WELD COUNTY - CRIME LAB STORAGE BUILDING

Project Location: 2329 115TH AVE., GREELEY, CO

Project No.: RSA02.30

PREPARED FOR:
WELD COUNTY

PREPARED BY:
Galloway & Company, Inc.
6162 S. Willow Drive, Suite 320
Greenwood Village, CO 80126

DATE:
January 24, 2020
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LOCAL INFORMATION
<table>
<thead>
<tr>
<th>LOCAL BUILDING DESIGN INFO</th>
</tr>
</thead>
</table>

**SITE NAME:** Crime Lab Storage Building  
**SITE ADDRESS:** 12111 West 28th Street  
Greeley, CO

**THE FOLLOWING INFORMATION WAS OBTAINED VIA:**  
[ ] City Website  
[ ] Contacting City Engineer Department

<table>
<thead>
<tr>
<th>CITY BUILDING DEPARTMENT WEBSITE:</th>
<th>CITY ENGINEERING DEPARTMENT CONTACT USED:</th>
</tr>
</thead>
<tbody>
<tr>
<td>[ ] City Website</td>
<td>[ ] Contacting City Engineer Department</td>
</tr>
</tbody>
</table>

**BUILDING CODE:**  
**RISK CATEGORY (ASCE 7-10: Table 1.5-1):**

<table>
<thead>
<tr>
<th>IBC 2015</th>
<th>II</th>
</tr>
</thead>
</table>

**SNOW:**  
- GROUND: 30 psf
- UNIFORM ROOF: 20 psf minimum
- FLAT ROOF: NA

**WIND:**  
- EXPOSURE: C
- 3 SEC GUST ($V_{UL}$) = 115 mph
- NOMINAL WIND SPEED ($V_{ASD}$) = 89.08 mph

**SEISMIC:**  
- DESIGN CATEGORY: B

**SOILS:**  
- FROST DEPTH: 30 in
- SITE CLASS: Class D - Stiff Soil
DEAD & LIVE LOADS
### Dead Load Calculations

#### Roof

<table>
<thead>
<tr>
<th>Material</th>
<th>Density</th>
<th>Multiplier (Ex. Layers of Material)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single-ply, sheet</td>
<td>None</td>
<td>0.7 psf 1.00 0.7 psf</td>
</tr>
<tr>
<td>Polystyrene foam</td>
<td>None</td>
<td>0.2 psf 3.00 0.2 psf</td>
</tr>
<tr>
<td>Metal Decking Type B 22 gauge</td>
<td>21.5 psf 1.00 21.5 psf</td>
<td></td>
</tr>
<tr>
<td>Metal Joists Type K Metal Joists</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Steel</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Mechanical Duct Allowance</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Uniform sprinkler load</td>
<td>None</td>
<td>1.5 psf 1.00 1.5 psf</td>
</tr>
<tr>
<td>1/2&quot; Plywood</td>
<td>None</td>
<td>1.2 psf 1.00 1.2 psf</td>
</tr>
<tr>
<td><strong>Manually input additional materials</strong></td>
<td></td>
<td>0 psf 1.00 0 psf</td>
</tr>
</tbody>
</table>

**Total Roof Dead Load = 11.61 psf**

**Use = 20 psf**

#### Live Loads

**Total Floor Dead Load = 43.78 psf**

**Use = 45 psf**

#### Elevated Vault Concrete Ceiling

<table>
<thead>
<tr>
<th>Material</th>
<th>Density</th>
<th>Multiplier (Ex. Layers of Material)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>None</td>
<td>12 psf 3.50 12 psf</td>
</tr>
<tr>
<td>Regular Concrete/Fill Finish Concrete (1&quot; Thick)</td>
<td>None</td>
<td>22.2 psf 1.00 22.2 psf</td>
</tr>
<tr>
<td>Light Gage Wall</td>
<td>None</td>
<td>1.7 psf 1.00 1.7 psf</td>
</tr>
<tr>
<td>Joist 4 inch</td>
<td>None</td>
<td>2.8 psf 1.00 2.8 psf</td>
</tr>
<tr>
<td>Gage 36</td>
<td>None</td>
<td>1.5 psf 1.00 1.5 psf</td>
</tr>
<tr>
<td>16 inch o.c.</td>
<td>None</td>
<td>1.0 psf 1.00 1.0 psf</td>
</tr>
<tr>
<td><strong>Manually input additional materials</strong></td>
<td></td>
<td>1.0 psf 1.00 1.0 psf</td>
</tr>
</tbody>
</table>

**Total Interior Wall Dead Load = 9.99 psf**

**Use = 10 psf**

#### Interior Masonry Wall - 8" (Grouted at 48" O.C.)

<table>
<thead>
<tr>
<th>Material</th>
<th>Density</th>
<th>Multiplier (Ex. Layers of Material)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Wall</td>
<td>None</td>
<td>42 psf 1.00 42 psf</td>
</tr>
<tr>
<td>Hollow concrete masonry</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Wythe 8 inch thickness 125</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>18-in. o.c. 125 &amp; 8 inc.</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td><strong>Manually input additional materials</strong></td>
<td></td>
<td>1.5 psf 1.00 1.5 psf</td>
</tr>
</tbody>
</table>

**Total Interior Wall Dead Load = 46.6 psf**

**Use = 50 psf**

#### Interior Vault Wall - 8" Fully Grouted Wall

<table>
<thead>
<tr>
<th>Material</th>
<th>Density</th>
<th>Multiplier (Ex. Layers of Material)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry Wall</td>
<td>None</td>
<td>81 psf 1.00 81 psf</td>
</tr>
<tr>
<td>Hollow concrete masonry</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Wythe 8 inch thickness 125</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td>Full grout 125 &amp; 8 inc.</td>
<td>None</td>
<td>0 psf 1.00 0 psf</td>
</tr>
<tr>
<td><strong>Manually input additional materials</strong></td>
<td></td>
<td>1.5 psf 1.00 1.5 psf</td>
</tr>
</tbody>
</table>

**Total Interior Wall Dead Load = 83.6 psf**

**Use = 85 psf**
SNOW SURCHARGE
(AT GARAGE AREA ONLY)
LOADING IN THIS AREA HAS BEEN DETERMINED BY OTHERS. THIS SECTION OF BLDG IS A PRE-FABRICATED METAL BUILDING AND DESIGNED BY THE METAL BUILDING MANUFACTURER.
SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details
Roof type
Flat
Width of roof
b = 32.00 ft

Ground snow load
Ground snow load (Figure 7.2-1)

\[ p_g = 30.00 \text{ lb/ft}^2 \]

Density of snow
\[ \gamma = \min(0.13 \times p_g / \text{1 ft} + 14 \text{lb/ft}^3, 30 \text{lb/ft}^3) = 17.90 \text{ lb/ft}^3 \]

Terrain type
Sect. 26.7

Exposure condition (Table 7.3-1)

\[ C_e = 0.90 \]

Thermal condition (Table 7.3-2)

\[ C_t = 1.20 \]

Importance category (Table 1.5-1)

\[ I_s = 1.00 \]

Min snow load for low slope roofs (Sect 7.3.4)

\[ p_{r_min} = I_s \times 20 \text{ lb/ft}^2 = 20.00 \text{ lb/ft}^2 \]

Balanced snow load at ground level (Sect 7.2)

\[ p_{s_ground} = I_s \times p_g = 30.00 \text{ lb/ft}^2 \]

Flat roof snow load (Sect 7.3)

\[ p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 22.68 \text{ lb/ft}^2 \]

Left parapet
Balanced snow load height

\[ h_b = p_f / \gamma = 1.27 \text{ ft} \]

Height of left parapet

\[ h_{pptL} = 4.50 \text{ ft} \]

Height from balance load to top of left parapet

\[ h_{u_{pptL}} = h_{pptL} - h_b = 3.23 \text{ ft} \]

Length of roof - left parapet

\[ l_{u_{pptL}} = b = 32.00 \text{ ft} \]

Drift height windward drift - left parapet

\[ h_{d_{L_{pptL}}} = \sqrt{(I_s) \times 0.75 \times (0.43 \times \max(20 \text{ ft}, l_{u_{pptL}}) \times 1 \text{ft}^2)^{1/3} \times (p_g / 1 \text{lb/ft}^2 + 10)^{1/4} \times 1.5 \text{ ft}} = 1.45 \text{ ft} \]

Drift height - left parapet

\[ h_{d_{pptL}} = \min(h_{d_{L_{pptL}}}, h_{pptL} - h_b) = 1.45 \text{ ft} \]

Drift width

\[ W_{d_{pptL}} = \min(4 \times h_{d_{L_{pptL}}}, 8 \times (h_{pptL} - h_b), b) = 5.80 \text{ ft} \]

Drift surcharge load - left parapet

\[ p_{d_{pptL}} = h_{d_{pptL}} \times \gamma = 25.95 \text{ lb/ft}^2 \]

Right parapet
Height of right parapet

\[ h_{pptR} = 4.50 \text{ ft} \]

Height from balance load to top of right parapet

\[ h_{u_{pptR}} = h_{pptR} - h_b = 3.23 \text{ ft} \]

Length of roof - right parapet

\[ l_{u_{pptR}} = b = 32.00 \text{ ft} \]

Drift height windward drift - right parapet

\[ h_{d_{L_{pptR}}} = \sqrt{(I_s) \times 0.75 \times (0.43 \times \max(20 \text{ ft}, l_{u_{pptR}}) \times 1 \text{ft}^2)^{1/3} \times (p_g / 1 \text{lb/ft}^2 + 10)^{1/4} \times 1.5 \text{ ft}} = 1.45 \text{ ft} \]

Drift height - right parapet

\[ h_{d_{pptR}} = \min(h_{d_{L_{pptR}}}, h_{pptR} - h_b) = 1.45 \text{ ft} \]

Drift width

\[ W_{d_{pptR}} = \min(4 \times h_{d_{L_{pptR}}}, 8 \times (h_{pptR} - h_b), b) = 5.80 \text{ ft} \]

Drift surcharge load - right parapet

\[ p_{d_{pptR}} = h_{d_{pptR}} \times \gamma = 25.95 \text{ lb/ft}^2 \]
SNOW LOADING

In accordance with ASCE7-16

Tedds calculation version 1.0.09

Building details

- Roof type: Flat
- Width of roof: b = 80.00 ft

Ground snow load

- Ground snow load (Figure 7.2-1): \( p_g = 30.00 \text{ lb/ft}^2 \)
- Density of snow: \( \gamma = \min(0.13 \times p_g / 1 \text{ ft} + 14 \text{ lb/ft}^3, 30 \text{ lb/ft}^3) = 17.90 \text{ lb/ft}^3 \)
- Terrain type: Sect. 26.7, C
- Exposure condition (Table 7.3-1): Fully exposed
- Exposure factor (Table 7.3-1): \( C_e = 0.90 \)
- Thermal condition (Table 7.3-2): Unheated structures
- Thermal factor (Table 7.3-2): \( C_t = 1.20 \)
- Importance category (Table 1.5-1): II
- Importance factor (Table 1.5-2): \( I_s = 1.00 \)

Min snow load for low slope roofs (Sect 7.3.4): \( p_{r,\min} = I_s \times 20 \text{ lb/ft}^2 = 20.00 \text{ lb/ft}^2 \)

Balanced snow load at ground level (Sect 7.2): \( p_{s,\text{ground}} = I_s \times p_g = 30.00 \text{ lb/ft}^2 \)

Flat roof snow load (Sect 7.3): \( p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 22.68 \text{ lb/ft}^2 \)

Left parapet

- Balanced snow load height: \( h_b = p_f / \gamma = 1.27 \text{ ft} \)
- Height of left parapet: \( h_{\text{pptL}} = 4.50 \text{ ft} \)
- Height from balance load to top of left parapet: \( h_{\text{u pptL}} = h_{\text{pptL}} - h_b = 3.23 \text{ ft} \)
- Length of roof - left parapet: \( l_{\text{u pptL}} = b = 80.00 \text{ ft} \)
- Drift height windward drift - left parapet: \( h_{d,\text{l pptL}} = \sqrt{(I_s) \times 0.75 \times (0.43 \times (\text{max}(20 \text{ ft}, l_{\text{u pptL}}) \times 1 \text{ ft}^2)\frac{1}{3} \times (p_g / 1 \text{ lb/ft}^2 + 10)}^{\frac{1}{4}} - 1.5 \text{ ft} = 2.37 \text{ ft} \)
- Drift height - left parapet: \( h_{d,\text{pptL}} = \min(h_{d,\text{l pptL}}, h_{\text{pptL}} - h_b) = 2.37 \text{ ft} \)
- Drift width: \( W_{d,\text{pptL}} = \min(4 \times h_{d,\text{l pptL}}, 8 \times (h_{\text{pptL}} - h_b), b) = 9.48 \text{ ft} \)
- Drift surcharge load - left parapet: \( p_{d,\text{pptL}} = h_{d,\text{pptL}} \times \gamma = 42.42 \text{ lb/ft}^2 \)

Right parapet

- Height of right parapet: \( h_{\text{pptR}} = 4.50 \text{ ft} \)
- Height from balance load to top of right parapet: \( h_{\text{u pptR}} = h_{\text{pptR}} - h_b = 3.23 \text{ ft} \)
- Length of roof - right parapet: \( l_{\text{u pptR}} = b = 80.00 \text{ ft} \)
- Drift height windward drift - right parapet: \( h_{d,\text{l pptR}} = \sqrt{(I_s) \times 0.75 \times (0.43 \times (\text{max}(20 \text{ ft}, l_{\text{u pptR}}) \times 1 \text{ ft}^2)\frac{1}{3} \times (p_g / 1 \text{ lb/ft}^2 + 10)}^{\frac{1}{4}} - 1.5 \text{ ft} = 2.37 \text{ ft} \)
- Drift height - right parapet: \( h_{d,\text{pptR}} = \min(h_{d,\text{l pptR}}, h_{\text{pptR}} - h_b) = 2.37 \text{ ft} \)
- Drift width: \( W_{d,\text{pptR}} = \min(4 \times h_{d,\text{l pptR}}, 8 \times (h_{\text{pptR}} - h_b), b) = 9.48 \text{ ft} \)
- Drift surcharge load - right parapet: \( p_{d,\text{pptR}} = h_{d,\text{pptR}} \times \gamma = 42.42 \text{ lb/ft}^2 \)
Roof elevation

Parapet

Parapet

Balanced load

65.1 psf

22.7 psf

9' 5.7"

9' 5.7"

65.1 psf

22.7 psf

4' 6"

4' 6"

80'

30.0 psf
SEISMIC CALCS
(AT GARAGE AREA ONLY)
CRIME LAB (GREELEY) - SEISMIC BUILDING WEIGHT CALCULATIONS (GARAGE AREA ONLY)

EFFECTIVE SEISMIC WEIGHT (ASCE 7-10, Section 12.7.2)

LOADING

<table>
<thead>
<tr>
<th></th>
<th>Dead Load</th>
<th>Roof Live Load</th>
<th>Flat Roof Snow Load</th>
<th>Storage Load</th>
<th>Partition Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20.0 psf</td>
<td>20 psf</td>
<td>30 psf</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Floor</td>
<td>0 psf</td>
<td>0 psf</td>
<td>0 psf</td>
<td>NO</td>
<td>0 psf</td>
</tr>
<tr>
<td>Interior Wall</td>
<td>50 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #1</td>
<td>15 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #2</td>
<td>0 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #3</td>
<td>0 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

BUILDING PLAN SHAPE

Note: Number of levels = Finished Floor + #Floors + Roof. Example: Single Story Retail = 1 Finished Floor + 1 Roof, therefore #Levels = 2. Example: Single Story Retail = 1 Finished Floor + 1 Roof, therefore #Levels = 2.

Number of Levels: 2
Parapet: YES
Is the Building Rectangular? YES
Is the building L-Shaped? NO

North-South Length: 80.00 ft
East-West Length: 33.00 ft
Short North-South Length: 0.00 ft
Short East-West Length: 0.00 ft

Typical Roof/Floor Area: 2640.00 sf
Typical Perimeter Length: 226.00 ft
Mezzanine Area: 0.00 sf

IRREGULAR SHAPED BUILDING

Note: Manually type in typical area and perimeter length if required per note above.

Typical Roof/Floor Area: 0.00 sf
Typical Perimeter Lengths: 0.00 ft

LEVEL HEIGHTS & WALLS

Note: Where NO level is shown, type NA
Note: Wall Type is wall for tributary length.
Note: Where NO Interior Walls are shown, type NA

Height Above Finished Floor Elevation (ft) | Exterior Wall Type Per Level | Ext. Wall Tributary Area Between Floors | Length of Interior Walls at Level | Typical Height of Interior Walls
--- | --- | --- | --- | ---
Parapet: 20.67 ft | - NA | - | - NA | 10.00 ft
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
- NA | - NA | - | - NA
Level 2: 16.17 ft | Level 2: Exterior Wall #1 | Level 2: 12.58 ft | Level 2: 36.00 ft
Level 1: 0.00 ft | Level 1: Exterior Wall #1 | Level 1: 8.08 ft | Level 1: 36.00 ft

PROJECT NAME: CRIME LAB
GREELEY, CO

PROJECT NUMBER: RSA02.30
CHECKED BY: KEJ
1/24/2020
### PERMANENT ROOF EQUIPMENT OR LANDSCAPING (ASCE 7-10: Section 3.1.3 & Section 12.7.2.1.3&5)

Note: Where no equipment is required, type 0
Note: Where no landscaping is shown, type 0

<table>
<thead>
<tr>
<th>RTU Actual Weight</th>
<th>Misc. Roof Equipment</th>
<th>Landscaping DL in Addition to Above Typ. Roof DL</th>
</tr>
</thead>
<tbody>
<tr>
<td>RTU #1: 2000.0 lbs</td>
<td>Misc. #1: 0.0 lbs</td>
<td>0 psf</td>
</tr>
<tr>
<td>RTU #2: 2000.0 lbs</td>
<td>Misc. #2: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #3: 2000.0 lbs</td>
<td>Misc. #3: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #4: 2000.0 lbs</td>
<td>Misc. #4: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #5: 2000.0 lbs</td>
<td>Misc. #5: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #6: 2000.0 lbs</td>
<td>Misc. #6: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #7: 0.0 lbs</td>
<td>Misc. #7: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #8: 0.0 lbs</td>
<td>Misc. #8: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #9: 0.0 lbs</td>
<td>Misc. #9: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #10: 0.0 lbs</td>
<td>Misc. #10: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>12000.00 lbs</td>
<td></td>
<td>0.0 lbs</td>
</tr>
</tbody>
</table>

### SEISMIC BUILDING WEIGHT BY LEVEL

Note: Level 1 = Finished Floor Elevation. Finished Floor not applied to Seismic or Wind Calculations.
Note: Roof and parapet load applied to top level. Floor loads are applied to all floors. If a-typical floor loading is required, go to Irregular Shaped Building Tab.

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Live Load</th>
<th>FOR ENERCALC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor / Roof</td>
<td>Exterior Wall</td>
<td>Interior Wall</td>
</tr>
<tr>
<td>Level 2: 52800.0 lbs</td>
<td>42657.6 lbs</td>
<td>18000.0 lbs</td>
</tr>
<tr>
<td>Level 1: 0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
</tr>
</tbody>
</table>
SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.1.00

Site parameters

Site class: D

Mapped acceleration parameters (Section 11.4.1)

- At short period: $S_S = 0.157$
- At 1 sec period: $S_1 = 0.054$

Site coefficient at short period (Table 11.4-1): $F_a = 1.600$

Site coefficient at short period (Table 11.4-2): $F_v = 2.400$

Spectral response acceleration parameters

- At short period (Eq. 11.4-1): $S_{MS} = F_a \times S_S = 0.251$
- At 1 sec period (Eq. 11.4-2): $S_{M1} = F_v \times S_1 = 0.130$

Design spectral acceleration parameters (Sect 11.4.4)

- At short period (Eq. 11.4-3): $S_{DS} = 2 / 3 \times S_{MS} = 0.167$
- At 1 sec period (Eq. 11.4-4): $S_{D1} = 2 / 3 \times S_{M1} = 0.086$

Seismic design category

- Risk category (Table 1.5-1): II
- Seismic design category based on short period response acceleration (Table 11.6-1): B
- Seismic design category based on 1 sec period response acceleration (Table 11.6-2): B

Approximate fundamental period

- Height above base to highest level of building: $h_n = 16.17$ ft

From Table 12.8-2:
- All other systems
- $C_t = 0.02$
- $x = 0.75$

Approximate fundamental period (Eq 12.8-7): $T_a = C_t \times (h_n)^x \times 1 \text{sec} / (1 \text{ft})^x = 0.161$ sec

Building fundamental period (Sect 12.8.2): $T = T_a = 0.161$ sec

Long-period transition period: $T_L = 4$ sec

Seismic response coefficient

- Seismic force-resisting system (Table 12.2-1): A. Bearing_Wall_Systems
- 9. Ordinary reinforced masonry shear walls
- $R = 2$
- Seismic importance factor (Table 1.5-2): $I_e = 1.000$
- Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-3): $C_{s_{calc}} = S_{DS} / (R / I_e) = 0.0837$

Maximum (Eq 12.8-3): $C_{s_{max}} = S_{D1} / ((T / 1 \text{ sec}) \times (R / I_e)) = 0.2679$

Minimum (Eq 12.8-5): $C_{s_{min}} = max(0.044 \times S_{DS} \times I_e, 0.01) = 0.0100$
Seismic response coefficient \( C_s = 0.0837 \)

**Seismic base shear (Sect 12.8.1)**
- Effective seismic weight of the structure \( W = 125.5 \) kips
- Seismic response coefficient \( C_s = 0.0837 \)
- Seismic base shear (Eq 12.8-1) \( V = C_s \times W = 10.5 \) kips

**Vertical distribution of seismic forces (Sect 12.8.3)**
- Vertical distribution factor (Eq 12.8-12) \( C_{vx} = w_x \times h_x^k / \Sigma(w_i \times h_i^k) \)
- Lateral force induced at level \( i \) (Eq 12.8-11) \( F_x = C_{vx} \times V \)

**Vertical force distribution table**

<table>
<thead>
<tr>
<th>Level</th>
<th>Height from base to Level ( i ) (ft), ( h_x )</th>
<th>Portion of effective seismic weight assigned to Level ( i ) (kips), ( w_x )</th>
<th>Distribution exponent related to building period, ( k )</th>
<th>Vertical distribution factor, ( C_{vx} )</th>
<th>Lateral force induced at Level ( i ) (kips), ( F_x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.2;</td>
<td>125.5;</td>
<td>1.00;</td>
<td>1.000;</td>
<td>10.5</td>
</tr>
</tbody>
</table>
BUILDING PLAN VIEW - ROOF LEVEL

Lateral Force Induced at Level (per TEDDS) = 10.5 kips

BUILDING PLAN VIEW - ROOF LEVEL

5.3 kips

318.18 plf

33.00 ft

East-West Shear Walls

5.3 kips

131.25 plf

80.00 ft

North-South Shear Walls

5.3 kips

65.63 plf

5.3 kips

159.09 plf

East-West Shear Walls

5.3 kips
## What Load Combinations were used for design numbers above? ASD

Use above values for diaphragm design

### ROOF DIAPHRAGM

**Deck Type:** Vulcraft 1.5B Deck 22 Gauge  
**Deck Span:** 5.08 ft  
**Support Fasteners:** Hilti X-ENP19  
**Sidelap Fasteners:** #10 TEK screws  
**Allowable Shear:** 402.0 plf

<table>
<thead>
<tr>
<th>Design Method</th>
<th>SDS</th>
<th>Support Fasteners</th>
<th>Fastener Pattern</th>
<th>Sidelap Fasteners</th>
<th># of Sidelap Fasteners</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Type</td>
<td>1.5B</td>
<td>Hilti X-ENP19</td>
<td>36/5</td>
<td>#10 TEK screws</td>
<td>5</td>
</tr>
<tr>
<td>Deck Span</td>
<td>5.08 ft</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of Spans</td>
<td>3</td>
<td>Tables Generated Based on:</td>
<td>Num. of Sidelap Attachments per Span</td>
<td>15.00</td>
<td></td>
</tr>
<tr>
<td>H0NF3 at Wind Uplift (psf)</td>
<td>15.00</td>
<td></td>
<td>1.00</td>
<td>0.50</td>
<td></td>
</tr>
</tbody>
</table>

Please refer to the Vulcraft Deck Catalog for product availability.

