STRUCTURAL CALCULATIONS

FOR

TIA ANA'S SUSTAINABLE DEVELOPMENT

TIA ANA'S

JUNE 22, 2016

PREPARED BY:

DANIELLE RUSTAGI

NOT FOR CONSTRUCTION, TO BE REVIEWED AND APPROVED BY IN COUNTRY ENGINEER.
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PROJECT DESCRIPTION

Tia Ana’s Sustainable Development is located in the El Salvadorian countryside, just outside of the capital city San Salvador. Currently, Tia Ana’s is located in the city of San Salvador, where there is little room for the children and a significant amount of violence and instability. In order to make the relocation of the orphanage feasible financially, a portion of the countryside landscape will contain a small hotel and numerous small dwellings nested in the tropical surroundings. By relocating the orphanage to the countryside, the children housed by Tia Ana will experience a safer and more stable environment and this sustainable development would be the home of a new kind of tourist destination in El Salvador.

CONSTRUCTION SEQUENCE

The building materials used for each structure are identical, cast in place concrete, CMU masonry blocks, architectural masonry blocks, reinforcing steel bars, steel tube framing, and corrugated metal decking. With these six main materials, I was able to develop a consistent framing system from one structure to the next. After preparing the site for construction, each structure begins with a poured concrete foundation consisting of isolated (pad) footings and slab on grade. In order to connect the concrete floor slab to the masonry walls, reinforcing steel must be embedded into the slab and hooked so the CMU blocks can be set with reinforcement in the proper cells. The structural and architectural masonry walls require fully grouted cells and reinforcement to resist the lateral loads that come with being in an area of high seismic activity. Once the walls are constructed, anchor bolts embedded in the top course of the CMU wall will connect tube steel columns to the walls below. From these columns, tube steel girders and beams can be welded to form the roof framing. The final step in the construction sequence is to fasten corrugated metal decking to the roof framing system. This sequence can be used to construct buildings as small as the Casita’s where the total square footage is only 620 sqft all the way up to structure as large as Tia Ana’s where the building footprint is just under 8,000 sqft.

DESIGN CRITERIA

When determining the lateral loads on the structure, I assumed seismic would govern. I was able to make this assumption because the site is located on the side of an active volcano in a high seismic zone, the design of each structure is such that architectural masonry walls provide enough openings to prevent a buildup of wind pressure on the structural walls and the surrounding trees and other natural features protect the buildings from high winds. Using the USGS Worldwide Beta, I found the Sds and Sd1 values for the site were 1.72 and 0.96 respectively to produce a Cs values of 0.344. These values are comparable to places in California with high seismic activity such as Los Angeles and San Francisco, both of which sit on major known faults. For this reason I can justify using the same building codes as those used in California and the United States for design.

These codes are as follows: 2012 International Building Code, 2010 American Society of Civil Engineers 7-10, American Institute of Steel Construction (14th Edition), The 2013 Masonry Specification 402, and 2014 American Concrete Institute 318.
ROOF SECTION:
(N.T.S)

IBC 2015

Roof live load: 20 psf

Dead load takeoff:

24 ga. shallow verlor deck
2.4 psf

Rigid insulation
4.0 psf

Vapor barrier
0.5 psf

Total to deck
5.9 psf

Beams @ 3'-0" O.C. (HSS 7"x7"x3/8")
10.9 psf

Total to beams
10.7 psf

Girders (HSS 10"x10"x3/4")
1.8 psf

Total to girders
18.5 psf

Columns (HSS 4"x4"x1/2")
0.6 psf

Total to columns
19.1 psf

(DL = 20 psf)
Decking:
\[ W_o = 5.8 \text{ psf} \]
\[ W_L = 20 \text{ psf} \]
\[ W_u = 20 \text{ psf} \]

1/180 Deflection Criteria, 3'-0" span, 3' spans

Wall Angle = 60° psf

60° psf > 20° psf \( \checkmark \)

24 ga Shallow Vertor Decking, Passed for Gravity Loads

Beams @ 3' o.c.
(TRY HSS 7" x 7" x 3/16")

Load Combinations:
\[ W_u = 1.2D + 1.6L \quad (1) \]
\[ W_u = 1.4D \quad (2) \quad \text{(CUPERT)} \]

(Results via RISA 2-D Envelope Analysis)

<table>
<thead>
<tr>
<th>Mu</th>
<th>17.57 k·ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vn</td>
<td>2.4 k</td>
</tr>
<tr>
<td>AMax</td>
<td>1.93&quot; @ 30' span</td>
</tr>
</tbody>
</table>

Exam:
- A = 2.09 k
- B = 3.12 k
- C = 3.95 k
- D = 2.89 k
**Beam Capacity**

**Try HSS 7x7x3/8**

\[ \phi M_n = 70.2 \, k \cdot ft \]

\[ \phi M_n > M_n \]

\[ 70.2 \, k \cdot ft > 17.57 \, k \cdot ft \]

\[ \phi V_n = \phi A_n \, F_t \, C_v \]

\[ C_v = ? \]

\[ h/w_o = 17.1 \]

\[ \frac{1.10 \, k \cdot ft \, T}{1.10 \, k \cdot ft} = \frac{1.10 \, k \cdot ft \, T}{29000 \, k \cdot ft} = 0.017 \]

\[ \therefore \quad C_v = 1.0 \]

\[ \phi V_n = 0.7 \times (0.6) \times (1.0) \times (5.3125) \times (0.375) \times (1.0) \]

\[ \phi V_n = 49.5 \, k \]

\[ \phi V_n > V_u \]

\[ 49.5 \, k > 24 \, k \]

\[ \Delta_{poof} = 4\text{'}18\text{"} \]

\[ \Delta_{all} = 2\text{"} \]

\[ \Delta_{all} < \Delta_{max} \]

\[ 2\text{"} > 1.93\text{"} \]

**HSS 7x7x3/8" is sufficient in shear, flexure and deflection as a roof beam @ 3' o.c.**
GIRDER CAPACITY

TYPICAL SINGLE BAY BEAM, WORST CASE AT GRID 3

TRIBUTARY WIDTH = 25'

\[ \frac{w_u}{3.95} = 1.32 \text{ k/ft} \]

\[ M_u = 24.71 \text{ k-ft} \]

\[ V_u = 8.06 \text{ k} \]

\[ \Delta_{max} = 0.204 \text{ in} \]

TRY HSS 9 x 9 x 1/4

\[ \phi M_u = 77.8 \text{ k-ft} \]

\[ \phi M_n = M_n \]

\[ \frac{77.8 \text{ k-ft}}{24.71 \text{ k-ft}} \geq 3.13 \]

\[ \phi V_n = \phi 0.6 F_t A_{wrf} \]

\[ C_v = 1.0 \]

\[ \frac{35.6}{8.06} \geq 0.76 \]

\[ \phi V_n = 0.9(0.6)(90)(7.875)(2)(1.0) \]

\[ \phi V_n = V_u \]

\[ 48.9 \text{ k} \geq 8.06 \text{ k} \]

\[ y_{180} = 0.817'' \]

\[ \Delta_{act} = \Delta_{act} \]

\[ 0.817 \geq 0.204 \]

HSS 9 x 9 x 1/4 IS SUFFICIENT IN FLEXURE, SHEAR AND DEFLECTION, AS A GIRDER FOR ALL 12'-3'' SPAN LOCATIONS.
Roof Beam 1
Shear Envelope (Kips)

Moment Envelope (Kip-Feet)

Roof Beam 2
Shear Envelope (Kips)

Moment Envelope (Kip-Feet)
GIRDER CAPACITY

DOUBLE SPAN GIRDER @ GRID 1

FLAT WIDTH = 16'

\[ W_u = 0.976 \text{ k-ft} \quad (\text{FROM BEAM RN}) \]

\[ I_{PQ} = 17_1 \text{ in}^4 \]

TRY HSS 9" x 9.5" (A.I. = 183 in^4)

\[ V_P = 0.7 F_f A_{sw} C_u \]

\[ V_P = 83.84 \text{ k} \]  

HSS 9" x 9.5" IS SUFFICIENT FOR:
DEFLECTION, FLEXURE AND SHEAR TO SPAN TWO BAYS AT GRID 1.
Roof Girder

Shear Envelope (Kips)

Moment Envelope (Kip-Feet)
COLUMN DESIGN

DEMAND FORCES

\[ P_u = 20.23 \text{kN} \quad (\text{COLUMN @ 1F}) \]

\[ H = 18' \quad K = 1.0 \quad (\text{PIN PIN}) \]

TPH: HSS 4x4x1/2

\[ \phi P_n = 58.0 \text{kN} \]

\[ \phi P_n > P_u \]

58 kN \( > \) 20.23 kN

COLUMN PASSES FOR AXIAL LOADS
SEISMIC DESIGN VALUES

\[ V = C_s \cdot W \]

\[ C_s = \frac{S_{ds}}{T^{\frac{1}{2}}} \text{ or } \frac{S_{p1}}{T^{\frac{2}{3}}} \]

\[ P = 5 \]

\[ I_c = 1.0 \]

\[ S_{ds} = 1.72 \]

\[ S_{p1} = 0.69 \]

\[ T = C_t \cdot h_n \]

\[ T = 0.02 \cdot (15)^{0.75} \]

\[ C_t = 0.02 \]

\[ h_n = 15' \]

\[ z = 0.75 \]

\[ C_s = 0.344 \]

\[ T = 0.152 \]

\[ C_s = 0.344 \text{ or } 0.900 \]

\[ C_s = 0.344 \]

SEISMIC WEIGHT

ROOF: \[ 20 \text{ psf} \left(104' \times 75.5'\right) - \left(6.5' \times 18.5'\right) \] \[ 154.64 \text{ k} \]

NS WALL: \[ 92 \text{ psf} \left(3.416 \times 4'\right) \] \[ 127.33 \text{ k} \]

EW WALL: \[ 92 \text{ psf} \left(193.5' \times 4'\right) \] \[ 5.52 \text{ k} \]

\[ W_{ns} = 287.6 \text{ k} \]

\[ W_{bw} = 271.9 \text{ k} \]

BASE SHEAR

\[ V = C_s \cdot W \]

\[ V = 0.344(104.8) \text{ k} \]

\[ V = 139.25 \text{ k} \]
**N-S DIRECTION**

**V**

\[ V_{\text{max}} = 32.6 \text{ kN} \]

\[ V_{\text{min}} = 0.432 \text{ kLbf} \]

**M**

\[ M_{\text{max}} = 225 \text{ k-ft} \]

**P**

\[ P_{\text{max}} = 3.57 \text{ kN} \]
E-W DIRECTION

\begin{align*}
V &= 15.9 \\
\frac{V}{L} &= 0.037 \\
\frac{M}{L^2} &= 0.010 \\
\frac{P}{L^3} &= 0.004 \\
V_{max} &= 33.5 \text{ kN} \\
\frac{V}{L} &\leq 0.323 \text{ kN/m} \\
\frac{M}{L^2} &\leq 264.7 \text{ kN/m} \\
\frac{P}{L^3} &\leq 2.71 \text{ kN/m} \\
\end{align*}
**Tia Ana's Sustainable Development**

**Subject:** Tia Ana Roof Diaphragm

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**Shear Capacity**

24 GA Shallow Metal Deck with #10's @ 6" O.C.

(30/7 Screw Pattern)

\[ \phi V_n = 0.9 (0.535) \]

\[ \phi V_n = 0.4815 \text{ klf} \]

\[ \phi V_n \geq V_n \]

0.4815 \( \geq \) 0.432 klf

*Deck Passes for Shear*

Column Shear Capacity (HSS 4x4 8/2)

\[ \phi V_n = \phi 0.6 F_y A_w G^{1.0} \]

\[ = 0.9 (0.6) (46) (6.02 / 2) (1.0) \]

\[ A_w = 9.74 \text{ in}^2 \]

\[ A_{w, B} = A_w / 2 \]

\[ V_n = 139.25 \text{ klf} \]

4k

*Assumes small four columns... transfer shear (conservative)*

\[ \phi V_n = 74.77 \text{ klf} \]

\[ \phi V_n \geq V_n \]

74.77 klf \( \geq \) 34.81 klf

*Column Passes in Shear*
### Plan North

#### Key Plan

<table>
<thead>
<tr>
<th>N-S Direction</th>
<th>Grid A/I</th>
<th>Grid C/G</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>46'-10&quot;</td>
<td>42'-4&quot;</td>
</tr>
<tr>
<td>Wall Width</td>
<td>15'-3&quot;</td>
<td>36'-8&quot;</td>
</tr>
<tr>
<td>Volume (Vw)</td>
<td>20.47 k</td>
<td>49.21 k</td>
</tr>
<tr>
<td>Unit Cost (Vn)</td>
<td>0.497 KLF</td>
<td>1.102 KLF</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>E-W Direction</th>
<th>Grid 2</th>
<th>Grid 3</th>
<th>Grid 4</th>
<th>Grid 5</th>
<th>Grid 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>48'-10&quot;</td>
<td>38'-10&quot;</td>
<td>50'-2&quot;</td>
<td>74'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>Wall Width</td>
<td>25'-0&quot;</td>
<td>15'-9&quot;</td>
<td>20'-0&quot;</td>
<td>15'-3&quot;</td>
<td></td>
</tr>
<tr>
<td>Volume (Vw)</td>
<td>46.21 k</td>
<td>29.11 k</td>
<td>38.97 k</td>
<td>28.19 k</td>
<td></td>
</tr>
<tr>
<td>Unit Cost (Vn)</td>
<td>0.7410 KLF</td>
<td>3.750 KLF</td>
<td>3.757 KLF</td>
<td>3.381 KLF</td>
<td></td>
</tr>
</tbody>
</table>
GRID A + I

9.91 k → 2.8 k → 2.816 k

22' - 8"

GRID C + G

3.635 k → 3.6 k → 3.866 k

22' - 8"

H = H_{u} = 8' \text{ (const.)}

(V = \frac{V_{u}}{H_{u}})

Pu = 5.60 k
Vu = 9.91 k

Pu = 6.53 k
Vu = 10.50 k

Pu = 7.46 k
Vu = 26.35 k

Pu = 6 k
Vu = 22.80 k
### TIA ANA - SHEAR WALL DESIGN

#### SW A, I (1)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
<td>Pu = 5.66 k</td>
<td>S&lt;sub&gt;max&lt;/sub&gt; = 48.000 in</td>
</tr>
<tr>
<td>L = 272 in</td>
<td>Vu = 9.91 k</td>
<td>A&lt;sub&gt;min&lt;/sub&gt; = 1.452 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>d&lt;sub&gt;v&lt;/sub&gt; = 268 in</td>
<td>Mu = 79.28 k*ft</td>
<td>As = 0.105 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>t = 7.625 in</td>
<td></td>
<td>Mn = 950.88 k*ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Values</th>
<th>Shear Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>A&lt;sub&gt;req&lt;/sub&gt; = 1.452 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vnm = 222.644 k</td>
</tr>
<tr>
<td>A&lt;sub&gt;v&lt;/sub&gt; = 0.2 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vns = 44.66667 k</td>
</tr>
<tr>
<td>A&lt;sub&gt;tot&lt;/sub&gt; = 2.4 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vnmax = 374.589 k</td>
</tr>
<tr>
<td>S = 24 in</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Max Shear</th>
<th>Mu/Vu*d = 0.4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mu/Vu = 79.28 k*ft</td>
</tr>
<tr>
<td></td>
<td>Vnmin = 148.5750 k</td>
</tr>
<tr>
<td></td>
<td>Vnmax = 374.59 k</td>
</tr>
<tr>
<td></td>
<td>phiVn = 213.85 k</td>
</tr>
</tbody>
</table>

#### SW A, I (2)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
<td>Pu = 6.53 k</td>
<td>S&lt;sub&gt;max&lt;/sub&gt; = 48.000 in</td>
</tr>
<tr>
<td>L = 296 in</td>
<td>Vu = 10.56 k</td>
<td>A&lt;sub&gt;min&lt;/sub&gt; = 1.580 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>d&lt;sub&gt;v&lt;/sub&gt; = 292 in</td>
<td>Mu = 84.48 k*ft</td>
<td>As = 0.078 in&lt;sup&gt;2&lt;/sup&gt;</td>
</tr>
<tr>
<td>t = 7.625 in</td>
<td></td>
<td>Mn = 1030.32 k*ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Values</th>
<th>Shear Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>A&lt;sub&gt;req&lt;/sub&gt; = 1.580 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vnm = 246.059 k</td>
</tr>
<tr>
<td>A&lt;sub&gt;v&lt;/sub&gt; = 0.2 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vns = 48.66667 k</td>
</tr>
<tr>
<td>A&lt;sub&gt;tot&lt;/sub&gt; = 2.4 in&lt;sup&gt;2&lt;/sup&gt;</td>
<td>Vnmax = 413.244 k</td>
</tr>
<tr>
<td>S = 24 in</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Max Shear</th>
<th>Mu/Vu*d = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mu = 84.48 k*ft</td>
</tr>
<tr>
<td></td>
<td>Vnmin = 160.9875 k</td>
</tr>
<tr>
<td></td>
<td>Vnmax = 413.24 k</td>
</tr>
<tr>
<td></td>
<td>phiVn = 235.78 k</td>
</tr>
</tbody>
</table>
### SW C, G (1)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>S_max = 48,000 in</td>
</tr>
<tr>
<td>H = 8 ft</td>
<td>Pu = 7.46 k</td>
<td>As_min = 1.452 in²</td>
</tr>
<tr>
<td>L = 272 in</td>
<td>Vu = 26.35 k</td>
<td>As = 0.469 in²</td>
</tr>
<tr>
<td>d_y = 268 in</td>
<td>Mu = 210.8 k*ft</td>
<td>Mn = 950.88 k*ft</td>
</tr>
<tr>
<td>t = 7.625 in</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>fy = 40 ksi</td>
<td>As_req = 1.452 in²</td>
<td></td>
</tr>
<tr>
<td>f'm = 1000 psi</td>
<td>A_v = 0.2 in²</td>
<td></td>
</tr>
<tr>
<td>ε_{mu} = 0.0025 psi</td>
<td># of Bars = 12</td>
<td></td>
</tr>
<tr>
<td>ε_y = 0.0021 psi</td>
<td>A_{total} = 2.4 in²</td>
<td></td>
</tr>
<tr>
<td>α = 1.86</td>
<td>S = 24 in</td>
<td></td>
</tr>
</tbody>
</table>

Max Shear

<table>
<thead>
<tr>
<th>Mu/Vu*d = 0.4</th>
<th>ρ_{act} = 0.0012</th>
<th>ρ_{max} = 0.209</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mu = 210.8 k*ft</td>
<td>φMn = 855.792 k*ft</td>
</tr>
<tr>
<td></td>
<td>Vu = 26.35 k</td>
<td>φVn = 214.21 k</td>
</tr>
<tr>
<td></td>
<td>Vn_{min} = 148.5750 k</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Vn_{max} = 374.59 k</td>
<td></td>
</tr>
</tbody>
</table>

### SW C, G (2)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pu = 6 k</td>
<td>S_max = 48,000 in</td>
</tr>
<tr>
<td>H = 8 ft</td>
<td>Vu = 22.86 k</td>
<td>As_min = 1.281 in²</td>
</tr>
<tr>
<td>L = 240 in</td>
<td>Mu = 182.88 k*ft</td>
<td>As = 0.496 in²</td>
</tr>
<tr>
<td>d_y = 236 in</td>
<td></td>
<td>Mn = 695.42 k*ft</td>
</tr>
<tr>
<td>t = 7.625 in</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>fy = 40 ksi</td>
<td>As_{req} = 1.281 in²</td>
<td></td>
</tr>
<tr>
<td>f'm = 1000 psi</td>
<td>A_v = 0.2 in²</td>
<td></td>
</tr>
<tr>
<td>ε_{mu} = 0.0025 psi</td>
<td># of Bars = 10</td>
<td></td>
</tr>
<tr>
<td>ε_y = 0.0021 psi</td>
<td>A_{total} = 2 in²</td>
<td></td>
</tr>
<tr>
<td>α = 2.02</td>
<td>S = 24 in</td>
<td></td>
</tr>
</tbody>
</table>

Max Shear

<table>
<thead>
<tr>
<th>Mu/Vu*d = 0.4</th>
<th>ρ_{act} = 0.0011</th>
<th>ρ_{max} = 0.200</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mu = 182.88 k*ft</td>
<td>φMn = 625.878 k*ft</td>
</tr>
<tr>
<td></td>
<td>Vu = 22.86 k</td>
<td>φVn = 184.89 k</td>
</tr>
<tr>
<td></td>
<td>Vn_{min} = 108.6594 k</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Vn_{max} = 323.02 k</td>
<td></td>
</tr>
</tbody>
</table>
### SW 2

**Wall Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>8 ft</td>
</tr>
<tr>
<td>L</td>
<td>304 in</td>
</tr>
<tr>
<td>d_v</td>
<td>300 in</td>
</tr>
<tr>
<td>t</td>
<td>7.625 in</td>
</tr>
</tbody>
</table>

**Demand Forces**

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu</td>
<td>10 k</td>
</tr>
<tr>
<td>Vu</td>
<td>23.11 k</td>
</tr>
<tr>
<td>Mu</td>
<td>184.88 k*ft</td>
</tr>
</tbody>
</table>

**Flexural Design**

<table>
<thead>
<tr>
<th>Moment</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>S_max</td>
<td>48,000 in</td>
</tr>
<tr>
<td>As_min</td>
<td>1.623 in²</td>
</tr>
<tr>
<td>As</td>
<td>0.264 in²</td>
</tr>
<tr>
<td>Mn</td>
<td>1238.02 k*ft</td>
</tr>
</tbody>
</table>

**Design Values**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>As_eq</td>
<td>1.623 in²</td>
</tr>
<tr>
<td>A_u</td>
<td>0.2 in²</td>
</tr>
<tr>
<td>N</td>
<td>14</td>
</tr>
<tr>
<td>A_tot</td>
<td>2.8 in²</td>
</tr>
<tr>
<td>S</td>
<td>24 in</td>
</tr>
</tbody>
</table>

**Max Shear**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu/Vu*d</td>
<td>0.3</td>
</tr>
</tbody>
</table>

**Shear Design**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vn_min</td>
<td>193.4406 k</td>
</tr>
<tr>
<td>Vn_max</td>
<td>426.13 k</td>
</tr>
</tbody>
</table>

### SW 3

**Wall Properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>8 ft</td>
</tr>
<tr>
<td>L</td>
<td>240 in</td>
</tr>
<tr>
<td>d_v</td>
<td>236 in</td>
</tr>
<tr>
<td>t</td>
<td>9.627 in</td>
</tr>
</tbody>
</table>

**Demand Forces**

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu</td>
<td>6.72 k</td>
</tr>
<tr>
<td>Vu</td>
<td>14.56 k</td>
</tr>
<tr>
<td>Mu</td>
<td>116.48 k*ft</td>
</tr>
</tbody>
</table>

**Flexural Design**

<table>
<thead>
<tr>
<th>Moment</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>S_max</td>
<td>48,000 in</td>
</tr>
<tr>
<td>As_min</td>
<td>1.281 in²</td>
</tr>
<tr>
<td>As</td>
<td>0.243 in²</td>
</tr>
<tr>
<td>5n</td>
<td>67/42 k*ft</td>
</tr>
</tbody>
</table>

