

12/30/2014

Sapsis Rigging Inc. 223 N. Landsdowne Avenue N. Landsdowne, PA 19050 Attn: Bill Sapsis

RE: Chain Hoist Stand

CRE Proj. No.: 14.319.04

Dear Bill,

Per your request, we have reviewed the Chain Hoist Stand. Attached are the plans and analysis for the structure. Attached are the plans and analysis for the structure. Our review has been performed in accordance with the structural provisions of the 2012 International Building Code and 2014 AISC Manual. The maximum load applied to the system is no larger than 1 ton.

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We trust this information is suitable for your needs at this time. If you have any questions, please do not hesitate to contact our office.

Regards,

Clark-Reder Engineering, Inc.

Scott Horn, E.I.T.



Jeffrey M. Reder, P.E.



#### **GENERAL STRUCTURAL NOTES**

#### CODES AND REFERENCE

- 1. 2012 INTERNATIONAL BUILDING CODE
- 2. ASCE 7-10 MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES
- 3. AISC STEEL MANUAL, 14<sup>TH</sup> EDITION

#### **DESIGN LOADS**

- 1. DEAD LOAD: SELFWEIGHT OF STRUCTURE
- 2. RIGGING LOADS: MAXIMUM LOAD SUPPORTED BY CHAIN MOTOR OR PEAR RING = 2200#

### STRUCTURAL STEEL

- 1. STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING UNLESS NOTED OTHERWISE ON THE DRAWINGS:
  - A. ROLLED WIDE FLANGE SHAPES: ASTM A992, FY = 50 KSI
  - B. MISC PLATE, BAR, ANGLES AND CHANNELS: ASTM A36, FY = 36 KSI
  - C. PIPE SHAPES: ASTM A53, TYPE E OR S, GRADE B, FY = 35 KSI
  - D. HSS TUBES: ASTM A500 GR B, FY = 46 KSI
  - E. HSS ROUND: ASTM A500 GR B, FY = 42KSI
  - F. BOLTS OR SCAFFOLD CONNECTION PINS: SAE J429 GRADE 5 BOLTS (FY=92 KSI)
  - G. TRUSS TO TRUSS CONNECTION PINS: A449
- 2. WELDING SHALL BE IN ACCORDANCE WITH THE AMERICAN WELDING SOCIETY LATEST EDITION.

#### **INSPECTIONS**

1. DAMAGED OR CORRODED EQUIPMENT SHALL NOT BE USED. FIELD MODIFICATIONS SHALL BE APPROVED BY THE ENGINEER OF RECORD PRIOR TO INSTALLATION.



#### Chain Hoist Stand

#### **Codes and Referenced Standards**

- 2012 International Building Code
- American Institute of Steel Construction, Steel Construction Manual 14th Edition
- American Society of Civil Engineers 7-10 (ASCE 7-10) "Minimum Design Loads for Buildings and Other Structures"

#### **Project Description**

The chain hoist stand, composed of steel members, is to support a total load of 1-Ton (2200#). There are (2) possible load points for the stand, one from a motor, and one from a pear ring. A lateral load will also be considered on the stand.

