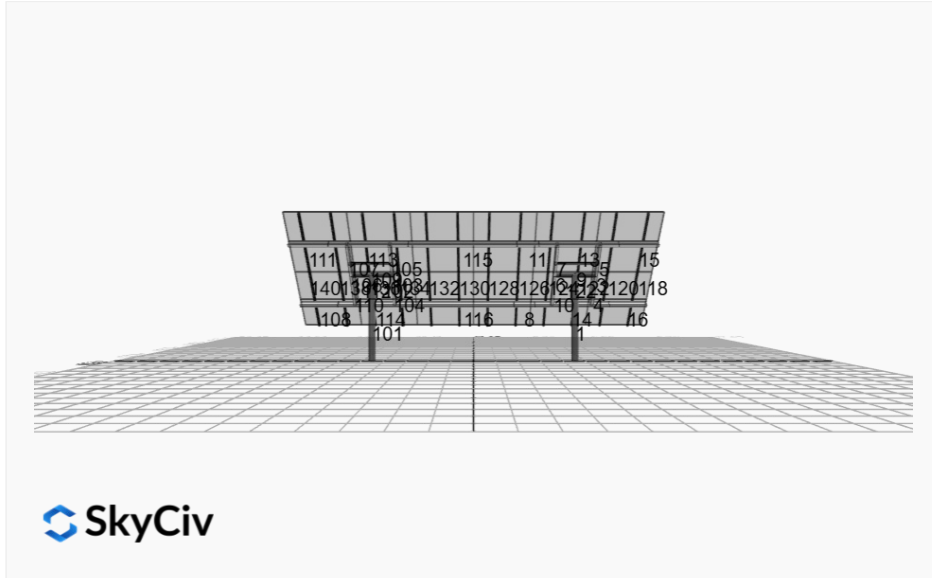


# Project Details



**Project Name:** Travis Zuroff **Date:** Tue Apr 01 2025  
**Location:** 1475 Cedar Canyon Rd, Billings, MT 59101, USA **Number of Modules:** 24  
**Unique ID:** 2P-19.75-6TOP-SD-45-L-4Hx6W-E7CJ **Number of Poles:** 2  
**Dealer:** \_\_\_\_\_ **Date Sold:** \_\_\_\_\_



Array Dimensions N/S	14.00 ft
Array Dimensions E/W	34.90 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

## MT Solar Bill of Materials (2P-19.75-6TOP-SD-45-L-4Hx6W-E7CJ)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-SD	H-Frame Assembly-SD	2
MTS-SD-Wing-45	45IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	2
MTS-SD-Splice-57	57IN SD Splice	2
MTS-CLAMP-HOOK-4PK	Hook Clamp	6

## Rail Bill of Materials

Part	Qty
Rails (168in)	12
Rail Attachment	24
Module Mid Clamp	36
Module End Clamp	24
Ground Lug	6

## Site Details:



**Site Address:** 1475 Cedar Canyon Rd, Billings, MT 59101, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	41.50 in
<b>Module Length:</b>	68.80in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	6
<b>Total Number of Modules:</b>	24
<b>Winter Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	3
<b>Total Array Height at Tilt:</b>	13.72 ft
<b>Total Frame Length:</b>	34.75 ft
<b>Module Info/Notes:</b>	Mission Solar Energy
<b>Array Dimensions N/S:</b>	14.00 ft
<b>Array Dimensions E/W:</b>	34.90 ft
<b>Rail Length:</b>	168.00 in
<b>Rail Spacing:</b>	2.91 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	8.36 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	19.75 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>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	1475 Cedar Canyon Rd, Billings, MT 59101, USA
<b>Wind Speed:</b>	103 mph
<b>Snow Load:</b>	20 psf

### **Design Disclaimer**

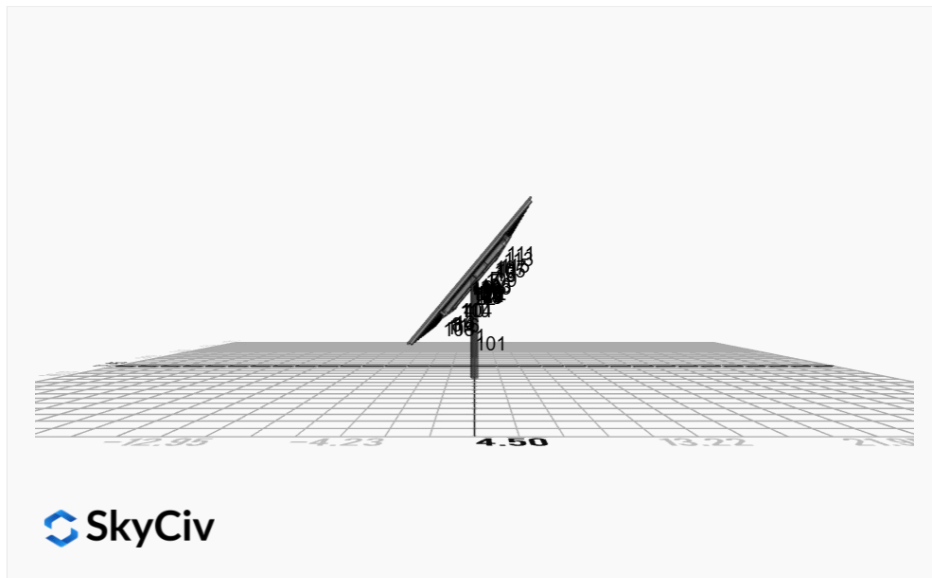
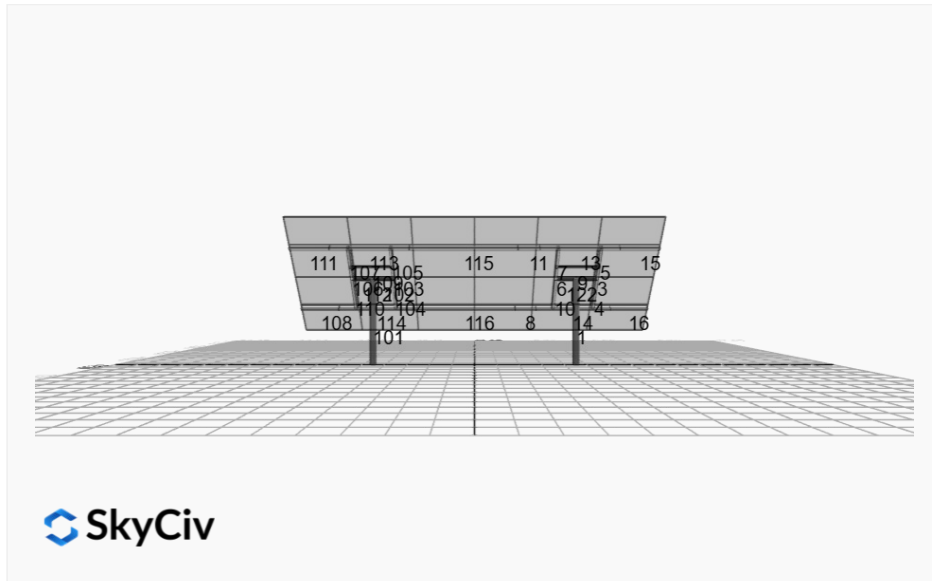
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

