Wine History Pavilion: FLOW

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

By

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December, 2019

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General Information

Introduction

The Wine History Pavilion has been designed using Integrated Project Delivery, a delivery approach that has been gaining popularity that involves the participation of multiple disciplines at every stage of design. For this group, that entails the participation of students from the departments of Architecture, Architectural Engineering, and Construction Management working in tandem to handle every facet of the design.

Background

The goal of this project is to design a pavilion for the Wine History Project, who needed a display space to house a variety of exhibits showcasing artifacts relevant to the history of wine. The pavilion will originally be installed at the Saucelito Canyon Tasting Room, but will be relocated to various other sites in the area.

Objective

The Wine History Pavilion is intended for the display of wine-related historical artifacts by the Wine History Project. It will need to accomodate various exhibits of differing size and organization. The pavilion will need to provide protection from the elements to both the visitors and exhibits housed within. A major consideration for the pavilion will be the need to easily disassemble, transport, and reassemble it for relocation to other sites. For this reason, the design emphasizes ease of assembly, lightweight construction, and the minimizing of long-term impacts to the site.

Credits

This project would not have been possible without the involvement of students from multiple disciplines. In addition, these students were all responsible for the information found in the Relevant Project Information.

Architecture:
Isha Sharma
Khanh Nguyen

Construction Management:
Anthony Cumpian
Antonio Rosales

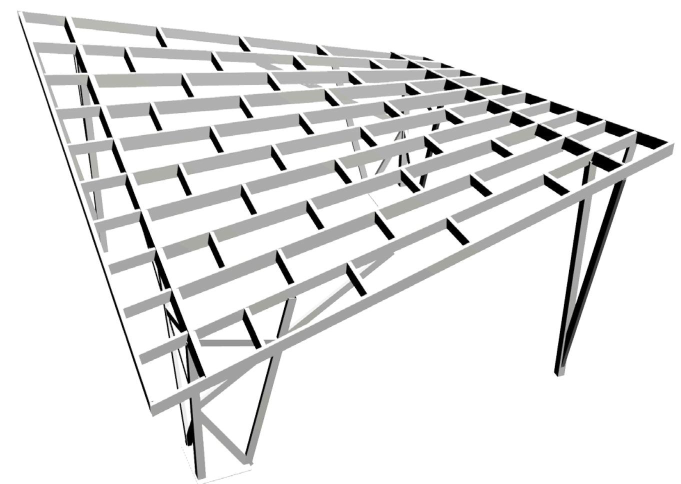
FLOW

RENDERINGS

DESIGN CONCEPT

TRANSLUCENT PANELS FOR AMBIENT LIGHTING

PANELS FOR ROOFING OVER STRUCTURE TO ALLOWS AND DIFFUSES DAYLIGHT FOR EVEN GLOW WITHIN SPACE.



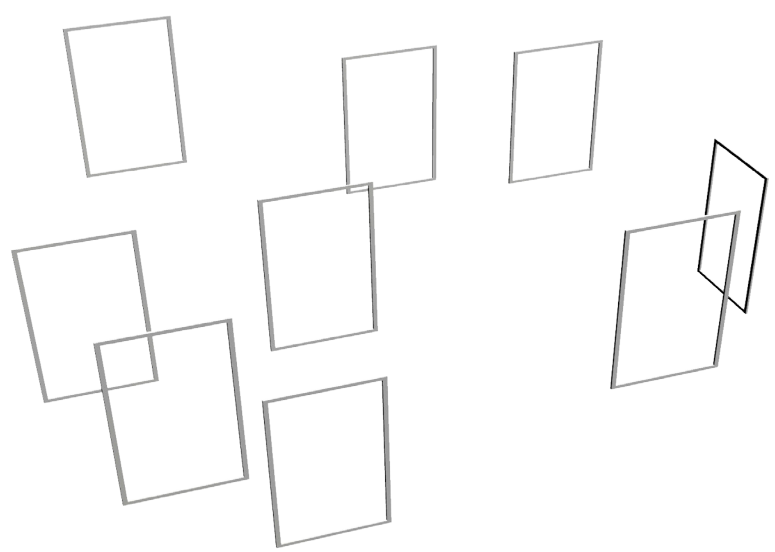
SEGMENTED AND COLLAPSIBLE

LIGHTWEIGHT ALUMINUM STRUCTURE FOR EASY ASSEMBLY AND TRANSPORTATION

SEGMENTED STRUCTURE THAT IS IN PARTS AND COLLAPSES EASILY

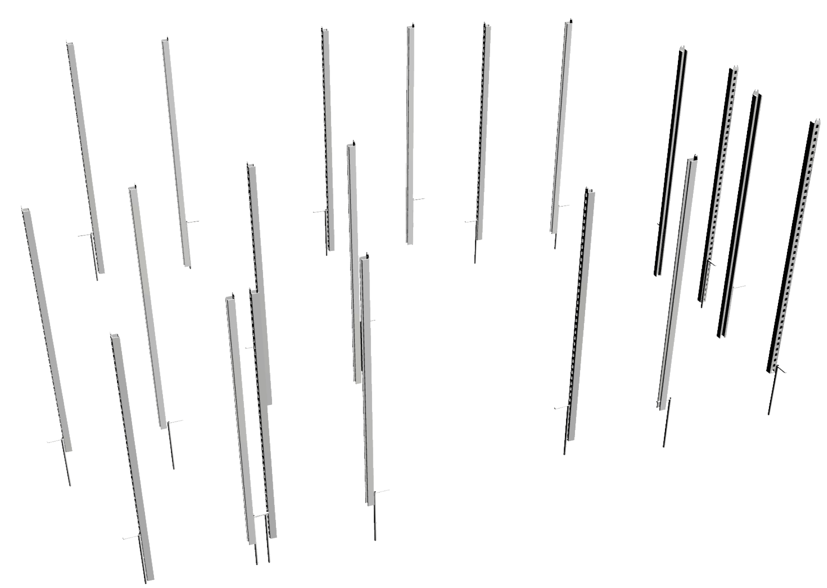
SEGMENTED AND COLLAPSIBLE

SWISSTRAX FLOOR THAT IS PRE-FABRICATED AND EASILY ASSEMBLED TO ALLOW FOR EVEN WALKING GROUND.



VERSATILE DISPLAY AND VARIETY IN CIRCULATION

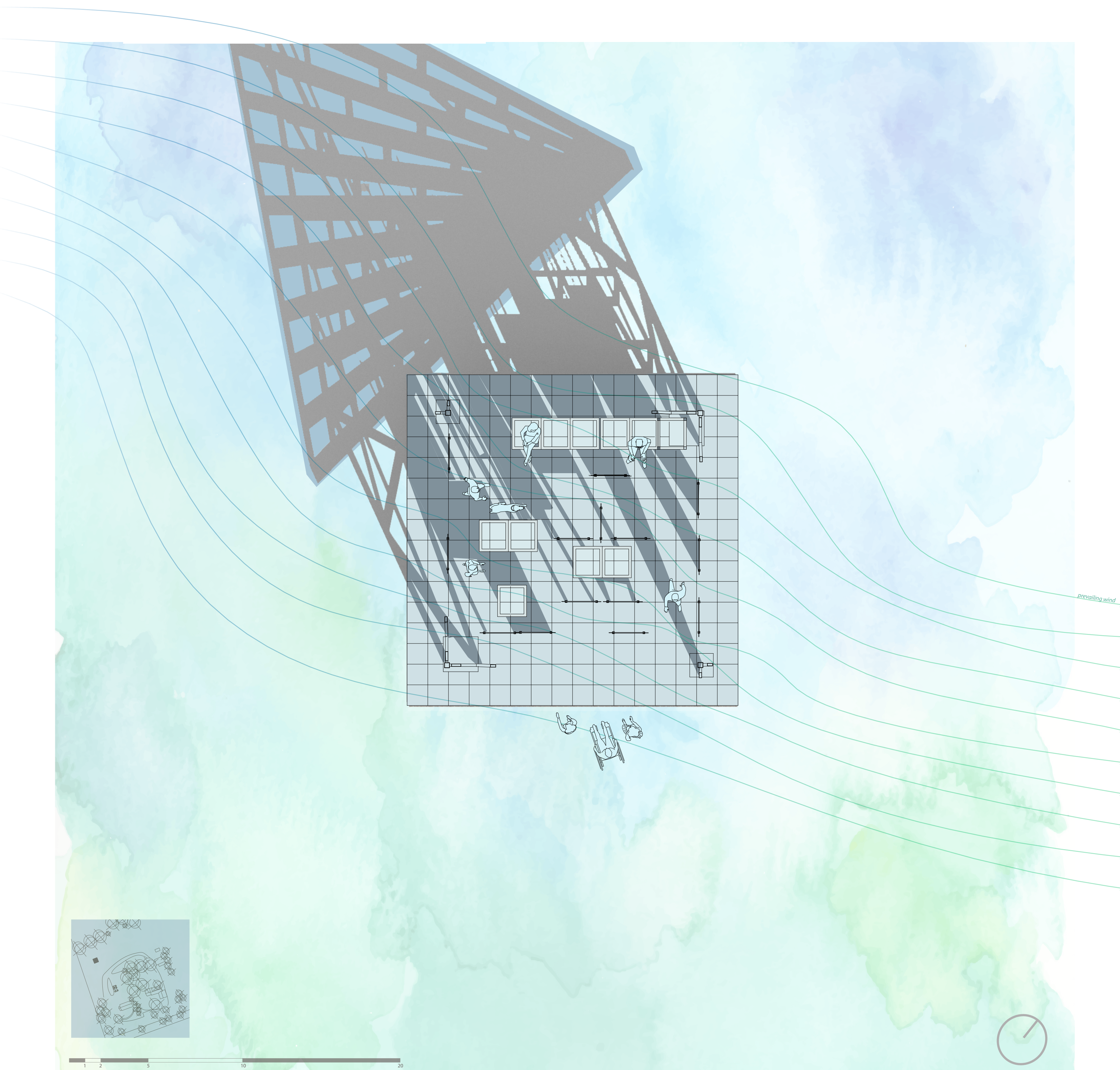
PANELS FOR DISPLAY AND TO HANG OFF CEILING FOR CIRCULATION DIVISION WITHIN AN OPEN SPACE, REMOVABLE FOR A LARGE GATHERING

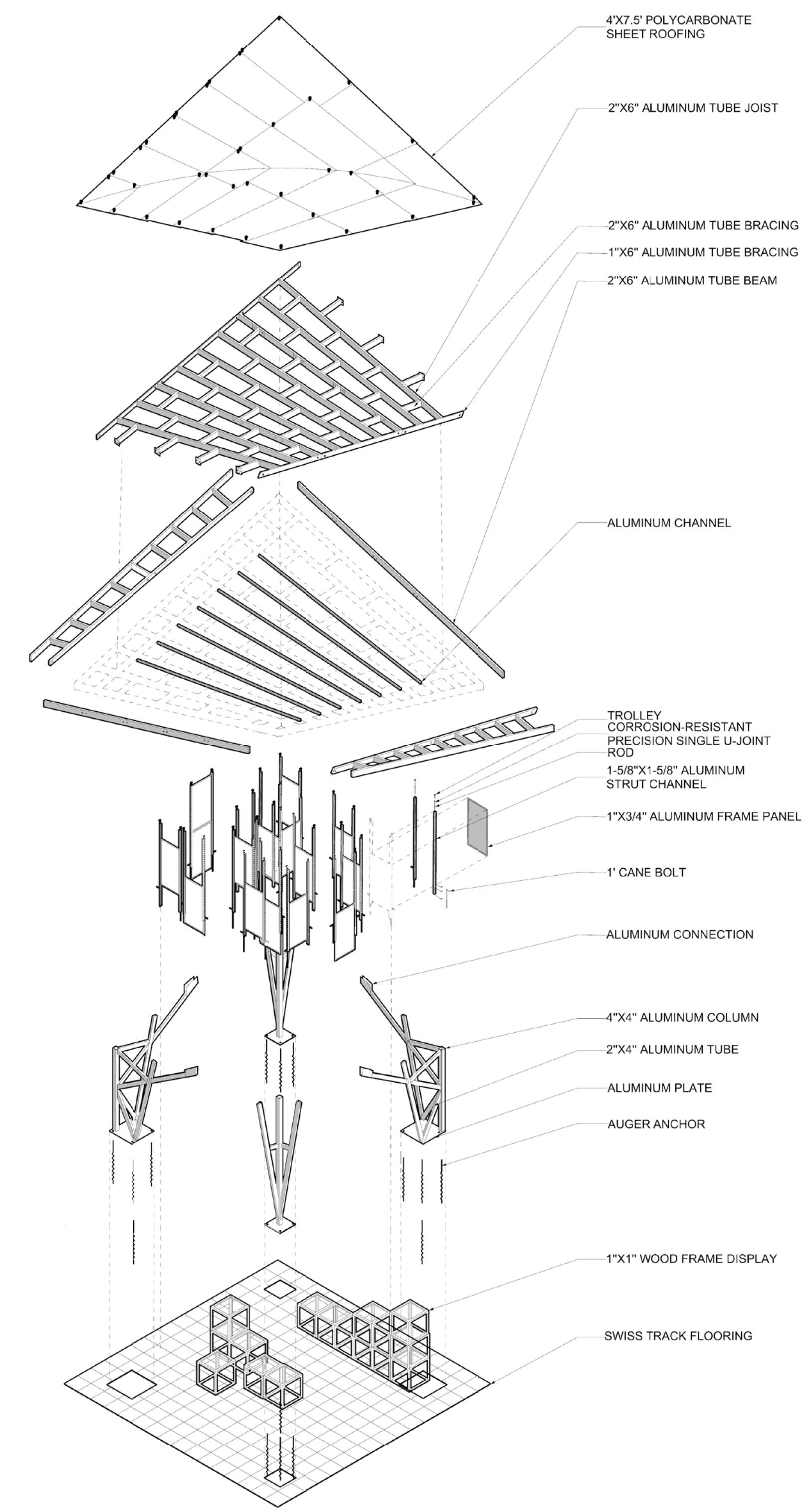


VERSATILE DISPLAY AND VARIETY IN CIRCULATION

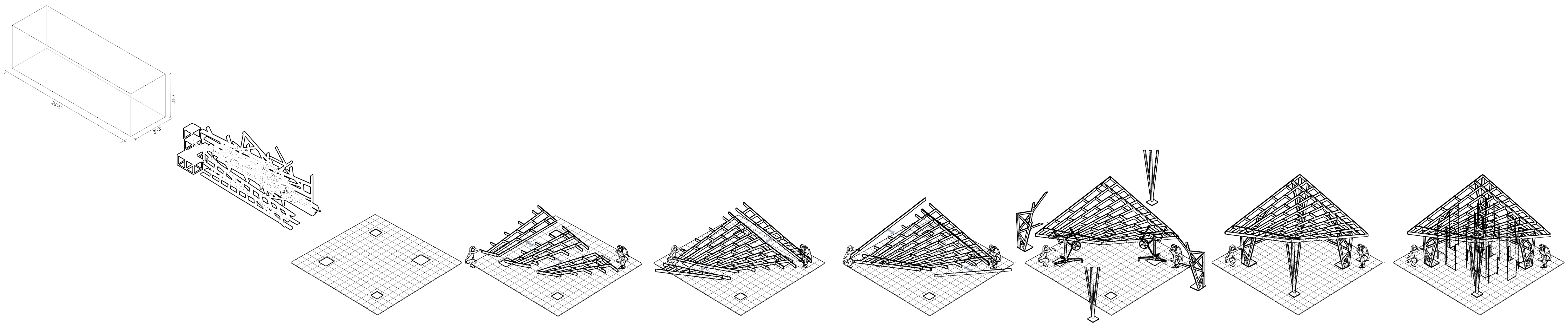
STRUTS TO ALLOW VERSATALITY IN PANEL ARRANGEMENT AND TO CHANGE CIRCULATION AND DISPLAY SPACE AS REQUIRED.

SITE PLAN AND EXPLODED AXONOMETRIC





ASSEMBLY DIAGRAM AND SECTION DRAWINGS



Wine History Pavilion

Structural Calculations per IBC 2018

Client: Wine History Project Designer: Isaac Cameron

Structural Calculations

for

Wine History Pavilion

November 22, 2019

Project Enginner: Isaac Cameron
Client: Wine History Project

Project Data .... P1 - P2
Key Plan K1
Gravity Load Take-off D1
Wind Loading .... W1 - W2
Member Design .... M1 - M16
Foundation Design .... F1 - F9
Connection Design .... C1 - C17
Appendix .... A1 - A16

Project Description

This project strives to create a lightweight, easily assembled pavilion space that can be disassembled and transported to a number of sites. This particular design strove to include a series of adjustable panels to accomodate a variety of circulation plans. These panels are designed as nonstructural elements that are supported from above by a relatively simple structural framework.

The pavilion will initially be installed in Saucelito Canyon, but will be relocated to various sites as needed by the Wine History Project.

Design Criteria

The main structure of the pavilion is a 20' x 20' aluminum framed roof, sheathed with polycarbonate roofing panels. The slope of the roof varies with each beam to create a hyperbolic paraboloid with a height varying from 6' to 12'. The roof is supported by square HSS aluminum columns as well as a series of K-shaped brace frames. At the ground level, these members are attached to an aluminum plate that distributes the loads to the ground through arrowhead anchors anchors.

Codes Used:

Wind/Live Loading Criteria	- ASCE 7-16
Design Criteria, Prescribed Soil Properties	- 2018 IBC
Aluminum Design	- Aluminum Design Manual 2010

Material Criteria

Aluminum Alloy 6061-T6

Ftu = 42 ksi	Ftuw = 24 ksi
Fty = 35 ksi	Ftyw = 15 ksi
Fcy = 35 ksi	Fcyw = 15 ksi
Fsu = 27 ksi	Fsuw = 15 ksi
E = 10,100 ksi	

5356 Filler Material

Ftu = 35 ksi
Fsu = 17 ksi

Prescibed Soil Loads
(Silty Sand or Clay assumed)
pmax = 1,000 psf
plat = 100 psf/ft
CoF = 0.25
Cohesion = 130 psf

10” Arrowhead Anchors
Pullout Load = 4000 lb

Modeling Criteria

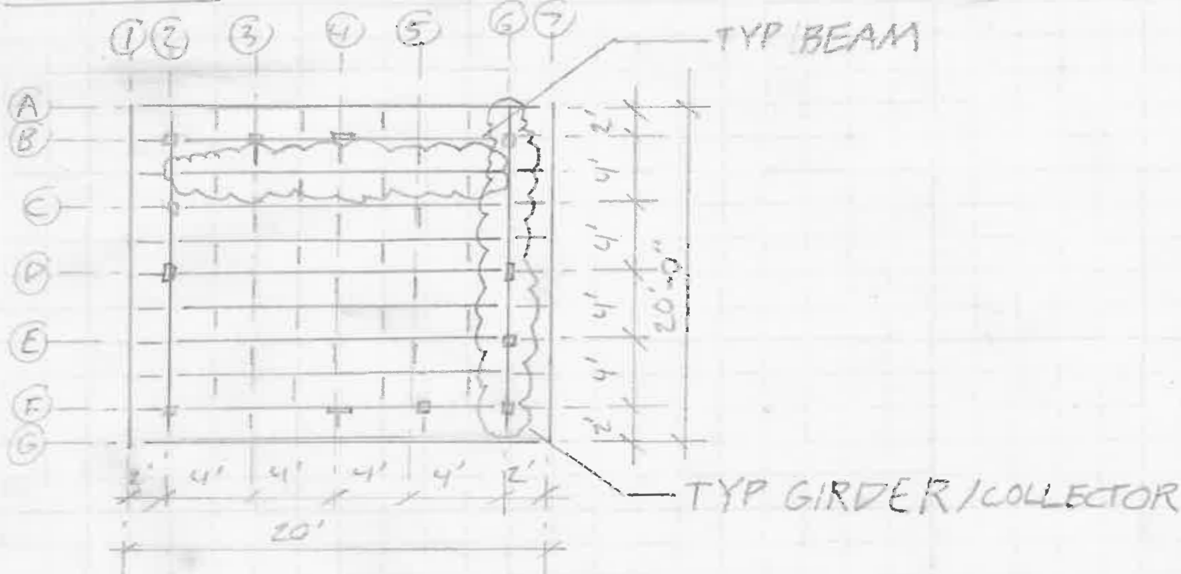
The structure was analyzed for design loading using SAP2000. All connections in the roof and lateral system were idealized as having pinned supports. LRFD load combos 1, 3, 4, and 5 were applied to the structure using applied point loads on the roof. Where the load combos contained wind loads, two separate wind load cases were created along with split load combos for uplift and downdraft on the roof respectively. Another set of load combos was also created using ASD formulas in order to obtain design loads to the foundation system.