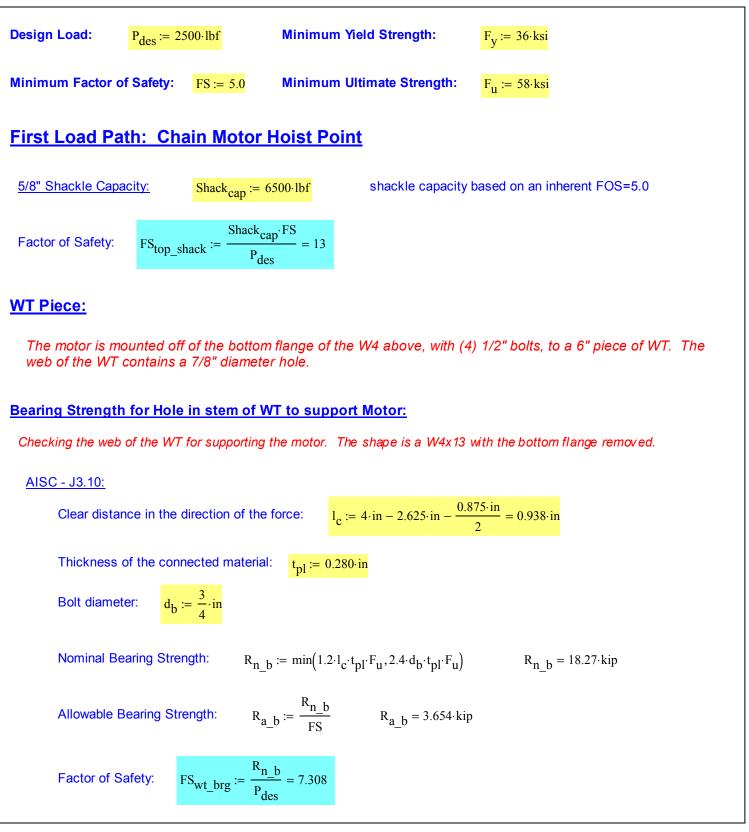
The top most beam is a W4x13, measuring 3'-0". Located in the center of the beam is a 1-Ton capacity, non-inverted, CM Loadstar Chain Motor. It is mounted to a T shape hoist point, 6" long, on a 4" wide by 4" deep member, 1/2" thick. The top beam is supported on (2) HSS3x3x1/4 tubes, measuring 3'-7-1/2" high each. Four angle braces, L2x2x3/16 members, are located 3'-0-1/2" high up the HSS members, and 1'-3" along the bottom base members. They measure 3'-1" long each. Attachment points for the braces are all L2x2x3/16 pieces. The base members, and leg cross braces, are C3x4.1 channels, measuring 5'-0" long each, one per side. Grade 5 bolts, measuring 3/8" or 1/2", are used throughout.

A 5/8" pear ring, supported on (2) 3/8" 7x19 aircraft cables and 5/8" shackles, is centered on the hoist stand as well. The cables are tied to a 3/8" gusset plate, 4" by 4". A 13/16" hole is located in the center of the plate to hold the pin for the attached shackles. These gusset plates are welded to the column tubes and the plates at the top, the measure 10" by 4", 3/8" thick steel, with (4) bolt holes.

#### Analysis Assumptions/Design Criteria

- All steel is A36 minimum, unless otherwise noted below.
- The minimum Factor of Safety for all pieces and members is 5.0, unless otherwise noted.
- The maximum load considered to be supported by the system is 2500#.
- A 10% of the maximum load (250#), applied as a lateral load, will also be considered.





