CALCULATIONS FOR

Job: 18'-0" x 18'-2" x 10'-0" Max Height - 4 Post Canopy - State of Utah

Address: 2006 International Building Code
(20 psf Red., 90 mph, Seismic Design Category D)
(These calculations apply to the job at this address only.)

Client: Western States Decking, Inc

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THESE STRUCTURAL CALCULATIONS MUST BE SUBMITTED WITH WET SEAL DATED NOT OVER 180 DAYS PRIOR TO PERMIT APPLICATIONS.

Copyright 11/09 Project Engineer G Starks

Job # 1304-09

S.E. CONSULTANTS, INC.
BASIS FOR DESIGN

DEAD LOADS

LIVE LOADS

Roofs 20.0 psf.,

LATERAL

Wind

Wind Load Basis: 90 mph (3 sec. Gust)
Exposure: C

Seismic Use Group: I
Seismic Design Cat: D

CODE:

2006 International Building Code
2001 Edition of Cold-Formed steel design manual
W/ 2004 supplement

STRESSES OF MATERIALS

CONCRETE

Footings $f_c = 2500$ psi.

STEEL

Reinforcing
Weldable Reinforcing
Roof Deck
Cold formed Steel

fy = 40000 psi. A-615, Grade 40
fy = 40000 psi. A-706, Grade 40
fy = 80000 psi. A-653, Grade 80
fy = 55000 psi. A-653, Grade 65

SOIL

Allow. Soil Bearing 1500 psf  Passive Pressure 150 psf / ft

Material Class = 5 [Table 1802.4]

Soil Class = D

Structural

Wide Flange
Tube
Pipe

fy = 36000 psi. A-36   or
fy = 50000 psi. A-572
fy = 46000 psi. A-500
fy = 36000 psi. A-501

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

S.E. CONSULTANTS, INC.
**Design Wind Loads for Monoslope Free Roofs**

- Wind direction: \( \gamma = 0^\circ \) to \( \gamma = 180^\circ \)
- \( V \), wind speed = 90 mph
- Exposure Category = C
- \( \theta \), roof angle = 0.6 \( ^\circ \)
- \( h \), mean roof height = 10.0 ft

### Main Wind Force Resisting System

- \( K_x \), exposure coeff. = 0.85 ASCE 6.5.6.6, Table 6-3
- \( K_{th} \), topography factor = 1.00 ASCE 6.5.7.2, Figure 6-4
- \( K_d \), directionality factor = 0.85 ASCE 6.5.4.4, Table 6-4
- \( I_w \), wind factor = 1.00 ASCE 6.5.5, Table 6-1
- \( G \), gust effect factor = 0.85 ASCE 6.5.8

\[ P = qzGCN = 12.73 \text{ (C}_N\text{) psf} \]

### Wind Load Coefficients

<table>
<thead>
<tr>
<th>Roof Angle, ( \theta )</th>
<th>Load Case</th>
<th>Clear Flow</th>
<th>Obstructed Flow</th>
<th>Clear Flow</th>
<th>Obstructed Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Wind Direction, ( \gamma = 0^\circ )</td>
<td>Wind Direction, ( \gamma = 180^\circ )</td>
<td>Wind Direction, ( \gamma = 0^\circ )</td>
<td>Wind Direction, ( \gamma = 180^\circ )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( C_{NW} )</td>
<td>( C_{NL} )</td>
<td>( C_{NW} )</td>
<td>( C_{NL} )</td>
</tr>
<tr>
<td>0°</td>
<td>A</td>
<td>1.2</td>
<td>0.3</td>
<td>-0.5</td>
<td>-1.2</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.1</td>
<td>-0.1</td>
<td>-1.1</td>
<td>-0.6</td>
</tr>
<tr>
<td>7.5°</td>
<td>A</td>
<td>-0.6</td>
<td>1.0</td>
<td>-1</td>
<td>-1.5</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.4</td>
<td>-0.1</td>
<td>-1.7</td>
<td>-0.8</td>
</tr>
<tr>
<td>15°</td>
<td>A</td>
<td>-0.9</td>
<td>-1.3</td>
<td>-1.1</td>
<td>-1.5</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-1.9</td>
<td>0.5</td>
<td>-2.1</td>
<td>-0.6</td>
</tr>
<tr>
<td>22.5°</td>
<td>A</td>
<td>-1.5</td>
<td>-1.6</td>
<td>-1.5</td>
<td>-1.7</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-2.4</td>
<td>-0.3</td>
<td>-2.3</td>
<td>-0.9</td>
</tr>
<tr>
<td>30°</td>
<td>A</td>
<td>-1.8</td>
<td>-1.8</td>
<td>-1.5</td>
<td>-1.8</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-2.5</td>
<td>-0.5</td>
<td>-2.3</td>
<td>-1.1</td>
</tr>
<tr>
<td>37.5°</td>
<td>A</td>
<td>-1.8</td>
<td>-1.8</td>
<td>-1.5</td>
<td>-1.8</td>
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<tr>
<td></td>
<td>B</td>
<td>-2.4</td>
<td>-0.6</td>
<td>-2.2</td>
<td>-1.1</td>
</tr>
<tr>
<td>45°</td>
<td>A</td>
<td>-1.6</td>
<td>-1.8</td>
<td>-1.3</td>
<td>-1.8</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>-2.3</td>
<td>-0.7</td>
<td>-1.9</td>
<td>-1.2</td>
</tr>
</tbody>
</table>

**Linearly interpolated coefficients from Figure 6-18A shown below.**

<table>
<thead>
<tr>
<th>0.6</th>
<th>A</th>
<th>1.20</th>
<th>0.30</th>
<th>-0.50</th>
<th>-1.20</th>
<th>1.20</th>
<th>0.30</th>
<th>-0.50</th>
<th>-1.20</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>-1.10</td>
<td>-0.10</td>
<td>-1.10</td>
<td>-0.60</td>
<td>-1.10</td>
<td>-0.10</td>
<td>-1.10</td>
<td>-0.60</td>
<td></td>
</tr>
</tbody>
</table>

Is flow obstructed (Yes/No)?

- Yes

**Load Case A (\( \gamma = 0^\circ \))**

- \( P \), windward roof = 15.28 psf
- \( P \), leeward roof = 3.82 psf

**Load Case B (\( \gamma = 0^\circ \))**

- \( P \), windward roof = -14.01 psf
- \( P \), leeward roof = -1.27 psf

**Load Case A (\( \gamma = 180^\circ \))**

- \( P \), windward roof = 15.28 psf
- \( P \), leeward roof = 3.82 psf

**Load Case B (\( \gamma = 180^\circ \))**

- \( P \), windward roof = -14.01 psf
- \( P \), leeward roof = -1.27 psf

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PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

J.b: 4  09  Date: 11  09  By: 0CS  J.b No.: 104  09  SH.: 2

S.E. CONSULTANTS, INC.
International Building Code - 2006

Soil Site Class = D
Seismic Use Group = I

\[ S_s = 178.7 \% \]
\[ S_I = 75.0 \% \]

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Table 1615.1.2 (1) (( F_a ))</th>
<th>Mapped spectral response acceleration at short periods (( S_s ))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.25</td>
<td>0.50</td>
</tr>
<tr>
<td>A</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>B</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>C</td>
<td>1.20</td>
<td>1.20</td>
</tr>
<tr>
<td>D</td>
<td>1.60</td>
<td>1.40</td>
</tr>
<tr>
<td>E</td>
<td>2.50</td>
<td>1.70</td>
</tr>
<tr>
<td>F</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

\[ F_a = 1.000 \quad (\text{interpolated}) \]

\[ S_{M_S} = F_a \times S_s = 1.787 \]

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Table 1615.1.2 (2) (( F_v ))</th>
<th>Mapped spectral response acceleration at 1 sec. periods (( S_I ))</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>A</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>B</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>C</td>
<td>1.70</td>
<td>1.60</td>
</tr>
<tr>
<td>D</td>
<td>2.40</td>
<td>2.00</td>
</tr>
<tr>
<td>E</td>
<td>3.50</td>
<td>3.20</td>
</tr>
<tr>
<td>F</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

\[ F_v = 1.5 \quad (\text{interpolated}) \]

\[ S_{M_I} = F_v \times S_I = 1.125 \]

\[ S_{D_S} = 2/3 \times S_{M_S} = 1.191 \]
\[ S_{D_I} = 2/3 \times S_{M_I} = 0.750 \]

<table>
<thead>
<tr>
<th>Sds</th>
<th>Seismic Use Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;</td>
<td>I</td>
</tr>
<tr>
<td>0.000</td>
<td>A</td>
</tr>
<tr>
<td>0.167</td>
<td>B</td>
</tr>
<tr>
<td>0.330</td>
<td>C</td>
</tr>
<tr>
<td>0.500</td>
<td>D</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sdi</th>
<th>Seismic Use Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;</td>
<td>I</td>
</tr>
<tr>
<td>0.000</td>
<td>A</td>
</tr>
<tr>
<td>0.067</td>
<td>B</td>
</tr>
<tr>
<td>0.133</td>
<td>C</td>
</tr>
<tr>
<td>0.200</td>
<td>D</td>
</tr>
</tbody>
</table>