PROJECT: WINE HISTORY PAVILION

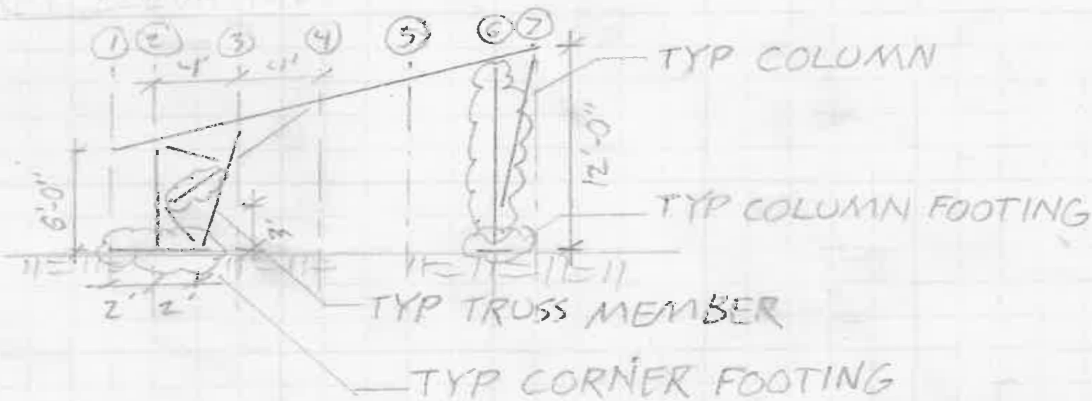
PREPARED BY: ISAAC CAMERON **PAGE:** K1

SUBJECT: KEY PLANS **DATE:** 11/22/19

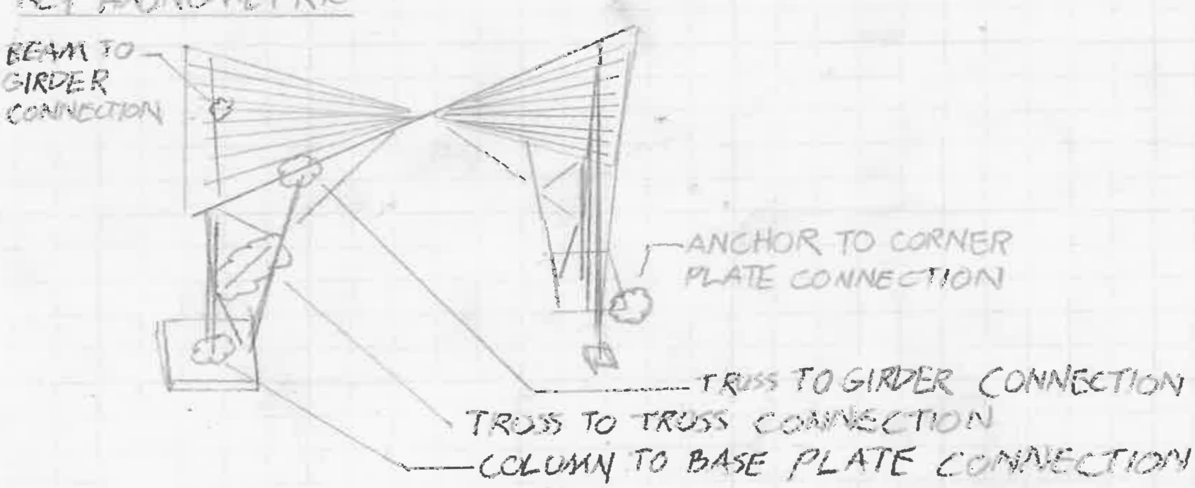
KEY PLAN



KEY SECTION (THROUGH C)



KEY AXONOMETRIC



LOADING

TYPICAL DEAD LOADS		
Description		**W (psf)**
Roof Sheathing	1/8" Polycarbonate Panels	0.78
Blocking	Aluminum HSS4x2x1/8 @ 4' o.c.	0.42
Beams	Aluminum HSS4x2x1/8 @ 2' o.c.	0.85
Panel Tracks	1-5/8"x1-5/8" 12ga strut channel @2' o.c.	0.33
Misc.	5% Miscellaneous	0.13
	Total to Beams (static):	**2.50**
Panel Frames	(2) 8'x1-5/8"x1-5/8" 12ga strut channel @2' o.c.	1.31
Misc.	91% Miscellaneous	1.19
	Total to Beams (adj.):	**2.50**
Girders	Aluminum HSS6x2x1/8 @ 16' o.c.	0.23
Misc.	9% Miscellaneous	0.02
	Total to Girders:	**5.25**
Columns	Aluminum HSS 4x4x1/8 @ 16' o.c.	0.20
Misc.	22% Miscellaneous	0.05
	Total to Columns:	**5.50**

TYPICAL LIVE LOADS		
Description		**W (psf)**
Live Load	Unoccupied attic without storage	10.00

ASCE 7-16 WIND LOAD CALCULATIONS

Following ch. 27 directional procedure (Table 27.2-1)

Step 1

Per T1.5-1
Risk category II

Step 2

Per fig 26.5-1B
$V = 95$ mph

Step 3

Surface roughness = C per 26.7.2
Exposure category = C per 26.7.3

$K_d = 0.85$ per T26.6-1

$K_{zt} = 1.0$ per 26.8.2

$K_e = 1.0$ per 26.9

$G = 0.85$ per 26.11.1

Structure is partially enclosed per 26.12

For partially-enclosed, per T26.13-1
$GC_{pi} = \pm 0.55$

Step 4

For Exposure C and $h < 15'$ per T26.10-1
$K_z = 0.85$

Step 5

$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 [psf/mph^2]$ (Eq. 26.10-1)
$= 0.00256(0.85)(1.0)(0.85)(1.0)(95)^2$ psf

$q_z = 16.69$ psf

Step 6

Per Fig. 27.3-1

$C_p = 0.8$ (windward)
$C_p = -0.3$ (windward)
$h/L = 12'/20'$
$= 0.6$
$C_p = -0.98$ or -0.18 (uplift)

Step 7

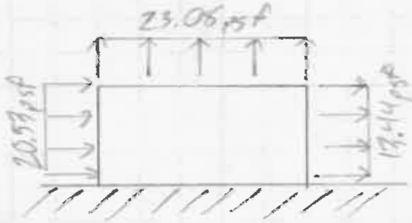
$p = qGC_p - q_i(GC_{pi})$ (Eq. 27.3-1)
$= 16.69\,psf(0.85)C_p - 16.69\,psf(\pm 0.55)$
$= 16.69\,psf(0.85C_p \pm 0.55)$

$p_{ww} = 16.69\,psf[0.85(0.8)+0.55]$
$= 20.53\,psf$

$p_{lw} = 16.69\,psf[0.85(-0.3)-0.55]$
$= -13.44\,psf$

$p_{ul} = 16.69\,psf[0.85(-0.98)-0.55]$
$= -23.08\,psf$
or
$p_{ul} = 16.69\,psf[0.85(-0.18)+0.55]$
$= 6.63\,psf$

FINAL DESIGN VALUES

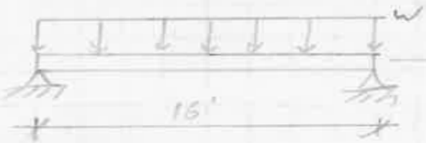


OR


STRUCTURAL MEMBERS

PROJECT: WINE HISTORY PAVILION			
PREPARED BY: ISAAC CAMERON		**PAGE:**	M1
SUBJECT: TYPICAL BEAM		**DATE:**	11/22/19

BEAM DESIGN - GRAVITY



Al. HSS4×2×1/8

trib width = 2'
$L_b = 4'$

$b = 2''$
$d = 4''$
$t = 0.125''$
$A = 1.44\ in^2$
$J = 2.3\ in^4$

$I_x = 2.96\ in^4$
$S_x = 1.49\ in^3$
$r_x = 1.44''$
$I_y = 0.992\ in^4$
$S_y = 0.992\ in^3$
$r_y = 0.831''$

DEMAND

From load take-off

$DL = 5.0\ psf$
$LL = 10.0\ psf$

From wind calcs

$WL = -23.1\ psf$ or $6.6\ psf$

Checking ASCE 7 load combos

Governing combos are 3 + 5

3. $1.2D + 1.6L_r + 0.5W$
$= 1.2(5\ psf) + 1.6(10\ psf) + 0.5(6.6\ psf)$
$= 25.3\ psf$

5. $0.9D + 1.0W$
$= 0.9(5\ psf) + 1.0(-23.1\ psf)$
$= -18.6\ psf$

Using 25.3 psf

Finding demand moment / shear

$w = 25.3\ psf(2')$
$= 50.6\ plf$

$M_u = wL^2/8$
$= 50.6\ plf(16')^2/8\ (1k/1000\#)$

$M_u = 1.62\ k\text{-}ft$

$V_u = wL/2$
$= 50.6\ plf(16')/2\ (1k/1000\#)$

$V_u = 0.405\ k$

CAPACITY

Using 2010 Aluminum Design Manual

Per TA.3.5 for 6061-T6, standard and weld-affected

$F_{tu} = 42$ ksi $F_{tuw} = 24$ ksi
$F_{ty} = 35$ ksi $F_{tyw} = 15$ ksi
$F_{cy} = 35$ ksi $F_{cyw} = 15$ ksi
$F_{su} = 27$ ksi $F_{suw} = 15$ ksi
$E = 10{,}100$ ksi

USING WELD-AFFECTED PROPERTIES FOR THE ENTIRE CROSS-SECTION IS REQUIRED PER F.9.1 DUE TO BLOCKING USING TRANSVERSE WELDS.

Finding buckling constants (TB.4.2, TB.4.1)

$k = 1.0$ ksi

$B_c = F_{cy}\left[1 + \left(F_{cy}/2250k\right)^{1/2}\right]$
$= 35\text{ksi}\left[1 + \sqrt{35/2250(1)}\right]$
$B_c = 39.37$

$D_c = \frac{B_c}{10}\left(\frac{B_c}{E}\right)^{1/2}$
$= \frac{39.37}{10}\left(\frac{39.37}{10100}\right)^{1/2}$
$D_c = 0.25$

$C_c = 0.41\, B_c/D_c$
$= 0.41(39.37/0.25)$
$C_c = 65.67$

weld-effected:

$B_c = F_{cy}\left[1 + \left(F_{cy}/1000k\right)^{1/2}\right]$
$= 15\text{ksi}\left[1 + (15/1000)^{1/2}\right]$
$B_c = 16.8$

$D_c = \frac{B_c}{10}\left(\frac{B_c}{E}\right)^{1/2}$
$= \frac{16.8}{10}\left(\frac{16.8}{10100}\right)^{1/2}$
$D_c = 0.069$

$C_c = \frac{2B_c}{3D_c}$
$= 2/3\,(16.8/0.069)$
$= 163.3$

Designing for Flexure

Per F.1

$\phi_{rupture} = 0.75$
$\phi = 0.9$

Per F.1.1a

$C_b = 1$

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** M3

SUBJECT: TYPICAL BEAM **DATE:** 11/22/19

Per F.3.1

$S_2 = (C_c/1.6)^2$
$= (163.3/1.6)^2$
$= 10414$

$\frac{2L_bS_c}{C_b\sqrt{I_yJ}} = \frac{2(4')(12''/1')(1.49 in^3)}{1.0\sqrt{0.992 in^4 (2.3 in^4)}}$
$= 98.9$

98.9<10414 ∴ use inelastic buckling

$F_b = B_c - 1.6D_c\sqrt{\frac{2L_bS_c}{C_b\sqrt{I_yJ}}}$
$= 16.8 - 1.6(0.069)\sqrt{98.9}$

$F_b = 15.7$ ksi $> F_{cy}$
∴ use 15 ksi

$M_n = F_b S_c$
$= 15 ksi (1.49 in^3)(1'/12'')$
$M_n = 1.86$ kft

Checking flexure

$M_{nr} = F_{tuw} S_c$
$= 24 ksi (1.49 in^3)(1'/12'')$
$M_{nr} = 2.98$ ksi

$\varnothing M_n = 0.9(1.86 kft)$
$= 1.68$ kft

or

$\varnothing M_n = 0.75(2.98 ksi)$
$= 2.24$ kft

1.68 kft > 1.62 kft
∴ design passes

Designing for shear

$\varnothing = 0.9$ per G.1

$V_n = F_s A_w$
$= 15 ksi (4'')(0.125'') = 7.5 k$

$\varnothing V_n = 0.9(7.5 k)$
$= 6.7k > 0.4k$ ∴ passes

PROJECT: WINE HISTORY PAVILION	
PREPARED BY: ISAAC CAMERON	**PAGE:** M4
SUBJECT: TYPICAL BEAM	**DATE:** 11/22/19

Checking deflection

Per 2012 IBC T1604.3

$\Delta_{max} = L/180 = 16'/180 (12''/1') = 1.07''$ for L

or

$\Delta_{max} = L/120 = 16'/120 (12''/1') = 1.6''$ for D+L

$w_L = 10\,psf (2') = 20\,plf$

$w_{D+L} = (5\,psf + 10\,psf)(2') = 30\,plf$

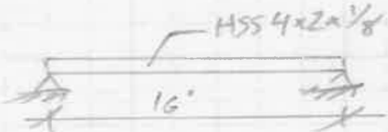
$\Delta_L = \frac{5wL^4}{384EI} = \frac{5(20\,plf)(16')^4}{384(10100\,ksi)(2.98\,in^4)} (12''/1')^3 (1k/1000\#)$

$\Delta_L = 0.98'' < 1.07''$ ✓

$\Delta_{D+L} = \frac{5wL^4}{384EI} = \frac{5(30\,plf)(16')^4}{384(10100\,ksi)(2.98\,in^4)} (12''/1')^3 (1k/1000\#)$

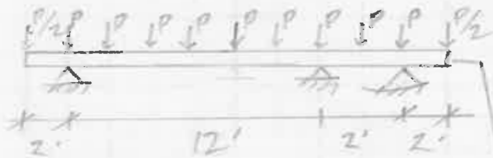
$\Delta_{D+L} = 1.47'' < 1.6''$ ✓

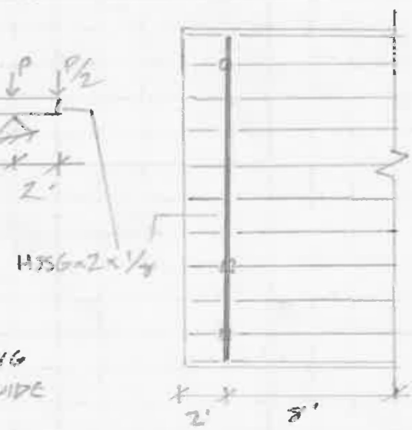
FINAL DESIGN



$\phi M_n = 1.68\,k\text{-}ft$ | $M_u = 1.62\,k\text{-}ft$
$\phi V_n = 6.7\,k$ | $V_u = 0.4\,k$
$\Delta_{ALL} = 1.07''$ | $\Delta_L = 0.98''$
$\Delta_{ALL} = 1.6''$ | $\Delta_{D+L} = 1.47''$

GIRDER DESIGN - GRAVITY





$b = 2"$
$d = 6"$
$t = 0.125"$
$A = 1.94"$

$J = 3.91 in^4$

$I_x = 8.28 in^4$
$S_x = 2.76 in^3$
$r_x = 2.07 in$
$I_y = 1.43 in^4$
$S_y = 1.43 in^3$
$r_y = 0.86"$

NOTE: ALTHOUGH THERE IS TECHNICALLY A SUPPORT AT MIDSPAN, WE WILL IGNORE IT BECAUSE THE CONTRIBUTING BRACE IS TOO ANGLED TO PROVIDE MEANINGFUL STIFFNESS.

DEMAND

From load take-off

$DL = 5.25 psf$
$LL = 10.0 psf$

From wind calculations

$WL = -23.1 psf$ or $6.6 psf$

Checking ASCE 7 load combos

Governing combos are 3 + 5

3. $1.2D + 1.6L_r + 0.5W$
$= 1.2(5.25psf) + 1.6(10psf) + 0.5(6.6psf)$
$= 25.6 psf$

5. $0.9D + 1.0W$
$= 0.9(5.25psf) + 1.0(-23.1psf)$
$= -18.4 psf$

Finding demand moment/shear

$w = 25.6 psf(2' + 8')$
$= 256 plf$

The moment will conservatively be taken using the largest span ignoring support from continuity to the rest of the girder.

$M_u = wL^2/8$
$= 256 plf(12')^2/8 \; (1k/1000\#)$
$M_u = 4.61 kft$

$V_u = wL/2$
$= 256 plf(12')/2$
$= 1.54k$

CAPACITY

Using 2010 Aluminum Design Manual

Per TA3.4 for 6061-T6

$F_{tu} = 42$ ksi $F_{su} = 27$ ksi
$F_{ty} = 35$ ksi
$F_{cy} = 35$ ksi $E = 10{,}100$ ksi

Finding buckling constants

These are based on material, so values are taken from beam calculations

$B_c = 39.37$
$D_c = 0.25$
$C_c = 65.67$

Finding flexural capacity

$\phi_{rupture} = 0.75$ per F.1
$\phi = 0.9$

$C_b = 1.0$ per F.1.1a

Per F.3.1

$$S_2 = (C_c/1.6)^2$$
$$= (65.67/1.6)^2$$
$$= 1685$$

$$\frac{2 L_b S_c}{C_b \sqrt{I_y J}} = \frac{2(2')(12''/1')(2.76 in^3)}{1.0\sqrt{1.43 in^4 (3.91 in^4)}}$$
$$= 56.0 < 1685 \therefore \text{use inelastic buckling}$$

$$F_b = B_c - 1.6 D_c \sqrt{\frac{2 L_b S_c}{C_b \sqrt{I_y J}}}$$
$$= 39.4 - 1.6(0.25)\sqrt{56.0}$$
$$= 36.4 \text{ ksi}$$

36.4 ksi ∴ governed by yielding, not buckling

$F_b = 35$ ksi

Checking flexure

$M_n = F_b S_c$
$= 35 ksi (2.76 in^3)(1'/12")$
$M_n = 8.05$ kft

$\phi M_n = 0.9(8.05$ kft$)$
$= 7.25$ kft

$M_{nr} = F_{tr} S$
$= 42 ksi (2.76 in^3)(1'/12")$
$M_{nr} = 9.66$ kft

$\phi_r M_{nr} = 0.75(9.66$ kft$)$
$= 7.25$ kft

7.25 kft > 4.61 kft ✓

Checking shear

$\phi = 0.9$ per G.1

$V_n = F_v A_w$
$= 27 ksi (6")(0.125")$
$= 20.3$ k

$\phi V_n = 0.9(20.3$ k$)$
$= 18.2$ k

18.2k > 1.54k ✓

Checking deflection

Per IBC 2012 T1604.3

$\Delta_{max,L} = L/180$
$= 12'/180 (12"/1')$
$= 0.8"$

$\Delta_{max,D+L} = L/120$
$= 12'/120 (12"/1')$
$= 1.2"$

$w_L = 10 psf (10')$
$= 100 plf$

$w_{D+L} = (10 psf + 5.25 psf)(10')$
$= 153 plf$

$\Delta_L = \frac{5wL^4}{384EI}$
$= \frac{5(0.1 klf)(12')^4(12"/1')^3}{384(10100 ksi)(8.28 in^4)}$

$\Delta_L = 0.56"$

$\Delta_{D+L} = \frac{5wL^4}{384EI}$
$= \frac{5(0.153 klf)(12')^4(12"/1')^3}{384(10100 ksi)(8.28 in^4)}$

$\Delta_{D+L} = 0.85"$

0.56" < 0.8" ✓

0.85" < 1.2" ✓

FINAL DESIGN

$\phi M_n = 7.25$ kft

$\phi V_n = 18.2$ k

$\Delta_{ALLL} = 0.8"$

$\Delta_{ALLDL} = 1.2"$

$M_u = 4.61$ kft

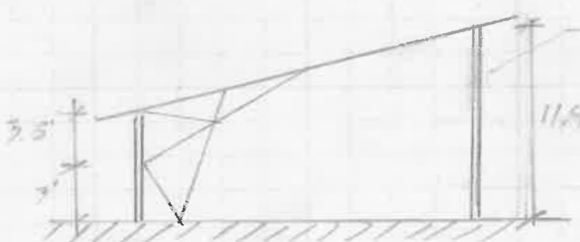
$V_u = 1.54$ k

$\Delta_L = 0.56"$

$\Delta_{D+L} = 0.85"$

COLUMN DESIGN

DUE TO THE COMPLEXITY OF THE LATERAL FORCE RESISTING SYSTEM, IT IS NOT FEASIBLE TO FIND DEMAND FORCES ON THE COLUMN THAT WE CAN BE CONFIDENT ARE GOVERNING THE DESIGN USING TRADITIONAL HAND CALCULATION METHODS. THIS MEANS THAT THE COLUMN DESIGN WILL BE DONE USING TABULATED FORCES FROM A SAP ANALYSIS. IN ADDITION, THE COLUMNS ARE FURTHER COMPLICATED BY THE PRESENCE OF COMBINED AXIAL FORCES AND BENDING MOMENTS. BECAUSE THE WORST-CASE LOADING ISN'T NECESSARILY THE ONE WITH THE WORST AXIAL LOAD, THE DESIGN CAPACITIES WERE DETERMINED FIRST BEFORE THE WORST-CASE LOADS WERE SELECTED USING THE WORST DEMAND/CAPACITY RATIO FROM EQUATION H.1-1.



$b = 4''$ $\quad I = 4.85\ in^4$
$d = 4''$ $\quad S = 2.43\ in^3$
$t = 0.125''$ $\quad r = 1.58''$
$A = 1.94\ in^2$ $\quad J = 7.27\ in^4$

$F_{ty} = 35\ ksi$ $\quad F_{tu} = 42\ ksi$ ← 6061-T6
$F_{cy} = 35\ ksi$ $\quad E = 10{,}100\ ksi$ per TA.3.4

CAPACITY

Short Column flexural capacity ($L_b = 3.6'$)

$B_c = 39.37$ SEE BEAM CALCS
$D_c = 0.25$ FOR EQUATIONS
$C_c = 65.67$

$\phi = 0.9$ per F.1
$C_b = 1.0$ per F.1.1a

$S_2 = 1685$ per BEAM CALCS

$$\frac{2L_bS_c}{C_b\sqrt{I_yJ}} = \frac{2(3.6')(2.43in^3)(12''/')}{1.0\sqrt{4.85in^4(7.27in^4)}} = 35.4 < 1685 \therefore \text{inelastic}$$

$$F_b = B_c - 1.6D_c\sqrt{2L_bS_c/C_b\sqrt{I_yJ}} = 39.4 - 1.6(0.25)\sqrt{35.4} = 37.0\ ksi$$

Use $F_b = F_{cy} = 35 ksi$

$$M_n = F_bS_c = 35ksi(2.43in^3) = 85.05\ k\,in$$

$$\phi M_n = 0.9(90 k\,in) = 76.55\ k\,in$$

Short column axial capacity ($L_b = 3.6'$)

$S_2 = C_c$
$= 65.67$

$k = 1.0$ per E.2

$kL/r = 1.0(3.6')(12''/1')/1.58''$
$= 27.34$

$27.34 < 65.67$ ∴ inelastic

$F_c = 0.85(B_c - D_c kL/r)$
$= 0.85(39.57 - 0.25(27.34))$
$= 27.75$ ksi

$P_{nc} = F_c A$
$= 27.75\,ksi\,(1.94\,in^2)$
$= 53.8\,k$

$P_{nt} = F_{yt} A$
$= 35\,ksi\,(1.94\,in^2)$
$= 67.9\,k$

$\phi P_{nc} = 0.9(53.8\,k)$
$= 48.4\,k$

$\phi P_{nt} = 0.9(67.9\,k)$
$= 61.1\,k$

Tall column flexural capacity ($L_b = 11.4'$)

Material-based constants are same as short column

$\frac{2 L_b S_c}{C_b \sqrt{I_y J}} = \frac{2(11.4')(12''/1')(2.43\,in^3)}{1.0\sqrt{4.75\,in^4 (7.27\,in^4)}}$
$= 112.0 < 1685$ ∴ inelastic

$F_b = B_c - 1.6 D_c \sqrt{2 L_b S_c / C_b \sqrt{I_y J}}$
$= 39.4 - 1.6(0.25)\sqrt{112.0}$
$= 35.2$ ksi

$35.2\,ksi > F_{cy}$ ∴ use $F_b = F_{cy} = 35\,ksi$

$M_n = F_b S_c$
$= 35\,ksi\,(2.43\,in^3) = 85.05$ kin

$\phi M_n = 0.9(85.05\,kin)$
$= 76.55$ kin

Tall column axial capacity (L_b=11.4')

$S_2 = C_c$
$= 65.67$

$k = 1.0$ per E.2

$kL/r = 1.0(11.4')(12''/') / 1.58''$
$= 86.58$

$86.58 \geq 65.67 \therefore$ elastic

$F_c = \frac{0.85\pi^2 E}{(kL/r)^2}$ E.3-3

$= 0.85\pi^2(10100\,ksi)/86.58^2$
$F_c = 11.3\,ksi$

$P_{nc} = F_c A$
$= 11.3\,ksi(1.94\,in^2)$
$= 21.9k$

$P_{nt} = F_{yt} A$
$= 35\,ksi(1.94\,in^2)$
$= 67.9k$

$\emptyset P_n = 0.9(21.9k)$
$= 19.7k$

$\emptyset P_{nt} = 0.9(67.9k)$
$= 61.1k$

AT THIS STAGE, AN EXCEL SPREADSHEET WAS GENERATED WITH DEMAND FORCES FROM SAP. A FORMULA WAS THEN ADDED TO DETERMINE THE GOVERNING FORCES USING EQUATION H.1-1. THE SPREADSHEET CAN BE FOUND IN APPENDIX A7

Governing load case: 5 (0.9D + W)

$P = -13.4k$ (compression, short column)
$M_2 = 2.3\,kin$
$M_3 = -0.7\,kin$

Checking Eq. H.1-1

$$\frac{P_n}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$$

$$\frac{13.4k}{48.4k} + \frac{2.3\,kin}{76.5\,kin} + \frac{0.7\,kin}{76.5\,kin} \leq 1.0$$

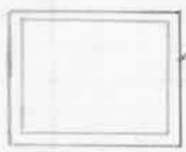
$0.317 \leq 1.0$ ✓

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** M12

SUBJECT: TYPICAL COLUMN **DATE:** 11/22/19

FINAL DESIGN



$P_u = 13.4k$

$\phi P_n = 48.4k$

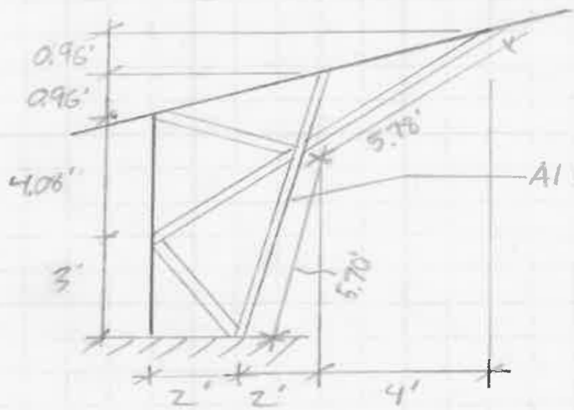
$M_{u1} = 2.3 \text{ kin}$

$M_{u2} = 0.7 \text{ kin}$

$\phi M_n = 76.5 \text{ kin}$

TRUSS DESIGN

THE TRUSS WAS DESIGNED IN A SIMILAR FASHION TO THE COLUMN, WHERE CAPACITIES WERE FOUND IN ORDER TO FIND GOVERNING FORCES FROM A SPREADSHEET OF SAP-GENERATED VALUES USING EQUATION H.1-1.



