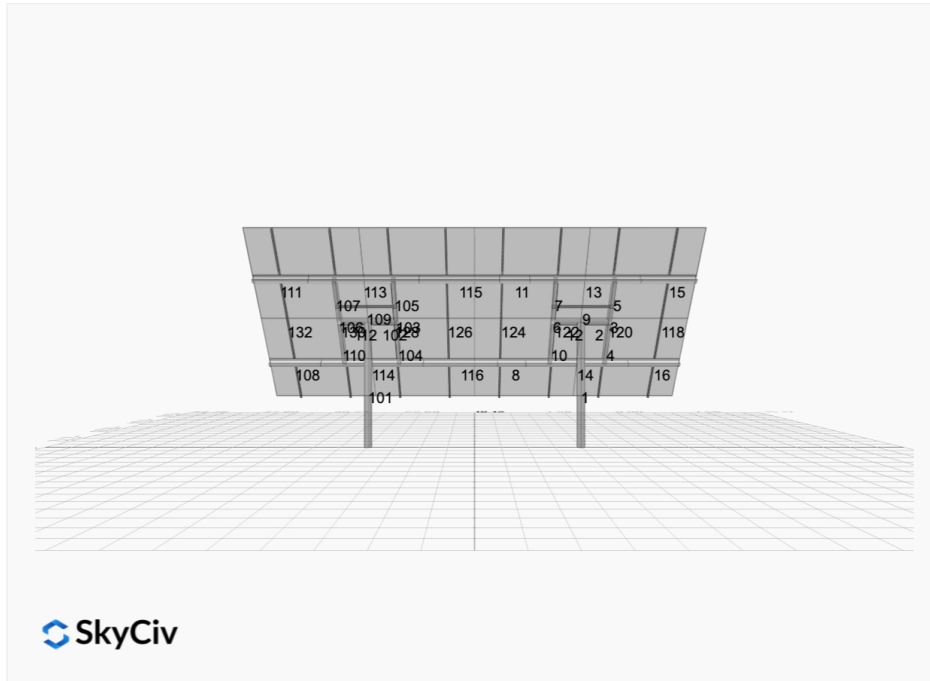


**Project Name:** Wade (16) 580W  
**Location:** 2130 Old Railroad Grade Rd, Libby, MT 59923, USA  
**Unique ID:** 2P-15-6TOP-SD-45-L-4Hx4W-G216  
**Dealer:** \_\_\_\_\_

**Date:** Fri Aug 08 2025  
**Number of Modules:** 16  
**Number of Poles:** 2  
**Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	15.05 ft
<b>Array Dimensions E/W</b>	30.24 ft
<b>Winter Tilt Angle</b>	50
<b>Front Edge Clearance</b>	3 ft

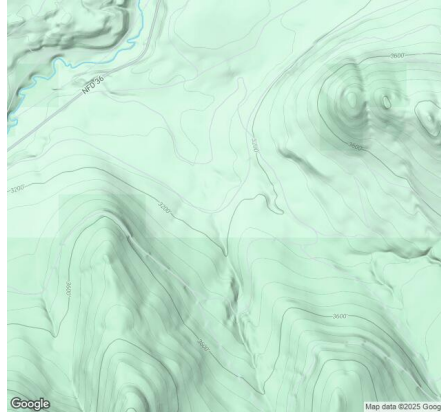
### MT Solar Bill of Materials (2P-15-6TOP-SD-45-L-4Hx4W-G216)

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

### Rail Bill of Materials

Part	Qty
Rails (181in)	8
Rail Attachment	16
Module Mid Clamp	24
Module End Clamp	16
Ground Lug	4

## Site Details:



**Site Address:** 2130 Old Railroad Grade Rd, Libby, MT 59923, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	44.65 in
<b>Module Length:</b>	89.72in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	4
<b>Total Number of Modules:</b>	16
<b>Winter Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	3
<b>Total Array Height at Tilt:</b>	14.53 ft
<b>Total Frame Length:</b>	30.00 ft
<b>Module Info/Notes:</b>	
<b>Array Dimensions N/S:</b>	15.05 ft
<b>Array Dimensions E/W:</b>	30.24 ft
<b>Rail Length:</b>	180.60 in
<b>Rail Spacing:</b>	3.78 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	8.76 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	15 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.75 ft Pile 2: 5.75 ft
<b>Foundation Volume:</b>	6.815 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	2130 Old Railroad Grade Rd, Libby, MT 59923, USA
<b>Wind Speed:</b>	110 mph
<b>Snow Load:</b>	98 psf

