

# Your Project Calculations



Project Name: Werner MB

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=Werner%20MB&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/9\\_2023](https://platform.skyciv.com/structural?preload_name=Werner%20MB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=y3BryrPtX83wbQimdvh0zJggYji0YPyorrbcURPU7ye13IGb44XloVYOLFde3tUu](https://platform.skyciv.com/structural-viewer?project_id=y3BryrPtX83wbQimdvh0zJggYji0YPyorrbcURPU7ye13IGb44XloVYOLFde3tUu)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	3P-22.5-6TOP-SD-72-L-4Hx11W-DFKL
<b>Duty Classification:</b>	SD
<b>Module Width:</b>	41.00 in
<b>Module Length:</b>	70.00in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	11
<b>Total Number of Modules:</b>	44
<b>Desired Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	5
<b>Total Array Height at Tilt:</b>	15.53 ft
<b>Total Frame Length:</b>	64.50 ft
<b>Frame Weight:</b>	2505 lbs
<b>Array Dimensions N/S:</b>	13.83 ft
<b>Array Dimensions E/W:</b>	65.08 ft
<b>Rail Length:</b>	166.00 in
<b>Rail Spacing:</b>	2.92 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 80
<b>Pole Length above Grade:</b>	10.30 ft
<b>Number of Poles:</b>	3
<b>Pole Spacing:</b>	22.5 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.50 ft Pile 2: 6.50 ft Pile 3: 6.50 ft
<b>Foundation Volume:</b>	11.556 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.199914 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	2 Timber Ln, Clancy, MT 59634, USA
<b>Wind Speed:</b>	110 mph
<b>Snow Load:</b>	50 psf
<b>Design Uplift Pressure:</b>	0.020791 ksf
<b>Design Downforce Pressure:</b>	-0.020791 ksf
<b>Design Snow Pressure:</b>	0.010996 ksf



### Design Disclaimer

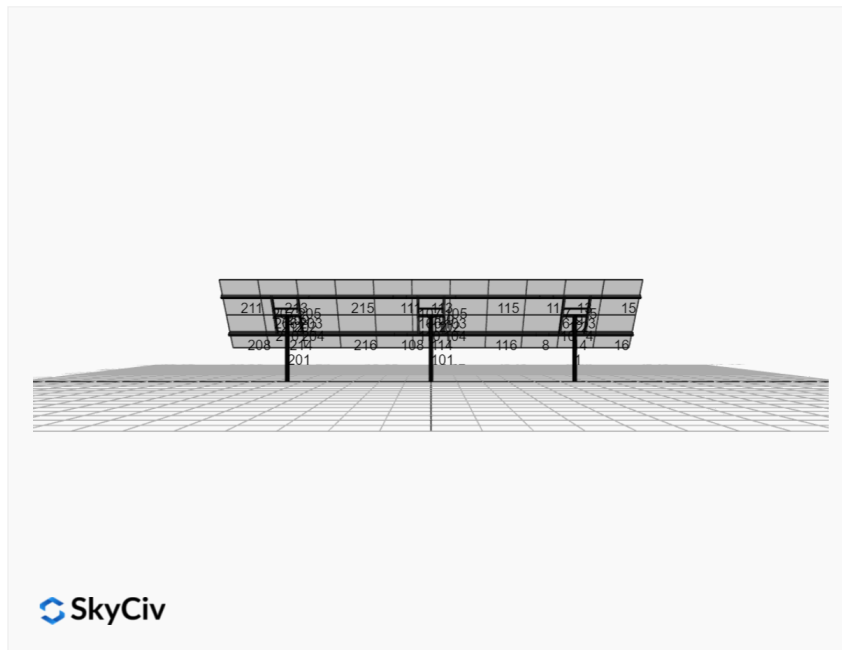
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