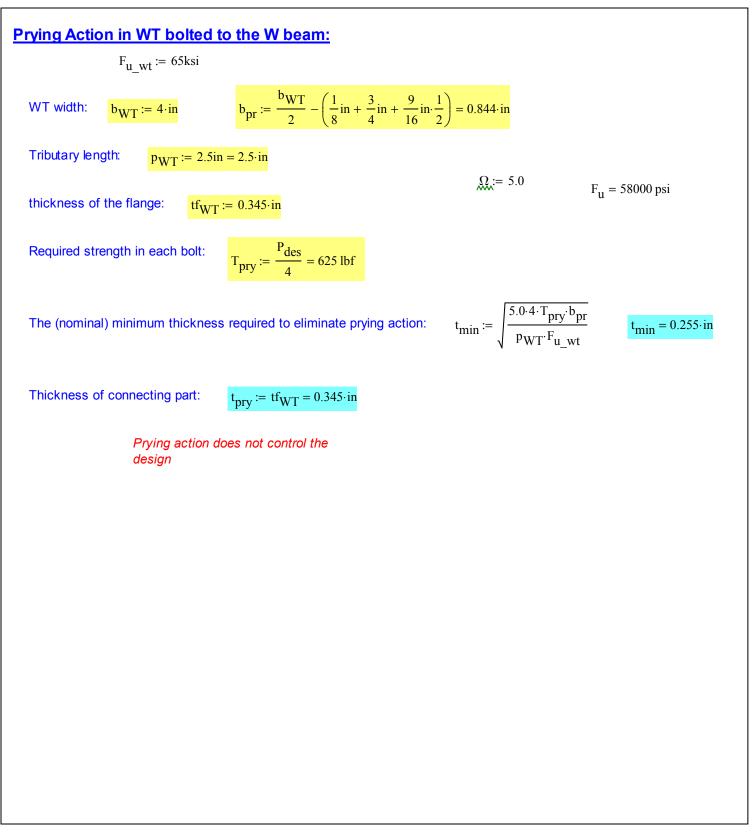


# **Plate Tension Values:** AISC - J4.1: $A_{g pl} := 6 \cdot in \cdot t_{pl} = 1.68 \cdot in^2$ Minimum gross area for plate with bolt hole: $\mathbf{A}_{e\_pl} \coloneqq \left\lceil 6 \cdot \operatorname{in} - \left(\frac{7}{8} \cdot \operatorname{in} + \frac{1}{16} \cdot \operatorname{in}\right) \right\rceil \cdot \mathbf{t}_{pl} = 1.417 \cdot \operatorname{in}^2$ Minimum effective net area for plate with bolt hole: (a) Tensile yielding: $R_{n ty} := F_y \cdot A_{g pl}$ $R_{n ty} = 60.48 \cdot kip$ $R_{a\_ty} := \frac{R_{n\_ty}}{FS} \qquad \qquad R_{a\_ty} = 12.096 \cdot kip$ (b) Tensile rupture: $R_{n tr} := F_{u} \cdot A_{e pl}$ $R_{n tr} = 82.215 \cdot kip$ $R_{a\_tr} := \frac{R_{n\_tr}}{FS} \qquad \qquad R_{a\_tr} = 16.443 \cdot kip$ $FS_{wt\_pl\_t} \coloneqq \frac{min(R_{n\_ty}, R_{n\_tr})}{P_{des}} = 24.192$ Factor of Safety:



Capacity of 1/2" E	Grade 5 bolts, each i	measuring 1-1/4" long
Yield strength:	Fy <sub>bolt</sub> := 92·ksi	
Tensile strength:	Fu <sub>bolt</sub> := 120·ksi	
Bolt diameter:	$d_{b1} \coloneqq \frac{1}{2} \cdot in \qquad A_b \coloneqq \frac{d_{b1}^2 \cdot \pi}{4}$	
Bolt tensile streng	th: $Tn_{bolt} := 0.75 \cdot Fu_{bolt} \cdot A_b$	$Tn_{bolt} = 17.671 \cdot kip$
Bolt shear strengt	h: $Vn_{bolt} := 0.4 \cdot Fu_{bolt} \cdot A_b$	$Vn_{bolt} = 9.425 \cdot kip$
Maximum tensile	load applied to each bolt:	$P_{max\_bolt} := \frac{P_{des}}{4} = 625  lbf$
Factor of Safety:	$FS_{tm\_bolt} := \frac{Tn_{bolt}}{P_{max\_bolt}} = 28.$	274





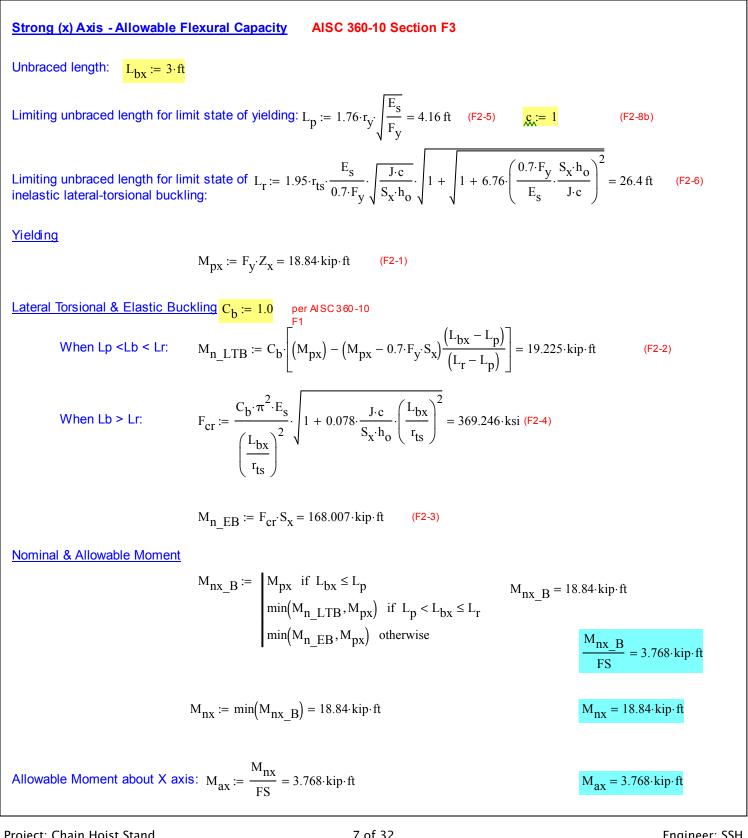


#### Crossbar Beam:

The top most beam of the stand is a W4x13, which supports the motor at the center from the lower flange.