Seismic Design Category = D
Earthquake Lateral/Longitudinal Load to Column - 2006 IBC

| Seismic Use Group | = | I |
| Seismic Design Category | = | D |
| I | = | 1.00 |
| Ss | = | 178.68 % |
| S1 | = | 76.03 % |
| S_{M5} | = | 1.787 |
| S_{M1} | = | 1.125 |
| R | = | 2.50 |
| Height | = | 10 R |
| T | = | 0.02 \times (hn)^{0.75} |
| r Reliability/redundancy Factor | = | 1.00 |
| S_{D5} | = | 1.191 |
| S_{D1} | = | 0.750 |

(16-35)  
\[ V = \frac{S_{D5}}{[R/I]} = 0.476 \text{ W} \]

(16-36)  
\[ V = \frac{S_{D1}}{[R/I]T} = 2.668 \text{ W} \]

(16-37)  
\[ V = 0.044 \times S_1 / [R/I] = 0.013 \text{ W} \]

(16-38)  
\[ V = 0.5 \times S_1 / [R/I] = 0.150 \text{ W} \]

\[ V (\text{Controls}) = 0.476 \text{ W} \quad \text{kips} \]
ROOF FRAMING & FOUNDATION PLAN

FRONT ELEVATION

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4_5346  Date: 11-09  By: GCS  Job No.: 130409  SH.: 5

S.E. CONSULTANTS, INC.
.26 ga Western Span

\begin{align*}
F_y, \text{ yield strength} &= 90.0 \text{ ksi} \\
F_u, \text{ tensile strength} &= 60.0 \text{ ksi} \\
I_x, \text{ moment of inertia} &= 0.07 \text{ in}^4/\text{ft} \\
S_y(t), \text{ section modulus} &= 0.07 \text{ in}^3/\text{ft} \\
S_y(b), \text{ section modulus} &= 0.06 \text{ in}^3/\text{ft}
\end{align*}

See Manufacturer's Specifications.
### Section Properties

<table>
<thead>
<tr>
<th>gauge</th>
<th>fy</th>
<th>wt</th>
<th>ix</th>
<th>sx</th>
<th>ma</th>
<th>ix</th>
<th>sx</th>
<th>ma</th>
</tr>
</thead>
<tbody>
<tr>
<td>29</td>
<td>80</td>
<td>0.62</td>
<td>0.0399</td>
<td>0.0407</td>
<td>1.4635</td>
<td>0.0372</td>
<td>0.0847</td>
<td>1.2481</td>
</tr>
<tr>
<td>26</td>
<td>80</td>
<td>0.86</td>
<td>0.0658</td>
<td>0.0743</td>
<td>2.6653</td>
<td>0.0621</td>
<td>0.0624</td>
<td>2.2419</td>
</tr>
<tr>
<td>24</td>
<td>50</td>
<td>1.10</td>
<td>0.0917</td>
<td>0.1090</td>
<td>3.9168</td>
<td>0.0911</td>
<td>0.0979</td>
<td>3.5208</td>
</tr>
<tr>
<td>22</td>
<td>50</td>
<td>1.40</td>
<td>0.1249</td>
<td>0.1521</td>
<td>5.4630</td>
<td>0.1249</td>
<td>0.1375</td>
<td>4.9392</td>
</tr>
</tbody>
</table>

### NOTES:

1. Effective section properties are calculated in accordance with the 2004 North American Specifications for the design of Cold-Formed Steel Structural Members.
2. Ix is for the determination of deflection.
3. Sx and Ma are for stress determination.
### Gravity - Total Allowable Load in psf, (span in feet)

<table>
<thead>
<tr>
<th>Span Type</th>
<th>Load Type</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single span</td>
<td>Stress</td>
<td>197.4</td>
<td>145.1</td>
<td>111.1</td>
<td>87.7</td>
<td>71.1</td>
<td>49.4</td>
<td>36.3</td>
<td>27.8</td>
<td>21.9</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>209.4</td>
<td>131.9</td>
<td>88.3</td>
<td>62.0</td>
<td>45.2</td>
<td>26.2</td>
<td>16.5</td>
<td>11.0</td>
<td>7.8</td>
<td>5.7</td>
</tr>
<tr>
<td>2 Spans</td>
<td>Stress</td>
<td>166.1</td>
<td>122.0</td>
<td>93.4</td>
<td>73.8</td>
<td>59.8</td>
<td>41.5</td>
<td>30.5</td>
<td>23.4</td>
<td>18.5</td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>476.1</td>
<td>299.8</td>
<td>200.8</td>
<td>141.1</td>
<td>102.8</td>
<td>59.5</td>
<td>37.5</td>
<td>25.1</td>
<td>17.6</td>
<td>12.9</td>
</tr>
<tr>
<td>3 Spans or more</td>
<td>Stress</td>
<td>193.8</td>
<td>142.4</td>
<td>109.0</td>
<td>86.1</td>
<td>69.8</td>
<td>48.5</td>
<td>35.6</td>
<td>27.3</td>
<td>21.5</td>
<td>17.4</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>395.9</td>
<td>249.3</td>
<td>167.0</td>
<td>117.3</td>
<td>85.5</td>
<td>49.5</td>
<td>31.2</td>
<td>20.9</td>
<td>14.7</td>
<td>10.7</td>
</tr>
</tbody>
</table>

### Uplift - Total Allowable Load in psf, (span in feet)

<table>
<thead>
<tr>
<th>Span Type</th>
<th>Load Type</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single span</td>
<td>Stress</td>
<td>166.1</td>
<td>122.0</td>
<td>93.4</td>
<td>73.8</td>
<td>59.8</td>
<td>41.5</td>
<td>30.5</td>
<td>23.4</td>
<td>18.5</td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>197.6</td>
<td>124.5</td>
<td>83.4</td>
<td>58.6</td>
<td>42.7</td>
<td>24.7</td>
<td>15.6</td>
<td>10.4</td>
<td>7.3</td>
<td>5.3</td>
</tr>
<tr>
<td>2 Spans</td>
<td>Stress</td>
<td>197.4</td>
<td>145.1</td>
<td>111.1</td>
<td>87.7</td>
<td>71.1</td>
<td>49.4</td>
<td>36.3</td>
<td>27.8</td>
<td>21.9</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>504.4</td>
<td>317.7</td>
<td>212.8</td>
<td>149.5</td>
<td>109.0</td>
<td>63.1</td>
<td>39.7</td>
<td>26.6</td>
<td>18.7</td>
<td>13.6</td>
</tr>
<tr>
<td>3 Spans or more</td>
<td>Stress</td>
<td>230.4</td>
<td>169.3</td>
<td>129.6</td>
<td>102.4</td>
<td>83.0</td>
<td>57.6</td>
<td>42.3</td>
<td>32.4</td>
<td>25.6</td>
<td>20.7</td>
</tr>
<tr>
<td></td>
<td>Deflection</td>
<td>419.5</td>
<td>264.2</td>
<td>177.0</td>
<td>124.3</td>
<td>90.6</td>
<td>52.4</td>
<td>33.0</td>
<td>22.1</td>
<td>15.5</td>
<td>11.3</td>
</tr>
</tbody>
</table>

**NOTES:**

1. Allowable loads are based on section properties in accordance with the 2004 North American Specification for the Design of Cold-Formed Steel Structural Members.

2. Load capacities do not include the panel self weight. Capacities are for the panel itself and do not include the capacities for the panel attachments or panel supports. (fastener pullout or panel pullover is not included in capacities)

3. A 1/3 stress increase for wind is not included in allowables shown.

4. Allowable loads are based on Fy = 60 ksi.

5. A ratio of L/180 was used for all deflection calculations.

**ISSUED: APRIL 18, 2008**
THE FOLLOWING IS BASED ON THE 2001 EDITION OF THE COLD-FORMED STEEL DESIGN MANUAL

DEFINITIONS:

\[ d = \text{Nominal screw diameter} \]
\[ dw = \text{Larger of Head or Washer Diameter, not larger than 1/2"} \]
Factor of Safety = 3.0

\[ \text{Pas} = \text{allowable shear force per screw} \]
\[ \text{Pns} = \text{nominal shear strength per screw} \]
\[ \text{Pat} = \text{allowable tension force per screw} \]
\[ \text{Pnt} = \text{nominal tension strength per screw} \]

\[ \text{Pnot} = \text{pull-out per screw} \]
\[ \text{Pnov} = \text{pull-over per screw} \]

\[ t1 = \text{thickness of member in contact with the screw head} \]
\[ t2 = \text{thickness of member not in contact with the screw head} \]

\[ \text{Fu1} = \text{tensile strength of member in contact with screw} \]
\[ \text{Fu2} = \text{tensile strength of member not in contact with screw} \]

SECTION E4.3.1 CONNECTION SHEAR GOVERNED BY BASE METALS

\[ t2/t1 = 4.1732 \]
\[ \text{t2/t1} \geq 2.5, \text{USE EQUATIONS E4.3.4 & E4.3.5} \]

\[ \text{WHEN } t2/t1 < 1.0 \]
\[ \text{WHEN } t2/t1 \geq 2.5 \]

\[ \text{Equation E4.3.1} \]
\[ = 2590.4 \text{ lbs/screw} \]

\[ \text{Equation E4.3.2} \]
\[ = 939.5 \text{ lbs/screw} \]

\[ \text{Equation E4.3.3} \]
\[ = 2831.7 \text{ lbs/screw} \]

\[ \text{Pns (smallest of the above:)} \]
\[ 939.5 \text{ lbs/screw} \]

\[ \text{Pas = Allow. Shear per SCREW} = \frac{\text{Pns}}{\text{F.S. of 3.0}} = 313.2 \text{ lbs/screw} \]

