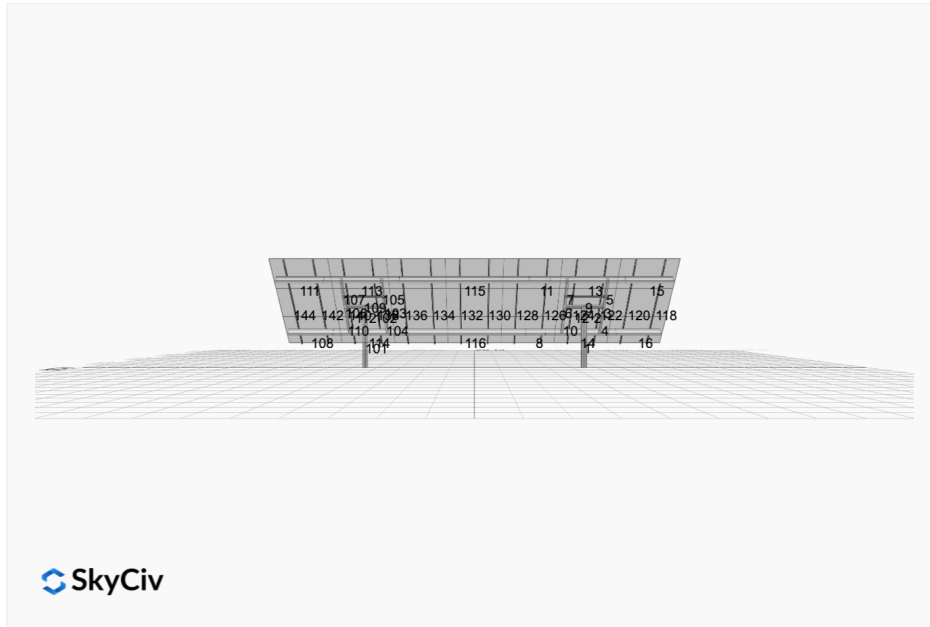


**Project Name:** MTSOLAR\_9BJ1862KL72E      **Date:** Wed Aug 06 2025  
**Location:** 42755 Fish Hatchery Dr, Scio, OR 97374, USA      **Number of Modules:** 21  
**Unique ID:** 2P-22.5-6TOP-HD-57-L-3Hx7W-7BAD      **Number of Poles:** 2  
**Dealer:** \_\_\_\_\_      **Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	11.98 ft
<b>Array Dimensions E/W</b>	40.27 ft
<b>Winter Tilt Angle</b>	45
<b>Front Edge Clearance</b>	2 ft

### MT Solar Bill of Materials (2P-22.5-6TOP-HD-57-L-3Hx7W-7BAD)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-HD	H-Frame Assembly-HD	2
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	7

### Rail Bill of Materials

Part	Qty
Rails (144in)	14
Rail Attachment	28
Module Mid Clamp	28
Module End Clamp	28
Ground Lug	7

## Site Details:



**Site Address:** 42755 Fish Hatchery Dr, Scio, OR 97374, USA

### Array Specification

<b>Duty Classification:</b>	HD
<b>Module Width:</b>	47.44 in
<b>Module Length:</b>	68.03in
<b>Number of Rows:</b>	3
<b>Number of Columns:</b>	7
<b>Total Number of Modules:</b>	21
<b>Winter Tilt Angle:</b>	45
<b>Front Edge Clearance:</b>	2
<b>Total Array Height at Tilt:</b>	10.47 ft
<b>Total Frame Length:</b>	39.50 ft
<b>Module Info/Notes:</b>	RE-460AAPure RX
<b>Array Dimensions N/S:</b>	11.98 ft
<b>Array Dimensions E/W:</b>	40.27 ft
<b>Rail Length:</b>	143.82 in
<b>Rail Spacing:</b>	2.88 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	6.24 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	22.5 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.50 ft Pile 2: 5.50 ft
<b>Foundation Volume:</b>	6.519 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	42755 Fish Hatchery Dr, Scio, OR 97374, USA
<b>Wind Speed:</b>	110 mph
<b>Snow Load:</b>	60 psf

