Wine history Project Pavilion
Primitivo edition

A Senior Project
presented to
the Faculty of the Architectural Engineering Department
California Polytechnic State University – San Luis Obispo

In Partial Fulfillment
of the Requirements for the Degree
Bachelor of Science in Architectural Engineering

By

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SPECIAL THANKS TO:

Architecture Students: Brandon Fimbrez, Ky Huynh and Toby Peters-Bleck for coordinating successfully with me in order to realize an elegant and useful design for our clients. Also, for working endless hours on the project (Day & Night), being open-minded (Willing to Compromise), and producing the highest degree of work (Great attention to presentation both in the scaled models and diagrams created).

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Architectural Engineering Professor: S.E. Dennis Bashaw for working with me to realize a solution whenever I confronted a road block. Also, for directing me in the right direction throughout the design process (Identifying structural issues in the project and setting up deadlines to create an even distribution of work throughout the whole quarter).

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LPA, Irvine CA: For funding our class and providing office hours for our team to use.

Wine History Project: for giving us the opportunity to design your future pavilion and scheduling meetings with our class, so that we may create the ideal design for you.
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INTRODUCTION

Background: The Wine History Project of San Luis Obispo County is a local organization that documents, preserves, and aims to educate the public about San Luis Obispo County's distinctive wine and food history. Currently, they are looking to invest in a Pavilion that will aid them in both attracting and informing the public about the story of wine in the county.

Team: Our team was compromised of three Architectural students (ARCH), one Architectural Engineer student (ARCE), and one Construction Management student (CM). Our goal was to work together using an IPD design approach in order to create the best Pavilion Design for our client, the Wine History Project.

IPD Approach: Integrated Project Delivery (IPD) is a project delivery approach that integrates all involved parties in the Project to come together from the very beginning to realize the optimal solution for the project. The idea is that every discipline/party is contributing their insight in order to maximize efficiency in the building, reduce waste, lower costs, and come to a solution that everyone can agree to. In our case, our CM student and I where able to have a big influence in the schematic design phase of the pavilion as we worked with ARCH students. We were able to create a design that was structurally sound, constructible, and aesthetically pleasing within a short span of 12 weeks.

Constraints: Our ambitious and creative clients wanted a pavilion with the following requirements: a 400sf requirement, provides security, modular, lightweight framing, easy constructibility, is temporary, can be assembled by college students, all pieces fit in a U-Haul, no cranes during construction, can not dig into the ground, integrated shelving, materials are all local, and design is aesthetically pleasing.

Client Interaction: Every 3 weeks at least one meeting, face-to-face was held in order guide our project towards the clients optimal pavilion design.
PROJECT DESCRIPTION

The pavilion is located on Saucelito Canyon Winery’s site. This project seeks to attract and enlighten guests as they experience both the pavilion and the Wine History showcase. The space provided intends to enhance the guest’s overall experience during their stay at the Saucelito Canyon Winery. This is achieved through first attracting guests to the pavilion through visual appeal of the pavilion and secondly, enlightening the guests about the history of winery through a laid out program provided by the Wine History Project.

This Pavilion will be built on a relatively flat grade. Being near the roadside, the building will attract fellow travelers to its space. Furthermore, the pavilion will be as open as it can be to the guests and nature. Meaning that there is no flooring and there is lots of wind circulation to allow for a more natural experience while inside the pavilion.

The pavilion stands with a ceiling height of 9’ and a grand entrance of 24’ wide. This pavilion is wide open to guests and allows for a circular path throughout the space. The total square footage of the pavilion is estimated to be 600 square feet.

Paneling is composed of a white fabric wrapping around the aluminum framing. Interior paneling is made up of plywood attached to the aluminum framing as well.

Design Assumptions for the lateral load resisting system

1) Grid shell composed of aluminum framing is the main lateral resisting system in both directions.
WESTERN ELEVATION SECTION

Western Elevation Section
1/2" = 1'-0"
Once all individual sections of the structure arrive on site, assembly can begin from the ground-up. Disassembly of the structure will be done in reverse order of assembly.

1. Stake out the site and place foundations into the ground.
2. Assemble the structural system in order of the major sections starting at the floor.
3. Unfold each section of fabric and enclose the exterior.
4. Attach the wood panels into the interior.
5. Place the seating and table assembly into the space and add any final touches.
6. Place artiflacts and attach posters to the pavilion and enjoy the exhibit.

**PRIMITIVO**

**assembly & disassembly**

**PRIMITIVO**

**transportation guideline**

Sizes that fit within the dimensions of a 26' U-Haul truck:
Largest size of an individual section: 23 ft. long, 7 ft. wide, and 6 ft. tall

Weight of each section that four movers can carry:
Max weight per person: 50 lbs.
Average weight of structural tubing: 3 lbs/ft
Total weight of one section carried by four people: 200 lbs.
Max total length of structural tubing per section: 86 ft.

We divided the whole structure into a few major sections comprised of even smaller segments, making sure that each piece carefully fits inside a 26' U-Haul truck. We also considered a maximum weight of 50 lbs. for a single person to carry. If we assume that there are at least four movers, each segment is limited to a maximum of 200 lbs.
**INTERIOR PANELING**

A. The raised display area allows guests to see exhibitions up close while also maintaining distance for protection. It is compatible with a glass case for security purposes, but can be used without.

B. Pavilion integrates seating for up to five in ribbon form. Located 18” off the ground and made with panel wood over honeycomb structure.

C. Interior facade of pavilion created with panel wood, offset 1/2”. Attached to wall with hook system making assembly and replacement easy. Typical panel is 1” x 7/16 “ x 11 1/2”.

D. Shelving for placement of exhibitions highly customizable.
STRUCTURAL CALCULATIONS

PRIMITIVO PAVILION DESIGN

SAN LUIS OBISPO, CALIFORNIA

Prepared by:

Erick Vazquez

December 13, 2019
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APPENDIX -------------------------------------------------- A1 - A7
1) Applicable Codes:
   a. International Building Code 2018
   c. American Society of Civil Engineers 7-16
   d. Aluminum Design Manual 2010

2) Design Loads:
   a. Dead Loads – Actual in-place weights of all materials shown on the drawings and specifications
   b. Live Loads – uniform as follows:
      i) Roof: 10 psf
   c. Wind Loads: Based on CBC section 1609 with exposure C condition with a basic wind speed of 92mph. Design Process based on ASCE 7-16 Section 26 and 27
      i) \( q_z = 0.00256K_z*K_{zt}*K_d*K_e*V^2 \)
   d. Seismic Load: based on CBC Section 1613 and ASCE 7-16 Section 12.8
      i) To be determined at a later date

3) Foundation Design
   a. No geotechnical report provided. The recommended design soil values are as follows by CBC TABLE 1806.2. Since class of soil is unknown, will go with worst case soil.