### **Design Disclaimer**

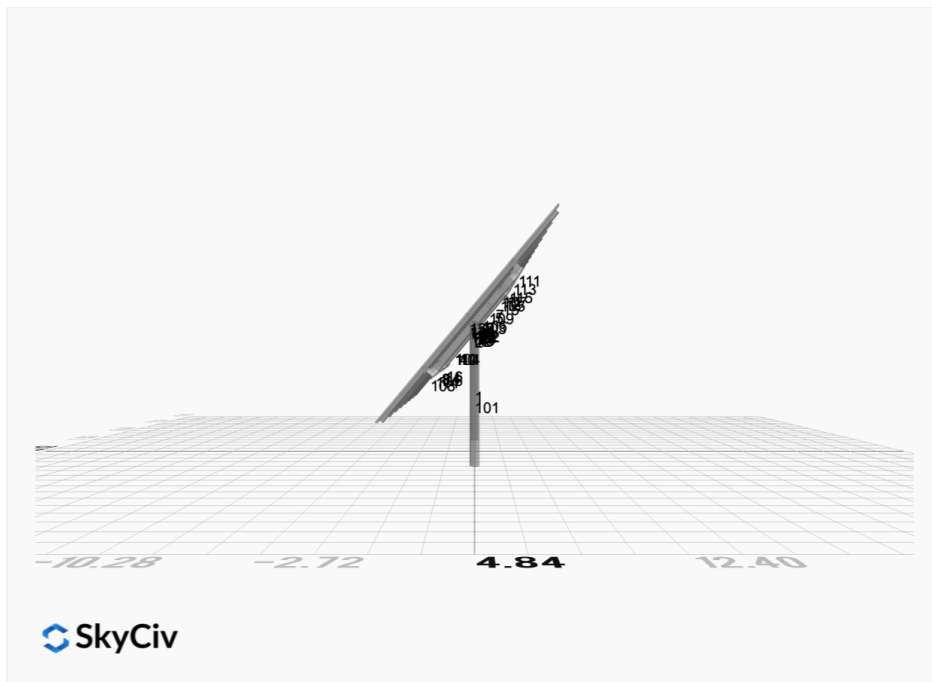
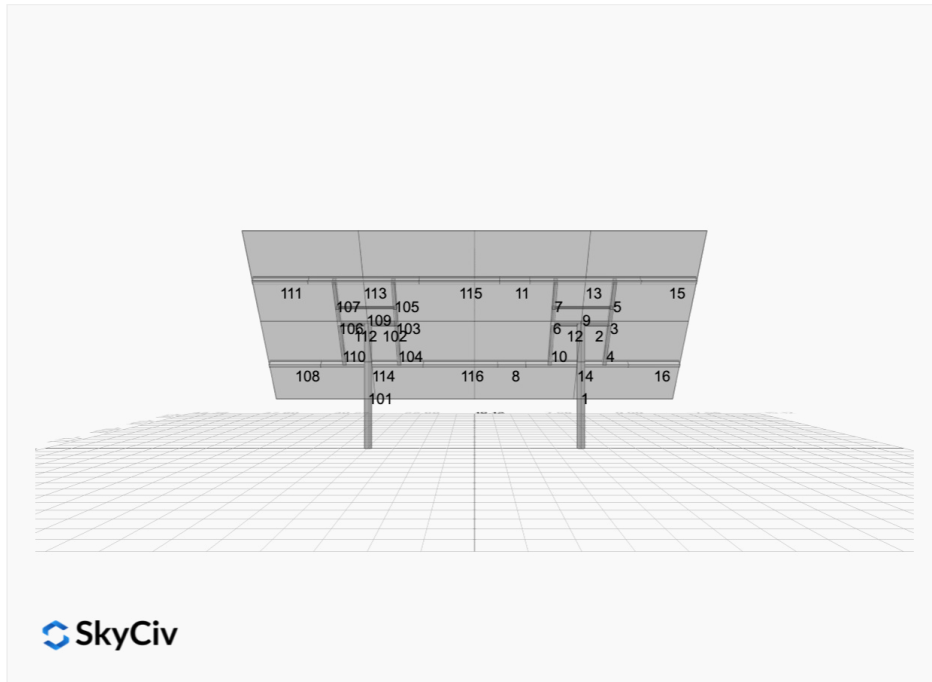
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

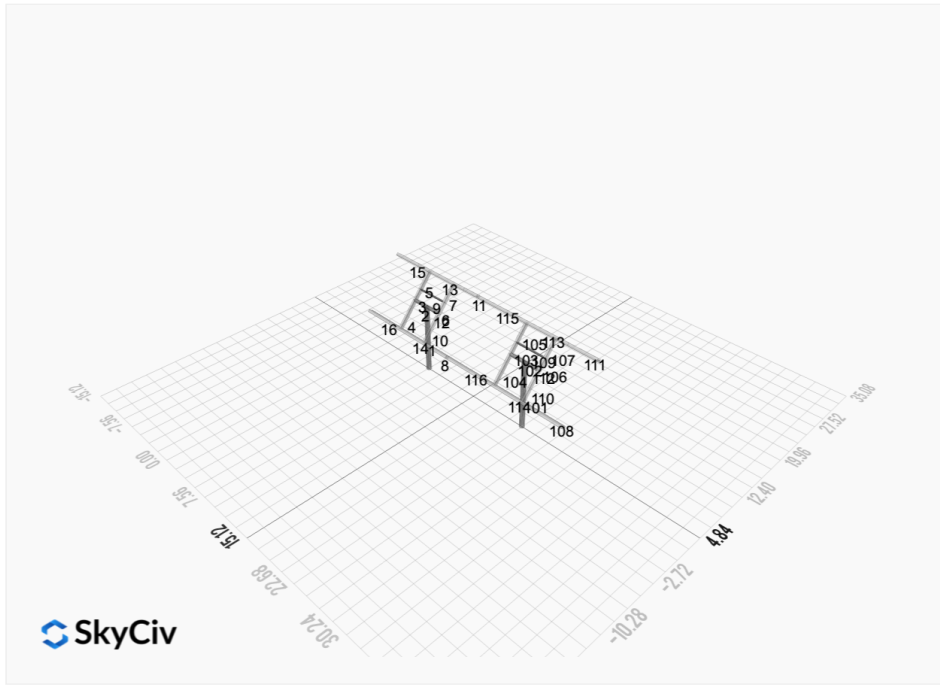
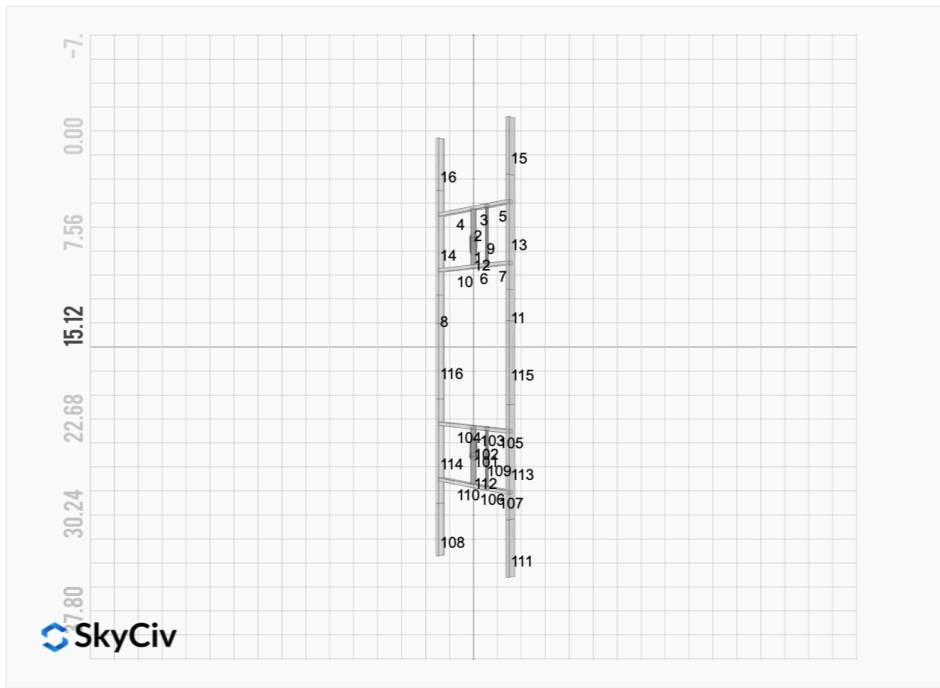
## AutoDesigner Input

