

# Your Project Calculations



Project Name: LanoEquipment-JB-RevB

S3D Model Link:  
[https://platform.skyciv.com/structural?preload\\_name=LanoEquipment-JB-RevB&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/7\\_2023](https://platform.skyciv.com/structural?preload_name=LanoEquipment-JB-RevB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023)

Public Model Link:  
[https://platform.skyciv.com/structural-viewer?project\\_id=OYDqpSuFXz3j8ymaAcTAJjTHJ1AcwSXMtrWfOS3FfhivETwVWVpuF0N7RSiww0W](https://platform.skyciv.com/structural-viewer?project_id=OYDqpSuFXz3j8ymaAcTAJjTHJ1AcwSXMtrWfOS3FfhivETwVWVpuF0N7RSiww0W)

## Array Specification

Product:	Beam
Unique ID:	3P-19.75-6TOP-SD-57-L-4Hx8W-E5JE
Duty Classification:	SD
Module Width:	41.14 in
Module Length:	87.24in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Desired Tilt Angle:	46
Front Edge Clearance:	5
Total Array Height at Tilt:	14.92 ft
Total Frame Length:	56.50 ft
Frame Weight:	2332 lbs
Array Dimensions N/S:	13.88 ft
Array Dimensions E/W:	58.83 ft
Rail Length:	166.56 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

## Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	9.99 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

## Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.00 ft Pile 3: 6.00 ft
Foundation Volume:	10.667 y <sup>3</sup>
Foundation Result:	PASSED
Mount Twist:	0.120903 kip

## Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1015 US-212, Norwood Young America, MN 55368, USA
Wind Speed:	102 mph
Snow Load:	50 psf
Design Uplift Pressure:	0.018991 ksf
Design Downforce Pressure:	-0.018991 ksf
Design Snow Pressure:	0.013196 ksf



### Design Disclaimer

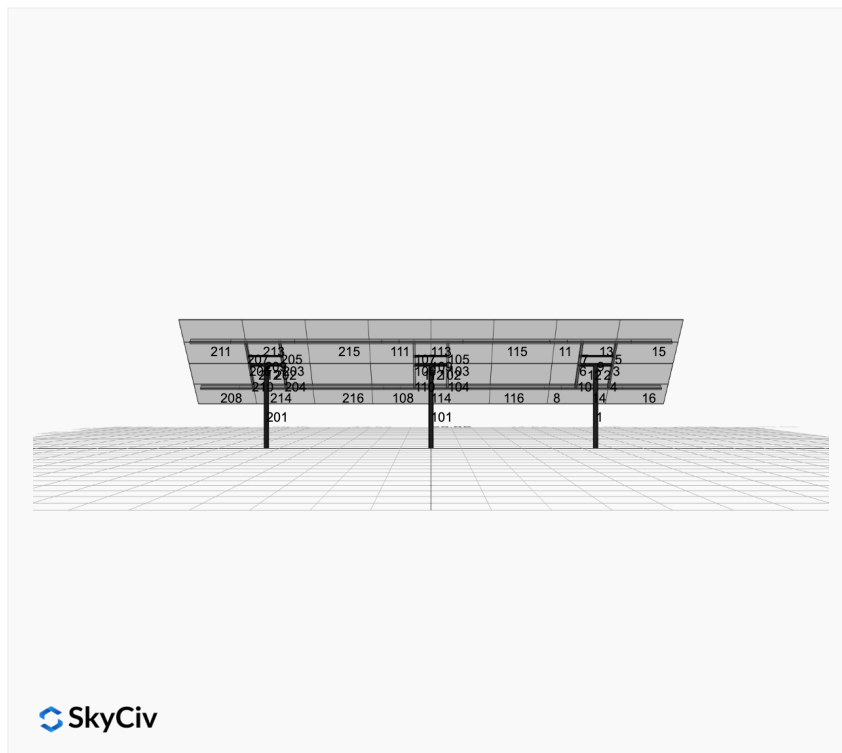
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