**Design Values**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>As_eq</td>
<td>1.281 in²</td>
</tr>
<tr>
<td>A_u</td>
<td>0.2 in²</td>
</tr>
<tr>
<td>N</td>
<td>10</td>
</tr>
<tr>
<td>A_tot</td>
<td>2 in²</td>
</tr>
<tr>
<td>S</td>
<td>24 in</td>
</tr>
</tbody>
</table>

**Max Shear**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 u/Vu*d</td>
<td>0.4</td>
</tr>
</tbody>
</table>

**Shear Design**

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vn_min</td>
<td>108.67 M4 k</td>
</tr>
<tr>
<td>Vn_max</td>
<td>323.02 k</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Value</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu</td>
<td>116.48 k*ft</td>
</tr>
<tr>
<td>phiMn</td>
<td>625.878 k*ft</td>
</tr>
</tbody>
</table>
### TIA ANA - SHEAR WALL DESIGN

#### SW 5 (1)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
<td>Pu = 8.39 k</td>
<td>( S_{max} = 48.000 \text{ in} )</td>
</tr>
<tr>
<td>L = 144 in</td>
<td>Vu = 9.15 k</td>
<td>( A_{min} = 0.769 \text{ in}^2 )</td>
</tr>
<tr>
<td>( d_v = 140 \text{ in} )</td>
<td>Mu = 73.2 k*ft</td>
<td>( A_s = 0.226 \text{ in}^2 )</td>
</tr>
<tr>
<td>( t = 7.625 \text{ in} )</td>
<td></td>
<td>( M_n = 250.78 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Values</th>
<th>Shear Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y = 40 \text{ ksi} )</td>
<td>Vnm = 99.319 k</td>
</tr>
<tr>
<td>( f'm = 1000 \text{ psi} )</td>
<td>Vns = 23.3333 k</td>
</tr>
<tr>
<td>( \varepsilon_{mu} = 0.0025 \text{ psi} )</td>
<td>Vnmax = 167.987 k</td>
</tr>
<tr>
<td>( \varepsilon_y = 0.0021 \text{ psi} )</td>
<td>Mu = 73.2 k*ft</td>
</tr>
<tr>
<td>( \alpha = 2.95 )</td>
<td>( \phi M_n = 225.702 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

Max Shear
- \( Mu/Vu*d = 0.7 \)
- \( \rho_{act} = 0.0011 \)
- \( \rho_{max} = 0.094 \)
- \( Vn_{min} = 39.1844 \text{ k} \)
- \( Vn_{max} = 167.99 \text{ k} \)

#### SW 5 (2)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
<td>Pu = 11.88 k</td>
<td>( S_{max} = 48.000 \text{ in} )</td>
</tr>
<tr>
<td>L = 304 in</td>
<td>Vu = 18.67 k</td>
<td>( A_{min} = 1.623 \text{ in}^2 )</td>
</tr>
<tr>
<td>( d_v = 300 \text{ in} )</td>
<td>Mu = 149.36 k*ft</td>
<td>( A_s = 0.118 \text{ in}^2 )</td>
</tr>
<tr>
<td>( t = 7.625 \text{ in} )</td>
<td></td>
<td>( M_n = 1238.02 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Values</th>
<th>Shear Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y = 40 \text{ ksi} )</td>
<td>Vnm = 255.127 k</td>
</tr>
<tr>
<td>( f'm = 1000 \text{ psi} )</td>
<td>Vns = 50 k</td>
</tr>
<tr>
<td>( \varepsilon_{mu} = 0.0025 \text{ psi} )</td>
<td>Vnmax = 426.127 k</td>
</tr>
<tr>
<td>( \varepsilon_y = 0.0021 \text{ psi} )</td>
<td>Mu = 149.36 k*ft</td>
</tr>
<tr>
<td>( \alpha = 1.73 )</td>
<td>( \phi M_n = 1114.218 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

Max Shear
- \( Mu/Vu*d = 0.3 \)
- \( \rho_{act} = 0.0012 \)
- \( \rho_{max} = 0.201 \)
- \( Vn_{min} = 193.4406 \text{ k} \)
- \( Vn_{max} = 426.13 \text{ k} \)
### Wall Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>8 ft</td>
</tr>
<tr>
<td>L</td>
<td>448 in</td>
</tr>
<tr>
<td>d_v</td>
<td>444 in</td>
</tr>
<tr>
<td>t</td>
<td>7.625 in</td>
</tr>
</tbody>
</table>

### Demand Forces

<table>
<thead>
<tr>
<th>Force</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pu</td>
<td>10.96 k</td>
</tr>
<tr>
<td>V_u</td>
<td>14.1 k</td>
</tr>
<tr>
<td>Mu</td>
<td>112.8 k*ft</td>
</tr>
</tbody>
</table>

### Flexural Design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>S_max</td>
<td>48.000 in</td>
</tr>
<tr>
<td>As_min</td>
<td>2.391 in^2</td>
</tr>
<tr>
<td>As</td>
<td>-0.062 in^2</td>
</tr>
<tr>
<td>M_n</td>
<td>1320.65 k*ft</td>
</tr>
</tbody>
</table>

### Design Values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_y</td>
<td>40 ksi</td>
</tr>
<tr>
<td>f_m</td>
<td>1000 psi</td>
</tr>
<tr>
<td>epsilon_m</td>
<td>0.0025 psi</td>
</tr>
<tr>
<td>epsilon_y</td>
<td>0.0021 psi</td>
</tr>
<tr>
<td>alpha</td>
<td>1.50</td>
</tr>
</tbody>
</table>

### Max Shear

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu/V_u*d</td>
<td>0.2</td>
</tr>
</tbody>
</table>

### Shear Design

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_nm</td>
<td>393.960 k</td>
</tr>
<tr>
<td>V_ns</td>
<td>37 k</td>
</tr>
</tbody>
</table>

### V_nmax

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_nmax</td>
<td>648.140 k</td>
</tr>
</tbody>
</table>

### M_n

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mu</td>
<td>112.8 k*ft</td>
</tr>
<tr>
<td>phi_Mn</td>
<td>1188.585 k*ft</td>
</tr>
</tbody>
</table>

### V_u

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_u</td>
<td>14.1 k</td>
</tr>
</tbody>
</table>

### phi_V_n

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>phi_V_n</td>
<td>344.77 k</td>
</tr>
</tbody>
</table>

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**PROJECT:** Tia Ana's Sustainable Development  
**SUBJECT:** TIA ANA - SHEAR WALL DESIGN  
**PREPARED BY:** Dani Rustagi  
**DATE:** 22  
**SHEET:**
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:02:12

File: U:\453 - Senior Project\TIA\ANA\SW AI 1.col
Project:
Column:
f'c = 1.5 ksi  
Ec = 2208 ksi  
f = 1.275 ksi  
e_u = 0.003 in/in  
Beta1 = 0.8
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Engineer:
Ag = 2176 in^2  
As = 2.40 in^2  
Xo = 0.00 in  
Yo = 0.00 in  
Min clear spacing = 23.50 in  
Clear cover = 3.75 in

rho = 0.11%
I = 1.34158e+007 in^4
I = 11605.3 in^4
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:03:07

spColumn v4.81. Licensed to: Cal Poly University. License ID: 62481-1042880-4-26E35-1F2A0

File: U:\453 - Senior Project\TIA ANA\SW AI 2.col
Project:
Column:
\( f_c = 1.5 \text{ ksi} \quad f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \quad E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.003 \text{ in/in} \)
Beta1 = 0.8
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Engineer:
Ag = 2368 in^2
As = 2.40 in^2
\( \rho = 0.10\% \)
Xo = 0.00 in
Yo = 0.00 in
\( \xi = 1.72896e+007 \text{ in}^4 \)
\( \gamma = 12629.3 \text{ in}^4 \)
Min clear spacing = 23.50 in
Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:04:03

File: U:\453 - Senior Project\TIA\ANA\SW CG 1.col
Project:
Column:
f_c = 1.5 ksi  \quad f_y = 40 ksi
E_c = 2208 ksi  \quad E_s = 29000 ksi
f_c = 1.275 ksi  
\varepsilon_u = 0.003 in/in
Beta1 = 0.8
Confinement: Tied
\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65

Engineer:
Ag = 2176 in^2  \quad As = 2.40 in^2
\rho_o = 0.11%  
X_o = 0.00 in  \quad I_x = 1.34158e+007 in^4
Y_o = 0.00 in  \quad I_y = 11605.3 in^4
Min clear spacing = 23.50 in  Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:05:35

spColumn v4.81. Licensed to: Cal Poly University. License ID: 62481-1042880-4-26E35-1F2A0

File: U:\453 - Senior Project\TIA ANA\SW CG 2.col
Project:
Column:
\( f_c = 1.5 \text{ ksi} \)
\( f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \)
\( E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.003 \text{ in/in} \)
\( \text{Beta1} = 0.8 \)

Engineer:
\( A_g = 1920 \text{ in}^2 \)
\( A_s = 2.00 \text{ in}^2 \)
\( \rho_c = 0.10\% \)
\( X_0 = 0.00 \text{ in} \)
\( I_x = 9.216e+006 \text{ in}^4 \)
\( Y_0 = 0.00 \text{ in} \)
\( I_y = 10240 \text{ in}^4 \)
\( \text{Min clear spacing} = 23.50 \text{ in} \)
\( \text{Clear cover} = 3.75 \text{ in} \)

Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 13:56:55

File: u:\453 - senior project\tia analysis\2.col
Project:
Column:
\( f_c = 1.5 \text{ ksi} \quad f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \quad E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.003 \text{ in/in} \)
\( \beta_1 = 0.8 \)
Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)
Engineer:
\( A_g = 2432 \text{ in}^2 \)
\( A_s = 2.80 \text{ in}^2 \)
\( \rho = 0.12\% \)
\( X_o = 0.00 \text{ in} \)
\( Y_o = 0.00 \text{ in} \)
Min clear spacing = 15.50 in
Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 13:57:31

File: U:\453 - Senior Project\TIA ANALYSIS\SW 3.col

Project:
Column:
f'c = 1.5 ksi  fy = 40 ksi
Ec = 2208 ksi  Es = 29000 ksi
fc = 1.275 ksi
e_u = 0.003 in/in
Beta1 = 0.8
Confinement: Tied

Engineer:
Ag = 1920 in^2  10 #4 bars
As = 2.00 in^2  rho = 0.10%
Xo = 0.00 in  lx = 9.216e+006 in^4
Yo = 0.00 in  ly = 10240 in^4
Min clear spacing = 23.50 in  Clear cover = 3.75 in

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 13:58:11

SpColumn v4.81. Licensed to: Cal Poly University. License ID: 62481-1042880-4-26E35-1F2A0

File: U:\453 - Senior Project\TIA ANA\SW 5 1.col
Project:
Column:
\( f_c = 1.5 \text{ ksi} \quad f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \quad E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.003 \text{ in/in} \)
Beta1 = 0.8
Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Engineer:
\( A_g = 1152 \text{ in}^2 \quad 6 \#4 \text{ bars} \)
\( A_s = 1.20 \text{ in}^2 \quad \rho = 0.10\% \)
\( x_0 = 0.00 \text{ in} \quad l_x = 1.99066e+006 \text{ in}^4 \)
\( y_0 = 0.00 \text{ in} \quad l_y = 6144 \text{ in}^4 \)
Min clear spacing = 23.50 in Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 13:59:00

File: U:\453 - Senior Project\TIA ANALSW 5 2.col
Project:
Column:
f'c = 1.5 ksi
Ec = 2208 ksi
fc = 1.275 ksi
e_u = 0.003 in/in
Beta1 = 0.8
Confinement: Tied
fy = 40 ksi
Es = 29000 ksi

Engineer:
Ag = 2432 in^2
As = 2.80 in^2
Xo = 0.00 in
Yo = 0.00 in
Min clear spacing = 15.50 in

14 #4 bars
rho = 0.12%
x = 1.87296e+007 in^4
ly = 12970.7 in^4
Clear cover = 3.75 in

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:00:24

File: U:\453 - Senior Project\TIA ANA\SW 8.col
Project:
Column:
f'c = 1.5 ksi  
Ec = 2208 ksi  
f = 1.275 ksi  
e_u = 0.003 in/in  
Beta1 = 0.8  
Confinement: Tied  
fy = 40 ksi  
Es = 29000 ksi  
fc = 1.275 ksi  
e_u = 0.003 in/in  
Beta1 = 0.8  
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Engineer:
Ag = 3584 in^2  
As = 2.00 in^2  
Xo = 0.00 in  
Yo = 0.00 in  
Min clear spacing = 23.50 in  
Clear cover = 3.75 in  
rho = 0.06%
Ix = 5.99436e+007 in^4  
Iy = 19114.7 in^4
**PROJECT: Tia Ana's Sustainable Development**

**SUBJECT: Tia Ana Out of Plane Shear Wall**

**PREPARED BY: Dani Rustagi**

**DATE:**

**SHEET:** 32 of

---

### WALL DEMANDS:

<table>
<thead>
<tr>
<th><strong>DEMAND FORCES</strong></th>
<th><strong>WALL WEIGHT</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{ps}$: 1.72 G</td>
<td>WT: 0.34 Kips</td>
</tr>
<tr>
<td>I: 1</td>
<td></td>
</tr>
</tbody>
</table>

| FP: 57.79 PSF | ME: 0.462 K-FT |

### LOAD COMBINATIONS:

- **1.2D + 1.6L**
- **(1.2 + 2S_{ps})D + 1.0E**
- **(0.9 - 2S_{ps})D + 1.0E**

<table>
<thead>
<tr>
<th><strong>DEMANDS TO USE</strong></th>
<th><strong>CAPACITY:</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>L: 2.41 KIPS</td>
<td><strong>WALL SECTION PROPERTIES:</strong></td>
</tr>
<tr>
<td>SHEAR: 8 KIPS</td>
<td><strong>BLOCK DEPTH:</strong> D</td>
</tr>
<tr>
<td>MOMENT: 0.26 KIP-FEET</td>
<td><strong>BLOCK WIDTH:</strong> B</td>
</tr>
<tr>
<td></td>
<td><strong>BLOCK HEIGHT:</strong> H</td>
</tr>
<tr>
<td></td>
<td><strong>WALL HEIGHT:</strong> H</td>
</tr>
<tr>
<td></td>
<td><strong>WALL WEIGHT:</strong> W_T</td>
</tr>
<tr>
<td></td>
<td><strong>BAR COVER:</strong> $B_{COVER}$</td>
</tr>
<tr>
<td></td>
<td><strong>VERTICAL REINFORCEMENT:</strong> #4 BARS</td>
</tr>
<tr>
<td></td>
<td><strong>NUMBER OF BARS:</strong> 1 BARS</td>
</tr>
<tr>
<td></td>
<td><strong>AREA REINFORCEMENT:</strong> 0.2 In^2</td>
</tr>
<tr>
<td></td>
<td><strong>BAR SPACING:</strong> 48 &quot; O.C.</td>
</tr>
<tr>
<td></td>
<td><strong>AREA TOTAL:</strong> 0.05 In^2/FT</td>
</tr>
<tr>
<td></td>
<td><strong>DIAMETER BAR:</strong> 0.5 In</td>
</tr>
</tbody>
</table>

| | **MASONRY PROPERTIES:** |
| | **COMPRESSIVE STRESS:** F'M: 2.00 ksi |
| | **MASONRY STRAIN:** $e_{ma}$: 0.0025 IN/IN |
| | **MODULUS OF RUP:** $F_x$: 163 PSI |

| | **REINFORCEMENT PROPERTIES:** |
| | **YIELD STRENGTH:** $F_y$: 40.00 ksi |
| | **MODULUS OF E:** E: 29000 ksi |

| | **HORIZONTAL REINFORCEMENT:** |
| | **BAR SPACING:** |
| | **AREA REINFORCEMENT:** |
| | **DIAMETER BAR:** |

| | **MOMENT OF INERTIA:** |
| | **I_o: 577.43 In^4** |

| | **CRACKING MOMENT:** |
| | **M_{cr}: 2.057 K-FT** |
DEPTH COMPRESSION BLOCK:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.18 in</td>
</tr>
<tr>
<td>C</td>
<td>0.22 in</td>
</tr>
</tbody>
</table>

CRACKING MOMENT OF INERTIA:

| N  | 16.1 |
| l_cr | 10.4 in^4 |

1ST ITERATION MOMENT WITH \( \xi = 0 \):

\[ M_{U1} = 0.462 \text{ k-ft} \]

FIND \( \xi \):

\[ \xi = 0.005 \text{ in} \]

2ND ITERATION MOMENT WITH \( \xi = 0.005 \):

\[ M_{U2} = 0.463 \text{ k-ft} \]

MOMENT CAPACITY:

| M_N  | 1.37 k-ft |
| \( \Phi M_N \) | 1.23 k-ft |

1.23 k-ft > 0.26 k-ft. Wall ok for out of plane.
TYPICAL WALL FOOTING (F-1)

WORST CASE AXIAL LOAD : 1.34 klf
WORST CASE SHEAR FORCE : 1.16 klf
SHORTEST WALL : 12’-0”

\[ W_d = 1.34 \text{ klf (self-wt included)} \]
\[ W_L = 100 \text{ psf}(20’)=2 \text{ klf} \]

\[ APFG = \frac{P_d + P_L}{2} \quad 12.95 \text{ k} \]

\[ P_d = 1.34(12’)=16.08 \text{ k} \]
\[ P_L = 2(12’)=24 \text{ k} \]

\[ q_{BL} = 1,500 \text{ psf} \]
\[ \text{APFG} = 20.72 \text{ ft}^2 \]

TRY 14’x3’ FOOTING

\[ q_u = 1.2 \left( 16.08 \right) + 1.6 \left( 24 \right) \]
\[ (14x3) \]

\[ Vu = q_u \left( L - d_{min} \right) - d_{min} \]
\[ = 1.37 \left( 14 - \left( 8/12 \right) \right) - d_{min} \]
\[ d_{min} = 8” \]
\[ Vu = 8.22 \text{ k} \]

SEE SPREADSHEET FOR FLEXURAL CAPACITY AND FINAL DESIGN
CONCENTRIC LOADED - TYPICAL WALL FOOTING

REFERENCE

**GIVENS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_{c}$</td>
<td>3000.00 psi</td>
</tr>
<tr>
<td>$f_{y}$</td>
<td>60000.00 psi</td>
</tr>
<tr>
<td>$d_{col}$</td>
<td>8.00 in</td>
</tr>
<tr>
<td>$b_{col}$</td>
<td>6.00 in</td>
</tr>
<tr>
<td>$q_{allow}$</td>
<td>1500.00 psf</td>
</tr>
</tbody>
</table>

**DETERMINE FOOTING DIMENSIONS (L,W)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>14.00 ft</td>
</tr>
<tr>
<td>W</td>
<td>3.00 ft</td>
</tr>
<tr>
<td>Aftg</td>
<td>42.00 ft²</td>
</tr>
</tbody>
</table>

**DETERMINE FOOTING THICKNESS (h) BASED ON SHEAR**

**A) CHECK ONE WAY BEAM SHEAR AT CRITICAL SECTION**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_u$</td>
<td>1370.00 psf</td>
</tr>
<tr>
<td>$d_{ftg}$</td>
<td>8.00 in</td>
</tr>
<tr>
<td>$V_u$</td>
<td>685.00 #/ft</td>
</tr>
<tr>
<td>$\phi_i/V_n$</td>
<td>7887.20 #/ft</td>
</tr>
</tbody>
</table>

**ACI 13.3.1.2**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>h</td>
<td>12.00 in</td>
</tr>
<tr>
<td>TRY h</td>
<td>12.00 in</td>
</tr>
<tr>
<td>TRY d</td>
<td>8.00 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{min}$ (effective depth of reinforcement = 6&quot;)</td>
<td>10.00 in</td>
</tr>
<tr>
<td>$\phi_i/V_n$</td>
<td>7887.20 klf</td>
</tr>
</tbody>
</table>

**B) CHECK TWO WAY PUNCHING SHEAR**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_u$</td>
<td>10465.28 #</td>
</tr>
<tr>
<td>$b_o$</td>
<td>60.00</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.33</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>phiVc</td>
<td>98590.06 #</td>
</tr>
<tr>
<td>144598.76 #</td>
<td></td>
</tr>
<tr>
<td>105162.73 #</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>phiVc &gt; Vu</td>
<td>d = OK</td>
</tr>
</tbody>
</table>

**FLEXURAL CAPACITY**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_u$</td>
<td>932.36 #-in</td>
</tr>
<tr>
<td>0.08 k-ft</td>
<td></td>
</tr>
</tbody>
</table>

**ESTIMATE (d-a/2) = 0.9d**

**ASSUME PHI = 0.90**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$</td>
<td>0.0024 in²/FT</td>
</tr>
</tbody>
</table>

**TRY #4 @ 16" O.C.**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$</td>
<td>0.48 in²</td>
</tr>
<tr>
<td>0.26 in²</td>
<td></td>
</tr>
<tr>
<td>0.26 in²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$</td>
<td>1.80 in²</td>
</tr>
<tr>
<td>d</td>
<td>8.75 in</td>
</tr>
<tr>
<td>a</td>
<td>3.53 in</td>
</tr>
<tr>
<td>c</td>
<td>4.15 in</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_n$</td>
<td>754411.76 #</td>
</tr>
<tr>
<td>62.87 k-ft</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_t$</td>
<td>0.0033 &gt; 0.0021</td>
</tr>
<tr>
<td>&gt;0.005</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi_iM_n$</td>
<td>56.58 k-ft</td>
</tr>
</tbody>
</table>

**phiMn > Mu**

use #4 @ 16" o.c. one way with 14'X3' footing 12" deep.
CONCENTRIC LOADED - TYPICAL COLUMN FOOTING

GIVENS

\[
\begin{align*}
P_c &= 3000 \text{ psi} & A_{TRG} &= 329.99 & d_{COL} &= 6.00 \text{ in} \\
f &= 60000 \text{ psi} & d_{COL} &= 6.00 \text{ in} \\
q_{allow} &= 1500 \text{ psf} & b_{COL} &= & \\
\text{DEAD} &= 20 \text{ psf} & P_0 &= 6599.88 \# \\
\text{LIVE} &= 100 \text{ psf} & P_l &= 32999.40 \# \\
\end{align*}
\]

DETERMINE FOOTING DIMENSIONS (L,W)

\[
L^*W = \frac{ASD \text{ LOAD}}{q_{allow}} = 26.40 \text{ ft}^2
\]

\[
\begin{align*}
L &= W = & 5.14 \text{ ft} \\
L &= W = & 6.00 \text{ ft} \\
A_{TRG} &= & 36.00 \text{ ft}^2
\end{align*}
\]

DETERMINE FOOTING THICKNESS (h) BASED ON SHEAR

A) CHECK ONE WAY BEAM SHEAR AT CRITICAL SECTION

\[
\begin{align*}
q_v &= 1686.64 \text{ psf} & d_{TRG} &= 10.75 \text{ in} \\
\phi V_v &= 3127.30 \#/\text{ft} & \phi &= 0.75 \\
\phi V_v &= 10598.43 \#/\text{ft} \\
\end{align*}
\]

\[
\begin{align*}
h &= d + 4" &= 14.75 \text{ in} \\
\text{TRY} h &= & 18.00 \text{ in} \\
\text{TRY} d &= & 14.00 \text{ in}
\end{align*}
\]

\[
\begin{align*}
h_{min} &= \text{effective depth of reinforcement} = 6" \\
\end{align*}
\]

\[
\begin{align*}
\phi V_v &= 276052.17 \# \\
\phi V_v &= 245379.71 \# \\
\phi V_v &> V_v & d = \text{OK}
\end{align*}
\]

\[
\phi V_v > V_v
\]