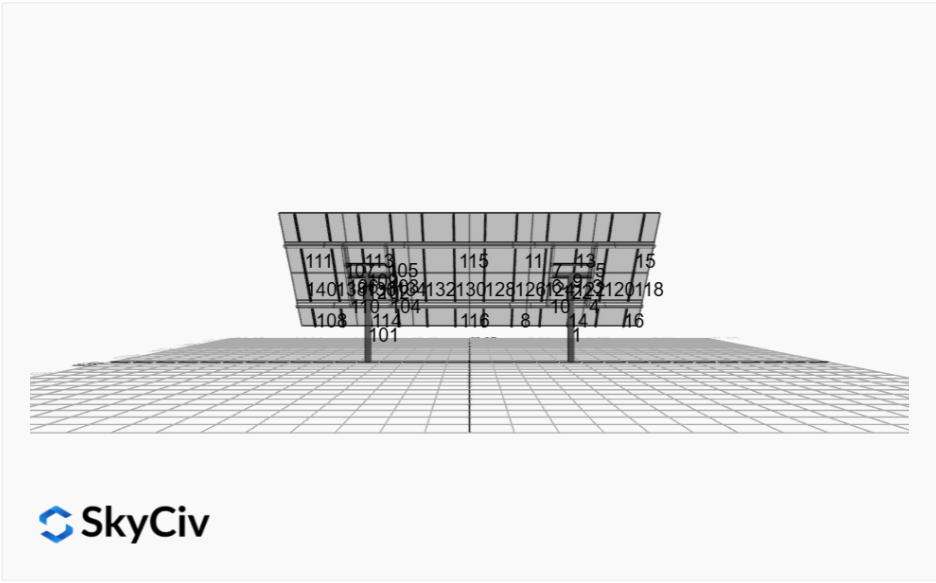
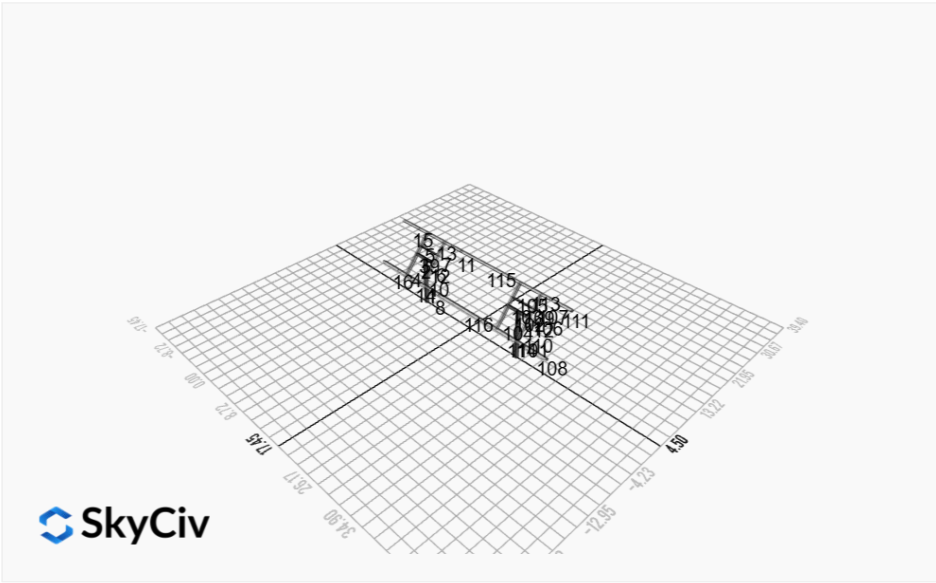
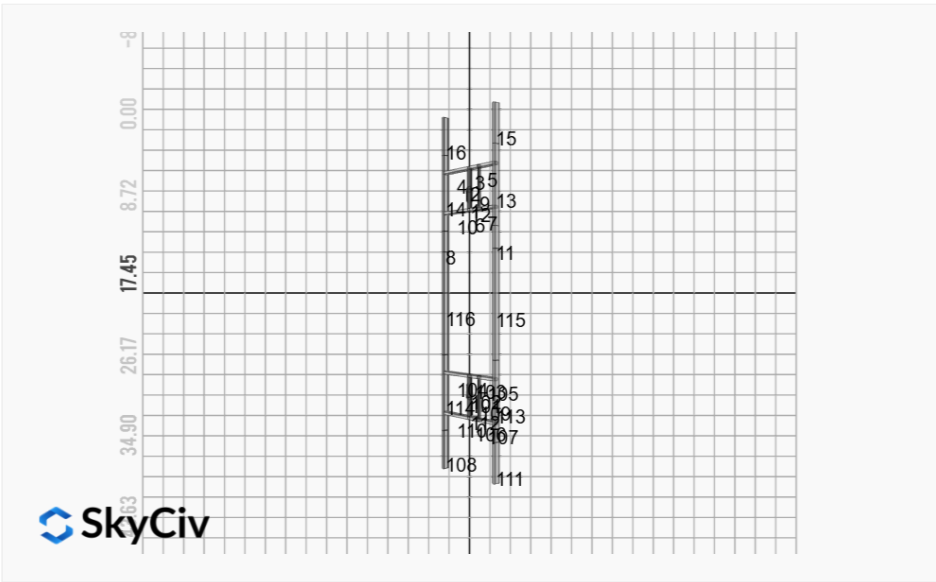
## AutoDesigner Input