* Allow. Shear Capacity of Screw * 625.0 lbs/screw

* Based on "HILTI" w/ F.S. of 3.0

SECTION E4.4, TENSION

\[ \text{Equation E4.4.1.1, Pullout Force, Pnot} = 891.5 \text{ lbs/screw} \]
\[ \text{Equation E4.4.2.1, Pullover Force, Pnov} = 755.2 \text{ lbs/screw} \]

\[ \text{Pnt} = \text{nominal tension strength per screw} = \text{lesser of Pnot & Pnov} \]

\[ \text{Pnt} = 755.2 \text{ lbs/screw} \]

\[ \text{Pat = Allow. Tension per SCREW = Pnt/ (F.S. of 3.0) = 251.7 lbs/screw} \]

\[ \text{NOTE: Shear Governed By Base Metals.} \]

\[ \text{NOTE:} \]
1. Minimum Spacing of screws shall not be less than 3d
2. Minimum Edge Distance of screws shall not be less than 3d
   (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)
3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.
4. Values may be increased 33% for wind or earthquake loads.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

bb: 4 Date: 11-05 By: Q.C. bb No.: 10439 SH: 9

S.E. CONSULTANTS, INC.
DEFINITIONS:

d  =  Nominal screw diameter

dw  =  Larger of Head or Washer Diameter, not larger than 1/2"

Factor of Safety = 3.0

Pas  =  allowable shear force per screw

Pns  =  nominal shear strength per screw

Pat  =  allowable tension force per screw

Pnt  =  nominal tension strength per screw

Pnot  =  pull-out per screw

Pnov  =  pull-over per screw

t1  =  thickness of member in contact with the screw head

t2  =  thickness of member not in contact with the screw head

Fu1  =  tensile strength of member in contact with screw

Fu2  =  tensile strength of member not in contact with screw

SECTION E4.3.1 CONNECTION SHEAR GOVERNED BY BASE METALS

\[
\begin{align*}
t2/t1 & = 1 & \text{when } t2/t1 \leq 1.0, \text{ use equations E4.3.1 to E4.3.3} \\
\text{Equation E4.3.1} & = 2590.4 \text{ l/screw} & \text{Equation E4.3.4} & = 2831.7 \text{ l/screw} \\
\text{Equation E4.3.2} & = 2831.7 \text{ l/screw} & \text{Equation E4.3.5} & = 2831.7 \text{ l/screw} \\
\text{Pns (smallest of the above)} & = 2590.4 \text{ l/screw} & \text{Pns (smallest of the above)} & = 2831.7 \text{ l/screw} \\
\text{Pas} & = \text{Allow. Shear per SCREW} = \frac{\text{Pns}}{\text{F.S. of 3.0}} = 863.5 \text{ l/screw} & \text{* Allow. Shear Capacity of Screw} & = 625.0 \text{ #/screw} \\
\text{* Based on "HILTI" w/ F.S. of 3.0}
\end{align*}
\]

SECTION E4.4, TENSION

\[
\begin{align*}
\text{Equation E4.4.1.1, Pullout Force, Pnot} & = 891.5 \text{ l/screw} \\
\text{Equation E4.4.2.1, Pullover Force, Pnov} & = 2276.0 \text{ l/screw} \\
\text{Pnt} & = \text{nominal tension strength per screw} = \text{lesser of } \text{Pnot} \& \text{Pnov} \\
\text{Pnt} & = 891.5 \text{ l/screw} \\
\text{Pat} & = \text{Allow. Tension per SCREW} = \frac{\text{Pnt}}{\text{F.S. of 3.0}} = 297.2 \text{ l/screw} \\
\end{align*}
\]

\[
\begin{align*}
\text{NOTE: Governed By Shear Capacity Of Screws} & = 625.0 \text{ l/screw} \\
\text{Allowable Tension per Screw} & = 297.2 \text{ l/screw} \\
\end{align*}
\]

NOTE:
1. Minimum Spacing of screws shall not be less than 3d
2. Minimum Edge Distance of screws shall not be less than 3d
   (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)
3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.
4. Values may be increased 33% for wind or earthquake loads.
SCREW CAPACITIES IN LIGHT GAUGE COLD FORMED STEEL

THE FOLLOWING IS BASED ON THE 2001 EDITION OF THE COLD-FORMED STEEL DESIGN MANUAL

DEFINITIONS:

d = Nominal screw diameter

dw = Larger of Head or Washer Diameter, not larger than 1/2"

Factor of Safety = 3.0

Pas = allowable shear force per screw

Pns = nominal shear strength per screw

Pat = allowable tension force per screw

Pnt = nominal tension strength per screw

Pnot = pull-out per screw

Pnov = pull-over per screw

t1 = thickness of member in contact with the screw head

t2 = thickness of member not in contact with the screw head

Fu1 = tensile strength of member in contact with screw

Fu2 = tensile strength of member not in contact with screw

SECTION E4.3.1 CONNECTION SHEAR

WHEN t2/t1 <= 1.0

Equation E4.3.1 Pns = 4.2(t2^3 * d) * 0.5 * Fu2

Equation E4.3.2 Pns = 2.7 t1 * d * Fu1

Equation E4.3.3 Pns = 2.7 t2 * d * Fu2

WHEN t2/t1 >= 2.5

Equation E4.3.4 Pns = 2.7 t1 * d * Fu1

Equation E4.3.5 Pns = 2.7 t2 * d * Fu2

Pas = ALLOW. SHEAR PER SCREW = Pns/ (F.S. of 3.0)

SECTION E4.4, TENSION

Equation E4.4.1.1, Pnot, Pull-out force = 0.85 t0c d Fu2

Equation E4.4.2.1, Pnov, Pull-over force = 1.5 t1 dw Fu1

Pnt = nominal tension strength per screw = lesser of Pnot & Pnov

Pat = ALLOW. TENSION PER SCREW = Pnt/ (F.S. of 3.0)

NOTES:
1. Minimum Spacing of screws shall not be less than 3d
2. Minimum Edge Distance of screws shall not be less than 3d
   (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)
3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.
4. Values may be increased 33% for wind or earthquake loads.

S.E. CONSULTANTS, INC.
Structures Basic Geometry

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof slope</td>
<td>0.125 ( /12 )</td>
</tr>
<tr>
<td>Length</td>
<td>18.17 feet</td>
</tr>
<tr>
<td>Tributary Width</td>
<td>9.00 feet</td>
</tr>
<tr>
<td>Total Tributary Area</td>
<td>163.50 Sq. Ft.</td>
</tr>
<tr>
<td>Total Live Load</td>
<td>20.00 psf</td>
</tr>
<tr>
<td>Dead Load</td>
<td>2.00 psf</td>
</tr>
<tr>
<td>Live Load (reduced)</td>
<td>20.00 psf</td>
</tr>
<tr>
<td>Snow Load</td>
<td>0.00 psf</td>
</tr>
<tr>
<td>Total Gravity Load</td>
<td>22.00 psf</td>
</tr>
<tr>
<td>Wind Uplift</td>
<td>14.01 psf</td>
</tr>
<tr>
<td>( w ) (dead) = D.L. \times L.)</td>
<td>18.00 lbs/ft.</td>
</tr>
<tr>
<td>( w ) (live) = L.L. \times L.)</td>
<td>180.00 lbs/ft.</td>
</tr>
<tr>
<td>( w ) (snow) = S.L. \times L.)</td>
<td>0.00 lbs/ft.</td>
</tr>
<tr>
<td>( w ) (wind) = W.L. \times L.)</td>
<td>126.09 lbs/ft.</td>
</tr>
<tr>
<td>( w ) (T.L.) = Uniform Gravity Load</td>
<td>198.00 lbs/ft.</td>
</tr>
<tr>
<td>( w ) (U.L.) = Uniform Uplift Load</td>
<td>108.09 lbs/ft.</td>
</tr>
</tbody>
</table>

**Moments Due to Gravity Loads**

\[
M \text{ (Simple)} = w \times (T.L.) \times L^2 / 8 = 8.168 \text{ ft-kips}
\]

**Moments Due to Wind Loads**

\[
M \text{ (Simple)} = w \times (W.L.) \times L^2 / 8 = 4.459 \text{ ft-kips}
\]

**Maximum End Reactions Due to Gravity Loads**

\[
R \text{ (Simple)} = w \times (T.L.) \times L / 2 = 1.799 \text{ kips}
\]

**Maximum End Reactions Due to Wind Uplift Loads**

\[
R \text{ (Simple)} = w \times (U.L.) \times L / 2 = 0.982 \text{ kips}
\]

**Minimum Number of Screws Required for End Support**

- Maximum capacity of #12 screw in shear = 0.618 kips
- Minimum number of screws required = 3 - #12 screws

See detail for actual number of screws required.