### **Design Disclaimer**

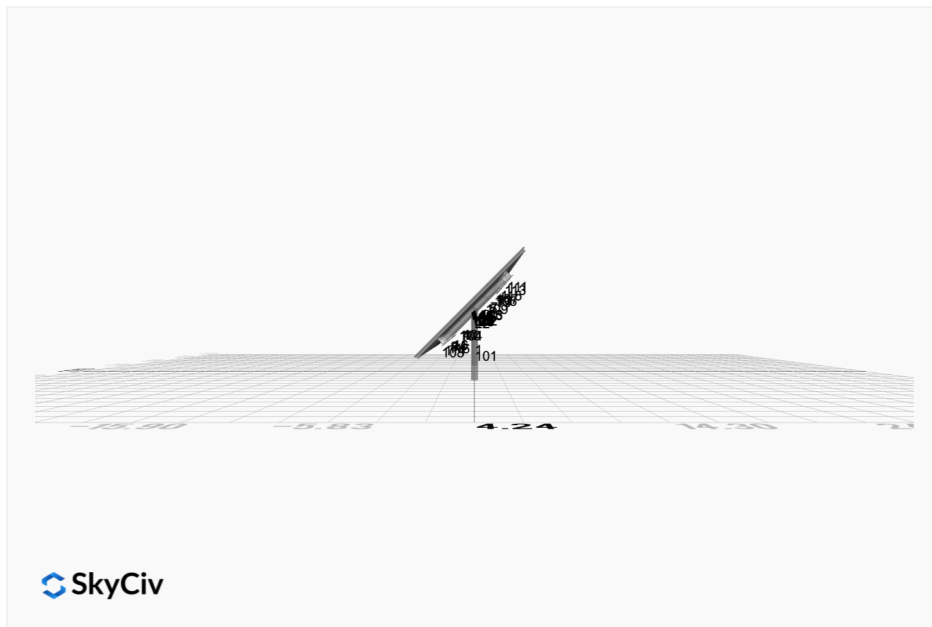
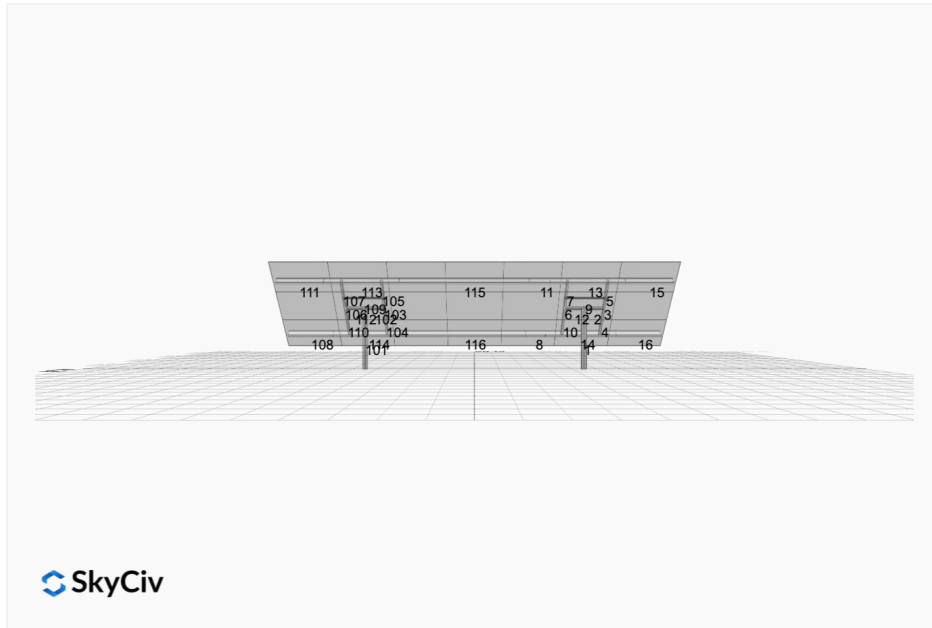
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