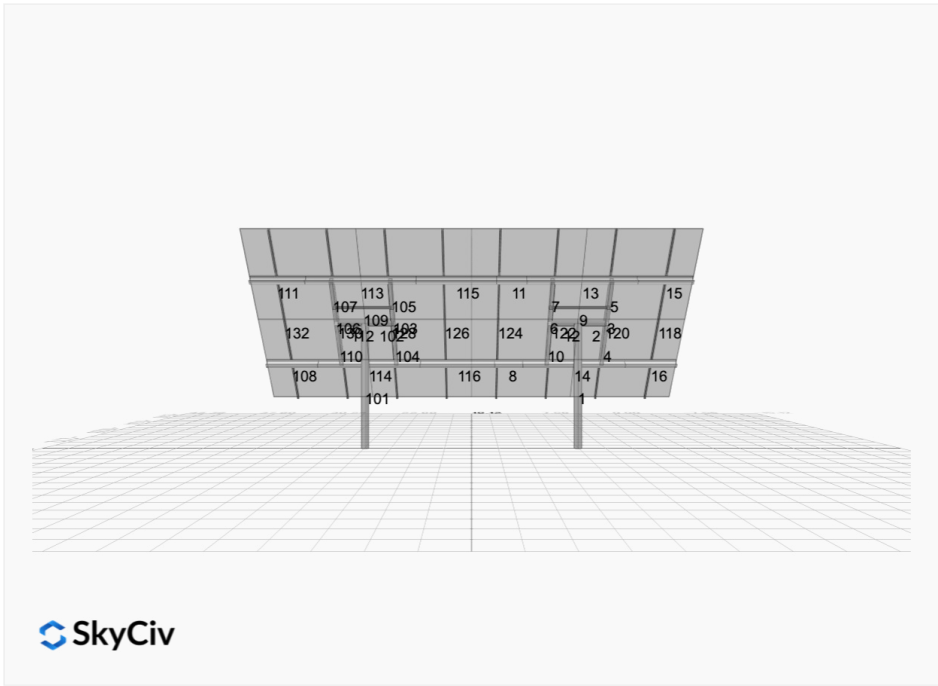
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## Design Notes:

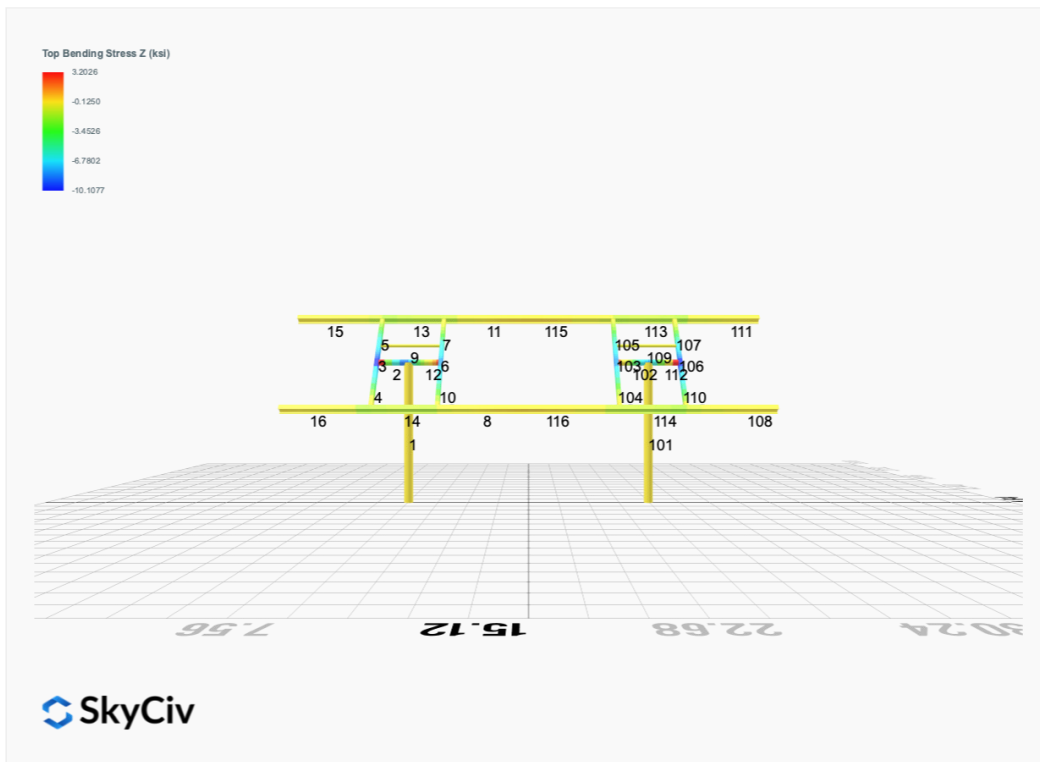
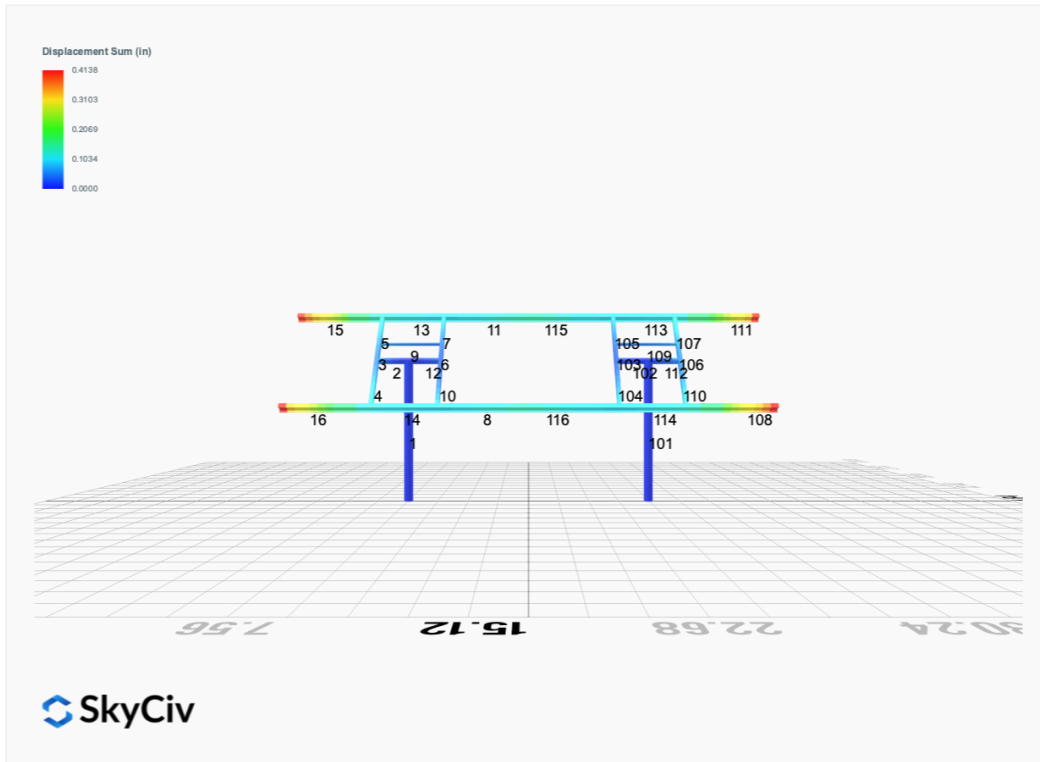
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



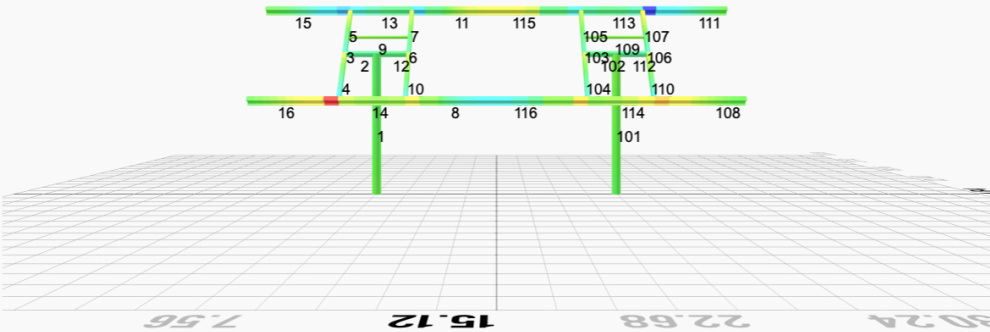




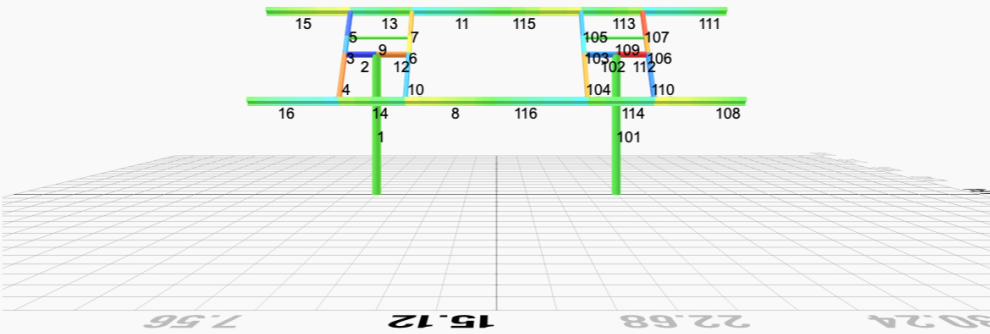
# FEM Results (Envelope Worst Case for each member)