The vertical gravity load capacity of the deck based on bending stress and applicable deflection criteria must be checked separately.

Use selected support attachment type for both perpendicular attachment and parallel attachment of steel deck.

**Notes:** Support Steel Thickness >= 1/8 in.

### Vulcraft Deck Diaphragm Shear & Stiffness

**Per SDS 030122**  
In accordance with 2015 IRC Section 2210.7.2A,B  
**Calculation** generated on: 4/12/2019  
**Using Calculation V1.1**

#### Input Design Criteria

<table>
<thead>
<tr>
<th>Load System</th>
<th>Imperial Deck to Support Attachment Type</th>
<th>Hilti X-ENP19</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Option</td>
<td>Base Deck</td>
<td>Support Number</td>
</tr>
<tr>
<td>Deck Type</td>
<td>1.5B-36</td>
<td>Perpendicular Attachment Pattern</td>
</tr>
<tr>
<td>Deck Gauge</td>
<td>36</td>
<td>Sidelap Attachment Pattern</td>
</tr>
<tr>
<td>Deck Grade</td>
<td>36</td>
<td>Table 10 Generator Formulation</td>
</tr>
</tbody>
</table>

#### ASD Diaphragm Shear Strength (plf)

<table>
<thead>
<tr>
<th>Num. of Sidelap Attachments per Span</th>
<th>ASD Diaphragm Shear Strength (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (Ft. in.)</td>
<td></td>
</tr>
<tr>
<td>4-9'</td>
<td>5-9'</td>
</tr>
<tr>
<td>1</td>
<td>341</td>
</tr>
<tr>
<td>2</td>
<td>388</td>
</tr>
<tr>
<td>3</td>
<td>431</td>
</tr>
<tr>
<td>4</td>
<td>470</td>
</tr>
<tr>
<td>5</td>
<td>501</td>
</tr>
<tr>
<td>6</td>
<td>537</td>
</tr>
<tr>
<td>7</td>
<td>565</td>
</tr>
</tbody>
</table>

#### ASD Diaphragm Shear Strength & Wind Uplift Interaction (psf)

<table>
<thead>
<tr>
<th>Num. of Sidelap Attachments per Span</th>
<th>ASD Diaphragm Shear Strength &amp; Wind Uplift Interaction (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (Ft. in.)</td>
<td></td>
</tr>
<tr>
<td>4-9'</td>
<td>5-9'</td>
</tr>
<tr>
<td>1</td>
<td>461</td>
</tr>
<tr>
<td>2</td>
<td>413</td>
</tr>
<tr>
<td>3</td>
<td>450</td>
</tr>
<tr>
<td>4</td>
<td>460</td>
</tr>
<tr>
<td>5</td>
<td>537</td>
</tr>
<tr>
<td>6</td>
<td>571</td>
</tr>
<tr>
<td>7</td>
<td>641</td>
</tr>
</tbody>
</table>

#### ASD Diaphragm Stiffness, G (Kips/In)

<table>
<thead>
<tr>
<th>Num. of Sidelap Attachments per Span</th>
<th>ASD Diaphragm Stiffness, G (Kips/In)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (Ft. in.)</td>
<td></td>
</tr>
<tr>
<td>4-9'</td>
<td>5-9'</td>
</tr>
<tr>
<td>1</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>5</td>
<td>16</td>
</tr>
<tr>
<td>6</td>
<td>16</td>
</tr>
<tr>
<td>7</td>
<td>16</td>
</tr>
</tbody>
</table>

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CHECK DIAPHRAGM SHEARS CONT.

ASD Load Combinations are being used in this design

<table>
<thead>
<tr>
<th>North-South</th>
<th>East-West</th>
<th>North-South</th>
<th>East-West</th>
<th>North-South</th>
<th>East-West</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.250 kips</td>
<td>5.250 kips</td>
<td>65.64 plf</td>
<td>159.09 plf</td>
<td>65.64 plf</td>
<td>159.09 plf</td>
</tr>
</tbody>
</table>

CHECK CHORD FORCES

North-South Chord (LRFD Loading)

\[ M_{u,N-S} = \frac{wL^2}{8} = 43.3 \text{ kip-ft} \]

\[ T_{u,N-S} = C_{u,N-S} = \frac{M_{u,N-S}}{b} = 0.54 \text{ kips} \]

<table>
<thead>
<tr>
<th>OPTION 1</th>
<th>OPTION 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>15x3x1/4</td>
<td>15x3x1/4</td>
</tr>
<tr>
<td>1.940 in²</td>
<td>1.930 in²</td>
</tr>
<tr>
<td>( F_y )</td>
<td>( F_y )</td>
</tr>
<tr>
<td>46 ksi</td>
<td>36 ksi</td>
</tr>
</tbody>
</table>

GOOD \hspace{1cm} GOOD

Option 1, \( f \_u = \frac{T_{u,N-S}}{A_s} = 0.28 \text{ ksi} \)

\[ f_{\text{allow}} = \phi F_y = 41.40 \text{ ksi} \]

GOOD

Option 2, \( f \_u = \frac{T_{u,N-S}}{A_s} = 0.28 \text{ ksi} \)

\[ f_{\text{allow}} = \phi F_y = 32.40 \text{ ksi} \]

GOOD

East-West Chord (LRFD Loading)

\[ M_{u,E-W} = \frac{wL^2}{8} = 105.0 \text{ kip-ft} \]

\[ T_{u,E-W} = C_{u,E-W} = \frac{M_{u,E-W}}{b} = 3.18 \text{ kips} \]

<table>
<thead>
<tr>
<th>OPTION 1</th>
<th>OPTION 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>15x3x1/4</td>
<td>15x3x1/4</td>
</tr>
<tr>
<td>1.940 in²</td>
<td>1.930 in²</td>
</tr>
<tr>
<td>( F_y )</td>
<td>( F_y )</td>
</tr>
<tr>
<td>46 ksi</td>
<td>36 ksi</td>
</tr>
</tbody>
</table>

GOOD \hspace{1cm} GOOD

Option 1, \( f \_u = \frac{T_{u,E-W}}{A_s} = 1.64 \text{ ksi} \)

\[ f_{\text{allow}} = \phi F_y = 41.40 \text{ ksi} \]

GOOD

Option 2, \( f \_u = \frac{T_{u,E-W}}{A_s} = 1.65 \text{ ksi} \)

\[ f_{\text{allow}} = \phi F_y = 32.40 \text{ ksi} \]

GOOD
SEISMIC CALCS
(AT VAULT AREA ONLY)
Effectiveness of Seismic Weight (ASCE 7-10, Section 12.7.2)

### Loading

<table>
<thead>
<tr>
<th></th>
<th>Dead Load</th>
<th>Roof Live Load</th>
<th>Live Load</th>
<th>Flat Roof Snow Load</th>
<th>Storage Load</th>
<th>Partition Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof/Vault Ceiling</td>
<td>0 psf</td>
<td>0 psf</td>
<td>-</td>
<td>0 psf</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Floor/Vault Ceiling</td>
<td>45 psf</td>
<td>-</td>
<td>125 psf</td>
<td>0 psf</td>
<td>YES</td>
<td>10 psf</td>
</tr>
<tr>
<td>Interior Wall</td>
<td>10 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #1</td>
<td>85 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #2</td>
<td>0 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Exterior Wall #3</td>
<td>0 psf</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

### Building Plan Shape

- **Number of Levels:** 3
- **Parapet:** NO
- **Is the Building Rectangular?** YES
- **Is the Building L-Shaped?** NO

| North-South Length: | 33.00 ft |
| East-West Length:   | 26.00 ft |
| Short North-South Length: | 0.00 ft |
| Short East-West Length: | 0.00 ft |

Typical Roof/Floor Area: 858.00 sf
Typical Perimeter Length: 118.00 ft

### Irregular Shaped Building

Note: Manually type in typical area and perimeter length if required per note above.

| Typical Roof/Floor Area: | 0.00 sf |
| Typical Perimeter Length: | 0.00 ft |

### Level Heights & Walls

- **Note:** Where NO floor level is shown, type NA
- **Note:** Wall Type is wall for tributary length.
- **Note:** Where NO Interior Walls are shown, type NA

<table>
<thead>
<tr>
<th>Height Above Finished Floor Elevation (ft)</th>
<th>Exterior Wall Type Per Level</th>
<th>Ext. Wall Tributary Area Between Floors</th>
<th>Length of Interior Walls at Level</th>
<th>Typical Height of Interior Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>- NA</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>12.00 ft</td>
</tr>
<tr>
<td>- NA</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>- NA</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>- NA</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Level 3: 12.25 ft</td>
<td>Level 3: Exterior Wall #1</td>
<td>Level 3: 0.13 ft</td>
<td>Level 3: 0.00 ft</td>
<td>-</td>
</tr>
<tr>
<td>Level 2: 12.00 ft</td>
<td>Level 2: Exterior Wall #1</td>
<td>Level 2: 6.13 ft</td>
<td>Level 2: 100.00 ft</td>
<td>-</td>
</tr>
<tr>
<td>Level 1: 0.00 ft</td>
<td>Level 1: Exterior Wall #1</td>
<td>Level 1: 6.00 ft</td>
<td>Level 1: 100.00 ft</td>
<td>-</td>
</tr>
</tbody>
</table>

1/24/2020
PERMANENT ROOF EQUIPMENT OR LANDSCAPING (ASCE 7-10: Section 3.1.3 & Section 12.7.2.1.3&5)

Note: Where no equipment is required, type 0
Note: Where no landscaping is shown, type 0

<table>
<thead>
<tr>
<th>RTU Actual Weight</th>
<th>Misc. Roof Equipment</th>
<th>Landscaping DL in Addition to Above Typ. Roof DL</th>
</tr>
</thead>
<tbody>
<tr>
<td>RTU #1: 0.0 lbs</td>
<td>Misc. #1: 0.0 lbs</td>
<td>0 psf</td>
</tr>
<tr>
<td>RTU #2: 0.0 lbs</td>
<td>Misc. #2: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #3: 0.0 lbs</td>
<td>Misc. #3: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #4: 0.0 lbs</td>
<td>Misc. #4: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #5: 0.0 lbs</td>
<td>Misc. #5: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #6: 0.0 lbs</td>
<td>Misc. #6: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #7: 0.0 lbs</td>
<td>Misc. #7: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #8: 0.0 lbs</td>
<td>Misc. #8: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>RTU #9: 0.0 lbs</td>
<td>Misc. #9: 0.0 lbs</td>
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</tr>
<tr>
<td>RTU #10: 0.0 lbs</td>
<td>Misc. #10: 0.0 lbs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.00 lbs</td>
</tr>
</tbody>
</table>

SEISMIC BUILDING WEIGHT BY LEVEL

Note: Level 1 = Finished Floor Elevation. Finished Floor not applied to Seismic or Wind Calculations.
Note: Roof and parapet load applied to top level. Floor loads are applied to all floors. If a-typical floor loading is required, go to Irregular Shaped Building Tab.
Note: Partition loads, if applicable, are applied at Floor / Roof column.

<table>
<thead>
<tr>
<th>Level</th>
<th>Floor / Roof</th>
<th>Exterior Wall</th>
<th>Interior Wall</th>
<th>Roof Equip.</th>
<th>Landscaping</th>
<th>Storage</th>
<th>Snow</th>
<th>FOR ENERCALC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Level 3</td>
<td>0.0 lbs</td>
<td>1253.8 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>Level 2</td>
<td>47190.0 lbs</td>
<td>61433.8 lbs</td>
<td>12000.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>26812.5 lbs</td>
<td>0.0 lbs</td>
<td></td>
</tr>
<tr>
<td>Level 1</td>
<td>47190.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>0.0 lbs</td>
<td>26812.5 lbs</td>
<td>0.0 lbs</td>
<td></td>
</tr>
</tbody>
</table>

Level 3: 12.25 ft 1.254 kips
Level 2: 12.00 ft 147.436 kips
Level 1: 0.00 ft 74.003 kips
SEISMIC FORCES (ASCE 7-10)

Tedds calculation version 3.1.00

Site parameters

Site class D

Mapped acceleration parameters (Section 11.4.1)

at short period $S_S = 0.157$
at 1 sec period $S_1 = 0.054$

Site coefficient at short period (Table 11.4-1) $F_a = 1.600$
at 1 sec period (Table 11.4-2) $F_v = 2.400$

Spectral response acceleration parameters

at short period (Eq. 11.4-1) $S_{MS} = F_a \times S_S = 0.251$
at 1 sec period (Eq. 11.4-2) $S_{M1} = F_v \times S_1 = 0.130$

Design spectral acceleration parameters (Sect 11.4.4)

at short period (Eq. 11.4-3) $S_{DS} = 2 / 3 \times S_{MS} = 0.167$
at 1 sec period (Eq. 11.4-4) $S_{D1} = 2 / 3 \times S_{M1} = 0.086$

Seismic design category

Risk category (Table 1.5-1) II

Seismic design category based on short period response acceleration (Table 11.6-1) B

Seismic design category based on 1 sec period response acceleration (Table 11.6-2) B

Seismic design category B

Approximate fundamental period

Height above base to highest level of building $h_n = 12$ ft

From Table 12.8-2:

Structure type All other systems
Building period parameter $C_t$ $C_t = 0.02$
Building period parameter $x$ $x = 0.75$

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1 \text{sec} / (1 \text{ft})^x = 0.129$ sec
Building fundamental period (Sect 12.8.2) $T = T_a = 0.129$ sec
Long-period transition period $T_L = 4$ sec

Seismic response coefficient

Seismic force-resisting system (Table 12.2-1) A. Bearing_Wall_Systems
9. Ordinary reinforced masonry shear walls

Response modification factor (Table 12.2-1) $R = 2$
Seismic importance factor (Table 1.5-2) $I_e = 1.000$

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-3) $C_s,_{calc} = S_{DS} / (R / I_e) = 0.0837$
Maximum (Eq 12.8-3) $C_s,_{max} = S_{D1} / ((T / 1 \text{sec}) \times (R / I_e)) = 0.3350$
Minimum (Eq 12.8-5) $C_s,_{min} = \max(0.044 \times S_{DS} \times I_e,0.01) = 0.0100$
Seismic response coefficient \( C_s = 0.0837 \)

**Seismic base shear (Sect 12.8.1)**

Effective seismic weight of the structure \( W = 148.0 \text{ kips} \)

Seismic response coefficient \( C_s = 0.0837 \)

Seismic base shear (Eq 12.8-1) \( V = C_s \times W = 12.4 \text{ kips} \)

**Vertical distribution of seismic forces (Sect 12.8.3)**

Vertical distribution factor (Eq 12.8-12) \( C_{vx} = w_x \times h_x^k / \sum (w_i \times h_i^k) \)

Lateral force induced at level i (Eq 12.8-11) \( F_x = C_{vx} \times V \)

**Vertical force distribution table**

<table>
<thead>
<tr>
<th>Level</th>
<th>Height from base to Level i (ft), ( h_x )</th>
<th>Portion of effective seismic weight assigned to Level i (kips), ( w_x )</th>
<th>Distribution exponent related to building period, ( k )</th>
<th>Vertical distribution factor, ( C_{vx} )</th>
<th>Lateral force induced at Level i (kips), ( F_x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.0</td>
<td>148.0</td>
<td>1.00</td>
<td>1.000</td>
<td>12.4</td>
</tr>
</tbody>
</table>
BUILDING PLAN VIEW - 2ND FLOOR

Lateral Force Induced at Level (per TEDDS) = **12.4 kips**

![Building Plan View Diagram]

BUILDING PLAN VIEW - 2ND FLOOR

![Building Plan View Diagram]
CHECK DIAPHRAGM SHEARS

What Load Combinations were used for design numbers above? ASD

Use above values for diaphragm design

<table>
<thead>
<tr>
<th>FLOOR/VAULT ROOF DIAPHRAGMS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck Type: Vulcraft 22 ga. 2VLI Deck</td>
</tr>
<tr>
<td>Support Fasteners: Hilti X-ENP19 PAF</td>
</tr>
<tr>
<td>Sidelap Fasteners: Button Punch</td>
</tr>
<tr>
<td>Sidelap Fastener Spacing: 18&quot; o.c.</td>
</tr>
<tr>
<td>Deck Span: 7.00 ft</td>
</tr>
<tr>
<td>Fastening Pattern: 36/4</td>
</tr>
<tr>
<td>Allowable Shear: 1941 plf</td>
</tr>
</tbody>
</table>

22 ga 2VLI-36 Grade 50 Composite Deck-Slab
4.5 in. Total Slab Depth, Fc = 4000 psi, 150 pcf NWC

Hilti X-ENP19 PAF Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
Button Punch Sidelap Connections

<table>
<thead>
<tr>
<th>ASD Allowable Diaphragm Shear Strength Sn/f, plf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sidelap Connection</td>
</tr>
<tr>
<td>Spacing (in.)</td>
</tr>
<tr>
<td>18</td>
</tr>
<tr>
<td>36</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Diaphragm Shear Stiffness, G, kip/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sidelap Connection</td>
</tr>
<tr>
<td>Spacing (in.)</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>18</td>
</tr>
<tr>
<td>24</td>
</tr>
<tr>
<td>36</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Average Connection Spacing to Supports at all Parallel and Perpendicular Chords &amp; Collectors (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sidelap Connection</td>
</tr>
<tr>
<td>Spacing (in.)</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>6</td>
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<tr>
<td>8</td>
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<tr>
<td>12</td>
</tr>
<tr>
<td>18</td>
</tr>
<tr>
<td>24</td>
</tr>
<tr>
<td>36</td>
</tr>
</tbody>
</table>

Tables generated using calculator V2.0.1 based on AISI S310-16. Date: 12/23/2019
### SIDE LAP FASTENER SUBSTITUTION

Diaphragm Shear Capacity w/ New Sidetap Fasteners = 2053 plf

Required Diaphragm Shear Capacity = 238.46 plf

<table>
<thead>
<tr>
<th>ASD LOAD COMBINATIONS</th>
<th>DIAPHRAGM TYPE WORKS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>YES</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ASD SEISMIC REACTION FORCES</th>
<th>ASD SEISMIC MAX SHEAR AT DIAPHRAGM</th>
<th>ASD LOAD COMBINATIONS SEISMIC MAX SHEAR AT DIAPHRAGM</th>
<th>DIAPHRAGM TYPE WORKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>North-South</td>
<td>East-West</td>
<td>North-South East-West</td>
<td>YES</td>
</tr>
<tr>
<td>6.200 kips</td>
<td>6.200 kips</td>
<td>187.88 plf 238.46 plf</td>
<td></td>
</tr>
<tr>
<td>0.000 kips</td>
<td>0.000 kips</td>
<td>0.00 plf 0.00 plf</td>
<td></td>
</tr>
</tbody>
</table>
WIND CALCS
(AT GARAGE AREA ONLY)
WIND LOADING

In accordance with ASCE7-10

Using the directional design method

Tedds calculation version 2.1.05

Building data
Type of roof: Flat
Length of building: \( b = 33.00 \) ft
Width of building: \( d = 81.00 \) ft
Height to eaves: \( H = 16.17 \) ft
Height of parapet: \( h_p = 4.50 \) ft
Mean height: \( h = 16.17 \) ft

General wind load requirements
Basic wind speed: \( V = 115.0 \) mph
Risk category: II
Velocity pressure exponent coef (Table 26.6-1): \( K_d = 0.85 \)
Exposure category (cl.26.10): Enclosed buildings
Enclosure classification (cl.26.10): Enclosed buildings
Internal pressure coef +ve (Table 26.11-1): \( GC_{pl,p} = 0.18 \)
Internal pressure coef –ve (Table 26.11-1): \( GC_{pl,n} = -0.18 \)
Gust effect factor: \( G_f = 0.85 \)
Minimum design wind loading (cl.27.4.7): \( p_{min,r} = 8 \) lb/ft²

Topography
Topography factor not significant: \( K_{zt} = 1.0 \)
Velocity pressure equation:
\[
q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 \text{psf/mph}^2
\]
Velocity pressures table

<table>
<thead>
<tr>
<th>z (ft)</th>
<th>K_z (Table 27.3-1)</th>
<th>q_z (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.00</td>
<td>0.85</td>
<td>24.46</td>
</tr>
<tr>
<td>16.17</td>
<td>0.86</td>
<td>24.80</td>
</tr>
<tr>
<td>16.17</td>
<td>0.86</td>
<td>24.80</td>
</tr>
<tr>
<td>20.67</td>
<td>0.91</td>
<td>26.05</td>
</tr>
<tr>
<td>16.17</td>
<td>0.86</td>
<td>24.80</td>
</tr>
<tr>
<td>20.67</td>
<td>0.91</td>
<td>26.05</td>
</tr>
<tr>
<td>16.17</td>
<td>0.86</td>
<td>24.80</td>
</tr>
</tbody>
</table>

Peak velocity pressure for internal pressure
Peak velocity pressure – internal (as roof press.)  

\[ q_i = 24.80 \text{ psf} \]

