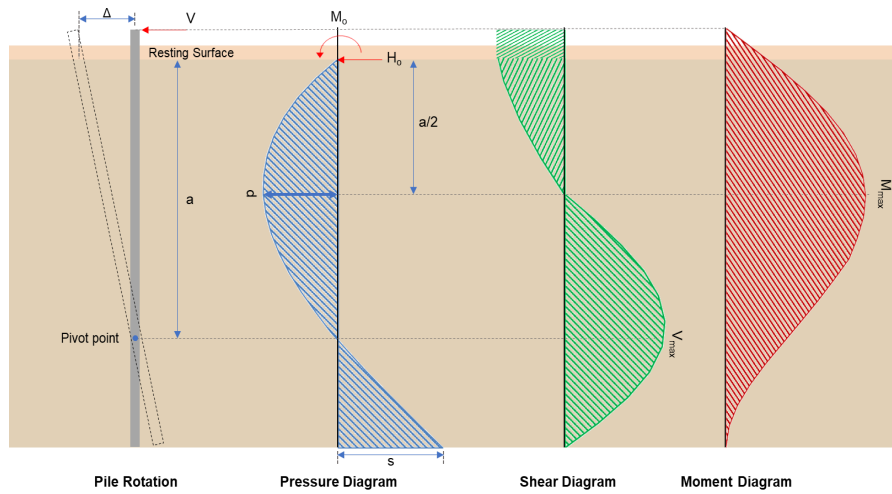
Status: **PASS**
Ratio: **0.600**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.748 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94984$	<p>Status: PASS Ratio: 0.950</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.0030255 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0057325 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0057325 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0057325 \text{ kipft/ft})) + (4 \times (0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.7838 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0057325 \text{ kipft/ft})) + (3 \times (0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.0057325 \text{ kipft/ft})) + (2 \times (0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = 0.0027684 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0057325 \text{ kipft/ft})) + ((0.0030255 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = 0.0059535 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.7838 \text{ ft})}{2}$ $p_a = 0.28379 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0027684 \text{ kip/ft}^2)}{(0.28379 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0097553$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.010</p>

$$\text{Ratio} = \frac{(0.0059535 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00756$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.588 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.584 \text{ kipft}) + ((-3.588 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.3924 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3924 \text{ kipft/ft})}{(-0.57134 \text{ kip/ft})}$$

$$E = 7.6878 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3924 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.57134 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.3924 \text{ kipft/ft})) + (4 \times (-0.57134 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6369 \text{ ft}$$

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

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

$$V_{max} = ((-0.57134 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.6878 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6369 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.6878 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6369 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.4286 \text{ kip}$$

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

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

$$M_{max} = ((-0.57134 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(7.6878 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.6878 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.6878 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6369 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.413 \text{ kipft}$$

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,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((0.032 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0095541 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0095541 \text{ kipft/ft})}{(0.0050955 \text{ kip/ft})}$$

$$E = 1.875 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0095541 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0050955 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.0095541 \text{ kipft/ft})) + (4 \times (0.0050955 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7849 \text{ ft}$$

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

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

$$V_{max} = ((0.0050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7849 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7849 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.026532 \text{ kip}$$

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

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

$$M_{max} = ((0.0050955 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(1.875 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.875 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7849 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.060142 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(3.421 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.482 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.482 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(3.421 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0012788$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 3.421 \text{ kip} \rightarrow 3421 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3421 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 118.94 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (118.94 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 118.94 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((118.94 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.39 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.4286 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.4286 \text{ kip})}{(110.39 \text{ kip})}$ $\text{Ratio} = 0.067292$ <p>Considering z-direction:</p> <p>$V_{max} = 0.026532 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.026532 \text{ kip})}{(110.39 \text{ kip})}$ $\text{Ratio} = 0.00024034$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 18.413 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(18.413 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.073771$	<p>Status: PASS Ratio: 0.070</p>
	<p>Considering z-direction: $M_{max} = 0.060142 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.060142 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00024095$	<p>Status: PASS Ratio: 0.000</p>