---

See CFS computer run for design size
Section Inputs

Material: A607 Class 1 Grade 55
No strength increase from cold work of forming.
Modulus of Elasticity, E (ksi) = 29500
Yield Strength, Fy (ksi) = 55
Tensile Strength, Fu (ksi) = 70
Warping Constant Override, Cw (in^6) = 0
Torsion Constant Override, J (in^4) = 0

Cee, Thickness 0.075 in
Placement of Part from Origin:
X to center of gravity = 0 in
Y to center of gravity = 0 in

Outside dimensions, Open shape
Length (in)
1 1.0000
2 2.5000
3 10.0000
4 2.5000
5 1.0000
Angle (deg)
1 270.000
2 180.000
3 90.000
4 0.000
5 -90.000
Radius (in)
1 0.13600
2 0.13600
3 0.13600
4 0.13600
5 0.13600
Web
1 None
2 Single
3 Single
4 Single
5 None
k Coef.
1 0.000
2 0.000
3 0.000
4 0.000
5 0.000
Hole Size (in)
1 0.000
2 0.000
3 0.000
4 0.000
5 0.000
Distance (in)
1 0.5000
2 1.2500
3 5.0000
4 1.2500
5 0.5000
## Analysis Inputs

### Members

<table>
<thead>
<tr>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Braced Flange</th>
<th>R (in)</th>
<th>ex (in)</th>
<th>ey (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.000</td>
<td>18.167</td>
<td>Top</td>
<td>0.4700</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

### Supports

<table>
<thead>
<tr>
<th>Type</th>
<th>Location (ft)</th>
<th>Bearing Location (in)</th>
<th>Fastened</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 XYT</td>
<td>0.333</td>
<td>1.000</td>
<td>No</td>
<td>1.0000</td>
</tr>
<tr>
<td>2 XYT</td>
<td>17.833</td>
<td>1.000</td>
<td>No</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

### Loading: Dead Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Distributed</td>
<td>90.000</td>
<td>0.000</td>
<td>18.167</td>
<td>-0.01800</td>
<td>-0.01800 k/ft</td>
</tr>
</tbody>
</table>

### Loading: Wind Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Distributed</td>
<td>90.000</td>
<td>0.000</td>
<td>18.167</td>
<td>0.12600</td>
<td>0.12600 k/ft</td>
</tr>
</tbody>
</table>

### Loading: Live Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Distributed</td>
<td>90.000</td>
<td>0.000</td>
<td>18.167</td>
<td>-0.18000</td>
<td>-0.18000 k/ft</td>
</tr>
</tbody>
</table>

**Load Combination:** ASD 2: D+L

**Specification:** 2004 North American Specification - US (ASD)

**Inflection Point Bracing:** No

**Loading Factor**

1. Beam Self Weight: 1.0000
2. Dead Load: 1.0000
3. Live Load: 1.0000
Load Combination: ASD 2: 6D+W
Inflection Point Bracing: Yes

<table>
<thead>
<tr>
<th>Loading</th>
<th>Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Beam Self Weight</td>
<td>0.6000</td>
</tr>
<tr>
<td>2 Dead Load</td>
<td>0.6000</td>
</tr>
<tr>
<td>3 Wind Load</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

Member Check - 2004 North American Specification - US (ASD)

Load Combination: ASD 2: D+L
Design Parameters at 9.083 ft:
\[
\begin{array}{llll}
L_x & 17.500 \text{ ft} & L_y & 17.500 \text{ ft} & L_t & 17.500 \text{ ft} \\
K_x & 1.0000 & K_y & 1.0000 & K_t & 1.0000 \\
\end{array}
\]

Section: 10x2.5-14 Gage Cee- unstiffened.sct
\[
\begin{array}{llll}
C_{bx} & 1.1366 & C_{by} & 1.0000 & ex & 0.0000 \text{ in} \\
C_{mx} & 1.0000 & C_{my} & 1.0000 & ey & 0.0000 \text{ in} \\
\end{array}
\]

Braced Flange: Top Moment Reduction, R: 0.4700

Loads:
\[
\begin{array}{lllll}
P & M_x & V_y & M_y & V_x \\
(k) & (k-ft) & (k) & (k-ft) & (k) \\
\end{array}
\]

<table>
<thead>
<tr>
<th>Total</th>
<th>Applied</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>8.2919</td>
</tr>
<tr>
<td>7.7286</td>
<td>0.0000</td>
<td>9.3885</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>3.9195</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>0.7618</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>6.4288</td>
</tr>
</tbody>
</table>

Effective section properties at applied loads:
\[
\begin{array}{llll}
A_e & 1.23016 \text{ in}^2 & I_{xe} & 17.424 \text{ in}^4 & I_{ye} & 1.021 \text{ in}^4 \\
& & S_{xe} & 3.4848 \text{ in}^3 & S_{ye} & 1.5502 \text{ in}^3 \\
& & S_{xe(t)} & 3.4848 \text{ in}^3 & S_{ye(l)} & 0.5545 \text{ in}^3 \\
\end{array}
\]

Interaction Equations
\[
\begin{array}{l}
\text{NAS Eq. C5.2.1-1} \quad (P, M_x, M_y) = 0.000 + 0.823 + 0.000 = 0.823 \leq 1.0 \\
\text{NAS Eq. C5.2.1-2} \quad (P, M_x, M_y) = 0.000 + 0.823 + 0.000 = 0.823 \leq 1.0 \\
\text{NAS Eq. C3.3.1-1} \quad (M_x, V_y) = Sqrt(0.678 + 0.000) = 0.823 \leq 1.0 \\
\text{NAS Eq. C3.3.1-1} \quad (M_y, V_x) = Sqrt(0.000 + 0.000) = 0.000 \leq 1.0 \\
\end{array}
\]

Member Check - 2004 North American Specification - US (ASD)

Load Combination: ASD 2: 6D+W
Design Parameters at 9.083 ft:
\[
\begin{array}{llll}
L_x & 17.500 \text{ ft} & L_y & 17.487 \text{ ft} & L_t & 17.487 \text{ ft} \\
K_x & 1.0000 & K_y & 1.0000 & K_t & 1.0000 \\
\end{array}
\]

Section: 10x2.5-14 Gage Cee- unstiffened.sct
\[
\begin{array}{llll}
C_{bx} & 1.1364 & C_{by} & 1.0000 & ex & 0.0000 \text{ in} \\
C_{mx} & 1.0000 & C_{my} & 1.0000 & ey & 0.0000 \text{ in} \\
\end{array}
\]

Braced Flange: Top Moment Reduction, R: 0.4700

Loads:
\[
\begin{array}{lllll}
P & M_x & V_y & M_y & V_x \\
(k) & (k-ft) & (k) & (k-ft) & (k) \\
\end{array}
\]

<table>
<thead>
<tr>
<th>Total</th>
<th>Applied</th>
<th>Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>8.2919</td>
</tr>
<tr>
<td>-4.3077</td>
<td>0.0000</td>
<td>4.4126</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>3.9195</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>0.7627</td>
</tr>
<tr>
<td>0.0000</td>
<td>0.0000</td>
<td>6.4288</td>
</tr>
</tbody>
</table>

Effective section properties at applied loads:
\[
\begin{array}{llll}
A_e & 1.23016 \text{ in}^2 & I_{xe} & 17.424 \text{ in}^4 & I_{ye} & 1.021 \text{ in}^4 \\
& & S_{xe} & 3.4848 \text{ in}^3 & S_{ye} & 1.5502 \text{ in}^3 \\
\end{array}
\]
Interaction Equations

\[ S_{x(e)}(b) = 3.4848 \text{ in}^3 \quad S_{y(e)}(r) = 0.5545 \text{ in}^3 \]

**NAS Eq. C5.2.1-1** \( (P, M_x, M_y) \)
\[ 0.000 + 0.976 + 0.000 = 0.976 \leq 1.0 \]

**NAS Eq. C5.2.1-2** \( (P, M_x, M_y) \)
\[ 0.000 + 0.976 + 0.000 = 0.976 \leq 1.0 \]

**NAS Eq. C3.3.1-1** \( (M_x, V_y) \)
\[ \sqrt{0.211 + 0.000} = 0.459 \leq 1.0 \]

**NAS Eq. C3.3.1-1** \( (M_y, V_x) \)
\[ \sqrt{0.000 + 0.000} = 0.000 \leq 1.0 \]
COLUMN LOADS - 2006 IBC

GRAVITY LOADS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Slope</td>
<td>0.125 /12</td>
</tr>
<tr>
<td>Length of Column</td>
<td>10 feet</td>
</tr>
<tr>
<td>Width of Building</td>
<td>18 feet</td>
</tr>
<tr>
<td>Average Bay Spacing</td>
<td>9.08333 feet</td>
</tr>
<tr>
<td>Number of Lateral Columns</td>
<td>2</td>
</tr>
<tr>
<td>Tributary Area to Column</td>
<td>81.75 sq. ft.</td>
</tr>
<tr>
<td>Dead Load</td>
<td>2.00 psf</td>
</tr>
<tr>
<td>Live Load</td>
<td>20.00 psf</td>
</tr>
<tr>
<td>Live Load (reduced)</td>
<td>20.00 psf</td>
</tr>
<tr>
<td>Total Dead Load to Column</td>
<td>0.164 kips</td>
</tr>
<tr>
<td>Total Live Load to Column</td>
<td>1.635 kips</td>
</tr>
<tr>
<td>Total Gravity Load to Column</td>
<td>1.799 kips</td>
</tr>
</tbody>
</table>