~				
Section Properties for	SHAPE_NAME = "W4X	113"		
Weight:	wt = $13 \cdot \text{plf}$	Moment of inertia:	$I_x = 11.3 \cdot in^4$	$I_y = 3.86 \cdot i$
Area:	$A = 3.83 \cdot in^2$	Section modulus:	$S_x = 5.46 \cdot in^3$	$S_v = 1.9$ ·ir
Depth:	$d = 4.16 \cdot in$	Radius of gyration:	$r_x = 1.72 \cdot in$	$r_v = 1 \cdot in$
Web thickness:	$t_w = 0.28 \cdot in$		A	5
Flange width:	$b_f = 4.06 \cdot in$	Plastic Section modulus:	$Z_{\rm X} = 6.28 \cdot \text{in}^3$	$Z_y = 2.92$
Flange thickness:	$t_f = 0.345 \cdot in$	Effective radius of gyration:	$r_{ts} = 1.16 \cdot in$	
Fillet:	$\mathbf{k} = 0.595 \cdot \mathbf{in}$	$\mathbf{k}_1 = 0.5 \cdot \mathbf{in}$		
Distance between fillet:	$T = 2.97 \cdot in$	Distance between flg centriods:	h = 2.92 in	
b/2t:	b_t = 5.88	bistance between ny centhous.	$h_0 = 3.82 \cdot in$	
h/t:	h_t = 10.6	Torsional constant:	$J = 0.151 \cdot in^4$	
		Warping constant:	$C_w = 14 \cdot in^6$	