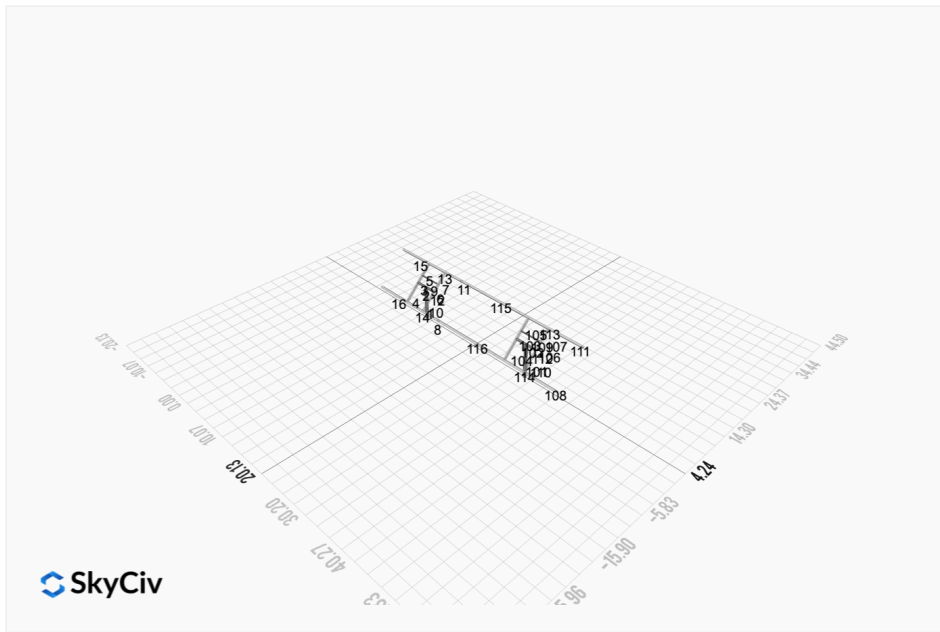
## AutoDesigner Input

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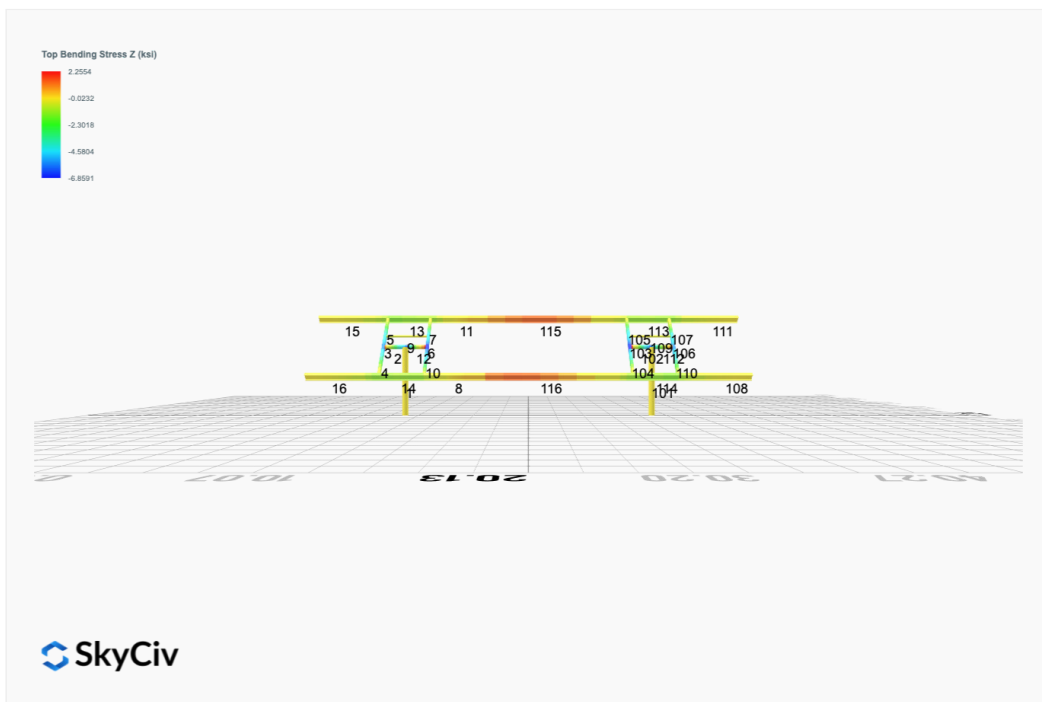
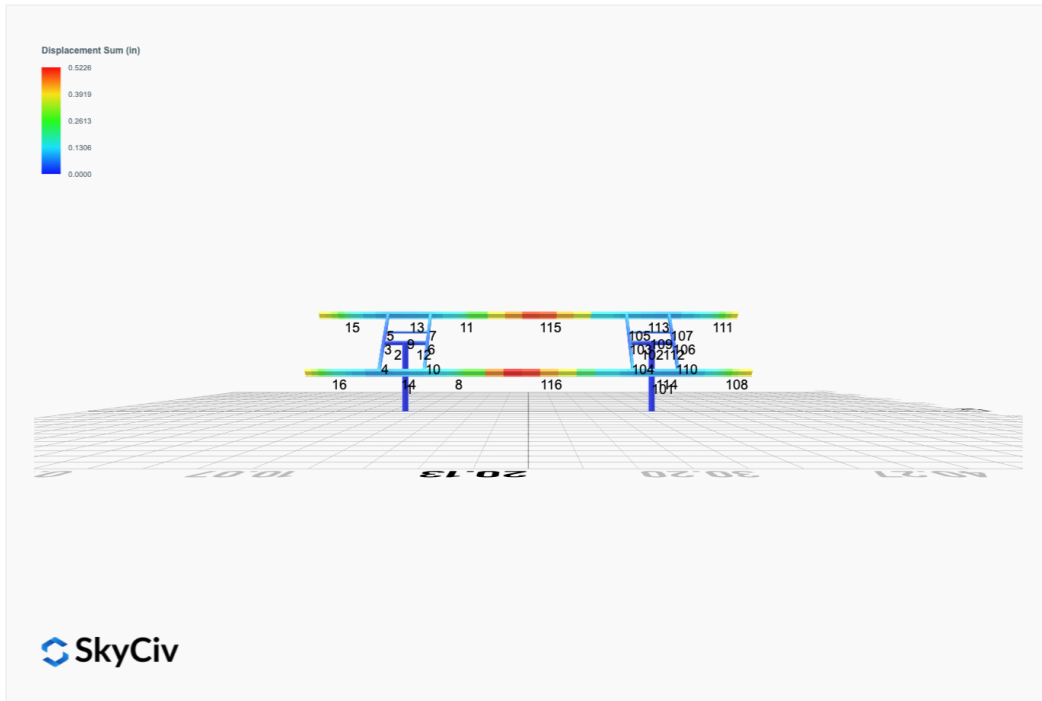
## Design Notes:

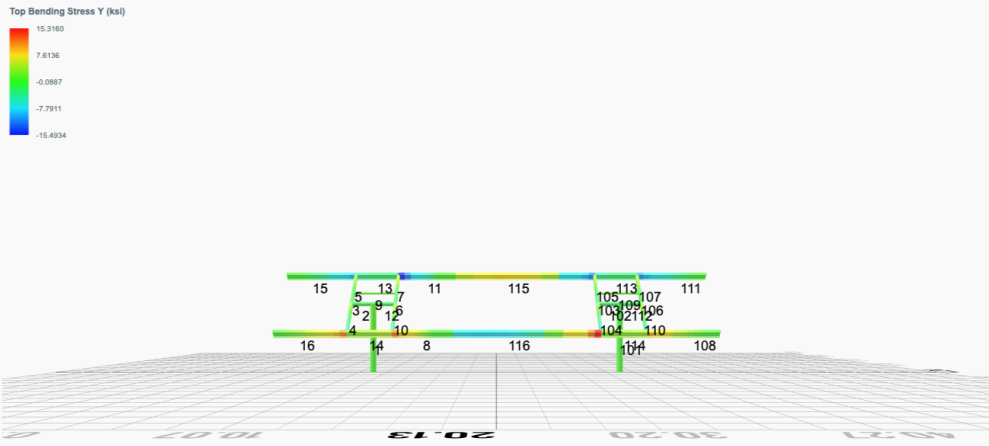
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