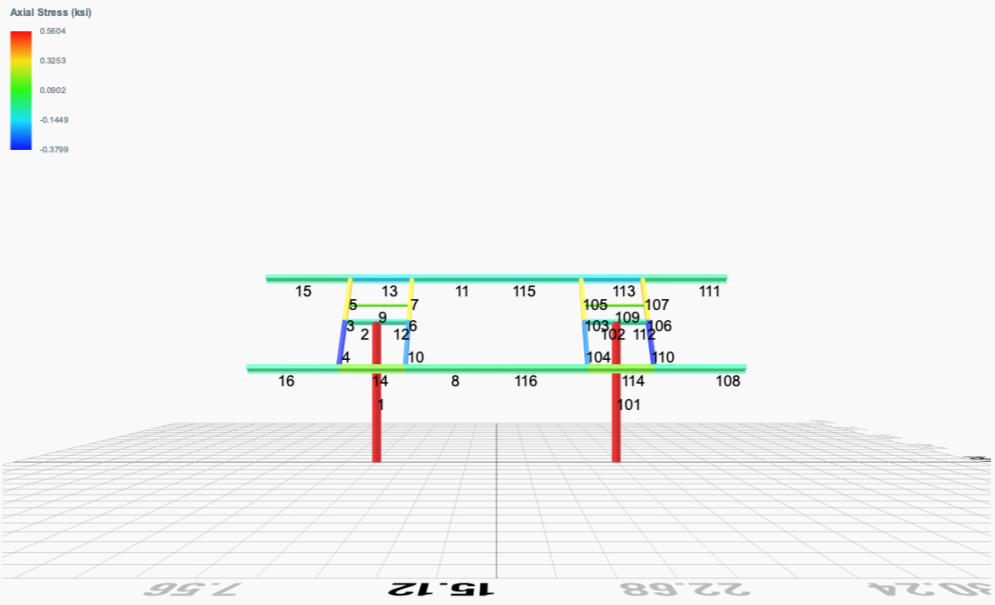


Top Bending Stress Y (ksi)



Shear Stress Y (ksi)





## 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	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 2. D + L	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 3. D + (S or Lr or R)	-0.0000	4.7460	-0.1068	-0.2716	0.2207	0.0337
ULS: 3. D + (S or Lr or R)	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.9641	-0.0879	-0.2237	0.1818	0.0292
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 5b. D + 0.7E	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.9641	-0.0879	-0.2237	0.1818	0.0292
ULS: 8. 0.6D + 0.7E	-0.0000	0.9711	-0.0189	-0.0480	0.0391	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3202	3.5653	-0.1241	-0.3083	0.3598	20.6545
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3202	-0.3284	0.0610	0.1474	-0.2298	-20.0269
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7402	5.4243	-0.1574	-0.3949	0.4027	15.5084
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	3.9641	-0.0879	-0.2237	0.1818	0.0292
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7401	2.5039	-0.0186	-0.0531	-0.0394	-15.0026
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	3.9641	-0.0879	-0.2237	0.1818	0.0292
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7402	3.0786	-0.1009	-0.2512	0.2861	15.4948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7402	0.1583	0.0379	0.0905	-0.1560	-15.0163
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.6184	-0.0315	-0.0801	0.0652	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3202	2.9180	-0.1115	-0.2763	0.3337	20.6483
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.9711	-0.0189	-0.0480	0.0391	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3202	-0.9758	0.0736	0.1794	-0.2559	-20.0331
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.9711	-0.0189	-0.0480	0.0391	0.0093

### 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	8.5686
Shear X	-3.8671
Shear Z	-0.2357
Moment X	-0.5947
Moment Y (Twist)	0.6458
Moment Z	35.0493

### 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	5.4243
Shear X	-2.3202
Shear Z	-0.1574
Moment X	-0.3949
Moment Y (Twist)	0.4027
Moment Z	20.6545

## 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	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 2. D + L	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 3. D + (S or Lr or R)	0.0000	4.7460	0.1068	0.2714	-0.2211	0.0332
ULS: 3. D + (S or Lr or R)	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.9641	0.0879	0.2236	-0.1822	0.0287

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 5b. D + 0.7E	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.9641	0.0879	0.2236	-0.1822	0.0287
ULS: 8. 0.6D + 0.7E	0.0000	0.9711	0.0189	0.0480	-0.0392	0.0092
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3202	3.5653	0.1241	0.3078	-0.3602	20.6540
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3202	-0.3284	-0.0610	-0.1470	0.2300	-20.0267
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7401	5.4242	0.1574	0.3944	-0.4034	15.5077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.9641	0.0879	0.2236	-0.1822	0.0287
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7402	2.5040	0.0186	0.0533	0.0393	-15.0028
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.9641	0.0879	0.2236	-0.1822	0.0287
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7401	3.0786	0.1009	0.2508	-0.2865	15.4944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7401	0.1583	-0.0379	-0.0902	0.1562	-15.0161
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6184	0.0315	0.0800	-0.0653	0.0154
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3202	2.9179	0.1115	0.2758	-0.3341	20.6479
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.9711	0.0189	0.0480	-0.0392	0.0092
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3202	-0.9758	-0.0736	-0.1790	0.2561	-20.0328
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.9711	0.0189	0.0480	-0.0392	0.0092