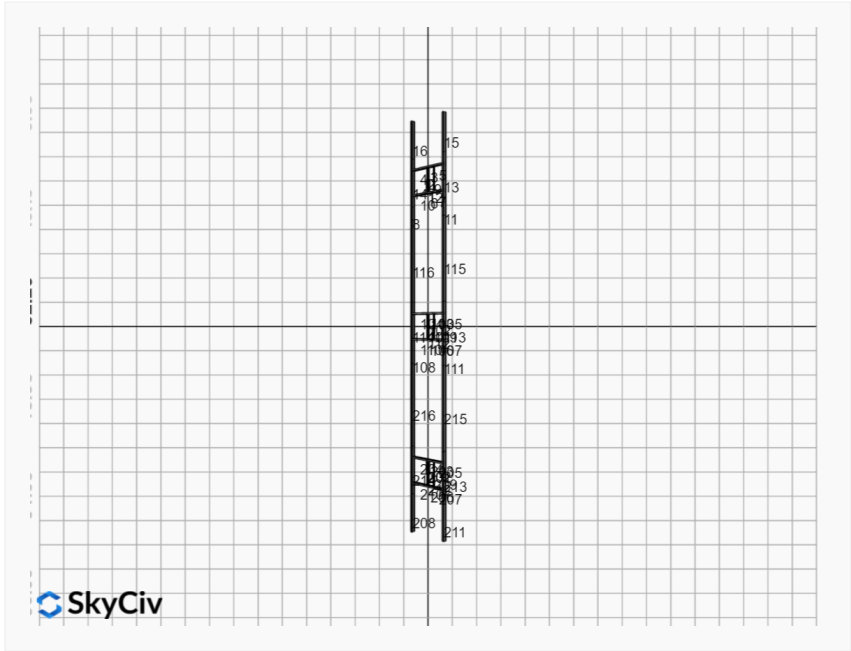
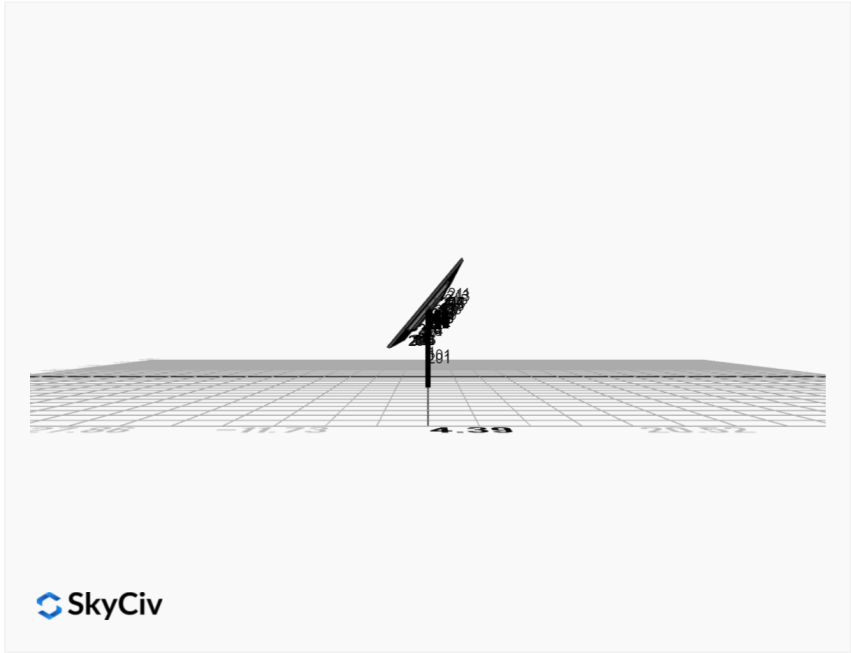
### AutoDesigner Input

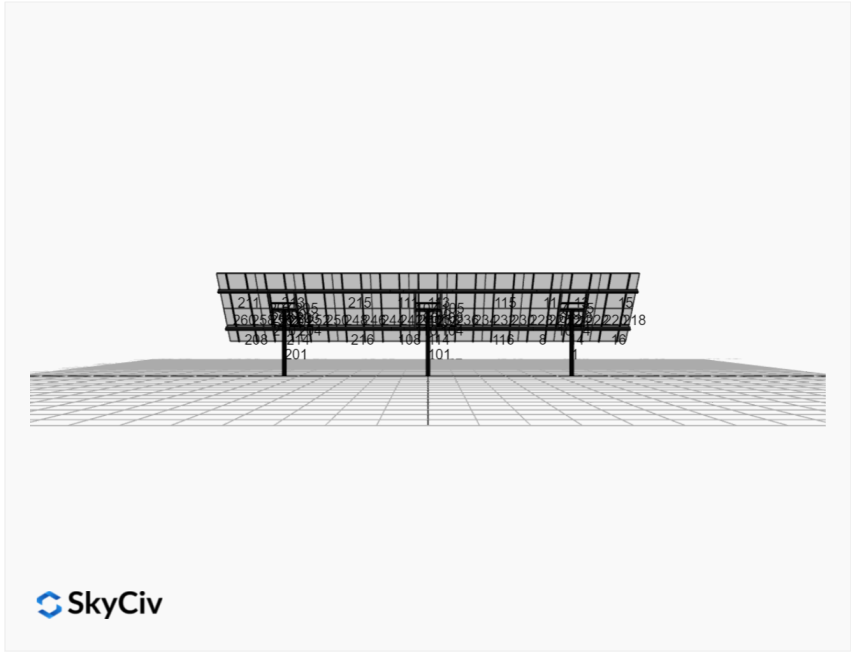
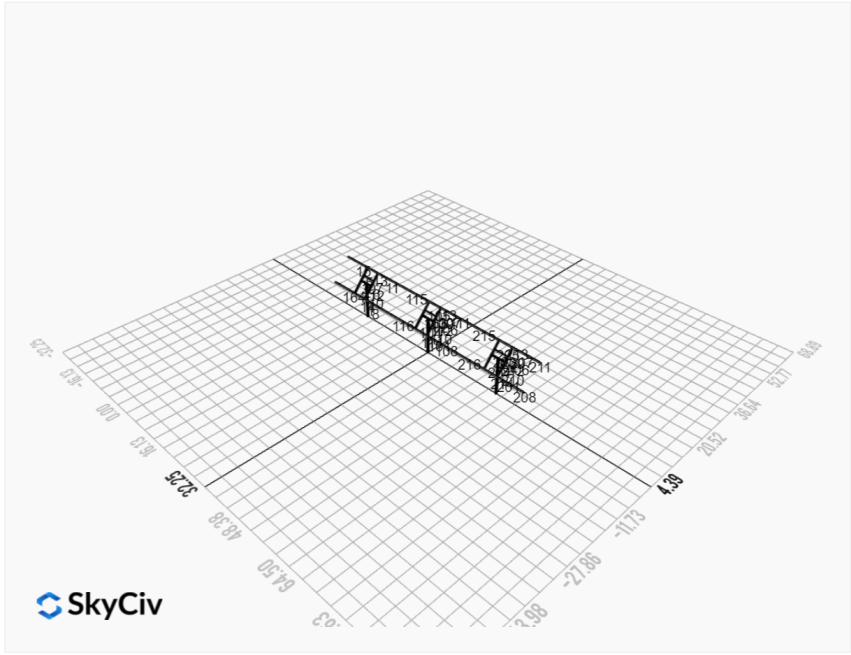
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  "project_id": "Werner MB",
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  "core_pipe_width": 65,
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  "customer_foundation_override": "48_Square",
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### Design Notes:

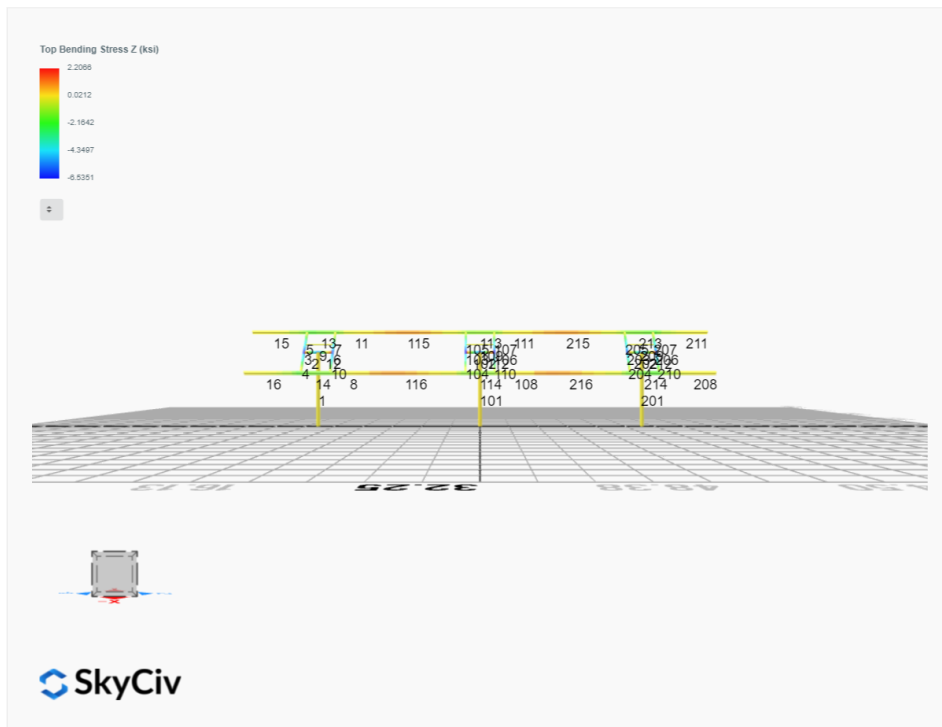
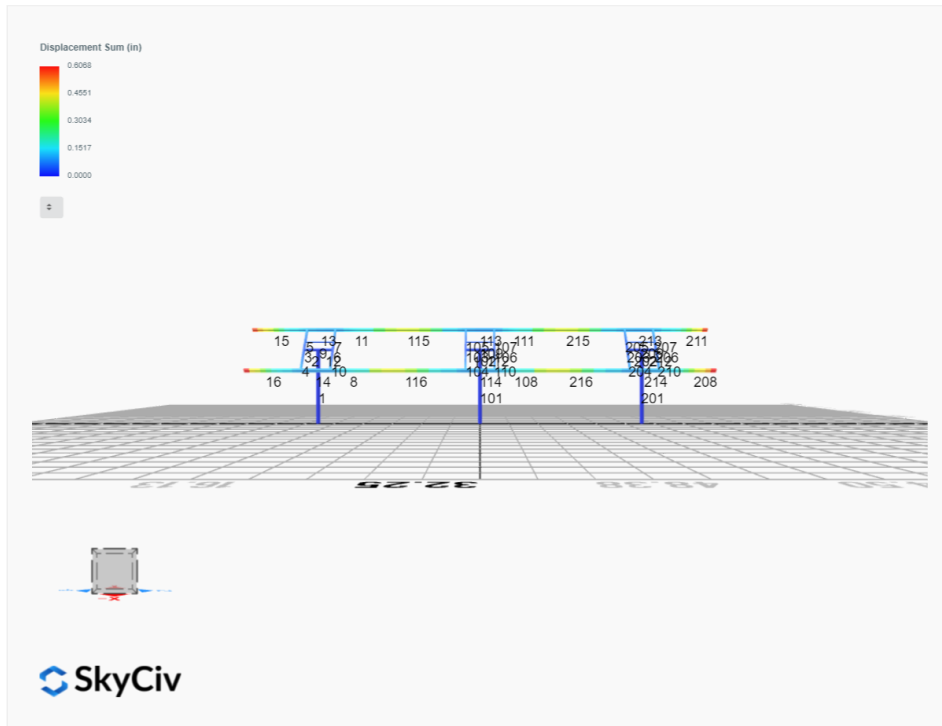
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Foundation Design and Sizing is approximate only