<table>
<thead>
<tr>
<th>Vertical Foundation Pressure</th>
<th>Lateral Bearing pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,500 psf</td>
<td>400 psf - 100 psf</td>
</tr>
</tbody>
</table>
MATERIAL & MODELING CRITERIA

Material Criteria

1) Aluminum
   a. For Frames: Using 6061 T6 Aluminum Round Tubing
   b. For Connections: Aluminum plates, Auger Anchors, Aluminum Bolts, 6061 T6 Custom Tube Sleeves
   c. For foundations: Aluminum 6061 T6 Base Plates

3) FOUNDATIONS
   a. 8"X8"X12" CMU BLOCKS
   b. PE46-Hex (American Earth Anchors - Auger Anchor)
   c. Non Shrink Grout

SAP2000 Modeling Criteria

The actual structure is composed of curved aluminum members, but in the sap model the members are all modeled as straight continuous frame members. This is allowable to do because there are multiple members that are generating the curve of the structure. Most eccentricities between the straight member and actual member fall between 0 3/8" – 1 1/2" distance of eccentricity. These are relatively small distances, which causes the overall model to be reasonably accurate. Only a few members exceed that eccentricity with a distance of around 2". To account for this eccentricity difference, SCI PUBLICATION P281 by Charles King & David Brown suggests applying a moment created by the axial force of that member times the eccentricity, to the ends of that member. This will provide a more accurate model.

Every connection is a fixed connection between the members while the supports are all pinned connections.

All loads are applied to the joint by multiplying the tributary area of that joint by the given design load(dead, live & wind). Wind load has an applied windward pressure, leeward pressure, and uplift pressure.

Furthermore, the model is running both LRFD design code checks and ASD force analysis for foundation design.
Design Loads
**Tabulation of Paneling Dead Load**

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>WEIGHT</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Searman 8421 Polyester Fabric w/ Acrylic Coating</td>
<td>22.02 oz</td>
<td>(1)</td>
</tr>
<tr>
<td>2. 1/2&quot; Sanded Plywood</td>
<td>0.93 lb/ft²</td>
<td>(2)</td>
</tr>
<tr>
<td>3. 3/4&quot; OD x 1/8&quot; wall 6001-T6 Alumina Round Tubing</td>
<td>1.20 lb/ft</td>
<td>(3)</td>
</tr>
<tr>
<td>4. 1/8&quot; Vinyl Coated Stainless Steel Cable Reel</td>
<td>0.042 lb/ft²</td>
<td>(4)</td>
</tr>
</tbody>
</table>

**References**

1. SLO Sail & Canvas
2. Home Depot
3. Speedy Metals
4. E-Rigging

**SAP Model Dead Load**

To account for the dead load, a self-weight multiplier of 25 was used between the aluminum round tubing & every other item in the structure. First, only materials that play a big role into the weight of the building are aluminum round tubing, pinned connections, & the interior plywood.

- Comparing aluminum round tubing to plywood being used:

**Aluminum**

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 1/2&quot; x 12&quot; x 1/8&quot;</td>
<td>0.0927 psf</td>
</tr>
</tbody>
</table>

**Plywood**

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>12&quot; x 12&quot; x 1/8&quot;</td>
<td>0.0927 psf</td>
</tr>
</tbody>
</table>

Looking at the worst case to of the model, there is a case where there is 112 of plywood & 80" of aluminum.

- Weight of plywood:
  \[ \text{Weight of plywood} = 0.0108 \text{ psf} \times (11.12^2 \times 1/8) \times (1/12) \]
  \[ = 18.3 \text{ lb} \]

- Weight of aluminum:
  \[ \text{Weight of aluminum} = 0.0927 \text{ psf} \times (10.8 \text{ ft}) \times (0.5 \text{ in} \times 1/8" \text{ in}) \]
  \[ = 8 \text{ lb} \]
Frame Analysis
FINDING CAPACITIES ASSOCIATED WITH 2 3/4" OD X 1/8" THK ALUMINUM ROUND TUBE 2024-T6 (LONGEST FRAME MEMBER)

MATERIAL PROPERTIES
- $E = 10,100 ksi$
- $F_{ty} = 35 ksi$
- $F_{tu} = 38 ksi$
- $A_g = 1.03 in^2$
- $I_x = I_y = 0.89 in^4$

TENSILE STRENGTH (ADM 2010 D.2)

- YIELDING
  \[ P_n = F_{ty} A_g (D^2 - 1) \]
  \[ = 35 ksi (1.03 in^2) \]
  \[ = 34.53 kips \]
  \[ \phi P_n = 0.9 (34.53 kips) \]
  \[ = 32.5 kips \]

- Rupture
  For Unwelded Members
  \[ P_n = F_{tu} A_e / k_e (D^2 - 3) \]
  \[ = 38 ksi (1.03 in^2 - .125 in^2(1/2)) \]
  \[ = 31.39 kips \]
  \[ \phi P_n = 0.75 (31.39 kips) \]
  \[ = 23.5 kips \]
**COMPRESSION STRENGTH (AISC E)**

- **E 2 EFFECTIVE LENGTH (6.3 \( \rightarrow k = 1.0 \))**
- **E.3 MEMBER BUCKLING**
  
  \[ P_n = f_c A_g \]  
  
  \[ C_c \rightarrow S_2 \rightarrow V_{ry} = S_2 \rightarrow f_c \]

  \[ C_c = 0.41 \text{ ksi} \]  
  
  \[ C_c = 0.41 \text{ ksi} \]  
  
  \[ \text{TABLE 84.2} \]

  **NOTE**: FOR COMPRESSION IN COLUMNS \& BEAM FLANGES, ALL FRAME ELEMENTS ARE EXPOSED TO COMBINED BENDING \& AXIAL FORCES

\[ P_c = f_{cy} \left( 1 + \left( \frac{f_{cy}}{2250 \text{ ksi}} \right)^{1/2} \right) = 35 \text{ ksi} \left( 1 + \left( \frac{35 \text{ ksi}}{2250 \text{ ksi}} \right)^{1/2} \right) = 39.37 \text{ ksi} \]

\[ D_c = \frac{8c}{10} \left( \frac{8c}{10} \right)^{1/2} = \frac{39.37 \text{ ksi}}{10} \left( \frac{39.37 \text{ ksi}}{10,000 \text{ ksi}} \right)^{1/2} = 0.246 \text{ ksi} \]

\[ C_c > 0.41 \left( \frac{39.37 \text{ ksi}}{0.246 \text{ ksi}} \right) = 65.67 \]

\[ V_L = \frac{10 \times (4' - L')}{0.93} = 58.06 \]

**SINCE** 58.06 < 65.67

\[ f_c = 0.85 (\theta_c - \theta_c (k + V)) \leq f_{cy} \]

\[ = 0.85 \left( 39.37 \text{ ksi} - 0.246 \text{ ksi} (58.06) \right) \leq 35 \text{ ksi} \]

\[ = 21.32 \text{ ksi} \geq 35 \text{ ksi} \)

\[ P_n = f_c A_g \]

\[ = 21.32 \text{ ksi} \times (1.03 \text{ in}^2) = 21.96 \text{ k} \]

\[ \Phi P = 0.9 (21.96 \text{ k}) = 19.77 \text{ k} \]