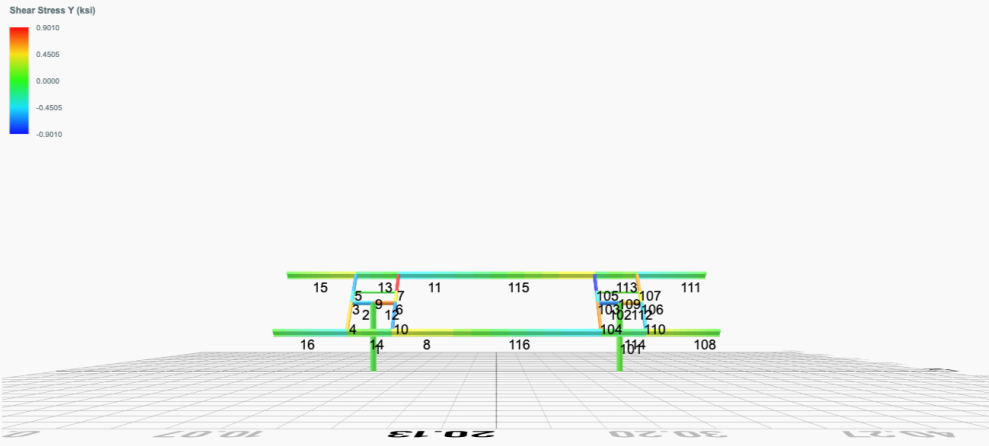


# FEM Results (Envelope Worst Case for each member)

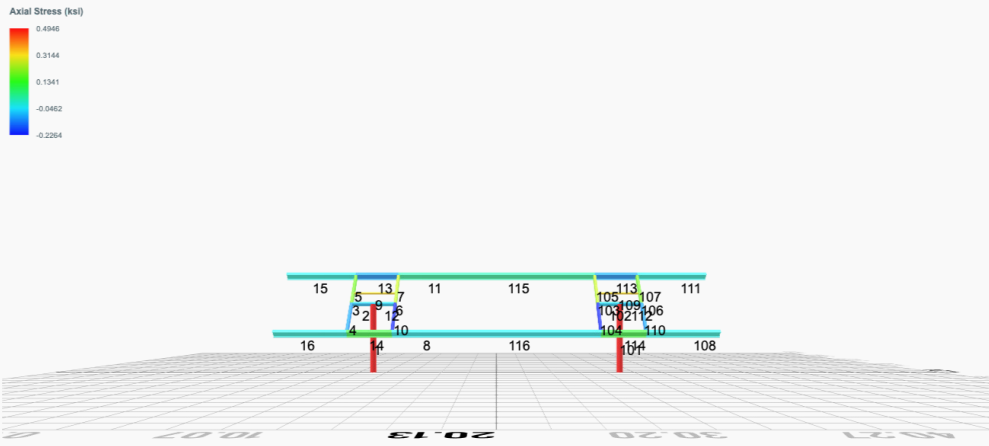




SkyCiv



SkyCiv



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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.9033	0.0512	0.0796	-0.0300	0.0222
ULS: 2. D + L	0.0000	1.9033	0.0512	0.0796	-0.0300	0.0222
ULS: 3. D + (S or Lr or R)	0.0000	4.6641	0.1417	0.2208	-0.0834	0.0322
ULS: 3. D + (S or Lr or R)	0.0000	1.9033	0.0512	0.0796	-0.0300	0.0222
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.9739	0.1191	0.1855	-0.0700	0.0297
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.9033	0.0512	0.0796	-0.0300	0.0222
ULS: 5b. D + 0.7E	0.0000	1.9033	0.0512	0.0796	-0.0300	0.0222
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.9739	0.1191	0.1855	-0.0700	0.0297
ULS: 8. 0.6D + 0.7E	0.0000	1.1420	0.0307	0.0478	-0.0180	0.0133
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9789	4.8822	0.1789	0.2533	-0.3265	19.8423
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.9789	4.8822	0.1789	0.2533	-0.3265	19.8423
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1549	-0.2516	-0.0405	-0.0451	0.1836	-12.6453
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.9014	0.0019	-0.0304	-0.0313	0.1599	-17.0723
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2342	6.2080	0.2148	0.3158	-0.2924	14.8947
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2342	6.2080	0.2148	0.3158	-0.2924	14.8947
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6162	2.3577	0.0503	0.0919	0.0902	-9.4710
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4261	2.5478	0.0579	0.1023	0.0724	-12.7912
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2342	4.1375	0.1470	0.2099	-0.2523	14.8873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2342	4.1375	0.1470	0.2099	-0.2523	14.8873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6162	0.2871	-0.0176	-0.0139	0.1302	-9.4784
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4261	0.4773	-0.0100	-0.0036	0.1124	-12.7986
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9789	4.1209	0.1584	0.2215	-0.3145	19.8334
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.9789	4.1209	0.1584	0.2215	-0.3145	19.8334
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1549	-1.0129	-0.0610	-0.0770	0.1956	-12.6542
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.9014	-0.7594	-0.0509	-0.0632	0.1719	-17.0812

### 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	9.1836
Shear X	-4.9648
Shear Z	0.3198
Moment X	0.4698
Moment Y (Twist)	0.5558
Moment Z	33.4094

### 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	6.2080
Shear X	-2.9789
Shear Z	0.2148
Moment X	0.3158
Moment Y (Twist)	0.3265
Moment Z	19.8423