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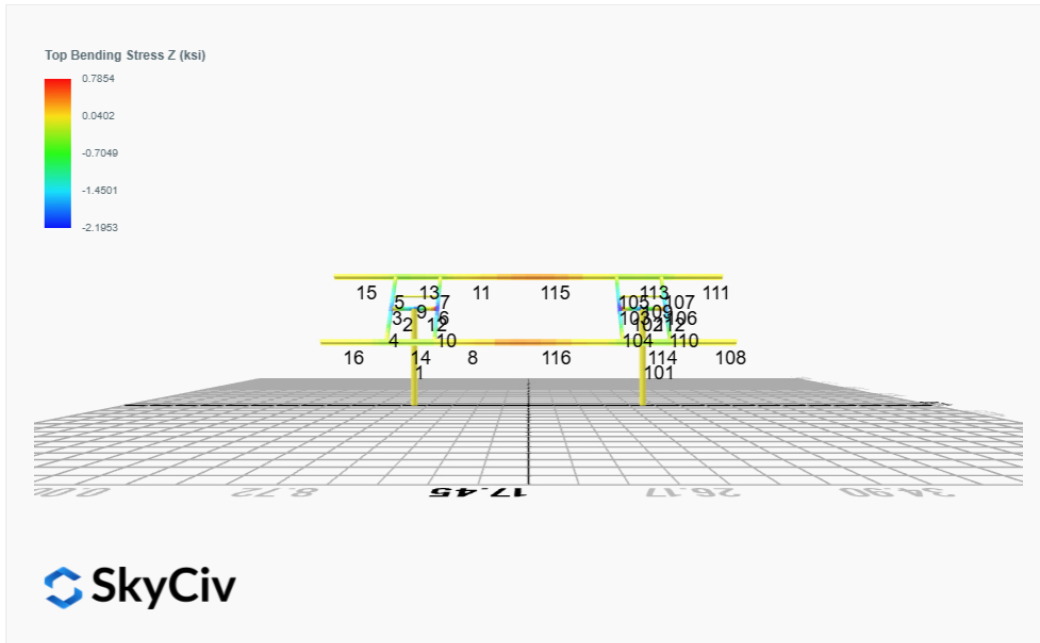
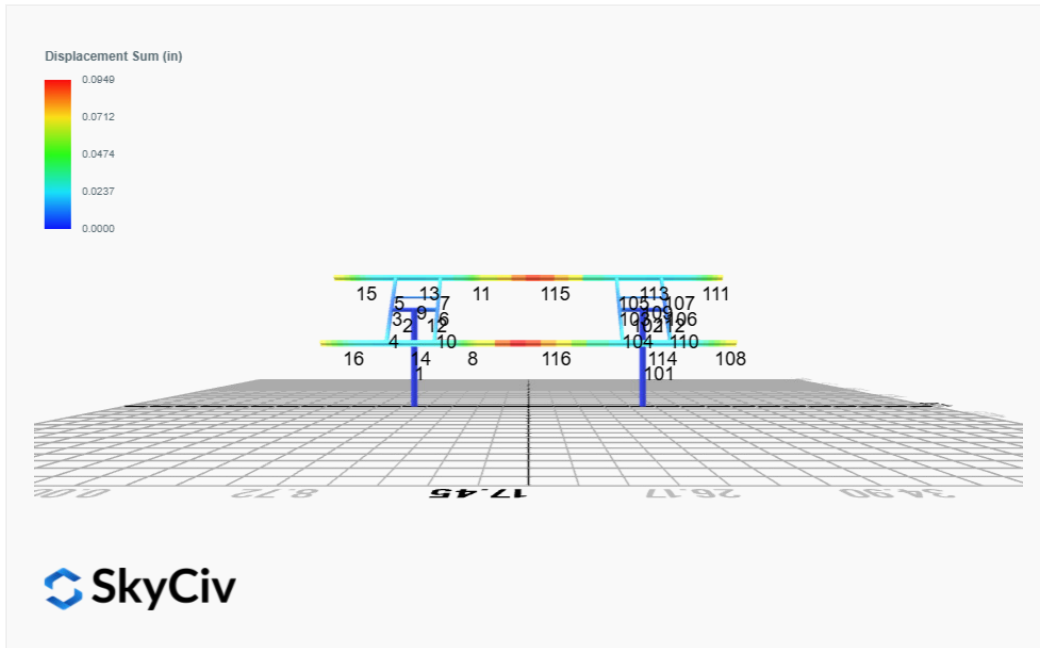
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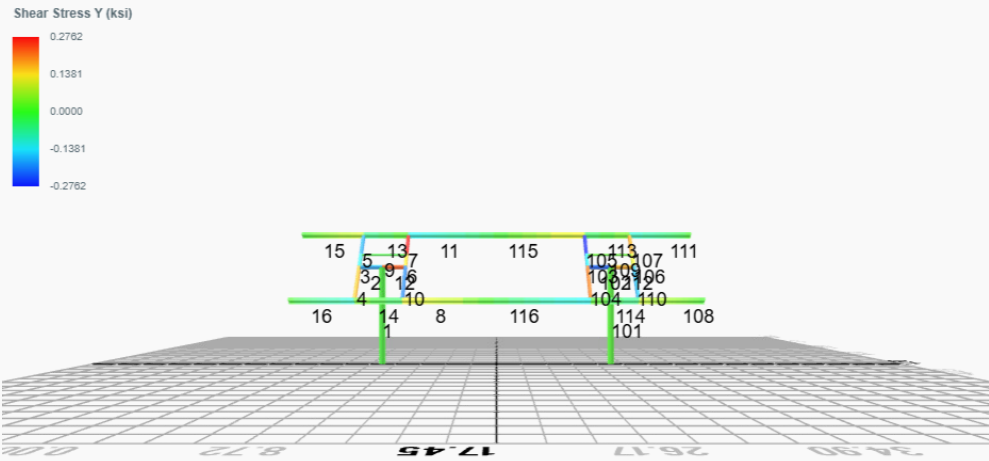
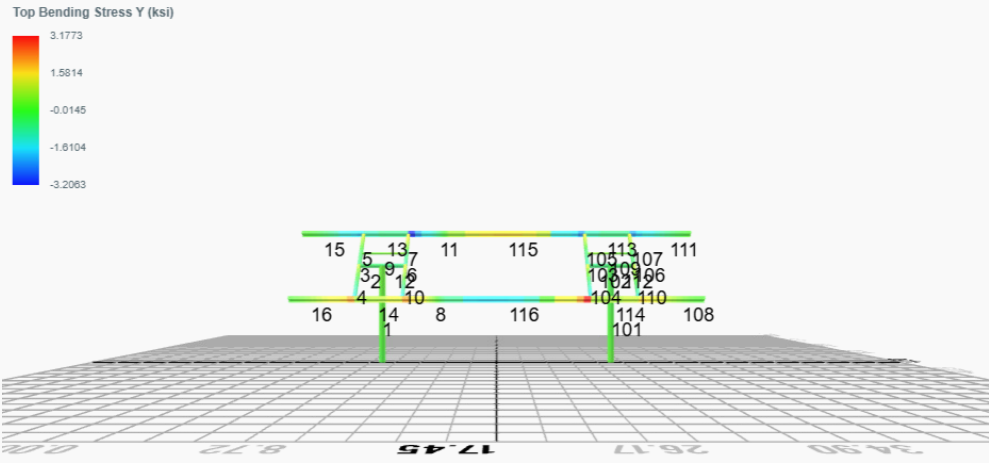
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

