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# Structural Calculations

## 51455 Huntington Rd.
## LaPine, OR 97739

**Prepared For**: Tavares
**Work Order**: 23038
**Date**: 11-15-22

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**LOADS:**

| | |
|---|---|
| **Roof Live / Snow** | **55 psf** |
| **Rood Dead** | **17 psf office, 15 psf Porch** |
| **Floor Live** | **50 psf, 2000 lb o/ 2 1/2' Sq.** |
| **Floor Dead** | **15 psf** |
| **Wind** | **97 mph, Exp C** |
| **Seismic Design Category** | **D** |
| **Allowable Soil Bearing** | **1500 psf** |

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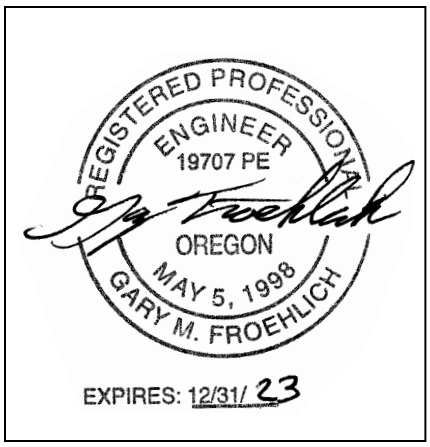


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Gary M Froehlich

Digitally signed by Gary M Froehlich
DN: C=US, E=garyf@mortierang.com, O=Mortier Ang Engineers, OU=MAE, CN=Gary M Froehlich
Reason: I am the author of this document
Date: 2022.11.15 08:52:04-08'00'

# Multiple Simple Beam

Project File: 23038.ec6



**Description :** Headers - Beams

## Wood Beam Design : Porch roof beam

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **8x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.2

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

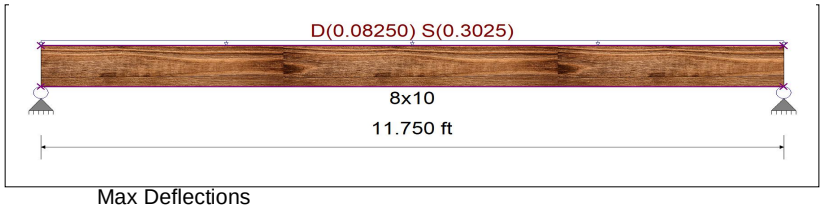
Unif Load: D = 0.0150, S = 0.0550 k/ft, Trib= 5.50 ft

**Design Summary**

Max fb/Fb Ratio = **0.702** : 1
fb : Actual : 706.76 psi at 5.875 ft in Span # 1
Fb : Allowable : 1,006.25 psi
Load Comb : +D+S

Max fv/FvRatio = **0.211** : 1
fv : Actual : 41.27 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.48 | | | 1.78 | | | |
| Right Support | 0.48 | | | 1.78 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.187 in | Total Downward | 0.238 in |
| Ratio | 753 | Ratio | 591 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

## Wood Beam Design : Porch roof beam at dormer post

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **8x10, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending
Wood Species : Douglas Fir-Larch Wood Grade : No.1

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Fb - Tension | 1,350.0 psi | Fc - Prll | 925.0 psi | Fv | 170.0 psi | Ebend- xx | 1,600.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 1,350.0 psi | Fc - Perp | 625.0 psi | Ft | 675.0 psi | Eminbend - xx | 580.0 ksi | | |

*Applied Loads*

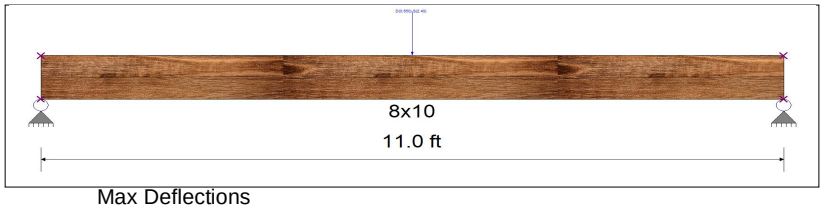
1Point: D = 0.650, S = 2.40 k @ 5.50 ft

**Design Summary**

Max fb/Fb Ratio = **0.578** : 1
fb : Actual : 892.19 psi at 5.500 ft in Span # 1
Fb : Allowable : 1,544.74 psi
Load Comb : +D+S

Max fv/FvRatio = **0.164** : 1
fv : Actual : 32.11 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.33 | | | 1.20 | | | |
| Right Support | 0.33 | | | 1.20 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.135 in | Total Downward | 0.171 in |
| Ratio | 979 | Ratio | 770 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

# Multiple Simple Beam

Project File: 23038.ec6



## Wood Beam Design : 6 ft headers

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **6x12, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

Unif Load: D = 0.0170, S = 0.0550 k/ft, Trib= 22.0 ft

**Design Summary**

Max fb/Fb Ratio = **0.823** : 1
fb : Actual : 828.07 psi at 3.250 ft in Span # 1
Fb : Allowable : 1,006.25 psi
Load Comb : +D+S

Max fv/FvRatio = **0.441** : 1
fv : Actual : 86.27 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 1.22 | | | 3.93 | | | |
| Right Support | 1.22 | | | 3.93 | | | |


Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.054 in | Total Downward | 0.071 in |
| Ratio | 1446 | Ratio | 1105 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

## Wood Beam Design : 4 ft headers

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **4x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

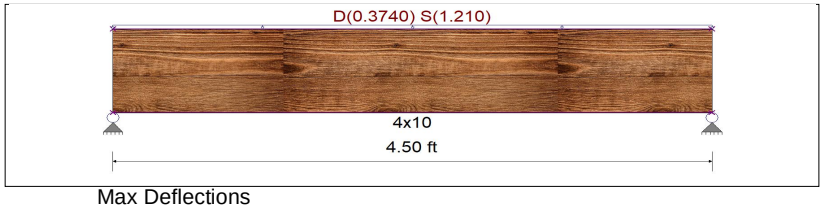
Unif Load: D = 0.0170, S = 0.0550 k/ft, Trib= 22.0 ft

**Design Summary**

Max fb/Fb Ratio = **0.798** : 1
fb : Actual : 963.99 psi at 2.250 ft in Span # 1
Fb : Allowable : 1,207.50 psi
Load Comb : +D+S

Max fv/FvRatio = **0.557** : 1
fv : Actual : 108.98 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.84 | | | 2.72 | | | |
| Right Support | 0.84 | | | 2.72 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.037 in | Total Downward | 0.049 in |
| Ratio | 1443 | Ratio | 1102 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

# Multiple Simple Beam

Project File: 23038.ec6



## Wood Beam Design : Porch Roof Decking

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **5.50 X 1.50, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

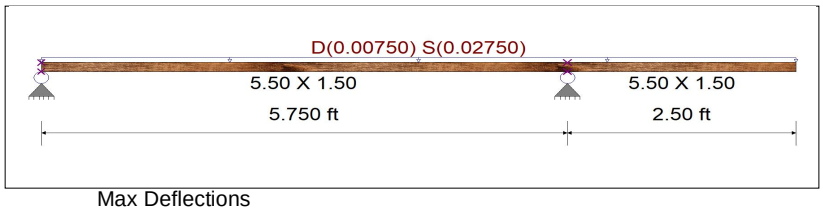
Unif Load: D = 0.0150, S = 0.0550 k/ft, Trib= 0.50 ft

**Design Summary**

Max fb/Fb Ratio = **0.486** : 1
fb : Actual : 636.36 psi at 5.750 ft in Span # 1
Fb : Allowable : 1,308.13 psi
Load Comb : +D+S

Max fv/FvRatio = **0.108** : 1
fv : Actual : 21.02 psi at 5.635 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.02 | | | 0.06 | | | |
| Right Support | 0.04 | | | 0.16 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.188 in | Total Downward | 0.239 in |
| Ratio | 367 | Ratio | 288 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | -0.018 in | Total Upward | -0.023 in |
| Ratio | 3334 | Ratio | 2620 |
| | LC: S Only | | LC: +D+S |

## Wood Beam Design : Out lookers

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **2x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

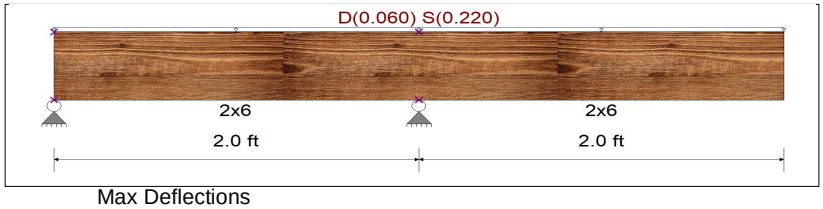
Unif Load: D = 0.0150, S = 0.0550 k/ft, Trib= 4.0 ft

**Design Summary**

Max fb/Fb Ratio = **0.692** : 1
fb : Actual : 888.60 psi at 2.000 ft in Span # 1
Fb : Allowable : 1,283.77 psi
Load Comb : +D+S

Max fv/FvRatio = **0.521** : 1
fv : Actual : 101.82 psi at 2.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | -0.00 | | | -0.00 | | | |
| Right Support | 0.24 | | | 0.88 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.056 in | Total Downward | 0.071 in |
| Ratio | 854 | Ratio | 670 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | -0.004 in | Total Upward | -0.006 in |
| Ratio | 5390 | Ratio | 4235 |
| | LC: S Only | | LC: +D+S |

# Multiple Simple Beam

Project File: 23038.ec6



## Wood Beam Design : Floor Joist plumbing area

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **2x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

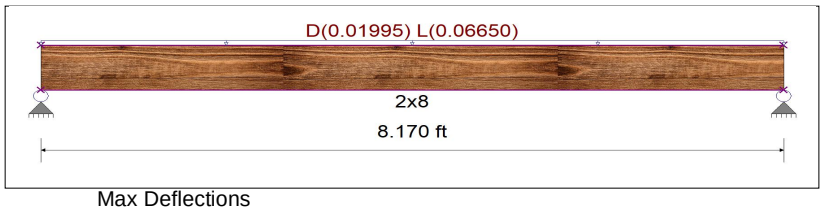
Unif Load: D = 0.0150, L = 0.050 k/ft, Trib= 1.330 ft

**Design Summary**

Max fb/Fb Ratio = **0.627** : 1
fb : Actual : 658.69 psi at 4.085 ft in Span # 1
Fb : Allowable : 1,050.00 psi
Load Comb : +D+L

Max fv/FvRatio = **0.245** : 1
fv : Actual : 41.57 psi at 7.571 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.08 | | 0.27 | | | | |
| Right Support | 0.08 | | 0.27 | | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.108 in | Total Downward | 0.141 in |
| Ratio | 905 | Ratio | 696 |
| | LC: L Only | | LC: +D+L |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

## Wood Beam Design : Floor Joist plumbing area w pt.

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **2x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

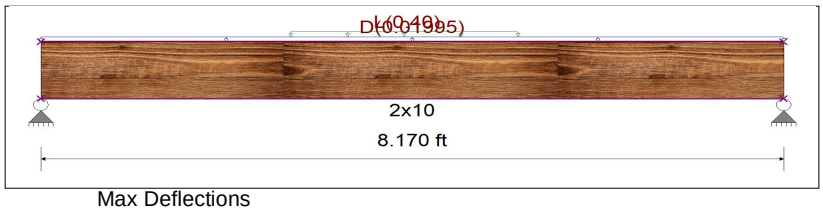
Unif Load: D = 0.0150 k/ft, Trib= 1.330 ft
Unif Load: L = 0.40 k/ft, 2.750 to 5.250 ft, Trib= 1.0 ft

**Design Summary**

Max fb/Fb Ratio = **0.961** : 1
fb : Actual : 1,063.46 psi at 4.031 ft in Span # 1
Fb : Allowable : 1,106.88 psi
Load Comb : +D+L

Max fv/FvRatio = **0.367** : 1
fv : Actual : 62.34 psi at 0.000 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.08 | | 0.51 | | | | |
| Right Support | 0.08 | | 0.49 | | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.147 in | Total Downward | 0.162 in |
| Ratio | 668 | Ratio | 603 |
| | LC: L Only | | LC: +D+L |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

# Multiple Simple Beam

Project File: 23038.ec6



## Wood Beam Design : Floor Joist plumbing area w pt. at end

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **2x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : | No.2 | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

Unif Load: D = 0.0150 k/ft, Trib= 1.330 ft

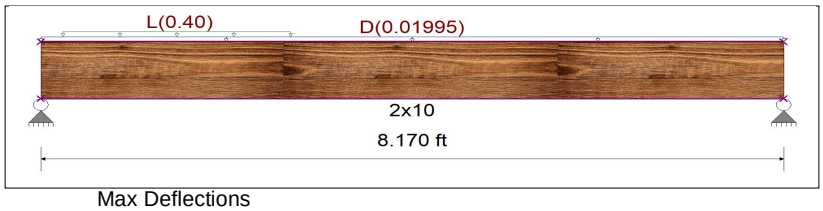
Unif Load: L = 0.40 k/ft, 0.250 to 2.750 ft, Trib= 1.0 ft