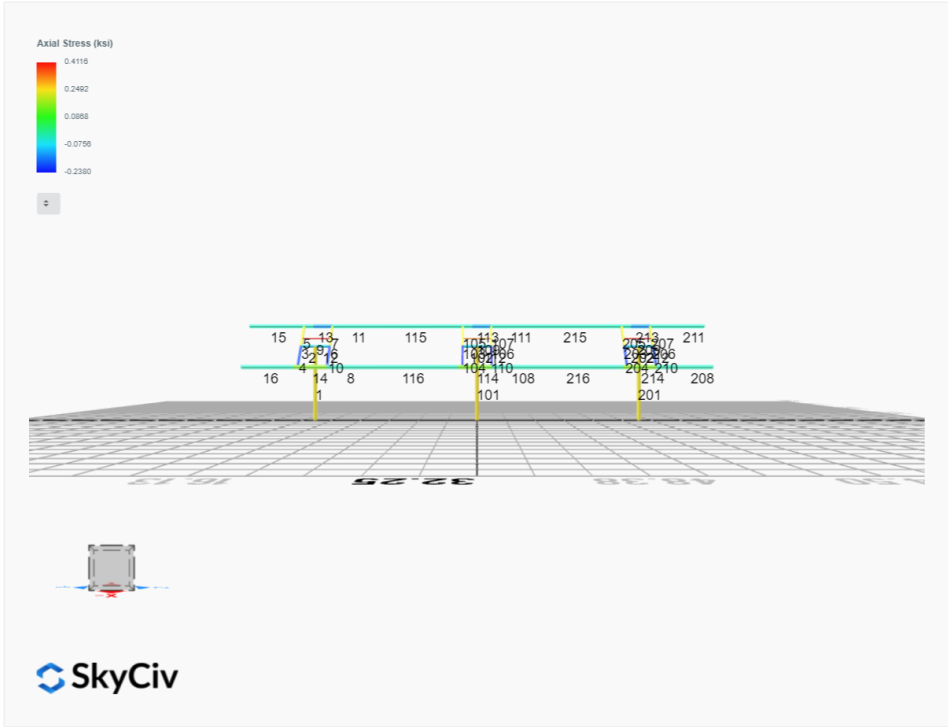




## FEM Results (Envelope Worst Case for each member)







## 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	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 2. D + L	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 3. D + (S or Lr or R)	-0.0127	4.2780	-0.0139	-0.0364	0.1088	0.1458
ULS: 3. D + (S or Lr or R)	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0110	3.7631	-0.0121	-0.0315	0.0942	0.1276
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 5b. D + 0.7E	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0110	3.7631	-0.0121	-0.0315	0.0942	0.1276
ULS: 8. 0.6D + 0.7E	-0.0036	1.3311	-0.0039	-0.0102	0.0303	0.0439
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8410	4.5922	-0.0223	-0.0638	0.1075	29.8089
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8284	-0.1550	0.0100	0.0315	-0.0095	-28.6500
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1373	5.5434	-0.0239	-0.0667	0.1369	22.4293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0110	3.7631	-0.0121	-0.0315	0.0942	0.1276
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1147	1.9830	0.0003	0.0048	0.0492	-21.4148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0110	3.7631	-0.0121	-0.0315	0.0942	0.1276
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1322	3.9988	-0.0183	-0.0521	0.0932	22.3750
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1198	0.4384	0.0058	0.0194	0.0055	-21.4692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0059	2.2185	-0.0065	-0.0170	0.0505	0.0732
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8386	3.7049	-0.0197	-0.0570	0.0872	29.7796
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0036	1.3311	-0.0039	-0.0102	0.0303	0.0439
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8308	-1.0424	0.0126	0.0383	-0.0297	-28.6793
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0036	1.3311	-0.0039	-0.0102	0.0303	0.0439

### 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	7.9357
Shear X	-4.7364
Shear Z	-0.0374
Moment X	-0.1068
Moment Y (Twist)	0.1999
Moment Z	50.5956

### 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	5.5434
Shear X	-2.8410
Shear Z	-0.0239
Moment X	-0.0667
Moment Y (Twist)	0.1369
Moment Z	29.8089

## 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	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 2. D + L	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 3. D + (S or Lr or R)	0.0255	4.5166	-0.0000	0.0000	-0.0000	-0.2156
ULS: 3. D + (S or Lr or R)	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0221	3.9696	-0.0000	0.0000	0.0000	-0.1853
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 5b. D + 0.7E	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0221	3.9696	-0.0000	0.0000	0.0000	-0.1853
ULS: 8. 0.6D + 0.7E	0.0071	1.3973	-0.0000	0.0000	0.0000	-0.0567
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9215	4.8005	-0.0000	0.0000	0.0000	30.6107
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9466	-0.1433	-0.0000	0.0000	0.0000	-29.7408
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1779	5.8234	-0.0000	0.0000	0.0000	22.8436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0221	3.9696	-0.0000	0.0000	0.0000	-0.1853
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2232	2.1155	-0.0000	0.0000	0.0000	-22.4201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0221	3.9696	-0.0000	0.0000	0.0000	-0.1853
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1882	4.1826	-0.0000	0.0000	0.0000	22.9344
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2129	0.4747	-0.0000	0.0000	0.0000	-22.3292
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0118	2.3289	-0.0000	0.0000	0.0000	-0.0944
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9262	3.8689	-0.0000	0.0000	0.0000	30.6485
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0071	1.3973	-0.0000	0.0000	0.0000	-0.0567
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9419	-1.0749	-0.0000	0.0000	0.0000	-29.7030
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0071	1.3973	-0.0000	0.0000	0.0000	-0.0567

#### 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	8.3543
Shear X	-4.9110
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	51.9575

#### 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	5.8234
Shear X	-2.9466
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	30.6485

#### 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	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 2. D + L	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 3. D + (S or Lr or R)	-0.0127	4.2780	0.0139	0.0364	-0.1088	0.1458
ULS: 3. D + (S or Lr or R)	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0110	3.7631	0.0121	0.0316	-0.0942	0.1276
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 5b. D + 0.7E	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0110	3.7631	0.0121	0.0316	-0.0942	0.1276
ULS: 8. 0.6D + 0.7E	-0.0036	1.3311	0.0039	0.0102	-0.0303	0.0439
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8410	4.5922	0.0223	0.0638	-0.1074	29.8089
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8284	-0.1550	-0.0100	-0.0315	0.0095	-28.6500
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1373	5.5434	0.0239	0.0667	-0.1369	22.4293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0110	3.7631	0.0121	0.0316	-0.0942	0.1276
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1147	1.9830	-0.0003	-0.0048	-0.0492	-21.4148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0110	3.7631	0.0121	0.0316	-0.0942	0.1276