$b = 2''$
$d = 4''$
$t = 0.125''$
$A = 1.44\,in^2$
$J = 2.3\,in^4$

$I_x = 2.98\,in^4$
$S_x = 1.49\,in^3$
$r_x = 1.44''$
$I_y = 0.992\,in^4$
$S_y = 0.992\,in^3$
$r_y = 0.831''$

CAPACITY

TRUSS IS BRACED AT THE MIDPOINT INTERSECTION, SO THE GOVERNING LENGTH IS THE 5.78' TOP BRACE.

Flexural capacity

$B_c = 39.37$ SEE BEAM CALCS
$D_c = 0.25$ FOR EQUATIONS
$C_c = 65.67$

$\phi = 0.9$ per F.1
$C_b = 1.0$ per F.1.1a

$S_2 = 1685$ per beam calcs

$$\frac{2L_bS_c}{C_b\sqrt{I_yJ}} = \frac{2(5.78')(1.49\,in^3)(12''/1')}{1.0\sqrt{0.992\,in^4(2.3\,in^4)}} = 136.8 < 1685 \therefore \text{inelastic}$$

$$F_b = B_c - 1.6D_c\sqrt{\frac{2L_bS_c}{C_b\sqrt{I_yJ}}} = 39.4 - 1.6(0.25)\sqrt{136.8}$$

$F_b = 34.77$ ksi

$M_n = F_b S_c$
$= 34.77\,ksi\,(1.49\,in^3)$
$= 51.8\,k\,in$

$\phi M_n = 0.9\,(51.8\,k\,in)$
$= 46.6\,k\,in$

Axial capacity

$S_2 = C_c$
$= 65.67$

$k = 1.0$ per E.2

$kL/r = 1.0\,(5.78')(12''/1')/0.831''$
$= 83.47$

$83.47 \geq 65.67 \therefore$ elastic

$F_c = \dfrac{0.85\pi^2 E}{(kL/r)^2}$ E.3-3
$= 0.85\pi^2(10100\,ksi)/83.47^2$
$F_c = 12.16\,ksi$

$P_{nc} = F_c A$
$= 12.16\,ksi\,(1.44\,in^2)$
$= 17.5\,k$

$\phi P_{nc} = 0.9\,(17.5\,k)$
$= 15.8\,k$

$P_{nt} = F_{yt} A$
$= 35\,ksi\,(1.44\,in^2)$
$= 50.4\,k$

$\phi P_{nt} = 0.9\,(50.4\,k)$
$= 45.4\,k$

Flexural capacity (out of plane)

In weak-axis bending, lateral torsional buckling obviously won't govern.

$M_n = F_{yc} S_y$
$= 35\,ksi\,(0.992\,in^3)$
$= 34.72\,k\,in$

$\phi M_n = 0.9\,(34.72\,k\,in)$
$= 31.2\,k\,in$

AT THIS STAGE, CAPACITIES HAVE BEEN ENTERED IN THE SPREADSHEET TO DETERMINE GOVERNING LOADS. (SEE APPENDIX A8)

$P = -6.718k$ (compression)
$M = 0.763$ kin
$M_{oop} = 2.289$ kin

Checking Eq. H.1-1

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \leq 1.0$$

$$\frac{6.72k}{15.76k} + \frac{0.76\,kin}{46.62\,kin} + \frac{2.29\,kin}{31.25\,kin} \leq 1.0$$

$0.516 \leq 1.0$ ✓

FINAL DESIGN

HSS4×2×1/8

$P_u = 6.72k$
$\phi P_n = 15.76k$

$M_{u1} = 0.76$ kin
$\phi M_{n1} = 46.62$ kin

$M_{u2} = 2.29$ kin
$\phi M_{n2} = 31.25$ kin

FLOW

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** M16

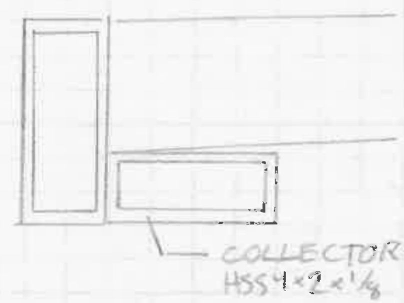
SUBJECT: TYPICAL COLLECTOR **DATE:** 11/22/19

COLLECTOR DESIGN

THE COLLECTORS ARE HSS4×2×1/8 (SAME AS BEAM) MEMBERS ORIENTED WEAK AXIS AND ATTACHED TO THE SIDE OF THE GIRDERS IN ORDER TO TRANSFER WIND LOADS FROM THE ROOF TO THE TRUSSES. THOUGH UNORTHODOX, THEY ARE NECESSARY DUE TO THE LACK OF A STRUCTURAL DIAPHRAGM.

THE LOADS ON THE COLLECTORS ARE TAKEN FROM SAP OUTPUT FOUND IN APPENDIX A10.

THE COLLECTORS ARE ATTACHED TO THE GIRDER BETWEEN BEAMS. MAKE SURE TO AVOID WELDING THE GIRDER.



DEMAND

$M_u = 9.87 \text{ k in } \left(\frac{1'}{12''}\right)$ per A10

$= 0.82 \text{ kft}$

All other loads are taken by the girder.

CAPACITY

SINCE THE BEAMS HAVE THE SAME SECTION PROPERTIES (ALBEIT ORIENTED DIFFERENTLY) AND BOTH MEMBERS ARE ASSUMED TO BE WELD-AFFECTED, WE CAN REUSE THE VALUES FOUND IN THE BEAM DESIGN.

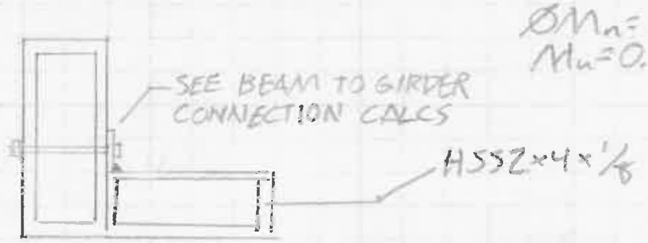
$\phi M_n = 1.68 \text{ kft}$ per beam design

$1.68 \text{ kft} > 0.82 \text{ kft}$ ✓

FINAL DESIGN

$\phi M_n = 1.68 \text{ kft}$

$M_u = 0.82 \text{ kft}$



FOUNDATIONS

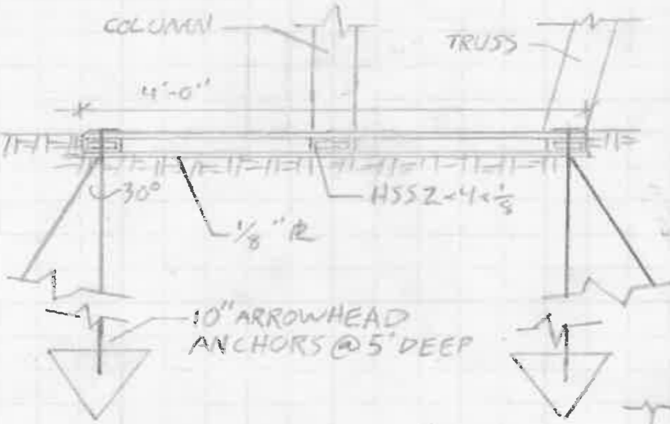
FLOW

PROJECT: WINE HISTORY PAVILION

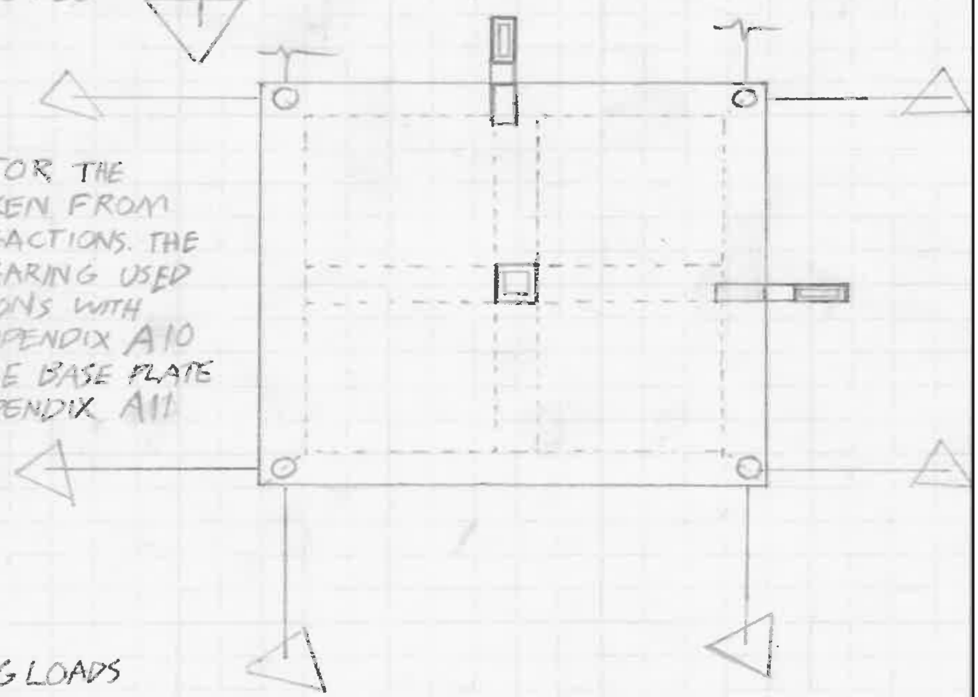
PREPARED BY: ISAAC CAMERON **PAGE:** F1

SUBJECT: CORNER FOUNDATION **DATE:** 11/22/19

CORNER FOUNDATION DESIGN

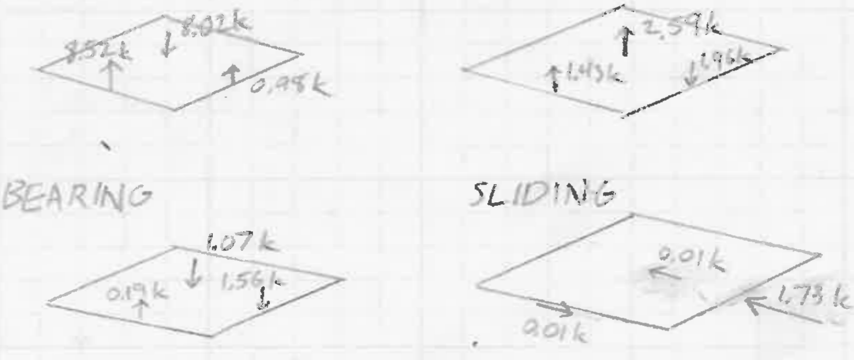


THE FOUNDATION RELIES ON ARROWHEAD ANCHORS TIED BY AIRCRAFT CABLE TO A SANDWICHED ALUMINUM PLATE IN ORDER TO RESIST UPLIFT AND OVERTURNING. THE ANCHORS ARE INSTALLED AT A 30 DEGREE ANGLE TO RESIST SLIDING. DETAILS ABOUT THE ANCHORING SYSTEM CAN BE FOUND IN APPENDIX A14

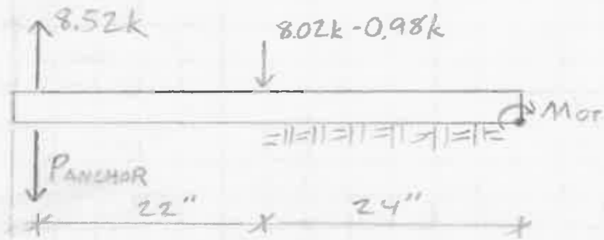


DESIGN LOADS FOR THE FOOTING WERE TAKEN FROM SAP USING JOINT REACTIONS. THE ANCHORS AND SOIL BEARING USED ASD LOAD COMBINATIONS WITH VALUES FOUND IN APPENDIX A10 LRFD LOADS FOR THE BASE PLATE CAN BE FOUND IN APPENDIX A11

ASD GOVERNING LOADS



CHECKING OVERTURNING



Finding anchor capacity

$\Omega = 2$ for hardware by engineering judgement

Per appendix A14, the pullout strength of the anchor is

$P_n = 4000\#$ for any compacted soil
$= 4k$

Adjusting for the angle of the cable and Ω

$P_{ALL} = P_n \cos\theta / \Omega$
$= 4k \cos(30)/2$
$= 1.73k$

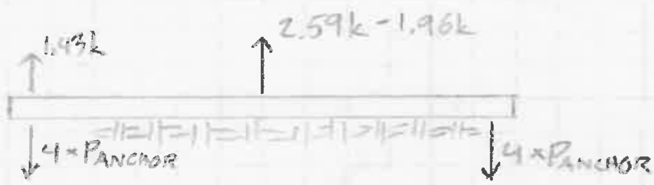
Checking M_{OT} (note: P_{ANCHOR} represents 4 anchors)

$M_{OT} = 8.52k(46") - (8.06k - 0.98k)(24") - 4(1.73k)(46")$

$M_{OT} = -96.3 \text{ kin}$

$-96.3 \text{ kin} < 0$ ∴ passes OT

CHECKING UPLIFT



From the overturning calculations, we know that for the anchors,

$P_{ALL} = 1.73k$

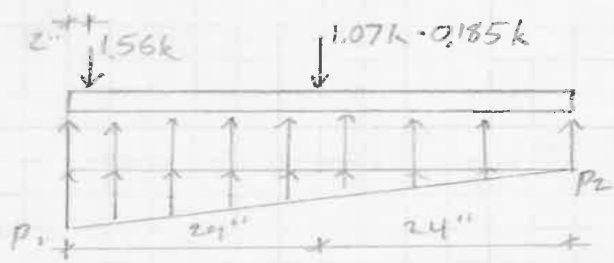
Checking uplift

$\Sigma F_y = 0 = 1.43k + (2.59k - 1.96k) - 8 P_{ANCHOR}$

$P_{ANCHOR} = 0.258k$

$0.258k < 1.73k$ ∴ passes uplift

CHECKING BEARING



Per 2018 IBC T1806.2

$p_{ALL} = 1000\ psf$

SINCE THE SOIL CONDITION IS UNKNOWN, WE'RE USING THE MOST CONSERVATIVE TABULATED VALUE.

$\Sigma M_o = 0 = -1.56k(22") + p_1(48")^2/2(8")$

$p_1 = 0.00372\ ksi$

$\Sigma F_y = 0 = -1.56k - 1.07k + 0.185k + p_2(48")^2 + (0.00372\ ksi)(48")^2/2$

$p_2 = -0.000801\ ksi$

$p_{max} = p_1 + p_2$

$= (0.00372\ ksi - 0.000801\ ksi)(1000\ \#/k)(12"/1')^2$

$p_{max} = 421\ psf$

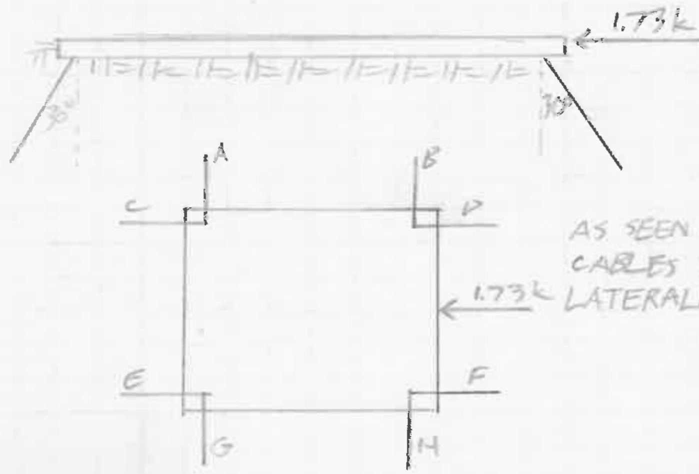
$421\ psf < 1000\ psf$ ∴ passes bearing

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** F4

SUBJECT: CORNER FOUNDATION **DATE:** 11/22/19

CHECKING SLIDING



AS SEEN FROM ABOVE, ONLY CABLES D + F ARE SUPPORTING LATERALLY

$P_{ALLOW} = P_n \sin\theta / \Omega$
$= 4k \sin(30°)/2$
$= 1k$

The total resisting force

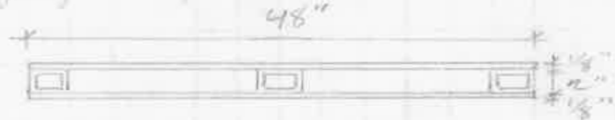
$P_n / \Omega = 2(1k)$
$= 2k > 1.73k$ ∴ passes sliding

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** F5

SUBJECT: CORNER FOUNDATION **DATE:** 11/22/19

Designing base plate



For the HSS 2×4×1/8

$d = 2"$
$b = 4"$
$A = 1.44 \text{ in}^2$
$I_x = 0.992 \text{ in}^4$

Finding net section properties

$A = 2(48")(1/8") + 3(1.44 \text{ in}^2)$
$= 16.3 \text{ in}^2$

$I = 3(0.992 \text{ in}^4) + [48"(1/8")^3/12 + 48"(1/8")(1" + 1/16")^2]2$
$I = 16.5 \text{ in}^4$

Finding moment capacity

$F_y = 35$ ksi per TA.3.4 (2010 ADM)

$M_n = F_y I / y$
$= 35\text{ksi}(16.5 \text{ in}^4)/(1" + 0.125")$
$= 514.5$ kin

$\varnothing = 0.9$ per F.1

$\varnothing M_n = 0.9(514.5 \text{ kin})(1/12")$
$\varnothing M_n = 38.6$ kft

Finding shear capacity

Limiting shear capacity to sides of 2x4s

$\varnothing = 0.9$ per G.1
$F_s = 27$ksi per TA.3.4

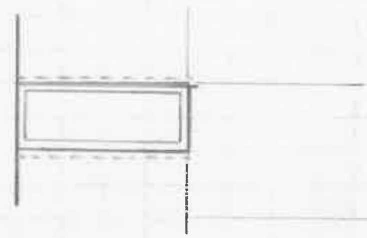
$V_n = F_s A_w$
$= 27 \text{ ksi}(6)(1/8")(2")$
$= 36$k

$\varnothing V_n = 0.9(36\text{k})$

$\varnothing V_n = 32.4$ k

Finding punching shear capacity

Relying entirely on edge HSS to resist



$V_n = F_s A_w$
$= 27ksi(4)(1/8")(2")$
$= 24k$

$\emptyset V_n = 0.9(24k)$
$= 21.6k$

Finding demands

DEMAND FORCE IS TAKEN FROM APPENDIX A11

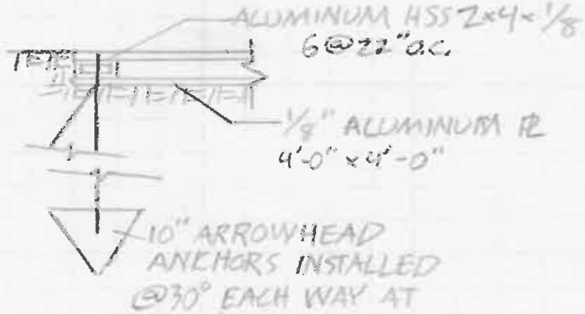
$V_u = 14.3k$

$M_u = 14.3k(2')$
$= 28.6k$

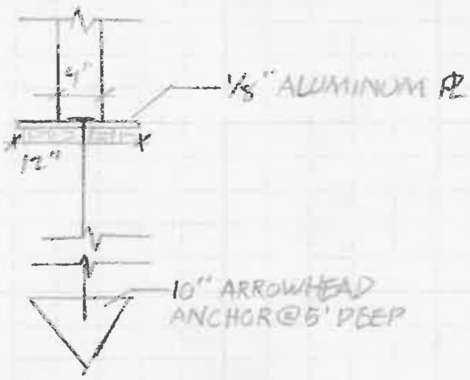
Checking base plate

Moment:	28.6k < 38.6k	✓
Shear:	14.3k < 32.4k	✓
Punching shear:	14.3k < 21.6k	✓

FINAL DESIGN



COLUMN FOUNDATION DESIGN



SIMILAR TO THE CORNER FOOTING, THE COLUMN IS HELD DOWN BY AN ARROWHEAD ANCHOR, FOUND IN APPENDIX A12, TO DEAL WITH UPLIFT. BEARING IS HANDLED BY A 1 FOOT PLATE AND LATERAL ISN'T A CONCERN.