## 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	1.9033	-0.0512	-0.0796	0.0300	0.0223
ULS: 2. D + L	-0.0000	1.9033	-0.0512	-0.0796	0.0300	0.0223
ULS: 3. D + (S or Lr or R)	-0.0000	4.6641	-0.1417	-0.2209	0.0835	0.0322
ULS: 3. D + (S or Lr or R)	-0.0000	1.9033	-0.0512	-0.0796	0.0300	0.0223
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.9739	-0.1191	-0.1856	0.0701	0.0297

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.9033	-0.0512	-0.0796	0.0300	0.0223
ULS: 5b. D + 0.7E	-0.0000	1.9033	-0.0512	-0.0796	0.0300	0.0223
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.9739	-0.1191	-0.1856	0.0701	0.0297
ULS: 8. 0.6D + 0.7E	-0.0000	1.1420	-0.0307	-0.0478	0.0180	0.0134
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9789	4.8822	-0.1789	-0.2534	0.3265	19.8423
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.9789	4.8822	-0.1789	-0.2534	0.3265	19.8423
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1549	-0.2516	0.0405	0.0451	-0.1836	-12.6453
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.9014	0.0019	0.0304	0.0313	-0.1599	-17.0723
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2342	6.2080	-0.2148	-0.3159	0.2925	14.8948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2342	6.2080	-0.2148	-0.3159	0.2925	14.8948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6162	2.3577	-0.0503	-0.0920	-0.0901	-9.4709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4261	2.5478	-0.0579	-0.1024	-0.0723	-12.7912
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2342	4.1375	-0.1470	-0.2099	0.2524	14.8873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.2342	4.1375	-0.1470	-0.2099	0.2524	14.8873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6162	0.2871	0.0176	0.0139	-0.1302	-9.4784
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4261	0.4773	0.0100	0.0036	-0.1124	-12.7986
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9789	4.1209	-0.1584	-0.2215	0.3145	19.8334
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.9789	4.1209	-0.1584	-0.2215	0.3145	19.8334
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1549	-1.0129	0.0610	0.0770	-0.1956	-12.6542
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.9014	-0.7594	0.0509	0.0631	-0.1719	-17.0812

### 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	9.1836
Shear X	-4.9648
Shear Z	-0.3198
Moment X	-0.4706
Moment Y (Twist)	0.5561
Moment Z	33.4103

### 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	6.2080
Shear X	-2.9789
Shear Z	-0.2148
Moment X	-0.3159
Moment Y (Twist)	0.3265
Moment Z	19.8423

# Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States  
  
 User Name: sales@mtsolar.us  
 Project Name: MTSOLAR\_9BJ1862KL72E  
 Unit System: imperial



## Design Input Information

Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
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 <sup>2</sup> )	J (in <sup>4</sup> )	$I_{y0}$ (in <sup>4</sup> )	$I_{z0}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{y0}$ (in <sup>3</sup> )	$S_{z0}$ (in <sup>3</sup> )



113	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.11,1.09,1.09,1.09,1.11,1.09,1.09,1.08,1.05,1.09,1.09,1.10,1.11,1.09,1.09,1.09,1.10	300	200	1
114	19	4.88	4.00	7.50	1.08,1.09,1.08,1.09,1.08,1.08,1.09,1.09,1.10,3.65,1.09,1.09,1.10,1.04,1.09,1.09,1.07,1.09,1.09,1.10,1.57,1.09,1.09,1.09,1.05	300	200	1
115	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.14,1.14,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13	300	200	1
116	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,2.44,1.13,1.13,1.13,1.11,1.13,1.13,1.14,1.14,1.13,1.13,1.13,1.19,1.13,1.13,1.13,1.12	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	175.53	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	123.95	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	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	23.94	6.12	40.24	43.62
14	133.20	85.85	23.73	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	175.53	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	32.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	32.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.96	6.12	40.24	43.62
114	133.20	85.85	23.76	6.12	40.24	43.62
115	133.20	46.28	12.23	6.12	40.24	43.62
116	133.20	46.28	12.03	6.12	40.24	43.62

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	$\delta$	Status
1	0.052	0.790	0.036	0.066	0.004	0.825	#13	0.350	Not Required	Pass
2	0.005	0.289	0.213	0.069	0.041	0.501	#13	0.035	Not Required	Pass
3	0.009	0.555	0.046	0.055	0.007	0.565	#13	0.045	Not Required	Pass
4	0.008	0.501	0.174	0.050	0.027	0.575	#13	0.040	Not Required	Pass