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	-2.1322	3.9988	0.0183	0.0521	-0.0932	22.3750
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1198	0.4384	-0.0058	-0.0194	-0.0055	-21.4692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0059	2.2185	0.0065	0.0170	-0.0505	0.0732
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8386	3.7049	0.0197	0.0570	-0.0872	29.7796
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0036	1.3311	0.0039	0.0102	-0.0303	0.0439
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8308	-1.0424	-0.0126	-0.0383	0.0297	-28.6793
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0036	1.3311	0.0039	0.0102	-0.0303	0.0439

### 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	7.9357
Shear X	-4.7363
Shear Z	0.0374
Moment X	0.1069
Moment Y (Twist)	0.1999
Moment Z	50.5963

### 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	5.5434
Shear X	-2.8410
Shear Z	0.0239
Moment X	0.0667
Moment Y (Twist)	0.1369
Moment Z	29.8089

## Project Details

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

User Name: sales@mtsolar.us  
 Project Name: Werner MB  
 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

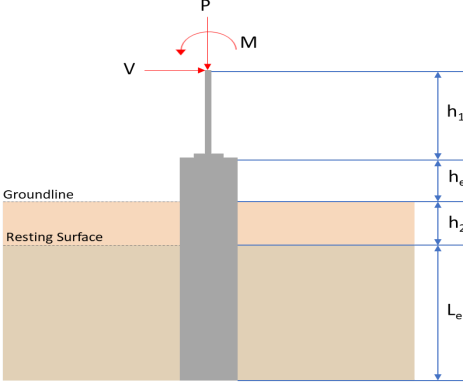
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	135.82	62.23	62.23	113.47	113.47
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	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	18.84	6.45	30.09	45.74
14	120.60	98.23	19.01	6.45	30.09	45.74
15	120.60	21.74	23.36	6.45	30.09	45.74
16	120.60	21.74	23.36	6.45	30.09	45.74
101	378.22	135.82	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	17.96	6.45	30.09	45.74
114	120.60	98.23	17.96	6.45	30.09	45.74
115	120.60	48.60	11.32	6.45	30.09	45.74
116	120.60	48.60	11.32	6.45	30.09	45.74
201	378.22	135.82	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	21.74	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	21.74	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	18.84	6.45	30.09	45.74
214	120.60	98.23	19.01	6.45	30.09	45.74
215	120.60	48.60	11.72	6.45	30.09	45.74
216	120.60	48.60	11.72	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	0.058	0.813	0.004	0.042	0.000	0.843	#13	0.591	Not Required	Pass
2	0.006	0.387	0.311	0.089	0.056	0.699	#13	0.034	Not Required	Pass
3	0.014	0.721	0.105	0.072	0.001	0.790	#13	0.044	Not Required	Pass
4	0.013	0.717	0.439	0.072	0.052	0.869	#13	0.078	Not Required	Pass
5	0.014	0.447	0.465	0.072	0.068	0.515	#13	0.073	Not Required	Pass
6	0.015	0.705	0.110	0.070	0.001	0.772	#13	0.044	Not Required	Pass
7	0.015	0.439	0.456	0.070	0.069	0.511	#13	0.073	Not Required	Pass
8	0.000	0.088	0.192	0.049	0.016	0.259	#21	0.088	Not Required	Pass
9	0.027	0.056	0.076	0.001	0.000	0.139	#13	0.198	Not Required	Pass
10	0.013	0.697	0.432	0.070	0.053	0.867	#13	0.078	Not Required	Pass
11	0.000	0.090	0.196	0.050	0.016	0.263	#21	0.088	Not Required	Pass
12	0.006	0.369	0.300	0.088	0.055	0.671	#13	0.034	Not Required	Pass
13	0.009	0.311	0.460	0.061	0.020	0.705	#21	0.265	Not Required	Pass
14	0.009	0.314	0.460	0.061	0.020	0.705	#21	0.177	Not Required	Pass
15	0.000	0.149	0.275	0.039	0.013	0.395	#21	Not Required	Not Required	Pass
16	0.000	0.149	0.275	0.039	0.013	0.395	#21	Not Required	Not Required	Pass
101	0.062	0.835	0.000	0.043	0.000	0.864	#13	0.591	Not Required	Pass
102	0.005	0.391	0.308	0.093	0.057	0.701	#13	0.034	Not Required	Pass
103	0.015	0.739	0.128	0.074	0.006	0.820	#13	0.044	Not Required	Pass
104	0.014	0.738	0.414	0.074	0.050	0.901	#13	0.078	Not Required	Pass
105	0.015	0.459	0.437	0.073	0.065	0.526	#13	0.073	Not Required	Pass
106	0.015	0.739	0.128	0.074	0.006	0.820	#13	0.044	Not Required	Pass
107	0.015	0.459	0.437	0.073	0.065	0.526	#13	0.073	Not Required	Pass
108	0.000	0.064	0.192	0.047	0.016	0.244	#21	0.088	Not Required	Pass
109	0.023	0.048	0.070	0.001	0.000	0.124	#13	0.198	Not Required	Pass
110	0.014	0.738	0.414	0.074	0.050	0.901	#13	0.078	Not Required	Pass
111	0.000	0.060	0.195	0.047	0.016	0.246	#21	0.088	Not Required	Pass
112	0.005	0.391	0.308	0.093	0.057	0.701	#13	0.034	Not Required	Pass
113	0.009	0.241	0.419	0.058	0.020	0.608	#21	0.265	Not Required	Pass
114	0.008	0.252	0.416	0.058	0.020	0.607	#21	0.265	Not Required	Pass
115	0.000	0.315	0.235	0.047	0.016	0.487	#21	0.557	Not Required	Pass
116	0.000	0.316	0.237	0.047	0.016	0.491	#21	0.557	Not Required	Pass
201	0.058	0.813	0.004	0.042	0.000	0.843	#13	0.591	Not Required	Pass
202	0.006	0.369	0.300	0.088	0.055	0.671	#13	0.034	Not Required	Pass
203	0.015	0.705	0.110	0.070	0.001	0.772	#13	0.044	Not Required	Pass
204	0.013	0.697	0.432	0.070	0.053	0.867	#13	0.078	Not Required	Pass
205	0.015	0.438	0.456	0.070	0.069	0.511	#13	0.073	Not Required	Pass
206	0.014	0.721	0.105	0.072	0.001	0.790	#13	0.044	Not Required	Pass
207	0.014	0.447	0.465	0.072	0.068	0.515	#13	0.073	Not Required	Pass
208	0.000	0.149	0.275	0.039	0.013	0.395	#21	Not Required	Not Required	Pass
209	0.027	0.056	0.076	0.001	0.000	0.139	#13	0.198	Not Required	Pass
210	0.013	0.717	0.439	0.072	0.052	0.869	#13	0.078	Not Required	Pass
211	0.000	0.149	0.275	0.039	0.013	0.395	#21	Not Required	Not Required	Pass
212	0.006	0.387	0.311	0.089	0.056	0.699	#13	0.034	Not Required	Pass
213	0.009	0.311	0.460	0.061	0.020	0.705	#21	0.177	Not Required	Pass
214	0.009	0.314	0.460	0.061	0.020	0.705	#21	0.265	Not Required	Pass
215	0.000	0.314	0.235	0.050	0.016	0.484	#21	0.557	Not Required	Pass
216	0.000	0.311	0.238	0.049	0.016	0.487	#21	0.557	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 = 6.5</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 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.543</td> <td>7.936</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.841</td> <td>-4.736</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.024</td> <td>-0.037</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.067</td> <td>-0.107</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>29.809</td> <td>50.596</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)	5.543	7.936	$V_x$ (kip)	-2.841	-4.736	$V_z$ (kip)	-0.024	-0.037	$M_x$ (kipft)	-0.067	-0.107	$M_z$ (kipft)	29.809	50.596	
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$M_z$ (kipft)	29.809	50.596																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.841 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.45239 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(29.809 \text{ kipft}) + ((-2.841 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7467 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.0064 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010669 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.86827 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.0064 \text{ ft}), (0.86827 \text{ ft})]$$