Parapet pressures and forces

Velocity pressure at top of parapet  

\[ q_p = 26.05 \text{ psf} \]
Combining net pressure coefficient, leeward  

\[ GC_{pl} = -1.0 \]
Combining net parapet pressure, leeward  

\[ p_{pl} = q_p \times GC_{pl} = 26.05 \text{ psf} \]
Combining net pressure coefficient, windward  

\[ GC_{pwl} = 1.5 \]
Combining net parapet pressure, windward  

\[ p_{pw} = q_p \times GC_{pwl} = 39.08 \text{ psf} \]

Wind direction 0 deg:
Leeward parapet force  

\[ F_{w,pl_0} = p_{pl} \times h_p \times b = -3.9 \text{ kips} \]
Windward parapet force  

\[ F_{w,pw_0} = p_{pw} \times h_p \times b = 5.8 \text{ kips} \]

Wind direction 90 deg:
Leeward parapet force  

\[ F_{w,pl_90} = p_{pl} \times h_p \times d = -9.5 \text{ kips} \]
Windward parapet force  

\[ F_{w,pw_90} = p_{pw} \times h_p \times d = 14.2 \text{ kips} \]

Pressures and forces

Net pressure  

\[ p = q \times G_f \times C_{pe} - q_i \times GC_{pi} \]
Net force  

\[ F_w = p \times A_{ref} \]

Roof load case 1 - Wind 0, GC_{pi} 0.18, + c_{pe}

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient c_{pe}</th>
<th>Peak velocity pressure q_p (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area A_{ref} (ft^2)</th>
<th>Net force F_w (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>266.75</td>
<td>2.20</td>
</tr>
<tr>
<td>B (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>266.75</td>
<td>2.20</td>
</tr>
<tr>
<td>C (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>533.50</td>
<td>4.41</td>
</tr>
<tr>
<td>D (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>1606.00</td>
<td>13.26</td>
</tr>
</tbody>
</table>

Total vertical net force  

\[ F_{w,v} = -22.07 \text{ kips} \]
Total horizontal net force  

\[ F_{w,h} = 0.00 \text{ kips} \]

Walls load case 1 - Wind 0, GC_{pi} 0.18, + c_{pe}

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient c_{pe}</th>
<th>Peak velocity pressure q_p (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area A_{ref} (ft^2)</th>
<th>Net force F_w (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_1</td>
<td>15.00</td>
<td>0.80</td>
<td>24.46</td>
<td>12.17</td>
<td>495.00</td>
<td>6.02</td>
</tr>
<tr>
<td>A_2</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>38.50</td>
<td>0.48</td>
</tr>
</tbody>
</table>
### Project Details

**Project Name:** Crime Lab Additon  
**Weld County - Crime Lab Addition**

**Job Ref.** RSA02.30  
**Section** WIND LOADING - DIRECTIONAL METHOD

**Calc. by** KEJ  
**Date** 1/6/2020  
**Chk'd by** RRH  
**App'd by** RRH  
**Date** 1/10/2020

### Loadings

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient cpe</th>
<th>Peak velocity pressure qpe (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area Aref (ft²)</th>
<th>Net force Fw (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A3</td>
<td>20.67</td>
<td>0.80</td>
<td>26.05</td>
<td>13.25</td>
<td>148.50</td>
<td>1.97</td>
</tr>
<tr>
<td>A4</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>-148.50</td>
<td>-1.84</td>
</tr>
<tr>
<td>B</td>
<td>16.17</td>
<td>-0.28</td>
<td>24.80</td>
<td>-10.31</td>
<td>533.50</td>
<td>-5.50</td>
</tr>
<tr>
<td>C</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>1309.50</td>
<td>-25.17</td>
</tr>
<tr>
<td>D</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>1309.50</td>
<td>-25.17</td>
</tr>
</tbody>
</table>

### Overall Loading

- **Projected vertical plan area of wall:**  
  \[ A_{vert,w_0} = b \times (H + h_p) = 682.00 \text{ ft}^2 \]
- **Projected vertical area of roof:**  
  \[ A_{vert,r_0} = 0.00 \text{ ft}^2 \]
- **Minimum overall horizontal loading:**  
  \[ F_{w,total_min} = p_{min,w} \times A_{vert,w_0} + p_{min,r} \times A_{vert,r_0} = 10.91 \text{ kips} \]
- **Leeward net force:**  
  \[ F_l = F_{w,wB} + F_{w,wpl_0} = -9.4 \text{ kips} \]
- **Windward net force:**  
  \[ F_w = F_{w,A_1} + F_{w,A_2} + F_{w,A_3} + F_{w,A_4} + F_{w,wpw_0} = 12.4 \text{ kips} \]
- **Overall horizontal loading:**  
  \[ F_{w,total} = \max(F_w - F_l, F_w,h) = 21.8 \text{ kips} \]

### Roof load case 2 - Wind 0, GC\( \pi \) 0.18, - cpe

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient cpe</th>
<th>Peak velocity pressure qpe (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area Aref (ft²)</th>
<th>Net force Fw (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (-ve)</td>
<td>16.17</td>
<td>-0.90</td>
<td>24.80</td>
<td>-23.43</td>
<td>266.75</td>
<td>-6.25</td>
</tr>
<tr>
<td>B (-ve)</td>
<td>16.17</td>
<td>-0.90</td>
<td>24.80</td>
<td>-23.43</td>
<td>266.75</td>
<td>-6.25</td>
</tr>
<tr>
<td>C (-ve)</td>
<td>16.17</td>
<td>-0.50</td>
<td>24.80</td>
<td>-15.00</td>
<td>533.50</td>
<td>-8.00</td>
</tr>
<tr>
<td>D (-ve)</td>
<td>16.17</td>
<td>-0.30</td>
<td>24.80</td>
<td>-10.79</td>
<td>1606.00</td>
<td>-17.32</td>
</tr>
</tbody>
</table>

- **Total vertical net force:**  
  \[ F_{w,v} = -37.83 \text{ kips} \]
- **Total horizontal net force:**  
  \[ F_{w,h} = 0.00 \text{ kips} \]

### Walls load case 2 - Wind 0, GC\( \pi \) 0.18, - cpe

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient cpe</th>
<th>Peak velocity pressure qpe (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area Aref (ft²)</th>
<th>Net force Fw (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>15.00</td>
<td>0.80</td>
<td>24.46</td>
<td>12.17</td>
<td>495.00</td>
<td>6.02</td>
</tr>
<tr>
<td>A2</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>38.50</td>
<td>0.48</td>
</tr>
<tr>
<td>A3</td>
<td>20.67</td>
<td>0.80</td>
<td>26.05</td>
<td>13.25</td>
<td>148.50</td>
<td>1.97</td>
</tr>
<tr>
<td>A4</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>-148.50</td>
<td>-1.84</td>
</tr>
<tr>
<td>B</td>
<td>16.17</td>
<td>-0.28</td>
<td>24.80</td>
<td>-10.31</td>
<td>533.50</td>
<td>-5.50</td>
</tr>
<tr>
<td>C</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>1309.50</td>
<td>-25.17</td>
</tr>
<tr>
<td>D</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>1309.50</td>
<td>-25.17</td>
</tr>
</tbody>
</table>

### Overall Loading

- **Projected vertical plan area of wall:**  
  \[ A_{vert,w_0} = b \times (H + h_p) = 682.00 \text{ ft}^2 \]
- **Projected vertical area of roof:**  
  \[ A_{vert,r_0} = 0.00 \text{ ft}^2 \]
- **Minimum overall horizontal loading:**  
  \[ F_{w,total_min} = p_{min,w} \times A_{vert,w_0} + p_{min,r} \times A_{vert,r_0} = 10.91 \text{ kips} \]
- **Leeward net force:**  
  \[ F_l = F_{w,wB} + F_{w,wpl_0} = -9.4 \text{ kips} \]
Windward net force
Overall horizontal loading

\[ F_w = F_{wA_1} + F_{wA_2} + F_{wA_3} + F_{wA_4} + F_{wpw_0} = 12.4 \text{ kips} \]
\[ F_{w,\text{total}} = \max(F_w - F_l + F_{w,h}, F_{w,\text{total min}}) = 21.8 \text{ kips} \]

### Roof load case 3 - Wind 90, GC\(_{pi}\) 0.18, + \(c_{pe}\)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient (c_{pe})</th>
<th>Peak velocity (c_p) (psf)</th>
<th>Net pressure (p) (psf)</th>
<th>Area (A_{\text{ref}}) (ft(^2))</th>
<th>Net force (F_w) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>654.75</td>
<td>-5.41</td>
</tr>
<tr>
<td>B (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>654.75</td>
<td>-5.41</td>
</tr>
<tr>
<td>C (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>1309.50</td>
<td>-10.81</td>
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<tr>
<td>D (+ve)</td>
<td>16.17</td>
<td>-0.18</td>
<td>24.80</td>
<td>-8.26</td>
<td>54.00</td>
<td>-0.45</td>
</tr>
</tbody>
</table>

Total vertical net force
\[ F_{w,v} = -22.07 \text{ kips} \]
Total horizontal net force
\[ F_{w,h} = 0.00 \text{ kips} \]

### Walls load case 3 - Wind 90, GC\(_{pi}\) 0.18, + \(c_{pe}\)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient (c_{pe})</th>
<th>Peak velocity (c_p) (psf)</th>
<th>Net pressure (p) (psf)</th>
<th>Area (A_{\text{ref}}) (ft(^2))</th>
<th>Net force (F_w) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A(_1)</td>
<td>15.00</td>
<td>0.80</td>
<td>24.46</td>
<td>12.17</td>
<td>1215.00</td>
<td>14.79</td>
</tr>
<tr>
<td>A(_2)</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>94.50</td>
<td>1.17</td>
</tr>
<tr>
<td>A(_3)</td>
<td>20.67</td>
<td>0.80</td>
<td>26.05</td>
<td>13.25</td>
<td>364.50</td>
<td>4.83</td>
</tr>
<tr>
<td>A(_4)</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>-364.50</td>
<td>-4.52</td>
</tr>
<tr>
<td>B</td>
<td>16.17</td>
<td>-0.50</td>
<td>24.80</td>
<td>-15.00</td>
<td>1309.50</td>
<td>-19.65</td>
</tr>
<tr>
<td>C</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>533.50</td>
<td>-10.25</td>
</tr>
<tr>
<td>D</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>533.50</td>
<td>-10.25</td>
</tr>
</tbody>
</table>

Overall loading

Projected vertical plan area of wall
\[ A_{\text{vert, w}} = d \times (H + h_p) = 1674.00 \text{ ft}^2 \]

Projected vertical area of roof
\[ A_{\text{vert, r}} = 0.00 \text{ ft}^2 \]

Minimum overall horizontal loading
\[ F_{w,\text{total min}} = p_{\text{min, w}} \times A_{\text{vert, w}} + p_{\text{min, r}} \times A_{\text{vert, r}} = 26.78 \text{ kips} \]

Leeward net force
\[ F_l = F_{wA_1} + F_{wpw_90} = -29.1 \text{ kips} \]

Windward net force
\[ F_w = F_{wA_1} + F_{wA_2} + F_{wA_3} + F_{wA_4} + F_{wpw_90} = 30.5 \text{ kips} \]

Overall horizontal loading
\[ F_{w,\text{total}} = \max(F_w - F_l + F_{w,h}, F_{w,\text{total min}}) = 59.7 \text{ kips} \]

### Roof load case 4 - Wind 90, GC\(_{pi}\) 0.18, - \(c_{pe}\)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient (c_{pe})</th>
<th>Peak velocity (c_p) (psf)</th>
<th>Net pressure (p) (psf)</th>
<th>Area (A_{\text{ref}}) (ft(^2))</th>
<th>Net force (F_w) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (-ve)</td>
<td>16.17</td>
<td>-0.90</td>
<td>24.80</td>
<td>-23.43</td>
<td>654.75</td>
<td>-15.34</td>
</tr>
<tr>
<td>B (-ve)</td>
<td>16.17</td>
<td>-0.90</td>
<td>24.80</td>
<td>-23.43</td>
<td>654.75</td>
<td>-15.34</td>
</tr>
<tr>
<td>C (-ve)</td>
<td>16.17</td>
<td>-0.50</td>
<td>24.80</td>
<td>-15.00</td>
<td>1309.50</td>
<td>-19.65</td>
</tr>
<tr>
<td>D (-ve)</td>
<td>16.17</td>
<td>-0.30</td>
<td>24.80</td>
<td>-10.79</td>
<td>54.00</td>
<td>-0.58</td>
</tr>
</tbody>
</table>

Total vertical net force
\[ F_{w,v} = -50.91 \text{ kips} \]
Total horizontal net force
\[ F_{w,h} = 0.00 \text{ kips} \]

---

**PROJECT NAME:** CRIME LAB  |  **PROJECT NUMBER:** RSA02.30  |  **CHECKED BY:** KEJ  |  **1/24/2020**  |  **Page 33**
### Walls load case 4 - Wind 90, GC, 0.18, - cpe

<table>
<thead>
<tr>
<th>Zone</th>
<th>Ref. height (ft)</th>
<th>Ext pressure coefficient cpe</th>
<th>Peak velocity pressure p_b (psf)</th>
<th>Net pressure p (psf)</th>
<th>Area A_{ref} (ft²)</th>
<th>Net force F_w (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A_1</td>
<td>15.00</td>
<td>0.80</td>
<td>24.46</td>
<td>12.17</td>
<td>1215.00</td>
<td>14.79</td>
</tr>
<tr>
<td>A_2</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>94.50</td>
<td>1.17</td>
</tr>
<tr>
<td>A_3</td>
<td>20.67</td>
<td>0.80</td>
<td>26.05</td>
<td>13.25</td>
<td>364.50</td>
<td>4.83</td>
</tr>
<tr>
<td>A_4</td>
<td>16.17</td>
<td>0.80</td>
<td>24.80</td>
<td>12.40</td>
<td>-364.50</td>
<td>-4.52</td>
</tr>
<tr>
<td>B</td>
<td>16.17</td>
<td>-0.50</td>
<td>24.80</td>
<td>-15.00</td>
<td>1309.50</td>
<td>-19.65</td>
</tr>
<tr>
<td>C</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>533.50</td>
<td>-10.25</td>
</tr>
<tr>
<td>D</td>
<td>16.17</td>
<td>-0.70</td>
<td>24.80</td>
<td>-19.22</td>
<td>533.50</td>
<td>-10.25</td>
</tr>
</tbody>
</table>

**Overall loading**

- **Projected vertical plan area of wall**: \( A_{vert, w, 90} = d \times (H + h_p) = 1674.00 \text{ ft}^2 \)
- **Projected vertical area of roof**: \( A_{vert, r, 90} = 0.00 \text{ ft}^2 \)
- **Minimum overall horizontal loading**: \( F_{w, \text{total min}} = p_{\text{min, w}} \times A_{vert, w, 90} + p_{\text{min, r}} \times A_{vert, r, 90} = 26.78 \text{ kips} \)
- **Leeward net force**: \( F_I = F_{w, wB} + F_{w, wpl, 90} = -29.1 \text{ kips} \)
- **Windward net force**: \( F_w = F_{w, wA_1} + F_{w, wA_2} + F_{w, wA_3} + F_{w, wA_4} + F_{w, wpw, 90} = 30.5 \text{ kips} \)
- **Overall horizontal loading**: \( F_{w, \text{total}} = \max(F_w - F_I, F_{w, \text{total min}}) = 59.7 \text{ kips} \)
PROJECT NAME: CRIME LAB
GREELEY, CO

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Wind - 90°

Windward face

Leeward face

Side face
### Project
- **Weld County - Crime Lab Addition**

### Job Ref.
- RSA02.30

### Section
- **WIND LOADING - DIRECTIONAL METHOD**

<table>
<thead>
<tr>
<th>Calc. by</th>
<th>Date</th>
<th>CHK'd by</th>
<th>Date</th>
<th>App'd by</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>KEJ</td>
<td>1/6/2020</td>
<td>RRH</td>
<td>1/10/2020</td>
<td>RRH</td>
<td>1/10/2020</td>
</tr>
</tbody>
</table>

### Sheet no./rev.
- 7

### Diagram

**Windward face**

- **A**
  - **A1**
  - **A2**
  - **A3**
  - **A4**

**Leeward face**

- **B**

**Side face**

- **C**

---

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**GREELEY, CO**

**PROJECT NUMBER:** RSA02.30  
**CHECKED BY:** KEJ  
**1/24/2020**

**Page 36**
HORIZONTAL DIAPHRAGM DESIGN - WIND (GARAGE ONLY)

ROOF LEVEL DIAPHRAGM DESIGN

### ROOF @ 0°

<table>
<thead>
<tr>
<th>LOAD CASE 1</th>
<th>LOAD CASE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₁ 12.17 psf</td>
<td>12.17 psf</td>
</tr>
<tr>
<td>A₂ 12.40 psf</td>
<td>12.40 psf</td>
</tr>
<tr>
<td>A₃ 13.25 psf</td>
<td>13.25 psf</td>
</tr>
<tr>
<td>B 10.31 psf</td>
<td>10.31 psf</td>
</tr>
</tbody>
</table>

Max Wall Loading = 22.71 psf

### ROOF @ 90°

<table>
<thead>
<tr>
<th>LOAD CASE 3</th>
<th>LOAD CASE 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₁ 12.17 psf</td>
<td>12.17 psf</td>
</tr>
<tr>
<td>A₂ 12.40 psf</td>
<td>12.40 psf</td>
</tr>
<tr>
<td>A₃ 13.25 psf</td>
<td>13.25 psf</td>
</tr>
<tr>
<td>A₄ 15.00 psf</td>
<td>15.00 psf</td>
</tr>
</tbody>
</table>

Max Wall Loading = 27.40 psf

### Typical Wall Total Loading

- **LOAD CASE 1**: 1.0*Windward + 1.0*Leeward
- **LOAD CASE 2**: 1.0*Windward + 1.0*Leeward
- **LOAD CASE 3**: 1.5*Windward + 1.0*Leeward
- **LOAD CASE 4**: 1.5*Windward + 1.0*Leeward

### Parapet Total Loading

- **LOAD CASE 1**: 1.0*Windward + 1.0*Leeward
- **LOAD CASE 2**: 1.0*Windward + 1.0*Leeward
- **LOAD CASE 3**: 1.5*Windward + 1.0*Leeward
- **LOAD CASE 4**: 1.5*Windward + 1.0*Leeward

### Line Load (0°)

- ASD Line Load (0°) = 0.6(Line Load)
- Line Load (0°) = 378.42 plf

### Line Load (90°)

- ASD Line Load (90°) = 0.6(Line Load)
- Line Load (90°) = 227.05 plf

---

**PROJECT NAME: CRIME LAB**
**GREELEY, CO**
**PROJECT NUMBER: RSA02.30**
**CHECKED BY: KEJ**
**1/24/2020**
CHECK DIAPHRAGM SHEARS

What Load Combinations were used for design numbers above? ASD

Use above values for diaphragm design

ROOF #2 DIAPHRAGM

Deck Type: Vulcraft 1.5B Deck 22 Gauge
Deck Span: 5.08 ft
Support Fasteners: Hilti X-ENP19
Deck Type: 1.58/36
Fastener Pattern 36/5
Sidetap Fasteners: #10 TEK screws
Allowable Shear: 410.0 plf
# of Sidetap Fasteners: 5

Vulcraft Deck Diaphragm Shear & Stiffness

In accordance with 2015 IBC Section 2310 ANSI/SDI-1.0, NC1-0 & C-011
Calculation Generated on 4/22/2019 Using Calculator V1.1

Input Design Criteria

Unit System
Imperial

Design Method
ASD

Deck Option
Roof Deck

Deck Grade
Grade 33

Number of Spaces
3

Table Generated Based on
15.00

Sidelap Fasteners

Allowable Shear:
# of Sidelap Fasteners:
5

Start Number of Sidelap Attachments per Span:
1

Note: Support Steel Thickness => 1/4 in.

The vertical gravity load capacity of the deck based on bending stress and applicable deflection criteria must be checked separately.

Use selected support attachment type for both perpendicular attachment and parallel attachment of steel deck.

VULCRAFT GROUP

Input Design Criteria

Unit System
Imperial

Design Method
ASD

Deck Option
Roof Deck

Deck Grade
Grade 33

Number of Spaces
3

Table Generated Based on
15.00

Sidelap Fasteners

Allowable Shear:
# of Sidelap Fasteners:
5

Start Number of Sidelap Attachments per Span:
1

Note: Support Steel Thickness => 1/4 in.

The vertical gravity load capacity of the deck based on bending stress and applicable deflection criteria must be checked separately.

Use selected support attachment type for both perpendicular attachment and parallel attachment of steel deck.

VULCRAFT GROUP

Diaphragm Shear Stiffness, G" (kip/ln)

Spant (ft - in) 4-0" 5-0" 6-0" 7-0" 8-0"
1 163 222 281 330 380 430 480 530 580 630 680 730 780 830 880 930 980 1030 1080 1130 1180 1230 1280 1330 1380 1430 1480 1530 1580 1630 1680 1730 1780 1830 1880 1930 1980 2030 2080 2130 2180 2230 2280 2330 2380 2430 2480 2530 2580 2630 2680 2730 2780 2830 2880 2930 2980 3030 3080 3130 3180 3230 3280 3330 3380 3430 3480 3530 3580 3630 3680 3730 3780 3830 3880 3930 3980 4030 4080 4130 4180 4230 4280 4330 4380 4430 4480 4530 4580 4630 4680 4730 4780 4830 4880 4930 4980 5030 5080 5130 5180 5230 5280 5330 5380 5430 5480 5530 5580 5630 5680 5730 5780 5830 5880 5930 5980 6030 6080 6130 6180 6230 6280 6330 6380 6430 6480 6530 6580 6630 6680 6730 6780 6830 6880 6930 6980 7030 7080

The vertical gravity load capacity of the deck based on bending stress and applicable deflection criteria must be checked separately.

Use selected support attachment type for both perpendicular attachment and parallel attachment of steel deck.

VULCRAFT GROUP

Diaphragm Shear Stiffness, G" (kip/ln)

Spant (ft - in) 4-0" 5-0" 6-0" 7-0" 8-0"
1 163 222 281 330 380 430 480 530 580 630 680 730 780 830 880 930 980 1030 1080 1130 1180 1230 1280 1330 1380 1430 1480 1530 1580 1630 1680 1730 1780 1830 1880 1930 1980 2030 2080 2130 2180 2230 2280 2330 2380 2430 2480 2530 2580 2630 2680 2730 2780 2830 2880 2930 2980 3030 3080 3130 3180 3230 3280 3330 3380 3430 3480 3530 3580 3630 3680 3730 3780 3830 3880 3930 3980 4030 4080 4130 4180 4230 4280 4330 4380 4430 4480 4530 4580 4630 4680 4730 4780 4830 4880 4930 4980 5030 5080 5130 5180 5230 5280 5330 5380 5430 5480 5530 5580 5630 5680 5730 5780 5830 5880 5930 5980 6030 6080 6130 6180 6230 6280 6330 6380 6430 6480 6530 6580 6630 6680 6730 6780 6830 6880 6930 6980 7030 7080

The vertical gravity load capacity of the deck based on bending stress and applicable deflection criteria must be checked separately.