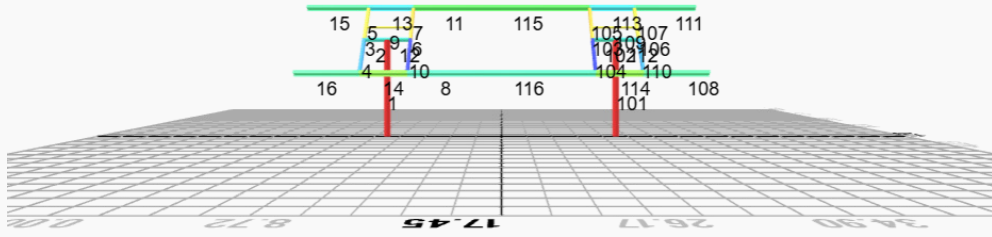
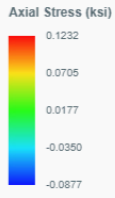




# FEM Results (Envelope Worst Case for each member)







## 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	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 2, D + L	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 3, D + (S or Lr or R)	0.0000	2.4688	0.0549	0.1349	-0.0393	0.0169
ULS: 3, D + (S or Lr or R)	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	0.0000	2.2969	0.0505	0.1241	-0.0361	0.0167
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 5b, D + 0.7E	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 6b, D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.2969	0.0505	0.1241	-0.0361	0.0167
ULS: 8, 0.6D + 0.7E	0.0000	1.0687	0.0224	0.0551	-0.0160	0.0096
ULS: 5a, D + 0.6W_Wind downforce Case A only	-2.2112	3.6365	0.1066	0.2496	-0.2453	18.7455
ULS: 5a, D + 0.6W_Wind downforce Case B only	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 5a, D + 0.6W_Wind uplift Case A only	2.2112	-0.0743	-0.0315	-0.0650	0.1917	-18.2429
ULS: 5a, D + 0.6W_Wind uplift Case B only	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6584	3.6885	0.1024	0.2424	-0.2001	14.0638
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.2969	0.0505	0.1241	-0.0361	0.0167
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6584	0.9053	-0.0012	0.0065	0.1276	-13.6775
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.2969	0.0505	0.1241	-0.0361	0.0167
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6584	3.1726	0.0893	0.2102	-0.1906	14.0631
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6584	0.3895	-0.0143	-0.0258	0.1371	-13.6782
ULS: 6a, D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.7811	0.0374	0.0919	-0.0267	0.0160
ULS: 7, 0.6D + 0.6W_Wind downforce Case A only	-2.2112	2.9241	0.0916	0.2129	-0.2346	18.7391
ULS: 7, 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0687	0.0224	0.0551	-0.0160	0.0096
ULS: 7, 0.6D + 0.6W_Wind uplift Case A only	2.2112	-0.7868	-0.0465	-0.1018	0.2023	-18.2493
ULS: 7, 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0687	0.0224	0.0551	-0.0160	0.0096