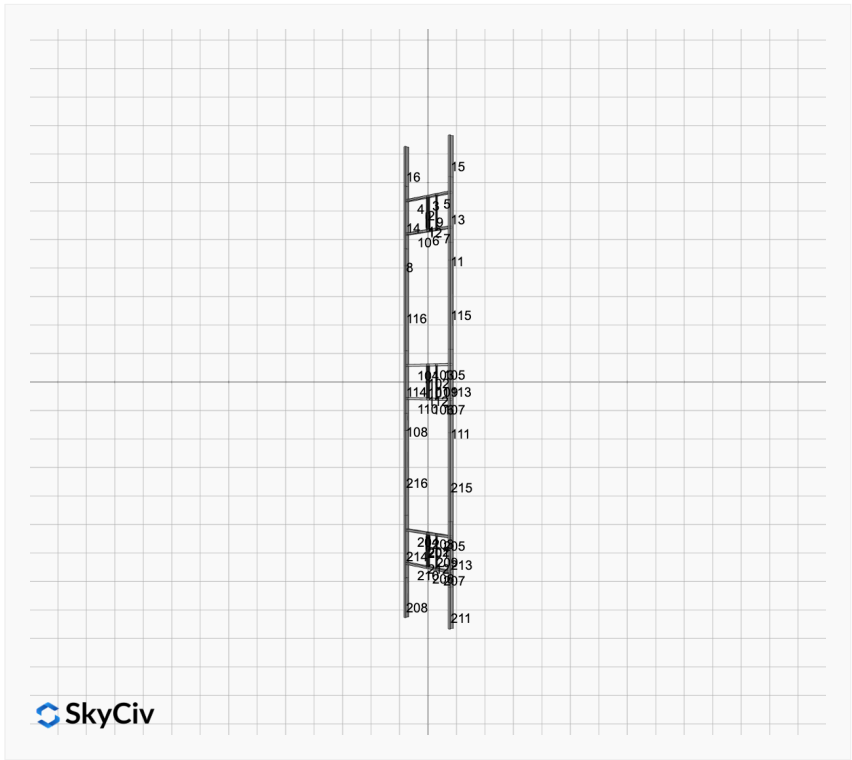
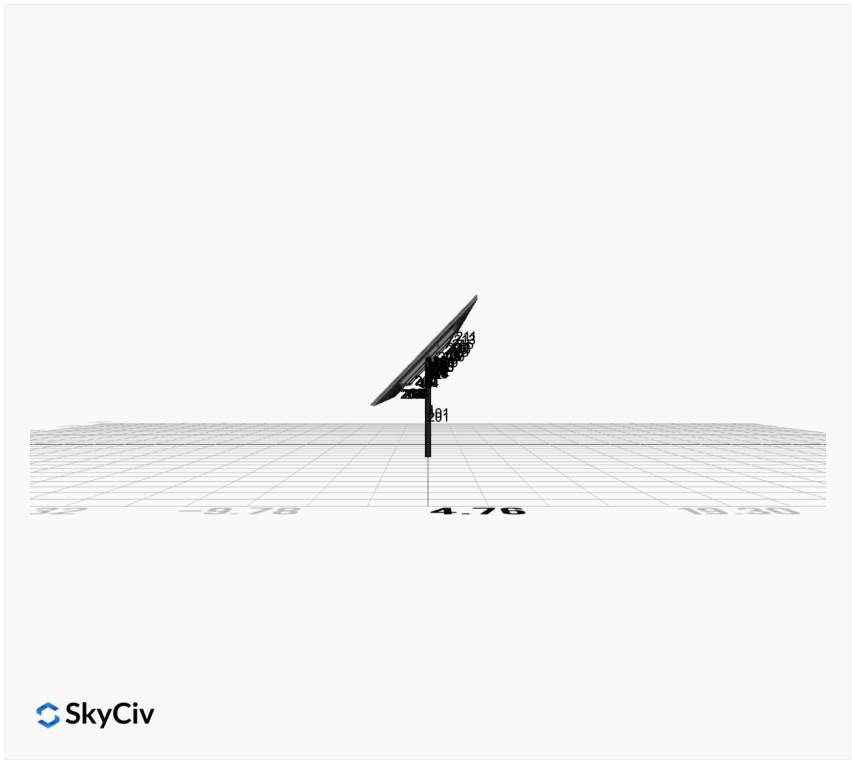
### AutoDesigner Input

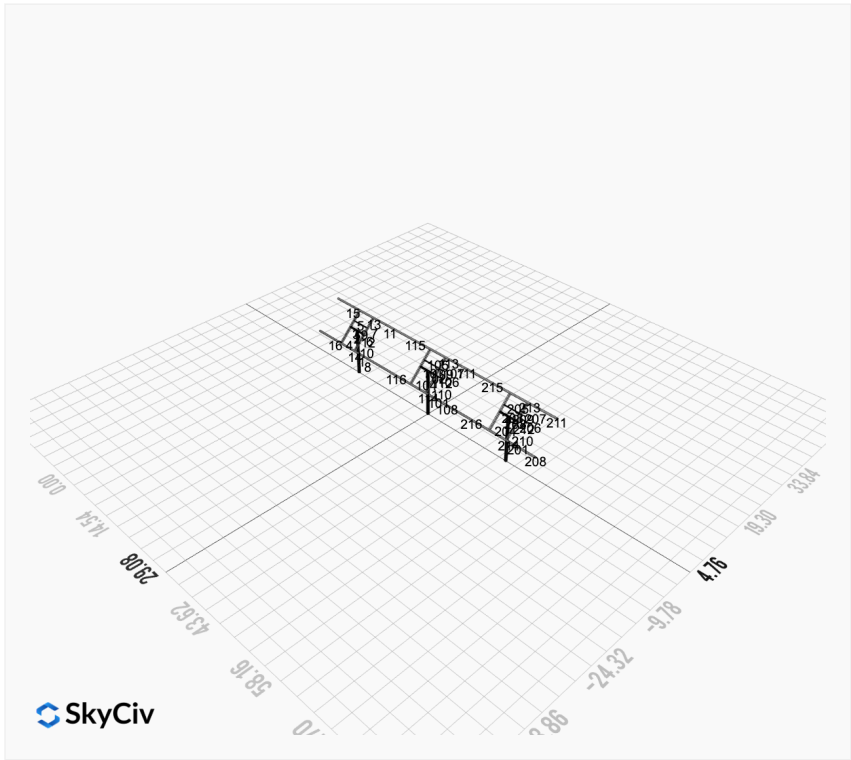
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  "adjuster_section": "2_40",
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  "tilt_angle": 46,
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  "exposure_category": "C",
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  "pole_override": "auto",
  "soil_type": "sand",
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### Design Notes:

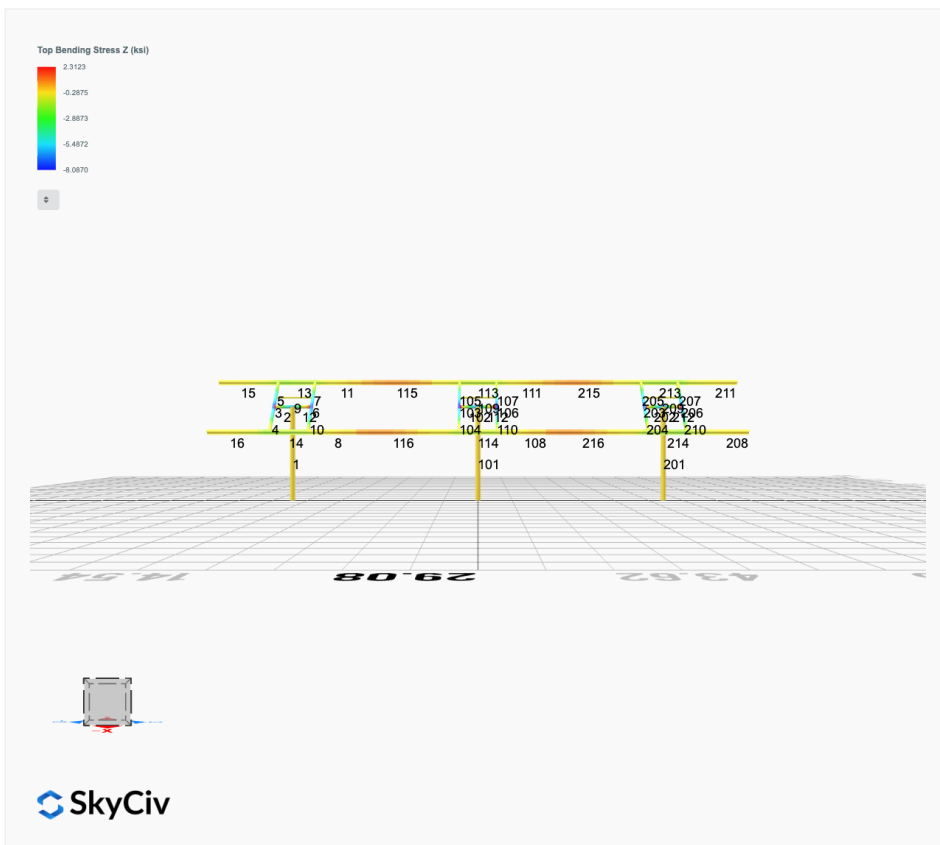
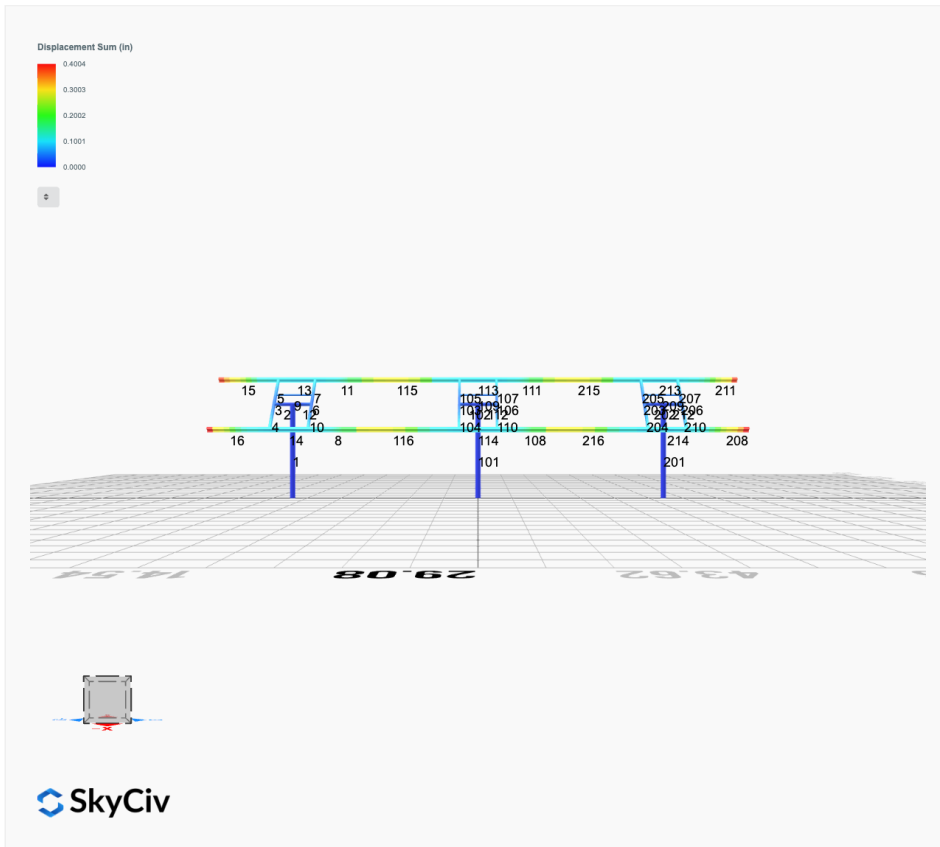
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



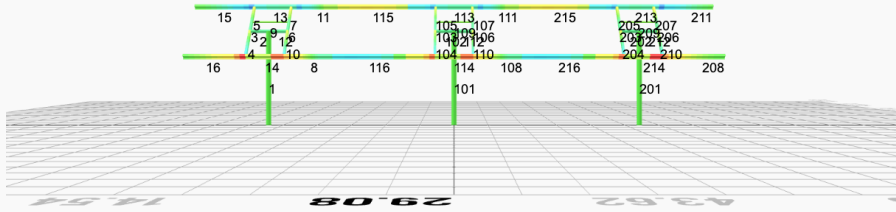
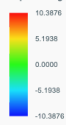




# FEM Results (Envelope Worst Case for each member)

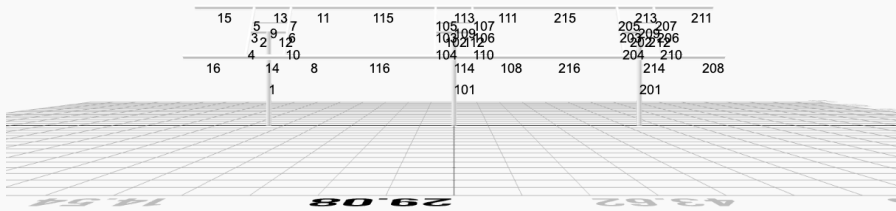


Top Bending Stress Y (ksi)