**SUMMARY**: **COMPRESSION STRENGTH**

\[ \Phi P = 19.77 \text{ k} \]
**Flexural Strength (AOM F)**

\[ M_a = 1.17 Fy S \quad \text{(Tensile Yielding & Compressive Yielding)} \quad (F.6.1) \]

\[ = 1.17 (35ksi) (0.047in^3) \]

\[ = 26.5 ksi \]

\[ \Delta M_a = 0.9 (26.5 ksi) \rightarrow 24.6 ksi \]

\[ M_a = 1.24 Fy S / K_s \quad \text{(Tensile Rupture)} \quad (F.6.1) \]

\[ = 1.24 (38 ksi) (0.047 in^3) / 1.1 \]

\[ = 27.72 ksi \]

\[ \Delta M_a = 0.75 (27.72 ksi) = 20.79 ksi \]

\[ M_n = \frac{F_b S}{L} \quad \text{(Local Buckling)} \quad (F.6.2) \]

\[ F_b = \frac{\pi^2 E}{(16(L / 2s))^2} \]

\[ = \frac{\pi^2 (10,100 ksi)}{16(L / 2s)^2 (1 + (L / 2s))^2} \]

\[ = 472.5 ksi \]

\[ M_n = 472.5 ksi (0.047 in^3) \]

\[ = 305.75 ksi \]

\[ \Delta M_n = 0.9 (305.75 ksi) \]

\[ = 275 ksi \]

**Summary: Flexural Strength**

\[ \delta M_a = 20.79 ksi \]
SHEAR STRENGTH (ADM 6)

V_n = F_s A_g / 2  Nominal Shear Strength (4.3.1)

F_s = 0.6 f_y T (TABLE A.2.1)
    = 0.6 (35 ksi)
    = 21 ksi

F_s = F_y * V_n = 21 ksi (1.03 in^2) / 2
    = 20.4 kips (Yielding)

\[ \Delta V_n = 0.9 (20.4 \text{kips}) \]
\[ = 18.35 \text{kips} \]

CONCLUSION: Shear Capacity
\[ \Delta V_n = 18.35 \text{kips} \]
STRESSES IN CURVED ELEMENTS (ADH 43.2)

(EQUATION H3.2)

\[
\frac{f_c}{\phi_f} + \frac{f_b}{\phi_b} + \left[\frac{f_s}{\phi_s}\right]^2 \leq 1.0 \quad \Rightarrow \quad \frac{P_d}{\phi_P} + \frac{M_d}{\phi_M} + \left(\frac{V_d}{\phi_V}\right)^2 \leq 1.0
\]

FROM PREVIOUS PAGES

\phi_P = 19.77 K
\phi_M = 20.79 K
\phi_V = 18.25 K

FROM SAP2000 MODEL

WORST AXIAL FORCE \( P_d = 3.685 K \) — (FRAME 276 LOAD COMBO LRFD EQU 4)
WORST SHEAR FORCE \( V_d = 0.525 K \) — (FRAME 145 LOAD COMBO LRFD EQU 4)
WORST MOMENT FORCE \( M_d = 6.657 K \) — (FRAME 12 LOAD COMBO LRFD EQU 3)

PLUGGING INTO EQU (H3.2)

\[
\frac{P_d}{\phi_P} + \frac{M_d}{\phi_M} + \left(\frac{V_d}{\phi_V}\right)^2 \leq 1.0
\]

\[
\frac{3.685 K}{19.77 K} + \frac{6.657 K}{20.79 K} + \left(\frac{0.525 K}{18.25 K}\right)^2 \leq 1.0
\]

0.186 + 0.320 + 0.0008 ≤ 1.0

0.507 ≤ 1.0 ALL MEMBERS OKAY

SUMMARY:

2 3/4" OD x 1/8" THK
ALUMINUM 6061-T6 ROUND TUBING IS SAFE TO USE FOR ALL MEMBERS IN STRUCTURE
FRAME DESIGN CONTINUED

FRAME 340, 12, 342 FROM SAP2000, ARE THE ONLY MEMBERS THAT EXCEED 4'6" DISTANCE, SO THE COMpressive STRENGTH WILL HAVE TO BE REVISITED.

ΩM = 20.7k/in (STAYS THE SAME)  P.9
ΩM = 18.35k (STAYS THE SAME)  P.9

FOR FRAME MEMBER, COMPRESSIVE STRENGTH IS AS FOLLOWS (ADME)

\[ P_n = F_c A_g (2.3 - 1) \]

\[ C_c = 65.67 = 52 \]  P.9

\[ \frac{K_L}{r} = \frac{(1.0)(6.81)}{0.93} = 83.87 \]  FRAME (340)  \[ \frac{K_L}{r} > C_c \]

\[ \frac{K_L}{r} = \frac{1.0(5.61)}{0.93} = 70.96 \]  FRAME (L2)  \[ \frac{K_L}{r} > C_c \]

\[ \frac{K_L}{r} = \frac{1.0(5.61)}{0.93} = 64.52 \]  FRAME (342)  \[ \frac{K_L}{r} < C_c \]

\[ F_c = 0.85 \left( 39.37ksi - 0.246 \left( 64.52 \right) \right) \]

\[ = 19.97 ksi < 35 ksi = F_{cy} \]

\[ \sigma_{Pn} = F_c A_g \]

\[ = 0.9 \left( 19.97 ksi \right) \left( 1.03 \right)^2 \]

\[ \sigma_{Pn} = 18.5 ksi \]

FROM SAP2000 MODEL, P = 0.725k (LRFD EQ 3)

\[ V = 0.035k (LRFD EQ 3) \]

\[ M = 1.85k/in \] (LRFD EQ 3)

ADN EQN H3-3

\[ \frac{P}{\sigma_{Pn}} + \frac{M}{\sigma_{Pn}} + \left( \frac{V}{\sigma_{Pn}} \right)^2 \leq 1.0 \]

\[ \frac{0.725k}{18.5k} + \frac{1.85k/in}{20.7k/in} + \left( \frac{0.035k}{18.5k} \right)^2 \]

\[ = 0.128 \leq 1.0 \]  FRAME 342 IS OK
FRAME DESIGN CONTINUED

FRAME C2

\[ F_c = \frac{0.85 \pi^2 E}{(\frac{KL}{V})^2} = \frac{0.85 \pi^2 (10,100 ksi)}{(70.96)^2} = 16.83 ksi \]

\[ \sigma_{Pa} = F_c A_g \]

\[ = 16.83 ksi \times (1.03 in^2) (0.9) = 15.60 kips \]

\[
\text{ADM EQU H3-3} \\
\frac{P_d}{\sigma_{Pa}} + \frac{M_d}{OMa} + \left( \frac{V_d}{OMa} \right)^2 \leq 1.0 
\]

\[ 0.218k + \frac{3.078k}{20.79k} + \frac{(0.048k)^2}{18.35k} \leq 1.0 \]

\[ = 0.277 \leq 1.0 \text { FRAME C2 OK } \checkmark \]