### 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	5.5735
Shear X	-3.6853
Shear Z	0.1689
Moment X	0.3950
Moment Y (Twist)	0.4020
Moment Z	31.6000

### 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	3.6885
Shear X	-2.2112
Shear Z	0.1066
Moment X	0.2496
Moment Y (Twist)	0.2453
Moment Z	18.7455

## 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	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 2, D + L	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 3, D + (S or Lr or R)	-0.0000	2.4688	-0.0549	-0.1349	0.0393	0.0169
ULS: 3, D + (S or Lr or R)	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 4, D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.2969	-0.0505	-0.1241	0.0361	0.0167

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 5b. D + 0.7E	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.2969	-0.0505	-0.1241	0.0361	0.0167
ULS: 8. 0.6D + 0.7E	-0.0000	1.0687	-0.0224	-0.0551	0.0160	0.0096
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2112	3.6365	-0.1066	-0.2496	0.2453	18.7455
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2112	-0.0743	0.0315	0.0650	-0.1917	-18.2429
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6584	3.6885	-0.1024	-0.2424	0.2001	14.0638
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.2969	-0.0505	-0.1241	0.0361	0.0167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6584	0.9053	0.0012	-0.0065	-0.1276	-13.6775
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.2969	-0.0505	-0.1241	0.0361	0.0167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6584	3.1726	-0.0893	-0.2102	0.1906	14.0631
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6584	0.3895	0.0143	0.0258	-0.1371	-13.6782
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.7811	-0.0374	-0.0919	0.0267	0.0160
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2112	2.9241	-0.0916	-0.2129	0.2346	18.7391
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.0687	-0.0224	-0.0551	0.0160	0.0096
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2112	-0.7868	0.0465	0.1018	-0.2023	-18.2493
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.0687	-0.0224	-0.0551	0.0160	0.0096

### 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	5.5735
Shear X	-3.6853
Shear Z	-0.1689
Moment X	-0.3951
Moment Y (Twist)	0.4019
Moment Z	31.6007

### 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	3.6885
Shear X	-2.2112
Shear Z	-0.1066
Moment X	-0.2496
Moment Y (Twist)	0.2453
Moment Z	18.7455

## Project Details

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

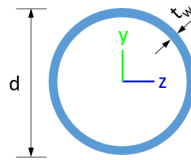


## Design Input Information

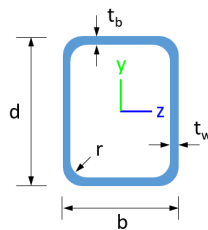
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Design Materials			
ID	E (ksi)	$F_y$ (ksi)	$F_u$ (ksi)
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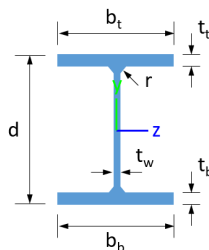
### Section Dimensions



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 <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------



113	18	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.15,1.11,1.12,1.11,1.13,1.11,1.12,1.11,1.12,1.11	300	200	1
114	18	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.11,1.11,1.15,1.11,1.11,1.11,1.13,1.11,1.12,1.11,1.12,1.11	300	200	1
115	18	6.63	6.63	10.20	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.12,1.13,1.11,1.13,1.12,1.13,1.12,1.13,1.12,1.13	300	200	1
116	18	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.12,1.12,1.12	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	131.91	42.30	42.30	75.35	75.35
2	142.83	140.22	16.17	16.17	42.85	42.85
3	79.65	74.89	10.99	6.26	29.14	16.61
4	79.65	72.84	10.99	6.26	29.14	16.61
5	79.65	74.30	10.99	6.26	29.14	16.61
6	79.65	74.89	10.99	6.26	29.14	16.61
7	79.65	74.30	10.99	6.26	29.14	16.61
8	120.60	115.40	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.84	10.99	6.26	29.14	16.61
11	120.60	115.40	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.03	19.56	6.45	30.09	45.74
14	120.60	84.03	19.51	6.45	30.09	45.74
15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	23.36	6.45	30.09	45.74
101	251.16	131.91	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.89	10.99	6.26	29.14	16.61
104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	54.44	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	54.44	23.36	6.45	30.09	45.74
112	142.83	140.22	16.17	16.17	42.85	42.85
113	120.60	84.03	19.56	6.45	30.09	45.74
114	120.60	84.03	19.51	6.45	30.09	45.74
115	120.60	68.63	15.22	6.45	30.09	45.74
116	120.60	68.63	15.21	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	$\delta$	Status
1	0.042	0.747	0.024	0.049	0.002	0.778	#13	0.469	Not Required	Pass
2	0.002	0.244	0.213	0.057	0.042	0.459	#13	0.053	Not Required	Pass
3	0.007	0.503	0.042	0.050	0.004	0.531	#13	0.044	Not Required	Pass
4	0.006	0.501	0.114	0.050	0.020	0.576	#13	0.078	Not Required	Pass

