

# Your Project Calculations



Project Name: Laurel Flynn

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=Laurel%20Flynn&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/7\\_2023](https://platform.skyciv.com/structural?preload_name=Laurel%20Flynn&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=DV3up0FYTwWCzEU4UIEFB5lpB15znglbruNwNxlqSbFOtFGyQterNtavupX17GO](https://platform.skyciv.com/structural-viewer?project_id=DV3up0FYTwWCzEU4UIEFB5lpB15znglbruNwNxlqSbFOtFGyQterNtavupX17GO)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	2P-17-6TOP-SD-24-L-4Hx5W-IG54
<b>Duty Classification:</b>	SD
<b>Module Width:</b>	44.65 in
<b>Module Length:</b>	67.80in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	5
<b>Total Number of Modules:</b>	20
<b>Desired Tilt Angle:</b>	45
<b>Front Edge Clearance:</b>	5
<b>Total Array Height at Tilt:</b>	15.58 ft
<b>Total Frame Length:</b>	28.50 ft
<b>Frame Weight:</b>	1407 lbs
<b>Array Dimensions N/S:</b>	15.05 ft
<b>Array Dimensions E/W:</b>	28.67 ft
<b>Rail Length:</b>	180.60 in
<b>Rail Spacing:</b>	2.82 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

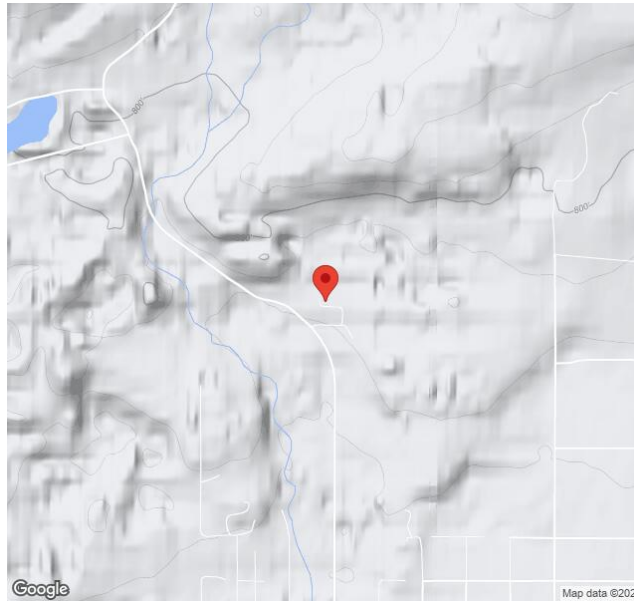
<b>Pole Size:</b>	6in Pipe Sch 80
<b>Pole Length above Grade:</b>	10.32 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	17 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.75 ft Pile 2: 6.75 ft
<b>Foundation Volume:</b>	8.000 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.879488 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	21419 Doodah Ave, Sutton-Alpine, AK 99674, USA
<b>Wind Speed:</b>	113 mph
<b>Snow Load:</b>	0 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.000000 ksf



### Design Disclaimer

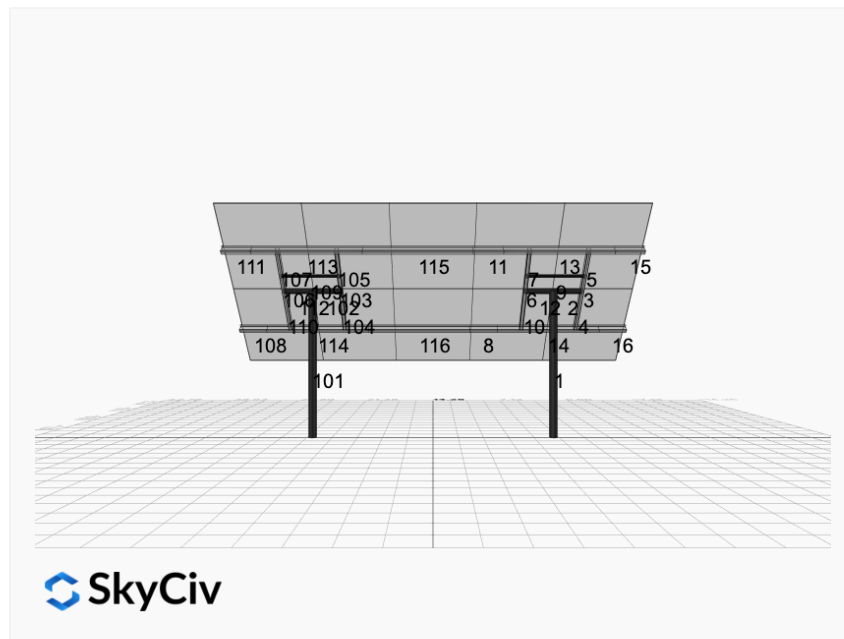
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