ANCHOR CAPACITY: 4000#
Ω: 2

p_{ALL} = 1000 psf per 2018 IBC T1806.2

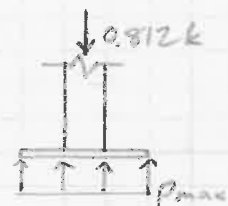
FINDING DEMAND

DEMAND LOADS ARE TAKEN FROM APPENDIX A10 AND A11 AND ARE IN ASD AND LRFD LOAD COMBOS RESPECTIVELY

	ASD	LRFD
BEARING	0.812k	1.26k
UPLIFT	0.771 k	N/A

CHECKING BEARING

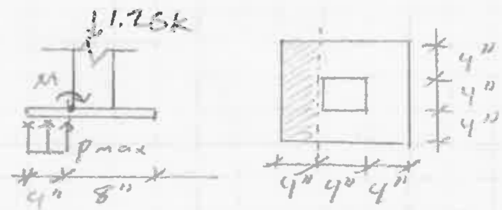
$P_n < p_{ALL} A$
$= 1500\text{ psf}(1')(1')(1^k/1000\#)$
$P_n = 1.5\text{ k} > 0.812$ ✓



CHECKING UPLIFT

$P_n = P_{ALL}/\Omega$
$= 4000\#/2\ (1^k/1000\#)$
$P_n/\Omega = 2k > 0.771\text{ k}$ ✓

CHECKING BASE PL



Moment demand

$P_{max} = 1.26k / (12")^2$
$= 0.00875$ ksi

$M_u = 0.00875(12")(4")^2/2$
$= 0.84$ kin

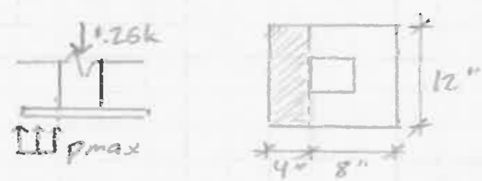
Moment capacity

$\phi = 0.9$ per F.1
$F_{ty} = 35$ksi per TA.3.4

$M_n = F_{ty} I / y$
$= 35ksi(12")(1/8")^3/12 / (1/16")$
$= 1.09$ kin

$\phi M_n = 0.9(1.09 kin)$

$\phi M_n = 0.98$ kin $>$ 0.84 kin ✓



Shear demand

$V_u = 0.00875ksi(4")(12")$
$= 0.42k$

Shear capacity

$F_{su} = 27$ ksi per TA.3.4

$V_n = F_{su} A$
$= 27ksi(12")(1/8")$
$= 40.5k$

$\phi = 0.9$ per G.1

$\phi V_n = 0.9(40.5k)$

$\phi V_n = 36.4k > 0.42k$ ✓

PROJECT: WINE HISTORY PAVILION

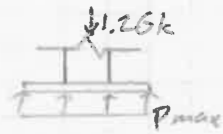
PREPARED BY: ISAAC CAMERON **PAGE:** F9

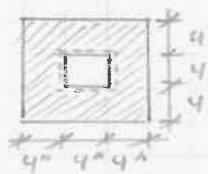
SUBJECT: COLUMN FOOTING **DATE:** 11/22/19

Block shear demand

$V_u = 0.00875\ ksi\ [(12")^2 - (4")^2]$

$= 1.12k$





Block shear capacity

$V_n = F_{su} A$

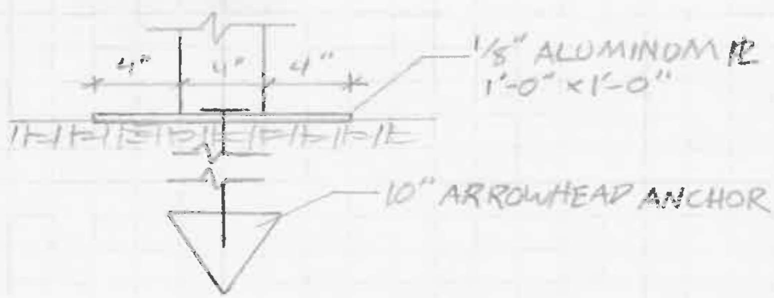
$= 27 ksi (16")(1/8")$

$= 54k$

$\phi V_n = 0.9(54k)$

$\phi V_n = 48.6k \quad > 1.12k$ ✓

FINAL DESIGN



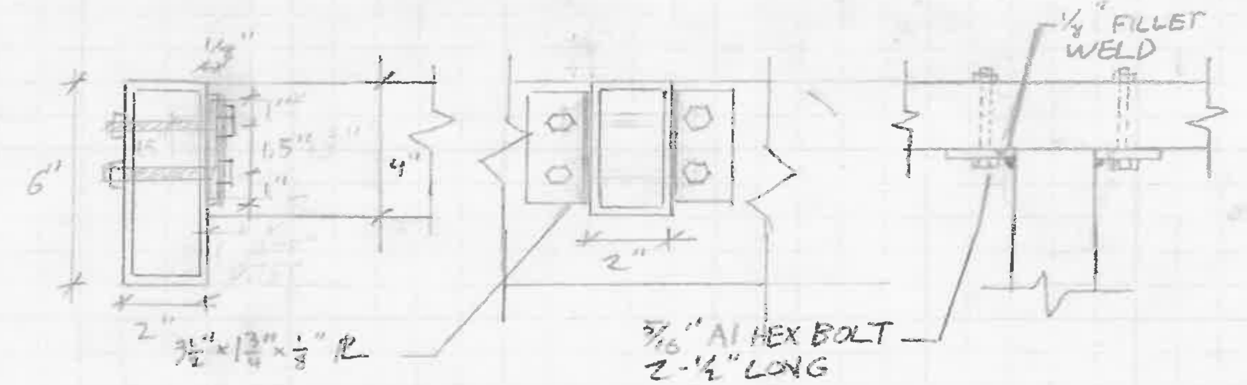
CONNECTIONS

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** C1

SUBJECT: BEAM-GIRDER CONNECTION **DATE:** 11/22/19

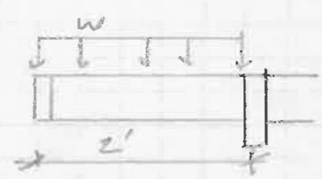
BEAM TO GIRDER CONNECTION



DEMAND

From beam calculation: $V_u = 0.405$ k

There will also be a small moment demand from the cantilevered elements.

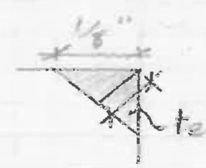


w = 50.6 plf per beam calculation

$M_u = 50.6\,\text{plf}\,(2')^2/2\,(1\text{k}/1000\#)(12''/1')$
$= 1.21$ kin

CAPACITY

Checking weld shear capacity



Per TA.3.4 for base aluminum

$F_{tu} = 42$ ksi $F_{ty} = 35$ ksi
$F_{su} = 27$ ksi $F_{cy} = 35$ ksi
$E = 10100$ ksi

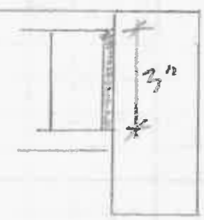
Per TA.3.5 for weld-affected aluminum (≤ 1" from weld)

$F_{tuw} = 24$ ksi $F_{tyw} = 15$ ksi
$F_{suw} = 15$ ksi $F_{cyw} = 15$ ksi

Per TM.9.1, use filler material 5356 for alloy 6061

Per TJ.2.1 for weld material

$F_{tu} = 35$ ksi
$F_{su} = 17$ ksi



$F_w = F_{su}\, t_e$ per J.2.2a
$= 17\,ksi\,(1/8")/\sqrt{2}$
$= 1.5\ k/in$

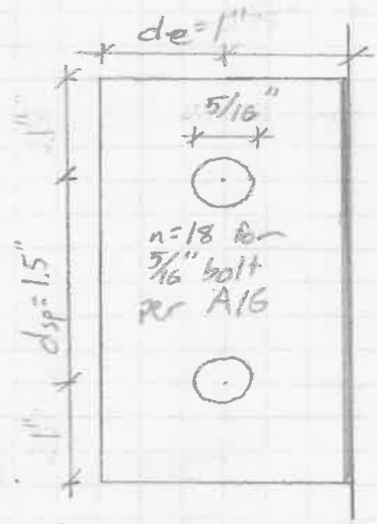
$R_n = F_w L_{eff}$
$= 1.5\ k/in\,(2)(3.5")$
$= 10.5\ k$

$\phi = 0.75$ per J.2

$\phi R_n = 0.75(10.5k)$

$\phi R_n = 7.88\ k > 0.405k$ ✓

Checking bolt geometry



$de_{min} = 1.5D$ per J.3.4
$= 1.5(5/16")$
$= 0.469" < 1"$ ✓

$d_{smin} = 2.5D$ per J.3.3
$= 2.5(5/16")$
$= 0.781" < 1.5"$ ✓

Checking bolt shear rupture

$R_n = \pi(D - 1.91/n)^2 (\tfrac{1}{4}) F_{su}$
$= \pi(5/16" - 1.91/18)^2(\tfrac{1}{4})(24ksi)$ per Eq. J.3-2
$= 0.803\ k$

$V_n = R_n\,(4\ bolts)$
$= 3.21\ k$

$\phi = 0.65$ per J.3.6

$\phi V_n = 0.65(3.21k)$

$\phi V_n = 2.09\ k > 0.405\ k$ ✓

Checking bolt bearing

$\phi = 0.75$ for bearing per J.3.7

Per Eq J.3-4

$R_n = l_c t F_{u,w}$
$= 1.0''(0.125'')(24 ksi)$
$= 3 k$

or

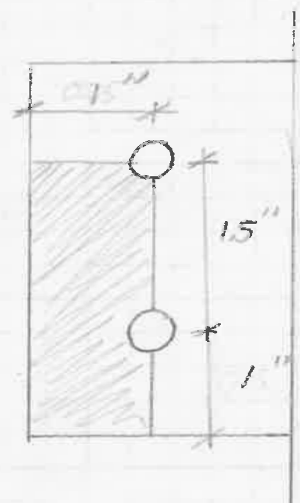
$R_n = 2 D t F_{u,w}$
$= 2(5/16'')(1/8'')(24 ksi)$
$= 1.88 k$

$V_n = 1.88k$ (4 bolts)
$= 7.5 k$

$\phi V_n = 0.75(7.5k)$

$\phi V_n = 5.63k > 0.405k$ ✓

Checking block shear



$L_s = 1.5'' + 2(1'') - 2(5/16'' + 1/16'')$
$= 2.75''$

$V_n = 2 F_{su} L_s t$
$= 2(15 ksi)(2.75'')(0.125'')$
$= 10.3 k$

$\phi = 0.9$ for shear per G.1

$\phi V_n = 0.9(10.3k)$

$\phi V_n = 9.28 k > 0.405k$ ✓

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** C4

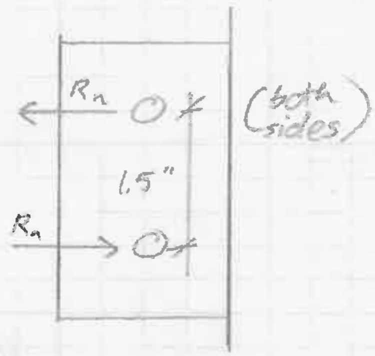
SUBJECT: BEAM-GIRDER CONNECTION **DATE:** 11/22/19

Checking moment resistance in bolt

$\phi M_n = 2(\phi R_n)(1.5")$
$= 2(0.803\,k)(1.5")$

$\phi M_n = 1.29$ kin > 1.21 kin ✓

Note: The bolts may be in tension, but since $F_{su} < F_{tu}$, this is fine.



Checking moment capacity in weld

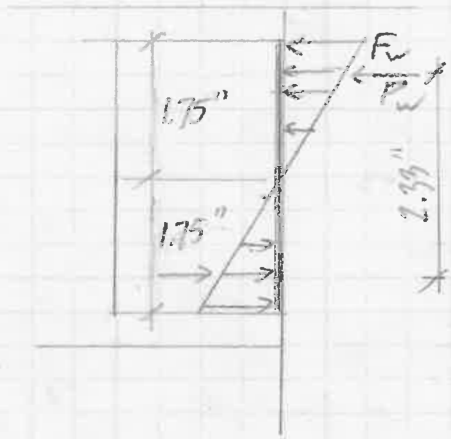
$P_w = F_w(1.75")/2$
$= 1.56\,k/in\,(1.75")/2$
$= 1.31\,k$

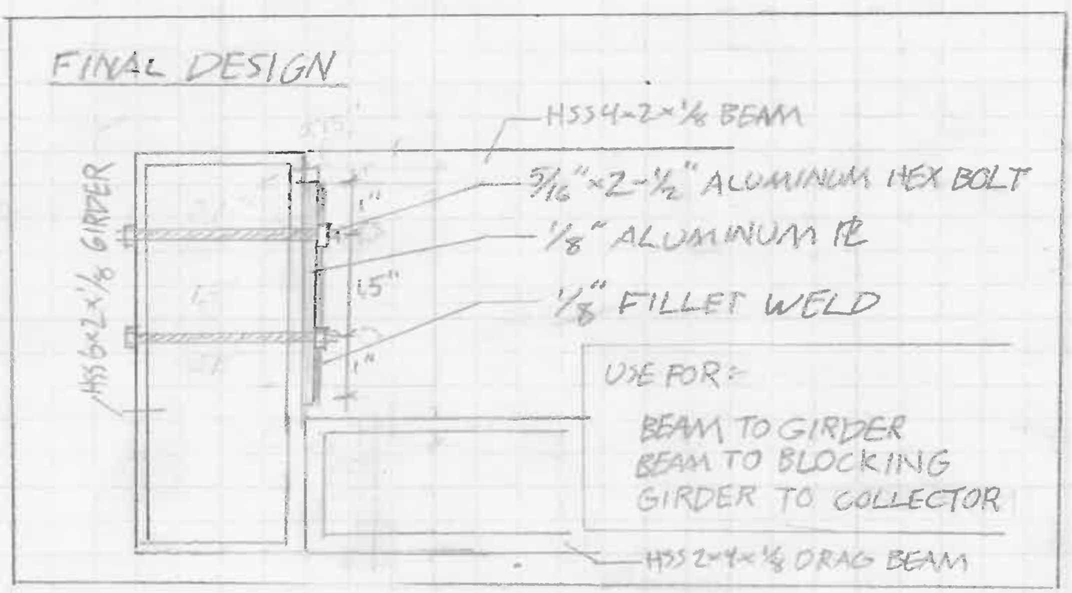
$M_n = 2(1.31\,k)(2.33")$
$= 6.13$ kin

$\phi = 0.75$ for weld per J.2

$\phi M_n = 0.75(6.13$ kin$)$

$\phi M_n = 4.59$ kin > 1.21 kin ✓



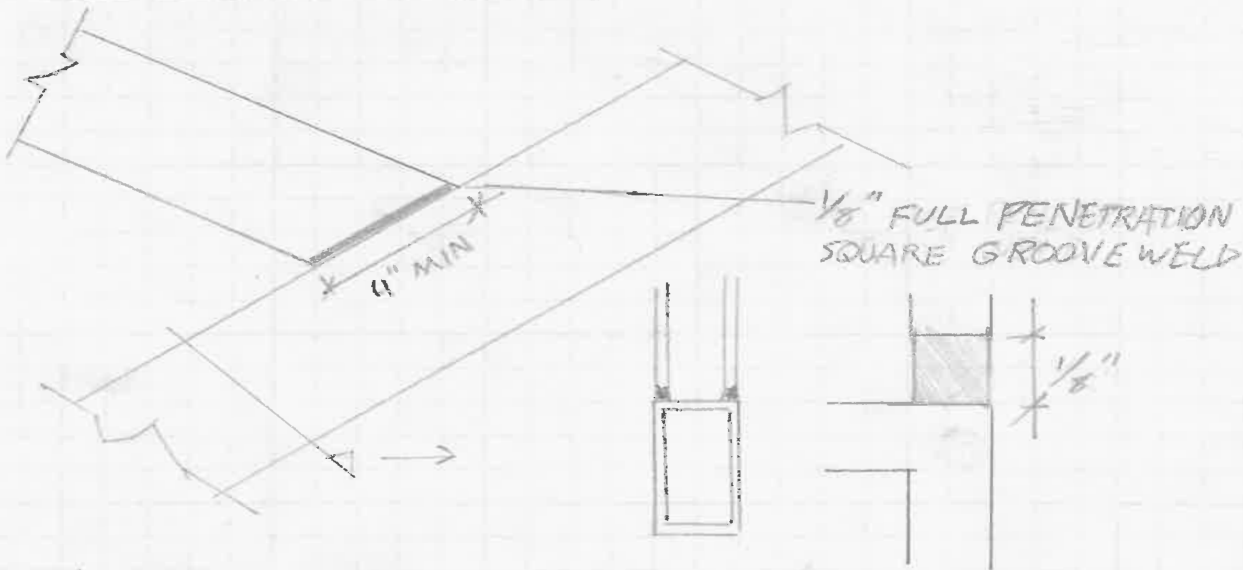


PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON — PAGE: C5

SUBJECT: TRUSS CONNECTION — DATE: 11/22/19

TRUSS TO TRUSS CONNECTION



DEMAND

$P_u = 8.127k$ per Appendix A12

CAPACITY

Checking weld tensile capacity

Per J.2.1.3, F_{tuw} is the lesser value of tensile rupture for either the base material or the filler

$F_{tuw} = 24ksi$ for base per TA.3.5
$F_{tu} = 35 ksi$ for fill per TJ.2.1

Use $F_{tuw} = 24 ksi$

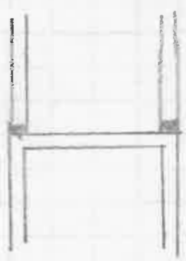
$R_n = F_{tuw} A_{we}$
$= 24ksi (2)(4")(1/8")$
$= 24k$

$\phi = 0.75$ for welds per J.2

$\phi R_n = 0.75(24k)$

$\phi R_n = 18k > 8.127k$ ✓

FINAL DESIGN



1/8" FULL-PEN
SQ GROOVE WELD

$\phi R_n = 18\ k$
$R_u = 8.127\ k$

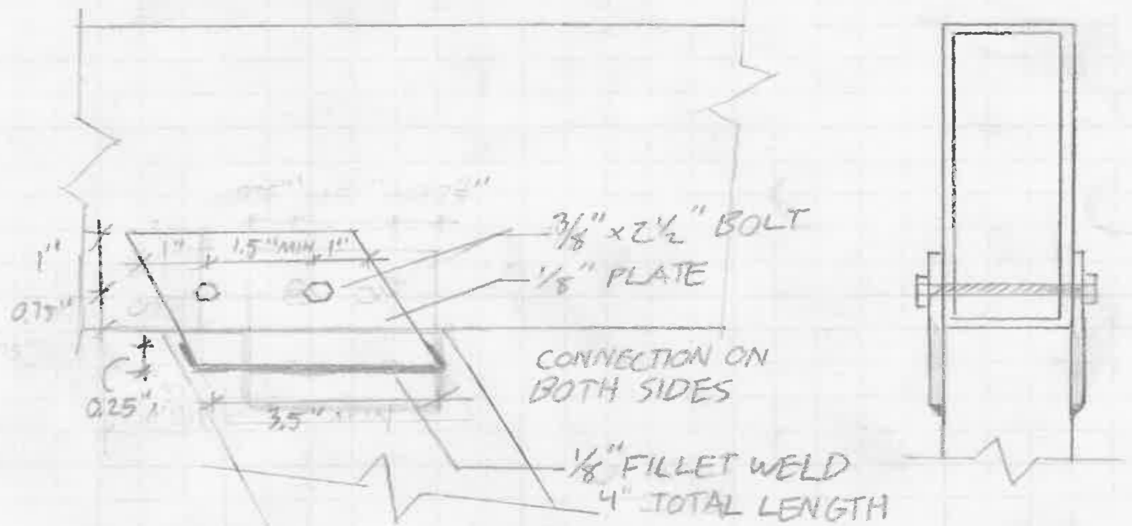
4" LONG, BOTH SIDES

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON PAGE: C7

SUBJECT: TRUSS-GIRDER CONNECTION DATE: 11/22/19

TRUSS TO GIRDER CONNECTION



DEMAND

APPLIED FORCES TAKEN FROM APPENDIX A13

$V_u = 4.21k$ $T_u = 0.45k$ $C_u = 0.75k$

$P_u = \sqrt{(4.21k)^2 + (0.75k)^2}$

$P_u = 4.28k$

CAPACITY

Per TA3.4 for base aluminum

$F_{tu} = 42\ ksi$ $F_{ty} = 35\ ksi$
$F_{su} = 27\ ksi$ $F_{cy} = 35\ ksi$
$E = 10100\ ksi$

Per TA.3.5 for weld-affected aluminum

$F_{tuw} = 24\ ksi$ $F_{tyw} = 15\ ksi$
$F_{suw} = 15\ ksi$ $F_{cyw} = 15\ ksi$

Per TM.9.1, use filler 5356

Per TJ.2.1 for weld

$F_{tu} = 35\ ksi$
$F_{su} = 17\ ksi$

Checking weld shear capacity

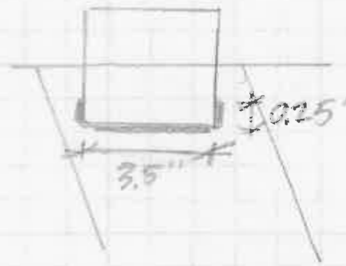
$F_w = F_{su} t_e$ per J.2.2a
$= 17 k_{si} (1/4'')/\sqrt{2}$
$= 1.5 k/in$

$R_n = F_w L_{eff}$
$= 1.5 k/in [3.5'' + 2(0.25'')]$
$= 6.0 k$

$\phi = 0.75$ for welds per J.2

$\phi R_n = 0.75(6.0 k)$

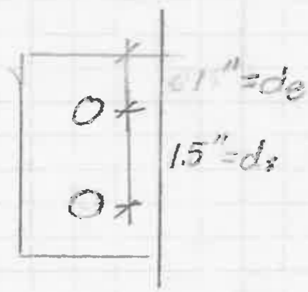
$\phi R_n = 4.50 k > 4.28 k$ ✓



Checking bolt geometry

$d_{emin} = 1.5D$
$= 1.5(1/2'')$
$= 0.75'' \le 1.0''$ ✓

$d_{smin} = 2.5D$
$= 2.5(1/2'')$
$= 1.25'' < 1.5''$ ✓



Checking bolt shear rupture using Eq J.3-2

$R_n = \pi (D - 1.91/n)^2 (1/4) F_{su}$
$= \pi (0.5'' - 1.91/13)^2 (1/4)(24 ksi)$
$= 2.35 k$

n = 13 for 1/2" BOLTS per A16

$V_n = 2.35 k (4 bolts)$
$= 9.40 k$

$\phi = 0.65$ for bolts per J.3.6

$\phi V_n = 0.65(9.4 k)$

$\phi V_n = 6.11 k > 4.28 k$ ✓

Checking bolt bearing

$\phi = 0.75$ for bearing per J.3.7

Per Eq J.3-4

$R_n = d e t F_{tu}$
$= 1.0''(0.125'')(42\,ksi)$ or
$= 5.25\,k$

$R_n = 2Dt F_{tu}$
$= 2(0.5'')(0.125'')(42\,ksi)$
$= 5.25\,k$

$V_n = 5.25\,k$ (4 bolt supports)
$= 21.0\,k$

$\phi V_n = 0.75(21.0\,k)$

$\phi V_n = 15.75\,k > 4.28\,k$ ✓

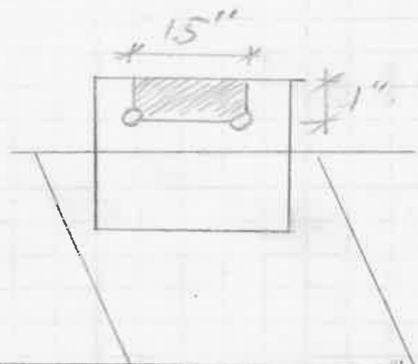
Checking block shear

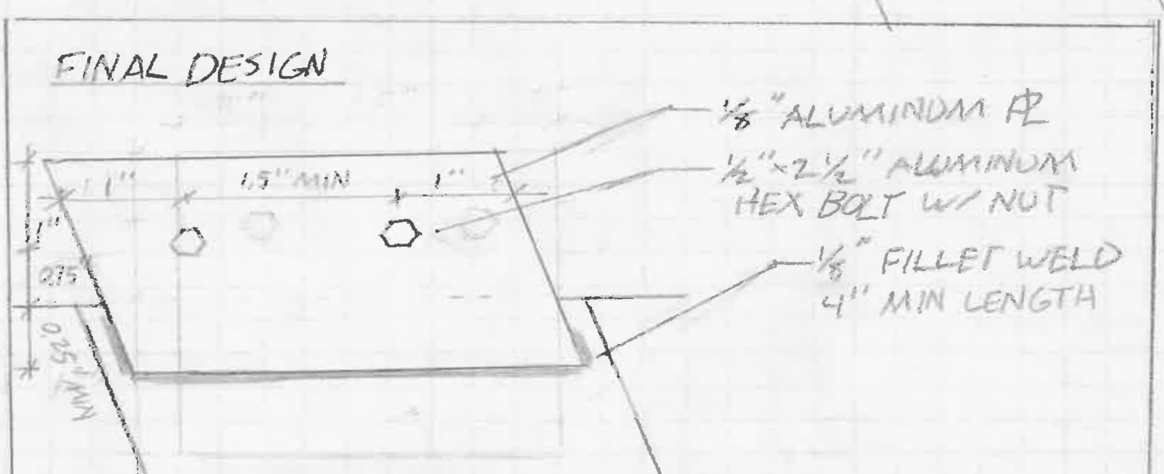
$L_s = 1.5'' + 2(1.0'') - 2(0.5'')$
$= 2.5''$