5	0.007	0.312	0.105	0.050	0.021	0.328	#13	0.073	Not Required	Pass
6	0.009	0.565	0.077	0.057	0.013	0.629	#13	0.044	Not Required	Pass
7	0.009	0.351	0.150	0.056	0.030	0.381	#13	0.073	Not Required	Pass
8	0.001	0.074	0.071	0.036	0.008	0.097	#13	0.088	Not Required	Pass
9	0.006	0.035	0.061	0.002	0.001	0.097	#13	0.198	Not Required	Pass
10	0.009	0.563	0.143	0.057	0.025	0.621	#13	0.078	Not Required	Pass
11	0.001	0.073	0.073	0.036	0.008	0.094	#13	0.088	Not Required	Pass
12	0.001	0.307	0.238	0.068	0.046	0.546	#13	0.034	Not Required	Pass
13	0.003	0.149	0.194	0.046	0.011	0.277	#21	0.265	Not Required	Pass
14	0.004	0.152	0.191	0.046	0.011	0.273	#21	0.177	Not Required	Pass
15	0.000	0.053	0.069	0.022	0.005	0.106	#21	Not Required	Not Required	Pass
16	0.000	0.053	0.069	0.022	0.005	0.106	#21	Not Required	Not Required	Pass
101	0.042	0.747	0.024	0.049	0.002	0.778	#13	0.469	Not Required	Pass
102	0.001	0.307	0.238	0.068	0.046	0.546	#13	0.034	Not Required	Pass
103	0.009	0.565	0.077	0.057	0.013	0.629	#13	0.044	Not Required	Pass
104	0.009	0.563	0.143	0.057	0.025	0.621	#13	0.078	Not Required	Pass
105	0.009	0.350	0.150	0.056	0.030	0.381	#13	0.073	Not Required	Pass
106	0.007	0.503	0.042	0.050	0.004	0.531	#13	0.044	Not Required	Pass
107	0.007	0.312	0.105	0.050	0.021	0.328	#13	0.073	Not Required	Pass
108	0.000	0.053	0.069	0.022	0.005	0.106	#21	Not Required	Not Required	Pass
109	0.006	0.035	0.061	0.002	0.001	0.097	#13	0.198	Not Required	Pass
110	0.006	0.501	0.114	0.050	0.020	0.576	#13	0.078	Not Required	Pass
111	0.000	0.053	0.069	0.022	0.005	0.106	#21	Not Required	Not Required	Pass
112	0.002	0.244	0.213	0.057	0.042	0.459	#13	0.053	Not Required	Pass
113	0.003	0.149	0.194	0.046	0.011	0.277	#21	0.177	Not Required	Pass
114	0.004	0.152	0.191	0.046	0.011	0.273	#21	0.265	Not Required	Pass
115	0.002	0.207	0.112	0.036	0.008	0.289	#13	0.439	Not Required	Pass
116	0.001	0.208	0.112	0.036	0.008	0.291	#13	0.439	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 <sub>z</sub> ,M <sub>y</sub> )	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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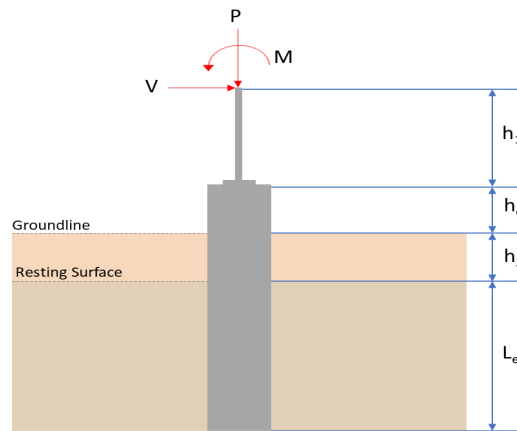
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5.5$  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)	3.688	5.574
$V_x$ (kip)	-2.211	-3.685
$V_z$ (kip)	0.107	0.169
$M_x$ (kipft)	0.250	0.395
$M_z$ (kipft)	18.746	31.600

### 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,

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

$$H_o = \frac{(-2.211 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.35207 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.746 \text{ kipft}) + ((-2.211 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.985 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.084 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.107 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.017038 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.25 \text{ kipft}) + ((0.107 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.039809 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7013 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.084 \text{ ft}), (1.7013 \text{ ft})]$$

$$L_{e,req} = 5.084 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.084 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.92436$$

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

**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

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

$$q = \frac{(3.688 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2305 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.2305 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.11525$$

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

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:**

$H_o = -0.35207 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.985 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.985 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.985 \text{ kipft/ft})) + (4 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.805 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.985 \text{ kipft/ft})) + (3 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.985 \text{ kipft/ft})) + (2 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.18337 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.985 \text{ kipft/ft})) + ((-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.80007 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.805 \text{ ft})}{2}$$

$$p_a = 0.28538 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18337 \text{ kip/ft}^2)}{(0.28538 \text{ kip/ft}^2)}$$

$$Ratio = 0.64256$$

$p_s$  - 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

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

$$Ratio = \frac{(0.80007 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.96978$$

Status: **PASS**  
Ratio: **0.640**

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

#### Considering z-direction:

$H_o = 0.017038 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.039809 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.039809 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.017038 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.039809 \text{ kipft/ft})) + (4 \times (0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9466 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.039809 \text{ kipft/ft})) + (3 \times (0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 [(3 \times (0.039809 \text{ kipft/ft})) + (2 \times (0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.015669 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 [(2 \times (0.039809 \text{ kipft/ft})) + ((0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.034379 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9466 \text{ ft})}{2}$$

$$p_a = 0.296 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.015669 \text{ kip/ft}^2)}{(0.296 \text{ kip/ft}^2)}$$