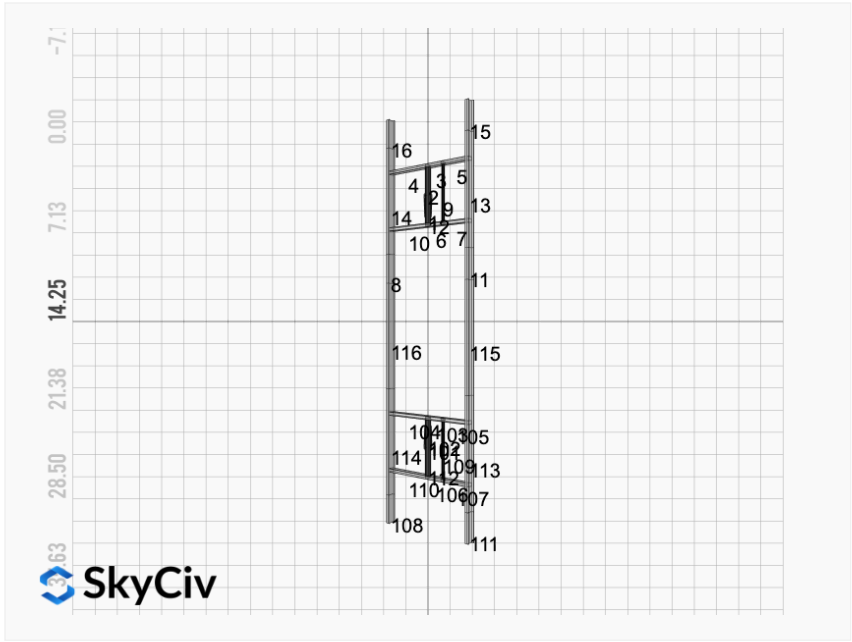
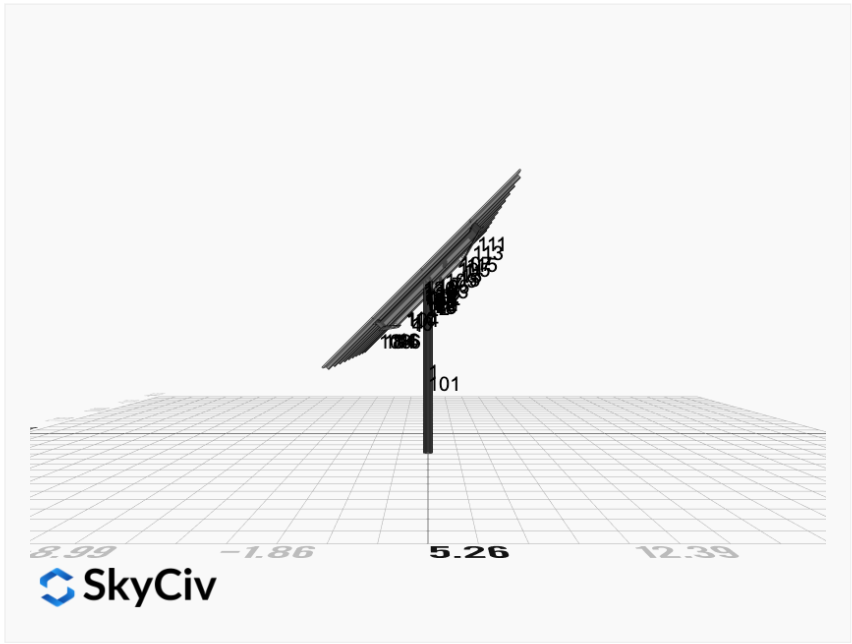
### AutoDesigner Input

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{
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  "snow_load_override": null,
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  "project_id": "Laurel Flynn",
  "site_address": "21419 Doodah Ave, Sutton-Alpine, AK 99674, USA",
  "module_width": 44.65,
  "module_length": 67.8,
  "number_rows": 4,
  "number_columns": 5,
  "pole_mount_section": "4_40",
  "core_pipe_width": 65,
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  "core_beam_height": 65,
  "core_beam_section": "HSS3x2x1/8",
  "main_pipe_section": "2_12GA",
  "pole_spacing": 15,
  "tilt_angle": 45,
  "ground_clearance": 5,
  "risk_category": "I",
  "exposure_category": "B",
  "frame_duty_override": "auto",
  "pole_override": "auto",
  "soil_type": "sand",
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### Design Notes:

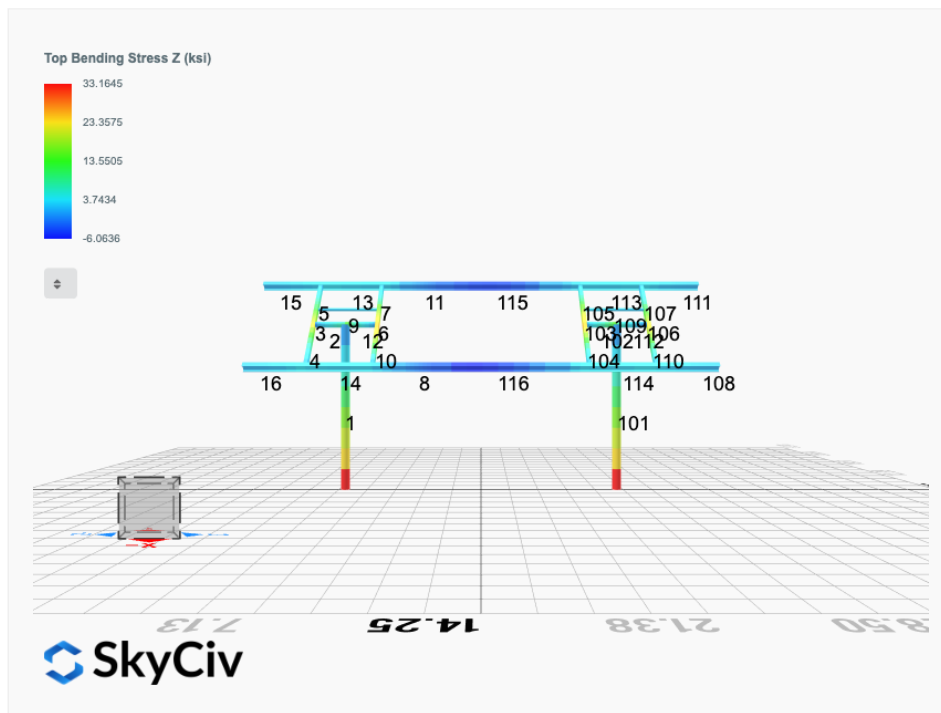
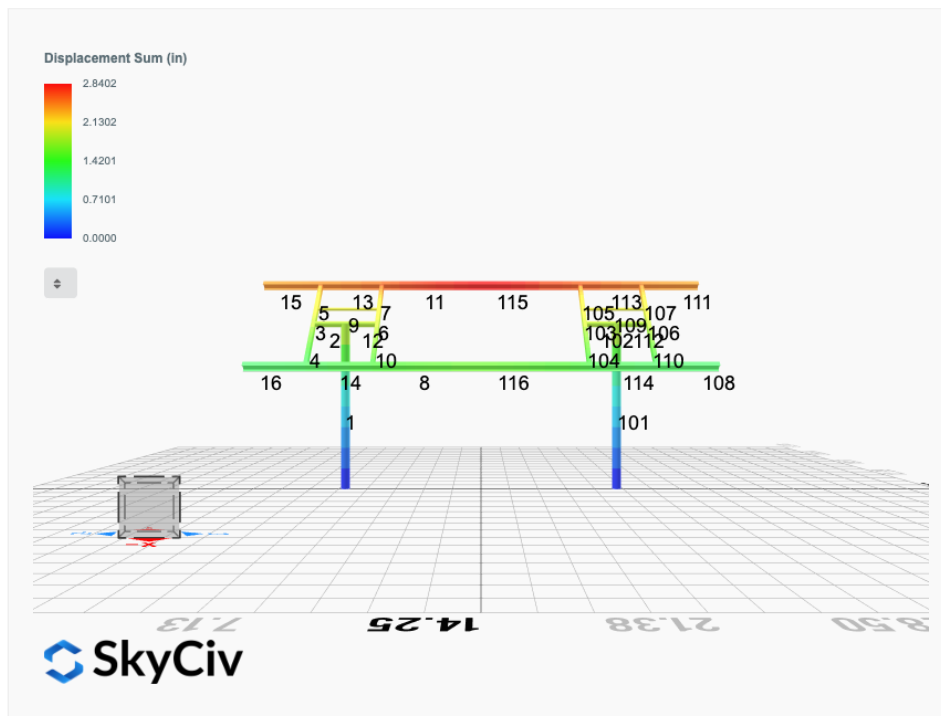
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