Use selected support attachment type for both perpendicular attachment and parallel attachment of steel deck.

VULCRAFT GROUP
CHECK DIAPHRAGM SHEARS CONT.

<table>
<thead>
<tr>
<th>ASD WIND REACTION FORCES</th>
<th>ASD WIND MAX SHEAR AT DIAPHRAGM</th>
<th>ASD LOAD COMBINATIONS</th>
<th>DIAPHRAGM TYPE WORKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>North-South</td>
<td>East-West</td>
<td>North-South</td>
<td>East-West</td>
</tr>
<tr>
<td>3.162 kips</td>
<td>9.062 kips</td>
<td>39.53 plf</td>
<td>275.22 plf</td>
</tr>
</tbody>
</table>

CHECK CHORD FORCES

North-South Chord (LRFD Loading)

\[ M_{u,N-S} = \frac{Wl^2}{8} = 26.1 \text{ kip-ft} \]

\[ T_{u,N-S} = C_{u,N-S} = \frac{M_{u,N-S}}{p} = 0.33 \text{ kips} \]

<table>
<thead>
<tr>
<th>OPTION 1</th>
<th>OPTION 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSS2 1/2x1/3/16&quot;</td>
<td>HSS2 1/2x1/3/16&quot;</td>
</tr>
<tr>
<td>Area of Steel, ( A_s )</td>
<td>Area of Steel, ( A_s )</td>
</tr>
<tr>
<td>1.020 in^2</td>
<td>1.930 in^2</td>
</tr>
<tr>
<td>( f_y )</td>
<td>( f_y )</td>
</tr>
<tr>
<td>46 ksi</td>
<td>36 ksi</td>
</tr>
</tbody>
</table>

GOOD

\[ f_{y,\text{ALLOW}} = \phi f_y = 41.40 \text{ ksi} \]

\[ f_{y,\text{ALLOW}} = \phi f_y = 41.40 \text{ ksi} \]

GOOD

\[ f_{y,\text{ALLOW}} = \phi f_y = 32.40 \text{ ksi} \]

\[ f_{y,\text{ALLOW}} = \phi f_y = 32.40 \text{ ksi} \]

GOOD

East-West Chord (LRFD Loading)

\[ M_{u,E-W} = \frac{Wl^2}{8} = 181.6 \text{ kip-ft} \]

\[ T_{u,N-S} = C_{u,N-S} = \frac{M_{u,N-S}}{p} = 5.50 \text{ kips} \]

<table>
<thead>
<tr>
<th>OPTION 1</th>
<th>OPTION 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSS2 1/2x1/3/16&quot;</td>
<td>HSS2 1/2x1/3/16&quot;</td>
</tr>
<tr>
<td>Area of Steel, ( A_s )</td>
<td>Area of Steel, ( A_s )</td>
</tr>
<tr>
<td>1.020 in^2</td>
<td>1.930 in^2</td>
</tr>
<tr>
<td>( f_y )</td>
<td>( f_y )</td>
</tr>
<tr>
<td>46 ksi</td>
<td>36 ksi</td>
</tr>
</tbody>
</table>

GOOD

\[ f_{y,\text{ALLOW}} = \phi f_y = 41.40 \text{ ksi} \]

\[ f_{y,\text{ALLOW}} = \phi f_y = 41.40 \text{ ksi} \]

GOOD

\[ f_{y,\text{ALLOW}} = \phi f_y = 32.40 \text{ ksi} \]

\[ f_{y,\text{ALLOW}} = \phi f_y = 32.40 \text{ ksi} \]

GOOD

---

PROJECT NAME: CRIME LAB
GREELEY, CO

PROJECT NUMBER: RSA02.30

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1/24/2020 Page 39
WIND LOADING

In accordance with ASCE7-10

Using the components and cladding design method

Tedds calculation version 2.1.05

Building data
Type of roof: Flat
Length of building: b = 33.00 ft
Width of building: d = 81.00 ft
Height to eaves: H = 16.17 ft
Height of parapet: h_p = 4.50 ft
Mean height: h = 16.17 ft

General wind load requirements
Basic wind speed: V = 115.0 mph
Risk category: II
Velocity pressure exponent coef (Table 26.6-1): K_d = 0.85
Exposure category (cl 26.7.3): C
Enclosure classification (cl 26.10): Enclosed buildings
Internal pressure coef +ve (Table 26.11-1): G_{C,PL} = 0.18
Internal pressure coef -ve (Table 26.11-1): G_{C,PL,N} = -0.18
Parapet internal pressure coef +ve (Table 26.11-1): G_{C,PL,PP} = 0.55
Parapet internal pressure coef -ve (Table 26.11-1): G_{C,PL,NP} = -0.55
Gust effect factor: G_{R} = 0.85

Topography
Topography factor not significant: K_{zt} = 1.0
Velocity pressure
Velocity pressure coefficient (T.30.3-1) \( K_z = 0.86 \)
Velocity pressure
\[ q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 \text{psf/mph}^2 = 24.8 \text{ psf} \]

Velocity pressure at parapet
Velocity pressure coefficient (T.30.3-1) \( K_z = 0.91 \)
Velocity pressure
\[ q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1 \text{psf/mph}^2 = 26.1 \text{ psf} \]

Peak velocity pressure for internal pressure
Peak velocity pressure – internal (as roof press.) \( q_i = 24.80 \text{ psf} \)

Equations used in tables
Net pressure \[ p = q_h \times [G_{C_p} - G_{C_{pi}}] \]
Parapet net pressure \[ p = q_p \times [G_{C_p} - G_{C_{pi}}] \]

Components and cladding pressures - Wall (Table 30.4-1 and Figure 30.4-2A)

<table>
<thead>
<tr>
<th>Component</th>
<th>Zone</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Eff. area (ft²)</th>
<th>+GCₚ</th>
<th>-GCₚ</th>
<th>Pres (+ve) (psf)</th>
<th>Pres (-ve) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 sf</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
<td>0.90</td>
<td>-0.99</td>
<td>26.8</td>
<td>-29.0</td>
</tr>
<tr>
<td>10 sf</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
<td>0.90</td>
<td>-1.26</td>
<td>26.8</td>
<td>-35.7</td>
</tr>
<tr>
<td>20 sf</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>0.85</td>
<td>-0.94</td>
<td>25.6</td>
<td>-27.8</td>
</tr>
<tr>
<td>20 sf</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>0.85</td>
<td>-1.16</td>
<td>25.6</td>
<td>-33.3</td>
</tr>
<tr>
<td>50 sf</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>50.0</td>
<td>0.79</td>
<td>-0.88</td>
<td>24.0</td>
<td>-26.3</td>
</tr>
<tr>
<td>50 sf</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>50.0</td>
<td>0.79</td>
<td>-1.04</td>
<td>24.0</td>
<td>-30.2</td>
</tr>
<tr>
<td>100 sf</td>
<td>4</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>0.74</td>
<td>-0.83</td>
<td>22.8</td>
<td>-25.1</td>
</tr>
<tr>
<td>100 sf</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>0.74</td>
<td>-0.94</td>
<td>22.8</td>
<td>-27.8</td>
</tr>
</tbody>
</table>

Components and cladding pressures - Roof (Figure 30.4-2A)

<table>
<thead>
<tr>
<th>Component</th>
<th>Zone</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Eff. area (ft²)</th>
<th>+GCₚ</th>
<th>-GCₚ</th>
<th>Pres (+ve) (psf)</th>
<th>Pres (-ve) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 sf</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
<td>0.30</td>
<td>-1.00</td>
<td>11.9 *</td>
<td>-29.3</td>
</tr>
</tbody>
</table>
# The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction

<table>
<thead>
<tr>
<th>Component</th>
<th>Zone</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Eff. area (ft²)</th>
<th>+GCₚ</th>
<th>-GCₚ</th>
<th>Pres (+ve) (psf)</th>
<th>Pres (-ve) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 sf</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
<td>0.90</td>
<td>-1.80</td>
<td>26.8</td>
<td>-49.1</td>
</tr>
<tr>
<td>10 sf</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>10.0</td>
<td>0.90</td>
<td>-1.80</td>
<td>26.8</td>
<td>-49.1</td>
</tr>
<tr>
<td>20 sf</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>0.27</td>
<td>-0.97</td>
<td>11.2 #</td>
<td>-28.5</td>
</tr>
<tr>
<td>20 sf</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>0.85</td>
<td>-1.59</td>
<td>25.6</td>
<td>-43.9</td>
</tr>
<tr>
<td>20 sf</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>20.0</td>
<td>0.85</td>
<td>-1.59</td>
<td>25.6</td>
<td>-43.9</td>
</tr>
<tr>
<td>50 sf</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>50.0</td>
<td>0.23</td>
<td>-0.93</td>
<td>10.2 #</td>
<td>-27.5</td>
</tr>
<tr>
<td>50 sf</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>50.0</td>
<td>0.79</td>
<td>-1.31</td>
<td>24.0</td>
<td>-37.0</td>
</tr>
<tr>
<td>50 sf</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>50.0</td>
<td>0.79</td>
<td>-1.31</td>
<td>24.0</td>
<td>-37.0</td>
</tr>
<tr>
<td>100 sf</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>0.20</td>
<td>-0.90</td>
<td>9.4 #</td>
<td>-26.8</td>
</tr>
<tr>
<td>100 sf</td>
<td>2</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>0.74</td>
<td>-1.10</td>
<td>22.8</td>
<td>-31.7</td>
</tr>
<tr>
<td>100 sf</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>100.0</td>
<td>0.74</td>
<td>-1.10</td>
<td>22.8</td>
<td>-31.7</td>
</tr>
</tbody>
</table>

Note: # indicates the final net design wind pressure, including all permitted reductions, used in the design.
SHEARWALL CALCS
(AT GARAGE AREA ONLY)
LOADING IN THIS AREA HAS BEEN DETERMINED BY OTHERS. THIS SECTION OF BUILDING IS A PRE-FABRICATED METAL BUILDING AND DESIGNED BY THE METAL BUILDING MANUFACTURER.

SEISMIC = 5.3 kips

WIND = 9.1 kips

WIND = 9.1 kips

WIND = 3.2 kips

SW #1
#5 rebar at 48" o.c. max

SW #2
#5 rebar in each cell
#3 ties at 8" o.c.

SW #2
#5 rebar in each cell
#3 ties at 8" o.c.

SW #3
#5 rebar at 48" o.c. max

SW #3
#5 rebar at 48" o.c. max

SW #3
#5 rebar at 48" o.c. max

SW #3
#5 rebar at 48" o.c. max

***LOADING SHOWN IS ASD

RSA02.30 - CRIME LAB ADDITION

1/8/2019

LATERAL KEYPLAN
**Column Axial Load**

- Wind Load = 3.2 kips
- Earthquake Load = 5.3 kips
- ASD Total Shear Load = 3.7 kips

**Column Moment Load (x-direction)**

- Moment = (Roof DL + Roof SL + Snow Drift) * (0.5 ft eccentricity) = 6.84 kips-ft

**Column Load**

- ASD Total Axial Load at 2'-0" column = DL + LL = 22.3 kips

**Column Moment Load (y-direction)**

- ASD Total Shear Load = 3.7 kips
- Moment = 3.7 kips (12 ft column height) = 44.4 kips-ft

---

**Roof DL = 20 psf * (33 ft/2) * 12 ft trib = 3.96 kips**

- **Wall DL = 80 psf * (12 ft trib) * (9 ft wall height) = 864 kips**

- **Roof Live Load = 20 psf * (33 ft trib / 2) * 12 ft trib = 3.96 kips**

- **Roof Snow Load = 30 psf * (33 ft trib / 2) * 12 ft trib = 5.94 kips**

- **Roof Snow Drift = 0.5 * 66 psf snow drift * 9.5 ft trib * 12 ft trib = 3.77 kips**

- **ASD Total Axial Load at 2'-0" column = DL + LL = 22.3 kips**

- **ASD Total Shear Load = 3.7 kips**

---

**Shear and Moment Loads / 2 in TEDDS since at least (2) columns will be used to resist lateral loads**
MASONRY WALL PANEL DESIGN TO TMS 402/602-16

Using the allowable stress design method

Tedds calculation version 2.2.04

Masonry wall panel details
Masonry Shear Wall - SW1 - Reinforced single-wythe wall with a parapet, the wall is pinned at the top and at the bottom for out of plane loads
The wall is fixed at the bottom and free at the top for in plane loads

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Panel length</td>
<td>L = 20 ft</td>
</tr>
<tr>
<td>Panel height</td>
<td>h = 16.25 ft</td>
</tr>
<tr>
<td>Parapet height</td>
<td>h_p = 4.5 ft</td>
</tr>
</tbody>
</table>

Seismic properties
Seismic design category | B
Seismic importance factor (ASCE7 Table 1.5-2) | I_p = 1
Design spectral response acceleration parameter, short periods (ASCE7 11.4.4) | $S_{os} = 0.167$

Shear wall designation | Ordinary reinforced
Redundancy factor, on in-plane load | $\rho_{vi} = 1.0$
Redundancy factor, on out-of-plane load | $\rho_{vo} = 1.0$

Construction details
Wall thickness | t = 7.625 in
Masonry details

Open ended hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class S mortar

Compressive strength of unit \( f'_{cu} = 1900 \) psi

Density of masonry units \( \gamma_{block} = 115 \) lb/ft\(^3\)

Height of masonry units \( h_b = 7.625 \) in

Length of masonry units \( l_b = 15.625 \) in

Number of internal webs \( N_{web} = 1 \)

Number of end webs \( N_{end} = 1 \)

Internal web thickness \( t_{bw} = 1.125 \) in

Face shell thickness \( t_{bf} = 1.25 \) in

End web thickness \( t_{be} = 1.25 \) in

Area of block
\[
A_{block} = \left( t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf}) \right) / l_b = 39.35 \text{ in}^2/\text{ft}
\]

Area of grout
\[
A_{grout} = [0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})] / l_b = 8.87 \text{ in}^2/\text{ft}
\]

Density of grout \( \gamma_{grout} = 140 \) lb/ft\(^3\)

Self weight of wall
\[
w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout} = 40.04 \text{ psf}
\]
From TMS 602-16 Table 2 - Compressive strength of masonry
Net compressive strength of masonry \( f_m = 1900 \text{ psi} \)
Modulus of elasticity for masonry \( E_m = 900 \times f_m = 1709997 \text{ psi} \)
Shear modulus of masonry \( G_v = 0.4 \times E_m = 683999 \text{ psi} \)

From TMS 402-16 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry
Allowable flexural tensile stress normal to bed \( F_{t_{\text{norm}}} = 38 \text{ psi} \)
Allowable flexural tensile stress parallel to bed \( F_{t_{\text{para}}} = 66 \text{ psi} \)

Reinforcement details
Yield strength of reinforcement \( f_y = 60000 \text{ psi} \)
Allowable tensile stress in reinforcement \( F_s = 32000 \text{ psi} \)
Modulus of elasticity for reinforcement \( E_s = 29000000 \text{ psi} \)
Vertical reinforcement provided
\( \text{No.5 bars at 48 in centers} \)
Area of vertical reinforcement
\( A_s = \pi \times \text{Dia}^2 / (4 \times s) = 0.08 \text{ in}^2/\text{ft} \)
Yield strength of horizontal reinforcement \( f_{yv} = 70000 \text{ psi} \)
Allowable tensile stress in horizontal reinforcement \( F_{sv} = 30000 \text{ psi} \)
Horizontal reinforcement provided
\( (2) \text{ W1.7 wires at 16 in centers} \)
Area of horizontal reinforcement
\( A_v = 2 \times \pi \times \text{HDia}^2 / (4 \times s_v) = 0.03 \text{ in}^2/\text{ft} \)
Minimum area of vertical reinforc. (8.3.5.2.2)
\( A_{s_{\text{min}}} = A_v / 3 = 0.01 \text{ in}^2/\text{ft} \)
PASS - Area of vertical reinforcement provided exceeds the minimum

Seismic reinforcement requirements
Minimum vertical reinforc., seismic (Ch. 7)
\( \text{No. 4 bars at 120 in} \)
PASS - Vertical distributed reinforcement meets minimum seismic requirements
Minimum horizontal reinforc., seismic (Ch. 7)
\( (2) \text{ W1.7 wires at 16 in} \)
PASS - Horizontal distributed reinforcement meets minimum seismic requirements

Lateral out-of-plane loads
Wind load on panel \( W = 28 \text{ psf} \)
Wind load on parapet \( W_p = 35 \text{ psf} \)
Seismic load factor (ASCE7 12.11.1) \( F_p = 0.4 \times S_{DS} \times I_e = 0.067 \)
Seismic load from wall \( E_{\text{wall}} = \max(F_p,0.1) \times w_{SW} = 4 \text{ psf} \)
Additional seismic load \( E_{\text{add}} = 0 \text{ psf} \)
Seismic lateral load on panel \( E = E_{\text{wall}} + E_{\text{add}} = 4 \text{ psf} \)

Lateral in-plane loads
Wind shear load on wall \( V_W = 3200 \text{ lbs} \)
Seismic shear load on wall \( V_e = 5300 \text{ lbs} \)

Vertical loading details
Dead load at supported level \( DL = 330 \text{ lb/ft} \) at an eccentricity of 6 in
Live roof load at supported level \( LL = 330 \text{ lb/ft} \) at an eccentricity of 6 in
Snow load at supported level \( SL = 810 \text{ lb/ft} \) at an eccentricity of 6 in
Vertical seismic load factor applied to dead load \( F_{Ev} = 0.2 \times S_{DS} = 0.033 \)

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)
Load combination no.1 \( DL \) (0.190)
Load combination no.2 \( DL + LL \) (0.190)
Load combination no.3 \( DL + (LL \text{ or SL or RL}) \) (0.520)
Load combination no.4  DL + 0.75 × LL + 0.75 × (LLr or SL or RL) (0.451)
Load combination no.5  DL + 0.6 × W (0.435)
Load combination no.6  DL + 0.7 × En + 0.7 × Ev (0.226)
Load combination no.7  DL + 0.75 × LL + 0.45 × W + 0.75 × (LLr or SL or RL) (0.604)
Load combination no.8  DL + 0.75 × LL + 0.525 × Eh + 0.525 × Ev + 0.75 × SL (0.473)
Load combination no.9  0.6 × DL + 0.6 × W (0.469)
Load combination no.10  0.6 × DL + 0.7 × En - 0.7 × Ev (0.153)

Properties of masonry section
Cross-sectional area
A = [t × lb - 0.83 × (lb - Nweb × tbw - Nend × tbe) × (t - 2 × tbf)] / lb = 48.2 in²/ft

Properties for walls loaded out-of-plane:
Moment of inertia
I = t³ / 12 - 0.83 × (lb - Nweb × tbw - Nend × tbe) × (t - 2 × tbf)³ / (12 × lb) = 348.6 in⁴/ft

Section modulus
S = I / c = 91.4 in³/ft

Radius of gyration
r = √[I / A] = 2.689 in

Effective height factor
K = 1

Properties for walls loaded in-plane:
Net moment of inertia
Iₓ_net = t × L³ / 12 - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) - 2 × (lx_cell + Acell × xcell²) = 5152110 in⁴

Net section modulus
Sₓ_net = Iₓ_net / (L / 2) = 42934 in³

Consider wall at top under load combination no.7
Axial load at top of panel

\[ P = 1118 \text{ lb/ft} \]

Compressive stress due to axial load

\[ f_a = \frac{P}{A} = 23.2 \text{ psi} \]

Slenderness ratio

\[ \frac{(K \times h)}{r} = 72.522 < 99 \]

Allowable compressive stress due to axial load

\[ f_a = \frac{1}{4} \times f_m \times \left[ 1 - \left( \frac{(K \times h)}{(140 \times r)} \right)^2 \right] = 347.5 \text{ psi} \]

\[ f_a / F_a = 0.067 \]

**PASS - Allowable compressive stress exceeds compressive stress due to axial loads**

Allowable compressive force

\[ P_a = \left( 0.25 \times f_m \times (A - A_a) + 0.65 \times A_a \times F_s \times \left[ 1 - \left( \frac{(K \times h)}{(140 \times r)} \right)^2 \right] \right) \times (1 - k_{bal}) \]

\[ P_a = 17897 \text{ lb/ft} \]

\[ P / P_a = 0.062 \]

**PASS - Allowable compressive force exceeds axial load**

Bending moment at top of panel

\[ M = 7539 \text{ lb}_\text{in}/\text{ft} \]

Depth of reinforcement

\[ d = 3.812 \text{ in} \]

Modular ratio

\[ n = E_s / E_m = 16.959 \]

Allowable compressive stress due to flexural load

\[ F_s = (0.45) \times f_m \times 855 \text{ psi} \]

Balance point

\[ k_{bal} = n / (F_s / F_s + n) = 0.312 \]

Tensile strain in reinforcement

\[ \varepsilon_s = F_s / E_s = 0.001103 \]

Compressive strain in masonry

\[ \varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) = 0.000500 \]

Compressive stress at balance point

\[ f_{bal} = \varepsilon_m \times E_m = 854.999 \text{ psi} \]

Tension at balance point

\[ T_{bal} = A_s \times F_s = 2454 \text{ lb/ft} \]

Compression at balance point

\[ C_{bal} = k_{bal} \times d \times f_{bal} / 2 = 6099 \text{ lb/ft} \]

Axial load at balance point

\[ P_{bal} = C_{bal} - T_{bal} = 3644 \text{ lb/ft} \]
Moment at balance point

\[ M_{\text{bal}} = T_{\text{bal}} \times (d - t / 2) + C_{\text{bal}} \times (t / 2 - k_{\text{bal}} \times d / 3) = 20835 \, \text{lb_in/ft} \]

Maximum moment from interaction diagram

\[ M_c = 12486 \, \text{lb_in/ft} \]

\[ M / M_c = 0.604 \]