4	0.000	0.301	0.174	0.050	0.057	0.373	#13	0.000	Not Required	Pass
5	0.008	0.344	0.167	0.055	0.043	0.364	#13	0.074	Not Required	Pass
6	0.011	0.623	0.078	0.063	0.010	0.672	#13	0.045	Not Required	Pass
7	0.011	0.386	0.231	0.062	0.059	0.422	#13	0.074	Not Required	Pass
8	0.002	0.064	0.235	0.042	0.019	0.260	#21	0.095	Not Required	Pass
9	0.019	0.043	0.070	0.002	0.002	0.111	#13	0.204	Not Required	Pass
10	0.012	0.565	0.213	0.057	0.044	0.620	#13	0.080	Not Required	Pass
11	0.003	0.069	0.242	0.046	0.019	0.271	#21	0.095	Not Required	Pass
12	0.004	0.356	0.239	0.083	0.043	0.595	#13	0.035	Not Required	Pass
13	0.007	0.227	0.513	0.057	0.024	0.673	#21	0.286	Not Required	Pass
14	0.010	0.211	0.505	0.052	0.024	0.653	#21	0.190	Not Required	Pass
15	0.000	0.085	0.208	0.029	0.012	0.279	#21	Not Required	Not Required	Pass
16	0.000	0.077	0.208	0.027	0.012	0.275	#21	Not Required	Not Required	Pass
101	0.052	0.790	0.036	0.066	0.004	0.825	#13	0.350	Not Required	Pass
102	0.004	0.356	0.239	0.083	0.043	0.595	#13	0.035	Not Required	Pass
103	0.011	0.623	0.078	0.063	0.010	0.672	#13	0.045	Not Required	Pass
104	0.012	0.565	0.213	0.057	0.044	0.620	#13	0.080	Not Required	Pass
105	0.011	0.386	0.231	0.062	0.059	0.422	#13	0.074	Not Required	Pass
106	0.009	0.555	0.046	0.055	0.007	0.565	#13	0.045	Not Required	Pass
107	0.008	0.344	0.167	0.055	0.043	0.364	#13	0.074	Not Required	Pass
108	0.000	0.077	0.208	0.027	0.012	0.275	#21	Not Required	Not Required	Pass
109	0.019	0.043	0.070	0.002	0.002	0.111	#13	0.204	Not Required	Pass
110	0.008	0.501	0.174	0.050	0.037	0.575	#13	0.080	Not Required	Pass
111	0.000	0.085	0.208	0.029	0.012	0.279	#21	Not Required	Not Required	Pass
112	0.005	0.289	0.213	0.069	0.041	0.501	#13	0.035	Not Required	Pass
113	0.007	0.227	0.513	0.057	0.024	0.673	#21	0.190	Not Required	Pass
114	0.010	0.211	0.505	0.052	0.024	0.653	#21	0.286	Not Required	Pass
115	0.009	0.480	0.279	0.046	0.019	0.683	#21	0.601	Not Required	Pass
116	0.003	0.440	0.279	0.042	0.019	0.660	#21	0.601	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
$F_y$	Specified minimum yield stress
$F_u$	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
$I_{yp}$	Moment of inertia about the Y axes
$I_{zp}$	Moment of inertia about the Z axes
$I_w$	Warping constant
$S_{yp}$	Plastic section modulus about the Y axis
$S_{zp}$	Plastic section modulus about the Z axis
KL	Effective length
$C_b$	Buckling modification factor (from all load combinations)
$L_b$	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
$P_n$	Nominal axial strength (tension/compression)
$M_n$	Nominal flexural strength (about Z/Y axis)
$V_n$	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
$M_z$	Design ratio in case of bending about Z axis
$M_y$	Design ratio in case of bending about Y axis
$V_y$	Design ratio in case of shear along Y axis

$V_z$	Design ratio in case of shear along Z axis
$(P, M_z, M_y)$	Design ratio in case of axial force and bending action
$KL/r$	Design ratio in case of section slenderness
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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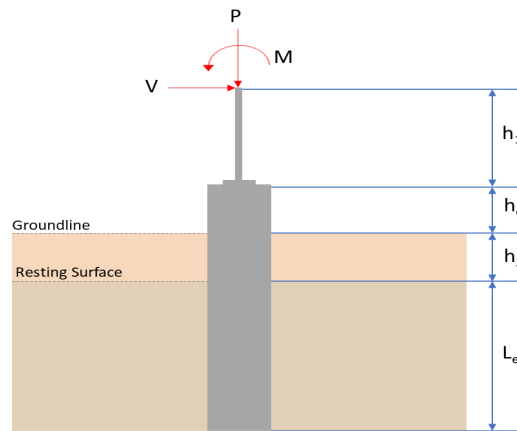
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	6.208	9.184
$V_x$ (kip)	-2.979	-4.965
$V_z$ (kip)	0.215	0.320
$M_x$ (kipft)	0.316	0.470
$M_z$ (kipft)	19.842	33.409

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.979 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47436 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(19.842 \text{ kipft}) + ((-2.979 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1596 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8565 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.215 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.034236 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.316 \text{ kipft}) + ((0.215 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.050318 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.0134 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.8565 \text{ ft}), (2.0134 \text{ ft})]$$

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

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.856 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.88291$$

Status: **PASS**  
Ratio: **0.880**

### 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

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

$$q = \frac{(6.208 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.388 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.388 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.194$$

Status: **PASS**  
Ratio: **0.190**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.47436 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.1596 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1596 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (3.1596 \text{ kipft/ft})) + (4 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8294 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1596 \text{ kipft/ft})) + (3 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (3.1596 \text{ kipft/ft})) + (2 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.1347 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (3.1596 \text{ kipft/ft})) + ((-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.73589 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8294 \text{ ft})}{2}$$

$$p_a = 0.2872 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.1347 \text{ kip/ft}^2)}{(0.2872 \text{ kip/ft}^2)}$$

$$Ratio = 0.46901$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.73589 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.892$$