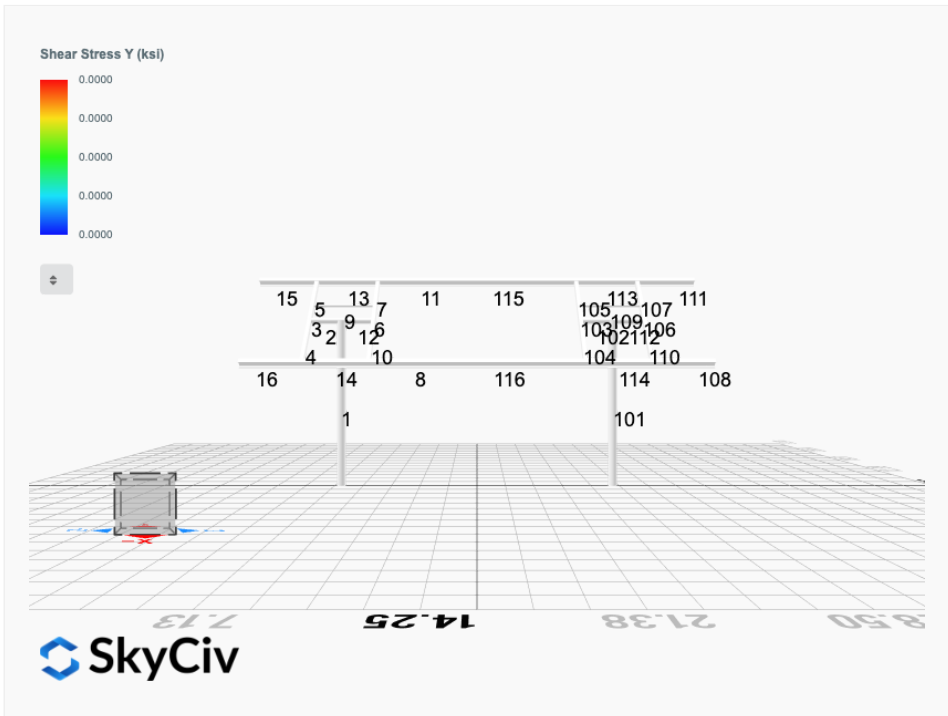
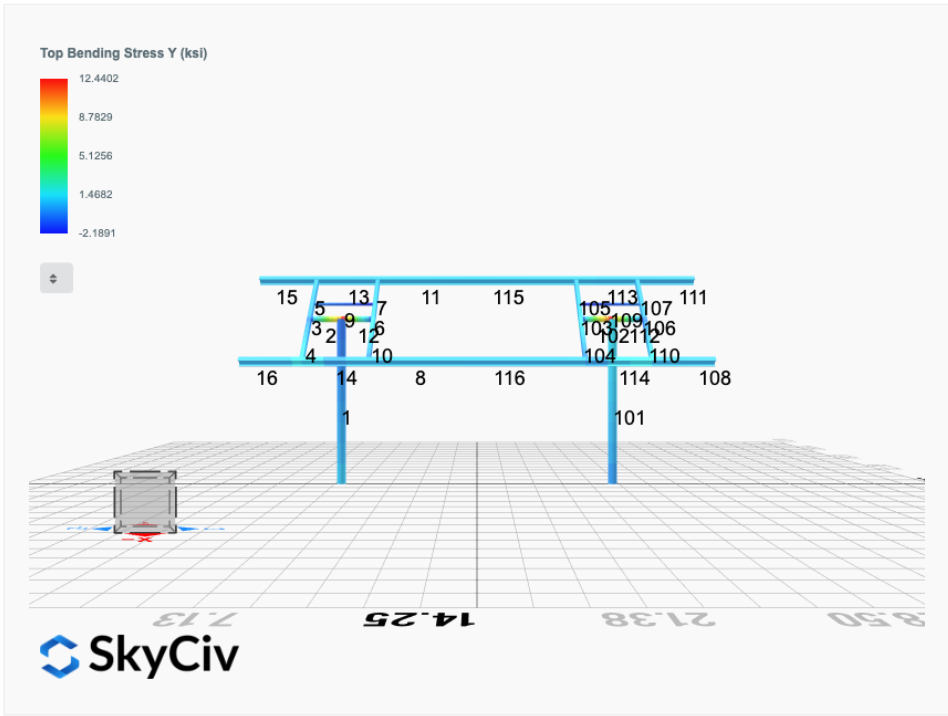