**PASS - Combination of applied axial load and flexure is acceptable**

Consider wall at bottom under load combination no.5
Shear force: $V = 113.3$ lb/ft
Net shear area: $A_{nv} = d \times b / ((N_{web} + 1) \times s_{grout}) = 7.4$ in$^2$/ft
Shear stress: $f_v = V / A_{nv} = 15.2$ psi
Compressive force: $N_v = 1.00 \times P_{DL,b,out} / 1 \text{ ft} = 1160.9$ lb/ft
Moment: $M = 0$ lb_in/ft
Allowable masonry shear stress: $F_{vm} = 0.5 \times [4 - 1.75 \times \min((M / (V \times d)),1.0)] \times \sqrt{(f_m \times 1 \text{ psi}) + 0.25 \times N_v / A}$ $= 93.2$ psi
Allowable shear stress: $F_v = \min(F_{vm}, 3 \times \sqrt{(f_m \times 1 \text{ psi})}) = 93.2$ psi
Shear utilization: $f_v / F_v = 0.163$

PASS - Allowable shear stress exceeds applied shear stress
Masonry Column Design

In accordance with TMS 402/602-16

Tedds calculation version 1.1.01

Column geometry

Column width \( b = 15.625 \) in
Column depth \( d = 15.625 \) in
Effective height of column \( h = 14 \) ft
Gross column area \( A_g = b \times d = 244 \) in\(^2\)
Net column area \( A_n = A_d = 244 \) in\(^2\)
Moment of inertia (x axis) \( I_{gx} = b \times d^3 / 12 = 4967 \) in\(^4\)
Radius of gyration (x axis) \( r_x = \sqrt{(I_{gx} / A_g)} = 4.51 \) in
Moment of inertia (y axis) \( I_{gy} = d \times b^3 / 12 = 4967 \) in\(^4\)
Radius of gyration (y axis) \( r_y = \sqrt{(I_{gy} / A_g)} = 4.51 \) in
Geometry condition \( 5.3.1.1 \)
\( h \leq 99 \times \min(r_x, r_y) \)

PASS - Column meets dimensional requirements

Masonry unit details

Thickness of masonry unit \( t = 7.625 \) in
Height of masonry unit \( h_b = 7.625 \) in
Masonry unit type Hollow

Masonry details

Pattern bond Running
Masonry type Concrete
Density \( \gamma = 115 \) lb/ft\(^3\)
Mortar PCL
Mortar type S
Grout type Coarse
Unit strength \( f'_{cu} = 1900 \) psi
Net area compr. strength (Table 2 TMS 602-16) \( f'_m = 1900 \) psi
Modulus of elasticity (4.2.2) \( E_m = 900 \times f'_m = 1710000 \) psi
Reinforcement details
Reinforcement yield strength \( f_y = 60000 \) psi
Modulus of elasticity of reinforcement \( E_s = 29000000 \) psi
Vertical reinforcement 4 No.5 bars
Minimum clear cover to reinforcement \( c_c = 1.5 \) in
Area of vertical reinforcement \( A_{st,\text{vert}} = 1.24 \) in\(^2\)
Min. area of vert. reinforcement (5.3.1.3) \( A_{st,\text{min}} = 0.0025 \times A_n = 0.61 \) in\(^2\)
Max. area of vert. reinforcement (5.3.1.3) \( A_{st,\text{max}} = 0.04 \times A_n = 9.77 \) in\(^2\)
Horizontal tie reinforcement No.3 @ 8 in
Maximum allowable tie spacing (5.3.1.4) \( s_{\text{tie, max}} = \min(16 \times d_{b,\text{vert}}, 48 \times d_{b,\text{tie}}, \min(b,d)) = 10 \) in
PASS - Tie spacing is less than maximum allowable spacing

Allowable stresses
Allowable mas. compr. stress (2.3.3.2.2) \( F_b = 0.45 \times f_m = 855 \) psi
Allowable reinf. compr. stress (8.3.3) \( F_{sc} = 32000 \) psi
Allowable reinf. tensile stress (8.3.3) \( F_{st} = 32000 \) psi

Design loads
Design axial load \( P = 22.3 \) kips
Design shear load \( V = 1.9 \) kips
Design moment load \( M_{x,\text{act}} = 14.8 \) kip\_ft
Ratio of moment due to eccentricity \( f_{pe} = 1 \)
Minimum moments (8.3.4.3)
Minimum eccentricity, along x axis \( e_{x,\text{min}} = 0.1 \times b = 1.56 \) in
Minimum moment about y axis \( M_{y,\text{min}} = P \times e_{x,\text{min}} = 2.9 \) kip\_ft
Design moment load, about y axis \( M_y = M_{y,\text{min}} = 2.9 \) kip\_ft
Minimum eccentricity, along y axis \( e_{y,\text{min}} = 0.1 \times d = 1.56 \) in
Minimum moment about x axis \( M_{x,\text{min}} = P \times e_{y,\text{min}} = 2.9 \) kip\_ft
Ratio of moments due to eccentricity \( f_{pe} = 1 \)
Moment due to eccentricity, about x axis \( M_{x,\text{e}} = f_{pe} \times M_{x,\text{act}} = 14.8 \) kip\_ft
Moment due to transverse loads, about x axis \( M_{x,\text{tr}} = (1 - f_{pe}) \times M_{x,\text{act}} = 0 \) kip\_ft
New design moment load, about x axis \( M_x = \max(M_{x,\text{min}}, M_{x,\text{e}}) + M_{x,\text{tr}} = 14.8 \) kip\_ft

Check pure axial capacity
Compressive stress from axial load \( f_a = P / A_g = 91 \) psi
Slenderness ratio \( h / r = 37.2 \) not greater than 99
Allowable compressive stress (Eqn. 8-13) \( F_a = 0.25 \times f_m \times [1 - (h / (140 \times r))^2] = 441 \) psi
\( f_a / F_a = 0.207 \)
PASS - Allowable axial stress exceeds compressive stress from axial load, see 8.3.4.2.2
Allowable compressive force (Eqn. 8-18) \( P_a = (0.25 \times f_m \times A_n + 0.65 \times A_{st} \times F_{sc}) \times [1 - (h / (140 \times r))^2] = 131.7 \) kips
\( P / P_a = 0.169 \)
PASS - Allowable compressive force exceeds axial load

Determine moment design strength about the x axis under design axial load
Design moment load \( M_x = 14.8 \) kip\_ft
Distance to neutral axis ratio, balance pt. \( k_b = 0.31 \)

From strain compatibility analysis:
- Depth to extreme tension layer \( d_t = 11.81 \) in
- Distance to neutral axis ratio, by iteration \( k_x = 0.42 \)
- Strain in extreme compression fiber \( k_y > k_b \), therefore
  \[ \varepsilon_m = \frac{F_b}{E_m} = 0.0005 \]

Nominal compression in masonry
\[ P_{x\text{mas}} = k_x \times d_t \times b \times \varepsilon_m \times E_m / 2 = 32.9 \text{ kips} \]

Nominal axial force in reinf.
\[ P_{xs} = -10.6 \text{ kips} \]

Nominal axial strength
\[ P_{nx} = P_{x\text{mas}} + P_{xs} = 22.3 \text{ kips} \]

Nominal moment from masonry
\[ M_{x\text{mas}} = P_{x\text{mas}} \times (d / 2 - k_x \times d_t / 3) = 17 \text{ kip ft} \]

Nominal moment from reinf.
\[ M_{xs} = 5 \text{ kip ft} \]

Nominal moment strength
\[ M_{nx} = M_{x\text{mas}} + M_{xs} = 22 \text{ kip ft} \]

Bending utilization
\[ M_y / M_{nx} = 0.680 \]

**PASS - Design moment strength exceeds design moment load**

**Determine moment design strength about the y axis under design axial load**

Design moment load
\[ M_y = 2.9 \text{ kip ft} \]

From strain compatibility analysis:
- Depth to extreme tension layer \( d_t = 11.81 \) in
- Distance to neutral axis ratio, by iteration \( k_y = 0.42 \)
- Strain in extreme compression fiber \( k_y > k_b \), therefore
  \[ \varepsilon_m = \frac{F_b}{E_m} = 0.0005 \]

Nominal compression in masonry
\[ P_{y\text{mas}} = k_y \times d_t \times d \times \varepsilon_m \times E_m / 2 = 32.9 \text{ kips} \]

Nominal axial force in reinf.
\[ P_{ys} = -10.6 \text{ kips} \]

Nominal axial strength
\[ P_{ny} = P_{y\text{mas}} + P_{ys} = 22.3 \text{ kips} \]

Nominal moment from masonry
\[ M_{y\text{mas}} = P_{y\text{mas}} \times (b / 2 - k_y \times d_t / 3) = 17 \text{ kip ft} \]

Nominal moment from reinf.
\[ M_{ys} = 5 \text{ kip ft} \]

Nominal moment strength
\[ M_{ny} = M_{y\text{mas}} + M_{ys} = 22 \text{ kip ft} \]

Bending utilization
\[ M_y / M_{ny} = 0.133 \]

**PASS - Design moment strength exceeds design moment load**

**Check shear capacity**

Design shear load
\[ V = 1.9 \text{ kips} \]

Net shear area
\[ A_{nv} = b \times d = 244.14 \text{ in}^2 \]

Design shear stress (Eqn. 8-21)
\[ f_v = V / A_{nv} = 7.6 \text{ psi} \]

Conservatively set \( M/Vd \) to 1.0

Maximum allowable shear stress (Eqn. 8-24)
\[ F_{v,\text{max}} = 2 \times \sqrt{(f'_{m} \times 1 \text{ psi})} = 87.2 \text{ psi} \]

Shear utilization
\[ f_v / F_{v,\text{max}} = 0.087 \]

**PASS - Maximum allowable shear stress exceeds design shear stress**

Masonry shear strength (Eqn. 8-26)
\[ F_{vm} = 1.125 \times \sqrt{(f'_{m} \times 1 \text{ psi})} + 0.25 \times P / A_n \]
\[ F_{vm} = 71.9 \text{ psi} \]

Masonry shear utilization
\[ f_v / F_{vm} = 0.105 \]

Shear reinforcement is not required

**PASS – Maximum allowable shear stress exceeds design shear stress**
INTERACTION DIAGRAM - X AXIS BENDING ONLY
MASONRY WALL PANEL DESIGN TO TMS 402/602-16

Using the allowable stress design method

Tedds calculation version 2.2.04

Masonry wall panel details
Masonry Shear Wall - SW1 - Reinforced single-wythe wall with a parapet, the wall is pinned at the top and at the bottom for out of plane loads.
The wall is fixed at the bottom and free at the top for in plane loads.

Panel length \( L = 15 \text{ ft} \)
Panel height \( h = 16.25 \text{ ft} \)
Parapet height \( h_p = 4.5 \text{ ft} \)

Seismic properties
Seismic design category \( B \)
Seismic importance factor (ASCE7 Table 1.5-2) \( I_e = 1 \)
Design spectral response acceleration parameter, short periods (ASCE7 11.4.4) \( S_{os} = 0.167 \)
Shear wall designation Ordinary reinforced
Redundancy factor, on in-plane load \( \rho_{ve} = 1.0 \)
Redundancy factor, on out-of-plane load \( \rho_{E} = 1.0 \)

Construction details
Wall thickness \( t = 7.625 \text{ in} \)
**Masonry details**

Open ended hollow concrete units grouted at 48 in on center in running bond fully bedded with PCL class S mortar

Compressive strength of unit \( f'_{cu} = 1900 \) psi

Density of masonry units \( \gamma_{block} = 115 \) lb/ft\(^3\)

Height of masonry units \( h_b = 7.625 \) in

Length of masonry units \( l_b = 15.625 \) in

Number of internal webs \( N_{web} = 1 \)

Number of end webs \( N_{end} = 1 \)

Internal web thickness \( t_{bw} = 1.125 \) in

Face shell thickness \( t_{bf} = 1.25 \) in

End web thickness \( t_{be} = 1.25 \) in

Area of block \( A_{block} = \frac{[t \times l_b - (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})]}{l_b} = 39.35 \) in\(^2\)/ft

Area of grout \( A_{grout} = \frac{[0.17 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})]}{l_b} = 8.87 \) in\(^2\)/ft

Density of grout \( \gamma_{grout} = 140 \) lb/ft\(^3\)

Self weight of wall \( w_{SW} = A_{block} \times \gamma_{block} + A_{grout} \times \gamma_{grout} = 40.04 \) psf
From TMS 602-16 Table 2 - Compressive strength of masonry
Net compressive strength of masonry \( f'_m = 1900 \text{ psi} \)
Modulus of elasticity for masonry \( E_m = 900 \times f'_m = 1709997 \text{ psi} \)
Shear modulus of masonry \( G_v = 0.4 \times E_m = 683999 \text{ psi} \)

From TMS 402 -16 Table 8.2.4.2 - Allowable flexural tensile stresses for clay and concrete masonry
Allowable flexural tensile stress normal to bed \( F_{t\text{,norm}} = \frac{38}{0.08} \text{ psi} \)
Allowable flexural tensile stress parallel to bed \( F_{t\text{,para}} = \frac{66}{0.03} \text{ psi} \)

Reinforcement details
Yield strength of reinforcement \( f_y = 60000 \text{ psi} \)
Allowable tensile stress in reinforcement \( F_s = 32000 \text{ psi} \)
Modulus of elasticity for reinforcement \( E_s = 29000000 \text{ psi} \)
Vertical reinforcement provided \( \text{No.5 bars at 48 in centers} \)
Area of vertical reinforcement \( A_s = \frac{\pi \times \text{Dia}^2}{4} \times \frac{1}{s} = 0.08 \text{ in}^2/\text{ft} \)
Yield strength of horizontal reinforcement \( f_{yh} = 70000 \text{ psi} \)
Allowable tensile stress in horizontal reinforcement \( F_{sv} = 30000 \text{ psi} \)
Horizontal reinforcement provided \( (2) \text{W1.7 wires at 16 in centers} \)
Area of horizontal reinforcement \( A_v = \frac{2 \times \pi \times \text{HDia}^2}{4} \times \frac{1}{s_v} = 0.03 \text{ in}^2/\text{ft} \)
Minimum area of vertical reinf. (8.3.5.2.2) \( A_{s,\text{min}} = \frac{A_v}{3} = 0.01 \text{ in}^2/\text{ft} \)

Seismic reinforcement requirements
Minimum vertical reinf., seismic (Ch. 7) \( \text{No. 4 bars at 120 in} \)
Minimum horizontal reinf., seismic (Ch. 7) \( (2) \text{W1.7 wires at 16 in} \)

Lateral out-of-plane loads
Wind load on panel \( W = 28 \text{ psf} \)
Wind load on parapet \( W_p = 35 \text{ psf} \)
Seismic load factor (ASCE7 12.11.1) \( F_p = 0.4 \times S_{DS} \times \lambda_e = 0.067 \)
Seismic load from wall \( E_{wall} = \max(F_p,0.1) \times w_{SW} = 4 \text{ psf} \)
Additional seismic load \( E_{add} = 0 \text{ psf} \)
Seismic lateral load on panel \( E = E_{\text{wall}} + E_{\text{add}} = 4 \text{ psf} \)

Lateral in-plane loads
Wind shear load on wall \( V_W = 9100 \text{ lbs} \)
Seismic shear load on wall \( V_s = 5300 \text{ lbs} \)

Vertical loading details
Dead load at supported level \( DL = 60 \text{ lb/ft} \text{ at an eccentricity of 6 in} \)
Live roof load at supported level \( LL = 60 \text{ lb/ft} \text{ at an eccentricity of 6 in} \)
Snow load at supported level \( SL = 288 \text{ lb/ft} \text{ at an eccentricity of 6 in} \)
Vertical seismic load factor applied to dead load \( F_{Ev} = 0.2 \times S_{DS} = 0.033 \)

From ASCE 7-10 cl.2.4.1 - Combining nominal loads using allowable stress design (Utilization)
Load combination no.1 \( DL \text{ (0.053)} \)
Load combination no.2 \( DL + LL \text{ (0.053)} \)
Load combination no.3 \( DL + (LL \text{ or SL or RL) } \text{ (0.199)} \)
<table>
<thead>
<tr>
<th>Load combination no.4</th>
<th>DL + 0.75 × LL + 0.75 × (LLr or SL or RL) (0.162)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load combination no.5</td>
<td>DL + 0.6 × W (0.492)</td>
</tr>
<tr>
<td>Load combination no.6</td>
<td>DL + 0.7 × Ex + 0.7 × Ev (0.190)</td>
</tr>
<tr>
<td>Load combination no.7</td>
<td>DL + 0.75 × LL + 0.45 × W + 0.75 × (LLr or SL or RL) (0.349)</td>
</tr>
<tr>
<td>Load combination no.8</td>
<td>DL + 0.75 × LL + 0.525 × Ex + 0.525 × Ev + 0.75 × SL (0.187)</td>
</tr>
<tr>
<td>Load combination no.9</td>
<td>0.6 × DL + 0.6 × W (0.539)</td>
</tr>
<tr>
<td>Load combination no.10</td>
<td>0.6 × DL + 0.7 × Ex - 0.7 × Ev (0.216)</td>
</tr>
</tbody>
</table>

**Properties of masonry section**

Cross-sectional area

\[ A = \left[ t \times l_b - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \right] \times \left( t - 2 \times t_{bf} \right) / l_b = 48.2 \text{ in}^2/\text{ft} \]

Properties for walls loaded out-of-plane:

Moment of inertia

\[ I = \frac{l^3}{12} - 0.83 \times (l_b - N_{web} \times t_{bw} - N_{end} \times t_{be}) \times (t - 2 \times t_{bf})^3 / (12 \times l_b) = 348.6 \text{ in}^4/\text{ft} \]

Section modulus

\[ S = \frac{I}{c} = 91.4 \text{ in}^3/\text{ft} \]

Radius of gyration

\[ r = \sqrt{\frac{I}{A}} = 2.689 \text{ in} \]

Effective height factor

\[ K = 1 \]

Properties for walls loaded in-plane:

Net moment of inertia

\[ I_{x_{net}} = \frac{l \times L^3}{12} - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell1}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell2}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell3}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell4}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell5}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell6}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell7}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell8}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell9}^2) - 2 \times (I_{x_{cell}} + A_{cell} \times x_{cell10}^2) = 2186761 \text{ in}^4 \]

Net section modulus

\[ S_{x_{net}} = \frac{I_{x_{net}}}{(L / 2)} = 24297 \text{ in}^3 \]

Consider wall at maximum moment location under load combination no.9
Axial force, out of plane - Combination No.9 - lbs/ft
Shear force, out of plane - Combination No.9 - lbs/ft
Moment force, out of plane - Combination No.9 - lb_in

Maximum moment location 7.28 ft
Axial load at max moment loc. of panel P = 360 lb/ft
Compressive stress due to axial load $f_a = P / A = 7.5$ psi
Slenderness ratio $(K \times h) / r = 72.522 < 99$
Allowable compressive stress due to axial load $F_a = (1 / 4) \times f_m \times [1 - ((K \times h) / (140 \times r))^2] = 347.5$ psi
$f_a / F_a = 0.021$

**PASS - Allowable compressive stress exceeds compressive stress due to axial loads**

Allowable compressive force $P_a = (0.25 \times f_m \times (A - A_s) + 0.65 \times A_s \times F_s) \times [1 - ((K \times h) / (140 \times r))^2] =$
17897 lb/ft
$P / P_a = 0.020$

**PASS - Allowable compressive force exceeds axial load**

Bending moment at max. moment loc. of panel $M = 5343 \text{ lb}_\text{in/ft}$
Depth of reinforcement $d = 3.812 \text{ in}$
Modular ratio $n = E_s / E_m = 16.959$
Allowable compressive stress due to flexural load $F_s = (0.45) \times f_m = 855$ psi
Balance point $k_{bal} = n / (F_s / F_a + n) = 0.312$
Tensile strain in reinforcement $\varepsilon_s = F_s / E_s = 0.001103$
Compressive strain in masonry $\varepsilon_m = \varepsilon_s \times k_{bal} / (1 - k_{bal}) = 0.000500$
Compressive stress at balance point $f_{bal} = \varepsilon_m \times E_m = 854.999$ psi
Tension at balance point $T_{bal} = A_s \times F_s = 2454 \text{ lb/ft}$
Compression at balance point $C_{bal} = k_{bal} \times d \times f_{bal} / 2 = 6099 \text{ lb/ft}$
Axial load at balance point  \( P_{bal} = C_{bal} - T_{bal} = 3644 \, \text{lb/ft} \)

Moment at balance point  \( M_{bal} = T_{bal} \times (d - t / 2) + C_{bal} \times (t / 2 - k_{bal} \times d / 3) = 20835 \, \text{lb_in/ft} \)

Maximum moment from interaction diagram  \( M_{c} = 9909 \, \text{lb_in/ft} \)

\[
\frac{M}{M_{c}} = 0.539
\]

PASS - Combination of applied axial load and flexure is acceptable

Consider wall at bottom under load combination no.9
Shear force: $V = 5460$ lbs

Depth to reinforcement: $d_n = L - t_b / 4 = 14.7$ ft

Net shear area: $A_{nv} = 2 \times d_n \times t_b + d_n / s_{group} \times (L - 2 \times t_b) = 587.1$ in$^2$

Shear stress: $f_v = V / A_{nv} = 9.3$ psi

Compressive force: $N_v = 0.6 \times P_{DL,b,in} = 8018.1$ lbs

Moment: $M_v = 88725$ lb$\cdot$ft

Allowable masonry shear stress: $F_{vm} = 0.5 \times [(4 - 1.75 \times \min(M_v / (V \times d_n), 1.0)) \times \sqrt{(f_m \times 1 psi) + 0.25 \times N_v / (A \times L)} = 51.8$ psi

Allowable shear stress: $F_v = \min(F_{vm}, 2 \times \sqrt{(f_m \times 1 psi)}) \times 0.75 = 38.9$ psi

Shear utilization: $f_v / F_v = 0.239$

Shear reinforcement is not required for strength.

allowable steel shear stress: $F_{vs} = 0.5 \times (A_{nv} \times F_{sv} \times d_n / (A_{nv} / d_n \times 1 \text{ ft})) = 142$ psi

allowable shear stress: $F_v = \min(F_{vm}, F_{vs}, 2 \times \sqrt{(f_m \times 1 psi)}) \times 0.75 = 65.4$ psi

Shear utilization: $f_v / F_v = 0.142$

PASS - Allowable shear stress exceeds applied shear stress.

Coefficient of friction: $\mu = 0.7$

Allowable shear friction stress: $F_f = \max(0.65 \times (0.6 \times A_{nv} \times d_n \times F_s + N_v) / (A_{nv}), 0 \text{ psi}) = 32.8$ psi

Shear friction utilization: $f_v / F_f = 0.284$

PASS - Allowable shear friction stress exceeds applied shear stress.