$$L_{e,req} = 6.006 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.006 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.924$$

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

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

$$q = \frac{(5.543 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.34644 \text{ kip/ft}^2$$

$$q = 0.34644 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34644 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17322$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.45239 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.7467 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.7467 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.7467 \text{ kipft/ft})) + (4 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4917 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (4.7467 \text{ kipft/ft})) + (3 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (4.7467 \text{ kipft/ft})) + (2 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21943 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 [(2 \times (4.7467 \text{ kipft/ft})) + ((-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.93057 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4917 \text{ ft})}{2}$$

$$p_a = 0.33687 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21943 \text{ kip/ft}^2)}{(0.33687 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65138$$

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

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.93057 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95443$$

Status: **PASS**  
Ratio: **0.950**

**Considering z-direction:**

$H_o = -0.0038217 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.010669 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.010669 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.010669 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6628 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010669 \text{ kipft/ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.010669 \text{ kipft/ft})) + (2 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = -0.0010186 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.010669 \text{ kipft/ft})) + ((-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = -0.00049749 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6628 \text{ ft})}{2}$$

$$p_a = 0.34971 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0010186 \text{ kip/ft}^2)}{(0.34971 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0029128$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

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

$$Ratio = \frac{(-0.00049749 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = -0.00051025$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.736 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.75414 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(50.596 \text{ kipft}) + ((-4.736 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.0567 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.0567 \text{ kipft/ft})}{(-0.75414 \text{ kip/ft})}$$

$$E = 10.683 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.0567 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.75414 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.0567 \text{ kipft/ft})) + (4 \times (-0.75414 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4896 \text{ ft}$$

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

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

$$V_{max} = ((-0.75414 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4896 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4896 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.762 \text{ kip}$$

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

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

$$M_{max} = ((-0.75414 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(10.683 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.199 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0058917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.107 \text{ kipft}) + ((-0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017038 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.017038 \text{ kipft/ft})}{(-0.0058917 \text{ kip/ft})}$$

$$E = 2.8919 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.017038 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.017038 \text{ kipft/ft})) + (4 \times (-0.0058917 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6582 \text{ ft}$$

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

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

$$V_{max} = ((-0.0058917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.6582 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.6582 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.034283 \text{ kip}$$

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

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

$$M_{max} = ((-0.0058917 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(2.8919 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.097779 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.936 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.332 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.332 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(7.936 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0029665$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.936 \text{ kip} \rightarrow 7936 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7936 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