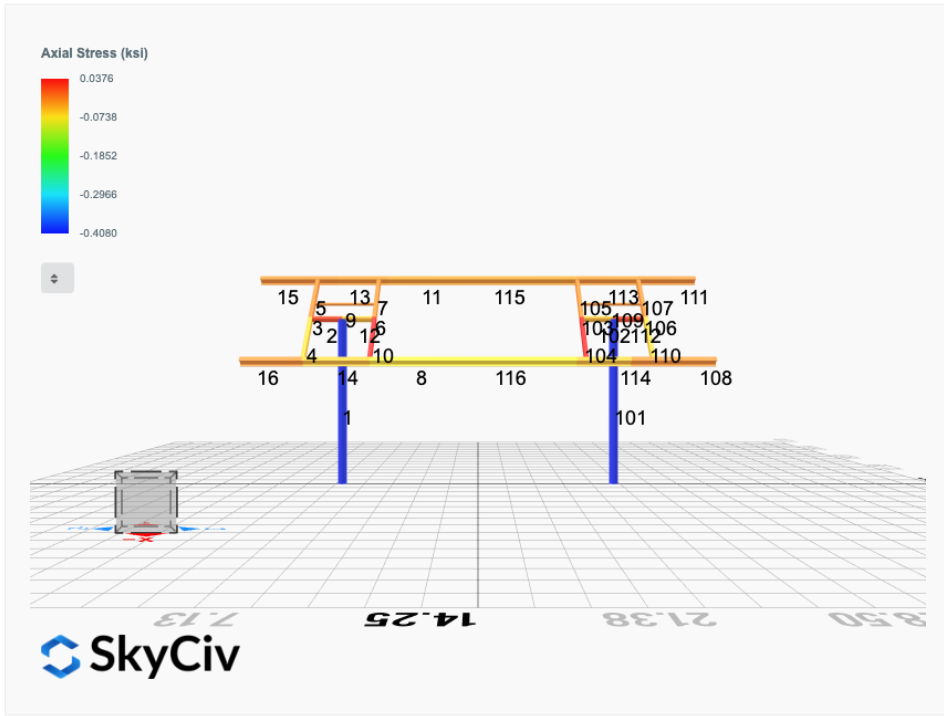




## FEM Results (Envelope Worst Case for each member)







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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 2. D + L	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 3. D + (S or Lr or R)	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 3. D + (S or Lr or R)	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 5b. D + 0.7E	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.7161	0.0473	0.1471	-0.0476	0.0168
ULS: 8. 0.6D + 0.7E	0.0000	1.0297	0.0284	0.0883	-0.0285	0.0101
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8441	4.5602	0.1967	0.5896	-0.5419	30.9858
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.8441	4.5602	0.1967	0.5896	-0.5419	30.9858
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0575	-0.3413	-0.0602	-0.1705	0.3092	-20.2787
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8154	-0.0993	-0.0483	-0.1355	0.2698	-23.5118
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1331	3.8492	0.1594	0.4790	-0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1331	3.8492	0.1594	0.4790	-0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5431	0.1730	-0.0333	-0.0911	0.2200	-15.2048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3615	0.3546	-0.0244	-0.0648	0.1904	-17.6296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1331	3.8492	0.1594	0.4790	-0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1331	3.8492	0.1594	0.4790	-0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5431	0.1730	-0.0333	-0.0911	0.2200	-15.2048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3615	0.3546	-0.0244	-0.0648	0.1904	-17.6296
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8441	3.8738	0.1778	0.5308	-0.5228	30.9791
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.8441	3.8738	0.1778	0.5308	-0.5228	30.9791
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0575	-1.0278	-0.0791	-0.2293	0.3282	-20.2855
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8154	-0.7857	-0.0672	-0.1943	0.2888	-23.5185

### 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	6.7996
Shear X	-4.7402
Shear Z	0.3056
Moment X	0.9144
Moment Y (Twist)	0.8797
Moment Z	52.1572

### 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	4.5602
Shear X	-2.8441
Shear Z	0.1967
Moment X	0.5896
Moment Y (Twist)	0.5419
Moment Z	30.9858

## Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 2. D + L	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 3. D + (S or Lr or R)	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 3. D + (S or Lr or R)	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 5b. D + 0.7E	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.7161	-0.0473	-0.1471	0.0476	0.0168
ULS: 8. 0.6D + 0.7E	-0.0000	1.0297	-0.0284	-0.0883	0.0285	0.0101
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8441	4.5602	-0.1967	-0.5896	0.5419	30.9858
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.8441	4.5602	-0.1967	-0.5896	0.5419	30.9858
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0575	-0.3413	0.0602	0.1705	-0.3092	-20.2787
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8154	-0.0993	0.0483	0.1355	-0.2697	-23.5118
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1331	3.8492	-0.1594	-0.4790	0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1331	3.8492	-0.1594	-0.4790	0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5431	0.1730	0.0333	0.0911	-0.2200	-15.2048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3615	0.3546	0.0244	0.0648	-0.1904	-17.6296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1331	3.8492	-0.1594	-0.4790	0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1331	3.8492	-0.1594	-0.4790	0.4183	23.2436
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5431	0.1730	0.0333	0.0911	-0.2200	-15.2048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3615	0.3546	0.0244	0.0648	-0.1904	-17.6296
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8441	3.8738	-0.1778	-0.5308	0.5228	30.9791
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.8441	3.8738	-0.1778	-0.5308	0.5228	30.9791
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0575	-1.0278	0.0791	0.2293	-0.3282	-20.2854
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8154	-0.7857	0.0672	0.1943	-0.2888	-23.5185

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 Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	6.7996
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Shear Z	-0.3056
Moment X	-0.9145
Moment Y (Twist)	0.8795
Moment Z	52.1581

#### 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	4.5602
Shear X	-2.8441
Shear Z	-0.1967
Moment X	-0.5896
Moment Y (Twist)	0.5419
Moment Z	30.9858

## Project Details

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

User Name: sales@mtsolar.us  
 Project Name: Laurel Flynn  
 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

8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K <sub>z</sub> L (ft)	K <sub>y</sub> L (ft)	L <sub>b</sub> (ft)	C <sub>b</sub>	L S T	L S C	L D
1	8	21.67	21.67	10.32	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17	300	200	1
4	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.64,1.67,1.67,1.65,1.81,1.67,1.67,1.65,1.81,1.67,1.67,1.66,1.65	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.66,1.66	300	200	1
8	18	1.33	1.33	2.05	1.30,1.30,1.30,1.30,1.30,1.30,1.29,1.29,1.29,1.23,1.29,1.29,1.29,1.23,1.29,1.29,1.28,1.36,1.29,1.29,1.28,1.36,1.29,1.29,1.29,1.24	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.66,2.02,1.67,1.67,1.66,1.66	300	200	1
11	18	1.33	1.33	2.05	1.31,1.31,1.31,1.31,1.31,1.31,1.30,1.30,1.29,1.30,1.30,1.30,1.29,1.30,1.30,1.30,1.28,1.30,1.30,1.30,1.28,1.30,1.30,1.30,1.29,1.30	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.24,1.24,1.29,1.24,1.24,1.24,1.29,1.24,1.24,1.25,1.31,1.24,1.24,1.25,1.31,1.24,1.24,1.24,1.28	300	200	1
14	18	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.21,1.22,1.22,1.24,1.08,1.22,1.22,1.24,1.08,1.22,1.22,1.26,2.05,1.22,1.22,1.26,2.05,1.22,1.22,1.24,1.08	300	200	1
15	18	4.20	4.20	2.00	2.33,2.33	300	200	1
16	18	4.20	4.20	2.00	2.33,2.33	300	200	1
101	8	21.67	21.67	10.32	-	300	200	1
102	4	1.30	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.66,2.02,1.67,1.67,1.66,1.66	300	200	1
105	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.66,1.66	300	200	1
106	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17	300	200	1
107	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1