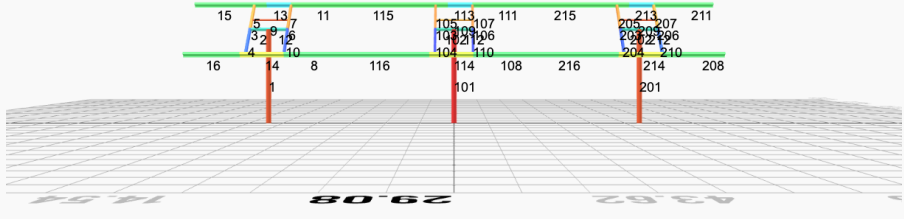
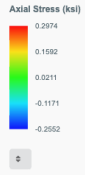


SkyCiv

Shear Stress Y (ksi)



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	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 2. D + L	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 3. D + (S or Lr or R)	-0.0103	4.3363	-0.0045	-0.0073	0.0829	0.1204
ULS: 3. D + (S or Lr or R)	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0088	3.7502	-0.0038	-0.0063	0.0705	0.1039
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 5b. D + 0.7E	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0088	3.7502	-0.0038	-0.0063	0.0705	0.1039
ULS: 8. 0.6D + 0.7E	-0.0025	1.1951	-0.0011	-0.0019	0.0200	0.0328
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2063	4.1115	-0.0046	-0.0113	0.0364	22.4104
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1974	-0.1275	0.0014	0.0065	0.0280	-21.6578
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6604	5.3399	-0.0059	-0.0124	0.0729	16.8708
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0088	3.7502	-0.0038	-0.0063	0.0705	0.1039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6424	2.1606	-0.0014	0.0009	0.0666	-16.1804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0088	3.7502	-0.0038	-0.0063	0.0705	0.1039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6557	3.5815	-0.0039	-0.0093	0.0356	16.8214
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6470	0.4023	0.0006	0.0041	0.0294	-16.2297
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0041	1.9918	-0.0018	-0.0031	0.0333	0.0546
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2046	3.3147	-0.0039	-0.0101	0.0231	22.3886
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0025	1.1951	-0.0011	-0.0019	0.0200	0.0328
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1990	-0.9243	0.0022	0.0077	0.0147	-21.6796
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0025	1.1951	-0.0011	-0.0019	0.0200	0.0328

### 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	7.9079
Shear X	-3.6789
Shear Z	-0.0085
Moment X	-0.0185
Moment Y (Twist)	0.1210
Moment Z	37.9848

### 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	5.3399
Shear X	-2.2063
Shear Z	-0.0059
Moment X	-0.0124
Moment Y (Twist)	0.0829
Moment Z	22.4104

## 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	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 2. D + L	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 3. D + (S or Lr or R)	0.0206	4.5943	0.0000	0.0000	0.0000	-0.1565
ULS: 3. D + (S or Lr or R)	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0175	3.9694	-0.0000	0.0000	0.0000	-0.1315
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 5b. D + 0.7E	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0175	3.9694	-0.0000	0.0000	0.0000	-0.1315
ULS: 8. 0.6D + 0.7E	0.0050	1.2568	-0.0000	0.0000	0.0000	-0.0339
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2799	4.3183	-0.0000	0.0000	0.0000	23.1024
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2977	-0.1293	-0.0000	0.0000	0.0000	-22.5410
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6986	5.6371	-0.0000	0.0000	0.0000	17.2377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0175	3.9694	-0.0000	0.0000	0.0000	-0.1315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7346	2.3014	-0.0000	0.0000	0.0000	-16.9948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0175	3.9694	-0.0000	0.0000	0.0000	-0.1315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7079	3.7624	-0.0000	0.0000	0.0000	17.3126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7254	0.4267	-0.0000	0.0000	0.0000	-16.9198
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0083	2.0947	-0.0000	0.0000	0.0000	-0.0565
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2833	3.4804	-0.0000	0.0000	0.0000	23.1250
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0050	1.2568	-0.0000	0.0000	0.0000	-0.0339
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2944	-0.9672	-0.0000	0.0000	0.0000	-22.5184
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0050	1.2568	-0.0000	0.0000	0.0000	-0.0339

#### 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	8.3657
Shear X	-3.8307
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	39.1371

#### 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	5.6371
Shear X	-2.2977
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	23.1250

#### Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

##### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 2. D + L	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 3. D + (S or Lr or R)	-0.0103	4.3363	0.0045	0.0073	-0.0829	0.1204
ULS: 3. D + (S or Lr or R)	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0088	3.7502	0.0038	0.0063	-0.0705	0.1039
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 5b. D + 0.7E	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0088	3.7502	0.0038	0.0063	-0.0705	0.1039
ULS: 8. 0.6D + 0.7E	-0.0025	1.1951	0.0011	0.0019	-0.0200	0.0328
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2063	4.1115	0.0046	0.0113	-0.0364	22.4104
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1974	-0.1275	-0.0014	-0.0065	-0.0280	-21.6578
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6604	5.3399	0.0059	0.0124	-0.0729	16.8708
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0088	3.7502	0.0038	0.0063	-0.0705	0.1039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6424	2.1607	0.0014	-0.0009	-0.0666	-16.1803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0088	3.7502	0.0038	0.0063	-0.0705	0.1039

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6557	3.5815	0.0039	0.0093	-0.0356	16.8215
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6470	0.4023	-0.0006	-0.0041	-0.0293	-16.2297
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0041	1.9918	0.0018	0.0031	-0.0333	0.0546
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2046	3.3147	0.0039	0.0101	-0.0231	22.3886
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0025	1.1951	0.0011	0.0019	-0.0200	0.0328
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1990	-0.9243	-0.0022	-0.0077	-0.0147	-21.6796
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0025	1.1951	0.0011	0.0019	-0.0200	0.0328