Project: Chain Hoist Stand CRE Project #: 14.319.04

Engineer: SSH 12/30/2014

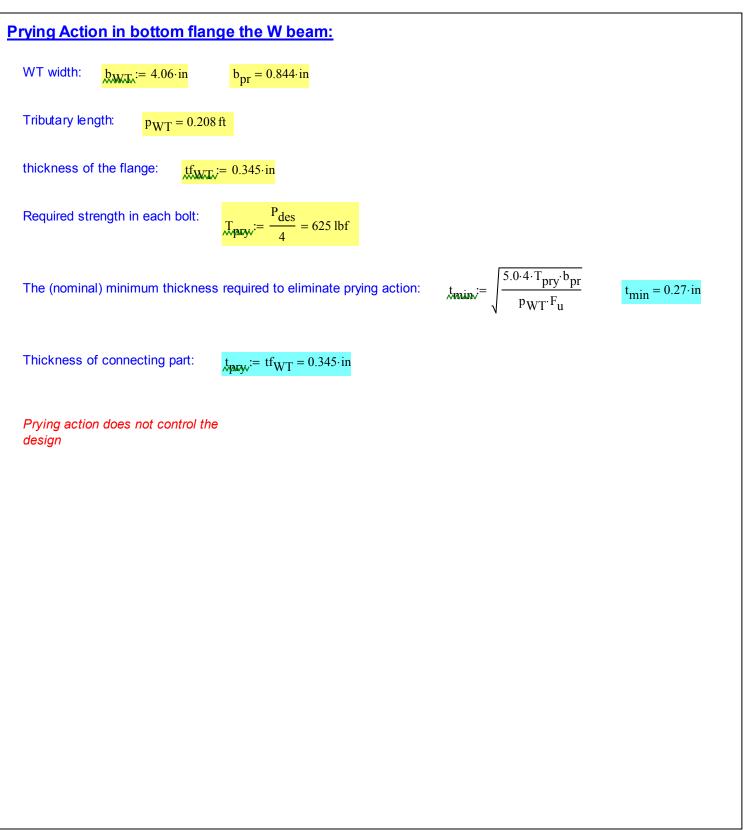


Strong (y) Axis - Allowable S	hear Canacity	AISC 360-4	10 Section G2	
		AI3C 300-		
Distance between flanges:	$h := T = 2.97 \cdot in$		E	
Web buckling coefficient:		if $\frac{h}{t_W} = 10.6$ < 2.24	N J	
Web Shear Coefficient:		$\leq 1.10 \sqrt{\frac{k_{v} \cdot E_{s}}{F_{y}}}$ $\frac{\overline{E_{s}}}{\underline{y}}  \text{if } 1.10 \sqrt{\frac{k_{v} \cdot E_{s}}{F_{y}}} < \frac{h}{t_{w}} \leq$	(G2-3) $1.37 \sqrt{\frac{k_v \cdot E_s}{F_y}}$ (G2-4)	
	$\frac{1.51 \cdot E_{s} \cdot k_{v}}{\left(\frac{h}{t_{w}}\right)^{2} \cdot F_{y}}$		(G2-5)	C <sub>V</sub> = 1.0
Shear Area:	$A_W := d \cdot t_W = 1.16$	$65 \cdot in^2$		
Nominal & Allowable Shear				
	$V_{ny} := 0.6 \cdot F_y \cdot A_w$	$C_{\rm V} = 25.16 \text{ kip}$ (G2-1)		
Allowable Y axis Shear:			$V_{ay} := \frac{V_{ny}}{FS} = 5.032 \cdot kip$	



Allowable Capacity Su	mmary for W4x13:	
Strong axis bending:	$M_{ax} = 3.768 \cdot kip \cdot ft$	
Weak axis bending	M <sub>ay</sub> := 0 Weak axis shear:	$V_{ax} := 0$
Strong axis shear:	$V_{ay} = 5.032 \cdot kip$	
Compression :	$C_a := 0$	
<u>Member Check</u>		
Strong shear: $V_y := 1$	$P_{des} = 2500  lbf$	
Factor of Safety: FS	$S_{V_W4} \coloneqq \frac{V_{ny}}{V_y} = 10.064$	
Strong moment: M <sub>X</sub> :=	$\frac{P_{\text{des}} \cdot 3 \cdot ft}{4} = 1.875 \cdot \text{kip} \cdot ft$	
Factor of Safety:	$FS_{Mx}W4 := \frac{M_{nx}}{M_{x}} = 10.048$	





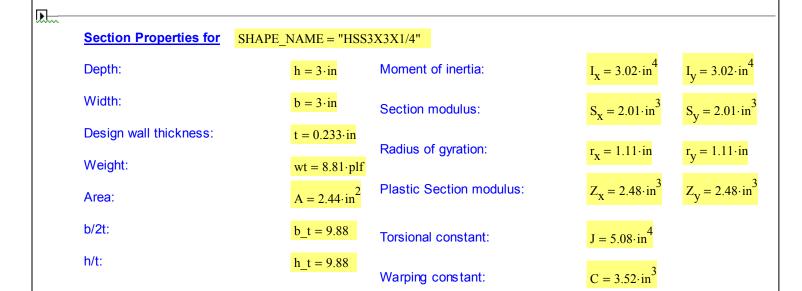


#### **Uprights - HSS columns**

The upright columns are HSS3x3x1/4 members that are each 3'-7-1/2" high.

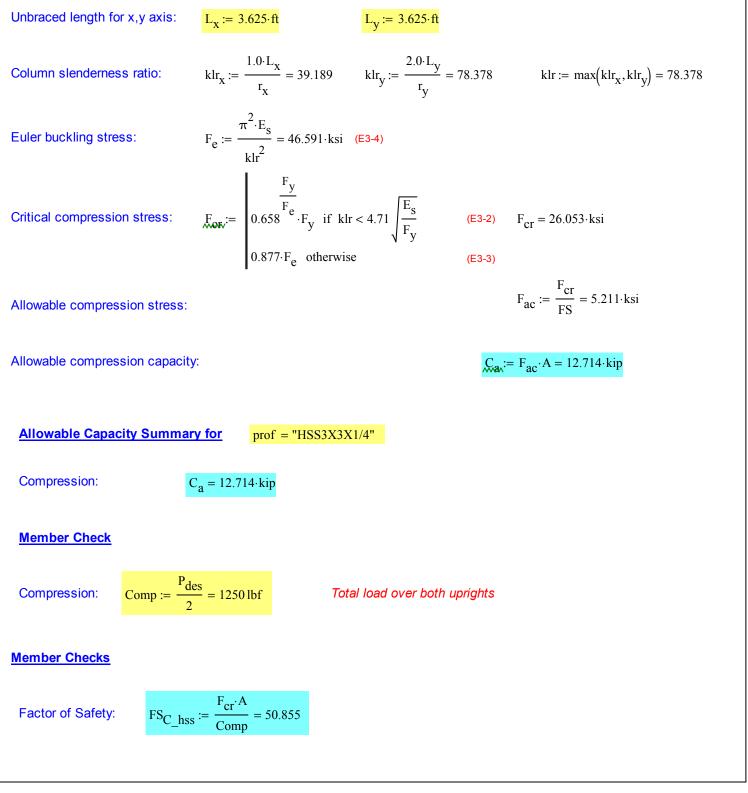
Total length:  $L_{tot upr} := 3 \cdot ft + 7.5 \cdot in = 3.625 ft$ 