115	120.60	89.27	19.09	6.45	30.09	45.74
116	120.60	89.27	18.91	6.45	30.09	45.74

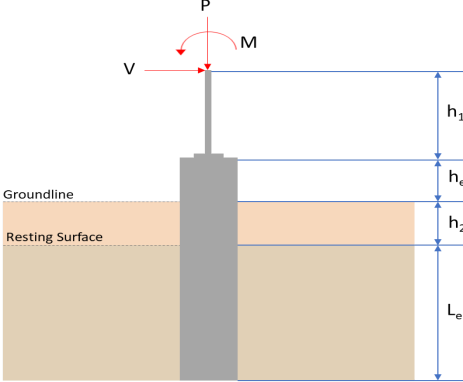
## Design Ratio

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	0.050	0.838	0.036	0.042	0.003	0.878	#13	0.592	Not Required	Pass
2	0.001	0.269	0.246	0.064	0.052	0.515	#13	0.052	Not Required	Pass
3	0.004	0.637	0.039	0.063	0.005	0.656	#13	0.044	Not Required	Pass
4	0.004	0.565	0.088	0.057	0.012	0.654	#13	0.078	Not Required	Pass
5	0.004	0.395	0.040	0.064	0.005	0.400	#13	0.073	Not Required	Pass
6	0.005	0.781	0.079	0.080	0.013	0.862	#13	0.044	Not Required	Pass
7	0.005	0.485	0.092	0.078	0.013	0.508	#13	0.073	Not Required	Pass
8	0.002	0.133	0.029	0.041	0.005	0.159	#13	0.088	Not Required	Pass
9	0.000	0.057	0.068	0.002	0.002	0.125	#13	0.198	Not Required	Pass
10	0.006	0.702	0.100	0.071	0.014	0.713	#13	0.078	Not Required	Pass
11	0.001	0.146	0.029	0.045	0.005	0.169	#13	0.088	Not Required	Pass
12	0.001	0.407	0.300	0.086	0.059	0.708	#13	0.052	Not Required	Pass
13	0.001	0.119	0.091	0.062	0.006	0.156	#13	0.265	Not Required	Pass
14	0.002	0.109	0.091	0.056	0.006	0.144	#13	0.177	Not Required	Pass
15	0.000	0.025	0.014	0.019	0.002	0.036	#13	Not Required	Not Required	Pass
16	0.000	0.022	0.014	0.017	0.002	0.034	#13	Not Required	Not Required	Pass
101	0.050	0.838	0.036	0.042	0.003	0.878	#13	0.592	Not Required	Pass
102	0.001	0.407	0.300	0.086	0.059	0.708	#13	0.052	Not Required	Pass
103	0.005	0.781	0.079	0.080	0.013	0.862	#13	0.044	Not Required	Pass
104	0.006	0.702	0.100	0.070	0.014	0.713	#13	0.078	Not Required	Pass
105	0.005	0.485	0.092	0.078	0.013	0.508	#13	0.073	Not Required	Pass
106	0.004	0.637	0.039	0.063	0.005	0.656	#13	0.044	Not Required	Pass
107	0.004	0.395	0.040	0.064	0.005	0.400	#13	0.073	Not Required	Pass
108	0.000	0.022	0.014	0.017	0.002	0.034	#13	Not Required	Not Required	Pass
109	0.000	0.056	0.068	0.002	0.002	0.125	#13	0.198	Not Required	Pass
110	0.004	0.565	0.088	0.057	0.012	0.653	#13	0.078	Not Required	Pass
111	0.000	0.025	0.014	0.019	0.002	0.036	#13	Not Required	Not Required	Pass
112	0.001	0.269	0.246	0.064	0.052	0.515	#13	0.052	Not Required	Pass
113	0.001	0.119	0.091	0.062	0.006	0.156	#13	0.177	Not Required	Pass
114	0.002	0.109	0.091	0.056	0.006	0.144	#13	0.265	Not Required	Pass
115	0.002	0.231	0.054	0.045	0.005	0.276	#13	0.321	Not Required	Pass
116	0.002	0.210	0.054	0.041	0.005	0.258	#13	0.321	Not Required	Pass

## Definitions

Φ <sub>t</sub>	Safety factor for tensile
Φ <sub>c</sub>	Safety factor for compression
Φ <sub>b</sub>	Safety factor for flexure
Φ <sub>v</sub>	Safety factor for shear
E	Modulus of elasticity
F <sub>y</sub>	Specified minimum yield stress
F <sub>u</sub>	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I <sub>yp</sub>	Moment of inertia about the Y axes
I <sub>zp</sub>	Moment of inertia about the Z axes
I <sub>w</sub>	Warping constant
S <sub>yp</sub>	Plastic section modulus about the Y axis
S <sub>zp</sub>	Plastic section modulus about the Z axis
KL	Effective length
C <sub>n</sub>	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 = 6.75</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>4.560</td> <td>6.800</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.844</td> <td>-4.740</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.197</td> <td>0.306</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.590</td> <td>0.914</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>30.986</td> <td>52.157</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)	4.560	6.800	$V_x$ (kip)	-2.844	-4.740	$V_z$ (kip)	0.197	0.306	$M_x$ (kipft)	0.590	0.914	$M_z$ (kipft)	30.986	52.157	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.844 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.45287 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(30.986 \text{ kipft}) + ((-2.844 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9341 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1143 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.197 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.031369 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.59 \text{ kipft}) + ((0.197 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.09395 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.2767 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1143 \text{ ft}), (2.2767 \text{ ft})]$$

$$L_{e,req} = 6.114 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.114 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.90578$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.56 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.285 \text{ kip/ft}^2$$

$$q = 0.285 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.285 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1425$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.45287 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.9341 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.9341 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9341 \text{ kipft/ft})) + (4 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6644 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9341 \text{ kipft/ft})) + (3 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9341 \text{ kipft/ft})) + (2 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.2115 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.9341 \text{ kipft/ft})) + ((-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.89696 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6644 \text{ ft})}{2}$$

$$p_a = 0.34983 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.2115 \text{ kip/ft}^2)}{(0.34983 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.60458$$