### 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	7.9079
Shear X	-3.6789
Shear Z	0.0085
Moment X	0.0185
Moment Y (Twist)	0.1209
Moment Z	37.9854

### 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	5.3399
Shear X	-2.2063
Shear Z	0.0059
Moment X	0.0124
Moment Y (Twist)	0.0829
Moment Z	22.4104

## Project Details

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

User Name: sales@mtsolar.us  
 Project Name: LanoEquipment-JB-RevB  
 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 <sub>y</sub> (ksi)	F <sub>u</sub> (ksi)
1	29000	50	65

**Section Dimensions**

ID	Name	d (in)	t <sub>w</sub> (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
8	6in Pipe Sch 80	6.63	0.43				

ID	Name	d (in)	b (in)	t <sub>w</sub> (in)	t <sub>b</sub> (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	

ID	Name	d (in)	t <sub>w</sub> (in)	b <sub>t</sub> (in)	b <sub>b</sub> (in)	t <sub>t</sub> (in)	t <sub>b</sub> (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 <sub>yp</sub> (in <sup>4</sup> )	I <sub>zp</sub> (in <sup>4</sup> )	I <sub>w</sub> (in <sup>6</sup> )	S <sub>yp</sub> (in <sup>3</sup> )	S <sub>zp</sub> (in <sup>3</sup> )
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31





## Member Design Capacity

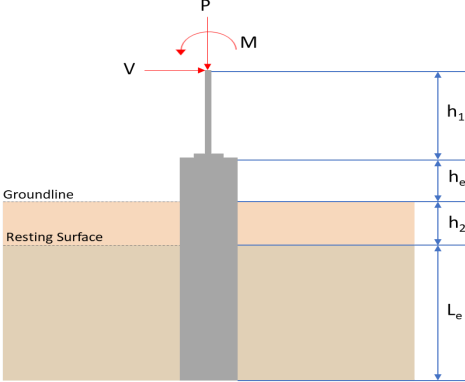
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	144.27	62.23	62.23	113.47	113.47
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.19	6.45	30.09	45.74
14	120.60	98.23	19.19	6.45	30.09	45.74
15	120.60	34.69	23.36	6.45	30.09	45.74
16	120.60	34.69	23.36	6.45	30.09	45.74
101	378.22	144.27	62.23	62.23	113.47	113.47
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.13	6.45	30.09	45.74
114	120.60	98.23	18.31	6.45	30.09	45.74
115	120.60	68.63	15.30	6.45	30.09	45.74
116	120.60	68.63	15.57	6.45	30.09	45.74
201	378.22	144.27	62.23	62.23	113.47	113.47
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	34.69	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	34.69	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	19.19	6.45	30.09	45.74
214	120.60	98.23	19.19	6.45	30.09	45.74
215	120.60	68.63	15.71	6.45	30.09	45.74
216	120.60	68.63	15.98	6.45	30.09	45.74