**Design Summary**

Max fb/Fb Ratio = **0.595** : 1
fb : Actual : 658.10 psi at 2.369 ft in Span # 1
Fb : Allowable : 1,106.88 psi
Load Comb : +D+L

Max fv/FvRatio = **0.431** : 1
fv : Actual : 73.26 psi at 0.000 ft in Span # 1
Fv : Allowable : 170.00 psi
Load Comb : +D+L

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.08 | | 0.82 | | | | |
| Right Support | 0.08 | | 0.18 | | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.080 in | Total Downward | 0.095 in |
| Ratio | 1232 | Ratio | 1032 |
| | LC: L Only | | LC: +D+L |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

## Wood Beam Design : 4 ft headers Gable Ends

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **2-2x6, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : | No.2 | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

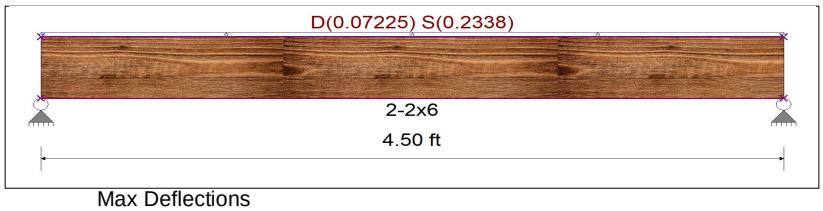
Unif Load: D = 0.0170, S = 0.0550 k/ft, Trib= 4.250 ft

**Design Summary**

Max fb/Fb Ratio = **0.470** : 1
fb : Actual : 614.53 psi at 2.250 ft in Span # 1
Fb : Allowable : 1,308.13 psi
Load Comb : +D+S

Max fv/FvRatio = **0.256** : 1
fv : Actual : 50.07 psi at 0.000 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.16 | | | 0.53 | | | |
| Right Support | 0.16 | | | 0.53 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.040 in | Total Downward | 0.052 in |
| Ratio | 1346 | Ratio | 1028 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

# Multiple Simple Beam

Project File: 23038.ec6



## Wood Beam Design : Ridge Dormer

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **8x8, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875.0 psi | Fc - Prll | 600.0 psi | Fv | 170.0 psi | Ebend- xx | 1,300.0 ksi | Density | 31.210 pcf |
| Fb - Compr | 875.0 psi | Fc - Perp | 625.0 psi | Ft | 425.0 psi | Eminbend - xx | 470.0 ksi | | |

*Applied Loads*

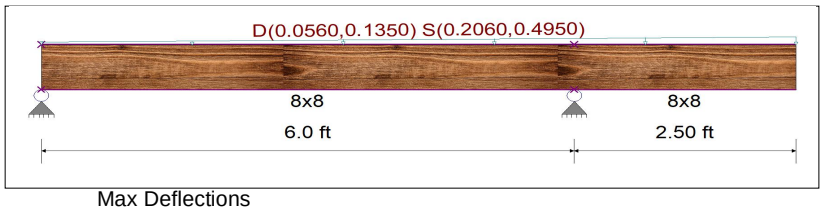
Unif Load: D= 0.0560->0.1350, S = 0.2060->0.4950 k/ft, 0.0 to 8.50 ft

**Design Summary**

Max fb/Fb Ratio = **0.315** : 1
fb : Actual : 316.76 psi at 6.000 ft in Span # 1
Fb : Allowable : 1,006.25 psi
Load Comb : +D+S

Max fv/FvRatio = **0.179** : 1
fv : Actual : 34.92 psi at 5.400 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.16 | | | 0.58 | | | |
| Right Support | 0.65 | | | 2.40 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.013 in | Total Downward | 0.016 in |
| Ratio | 4746 | Ratio | 3726 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | 0.000 in | Total Upward | 0.000 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: | | LC: |

## Wood Beam Design : Porch Valley beam at dormer

**Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16**

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2021 Load Combinations, Major Axis Bending

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Wood Species : | Douglas Fir-Larch | | | | Wood Grade : No.2 | | | | |
| Fb - Tension | 875 psi | Fc - Prll | 600 psi | Fv | 170 psi | Ebend- xx | 1300 ksi | Density | 31.21 pcf |
| Fb - Compr | 875 psi | Fc - Perp | 625 psi | Ft | 425 psi | Eminbend - xx | 470 ksi | | |

*Applied Loads*

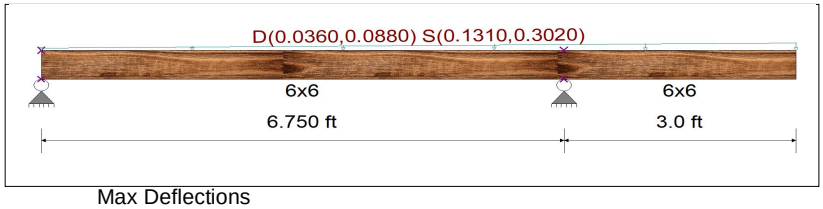
Unif Load: D= 0.0360->0.0880, S = 0.1310->0.3020 k/ft, 0.0 to 9.750 ft

**Design Summary**

Max fb/Fb Ratio = **0.711** : 1
fb : Actual : 714.95 psi at 6.750 ft in Span # 1
Fb : Allowable : 1,006.25 psi
Load Comb : +D+S

Max fv/FvRatio = **0.293** : 1
fv : Actual : 57.31 psi at 6.750 ft in Span # 1
Fv : Allowable : 195.50 psi
Load Comb : +D+S

| Max Reactions (k) | D | Lr | L | S | W | E | H |
|---|---|---|---|---|---|---|---|
| Left Support | 0.11 | | | 0.39 | | | |
| Right Support | 0.50 | | | 1.73 | | | |




Max Deflections

| | | | |
|---|---|---|---|
| Transient Downward | 0.071 in | Total Downward | 0.093 in |
| Ratio | 1010 | Ratio | 776 |
| | LC: S Only | | LC: +D+S |
| Transient Upward | -0.001 in | Total Upward | -0.001 in |
| Ratio | 9999 | Ratio | 9999 |
| | LC: S Only | | LC: +D+S |

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| | | | |
|---|---|---|---|
| Company: | | Date: | 10/29/2022 |
| Engineer: | | Page: | 1/5 |
| Project: | | | |
| Address: | | | |
| Phone: | | | |
| E-mail: | | | |

## 1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

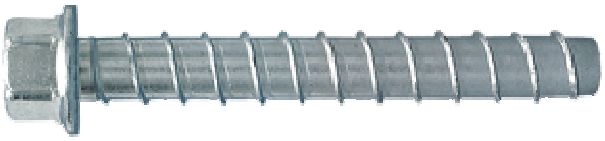
## 2. Input Data & Anchor Parameters

**General**
Design method:ACI 318-14
Units: Imperial units

**Anchor Information:**
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.500
Nominal Embedment depth (inch): 3.250
Effective Embedment depth, $h_{ef}$ (inch): 2.350
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
$h_{min}$ (inch): 5.00
$c_{ac}$ (inch): 3.56
$C_{min}$ (inch): 1.75
$S_{min}$ (inch): 3.00

**Base Material**
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, $f'_c$ (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

**Recommended Anchor**
Anchor Name: Titen HD® - 1/2"Ø Titen HD, hnom:3.25" (83mm)
Code Report: ICC-ES ESR-2713

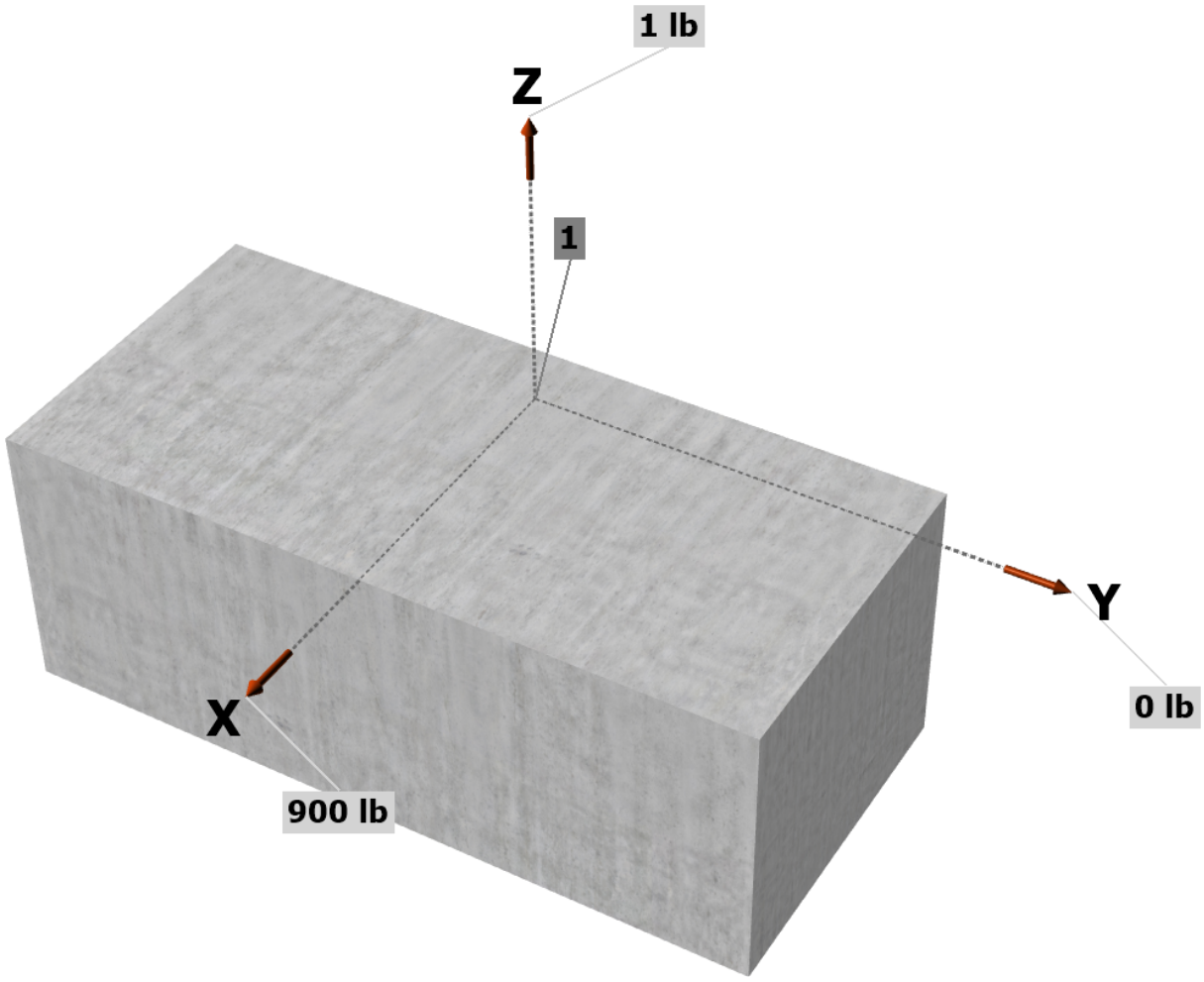


**Load and Geometry**
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

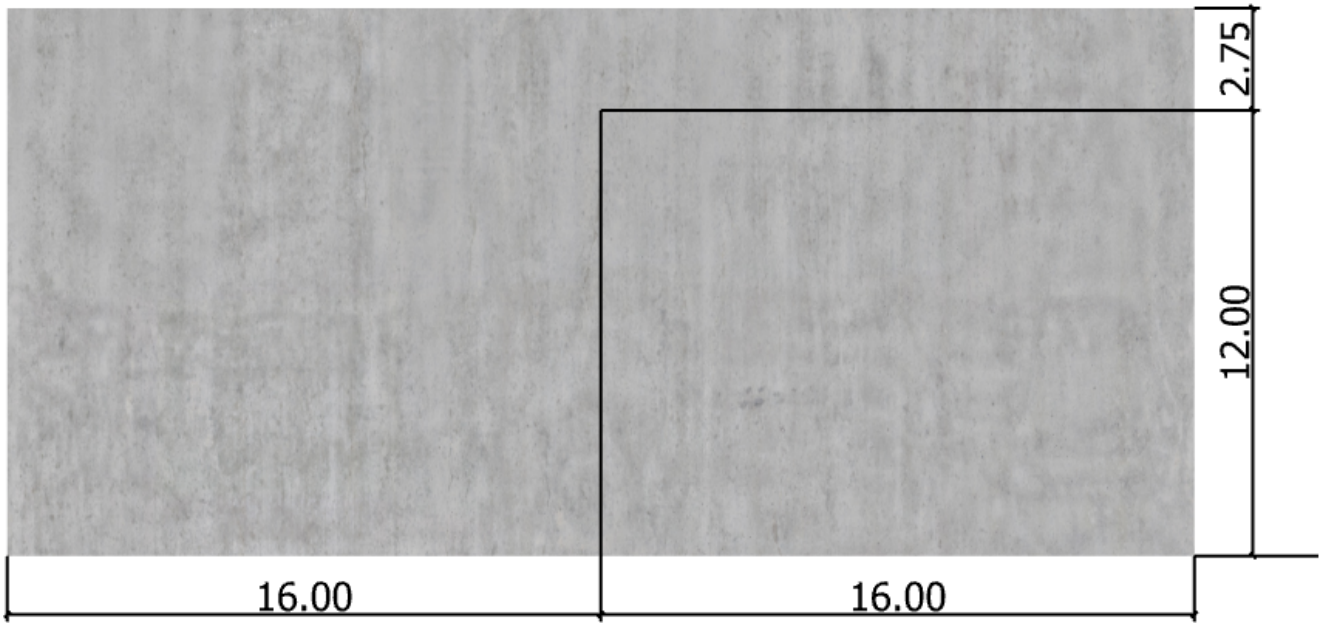
Strength level loads:

$N_{ua}$ [lb]: 1
$V_{uax}$ [lb]: 900
$V_{uay}$ [lb]: 0

<Figure 1>



<Figure 2>



## 3. Resulting Anchor Forces

| Anchor | Tension load, $N_{ua}$ (lb) | Shear load x, $V_{uax}$ (lb) | Shear load y, $V_{uay}$ (lb) | Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb) |
|---|---|---|---|---|
| 1 | 1.0 | 900.0 | 0.0 | 900.0 |
| Sum | 1.0 | 900.0 | 0.0 | 900.0 |

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, $e'_{Nx}$ (inch): 0.00
Eccentricity of resultant tension forces in y-axis, $e'_{Ny}$ (inch): 0.00
Eccentricity of resultant shear forces in x-axis, $e'_{Vx}$ (inch): 0.00
Eccentricity of resultant shear forces in y-axis, $e'_{Vy}$ (inch): 0.00

## 4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

| $N_{sa}$ (lb) | $\phi$ | $\phi N_{sa}$ (lb) |
|---|---|---|
| 20130 | 0.65 | 13085 |

## 5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

| $k_c$ | $\lambda_a$ | $f'_c$ (psi) | $h_{ef}$ (in) | $N_b$ (lb) |
|---|---|---|---|---|
| 17.0 | 1.00 | 2500 | 2.350 | 3062 |

$\phi N_{cb} = \phi (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

| $A_{Nc}$ (in$^2$) | $A_{Nco}$ (in$^2$) | $c_{a,min}$ (in) | $\Psi_{ed,N}$ | $\Psi_{c,N}$ | $\Psi_{cp,N}$ | $N_b$ (lb) | $\phi$ | $\phi N_{cb}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 44.24 | 49.70 | 2.75 | 0.934 | 1.00 | 1.000 | 3062 | 0.65 | 1655 |

## 8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

| $V_{sa}$ (lb) | $\phi_{grout}$ | $\phi$ | $\phi_{grout}\phi V_{sa}$ (lb) |
|---|---|---|---|
| 7455 | 1.0 | 0.60 | 4473 |

## 9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

***Shear perpendicular to edge in x-direction:***

$V_{bx} = \min\left|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}\right|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

| $l_e$ (in) | $d_a$ (in) | $\lambda_a$ | $f'_c$ (psi) | $c_{a1}$ (in) | $V_{bx}$ (lb) |
|---|---|---|---|---|---|
| 2.35 | 0.500 | 1.00 | 2500 | 10.67 | 11749 |

$\phi V_{cbx} = \phi (A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

| $A_{Vc}$ (in$^2$) | $A_{Vco}$ (in$^2$) | $\Psi_{ed,V}$ | $\Psi_{c,V}$ | $\Psi_{h,V}$ | $V_{bx}$ (lb) | $\phi$ | $\phi V_{cbx}$ (lb) |
|---|---|---|---|---|---|---|---|
| 384.00 | 512.00 | 1.000 | 1.000 | 1.155 | 11749 | 0.70 | 7123 |

***Shear parallel to edge in y-direction:***

$V_{bx} = \min\left|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}\right|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

| $l_e$ (in) | $d_a$ (in) | $\lambda_a$ | $f'_c$ (psi) | $c_{a1}$ (in) | $V_{bx}$ (lb) |
|---|---|---|---|---|---|

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 2.35 | 0.500 | 1.00 | 2500 | 8.00 | 7631 | | |

$\phi V_{cby} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

| $A_{Vc}$ (in$^2$) | $A_{Vco}$ (in$^2$) | $\Psi_{ed,V}$ | $\Psi_{c,V}$ | $\Psi_{h,V}$ | $V_{bx}$ (lb) | $\phi$ | $\phi V_{cby}$ (lb) |
|---|---|---|---|---|---|---|---|
| 177.00 | 288.00 | 1.000 | 1.000 | 1.000 | 7631 | 0.70 | 6566 |

## 10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

| $k_{cp}$ | $A_{Nc}$ (in$^2$) | $A_{Nco}$ (in$^2$) | $\Psi_{ed,N}$ | $\Psi_{c,N}$ | $\Psi_{cp,N}$ | $N_b$ (lb) | $\phi$ | $\phi V_{cp}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1.0 | 44.24 | 49.70 | 0.934 | 1.000 | 1.000 | 3062 | 0.70 | 1782 |

## 11. Results

### Interaction of Tensile and Shear Forces (Sec. 17.6.)

| Tension | Factored Load, $N_{ua}$ (lb) | Design Strength, ø$N_n$ (lb) | Ratio | Status |
|---|---|---|---|---|
| Steel | 1 | 13085 | 0.00 | Pass |
| **Concrete breakout** | **1** | **1655** | **0.00** | **Pass (Governs)** |

| Shear | Factored Load, $V_{ua}$ (lb) | Design Strength, ø$V_n$ (lb) | Ratio | Status |
|---|---|---|---|---|
| Steel | 900 | 4473 | 0.20 | Pass |
| T Concrete breakout x+ | 900 | 7123 | 0.13 | Pass |
| \|\| Concrete breakout y+ | 900 | 6566 | 0.14 | Pass |
| **Pryout** | **900** | **1782** | **0.51** | **Pass (Governs)** |

| Interaction check | $N_{ua}/\phi N_n$ | $V_{ua}/\phi V_n$ | Combined Ratio | Permissible | Status |
|---|---|---|---|---|---|
| Sec. 17.6..2 | 0.00 | 0.51 | 50.5% | 1.0 | Pass |

**1/2"Ø Titen HD, hnom:3.25" (83mm) meets the selected design criteria.**

## 12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

## 1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

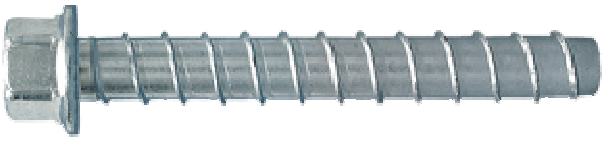
## 2. Input Data & Anchor Parameters

**General**
Design method:ACI 318-14
Units: Imperial units

**Anchor Information:**
Anchor type: Concrete screw
Material: Carbon Steel
Diameter (inch): 0.500
Nominal Embedment depth (inch): 3.250
Effective Embedment depth, $h_{ef}$ (inch): 2.350
Code report: ICC-ES ESR-2713
Anchor category: 1
Anchor ductility: No
$h_{min}$ (inch): 5.00
$c_{ac}$ (inch): 3.56
$C_{min}$ (inch): 1.75
$S_{min}$ (inch): 3.00

**Base Material**
Concrete: Normal-weight
Concrete thickness, h (inch): 12.00
State: Cracked
Compressive strength, $f'_c$ (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

**Recommended Anchor**
Anchor Name: Titen HD® - 1/2"Ø Titen HD, hnom:3.25" (83mm)
Code Report: ICC-ES ESR-2713



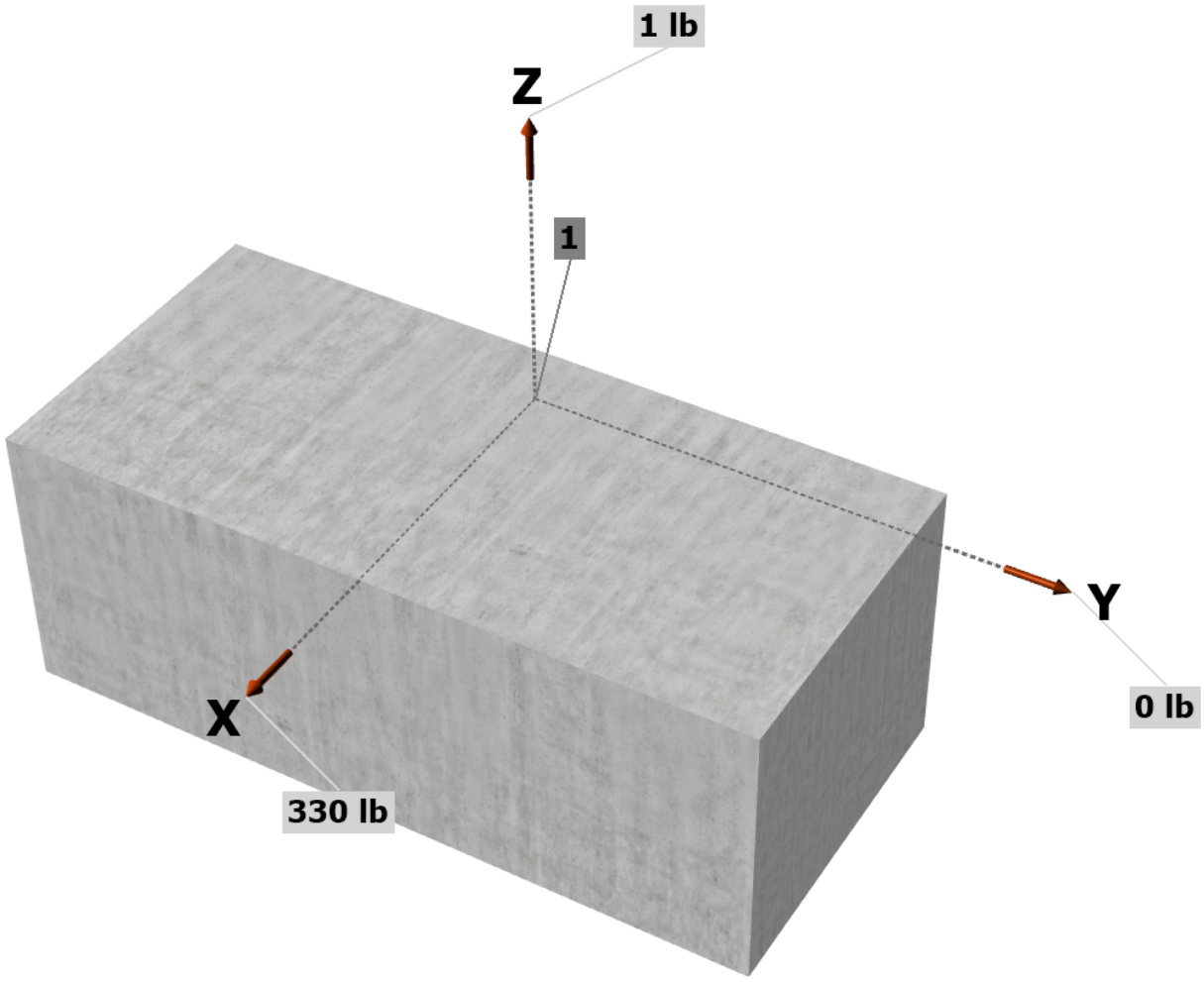
| | | | |
|---|---|---|---|
| Company: | | Date: | 10/29/2022 |
| Engineer: | | Page: | 2/5 |
| Project: | | | |
| Address: | | | |
| Phone: | | | |
| E-mail: | | | |