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

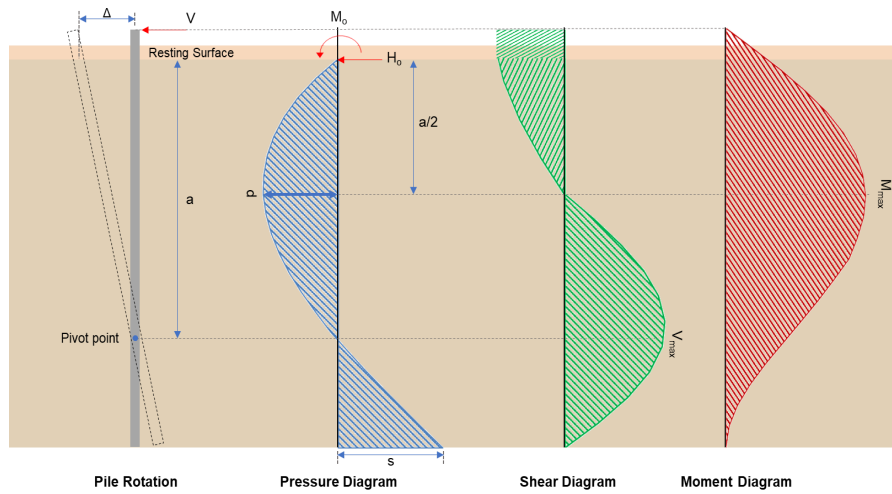
Status: **PASS**  
Ratio: **0.600**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89696 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.88589$	Status: <b>PASS</b> Ratio: <b>0.890</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.031369 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.09395 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.09395 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.031369 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.09395 \text{ kipft/ft})) + (4 \times (0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 4.8377 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.09395 \text{ kipft/ft})) + (3 \times (0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.09395 \text{ kipft/ft})) + (2 \times (0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = 0.023855 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.09395 \text{ kipft/ft})) + ((0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = 0.052628 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.8377 \text{ ft})}{2}$ $p_a = 0.36283 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.023855 \text{ kip/ft}^2)}{(0.36283 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.065748$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.070</b>

$$Ratio = \frac{(0.052628 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.051978$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.74 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.75478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(52.157 \text{ kipft}) + ((-4.74 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3053 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.3053 \text{ kipft/ft})}{(-0.75478 \text{ kip/ft})}$$

$$E = 11.004 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.3053 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.75478 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.3053 \text{ kipft/ft})) + (4 \times (-0.75478 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6633 \text{ ft}$$

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

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

$$V_{max} = ((-0.75478 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6633 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6633 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.7 \text{ kip}$$

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

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

$$M_{max} = ((-0.75478 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(11.004 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.263 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.306 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.048726 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.914 \text{ kipft}) + ((0.306 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14554 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.14554 \text{ kipft/ft})}{(0.048726 \text{ kip/ft})}$$

$$E = 2.9869 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.14554 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.048726 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.14554 \text{ kipft/ft})) + (4 \times (0.048726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8381 \text{ ft}$$

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

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

$$V_{max} = ((0.048726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.8381 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.8381 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.28272 \text{ kip}$$

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

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

$$M_{max} = ((0.048726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(2.9869 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.83701 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(6.8 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.37 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.37 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(6.8 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0025419$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 6.8 \text{ kip} \rightarrow 6800 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6800 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.39 \text{ kip}$$

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

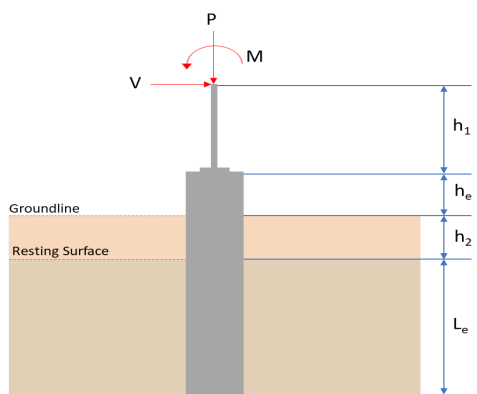
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.39 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.39 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.69 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.7 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.7 \text{ kip})}{(110.69 \text{ kip})}$ $\text{Ratio} = 0.096668$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.28272 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.28272 \text{ kip})}{(110.69 \text{ kip})}$ $\text{Ratio} = 0.0025542$	<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, LFRD)</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} = 34.263 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(34.263 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13727$	<p>Status: <b>PASS</b>  Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.83701 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.83701 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0033534$	<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 = 6.75</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="414 1097 1189 1198"> <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="670 1288 933 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.560</td> <td>6.800</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.844</td> <td>-4.740</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.197</td> <td>-0.306</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.590</td> <td>-0.914</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>30.986</td> <td>52.158</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)	4.560	6.800	$V_x$ (kip)	-2.844	-4.740	$V_z$ (kip)	-0.197	-0.306	$M_x$ (kipft)	-0.590	-0.914	$M_z$ (kipft)	30.986	52.158	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.844 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.45287 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(30.986 \text{ kipft}) + ((-2.844 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9341 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1143 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.197 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.031369 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.59 \text{ kipft}) + ((-0.197 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.09395 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6418 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1143 \text{ ft}), (1.6418 \text{ ft})]$$