Axial load: $P = 8018$ lb

By iteration, tension at balance point: $T_{bal} = 17780$ lb
Compression at balance point
Axial load at balance point
By iteration, moment at balance point
Maximum moment from interaction diagram

\[ C_{\text{bal}} = 178897 \text{ lb} \]
\[ P_{\text{bal}} = C_{\text{bal}} - T_{\text{bal}} = 161117 \text{ lb} \]
\[ M_{\text{bal}} = 13877161 \text{ lb_in} \]
\[ M_c = 3371105 \text{ lb_in} \]
\[ M_v / M_c = 0.316 \]

**PASS - Combination of applied axial load and flexure is acceptable**

Allowable stress interaction diagram
VAULT
GRAVITY CALCULATIONS
CRIME LAB - ELEVATED VAULT ROOF

STEEL DECK: Vault Roof

<table>
<thead>
<tr>
<th>Load Acting on Joist</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prior DL psf</td>
</tr>
<tr>
<td>Prior LL psf</td>
</tr>
<tr>
<td>Flat Roof LL psf</td>
</tr>
<tr>
<td>Total Load on Deck</td>
</tr>
<tr>
<td>Uniform Dead Load</td>
</tr>
<tr>
<td>Uniform Live Load</td>
</tr>
<tr>
<td>Drift Pressure (psf)</td>
</tr>
</tbody>
</table>

Project Name: CRIME LAB
Greeley, CO

Project Number: RSA02.30
Checked By: KEJ
1/24/2020

2VLI COMPOSITE DECK

SECTION PROPERTIES

<table>
<thead>
<tr>
<th>Deck</th>
<th>Span</th>
<th>Design Thickness</th>
<th>Deck Weight</th>
<th>Section Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>22</td>
<td>7.0</td>
<td>0.250</td>
<td>0.0</td>
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</tr>
<tr>
<td>20</td>
<td>6.0</td>
<td>0.225</td>
<td>0.314</td>
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<tr>
<td>18</td>
<td>7.0</td>
<td>0.375</td>
<td>0.244</td>
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</tr>
<tr>
<td>19</td>
<td>5.75</td>
<td>0.375</td>
<td>0.459</td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>5.0</td>
<td>0.375</td>
<td>0.662</td>
<td></td>
</tr>
</tbody>
</table>

Vault Roof

VULCRAFT

2VLI NORMAL WEIGHT CONCRETE (145 PCF)

<table>
<thead>
<tr>
<th>TOTAL</th>
<th>1 SPAN</th>
<th>2 SPAN</th>
<th>3 SPAN</th>
<th>4 SPAN</th>
<th>5 SPAN</th>
<th>6 SPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td>125</td>
<td>170</td>
<td>215</td>
<td>260</td>
<td>305</td>
<td>350</td>
<td>405</td>
</tr>
<tr>
<td>120</td>
<td>170</td>
<td>215</td>
<td>260</td>
<td>305</td>
<td>350</td>
<td>405</td>
</tr>
<tr>
<td>115</td>
<td>170</td>
<td>215</td>
<td>260</td>
<td>305</td>
<td>350</td>
<td>405</td>
</tr>
<tr>
<td>110</td>
<td>170</td>
<td>215</td>
<td>260</td>
<td>305</td>
<td>350</td>
<td>405</td>
</tr>
<tr>
<td>105</td>
<td>170</td>
<td>215</td>
<td>260</td>
<td>305</td>
<td>350</td>
<td>405</td>
</tr>
</tbody>
</table>

Notes:
1. Section properties are determined based on allowable stresses on page 43 with the Vulcraf Composite Span Calculator available at www.vulcraf.com/.
2. The following conditions are required to meet the maximum unshored span shown.
3. Allowable bearing length of 12" for 25 and 30 psf. Allowable bearing varies from 12" to 14" for 22 and 25 psf, depending on soil thickness.
4. All loadings are for 2VLI bearings with unshored loads in excess of 165 psf. Loadings larger than 165 psf should be subject to an upper live load limit of 250 psf.

Kevin E. Johnson
11/9/2020

Page 66
STEEL BEAM ANALYSIS & DESIGN (AISC360-16)

In accordance with AISC360-16 using the LRFD method

Tedds calculation version 3.0.14
Support conditions

Support A
- Vertically restrained
- Rotationally free

Support B
- Vertically restrained
- Rotationally free

Applied loading

Beam loads
- Dead self weight of beam $\times$ 1
- DL Composite Deck (45 psf @ 7 ft trib) - Dead full UDL 0.315 kips/ft
- LL (125 psf storage @ 7 ft trib) - Live full UDL 0.875 kips/ft
<table>
<thead>
<tr>
<th>Load combinations</th>
<th>Support A</th>
<th>Support B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load combination 1 - 1.4D</td>
<td>Dead × 1.40</td>
<td>Dead × 1.40</td>
</tr>
<tr>
<td>Load combination 2 - 1.2D+1.6L+ 0.5Lr</td>
<td>Dead × 1.20</td>
<td>Dead × 1.20</td>
</tr>
<tr>
<td>Load combination 3 - 1.2D+1.6L+0.5S</td>
<td>Dead × 1.20</td>
<td>Dead × 1.20</td>
</tr>
<tr>
<td>Load combination 4 - 1.2D+1.6Lr+1.0L</td>
<td>Dead × 1.20</td>
<td>Dead × 1.20</td>
</tr>
<tr>
<td>Load combination 5 - 1.2D+1.6Lr+0.5W</td>
<td>Dead × 1.20</td>
<td>Dead × 1.20</td>
</tr>
<tr>
<td>Load combination 6 - 1.2D+1.6S+1.0L</td>
<td>Dead × 1.20</td>
<td>Dead × 1.20</td>
</tr>
<tr>
<td>Load combination</td>
<td>Support A</td>
<td>Support B</td>
</tr>
<tr>
<td>------------------</td>
<td>-----------</td>
<td>-----------</td>
</tr>
<tr>
<td>7 - 1.2D+1.6S+0.5W</td>
<td>Dead × 1.2</td>
<td>Snow × 1.6</td>
</tr>
<tr>
<td></td>
<td>Snow × 1.6</td>
<td>Live × 1.0</td>
</tr>
<tr>
<td></td>
<td>Wind × 0.5</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Snow × 1.6</td>
<td>Wind × 0.5</td>
</tr>
<tr>
<td></td>
<td>Wind × 0.5</td>
<td>Snow × 1.6</td>
</tr>
<tr>
<td>8 - 1.2D+1.0W+1.0L+0.5Lr</td>
<td>Dead × 1.2</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Live × 1.0</td>
<td>Live × 1.0</td>
</tr>
<tr>
<td></td>
<td>Roof live × 0.5</td>
<td>Roof live × 0.5</td>
</tr>
<tr>
<td></td>
<td>Wind × 1.0</td>
<td>Wind × 1.0</td>
</tr>
<tr>
<td></td>
<td>Dead × 1.2</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Live × 1.0</td>
<td>Live × 1.0</td>
</tr>
<tr>
<td></td>
<td>Roof live × 0.5</td>
<td>Roof live × 0.5</td>
</tr>
<tr>
<td></td>
<td>Wind × 1.0</td>
<td>Wind × 1.0</td>
</tr>
<tr>
<td>9 - 1.2D+1.0W+1.0L+0.5S</td>
<td>Dead × 1.2</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Live × 1.0</td>
<td>Live × 1.0</td>
</tr>
<tr>
<td></td>
<td>Snow × 0.5</td>
<td>Snow × 0.5</td>
</tr>
<tr>
<td></td>
<td>Wind × 1.0</td>
<td>Wind × 1.0</td>
</tr>
<tr>
<td></td>
<td>Dead × 1.2</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Live × 1.0</td>
<td>Live × 1.0</td>
</tr>
<tr>
<td></td>
<td>Snow × 0.5</td>
<td>Snow × 0.5</td>
</tr>
<tr>
<td></td>
<td>Wind × 1.0</td>
<td>Wind × 1.0</td>
</tr>
<tr>
<td>10 - 1.2D+1.0E+1.0L+0.2S</td>
<td>Dead × 1.2</td>
<td>Dead × 1.2</td>
</tr>
<tr>
<td></td>
<td>Live × 1.0</td>
<td>Live × 1.0</td>
</tr>
</tbody>
</table>

**Load Combinations:**
- Load combination 1:
- Load combination 2:
- Load combination 3:
- Load combination 4:
- Load combination 5:
- Load combination 6:
- Load combination 7:
- Load combination 8:
- Load combination 9:
- Load combination 10:
Snow × 0.20  
Seismic × 1.00  
Dead × 1.20  
Live × 1.00  
Snow × 0.20  
Seismic × 1.00

Support B

Dead × 1.20  
Live × 1.00  
Snow × 0.20  
Seismic × 1.00

Load combination 11 - 0.9D+1.0W

Support A

Dead × 0.90  
Wind × 1.00  
Dead × 0.90  
Wind × 1.00

Support B

Dead × 0.90  
Wind × 1.00

Load combination 12 - 0.9D+1.0E

Support A

Dead × 0.90  
Seismic × 1.00  
Dead × 0.90  
Seismic × 1.00

Support B

Dead × 0.90  
Seismic × 1.00

Analysis results

Maximum moment

\[ M_{\text{max}} = 153.8 \text{ kips}_\text{ft} \]
\[ M_{\text{min}} = 0 \text{ kips}_\text{ft} \]

Maximum shear

\[ V_{\text{max}} = 23.7 \text{ kips} \]
\[ V_{\text{min}} = -23.7 \text{ kips} \]

Deflection

\[ \delta_{\text{max}} = 0.9 \text{ in} \]
\[ \delta_{\text{min}} = 0 \text{ in} \]

Maximum reaction at support A

\[ R_{A_{\text{max}}} = 23.7 \text{ kips} \]
\[ R_{A_{\text{min}}} = 4.1 \text{ kips} \]

Unfactored dead load reaction at support A

\[ R_{A_{\text{Dead}}} = 4.6 \text{ kips} \]

Unfactored live load reaction at support A

\[ R_{A_{\text{Live}}} = 11.4 \text{ kips} \]

Maximum reaction at support B

\[ R_{B_{\text{max}}} = 23.7 \text{ kips} \]
\[ R_{B_{\text{min}}} = 4.1 \text{ kips} \]

Unfactored dead load reaction at support B

\[ R_{B_{\text{Dead}}} = 4.6 \text{ kips} \]

Unfactored live load reaction at support B

\[ R_{B_{\text{Live}}} = 11.4 \text{ kips} \]

Section details

Section type

\[ \text{W 18x35 (AISC 15th Edn (v15.0))} \]

ASTM steel designation

\[ \text{A992} \]

Steel yield stress

\[ F_y = 50 \text{ ksi} \]

Steel tensile stress

\[ F_u = 65 \text{ ksi} \]

Modulus of elasticity

\[ E = 29000 \text{ ksi} \]
Resistance factors

- Resistance factor for tensile yielding: $\phi_y = 0.90$
- Resistance factor for tensile rupture: $\phi_r = 0.90$
- Resistance factor for compression: $\phi_c = 0.90$
- Resistance factor for flexure: $\phi_b = 0.90$

**Classification of sections for local buckling - Section B4.1**

**Classification of flanges in flexure - Table B4.1b (case 10)**

- Width to thickness ratio: $b_f / (2 \times t) = 7.06$
- Limiting ratio for compact section: $\lambda_{eff} = 0.38 \times \sqrt{\frac{E}{F_y}} = 9.15$
- Limiting ratio for non-compact section: $\lambda_{eff} = 1.0 \times \sqrt{\frac{E}{F_y}} = 24.08$ (Compact)

**Classification of web in flexure - Table B4.1b (case 15)**

- Width to thickness ratio: $(d - 2 \times k) / t_w = 53.49$
- Limiting ratio for compact section: $\lambda_{web} = 3.76 \times \sqrt{\frac{E}{F_y}} = 90.55$
- Limiting ratio for non-compact section: $\lambda_{web} = 5.70 \times \sqrt{\frac{E}{F_y}} = 137.27$ (Compact)

**Design of members for shear - Chapter G**

- Required shear strength: $V_r = \max(\abs{V_{\text{max}}}, \abs{V_{\text{min}}}) = 23.661$ kips
- Web area: $A_w = d \times t_w = 5.31$ in$^2$
- Web plate buckling coefficient: $k_v = 5.34$
- Web shear coefficient - eq G2-3: $C_vt = 1$
- Nominal shear strength – eq G6-1: $V_n = 0.6 \times F_y \times A_w \times C_vt = 159.300$ kips
- Resistance factor for shear: $\phi_v = 1.00$
- Design shear strength: $V_c = \phi_v \times V_n = 159.300$ kips

**Section is compact in flexure**

PASS - Design shear strength exceeds required shear strength
Design of members for flexure in the major axis - Chapter F

Required flexural strength

\[ M_r = \max(\text{abs}(M_{s1_{\text{max}}}), \text{abs}(M_{s1_{\text{min}}})) = 153.795 \text{ kips}_\text{ft} \]

Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1

\[ M_{nyld} = M_p = F_y \times Z_x = 277.083 \text{ kips}_\text{ft} \]

Nominal flexural strength

\[ M_n = M_{nyld} = 277.083 \text{ kips}_\text{ft} \]

Design flexural strength

\[ M_d = \phi_b \times M_n = 249.375 \text{ kips}_\text{ft} \]

PASS - Design flexural strength exceeds required flexural strength

Design of members for vertical deflection

Consider deflection due to dead and live loads

Limiting deflection

\[ \delta_{\text{lim}} = \min(1 \text{ in}, L_{s1} / 240) = 1 \text{ in} \]

Maximum deflection span 1

\[ \delta = \max(\text{abs}(\delta_{\text{max}}), \text{abs}(\delta_{\text{min}})) = 0.852 \text{ in} \]

PASS - Maximum deflection does not exceed deflection limit
MASONRY COLUMN DESIGN

In accordance with TMS 402/602-16

Tedds calculation version 1.1.01

Plan of column

Column geometry

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column width</td>
<td>b = 8 in</td>
</tr>
<tr>
<td>Column depth</td>
<td>d = 23.625 in</td>
</tr>
<tr>
<td>Effective height of column</td>
<td>h = 12 ft</td>
</tr>
<tr>
<td>Gross column area</td>
<td>A₀ = b × d = 189 in²</td>
</tr>
<tr>
<td>Net column area</td>
<td>Aₙ = A₀ = 189 in²</td>
</tr>
<tr>
<td>Moment of inertia (x axis)</td>
<td>Iₓ = b × d² / 12 = 8791 in⁴</td>
</tr>
<tr>
<td>Radius of gyration (x axis)</td>
<td>rₓ = √(Iₓ / A₀) = 6.82 in</td>
</tr>
<tr>
<td>Moment of inertia (y axis)</td>
<td>Iᵧ = d × b² / 12 = 1008 in⁴</td>
</tr>
<tr>
<td>Radius of gyration (y axis)</td>
<td>rᵧ = √(Iᵧ / A₀) = 2.31 in</td>
</tr>
<tr>
<td>Geometry condition (5.3.1.1)</td>
<td>h &lt;= 99 × min(rₓ, rᵧ)</td>
</tr>
</tbody>
</table>

PASS - Column meets dimensional requirements

Masonry unit details

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of masonry unit</td>
<td>t = 7.625 in</td>
</tr>
<tr>
<td>Height of masonry unit</td>
<td>hₘ = 7.625 in</td>
</tr>
<tr>
<td>Masonry unit type</td>
<td>Hollow</td>
</tr>
</tbody>
</table>

Masonry details

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pattern bond</td>
<td>Running</td>
</tr>
<tr>
<td>Masonry type</td>
<td>Concrete</td>
</tr>
<tr>
<td>Density</td>
<td>γ = 115 lb/ft³</td>
</tr>
<tr>
<td>Mortar</td>
<td>PCL</td>
</tr>
<tr>
<td>Mortar type</td>
<td>S</td>
</tr>
<tr>
<td>Grout type</td>
<td>Coarse</td>
</tr>
<tr>
<td>Unit strength</td>
<td>f'ₘₜ = 1900 psi</td>
</tr>
<tr>
<td>Net area compr. strength (Table 2 TMS 602-16)</td>
<td>fₘ = 1900 psi</td>
</tr>
<tr>
<td>Modulus of elasticity (4.2.2)</td>
<td>Eₘ = 900 × fₘ = 1710000 psi</td>
</tr>
</tbody>
</table>
Maximum usable strain (9.3.2(c))
\[ \varepsilon_{mu} = 0.0025 \]

**Reinforcement details**
- Reinforcement yield strength: \( f_y = 60000 \) psi
- Modulus of elasticity of reinforcement: \( E_s = 29000000 \) psi
- Vertical reinforcement: 6 No.5 bars
- Minimum clear cover to reinforcement: \( c = 1.75 \) in
- Area of vertical reinforcement: \( A_{st} = N_{vert} \times A_{b_vert} = 1.86 \) in\(^2\)
- Minimum area of vertical reinforcement (5.3.1.3): \( A_{st_min} = 0.0025 \times A_n = 0.47 \) in\(^2\)

**Design Results for Load Case 5**
- Design axial load: \( P_{LCS} = 16.9 \) kips
- Design moment load: \( M_{LCS} = 15.6 \) kip*ft
- Design shear load: \( V_{LCS} = 20 \) kips
- Ultimate axial load: \( P_{U,As} = P_D + 0.75 \times P_L + 0.525 \times P_E = 13.2 \) kips
- Tensile strain factor: \( \alpha = 1.5 \)
- Maximum masonry strain (9.3.2(c)): \( \varepsilon_{mu} = 0.0025 \)
- Strain in extreme tensile reinforcement: \( \varepsilon_{max} = \alpha \times \varepsilon_y = 0.0031 \)
- From strain compatibility analysis: \( A_{vert, max} = 0.71 \) in\(^2\)
- Design axial load: \( P_{LCS} = 16.9 \) kips
Axial strength reduction factor (9.1.4.4) \( \phi = 0.9 \)
Slenderness ratio \( h / r = 62.4 \) not greater than 99
Nominal axial design strength (9.3.4.1.1) \( P_n = 0.8 \times 0.8 \times f_m \times (A_n - A_{st}) \times [1 - (h / (140 \times r))^2] = 254 \) kips
Axial design strength \( \phi P_n = 228.6 \) kips
Axial utilization \( P_{LCS} / \phi P_n = 0.074 \)

**PAss - Axial design strength exceeds ultimate axial load**

Determine moment design strength about x axis under design axial load

Ultimate axial load \( P_{LCS} = 16.9 \) kips
Ultimate moment load \( M_{LCSx} = 15.6 \) kip_ft
Depth to extreme tension layer \( d_t = 21.19 \) in
From strain compatibility analysis:
Distance to neutral axis, by iteration \( c_x = 6.79 \) in
Depth of equivalent stress block (9.3.2) \( a = 0.8 \times \min(c_x,d) = 5.43 \) in
Nominal compression in masonry \( P_{xmas} = 0.8 \times f_m \times (a \times b) = 66 \) kips
Nominal axial force in reinf. \( P_{as} = -42.6 \) kips
Nominal axial strength \( P_{nx} = P_{xmas} + P_{as} = 23.4 \) kips
Nominal moment from masonry \( M_{xmas} = P_{xmas} \times (d / 2 - a / 2) = 50 \) kip_ft
Nominal moment in from reinf \( M_{as} = 51 \) kip_ft
Nominal moment strength \( M_{nx} = M_{xmas} + M_{as} = 101 \) kip_ft
Strength reduction factor (9.1.4.4) \( \phi = 0.9 \)
Axial design strength, adjusted for slenderness \( \phi P_{nx} = \phi \times P_{nx} \times [1 - (h / (140 \times r))^2] = 16.9 \) kips
Bending design strength \( \phi M_{inx} = 91 \) kip_ft
Bending utilization \( M_{LCSx} / \phi M_{inx} = 0.172 \)

**PAss - Design moment strength exceeds ultimate moment load**

Strain compatibility analysis of reinforcement for \( (P_{nx},M_{nx}) \)

Equations used to determine table contents:
Strain \( (\varepsilon) \) \( \varepsilon_{mu} \times (c_x - y_i) / c_x \)
Stress, bars inside comp. block \( (\sigma) \) \( \min(f_y, \varepsilon E_s) - 0.8f_m \)
Stress, bars outside comp. block \( (\sigma) \) \( \min(f_y, \varepsilon E_s) \)
Force \( (P_i) \) No. of bars \( \times A_{bar,i} \times \sigma_i \)
Moment \( (M_i) \) \( P_i \times (d / 2 - y_i) \)

Table of reinforcement data

<table>
<thead>
<tr>
<th>Row</th>
<th>Location (in)</th>
<th>Strain</th>
<th>Stress (psi)</th>
<th>Force (kips)</th>
<th>Moment (kip_ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.438</td>
<td>0.001602</td>
<td>44949.5</td>
<td>27.9</td>
<td>21.8</td>
</tr>
<tr>
<td>2</td>
<td>11.813</td>
<td>-0.001850</td>
<td>-53647.9</td>
<td>-33.3</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>21.188</td>
<td>-0.005302</td>
<td>-60000</td>
<td>-37.2</td>
<td>29.1</td>
</tr>
</tbody>
</table>

Notes:
- Negative force indicates reinforcement in tension.
- Stress in compression reinforcement is reduced by masonry stress to compensate for replaced area used in calculating masonry compression force.