WIND LOAD

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind Uplift</td>
<td>14.01 psf</td>
</tr>
<tr>
<td>Dead Load</td>
<td>2.00 psf</td>
</tr>
<tr>
<td>Net Wind Uplift (W.L. - D.L.)</td>
<td>12.01 psf</td>
</tr>
<tr>
<td>Total Wind Load</td>
<td>1.145 kips</td>
</tr>
<tr>
<td>Total Wind Uplift to Column</td>
<td>0.982 kips</td>
</tr>
</tbody>
</table>

LATERAL WIND LOAD

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind, Lateral</td>
<td>14.01 psf</td>
</tr>
<tr>
<td>Height of Vertical Surface</td>
<td>1.23 feet</td>
</tr>
<tr>
<td>Wind Load per Column</td>
<td>0.076 kips</td>
</tr>
<tr>
<td>Wind Moment to Column</td>
<td>0.782 ft-kips</td>
</tr>
</tbody>
</table>

EARTHQUAKE LATERAL/LONGITUDINAL LOAD TO COLUMN - 2003 IBC

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEISMIC USE GROUP</td>
<td>I       (1616-3(1))</td>
</tr>
<tr>
<td>SEISMIC DESIGN CAT</td>
<td>D       (1616-3(1))</td>
</tr>
<tr>
<td>IMPORTANCE FACTOR</td>
<td>1.00</td>
</tr>
<tr>
<td>SEISMIC BASE SHEAR V</td>
<td>Cs*W    (1617-5)</td>
</tr>
<tr>
<td>R=Response modification factor</td>
<td>2.50    (1617-6) Cantilever Column Sys.</td>
</tr>
<tr>
<td>W=Effective Seismic Weight</td>
<td>0.164 kips (1617-5-1)</td>
</tr>
<tr>
<td>T = 0.02 * h n^(3/4)</td>
<td>0.112</td>
</tr>
<tr>
<td>Ss</td>
<td>178.7 %</td>
</tr>
<tr>
<td>S1</td>
<td>75.0 %</td>
</tr>
<tr>
<td>Smax = 1.787</td>
<td></td>
</tr>
<tr>
<td>Sds = 1.125</td>
<td></td>
</tr>
<tr>
<td>Sds = 2/3 * Smax</td>
<td>1.191</td>
</tr>
<tr>
<td>Smax = 2/3 * Smi</td>
<td>0.750</td>
</tr>
<tr>
<td>Fa</td>
<td>1.000   (1615-1-2(1))</td>
</tr>
<tr>
<td>Fv</td>
<td>1.500   (1615-1-2(2))</td>
</tr>
<tr>
<td>Cs = Sds / (R/I)</td>
<td>0.477   (Equation 16-35)</td>
</tr>
<tr>
<td>Cs = Sds / [(R/I)T]</td>
<td>2.667   (Equation 16-36)</td>
</tr>
<tr>
<td>Cs = 0.044 * Sds * I</td>
<td>0.033   (Equation 16-37)</td>
</tr>
<tr>
<td>Cs = 0.5*S1 / (R/I)</td>
<td>N/A     (Equation 16-38)</td>
</tr>
<tr>
<td>V (controls)</td>
<td>0.076 kips</td>
</tr>
<tr>
<td>Mh = V*h</td>
<td>0.779 ft-kips</td>
</tr>
</tbody>
</table>

Wind Governs See computer run for design size

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

S.E. CONSULTANTS, INC.
Section Inputs

Material: A607 Class 2 Grade 55
No strength increase from cold work of forming.
Modulus of Elasticity, E 29500 ksi
Yield Strength, Fy 55 ksi
Tensile Strength, Fu 65 ksi
Warping Constant Override, Cw 0 in^6
Torsion Constant Override, J 0 in^4

Tube, Thickness 0.0713 in (14 Gauge)
Placement of Part from Origin:
X to center of gravity 0 in
Y to center of gravity 0 in
Outside dimensions, Closed shape

<table>
<thead>
<tr>
<th>Length (in)</th>
<th>Angle (deg)</th>
<th>Radius (in)</th>
<th>Web Coef.</th>
<th>Hole Size (in)</th>
<th>Distance (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.0000</td>
<td>0.0000</td>
<td>0.13600</td>
<td>Single</td>
<td>0.0000</td>
</tr>
<tr>
<td>2</td>
<td>4.0000</td>
<td>90.0000</td>
<td>0.13600</td>
<td>Single</td>
<td>0.0000</td>
</tr>
<tr>
<td>3</td>
<td>4.0000</td>
<td>180.0000</td>
<td>0.13600</td>
<td>Single</td>
<td>0.0000</td>
</tr>
<tr>
<td>4</td>
<td>4.0000</td>
<td>-90.0000</td>
<td>0.13600</td>
<td>Single</td>
<td>0.0000</td>
</tr>
</tbody>
</table>
## Analysis Inputs

<table>
<thead>
<tr>
<th>Members</th>
<th>Section File</th>
<th>Revision Date and Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 TS 4x4x 14 ga.sct</td>
<td>6/12/2006 8:52:01 AM</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Braced</th>
<th>R (in)</th>
<th>ex (in)</th>
<th>ey (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10.500</td>
<td>None</td>
<td>0.4000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Supports</th>
<th>Type</th>
<th>Location (ft)</th>
<th>Bearing (in)</th>
<th>Fastened</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>XYTRx Ry</td>
<td>0.000</td>
<td>5.000</td>
<td>No</td>
<td>2.0000</td>
</tr>
<tr>
<td>2</td>
<td>XT</td>
<td>10.000</td>
<td>1.000</td>
<td>No</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

### Loading: Dead Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Axial</td>
<td>NA</td>
<td>0.000</td>
<td>10.000</td>
<td>0.1640</td>
<td>0.1640 k</td>
</tr>
<tr>
<td>2 Concentrated</td>
<td>90.000</td>
<td>9.958</td>
<td>NA</td>
<td>0.0000</td>
<td>NA k</td>
</tr>
</tbody>
</table>

Bearing Length 1.000 in

### Loading: Roof Live Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Axial</td>
<td>NA</td>
<td>0.000</td>
<td>10.000</td>
<td>1.6350</td>
<td>1.6350 k</td>
</tr>
<tr>
<td>2 Concentrated</td>
<td>90.000</td>
<td>9.958</td>
<td>NA</td>
<td>0.0000</td>
<td>NA k</td>
</tr>
</tbody>
</table>

Bearing Length 1.000 in

### Loading: Wind Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Axial</td>
<td>NA</td>
<td>0.000</td>
<td>10.000</td>
<td>-1.1450</td>
<td>-1.1450 k</td>
</tr>
<tr>
<td>2 Concentrated</td>
<td>90.000</td>
<td>9.958</td>
<td>NA</td>
<td>0.0780</td>
<td>NA k</td>
</tr>
</tbody>
</table>

Bearing Length 1.000 in

| 3 Concentrated | 90.000   | 9.958           | NA            | 0.0000          | NA k          |

Bearing Length 1.000 in
Loading: Earthquake Load

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concentrated</td>
<td>90.000</td>
<td>9.958</td>
<td>NA</td>
<td>0.0780</td>
<td>NA k</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bearing Length</td>
<td>1.000 in</td>
</tr>
</tbody>
</table>

Loading: UNBAL

<table>
<thead>
<tr>
<th>Type</th>
<th>Angle (deg)</th>
<th>Start Loc. (ft)</th>
<th>End Loc. (ft)</th>
<th>Start Magnitude</th>
<th>End Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial</td>
<td>NA</td>
<td>0.000</td>
<td>10.000</td>
<td>0.0000</td>
<td>0.0000 k</td>
</tr>
<tr>
<td>Concentrated</td>
<td>90.000</td>
<td>9.958</td>
<td>NA</td>
<td>0.0000</td>
<td>NA k</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bearing Length</td>
<td>1.000 in</td>
</tr>
</tbody>
</table>

Load Combination: ASD 2: D+L+R
Inflection Point Bracing: Yes
Loading Factor
1 Dead Load 1.0000
2 Roof Live Load 1.0000

Load Combination: ASD 3: .6D+W
Inflection Point Bracing: No
Loading Factor
1 Dead Load 0.6000
2 Wind Load 1.0000

Load Combination: ASD 4: D + 0.7E
Inflection Point Bracing: No
Loading Factor
1 Beam Self Weight 1.0000
2 Dead Load 1.0000
3 Earthquake Load 0.7000

Load Combination: DL+UNBAL
Inflection Point Bracing: No
Loading Factor
1 Beam Self Weight 1.0000
2 Dead Load 1.0000
3 UNBAL 1.0000
Member Check - 2004 North American Specification - US (ASD)

Load Combination: ASD 2: D+L+R
Design Parameters at 0.000 ft, Right side:
Lx  10.500 ft  Ly  10.000 ft  Lt  10.000 ft
Kx  2.0000    Ky  2.0000    Kt  2.0000

Section: TS 4x4x 14 ga.sct
Cbx  1.0000  Cby  1.0000  ex  0.0000 in
Cmx  1.0000  Cmy  1.0000  ey  0.0000 in
Braced Flange: None  Moment Reduction, R: 0.0000

Loads:

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>Mx</th>
<th>Vy</th>
<th>My</th>
<th>Vx</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>1.799</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
<tr>
<td>Applied</td>
<td>1.799</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