FRAME 340

\[ F_c = \frac{0.85 \pi^2 E}{(\frac{KL}{V})^2} = \frac{0.85 \pi^2 (10,100 ksi)}{(83.87)^2} = 12.05 ksi \]

\[ \sigma_{Pa} = F_c A_g \]

\[ = 0.9 (12.05 ksi) (1.03 in^2) = 11.166 kips \]

\[
\text{ADM EQU H3-3} \\
\frac{P_d}{\sigma_{Pa}} + \frac{M_d}{OMa} + \left( \frac{V_d}{OMa} \right)^2 \leq 1.0 
\]

\[ 0.959 kip + \frac{4.85 k}{20.79 k} + \frac{(0.072 k)^2}{18.35 k} \leq 1.0 \]

\[ = 0.319 \leq 1.0 \text { FRAME 340 OK } \checkmark \]

SUMMARY: FRAMES 340, 621, 2 342 PASS COMBINED STRESS CHECKS.
Wind Analysis
**ASCE 7-16 Ch. 24 General Requirements**

**Basic wind speed** $v = 92$ mph (7-16, Fig 24.5-18)

**Wind Directionality Factor** $K_d = 0.95$ (7-16, Table 24.6-1)

- For round or octagonal structures $U/H$ non-axisymmetric structural system
- Exposure C (7-16, 24.7.3)
- Surface Roughness C

**Topographic Factor** $K_t = 1.0$ (7-16, 26.8)

- No Escarpment or 2-Wedge

**Ground Elevation Factor** $K_e = 1.0$ (7-16, Table 26.9-1)

**Velocity Pressure** $q_z = 17.5$ psf (7-16, 26.10)

$$q_z = 0.00256 (0.86)(1.0)(0.95)(1.0)(92 	ext{ mph})^2$$

$$= 17.5 	ext{ psf}$$

$U/H = 1.6$, $K_z = 0.85$ (7-16, Table 26.10-1)

**Gust Effect Factor** $= 0.85$ (7-14, 24.11)

- Low-rise buildings are permitted to be considered rigid

**Enclosure Classification** (7-16, 26.2)

- Partially Enclosed

**Internal Pressure Coefficient** $C_{pi} = +0.85$ (7-16, Table 26.13-1)

**ASCE 7-16 Ch. 27 Directional Procedure**

**Step (1-3) Shown Above**

**Step (4)**

\[ C_p \]

- Using (ASCE 7-16, Fig 27.3-3) → Arcued Roof

- Rise to Span Ratio: $9'/24' = 0.375 = r$

**Step (5)** Use (ASCE 7-16, Eqn 27.3-1)

\[ p = g_c C_p - q_z (6 C_p l) \]

$\rho = 9_c C_P - q_z (6 C_P l)$

<table>
<thead>
<tr>
<th>Unloaded Quarter</th>
<th>Center Half</th>
<th>Leeward Quarter</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4</td>
<td>-0.75</td>
<td>0.5</td>
</tr>
<tr>
<td>[ C_p ]</td>
<td>0.525</td>
<td>-1.075</td>
</tr>
<tr>
<td>0.525</td>
<td>-1.075</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Step (7)**

\[ p = g_c C_p - q_z (6 C_p l) \]

$\rho = 9_c C_P - q_z (6 C_P l)$

<table>
<thead>
<tr>
<th>Unloaded Quarter</th>
<th>Center Half</th>
<th>Leeward Quarter</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.4 psf</td>
<td>-23.6 psf</td>
<td>17.1 psf</td>
</tr>
</tbody>
</table>

Engineer: EAU
Serviceability Check
DEFLECTION LIMITATIONS (VERTICAL) 

[IBC 2018 TABLE 1604.2]

For Roof Members

SUPPORTING NONPLASTER CEILING

LIVE = 1/120
DEAD + LIVE = 1/180

WORST DEFLECTIONS OCCUR IN THIS AREA OF THE BUILDING

PLAN VIEW

\[ \frac{24\text{'}(13\%)}{240} = 1.2\text{'} \]

DEAD + LIVE

\[ \frac{241(12\%)}{180} = 1.6\text{'} \]

FROM SAP2000 MODEL, WORST DEFLECTIONS

D+L = -1.67" \[ \frac{1.67}{1.2} = 1.04 \text{ } \checkmark \text{ OK} \]
L = -0.96" \[ \frac{0.96}{1.2} = 0.8 \text{ } \checkmark \]

- THE D+L IS WITHIN 4% OF RECOMMENDED IBC DEFLECTION LIMIT. I WOULD SAY THIS IS OK.

VERTICAL DEFLECTIONS ARE OKAY \[ \checkmark \]
NOTE: AS STATED IN WEST & FISHER'S ARTICLE, "SERVICEABILITY LIMITS STIPULATED UNDER WIND LOAD", PUBLISHED IN 2003, "NONE OF THE THREE NATIONAL BUILDING CODES IN THE UNITED STATES SPECIFY A LIMIT TO LATERAL FRAME DEFORMATION UNDER WIND LOAD."

THIS IS IMPORTANT BECAUSE THIS LIMIT IS LEFT TO ENGINEERING JUDGEMENT. CURRENTLY WEST & FISHER RECOMMEND A LIMIT FROM H/100 TO H/600, WHERE H/100 APPLIES TO METAL PANELS, THE REASON FOR THIS MAY BE DUE TO THE METAL PANELS BEING MORE RIGID THAN FLEXIBLE AS COMPARED TO OTHER MATERIALS. THIS PROJECT USES FABRIC WHICH IS VERY FLEXIBLE THAT IS WHY MY LIMITED ENGINEERING JUDGEMENT SAYS TO USE H/100 ~ H/60. THIS LIMIT WAS MADE TO PROTECT YOUR EXTERIOR CLADDING WHICH IN MY CASE IS FABRIC.

W/ A HEIGHT OF H=9', I CONCLUDE DRIFT LIMITATIONS ARE LIMITED TO A RANGE FROM 1.0'' TO 1.09'' & LOWER.