Status: **PASS**  
Ratio: **0.470**

Status: **PASS**  
Ratio: **0.890**

#### Considering z-direction:

$H_o = 0.034236 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.050318 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.050318 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.034236 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.050318 \text{ kipft/ft})) + (4 \times (0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.994 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.050318 \text{ kipft/ft})) + (3 \times (0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 [(3 \times (0.050318 \text{ kipft/ft})) + (2 \times (0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.027588 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (0.050318 \text{ kipft/ft})) + ((0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.057309 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.994 \text{ ft})}{2}$$

$$p_a = 0.29954 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.027588 \text{ kip/ft}^2)}{(0.29954 \text{ kip/ft}^2)}$$

$$Ratio = 0.092101$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

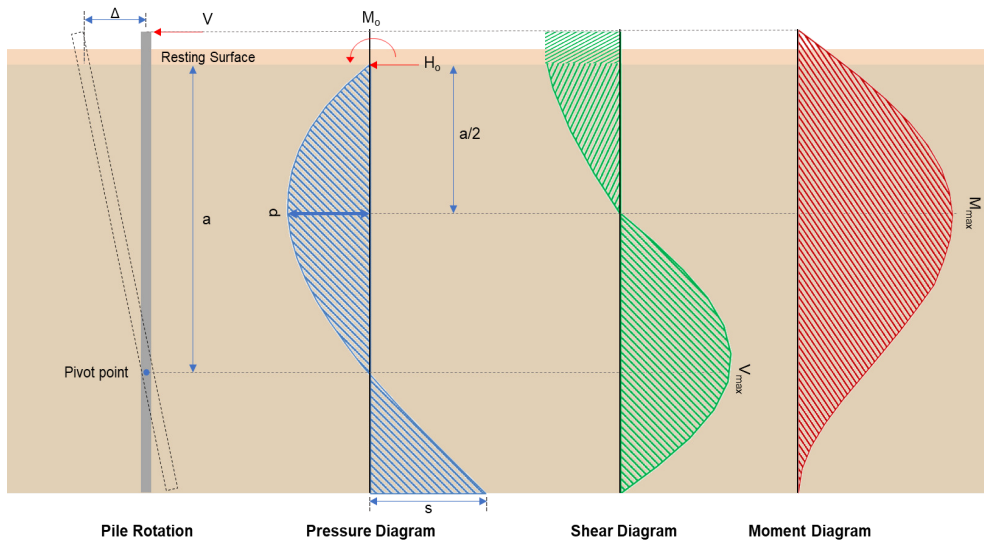
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.057309 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.069466$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.965 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.79061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(33.409 \text{ kipft}) + ((-4.965 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.3199 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.3199 \text{ kipft/ft})}{(-0.79061 \text{ kip/ft})}$$

$$E = 6.7289 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.3199 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.79061 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times 5.3199) + (4 \times (-0.79061) \times 5.5)}$$

$$a = \frac{(-0.79061 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.3199 \text{ kipft/ft})) + (4 \times (-0.79061 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8283 \text{ ft}$$

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

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

$$V_{max} = ((-0.79061 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.7289 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8283 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6.7289 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8283 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.9323 \text{ kip}$$

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

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

$$M_{max} = ((-0.79061 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(6.7289 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8283 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.7289 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8283 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (6.7289 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8283 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 22.992 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.050955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.47 \text{ kipft}) + ((0.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.074841 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.074841 \text{ kipft/ft})}{(0.050955 \text{ kip/ft})}$$

$$E = 1.4688 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.074841 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.050955 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.074841 \text{ kipft/ft})) + (4 \times (0.050955 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.994 \text{ ft}$$

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

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

$$V_{max} = ((0.050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.4688 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.994 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (1.4688 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.994 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23342 \text{ kip}$$

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

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

$$M_{max} = ((0.050955 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(1.4688 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.994 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (1.4688 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.994 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (1.4688 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.994 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.54267 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

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

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

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

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

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(9.184 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.291 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.291 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.184 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.003433$	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 9.184 \text{ kip} \rightarrow 9184 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9184 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.71 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.89 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.9323 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(8.9323 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.080549$$

**Considering z-direction:**

$V_{max} = 0.23342 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.23342 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.0021049$$

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

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

**Flexural Strength (ACI 318-19, LRFD)**

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 22.992 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(22.992 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.092116$$

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

**Considering z-direction:**

$M_{max} = 0.54267 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.54267 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0021742$$

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

REFERENCES	CALCULATIONS	RESULTS
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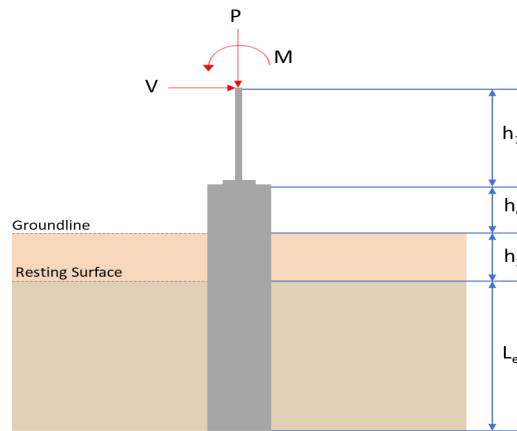
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	6.208	9.184
$V_x$ (kip)	-2.979	-4.965
$V_z$ (kip)	-0.215	-0.320
$M_x$ (kipft)	-0.316	-0.471
$M_z$ (kipft)	19.842	33.410

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.979 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47436 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(19.842 \text{ kipft}) + ((-2.979 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1596 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8565 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.215 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.034236 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.316 \text{ kipft}) + ((-0.215 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.050318 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.1733 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.8565 \text{ ft}), (1.1733 \text{ ft})]$$

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

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.856 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.88291$$