## 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	δ	Status
1	0.055	0.610	0.001	0.032	0.000	0.635	#13	0.574	Not Required	Pass
2	0.004	0.372	0.241	0.088	0.043	0.593	#13	0.034	Not Required	Pass
3	0.014	0.627	0.121	0.063	0.009	0.700	#13	0.044	Not Required	Pass
4	0.013	0.624	0.337	0.063	0.041	0.783	#21	0.078	Not Required	Pass
5	0.013	0.389	0.355	0.062	0.051	0.434	#21	0.073	Not Required	Pass
6	0.014	0.625	0.133	0.063	0.011	0.703	#13	0.044	Not Required	Pass
7	0.014	0.388	0.359	0.062	0.053	0.443	#21	0.073	Not Required	Pass
8	0.000	0.046	0.126	0.040	0.015	0.161	#21	0.059	Not Required	Pass
9	0.016	0.049	0.057	0.001	0.000	0.110	#13	0.198	Not Required	Pass
10	0.013	0.618	0.345	0.062	0.043	0.796	#21	0.078	Not Required	Pass
11	0.000	0.045	0.129	0.041	0.015	0.164	#21	0.088	Not Required	Pass
12	0.004	0.368	0.236	0.089	0.043	0.585	#13	0.034	Not Required	Pass
13	0.007	0.221	0.359	0.052	0.019	0.550	#21	0.265	Not Required	Pass
14	0.007	0.223	0.359	0.052	0.019	0.550	#21	0.177	Not Required	Pass
15	0.000	0.094	0.192	0.031	0.011	0.276	#21	Not Required	Not Required	Pass
16	0.000	0.094	0.192	0.031	0.011	0.276	#21	Not Required	Not Required	Pass
101	0.058	0.629	0.000	0.034	0.000	0.655	#13	0.574	Not Required	Pass
102	0.003	0.389	0.243	0.094	0.045	0.608	#13	0.034	Not Required	Pass
103	0.015	0.653	0.147	0.066	0.016	0.745	#21	0.044	Not Required	Pass
104	0.014	0.653	0.329	0.066	0.041	0.822	#21	0.078	Not Required	Pass
105	0.015	0.406	0.343	0.065	0.050	0.452	#13	0.073	Not Required	Pass
106	0.015	0.653	0.147	0.066	0.016	0.745	#21	0.044	Not Required	Pass
107	0.015	0.406	0.343	0.065	0.050	0.452	#13	0.073	Not Required	Pass
108	0.000	0.056	0.125	0.039	0.015	0.152	#21	0.059	Not Required	Pass
109	0.013	0.044	0.055	0.001	0.000	0.103	#13	0.198	Not Required	Pass
110	0.014	0.653	0.329	0.066	0.041	0.822	#21	0.078	Not Required	Pass
111	0.000	0.058	0.127	0.038	0.015	0.154	#21	0.088	Not Required	Pass
112	0.003	0.389	0.243	0.094	0.045	0.608	#13	0.034	Not Required	Pass
113	0.007	0.172	0.336	0.050	0.019	0.484	#21	0.265	Not Required	Pass
114	0.007	0.181	0.334	0.050	0.019	0.483	#21	0.177	Not Required	Pass
115	0.000	0.179	0.193	0.038	0.015	0.353	#21	0.439	Not Required	Pass
116	0.000	0.178	0.195	0.039	0.015	0.354	#21	0.293	Not Required	Pass
201	0.055	0.610	0.001	0.032	0.000	0.635	#13	0.574	Not Required	Pass
202	0.004	0.368	0.236	0.089	0.043	0.585	#13	0.034	Not Required	Pass
203	0.014	0.625	0.133	0.063	0.011	0.703	#13	0.044	Not Required	Pass
204	0.013	0.618	0.345	0.062	0.043	0.796	#21	0.078	Not Required	Pass
205	0.014	0.388	0.359	0.062	0.053	0.443	#21	0.073	Not Required	Pass
206	0.014	0.627	0.121	0.063	0.009	0.700	#13	0.044	Not Required	Pass
207	0.013	0.389	0.355	0.062	0.051	0.434	#21	0.073	Not Required	Pass
208	0.000	0.094	0.192	0.031	0.011	0.276	#21	Not Required	Not Required	Pass
209	0.016	0.049	0.057	0.001	0.000	0.110	#13	0.198	Not Required	Pass
210	0.013	0.624	0.337	0.063	0.041	0.783	#21	0.078	Not Required	Pass
211	0.000	0.094	0.192	0.031	0.011	0.276	#21	Not Required	Not Required	Pass
212	0.004	0.372	0.241	0.088	0.043	0.593	#13	0.034	Not Required	Pass
213	0.007	0.221	0.359	0.052	0.019	0.550	#21	0.177	Not Required	Pass
214	0.007	0.223	0.359	0.052	0.019	0.550	#21	0.177	Not Required	Pass
215	0.000	0.177	0.193	0.041	0.015	0.350	#21	0.439	Not Required	Pass
216	0.000	0.176	0.195	0.040	0.015	0.352	#21	0.293	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																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.340</td> <td>7.908</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.206</td> <td>-3.679</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.006</td> <td>-0.008</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.012</td> <td>-0.018</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>22.410</td> <td>37.985</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	5.340	7.908	$V_x$ (kip)	-2.206	-3.679	$V_z$ (kip)	-0.006	-0.008	$M_x$ (kipft)	-0.012	-0.018	$M_z$ (kipft)	22.410	37.985	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.206 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.35127 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(22.41 \text{ kipft}) + ((-2.206 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.5685 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5283 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.006 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00095541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.012 \text{ kipft}) + ((-0.006 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0019108 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.49876 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.5283 \text{ ft}), (0.49876 \text{ ft})]$$

$$L_{e,req} = 5.528 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.528 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92133$$

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

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

$$q = \frac{(5.34 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33375 \text{ kip/ft}^2$$

$$q = 0.33375 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16687$$

Status: **PASS**  
Ratio: **0.170**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.35127 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.5685 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.5685 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.5685 \text{ kipft/ft})) + (4 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1413 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.5685 \text{ kipft/ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.5685 \text{ kipft/ft})) + (2 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.20293 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.5685 \text{ kipft/ft})) + ((-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.83822 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1413 \text{ ft})}{2}$$

$$p_a = 0.31059 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20293 \text{ kip/ft}^2)}{(0.31059 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65336$$

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

Status: **PASS**  
Ratio: **0.650**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.83822 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93135$	Status: <b>PASS</b> Ratio: <b>0.930</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.00095541 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.0019108 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0019108 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0019108 \text{ kipft/ft})) + (4 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3333 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0019108 \text{ kipft/ft})) + (3 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.0019108 \text{ kipft/ft})) + (2 \times (-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = -0.00033174 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0019108 \text{ kipft/ft})) + ((-0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = -0.00031847 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3333 \text{ ft})}{2}$ $p_a = 0.325 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.00033174 \text{ kip/ft}^2)}{(0.325 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0010207$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.000</b>

$$Ratio = \frac{(-0.00031847 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = -0.00035386$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.679 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58583 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(37.985 \text{ kipft}) + ((-3.679 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.0486 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.0486 \text{ kipft/ft})}{(-0.58583 \text{ kip/ft})}$$

$$E = 10.325 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.0486 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.58583 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.0486 \text{ kipft/ft})) + (4 \times (-0.58583 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1396 \text{ ft}$$

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

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

$$V_{max} = ((-0.58583 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1396 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1396 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6809 \text{ kip}$$

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

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

$$M_{max} = ((-0.58583 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(10.325 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 24.766 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0012739 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.018 \text{ kipft}) + ((-0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0028662 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.0028662 \text{ kipft/ft})}{(-0.0012739 \text{ kip/ft})}$$

$$E = 2.25 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.0028662 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.0012739 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0028662 \text{ kipft/ft})) + (4 \times (-0.0012739 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.32 \text{ ft}$$

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

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

$$V_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.25 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.32 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.25 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.32 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0067913 \text{ kip}$$

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

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

$$M_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(2.25 \text{ ft})}{(6 \text{ ft})} + \frac{(4.32 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.25 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.32 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.25 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.32 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.017657 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.908 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.333 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.333 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.908 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0029561$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.908 \text{ kip} \rightarrow 7908 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7908 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.54 \text{ kip}$$