$$Ratio = 0.052937$$

$p_s$  - 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

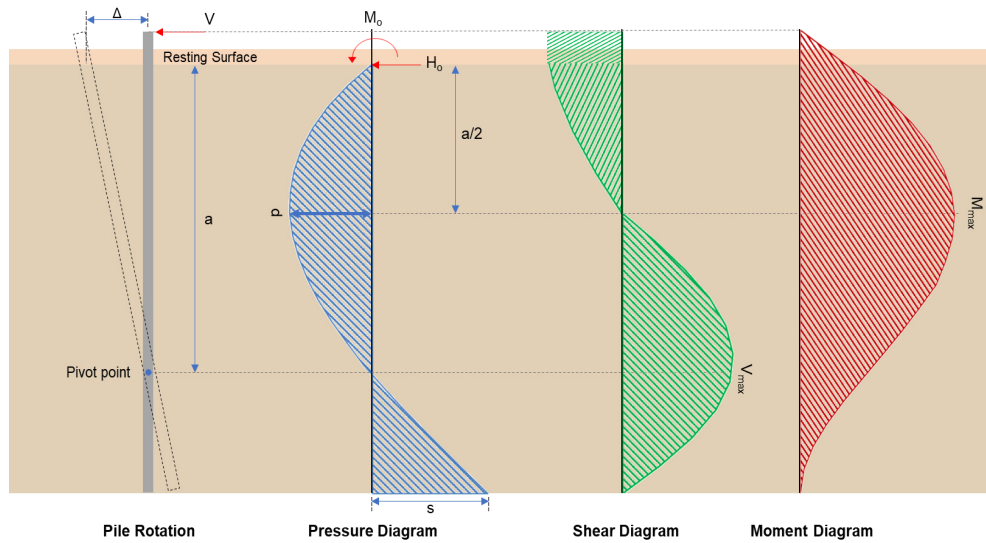
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.034379 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.041672$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.685 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58678 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(31.6 \text{ kipft}) + ((-3.685 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0318 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.0318 \text{ kipft/ft})}{(-0.58678 \text{ kip/ft})}$$

$$E = 8.5753 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.0318 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.58678 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times 5.0318) + (4 \times (-0.58678) \times 5.5)}$$

$$a = \frac{(-0.58678 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.0318 \text{ kipft/ft})) + (4 \times (-0.58678 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8039 \text{ ft}$$

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

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

$$V_{max} = ((-0.58678 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.5753 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8039 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.5753 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8039 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0232 \text{ kip}$$

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

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

$$M_{max} = ((-0.58678 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(8.5753 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8039 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.5753 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8039 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (8.5753 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8039 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 20.893 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.026911 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.395 \text{ kipft}) + ((0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.062898 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.062898 \text{ kipft/ft})}{(0.026911 \text{ kip/ft})}$$

$$E = 2.3373 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.062898 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.026911 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.062898 \text{ kipft/ft})) + (4 \times (0.026911 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9466 \text{ ft}$$

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

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

$$V_{max} = ((0.026911 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9466 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9466 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.15284 \text{ kip}$$

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

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

$$M_{max} = ((0.026911 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(2.3373 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.36763 \text{ kipft}$$

### 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,  $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.574 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.411 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.411 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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} = Minimum\ spacing\ of\ reinforcement,</math></p> $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625\ in))]$ $s_{rebar} = 1.5\ in$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq No. 10\phi</math>: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625\ in)), (48 \times (0.375\ in)), Min((48\ in), (48\ in))]$ $s_{ties} = 10\ in$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5\ ksi) \times [(2304\ in^2) - (4.2951\ in^2)]) + ((60\ ksi) \times (4.2951\ in^2))]$ $\phi P_N = 2675.2\ kip$ <p><i>Ratio - Capacity</i></p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(5.574\ kip)}{(2675.2\ kip)}$ $Ratio = 0.0020836$	<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\ in</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48\ in)$ $d = 38.4\ in$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4\ in)}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5\ ksi \rightarrow 2500\ psi</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ psi)} \times (48\ in) \times (38.4\ in)$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.574 \text{ kip} \rightarrow 5574 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5574 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.23 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.23 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.23 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - 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

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

$$\phi V_n = (0.65) \times ((119.23 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.58 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.0232 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.0232 \text{ kip})}{(110.58 \text{ kip})}$$

$$Ratio = 0.072556$$

**Considering z-direction:**

$V_{max} = 0.15284 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.15284 \text{ kip})}{(110.58 \text{ kip})}$$

$$Ratio = 0.0013822$$

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

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:**

$M_{max} = 20.893 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(20.893 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.083707$$

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

**Considering z-direction:**

$M_{max} = 0.36763 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(0.36763 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0014729$$

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

REFERENCES	CALCULATIONS	RESULTS
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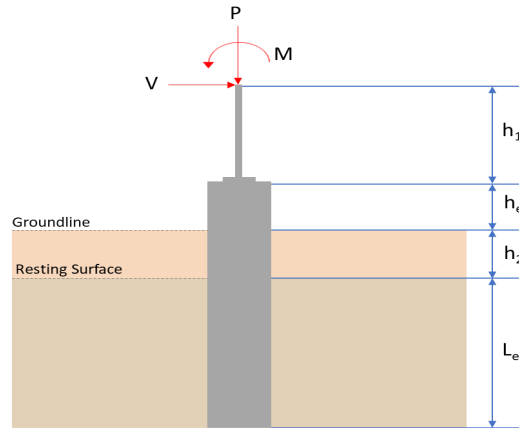
## SkyCiv Foundation Design

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)	3.688	5.574
$V_x$ (kip)	-2.211	-3.685
$V_z$ (kip)	-0.107	-0.169
$M_x$ (kipft)	-0.250	-0.395
$M_z$ (kipft)	18.746	31.601

### 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,

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

$$H_o = \frac{(-2.211 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.35207 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.746 \text{ kipft}) + ((-2.211 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.985 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.084 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.107 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.017038 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.25 \text{ kipft}) + ((-0.107 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.039809 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2418 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.084 \text{ ft}), (1.2418 \text{ ft})]$$

$$L_{e,req} = 5.084 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.084 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.92436$$

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

**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

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

$$q = \frac{(3.688 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2305 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.2305 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.11525$$