Status: **PASS**  
Ratio: **0.880**

### 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

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

$$q = \frac{(6.208 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.388 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.388 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.194$$

Status: **PASS**  
Ratio: **0.190**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.47436 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.1596 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1596 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (3.1596 \text{ kipft/ft})) + (4 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8294 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1596 \text{ kipft/ft})) + (3 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (3.1596 \text{ kipft/ft})) + (2 \times (-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.1347 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (3.1596 \text{ kipft/ft})) + ((-0.47436 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.73589 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8294 \text{ ft})}{2}$$

$$p_a = 0.2872 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.1347 \text{ kip/ft}^2)}{(0.2872 \text{ kip/ft}^2)}$$

$$Ratio = 0.46901$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.73589 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.892$$

Status: **PASS**  
Ratio: **0.470**

Status: **PASS**  
Ratio: **0.890**

#### Considering z-direction:

$H_o = -0.034236 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.050318 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.050318 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.034236 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.050318 \text{ kipft/ft})) + (4 \times (-0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.994 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.050318 \text{ kipft/ft})) + (3 \times (-0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.050318 \text{ kipft/ft})) + (2 \times (-0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.014528 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (0.050318 \text{ kipft/ft})) + ((-0.034236 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = -0.017387 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.994 \text{ ft})}{2}$$

$$p_a = 0.29954 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.014528 \text{ kip/ft}^2)}{(0.29954 \text{ kip/ft}^2)}$$

$$Ratio = -0.048502$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

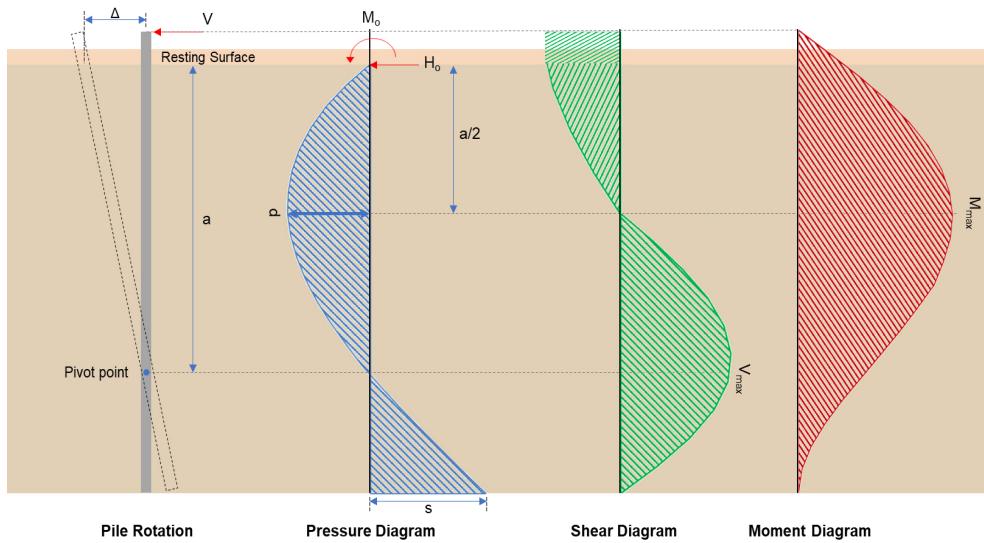
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.017387 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = -0.021075$$

Status: **PASS**  
Ratio: **-0.050**

Status: **PASS**  
Ratio: **-0.020**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.965 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.79061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(33.41 \text{ kipft}) + ((-4.965 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.3201 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.3201 \text{ kipft/ft})}{(-0.79061 \text{ kip/ft})}$$

$$E = 6.7291 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.3201 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.79061 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times 5.3201 \text{ kipft/ft}) + (4 \times (-0.79061 \text{ kip/ft}) \times 5.5 \text{ ft})}$$

$$a = \frac{6 \times (5.3201 \text{ kipft/ft}) + (4 \times (-0.79061 \text{ kip/ft}) \times (5.5 \text{ ft}))}{(6 \times (5.3201 \text{ kipft/ft}) + (4 \times (-0.79061 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8283 \text{ ft}$$

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

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

$$V_{max} = ((-0.79061 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.7291 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8283 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6.7291 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8283 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.9325 \text{ kip}$$

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

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

$$M_{max} = ((-0.79061 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(6.7291 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8283 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.7291 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8283 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (6.7291 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8283 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 22.993 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.050955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.471 \text{ kipft}) + ((-0.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.075 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.075 \text{ kipft/ft})}{(-0.050955 \text{ kip/ft})}$$

$$E = 1.4719 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.050955 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.075 \text{ kipft/ft})) + (4 \times (-0.050955 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9937 \text{ ft}$$

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

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

$$V_{max} = ((-0.050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.4719 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9937 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (1.4719 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9937 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23362 \text{ kip}$$

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

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

$$M_{max} = ((-0.050955 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(1.4719 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9937 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (1.4719 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9937 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (1.4719 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9937 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.54322 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

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

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

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

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

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(9.184 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.291 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.291 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.184 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.003433$	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 9.184 \text{ kip} \rightarrow 9184 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9184 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.71 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.89 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.9325 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(8.9325 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.080551$$

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

**Considering z-direction:**

$V_{max} = 0.23362 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.23362 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.0021067$$

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

**Flexural Strength (ACI 318-19, LRFD)**

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 22.993 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(22.993 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.092118$$

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

**Considering z-direction:**

$M_{max} = 0.54322 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.54322 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0021764$$

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