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

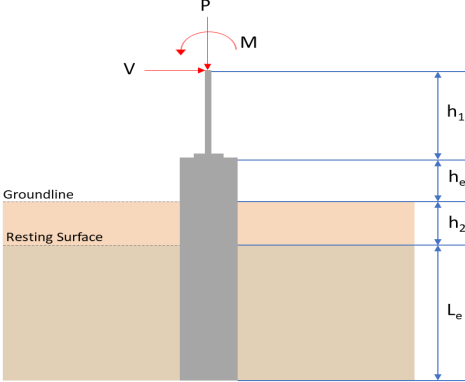
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.54 \text{ kip}$$

<p>22.5.1.2</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><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ytks} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.78 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.6809 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.6809 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.07836$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.0067913 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.0067913 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.000061304$	<p>Status: <b>PASS</b> Ratio: <b>0.080</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 24.766 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.766 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.099223$	<p>Status: <b>PASS</b>  Ratio: <b>0.100</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.017657 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.017657 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.000070742$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.637</td> <td>8.366</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.298</td> <td>-3.831</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>23.125</td> <td>39.137</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	5.637	8.366	$V_x$ (kip)	-2.298	-3.831	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	23.125	39.137	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.298 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.36592 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(23.125 \text{ kipft}) + ((-2.298 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.6823 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation:  <math>L_{e,x} = 5.5655 \text{ ft}</math> - Required depth in x-direction,</p> <p><b>Considering z-direction:</b>  <math>L_{e,z} = 0 \text{ ft}</math> - Required depth in z-direction,</p> <p><b>Minimum embedded depth required:</b>  <math>L_{e,req}</math> - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.5655 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.566 \text{ ft}$ <p><math>L_e</math> - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.566 \text{ ft})}{(6 \text{ ft})}$ $\text{Ratio} = 0.92767$	<p>Status: <b>PASS</b>  Ratio: <b>0.930</b></p>
	<p><b>End-bearing Capacity (ASD)</b>  A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(5.637 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.35231 \text{ kip/ft}^2$ <p><b>Check bearing capacity ratio:</b>  <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.35231 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.17616$	<p>Status: <b>PASS</b>  Ratio: <b>0.180</b></p>
<p>Czerniak</p>	<p><b>Lateral Soil Pressure (ASD):</b>  L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.36592$  kip/ft - Lateral force per length of pile,

$M_o = 3.6823$  kipft/ft - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.6823 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.36592 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.6823 \text{ kipft/ft})) + (4 \times (-0.36592 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1422 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6823 \text{ kipft/ft})) + (3 \times (-0.36592 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.6823 \text{ kipft/ft})) + (2 \times (-0.36592 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.20753 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.6823 \text{ kipft/ft})) + ((-0.36592 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.86152 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1422 \text{ ft})}{2}$$

$$p_a = 0.31067 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20753 \text{ kip/ft}^2)}{(0.31067 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66802$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

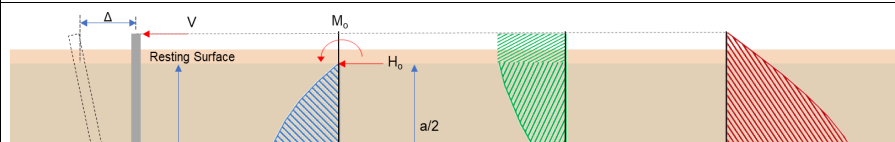
$$\text{Ratio} = \frac{s}{p_s}$$

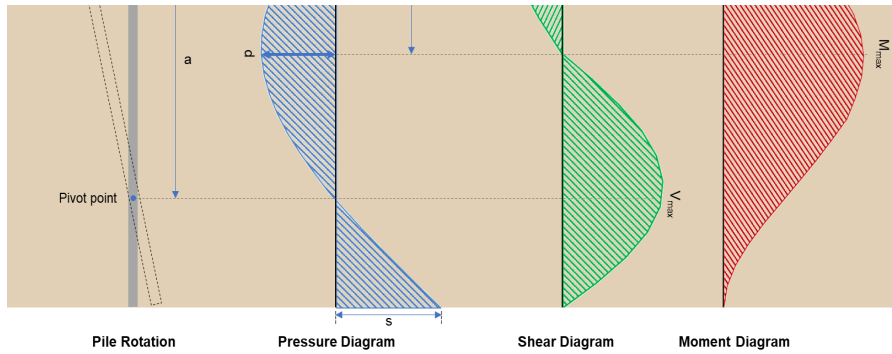
$$\text{Ratio} = \frac{(0.86152 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95724$$

Status: **PASS**  
Ratio: **0.670**

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





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.831 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.61003 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(39.137 \text{ kipft}) + ((-3.831 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.232 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.232 \text{ kipft/ft})}{(-0.61003 \text{ kip/ft})}$$

$$E = 10.216 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.232 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.61003 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.232 \text{ kipft/ft})) + (4 \times (-0.61003 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1407 \text{ ft}$$

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

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

$$V_{max} = ((-0.61003 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.216 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1407 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.216 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1407 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.961 \text{ kip}$$

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

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

$$M_{max} = ((-0.61003 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(10.216 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1407 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.216 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1407 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.216 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1407 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.554 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.366 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.318 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.318 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3  $s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

**Axial Compression Strength (ACI 318-19, LRFD)**22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.366 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0031273$$

Status: **PASS**  
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$  - Effective width,22.5.2.2  $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,22.5.5.1.1  $V_{c,max}$  - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.366 \text{ kip} \rightarrow 8366 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8366 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.6 \text{ kip}$$

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

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

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

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

 $V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.6 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.6 \text{ kip}$$

<p>22.5.1.2</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><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.6 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.82 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.961 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.961 \text{ kip})}{(110.82 \text{ kip})}$ $\text{Ratio} = 0.08086$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</b></p>
<p>14.5.2.1b</p>	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:</p> <p><math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$$\phi M_{n,2} = \phi F_y Z_{ek} S_m$$