**Load and Geometry**
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

$N_{ua}$ [lb]: 1
$V_{uax}$ [lb]: 330
$V_{uay}$ [lb]: 0

<Figure 1>



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| | | | |
|---|---|---|---|
| Company: | | Date: | 10/29/2022 |
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| Address: | | | |
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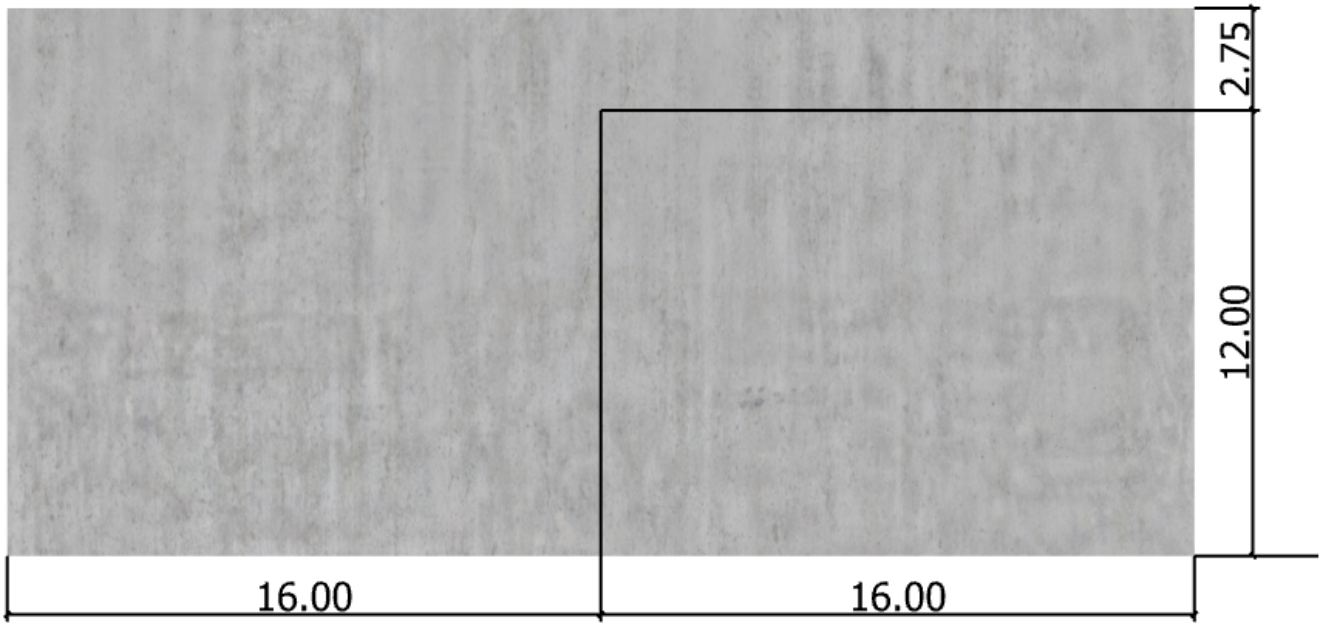
| E-mail: | | | |

<Figure 2>



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| | | | |
|---|---|---|---|
| Company: | | Date: | 10/29/2022 |
| Engineer: | | Page: | 4/5 |
| Project: | | | |
| Address: | | | |
| Phone: | | | |
| E-mail: | | | |

## 3. Resulting Anchor Forces

| Anchor | Tension load, $N_{ua}$ (lb) | Shear load x, $V_{uax}$ (lb) | Shear load y, $V_{uay}$ (lb) | Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb) |
|---|---|---|---|---|
| 1 | 1.0 | 330.0 | 0.0 | 330.0 |
| Sum | 1.0 | 330.0 | 0.0 | 330.0 |

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, $e'_{Nx}$ (inch): 0.00
Eccentricity of resultant tension forces in y-axis, $e'_{Ny}$ (inch): 0.00
Eccentricity of resultant shear forces in x-axis, $e'_{Vx}$ (inch): 0.00
Eccentricity of resultant shear forces in y-axis, $e'_{Vy}$ (inch): 0.00

## 4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

| $N_{sa}$ (lb) | $\phi$ | $\phi N_{sa}$ (lb) |
|---|---|---|
| 20130 | 0.65 | 13085 |

## 5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

| $k_c$ | $\lambda_a$ | $f'_c$ (psi) | $h_{ef}$ (in) | $N_b$ (lb) |
|---|---|---|---|---|
| 17.0 | 1.00 | 2500 | 2.350 | 3062 |

$\phi N_{cb} = \phi (A_{Nc} / A_{Nco}) \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1a)

| $A_{Nc}$ (in$^2$) | $A_{Nco}$ (in$^2$) | $c_{a,min}$ (in) | $\Psi_{ed,N}$ | $\Psi_{c,N}$ | $\Psi_{cp,N}$ | $N_b$ (lb) | $\phi$ | $\phi N_{cb}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 44.24 | 49.70 | 2.75 | 0.934 | 1.00 | 1.000 | 3062 | 0.65 | 1655 |

## 8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

| $V_{sa}$ (lb) | $\phi_{grout}$ | $\phi$ | $\phi_{grout}\phi V_{sa}$ (lb) |
|---|---|---|---|
| 7455 | 1.0 | 0.60 | 4473 |

## 9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

***Shear perpendicular to edge in x-direction:***

$V_{bx} = \min\left|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}\right|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

| $l_e$ (in) | $d_a$ (in) | $\lambda_a$ | $f'_c$ (psi) | $c_{a1}$ (in) | $V_{bx}$ (lb) |
|---|---|---|---|---|---|
| 2.35 | 0.500 | 1.00 | 2500 | 10.67 | 11749 |

$\phi V_{cbx} = \phi (A_{Vc} / A_{Vco}) \Psi_{ed,V} \Psi_{c,V} \Psi_{h,V} V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

| $A_{Vc}$ (in$^2$) | $A_{Vco}$ (in$^2$) | $\Psi_{ed,V}$ | $\Psi_{c,V}$ | $\Psi_{h,V}$ | $V_{bx}$ (lb) | $\phi$ | $\phi V_{cbx}$ (lb) |
|---|---|---|---|---|---|---|---|
| 384.00 | 512.00 | 1.000 | 1.000 | 1.155 | 11749 | 0.70 | 7123 |

***Shear parallel to edge in y-direction:***

$V_{bx} = \min\left|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}\right|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

| $l_e$ (in) | $d_a$ (in) | $\lambda_a$ | $f'_c$ (psi) | $c_{a1}$ (in) | $V_{bx}$ (lb) |
|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| 2.35 | 0.500 | 1.00 | 2500 | 8.00 | 7631 |

$\phi V_{cby} = \phi\ (2)(A_{Vc} / A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

| $A_{Vc}$ (in$^2$) | $A_{Vco}$ (in$^2$) | $\Psi_{ed,V}$ | $\Psi_{c,V}$ | $\Psi_{h,V}$ | $V_{bx}$ (lb) | $\phi$ | $\phi V_{cby}$ (lb) |
|---|---|---|---|---|---|---|---|
| 177.00 | 288.00 | 1.000 | 1.000 | 1.000 | 7631 | 0.70 | 6566 |

## 10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cp} = \phi k_{cp} N_{cb} = \phi k_{cp}(A_{Nc} / A_{Nco})\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1a)

| $k_{cp}$ | $A_{Nc}$ (in$^2$) | $A_{Nco}$ (in$^2$) | $\Psi_{ed,N}$ | $\Psi_{c,N}$ | $\Psi_{cp,N}$ | $N_b$ (lb) | $\phi$ | $\phi V_{cp}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1.0 | 44.24 | 49.70 | 0.934 | 1.000 | 1.000 | 3062 | 0.70 | 1782 |

## 11. Results

### Interaction of Tensile and Shear Forces (Sec. 17.6.)

| Tension | Factored Load, $N_{ua}$ (lb) | Design Strength, ø$N_n$ (lb) | Ratio | Status |
|---|---|---|---|---|
| Steel | 1 | 13085 | 0.00 | Pass |
| **Concrete breakout** | **1** | **1655** | **0.00** | **Pass (Governs)** |

| Shear | Factored Load, $V_{ua}$ (lb) | Design Strength, ø$V_n$ (lb) | Ratio | Status |
|---|---|---|---|---|
| Steel | 330 | 4473 | 0.07 | Pass |
| T Concrete breakout x+ | 330 | 7123 | 0.05 | Pass |
| \|\| Concrete breakout y+ | 330 | 6566 | 0.05 | Pass |
| **Pryout** | **330** | **1782** | **0.19** | **Pass (Governs)** |

| Interaction check | $N_{ua}/\phi N_n$ | $V_{ua}/\phi V_n$ | Combined Ratio | Permissible | Status |
|---|---|---|---|---|---|
| Sec. 17.6..2 | 0.00 | 0.19 | 18.5% | 1.0 | Pass |

**1/2"Ø Titen HD, hnom:3.25" (83mm) meets the selected design criteria.**

## 12. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer's product literature for hole cleaning and installation instructions.

**MAE Engineers**
1355 Oak St., Suite 200
Eugene, OR 97401
(541) 484- 9080

Project: Tavares - Office
Job No.: WO23038
By: GF
Date: 11/8/2022

## BOLT CONNECTION DESIGN (NDS 2018)

| | | | | |
|---|---|---|---|---|
| DURATION FACTOR: | | 1.00 | | |
| OTHER FACTORS : | | 1.00 | | |
| SIDE MEMBER (S-STEEL, W-WOOD) | | W | | |
| NUMBER OF SHEAR FACES | | 1 | | |
| THICKNESS OF MAIN MEMBER, tm | | 3.50 | in. | |
| THICKNESS OF ONE SIDE MEMBER, ts | | 1.50 | in. | |
| DOWEL BEARING STRENGTH OF MAIN MEMBER, Fem | | 7500 | psi. | Per Table 11E |
| FOR LOAD PARALLEL TO GRAIN, Fem1 | | 7500 | psi. | |
| FOR LOAD PERPENDICULAR TO GRAIN, Fem2 | | 7500 | psi. | |
| MAXIMUM ANGLE OF LOAD TO GRAIN | T | 0.00 | degree | |
| DOWEL BEARING STRENGTH OF SIDE MEMBER(S), Fes | | 5600 | psi. | |
| FOR LOAD PARALLEL TO GRAIN, Fes1 | | 5600 | psi. | 2 x ledger |
| FOR LOAD PERPENDICULAR TO GRAIN, Fes2 | | 3150 | psi. | |
| MAXIMUM ANGLE OF LOAD TO GRAIN | 1 | 0.00 | degree | |
| NOMINAL BOLT DIAMETER, D | | 0.500 | in. | |
| BENDING YIELD STRENGTH OF BOLT, Fyb | | 45000 | psi. | (A307 Bolts) |

| FAILURE MODES | NOMINAL LATERAL DESIGN VALUES | | |
|---|---|---|---|
| MODE Im | Z = | 3281 | lb. |
| MODE Is | Z = | 1050 | lb. |
| MODE II | Z = | 1153 | lb. |
| MODE IIIm | Z = | 1373 | lb. |
| MODE IIIs | Z = | 648 | lb. |
| MODE IV | Z = | 766 | lb. |

ALLOWABLE LATERAL DESIGN VALUE  Z' = 648 lb.

SPACING REQUIREMENTS

| | | | | |
|---|---|---|---|---|
| LENGTH OF BOLTS IN MAIN MEMBER | L = | 3.50 | in. | |
| LENGTH OF BOLTS IN SIDE MEMBER(S) | L = | 1.50 | in. | |

| | Min. Edge Distance (in.) | Min. End Distance (in.) | Spacing in a Row (in.) | Row Spacing (in.) |
|---|---|---|---|---|
| Parallel to Grain | | | | |
| Full Value | 0.750 | 2.000 | 2.000 | 0.750 |
| Reduced Value | N/A | 1.000 | 1.500 | N/A |
| Perpendicular to Grain | | | | |
| Full Value | 2.000 LOADED | 3.500 | | 1.563 |
| Reduced Value | 0.750 UNLD | 1.750 | 1.500 | N/A |

MORTIER ANG
ENGINEERS

**CALCULATIONS**

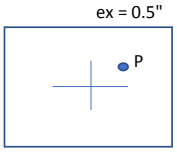
STRUCTURAL
BUILDING DESIGN • FIRE PROTECTION
CODE CONSULTANT • PLAN CHECKING
CONSTRUCTION INSPECTION

| WORK ORDER# | PAGE # |
|---|---|
| | |
| PROJECT NAME | BY/CHECKED |
| | |
| REF | DATE |
| | 1/10/2018 |

Column Allowable Loads (kips) :
eccentricity = 0.5" (x and y directions)

DF-L No.2 design values

| | | | |
|---|---|---|---|
| fb | = | 875 | psi |
| fc (parallel) | = | 1300 | psi |
| E | = | 1600000 | psi |