Effective section properties at applied loads:

Ae  1.09945 in^2
Ixe  2.7997 in^4
Iye  2.7997 in^4
Sxe(t)  1.3998 in^3
Sxe(b)  1.3998 in^3
Sye(l)  1.3998 in^3
Sye(r)  1.3998 in^3

Interaction Equations

NAS Eq. C5.2.1-1  (P, Mx, My)  0.288 + 0.000 + 0.000 = 0.288 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.074 + 0.000 + 0.000 = 0.074 <= 1.0
NAS Eq. C3.3.1-1  (Mx, Vy)  Sqrt(0.000 + 0.000) = 0.000 <= 1.0
NAS Eq. C3.3.1-1  (My, Vx)  Sqrt(0.000 + 0.000) = 0.000 <= 1.0

Member Check - 2004 North American Specification - US (ASD)

Load Combination: ASD 3: .6D+W
Design Parameters at 0.000 ft, Right side:
Lx  10.500 ft  Ly  10.000 ft  Lt  10.000 ft
Kx  2.0000    Ky  2.0000    Kt  2.0000

Section: TS 4x4x 14 ga.sct
Cbx  1.0000  Cby  1.0000  ex  0.0000 in
Cmx  1.0000  Cmy  1.0000  ey  0.0000 in
Braced Flange: None  Moment Reduction, R: 0.0000

Loads:

<table>
<thead>
<tr>
<th></th>
<th>P</th>
<th>Mx</th>
<th>Vy</th>
<th>My</th>
<th>Vx</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>-1.047</td>
<td>0.7800</td>
<td>-0.078</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
<tr>
<td>Applied</td>
<td>-1.047</td>
<td>0.7800</td>
<td>-0.078</td>
<td>0.0000</td>
<td>0.0000</td>
</tr>
</tbody>
</table>

Effective section properties at applied loads:

Ae  1.09945 in^2
Ixe  2.7997 in^4
Iye  2.7997 in^4
Sxe(t)  1.3998 in^3
Sxe(b)  1.3998 in^3
Sye(l)  1.3998 in^3
Sye(r)  1.3998 in^3

Interaction Equations

NAS Eq. C5.1.1-1  (Mx, My, T)  0.203 + 0.000 + 0.029 = 0.232 <= 1.0
NAS Eq. C5.1.1-2  (Mx, My, T)  0.244 + 0.000 - 0.029 = 0.215 <= 1.0
NAS Eq. C3.3.1-1  (Mx, Vy)  Sqrt(0.060 + 0.000) = 0.244 <= 1.0
NAS Eq. C3.3.1-1  (My, Vx)  Sqrt(0.000 + 0.000) = 0.000 <= 1.0
Member Check - 2004 North American Specification - US (ASD)

Load Combination: ASD 4: D + 0.7E
Design Parameters at 0.000 ft, Right side:

<table>
<thead>
<tr>
<th>Lx</th>
<th>Ly</th>
<th>Lt</th>
<th>Kx</th>
<th>Ky</th>
<th>Kt</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.500 ft</td>
<td>10.000 ft</td>
<td>10.000 ft</td>
<td>2.0000</td>
<td>2.0000</td>
<td>2.0000</td>
</tr>
</tbody>
</table>

Section: TS 4x4x 14 ga.sct
Cb 1.0000  Cby 1.0000  ex 0.0000 in
Cmx 1.0000  Cmy 1.0000  ey 0.0000 in
Braced Flange: None  Moment Reduction, R: 0.0000

Loads:

<table>
<thead>
<tr>
<th>Loads</th>
<th>P(k)</th>
<th>Mx(k-ft)</th>
<th>Vy(k)</th>
<th>My(k-ft)</th>
<th>Vx(k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>0.164</td>
<td>0.3399</td>
<td>-0.015</td>
<td>0.0000</td>
<td>0.000</td>
</tr>
<tr>
<td>Applied</td>
<td>0.164</td>
<td>0.3399</td>
<td>-0.015</td>
<td>0.0000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

Effective section properties at applied loads:

| Ae 1.09945 in^2 | Ixe 2.7997 in^4 | Iye 2.7997 in^4 | Sxe(t) 1.3998 in^3 | Sye(l) 1.3998 in^3 | Sxe(b) 1.3998 in^3 | Sye(r) 1.3998 in^3 |

Interaction Equations

NAS Eq. C5.2.1-1  \( P, M_x, M_y \)  0.026 + 0.109 + 0.000 = 0.135 <= 1.0
NAS Eq. C5.2.1-2  \( P, M_x, M_y \)  0.007 + 0.106 + 0.000 = 0.113 <= 1.0
NAS Eq. C3.3.1-1  \( M_x, V_y \)  Sqrt(0.011 + 0.000) = 0.106 <= 1.0
NAS Eq. C3.3.1-1  \( M_y, V_x \)  Sqrt(0.000 + 0.000) = 0.000 <= 1.0
ECCENTRIC LOADS ON FASTENER GROUPS

\[ a \]

\[ b \]

\[ n = \text{No. of fasteners in vertical row} \]
\[ m = \text{No. of fasteners in horizontal row} \]
\[ P = \text{applied load} \]
\[ rv = \text{allowable shear/bearing for one fastener} \]
\[ lp = \text{polar moment of inertia} = I_{xx} + I_{yy} \]
\[ I_{xx} = \left[ nb^2(n^2-1)/12 \right] \times \text{no. of vertical rows} \]
\[ I_{yy} = \left[ mb^2(m^2-1)/12 \right] \times \text{no. of horizontal rows} \]
\[ f_1 = \frac{P}{(mn)} \]
\[ f_2 = \text{(moment)}xb/2lp \]
\[ f_3 = \text{(moment)x(n-1)b/2lp} \]
\[ fr = \text{actual load on bolts} = [(f_3)^2 + (f_1 + f_2)^2]^{.5} \]

Beam to Column Connection is Designed for Total Load and Moment

Total Gravity Load = 1.799 kips
Total Gravity Moment = 0.000 k-ft
Total Uplift Load = 1.145 kips
Total Uplift Moment = 0.782 k-ft

Total Seismic Load = 0.164 kips
Total Seismic Moment = 0.779 k-ft

Connection Design (half load to each side)

\[ t, \text{Thickness of material} = 0.0747 \text{ inches} \]
\[ F_{u}, \text{of joined materials} = 70 \text{ ksi} \]
Screw Size = No. 12

No. of vertical rows = 2
No. of fasteners in vertical row = 4
No. of horizontal rows = 4
No. of fasteners in horizontal row = 2
Vertical Dimension = 1.833 inches
Horizontal Dimension = 4 inches
\[ I_{xx} = 33.599 \]
\[ I_{yy} = 32 \]
\[ lp = 65.599 \]

Use:
Type of Screws: No. 12
Horizontal Spacing: 4 inches
Vertical Spacing: 1.833 inches
Number of Screws: 8

Gravity | Seismic | Wind
--- | --- | ---
0.112 kips | 0.010 kips | 0.072 kips
0.000 kips | 0.143 kips | 0.143 kips
0.000 kips | 0.196 kips | 0.197 kips
0.112 kips | 0.248 kIPS | 0.291 kips

Allowable Shear = 0.625 kips

rv, Allowable w/ 33\% increase = 0.625 kips

Increase Aloud (y, n) N

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

S.E. CONSULTANTS, INC.
Base Plate Design

Base Plate to Footing Connection is Designed for Total Load and Moment

- Total Gravity Load = 1.80 kips
- Total Gravity Moment = 0.00 k-ft
- Total Uplift Load = -1.15 kips
- Total Uplift Moment = 0.78 k-ft
- Total Seismic Load = 0.16 kips
- Total Seismic Moment = 0.78 k-ft

Tension in Bolts

- Due to Gravity = -0.45 kips
- Due to Uplift = 0.81 kips
- Due to Seismic = 0.48 kips

Wind Controls

Checking Plate

- Bending Moment = 4.04 k-in
- t, Thickness of material = 0.50 in
- Fy, of base plate = 50.0 ksi
- Sx, Section Modulus = 0.50 in^3
- Fb = 37.5 ksi
- fb = 8.1 ksi o.k.
Anchor Calculations

Anchor Designer for ACI 318 (Version 4.2.0.1)

Job Name: Anchor Design  Date/Time: 11/16/2009 11:29:58 AM

1) Input

Calculation Method: ACI 318 Appendix D For Cracked Concrete
Calculation Type: Analysis

a) Layout

Anchor: 3/4" Titen HD  Number of Anchors: 1
Embedment Depth: 5.5 in
Built-up Grout Pads: No

Anchor Layout Dimensions:
- \( c_{x1} \): 6 in
- \( c_{x2} \): 6 in
- \( c_{y1} \): 6 in
- \( c_{y2} \): 6 in
- \( b_{x1} \): 1.5 in
- \( b_{x2} \): 1.5 in
- \( b_{y1} \): 1.5 in
- \( b_{y2} \): 1.5 in

b) Base Material

about:blank
Concrete: Normal weight
Cracked Concrete: Yes
Condition: B tension and shear
Thickness, $h$: 24 in
Supplementary edge reinforcement: No

c) Factored Loads
Load factor source: ACI 318 Appendix C
$N_{ua}$: 4552 lb
$V_{uay}$: 0 lb
$M_{uy}$: 0 lb*ft
$e_x$: 0 in
$e_y$: 0 in
Moderate/high seismic risk or intermediate/high design category: No
Apply entire shear load at front row for breakout: No

d) Anchor Parameters
From C-SAS-2009:
Anchor Model = THD75 $d_o = 0.75$ in
Category = 1 $h_{ef} = 4.22$ in
$h_{min} = 8.75$ in $c_{ac} = 6.375$ in
$c_{min} = 1.75$ in $s_{min} = 3$ in
Ductile: No