FLEXURAL CAPACITY

\[
\begin{align*}
M_p &= 6377.59 \#-\text{in} & 0.53 \text{ k-ft}
\end{align*}
\]

ESTIMATE \(d-a/2 = 0.9d\) 
ASSUME PHI = 0.90

\[
\begin{align*}
A_{S\text{req}} &= 0.01 \text{ in}^2/\text{ft} \\
A_{S\text{min}} &= 0.48 \text{ in}^2 \\
A_{S\text{min}} &= 0.46 \text{ in}^2 \\
A_{S\text{min}} &= 0.39 \text{ in}^2
\end{align*}
\]

TRY #4 @ 16" O.C.

\[
\begin{align*}
M_p &= 977294.12 \#-\text{in} & 81.44 \text{ k-ft} \\
E_i &= 0.01 > 0.0021 \text{ (>0.005)} \\
\phi M_p &= 73.30 \text{ k-ft}
\end{align*}
\]

\[
\phi M_p > M_p
\]

use #4 @ 6" o.c. each way with 3"x3" footing 14" deep.
ASSUME TRIBUTARY 1 FT SECTION: \( \# \cdot \# \cdot \# \cdot \# = \frac{0.7 \text{ in.} (12\text{ in.})}{1 \text{ ft}} = 0.15 \text{ in.}^2/\text{ft} \)

\( d = 1/8 \text{ in.} \)

\[ W_{tot} = 8 \left( \frac{100 \text{ lbs}}{1 \text{ ft}} \right) \left( \frac{3/8 \text{ ft}}{2} \right) \]

\[ W_{tot} = 320 \text{ lbs} \]

ACCOUNTS FOR OPENINGS IN BLOCKS

(SEE SPEEAMMENT FOR DESIGN ON NEXT PAGE)
### Project: Tia Ana's Sustainable Development

**Subject:** Out of Plane Screen Wall

**Prepared By:** Dani Rustagi

**Date:**

**Sheet:** 36 of 36

#### Demands

<table>
<thead>
<tr>
<th>Demand Forces</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Walls Weight:</strong></td>
</tr>
<tr>
<td>( W_T )</td>
</tr>
<tr>
<td>( S_{DG} )</td>
</tr>
<tr>
<td>( I )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Demand Routes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FP</strong></td>
</tr>
<tr>
<td><strong>ME</strong></td>
</tr>
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</table>

#### Axial

<table>
<thead>
<tr>
<th>Axial</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead:</strong></td>
</tr>
<tr>
<td><strong>Live:</strong></td>
</tr>
<tr>
<td><strong>Earthquake:</strong></td>
</tr>
</tbody>
</table>

#### Shear

<table>
<thead>
<tr>
<th>Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead:</strong></td>
</tr>
<tr>
<td><strong>Live:</strong></td>
</tr>
<tr>
<td><strong>Earthquake:</strong></td>
</tr>
</tbody>
</table>

#### Moment

<table>
<thead>
<tr>
<th>Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead:</strong></td>
</tr>
<tr>
<td><strong>Live:</strong></td>
</tr>
<tr>
<td><strong>Earthquake:</strong></td>
</tr>
</tbody>
</table>

#### Load Combinations

\[
(1.2D + 1.6L) (1.2 + 2S_{DG})D + 1.0E
\]

<table>
<thead>
<tr>
<th>Load Combination</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Axial:</strong></td>
</tr>
<tr>
<td><strong>Shear:</strong></td>
</tr>
<tr>
<td><strong>Moment:</strong></td>
</tr>
</tbody>
</table>

#### Demands To Use

<table>
<thead>
<tr>
<th>Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>L</strong></td>
</tr>
<tr>
<td><strong>Bar</strong></td>
</tr>
<tr>
<td><strong>Moment</strong></td>
</tr>
</tbody>
</table>

#### Capacity

**Wall Section Properties**

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Block Depth:</strong></td>
</tr>
<tr>
<td><strong>Block Width:</strong></td>
</tr>
<tr>
<td><strong>Block Height:</strong></td>
</tr>
<tr>
<td><strong>Wall Height:</strong></td>
</tr>
<tr>
<td><strong>Wall Weight:</strong></td>
</tr>
<tr>
<td><strong>Bar Cover</strong></td>
</tr>
</tbody>
</table>

#### Masonry Properties

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Compressive Stress:</strong></td>
</tr>
<tr>
<td><strong>Masonry Strain:</strong></td>
</tr>
<tr>
<td><strong>Modulus of Rupture:</strong></td>
</tr>
</tbody>
</table>

#### Reinforcement Properties

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yield Strength:</strong></td>
</tr>
<tr>
<td><strong>Modulus of Elasticity:</strong></td>
</tr>
</tbody>
</table>

#### Vertical Reinforcement

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number of Bars:</strong></td>
</tr>
<tr>
<td><strong>Area Reinforcement:</strong></td>
</tr>
<tr>
<td><strong>Bar Spacing:</strong></td>
</tr>
<tr>
<td><strong>Area Total:</strong></td>
</tr>
<tr>
<td><strong>Diameter Bar:</strong></td>
</tr>
</tbody>
</table>

#### Moment of Inertia

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>I_{G}</strong></td>
</tr>
</tbody>
</table>

#### Cracking Moment

<table>
<thead>
<tr>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>M_{cr}</strong></td>
</tr>
</tbody>
</table>
DEPTH COMPRESSION BLOCK:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.29 IN</td>
</tr>
<tr>
<td>C</td>
<td>0.36 IN</td>
</tr>
</tbody>
</table>

CRACKING MOMENT OF INERTIA:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>16.1</td>
</tr>
<tr>
<td>l_{cr}</td>
<td>29.0 IN²</td>
</tr>
</tbody>
</table>

1ST ITERATION MOMENT WITH $\xi = 0$:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{u1}</td>
<td>0.231 K-Ft</td>
</tr>
</tbody>
</table>

FIND $\xi$:

| $\xi$ | 0.003 IN |

2ND ITERATION MOMENT WITH $\xi = 0.003$

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>M_{u2}</td>
<td>0.231 K-Ft</td>
</tr>
</tbody>
</table>

MOMENT CAPACITY:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>M_n</td>
<td>2.20 K-Ft</td>
</tr>
<tr>
<td>$\Phi_{MN}$</td>
<td>1.98 K-Ft</td>
</tr>
</tbody>
</table>

1.98 K-Ft > 0.13 K-Ft Wall ok for out of plane.
Specifier's comments: Anchor design for all columns into CMU Wall

1 Input data

Anchor type and diameter: Heavy Square Head ASTM F 1554 GR. 36 7/8
Effective embedment depth: $h_{ef} = 12.000$ in.
Material: ASTM F 1554
Proof: Design method ACI 318-14 / CIP
Stand-off installation: $e_0 = 0.000$ in. (no stand-off); $t = 0.500$ in.
Anchor plate: $l \times l \times t = 8.000$ in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile: Square HSS (AISC); $(L \times W \times T) = 4.000$ in. x 4.000 in. x 0.500 in.
Base material: cracked concrete, 2500, $f'_c = 2500$ psi; $h = 420.000$ in.
Reinforcement: tension: condition A, shear: condition B; anchor reinforcement: tension edge reinforcement: > No. 4 bar

Geometry [in.] & Loading [lb, in.lb]
2 Load case/Resulting anchor forces

Load case: Design loads

**Anchor reactions [lb]**

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension force</th>
<th>Shear force</th>
<th>Shear force x</th>
<th>Shear force y</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4309</td>
<td>4658</td>
<td>3294</td>
<td>3294</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>4658</td>
<td>3294</td>
<td>3294</td>
</tr>
<tr>
<td>3</td>
<td>15650</td>
<td>4658</td>
<td>3294</td>
<td>3294</td>
</tr>
<tr>
<td>4</td>
<td>4309</td>
<td>4658</td>
<td>3294</td>
<td>3294</td>
</tr>
</tbody>
</table>

- max. concrete compressive strain: $1.03 \%$ 
- max. concrete compressive stress: $4492 \text{ [psi]}$
- resulting tension force in $(x/y)=(-1.612/1.612)$: $24269 \text{ [lb]}$
- resulting compression force in $(x/y)=(2.475/-2.475)$: $27869 \text{ [lb]}$

3 Tension load

<table>
<thead>
<tr>
<th>Steel Strength$^*$</th>
<th>Load $N_{sa}$ [lb]</th>
<th>Capacity $4N_s$ [lb]</th>
<th>Utilization $\beta_N = N_{sa} / 4N_s$</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>15650</td>
<td>20097</td>
<td>78</td>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

Pullout Strength$^*$
- $15650$
- $20510$
- $77$
- OK

Concrete Breakout Strength$^{**1}$
- $24269$
- $44305$
- $55$
- OK

Concrete Side-Face Blowout, direction $^{**}$
- N/A
- N/A
- N/A
- N/A

$^*$ anchor having the highest loading  
$^{**}$ anchor group (anchors in tension)  
$^{1}$ Tension Anchor Reinforcement has been selected!

3.1 Steel Strength

$N_{sa} = A_{sa,N} \cdot f_{sa}$

$\phi N_{sa} \geq N_{sa}$

**Variables**

- $A_{sa,N} \text{ [in.}^2\text{]} = 0.46$
- $f_{sa} \text{ [psi]} = 58000$

**Calculations**

$N_{sa} \text{ [lb]} = 26796$

**Results**

- $N_{sa} \text{ [lb]} = 26796$
- $\phi_{steel} = 0.750$
- $\phi N_{sa} \text{ [lb]} = 20097$
- $N_{sa} \text{ [lb]} = 15650$
### 3.2 Pullout Strength

\[ N_{pl} = \psi_{cp} N_p \quad \text{ACI 318-14 Eq. (17.4.3.1)} \]
\[ N_p = 8 A_{bar} f_c \quad \text{ACI 318-14 Eq. (17.4.3.4)} \]
\[ \phi N_{pl} \geq N_u \quad \text{ACI 318-14 Table 17.3.1.1} \]

**Variables**

<table>
<thead>
<tr>
<th>( \psi_{cp} )</th>
<th>( A_{bar} ) [in.²]</th>
<th>( \lambda_c )</th>
<th>( f_c ) [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
<td>1.47</td>
<td>1.000</td>
<td>2500</td>
</tr>
</tbody>
</table>

**Calculations**

\[ N_p \] [lb]

29300

**Results**

\[ N_{pl} \] [lb] \( \phi \) \( N_{pl} \) [lb] \( N_u \) [lb]

29300 \ 0.700 \ 20510 \ 15650

### 3.3 Concrete Breakout Strength

\[ N_{db} = \left( \frac{A_{nc}}{A_{nc,N}} \right) \psi_{c,N} \psi_{d,N} \psi_{c,N} \psi_{cp,N} N_b \quad \text{ACI 318-14 Eq. (17.4.2.1b)} \]
\[ \phi N_{db} \geq N_u \quad \text{ACI 318-14 Table 17.3.1.1} \]
\[ A_{nc} \quad \text{see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)} \]
\[ A_{nc,N} \quad \text{ACI 318-14 Eq. (17.4.2.1c)} \]

\[ \psi_{c,N} = \left( \frac{1}{1 + 2 \frac{e_{c,1}}{3 h_{nf}}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.4)} \]
\[ \psi_{d,N} = 0.7 + 0.3 \left( \frac{c_{min}}{1.5 h_{nf}} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.5b)} \]
\[ \psi_{cp,N} = \text{MAX} \left( \frac{c_{min}}{1.5 h_{nf}}, 1.5 h_{nf} \right) \leq 1.0 \quad \text{ACI 318-14 Eq. (17.4.2.7b)} \]
\[ N_b = 16 \lambda_c \sqrt{f_c h_{nf}^3} \quad \text{ACI 318-14 Eq. (17.4.2.2b)} \]

**Variables**

<table>
<thead>
<tr>
<th>( h_{nf} ) [in.]</th>
<th>( e_{c,1,N} ) [in.]</th>
<th>( e_{c,2,N} ) [in.]</th>
<th>( c_{min} ) [in.]</th>
<th>( \psi_{c,N} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.000</td>
<td>0.779</td>
<td>0.779</td>
<td>422.500</td>
<td>1.000</td>
</tr>
</tbody>
</table>

\[ c_{nc} \] [in.] \( k_c \) \( \lambda_c \) \( f_c \) [psi]

- \ 16 \ 1.000 \ 2500

**Calculations**

\[ A_{nc} \] [in.²] \( A_{nc,N} \) [in.²] \( \psi_{c,N} \) \( \psi_{d,N} \) \( \psi_{cp,N} \) \( N_b \) [lb]

1656.00 \ 1296.00 \ 0.959 \ 0.959 \ 1.000 \ 1.000 \ 50318

**Results**

\[ N_{db} \] [lb] \( \phi \) \( N_{db} \) [lb] \( N_u \) [lb]

59073 \ 0.750 \ 44305 \ 24269

Input data and results must be checked for agreement with the existing conditions and for plausibility!
4 Shear load

<table>
<thead>
<tr>
<th>Steel Strength*</th>
<th>4658</th>
<th>10450</th>
<th>45</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel failure (with lever arm)*</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Pryout Strength**</td>
<td>18632</td>
<td>91373</td>
<td>21</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete edge failure in direction x**</td>
<td>18632</td>
<td>1792125</td>
<td>2</td>
<td>OK</td>
</tr>
</tbody>
</table>

* anchor having the highest loading  **anchor group (relevant anchors)

4.1 Steel Strength

\[
\begin{align*}
V_{sp} &= 0.6 A_{p2-V} f_{sc} \\
\phi V_{steel} &= V_{ua}
\end{align*}
\]

ACI 318-14 Eq. (17.5.1.2b)

ACI 318-14 Table 17.3.1.1

Variables

<table>
<thead>
<tr>
<th>A_{p2-V} [in.²]</th>
<th>f_{sc} [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.46</td>
<td>58000</td>
</tr>
</tbody>
</table>

Calculations

\[
V_{ua} [lb] = 16078
\]

Results

<table>
<thead>
<tr>
<th>V_{ua} [lb]</th>
<th>\phi V_{ua} [lb]</th>
<th>V_{ua} [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>16078</td>
<td>10450</td>
<td>4658</td>
</tr>
</tbody>
</table>

4.2 Pryout Strength

\[
\begin{align*}
V_{cp2} &= K_{cp} \left[ \left( \frac{A_{p2}}{A_{c2}} \right) \psi_{c2-N} \psi_{c2-N} \psi_{c2-N} \psi_{c2-N} N_b \right] \\
\phi V_{c2} &= V_{ua}
\end{align*}
\]

ACI 318-14 Eq. (17.5.3.1b)

ACI 318-14 Table 17.3.1.1

Variables

<table>
<thead>
<tr>
<th>K_{cp}</th>
<th>h_{ref} [in.]</th>
<th>e_{c2-N} [in.]</th>
<th>e_{c2-N} [in.]</th>
<th>c_{s,min} [in.]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>12.00</td>
<td>0.000</td>
<td>0.000</td>
<td>422.500</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>\psi_{c2-N}</th>
<th>c_{sc} [in.]</th>
<th>k_{c}</th>
<th>\lambda_{a}</th>
<th>f_{c} [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
<td>-</td>
<td>16</td>
<td>1.000</td>
<td>2500</td>
</tr>
</tbody>
</table>

Calculations

<table>
<thead>
<tr>
<th>A_{c2} [in.²]</th>
<th>A_{c2-N} [in.²]</th>
<th>\psi_{c2-N}</th>
<th>\psi_{c2-N}</th>
<th>\psi_{c2-N}</th>
<th>\psi_{c2-N}</th>
<th>N_b [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1681.00</td>
<td>1296.00</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>50318</td>
</tr>
</tbody>
</table>

Results

<table>
<thead>
<tr>
<th>V_{cp2} [lb]</th>
<th>\phi V_{cp2} [lb]</th>
<th>V_{ua} [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>130532</td>
<td>0.700</td>
<td>18632</td>
</tr>
</tbody>
</table>
4.3 Concrete edge failure in direction x+

\[ V_{\text{edge}} = \left( \frac{A_{VC}}{A_{VCD}} \right) \psi_{EC,V} \psi_{LY,V} \psi_{FH,V} \psi_{\text{parallel},V} V_b \]

- \( \psi_{FH,V} \geq V_u \)  
- \( A_{VC} \) see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)
- \( A_{VCD} = 4.5 c_{st}^2 \)
- \( \psi_{EC,V} = \left( \frac{1}{1 + 2c_{st} / 3c_{cd}} \right) \leq 1.0 \)
- \( \psi_{LY,V} = 0.7 + 0.3 \left( \frac{c_{cd}}{1.5c_{st}} \right) \leq 1.0 \)
- \( \psi_{FH,V} = \frac{1.5c_{st}}{h_b} \geq 1.0 \)
- \( V_b = 9 \lambda_a \lambda_y c_{st}^2 \)

Variables

\[
\begin{array}{cccc}
 c_{st} & c_{cd} & \phi_{EC} & h_b \\
 281.667 & 422.500 & 0.000 & 1.200 \\
 420.000 &
\end{array}
\]

\[
\begin{array}{cccc}
 l_b & \lambda_a & d_b & f_y \\
 7.000 & 1.000 & 0.875 & 2500 \\
 & 1.000 & & 1.000 \\
\end{array}
\]

\[ \psi_{\text{parallel},V} \]

Calculations

\[
\begin{array}{cccc}
 A_{VC} & A_{VCD} & \psi_{EC,V} & \psi_{LY,V} \\
 357000.00 & 357012.50 & 1.000 & 1.000 \\
\end{array}
\]

Results

\[
\begin{array}{cccc}
 V_{\text{edge}} & \psi_{FH,V} & \psi_{LY,V} \\
 2560179 & 0.700 & 1792125 & 18632 \\
\end{array}
\]

5 Combined tension and shear loads

\[
\begin{array}{cccc}
 \beta_N & \beta_V & \zeta & Utilization \% \\
 0.779 & 0.446 & 5/3 & 92 \\
\end{array}
\]

\[
\beta_{NV} = \beta_N + \beta_V \leq 1
\]

6 Warnings

- Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading! Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The \( \Phi \) factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- The design of Anchor Reinforcement is beyond the scope of PROFIS Anchor. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Anchor calculations.

**Fastening meets the design criteria!**
7 Installation data

Anchor plate, steel:
Profile: Square HSS (AISC): 4.000 x 4.000 x 0.500 in.
Hole diameter in the fixture: \( d_h = 4.938 \) in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Heavy Square Head ASTM F 1554 GR. 36 7/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 12.000 in.
Minimum thickness of the base material: 13.052 in.

Coordinates Anchor in.

<table>
<thead>
<tr>
<th>Anchor</th>
<th>x</th>
<th>y</th>
<th>( c_x )</th>
<th>( c_{xx} )</th>
<th>( c_y )</th>
<th>( c_{yy} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-2.500</td>
<td>-2.500</td>
<td>422.500</td>
<td>427.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
<tr>
<td>2</td>
<td>2.500</td>
<td>-2.500</td>
<td>427.500</td>
<td>422.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
<tr>
<td>3</td>
<td>-2.500</td>
<td>2.500</td>
<td>422.500</td>
<td>427.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
<tr>
<td>4</td>
<td>2.500</td>
<td>2.500</td>
<td>427.500</td>
<td>422.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
</tbody>
</table>
General Information:
=======================================

File Name: U:\453 - Senior Project\TIA ANA\SW CG 2.col
Project: Column
Code: ACI 318-11
Run Option: Investigation
Run Axis: X-axis

Material Properties:

- $f'c = 1.5$ ksi
- $f_c = 2207.6$ ksi
- Ultimate strain = 0.003 in/in
- Betal = 0.6

Slenderness: Not considered
Column Type: Structural

Section:

- Rectangular: Width = 8 in
  Depth = 240 in
  Gross section area, $A_g = 1920$ in$^2$
  Ix = $9.216e+006$ in$^4$
  rx = 69.282 in
  xo = 0 in

Reinforcement:

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in$^2$)</th>
<th>#8</th>
<th>0.38</th>
<th>0.11</th>
<th>#7</th>
<th>0.50</th>
<th>0.20</th>
<th>#5</th>
<th>0.63</th>
<th>0.31</th>
</tr>
</thead>
<tbody>
<tr>
<td># 6</td>
<td>0.75</td>
<td></td>
<td>0.88</td>
<td>0.60</td>
<td># 8</td>
<td>1.00</td>
<td>0.79</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># 9</td>
<td>1.13</td>
<td></td>
<td>1.27</td>
<td>1.27</td>
<td># 11</td>
<td>1.41</td>
<td>1.56</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td># 14</td>
<td>1.69</td>
<td></td>
<td>2.26</td>
<td>2.26</td>
<td></td>
<td>4.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

Pattern: Irregular
Total steel area: $A_s = 2.00$ in$^2$ at rho = 0.10% (Note: rho < 0.50%)
Minimum clear spacing = 23.50 in

<table>
<thead>
<tr>
<th>Area in$^2$</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in$^2$</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in$^2$</th>
<th>X (in)</th>
<th>Y (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>-108.0</td>
<td>0.20</td>
<td>0.0</td>
<td>-120.0</td>
<td>0.20</td>
<td>0.0</td>
<td>-60.0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>36.0</td>
<td>0.20</td>
<td>0.0</td>
<td>60.0</td>
<td>0.20</td>
<td>0.0</td>
<td>84.0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>108.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Control Points:

<table>
<thead>
<tr>
<th>Bending about</th>
<th>Axial Load P (kip)</th>
<th>X-Moment k-ft</th>
<th>Y-Moment k-ft</th>
<th>NA depth in</th>
<th>Dt depth in</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>1641.5</td>
<td>-0.00</td>
<td>-0.00</td>
<td>422.04</td>
<td>228.00</td>
<td>0.00138</td>
<td>0.65</td>
</tr>
<tr>
<td>@ Allowable comp.</td>
<td>1313.2</td>
<td>2616.43</td>
<td>-0.00</td>
<td>240.27</td>
<td>228.00</td>
<td>0.00015</td>
<td>0.65</td>
</tr>
<tr>
<td>@ fs = 0.0</td>
<td>1245.9</td>
<td>2993.98</td>
<td>0.00</td>
<td>228.00</td>
<td>228.00</td>
<td>0.00000</td>
<td>0.65</td>
</tr>
<tr>
<td>@ fs = 0.5*fy</td>
<td>1009.4</td>
<td>3903.27</td>
<td>0.00</td>
<td>185.38</td>
<td>228.00</td>
<td>0.00069</td>
<td>0.65</td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>843.3</td>
<td>4713.24</td>
<td>0.00</td>
<td>156.19</td>
<td>228.00</td>
<td>0.00138</td>
<td>0.65</td>
</tr>
<tr>
<td>@ Tension control</td>
<td>606.4</td>
<td>4800.01</td>
<td>0.00</td>
<td>85.50</td>
<td>228.00</td>
<td>0.00500</td>
<td>0.90</td>
</tr>
<tr>
<td>@ Pure bending</td>
<td>0.00</td>
<td>695.42</td>
<td>0.00</td>
<td>9.41</td>
<td>228.00</td>
<td>0.06969</td>
<td>0.90</td>
</tr>
<tr>
<td>@ Max tension</td>
<td>-72.0</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>228.00</td>
<td>9.99999</td>
<td>0.90</td>
</tr>
</tbody>
</table>