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

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

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:**

$M_{max} = 25.554 \text{ kipft}$  - Maximum moment in the x-direction,

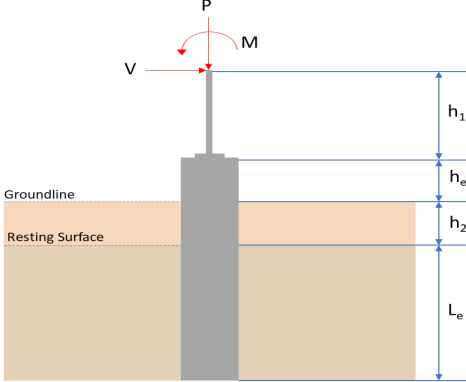
*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(25.554 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.10238$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.340</td> <td>7.908</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.206</td> <td>-3.679</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.006</td> <td>0.008</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.012</td> <td>0.019</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>22.410</td> <td>37.985</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	5.340	7.908	$V_x$ (kip)	-2.206	-3.679	$V_z$ (kip)	0.006	0.008	$M_x$ (kipft)	0.012	0.019	$M_z$ (kipft)	22.410	37.985	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.206 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.35127 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(22.41 \text{ kipft}) + ((-2.206 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.5685 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5283 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.006 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00095541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.012 \text{ kipft}) + ((0.006 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0019108 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.57048 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.5283 \text{ ft}), (0.57048 \text{ ft})]$$

$$L_{e,req} = 5.528 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.528 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92133$$

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

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

$$q = \frac{(5.34 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33375 \text{ kip/ft}^2$$

$$q = 0.33375 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16687$$

Status: **PASS**  
Ratio: **0.170**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.35127 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.5685 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.5685 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.5685 \text{ kipft/ft})) + (4 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1413 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.5685 \text{ kipft/ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.5685 \text{ kipft/ft})) + (2 \times (-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.20293 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.5685 \text{ kipft/ft})) + ((-0.35127 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.83822 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1413 \text{ ft})}{2}$$

$$p_a = 0.31059 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20293 \text{ kip/ft}^2)}{(0.31059 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65336$$

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

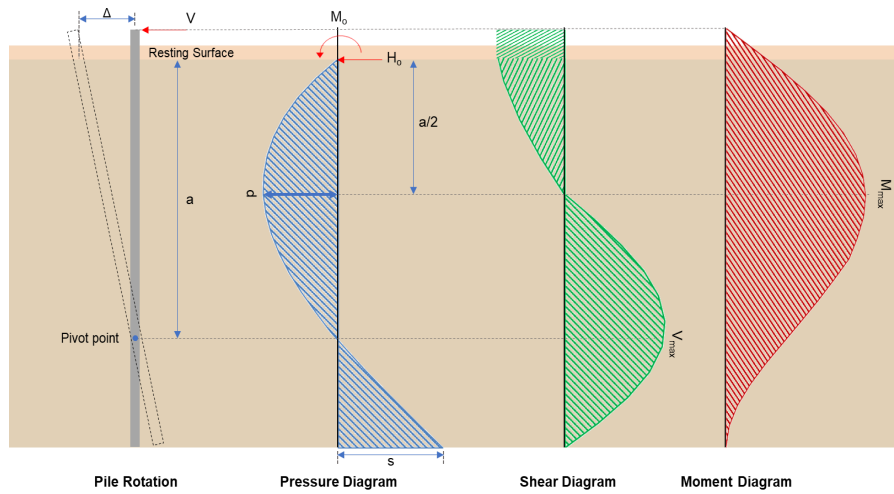
Status: **PASS**  
Ratio: **0.650**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.83822 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93135$	Status: <b>PASS</b> Ratio: <b>0.930</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.00095541 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.0019108 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0019108 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0019108 \text{ kipft/ft})) + (4 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3333 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0019108 \text{ kipft/ft})) + (3 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.0019108 \text{ kipft/ft})) + (2 \times (0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.00074752 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0019108 \text{ kipft/ft})) + ((0.00095541 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.0015924 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3333 \text{ ft})}{2}$ $p_a = 0.325 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00074752 \text{ kip/ft}^2)}{(0.325 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0023001$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.000</b>

$$Ratio = \frac{(0.0015924 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.0017693$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.679 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58583 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(37.985 \text{ kipft}) + ((-3.679 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.0486 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.0486 \text{ kipft/ft})}{(-0.58583 \text{ kip/ft})}$$

$$E = 10.325 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.0486 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.58583 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.0486 \text{ kipft/ft})) + (4 \times (-0.58583 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1396 \text{ ft}$$

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

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

$$V_{max} = ((-0.58583 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1396 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1396 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6809 \text{ kip}$$

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

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

$$M_{max} = ((-0.58583 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(10.325 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.325 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1396 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 24.766 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0012739 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.019 \text{ kipft}) + ((0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0030255 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.0030255 \text{ kipft/ft})}{(0.0012739 \text{ kip/ft})}$$

$$E = 2.375 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.0030255 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.0012739 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.0030255 \text{ kipft/ft})) + (4 \times (0.0012739 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3137 \text{ ft}$$

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

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

$$V_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.375 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.3137 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.375 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.3137 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0069763 \text{ kip}$$

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

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

$$M_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(2.375 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3137 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.375 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.3137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.375 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.3137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.01821 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.908 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.333 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.333 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.908 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0029561$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.908 \text{ kip} \rightarrow 7908 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7908 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.54 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.54 \text{ kip}$$

<p>22.5.1.2</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><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.78 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.6809 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.6809 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.07836$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.0069763 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.0069763 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.000062974$	<p>Status: <b>PASS</b> Ratio: <b>0.080</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 24.766 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.766 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.099223$	<p>Status: <b>PASS</b>  Ratio: <b>0.100</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.01821 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.01821 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.000072958$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>