$V_n = 2 F_{su} L_s t$
$= 2(24\,ksi)(2.5'')(0.125'')$
$= 15\,k$

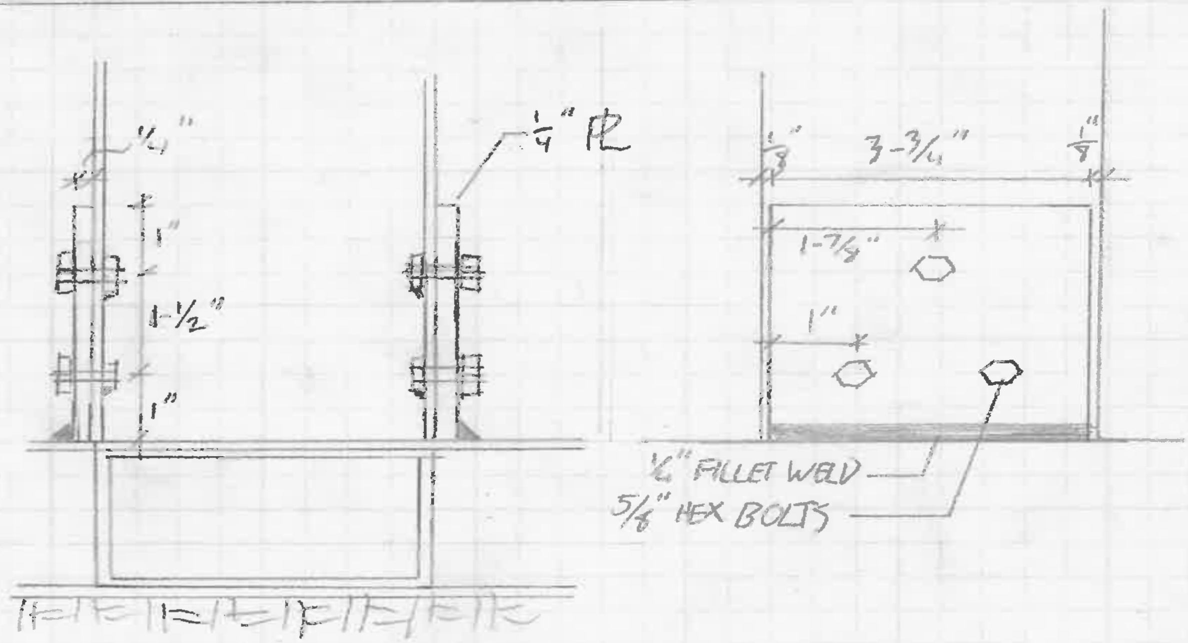
$\phi V_n = 0.9(15\,k)$

$\phi V_n = 13.5\,k > 4.28\,k$ ✓





COLUMN TO BASE PLATE CONNECTION



DEMAND

WORST CASE LOAD TAKEN FROM APPENDIX A11

$P_u = 14.3k$

CAPACITY

Per TA.3.4 for unwelded aluminum

$F_{tu} = 42 ksi$ $F_{ty} = 35 ksi$
$F_{su} = 27 ksi$ $F_{cy} = 35 ksi$
$E = 10100 ksi$

Per TA.3.5 for weld-affected aluminum

$F_{tuw} = 24 ksi$ $F_{tyw} = 15 ksi$
$F_{suw} = 15 ksi$ $F_{cyw} = 15 ksi$

Per TJ.2.1 for weld material

$F_{tu} = 35 ksi$
$F_{su} = 17 ksi$

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** C11

SUBJECT: COLUMN-BASE CONNECTION **DATE:** 11/22/19

Checking weld tension capacity

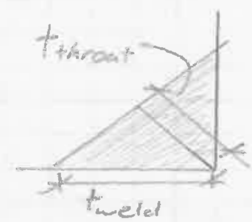
Finding governing value of F_{sw}

J.2.2.2a:

$F_{sw} = F_{sw} t_{throat}$
$= 17\,ksi(1/4")/\sqrt{2}$
$= 3.0\,k/in$

J.2.2.2c

$F_{sw} = F_{tuw} t_{weld}$
$= 24\,ksi(1/4")$
$= 6\,k/in$



$R_n = F_{sw} L_{eff}$ per Eq J.2-3
$= 3.0\,k/in(3.75")$
$= 11.27k$

$P_n = 11.27k(2) = 22.54k$

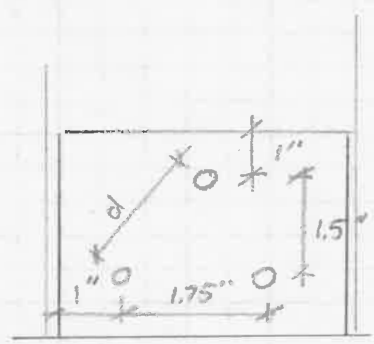
$\phi = 0.75$ for welds per J.2

$\phi P_n = 0.75(22.54k)$

$\phi P_n = 16.9k > 14.3k$ ✓

Checking bolt geometry

$d = \sqrt{(1.75"/2)^2 + (1.5")^2}$
$= 1.737"$



$d_{emin} = 1.5D$ per J.3.4
$= 1.5(5/8")$
$= 0.938" < 1"$ ✓

$d_{smin} = 2.5D$ per J.3.3
$= 2.5(5/8")$
$= 1.56" < 1.737"$ ✓

Checking bolt shear rupture

$R_n = \pi(D - 1.191/n)(1/4)F_{su}$ per Eq. J.3-2

$= \pi(5/8'' - 1.191/11)(1/4)(24 ksi)$ n=11 per Appendix A16

$R_n = 3.84k$

$V_n = R_n$ (6 bolt holes)

$= 3.84k(6)$

$= 23.0 k$

$\emptyset = 0.65$ for bolts per J.3.6

$\emptyset V_n = 0.65(23k)$

$\emptyset V_n = 15.0k > 14.3k$ ✓

Checking bolt bearing in column

$\emptyset = 0.75$ for bearing per J.3.7

Per Eq. J.3-4

$R_n = d_c t F_{un}$

$= 1''(1/8'')(42ksi)$ or

$= 5.25k$

$R_n = 2Dt F_{tu}$

$= 2(5/8'')(1/8'')(42ksi)$

$= 6.56k$

$P_n = R_n$ (6 bolt holes)

$= 5.25k(6)$

$= 31.5k$

$\emptyset P_n = 0.75(31.5k)$

$\emptyset P_n = 23.6k > 14.3k$ ✓

PROJECT: WINE HISTORY PAVILION		
PREPARED BY: ISAAC CAMERON	**PAGE:**	C13
SUBJECT: COLUMN-BASE CONNECTION	**DATE:**	11/22/19

Checking block shear in column

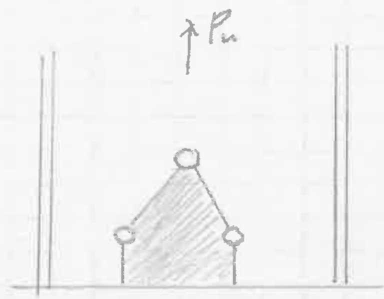
$L_s = 2(1") + 2(1.737") - 3(5/8" + 1/16")$
$= 3.41"$

$P_n = 2 F_{su} L_s t$
$= 2(24 ksi)(3.41")(0.125")$
$= 20.46 k$

$\phi = 0.9$ for shear per G.1

$\phi P_n = 0.9(20.46k)$

$\phi P_n = 18.4k > 14.3k$ ✓



Checking net tension in plate

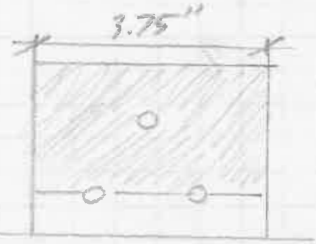
$W_{net} = 3.75" - 2(5/8" + 1/16")$
$= 2.38"$

$P_n = F_{ty} W_{net} t$
$= 35 ksi (2.38")(0.25")$
$= 20.78k$

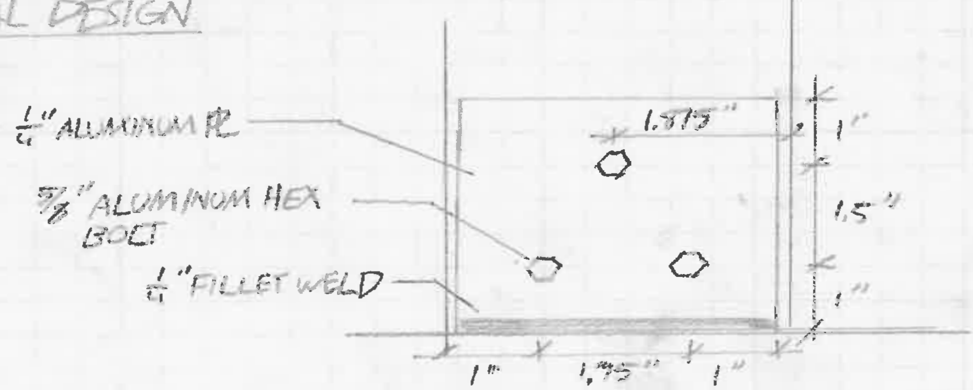
$\phi = 0.9$ for tension per D.1

$\phi P_n = 0.9(20.78k)$

$\phi P_n = 18.7k > 14.3k$ ✓



FINAL DESIGN

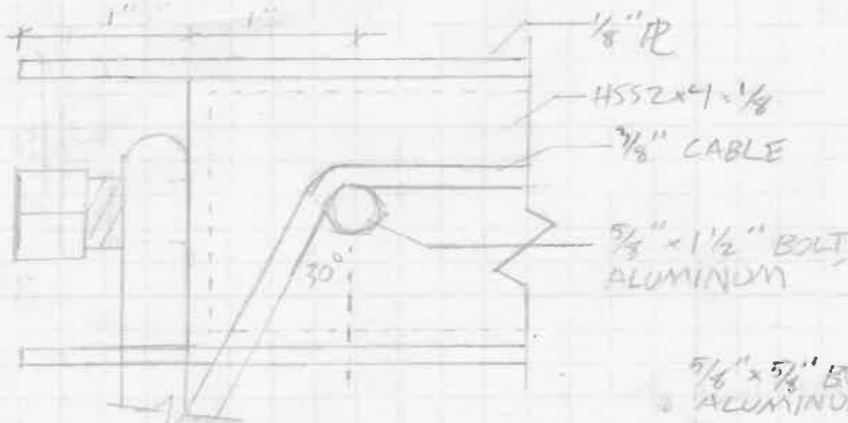


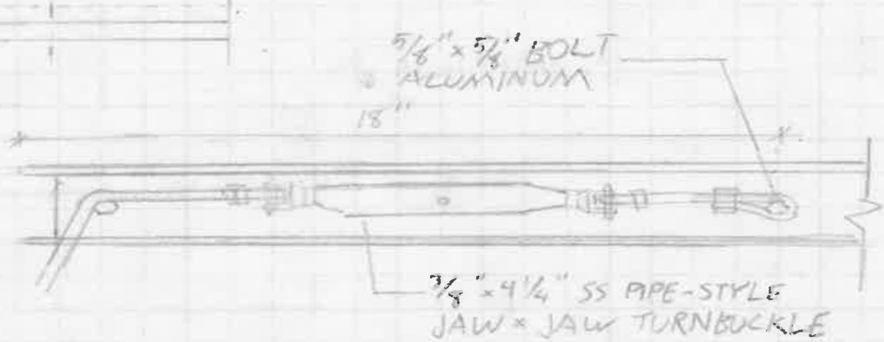
PROJECT: WINE HISTORY PAVILION

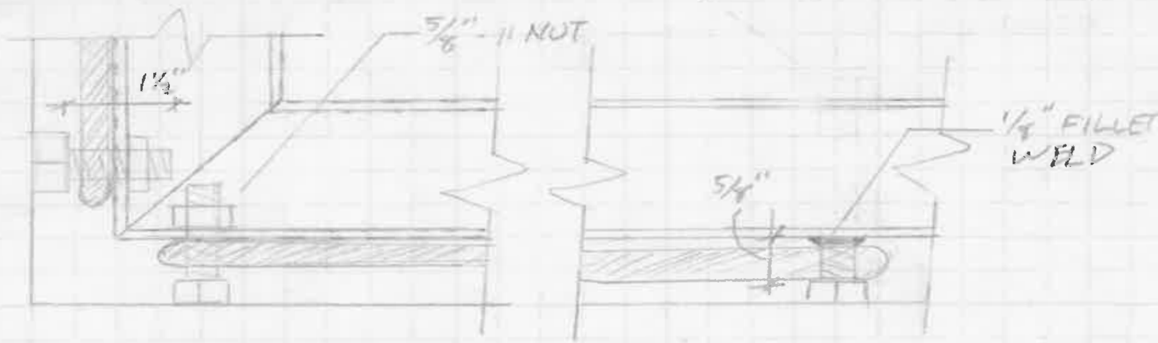
PREPARED BY: ISAAC CAMERON **PAGE:** C14

SUBJECT: ANCHOR-PLATE CONNECTION **DATE:** 11/22/19

ANCHOR TO CORNER PLATE CONNECTION







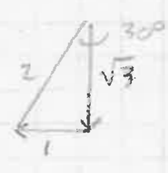
DEMAND

$V_u = 1.73^k/2$ For ASD
$= 0.865^k$ per Appendix A10

$V_u = 2.885^k/2$ For LRFD
$= 1.44^k$ per Appendix A11

FOR THE CORNER BOLT, WE NEED THE VERTICAL COMPONENT OF THE LOAD APPLIED BY THE CABLE.

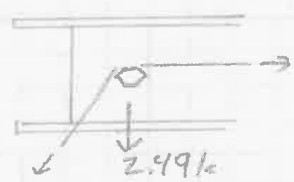
$V_u = 1.44k(\sqrt{3})$
$= 2.49\ k$



CAPACITY

Checking shear rupture in corner bolt

$\phi = 0.65$ per J.3.6
$n = 11$ per Appendix A16
$F_{su} = 24$ ksi per TA.3.4



$R_n = \pi(D - 1.191/n)^2(\frac{1}{4})F_{su}$ per Eq. J.3-2
$= \pi(0.625'' - 1.191/11)(1/4)(24 \text{ ksi})$
$= 3.84k$

$\phi R_n = 0.65(3.84k)$

$\phi R_n = 2.50k > 2.49k$ ✓

Checking bolt bearing in corner bolt

$\phi = 0.75$ per J.3.7
$F_{tuw} = 24$ ksi per TA.3.5

Per Eq. J.3-4

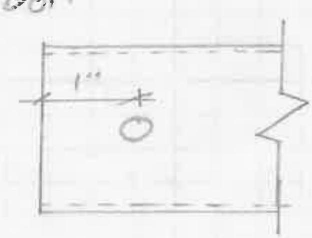
$R_n = 2Dt F_{tuw}$
$= 2(0.625'')(0.126'')(24 \text{ ksi})$
$= 3.75 k$

$\phi R_n = 0.75(3.75k)$

$\phi R_n = 2.81 k > 2.49k$ ✓

Checking edge spacing for corner bolt

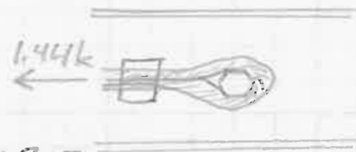
$d_{emin} = 1.5D$ per J.3.4
$= 1.5(5/8'')$



$d_{emin} = 0.938'' < 1.0''$ ✓

Checking shear rupture in center bolt (due to weld)

$F_{suw} = 15\ ksi$ per TA.3.5



$R_n = \pi[(D - 1.91/n)^2/4] F_{suw}$ per Eq. J.3-2

$= \pi(0.625" - 1.91/11)^2(1/4)(15ksi)$

$= 2.40k$

$\emptyset R_n = 0.65(2.4k)$

$\emptyset R_n = 1.56k > 1.44k$ ✓

Checking bearing due to center bolt

SINCE THE BOLT SIZE IS THE SAME AND BOTH PORTIONS OF THE BASE WERE CONSIDERED WELD-EFFECTED, THE CAPACITY HERE IS THE SAME AS THE CORNER BOLT.

$\emptyset R_n = 2.81k > 1.44k$ ✓

Checking shear in weld

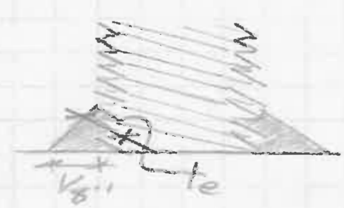
$F_{su} = 17ksi$ per TJ.2.1

$\emptyset = 0.75$ per J.2

$F_w = F_{su} t_e$ per J.2.2a

$= 17ksi(1/8")/\sqrt{2}$

$= 1.5\ k/in$



$R_n = F_w L_{eff}$

$= 1.5k/in(\pi)(0.625")$

$= 2.95k$

$\emptyset R_n = 0.75(2.95k)$

$\emptyset R_n = 2.21k > 1.44k$ ✓

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON

PAGE: C17

SUBJECT: ANCHOR-PLATE CONNECTION

DATE: 11/22/19

Checking turnbuckle capacity (ASD)

$\Omega = 2$ for hardware per engineering judgement

Per appendix A15, turnbuckle strength is:

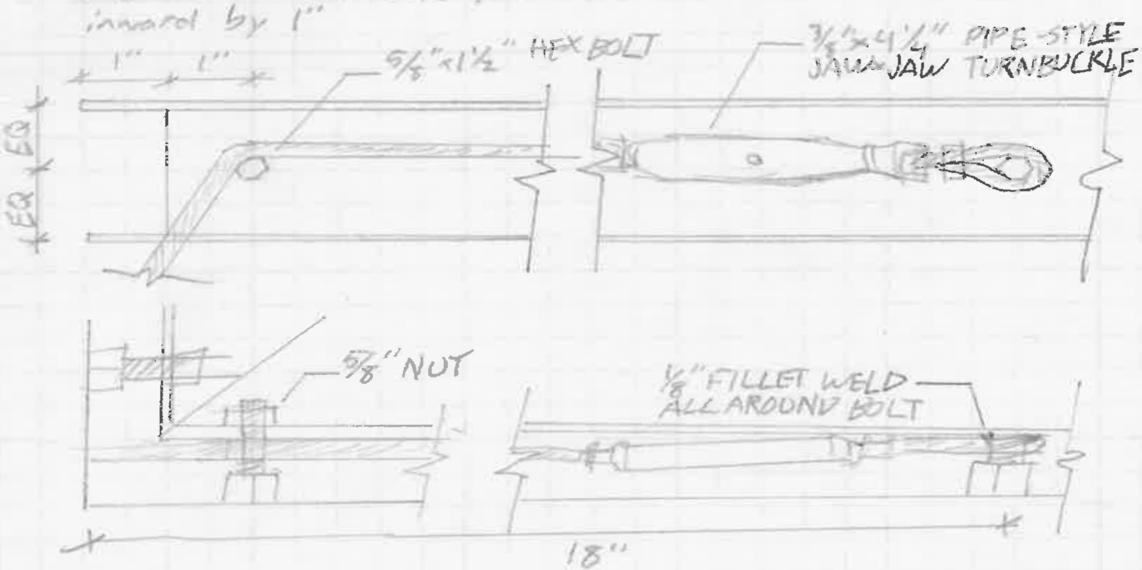
$P_n = 1900\#$

$P_n/\Omega = 1900\#/2 \; (1k/1000\#)$

$P_n/\Omega = 0.95k > 0.865k$ ✓

FINAL DESIGN

Move HSS's from base plate inward by 1"