Use the total length for unbraced length of the uprights.  $Lb_{upr} := L_{tot upr} = 3.625 \text{ ft}$ 

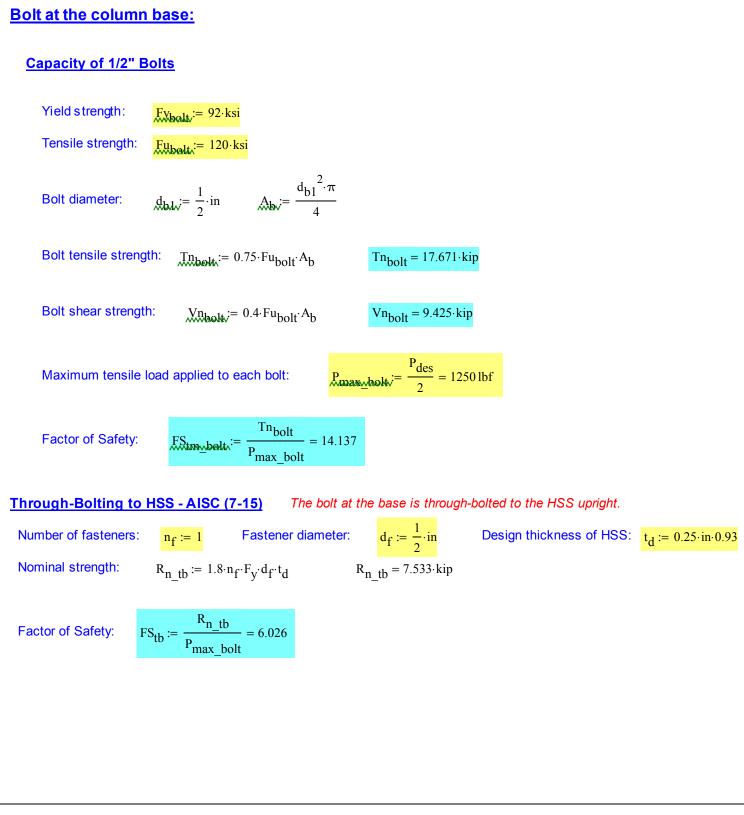




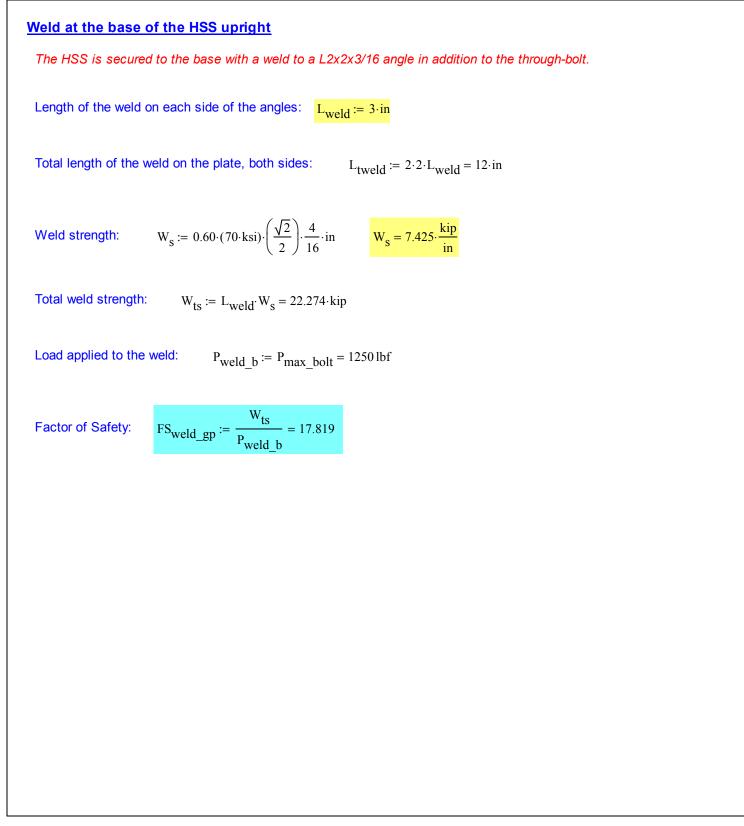














#### **Base Assembly Beams - Channels**

The HSS vertical members are bolted to the (2) base assembly beams, which are C3x4.1, that are 5'-0" in length. They are sitting on existing channels that are each 3" apart. The maximum load will be applied on a continuously supported base assembly beam.

Enter Shape: prof. = "C3X4.1"

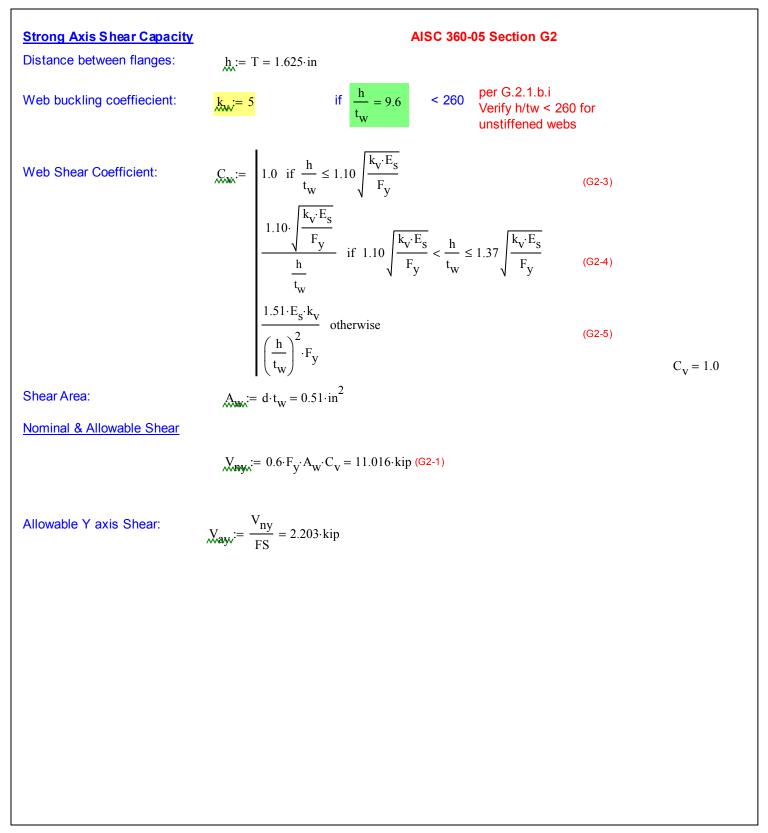
Section Properties for SHAPE_NAME = "C3X4.1"				
Area:	$A = 1.2 \cdot in^2$	Moment of inertia:	$I_x = 1.65 \cdot in^4$	$I_y = 0.191 \cdot in^4$
Depth:	$d = 3 \cdot in$	Section modulus:	$S_x = 1.1 \cdot in^3$	$S_y = 0.196 \cdot in^3$
Web thickness:	$t_w = 0.17 \cdot in$	Radius of gyration:	$r_x = 1.18 \cdot in$	$r_v = 0.398 \cdot in$
Flange width:	$b_f = 1.41 \cdot in$	radiae of gyration.	A	5
Flange thickness:	$t_f = 0.273 \cdot in$	Plastic Section modulus:	$Z_{\rm x} = 1.32 \cdot \text{in}^3$	$Z_y = 0.399 \cdot in^3$
Fillet:	$\mathbf{k} = 0.688 \cdot \mathbf{in}$	Torsional constant:	$J = 0.027 \cdot in^4$	
Distance between fillet:	$T = 1.625 \cdot in$	Warping constant:	$C_w = 0.307 \cdot in^6$	
Effective radius of gyration:	$r_{ts} = 0.469 \cdot in$	Polar radius of gyration: (about shear center)	$r_0 = 1.53 \cdot in$	
Distance between flg centriods:	$h_0 = 2.73 \cdot in$	Distance to centroid: (from web)	$x_{bar} = 0.437 \cdot in$	
Shear center:	$e_0 = 0.461 \cdot in$			
Flexural Constant:	H = 0.655			