### 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	8.5685
Shear X	-3.8669
Shear Z	0.2357
Moment X	0.5941
Moment Y (Twist)	0.6469
Moment Z	35.0491

### 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	5.4242
Shear X	-2.3202
Shear Z	0.1574
Moment X	0.3944
Moment Y (Twist)	0.4034
Moment Z	20.6540

## Project Details

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

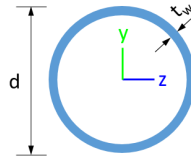


## Design Input Information

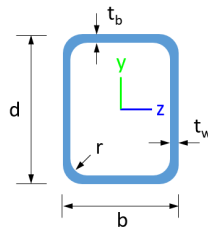
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$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
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Design Materials			
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1	29000	50	65

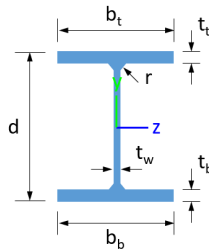
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	$t_w$ (in)	$t_b$ (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
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5	0.018	0.408	0.272	0.065	0.052	0.451	#21	0.073	Not Required	Pass
6	0.016	0.570	0.117	0.056	0.020	0.666	#21	0.044	Not Required	Pass
7	0.016	0.354	0.183	0.057	0.036	0.386	#21	0.073	Not Required	Pass
8	0.002	0.041	0.112	0.029	0.014	0.127	#21	0.088	Not Required	Pass
9	0.006	0.047	0.081	0.002	0.002	0.121	#13	0.198	Not Required	Pass
10	0.014	0.566	0.205	0.057	0.038	0.724	#21	0.078	Not Required	Pass
11	0.002	0.040	0.113	0.029	0.014	0.130	#21	0.059	Not Required	Pass
12	0.002	0.353	0.226	0.089	0.043	0.515	#13	0.052	Not Required	Pass
13	0.007	0.186	0.393	0.043	0.020	0.567	#21	0.177	Not Required	Pass
14	0.009	0.190	0.393	0.043	0.020	0.567	#21	0.177	Not Required	Pass
15	0.000	0.071	0.183	0.029	0.014	0.251	#21	Not Required	Not Required	Pass
16	0.000	0.071	0.183	0.029	0.014	0.251	#21	Not Required	Not Required	Pass
101	0.069	0.829	0.035	0.051	0.003	0.869	#13	0.492	Not Required	Pass
102	0.002	0.353	0.226	0.089	0.043	0.515	#13	0.052	Not Required	Pass
103	0.016	0.570	0.117	0.056	0.020	0.666	#21	0.044	Not Required	Pass
104	0.014	0.566	0.205	0.057	0.038	0.724	#21	0.078	Not Required	Pass
105	0.016	0.355	0.183	0.057	0.036	0.386	#21	0.073	Not Required	Pass
106	0.018	0.657	0.172	0.066	0.036	0.812	#21	0.044	Not Required	Pass
107	0.018	0.408	0.272	0.065	0.052	0.451	#21	0.073	Not Required	Pass
108	0.000	0.071	0.183	0.029	0.014	0.251	#21	Not Required	Not Required	Pass
109	0.006	0.047	0.081	0.002	0.002	0.121	#13	0.198	Not Required	Pass
110	0.018	0.654	0.260	0.066	0.045	0.799	#21	0.078	Not Required	Pass
111	0.000	0.071	0.183	0.029	0.014	0.251	#21	Not Required	Not Required	Pass
112	0.001	0.444	0.266	0.106	0.048	0.644	#13	0.079	Not Required	Pass
113	0.007	0.186	0.393	0.043	0.020	0.567	#21	0.177	Not Required	Pass
114	0.009	0.190	0.393	0.043	0.020	0.567	#21	0.265	Not Required	Pass
115	0.002	0.040	0.151	0.029	0.014	0.182	#21	0.156	Not Required	Pass
116	0.002	0.041	0.150	0.029	0.014	0.179	#21	0.235	Not Required	Pass

## Definitions

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

(P,M <sub>z</sub> ,M <sub>y</sub> )	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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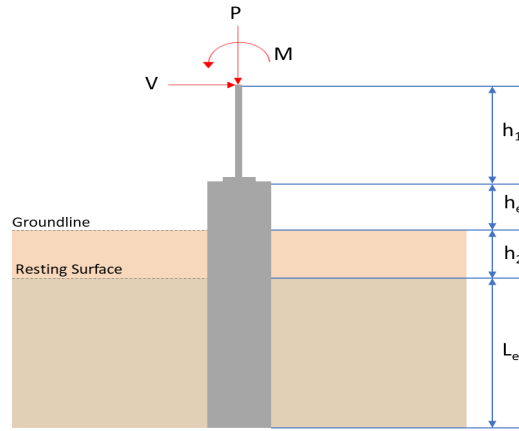
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5.75$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	5.424	8.569
$V_x$ (kip)	-2.320	-3.867
$V_z$ (kip)	-0.157	-0.236
$M_x$ (kipft)	-0.395	-0.595
$M_z$ (kipft)	20.655	35.049

### Material Properties

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

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

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36943 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(20.655 \text{ kipft}) + ((-2.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.289 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.2695 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.157 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.395 \text{ kipft}) + ((-0.157 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

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:

$L_{e,z} = 1.4251 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(5.2695 \text{ ft}), (1.4251 \text{ ft})]$$

$$L_{e,req} = 5.269 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(5.269 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.91635$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.424 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.339 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.339 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1695$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.36943 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.289 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.289 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.289 \text{ kipft/ft})) + (4 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9776 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.289 \text{ kipft/ft})) + (3 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.289 \text{ kipft/ft})) + (2 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18578 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.289 \text{ kipft/ft})) + ((-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.80826 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9776 \text{ ft})}{2}$$

$$p_a = 0.29832 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18578 \text{ kip/ft}^2)}{(0.29832 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.62276$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.80826 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93711$$

Status: **PASS**  
Ratio: **0.620**

Status: **PASS**  
Ratio: **0.940**

#### Considering z-direction:

$H_o = -0.025 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.062898 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.062898 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.025 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.062898 \text{ kipft/ft})) + (4 \times (-0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1226 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.062898 \text{ kipft/ft})) + (3 \times (-0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.062898 \text{ kipft/ft})) + (2 \times (-0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = -0.0074103 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.062898 \text{ kipft/ft})) + ((-0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = -0.0032582 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1226 \text{ ft})}{2}$$

$$p_a = 0.3092 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0074103 \text{ kip/ft}^2)}{(0.3092 \text{ kip/ft}^2)}$$

$$Ratio = -0.023966$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

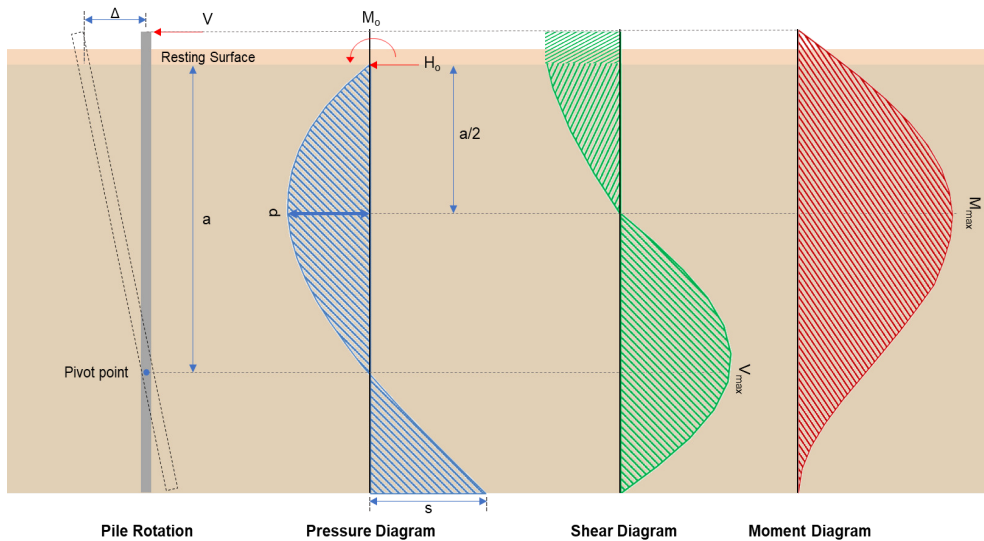
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0032582 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.0037776$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.867 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.61576 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(35.049 \text{ kipft}) + ((-3.867 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5811 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.5811 \text{ kipft/ft})}{(-0.61576 \text{ kip/ft})}$$

$$E = 9.0636 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.5811 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.5811 \text{ kipft/ft})) + (4 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (5.5811 \text{ kip/ft})) + (4 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9758 \text{ ft}$$

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

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

$$V_{max} = ((-0.61576 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9758 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9758 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.4942 \text{ kip}$$

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

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

$$M_{max} = ((-0.61576 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(9.0636 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.136 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.236 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.03758 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.595 \text{ kipft}) + ((-0.236 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.094745 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.094745 \text{ kipft/ft})}{(-0.03758 \text{ kip/ft})}$$

$$E = 2.5212 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.094745 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.03758 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.094745 \text{ kipft/ft})) + (4 \times (-0.03758 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1224 \text{ ft}$$

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

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

$$V_{max} = ((-0.03758 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.5212 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1224 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.5212 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1224 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21698 \text{ kip}$$

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

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

$$M_{max} = ((-0.03758 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.5212 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1224 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.5212 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1224 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.5212 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1224 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.54686 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

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

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

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

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

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.569 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.311 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.311 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = Min[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Min[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p>Ratio - Capacity</p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(8.569 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0032031</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.569 \text{ kip} \rightarrow 8569 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.63 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.63 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.63 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.63 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.84 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.4942 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.4942 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.076635$$

**Considering z-direction:**

$V_{max} = 0.21698 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.21698 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.0019576$$

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

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 23.136 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(23.136 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.092694$$

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

**Considering z-direction:**

$M_{max} = 0.54686 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.54686 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.002191$$

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

REFERENCES	CALCULATIONS	RESULTS
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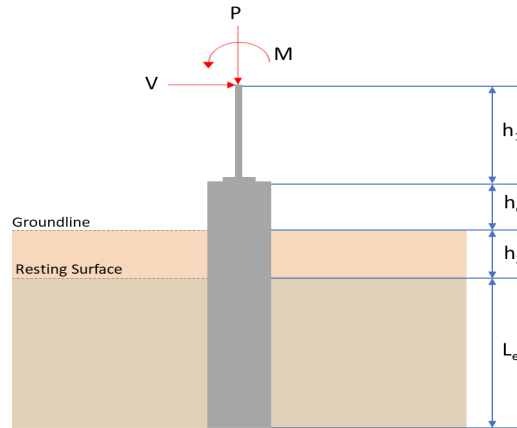
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5.75$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	5.424	8.569
$V_x$ (kip)	-2.320	-3.867
$V_z$ (kip)	0.157	0.236
$M_x$ (kipft)	0.394	0.594
$M_z$ (kipft)	20.654	35.049