FROM SAP2000 MODEL, WORST LATERAL DEFLECTIONS ARE:

NODE 30 1.24" (LRFD EQU 4)
NODE 133 0.73" (LRFD EQU 4)

1.24 IS WITHIN 1.8" & 1.24" & 1.08" ✔

WIND DRIFT SERVICEABILITY CHECK PASSES
Connection Design
Slotted Connection
CASE (1) (ALUMINUM ROUND TUBE FITTINGS)

2 3/4" OD x 1/8" thick G061-TG ROUND TUBE
3 3/4" OD x 1/8" thick G061-TG ROUND TUBE FITTING
1/8" OD G061-TG ALUMINUM BOLT

MATERIAL PROPERTIES
G061-TG → E = 10,100 ksi

Ftu = 38 ksi
Fsu = 24 ksi

CAPACITY SUMMARY

(BOLT TENSION) 6.47 kL
(SHEAR RUPTURE) 8.112 kL
(BOLT BEARING) 7.125 kL
(BLOCK SHEAR) 8.34 kL
(MOMENT CAPACITY) 6.50 kip
BOLT TENSION (ADM J3.5)

$$R_n = \frac{n(D - \frac{1.191}{n})^2}{\bar{A}_u} = \frac{n(\frac{1}{2} - \frac{1.191}{n})^2}{\bar{A}_u} = 38 \text{ksi}$$

$$D = \frac{1}{2}'' \text{ diameter of bolt}$$

$$n = 13 \text{ threads per inch}$$

$$\bar{A}_u = 0.13 \times 10^2 (38 \text{ ksi}) (2 \text{ bolts}) = 9.95 \text{ k}$$

$$D R_n = 0.65 (9.95 \text{ k}) = 6.47 \text{ k}$$

SHEAR RUPTURE (ADM J3.6)

$$R_n = \frac{n(D - \frac{1.191}{n})^2}{\bar{A}_u} = 0.13 \times 10^2 (24 \text{ ksi}) (2 \text{ bolts}) (2 \text{ shear planes}) (2 \text{ bolts})$$

$$= 12.48 \text{ k}$$

$$D R_n = 0.65 (12.48 \text{ k}) = 8.112 \text{ k}$$

BOLT BEARING (ADM J3.7)

$$R_n = \frac{d e}{t} F_{tu} \leq 2 d e F_{tu}$$

$$d e = 1'' \text{ edge distance}$$

$$t = \frac{3}{4}'' \text{ thickness of connected part}$$

$$R_n = 1'' (1/4'')(38 \text{ ksi}) \leq 2 (5/8'')(3/4'')(38 \text{ ksi})$$

$$9.5k \leq 11.875k$$

$$D R_n = 0.75 (9.5 \text{ k})$$

$$= 7.125 \text{ k}$$
BLOCK SHEAR (AISC 360-10, J4-9)

* NOTE: AISC DOES NOT ADDRESS BLOCK SHEAR EQN, BUT AISC 360-10 PROVIDES A WAY TO CHECK IT. STEEL CODE IS OKAY TO USE HERE BECAUSE THE ALUMINUM BEHAVES SIMILARLY.

\[ R_a = 0.60 F_{u,A} N + V_{d,U} f_{u, A} t \leq 0.60 F_{u,A} N + U_{d,U} f_{u, A} t \]

**BLOCK SHEAR FAILURE 1**

\[ A_{u,v} = 2(3)(\frac{1}{8}) = 0.75 \text{in}^2 \]
\[ A_{u,v} = A_{u,v} - 2(\sin(\frac{1}{8})\cos(1.5\theta)(1/2)) = 0.5625 \text{in}^2 \]
\[ A_{u,v} = 0 \text{ (really small so I used 0.)} \]

\[ P_0 = 0.60(38)(0.5625) + 1.0(38)(0) \leq 0.60(35)(0.75) + 1.0(38)(0) \]
\[ 12.825k \leq 15.75k \]

\[ 12.825k = 15.75k \]

**BLOCK SHEAR FAILURE 2**

**BLOCK SHEAR 2 WILL NOT GOVERN SINCE IT WILL RESULT IN A GREATER CAPACITY THAN BLOCK SHEAR 1 FAILURE**
MOMENT CAPACITY

USING THE BOLT TENSION CAPACITY

\[
\theta_{2a} = 0.47k \text{ (for 2 bolts)}
\]

\[
= 3.235k \text{ (for 1 bolt)}
\]

MOMENT CAPACITY

\[
\theta_{\text{Mcap}} = T_i \text{ (lbf-in)}
\]

\[
= 3.235k(2\text{ in})
\]

\[
= 6.47k\text{ in}
\]
SUMMARY REPORT FOR ALUMINUM ROUND TUBE FITTING w/ 1/4" BOLTS

BOLT TENSION: $\Delta P_{tu} = 6.47\, \text{kN}$

SHEAR RUPTURE: $\Delta P_{ru} = 8.11\, \text{kN}$

BOLT BEARING: $\Delta P_{bb} = 7.13\, \text{kN}$

BLOCK SHEAR: $\Delta P_{bs} = 8.34\, \text{kN}$

MOMENT CAPACITY: $\Delta M_{m} = 6.47\, \text{kNm}$

SAP2000 ELEMENT JOINT FORCE - MAX VALUES

<table>
<thead>
<tr>
<th>VALUE</th>
<th>JOINT</th>
<th>LOAD COMBO</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAX AXIAL</td>
<td>3.608k</td>
<td>85</td>
</tr>
<tr>
<td>MAX SHEAR</td>
<td>0.525k</td>
<td>70.75</td>
</tr>
<tr>
<td>MAX MOMENT</td>
<td>6.657k in</td>
<td>13</td>
</tr>
</tbody>
</table>

FITTING: $\Delta P_{tu} = 6.47\, \text{kN}$

\[
\frac{3.61k}{6.47k} = 0.56 < 1 \quad \text{within 5%}
\]

CONCLUSION: 3' 00" x 7/8" CUSTOM FITTING SLOT CONNECTION (\#)

1/2" Ø BOLTS WORKS FOR ANY FRAME CONNECTION.
NOTE: EVERY CONNECTION WILL BE CUSTOM MADE. EVERY FRAME WILL SLIDE INTO A SLOT WITH TWO 1/2" Ø BOLT HOLES SPACED APART 2" WITH 1" MIN EDGE DISTANCE. THE PREVIOUS PAGE SHOWS HOW THAT DETAIL WILL LOOK.

EXAMPLES OF JOINT CONNECTS.

**Joint 99**

**Joint 23**

3/4" x 1/2" Aluminum Pipe TYP.

1" min 2" 1"

1" min 2" 1"

1" min 2" 1"

1" min 2" 1"

*Some eccentricity is introduced*

*Worst detail
Plate Design
TYPES OF PLATES (NOT TO SCALE)

PLATE (A)

- 1 1/8" nodes
- 2" bolt holes
- Goes on every foundation node, except on node C4.

PLATE (B)

- 3/8" thick plate
- Goes on node C4.