TURNBUCKLE MIN. CAPACITY: 1,730#

APPENDIX

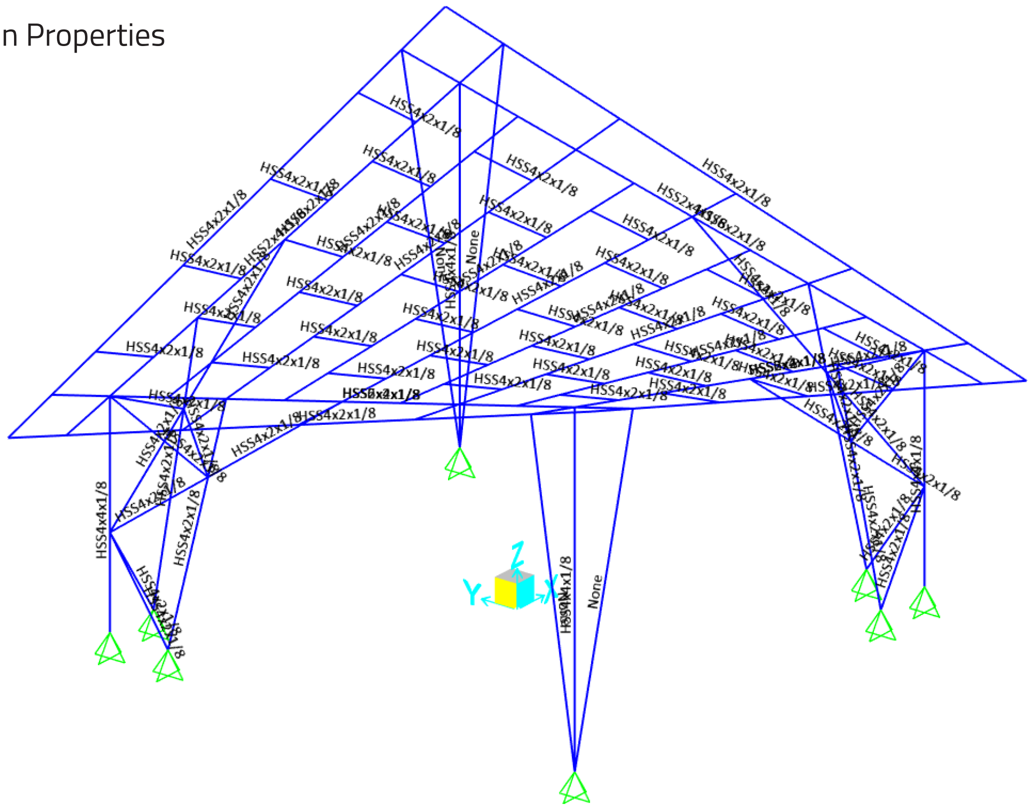
FLOW

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** A1

SUBJECT: SAP - SECTION ASSIGNMENTS **DATE:** 11/22/19

Section Properties

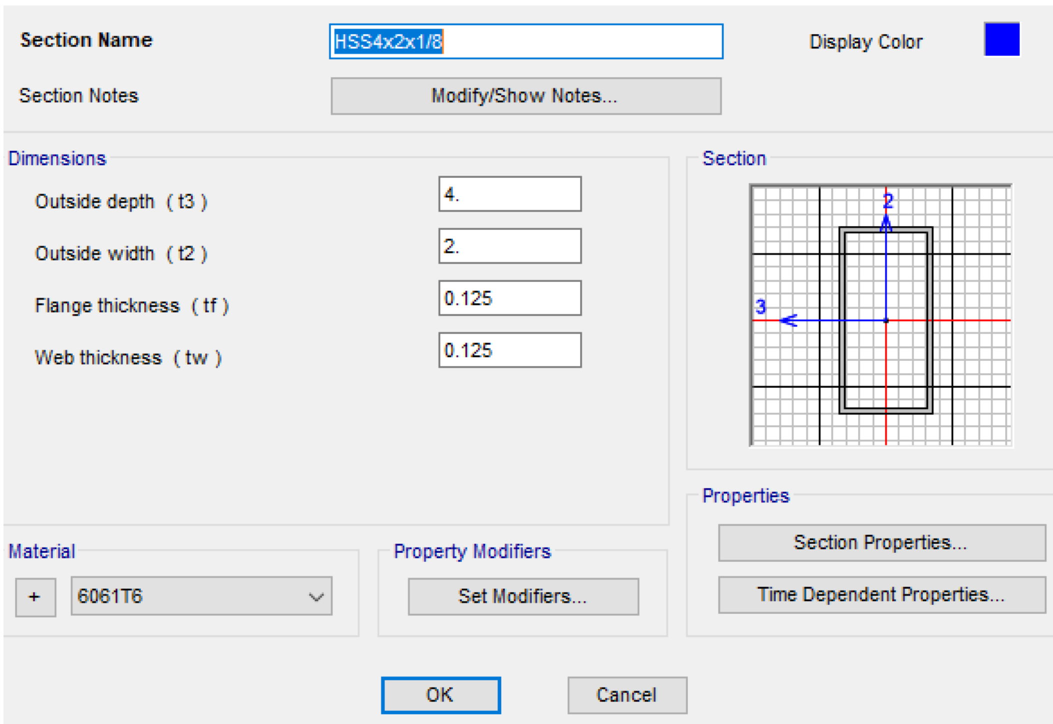


PROJECT: WINE HISTORY PAVILION

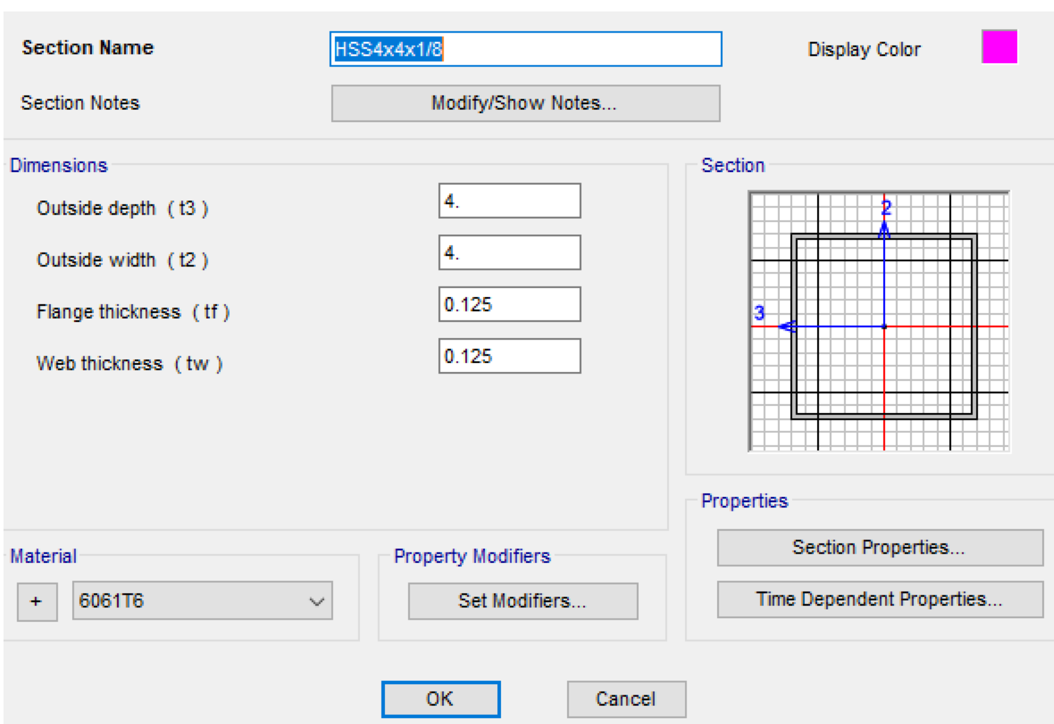
PREPARED BY: ISAAC CAMERON **PAGE:** A2

SUBJECT: SAP - SECTION PROPERTIES **DATE:** 11/22/19

HSS4X2X1/8



HSS4X4X1/8

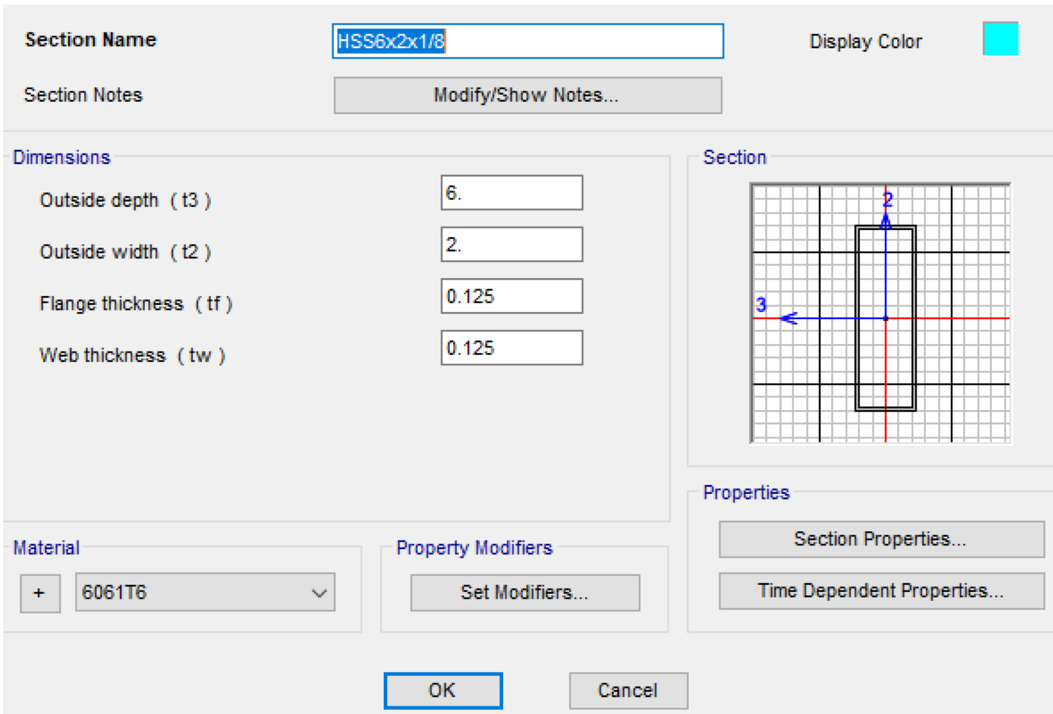


PROJECT: WINE HISTORY PAVILION

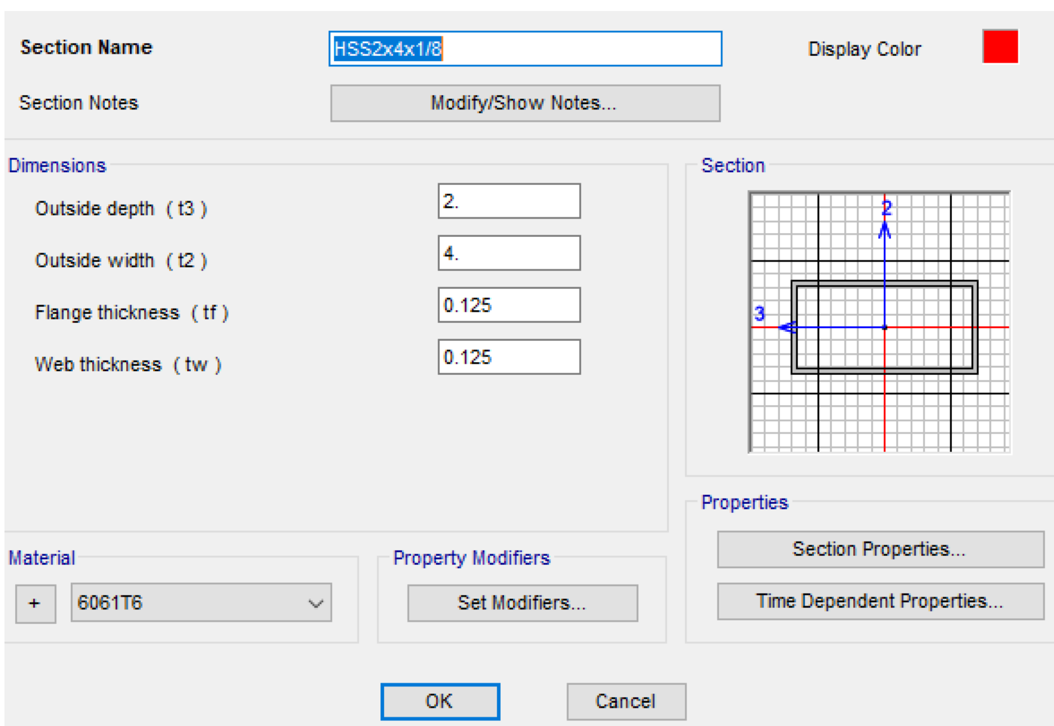
PREPARED BY: ISAAC CAMERON **PAGE:** A3

SUBJECT: SAP - SECTION PROPERTIES **DATE:** 11/22/19

HSS6X2X1/8



HSS2X4X1/8

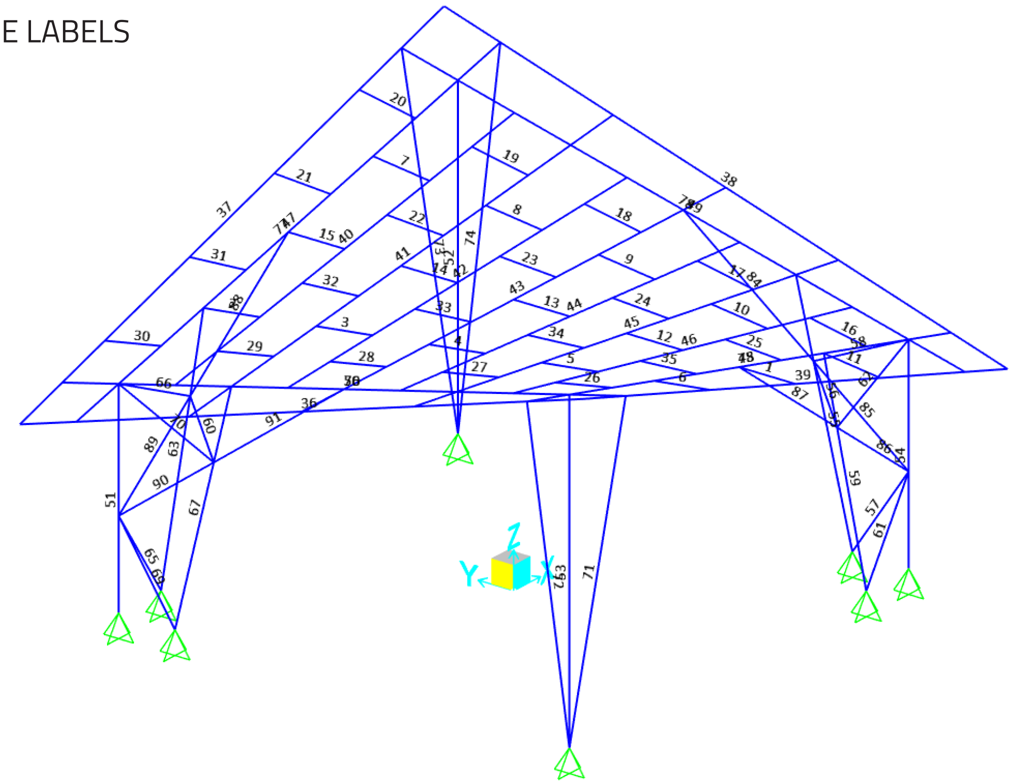


PROJECT: WINE HISTORY PAVILION

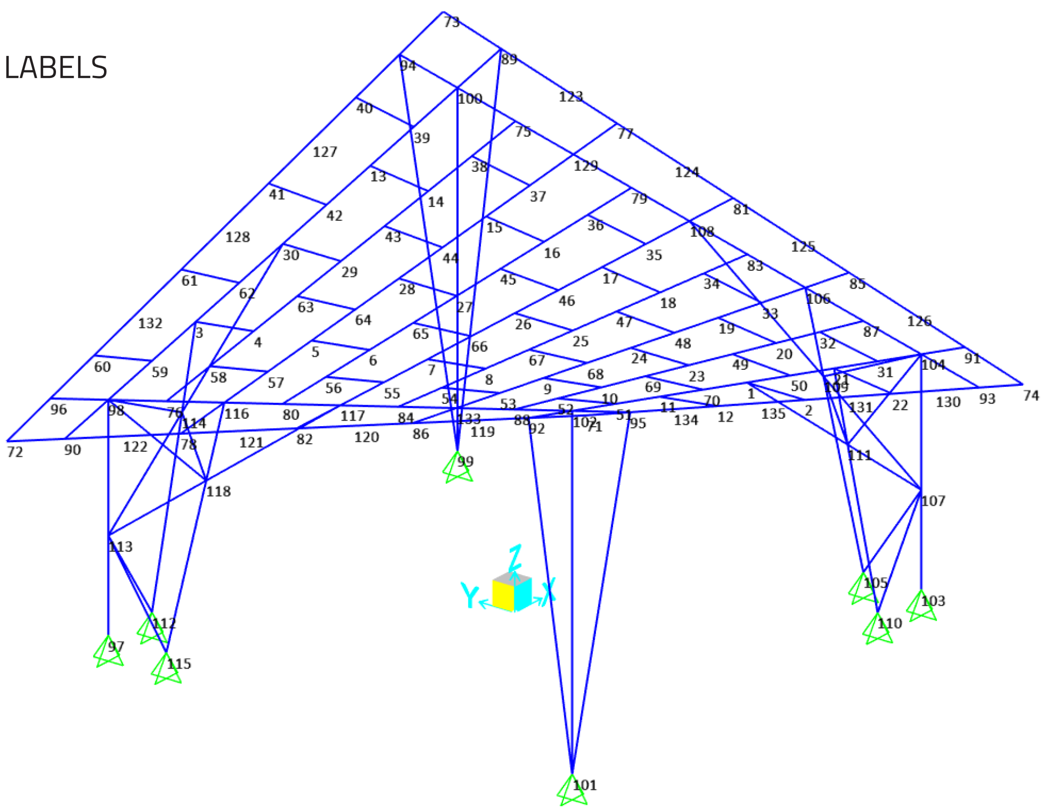
PREPARED BY: ISAAC CAMERON **PAGE:** A4

SUBJECT: SAP - OBJECT LABELS **DATE:** 11/22/19

FRAME LABELS



JOINT LABELS

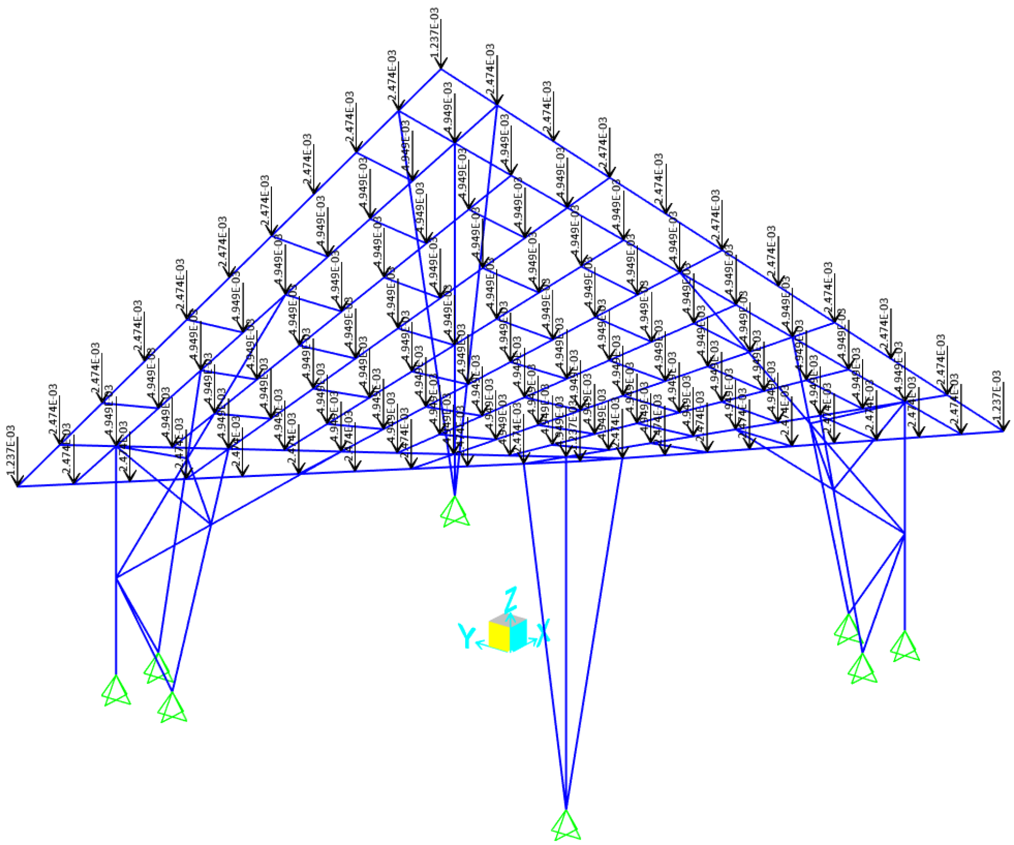


PROJECT: WINE HISTORY PAVILION

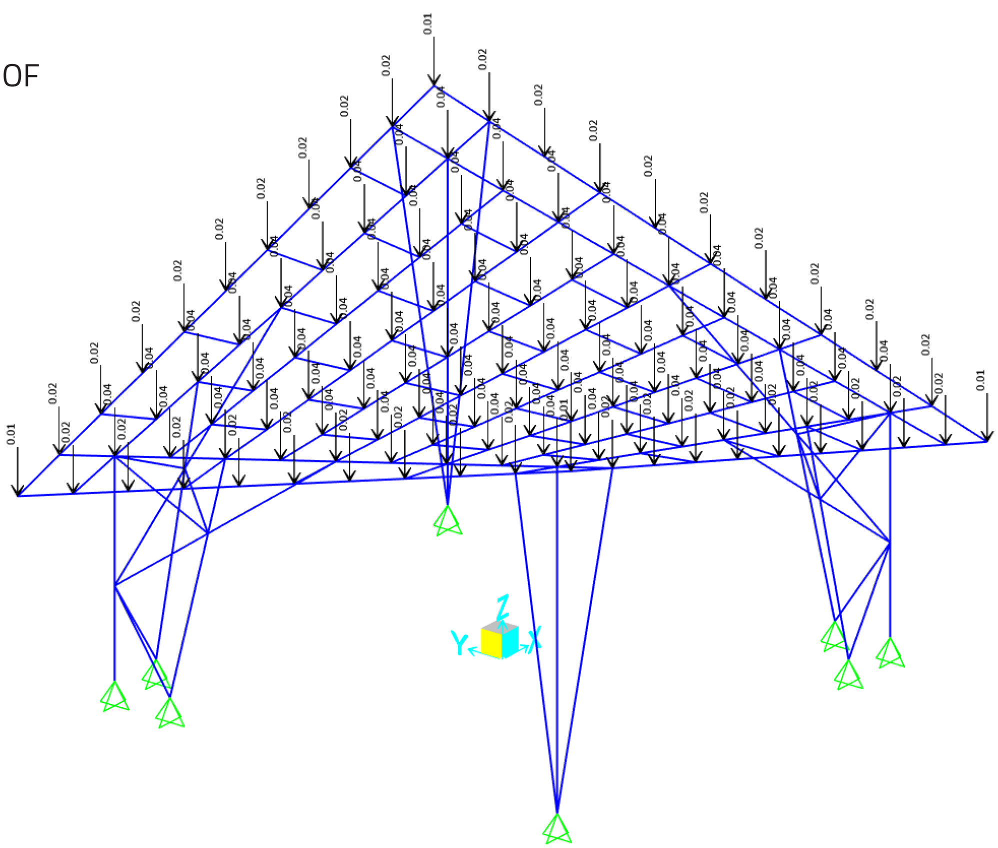
PREPARED BY: ISAAC CAMERON **PAGE:** A5

SUBJECT: SAP - LOAD ASSIGNMENTS **DATE:** 11/22/19

DEAD



LIVE ROOF

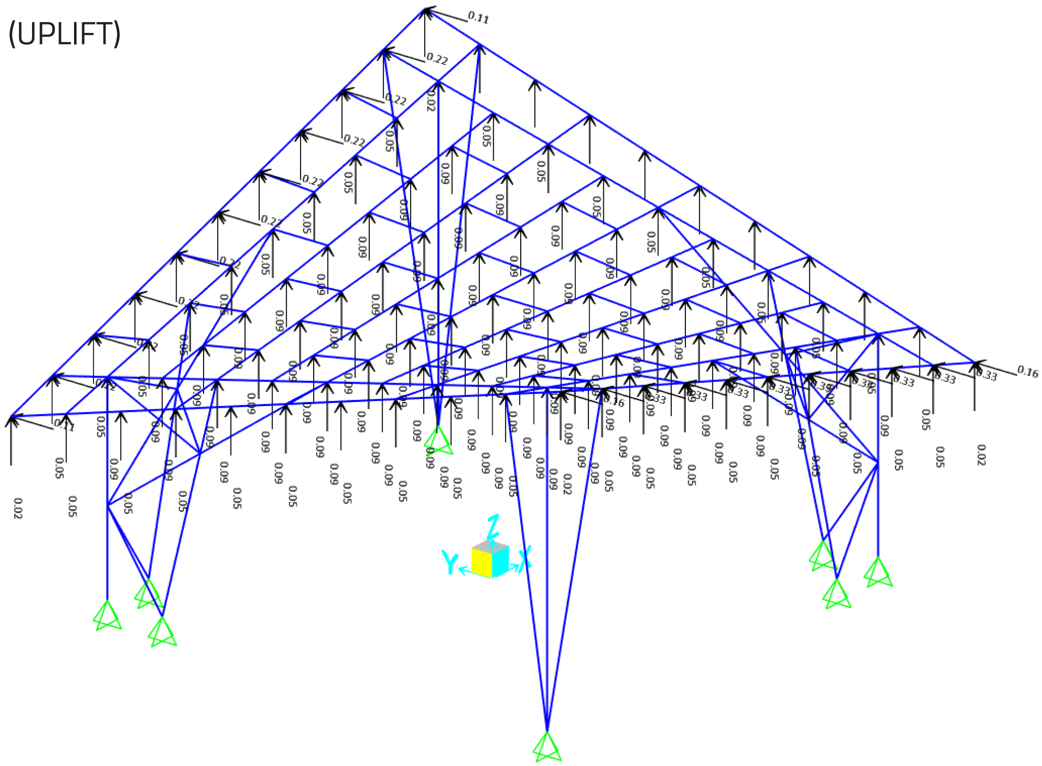


PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** A6

SUBJECT: SAP - LOAD ASSIGNMENTS **DATE:** 11/22/19

WIND (UPLIFT)



WIND (DOWNDRAFT)