- X @ Max compression
| @ Allowable comp. | 1313.2             | -2616.43      | 0.00          | 240.27      | 228.00      | 0.00015 | 0.65 |
| @ fs = 0.0     | 1245.9             | -2993.98      | 0.00          | 228.00      | 228.00      | 0.00000 | 0.65 |
| @ fs = 0.5*fy  | 1009.4             | -3903.27      | 0.00          | 185.38      | 228.00      | 0.00069 | 0.65 |
| @ Balanced point | 843.3             | -4713.24      | 0.00          | 156.19      | 228.00      | 0.00138 | 0.65 |
| @ Tension control | 606.4             | -4800.01      | 0.00          | 85.50       | 228.00      | 0.00500 | 0.90 |
| @ Pure bending  | 0.00               | -695.42       | 0.00          | 9.41        | 228.00      | 0.06969 | 0.90 |
| @ Max tension  | -72.0              | 0.00          | 0.00          | 0.00        | 228.00      | 9.99999 | 0.90 |

**End of output***
General Information:

- File Name: u:\453 - senior project\tia ana\sw 2.col
- Column:
  - Code: ACI 318-11
  - Run Option: Investigation
  - Run Axis: X-axis
- Material Properties:
  - $f'c = 1.5 \text{ ksi}$
  - $Ec = 22076 \text{ ksi}$
  - Ultimate strain = 0.003 in/in
  - Beta1 = 0.8
- Section:
  - Rectangular: Width = 8 in, Depth = 304 in
  - Gross section area, $A_g = 2432 \text{ in}^2$
  - $I_x = 1.87296 \times 10^5 \text{ in}^4$
  - $rx = 87.7572 \text{ in}$
  - $Xo = 0 \text{ in}$
- Reinforcement:
  - Bar Set: ASTM A615
  - Size Diam (in)  Area (in$^2$)  Size Diam (in)  Area (in$^2$)  Size Diam (in)  Area (in$^2$)
  - # 3 0.38 0.11  # 4 0.50 0.20  # 5 0.63 0.31
  - # 6 0.75 0.44  # 7 0.88 0.60  # 8 1.00 0.79
  - # 9 1.13 1.00  # 10 1.27 1.27  # 11 1.41 1.56
  - # 14 1.69 2.25  # 18 2.26 4.00
- Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
- $\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65$
- Pattern: Irregular
- Total steel area: $A_s = 2.80 \text{ in}^2$ at $\rho_o = 0.12\%$ (Note: $\rho < 0.50\%$)
- Minimum clear spacing = 15.50 in

Control Points:

Bending about:

<table>
<thead>
<tr>
<th>Axial Load P kip</th>
<th>X-Moment k-ft</th>
<th>Y-Moment NA depth in</th>
<th>X depth in</th>
<th>Y depth in</th>
<th>$\varepsilon_t$</th>
<th>$\Phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>2086.0</td>
<td>-0.00</td>
<td>-555.32</td>
<td>300.00</td>
<td>-0.00138</td>
<td>0.650</td>
</tr>
<tr>
<td>Allowable comp.</td>
<td>1668.8</td>
<td>-4223.91</td>
<td>-0.00</td>
<td>304.69</td>
<td>300.00</td>
<td>-0.00005</td>
</tr>
<tr>
<td>@ fs = 0.0</td>
<td>1643.0</td>
<td>-4417.69</td>
<td>-0.00</td>
<td>300.00</td>
<td>300.00</td>
<td>0.00000</td>
</tr>
<tr>
<td>@ fs = 0.5*fy</td>
<td>1331.2</td>
<td>-6149.72</td>
<td>-0.00</td>
<td>243.93</td>
<td>300.00</td>
<td>0.00069</td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>1111.9</td>
<td>-6728.00</td>
<td>-0.00</td>
<td>205.51</td>
<td>300.00</td>
<td>0.00138</td>
</tr>
<tr>
<td>@ Tension control</td>
<td>801.2</td>
<td>-7991.03</td>
<td>-0.00</td>
<td>112.50</td>
<td>300.00</td>
<td>0.00500</td>
</tr>
<tr>
<td>@ Pure bending</td>
<td>0.0</td>
<td>1238.02</td>
<td>-0.00</td>
<td>11.80</td>
<td>300.00</td>
<td>0.07330</td>
</tr>
<tr>
<td>@ Max tension</td>
<td>-100.8</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>300.00</td>
<td>9.99999</td>
</tr>
</tbody>
</table>

* End of output ***
General Information:
======================

File Name: U:\453 - Senior Project\TIA ANA\SW 3.col

Project:
--------

Code: ACI 318-11
Units: English

Material Properties:
=====================

f'c = 1.5 ksi
Ec = 2076.6 ksi
Ultimate strain = 0.003 in/in
Beta = 0.8

Section:
-------

Rectangular: Width = 8 in
Depth = 240 in

Gross section area, Ag = 1920 in^2
Iy = 10240 in^4
rx = 69.282 in
Xy = 2.3094 in
Yo = 0 in

Reinforcement:
---------------

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 3</td>
<td>0.38</td>
<td># 4</td>
<td>0.50</td>
</tr>
<tr>
<td># 6</td>
<td>0.75</td>
<td># 7</td>
<td>0.88</td>
</tr>
<tr>
<td># 9</td>
<td>1.13</td>
<td># 10</td>
<td>1.27</td>
</tr>
<tr>
<td># 14</td>
<td>1.69</td>
<td># 12</td>
<td>2.26</td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Pattern: Irregular

Total steel area: As = 2.00 in^2 at rho = 0.10% (Note: rho < 0.50%)

Minimum clear spacing = 25.30 in

Control Points:
-----------------

Bending about X @ Max compression:

Axial Load P \( \text{kip} \)
X-Moment \( \text{ft-kip} \)
Y-Moment \( \text{ft-kip} \)
NA depth \( \text{in} \)
Dt depth \( \text{in} \)
\( \text{eps}_T \)
Phi

- X @ Max compression:

- X @ Max compression:

** End of output **
General Information:

- Project: ACI 318-11
- Code: 8
- Run Option: Investigation
- Run Axis: X-axis
- Slenderness: Not considered
- Column Type: Structural

Material Properties:

\[ f'c = 1.5 \text{ ksi} \quad f_y = 40 \text{ ksi} \]
\[ E_c = 2207.6 \text{ ksi} \quad E_s = 29000 \text{ ksi} \]
Ultimate strain = 0.003 in/in
Beta = 0.8

Section:

- Rectangular: Width = 8 in
  Depth = 144 in
  Gross section area, \( A_g = 1152 \text{ in}^2 \)
  \( I_x = 1.99066e+006 \text{ in}^4 \)
  \( I_y = 6144 \text{ in}^4 \)
  \( r_x = 41.5692 \text{ in} \)
  \( r_y = 2.3094 \text{ in} \)
  \( X_0 = 0 \text{ in} \)
  \( Y_0 = 0 \text{ in} \)

Reinforcement:

- Bar Set: ASTM A615
- Size Diam (in) | Area (in^2) | Size Diam (in) | Area (in^2) | Size Diam (in) | Area (in^2)
- # 3 | 0.38 | 0.11 | # 4 | 0.50 | 0.20 | # 5 | 0.63 | 0.31
- # 6 | 0.75 | 0.44 | # 7 | 0.88 | 0.60 | # 8 | 1.00 | 0.79
- # 9 | 1.13 | 1.00 | # 10 | 1.27 | 1.27 | # 11 | 1.41 | 1.56
- # 14 | 1.69 | 2.25 | # 18 | 2.26 | 4.00

Confinement: Tied: #3 ties with #10 bars, #4 with larger bars.
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Pattern: Irregular

Total steel area: \( A_s = 1.20 \text{ in}^2 \) at rho = 0.10% (Note: rho < 0.50%)
Minimum clear spacing = 23.50 in

<table>
<thead>
<tr>
<th>Area in^2</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in^2</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in^2</th>
<th>X (in)</th>
<th>Y (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>-60.0</td>
<td>0.20</td>
<td>0.0</td>
<td>-36.0</td>
<td>0.20</td>
<td>0.0</td>
<td>-12.0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>12.0</td>
<td>0.20</td>
<td>0.0</td>
<td>36.0</td>
<td>0.20</td>
<td>0.0</td>
<td>60.0</td>
</tr>
</tbody>
</table>

Control Points:

<table>
<thead>
<tr>
<th>Bending about</th>
<th>Axial Load P kip</th>
<th>X-Moment k-ft</th>
<th>Y-Moment k-ft</th>
<th>NA depth in</th>
<th>Dt depth in</th>
<th>eps_t in</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>984.9</td>
<td>0.00</td>
<td>-0.00</td>
<td>244.34</td>
<td>132.00</td>
<td>-0.0138</td>
</tr>
<tr>
<td>@ Allowable comp.</td>
<td>787.9</td>
<td>941.32</td>
<td>-0.00</td>
<td>144.17</td>
<td>132.00</td>
<td>-0.0025</td>
</tr>
<tr>
<td>@ fs = 0.0</td>
<td>721.2</td>
<td>1155.40</td>
<td>-0.00</td>
<td>132.00</td>
<td>132.00</td>
<td>0.00000</td>
</tr>
<tr>
<td>@ fs = 0.5fy</td>
<td>583.8</td>
<td>1435.02</td>
<td>-0.00</td>
<td>107.33</td>
<td>132.00</td>
<td>0.00069</td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>486.7</td>
<td>1507.75</td>
<td>-0.00</td>
<td>90.43</td>
<td>132.00</td>
<td>0.00138</td>
</tr>
<tr>
<td>@ Tension control</td>
<td>349.6</td>
<td>1689.80</td>
<td>-0.00</td>
<td>49.50</td>
<td>132.00</td>
<td>0.00500</td>
</tr>
<tr>
<td>@ Pure bending</td>
<td>0.0</td>
<td>250.78</td>
<td>-0.00</td>
<td>5.88</td>
<td>132.00</td>
<td>0.06431</td>
</tr>
<tr>
<td>@ Max tension</td>
<td>-43.2</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>9.99999</td>
</tr>
</tbody>
</table>

-X @ Max compression | 984.9            | 0.00          | -0.00         | 244.34      | 132.00      | -0.0138   |
| @ Allowable comp. | 787.9            | -941.32       | 0.00          | 144.17      | 132.00      | -0.0025   |
| @ fs = 0.0      | 721.2            | -1155.40      | 0.00          | 132.00      | 132.00      | 0.00000   |
| @ fs = 0.5fy    | 583.8            | -1435.02      | 0.00          | 107.33      | 132.00      | 0.00069   |
| @ Balanced point| 486.7            | -1507.75      | 0.00          | 90.43       | 132.00      | 0.00138   |
| @ Tension control| 349.6           | -1689.80      | 0.00          | 49.50       | 132.00      | 0.00500   |
| @ Pure bending  | 0.0              | -250.78       | 0.00          | 5.88        | 132.00      | 0.06431   |
| @ Max tension   | -43.2            | 0.00          | 0.00          | 0.00        | 0.00        | 9.99999   |

*** End of output ***
General Information:

File Name: U:\453 - Senior Project\TIA ANA\SW 5.2.col

Project:
Code: ACI 318-11
Run Option: Investigation
Run Axis: X-axis

Material Properties:

- f'c = 1.5 ksi
- Ec = 22076 ksi
- Ultimate strain = 0.003 in/in
- Beta = 0.8

Section:

- Rectangular: Width = 8 in
  Depth = 304 in

- Gross section area, Ag = 2432 in^2
- Ix = 1.87296e+007 in^4
- rx = 87.7572 in
- Xo = 0 in

Reinforcement:

- Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Diam (in)</th>
<th>Area (in^2)</th>
<th>Diam (in)</th>
<th>Area (in^2)</th>
<th>Diam (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3 0.38</td>
<td>0.11</td>
<td>#4 0.50</td>
<td>0.20</td>
<td>#5 0.63</td>
<td>0.31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#6 0.75</td>
<td>0.44</td>
<td>#7 0.88</td>
<td>0.60</td>
<td>#8 1.00</td>
<td>0.79</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#9 1.13</td>
<td>1.00</td>
<td>#10 1.27</td>
<td>1.27</td>
<td>#11 1.41</td>
<td>1.56</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#14 1.69</td>
<td>2.25</td>
<td>#18 2.26</td>
<td>4.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Pattern: Irregular

Total steel area: As = 2.80 in^2 at rho = 0.12% (Note: rho < 0.50%) Minimum clear spacing = 15.50 in

Area in^2 | X (in) | Y (in) | Area in^2 | X (in) | Y (in) |
----------|--------|--------|----------|--------|--------|
| 0.20     | 0.0    | -148.0 | 0.20     | 0.0    | -132.0 |
| 0.20     | 0.0    | -84.0  | 0.20     | 0.0    | -60.0  |
| 0.20     | 0.0    | -12.0  | 0.20     | 0.0    | 12.0   |
| 0.20     | 0.0    | 60.0   | 0.20     | 0.0    | 84.0   |
| 0.20     | 0.0    | 132.0  | 0.20     | 0.0    | 148.0  |

Control Points:

Bending about:

<table>
<thead>
<tr>
<th>X @ Max compression</th>
<th>Axial Load P</th>
<th>X-Moment</th>
<th>Y-Moment</th>
<th>NA depth</th>
<th>Dt depth</th>
<th>eps_t</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>2086.0</td>
<td>0.00</td>
<td>-0.00</td>
<td>555.32</td>
<td>300.00</td>
<td>-0.00138</td>
<td>0.650</td>
</tr>
<tr>
<td>X @ Allowable comp.</td>
<td>1668.8</td>
<td>4223.91</td>
<td>-0.00</td>
<td>304.69</td>
<td>300.00</td>
<td>0.00005</td>
<td>0.650</td>
</tr>
<tr>
<td>X @ fs = 0.0</td>
<td>1643.0</td>
<td>4417.69</td>
<td>-0.00</td>
<td>300.00</td>
<td>300.00</td>
<td>0.00000</td>
<td>0.650</td>
</tr>
<tr>
<td>X @ fs = 0.5*fy</td>
<td>1331.3</td>
<td>5149.72</td>
<td>-0.00</td>
<td>243.93</td>
<td>300.00</td>
<td>0.00018</td>
<td>0.650</td>
</tr>
<tr>
<td>X @ Balanced point</td>
<td>801.2</td>
<td>7991.03</td>
<td>-0.00</td>
<td>112.50</td>
<td>300.00</td>
<td>0.00500</td>
<td>0.900</td>
</tr>
<tr>
<td>X @ Pure bending</td>
<td>0.0</td>
<td>1238.02</td>
<td>-0.00</td>
<td>11.80</td>
<td>300.00</td>
<td>0.07330</td>
<td>0.900</td>
</tr>
<tr>
<td>X @ Max tension</td>
<td>-100.8</td>
<td>0.00</td>
<td>0.00</td>
<td>300.00</td>
<td>9.999999</td>
<td>0.900</td>
<td></td>
</tr>
</tbody>
</table>

* End of output **
General Information:
---------------------

File Name: U:\453 - Senior Project\TIA ANA\SW 8.col

Project:
Code: ACI 318-11
Units: English

Run Option: Investigation
Run Axis: X-axis

Material Properties:
---------------------

\( f'c = 1.5 \text{ ksi} \)
\( f_y = 40 \text{ ksi} \)
\( E_c = 2072.5 \text{ ksi} \)
\( E_s = 29000 \text{ ksi} \)
\( \beta_2 = 0.8 \)

Section:
---------

Rectangular: Width = 8 in  Depth = 448 in

\( A_g = 3584 \text{ in}^2 \)
\( I_x = 5.99436e+007 \text{ in}^4 \)
\( I_y = 19114.7 \text{ in}^4 \)
\( I_{xy} = 2.3094 \text{ in}^4 \)
\( I_0 = 0 \text{ in} \)

Reinforcement:
---------------

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Dia (in)</th>
<th>Area (in^2)</th>
<th>Size Dia (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#3</td>
<td>0.38</td>
<td>#4</td>
<td>0.50</td>
</tr>
<tr>
<td>#6</td>
<td>0.75</td>
<td>#7</td>
<td>0.88</td>
</tr>
<tr>
<td>#9</td>
<td>1.13</td>
<td>#10</td>
<td>1.27</td>
</tr>
<tr>
<td>#14</td>
<td>1.69</td>
<td>#18</td>
<td>2.26</td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Pattern: Irregular

Total steel area: \( A_s = 2.00 \text{ in}^2 \) at \( \rho = 0.06\% \) (Note: \( \rho < 0.50\% \))

Minimum clear spacing = 23.50 in

Control Points:
----------------

Bending about X @ Max compression:
- Axial Load P = 320.6 kip
- X-Moment = 0.00 k-ft
- Y-Moment = -0.00 k-ft
- NA depth = 428.00 in
- Dt depth = -0.00 in
- \( \varepsilon_t = 0.00138 \)
- Phi = 0.65

-X @ Max compression:
- Axial Load P = -320.6 kip
- X-Moment = -0.00 k-ft
- Y-Moment = -0.00 k-ft
- NA depth = 428.00 in
- Dt depth = -0.00 in
- \( \varepsilon_t = 0.00138 \)
- Phi = 0.65

** End of output **
General Information:

File Name: U:\453 - Senior Project\TIA ANA\SW AI 1.col

Section:
Rectangular: Width = 8 in  Depth = 272 in

Material Properties:

- f'c = 1.5 ksi
- Ec = 22076.6 ksi
- fy = 40 ksi
- Es = 29000 ksi
- Ultimate strain = 0.003 in/in
- Beta = 0.8

Section:

Reinforcement:

Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
<th>Size Diam (in)</th>
<th>Area (in^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td># 3</td>
<td>0.38</td>
<td># 4</td>
<td>0.50</td>
<td># 5</td>
<td>0.63</td>
</tr>
<tr>
<td># 6</td>
<td>0.75</td>
<td># 7</td>
<td>0.88</td>
<td># 8</td>
<td>1.00</td>
</tr>
<tr>
<td># 9</td>
<td>1.13</td>
<td># 10</td>
<td>1.27</td>
<td># 11</td>
<td>1.41</td>
</tr>
<tr>
<td># 14</td>
<td>1.69</td>
<td># 18</td>
<td>2.26</td>
<td></td>
<td>1.56</td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

Pattern: Irregular

Steel area: As = 2.40 in^2 at rho = 0.11% (Note: rho < 0.50%)

Minimum clear spacing = 23.50 in

Control Points:

Bending about X @ Max compression: 1863.8 kip, Y-Moment NA depth: 268.00 in

Axial Load P kip, X-Moment k-ft, Y-Moment k-ft, NA depth in, Dt depth in, epa_t, Phi

** End of output **
General Information:

File Name: U:\453 - Senior Project\TIA ANA\SW AI 2.col
Project:
Code: ACI 318-11
Run Option: Investigation
Run Axis: X-axis
Engineer: Slenderness: Not considered
Units: English
Column Type: Structural

Material Properties:

f'c = 1.5 ksi
Ec = 2207.6 ksi
Ultimate strain = 0.003 in/in
Betal = 0.8
fy = 40 ksi
Es = 29000 ksi

Section:

Rectangular: Width = 8 in
Depth = 296 in

Gross section area, Ag = 2368 in^2
Ix = 1.72899e+007 in^4
Iy = 12629.3 in^4
rx = 85.4478 in
ry = 2.3094 in
Xo = 0 in
Yo = 0 in

Reinforcement:

Bar Set: ASTM A615
Size Diam (in) Area (in^2) Size Diam (in) Area (in^2) Size Diam (in) Area (in^2)
# 3 0.38 0.11 # 4 0.50 0.20 # 5 0.63 0.31
# 6 0.75 0.44 # 7 0.88 0.60 # 8 1.00 0.79
# 9 1.13 1.00 # 10 1.27 1.27 # 11 1.41 1.56
# 14 1.69 2.25 # 18 2.26 4.00

Confiment: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Pattern: Irregular
Total steel area: As = 2.40 in^2 at rho = 0.10% (Note: rho < 0.50%)
Minimum clear spacing = 23.50 in

Area in^2 X (in) Y (in) Area in^2 X (in) Y (in) Area in^2 X (in) Y (in)
0.20 0.0 136.0 0.20 0.0 112.0 0.20 0.0 88.0
0.20 0.0 64.0 0.20 0.0 40.0 0.20 0.0 12.0
0.20 0.0 12.0 0.20 0.0 40.0 0.20 0.0 64.0
0.20 0.0 88.0 0.20 0.0 112.0 0.20 0.0 136.0

Control Points:

Bending about Axial Load F X-Moment Y-Moment NA depth Dt depth eps_t Phi

X @ Max compression
@ Allowable comp. 2022.9 0.00 -0.00 525.70 284.00 -0.00138 0.650
@ fs = 0.0 1618.3 3978.39 -0.00 296.39 284.00 -0.00013 0.650
@ fs = 0.5fy 1550.5 4453.48 -0.00 284.00 284.00 0.00000 0.650
@ Balanced point 1256.5 5896.64 -0.00 230.92 284.00 0.00069 0.650
@ Tension control 756.0 7344.82 -0.00 106.50 284.00 0.00138 0.650
@ Pure bending -0.0 1030.32 -0.00 10.98 284.00 0.07458 0.900
@ Max tension -86.4 0.00 0.00 284.00 9.99999 0.900

-X @ Max compression
@ Allowable comp. 2022.9 -0.00 -0.00 525.70 284.00 -0.00138 0.650
@ fs = 0.0 1618.3 -3978.39 0.00 296.39 284.00 -0.00013 0.650
@ fs = 0.5fy 1550.5 -4453.47 0.00 284.00 284.00 0.00000 0.650
@ Balanced point 1256.5 -5896.64 0.00 230.92 284.00 0.00069 0.650
@ Tension control 756.0 -7344.82 0.00 106.50 284.00 0.00138 0.650
@ Pure bending -0.0 -1030.32 0.00 10.98 284.00 0.07458 0.900
@ Max tension -86.4 0.00 0.00 284.00 9.99999 0.900

**End of output***
General Information:

Title Name: U:\453 - Senior Project\TIA ANA\SW CG 1.col

Project:
  Code: ACI 318-11
  Units: English
  Run Option: Investigation
  Run Axis: X-axis
  Slenderness: Not considered
  Column Type: Structural

Material Properties:

- $f'c = 1.5$ ksi
- $f_y = 40$ ksi
- $E_c = 2207.6$ ksi
- $E_s = 29000$ ksi
- Ultimate strain = 0.003 in/in
- $\beta_{sl} = 0.8$

Section:

- Rectangular: Width = 8 in, Depth = 272 in
  - Gross section area, $A_g = 2176$ in$^2$
  - $I_x = 13415858.007$ in$^4$
  - $I_y = 116053.3$ in$^4$
  - $r_x = 78.5196$ in
  - $r_y = 2.3094$ in
  - $x_0 = 0$ in

Reinforcement:

- Bar Set: ASTM A615
- Size Diam (in) Area (in$^2$) | Size Diam (in) Area (in$^2$) | Size Diam (in) Area (in$^2$)
  - #3 0.38 0.11 | #4 0.50 0.20 | #5 0.63 0.31
  - #6 0.75 0.44 | #7 0.88 0.60 | #8 1.00 0.79
  - #9 1.13 1.00 | #10 1.27 1.27 | #11 1.41 1.56
  - #14 1.69 2.25 | #18 2.26 4.00

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
- $\phi(a) = 0.8$, $\phi(b) = 0.9$, $\phi(c) = 0.65$

Pattern: Irregular
- Total steel area: $A_s = 2.40$ in$^2$ at $\rho = 0.11\%$ (Note: $\rho < 0.50\%$)
- Minimum clear spacing = 23.50 in

Area in$^2$ X (in) Y (in) Area in$^2$ X (in) Y (in) Area in$^2$ X (in) Y (in)
- 0.20 0.0 132.0 0.20 0.0 108.0 0.20 0.0 84.0
- 0.20 0.0 60.0 0.20 0.0 36.0 0.20 0.0 12.0
- 0.20 0.0 -12.0 0.20 0.0 -36.0 0.20 0.0 -60.0
- 0.20 0.0 -84.0 0.20 0.0 -108.0 0.20 0.0 -132.0

Control Points:

- Bending about X
  - @ Max compression 1863.8 0.00 0.00 496.08 268.00 -0.00138 0.650
  - @ Allowable comp. 1491.0 3373.87 -0.00 272.48 268.00 -0.00005 0.650
  - @ fs = 0.0 1466.4 3538.55 -0.00 268.00 268.00 0.000000 0.650
  - @ fs = 0.5*fy 1186.4 4910.42 -0.00 217.91 268.00 0.000069 0.650
  - @ Balanced point 993.4 5362.30 -0.00 183.59 268.00 0.00138 0.650
  - @ Tension control 715.8 6354.19 -0.00 100.50 268.00 0.00500 0.900
  - @ Pure bending 0.0 950.88 -0.00 9.84 268.00 0.07874 0.900
  - @ Max tension -86.4 0.00 0.00 0.00 268.00 9.99999 0.900

- Bending about Y
  - @ Max compression 1863.8 0.00 0.00 496.08 268.00 -0.00138 0.650
  - @ Allowable comp. 1491.0 -3373.87 0.00 272.48 268.00 -0.00005 0.650
  - @ fs = 0.0 1466.4 -3538.55 0.00 268.00 268.00 0.000000 0.650
  - @ fs = 0.5*fy 1186.4 -4910.42 0.00 217.91 268.00 0.000069 0.650
  - @ Balanced point 993.4 -5362.30 0.00 183.59 268.00 0.00138 0.650
  - @ Tension control 715.8 -6354.19 0.00 100.50 268.00 0.00500 0.900
  - @ Pure bending 0.0 -950.88 0.00 9.84 268.00 0.07874 0.900
  - @ Max tension -86.4 0.00 0.00 0.00 268.00 9.99999 0.900

** End of output ***
# Shallow VERCOR™

## Allowable Uniform Loads (psf)

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| DOUBLE 26              |       |       |       |       |       |       |       |       |       |       |       |
| Stress 300             | 300   | 300   | 280   | 195   | 143   | 110   | 87    | 70    | 58    | 49    |       |
| L360                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L240                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L180                   | 241   | 123   | 71    | 45    | 30    | 21    | 15    | 12    | 9     | 7     | 7     |
| Stress 300             | 300   | 300   | 280   | 195   | 143   | 110   | 87    | 70    | 58    | 49    |       |
| L360                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L240                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L180                   | 241   | 123   | 71    | 45    | 30    | 21    | 15    | 12    | 9     | 7     | 7     |
| Stress 300             | 300   | 300   | 280   | 195   | 143   | 110   | 87    | 70    | 58    | 49    |       |
| L360                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L240                   | 237   | 121   | 70    | 44    | 30    | 21    | 15    | 11    | 9     | 7     | 7     |
| L180                   | 241   | 123   | 71    | 45    | 30    | 21    | 15    | 12    | 9     | 7     | 7     |

| TRIPLE 24              |       |       |       |       |       |       |       |       |       |       |       |
| Stress 300             | 300   | 300   | 300   | 243   | 179   | 137   | 108   | 88    | 72    | 61    |       |
| L360                   | 278   | 143   | 82    | 52    | 35    | 24    | 18    | 13    | 10    | 7     | 7     |
| L240                   | 186   | 95    | 55    | 35    | 23    | 16    | 12    | 9     | 7     | 7     |       |
| L180                   | 278   | 143   | 82    | 52    | 35    | 24    | 18    | 13    | 10    | 7     | 7     |

See footnotes on page 146.
### Shallow VERCOR™

- **36/7 Screw Pattern at Supports**
  #12 or #14 SDI Recognized Screws to Supports 0.0385" and thicker
- **Sidelaps Connected with #10 Screws**

#### REFERENCES

**DECKING SPECIFICATIONS**

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#### Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F ((in/lb)x10^6)

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<th>SPAN (ft-in)</th>
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<td>F</td>
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See footnotes on page 145.
STRUCTURAL CALCULATIONS

FOR

TIA ANA'S SUSTAINABLE DEVELOPMENT

CASITA

JUNE 22, 2016

PREPARED BY:

DANIELLE RUSTAGI

NOT FOR CONSTRUCTION, TO BE REVIEWED AND APPROVED BY IN COUNTRY ENGINEER.
INTRODUCTION.................................................................2
GRAVITY LOAD TAKE-OFF..................................................3
GRAVITY FRAMING DESIGN..................................................4
SEISMIC DESIGN FORCES..................................................11
ROOF DIAPHRAGM DESIGN..................................................12
SHEAR WALL DESIGN.........................................................15
    IN PLANE.................................................................17
    OUT OF PLANE...........................................................27
FOUNDATION DESIGN.......................................................29
SCREEN WALL OUT OF PLANCE CHECK..................................32
COLUMN ANCHORAGE DESIGN.............................................35
SHEAR WALL CAPACITIES FROM SP COLUMN..........................41
PROJECT DESCRIPTION

Tia Ana’s Sustainable Development is located in the El Salvadorian countryside, just outside of the capital city San Salvador. Currently, Tia Ana’s is located in the city of San Salvador, where there is little room for the children and a significant amount of violence and instability. In order to make the relocation of the orphanage feasible financially, a portion of the countryside landscape will contain a small hotel and numerous small dwellings nested in the tropical surroundings. By relocating the orphanage to the countryside, the children housed by Tia Ana will experience a safer and more stable environment and this sustainable development would be the home of a new kind of tourist destination in El Salvador.

CONSTRUCTION SEQUENCE

The building materials used for each structure are identical, cast in place concrete, CMU masonry blocks, architectural masonry blocks, reinforcing steel bars, steel tube framing, and corrugated metal decking. With these six main materials, I was able to develop a consistent framing system from one structure to the next. After preparing the site for construction, each structure begins with a poured concrete foundation consisting of isolated (pad) footings and slab on grade. In order to connect the concrete floor slab to the masonry walls, reinforcing steel must be embedded into the slab and hooked so the CMU blocks can be set with reinforcement in the proper cells. The structural and architectural masonry walls require fully grouted cells and reinforcement to resist the lateral loads that come with being in an area of high seismic activity. Once the walls are constructed, anchor bolts embedded in the top course of the CMU wall will connect tube steel columns to the walls below. From these columns, tube steel girders and beams can be welded to form the roof framing. The final step in the construction sequence is to fasten corrugated metal decking to the roof framing system. This sequence can be used to construct buildings as small as the Casita’s where the total square footage is only 620 sqft all the way up to structure as large as Tia Ana’s where the building footprint is just under 8,000 sqft.

DESIGN CRITERIA

When determining the lateral loads on the structure, I assumed seismic would govern. I was able to make this assumption because the site is located on the side of an active volcano in a high seismic zone; the design of each structure is such that architectural masonry walls provide enough openings to prevent a buildup of wind pressure on the structural walls and the surrounding trees and other natural features protect the buildings from high winds. Using the USGS Worldwide Beta, I found the Sds and Sd1 values for the site were 1.72 and 0.96 respectively to produce a Cs values of 0.344. These values are comparable to places in California with high seismic activity such as Los Angeles and San Francisco; both of which sit on major known faults. For this reason I can justify using the same building codes as those used in California and the United States for design.

These codes are as follows: 2012 International Building Code, 2010 American Society of Civil Engineers 7-10, American Institute of Steel Construction (14th Edition), The 2013 Masonry Specification 402, and 2014 American Concrete Institute 318.
ROOF SECTION:

(Vertex)

(ROOF LIVE LOAD = 20 psf)

DEAD LOAD TAKEOFF

2.4 CA: SHALLOW VERCO DECK
RIGID INSULATION (2")
VAPOR BARRIER
TOTAL TO DECK
BERMS (HSS 6"x6"x3/4") @ 3.5' O.C.
TOTAL TO BEAMS
GIRDERS (HSS 7"x7"x3/4")
TOTAL TO GIRDERS
COLUMNS (HSS)
DECKING

\[ w_g = 5.8 \text{ psf} \]
\[ w_L = 20 \text{ psf} \]
\[ w_u = D + L \]

1/180 DEFLECTION CRITERIA, 8'-0" SPAN, 3" SPANS

\[ w_{allowable} = 6.9 \text{ psf} \]

\[ GPSE > 24 \text{ psf} \]

24 GA SHALLOW VEECO DECKING

PASSES FOR GRAVITY LOADS

BEAMS @ 3'-0" D.C.

\[ W = W_u + W_L \]
\[ DL = 14.4 \text{ psf} \]
\[ LL = 20 \text{ psf} \]

\[ W_{u_l} = 1.2 D + 1.0 L \]
\[ W_{u_t} = 1.4 D \]

(1) (UPPER CONDITION)

(VIA RISA 2D ENVELOPE ANALYSIS)

\[ M_u = 60.0 \text{ klf} \]
\[ V_u = 1.4 \text{ k} \]

RNXS:

A = 2.67 k / -5.68 k ft (TORSION)
B = 1.29 k
C = 1.64 k / 1.15 k ft (TORSION)
BEAM CAPACITY

TRY HSS C6 x 6 x 3/4:

\[ \phi M_n = 38.7 \text{ k-ft} \]

\[ \phi M_n \geq M_n \]

\[ 38.7 \text{ k-ft} \geq 0 \text{ k-ft} \]

\[ \phi V_n = \frac{0.6 A_w F_y C_v}{C_n} \]

\[ C_n = ? \]

\[ C_n = 22.8 \]

\[ \frac{1.10 f_{y} E}{E_y} = 1.1 \frac{0.825 (0.025)}{180} = 61.70 \]

\[ 1.10 \frac{f_{y} E}{E_y} = 118 \frac{0.825 (0.025)}{180} = 61.70 \]

\[ \phi V_n = 28.72 \text{ k} \]

\[ \phi V_n > V_n \]

\[ 28.72 > 14 \text{ k} \]

PASSES IN SHEAR

\[ \Delta = \frac{4}{180} \]

\[ \Delta_{max} = 0.900'' \text{ (via PISA) @ CANTILEVER (L=8.5')} \]

\[ L_{EFR} = 2L = 17' \]

\[ \Delta = \frac{17'(12'')}{180} = 1.133'' \]

\[ \Delta_{max} > \Delta_{max} \]

1.133'' > 0.900''

PASSES IN DEFLECTION

HSS C6 x 6 x 3/4 IS SUFFICIENT

IN SHEAR, FLEXURE, AND DEFLECTION

AS A ROOF BEAM @ 3'-6" O.C.
Roof Beam 1
Shear Envelope (Kips)

Moment Envelope (Kip-Feet)
GIRDER (A" FROM BEAM FBD)

TRY HSS 7"x7"x 3/4"

\[ W = \frac{2.67''(T)}{27'} = 0.692 \text{ klf} \]

\[ W = 0.692 \text{ klf} \]

\[ + \left\{ \begin{array}{l}
5.08' \text{ k-ft moment} \\
@ 3'6" \text{o.c. from beams (T)}
\end{array} \right\} \]

TORSONAL CAPACITY

\[ \phi T_n = \phi F_{cr} \]

\[ \phi = 0.9 \]

\[ F_{cr} = 0.6 F_T \]

\[ W/T < 2.45 \sqrt{E/\alpha^2} \]

\[ 27 < 2.45 \sqrt{19000/410^2} \]

\[ 27 < 21.52 \]

\[ F_{cr} = 0.6(410) \]

\[ C = 2(0.75)(0.75)(0.25) = 22.72 \]

\[ \phi T_n = 0.9(27.6)(22.72) \]

\[ \phi T_n < T_n \]

\[ 504 \text{ k-ft} \geq 77(5.68) = 59.76 \text{ k-ft} \]

\[ T_n < 0.2 \phi T_n \]

RESULTS FROM PISA

\[ M_u = 9.22 \text{ k-ft} \]

\[ V_u = 4.72 \text{ k} \]

GIRDER CAPACITY

\[ \phi R_n = 53.10 \text{ k-ft} \]

\[ \phi R_n \geq M_u \]

\[ 53.10 \text{ k-ft} \geq 9.22 \text{ k-ft} \]
GIRDER CAPACITY

\[ \phi V_n = \phi V_t \cdot (1.1 \sqrt{E/F_y}) \cdot C_v \]

\[ C_v = 1 \]
\[ W_{tw} = 27 \]

\[ 1.1 \sqrt{E/F_y} = 51.7 \]

\[ h/w_{tw} \leq 1.1 \sqrt{E/F_y} \]
\[ \phi V_n = 0.9(0.6)(16)(5.875)(0.25)(1.0) \]
\[ \phi V_n = 36.48 \text{ k} \]

\[ \phi V_n \leq V_n \]
\[ 36.48 \text{ k} \geq \]

\[ \Delta_{act} = 0.225 \text{ in} \]
\[ \Delta_{all} = \frac{L}{180} = \frac{13.867}{180} = 0.091 \]
\[ \Delta_{all} = 0.911 \]

\[ \Delta_{act} \leq \Delta_{all} \]
\[ 0.225 \leq 0.911 \]

HSS 7x7 x \frac{7}{16} passes in shear, deflection, and flexure, as a roof girder.
Cantilever Roof Beam

Shear Envelope (Kips)

Moment Envelope (Kip-Feet)
COLUMN DESIGN (GRID: B2)

P_u = 7.67 kN (via FISA)
L = 12.7 m, k = 1.0 (PIN - PIN)

ΦPin = 38 kN/m (EL = 13') ↔ HSS 4×4×1/8
SEISMIC DESIGN VALUES

\[ V = C_s \cdot W \]

\[ C_s = \frac{S_{D2}}{T_{1A}} \cdot \frac{S_{D1}}{T_{1A}} \]

- \( R = 5 \) (SPECIAL REINF. MASONRY SW)
- \( T_{1A} = 1.0 \) (CAT. II BUILD)
- \( S_{D2} = 1.72 \text{ g} \)
- \( S_{D1} = 0.69 \text{ g} \)

\[ T_a = C_t \cdot h_a \cdot 0.75 \]

\[ C_t = 0.02 \quad h_a = 13.25' \quad z = 0.75 \]

\[ T_a = 0.14' \]

\[ C_s = \frac{1.72}{5} \quad \text{or} \quad 0.344 \]

\[ C_s = 0.344 \]

SEISMIC WEIGHT

- ROOF: 15.5 sf (27' x 34') 12.77 k
- N-S WALL: 92 sf (4' x 45') 10.56 k
- E-W WALL: 92 sf (4' x 48') 5.15 k

\[ W = 35.48 \text{ k} \] (TOTAL)

BASE SHEAR

\[ V = C_s \cdot W \]

\[ V = 0.344 \cdot (35.48) \]

\[ V = 12.21 \text{ k} \]
N-S DIRECTION

\[ \text{dcord} = 27' \]

\[ V_{\text{max}} = 4.48 \text{ k} \]

\[ V_{\text{max}} = 0.130 \text{ k/ft} \]

\[ M_{\text{max}} = 22.4 \text{ k-ft} \]

\[ P_{\text{max}} = 0.830 \text{ k} \]
E-W DIRECTION

\[ V = 1.11 \]

\[ U = 0.043 \]

\[ M = 1.61 \]

\[ P_{\text{Cross}} = 0.156 \]

\[ \delta_{\text{Choord}} = 20^\circ \]

\[ V_{\text{max}} = 7.78 \text{ k} \]

\[ U_{\text{max}} = 0.295 \text{ k/ft} \]

\[ M_{\text{max}} = 80.02 \text{ k-ft} \]

\[ P_{\text{Cross}} = 8.89 \text{ k} \]
**Shear Capacity**

24 GA. SHALLOW METAL DECK w/#10's @ 6" O.C. (36/7 SCREW PATTERN)

\[ \phi V_n = 0.9 \times 0.535 \]
\[ \phi V_n > V_n \]
\[ 0.482 > 0.295 \text{ klf} \checkmark \]

**Column Shear Capacity**

\[ \phi V_n = \phi V_u \times F_y \times A \]

TRY 4" x 4" x 1/2

\[ V_u = \frac{1221}{4} \]
\[ \phi V_u > V_u \]
\[ 74.77 > 3.05 \text{ k} \checkmark \]
Shear Wall Key Plan

N-S Direction

<table>
<thead>
<tr>
<th>Grid</th>
<th>Length</th>
<th>Shear Width</th>
<th>Vwall</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>10'-11&quot;</td>
<td>16'-10&quot;</td>
<td>7.01 k</td>
</tr>
<tr>
<td>F</td>
<td>21'-8&quot;</td>
<td>10'-2&quot;</td>
<td>4.60 k</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>Total: 12.21 k</strong></td>
</tr>
</tbody>
</table>

E-W Direction

<table>
<thead>
<tr>
<th>Grid</th>
<th>Length</th>
<th>Shear Width</th>
<th>Vwall</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>13'-8&quot;</td>
<td>8'-10&quot;</td>
<td>3.19 k</td>
</tr>
<tr>
<td>B</td>
<td>6'-2&quot;</td>
<td>25'-0&quot;</td>
<td>9.02 k</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td><strong>Total: 12.21 k</strong></td>
</tr>
</tbody>
</table>
IN PLANE LOADING

GRID B (1)

\( \Phi V_n > V_u \quad V_u = 2.48 \text{k} \quad A_{\text{m}} = V_u(h) \quad M_u = 19.84 \text{k-ft} \)

\( V_n < 2.5 V_u \quad 2.5 V_u = 0.2 \text{k} \)

\( V_n (V_{nm} + V_{ns}) \sigma_{\text{a}} \quad \sigma_{\text{a}} = 1.0 \quad \text{(FULLY GRouted)} \)

MINIMUM REINFORCING REQ.

\[ S_{\text{max}} = \frac{V_u}{0.3} \quad \text{or} \quad 4^\prime \]

\[ A_{\text{min}} = 0.0007 (A_{\text{req}} \text{in}^2) \]

\[ = 0.0007 (6.4^\prime \times 8"^\prime) \]

\[ 1.25 M_n < \Phi V_n ^ 2.5 V_u \]

DESIGN ESTIMATE FOR FLEXURE

\[ A_{\text{s}} = 2.5 M_n - \frac{P_n}{F_y} \]

\[ A_{\text{s}} = 2.5 \left( 19.84 \right) \left( 0.9 (60) (60) \right) - \frac{0.51}{60} \]

\[ A_{\text{s},\text{min}} = 0.075 \text{in}^2 \]

\[ A_{\text{req}} = 0.30 \text{in}^2 \text{VEEF} \]

TRY #4 @ 16" O.C.

(SEE SP COLUMN FOR CAPACITY)

PASSES FOR FLEXURE
IN-PLANE LOADING
GRID B(1)

SHEAR DESIGN

\[ V_{nm} = \left[ 4 - 1.75 \left( \frac{M_u}{V_{ud}} \right) \right] A_n \sqrt{f_m} + 0.25 P_n \]

\[ = \left[ 4 - 1.75 \left( \frac{19.84}{2.48} \right) \right] \frac{8 \times 0.04}{1000} + 0.25 \left( 0.51 \right) \quad V_{nm} = 21.38 \text{ kN} \]

\[ M_u = 1.60 \quad \therefore \quad V_n \leq 4A_n \sqrt{f_m} \]

\[ \frac{M_u}{V_{ud}} = 1.00 \quad \therefore \quad V_n \leq 10.19 \text{ kN} \leftarrow \text{CONVERNS} \]

\[ V_{ms} = 0.5 \frac{A_n P_{n}}{d} \]

\[ V_{ms} = 0.5 \times 0.2 \times 40 \times 10 \]

\[ = 15 \text{ kN} \]

\[ \phi V_n = 0.8 \left( 21.38 + 15 \right) \]

\[ \phi V_n = 29.104 \]

\[ \phi V_n > V_n \checkmark \quad \text{(MINIMUM REINFORCEMENT)} \]

\[ \text{PER TMS 402} \]

VERTICAL BARS @ 10" O.C. TO PRINT IN-PLANE SHEAR AND FLEXURE
SEE DRAWINGS FOR LAYOUT

FOR DESIGN OF ALL REMAINING SHEAR WALLS, SEE SPREADSHEETS AND SP-COLUMN RESULTS.
### SWB (1)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
<td>Pu = 6.51 k</td>
<td>S_max = 21.333 in</td>
</tr>
<tr>
<td>L = 64 in</td>
<td>Vu = 2.48 k</td>
<td>A_{min} = 0.342 in^2</td>
</tr>
<tr>
<td>d_y = 60 in</td>
<td>Mu = 19.84 k*ft</td>
<td>As = 0.113 in^2</td>
</tr>
<tr>
<td>t = 7.625 in</td>
<td></td>
<td>Mn = 73.04 k*ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Values</th>
<th>Shear Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_y = 40 \text{ ksi} )</td>
<td>( V_{nm} = 20.146 \text{ k} )</td>
</tr>
<tr>
<td>( f'_m = 1000 \text{ psi} )</td>
<td>( V_{ns} = 15 \text{ k} )</td>
</tr>
<tr>
<td>( \varepsilon_{mu} = 0.0025 \text{ psi} )</td>
<td>( V_{nm} = 61.728 \text{ k} )</td>
</tr>
<tr>
<td>( \varepsilon_y = 0.0021 \text{ psi} )</td>
<td>( \mu = 19.84 \text{ k*ft} )</td>
</tr>
<tr>
<td>( \alpha = 4.00 )</td>
<td>( \phi M_n = 65.736 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

Max Shear: \( \mu / V_u \times d = 1.6 \)

### SWB (2)

<table>
<thead>
<tr>
<th>Wall Properties</th>
<th>Demand Forces</th>
<th>Flexural Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>H = 8 ft</td>
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</tr>
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<td>t = 7.625 in</td>
<td></td>
<td>Mn = 73.04 k*ft</td>
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</tbody>
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<tr>
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<th>Shear Design</th>
</tr>
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<tbody>
<tr>
<td>( f_y = 40 \text{ ksi} )</td>
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<td>( V_{ns} = 15 \text{ k} )</td>
</tr>
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<td>( V_{nm} = 61.728 \text{ k} )</td>
</tr>
<tr>
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</tr>
<tr>
<td>( \alpha = 4.00 )</td>
<td>( \phi M_n = 65.736 \text{ k*ft} )</td>
</tr>
</tbody>
</table>