Check shear capacity

Ultimate shear load \( V_{LCS} = 20 \) kips
Net shear area
Conservatively set M/Vd to 1.0
Maximum nominal shear capacity (Eqn. 9-19) \[ V_{n_{\text{max}}}=4 \times A_{nv} \times \sqrt{f'_{m} \times 1 \text{ psi}} = 33 \text{ kips} \]
Nominal masonry shear strength (Eqn. 9-20) \[ V_{nm}=2.25 \times A_{nv} \times \sqrt{f'_{m} \times 1 \text{ psi}} + 0.25 \times P_{LC6}=19.6 \text{ kips} \]
Area of shear reinforcement 
Nominal reinf. shear strength (Eqn. 9-21) \[ V_{ns}=0.5 \times (A_{v} / s_{tie}) \times f_{y} \times d = 19.5 \text{ kips} \]
Nominal total shear strength (Eqn. 9-17) \[ V_{n}=\min(V_{nm} + V_{ns},V_{n_{\text{max}}}) = 33 \text{ kips} \]
Shear strength reduction factor (9.1.4.5) \[ \phi_{v}=0.8 \]
\[ \phi_{v}V_{n}=26.4 \text{ kips} \]
Design shear strength
Shear utilization \[ V_{LC5} / \phi_{v}V_{n}=0.759 \]

**PASS - Design shear strength exceeds ultimate shear load**
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

- Length of foundation \( L_x = 1 \text{ ft} \)
- Width of foundation \( L_y = 3 \text{ ft} \)
- Foundation area \( A = L_x \times L_y = 3 \text{ ft}^2 \)
- Depth of foundation \( h = 12 \text{ in} \)
- Depth of soil over foundation \( h_{soil} = 8 \text{ in} \)
- Density of concrete \( \gamma_{conc} = 150.0 \text{ lb/ft}^3 \)

Wall no.1 details
- Width of wall \( l_{y1} = 8 \text{ in} \)
- Position in y-axis \( y_1 = 18 \text{ in} \)

Soil properties
- Gross allowable bearing pressure \( q_{allow\_Gross} = 1.5 \text{ ksf} \)
- Density of soil \( \gamma_{soil} = 120.0 \text{ lb/ft}^3 \)
- Angle of internal friction \( \phi_b = 30.0 \text{ deg} \)
- Design base friction angle \( \delta_b = 30.0 \text{ deg} \)
- Coefficient of base friction \( \tan(\delta_b) = 0.577 \)

Foundation loads
- Self weight \( F_{swt} = h \times \gamma_{conc} = 150 \text{ psf} \)
- Soil weight \( F_{soil} = h_{soil} \times \gamma_{soil} = 80 \text{ psf} \)
Wall no.1 loads per linear foot
Dead load in z \( F_{Dz1} = 1.6 \) kips
Live load in z \( F_{Lz1} = 1.6 \) kips

Footings analysis for soil and stability
Load combinations per ASCE 7-16
1.0D (0.510)
1.0D + 1.0L (0.871)
1.0D + 1.0Lr (0.510)
1.0D + 1.0S (0.510)
1.0D + 1.0R (0.510)
1.0D + 0.75L + 0.75Lr (0.781)
1.0D + 0.75L + 0.75S (0.781)
1.0D + 0.75L + 0.75R (0.781)

Combination 2 results: 1.0D + 1.0L

Forces on foundation per linear foot
Force in z-axis \( F_{dz} = \gamma \times A \times (F_{swt} + F_{soil}) + \gamma \times F_{Dz1} + \gamma \times F_{Lz1} = 3.9 \) kips

Moments on foundation per linear foot
Moment in y-axis, about y is 0 \( M_{dy} = \gamma \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma \times (F_{Dz1} \times y_1) + \gamma \times (F_{Lz1} \times y_1) = 5.9 \) kip_ft

Uplift verification
Vertical force \( F_{dz} = 3.92 \) kips

Stability against sliding
Resistance due to base friction \( F_{RFriction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{ba}) = 2.263 \) kips

Bearing resistance
Eccentricity of base reaction
Eccentricity of base reaction in y-axis \( e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 0.000 \) in

Strip base pressures
\( q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 1.307 \) ksf
\( q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 1.307 \) ksf

Minimum base pressure

Maximum base pressure

Allowable bearing capacity
Allowable bearing capacity
\( Q_{allow} = Q_{allow, Gross} = 1.5 \) ksf
\( Q_{max} / Q_{allow} = 0.871 \)

FOOTING DESIGN (ACI318)
In accordance with ACI318-14
Material details
Compressive strength of concrete \( f_c = 3000 \) psi
Yield strength of reinforcement \( f_y = 600000 \) psi
Cover to reinforcement: 3 in
Concrete type: Normal weight
Concrete modification factor: λ = 1.00
Wall type: Masonry

Analysis and design of concrete footing

Load combinations per ASCE 7-16

1.4D (0.048)
1.2D + 1.6L + 0.5Lr (0.096)
1.2D + 1.6L + 0.5S (0.096)
1.2D + 1.6L + 0.5R (0.096)
1.2D + 1.0L + 1.6Lr (0.076)
1.2D + 1.0L + 1.6S (0.076)
1.2D + 1.0L + 1.6R (0.076)
1.2D + 1.6Lr + 0.5W (0.041)
1.2D + 1.6S + 0.5W (0.041)
1.2D + 1.6R + 0.5W (0.041)
1.2D + 1.0L + 0.5Lr + 1.0W (0.076)
1.2D + 1.0L + 0.5S + 1.0W (0.076)
1.2D + 1.0L + 0.5R + 1.0W (0.076)
(1.2 + 0.2 × S_d)D + 1.0L + 0.2S + 1.0E (0.082)
0.9D + 1.0W (0.031)
(0.9 - 0.2 × S_d)D + 1.0E (0.024)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation per linear foot

Ultimate force in z-axis

\[ F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = 5.4 \text{ kips} \]

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0

\[ M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = 8.0 \text{ kip*ft} \]

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

\[ e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0.000 \text{ in} \]

Strip base pressures

\[ q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 1.785 \text{ ksf} \]

\[ q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 1.785 \text{ ksf} \]

Minimum ultimate base pressure

\[ q_{umin} = \min(q_{u1}, q_{u2}) = 1.785 \text{ ksf} \]

Maximum ultimate base pressure

\[ q_{umax} = \max(q_{u1}, q_{u2}) = 1.785 \text{ ksf} \]

Shear diagram (kips)

\[ \text{Shear diagram (kips)} \]

\[ \text{2.3} \]

\[ \text{0} \]

\[ 0 \]

\[ -2.3 \]
**Moment design, y direction, positive moment**

Ultimate bending moment \( M_{u,y,max} = 1.341 \text{ kip}_{\text{ft}} \)

Tension reinforcement provided
No.5 bars at 10.0 in c/c bottom

Area of tension reinforcement provided \( A_{s,y,bot,prov} = 0.372 \text{ in}^2 \)

Minimum area of reinforcement (7.6.1.1)
\( A_{s,min} = 0.0018 \times L_x \times h = 0.259 \text{ in}^2 \)

**PASS - Area of reinforcement provided exceeds minimum**

Maximum spacing of reinforcement (7.7.2.3)
\( s_{max} = \min(3 \times h, 18 \text{ in}) = 18 \text{ in} \)

**PASS - Maximum permissible reinforcement spacing exceeds actual spacing**

Depth to tension reinforcement
\( d = h - c_{nom} - \phi_{y,bot} / 2 = 8.688 \text{ in} \)

Depth of compression block
\( a = A_{s,y,bot,prov} \times f_y / (0.85 \times f'_{c} \times L_x) = 0.729 \text{ in} \)

Neutral axis factor \( \beta_1 = 0.85 \)

Depth to neutral axis
\( c = a / \beta_1 = 0.858 \text{ in} \)

Strain in tensile reinforcement (7.3.3.1)
\( \varepsilon_t = 0.003 \times d / c - 0.003 = 0.02737 \)

**PASS - Tensile strain exceeds minimum required, 0.004**

Nominal moment capacity
\( M_n = A_{s,y,bot,prov} \times f_y \times (d - a / 2) = 15.48 \text{ kip}_{\text{ft}} \)

Flexural strength reduction factor
\( \phi = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900 \)

Design moment capacity
\( \phi M_n = \phi y \times M_n = 13.932 \text{ kip}_{\text{ft}} \)
\( M_{u,y,max} / \phi M_n = 0.096 \)

**PASS - Design moment capacity exceeds ultimate moment load**

One-way shear design, y direction

Ultimate shear force \( V_{u,y} = 0.668 \text{ kips} \)

Depth to reinforcement
\( d_y = h - c_{nom} - \phi_{y,bot} / 2 = 8.688 \text{ in} \)

Shear strength reduction factor \( \phi_s = 0.75 \)

Nominal shear capacity (Eq. 22.5.5.1)
\( V_n = 2 \times \lambda \times \phi (f'_c \times 1 \text{ psi}) \times L_x \times d_y = 11.42 \text{ kips} \)

Design shear capacity
\( \phi V_n = \phi y \times V_n = 8.565 \text{ kips} \)
\( V_{u,y} / \phi V_n = 0.078 \)

**PASS - Design shear capacity exceeds ultimate shear load**
No. 5 bars at 10 in c/c bottom
GARAGE ROOF JOIST CALCULATIONS
## JOIST CALCULATIONS

**ASD - OWSJ DESIGN WITH GRAVITY LOADING**

**Comment:** Roof joists are open web steel joists (OWSJ). This item is engineered by others per SJI specifications. Design loads are provided herein and noted on the CDs.

### JOIST MARK: J1

Unfactored Loading Acting on Joist

<table>
<thead>
<tr>
<th>Load</th>
<th>DL (psf)</th>
<th>LL (psf)</th>
<th>Flat Roof SL (psf)</th>
<th>Drift Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>29.4</td>
</tr>
<tr>
<td>Misc.</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Total Load on Joist = | 79.4 psf |
| Total Load = | 238.3 plf |
| Uniform Dead Load = | 60.0 psf |
| Uniform Live Load = | 90.0 psf |

**SIZE:** 32 ft

**LOOKUP:** 239 / 90

**JOIST MARK: J2**

Loading Acting on Joist

<table>
<thead>
<tr>
<th>Load</th>
<th>DL (psf)</th>
<th>LL (psf)</th>
<th>Flat Roof SL (psf)</th>
<th>Drift Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>15.8</td>
</tr>
<tr>
<td>Misc.</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Total Load on Joist = | 65.8 psf |
| Total Load = | 197.5 plf |
| Uniform Dead Load = | 60.0 psf |
| Uniform Live Load = | 137.5 psf |

**SIZE:** 32 ft

**LOOKUP:** 198 / 138

**JOIST MARK: J3**

Loading Acting on Joist

<table>
<thead>
<tr>
<th>Load</th>
<th>DL (psf)</th>
<th>LL (psf)</th>
<th>Flat Roof SL (psf)</th>
<th>Drift Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>23.9</td>
</tr>
<tr>
<td>Misc.</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| Total Load on Joist = | 52.3 psf |
| Total Load = | 222.1 plf |
| Uniform Dead Load = | 85.0 psf |
| Uniform Live Load = | 137.1 psf |

**SIZE:** 32 ft

**LOOKUP:** 223 / 138

**Comment:** Steel roof joists and joist girders are designed by others in accordance with loading information provided in the construction documents.
### JOIST CALCULATIONS

Comment: Roof joists are open web steel joists (OWSJ). This item is engineered by others per SJI specifications. Design loads are provided herein and noted on the CDs.

#### JOIST MARK: TYP. JOIST

**Loading Acting on Joist**

<table>
<thead>
<tr>
<th>Load</th>
<th>DL (psf)</th>
<th>LL (psf)</th>
<th>Flat Roof Sl (psf)</th>
<th>Drift Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>0.6</td>
</tr>
<tr>
<td>Misc.</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total Load on Joist = 50.6 psf  
Total Load = 265.4 psf  
Uniform Dead Load = 105.0 psf  
Uniform Live Load = 160.4 psf

**Comment:** Steel roof joists and joist girders are designed by others in accordance with loading information provided in the construction documents.

**LOADS**

- **Span (ft):** 32.0  
- **Roof Trib (ft):** 5.3  
- **Drift Pressure (psf):** 26.0  
- **Drift Width (ft):** 6.0  
- **Joist Perpendicular to Drift (Y/N):** Y  
- **Dist. Away from Parapet Wall (ft):** 0.0  

**SIZE:** 32 ft  
**LOOKUP:** 266 / 161  
**TYP. JOIST USE:** 22K5
FOUNDATION CALCULATIONS
FOUNDATION ANALYSIS & DESIGN (ACI 318)

In accordance with ACI 318-14

FOOTING ANALYSIS

- Length of foundation: \( L_x = 1 \text{ ft} \)
- Width of foundation: \( L_y = 2 \text{ ft} \)
- Foundation area: \( A = L_x \times L_y = 2 \text{ ft}^2 \)
- Depth of foundation: \( h = 12 \text{ in} \)
- Depth of soil over foundation: \( h_{\text{soil}} = 18 \text{ in} \)
- Density of concrete: \( \gamma_{\text{conc}} = 150.0 \text{ lb/ft}^3 \)

Wall no.1 details

- Width of wall: \( l_{y1} = 8 \text{ in} \)
- Position in y-axis: \( y_1 = 12 \text{ in} \)

Soil properties

- Net allowable bearing pressure: \( q_{\text{allow,Net}} = 1.5 \text{ ksf} \)
- Density of soil: \( \gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3 \)
- Angle of internal friction: \( \phi_b = 30.0 \text{ deg} \)
- Design base friction angle: \( \delta_{bb} = 30.0 \text{ deg} \)
- Coefficient of base friction: \( \tan(\delta_{bb}) = 0.577 \)
- Self weight: \( F_{\text{swt}} = h \times \gamma_{\text{conc}} = 150 \text{ psf} \)
- Soil weight: \( F_{\text{soil}} = h_{\text{soil}} \times \gamma_{\text{soil}} = 180 \text{ psf} \)
Wall no.1 loads per linear foot
- Dead load in z \( F_{Dz1} = 1.4 \text{ kips} \)
- Live roof load in z \( F_{Lrz1} = 0.3 \text{ kips} \)
- Snow load in z \( F_{Sz1} = 0.7 \text{ kips} \)
- Wind load in z \( F_{Wz1} = 0.4 \text{ kips} \)

Footing analysis for soil and stability

Load combinations per ASCE 7-10
- 1.0D (0.569)
- 1.0D + 1.0L (0.569)
- 1.0D + 1.0Lr (0.664)
- 1.0D + 1.0S (0.753)
- 1.0D + 1.0R (0.569)
- 1.0D + 0.75L + 0.75Lr (0.640)
- 1.0D + 0.75L + 0.75S (0.707)
- 1.0D + 0.75L + 0.75R (0.569)

Combination 4 results: 1.0D + 1.0S

Forces on foundation per linear foot
- Force in z-axis \( F_{dz} = \gamma D \times A \times (F_{swt} + F_{soil}) + \gamma D \times F_{Dz1} + \gamma S \times F_{Sz1} = 2.7 \text{ kips} \)

Moments on foundation per linear foot
- Moment in y-axis, about y is 0 \( M_{dy} = \gamma D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma D \times (F_{Dz1} \times y_1) + \gamma S \times (F_{Sz1} \times y_1) = 2.7 \text{ kip_ft} \)

Uplift verification
- Vertical force \( F_{dz} = 2.71 \text{ kips} \)

Stability against sliding
- Resistance due to base friction \( F_{RFriction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb}) = 1.565 \text{ kips} \)

Bearing resistance
- Eccentricity of base reaction \( e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 0.000 \text{ in} \)

Strip base pressures
- Minimum base pressure \( q_{min} = \min(q_1, q_2) = 1.355 \text{ ksf} \)
- Maximum base pressure \( q_{max} = \max(q_1, q_2) = 1.355 \text{ ksf} \)

Allowable bearing capacity
- Allowable bearing capacity \( q_{allow} = q_{allow_{Net}} + (h + h_{soil}) \times \gamma_{soil} = 1.8 \text{ ksf} \)
- \( q_{max} / q_{allow} = 0.753 \)

FOOTING DESIGN (ACI318)
- In accordance with ACI318-14
Material details
Compressive strength of concrete $f_c = 3000$ psi
Yield strength of reinforcement $f_y = 60000$ psi
Cover to reinforcement $c_{nom} = 3$ in
Concrete type Normal weight
Concrete modification factor $\lambda = 1.00$
Wall type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10
1.4D (0.019)
1.2D + 1.6L + 0.5Lr (0.017)
1.2D + 1.6L + 0.5S (0.019)
1.2D + 1.6L + 0.5R (0.016)
1.2D + 1.0L + 1.6Lr (0.021)
1.2D + 1.0L + 1.6S (0.026)
1.2D + 1.0L + 1.6R (0.016)
1.2D + 1.6Lr + 0.5W (0.018)
1.2D + 1.6S + 0.5W (0.023)
1.2D + 1.6S + 0.5W (0.028)
1.2D + 1.6R + 0.5W (0.018)
1.2D + 1.0L + 0.5Lr + 1.0W (0.021)
1.2D + 1.0L + 0.5S + 1.0W (0.023)
1.2D + 1.0L + 0.5R + 1.0W (0.020)
1.2D + 1.0L + 0.2S + 1.0E (0.018)
1.2D + 1.6Lr + 0.2S + 1.0E (0.018)
0.9D + 1.0W (0.016)
0.9 - 0.2 $\times$ S0S)D + 1.0L + 0.2S + 1.0E (0.011)

Combination 9 results: 1.2D + 1.6S + 0.5W

Forces on foundation per linear foot
Ultimate force in z-axis $F_{uz} = \gamma D \times A \times (F_{swt} + F_{soil}) + \gamma D \times F_{Dz1} + \gamma S \times F_{Sz1} + \gamma W \times F_{Wz1} = 3.7$ kips

Moments on foundation per linear foot
Ultimate moment in y-axis, about y is 0 $M_{uy} = \gamma D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma D \times (F_{Dz1} \times y_1) + \gamma S \times (F_{Sz1} \times y_1) + \gamma W \times (F_{Wz1} \times y_1) = 3.7$ kip-ft

Eccentricity of base reaction
Eccentricity of base reaction in y-axis $e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0.000$ in

Strip base pressures
$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 1.858$ ksf
$q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 1.858$ ksf

Minimum ultimate base pressure $q_{umin} = \min(q_{u1}, q_{u2}) = 1.858$ ksf
Maximum ultimate base pressure $q_{umax} = \max(q_{u1}, q_{u2}) = 1.858$ ksf
Moment design, y direction, positive moment

Ultimate bending moment \( M_{u,y,max} = 0.325 \text{ kip}_\text{ft} \)

Tension reinforcement provided

No.5 bars at 12.0 in c/c bottom

Area of tension reinforcement provided \( A_{\text{sy,bot,prov}} = 0.31 \text{ in}^2 \)

Minimum area of reinforcement (7.6.1.1) \( A_{s,\text{min}} = 0.0018 \times L_x \times h = 0.259 \text{ in}^2 \)

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (7.7.2.3) \( s_{\text{max}} = \min(3 \times h, 18 \text{ in}) = 18 \text{ in} \)

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement \( d = h - c_{\text{nom}} - \phi_y \cdot \text{bot} / 2 = 8.688 \text{ in} \)

Depth of compression block \( a = A_{\text{sy,bot,prov}} \times f_y / (0.85 \times f'_c \times L_x) = 0.608 \text{ in} \)

Neutral axis factor \( \beta_1 = 0.85 \)

Depth to neutral axis \( c = a / \beta_1 = 0.715 \text{ in} \)

Strain in tensile reinforcement (7.3.3.1) \( \varepsilon_t = 0.003 \times d / c - 0.003 = 0.03345 \)

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity \( M_n = A_{\text{sy,bot,prov}} \times f_y \times (d - a / 2) = 12.995 \text{ kip}_\text{ft} \)

Flexural strength reduction factor \( \phi = \min(\max(0.65 + (\varepsilon - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900 \)

Design moment capacity \( \phi M_n = \phi \times M_n = 11.695 \text{ kip}_\text{ft} \)

\( M_{u,y,max} / \phi M_n = 0.028 \)

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.
No. 5 bars at 12 in c/c bottom
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

FOOTING ANALYSIS

Length of foundation \( L_x = 1 \text{ ft} \)
Width of foundation \( L_y = 1 \text{ ft} \)
Foundation area \( A = L_x \times L_y = 1 \text{ ft}^2 \)
Depth of foundation \( h = 12 \text{ in} \)
Depth of soil over foundation \( h_{soil} = 0 \text{ in} \)
Density of concrete \( \gamma_{conc} = 150.0 \text{ lb/ft}^3 \)

Wall no.1 details

Width of wall \( l_{wl} = 8 \text{ in} \)
position in y-axis \( y_{wl} = 6 \text{ in} \)

Soil properties

Net allowable bearing pressure \( q_{allow\_Net} = 1.5 \text{ ksf} \)
Density of soil \( \gamma_{soil} = 120.0 \text{ lb/ft}^3 \)
Angle of internal friction \( \phi_b = 30.0 \text{ deg} \)
Design base friction angle \( \delta_{bb} = 30.0 \text{ deg} \)
Coefficient of base friction \( \tan(\delta_{bb}) = 0.577 \)

Foundation loads

Self weight \( F_{swt} = h \times \gamma_{conc} = 150 \text{ psf} \)
Wall no.1 loads per linear foot
Dead load in z

Footing analysis for soil and stability
Load combinations per ASCE 7-10
1.0D (0.401)  
1.0D + 1.0L (0.401)  
1.0D + 1.0Lr (0.401)  
1.0D + 1.0S (0.401)  
1.0D + 1.0R (0.401)  
1.0D + 0.75L + 0.75Lr (0.401)  
1.0D + 0.75L + 0.75S (0.401)  
1.0D + 0.75L + 0.75R (0.401)

Combination 1 results: 1.0D

Forces on foundation per linear foot
Force in z-axis

\[ F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times F_{Dz1} = 0.7 \text{ kips} \]

Moments on foundation per linear foot
Moment in y-axis, about y is 0

\[ M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y) = 0.3 \text{ kip-ft} \]

Uplift verification
Vertical force

\[ F_{dz} = 0.65 \text{ kips} \]

\[ \text{PASS - Foundation is not subject to uplift} \]

Stability against sliding
Resistance due to base friction

\[ F_{RFriction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb}) = 0.375 \text{ kips} \]

Bearing resistance

Eccentricity of base reaction
Eccentricity of base reaction in y-axis

\[ e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 0.000 \text{ in} \]

Strip base pressures

\[ q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 0.65 \text{ ksf} \]

\[ q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 0.65 \text{ ksf} \]

Minimum base pressure

\[ q_{min} = \min(q_1, q_2) = 0.65 \text{ ksf} \]

Maximum base pressure

\[ q_{max} = \max(q_1, q_2) = 0.65 \text{ ksf} \]

Allowable bearing capacity

Allowable bearing capacity

\[ q_{allow} = q_{allow, Net} + (h + h_{soil}) \times \gamma_{soil} = 1.62 \text{ ksf} \]

\[ q_{max} / q_{allow} = 0.401 \]

\[ \text{PASS - Allowable bearing capacity exceeds design base pressure} \]

FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details
Compressive strength of concrete

\[ f'_c = 3000 \text{ psi} \]

Yield strength of reinforcement

\[ f_y = 60000 \text{ psi} \]

Cover to reinforcement

\[ c_{nom} = 2 \text{ in} \]
Concrete type: Normal weight
Concrete modification factor: \( \lambda = 1.00 \)
Wall type: Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.000)
1.2D + 1.6L + 0.5Lr (0.000)
1.2D + 1.6L + 0.5S (0.000)
1.2D + 1.6L + 0.5R (0.000)
1.2D + 1.0L + 1.6Lr (0.000)
1.2D + 1.0L + 1.6S (0.000)
1.2D + 1.0L + 1.6R (0.000)
1.2D + 1.6Lr + 0.5W (0.000)
1.2D + 1.6S + 0.5W (0.000)
1.2D + 1.6R + 0.5W (0.000)
1.2D + 1.0L + 0.5Lr + 1.0W (0.000)
1.2D + 1.0L + 0.5S + 1.0W (0.000)
1.2D + 1.0L + 0.5R + 1.0W (0.000)
(1.2 + 0.2 \times S_{DB})D + 1.0L + 0.2S + 1.0E (0.000)
0.9D + 1.0W (0.000)
(0.9 - 0.2 \times S_{DB})D + 1.0E (0.000)

Combination 1 results: 1.4D

Forces on foundation per linear foot
Ultimate force in z-axis
\[ F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times F_{Dz1} = 0.9 \text{ kips} \]

Moments on foundation per linear foot
Ultimate moment in y-axis, about y is 0
\[ M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) = 0.5 \text{ kip\_ft} \]

Eccentricity of base reaction
Eccentricity of base reaction in y-axis
\[ e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0.000 \text{ in} \]

Strip base pressures
\[ q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 0.91 \text{ ksf} \]
\[ q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 0.91 \text{ ksf} \]

Minimum ultimate base pressure
\[ q_{umin} = \min(q_{u1}, q_{u2}) = 0.91 \text{ ksf} \]

Maximum ultimate base pressure
\[ q_{umax} = \max(q_{u1}, q_{u2}) = 0.91 \text{ ksf} \]

Shear diagram (kips)
One-way shear design, y direction

One-way shear design does not apply. Shear failure plane fall outside extents of foundation.