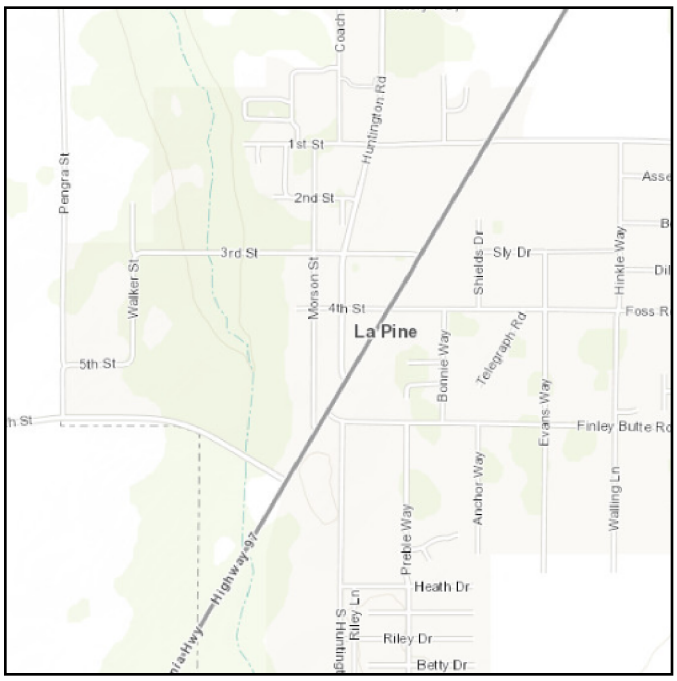

| Post Size | laterally supported (?) | | 8 ft. | | | 9 ft. | | | 10 ft. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | Strong | Weak | floor (100%) | roof (115%) | interaction | floor (100%) | roof (115%) | interaction | floor (100%) | roof (115%) | interaction |
| 2x4 | N | N | 0.42 | 0.42 | 0.93 | 0.34 | 0.34 | 0.88 | 0.29 | 0.29 | 0.92 |
| | N | Y | 2.00 | 2.25 | 0.91 | 1.50 | 1.50 | 0.93 | 1.30 | 1.30 | 0.93 |
| | Y | Y | 2.00 | 2.30 | 0.89 | 2.00 | 2.30 | 0.89 | 2.00 | 2.30 | 0.89 |
| DBL 2x4 | N | N | 2.00 | 2.25 | 0.91 | 1.50 | 1.50 | 0.93 | 1.30 | 1.30 | 0.93 |
| | N | Y | 2.20 | 2.30 | 0.92 | 1.90 | 2.00 | 0.91 | 1.60 | 1.70 | 0.85 |
| | Y | Y | 3.70 | 4.30 | 0.91 | 3.70 | 4.30 | 0.91 | 3.70 | 4.30 | 0.91 |
| TPL 2x4 | N | N | 3.30 | 3.50 | 0.90 | 2.90 | 3.00 | 0.92 | 2.50 | 2.60 | 0.90 |
| | N | Y | 3.40 | 3.60 | 0.90 | 3.00 | 3.10 | 0.93 | 2.50 | 2.70 | 0.92 |
| | Y | Y | 6.00 | 7.00 | 0.92 | 6.00 | 7.00 | 0.92 | 6.00 | 7.00 | 0.92 |
| 4x4 | N | N | 3.50 | 3.70 | 0.93 | 3.00 | 3.20 | 0.93 | 2.60 | 2.75 | 0.90 |
| | N | Y | 3.80 | 4.00 | 0.89 | 3.30 | 3.50 | 0.91 | 2.90 | 3.00 | 0.91 |
| | Y | Y | 7.00 | 8.00 | 0.89 | 7.00 | 8.00 | 0.89 | 7.00 | 8.00 | 0.89 |
| 4x6 | N | N | 5.80 | 6.20 | 0.91 | 5.00 | 5.30 | 0.89 | 4.40 | 4.60 | 0.89 |
| | N | Y | 9.20 | 10.20 | 0.91 | 8.50 | 9.40 | 0.91 | 8.00 | 8.50 | 0.92 |
| | Y | Y | 11.50 | 13.20 | 0.91 | 11.50 | 13.20 | 0.91 | 11.50 | 13.20 | 0.91 |
| 2x6 | N | N | 0.65 | 0.65 | 0.93 | 0.50 | 0.50 | 0.78 | 0.45 | 0.45 | 0.92 |
| | N | Y | 2.80 | 3.10 | 0.91 | 2.70 | 3.00 | 0.92 | 2.60 | 2.80 | 0.93 |
| | Y | Y | 3.10 | 3.50 | 0.91 | 3.10 | 3.50 | 0.91 | 3.10 | 3.50 | 0.91 |
| DBL 2x6 | N | N | 2.90 | 3.00 | 0.91 | 2.50 | 2.50 | 0.93 | 2.10 | 2.20 | 0.92 |
| | N | Y | 5.50 | 6.00 | 0.93 | 5.00 | 5.50 | 0.91 | 4.60 | 5.00 | 0.90 |
| | Y | Y | 6.50 | 7.50 | 0.91 | 6.50 | 7.50 | 0.91 | 6.50 | 7.50 | 0.91 |
| TPL 2x6 | N | N | 7.20 | 7.80 | 0.90 | 6.50 | 7.00 | 0.93 | 5.70 | 6.00 | 0.89 |
| | N | Y | 8.70 | 9.60 | 0.91 | 8.00 | 8.70 | 0.89 | 7.40 | 8.00 | 0.91 |
| | Y | Y | 10.70 | 12.20 | 0.91 | 10.70 | 12.20 | 0.91 | 10.70 | 12.20 | 0.91 |
| 6x6 | N | N | 12.70 | 14.00 | 0.90 | 11.80 | 12.80 | 0.91 | 10.80 | 11.60 | 0.90 |
| | N | Y | 14.00 | 15.20 | 0.92 | 12.80 | 14.00 | 0.90 | 11.90 | 12.80 | 0.90 |
| | Y | Y | 17.20 | 20.00 | 0.91 | 17.20 | 20.00 | 0.91 | 17.20 | 20.00 | 0.91 |

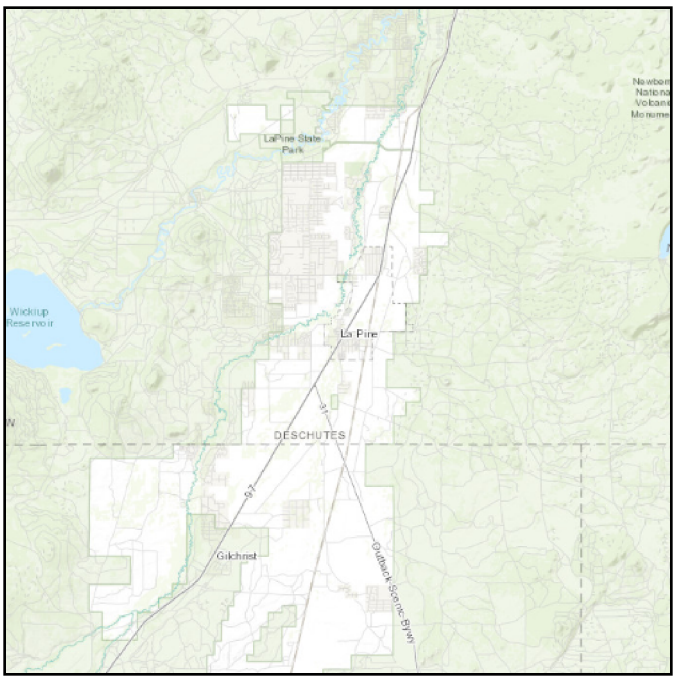
# ASCE 7 Hazards Report

**Address:**
51455 Huntington Rd
La Pine, Oregon
97739

**Standard:** ASCE/SEI 7-16
**Risk Category:** II
**Soil Class:** D - Default (see Section 11.4.3)

**Elevation:** 4234.8 ft (NAVD 88)
**Latitude:** 43.669489
**Longitude:** -121.505542




## Wind

**Results:**

| | |
|---|---|
| Wind Speed | 97 Vmph |
| 10-year MRI | 67 Vmph |
| 25-year MRI | 73 Vmph |
| 50-year MRI | 78 Vmph |
| 100-year MRI | 83 Vmph |

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4, and Section 26.5.2
Date Accessed: Thu Oct 27 2022

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

## Seismic

**Site Soil Class:** D - Default (see Section 11.4.3)

**Results:**

| | | | |
|---|---|---|---|
| $S_S$ : | 0.425 | $S_{D1}$ : | N/A |
| $S_1$ : | 0.215 | $T_L$ : | 16 |
| $F_a$ : | 1.46 | PGA : | 0.194 |
| $F_v$ : | N/A | $PGA_M$ : | 0.273 |
| $S_{MS}$ : | 0.62 | $F_{PGA}$ : | 1.413 |
| $S_{M1}$ : | N/A | $I_e$ : | 1 |
| $S_{DS}$ : | 0.413 | $C_v$ : | 0.983 |

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

**Data Accessed:** Thu Oct 27 2022

**Date Source:** USGS Seismic Design Maps

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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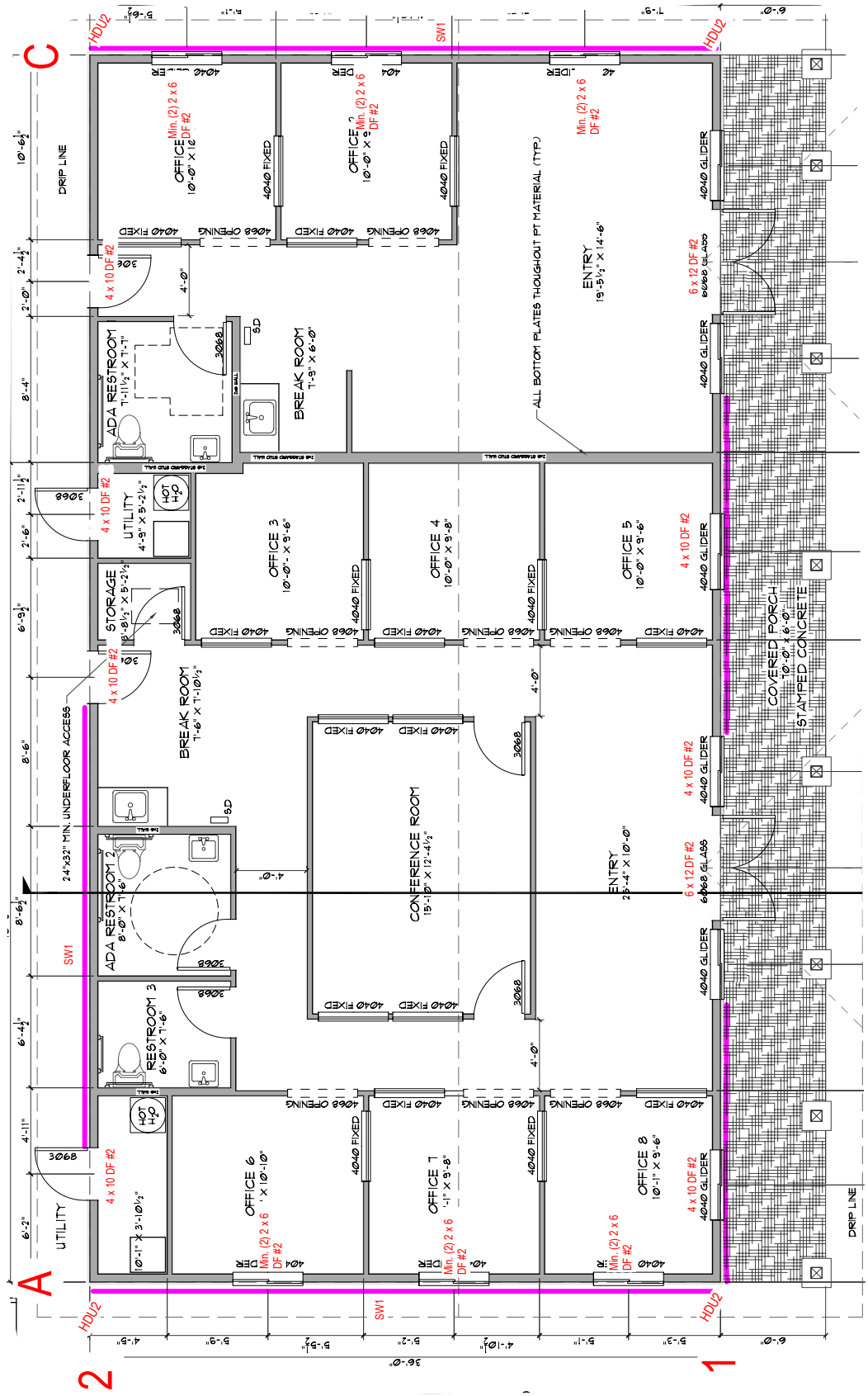
**MAE Engineering**
1355 OAK ST, SUITE #200
EUGENE, OR 97401
PHONE: (541) 484-9080 |WWW.MAE-ENGINEERS.COM



| WORK ORDER: 23038 | PROJECT: Tavares Realty Office | ENGR: gF | DATE: 11-14-22 |
|---|---|---|---|



# ASCE 7-16 Wind Forces Chpt 28, Pt2 & Chpt 30, Pt2





DESCRIPTION: Tavares Office

## General Design Values

Calculations per ASCE 7-16

| | | |
|---|---|---|
| V : Basic Wind Speed per Sect 26.5-1 or 2 | **97.0** | mph |
| User specified minimum design pressure | 10.0 | psf |
| Occupancy per Table 1.5-1 | II | All Buildings and other structures except those listed |
| Exposure Category per 26.7 | Exposure C | |
| Topographic Factor Kzt per 26.8 | **1.00** | |

*"Lambda" is interpolated between height tablular values.*

## Main Force Resisting System Values

| | | |
|---|---|---|
| MRH : Mean Roof Height | 16.0 | ft |
| Roof Rise:Run Ratio | 6:12 | |