PLATE (C)

- 1'-11 5/8" base plate on top of every foundation system.
PLATE DESIGN CASE (A)

MATERIAL PROPERTIES

- TENSILE YIELDING (ADM 0.2%)

\[ \sigma_p = F_{y} (A_0 - A_u) + F_{\text{weld}} A_u \]

VERTICALLY

\[ \sigma_p = \left[ 35 \text{ ksi} \right] \left[ \frac{1}{2} \text{ in.} \right] - \left( \frac{1}{2} \text{ in.} \right) + \left( \frac{1}{2} \text{ in.} \right) \times 0.9 = 103.4 \text{ kpsi} \]

HORIZONTALLY

\[ \sigma_p = \left[ 11 \text{ ksi} \right] \left[ \frac{1}{2} \text{ in.} \right] \times 0.9 = 59.4 \text{ kpsi} \]

- TENSILE RUPTURE (ADM 0.2%)

\[ \sigma_p = F_{u} (A_0 - A_u) / K_t + F_{\text{weld}} A_u \]

HORIZONTALLY

TWO RUPTURE PLANES

RUPTURE 1: \[ A_0 = 12(1/2) - 9(1/2 + 1/4)(1/2) = 4.875 \text{ in.} \]

RUPTURE 2: \[ A_0 = 12(1/2) - 5(1/2 + 1/4)(1/2) \times \frac{2(1.375)(4)}{4(1.375)} = 4.94 \text{ in.} \]

RUPTURE 1 GOVERNS

\[ \sigma_p = 0.75 \left[ 0.75 \text{ kpsi} (4.875 \text{ in.}) \right] \] (WELD AFFECTED)

\[ = 187.75 \text{ kpsi} \]

VERTICALLY

TWO RUPTURE PLANES

RUPTURE 2: \[ A_0 = 7.75(1/2) - (1/2 + 1/4)(1/2) = 3.0625 \text{ in.} \]

RUPTURE 1: \[ A_0 = 7.75(1/2) - 3(1/2)(1/2) + \frac{1.375}{1.375} + \left( \frac{3}{4} \right) (1/2) \times \frac{40}{3.875} = 2.96 \text{ in.} \]

\[ \sigma_p = \left[ 38 \times \left( 2.96 - (1/2) \right) \right] + 24(1)(1/2) \times 0.75 \]

\[ = 79.11 \text{ kpsi} \]
**Compression Strength (AASHTO)**

\[ \sigma_{ck} = \sigma_p \left[ 1 - \left( \frac{w_d}{A_g} \right)^{1/3} + P_{k0} \left( \frac{w_d}{A_g} \right)^{1/3} \right] \]  (AASHTO E20-1)

**Steps**

1. Find \( B_p \rightarrow D_p \rightarrow C_p \)

2. \( k/l \rightarrow F_c \)

3. \( P_{k0}, P_{uw} \rightarrow \sigma_{ck} \)

**\( C_p \) (Not Weld Affected)**

\[ B_p = \frac{F_{cy}(1 + (\frac{w_d}{A_g})^{1/3})}{E} \]  Table 8.4.2

\[ = 35(1 + (35/1500)^{1/3}) \]

\[ = 45 \]

\[ D_p = \frac{B_p}{10 (0.9/6)} \]

\[ = \frac{45}{10} \left( \frac{15}{10,100} \right)^{1/2} \]

\[ = 0.3 \]

\[ C_p = 0.41 \frac{B_p}{D_p} \]

\[ = 0.41(45)/0.3 \]

\[ = 61.42 = 52 \]

**Horizontal**

\[ \frac{k_l}{l} = \frac{0.01(12)}{61.42} = 0.01 \]

\[ F_c = 0.85 \frac{F_{cy}}{E} \left( \frac{10}{l} \right)^{1/2} = 0.85 \frac{F_{cy}}{E} \left( \frac{10}{61.42} \right)^{1/2} \]

\[ = 12.2 \text{kips} \]

\[ \sigma_{ck} = 0.9 \left[ 12.2k(12(\frac{w_d}{A_g})) \right] \]

\[ = 65.9 \text{kips} \]

**Vertical**

\[ k_l = \frac{(1.6)(7/4)}{0.144} = 50.3 \]

\[ P_{k0} = F_c A_g = 0.85 (B_p - D_p (w_d/A_g)) A_g \]

\[ = 0.85(45 - 0.3(35/1500))(7350.8) \]

\[ = 92.11 \text{kips} \]

\[ P_{uw} = 0.85 \frac{F_{cy}}{E} A_g \]

\[ = 0.85 \frac{F_{cy}}{E} (10/100) (7350.8) \]

\[ = 121.16 \text{kips} \]

\[ \sigma_{ck} = 52.11 (1 - \frac{1.5}{7350.8}) + 121.16 (\frac{1.5}{7350.8}) \] 0.9

\[ = 86.5 \text{kips} \]
FLEXURAL CAPACITY (AOMF)

TENSION (FLEXURE)

\[ F_{d} = 1.30 \left[ F_{y} (1 - A_{wz} / A_{s}) + F_{uy} A_{wz} / A_{s} \right] \]

**HORIZONTAL**

\[ F_{d} = 1.30 [11(15)] \]
\[ = 19.3 \text{ksi} \]

\[ \Delta M_{x} = F_{d} S_{x} = 0.9 (14.3 \text{ksi})(0.5) \]
\[ = 6.4 \text{ k} \text{in} \]

\[ \Delta M_{y} = 0.9 (14.3 \text{ksi})(12) \]
\[ = 154.4 \text{ k} \text{in} \]

**VERTICAL**

\[ F_{d} = 1.30 [35(1 - 0.425)(0.3)] \]
\[ = 41.2 \text{ ksi} \]

\[ \Delta M_{x} = F_{d} S_{x} = 0.9 (41.2 \text{ ksi})(0.3) \]
\[ = 11.2 \text{ k} \text{in} \]

\[ \Delta M_{y} = 0.9 (41.2 \text{ ksi})(4.38) \]
\[ = 162.4 \text{ k} \text{in} \]

RUPTURE (FLEXURE)

\[ F_{y} = 1.42 [F_{u} (1 - A_{wz} / A_{s}) / k_{t} + (F_{u} A_{wz} / A_{s})] \]

**HORIZONTAL**

\[ F_{d} = 1.42 (24 \text{ k} \text{si})(1) \]
\[ = 34 \text{ k} \text{si} \]

\[ \Delta M_{x} = F_{d} S_{x} = 0.75 (34.2 \text{ k} \text{si})(6.5) = 127.5 \text{ k} \text{in} \]

\[ \Delta M_{y} = F_{d} S_{y} = 0.75 (34.2 \text{ k} \text{si})(12) = 304 \text{ k} \text{in} \]

**VERTICAL**

\[ F_{d} = 1.42 \left[ 38(1 - 0.75)(0.3) / k_{t} + 29(0.75)(0.3) \right] \]
\[ = 51.2 \text{ k} \text{si} \]

\[ \Delta M_{x} = F_{d} S_{x} = 0.75 (51.2)(6.3) = 1752 \text{ k} \text{in} \]

\[ \Delta M_{y} = F_{d} S_{y} = 0.75 (51.2)(4.38) = 168.2 \text{ k} \text{in} \]

SHEAR CAPACITY (AOMS)

**HORIZONTAL**

\[ \Delta V_{u} = 0.9 (15 \text{ k} \text{si})(12)(1/2) = 81 \text{ k} \text{ips} \]

**VERTICAL**

\[ \Delta V_{u} = 0.9 [15(0.75)(0.3) + 29(1 - 0.75)(0.3)] = 74.25 \text{ k} \text{ips} \]
SUMMARY OF PLATE CASE (A)