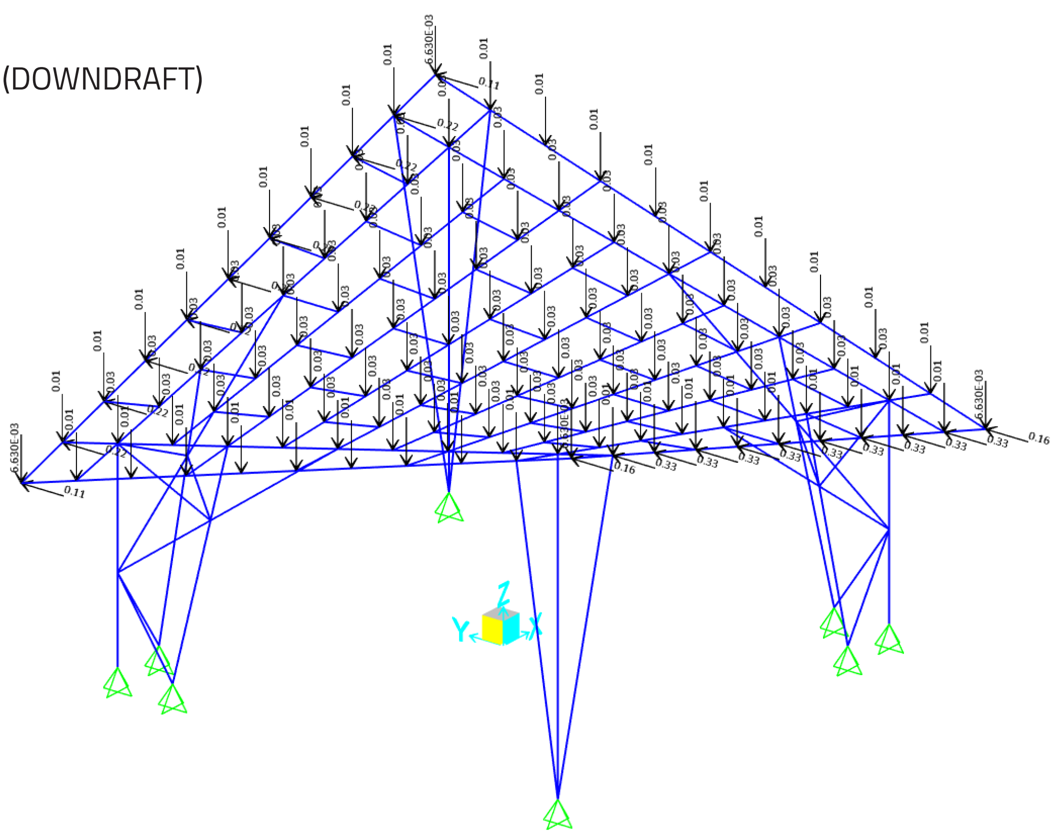


TABLE: Element Forces - Frames									
Frame	Station	OutputCase	P	V2	V3	T	M2	M3	Eq H.1-1
	in		Kip	Kip	Kip	Kip-in	Kip-in	Kip-in	
51	36.00	5a (0.9D+W)	-13.42	0.02	-0.06	0.00	2.30	-0.73	0.317
51	36.00	4a (1.2D+W+0.5Lr)	-12.13	0.01	-0.05	0.00	1.89	-0.52	0.282
51	0.00	5a (0.9D+W)	-13.43	0.02	-0.06	0.00	0.00	0.00	0.277
51	0.00	4a (1.2D+W+0.5Lr)	-12.14	0.01	-0.05	0.00	0.00	0.00	0.251
54	36.00	4b (1.2D+W+0.5Lr)	12.42	0.02	-0.05	0.00	1.96	-0.62	0.237
54	36.00	5b (0.9D+W)	11.07	0.01	-0.04	0.00	1.55	-0.40	0.207
54	0.00	4b (1.2D+W+0.5Lr)	12.41	0.02	-0.05	0.00	0.00	0.00	0.203
54	36.00	3b (1.2D+1.6Lr+0.5W)	9.71	0.03	-0.06	0.00	2.05	-0.89	0.197
54	0.00	5b (0.9D+W)	11.07	0.01	-0.04	0.00	0.00	0.00	0.181
54	0.00	3b (1.2D+1.6Lr+0.5W)	9.70	0.03	-0.06	0.00	0.00	0.00	0.159
51	36.00	5b (0.9D+W)	-6.89	-0.01	-0.01	0.00	0.22	0.31	0.149
51	0.00	5b (0.9D+W)	-6.90	-0.01	-0.01	0.00	0.00	0.00	0.142
51	36.00	4b (1.2D+W+0.5Lr)	-5.60	-0.01	0.01	0.00	-0.19	0.52	0.125
54	36.00	3a (1.2D+1.6Lr+0.5W)	6.30	0.01	-0.03	0.00	1.01	-0.33	0.121
51	0.00	4b (1.2D+W+0.5Lr)	-5.61	-0.01	0.01	0.00	0.00	0.00	0.116
54	0.00	3a (1.2D+1.6Lr+0.5W)	6.29	0.01	-0.03	0.00	0.00	0.00	0.103
54	36.00	4a (1.2D+W+0.5Lr)	5.60	-0.01	0.00	0.00	-0.12	0.49	0.100
54	0.00	4a (1.2D+W+0.5Lr)	5.60	-0.01	0.00	0.00	0.00	0.00	0.092
51	36.00	5a (0.9D+W)	-2.87	-0.01	0.04	-0.21	2.17	-0.31	0.092
51	42.48	5a (0.9D+W)	-2.87	-0.01	0.04	-0.21	1.88	-0.27	0.087
54	36.00	5a (0.9D+W)	4.25	-0.02	0.02	0.00	-0.53	0.72	0.086
51	36.00	4a (1.2D+W+0.5Lr)	-2.34	0.00	0.04	-0.19	1.80	-0.18	0.074
54	36.00	3b (1.2D+1.6Lr+0.5W)	2.59	-0.01	0.04	0.17	1.90	-0.51	0.074
51	42.48	4a (1.2D+W+0.5Lr)	-2.34	0.00	0.04	-0.19	1.56	-0.15	0.071
54	42.48	3b (1.2D+1.6Lr+0.5W)	2.60	-0.01	0.04	0.17	1.65	-0.44	0.070
54	0.00	5a (0.9D+W)	4.25	-0.02	0.02	0.00	0.00	0.00	0.070
54	36.00	4b (1.2D+W+0.5Lr)	2.31	-0.01	0.04	0.19	1.85	-0.26	0.065
52	0.00	3b (1.2D+1.6Lr+0.5W)	-1.26	0.00	0.00	0.00	0.00	0.00	0.064
52	65.52	3b (1.2D+1.6Lr+0.5W)	-1.25	0.00	0.00	0.00	0.00	0.00	0.063
52	131.04	3b (1.2D+1.6Lr+0.5W)	-1.23	0.00	0.00	0.00	0.00	0.00	0.062
54	42.48	4b (1.2D+W+0.5Lr)	2.31	-0.01	0.04	0.19	1.60	-0.22	0.062
51	36.00	3a (1.2D+1.6Lr+0.5W)	-2.71	-0.01	0.00	0.00	-0.13	0.28	0.061
51	84.96	5a (0.9D+W)	-2.86	-0.01	0.04	-0.21	0.00	0.00	0.059
51	0.00	3a (1.2D+1.6Lr+0.5W)	-2.72	-0.01	0.00	0.00	0.00	0.00	0.056
52	0.00	4b (1.2D+W+0.5Lr)	-1.09	0.00	0.00	0.00	0.00	0.00	0.055
52	65.52	4b (1.2D+W+0.5Lr)	-1.08	0.00	0.00	0.00	0.00	0.00	0.055
52	131.04	4b (1.2D+W+0.5Lr)	-1.06	0.00	0.00	0.00	0.00	0.00	0.054
54	36.00	5b (0.9D+W)	1.75	0.00	0.03	0.16	1.47	-0.11	0.049
51	84.96	4a (1.2D+W+0.5Lr)	-2.33	0.00	0.04	-0.19	0.00	0.00	0.048
54	42.48	5b (0.9D+W)	1.76	0.00	0.03	0.16	1.28	-0.10	0.047
51	36.00	3b (1.2D+1.6Lr+0.5W)	1.54	0.01	-0.02	0.05	-1.01	0.58	0.046
51	42.48	3b (1.2D+1.6Lr+0.5W)	1.55	0.01	-0.02	0.05	-0.88	0.50	0.043
54	84.96	3b (1.2D+1.6Lr+0.5W)	2.61	-0.01	0.04	0.17	0.00	0.00	0.043

ΦM_n	76.55 kin
ΦP_{n_T}	61.11 k
$\Phi P_{n_{C1}}$	48.45 k
$\Phi P_{n_{C2}}$	19.7 k

ADDITIONAL NON-GOVERNING RESULTS OMITTED

TABLE: Element Forces - Frames

Frame	Station	OutputCase	P	V2	V3	T	M2	M3	Eq H.1-1
	in		Kip	Kip	Kip	Kip-in	Kip-in	Kip-in	
55	68.41	4b (1.2D+W+0.5Lr)	-6.718	0.013	0.033	0.000	-2.289	-0.763	0.516
57	21.63	4b (1.2D+W+0.5Lr)	-7.756	0.000	0.000	0.000	0.000	0.022	0.493
57	0.00	4b (1.2D+W+0.5Lr)	-7.759	-0.002	0.000	0.000	0.000	0.000	0.492
57	43.27	4b (1.2D+W+0.5Lr)	-7.753	0.002	0.000	0.000	0.000	0.000	0.492
55	49.71	4b (1.2D+W+0.5Lr)	-6.721	0.012	0.033	0.000	-1.664	-0.535	0.491
55	68.41	3b (1.2D+1.6Lr+0.5W)	-6.616	0.015	0.019	0.000	-1.281	-0.960	0.481
55	49.71	3b (1.2D+1.6Lr+0.5W)	-6.619	0.015	0.019	0.000	-0.931	-0.678	0.464
57	21.63	5b (0.9D+W)	-7.267	0.000	0.000	0.000	0.000	0.016	0.461
57	0.00	5b (0.9D+W)	-7.270	-0.002	0.000	0.000	0.000	0.000	0.461
57	43.27	5b (0.9D+W)	-7.265	0.002	0.000	0.000	0.000	0.000	0.461
55	68.41	5b (0.9D+W)	-5.462	0.009	0.033	0.000	-2.237	-0.540	0.430
55	0.00	4b (1.2D+W+0.5Lr)	-6.729	0.010	0.033	0.000	0.000	0.000	0.427
55	0.00	3b (1.2D+1.6Lr+0.5W)	-6.627	0.013	0.019	0.000	0.000	0.000	0.420
55	49.71	5b (0.9D+W)	-5.464	0.008	0.033	0.000	-1.626	-0.378	0.407
90	25.32	5a (0.9D+W)	-5.498	0.000	0.000	0.524	0.000	0.033	0.350
90	0.00	5a (0.9D+W)	-5.500	-0.003	0.000	0.524	0.000	0.000	0.349
90	50.64	5a (0.9D+W)	-5.496	0.003	0.000	0.524	0.000	0.000	0.349
55	0.00	5b (0.9D+W)	-5.470	0.007	0.033	0.000	0.000	0.000	0.347
57	21.63	4a (1.2D+W+0.5Lr)	-5.304	0.000	0.000	0.000	0.000	0.022	0.337
57	0.00	4a (1.2D+W+0.5Lr)	-5.307	-0.002	0.000	0.000	0.000	0.000	0.337
57	43.27	4a (1.2D+W+0.5Lr)	-5.300	0.002	0.000	0.000	0.000	0.000	0.336
90	25.32	4a (1.2D+W+0.5Lr)	-5.184	0.000	0.000	0.430	0.000	0.043	0.330
90	0.00	4a (1.2D+W+0.5Lr)	-5.186	-0.003	0.000	0.430	0.000	0.000	0.329
90	50.64	4a (1.2D+W+0.5Lr)	-5.181	0.003	0.000	0.430	0.000	0.000	0.329
57	21.63	3b (1.2D+1.6Lr+0.5W)	-5.143	0.000	0.000	0.000	0.000	0.022	0.327
57	0.00	3b (1.2D+1.6Lr+0.5W)	-5.147	-0.002	0.000	0.000	0.000	0.000	0.327
57	43.27	3b (1.2D+1.6Lr+0.5W)	-5.140	0.002	0.000	0.000	0.000	0.000	0.326
57	21.63	5a (0.9D+W)	-4.815	0.000	0.000	0.000	0.000	0.016	0.306
57	0.00	5a (0.9D+W)	-4.817	-0.002	0.000	0.000	0.000	0.000	0.306
57	43.27	5a (0.9D+W)	-4.813	0.002	0.000	0.000	0.000	0.000	0.305
55	68.41	3a (1.2D+1.6Lr+0.5W)	-3.453	0.007	0.017	0.000	-1.145	-0.396	0.264
67	68.41	5a (0.9D+W)	7.769	-0.012	-0.030	0.000	2.054	0.926	0.257
58	22.28	4b (1.2D+W+0.5Lr)	-4.001	0.000	0.000	-0.227	0.000	0.038	0.255
58	0.00	4b (1.2D+W+0.5Lr)	-4.003	-0.003	0.000	-0.227	0.000	0.000	0.254
58	44.57	4b (1.2D+W+0.5Lr)	-4.000	0.003	0.000	-0.227	0.000	0.000	0.254
55	49.71	3a (1.2D+1.6Lr+0.5W)	-3.456	0.006	0.017	0.000	-0.832	-0.269	0.252
90	25.32	5b (0.9D+W)	-3.919	0.000	0.000	0.032	0.000	0.033	0.249
57	21.63	3a (1.2D+1.6Lr+0.5W)	-3.917	0.000	0.000	0.000	0.000	0.022	0.249
90	0.00	5b (0.9D+W)	-3.921	-0.003	0.000	0.032	0.000	0.000	0.249
57	0.00	3a (1.2D+1.6Lr+0.5W)	-3.920	-0.002	0.000	0.000	0.000	0.000	0.249
90	50.64	5b (0.9D+W)	-3.917	0.003	0.000	0.032	0.000	0.000	0.249
57	43.27	3a (1.2D+1.6Lr+0.5W)	-3.914	0.002	0.000	0.000	0.000	0.000	0.248
58	22.28	3b (1.2D+1.6Lr+0.5W)	-3.784	0.000	0.000	0.111	0.000	0.038	0.241

ΦMn_3	46.62 kin
ΦMn_2	31.25 kin
ΦPn_C	15.76 k
ΦPn_T	45.4 k

ADDITIONAL NON-GOVERNING RESULTS OMITTED

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON

PAGE: A9

SUBJECT: COLLECTOR FORCES

DATE: 11/22/19

TABLE: Element Forces - Frames								
Frame	Station	OutputCase	P	V2	V3	T	M2	M3
	in		Kip	Kip	Kip	Kip-in	Kip-in	Kip-in
75	98.73	4b (1.2D+W+0.5Lr)	-0.09	-0.02	-0.21	0.07	-9.87	0.11
75	98.73	4b (1.2D+W+0.5Lr)	0.08	0.00	0.22	0.03	-9.84	0.23
75	98.73	5b (0.9D+W)	-0.08	-0.01	-0.21	0.02	-9.84	0.17
75	98.73	5b (0.9D+W)	-0.01	-0.01	0.22	-0.01	-9.81	0.26
75	98.73	4a (1.2D+W+0.5Lr)	-0.39	-0.01	0.21	-0.20	-9.71	0.40
75	98.73	4a (1.2D+W+0.5Lr)	0.00	0.03	-0.21	-0.20	-9.71	0.43
75	98.73	5a (0.9D+W)	-0.48	-0.01	0.21	-0.25	-9.68	0.43
75	98.73	5a (0.9D+W)	0.02	0.04	-0.21	-0.26	-9.68	0.50
75	148.09	5a (0.9D+W)	0.01	0.03	0.08	-0.27	-6.36	-0.85
75	148.09	4a (1.2D+W+0.5Lr)	-0.01	0.02	0.08	-0.21	-6.27	-0.59
75	148.09	5a (0.9D+W)	0.00	0.03	-0.31	-0.28	-6.25	-0.61
75	148.09	4a (1.2D+W+0.5Lr)	-0.01	0.02	-0.31	-0.22	-6.19	-0.41
77	172.77	5a (0.9D+W)	0.07	-0.03	-0.25	-0.15	-6.13	-0.78
75	148.09	5b (0.9D+W)	-0.06	0.00	-0.33	0.03	-5.94	0.44
75	148.09	5b (0.9D+W)	-0.07	-0.01	0.05	0.02	-5.91	0.49
75	148.09	4b (1.2D+W+0.5Lr)	-0.08	0.00	-0.34	0.09	-5.87	0.65
75	148.09	4b (1.2D+W+0.5Lr)	-0.09	-0.01	0.04	0.08	-5.82	0.75
77	172.77	4a (1.2D+W+0.5Lr)	0.06	-0.02	-0.23	-0.08	-5.59	-0.51
75	49.36	4b (1.2D+W+0.5Lr)	0.20	0.05	0.28	0.13	-5.57	-0.54
75	49.36	5b (0.9D+W)	0.08	0.03	0.28	0.09	-5.48	-0.38
75	49.36	4b (1.2D+W+0.5Lr)	0.07	-0.02	-0.04	0.03	-5.26	-0.41
75	49.36	5b (0.9D+W)	-0.02	-0.02	-0.03	-0.02	-5.15	-0.30
75	49.36	4a (1.2D+W+0.5Lr)	-0.41	-0.03	0.28	-0.08	-5.14	0.25
75	49.36	5a (0.9D+W)	-0.53	-0.04	0.29	-0.12	-5.06	0.41
75	98.73	3b (1.2D+1.6Lr+0.5W)	-0.09	-0.04	-0.10	0.17	-5.02	-0.11
76	49.36	4b (1.2D+W+0.5Lr)	-0.57	-0.01	0.20	1.23	-5.02	0.90
75	98.73	3b (1.2D+1.6Lr+0.5W)	0.29	0.00	0.12	0.14	-4.99	0.03
75	98.73	3a (1.2D+1.6Lr+0.5W)	-0.05	-0.01	-0.11	0.04	-4.94	0.05
75	98.73	3a (1.2D+1.6Lr+0.5W)	0.05	0.00	0.11	0.02	-4.92	0.11
75	123.41	4b (1.2D+W+0.5Lr)	-0.09	-0.02	0.04	0.08	-4.79	0.37
76	49.36	5b (0.9D+W)	-0.57	0.00	0.18	0.75	-4.78	0.46
75	49.36	4a (1.2D+W+0.5Lr)	-0.39	-0.01	0.00	-0.19	-4.73	0.13
75	123.41	4b (1.2D+W+0.5Lr)	-0.09	-0.02	-0.21	0.07	-4.72	0.62
75	123.41	5b (0.9D+W)	-0.07	-0.01	0.05	0.02	-4.71	0.28
75	74.05	5a (0.9D+W)	-0.48	0.00	0.00	-0.24	-4.70	0.39
75	123.41	5b (0.9D+W)	-0.08	-0.01	-0.21	0.02	-4.67	0.43
75	74.05	4a (1.2D+W+0.5Lr)	-0.38	-0.01	0.00	-0.19	-4.65	0.33
75	49.36	5a (0.9D+W)	-0.48	-0.01	0.00	-0.24	-4.63	0.24
75	74.05	5a (0.9D+W)	-0.48	-0.02	0.21	-0.25	-4.61	0.10
75	74.05	4a (1.2D+W+0.5Lr)	-0.39	-0.01	0.21	-0.20	-4.58	0.09
77	24.68	4b (1.2D+W+0.5Lr)	0.41	-0.01	0.19	0.08	-4.56	0.28
75	74.05	5b (0.9D+W)	-0.01	-0.01	0.22	-0.01	-4.46	0.08
75	123.41	4a (1.2D+W+0.5Lr)	0.00	0.03	-0.21	-0.20	-4.46	-0.32
76	98.73	4b (1.2D+W+0.5Lr)	-0.70	0.06	0.15	0.17	-4.44	-1.01

ADDITIONAL NON-GOVERNING RESULTS OMITTED

TABLE: Joint Reactions							
Joint	OutputCase	F1	F2	F3	M1	M2	M3
Text	Text	Kip	Kip	Kip	Kip-in	Kip-in	Kip-in
97	3 (D+Lr)	0.013	-0.025	-2.837	0.000	0.000	0.000
99	3 (D+Lr)	0.000	0.000	0.633	0.000	0.000	0.000
101	3 (D+Lr)	0.000	0.000	0.615	0.000	0.000	0.000
103	3 (D+Lr)	-0.014	0.025	-2.958	0.000	0.000	0.000
105	3 (D+Lr)	-0.002	0.078	3.592	0.000	0.000	0.000
110	3 (D+Lr)	-0.086	-0.003	1.519	0.000	0.000	0.000
112	3 (D+Lr)	0.087	0.003	1.382	0.000	0.000	0.000
115	3 (D+Lr)	0.002	-0.078	3.592	0.000	0.000	0.000
97	5b (D+0.6W)	0.007	0.001	3.844	0.000	0.000	0.000
99	5b (D+0.6W)	0.000	0.000	0.572	0.000	0.000	0.000
101	5b (D+0.6W)	0.000	0.000	0.077	0.000	0.000	0.000
103	5b (D+0.6W)	-0.008	0.028	-6.945	0.000	0.000	0.000
105	5b (D+0.6W)	-0.020	-1.615	7.200	0.000	0.000	0.000
110	5b (D+0.6W)	-0.023	0.006	0.706	0.000	0.000	0.000
112	5b (D+0.6W)	0.060	0.008	0.952	0.000	0.000	0.000
115	5b (D+0.6W)	-0.015	-1.689	-3.279	0.000	0.000	0.000
97	6b (D+0.75(0.6W)+0.75Lr)	0.013	-0.015	1.074	0.000	0.000	0.000
99	6b (D+0.75(0.6W)+0.75Lr)	0.000	0.000	0.812	0.000	0.000	0.000
101	6b (D+0.75(0.6W)+0.75Lr)	0.000	0.000	0.430	0.000	0.000	0.000
103	6b (D+0.75(0.6W)+0.75Lr)	-0.015	0.037	-7.096	0.000	0.000	0.000
105	6b (D+0.75(0.6W)+0.75Lr)	-0.016	-1.161	7.674	0.000	0.000	0.000
110	6b (D+0.75(0.6W)+0.75Lr)	-0.073	0.003	1.463	0.000	0.000	0.000
112	6b (D+0.75(0.6W)+0.75Lr)	0.101	0.008	1.559	0.000	0.000	0.000
115	6b (D+0.75(0.6W)+0.75Lr)	-0.010	-1.317	-0.185	0.000	0.000	0.000
97	7a (0.6D+0.6W)	-0.012	0.038	8.020	0.000	0.000	0.000
99	7a (0.6D+0.6W)	0.000	0.000	-0.303	0.000	0.000	0.000
101	7a (0.6D+0.6W)	0.000	0.000	-0.771	0.000	0.000	0.000
103	7a (0.6D+0.6W)	0.012	-0.009	-2.589	0.000	0.000	0.000
105	7a (0.6D+0.6W)	-0.017	-1.731	1.959	0.000	0.000	0.000
110	7a (0.6D+0.6W)	0.105	0.010	-1.433	0.000	0.000	0.000
112	7a (0.6D+0.6W)	-0.070	0.003	-0.981	0.000	0.000	0.000
115	7a (0.6D+0.6W)	-0.018	-1.573	-8.521	0.000	0.000	0.000

Legend
Overturning
Uplift
Bearing
Sliding
Column Bearing
Column Uplift

TABLE: Joint Reactions

Joint	OutputCase	F1	F2	F3	M1	M2	M3
Text	Text	Kip	Kip	Kip	Kip-in	Kip-in	Kip-in
97	1 (1.4D)	0.004	-0.008	-0.893	0.000	0.000	0.000
99	1 (1.4D)	0.000	0.000	0.257	0.000	0.000	0.000
101	1 (1.4D)	0.000	0.000	0.250	0.000	0.000	0.000
103	1 (1.4D)	-0.005	0.008	-0.927	0.000	0.000	0.000
105	1 (1.4D)	0.000	0.023	1.176	0.000	0.000	0.000
110	1 (1.4D)	-0.026	-0.001	0.575	0.000	0.000	0.000
112	1 (1.4D)	0.026	0.001	0.537	0.000	0.000	0.000
115	1 (1.4D)	0.000	-0.023	1.177	0.000	0.000	0.000
97	3b (1.2D+1.6Lr+0.5W)	0.022	-0.032	-0.550	0.000	0.000	0.000
99	3b (1.2D+1.6Lr+0.5W)	0.000	0.000	1.263	0.000	0.000	0.000
101	3b (1.2D+1.6Lr+0.5W)	0.000	0.000	0.827	0.000	0.000	0.000
103	3b (1.2D+1.6Lr+0.5W)	-0.025	0.057	-9.704	0.000	0.000	0.000
105	3b (1.2D+1.6Lr+0.5W)	-0.019	-1.241	10.711	0.000	0.000	0.000
110	3b (1.2D+1.6Lr+0.5W)	-0.135	0.001	2.512	0.000	0.000	0.000
112	3b (1.2D+1.6Lr+0.5W)	0.166	0.010	2.531	0.000	0.000	0.000
115	3b (1.2D+1.6Lr+0.5W)	-0.011	-1.512	1.979	0.000	0.000	0.000
97	4b (1.2D+W+0.5Lr)	0.014	-0.005	5.605	0.000	0.000	0.000
99	4b (1.2D+W+0.5Lr)	0.000	0.000	1.093	0.000	0.000	0.000
101	4b (1.2D+W+0.5Lr)	0.000	0.000	0.263	0.000	0.000	0.000
103	4b (1.2D+W+0.5Lr)	-0.017	0.054	-12.414	0.000	0.000	0.000
105	4b (1.2D+W+0.5Lr)	-0.033	-2.668	12.984	0.000	0.000	0.000
110	4b (1.2D+W+0.5Lr)	-0.064	0.009	1.539	0.000	0.000	0.000
112	4b (1.2D+W+0.5Lr)	0.125	0.013	1.907	0.000	0.000	0.000
115	4b (1.2D+W+0.5Lr)	-0.025	-2.838	-4.481	0.000	0.000	0.000
97	5a (0.9D+W)	-0.020	0.064	13.431	0.000	0.000	0.000
99	5a (0.9D+W)	0.000	0.000	-0.523	0.000	0.000	0.000
101	5a (0.9D+W)	0.000	0.000	-1.303	0.000	0.000	0.000
103	5a (0.9D+W)	0.020	-0.015	-4.248	0.000	0.000	0.000
105	5a (0.9D+W)	-0.029	-2.887	3.182	0.000	0.000	0.000
110	5a (0.9D+W)	0.177	0.017	-2.430	0.000	0.000	0.000
112	5a (0.9D+W)	-0.118	0.005	-1.673	0.000	0.000	0.000
115	5a (0.9D+W)	-0.030	-2.619	-14.285	0.000	0.000	0.000