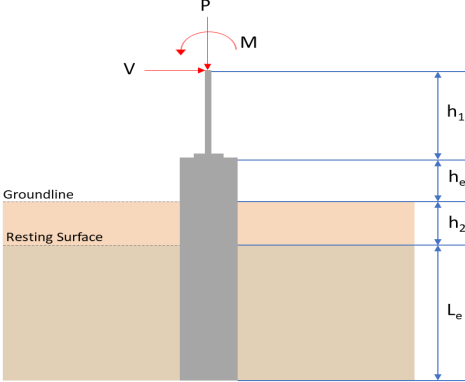
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

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

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

<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} = 10.762 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.762 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.097147$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.034283 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.034283 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.00030946$	<p>Status: <b>PASS</b> Ratio: <b>0.100</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} = 33.199 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(33.199 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13301$	<p>Status: <b>PASS</b>  Ratio: <b>0.130</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.097779 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.097779 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00039174$	<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 = 6.5</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 1193"> <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>5.823</td> <td>8.354</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.947</td> <td>-4.911</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>30.648</td> <td>51.957</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)	5.823	8.354	$V_x$ (kip)	-2.947	-4.911	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	30.648	51.957	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
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$M_x$ (kipft)	0.000	0.000																										
$M_z$ (kipft)	30.648	51.957																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.947 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.46927 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(30.648 \text{ kipft}) + ((-2.947 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.8803 \text{ kipft/ft}$ <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} = 6.0401 \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> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.0401 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.04 \text{ ft}$ <p><math>L_e</math> - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.04 \text{ ft})}{(6.5 \text{ ft})}$ $\text{Ratio} = 0.92923$	<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> $q = \frac{P_v}{A}$ $q = \frac{(5.823 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.36394 \text{ kip/ft}^2$ <p><b>Check bearing capacity ratio:</b>  <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.36394 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.18197$	<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> $L/D = \frac{L}{D}$ $L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.46927$  kip/ft - Lateral force per length of pile,

$M_o = 4.8803$  kipft/ft - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.8803 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.46927 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.8803 \text{ kipft/ft})) + (4 \times (-0.46927 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4927 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.8803 \text{ kipft/ft})) + (3 \times (-0.46927 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (4.8803 \text{ kipft/ft})) + (2 \times (-0.46927 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.22353 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.8803 \text{ kipft/ft})) + ((-0.46927 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.95294 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4927 \text{ ft})}{2}$$

$$p_a = 0.33695 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22353 \text{ kip/ft}^2)}{(0.33695 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66341$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

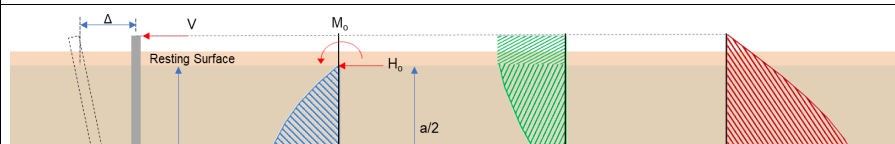
$$\text{Ratio} = \frac{s}{p_s}$$

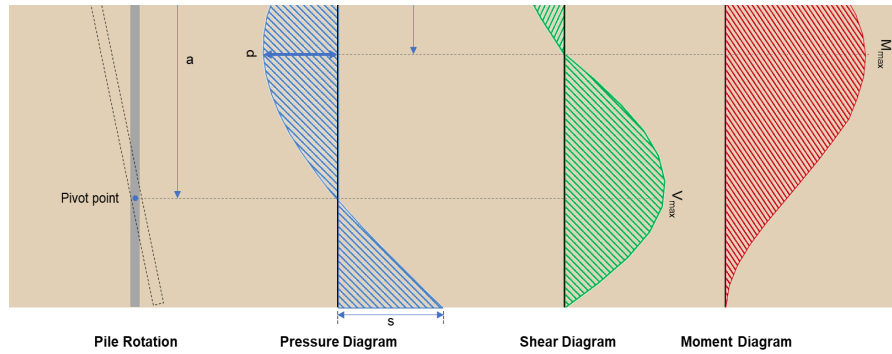
$$\text{Ratio} = \frac{(0.95294 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97737$$

Status: **PASS**  
Ratio: **0.660**

Status: **PASS**  
Ratio: **0.980**





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.911 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.78201 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(51.957 \text{ kipft}) + ((-4.911 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.2734 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.2734 \text{ kipft/ft})}{(-0.78201 \text{ kip/ft})}$$

$$E = 10.58 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.2734 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.78201 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.2734 \text{ kipft/ft})) + (4 \times (-0.78201 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4907 \text{ ft}$$

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

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

$$V_{max} = ((-0.78201 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.58 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4907 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.58 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4907 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.072 \text{ kip}$$

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

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

$$M_{max} = ((-0.78201 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(10.58 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4907 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.58 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4907 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.58 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4907 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.139 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.354 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.319 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.319 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.354 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0031228$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.354 \text{ kip} \rightarrow 8354 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8354 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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} = 11.072 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.072 \text{ kip})}{(110.82 \text{ kip})}$ $\text{Ratio} = 0.099908$	<p>Status: <b>PASS</b>  Ratio: <b>0.100</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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kip ft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$\phi M_{n,2} = \phi M_{n,1}$

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

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

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:**

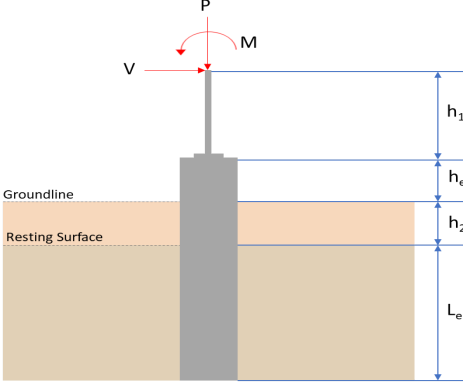
$M_{max} = 34.139 \text{ kipft}$  - Maximum moment in the x-direction,  
*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(34.139 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.13677$$