HORIZONTAL DIRECTION

TENSILE YIELDING  \( \sigma_{Pn} = 59.4K \)
TENSILE RUPTURE  \( \sigma_{Pn} = 87.5K \)
COMPRRESSIVE STRENGTH  \( \sigma_{Pn} = 65.9K \)
FLEXURAL TENSION YIELDING  \( \phi_{Mn} = 6.44Kin \)
FLEXURAL TENSILE RUPTURE  \( \phi_{Mn} = 12.75Kin \)
SHEAR CAPACITY  \( \phi_{Vn} = 81Kip \)

VERTICAL DIRECTION

TENSILE YIELDING  \( \sigma_{Pn} = 103.4K \)
TENSILE RUPTURE  \( \sigma_{Pn} = 79.1K \)
COMPRRESSIVE STRENGTH  \( \sigma_{Pn} = 86.5K \)
FLEXURAL TENSION YIELDING  \( \phi_{Mn} = 11.2Kin \)
FLEXURAL TENSILE RUPTURE  \( \phi_{Mn} = 11.52Kin \)
SHEAR CAPACITY  \( \phi_{Vn} = 74.3Kip \)

**These capacities can be used when analyzing each support connection. Except node 64, node 64 has another plate design.**
PLATE DESIGN (CASE (C))

TRYING 1/2" THICKNESS

SHEAR CAPACITY

\[ \sigma_{\text{shear}} = 0.75(24,000 \text{ psi})(23.025")(1/2") = 212.625 \text{kips} \]

FLEXURAL CAPACITY BY TENSION

\[ F_0 = 1.30 [F_{uyw}] = 1.3(111 \text{ ksi}) = 144.3 \text{ ksi} \]

\[ \sigma_{\text{min}} = F_0 s_y = 0.9(144.3 \text{ ksi})(0.9810^3) = 126.7 \text{kips} \]

Model was designed with pinned supports so a moment shall be resisted by base plate. Furthermore, 212.625 k for shear resists max lateral force of 2741 kips base plate is okay to use for 1/2" thickness.
Weld Connection
**WELD PROPERTIES**

**USING FILLER S356**

- \( F_{sw} = 17 \text{ksi} \)
- \( F_{uw} = 25 \text{ksi} \)

**BASE METAL 6061 T6**

- \( F_{sw} = 15 \text{ksi} \)
- \( F_{uw} = 24 \text{ksi} \)

---

**DETERMINING EFFECTIVE THREAT LENGTH \( L_e \)**

\[ P_{re} = 0.75 \cdot F_{sw} \cdot L_e \quad (A3M J.2.2.1) \]

**WHERE**

\( F_{sw} \) =

(a) \( F_{sw} \cdot (\text{effective threat}) = 17 \left( \frac{5}{2} \right) = 12 L_e \)
(b) \( F_{sw} \cdot (SW) = 15 \cdot (SW) = 15 \text{SW} \)
(c) \( F_{uw} \cdot (SW) = 24 \cdot (SW) = 24 \text{SW} \)

---

**FROM SAP2000 MODEL MAX SUPPORT FORCES ARE AS FOLLOWS**

- **MAX AXIAL** 2.039k FRAME 116 LRFD EQ 4
- **MAX SHEAR** 2.741k FRAME 135 LRFD EQ 4

\[ 2.741k \leq 0.75 \cdot F_{sw} \cdot L_e \leq 0.75 \cdot (12 \text{SW}) \cdot (12\,\text{in})(2.51025) \]
\[ \leq 21.6k \text{SW} \]

\[ 0.013\,\text{in} \leq SW \]

\( \frac{\text{SW}}{\text{in}} = 0.1 \quad \checkmark \)

---

**USING \( \frac{\text{SW}}{\text{in}} = 0.1 \) FOR FILLET WELD**

\[ P_{re} = 0.75 \left( 12 \cdot 0.0125 \right) \left( 12\,\text{in} \right)(2.51025) \]
\[ = 27 \text{ kips} \]

\[ \frac{2.741k}{27 \text{ kips}} = 0.1 \quad \checkmark \]

**USE FILLET WELD \( \frac{\text{SW}}{\text{in}} \) LEGS FOR 12" WELD EFFECTIVE LENGTH CENTERED AT BOTTOM OF PLATE ON EACH SIDE**
Foundation Design
*NOTE: FOUNDATION PROVIDED BELOW IS COSTLY AND NOT A PREFERRED FOUNDATION SYSTEM. OTHER FOUNDATION SYSTEMS THAT WOULD WORK WOULD HAVE BEEN:

1) USE A CONTINUOUS RING FOOTING CONNECTING ALL OF THE SUPPORTS TOGETHER.
2) USE BATTER PILE FOUNDATION SYSTEM.
3) IF AMERICAN EARTH ANCHORS PROVIDE LATERAL RESISTANCE FOR THEIR AUGER ANCHORS, AUGER ANCHORS CONNECTED TO A BASE PLATE MAY PROVIDE AN ADEQUATE FOUNDATION.

**FOUNDATION SYSTEM**

**MATERIALS**

<table>
<thead>
<tr>
<th>Item</th>
<th>Number</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 8&quot; x 8&quot; x 12&quot; CMU BLOCKS</td>
<td>30</td>
</tr>
<tr>
<td>1 1&quot; ALUMINUM BASE PLATE</td>
<td>1</td>
</tr>
<tr>
<td>2 PE46-HEX</td>
<td>2</td>
</tr>
<tr>
<td>2 2&quot; ALUMINUM PIPES</td>
<td>2</td>
</tr>
<tr>
<td>4 AMERICAN EARTH ANCHOR</td>
<td>4</td>
</tr>
<tr>
<td>2&quot; SLEEVES</td>
<td>2</td>
</tr>
<tr>
<td>4 1/2&quot; ALUMINUM ANCHOR BOLTS</td>
<td>4</td>
</tr>
<tr>
<td>1 Non Shrink Grout to Fill CMU Blocks</td>
<td>1</td>
</tr>
</tbody>
</table>

*THIS FOUNDATION SYSTEM WORKS FOR EVERY SUPPORT EXCEPT NODES G4 & III. FOR THOSE YOU JUST NEED TO ADD 1 MORE LAYER OF CMU BLOCKS, SO 36 CMU BLOCKS IN TOTAL.*
ALLOWABLE BEARING PRESSURE CHECK
FROM CBC TABLE 1808.2

VERTICAL FOUNDATION PRESSURE  LATERAL BEARING PRESSURE
1,500 PSF  1000 PSF - 400 PSF

* OUR TEACHER, DENNIS BASHAW (STRUCTURAL ENGINEER), RECOMMENDED TO USE A LATERAL BEARING PRESSURE OF 300 PSF FROM HIS EXPERIENCE. FURTHERMORE, HE RECOMMENDED THAT THE FIRST 12" OF TOP SOIL ARE NOT USABLE.