Overturning
Sliding

Column Bearing
Column Uplift

ADDITIONAL NON-GOVERNING RESULTS OMITTED

TABLE: Element Forces - Frames

Frame	Station	OutputCase	P	V2	V3	T	M2	M3	Eq H.1-1
	in		Kip	Kip	Kip	Kip-in	Kip-in	Kip-in	
69	43.27	5a (0.9D+W)	8.127	0.002	0.000	0.000	0.000	0.000	0.179
69	21.63	5a (0.9D+W)	8.125	0.000	0.000	0.000	0.000	0.016	0.179
69	0.00	5a (0.9D+W)	8.123	-0.002	0.000	0.000	0.000	0.000	0.179
67	68.41	5a (0.9D+W)	7.769	-0.012	-0.030	0.000	2.054	0.926	0.257
67	49.71	5a (0.9D+W)	7.766	-0.013	-0.030	0.000	1.492	0.687	0.234
67	0.00	5a (0.9D+W)	7.760	-0.015	-0.030	0.000	0.000	0.000	0.171
69	43.27	4a (1.2D+W+0.5Lr)	7.640	0.002	0.000	0.000	0.000	0.000	0.168
69	21.63	4a (1.2D+W+0.5Lr)	7.637	0.000	0.000	0.000	0.000	0.022	0.169
69	0.00	4a (1.2D+W+0.5Lr)	7.634	-0.002	0.000	0.000	0.000	0.000	0.168
67	68.41	4a (1.2D+W+0.5Lr)	6.513	-0.009	-0.029	0.000	2.000	0.709	0.223
67	49.71	4a (1.2D+W+0.5Lr)	6.510	-0.010	-0.029	0.000	1.454	0.534	0.201
67	0.00	4a (1.2D+W+0.5Lr)	6.502	-0.012	-0.029	0.000	0.000	0.000	0.143
69	43.27	5b (0.9D+W)	5.676	0.002	0.000	0.000	0.000	0.000	0.125
69	21.63	5b (0.9D+W)	5.674	0.000	0.000	0.000	0.000	0.016	0.125
69	0.00	5b (0.9D+W)	5.671	-0.002	0.000	0.000	0.000	0.000	0.125
85	0.00	4b (1.2D+W+0.5Lr)	5.265	-0.003	0.000	-0.453	0.000	0.000	0.116
85	25.32	4b (1.2D+W+0.5Lr)	5.262	0.000	0.000	-0.453	0.000	0.043	0.117
85	50.64	4b (1.2D+W+0.5Lr)	5.260	0.003	0.000	-0.453	0.000	0.000	0.116
69	43.27	4b (1.2D+W+0.5Lr)	5.188	0.002	0.000	0.000	0.000	0.000	0.114
69	21.63	4b (1.2D+W+0.5Lr)	5.185	0.000	0.000	0.000	0.000	0.022	0.115
69	0.00	4b (1.2D+W+0.5Lr)	5.182	-0.002	0.000	0.000	0.000	0.000	0.114
85	0.00	5b (0.9D+W)	4.950	-0.003	0.000	-0.358	0.000	0.000	0.109
85	25.32	5b (0.9D+W)	4.948	0.000	0.000	-0.358	0.000	0.033	0.110
85	50.64	5b (0.9D+W)	4.946	0.003	0.000	-0.358	0.000	0.000	0.109
70	44.57	5a (0.9D+W)	4.581	0.003	0.000	0.089	0.000	0.000	0.101
70	22.28	5a (0.9D+W)	4.580	0.000	0.000	0.089	0.000	0.029	0.102
70	0.00	5a (0.9D+W)	4.579	-0.003	0.000	0.089	0.000	0.000	0.101
70	44.57	4a (1.2D+W+0.5Lr)	3.893	0.003	0.000	0.172	0.000	0.000	0.086
70	22.28	4a (1.2D+W+0.5Lr)	3.892	0.000	0.000	0.172	0.000	0.038	0.087
70	0.00	4a (1.2D+W+0.5Lr)	3.890	-0.003	0.000	0.172	0.000	0.000	0.086
85	0.00	4a (1.2D+W+0.5Lr)	3.685	-0.003	0.000	0.044	0.000	0.000	0.081
85	25.32	4a (1.2D+W+0.5Lr)	3.683	0.000	0.000	0.044	0.000	0.043	0.082
85	50.64	4a (1.2D+W+0.5Lr)	3.680	0.003	0.000	0.044	0.000	0.000	0.081
85	0.00	3b (1.2D+1.6Lr+0.5W)	3.449	-0.003	0.000	-0.476	0.000	0.000	0.076
85	25.32	3b (1.2D+1.6Lr+0.5W)	3.447	0.000	0.000	-0.476	0.000	0.043	0.077
85	50.64	3b (1.2D+1.6Lr+0.5W)	3.444	0.003	0.000	-0.476	0.000	0.000	0.076
85	0.00	5a (0.9D+W)	3.370	-0.003	0.000	0.139	0.000	0.000	0.074
85	25.32	5a (0.9D+W)	3.368	0.000	0.000	0.139	0.000	0.033	0.075
85	50.64	5a (0.9D+W)	3.366	0.003	0.000	0.139	0.000	0.000	0.074
84	0.00	5a (0.9D+W)	3.347	-0.004	0.000	1.857	0.000	0.000	0.074
84	34.68	5a (0.9D+W)	3.345	0.000	0.000	1.857	0.000	0.061	0.075
84	69.36	5a (0.9D+W)	3.342	0.004	0.000	1.857	0.000	0.000	0.074
85	0.00	3a (1.2D+1.6Lr+0.5W)	2.659	-0.003	0.000	-0.227	0.000	0.000	0.059

ΦMn_3	46.62 kin
ΦMn_2	31.25 kin
ΦPn_C	15.76 k
ΦPn_T	45.4 k

TABLE: Element Joint Forces - Frames								
Frame	Joint	OutputCase	F1	F2	F3	M1	M2	M3
Text	Text	Text	Kip	Kip	Kip	Kip-in	Kip-in	Kip-in
51	98	1 (1.4D)	0.00	-0.01	0.34	0.00	0.00	0.01
51	98	3a (1.2D+1.6Lr+0.5W)	0.01	0.00	0.21	0.00	0.00	-0.02
51	98	3b (1.2D+1.6Lr+0.5W)	0.01	-0.02	1.55	0.00	0.00	0.05
51	98	4a (1.2D+W+0.5Lr)	0.00	0.04	-2.33	0.00	0.00	-0.19
51	98	4b (1.2D+W+0.5Lr)	0.01	0.00	0.37	0.00	0.00	-0.04
51	98	5a (0.9D+W)	-0.01	0.04	-2.86	0.00	0.00	-0.21
51	98	5b (0.9D+W)	0.01	0.01	-0.16	0.00	0.00	-0.07
66	98	1 (1.4D)	0.17	0.00	-0.07	0.00	0.00	0.00
66	98	3a (1.2D+1.6Lr+0.5W)	0.45	0.00	-0.20	0.42	0.00	-0.19
66	98	3b (1.2D+1.6Lr+0.5W)	0.98	0.00	-0.45	0.45	0.00	-0.21
66	98	4a (1.2D+W+0.5Lr)	-0.21	0.00	0.10	0.77	0.00	-0.35
66	98	4b (1.2D+W+0.5Lr)	0.85	0.00	-0.39	0.84	0.00	-0.39
66	98	5a (0.9D+W)	-0.42	0.00	0.20	0.76	0.00	-0.35
66	98	5b (0.9D+W)	0.64	0.00	-0.29	0.83	0.00	-0.38
70	98	1 (1.4D)	0.00	-0.43	-0.19	0.00	0.04	0.02
70	98	3a (1.2D+1.6Lr+0.5W)	0.00	0.15	0.07	0.00	0.27	0.13
70	98	3b (1.2D+1.6Lr+0.5W)	0.00	-1.44	-0.66	0.00	0.47	0.22
70	98	4a (1.2D+W+0.5Lr)	0.00	3.54	1.63	0.00	0.16	0.07
70	98	4b (1.2D+W+0.5Lr)	0.00	0.37	0.17	0.00	0.55	0.25
70	98	5a (0.9D+W)	0.00	4.16	1.91	0.00	0.08	0.04
70	98	5b (0.9D+W)	0.00	1.00	0.46	0.00	0.47	0.22
54	104	1 (1.4D)	0.00	0.01	0.36	0.00	0.00	0.01
54	104	3a (1.2D+1.6Lr+0.5W)	0.00	0.02	1.18	0.00	0.00	0.09
54	104	3b (1.2D+1.6Lr+0.5W)	-0.01	0.04	2.61	0.00	0.00	0.17
54	104	4a (1.2D+W+0.5Lr)	0.01	0.00	-0.53	0.00	0.00	0.03
54	104	4b (1.2D+W+0.5Lr)	-0.01	0.04	2.32	0.00	0.00	0.19
54	104	5a (0.9D+W)	0.01	-0.01	-1.09	0.00	0.00	0.00
54	104	5b (0.9D+W)	0.00	0.03	1.76	0.00	0.00	0.16
58	104	1 (1.4D)	0.00	0.43	-0.19	0.00	-0.05	0.02
58	104	3a (1.2D+1.6Lr+0.5W)	0.00	1.86	-0.85	0.00	0.10	-0.05
58	104	3b (1.2D+1.6Lr+0.5W)	0.00	3.44	-1.57	0.00	-0.10	0.05
58	104	4a (1.2D+W+0.5Lr)	0.00	0.48	-0.21	0.00	0.61	-0.28
58	104	4b (1.2D+W+0.5Lr)	0.00	3.64	-1.66	0.00	0.21	-0.10
58	104	5a (0.9D+W)	0.00	-0.15	0.07	0.00	0.69	-0.32
58	104	5b (0.9D+W)	0.00	3.01	-1.38	0.00	0.29	-0.13
62	104	1 (1.4D)	-0.18	0.00	-0.08	0.00	0.00	0.00
62	104	3a (1.2D+1.6Lr+0.5W)	-0.18	0.00	-0.08	0.48	0.00	0.22
62	104	3b (1.2D+1.6Lr+0.5W)	-0.75	0.00	-0.34	0.45	0.00	0.20
62	104	4a (1.2D+W+0.5Lr)	0.81	0.00	0.37	1.02	0.00	0.47
62	104	4b (1.2D+W+0.5Lr)	-0.32	0.00	-0.14	0.95	0.00	0.44
62	104	5a (0.9D+W)	1.03	0.00	0.48	1.03	0.00	0.47
62	104	5b (0.9D+W)	-0.09	0.00	-0.04	0.96	0.00	0.44
87	1	1 (1.4D)	0.18	0.00	-0.13	-0.01	0.00	0.01
87	1	3a (1.2D+1.6Lr+0.5W)	0.21	0.00	-0.15	-0.04	0.00	0.03

Shear
Tension
Compression

ADDITIONAL NON-GOVERNING RESULTS OMITTED

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** A14

SUBJECT: ANCHOR PULLOUT FORCES **DATE:** 11/22/19

American Earth Anchors
The best screw you will have in the dirt™

LOAD CAPACITY

Penetrators

PENETRATORS®

LOAD CAPACITY Pounds

Field-tested pullout resistance

SOIL CLASSIFICATION per ASTM D-2487/2488	9" PE9 PE-T9	10" PE10	14" 18" PE14 PE14-STD PE18 PE18-SQ	26" PE26	36" PE36	46" PE46-Hex PE46-Hex8 PE46-Guy
1 Hardpan Asphalt	400	1,000	2,500	4,500	8,400	14,000
2 Sandy gravel Very dense sand	200	700	1,700	3,100	6,000	9,500
3 Silty or clayey sand Silty gravel	100	350	600	1,100	2,100	3,300
4 Loose to med dense sands Loose sands Firm clays	Less than 100	200	350	630	1,000	2,000
5 Uncompacted fill	Less than 100	100	200	360	550	750

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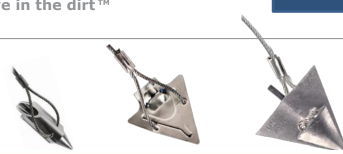
The best screw you will have in the dirt™

LOAD CAPACITY

Cabled Anchors
Bullets and Arrowheads



CABLED ANCHORS

LOAD CAPACITY Pounds

Field-tested pullout resistance

SOIL CLASSIFICATION per ASTM D-2487/2488	Bullets: Thimble loop or cable clamps	Bullets: Quickvise or disk + Quickvise	Steel arrowheads: Thimble loop or cable clamps	Steel arrowheads: Quickvise or disk + Quickvise	Steel arrowheads: Thimble loop or cable clamps	Steel arrowheads: Quickvise or disk + Quickvise	Aluminum mil-spec arrowheads: Thimble loop or cable clamps	Aluminum mil-spec arrowheads: Thimble loop or cable clamps	Aluminum mil-spec arrowheads: Thimble loop or cable clamps	Aluminum mil-spec arrowheads: Thimble loop or cable clamps
Minimum depth	2'	2'	2'	2'	2.5'	2.5'	2.5'	3.5'	4'	5'
Size	3"	3"	3"	3"	4"	4"	4"	6"	8"	10"
1 Hardpan Asphalt	2,000 lb	1,100 lb	2,000 lb	1,100 lb	3,500 lb	3,150 lb	3,500 lb	5,000 lb	9,000 lb	14,000 lb
2 Sandy gravel Very dense sand	1,800 lb	1,100 lb	1,800 lb	1,100 lb	2,200 lb	2,200 lb	2,200 lb	3,000 lb	6,500 lb	11,000 lb
3 Silty or clayey sand Silty gravel	1,700 lb	1,100 lb	1,700 lb	1,100 lb	1,900 lb	1,900 lb	1,900 lb	2,000 lb	3,500 lb	7,000 lb
4 Loose to med dense sands Loose sands Firm clays	600 lb	600 lb	600 lb	600 lb	900 lb	900 lb	900 lb	1,200 lb	2,200 lb	4,000 lb
5 Uncompacted fill	350 lb	350 lb	350 lb	350 lb	475 lb	475 lb	475 lb	600 lb	1,250 lb	2,400 lb

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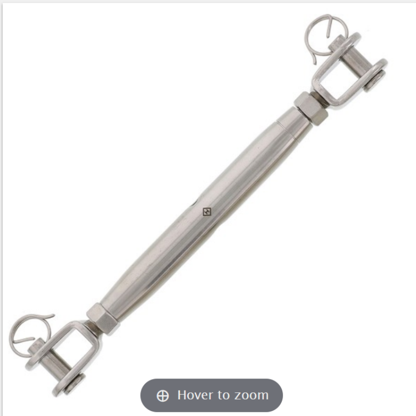
PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** A15

SUBJECT: TURNBUCKLE DESIGN VALUES **DATE:** 11/22/19



HAAS stainless

3/8" x 4-1/4" Stainless Steel Pipe Style Jaw x Jaw Turnbuckle

Part No.: 51609220

$20.77

Jaw x Jaw Turnbuckle, Pipe Style - 3/8 inch x 4-1/4 inch size, Stainless Steel, Type 316. Working Load Limit (WLL) : 1900 lbs.

Size

3/16" x 3" | 1/4" x 3-1/4" | 5/16" x 3-9/16"

3/8" x 4-1/4" | 1/2" x 5 | Catalog

Features

Material: Type 316 Stainless Steel

Weight: 0.52 lb

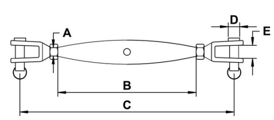
Working Load Limit (WLL): 1900 lbs

Size:3/8 inches

Length:4-1/4 inches

STAINLESS STEEL PIPE STYLE JAW X JAW TURNBUCKLES

3/16 inch x 1-15/16 inch - 1/2 inch x 5 inch



Dimensions are in inches unless otherwise noted.

Thread Diameter (A) x Take Up (B)	Closed Pin to Pin (C)	Pin Diameter (D)	Inside Jaw Width (E)	WLL (lbs)	Weight (lbs)
3/16" x 3"	5.12	0.20	0.26	400	0.100
1/4" x 3-1/4"	5.90	0.24	0.30	600	0.160
5/16" x 3-9/16"	6.50	0.31	0.43	1320	0.300
3/8" x 4-1/4"	7.48	0.35	0.47	1900	0.520
1/2" x 5"	9.65	0.47	0.55	2850	1.000

PROJECT: WINE HISTORY PAVILION

PREPARED BY: ISAAC CAMERON **PAGE:** A16

SUBJECT: HEX BOLT DESIGN VALUES **DATE:** 11/22/19

McMASTER-CARR

bolts

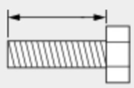
Clear All

System of Measurement
- Inch
- Metric

Thread Size
- 1/4"-20
- 5/16"-18
- 3/8"-16
- 1/2"-13
- 5/8"-11
- M6
- M8
- M10

Material
- Aluminum

Length

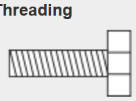


- 1/2"
- 3/4"
- 1"
- 1 1/4"
- 1 1/2"
- 2"
- 2 1/2"
- 3"
- 3 1/2"
- 4"
- 16mm
- 20mm
- 25mm
- 30mm
- 40mm
- 50mm

Thread Type
- UNC
- Metric

Thread Fit
- Class 2A
- Class 6g

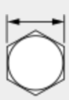
Threading



- Fully Threaded

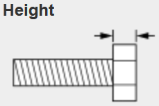
Drive Style
- External Hex

Head Width



- 7/16"
- 1/2"
- 9/16"
- 3/4"
- 15/16"
- 10mm
- 13mm
- 17mm

Head Height



- 5/32"
- 13/64"
- 15/64"
- 5/16"
- 25/64"
- 4mm
- 5.3mm
- 6.4mm

Thread Spacing
- Coarse

47 Products

About Hex Head Screws and Bolts

More

Aluminum Hex Head Screws

Aluminum screws are one-third the weight of steel and have good corrosion resistance. Length is measured from under the head.

Screws that meet **ISO 4017** (formerly DIN 933) comply with specifications for dimensional standards.

For technical drawings and 3-D models, click on a part number.

Lg.	Threading	Thread Spacing	Head Wd.	Head Ht.	Tensile Strength, psi	Pkg. Qty.		Pkg.
1/4"-20								
1/2"	Fully Threaded	Coarse	7/16"	5/32"	30,000	25	93306A538	$12.28
3/4"	Fully Threaded	Coarse	7/16"	5/32"	30,000	25	93306A540	13.35
1"	Fully Threaded	Coarse	7/16"	5/32"	30,000	25	93306A542	13.67
1 1/4"	Fully Threaded	Coarse	7/16"	5/32"	30,000	25	93306A543	14.79
1 1/2"	Fully Threaded	Coarse	7/16"	5/32"	30,000	10	93306A546	7.87
2"	Fully Threaded	Coarse	7/16"	5/32"	30,000	10	93306A550	9.82
2 1/2"	Fully Threaded	Coarse	7/16"	5/32"	30,000	10	93306A555	12.34
3"	Fully Threaded	Coarse	7/16"	5/32"	30,000	10	93306A558	13.69
5/16"-18								
3/4"	Fully Threaded	Coarse	1/2"	13/64"	30,000	10	93306A580	9.24
1"	Fully Threaded	Coarse	1/2"	13/64"	30,000	10	93306A583	8.84
1 1/4"	Fully Threaded	Coarse	1/2"	13/64"	30,000	10	93306A585	10.00
1 1/2"	Fully Threaded	Coarse	1/2"	13/64"	30,000	10	93306A587	11.48
2"	Fully Threaded	Coarse	1/2"	13/64"	30,000	5	93306A560	8.28
2 1/2"	Fully Threaded	Coarse	1/2"	13/64"	30,000	5	93306A562	10.21
3"	Fully Threaded	Coarse	1/2"	13/64"	30,000	5	93306A565	10.96
3/8"-16								
3/4"	Fully Threaded	Coarse	9/16"	15/64"	30,000	10	93306A622	10.04
1"	Fully Threaded	Coarse	9/16"	15/64"	30,000	10	93306A624	10.58
1 1/4"	Fully Threaded	Coarse	9/16"	15/64"	30,000	10	93306A626	12.33
1 1/2"	Fully Threaded	Coarse	9/16"	15/64"	30,000	10	93306A628	12.61
2"	Fully Threaded	Coarse	9/16"	15/64"	30,000	10	93306A632	16.51
3"	Fully Threaded	Coarse	9/16"	15/64"	30,000	5	93306A635	11.23
1/2"-13								
1"	Fully Threaded	Coarse	3/4"	5/16"	30,000	5	93306A712	12.21
1 1/2"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A716	3.05
2"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A720	3.36
2 1/2"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A722	3.89
3"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A725	4.21
3 1/2"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A728	4.39
4"	Fully Threaded	Coarse	3/4"	5/16"	30,000	1	93306A730	5.62
5/8"-11								
1 1/2"	Fully Threaded	Coarse	15/16"	25/64"	30,000	1	93306A805	4.28
2"	Fully Threaded	Coarse	15/16"	25/64"	30,000	1	93306A808	5.31
2 1/2"	Fully Threaded	Coarse	15/16"	25/64"	30,000	1	93306A811	6.02
3"	Fully Threaded	Coarse	15/16"	25/64"	30,000	1	93306A814	7.06

Lg., mm	Threading	Thread Spacing	Head, mm Wd.	Head, mm Ht.	Tensile Strength, psi	Specifications Met		Each
M6 × 1 mm								
16	Fully Threaded	Coarse	10	4	30,000	ISO 4017, DIN 933	92722A111	$2.80
20	Fully Threaded	Coarse	10	4	30,000	ISO 4017, DIN 933	92722A112	3.32
25	Fully Threaded	Coarse	10	4	30,000	ISO 4017, DIN 933	92722A113	3.08
30	Fully Threaded	Coarse	10	4	30,000	ISO 4017, DIN 933	92722A114	3.40
40	Fully Threaded	Coarse	10	4	30,000	ISO 4017, DIN 933	92722A115	3.62
M8 × 1.25 mm								
16	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A116	3.70
20	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A117	4.60
25	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A118	5.20
30	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A119	5.64
40	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A120	5.40
50	Fully Threaded	Coarse	13	5.3	30,000	ISO 4017, DIN 933	92722A121	6.96
M10 × 1.5 mm								
20	Fully Threaded	Coarse	17	6.4	30,000	ISO 4017, DIN 933	92722A122	7.04
25	Fully Threaded	Coarse	17	6.4	30,000	ISO 4017, DIN 933	92722A123	14.24
30	Fully Threaded	Coarse	17	6.4	30,000	ISO 4017, DIN 933	92722A124	12.80
40	Fully Threaded	Coarse	17	6.4	30,000	ISO 4017, DIN 933	92722A125	17.00