Max Shear: \( \mu / V_u \times d = 1.6 \)
### Wall Properties
- $H = 8$ ft
- $L = 260$ in
- $d_v = 256$ in
- $t = 7.625$ in

### Demand Forces
- $P_u = 14.48$ k
- $V_u = 4.6$ k
- $M_u = 36.8$ k*ft

### Design Values
- $f_y = 40$ ksi
- $f_m = 1000$ psi
- $\varepsilon_{mu} = 0.0025$ psi
- $\varepsilon_{y} = 0.0021$ psi
- $\alpha = 1.92$

### Flexural Design
- $S_{max} = 48,000$ in
- $A_{s_{min}} = 1.388$ in$^2$
- $A_s = -0.242$ in$^2$
- $M_n = 674.78$ k*ft

### Design Values
- $A_{s_{req}} = 1.388$ in$^2$
- $A_v = 0.2$ in$^2$
- $\#$ of Bars = 9
- $A_{tot} = 1.8$ in$^2$
- $S = 32$ in

### Shear Design
- $V_{n_{min}} = 105,434.4$ k
- $V_{n_{max}} = 355.26$ k
- $\rho_{act} = 0.0009$
- $\rho_{max} = 0.160$
- $\phi_{Mn} = 607.302$ k*ft

### Shear Design
- $V_n = 4.6$ k
- $\phi_{Vn} = 196.20$ k
### Wall Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>8 ft</td>
</tr>
<tr>
<td>L</td>
<td>164 in</td>
</tr>
<tr>
<td>d&lt;sub&gt;v&lt;/sub&gt;</td>
<td>160 in</td>
</tr>
<tr>
<td>t</td>
<td>7.625 in</td>
</tr>
</tbody>
</table>

### Demand Forces

- Pu = 10.26 k
- Vu = 3.19 k
- Mu = 25.52 k*ft

### Design Values

- f<sub>y</sub> = 40 ksi
- f<sub>m</sub> = 1000 psi
- ε<sub>mu</sub> = 0.0025 psi
- ε<sub>v</sub> = 0.0021 psi
- α = 2.67

Max Shear
- Mu/Vu*d = 0.6

### Flexural Design

- S<sub>max</sub> = 48.000 in
- A<sub>s,min</sub> = 0.875 in<sup>2</sup>
- A<sub>s</sub> = -0.124 in<sup>2</sup>
- Mn = 460.11 k*ft

### Shear Design

- V<sub>n,min</sub> = 119.221 k
- V<sub>n</sub> = 40 k
- V<sub>n,max</sub> = 200.358 k

- V<sub>n</sub> = 17 k
- V<sub>n</sub> = 27.762 k
- V<sub>n</sub> = 69.444 k

- Mu = 25.52 k*ft
- φ<sub>Mn</sub> = 414.099 k*ft

- μ = 3.19 k
- φ<sub>Vn</sub> = 127.38 k

---

### Wall Properties

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>H</td>
<td>8 ft</td>
</tr>
<tr>
<td>L</td>
<td>72 in</td>
</tr>
<tr>
<td>d&lt;sub&gt;v&lt;/sub&gt;</td>
<td>68 in</td>
</tr>
<tr>
<td>t</td>
<td>7.625 in</td>
</tr>
</tbody>
</table>

### Demand Forces

- Pu = 4.84 k
- Vu = 9.02 k
- Mu = 72.16 k*ft

### Design Values

- f<sub>y</sub> = 40 ksi
- f<sub>m</sub> = 1000 psi
- ε<sub>mu</sub> = 0.0025 psi
- ε<sub>v</sub> = 0.0021 psi
- α = 4.00

Max Shear
- Mu/Vu*d = 1.4
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:09:07

Engineer: DMR
Ag = 512 in^2
As = 0.80 in^2
Xo = 0.00 in
Yo = 0.00 in
Min clear spacing = 7.50 in
Clear cover = 3.75 in

f_c = 1.5 ksi
Ec = 2208 ksi
Es = 29000 ksi
f_c = 1.275 ksi
e_u = 0.003 in/in
Beta1 = 0.8
Confinement: Tied
\phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65

P (kip)
700
P_{max} - P_{min}
M_x (k-ft)
fs=0
fs=0.5fy

File: U:\453 - Senior Project\CASITA\SW B1.col
Project: Tia Ana
Column: SW B1
8 x 136 in

Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:10:04

File: U:\453 - Senior Project\CASITA\SW B2.col

Project:
Column:
\( f_c = 1.5 \text{ ksi} \)
\( f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \)
\( E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.0025 \text{ in/in} \)
\( \beta_1 = 0.8 \)
Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Engineer:
\( A_g = 1088 \text{ in}^2 \)
\( A_s = 1.00 \text{ in}^2 \)
\( \rho_0 = 0.09\% \)
\( X_0 = 0.00 \text{ in} \)
\( Y_0 = 0.00 \text{ in} \)
\( I_x = 1.67697e+006 \text{ in}^4 \)
\( I_y = 5802.67 \text{ in}^4 \)
Min clear spacing = 31.50 in
Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:10:50

File: U:\453 - Senior Project\CASITAISW C.col
Project:
Column:
f'_c = 1.5 ksi       fy = 40 ksi
Ec = 2208 ksi       Es = 29000 ksi
f_c = 1.275 ksi
e_u = 0.003 in/in
Beta1 = 0.8
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Engineer:
Ag = 2048 in^2       9 #4 bars
As = 1.80 in^2       rho = 0.09%
Xo = 0.00 in
Yo = 0.00 in
Min clear spacing = 27.50 in
Clear cover = 3.75 in
Code: ACI 318-11
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 06/17/16
Time: 14:07:00

File: u:\453 - senior project\casita\sw 1.col
Project:
Column:
\( f_c = 1.5 \text{ ksi} \)
\( f_y = 40 \text{ ksi} \)
\( E_c = 2208 \text{ ksi} \)
\( E_s = 29000 \text{ ksi} \)
\( f_c = 1.275 \text{ ksi} \)
\( e_u = 0.003 \text{ in/in} \)
\( \beta_1 = 0.8 \)
Confinement: Tied
\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Engineer:
\( A_g = 1280 \text{ in}^2 \)
\( A_s = 2.00 \text{ in}^2 \)
\( X_0 = 0.00 \text{ in} \)
\( Y_0 = 0.00 \text{ in} \)
\( \text{Min clear spacing} = 15.50 \text{ in} \)
\( \text{Clear cover} = 3.75 \text{ in} \)
### Wall Demands:

#### Demand Forces:

<table>
<thead>
<tr>
<th>S&lt;sub&gt;dp&lt;/sub&gt;</th>
<th>1.72 g</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>F&lt;sub&gt;p&lt;/sub&gt;</th>
<th>57.79 PSF</th>
</tr>
</thead>
<tbody>
<tr>
<td>M&lt;sub&gt;e&lt;/sub&gt;</td>
<td>0.462 K-FT</td>
</tr>
</tbody>
</table>

#### Wall Weight:

- WT: 0.34 Kips

### Axial:

<table>
<thead>
<tr>
<th>Dead:</th>
<th>4 Kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live:</td>
<td>4 Kips</td>
</tr>
<tr>
<td>Earthquake:</td>
<td>0 Kips</td>
</tr>
</tbody>
</table>

### Shear:

| Earthquake: | 8 Kips |

### Moment:

| Earthquake: | 0 Kip-Feet |

### Load Combinations:

- 1.2D+1.6L:
  - Axial: 11.60 Kips
  - Shear: 0 Kips
  - Moment: 0.00 Kip-Feet
  - (1.2+2S<sub>dp</sub>)D+1.0E:
    - Axial: 6.69 Kips
    - Shear: 8 Kips
    - Moment: 0.00 Kip-Feet
  - (0.2S<sub>dp</sub>)D+1.0E:
    - Axial: 2.4 Kips
    - Shear: 8 Kips
    - Moment: 0.257 Kip-Feet

### Demands To Use:

- Axial: 2.41 Kips
- Shear: 8 Kips
- Moment: 0.26 Kip-Feet

### Capacity:

<table>
<thead>
<tr>
<th>Wall Section Properties:</th>
</tr>
</thead>
</table>

### Masonry Properties:

- Compressive Stress: f<sup>'</sup>M = 2.00 KSI
- Masonry Strain: e<sub>Mr</sub> = 0.0025 in/in
- Modulus of Rup: f<sub>r</sub> = 163 PSI

### Reinforcement Properties:

- Yield Strength: f<sub>y</sub> = 40.00 KSI
- Modulus Of E: E = 29000 KSI

### Vertical Reinforcement:

- #4 Bars
- Number Of Bars: 1
- Area Reinforcement: 0.2 in<sup>2</sup>
- Bar Spacing: 48" O.C.
- Area Total: 0.05 in<sup>2</sup>/ft
- Diameter Bar: 0.5 in

### Moment Of Inertia:

- I<sub>g</sub> = 577.43 in<sup>4</sup>

### Cracking Moment:

- M<sub>cr</sub> = 2.057 K-FT
**FIND DEPTH COMPRESSION BLOCK:**

<table>
<thead>
<tr>
<th></th>
<th>A</th>
<th>0.18 IN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C</td>
<td>0.22 IN</td>
</tr>
</tbody>
</table>

**CRACKING MOMENT OF INERTIA:**

<table>
<thead>
<tr>
<th></th>
<th>N</th>
<th>16.1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I_{CR}</td>
<td>10.4 IN^4</td>
</tr>
</tbody>
</table>

**1ST ITERATION MOMENT WITH \( \xi = 0 \):**

|   | M_{u1} | 0.462 K-FT |

**FIND \( \xi \):**

|   | \( \xi \) | 0.005 IN |

**2ND ITERATION MOMENT WITH \( \xi = 0.005 \):**

|   | M_{u2} | 0.463 K-FT |

**MOMENT CAPACITY:**

<table>
<thead>
<tr>
<th></th>
<th>M_{N}</th>
<th>1.37 K-FT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \Phi M_{N} )</td>
<td>1.23 K-FT</td>
</tr>
</tbody>
</table>

1.23 K-FT > 0.26 K-FT wall OK for out of plane.
TYPICAL WALL FOOTING (F-1)

WORST CASE AXIAL LOAD: 139 kN
WORST CASE SHEAR FORCE: 110 kN
SHORTEST WALL: 12' - 0"

\[ W = 139 \text{ kN} \] (SELF-WT INCLUDED)

\[ W_L = 100 \text{ kN} \times 120 = 24 \text{ kN} \]

\[ A_{sv} = \frac{1.68 + 2.4}{1.6} \]

\[ P_0 = 1.34(12') = 16.08 \text{ kN} \]

\[ P_L = 2 \times 12' = 24 \text{ kN} \]

\[ q_0 = 1,500 \text{ kN/m} \]

\[ A_{tr} = 20.72 \text{ m}^2 \]

11' x 3' FOOTING

\[ q_0 = 1.3(16.08 + 1.6(24)) \]

\[ q_0 = 113.7 \text{ kN/m} \]

\[ V_u = \frac{q_0(L - h_{mic}) - d_{cm}}{2} \]

\[ V_u = 9.13 - 1.27d \]

\[ d = 8" \]

\[ V_u = 8.22 \text{ kN} \]

SEE SPREADSHEET FOR PLANTS' CAPACITY AND FINAL DESIGN
CONCENTRIC LOADED - TYPICAL WALL FOOTING

REFERENCE:

- $f'_c = 3000.00$ psi
- $f_c = 60000.00$ psi
- $q_{allow} = 1500.00$ psf
- $d_{col} = 8.00$ in
- $b_{col} = 6.00$ in

DETERMINING DIMENSIONS (W x L):

- $L = 14.00$ ft
- $W = 3.00$ ft
- $A_{fg} = 42.00$ ft$^2$

DETERMINING FOOTING VERTICAL REINFORCEMENT:

A) CHECK ONE WAY BEAM SHEAR AT CRITICAL SECTION

- $q_u = 1370.00$ psf
- $d_{fg} = 8.00$ in
- $\phi_{shear} = 0.75$
- $V_u = 685.00$ #/ft
- $\phi V_n = 7887.20$ #/ft
- $h = d + 4" = 12.00$ in
- $\text{TRY } h = 12.00$ in
- $\text{TRY } d = 8.00$ in

- $h_{min} (\text{effective depth of reinforcement } = 6")$
- $h_{min} = 10.00$ in < $h_{try}$
- $V_u = 685.00$ klf
- $\phi V_n = 7887.20$ klf
- $\phi V_n > V_u$

B) CHECK TWO WAY PUNCHING SHEAR

- $V_u = 10465.28$ #
- $b_o = 60.00$
- $\beta = 1.33$
- $\phi V_c = 98590.06$ #
- $144598.76$ #
- $105162.73$ #
- $\phi V_c > V_u$
- $d = \text{OK}$

DETERMINING CAPACITY:

- $M_u = 932.36$ #-in
- $0.08$ k-ft

ESTIMATE ($d - a/2) = 0.9d$

ASSUME $\phi = 0.90$

- $As_{REQ} = 0.0024$ in$^2$/FT
- $As_{MIN} = 0.48$ in$^2$
- $0.26$ in$^2$
- $0.26$ in$^2$

TRY #4 @ 16" O.C.

- $As = 1.80$ in$^2$
- $d = 8.75$ in
- $a = 3.53$ in
- $c = 4.15$ in

- $M_n = 754411.76$ #-in
- $62.87$ k-ft
- $E_t = 0.0033 > 0.0021$
- $> 0.0021$
- $> 0.005$
- $\phi M_n = 56.58$ k-ft
- $\phi M_n > M_u$

Remark: Use #4 @ 16" o.c. one way with 14"x3" footing 12" deep.
**CONCENTRIC LOADED - TYPICAL COLUMN FOOTING**

**DETERMINATION OF SHEAR**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_c' =</td>
<td>3000 \text{ psi}</td>
</tr>
<tr>
<td>$f_y =</td>
<td>60000 \text{ psi}</td>
</tr>
<tr>
<td>$q_{dead} =</td>
<td>1500 \text{ psf}</td>
</tr>
<tr>
<td>DEAD =</td>
<td>20 \text{ psf}</td>
</tr>
<tr>
<td>LIVE =</td>
<td>100 \text{ psf}</td>
</tr>
<tr>
<td>$P_0 =</td>
<td>65999.88 #</td>
</tr>
<tr>
<td>$P_i =</td>
<td>32999.40 #</td>
</tr>
</tbody>
</table>

**DETERMINATION OF SUPERIMPOSED LOAD**

$L = W = 5.14 \text{ ft}$

**TRY:**

$L = W = 6.00 \text{ ft}$

$A_{c,tg} = 36.00 \text{ ft}^2$

**DETERMINATION OF SHEAR**

**A) CHECK ONE WAY BEAM SHEAR AT CRITICAL SECTION**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_w =</td>
<td>1686.64 \text{ psf}</td>
</tr>
<tr>
<td>$V_w =</td>
<td>3127.30 #/\text{ft}</td>
</tr>
<tr>
<td>$\phi V_w =</td>
<td>10598.43 #/\text{ft}</td>
</tr>
<tr>
<td>$h =</td>
<td>d + 4'' = 14.75 \text{ in}</td>
</tr>
<tr>
<td>TRY $h =</td>
<td>18.00 \text{ in}</td>
</tr>
<tr>
<td>TRY $d =</td>
<td>14.00 \text{ in}</td>
</tr>
<tr>
<td>$h_{ef} =</td>
<td>\text{effective depth of reinforcement} = 6''</td>
</tr>
<tr>
<td>$h_{eff} =</td>
<td>10.00 \text{ in}</td>
</tr>
</tbody>
</table>

$\phi V_d > V_u$  

**B) CHECK TWO WAY PUNCHING SHEAR**

$V_d = 59032.26 \#$

$\beta = 80.00$

$\phi V_d = 276052.17 \#$

$\phi V_s = 245379.71 \#$

$\phi V_n > V_u$

**RECURSIVE SHEAR**

$M_n = 6377.59 \text{ ft}-\text{ft}$

$0.53 \text{ k-ft}$

**ESTIMATE ($d-a/2) = 0.90$**

**ASSUME PHI = 0.90**

$A_{sec} = 0.01 \text{ in}^2/\text{ft}$

$A_{nau} = 0.48 \text{ in}^2$

$0.46 \text{ in}^2$

$0.39 \text{ in}^2$

**TRY #4 @ 16'' O.C.**

$As = 1.20 \text{ in}^2$

$d = 14.75 \text{ in}$

$a = 2.35 \text{ in}$

$c = 2.77 \text{ in}$

$M_n = 977294.12 \text{ ft}-\text{ft}$

$81.44 \text{ k-ft}$

$E_s = 0.01 > 0.0021$

$>0.005$

$\phi M_n = 73.30 \text{ k-ft}$

$\phi M_n > M_u$

**use #4 @ 8'' o.c. each way with 3''x3'' footing 14'' deep.**
SCREEN WALL OUT OF PLANE LOAD

SCREEN WALL (H = 8", L = VARIOUS)

\[ H = 16" \text{ O.C. FULL HT.} \]

FOUNDATION

Typ. Elevation

16" O.C.

Assume Tributary 1 ft section: \[ H = 8\text{"} \text{ O.C.} \rightarrow \frac{0.2\text{ in}^2}{12\text{"}} = 0.15\text{ in}^2/\text{ft} \]

\[ d = 4" \]

\[ W_{self} = 8'(120 \text{ PER}) \times (8/12 \text{ PT}) \times 2 \]

\[ \text{ACCOUNTS FOR OPENINGS IN BLOCKS} \]

(SEE SPREADSHEET FOR RESULTS ON NEXT PAGE)
### Project: Tia Ana's Sustainable Development

#### Out of Plane Screen Wall

### Prepared By: Dani Rustagi

#### Date:

### Sheet: 33 of 33

#### Wall Demands:

#### Demand Forces:

- **S_{d}**: 1.72 G
- **FP**: 28.90 PSF
- **ME**: 0.231 K-FT

#### Axial:
- **Dead**: 2 KIPS
- **Live**: 0 KIPS
- **Earthquake**: 0 KIPS

#### Shear:
- **Dead**: 0 KIPS
- **Live**: 0 KIPS
- **Earthquake**: 0 KIPS

#### Moment:
- **Dead**: 0 KIP-Feet
- **Live**: 0 KIP-Feet
- **Earthquake**: 0 KIP-Feet

#### Load Combinations:

- **1.2D+1.6L**
- **(1.2+2S_{d})D+1.0E**
- **(9.2S_{d})D+1.0E**

#### Demands To Use:

- **L**: 1.21 KIPS
- **Shear**: 0 KIPS
- **Moment**: 0.13 KIP-Feet

### Capacity:

#### Wall Section Properties:

- **Block Depth**: D
- **Block Width**: B
- **Block Height**: H
- **Wall Height**: H
- **Wall Weight**: W_{w}
- **Bar Cover**: B_{cover}

- **7.63 INCHES**
- **15.63 INCHES**
- **7.63 INCHES**
- **8.00 FEET**
- **42.00 PSF**
- **3.8125 INCHES**

#### Masonry Properties:

- **Compressive Stress**: F'M 2.00 KSI
- **Masonry Strain**: \( \varepsilon_{mu} \) 0.0025 IN/IN
- **Modulus of Rupture**: \( F_{r} \) 163 PSI

#### Reinforcement Properties:

- **Yield Strength**: \( F_{y} \) 40.00 KSI
- **Modulus of Elasticity**: \( E \) 29000 KSI

#### Vertical Reinforcement:

- **#4 Bars**: 1
- **Number of Bars**: 1
- **Area Reinforcement**: 0.2 IN^2
- **Bar Spacing**: 16 " O.C.
- **Area Total**: 0.15 IN^2/FT
- **Diameter Bar**: 0.5 IN

#### Horizontal Reinforcement:

- **NONE Bars**
- **Bar Spacing**: 0 " O.C.
- **Area Reinforcement**: 0 IN^2
- **Diameter Bar**: 0 IN

#### Moment of Inertia:

- **I_{o}**: 577.43 IN^4

#### Cracking Moment:

- **M_{ck}**: 2.057 K-FT
PROJECT: Tia Ana's Sustainable Development

SUBJECT: Out of Plane Screen Wall

PREPARED BY: Dani Rustagi

DEEP COMPRESSION BLOCK:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.29 IN</td>
</tr>
<tr>
<td>C</td>
<td>0.36 IN</td>
</tr>
</tbody>
</table>

CRACKING MOMENT OF INERTIA:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>N</td>
<td>16.1</td>
</tr>
<tr>
<td>I_{cr}</td>
<td>29.0 \text{ IN}^4</td>
</tr>
</tbody>
</table>

1ST ITERATION MOMENT WITH $\xi = 0$:

$M_{u1}$ = 0.231 K-FT

FIND $\xi$:

$\xi$ = 0.003 IN

2ND ITERATION MOMENT WITH $\xi = 0.003$

$M_{u2}$ = 0.231 K-FT

MOMENT CAPACITY:

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_n$</td>
<td>2.20 K-FT</td>
</tr>
<tr>
<td>$\phi_{MN}$</td>
<td>1.98 K-FT</td>
</tr>
</tbody>
</table>

1.98 K-FT > 0.13 K-FT WALL OK FOR OUT OF PLANE
1 Input data

Anchor type and diameter: Heavy Square Head ASTM F 1554 GR. 36 7/8
Effective embedment depth: $h_{ef} = 12.000 \text{ in.}$
Material: ASTM F 1554
Proof: Design method ACI 318-14 / CIP
Stand-off installation: $e_s = 0.000 \text{ in.} \text{ (no stand-off); } t = 0.500 \text{ in.}$
Anchor plate: $l_x \times l_y \times t = 8.000 \text{ in.} \times 8.000 \text{ in.} \times 0.500 \text{ in.}; \text{ (Recommended plate thickness: not calculated}$
Profile: Square HSS (AISC); $(L \times W \times T) = 4.000 \text{ in.} \times 4.000 \text{ in.} \times 0.500 \text{ in.}$
Base material: cracked concrete, 2500, $f_{c}'' = 2500 \text{ psi; } h = 420.000 \text{ in.}$
Reinforcement: tension: condition A, shear: condition B; anchor reinforcement: tension
edge reinforcement: > No. 4 bar

Geometry [in.] & Loading [lb, in.lb]
2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension force</th>
<th>Shear force</th>
<th>Shear force x</th>
<th>Shearforce y</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4121</td>
<td>3189</td>
<td>2255</td>
<td>2255</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>3189</td>
<td>2255</td>
<td>2255</td>
</tr>
<tr>
<td>3</td>
<td>15338</td>
<td>3189</td>
<td>2255</td>
<td>2255</td>
</tr>
<tr>
<td>4</td>
<td>4121</td>
<td>3189</td>
<td>2255</td>
<td>2255</td>
</tr>
</tbody>
</table>

max. concrete compressive strain: 1.03 [%]
max. concrete compressive stress: 4489 [psi]
resulting tension force in (x/y)=(-1.626/1.626): 23581 [lb]
resulting compression force in (x/y)=(2.459/-2.459): 28421 [lb]

3 Tension load

<table>
<thead>
<tr>
<th></th>
<th>Load N_{td} [lb]</th>
<th>Capacity ( \Phi N_n ) [lb]</th>
<th>Utilization ( \beta_n = N_{ud}/\Phi N_n )</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel Strength*</td>
<td>15338</td>
<td>20097</td>
<td>77</td>
<td>OK</td>
</tr>
<tr>
<td>Pullout Strength*</td>
<td>15338</td>
<td>20510</td>
<td>75</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete Breakout Strength**</td>
<td>23581</td>
<td>44239</td>
<td>54</td>
<td>OK</td>
</tr>
<tr>
<td>Concrete Side-Face Blowout, direction **</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

* anchor having the highest loading  **anchor group (anchors in tension)

3.1 Steel Strength

\( N_{ud} = A_{se,N} f_{ud} \)

ACI 318-14 Eq. (17.4.1.2)

\( \Phi N_n \geq N_{ud} \)

ACI 318-14 Table 17.3.1.1

Variables

\( A_{se,N} \) [in.²]  
\( f_{ud} \) [psi]

0.46  
58000

Calculations

\( N_{ud} \) [lb]

26796

Results

\( N_{ud,\Phi} \) [lb]  
\( \Phi N_n \) [lb]  
\( N_{ud} \) [lb]

26796  
0.750  
20097  
15338

Input data and results must be checked for agreement with the existing conditions and for plausibility!