BEARING CAPACITY

\[ = 150.6 \times \left( \frac{23.625}{2} \right)^2 - 2\pi \left( \frac{1}{2} \right)^2 \times \left( \frac{14\,\text{in}}{12\,\text{in}} \right) \times 1000 \times \frac{\text{psf}}{\text{psf}} \]

\[ = 5.75 \text{ k} \]

FROM SAP2000 MODEL, HIGHEST COMPressive LOAD is AT SUPPORT NODE 64 WITH A LOAD OF 2.79 k. SUPPORT NODE CAPACITIES ON APPENDIX A3

\[ \frac{5.75 \text{ k}}{2.79 \text{ k}} = 2.06 > 2 = \text{SF. FOR ASD} \]

UPLIFT CHECK

PE46-HEX AUGER ANCHORS WILL TAKE THE UPLIFT FORCE

CAPACITY PER ANCHOR = 2000# OF UPLIFT \( \rightarrow \) SEE APPENDIX A4

WITH A SF OF 2, CAPACITY = 1000# \( \frac{1}{2} \) WITH 2 ANCHORS

TOTAL UPLIFT CAPACITY = 2000#

\[ \frac{2000 \#}{1.767 \text{ k}} = \frac{2000 \#}{1.767 \text{ k}} > 1.767 \text{ k} \checkmark \]

1.767 IS HIGHEST COMpressive LOAD AT SUPPORT NODE 138. RESULTS DISPLAYED ON APPENDIX A3
SLIDING CHECK (JUST RELYING ON THE CMU BLOCKS)

\[ P_p = \frac{3(h)^2 (b) (1.33)}{2} \]

\[ P_p = \text{LATERAL BEARING RESISTANCE} \]
\[ P_p = \frac{300 \text{PSF/ft} \times (2.8 \times 1/16) \times (23.6 \times 1/12) \times (1.33)}{2} \]

\[ b \]
\[ \text{WIND} \]

\[ P_p = 2.14 \text{ kips} \]

HIGHEST LATERAL SUPPORT REACTION, BESIDES THE ONES ON NODE 64 & 116 IS 0.733 kips - SEE APPENDIX PAGE A3

\[ \frac{2.14 \text{ kips}}{0.733} = 2.9 \]

ASD & CBC REQUIRE A SF OF 1.5 SO WE ARE }
OVERTURNING CHECK

Worst overturning resultant is 2.214 kft [ignoring all moments on nodes G, H & I of previous page A6]

\[ M_{\text{resisting}} = 2.14k(9.33/12) + 2(11.8/12) \approx 3.63 \text{kft} \]

\[ M_{\text{overturning}} = 0.719(40/12) - 0.185(11.8/12) \approx 2.21 \text{kft} \]

\[ M_{\text{resisting}} \div M_{\text{overturning}} = 1.64 > 1.5 \text{ Safe for overturning} \]

**Foundation is safe for overturning**
Appendices
SHEET: A3 of A7

**NODE LOCATIONS**

**SUPPORT ASD DESIGN FORCES**

**SUMMARY OF MAX RESULTS**

- $F_x = -1.127kN$ (X) 2.776kN (Y)
- $F_y = -0.913kN$ (Y) 0.773kN (X)
- $F_z = 0.199kN$ (Z) 0.189kN (Z)

**VALUES OBTAINED FROM SAP2000 MODEL ASD ANALYSIS**

**COLOR SCHEME**

- RED: WORST STRESS CONSIDERED
- GREEN: LIFFT CONDITION
- BLUE: WORST FORTRAN FORCE

**VALUE'S OBTAINED FROM SAP2000 MODEL ASD ANALYSIS**

- WORST TON FORCES ARE NOT VOWAN INTO ACCOUNT FOR MAX FORCES IN BOTH THE X, Y, AND Z DIRECTIONS. FOR EACH PENDED SUPPORT FOUNDATION.
American Earth Anchors
The best screw you will have in the dirt ™

PE46-Hex | Specifications

46" Penetrator™ with hex head
- Aircraft-quality cast aluminum 356 alloy
- Heat-treated to T6 specification
- Install with 2" or 51 mm socket
- Removable

LOAD CAPACITY
Pullout strength with flight fully embedded

<table>
<thead>
<tr>
<th>Soil Class 1</th>
<th>Soil Class 2</th>
<th>Soil Class 3</th>
<th>Soil Class 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hardpan Asphalt</td>
<td>Sandy gravel</td>
<td>Silty/clayey sand</td>
<td>Loose fine uncompacted sand</td>
</tr>
<tr>
<td>14,000 lb</td>
<td>9,500 lb</td>
<td>3,300 lb</td>
<td>2,000 lb</td>
</tr>
<tr>
<td>62.3 kN</td>
<td>42.3 kN</td>
<td>14.7 kN</td>
<td>8.90 kN</td>
</tr>
</tbody>
</table>

6-point socket (instead of 12-point socket) will minimize wear and rounding of hex head for repeated installation/removal

American Earth Anchors
info@americanea.com
americanearthanchors.com

Contact us for CUSTOM WORK
Size, length, shape, material, prototypes, cable assemblies

866-520-8511
+1 508-520-8511
PE46-Hex | Installation

Through asphalt

Drill PILOT HOLE through asphalt

Diameter: 2” (5 cm)

Asphalt

Soil

Installation methods

Watch the video:
Click here or visit aeavideo.com

Power take-off
Watch the video: Click here or visit aeavideo.com
(Video shows PE46-Guy)

Impact wrench

Attachment accessories

U-BRACKETS

For 4”x 4” post
(10 cm x 10 cm)

For 2” pipe
(5 cm)

SLEEVES

TIE-OFF CABLE
PE-TC46

Can be doubled over to make large loop around structural member

Contact us for CUSTOM WORK
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info@americanea.com
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We will make custom brackets or sleeves to your specification

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CR-PE46-Hex Rev 2018

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SUPPORT REACTIONS WITH OVERTURNING CALCULATIONS

THE FOLLOWING EXCEL SHEETS PROVIDES ALL SUPPORT REACTIONS EXCEPT NODES 116 AND 64. RESULTS ARE EXPORTED DIRECTLY FROM SAP2000.

THE OVERTURNING CALCULATIONS ARE BASED OFF THE FOUNDATION CALCULATION FOUND ON FN4.

<table>
<thead>
<tr>
<th>Joint</th>
<th>OutputCase</th>
<th>F1</th>
<th>F2</th>
<th>F3</th>
<th>OT w/Fl/F3</th>
<th>OT w/oFl/F3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A6</td>
<td>0.073</td>
<td>0.031</td>
<td>0.026</td>
<td>0.026</td>
<td>0.026</td>
<td>0.026</td>
</tr>
<tr>
<td>A7</td>
<td>0.073</td>
<td>0.031</td>
<td>0.026</td>
<td>0.026</td>
<td>0.026</td>
<td>0.026</td>
</tr>
</tbody>
</table>

Client: WINE HISTORY PROJECT
Project: PRIMITTO - APPENDIX
SUPPORT REACTIONS WITH OVERTURNING CALCULATIONS

Engineer: EAU