### Material Properties

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

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

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36943 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(20.654 \text{ kipft}) + ((-2.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2889 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.2693 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.157 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.394 \text{ kipft}) + ((0.157 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.062739 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0018 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.2693 \text{ ft}), (2.0018 \text{ ft})]$$

$$L_{e,req} = 5.269 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.269 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.91635$$

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

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.424 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.339 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.339 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.1695$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.36943 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.2889 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.2889 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.2889 \text{ kipft/ft})) + (4 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9776 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2889 \text{ kipft/ft})) + (3 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.2889 \text{ kipft/ft})) + (2 \times (-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18576 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.2889 \text{ kipft/ft})) + ((-0.36943 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.8082 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9776 \text{ ft})}{2}$$

$$p_a = 0.29832 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18576 \text{ kip/ft}^2)}{(0.29832 \text{ kip/ft}^2)}$$

$$Ratio = 0.62269$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.8082 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.93704$$

Status: **PASS**  
Ratio: **0.620**

Status: **PASS**  
Ratio: **0.940**

#### Considering z-direction:

$H_o = 0.025 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.062739 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.062739 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.025 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.062739 \text{ kipft/ft})) + (4 \times (0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1229 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.062739 \text{ kipft/ft})) + (3 \times (0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.062739 \text{ kipft/ft})) + (2 \times (0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.022193 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.062739 \text{ kipft/ft})) + ((0.025 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.048858 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1229 \text{ ft})}{2}$$

$$p_a = 0.30922 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.022193 \text{ kip/ft}^2)}{(0.30922 \text{ kip/ft}^2)}$$

$$Ratio = 0.07177$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

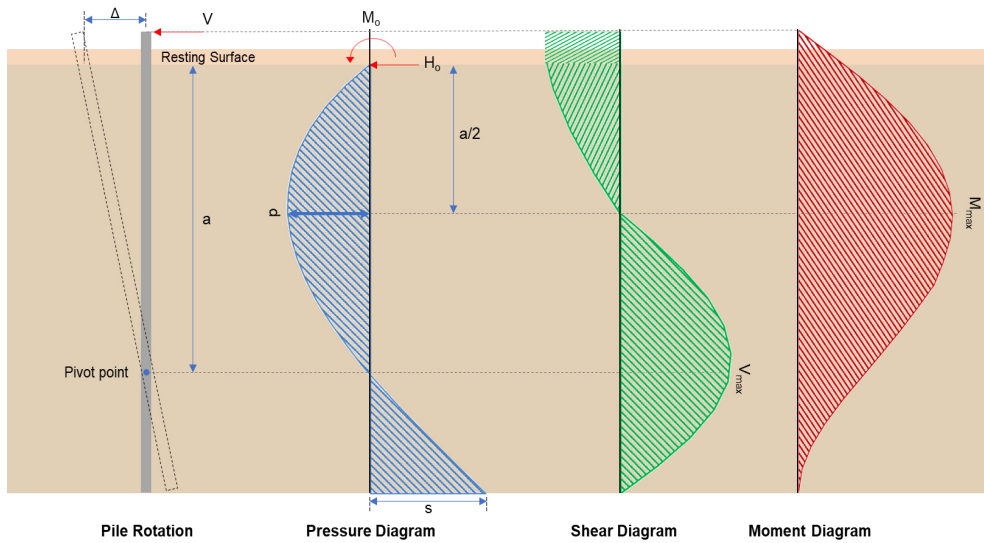
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.048858 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.056647$$

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

Status: **PASS**  
Ratio: **0.060**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.867 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.61576 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(35.049 \text{ kipft}) + ((-3.867 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5811 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.5811 \text{ kipft/ft})}{(-0.61576 \text{ kip/ft})}$$

$$E = 9.0636 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.5811 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.5811 \text{ kipft/ft})) + (4 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.61576 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.5811 \text{ kip/ft})) + (4 \times (-0.61576 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9758 \text{ ft}$$

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

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

$$V_{max} = ((-0.61576 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9758 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9758 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.4942 \text{ kip}$$

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

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

$$M_{max} = ((-0.61576 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(9.0636 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (9.0636 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9758 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.136 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.236 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.03758 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.594 \text{ kipft}) + ((0.236 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.094586 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.094586 \text{ kipft/ft})}{(0.03758 \text{ kip/ft})}$$

$$E = 2.5169 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.094586 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.03758 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.094586 \text{ kipft/ft})) + (4 \times (0.03758 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1226 \text{ ft}$$

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

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

$$V_{max} = ((0.03758 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.5169 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1226 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.5169 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1226 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21679 \text{ kip}$$

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

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

$$M_{max} = ((0.03758 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.5169 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1226 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.5169 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1226 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.5169 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1226 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.54631 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

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

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

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

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

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.569 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.311 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.311 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p>Ratio - Capacity</p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(8.569 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0032031</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.569 \text{ kip} \rightarrow 8569 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.63 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.63 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.63 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.63 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.84 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.4942 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.4942 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.076635$$

**Considering z-direction:**

$V_{max} = 0.21679 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.21679 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.0019559$$

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

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b  $\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 23.136 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(23.136 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.092694$$

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

**Considering z-direction:**

$M_{max} = 0.54631 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$\text{Ratio} = \frac{(0.54631 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0021887$$

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