PROFIS Anchor (c) 2003-2009 Hilti AG, FL-9494 Schaan  Hilti is a registered Trademark of Hilti AG, Schaan
3.2 Pullout Strength

\[ N_{pl} = \Psi_{cp} N_p \]
\[ N_p = 8 A_{bp} f_c \]
\[ \phi N_{pl} \geq N_{ua} \]

ACI 318-14 Eq. (17.4.3.1)
ACI 318-14 Eq. (17.4.3.4)
ACI 318-14 Table 17.3.1.1

**Variables**

<table>
<thead>
<tr>
<th>( \Psi_{cp} )</th>
<th>( A_{bp} \text{ [in.}^2 )</th>
<th>( \lambda_p )</th>
<th>( f_c \text{ [psi]} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.000</td>
<td>1.47</td>
<td>1.000</td>
<td>2500</td>
</tr>
</tbody>
</table>

**Calculations**

\[ N_p \text{ [lb]} \]
\[ 29300 \]

**Results**

\[ \Psi_{pl} \text{ [lb]} \]
\[ 29300 \]
\[ \phi \text{concrete} \]
\[ 0.700 \]
\[ \phi N_{pl} \text{ [lb]} \]
\[ 20510 \]
\[ N_{ua} \text{ [lb]} \]
\[ 15338 \]

3.3 Concrete Breakout Strength

\[ N_{cbq} = \left( \frac{A_{nc}}{A_{no}} \right) \Psi_{ec,N} \Psi_{ed,N} \Psi_{ec,F} N_p \]
\[ \phi N_{cbq} \geq N_{ua} \]

ACI 318-14 Eq. (17.4.2.1b)
ACI 318-14 Table 17.3.1.1
ACI 318-14 Eq. (17.4.2.1c)
ACI 318-14 Eq. (17.4.2.2)

**Variables**

<table>
<thead>
<tr>
<th>( h_{ef} \text{ [in.]} )</th>
<th>( e_{oc,1} \text{ [in.]} )</th>
<th>( e_{oc,2} \text{ [in.]} )</th>
<th>( c_{al,\text{min}} \text{ [in.]} )</th>
<th>( \Psi_{ec,N} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.000</td>
<td>0.793</td>
<td>0.793</td>
<td>422.500</td>
<td>1.000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>( c_{ec} \text{ [in.]} )</th>
<th>( k_c )</th>
<th>( \lambda_p )</th>
<th>( f_c \text{ [psi]} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>-</td>
<td>16</td>
<td>1.000</td>
<td>2500</td>
</tr>
</tbody>
</table>

**Calculations**

<table>
<thead>
<tr>
<th>( A_{nc} \text{ [in.}^2 )</th>
<th>( A_{no} \text{ [in.}^2 )</th>
<th>( \Psi_{ec,1,N} )</th>
<th>( \Psi_{ec,2,N} )</th>
<th>( \Psi_{ec,F} )</th>
<th>( \Psi_{ed,N} )</th>
<th>( \Psi_{ed,F} )</th>
<th>( N_p \text{ [lb]} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1656.00</td>
<td>1296.00</td>
<td>0.958</td>
<td>0.958</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>50318</td>
</tr>
</tbody>
</table>

**Results**

<table>
<thead>
<tr>
<th>( N_{cbq} \text{ [lb]} )</th>
<th>( \phi \text{concrete} )</th>
<th>( \phi N_{cbq} \text{ [lb]} )</th>
<th>( N_{ua} \text{ [lb]} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>58985</td>
<td>0.750</td>
<td>44239</td>
<td>23581</td>
</tr>
</tbody>
</table>
4 Shear load

<table>
<thead>
<tr>
<th>Load $V_{sa}$ [lb]</th>
<th>Capacity $\phi V_n$ [lb]</th>
<th>Utilization $\beta_V = V_{sa}/\phi V_n$</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>3189</td>
<td>10450</td>
<td>31</td>
<td>OK</td>
</tr>
</tbody>
</table>

Steel Strength*  
Steel failure (with lever arm)*  
Pryout Strength**  
Concrete edge failure in direction x**

* anchor having the highest loading  
** anchor group (relevant anchors)

4.1 Steel Strength

$$V_{sa} = 0.6 A_{soc,V} f_{soc}$$  
$$\phi V_{Steel} \geq V_{sa}$$  

ACI 318-14 Eq. (17.5.1.2b)  
ACI 318-14 Table 17.3.1.1

Variables

<table>
<thead>
<tr>
<th>$A_{soc,V}$ [in.$^2$]</th>
<th>$f_{soc}$ [psi]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.46</td>
<td>58000</td>
</tr>
</tbody>
</table>

Calculations

$$V_{sa} [lb]$$  
$$16078$$

Results

<table>
<thead>
<tr>
<th>$V_{sa} [lb]$</th>
<th>$\phi_{steel}$</th>
<th>$\phi V_{sa} [lb]$</th>
<th>$V_{sa} [lb]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>16078</td>
<td>0.650</td>
<td>10450</td>
<td>3189</td>
</tr>
</tbody>
</table>

4.2 Pryout Strength

$$V_{cpp} = k_{cpp} \left[ \frac{A_{soc,N}}{A_{soc,N}} \right] \psi_{soc,N} \psi_{ed,N} \psi_{c,N} \psi_{cpp,N} N_p$$

$$\phi V_{cpp} \geq V_{sa}$$

ACI 318-14 Eq. (17.5.3.1b)  
ACI 318-14 Table 17.3.1.1

$$A_{soc,N} = 9 h_{ef}^2$$

ACI 318-14 Eq. (17.4.2.1c)

$$\psi_{soc,N} = \frac{1}{1 + \frac{2.5}{3h_{ef}}} \leq 1.0$$

ACI 318-14 Eq. (17.4.2.4)

$$\psi_{ed,N} = 0.7 + 0.3 \left( \frac{c_{soc,N}}{1.5h_{ef}} \right) \leq 1.0$$

ACI 318-14 Eq. (17.4.2.6b)

$$\psi_{c,N} = \text{MAX} \left( \frac{c_{soc,N}}{1.5h_{ef}}, \frac{c_{soc,N}}{c_{soc,N}} \right) \leq 1.0$$

ACI 318-14 Eq. (17.4.2.7b)

$$N_p = 16 \lambda_a \sqrt{f_{c0} h_{ef}^3}$$

ACI 318-14 Eq. (17.4.2.2b)

Variables

<table>
<thead>
<tr>
<th>$k_{cpp}$</th>
<th>$h_{ef}$ [in.]</th>
<th>$c_{soc,N}$ [in.]</th>
<th>$c_{soc,N}$ [in.]</th>
<th>$c_{soc,N}$ [in.]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.000</td>
<td>0.000</td>
<td>0.000</td>
<td>422.500</td>
</tr>
</tbody>
</table>

Calculations

$$\psi_{c,N} \quad c_{soc,N} \quad k_{cpp} \quad \lambda_a \quad f_{c0} [psi]$$

| 1.000  | 16             | 1.000           | 2500            |

Results

<table>
<thead>
<tr>
<th>$V_{cpp} [lb]$</th>
<th>$\phi_{concrete}$</th>
<th>$\phi V_{cpp} [lb]$</th>
<th>$V_{sa} [lb]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>130532</td>
<td>0.700</td>
<td>91373</td>
<td>12756</td>
</tr>
</tbody>
</table>

Input data and results must be checked for agreement with the existing conditions and for plausibility!

PROFIS Anchor (c) 2003-2009 Hilti AG, FL-9494 Schaan  Hilti is a registered Trademark of Hilti AG, Schaan
4.3 Concrete edge failure in direction x+

\[
V_{deg} = \left(\frac{A_{vc}}{A_{v0}}\right) \psi_{c,v} \psi_{d,v} \psi_{f,v} V_{d,v} V_{par,v} V_b
\]

\[\Phi V_{deg} = V_{in}\]

\[A_{vc} \text{ see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)}\]

\[A_{v0} = 4.5 c_{a1}^2\]

\[\psi_{c,v} = \left(\frac{1}{1 + 2c_{a1}^2 / 3c_{a1}}\right) \leq 1.0\]

\[\psi_{d,v} = 0.7 + 0.3 \left(\frac{c_{a1}}{1.5c_{a1}}\right) \leq 1.0\]

\[\psi_{f,v} = \frac{1.5c_{a1}}{h_b} \geq 1.0\]

\[V_b = 9 \lambda \psi_{c,v} c_{a1}^2\]

Variables

<table>
<thead>
<tr>
<th>$c_{a1}$ [in.]</th>
<th>$c_{a2}$ [in.]</th>
<th>$e_{sv}$ [in.]</th>
<th>$\psi_{c,v}$</th>
<th>$h_b$ [in.]</th>
</tr>
</thead>
<tbody>
<tr>
<td>281.667</td>
<td>422.500</td>
<td>0.000</td>
<td>1.200</td>
<td>420.000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$h_b$ [in.]</th>
<th>$\lambda$</th>
<th>$d_b$ [in.]</th>
<th>$f_c$ [psi]</th>
<th>$V_{par,v}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.000</td>
<td>1.000</td>
<td>0.875</td>
<td>2500</td>
<td>1.000</td>
</tr>
</tbody>
</table>

Calculations

\[A_{vc} \text{ in.}^2 = 357000.00\]

\[A_{v0} \text{ in.}^2 = 357012.50\]

\[\psi_{c,v} = 1.000\]

\[\psi_{d,v} = 1.000\]

\[\psi_{f,v} = 1.003\]

\[V_b = 2127235\]

Results

\[V_{deg} \text{ lb} = 2560179\]

\[\Phi V_{deg} \text{ lb} = 1792125\]

\[V_{in} \text{ lb} = 12756\]

5 Combined tension and shear loads

<table>
<thead>
<tr>
<th>$\beta_N$</th>
<th>$\beta_V$</th>
<th>$\beta$</th>
<th>Utilization [%]</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.763</td>
<td>0.305</td>
<td>5/3</td>
<td>78</td>
<td>OK</td>
</tr>
</tbody>
</table>

\[\beta_{in} = \beta_N + \beta_V \leq 1\]

6 Warnings

- Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading! Input data and results must be checked for agreement with the existing conditions and for plausibility!

- Condition A applies when supplementary reinforcement is used. The $\Phi$ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.

- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

- The design of Anchor Reinforcement is beyond the scope of PROFIS Anchor. Refer to ACI 318-14, Section 17.4.2.9 for information about Anchor Reinforcement.

- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Anchor calculations.

Fastening meets the design criteria!
7 Installation data

Anchor plate, steel: 
Profile: Square HSS (AISC); 4,000 x 4,000 x 0.500 in.
Hole diameter in the fixture: d₁ = 0.938 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: Heavy Square Head ASTM F 1554 GR. 36 7/8
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 12.000 in.
Minimum thickness of the base material: 13.052 in.

Coordinates Anchor in.

<table>
<thead>
<tr>
<th>Anchor</th>
<th>x</th>
<th>y</th>
<th>cₓ</th>
<th>cᵧ</th>
<th>cₓᵧ</th>
<th>cᵧₓ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-2.500</td>
<td>-2.500</td>
<td>422.500</td>
<td>422.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
<tr>
<td>2</td>
<td>2.500</td>
<td>-2.500</td>
<td>422.500</td>
<td>422.500</td>
<td>422.500</td>
<td>427.500</td>
</tr>
<tr>
<td>3</td>
<td>-2.500</td>
<td>2.500</td>
<td>422.500</td>
<td>427.500</td>
<td>427.500</td>
<td>422.500</td>
</tr>
<tr>
<td>4</td>
<td>2.500</td>
<td>2.500</td>
<td>427.500</td>
<td>422.500</td>
<td>427.500</td>
<td>422.500</td>
</tr>
</tbody>
</table>

Input data and results must be checked for agreement with the existing conditions and for plausibility!
General Information:

- File Name: U:\453 - Senior Project\CASITA\SW 1.5.col
- Project: ACI 318-11
- Engineer: Units: English
- Slenderness: Not considered
- Column Type: Structural

Material Properties:

- f'c = 1.5 ksi
- E = 2207.6 ksi
- Ultimate strain = 0.003 in/in
- Beta1 = 0.8

Section:

- Rectangular: Width = 8 in
- Depth = 72 in
- Gross section area, Ag = 576 in^2
- Ix = 248832 in^4
- rx = 20.7846 in
- xo = 0 in

Reinforcement:

- Bar Set: ASTM A615
- Size Diam (in) | Area (in^2) | # | Size Diam (in) | Area (in^2) | # | Size Diam (in) | Area (in^2) |
- # 3 | 0.38 | 0.11 | 4 | 0.50 | 0.20 | 5 | 0.63 | 0.31
- # 6 | 0.75 | 0.44 | 7 | 0.88 | 0.60 | 8 | 1.00 | 0.79
- # 9 | 1.13 | 1.00 | 10 | 1.27 | 1.27 | 11 | 1.41 | 1.56
- # 14 | 1.69 | 2.25 | 18 | 2.26 | 4.00 |

Confinement: Tied: #3 ties with #10 bars, #4 with larger bars.

Pattern: Irregular
- Total steel area: As = 1.00 in^2 at rho = 0.17% (Note: rho < 0.50%)
- Minimum clear spacing = 15.50 in

Control Points:

- Axial Load P | X-Moment | Y-Moment | NA depth | Dt depth | eps, Phi |
- X @ Max compression | 502.5 | 0.00 | -0.00 | 125.87 | 68.00 | -0.00138 | 0.650
- @ Allowable comp. | 402.0 | 241.02 | -0.00 | 72.26 | 68.00 | -0.00018 | 0.650
- @ fs = 0.0 | 378.3 | 281.30 | -0.00 | 68.00 | 68.00 | 0.00000 | 0.650
- @ fs = 0.5fy | 305.2 | 365.47 | -0.00 | 55.29 | 68.00 | 0.00006 | 0.650
- @ Balanced point | 253.0 | 393.33 | -0.00 | 46.58 | 68.00 | 0.00013 | 0.650
- @ Tension control | 176.5 | 454.22 | -0.00 | 25.50 | 68.00 | 0.00050 | 0.900
- @ Pure bending | -0.0 | 101.83 | -0.00 | 3.95 | 68.00 | 0.04866 | 0.900
- @ Max tension | -36.0 | 0.00 | 0.00 | 68.00 | 9.99999 | 0.900

Y @ Max compression | 502.5 | -0.00 | -0.00 | 125.87 | 68.00 | -0.00138 | 0.650
- @ Allowable comp. | 402.0 | -241.02 | 0.00 | 72.26 | 68.00 | -0.00018 | 0.650
- @ fs = 0.0 | 378.3 | -281.30 | 0.00 | 68.00 | 68.00 | 0.00000 | 0.650
- @ fs = 0.5fy | 305.2 | -365.47 | 0.00 | 55.29 | 68.00 | 0.00006 | 0.650
- @ Balanced point | 253.0 | -393.33 | 0.00 | 46.58 | 68.00 | 0.00013 | 0.650
- @ Tension control | 176.5 | -454.22 | 0.00 | 25.50 | 68.00 | 0.00050 | 0.900
- @ Pure bending | -0.0 | -101.83 | 0.00 | 3.95 | 68.00 | 0.04866 | 0.900
- @ Max tension | -36.0 | 0.00 | 0.00 | 68.00 | 9.99999 | 0.900

*** End of output ***
General Information:

Object: u:\e53 - senior project\casita\sw 1.col

Run Option: Investigation
Run Axis: X-axis

Material Properties:

\( f'c = 1.5 \text{ ksi} \)
\( E = 2207.6 \text{ ksi} \)
Ultimate strain = 0.003 in/in
Beta = 0.8

Section:

Rectangular: Width = 8 in
Depth = 160 in

Gross section area, \( A_g = 1280 \text{ in}^2 \)
Ix = 2.73067e+06 in^4
Rx = 46.188 in
X0 = 0 in

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

\( \phi(a) = 0.8, \phi(b) = 0.9, \phi(c) = 0.65 \)

Total steel area: \( A_s = 2.00 \text{ in}^2 \) at \( \rho = 0.16\% \) (Note: \( \rho < 0.50\% \))
Minimum clear spacing = 15.50 in

Control Points:

Axial Load P kip
X-Moment k-ft
Y-Moment k-ft
NA depth in
Dt depth in
\( \varepsilon_t \)
Phi

** End of output **
General Information:

- **File Name:** U:\453 - Senior Project\CISITA\SW B1.col
- **Project:** Tia Ana
- **Column:** SW B1
- **Code:** ACI 318-11
- **Run Option:** Investigation
- **Run Axis:** X-axis
- **Engineer:** DMR
- **Units:** English

Material Properties:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'c$</td>
<td>1.5 ksi</td>
</tr>
<tr>
<td>$E_c$</td>
<td>2207.6 ksi</td>
</tr>
<tr>
<td>Ultimate strain</td>
<td>0.003 in/in</td>
</tr>
<tr>
<td>$(\gamma_0)$</td>
<td>0.8</td>
</tr>
<tr>
<td>$f_y$</td>
<td>40 ksi</td>
</tr>
<tr>
<td>$E_s$</td>
<td>29000 ksi</td>
</tr>
</tbody>
</table>

Section:

- **Rectangular:** Width = 8 in, Depth = 64 in
- **Gross section area, $A_g$:** 512 in$^2$
- **$I_x$:** 174763 in$^4$
- **$I_y$:** 2730.67 in$^4$
- **$I_{xy}$:** 2.3094 in
- **$I_y$:** 2.3094 in
- **$y_o$:** 0 in

Reinforcement:

- **Bar Set:** ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in$^2$)</th>
<th>Size Diam (in)</th>
<th>Area (in$^2$)</th>
<th>Size Diam (in)</th>
<th>Area (in$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.38</td>
<td>4</td>
<td>0.50</td>
<td>5</td>
<td>0.63</td>
</tr>
<tr>
<td>6</td>
<td>0.75</td>
<td>7</td>
<td>0.88</td>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td>9</td>
<td>1.13</td>
<td>10</td>
<td>1.27</td>
<td>11</td>
<td>1.41</td>
</tr>
<tr>
<td>10</td>
<td>1.69</td>
<td>18</td>
<td>2.26</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.

Pattern: Irregular

Total steel area: $A_s = 0.80$ in$^2$ at $\rho = 0.16\%$ (Note: $\rho < 0.50\%$)

Minimum clear spacing = 7.50 in

Control Points:

<table>
<thead>
<tr>
<th>Control Points</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in$^2$</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in$^2$</th>
<th>X (in)</th>
<th>Y (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>20.0</td>
<td></td>
<td>0.20</td>
<td>0.0</td>
<td></td>
<td>0.20</td>
<td>0.0</td>
</tr>
<tr>
<td>0.20</td>
<td>0.0</td>
<td>-20.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Control Points:

- **Axial Load $P$:**
  - $X = \text{Max compression:}$ 444.5 kip
  - $Y = \text{Max compression:}$ 444.5 kip

- **X-Moment $M_x$:**
  - $X = \text{Max compression:}$ 0.0 k-ft
  - $Y = \text{Max compression:}$ 0.0 k-ft

- **Y-Moment $M_y$:**
  - $X = \text{Max compression:}$ -0.0 k-ft
  - $Y = \text{Max compression:}$ -0.0 k-ft

- **NA depth $D$:**
  - $X = \text{Max compression:}$ 96.26 in
  - $Y = \text{Max compression:}$ 96.26 in

- **Depth $D_t$:**
  - $X = \text{Max compression:}$ 52.00 in
  - $Y = \text{Max compression:}$ 52.00 in

- **Eps$_t$:**
  - $X = \text{Max compression:}$ 0.0000
  - $Y = \text{Max compression:}$ 0.0000

- **Phi:**
  - $X = \text{Max compression:}$ 650
  - $Y = \text{Max compression:}$ 650

*** End of output ***
General Information:

- Column: ACI 318-11
- Run Option: Investigation
- Run Axis: X-axis
- Slenderness: Not considered
- Column Type: Structural

Material Properties:

- \( f'c = 1.5 \text{ ksi} \)
- \( fy = 40 \text{ ksi} \)
- \( Ec = 2207.6 \text{ ksi} \)
- \( Es = 29000 \text{ ksi} \)
- Ultimate strain = 0.0025 in/in
- \( Bt = 0.8 \)

Section:

- Rectangular: Width = 8 in
- Depth = 136 in
- Gross section area, \( A_g = 1088 \text{ in}^2 \)
- Iy = 5802.67 in\(^4\)
- \( Ix = 1.67697\times10^6 \text{ in}^4 \)
- \( rx = 39.2598 \text{ in} \)
- \( ry = 2.3094 \text{ in} \)
- \( Xo = 0 \text{ in} \)
- \( Yo = 0 \text{ in} \)

Reinforcement:

- Bar Set: ASTM A615

<table>
<thead>
<tr>
<th>Size Diam (in)</th>
<th>Area (in(^2))</th>
<th>Size Diam (in)</th>
<th>Area (in(^2))</th>
<th>Size Diam (in)</th>
<th>Area (in(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td># 3</td>
<td>0.38</td>
<td>0.11</td>
<td>0.50</td>
<td>0.20</td>
<td>0.63</td>
</tr>
<tr>
<td># 6</td>
<td>0.75</td>
<td>0.44</td>
<td>0.88</td>
<td>0.60</td>
<td>1.00</td>
</tr>
<tr>
<td># 9</td>
<td>1.13</td>
<td>1.00</td>
<td>1.27</td>
<td>1.27</td>
<td>1.14</td>
</tr>
<tr>
<td># 14</td>
<td>1.69</td>
<td>2.25</td>
<td>2.26</td>
<td>4.00</td>
<td></td>
</tr>
</tbody>
</table>

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
- \( \phi(a) = 0.8 \)
- \( \phi(b) = 0.9 \)
- \( \phi(c) = 0.65 \)

Pattern: Irregular
- Total steel area: \( A_s = 1.00 \text{ in}^2 \) at \( \rho = 0.09\% \) (Note: \( \rho < 0.50\% \))
- Minimum clear spacing = 31.50 in