No.5 bars at 24 in o.c. bottom
FOOTING ANALYSIS

Length of foundation \( L_x = 1 \text{ ft} \)
Width of foundation \( L_y = 1 \text{ ft} \)
Foundation area \( A = L_x \times L_y = 1 \text{ ft}^2 \)
Depth of foundation \( h = 12 \text{ in} \)
Depth of soil over foundation \( h_{soil} = 0 \text{ in} \)
Density of concrete \( \gamma_{conc} = 150.0 \text{ lb/ft}^3 \)

Wall no.1 details
Width of wall \( l_{y1} = 8 \text{ in} \)
position in y-axis \( y_1 = 6 \text{ in} \)

Soil properties
Net allowable bearing pressure \( q_{allow,\text{Net}} = 1.5 \text{ ksf} \)
Density of soil \( \gamma_{soil} = 120.0 \text{ lb/ft}^3 \)
Angle of internal friction \( \phi_b = 30.0 \text{ deg} \)
Design base friction angle \( \delta_{bb} = 30.0 \text{ deg} \)
Coefficient of base friction \( \tan(\delta_{bb}) = 0.577 \)
Self weight \( F_{swt} = h \times \gamma_{conc} = 150 \text{ psf} \)

Wall no.1 loads per linear foot
Dead load in z \( F_{dz1} = 0.9 \text{ kips} \)
Footing analysis for soil and stability

Load combinations per ASCE 7-10
1.0D (0.648)
1.0D + 1.0L (0.648)
1.0D + 1.0Lr (0.648)
1.0D + 1.0S (0.648)
1.0D + 1.0R (0.648)
1.0D + 0.75L + 0.75Lr (0.648)
1.0D + 0.75L + 0.75S (0.648)
1.0D + 0.75L + 0.75R (0.648)

Combination 1 results: 1.0D

Forces on foundation per linear foot
Force in z-axis
\[ F_{dz} = \gamma_D \times A \times F_{swt} + \gamma_D \times F_{Dz1} = 1.1 \text{ kips} \]

Moments on foundation per linear foot
Moment in y-axis, about y is 0
\[ M_{dy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) = 0.5 \text{ kip-ft} \]

Uplift verification
Vertical force
\[ F_{dz} = 1.05 \text{ kips} \]

**PASS - Foundation is not subject to uplift**

Stability against sliding
Resistance due to base friction
\[ F_{Friction} = \max(F_{dz}, 0 \text{ kN}) \times \tan(\delta_{bb}) = 0.606 \text{ kips} \]

Bearing resistance

Eccentricity of base reaction
\[ e_{dy} = M_{dy} / F_{dz} - L_y / 2 = 0.000 \text{ in} \]

Strip base pressures
\[ q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 1.05 \text{ ksf} \]
\[ q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1 \text{ ft}) = 1.05 \text{ ksf} \]

Minimum base pressure
\[ q_{min} = \min(q_1, q_2) = 1.05 \text{ ksf} \]

Maximum base pressure
\[ q_{max} = \max(q_1, q_2) = 1.05 \text{ ksf} \]

Allowable bearing capacity
Allowable bearing capacity
\[ q_{allow} = q_{allow, Net} + (h + h_{soil}) \times \gamma_{soil} = 1.62 \text{ ksf} \]
\[ q_{max} / q_{allow} = 0.648 \]

**PASS - Allowable bearing capacity exceeds design base pressure**

**FOOTING DESIGN (ACI318)**

In accordance with ACI318-14

Material details
Compressive strength of concrete
\[ f'_c = 3000 \text{ psi} \]
Yield strength of reinforcement
\[ f_y = 60000 \text{ psi} \]
Cover to reinforcement
\[ c_{nom} = 2 \text{ in} \]
Concrete type
Normal weight
Concrete modification factor
\[ \lambda = 1.00 \]
Wall type
Concrete
Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.002)
1.2D + 1.6L + 0.5Lr (0.001)
1.2D + 1.6L + 0.5S (0.001)
1.2D + 1.6L + 0.5R (0.001)
1.2D + 1.0L + 1.6Lr (0.001)
1.2D + 1.0L + 1.6S (0.001)
1.2D + 1.0L + 1.6R (0.001)
1.2D + 1.6Lr + 0.5W (0.001)
1.2D + 1.6S + 0.5W (0.001)
1.2D + 1.6R + 0.5W (0.001)
1.2D + 1.0L + 0.5Lr + 1.0W (0.001)
1.2D + 1.0L + 0.5S + 1.0W (0.001)
1.2D + 1.0L + 0.5R + 1.0W (0.001)
(1.2 + 0.2 × S_{D3})D + 1.0L + 0.2S + 1.0E (0.001)
0.9D + 1.0W (0.001)
(0.9 - 0.2 × S_{D3})D + 1.0E (0.001)

Combination 16 results: (0.9 - 0.2 × S_{D3})D + 1.0E

Forces on foundation per linear foot

Ultimate force in z-axis

\[ F_{uz} = \gamma D \times A \times F_{swt} + \gamma D \times F_{Dz1} = 0.9 \text{ kips} \]

Moments on foundation per linear foot

Ultimate moment in y-axis, about y is 0

\[ M_{uy} = \gamma D \times A \times F_{swt} \times L_y / 2 + \gamma D \times (F_{Dz1} \times y_1) = 0.5 \text{ kip\_ft} \]

Eccentricity of base reaction

Eccentricity of base reaction in y-axis

\[ e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0.000 \text{ in} \]

Strip base pressures

\[ q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 0.91 \text{ ksf} \]

\[ q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1 \text{ ft}) = 0.91 \text{ ksf} \]

Minimum ultimate base pressure

\[ q_{umin} = \min(q_{u1}, q_{u2}) = 0.91 \text{ ksf} \]

Maximum ultimate base pressure

\[ q_{umax} = \max(q_{u1}, q_{u2}) = 0.91 \text{ ksf} \]

Shear diagram (kips)
Moment design, y direction, positive moment

Ultimate bending moment \( M_{u,y,\text{max}} = 0.011 \text{ kip}_\text{ft} \)

Tension reinforcement provided No.5 bars at 14.0 in c/c bottom

Area of tension reinforcement provided \( A_{\text{sy,bot,prov}} = 0.266 \text{ in}^2 \)

Minimum area of reinforcement (7.6.1.1) \( A_{s,\text{min}} = 0.0018 \times L_x \times h = 0.259 \text{ in}^2 \)

**PASS - Area of reinforcement provided exceeds minimum**

Maximum spacing of reinforcement (7.7.2.3) \( s_{\text{max}} = \min(3 \times h, 18 \text{ in}) = 18 \text{ in} \)

**PASS - Maximum permissible reinforcement spacing exceeds actual spacing**

Depth to tension reinforcement \( d = h - c_{\text{nom}} - \phi_{\text{y,bot}} / 2 = 9.687 \text{ in} \)

Depth of compression block \( a = A_{\text{sy,bot,prov}} \times f_y / (0.85 \times f'_c \times L_x) = 0.521 \text{ in} \)

Neutral axis factor \( \beta_1 = 0.85 \)

Depth to neutral axis \( c = a / \beta_1 = 0.613 \text{ in} \)

Strain in tensile reinforcement (7.3.3.1) \( \varepsilon_t = 0.003 \times d / c - 0.003 = 0.04441 \)

**PASS - Tensile strain exceeds minimum required, 0.004**

Nominal moment capacity \( M_n = A_{\text{sy,bot,prov}} \times f_y \times (d - a / 2) = 12.524 \text{ kip}_\text{ft} \)

Flexural strength reduction factor \( \phi = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900 \)

Design moment capacity \( \phi M_n = \psi \times M_n = 11.272 \text{ kip}_\text{ft} \)

\( M_{u,y,\text{max}} / \phi M_n = 0.001 \)

**PASS - Design moment capacity exceeds ultimate moment load**

One-way shear design, y direction

*One-way shear design does not apply. Shear failure plane fall outside extents of foundation.*
METAL BLDG
FOUNDATION CALCS USING ASSUMED LOADS
FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-14

Tedds calculation version 3.2.09

FOOTING ANALYSIS

Length of foundation \( L_x = 7 \text{ ft} \)
Width of foundation \( L_y = 7 \text{ ft} \)
Foundation area \( A = L_x \times L_y = 49 \text{ ft}^2 \)
Depth of foundation \( h = 30 \text{ in} \)
Depth of soil over foundation \( h_{\text{soil}} = 0 \text{ in} \)
Density of concrete \( \gamma_{\text{conc}} = 150.0 \text{ lb/ft}^3 \)

Column no.1 details

Length of column \( l_{x1} = 14.00 \text{ in} \)
Width of column \( l_{y1} = 14.00 \text{ in} \)
position in x-axis \( x_1 = 42.00 \text{ in} \)
position in y-axis \( y_1 = 42.00 \text{ in} \)

Soil properties

Gross allowable bearing pressure \( Q_{\text{allow,Gross}} = 1.5 \text{ ksf} \)
Density of soil \( \gamma_{\text{soil}} = 120.0 \text{ lb/ft}^3 \)
Angle of internal friction \( \phi_c = 30.0 \text{ deg} \)
Design base friction angle \( \delta_{bb} = 30.0 \text{ deg} \)
Coefficient of base friction \( \tan(\delta_{bb}) = 0.577 \)
Design wall friction angle \( \delta_c = 15.0 \text{ deg} \)
### Foundation loads

#### Self weight

F\(_{swt}\) = h \times \gamma_{concrete} = 375 \text{ psf}

#### Column no.1 loads

- Dead load in z: F\(_{Dz1}\) = 16.0 \text{ kips}
- Live load in z: F\(_{Lz1}\) = 31.9 \text{ kips}
- Wind load in z: F\(_{Wz1}\) = 3.3 \text{ kips}
- Seismic load in x: F\(_{Ex1}\) = 3.0 \text{ kips}

#### Footing analysis for soil and stability

##### Load combinations per ASCE 7-16

- 1.0D (0.467)
- 1.0D + 1.0L (0.901)

#### Combination 2 results: 1.0D + 1.0L

##### Forces on foundation

Forces on z-axis

\[
F_{dz} = \gamma_b \times A \times F_{swt} + \gamma_b \times F_{Dz1} + \gamma_l \times F_{Lz1} = 66.3 \text{ kips}
\]

##### Moments on foundation

- Moment in x-axis, about x is 0
  \[
  M_{mx} = \gamma_b \times A \times F_{swt} \times L_x / 2 + \gamma_b \times (F_{Dz1} \times x_1) + \gamma_l \times (F_{Lz1} \times x_1) = 231.9 \text{ kip}\_\text{ft}
  \]
- Moment in y-axis, about y is 0
  \[
  M_{my} = \gamma_b \times A \times F_{swt} \times L_y / 2 + \gamma_b \times (F_{Dz1} \times y_1) + \gamma_l \times (F_{Lz1} \times y_1) = 231.9 \text{ kip}\_\text{ft}
  \]

##### Uplift verification

Vertical force

\[
F_{dz} = 66.255 \text{ kips}
\]

**PASS - Foundation is not subject to uplift**

### Bearing resistance

#### Eccentricity of base reaction

- Eccentricity of base reaction in x-axis: e\(_{dx}\) = M\(_{mx}\) / F\(_{dz}\) - L\(_x\) / 2 = 0 in
- Eccentricity of base reaction in y-axis: e\(_{dy}\) = M\(_{my}\) / F\(_{dz}\) - L\(_y\) / 2 = 0 in

#### Pad base pressures

\[
\begin{align*}
q_1 &= F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.352 \text{ ksf} \\
q_2 &= F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.352 \text{ ksf} \\
q_3 &= F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.352 \text{ ksf} \\
q_4 &= F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y) = 1.352 \text{ ksf}
\end{align*}
\]

Minimum base pressure

q\(_{min}\) = min(q\(_1\), q\(_2\), q\(_3\), q\(_4\)) = 1.352 \text{ ksf}

Maximum base pressure

q\(_{max}\) = max(q\(_1\), q\(_2\), q\(_3\), q\(_4\)) = 1.352 \text{ ksf}

#### Allowable bearing capacity

- Allowable bearing capacity
  \[
  q_{allow} = q_{allow, Gross} = 1.5 \text{ ksf}
  \]
- q\(_{allow}\) / q\(_{max}\) = 0.901

**PASS - Allowable bearing capacity exceeds design base pressure**
FOOTING DESIGN (ACI318)

In accordance with ACI318-14

Material details
Compressive strength of concrete $f'_c = 3000$ psi
Yield strength of reinforcement $f_y = 60000$ psi
Cover to reinforcement $c_{nom} = 3$ in
Concrete type Normal weight
Concrete modification factor $\lambda = 1.00$
Column type Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-16
1.4D (0.025)
1.2D + 1.6L + 0.5Lr (0.078)
1.2D + 1.6L + 0.5S (0.078)
1.2D + 1.6L + 0.5R (0.078)
1.2D + 1.0L + 1.6Lr (0.057)
1.2D + 1.0L + 1.6S (0.057)
1.2D + 1.0L + 1.6R (0.057)
1.2D + 1.6Lr + 0.5W (0.023)
1.2D + 1.6S + 0.5W (0.023)
1.2D + 1.6R + 0.5W (0.023)
1.2D + 1.0L + 0.5Lr + 1.0W (0.061)
1.2D + 1.0L + 0.5S + 1.0W (0.061)
1.2D + 1.0L + 0.5R + 1.0W (0.061)
(1.2 + 0.2 × SDS)D + 1.0L + 0.2S + 1.0E (0.061)
0.9D + 1.0W (0.020)
(0.9 - 0.2 × SDS)D + 1.0E (0.013)

Combination 2 results: 1.2D + 1.6L + 0.5Lr

Forces on foundation
Ultimate force in z-axis
$F_{uz} = \gamma_D \times A \times F_{swt} + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} = 92.3$ kips

Moments on foundation
Ultimate moment in x-axis, about x is 0
$M_{ux} = \gamma_D \times A \times F_{swt} \times L_x / 2 + \gamma_D \times (F_{Dz1} \times x_1) + \gamma_L \times (F_{Lz1} \times x_1) = 323.0$ kip-ft
Ultimate moment in y-axis, about y is 0
$M_{uy} = \gamma_D \times A \times F_{swt} \times L_y / 2 + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1) = 323.0$ kip-ft

Eccentricity of base reaction
Eccentricity of base reaction in x-axis
$e_{ux} = M_{ux} / F_{uz} - L_x / 2 = 0$ in
Eccentricity of base reaction in y-axis
$e_{uy} = M_{uy} / F_{uz} - L_y / 2 = 0$ in

Pad base pressures

Minimum ultimate base pressure
$q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 1.883$ ksf

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Maximum ultimate base pressure

\[ q_{\text{umax}} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) = 1.883 \text{ ksf} \]

Shear diagram, x axis (kips)

Moment diagram, x axis (kip_ft)

Moment design, x direction, positive moment

Ultimate bending moment

\[ M_{u,x,\text{max}} = 42.671 \text{ kip}_x \text{ ft} \]

Tension reinforcement provided

15 No.5 bottom bars (5.5 in c/c)

Area of tension reinforcement provided

\[ A_{x,\text{bot,prov}} = 4.65 \text{ in}^2 \]

Minimum area of reinforcement (8.6.1.1)

\[ A_{x,\text{min}} = 0.0018 \times L_y \times h = 4.536 \text{ in}^2 \]

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)

\[ s_{\text{max}} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in} \]

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement

\[ d = h - c_{\text{nom}} - \phi x_{\text{bot}} / 2 = 26.687 \text{ in} \]

Depth of compression block

\[ a = A_{x,\text{bot,prov}} \times f_y / (0.85 \times f'_c \times L_y) = 1.303 \text{ in} \]

Neutral axis factor

\[ \beta_1 = 0.85 \]

Depth to neutral axis

\[ c = a / \beta_1 = 1.532 \text{ in} \]

Strain in tensile reinforcement (8.3.3.1)

\[ \varepsilon_t = 0.003 \times d / c - 0.003 = 0.04925 \]

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity

\[ M_n = A_{x,\text{bot,prov}} \times f_y \times (d - a / 2) = 605.343 \text{ kip}_f \]

Flexural strength reduction factor

\[ \phi = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900 \]

Design moment capacity

\[ \phi M_n = \phi \times M_n = 544.808 \text{ kip}_f \]

\[ M_{u,x,\text{max}} / \phi M_n = 0.078 \]

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force

\[ V_{u,x} = 7.472 \text{ kips} \]

Depth to reinforcement

\[ d_v = h - c_{\text{nom}} - \phi x_{\text{bot}} / 2 = 26.687 \text{ in} \]

Shear strength reduction factor

\[ \phi_v = 0.75 \]

Nominal shear capacity (Eq. 22.5.5.1)

\[ V_n = 2 \times \lambda \times \sqrt{f'_c \times 1 \text{ psi}} \times L_y \times d_v = 245.571 \text{ kips} \]

Design shear capacity

\[ \phi V_n = \phi_v \times V_n = 184.179 \text{ kips} \]

\[ V_{u,x} / \phi V_n = 0.041 \]
PASS - Design shear capacity exceeds ultimate shear load

Shear diagram, y axis (kips)

Moment diagram, y axis (kip_ft)

Moment design, y direction, positive moment

Ultimate bending moment
- $M_{u,y,max} = 42.671 \text{ kip ft}$
Tension reinforcement provided
- 15 No.5 bottom bars (5.5 in c/c)
Area of tension reinforcement provided
- $A_{sy,bot,prov} = 4.65 \text{ in}^2$
Minimum area of reinforcement (8.6.1.1)
- $A_{a,min} = 0.0018 \times L_x \times h = 4.536 \text{ in}^2$

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (8.7.2.2)
- $s_{max} = \min(2 \times h, 18 \text{ in}) = 18 \text{ in}$

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement
- $d = h - c_{nom} - \phi_x,bot - \phi_y,bot / 2 = 26.063 \text{ in}$
Depth of compression block
- $a = A_{sy,bot,prov} \times f_y / (0.85 \times f'_c \times L_x) = 1.303 \text{ in}$
Neutral axis factor
- $\beta_1 = 0.85$
Depth to neutral axis
- $c = a / \beta_1 = 1.532 \text{ in}$
Strain in tensile reinforcement (8.3.3.1)
- $\varepsilon_t = 0.003 \times d / c - 0.003 = 0.04802$

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity
- $M_n = A_{sy,bot,prov} \times f_y \times (d - a / 2) = 590.811 \text{ kip ft}$
Flexural strength reduction factor
- $\phi = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) = 0.900$
Design moment capacity
- $\phi M_n = \phi \times M_n = 531.73 \text{ kip ft}$
- $M_{u,y,max} / \phi M_n = 0.080$

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, y direction

Ultimate shear force
- $V_{u,y} = 7.472 \text{ kips}$
Depth to reinforcement
- $d_r = h - c_{nom} - \phi_x,bot - \phi_y,bot / 2 = 26.063 \text{ in}$
Shear strength reduction factor
- $\phi_v = 0.75$
Nominal shear capacity (Eq. 22.5.5.1)
- $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi}) \times L_x \times d_r} = 239.82 \text{ kips}$
Design shear capacity
- $\phi V_n = \phi_v \times V_n = 179.865 \text{ kips}$
- $V_{u,y} / \phi V_n = 0.042$
**Two-way shear design at column 1**

- **Depth to reinforcement**: $d_v = 26.375$ in
- **Shear perimeter length (22.6.4)**: $l_{xp} = 40.375$ in
- **Shear perimeter width (22.6.4)**: $l_{yp} = 40.375$ in
- **Shear perimeter (22.6.4)**: $b_o = 2 \times (l_{x1} + d_v) + 2 \times (l_{y1} + d_v) = 161.500$ in
- **Shear area**: $A_p = l_{x,\text{perim}} \times l_{y,\text{perim}} = 1630.141$ in$^2$
- **Surcharge loaded area**: $A_{\text{sur}} = A_p - l_{x1} \times l_{y1} = 1434.141$ in$^2$
- **Ultimate bearing pressure at center of shear area**: $q_{\text{up.avg}} = 1.883$ ksf
- **Ultimate shear load**: $F_{up} = \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1} + \gamma_D \times A_p \times F_{swt} - q_{\text{up.avg}} \times A_p = 54.000$ kips
- **Ultimate shear stress from vertical load**: $v_{ug} = \max(F_{up} / (b_o \times d_v), 0 \text{ psi}) = 12.677$ psi
- **Column geometry factor (Table 22.6.5.2)**: $\beta = l_{y1} / l_{x1} = 1.00$
- **Column location factor (22.6.5.3)**: $\alpha_s = 40$
- **Concrete shear strength (22.6.5.2)**: 
  - $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = 328.634$ psi
  - $v_{cpb} = (\alpha_s \times d_v / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = 467.345$ psi
  - $v_{cp} = 4 \times \lambda \times \sqrt{(f'_c \times 1 \text{ psi})} = 219.089$ psi
  - $v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cp}) = 219.089$ psi
- **Shear strength reduction factor**: $\phi_v = 0.75$
- **Nominal shear stress capacity (Eq. 22.6.1.2)**: $v_n = v_{cp} = 219.089$ psi
- **Design shear stress capacity (8.5.1.1(d))**: $\phi v_n = \phi_v \times v_n = 164.317$ psi
- **$v_{ug} / \phi v_n = 0.077$**

**PASS - Design shear stress capacity exceeds ultimate shear stress load**