Lambda MWFRS: per Figure 26.8-1 **1.23**

## Component & Cladding Values

| | | |
|---|---|---|
| Effective Wind Area of Component & Cladding | 10.0 | ft^2 |
| Roof pitch for cladding pressure | Flat/Hip/Gable Roof | |
| LHD : Least Horizontal Dimension | 36.0 | ft |
| a = max (0.04 * LHD, 3, min(0.10 * LHD, 0.4*MRH)) | **3.60** | ft |

Lambda Componant & Cladding : per Figure 30.4-1 **1.23**

## Design Wind Pressures

### Horizontal Pressures . . .

Load Case # 1 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: A = | 23.00 | psf | Zone: C = | 16.62 | psf |
| Zone: B = | 10.00 | psf | Zone: D = | 10.00 | psf |

Load Case # 2 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: A = | 10.00 | psf | Zone: C = | 10.00 | psf |
| Zone: B = | 10.00 | psf | Zone: D = | 10.00 | psf |

### Vertical Pressures . . .

Load Case # 1 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: E = | -10.20 | psf | Zone: G = | -10.00 | psf |
| Zone: F = | -13.90 | psf | Zone: H = | -11.16 | psf |

Load Case # 2 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: E = | -10.00 | psf | Zone: G = | -10.00 | psf |
| Zone: F = | -10.00 | psf | Zone: H = | -10.00 | psf |

### Overhangs . . .

Load Case # 1 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: Eoh = | -19.05 | psf | Zone: Goh = | -16.21 | psf |

Load Case # 2 . . .

| | | | | | |
|---|---|---|---|---|---|
| Zone: Eoh = | 10.00 | psf | Zone: Goh = | 10.00 | psf |

ASCE 7-16 Section 28.5.4 Minimum Design Wind Loads requires that the load effects of the design wind pressures from Section 28.5.3 shall not be less than a minimum load defined by assuming the pressures, ps, for zones A and C equal to +16 psf, Zones B and D equal to +8 psf, while assuming ps for Zones E, F, G, and H are equal to 0 psf.

# ASCE 7-16 Wind Forces Chpt 28, Pt2 & Chpt 30, Pt2





DESCRIPTION: Tavares Office

## Component & Cladding Design Wind Pressures

*Design Wind Pressure = Lambda * Kzt * Ps30 per Eq 30.4-1*

| Roof Pressures | Positive | Negative | | Overhang Pressures | Negative | |
|---|---|---|---|---|---|---|
| Zone 1 | 10.000 | -33.127 | psf | Zone 1 | *** | psf |
| Zone 1' | 10.000 | -19.052 | psf | Zone 1' | *** | psf |
| Zone 2 | 10.000 | -43.621 | psf | Zone 2 | -38.987 | psf |
| Zone 2e | *** | *** | psf | Zone 2e | *** | psf |
| Zone 2n | *** | *** | psf | Zone 2n | *** | psf |
| Zone 2r | *** | *** | psf | Zone 2r | *** | psf |
| Zone 3 | 10.000 | -59.461 | psf | Zone 3 | -52.841 | psf |
| Zone 3e | *** | *** | psf | Zone 3e | *** | psf |
| Zone 3r | *** | *** | psf | Zone 3r | *** | psf |

Wall Pressures

| | | | |
|---|---|---|---|
| Wall Zone 4 : | 20.744 | -22.509 | psf |
| Wall Zone 5 : | 20.744 | -27.781 | psf |

'*** : There is no value in Figure 30.4-1 Tabular Values

# ASCE Seismic Base Shear



Lic. # : KW-06014291 — MAE ENGINEERING

DESCRIPTION: Tavares Office

## Tavares Office

### Risk Category

Calculations per ASCE 7-16

Risk Category of Building or Other Structure : "II" : All Buildings and other structures except those listed as Category I, III, and IV — *ASCE 7-16, Page 4, Table 1.5-1*

Seismic Importance Factor = 1 — *ASCE 7-16, Page 5, Table 1.5-2*

### USER DEFINED Ground Motion

ASCE 7-16 11.4.2

Max. Ground Motions, 5% Damping :

$S_S$ = 0.4250 g, 0.2 sec response
$S_1$ = 0.2150 g, 1.0 sec response

### Site Class, Site Coeff. and Design Category

Site Classification "D" : Shear Wave Velocity 600 to 1,200 ft/sec = D — *ASCE 7-16 Table 20.3-1*

Site Coefficients Fa & Fv (*using straight-line interpolation from table values*): Fa = 1.54, Fv = 2.03 — *ASCE 7-16 Table 11.4-1 & 11.4-2*

Maximum Considered Earthquake Acceleration
$S_{MS} = Fa * Ss$ = 0.655 — *ASCE 7-16 Eq. 11.4-1*
$S_{M1} = Fv * S1$ = 0.436 — *ASCE 7-16 Eq. 11.4-2*

Design Spectral Acceleration
$S_{DS} = S_{MS} * 2/3$ = 0.436 — *ASCE 7-16 Eq. 11.4-3*
$S_{D1} = S_{M1} * 2/3$ = 0.291 — *ASCE 7-16 Eq. 11.4-4*

Seismic Design Category = D — *ASCE 7-16 Table 11.6-1 & -2*

### Resisting System

*ASCE 7-16 Table 12.2-1*

Basic Seismic Force Resisting System . . . Bearing Wall Systems
15.Light-frame (wood) walls sheathed w/wood structural panels rated for shear resistance.

Response Modification Coefficient " R " = 6.50
System Overstrength Factor " Wo " = 2.50
Deflection Amplification Factor " Cd " = 4.00

*NOTE! See ASCE 7-16 for all applicable footnotes.*

Building height Limits :

| | |
|---|---|
| Category "A & B" Limit: | No Limit |
| Category "C" Limit: | No Limit |
| Category "D" Limit: | Limit = 65 |
| Category "E" Limit: | Limit = 65 |
| Category "F" Limit: | Limit = 65 |

### Lateral Force Procedure

*ASCE 7-16 Section 12.8.2*

Equivalent Lateral Force Procedure

The "Equivalent Lateral Force Procedure" is being used according to the provisions of ASCE 7-16 12.8

### Determine Building Period

*Use ASCE 12.8-7*

Structure Type for Building Period Calculation : All Other Structural Systems

" Ct " value = 0.020
" x " value = 0.75
" hn " : Height from base to highest level = 10.0 ft
" Ta " Approximate fundemental period using Eq. 12.8-7 : Ta = Ct * (hn ^ x) = 0.112 sec
"TL" : Long-period transition period per ASCE 7-16 Maps 22-14 -> 22-17 = 16.000 sec

Building Period " Ta " Calculated from Approximate Method selected = 0.112 sec

### " Cs " Response Coefficient

*ASCE 7-16 Section 12.8.1.1*

| | | | |
|---|---|---|---|
| $S_{DS}$: Short Period Design Spectral Response | = 0.436 | From Eq. 12.8-2, Preliminary Cs | = 0.067 |
| " R " : Response Modification Factor | = 6.50 | From Eq. 12.8-3 & 12.8-4 , Cs need not exceed | = 0.398 |
| " I " : Seismic Importance Factor | = 1 | From Eq. 12.8-5 & 12.8-6, Cs not be less than | = 0.019 |

Cs : Seismic Response Coefficient = = 0.0671

### Seismic Base Shear

*ASCE 7-16 Section 12.8.1*

Cs = 0.0671 from 12.8.1.1
W ( see Sum Wi below ) = 0.00 k
Seismic Base Shear V = Cs * W = 0.00 k

**MAE Engineers**
1355 NW Oak St.
Eugene, OR 97401
(541) 484 - 9080

Project : Tavares - Office
Proj. No.: WO23038
By : GF
Date : 10/28/2022
Page:



WOOD SHEARWALL DESIGN TABLE:

*ASCE 7*

| | | |
|---|---|---|
| *seismic approach* | S | (ELF) or (S)implified |
| *period timing* | 1 | (S)hort or (1) second |
| *seismic design category* | D | |
| *"Alpha" axis complies with ASCE 7 12.3-3* | Y | Y/N |
| *"Numeric" axis complies with ASCE 7 12.3-3* | Y | Y/N |

Wind Loads Calculated using OSSC2021 ASCE7-16

| | | |
|---|---|---|
| *Basic Wind Speed* | 96 | mph |
| *Exposure* | C | |
| *Importance Factor* | 1 | |
| *Wind Stagnation Pressure* | 36.0 | psf |
| *Height, Exposure and Gust Factor Coefficient* | 1.00 | |

*area of diaphragm above story, Ax* ..... $ft^2$

Main Story In-Plane Shear (WSD):

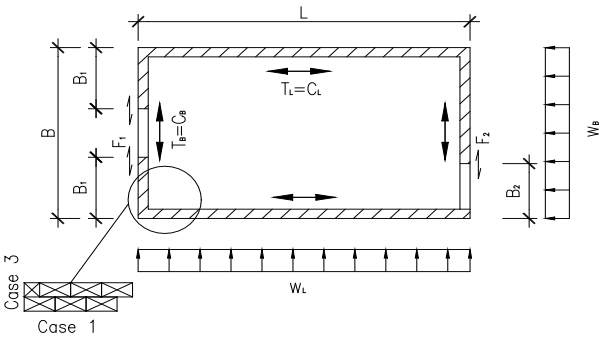
| shearwall | total wall (ft) | wall (ft) | perf. wall (ft) | sum of wall (ft) | max. opening (ft) | shear factor | wind (lb) | .7Fx seismic (lb) | ρ | ρ max *.7Fx seismic (lb) | unit (plf) | wall | wall segment 1 (perf wall = full wall length) segment (ft) | uniform (plf) | overtrng. (lb) | holdown |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | 24.00 | 10 | 36 | 24 | 4 | 0.83 | 6560 | 1556 | 1.3 | 1945 | 329 | 1 | 36.00 | 131 | 1399 | HDU2 |
| B | 0.1 | 0.1 | | | | 1.00 | 0 | 0 | 1.0 | 0 | 1 | 1 | 0.10 | 0 | 0 | |
| C | 24 | 10 | 36 | 24 | 4 | 0.83 | 6560 | 1556 | 1.3 | 1945 | 329 | 1 | 36 | 131 | 1399 | HDU2 |
| D | 0.1 | 0.1 | | | | 1.00 | 0 | 0 | 1.0 | 0 | 1 | | 0.1 | 0 | 0 | |
| E | 0.1 | 0.1 | | | | 1.00 | 0 | 0 | 1.0 | 0 | 1 | | 0 | 0 | 0 | |
| | | | | | | | | | | | | | | | | |
| 1 | 26.83 | 10 | 16 | 12 | 4 | 0.87 | 3740 | 1500 | 1.30 | 1875 | 160 | 1 | 16 | 420 | 0 | NR |
| 2 | 25.25 | 10 | | | | 1.00 | 3040 | 1650 | 1.30 | 2063 | 120 | 1 | 25.25 | 420 | 0 | NR |
| 3 | 0.1 | 0.1 | | | | 1.00 | 0 | 0 | 1.00 | 0 | 1 | 1 | 0.1 | 0 | 0 | |
| 4 | 0.1 | 0.1 | | | | 1.00 | 0 | 0 | 1.00 | 0 | 1 | | 0.1 | 0 | 0 | |

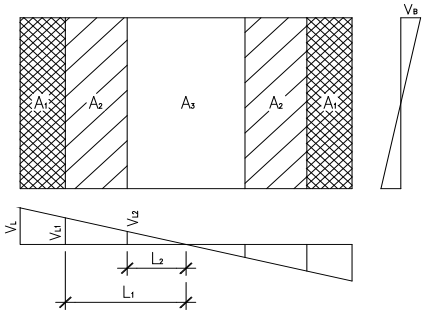
| | | | |
|---|---|---|---|
| PROJECT : | Tavares - Office | PAGE : | |
| CLIENT : | Tavares | DESIGN BY : | gf |
| JOB NO. : | WO 23038 DATE : 10/29/22 | REVIEW BY : | |

# Wood Diaphragm Design Based on NDS

## INPUT DATA

| | | | |
|---|---|---|---|
| LATERAL FORCE ALONG L SIDE: | $w_{L,\ WIND}$ = | 187 | plf,for wind |
| | $w_{L,\ SEISMIC}$ = | 45 | plf,for seismic |
| LATERAL FORCE ALONG B SIDE: | $w_{B,\ WIND}$ = | 169 | plf,for wind |
| | $w_{B,\ SEISMIC}$ = | 87 | plf,for seismic |
| DIMENSIONS: | L = 70 ft , B = | 36 | ft |
| | $B_1$ = 24 ft , $B_2$ = | 12 | ft |
| PANEL GRADE ( 0 or 1) | = 1 | <= Sheathing and Single-Floor | |
| MINIMUM NOMINAL FRAMING WITH ( 2 or 3) | = | 2 | in |
| MINIMUM NOMINAL PANEL THICKNESS | = | 15/32 | in |
| COMMON NAIL SIZE ( 0=6d, 1=8d, 2=10d ) | | 1 | 8d |
| SPECIFIC GRAVITY OF FRAMING MEMBERS | | 0.48 | |




## DESIGN SUMMARY

**A1:**

**A2:**

**A3:** (1) - 70.00 ft x 36 ft

UNBLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS @ 6" O.C. ALL EDGES / 12"O.CFIELD.