2) Tension Force on Each Individual Anchor
Anchor #1 $N_{ua1} = 4552.00$ lb
Sum of Anchor Tension $\Sigma N_{ua} = 4552.00$ lb
$a_x = 0.00$ in
$a_y = 0.00$ in
$e_{Nx} = 0.00$ in
$e_{Ny} = 0.00$ in

3) Shear Force on Each Individual Anchor
Resultant shear forces in each anchor:
Anchor #1 $V_{uax} = 0.00$ lb ($V_{uax1} = 0.00$ lb, $V_{uay1} = 0.00$ lb)
Sum of Anchor Shear $\Sigma V_{uax} = 0.00$ lb, $\Sigma V_{uay} = 0.00$ lb
$e_{Vx} = 0.00$ in
\( e'_{V_y} = 0.00 \text{ in} \)

**4) Steel Strength of Anchor in Tension [Sec. D.5.1]**

\( N_{sa} = nA_{se}f_{uta} \) [Eq. D-3]

Number of anchors acting in tension, \( n = 1 \)

\( N_{sa} = 45540 \text{ lb (for a single anchor)} \) [C-SAS-2009]

\( \phi = 0.70 \) [D.4.5]

\( \phi N_{sa} = 31878.00 \text{ lb (for a single anchor)} \)

**5) Concrete Breakout Strength of Anchor in Tension [Sec. D.5.2]**

\( N_{cb} = A_{Nco}/A_{Nc}^\Psi_{ed,N}^\Psi_{c,N}^\Psi_{cp,N}^N_b \) [Eq. D-4]

Number of influencing edges = 4

\( h_{ef} \) (adjusted for edges per D.5.2.3) = 4.000 in

\( A_{Nco} = 144.00 \text{ in}^2 \) [Eq. D-6]

\( A_{Nc} = 144.00 \text{ in}^2 \)

Smallest edge distance, \( c_{a,min} = 6.00 \text{ in} \)

\( \Psi_{ed,N} = 1.0000 \) [Eq. D-10 or D-11]

Note: Cracking shall be controlled per D.5.2.6

\( \Psi_{c,N} = 1.0000 \) [Sec. D.5.2.6]

\( \Psi_{cp,N} = 1.0000 \) [Eq. D-12 or D-13]

\( N_b = k_c \sqrt{f_c \cdot h_{ef}^{1.5}} = 6800.00 \text{ lb (Eq. D-7)} \)

\( k_c = 17 \) [Sec. D.5.2.6]

\( N_{cb} = 6800.00 \text{ lb (Eq. D-4)} \)

\( \phi = 0.75 \) [D.4.5]

\( \phi N_{cb} = 5100.00 \text{ lb (for a single anchor)} \)

**6) Pullout Strength of Anchor in Tension [Sec. D.5.3]**

\( N_{pn} = \Psi_{c,p} N_p \)

\( N_{pn} = 6070 \text{ lb (f_c/2,500 psi)}^{0.5} = 6070.00 \text{ lb} \)

\( \phi = 0.75 \)

\( \phi N_{pn} = 4552.50 \text{ lb} \)

**7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]**

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge, \( c_{a1} < 0.4h_{ef} \). Not applicable in this case.

**8) Steel Strength of Anchor in Shear [Sec. D.6.1]**
\( V_{sa} = 16840.00 \text{ lb (for a single anchor) [C-SAS-2009]} \)
\( \phi = 0.65 [D.4.5] \)
\( \phi V_{sa} = 10946.00 \text{ lb (for a single anchor)} \)

9) Concrete Breakout Strength of Anchor in Shear [Sec D.6.2]

Case 1: Anchor checked against total shear load
In x-direction...
\( V_{cbx} = A_{vcx}/A_{vcox} \Psi_{ed,v} \Psi_{c,v} V_{bx} \) [Eq. D-21]
\( c_{a1} = 6.00 \text{ in} \)
\( A_{vcx} = 108.00 \text{ in}^2 \)
\( A_{vcox} = 162.00 \text{ in}^2 \) [Eq. D-23]
\( \Psi_{ed,v} = 0.9000 \) [Eq. D-27 or D-28]
\( \Psi_{c,v} = 1.4000 \) [Sec. D.6.2.7]
\( V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \) [Eq. D-24]
\( l_e = 4.22 \text{ in} \)
\( V_{bx} = 6293.26 \text{ lb} \)
\( V_{cbx} = 5286.34 \text{ lb} \) [Eq. D-21]
\( \phi = 0.75 \)
\( \phi V_{cbx} = 3964.75 \text{ lb (for a single anchor)} \)

In y-direction...
\( V_{cby} = A_{vcy}/A_{vcoy} \Psi_{ed,v} \Psi_{c,v} V_{by} \) [Eq. D-21]
\( c_{a1} = 6.00 \text{ in} \)
\( A_{vcy} = 108.00 \text{ in}^2 \)
\( A_{vcoy} = 162.00 \text{ in}^2 \) [Eq. D-23]
\( \Psi_{ed,v} = 0.9000 \) [Eq. D-27 or D-28]
\( \Psi_{c,v} = 1.4000 \) [Sec. D.6.2.7]
\( V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \) [Eq. D-24]
\( l_e = 4.22 \text{ in} \)
\( V_{by} = 6293.26 \text{ lb} \)
\( V_{cby} = 5286.34 \text{ lb} \) [Eq. D-21]
\( \phi = 0.75 \)
\( \phi V_{cby} = 3964.75 \text{ lb (for a single anchor)} \)

Case 2: This case does not apply to single anchor layout
Case 3: Anchor checked for parallel to edge condition

Check anchors at $c_{1x}$ edge

$$V_{cbx} = A_{vcx}/A_{vcx} \Psi_{ed,V} \Psi_{c,V} V_{bx} \ [Eq. \ D-21]$$

$c_{a1} = 6.00 \text{ in}$

$A_{vcx} = 108.00 \text{ in}^2$

$A_{vcx} = 162.00 \text{ in}^2 \ [Eq. \ D-23]$

$\Psi_{ed,V} = 1.0000 \ [Sec. \ D.6.2.1(c)]$

$\Psi_{c,V} = 1.4000 \ [Sec. \ D.6.2.7]$

$V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \ [Eq. \ D-24]$

$l_e = 4.22 \text{ in}$

$V_{bx} = 6293.26 \text{ lb}$

$V_{cbx} = 5873.71 \text{ lb} \ [Eq. \ D-21]$

$V_{cby} = 2 * V_{cbx} \ [Sec. \ D.6.2.1(c)]$

$V_{cby} = 11747.42 \text{ lb}$

$\phi = 0.75$

$\phi V_{cby} = 8810.57 \text{ lb} \ (for \ a \ single \ anchor)$

Check anchors at $c_{y1}$ edge

$$V_{cby} = A_{vcy}/A_{vcy} \Psi_{ed,V} \Psi_{c,V} V_{by} \ [Eq. \ D-21]$$

$c_{a1} = 6.00 \text{ in}$

$A_{vcy} = 108.00 \text{ in}^2$

$A_{vcy} = 162.00 \text{ in}^2 \ [Eq. \ D-23]$

$\Psi_{ed,V} = 1.0000 \ [Sec. \ D.6.2.1(c)]$

$\Psi_{c,V} = 1.4000 \ [Sec. \ D.6.2.7]$

$V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \ [Eq. \ D-24]$

$l_e = 4.22 \text{ in}$

$V_{by} = 6293.26 \text{ lb}$

$V_{cby} = 5873.71 \text{ lb} \ [Eq. \ D-21]$

$V_{cbx} = 2 * V_{cby} \ [Sec. \ D.6.2.1(c)]$

$V_{cbx} = 11747.42 \text{ lb}$

$\phi = 0.75$

$\phi V_{cbx} = 8810.57 \text{ lb} \ (for \ a \ single \ anchor)$
Check anchors at \( c_{x2} \) edge

\[ V_{cbx} = A_{vcx}/A_{vcox} \Psi_{ed,v} \Psi_{c,v} V_{bx} \text{ [Eq. D-21]} \]

\( c_{a1} = 6.00 \text{ in} \)

\( A_{vcx} = 108.00 \text{ in}^2 \)

\( A_{vcox} = 162.00 \text{ in}^2 \text{ [Eq. D-23]} \)

\( \Psi_{ed,v} = 1.0000 \text{ [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]} \)

\( \Psi_{c,v} = 1.4000 \text{ [Sec. D.6.2.7]} \)

\( V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]} \)

\( l_e = 4.22 \text{ in} \)

\( V_{bx} = 6293.26 \text{ lb} \)

\( V_{cbx} = 5873.71 \text{ lb} \text{ [Eq. D-21]} \)