Status: **PASS**  
Ratio: **0.140**

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></p> <p>Pile shape: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 6.5</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 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.543</td> <td>7.936</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.841</td> <td>-4.736</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.024</td> <td>0.037</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.067</td> <td>0.107</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>29.809</td> <td>50.596</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><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)	5.543	7.936	$V_x$ (kip)	-2.841	-4.736	$V_z$ (kip)	0.024	0.037	$M_x$ (kipft)	0.067	0.107	$M_z$ (kipft)	29.809	50.596	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
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Load Component	ASD	LRFD																										
$P$ (kip)	5.543	7.936																										
$V_x$ (kip)	-2.841	-4.736																										
$V_z$ (kip)	0.024	0.037																										
$M_x$ (kipft)	0.067	0.107																										
$M_z$ (kipft)	29.809	50.596																										
	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><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></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.841 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.45239 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(29.809 \text{ kipft}) + ((-2.841 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7467 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.0064 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010669 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.029 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.0064 \text{ ft}), (1.029 \text{ ft})]$$

$$L_{e,req} = 6.006 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.006 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.924$$

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

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

$$q = \frac{(5.543 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.34644 \text{ kip/ft}^2$$

$$q = 0.34644 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34644 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17322$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.45239 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.7467 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.7467 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.7467 \text{ kipft/ft})) + (4 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4917 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (4.7467 \text{ kipft/ft})) + (3 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (4.7467 \text{ kipft/ft})) + (2 \times (-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21943 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 [(2 \times (4.7467 \text{ kipft/ft})) + ((-0.45239 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.93057 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4917 \text{ ft})}{2}$$

$$p_a = 0.33687 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21943 \text{ kip/ft}^2)}{(0.33687 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65138$$

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

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.93057 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95443$$

Status: **PASS**  
Ratio: **0.950**

**Considering z-direction:**

$H_o = 0.0038217 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.010669 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.010669 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.010669 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6628 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010669 \text{ kipft/ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.010669 \text{ kipft/ft})) + (2 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.0029848 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.010669 \text{ kipft/ft})) + ((0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0065579 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6628 \text{ ft})}{2}$$

$$p_a = 0.34971 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0029848 \text{ kip/ft}^2)}{(0.34971 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0085351$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

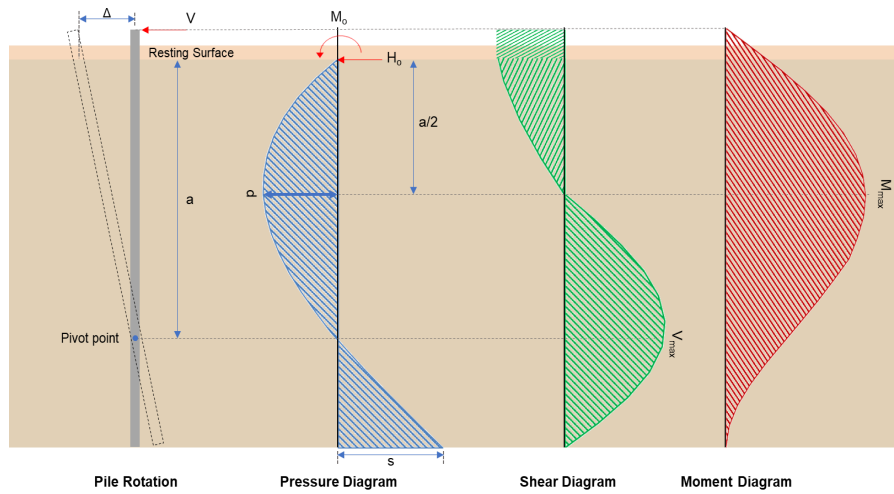
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0065579 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.006726$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.736 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.75414 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(50.596 \text{ kipft}) + ((-4.736 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.0567 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.0567 \text{ kipft/ft})}{(-0.75414 \text{ kip/ft})}$$

$$E = 10.683 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.0567 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.75414 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.0567 \text{ kipft/ft})) + (4 \times (-0.75414 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4896 \text{ ft}$$

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

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

$$V_{max} = ((-0.75414 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4896 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4896 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.762 \text{ kip}$$

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

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

$$M_{max} = ((-0.75414 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(10.683 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (10.683 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4896 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.199 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0058917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.107 \text{ kipft}) + ((0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017038 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.017038 \text{ kipft/ft})}{(0.0058917 \text{ kip/ft})}$$

$$E = 2.8919 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.017038 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.017038 \text{ kipft/ft})) + (4 \times (0.0058917 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6582 \text{ ft}$$

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

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

$$V_{max} = ((0.0058917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.6582 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.6582 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.034283 \text{ kip}$$

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

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

$$M_{max} = ((0.0058917 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(2.8919 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (2.8919 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.6582 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.097779 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.936 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.332 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.332 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(7.936 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0029665$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.936 \text{ kip} \rightarrow 7936 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7936 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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} = 10.762 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.762 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.097147$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.034283 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.034283 \text{ kip})}{(110.78 \text{ kip})}$ $\text{Ratio} = 0.00030946$	<p>Status: <b>PASS</b> Ratio: <b>0.100</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} = 33.199 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(33.199 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13301$	<p>Status: <b>PASS</b>  Ratio: <b>0.130</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.097779 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.097779 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00039174$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>