THE CHORD FORCES: $T_L = C_L$ = 3.19 k , $T_B = C_B$ = 0.39 k

THE DRAG STRUT FORCES $F_1$ = -1.09 k , $F_2$ = 2.19 k

THE MAXIMUM DIAPHRAGM DEFLECTION: $\Delta$ = 0.80 in

## ANALYSIS

THE DIAPHRAGM IS CONSIDER FLEXIBLE IF ITS MAXIMUM LATERAL DEFORMATION IS MORE THAN TWO TIMES THE AVERAGE SHEAR WALL DEFLECTION OF THE ASSOCIATED STORY. WITHOUT FURTHER CALCULATIONS, ASSUME A FLEXIBLE DIAPHRAGM HERE.

FROM THE TABLE 3.1 IN ASD MANUAL SUPP 01, PAGE SP-12, THE PANEL BENDING STRENGTH CAPACITY IS 355 in-lbs/ft, THAT IS THE DIAPHRAGM CAN RESISTS 59 psf GRAVITY LOADS (DL+LL) AT 2'-0" o.c. SPACING SUPPORTS.

THE MAX DIAPHRAGM DIMENSION RATIO L / B = 1.9 < 4, [satisfactory]

THE MAX SHEAR FORCE ALONG B SIDE $v_L$ = 182 plf, ( Boundary Spacing = 6 in, Edges ReqD = 6 in )

THE MAX SHEAR FORCE ALONG L SIDE $v_B$ = 43 plf, ( Required Boundary/Edges Nail Spacing for Case 3 = 6 in )

THE ALLOWABLE SHEAR FORCE FOR CASE 1 @ 6 in NAIL SPACING v1 = 270 plf, L1 = 35.0 ft

THE MAX ALLOWABLE UNBLOCKED SHEAR FORCE FOR CASE 1 v1 = 240 plf, L2 = 35.0 ft

THE SHEAR CAPACITIES PER IBC Table 2306.3.2 / UBC Table 23-II-H :

| Panel Grade | Common Nail | Min. Penetration (in) | Min. Thickness (in) | Member Width (in) | Blocked Nail Spacing Boundary / Other Edges | | | | Unblocked | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 6 / 6 | 4 / 6 | 2.5 / 4 | 2 / 3 | Case 1 | Others |
| Sheathing and Single-Floor | 8d | 1 1/2 | 15/32 | 2 | 270 | 360 | 530 | 600 | 240 | 180 |

Note: The indicated shear numbers have reduced by specific gravity factor per IBC note a / UBC note1 of the table.

THE CHORD FORCES: $T_L = C_L = (w_L L^2) / (8B)$ = 3.19 k $T_B = C_B = (w_B B^2) / (8L)$ = 0.39 k

THE DRAG STRUT FORCES: $F_1 = 0.5\,(B-2B_1)\ MAX(v_{1,\ WIND}, \Omega_0 v_{1,\ SEISMIC})$ = -1.09 k

$F_2 = B_2\ MAX(v_{1,\ WIND}, \Omega_0 v_{1,\ SEISMIC})$ = 2.19 k

$\Omega_0$ = 2.5 (UBC 1633.2.6, or IBC 1620.1.6)

THE MAXIMUM DIAPHRAGM DEFLECTION: (IBC 2305.2.2 , ASD MANUAL SUUP 01 Sec 3.3, Page SW-12)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ slip} + \Delta_{Chord\ splice\ slip} = \frac{5 v_L L^3}{8EAB} + \frac{v_L L}{4Gt} + 0.188 L e_n + \frac{\Sigma(D_C x)}{2B} = 0.802 \text{ in}$$

Where: $v_L$ = 182 plf, L = 70 ft, E = 1.7E+06 psi

A = 21.75 $in^2$, B = 36 ft, G = 9.0E+04 psi

t = 0.298 in,(UBC97 Page3-420) $e_n$ = 0.037 in, $\Sigma(D_C x)$ = 12.00 in

Note: The deflection, $\Delta$, above is based on completely blocked. For unblocked diaphragm, 2.4$\Delta$ should be used.


Technical References:

1. "National Design Specification, NDS", AF&AP, AWC.
2. Alan Williams: "Structural Engineering Reference Manual", Professional Publications, Inc, 2001.

## ENGINEERED SHEARWALL SCHEDULE

General notes:

- All shear walls panels shall land on framing members or blocking with all edges fastened per the shear wall schedule.
- All nails referenced in the shear wall schedule shall be of the following types and minimum sizes:
  8d common (2½" x 0.131"φ) or galvanized box (2½" x 0.113" φ), 10d common (3" x 0.148" φ) or galvanized box (3" x 0.128" φ)
- All sheathing shall lap onto and be "edge nailed" to all boundary members with attached holdowns.
- Foundation anchor bolts shall have a steel plate washer under each nut not less than 0.229" x 3" x 3" in size. The hole in the plate washer shall be permitted to have a 1-3/4" long diagonal slot with a width of up to 3/16" larger than the bolt diameter, provided a standard cut washer is placed between the plate washer and the nut. The plate washer shall extend to within 1/2" of the edge of the bottom plate on the side(s) with sheathing.

| SYM | OSB / plyw'd sheathing [1] | Fastening: sheathing to studs | | Mud sill A.B. | Rim Joists to | Plate to Rim | Freeze blocking | dbl. stud |
|---|---|---|---|---|---|---|---|---|
| | | Edges | Field | Size & Spacing | Plate Below | joist below | to top plate | fastening |
| 1 | 1 side | 8d @ 6" o.c. | 8d @ 12" o.c. | 1/2" φ @ 48" o.c. | LTP4 @ 48" o.c. | 16d @ 6" o.c. | RBC @ 24" o.c. | 2 rows 16d at 12" o.c. |
| 2 | 1 side | 8d @ 4" o.c. | 8d @ 12" o.c. | 1/2" φ @ 24" o.c.<br>5/8" φ @ 32" o.c. | LTP4 @ 32" o.c. | 16d @ 6" o.c. &<br>LTP4 @ 48" o.c. | RBC @ 12" o.c. | 2 rows 16d at 10" o.c. |
| 3 | 1 side | 8d @ 3" o.c. | 8d @ 12" o.c. | 1/2" φ @ 24" o.c.<br>5/8" φ @ 32" o.c. | LTP4 @ 24" o.c. | 16d @ 6" o.c. &<br>LTP4 @ 24" o.c. | RBC @ 10" o.c. | 2 rows 16d at 8" o.c. |

Notes:

1 Plywood or OSB sheathing 15/32" thick shall be used as shown in this table. 7/16" thick sheathing may be substituted provided studs are spaced a maximum of 16 inched on center or panels are applied with long dimensions across studs.

note not all symbols from schedule may be used on every project

# HOLD-DOWN SCHEDULE

REV 08.11.2021

| HOLD-DOWN TYPE | | ANCHOR BOLT | | | BOUNDARY MEMBER POST & FASTENING | |
|---|---|---|---|---|---|---|
| TYPE | SIMPSON CALLOUT | ANCHOR U.N.O. | MIN. EMBEDMENT | MIN. STEM WALL WIDTH | HOLD-DOWN FASTENING TO POST | MIN. POST SIZE, NUMBER & FASTENING OF BUILT UP POSTS |
| A | DTT2Z | ½"Ø HOOKED ANCHOR | 7" W/ 1¾" MIN. EDGE DISTANCE | 6" | (8) SDS ¼"x1½" SCREWS | (1) 2x WALL DEPTH STUD |
| A2 | LSTA36 | N.A. | N.A. | N.A. | (7) 10d COMMON EA END OF STRAP | (1) 2x WALL DEPTH STUD |
| B | HDU2 | SIMPSON SSTB20L | 16⅝" | 6" | (6) SDS ¼"x2½" SCREWS | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (12) 16d SINKERS |
| B2 | MSTC 40 | N.A. | N.A. | N.A. | (14) 10d COMMON EA END OF STRAP | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (18) 16d SINKERS |
| C | HDU4-SDS | SIMPSON SB ⅝x24 | 18" | 6" | (10) SDS ¼"x2½" SCREWS | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (18) 16d SINKERS |
| C2 | MSTC 52 | N.A. | N.A. | N.A. | (22) 10d COMMON EA END OF STRAP | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (18) 16d SINKERS |
| D | HDU5-SDS | SIMPSON SB ⅝x24 | 18" | 6" | (14) SDS ¼"x2½" SCREWS | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (24) 16d SINKERS |
| D2 | MSTC 66 | N.A. | N.A. | N.A. | (32) 10d COMMON EA END OF STRAP | (2) 2x WALL DEPTH STUD, FASTEN TOGETHER W/ (24) 16d SINKERS |
| E | HDU8-SDS | SIMPSON SB ⅞x24 | 18" | 8" | (20) SDS ¼"x2½" SCREWS | (1) 4x4 OR (3) 2x4 |
| F | HDU11-SDS | SIMPSON SB 1x30 | 24" | 8" | (30) SDS ¼"x2½" SCREWS | (1) 5½"x3½" OR (1) 7¼"x3½" AS NOTED ON PLAN |
| G | HDU14-SDS | 1"Ø PER PLANS | PER PLANS | PER PLANS | (36) SDS ¼"x2½" SCREWS | (1) 6x6 MIN. |

NOTES:
1. FASTEN HOLD-DOWNS TO THE BOUNDARY MEMBERS FOR THE SHEAR WALL AT THE LOCATIONS MARKED ON THE PLANS.
2. SHEAR WALL PANELS SHALL BE FASTENED TO THE BOUNDARY MEMBER POSTS PER THE PANEL EDGE SPACING ON THE SHEAR WALL SCHEDULE.
3. WHERE BOUNDARY MEMBERS ARE BUILT UP MEMBERS OR OVER 2" NOMINAL, EDGE NAILING SHALL BE STAGGERED INTO TWO ROWS.
4. ALL HOLD-DOWNS AND ANCHOR BOLTS SHALL BE INSTALLED PER THE MANUFACTURERS INSTRUCTIONS.
5. ALL HOLD-DOWNS AND BOUNDARY MEMBER POSTS SHALL BE INSTALLED TO FORM A CONTINUOUS LOAD PATH FROM EACH END OF THE SHEAR WALL TO THE FOUNDATION BELOW.

Note, not all symbols or hold down types are used on every project.

MAE ENGINEERING

HOLD-DOWN SCHEDULE

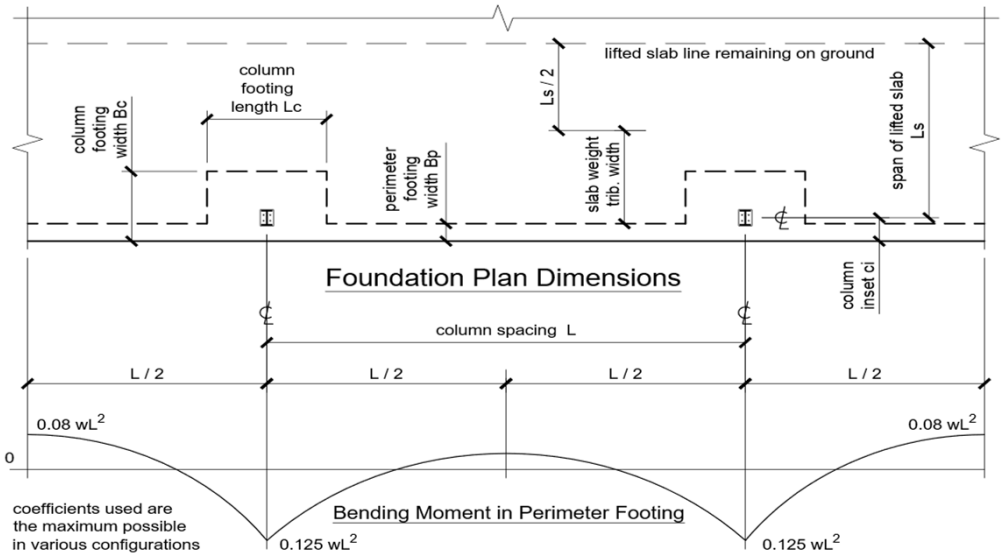


010008

Work Order: 23038 Project: Tavares - Office Uniform line Date: 10/29/2022

# Monopour Slab Edge Footing with Integral Spread Footing at Column

by SNS 2/2021 input in blue cells output in gray cells




**Dimensions**

| | | | |
|---|---|---|---|
| Footing Height | $h_f$ | 18 | in |
| Perimeter Footing Width | $B_p$ | 16 | in |
| Column Footing Width | $B_c$ | 1.33 | ft |
| Column Footing Length | $L_c$ | 1.33 | ft |
| Column Spacing | L | 1.33 | ft |
| Column Inset | $c_i$ | 3 | in |
| Slab Thickness | $h_s$ | 4 | in |
| Slab lifted span | $L_s$ | 2 | ft |