\( V_{cby} = 2 * V_{cbx} \text{ [Sec. D.6.2.1(c)]} \)

\( V_{cby} = 11747.42 \text{ lb} \)

\( \phi = 0.75 \)

\( \phi V_{cby} = 8810.57 \text{ lb (for a single anchor)} \)

Check anchors at \( c_{y2} \) edge

\[ V_{cby} = A_{vcy}/A_{vcoy} \Psi_{ed,v} \Psi_{c,v} V_{by} \text{ [Eq. D-21]} \]

\( c_{a1} = 6.00 \text{ in} \)

\( A_{vcy} = 108.00 \text{ in}^2 \)

\( A_{vcoy} = 162.00 \text{ in}^2 \text{ [Eq. D-23]} \)

\( \Psi_{ed,v} = 1.0000 \text{ [Sec. D.6.2.1(c)]} \)

\( \Psi_{c,v} = 1.4000 \text{ [Sec. D.6.2.7]} \)

\( V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]} \)

\( l_e = 4.22 \text{ in} \)

\( V_{by} = 6293.26 \text{ lb} \)

\( V_{cby} = 5873.71 \text{ lb} \text{ [Eq. D-21]} \)

\( V_{cbx} = 2 * V_{cby} \text{ [Sec. D.6.2.1(c)]} \)

\( V_{cbx} = 11747.42 \text{ lb} \)

\( \phi = 0.75 \)

\( \phi V_{cbx} = 8810.57 \text{ lb (for a single anchor)} \)

10) Concrete Pryout Strength of Anchor in Shear [Sec. D.6.3]
\[ V_{cp} = k_{cp} N_{cb} \]  [Eq. D-29]
\[ k_{cp} = 2 \]  [Sec. D.6.3.1]
\[ N_{cb} = 6800.00 \text{ lb (from Section (5) of calculations)} \]
\[ V_{cp} = 13600.00 \text{ lb} \]
\[ \phi = 0.75 \]  [D.4.5]
\[ \phi V_{cp} = 10200.00 \text{ lb (for a single anchor)} \]

11) **Check Demand/Capacity Ratios [Sec. D.7]**

**Tension**
- Steel : 0.1428
- Breakout : 0.8925
- Pullout : 0.9999
- Sidetface Blowout : N/A

**Shear**
- Steel : 0.0000
- Breakout (case 1) : 0.0000
- Breakout (case 2) : N/A
- Breakout (case 3) : 0.0000
- Pryout : 0.0000

\[ V_{Max}(0) \leq 0.2 \text{ and } T_{Max}(1) \leq 1.0 \]  [Sec D.7.1]

Interaction check: PASS

**Use 3/4" diameter Titen HD anchor(s) with 5.5 in. embedment**
NON CONSTRAINED POLE TYPE FOOTING PER INTERNATIONAL BUILDING CODE

DESCRIPTION OR LOCATION OF FOOTING: COLUMNS W/ D.L. & WIND
ALLOWABLE SOIL BEARING = 1500 PSF
ALLOWABLE LATERAL BEARING = 150 PSF/FT

CHECK LATERAL BEARING CAPACITY

\[
A = 4.36 \frac{h}{(1 + \left[ 1 + \frac{1}{A} \right]^{1/2})}
\]
\[
d = 2.34 \frac{P}{A}
\]

\[
A = \frac{S1 \times b}{y}
\]

Load Due To Wind or Seismic? (Y/N) = Y
M. Moment in Column = 0.782 ft. kips
P. Lateral load = M / h = 78 lbs.
h Dist. in feet from ground to "P" = 10 feet
d Depth of Concrete Footing = 3.00 feet
b Diameter of round footing = 2.00 feet
Allowable Lateral Bearing = 150 lbs/sq.ft.

IBC, Section 1804.3.1
Will structure be adversely affected by a
1/2" motion at the ground surface? (Y/N) = N
2 x Allowable Lateral Bearing, per sec. 1804.3.1 = 300.0 lbs/sq.ft.
S1 Allowable Lat. brg. based on 1/3 of "d" (limited to "d" = 12 feet) = 300.0 lbs/sq.ft.
Allowable Lat. Brg. w/ 33% Increase per footnote D = 400.0 lbs/sq.ft.

A = 0.229

REQUIRED "d" = 1.70 FEET < 3 FEET O.K.

Diameter of Pier = 24.00 inches
Depth of Concrete Footing = 3.00 feet

CHECK STRESSES OF CONCRETE PIER

\[f_c, \text{Compressive Strength of Concrete} = 2500 \text{ psi}\]
S, Section Modulus of the Round Concrete Pier = 1357 in.^3
f, Stress in Plain Concrete Pier = M/S = 7 psi.
ft, Allowable Tension = 1.6 \times \text{SQRT.} f_c = 80 psi.
ft, Increased for Wind or Seismic = 106.67 psi.

CHECK VERTICAL BEARING CAPACITY

P(t.l.), Gravity Load = 1.799 kips
Allowable Bearing at Min. Depth = 1500 psf
Allowable Bearing w/ Increase for Depth = 1500 psf

Allowable Bearing Value at Bottom of Pier = 1500 psf
A, Bearing Area at Bottom of Pier = 3.142 sq. ft.
qs, Gravity Load Soil Pressure = 572.639 psf.

O.K. - WITHOUT SKIN FRICTION
Allow. Skin Friction = Allow. Bearing / 6 = 250.0 psf
Skin Surface (Excluding top one foot) = 12.57 sq. ft.
Skin Friction = 3.14 kips
NOT USED

CHECK UPLIFT CAPACITY, IF APPLICABLE

F, Total Wind Uplift Load = 1.145 kips
Allowable Bearing Value at 1' depth of Pier = 1500 psf
Allow. Skin Friction = Allow. Bearing / 6 = 250.0 psf
Skin Surface (Excluding top one foot) = 12.57 sq. ft.
Skin Friction = 3.1 kips
Remove Earth Loads from Uplift Capacity = N
Dead Load of Pier = 1.4 kips
Dead Load of Earth = 3.4 kips

Total Uplift Resistance = 4.5 kips
O.K.
Safety Factor = 3.94

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

bb: 4 past January Date: 11-09 By: LCS bb No.: 1304-09 SH.: 32
S.E. CONSULTANTS, INC.
SPREAD FOOTING W/ COLUMN CENTERED IN FOOTING

DEAD LOAD MOMENT RESISTANCE AGAINST OVERTURNING SOIL PRESSURE

- Applied Overturning Moment = 0.782 ft-kips
- D.L. of Structure = 0.164 kips
- Length of Footing = 2 feet
- Width of Footing = 2 feet
- Thickness of Footing = 24 inches
- Dead Load of Footing = 1.2 kips
- Volume of Earth in 30 degree Cone = 9.2 cu. ft.
- Slab on Grade (y or n) = N
- Length of Turndown = 0 ft
- Depth of Turndown = 0 ft
- Dead Load of Turndown = 0.00 kips
- Dead Load of Earth = 0.925 kips
- Total D.L. = 2.289 kips
- D.L. Resisting Moment = 2.29 ft-kips
- Required Factor of Safety = 1.5
- F. of S. Against Overturning = 2.927
- Allowable Soil Pressure = 1500 psf
- Increases for Width OR Depth = 0
- Allowable S. P. w/ Increases = 1500
- Loads Due to Wind or Seismic? (Y/N) = N
- S.P. w/ 33% Increase = N/A
- Applied Overturning Moment = 0.782 ft-kips
- D.L. of Structure = 0.164 kips
- Footing D.L. = 1.2 kips
- Total Vert. Load = 1.364 kips
- e' = (M - OTM) / Total Vert. Load = -0.105 feet
- e = L/2 - e' = 0.333 feet
- RESULTANT IN MIDDLE 1/3
- S.P. = P/A +/- M/S
- Soil Pressure = 927.5 psf

DEAD LOAD RESISTANCE AGAINST UPLIFT

- Applied Uplift Load - Total Wind = 0.78 kips
- D.L. of Structure = 0.164 kips
- Volume of Earth in 30 degree Cone = 9.249 cu. ft.
- Dead Load of Earth = 0.925 kips
- Dead Load of Footing = 1.20 kips
- Dead Load of Earth = 0.925 kips
- Total Dead Load Resistance = 2.289 kips
- F. of S. Against Uplift = 2.927
- f'c = 2500 psi
- f'y = 4000 psi
- d = 21 inches
- M, Moment = 0.782 ft-kips
- Mu, Ultimate Moment = 1.7 M = 1.329 ft-kips
- Assume a one foot strip
- Mu, = 1.7 M / Width of Footing = 0.665 ft-kips
- Beta = 0.85
- Phi = 0.85
- rho, balanced = 0.031
- m = 18.824
- 1.773 psi = Rn
- rho, calc. = 0.0000
- rho, min. = 0.0000
- rho, Req'd. = 0.0000
- Req'd. As = 0.011 sq. in. / ft.
- Total As Req'd. = 0.022 sq. in.
- USE: 3 - # 5 As = 0.920 sq.in.

FOOTING REINFORCING

LENGTH OF FOOTING: 2 feet
WIDTH OF FOOTING: 2 feet
THICKNESS OF FOOTING: 24 inches
REINFORCING: 3 - #5 Bars
Use #4's @ 12" o.c. lateral

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

S.E. CONSULTANTS, INC.