<table>
<thead>
<tr>
<th>Area in(^2)</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in(^2)</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in(^2)</th>
<th>X (in)</th>
<th>Y (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20</td>
<td>0.20</td>
<td>32.0</td>
<td>0.020</td>
<td>0.00</td>
<td>0.00</td>
<td>0.020</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Control Points:

Bending about:

<table>
<thead>
<tr>
<th>Axial Load P kip</th>
<th>X-Moment k-ft</th>
<th>Y-Moment NA depth in</th>
<th>Dt depth in</th>
<th>( \epsilon_{t} )</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>926.9</td>
<td>0.00</td>
<td>-0.00</td>
<td>294.46</td>
<td>132.00</td>
</tr>
<tr>
<td>@ Allowable comp.</td>
<td>741.5</td>
<td>840.16</td>
<td>-0.00</td>
<td>136.53</td>
<td>132.00</td>
</tr>
<tr>
<td>@ fs = 0.0</td>
<td>716.7</td>
<td>921.45</td>
<td>-0.00</td>
<td>132.00</td>
<td>132.00</td>
</tr>
<tr>
<td>@ fs = 0.5#fy</td>
<td>559.6</td>
<td>1270.44</td>
<td>-0.00</td>
<td>103.46</td>
<td>132.00</td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>456.1</td>
<td>1349.80</td>
<td>-0.00</td>
<td>85.07</td>
<td>132.00</td>
</tr>
<tr>
<td>@ Tension control</td>
<td>311.0</td>
<td>1458.27</td>
<td>-0.00</td>
<td>44.00</td>
<td>132.00</td>
</tr>
<tr>
<td>@ Pure bending</td>
<td>-0.00</td>
<td>197.82</td>
<td>-0.00</td>
<td>3.95</td>
<td>132.00</td>
</tr>
<tr>
<td>@ Max tension</td>
<td>-36.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

- X @ Max compression:
<table>
<thead>
<tr>
<th>Axial Load P kip</th>
<th>X-Moment k-ft</th>
<th>Y-Moment NA depth in</th>
<th>Dt depth in</th>
<th>( \epsilon_{t} )</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>926.9</td>
<td>0.00</td>
<td>294.46</td>
<td>132.00</td>
<td>-0.00138</td>
<td>0.650</td>
</tr>
<tr>
<td>@ Allowable comp.</td>
<td>741.5</td>
<td>840.16</td>
<td>136.53</td>
<td>-0.00008</td>
<td>0.650</td>
</tr>
<tr>
<td>@ fs = 0.0</td>
<td>716.7</td>
<td>921.45</td>
<td>132.00</td>
<td>-0.00000</td>
<td>0.650</td>
</tr>
<tr>
<td>@ fs = 0.5#fy</td>
<td>559.6</td>
<td>1270.44</td>
<td>103.46</td>
<td>-0.00069</td>
<td>0.650</td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>456.1</td>
<td>1349.80</td>
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<td>-0.00138</td>
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<td>@ Tension control</td>
<td>311.0</td>
<td>1458.27</td>
<td>44.00</td>
<td>-0.00500</td>
<td>0.900</td>
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<tr>
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<td>-0.00</td>
<td>197.82</td>
<td>3.95</td>
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<td>-36.00</td>
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*** End of output ***
General Information:

- **File Name:** U:\453 - Senior Project\CASITA\SW C.col
- **Object:** Column
- **Code:** ACI 318-11
- **Run Option:** Investigation
- **Run Axis:** X-axis
- **Engineer:**
- **Units:** English
- **Slenderness:** Not considered
- **Column Type:** Structural

Material Properties:

- \( f'c = 1.5 \text{ ksi} \)
- \( E_c = 2207.6 \text{ ksi} \)
- Ultimate strain = 0.003 in/in
- \( \beta_1 = 0.8 \)

Section:

- **Rectangular:** Width = 8 in, Depth = 256 in
- Gross section area, \( A_g = 2048 \text{ in}^2 \)
- \( I_x = 1.11848 \times 10^7 \text{ in}^4 \)
- \( r_x = 73.9008 \text{ in} \)
- \( x_0 = 0 \text{ in} \)
- \( I_y = 10922.7 \text{ in}^4 \)
- \( r_y = 2.3094 \text{ in} \)
- \( y_0 = 0 \text{ in} \)

Reinforcement:

- **Bar Set:** ASTM A615
- **Area (in^2)**
- **Size Diam (in)**
- **Area (in^2)**

<table>
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<tr>
<th>#</th>
<th>0.38</th>
<th>0.11</th>
<th>0.50</th>
<th>0.20</th>
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<td>4</td>
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<td>8</td>
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<td>1.00</td>
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<td>1.27</td>
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<td>2.25</td>
<td>2.26</td>
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</tbody>
</table>

Confinement: Tied; 3 # tying bars, 4 # with larger bars.
- \( \phi_{a} = 0.8, \phi_{b} = 0.9, \phi_{c} = 0.65 \)
- **Pattern:** Irregular
- **Total steel area:** \( A_s = 1.80 \text{ in}^2 \) at \( \rho = 0.09\% \)

Minimum clear spacing = 27.50 in

<table>
<thead>
<tr>
<th>Area in^2</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in^2</th>
<th>X (in)</th>
<th>Y (in)</th>
<th>Area in^2</th>
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<th>Y (in)</th>
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<td>-96.0</td>
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Control Points:

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<tr>
<th>Bending about</th>
<th>Axial Load P kip</th>
<th>X-Moment k-ft</th>
<th>Y-Moment k-ft</th>
<th>NA depth in</th>
<th>Dt depth in</th>
<th>( \varepsilon_t )</th>
<th>Phi</th>
</tr>
</thead>
<tbody>
<tr>
<td>X @ Max compression</td>
<td>1742.6 &lt; 0.00</td>
<td>-0.00</td>
<td>-0.00</td>
<td>466.47</td>
<td>252.00</td>
<td>-0.00138</td>
<td>0.65</td>
</tr>
<tr>
<td>@ Allowable comp.</td>
<td>1394.1 &gt; 2973.23</td>
<td>-0.00</td>
<td>256.51</td>
<td>252.00</td>
<td>0.00005</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>@ fs = 0.5'fy</td>
<td>1369.5 &gt; 3128.11</td>
<td>-0.00</td>
<td>252.00</td>
<td>252.00</td>
<td>0.00000</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>@ Balanced point</td>
<td>1110.4 &gt; 4327.60</td>
<td>-0.00</td>
<td>204.90</td>
<td>252.00</td>
<td>0.00000</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>@ Tension control</td>
<td>929.3 &gt; 4715.19</td>
<td>-0.00</td>
<td>172.63</td>
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<td>0.65</td>
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<td>@ Pure bending</td>
<td>678.2 &gt; 5559.09</td>
<td>-0.00</td>
<td>94.50</td>
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<td>0.00000</td>
<td>0.65</td>
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</tr>
<tr>
<td>@ Max tension</td>
<td>-64.8 &gt; 0.00</td>
<td>0.00</td>
<td>6.97</td>
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<td>0.00000</td>
<td>0.65</td>
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</tbody>
</table>

- X @ Max compression | 1742.6 < 0.00 | -0.00 | -0.00 | 466.47 | 252.00 | -0.00138 | 0.65 |
| @ Allowable comp. | 1394.1 > 2973.23 | 0.00 | 256.51 | 252.00 | 0.00000 | 0.65 |
| @ fs = 0.5'fy | 1369.5 > 3128.11 | 0.00 | 252.00 | 252.00 | 0.00000 | 0.65 |
| @ Balanced point | 1110.4 > 4327.60 | 0.00 | 204.90 | 252.00 | 0.00000 | 0.65 |
| @ Tension control | 929.3 > 4715.19 | 0.00 | 172.63 | 252.00 | 0.00000 | 0.65 |
| @ Pure bending | 678.2 > 5559.09 | 0.00 | 94.50 | 252.00 | 0.00000 | 0.65 |
| @ Max tension | -64.8 > 0.00 | 0.00 | 6.97 | 252.00 | 0.00000 | 0.65 |

*** End of output ***
## DECKING SPECIFICATIONS

### Shallow VERCOR™

**Allowable Uniform Loads (psf)**

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<tr>
<th>DECK SPAN GAGE</th>
<th>CRITERIA</th>
<th>1'-0&quot;</th>
<th>1'-6&quot;</th>
<th>2'-0&quot;</th>
<th>2'-6&quot;</th>
<th>3'-0&quot;</th>
<th>3'-6&quot;</th>
<th>4'-0&quot;</th>
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<td>25</td>
<td>29</td>
<td>42</td>
</tr>
</tbody>
</table>

See footnotes on page 146.
# Shallow VERCOR™

- **36/7 Screw Pattern at Supports**
  - #12 or #14 SDI Recognized Screws
to Supports 0.0385” and thicker
- **Sidelpas Connected with #10 Screws**

## Allowable Diaphragm Shear Strength, q (plf) and Flexibility Factors, F (lb x 10^6)

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<thead>
<tr>
<th>DECK GAGE</th>
<th>SPAN (ft-in)</th>
<th>1'-0&quot;</th>
<th>1'-6&quot;</th>
<th>2'-0&quot;</th>
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<td>F</td>
<td>q</td>
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See footnotes on page 145.
TIA ANA'S SUSTAINABLE DEVELOPMENT

TIA ANA'S

DRAWINGS NOT FOR CONSTRUCTION, TO BE APPROVED BY IN COUNTRY ENGINEERS

2 FRONT ENTRANCE
GENERAL

1. REFER TO SPECIFICATIONS FOR COMPLETE REQUIREMENTS AND SPECIFICATIONS FOR CONSTRUCTION. METHODS WHERE INDICATION SHOWN ON DRAWINGS ARE IN SPECIFICATIONS AND IN THIS GUIDE.

2. CONSTRUCTION SHALL BE IN ACCORDANCE WITH THE 2012 CALIFORNIA BUILDING CODE CODE AND PROVISES FOR THE FOLLOWING LOADS

3. ALL DETAILS OF THE CONSTRUCTION NOT SHOWN OR SPECIFIED ON THE DRAWINGS ARE TO BE EXECUTED AS SHOWN ON THE DRAWINGS AND MAY BE REVISED BY THE CONTRACTOR. DRILLED HOLE DETAILS FOR DEFINITIONS OF DEVELOPMENT LENGTH AND SPLICE LENGTH

4. CONDITIONS WHICH ARE SHOWN AND NOTED.

5. CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS

6. CONTRACTOR SHALL BE SOLELY RESPONSIBLE FOR THE SAFETY AND PRESERVATION OF THE BUILDING AND CONTENTS DURING CONSTRUCTION AND SHALL BE WHOLLY RESPONSIBLE FOR PROVIDING A SAFE PLACE TO WORK. THE CONTRACTOR SHALL EXECUTE WORK TO ENSURE SAFETY OF PERSONS AND PROPERTY AGAINST DAMAGE BY FALLING DEBRIS AND OTHER HAZARDS IN THE WORK AREA. SHORING AND BRACING DURING ALL DEMOLITION AND CONSTRUCTION.

DESIGN BASIS

THE DESIGN IS IN ACCORDANCE WITH THE 2012 CALIFORNIA BUILDING CODE CODE AND PROVIDES FOR THE FOLLOWING LOADS

LIVE LOADS

POOFS 20 PSF

FLOORS RESIDENTIAL 40 PSF

SEISMIC LOADS

V = 132.5 W 41.44 PSF

R = 5 1.01

SOL. TYPE = O 72 lbs/ft

SH = 0.97

SEISMIC LOAD RESISTING SYSTEMS

SPC. REINFORCED MASONRY SHEAR WALLS (GBM) R = 5

EAST-WEST DIRECTION

SPC. REINFORCED MASONRY SHEAR WALLS (GBM) R = 5

FOUNTAINS

1. SLABS ON GRADE AND FOUNDATIONS SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE CONTRACTOR SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATIONS.

2. ALL STEEL TO BE CONSTRUCTED IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATIONS.

3. CONCRETE CASTING TO BE PERFORMED BY TECHNICIANS CERTIFIED IN THE FABRICATION AND INSTALLATION OF CONCRETE STRUCTURES.及び、金属構造の設計と工事における注意点を示す文書。
FOUNDATION PLAN

3/32" = 1'-0"

S.2.1
PLAN NOTES:
1. REFERENCE ARCHITECTURAL DRAWINGS FOR ROOF SLOPE AND ELEVATIONS
2. ALL STEEL MEMBERS TO BE JOINED WITH FULL PENETRATION WELDS UNLESS OTHERWISE NOTED

HSS 4X4X1/2 COLUMNS TYP.

HSS 7X7X3/8 BEAM TYP. U.N.O.

HSS 10X10X1/4

SAN SALVADOR, EL SALVADOR
TIA ANA’S

CAL POLY, SAN LUIS OBISPO
1 GRAND AVENUE
SAN LUIS OBISPO, CA 93401

REVISIONS

NO. DESC. DATE
1 EDIT 4/25/16

TIA ANA’S
SAN SALVADOR, EL SALVADOR

PLOT DATE:
6/22/2016 9:00:45 PM

SHEET NAME:
ROOF PLAN

DRAWN BY:
DR

CHECKED BY:
JAMES MWANGI

SCALE:
3/32" = 1'-0"

SHEET:
S.2.3
ALL SHEAR WALL REINFORCING SHALL BE #4 BARS UNLESS NOTED OTHERWISE.
FOUNDATION DETAILS

1. TYPICAL COLUMN FOOTING
   1/2" = 1'-0"

2. EXTERIOR SHEAR WALL FOOTING
   3/4" = 1'-0"

3. INTERIOR SHEAR WALL FOOTING
   3/4" = 1'-0"

FOOTING SCHEDULE

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<tr>
<th>TYPE</th>
<th>W</th>
<th>H</th>
<th>REINFORCING STEEL</th>
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<tbody>
<tr>
<td>F - 1</td>
<td>6'</td>
<td>1'</td>
<td>#4 @ 6&quot; O.C. EW</td>
</tr>
<tr>
<td>F - 2</td>
<td>3'</td>
<td>1'</td>
<td>#4 @ 16&quot; O.C. TRANS (3) #4 LONG</td>
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</table>

ANCHOR BOLT TYP.
REINFORCEMENT PER FOOTING SCHEDULE
SLAB ON GRADE
SEE 2/S.3-1
ROCK BASE
TIA ANA'S SUSTAINABLE DEVELOPMENT
CASITA

DRAWINGS NOT FOR CONSTRUCTION, TO BE APPROVED BY IN COUNTRY ENGINEERS

FRONT ENTRANCE

S.0.1
GENERAL
1. REFERENCE TO SPECIFICATIONS FOR COMPLETE REQUIREMENTS AND INSTRUCTIONS.
2. INFORMATION SHOWN ON DRAWINGS AND IN SPECIFICATIONS ARE IN ACCORDANCE WITH 2012 CALIFORNIA BUILDING CODE.
3. CONSTRUCTION SHALL BE IN ACCORDANCE WITH THE 2012 CALIFORNIA BUILDING CODE.
4. DETAILS OF THE CONSTRUCTION NOT FULLY SHOWN OR NOTED IN THIS DRAWING OR SPECIFICATIONS MAY BE SHOWN AND NOTIFIED AT THE CONTRACTOR'S OPTION. ALL SHOP AND FIELD WELDING SHALL BE INSPECTED BY THE CONTRACTOR'S TESTING AGENCY.
5. THE CONTRACTOR SHALL NOTIFY THE OWNER'S TESTING AGENCY FOR ALL RIGID MECHANICAL COUPLERS INSTALLED THAT DAY FOR TESTING. TESTING OF THE MECHANICAL COUPLERS INSTALLED AT RIGID JOINTS SHALL BE PERFORMED BY THE OWNER'S TESTING AGENCY. THE CONTRACTOR SHALL CORRECT ALL TYPE II MECHANICAL COUPLERS INSTALLED THAT DAY WHICH FAILED TO PASS THE OWNER'S TESTING AGENCY. THE CONTRACTOR SHALL CORRECT ALL MECHANICAL COUPLERS INSTALLED THAT FAILED THE OWNER'S TESTING AGENCY.

FOOTINGS: 1500 PSF PLACED, AND SHALL BE PUMPED OUT. IF BOTTOMS OF TRENCHES OR FORMS BEFORE OR AFTER CONCRETE IS.

ALLOWABLE BEARING PRESSURES FOR SPREAD FOOTINGS AND GRADE BEAMS SHALL BE CAST IN TRENCHES, UNLESS NOTED OTHERWISE ON THE DRAWINGS.

CONCRETE FOOTINGS
1. ALL CONCRETE SHALL BE REINFORCED. CONCRETE PLACEMENT SHOULD BE IN ACCORDANCE WITH THE MANUFACTURER'S SPECIFICATIONS.

METAL DECK
1. SEE CALCULATIONS FOR DECK Profiles.
2. STEEL DECK SHALL BE WELDED TO ALL STRUCTURAL STEEL STEPS UNLESS NOTED OTHERWISE.

A. REINFORCEMENT INTERSECTIONS BY STRUCTURAL STEEL STEPS, SHAPES, AND IN PLACE, IF NOT OTHERWISE NOTED ON THE DRAWINGS.

REVIEWS
1. EDIT: 4/25/16
2. DRAWING: CAL POLY, SAN LUIS OBISPO
3. SHEET NAME: S.1.1
4. SCALE: 1/4 IN = 1'-0"
5. DATE: 6/22/2016 8:56:27 PM
6. DRAWTED: SAN LUIS OBISPO, CALIFORNIA
7. CHECKED BY: JAMES MANGWANI
8. SHEET NUMBER: GENERAL NOTES

CONCRETE REINFORCEMENT
1. ALL CONCRETE SHALL BE REINFORCED. REINFORCEMENT DETAILS SHALL CONFORM TO ACI 318, "MANUAL OF STANDARD PRACTICE FOR DETERMINING CONCRETE STRESSED STRUCTURES".
2. CONCRETE COVER SHALL BE TO FACE OF BAR, MECHANICAL COUPLER, OR WELDED WIRE FABRIC. UNLESS NOTED OTHERWISE ON THE DRAWINGS.

GENERAL
1. CONSTRUCTION SHALL BE IN ACCORDANCE WITH THE DRAWINGS AND SPECIFICATIONS. IF FOOTINGS CANNOT BE CAST IN TRENCHES, THE CONTRACTOR SHALL VERIFY ALL EXISTING CONDITIONS AND DIMENSIONS AT JOB SITE. THE CONTRACTOR SHALL PROVIDE A SAFE PLACE TO WORK. THE CONTRACTOR SHALL ORDER ADEQUATE ADDITIONAL UNITS OF GRADE BEAMS, UNLESS OTHERWISE NOTED ON THE DRAWINGS.

CONCRETE
1. ALL STEEL TO STEEL CONNECTIONS SHALL BE FULL PENETRATION WELDED. UNLESS NOTED OTHERWISE.

CONCRETE FOOTINGS
1. FOOTINGS SHALL BE CAST IN TRENCHES USING INFORMATION SHOWN ON THE DRAWINGS AND MAY BE REVISED BY THE GEOTECHNICAL ENGINEER.
2. LOCATION OF EXISTING MASONRY SHEAR WALLS (SRM) - R = S
3. EAST/WEST DIRECTION
4. SPECIAL REINFORCED MASONRY SHEAR WALLS (SRM) - R = S
5. SPECIAL REINFORCED MASONRY SHEAR WALLS (SRM) - R = S
6. ALL STEEL TO STEEL CONNECTIONS SHALL BE FULL PENETRATION WELDED. UNLESS NOTED OTHERWISE

CONCRETE MASONRY UNITS
1. MINIMUM MASONRY COMPRESSIVE STRENGTH AT 28 DAYS, N = 150 psi, UNLESS OTHERWISE NOTED.
2. MASONRY MATERIALS SHALL CONFORM TO THE FOLLOWING:

CONCRETE ELEMENTS, UNLESS OTHERWISE NOTED. IN GENERAL, BAR SPLICES MAY BE DELETED AND CONTINUOUS REINFORCEMENT USED AT THE CONTRACTOR'S OPTION. LONG BARS OR BENT BARS SHOWN MAY BE WELDED UNLESS NOTED OTHERWISE ON THE DRAWINGS.

CONCRETE FOOTINGS
1. ALL STEEL TO STEEL CONNECTIONS SHALL BE FULL PENETRATION WELDED.

CONCRETE MASONRY UNITS
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CONCRETE
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2. MASONRY MATERIALS SHALL CONFORM TO THE FOLLOWING:

CONCRETE
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CONCRETE MASONRY UNITS
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2. MASONRY MATERIALS SHALL CONFORM TO THE FOLLOWING:

CONCRETE
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2. CONCRETE COVER SHALL BE TO FACE OF BAR, MECHANICAL COUPLER, OR WELDED WIRE FABRIC. UNLESS NOTED OTHERWISE ON THE DRAWINGS.
PLAN NOTES:

1. REFERENCE ARCHITECTURAL DRAWINGS FOR ROOF SLOPE AND ELEVATIONS
2. ALL STEEL MEMBERS TO BE JOINED WITH FULL PENETRATION WELDS UNLESS OTHERWISE NOTED
**FOOTING SCHEDULE**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>W</th>
<th>H</th>
<th>REINFORCING STEEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>F - 1</td>
<td>6' - 0&quot;</td>
<td>1' - 2&quot;</td>
<td>#4 @ 6&quot; O.C. EW</td>
</tr>
<tr>
<td>F - 2</td>
<td>3' - 0&quot;</td>
<td>1' - 0&quot;</td>
<td>#4 @ 16&quot; O.C. TRANS</td>
</tr>
</tbody>
</table>

(3) #4 LONG

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**SAN SALVADOR, EL SALVADOR**

**CASITA**

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**FOUNDATION DETAILS**

S.4.1
1. **TYPICAL BAR HOOKS AND BENDS**
   - 1/2" = 1'-0"

2. **TYPICAL SPLICE DEVELOPMENT LENGTHS**
   - 1/2" = 1'-0"

3. **TYPICAL STIRRUPS AND TIES**
   - 1/2" = 1'-0"

4. **TYPICAL SLAB ON GRADE**
   - 1/2" = 1'-0"

5. **TYPICAL SLAB ON GRADE JOINT**
   - 1/2" = 1'-0"

6. **TYPICAL CONCRETE DETAILS**

**NOTES:**
1. Hooks for (S), (D), and the closed tie are 135 degrees, TYP. U.O.N.
2. For typical bar hooks and bends, see "Typical Bar Hooks and Bends" detail.
3. Hangers for slabs, and the closed tie are 135 degrees, TYP. U.O.N.
4. For typical bar hooks and bends, see "Typical Bar Hooks and Bends" detail.

**CONCRETE BARS**
- SIZE
- POSITION
- Ldh
- Ld
- Ls

**TYPICAL CROSSIES NOTES:**
A. Size and spacing to match stirrups and ties, U.O.N.
B. For joints and beams where slab occurs on both sides of member, alternate placement of 135 deg. hooks at each consecutive crossie. Where slab occurs on one side only, place the 135 deg. hook on the side without the slab at each consecutive crossie.
C. For column, place crossies as shown in the column schedule. Alternate placement of hook for each consecutive tie.

**CONSTRUCTION JOINT**
- 5/8" dia. x 1"-6" smooth bars, apply bond breaker to one side, space to match typical reinforcing.