$$L_{e,req} = 6.114 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.114 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.90578$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.56 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.285 \text{ kip/ft}^2$$

$$q = 0.285 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.285 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1425$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.45287 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.9341 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.9341 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9341 \text{ kipft/ft})) + (4 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6644 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9341 \text{ kipft/ft})) + (3 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9341 \text{ kipft/ft})) + (2 \times (-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.2115 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.9341 \text{ kipft/ft})) + ((-0.45287 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.89696 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6644 \text{ ft})}{2}$$

$$p_a = 0.34983 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.2115 \text{ kip/ft}^2)}{(0.34983 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.60458$$

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

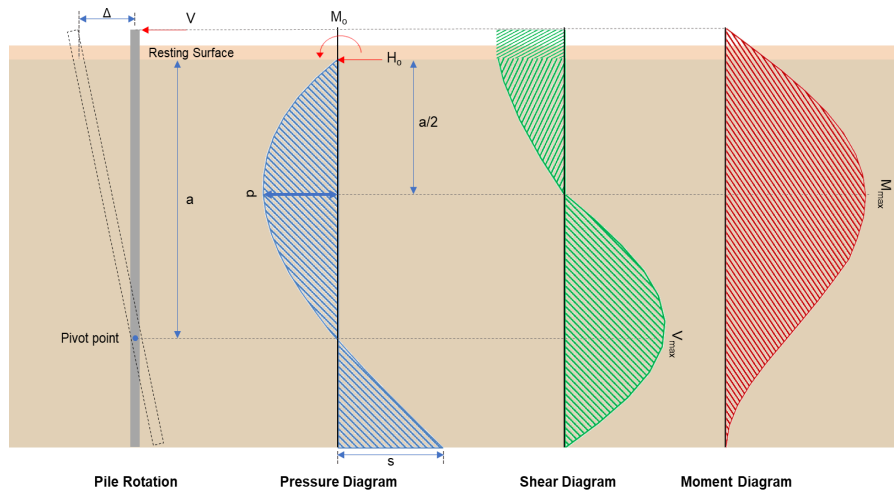
Status: **PASS**  
Ratio: **0.600**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89696 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.88589$	Status: <b>PASS</b> Ratio: <b>0.890</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.031369 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.09395 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.09395 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.031369 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.09395 \text{ kipft/ft})) + (4 \times (-0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 4.8377 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.09395 \text{ kipft/ft})) + (3 \times (-0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.09395 \text{ kipft/ft})) + (2 \times (-0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$ $p = -0.0078221 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.09395 \text{ kipft/ft})) + ((-0.031369 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$ $s = -0.0031402 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.8377 \text{ ft})}{2}$ $p_a = 0.36283 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0078221 \text{ kip/ft}^2)}{(0.36283 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.021559$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$ $p_s = 1.0125 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>-0.020</b>

$$\text{Ratio} = \frac{(-0.0031402 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0031014$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.74 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.75478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(52.158 \text{ kipft}) + ((-4.74 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3054 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.3054 \text{ kipft/ft})}{(-0.75478 \text{ kip/ft})}$$

$$E = 11.004 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.3054 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.75478 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.3054 \text{ kipft/ft})) + (4 \times (-0.75478 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6633 \text{ ft}$$

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

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

$$V_{max} = ((-0.75478 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6633 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6633 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.7 \text{ kip}$$

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

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

$$M_{max} = ((-0.75478 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(11.004 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (11.004 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6633 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.263 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.306 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.048726 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.914 \text{ kipft}) + ((-0.306 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14554 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.14554 \text{ kipft/ft})}{(-0.048726 \text{ kip/ft})}$$

$$E = 2.9869 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.14554 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.048726 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.14554 \text{ kipft/ft})) + (4 \times (-0.048726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8381 \text{ ft}$$

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

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

$$V_{max} = ((-0.048726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.8381 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.8381 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.28272 \text{ kip}$$

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

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

$$M_{max} = ((-0.048726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(2.9869 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.9869 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.8381 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.83701 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(6.8 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.37 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.37 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(6.8 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0025419$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 6.8 \text{ kip} \rightarrow 6800 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6800 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.39 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.39 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.39 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.69 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 10.7 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.7 \text{ kip})}{(110.69 \text{ kip})}$ $\text{Ratio} = 0.096669$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.28272 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.28272 \text{ kip})}{(110.69 \text{ kip})}$ $\text{Ratio} = 0.0025542$	<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, LFRD)</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} = 34.263 \text{ kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(34.263 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.13727$	<p>Status: <b>PASS</b>  Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.83701 \text{ kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.83701 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0033534$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>