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

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:**

$H_o = -0.35207 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.985 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.985 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.985 \text{ kipft/ft})) + (4 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.805 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.985 \text{ kipft/ft})) + (3 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.985 \text{ kipft/ft})) + (2 \times (-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.18337 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.985 \text{ kipft/ft})) + ((-0.35207 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.80007 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.805 \text{ ft})}{2}$$

$$p_a = 0.28538 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18337 \text{ kip/ft}^2)}{(0.28538 \text{ kip/ft}^2)}$$

$$Ratio = 0.64256$$

$p_s$  - 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

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

$$Ratio = \frac{(0.80007 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.96978$$

Status: **PASS**  
Ratio: **0.640**

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

#### Considering z-direction:

$H_o = -0.017038 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.039809 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.039809 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.017038 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.039809 \text{ kipft/ft})) + (4 \times (-0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9466 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.039809 \text{ kipft/ft})) + (3 \times (-0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.039809 \text{ kipft/ft})) + (2 \times (-0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.005418 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.039809 \text{ kipft/ft})) + ((-0.017038 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = -0.0027952 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9466 \text{ ft})}{2}$$

$$p_a = 0.296 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.005418 \text{ kip/ft}^2)}{(0.296 \text{ kip/ft}^2)}$$

$$Ratio = -0.018304$$

$p_s$  - 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

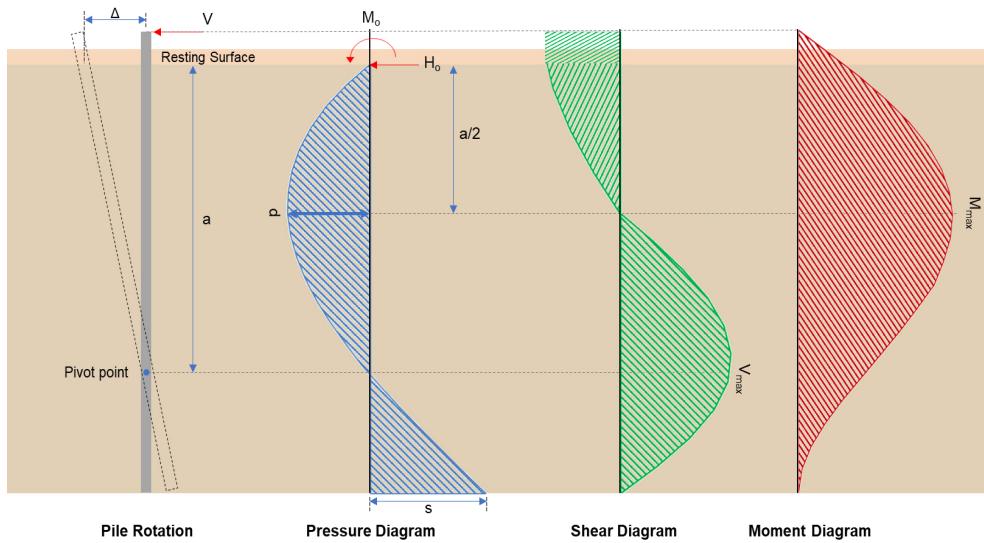
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0027952 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = -0.0033881$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.685 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58678 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(31.601 \text{ kipft}) + ((-3.685 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.032 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.032 \text{ kipft/ft})}{(-0.58678 \text{ kip/ft})}$$

$$E = 8.5756 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.032 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.58678 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (5.032 \text{ kipft/ft})) + (4 \times (-0.58678 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = \frac{(-0.58678 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.032 \text{ kipft/ft})) + (4 \times (-0.58678 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8039 \text{ ft}$$

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

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

$$V_{max} = ((-0.58678 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.5756 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8039 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.5756 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8039 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0234 \text{ kip}$$

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

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

$$M_{max} = ((-0.58678 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(8.5756 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8039 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.5756 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.8039 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (8.5756 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.8039 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 20.894 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026911 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.395 \text{ kipft}) + ((-0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.062898 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.062898 \text{ kipft/ft})}{(-0.026911 \text{ kip/ft})}$$

$$E = 2.3373 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.062898 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.026911 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.062898 \text{ kipft/ft})) + (4 \times (-0.026911 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9466 \text{ ft}$$

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

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

$$V_{max} = ((-0.026911 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9466 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9466 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.15284 \text{ kip}$$

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

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

$$M_{max} = ((-0.026911 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[ \left( \frac{(2.3373 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.3373 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.9466 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.36763 \text{ kipft}$$

### 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,  $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.574 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.411 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.411 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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 <math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> <p>25.7.2.2 Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p>25.7.2.1 <math>s_{ties}</math> - Maximum spacing of ties,</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p style="text-align: center;"><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p style="text-align: center;"><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <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 <math>\phi P_N</math> - Allowable axial compressive strength</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]</math></p> <p style="text-align: center;"><math>\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))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(5.574 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0020836</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2 <math>b_w</math> = 48 in - Effective width, <math>d</math> - Effective depth</p> <p>22.5.5.1.3 <math>\lambda_s</math> - size effect modification factor</p> <p>22.5.5.1.1 <math>V_{c,max}</math> - Max shear strength of concrete</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <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 style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.574 \text{ kip} \rightarrow 5574 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5574 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.23 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.23 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.23 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - 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

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

$$\phi V_n = (0.65) \times ((119.23 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.58 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.0234 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.0234 \text{ kip})}{(110.58 \text{ kip})}$$

$$Ratio = 0.072558$$

**Considering z-direction:**

$V_{max} = 0.15284 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.15284 \text{ kip})}{(110.58 \text{ kip})}$$

$$Ratio = 0.0013822$$

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

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:**

$M_{max} = 20.894 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(20.894 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.083709$$

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

**Considering z-direction:**

$M_{max} = 0.36763 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(0.36763 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0014729$$

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