**Loading**

| | | service level | | *load factor* | strength level | |
|---|---|---|---|---|---|---|
| Column Dead Load | $P_D$ | 0.63 | k | 1.2 | 0.8 | k |
| Column Live Load | $P_L$ | 1.61 | k | 1.6 | 2.6 | k |
| Column Wind Load Combination Net Uplift (indicate as positive) | $P_W$ | 0.05 | k | 1.67 | 0.1 | k |

**Soil Bearing**

| | | | | | |
|---|---|---|---|---|---|
| Allowable soil bearing pressure | $q_a$ | 1500 | psf | | |
| Footing Area, combined | | 1.8 | $ft^2$ | | |
| Centroid of combined Footing Area | | 8.0 | inches from outside edge | (compare to Column Inset) | |
| Eccentricity of column load on footing centroid | | -5.0 | in | | |
| Soil Bearing Pressure with Service Level Gravity Load | | 1484 | psf | < qa OK | |

**Weight of concrete resisting uplift**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| weight of column footing | | 0.38 | k | | | Density of Concrete | 145 pcf |
| weight of perimeter ftg between column ftgs | | 0.00 | k | | | | |
| weight of tributary slab between column ftgs | | 0.00 | k | | | | |
| weight of tributary slab at column ftgs | | -0.01 | k | | | | |
| | total weight | 0.4 | k | 0.6 total weight = | 0.2 k | > service level Pw | OK |

**Distributed Loads on footing beam** strength level

| | | | | |
|---|---|---|---|---|
| net upward pressure on footing due to gravity | | 1884 | psf | (soil pressure minus ftg weight) |
| uniform upward load on perimeter ftg due to gravity | $w_p$ | 2512 | plf | |
| uniform upward load on column ftg area inboard of perimeter ftg | $w_c$ | -6 | plf | |
| uniform downward load on footing due to wind , average | | 63 | plf | |

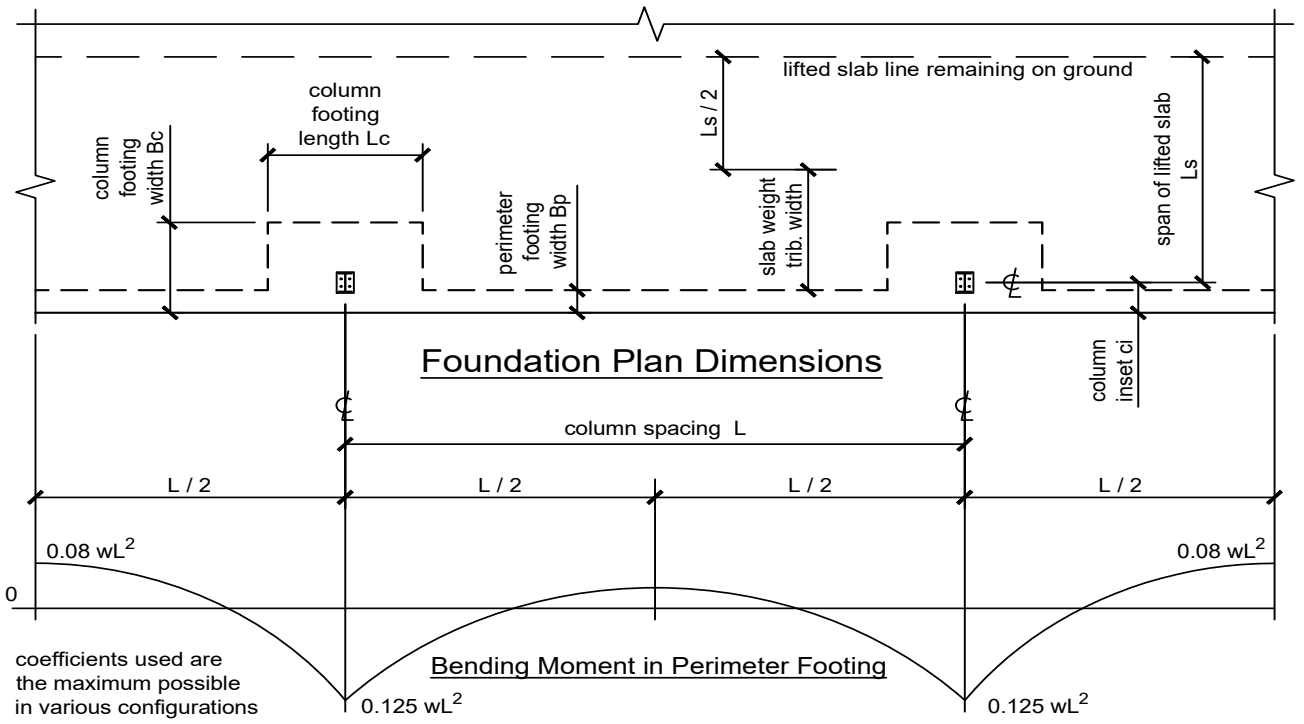
**Bending Moment in footing** strength level

| At Column | $M_u$ | | Between Columns | $M_u$ | |
|---|---|---|---|---|---|
| **positive moment due to gravity load:** | 1 | ft k | **positive moment due to wind load** | 0 | ft k |
| **negative moment due to wind load** | 0 | ft k | **negative moment due to gravity load** | 0 | ft k |

Work Order: 23038 Project: Tavares - Office header Point Date: 10/29/2022

# Monopour Slab Edge Footing with Integral Spread Footing at Column

by SNS 2/2021 input in blue cells output in gray cells




**Dimensions**

| | | | |
|---|---|---|---|
| Footing Height | $h_f$ | 18 | in |
| Perimeter Footing Width | $B_p$ | 12 | in |
| Column Footing Width | $B_c$ | 1 | ft |
| Column Footing Length | $L_c$ | 1 | ft |
| Column Spacing | L | 4 | ft |
| Column Inset | $c_i$ | 2.25 | in |
| Slab Thickness | $h_s$ | 4 | in |
| Slab lifted span | $L_s$ | 2 | ft |

**Loading**

| | | service level | | *load factor* | strength level | |
|---|---|---|---|---|---|---|
| Column Dead Load | $P_D$ | 1 | k | 1.2 | 1.2 | k |
| Column Live Load | $P_L$ | 3.56 | k | 1.6 | 5.7 | k |
| Column Wind Load Combination Net Uplift (indicate as positive) | $P_W$ | 0.1 | k | 1.67 | 0.2 | k |

**Soil Bearing**

| | | | | | |
|---|---|---|---|---|---|
| Allowable soil bearing pressure | $q_a$ | 1500 | psf | | |
| Footing Area, combined | | 4.0 | $ft^2$ | | |
| Centroid of combined Footing Area | | 6.0 | inches from outside edge (compare to Column Inset) | | |
| Eccentricity of column load on footing centroid | | -3.8 | in | | |
| Soil Bearing Pressure with Service Level Gravity Load | | 1358 | psf | < qa OK | |

**Weight of concrete resisting uplift**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| weight of column footing | | 0.22 | k | Density of Concrete | 145 | pcf | |
| weight of perimeter ftg between column ftgs | | 0.65 | k | | | | |
| weight of tributary slab between column ftgs | | 0.03 | k | | | | |
| weight of tributary slab at column ftgs | | 0.01 | k | | | | |
| | total weight | 0.9 | k | 0.6 total weight = | 0.5 | k | > service level Pw OK |

**Distributed Loads on footing beam** strength level

| | | | | |
|---|---|---|---|---|
| net upward pressure on footing due to gravity | | 1724 | psf | (soil pressure minus ftg weight) |
| uniform upward load on perimeter ftg due to gravity | $w_p$ | 1724 | plf | |
| uniform upward load on column ftg area inboard of perimeter ftg | $w_c$ | 0 | plf | |
| uniform downward load on footing due to wind , average | | 42 | plf | |

**Bending Moment in footing** strength level

| At Column | $M_u$ | | Between Columns | $M_u$ | |
|---|---|---|---|---|---|
| **positive moment due to gravity load:** | 3 | ft k | **positive moment due to wind load** | 0 | ft k |
| **negative moment due to wind load** | 0 | ft k | **negative moment due to gravity load** | 2 | ft k |

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**CALCULATIONS**

STRUCTURAL
BUILDING DESIGN • FIRE PROTECTION
CODE CONSULTANT • PLAN CHECKING
CONSTRUCTION INSPECTION

| WORK ORDER# | PAGE # |
|---|---|
| | |
| PROJECT NAME | BY/CHECKED |
| | |
| REF | DATE |
| | 10-11-22 |

**FOUNDATIONS**

Spread Footing Schedule: assumes **1500 psf** allowable soild bearing pressure

| Type | Size | Reinforcement | Allowable Load (kips) | Dead Load (kips) |
|---|---|---|---|---|
| 1 | 16" x 16" x 8" | (1)#4 E.W. bot. | 2.4 | 0.17 |
| 2 | 18" x 18" x 10" | (1)#4 E.W. bot. | 3.1 | 0.28 |
| 3 | 24" x 24" x 10" | (2) #4 E.W. bot. | 5.5 | 0.5 |
| 4 | 28" x 28" x 10" | (2) #4 E.W. bot. | 7.45 | 0.66 |
| 5 | 30" x 30" x 10" | (3) #4 E.W. bot. | 8.5 | 0.78 |
| 6 | 32" x 32" x 10" | (3) #4 E.W. bot. | 9.75 | 0.88 |
| 7 | 36" x 36" x 10" | (4) #4 E.W. bot. | 12.25 | 1.1 |
| 8 | 42" x 42" x 10" | (4) #4 E.W. bot. | 16.75 | 1.5 |
| 9 | 48" x 48" x 10" | (5) #4 E.W. bot. | 22 | 2 |
| 10 | 54" x 54" x 12" | (6) #4 E.W. bot. | 27 | 3 |
| 11 | 60" x 60" x 12" | (8) #4 E.W. bot. | 34.25 | 3.75 |
| 12 | 72" x 72" x 14" | (7) #5 E.W. bot. | 48 | 6.3 |

Spread Footing Schedule: assumes **1000 psf** allowable soild bearing pressure

| Type | Size | Reinforcement | Allowable Load (kips) | Dead Load (kips) |
|---|---|---|---|---|
| 1 | 16" x 16" x 8" | (1) #4 E.W. bot. | 1.5 | 0.17 |
| 2 | 18" x 18" x 10" | (1) #4 E.W. bot. | 2 | 0.28 |
| 3 | 24" x 24" x 10" | (2) #4 E.W. bot. | 3.5 | 0.5 |
| 4 | 28" x 28" x 10" | (2) #4 E.W. bot. | 4.7 | 0.66 |
| 5 | 30" x 30" x 10" | (3) #4 E.W. bot. | 5.6 | 0.78 |
| 6 | 32" x 32" x 10" | (3) #4 E.W. bot. | 6.2 | 0.88 |
| 7 | 36" x 36" x 10" | (4) #4 E.W. bot. | 8 | 1.1 |
| 8 | 42" x 42" x 10" | (4) #4 E.W. bot. | 10.5 | 1.5 |
| 9 | 48" x 48" x 10" | (5) #4 E.W. bot. | 14 | 2 |
| 10 | 54" x 54" x 12" | (6) #4 E.W. bot. | 17 | 3 |
| 11 | 60" x 60" x 12" | (8) #4 E.W. bot. | 21 | 3.75 |

Perimeter Footing Schedule: assumes **1500 psf** allowable soil bearing pressure

| No. of story | Foundation Wall | Footing Width | Footing Thickness | Capacity (klf) | Point Load (kips) |
|---|---|---|---|---|---|
| 1 story | 6" thick | 12" | 6" | 1.5 | 6 |
| 2 story | 8" thick | 15" | 7" | 1.875 | 7.5 |
| 3 story | 10" thick | 18" | 8" | 2.25 | 9 |

Round Footing Schedule: assumes **1500 psf** allowable soil bearing pressure

| Type | Diameter | Allowable Load (kips) |
|---|---|---|
| 1 | 24" | 4.7 |
| 2 | 30" | 7.3 |
| 3 | 36" | 10.6 |

**Slab:** 4" concrete (2500) psi slab w/ #3 @ 24" o.c. E.W. @ Center of Slab over 6mil. Poly. Over 6" of 3/4" minus crushed rock.
Gravel Compacted to 95% ASTM D1557. Reinforcement to be discontinuous at every other bar at control joints.
Control joints to be tooled or saw cut to 1/4 depth of slab. C.J. spacing in feet = 2.5 x slab thickness in inches

Rebar: Min. Lap: 12" (40ksi) = 40 diameters