Strong Axis Flexural CapacityAISC 360-05 Section F2Unbraced length:
$$\frac{1}{M_{AW}} = \frac{51}{2} - 10.5 \text{ m} = 1.625 \text{ ft}$$
Limiting unbraced length for limit state of yielding: $\int_{QW} = 1.76 \text{ ry} \sqrt{\frac{F_S}{F_Y}} = 1.66 \text{ ft}$  $(F2.6)$  $\int_{S_0^+} = \frac{h_0}{2} \sqrt{\frac{1}{V_W}}$  $(F2.6)$ Limiting unbraced length for limit state of inelastic lateral-torsional buckling: $\int_{WW} = 1.95 \text{ rtg} \cdot \frac{E_S}{0.7 \text{ Fy}} \cdot \sqrt{\frac{3}{5} \frac{1}{K_0}} \cdot \sqrt{1 + \sqrt{1 + 6.76} \left(\frac{0.77 \text{ Fy}}{E_S} \cdot \frac{5 \text{ k}}{1 \text{ c}}\right)^2} = 12.3 \text{ ft}$  $(F2.6)$ Yielding $M_p := F_y Z_x = 3.96 \text{ kip} \cdot ft$  $(F2.1)$ Lateral Torsional Buckling $\int_{W_0} = \frac{10}{2} \text{ ft} \cdot \frac{10}{1 \text{ per AISC330.05 F1}}$ When Lp  $M_{W_0} = \frac{10}{2} \text{ ft} \cdot \frac{1}{10} \text{ per AISC330.05 F1}$ When Lb > Lr. $\int_{W_0} \frac{1}{\sqrt{1 \text{ tb}} x^2} \cdot C_b \left[ (M_p) - (M_p - 0.77 \text{ Fy} S_x) \frac{(L_{bx} - L_p)}{(L_1 - L_p)} \right] = 3.965 \text{ kip} \cdot ft$  $M_{W_0} = \frac{10}{2} \text{ ft} \frac{1}{10} \text{ output } \frac{1}{2} \text{$ 



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Allowable Capacity Sur	nmary for prof = "C3X4.1"
Strong axis bending:	$M_{ax} = 0.792 \cdot \text{kip} \cdot \text{ft}$
Weak axis bending	$M_{ay} = 0 \cdot kip \cdot ft$
Weak axis shear:	$V_{ax} = 0 \cdot kip$
Strong axis shear:	$V_{ay} = 2.203 \cdot kip$
Compression:	$C_{a} := 0$
Member Check	
Strong shear:	$\frac{P_{des}}{2} = 1250  lbf$ Maximum axial force from the upright column
Factor of Safety: FS <sub>V</sub>	$v_{bab} := \frac{V_{ny}}{V_y} = 8.813$
Strong moment: Max =	$\frac{0.5 \cdot P_{des}}{5 \cdot ft} \cdot \frac{5 \cdot ft}{2} \cdot \frac{5 \cdot ft}{4} = 0.781 \cdot kip \cdot ft$ assuming that the the load from the column is continuously supported along the bottom from existing steel
Factor of Safety:	$S_{bab} := \frac{M_{nx}}{M_x} = 5.069$



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# Leg Cross Brace

The leg cross braces, also C3x4.1 channels, are 2'-9" in length and are framed into the base assembly beams. Using the load assumption above, that the assembly beams are continously supported on each side, the leg braces will see a bending force applied to them.

Maximum moment in the leg cross brace:

 $M_{\text{legb}} := \frac{P_{\text{des}}}{2} \cdot \frac{1}{2\text{ft} + 9\text{in}} \cdot \frac{(2\text{ft} + 9\text{in})^2}{8} = 0.43 \cdot \text{kip} \cdot \text{ft}$ 

Interaction:

INT<sub>M\_lcb</sub> := 
$$\frac{M_{legb}}{M_{ax}} = 0.543$$
  
FS<sub>M\_lcb</sub> :=  $\frac{M_{nx}}{M_{nx}} = 9.210$ 

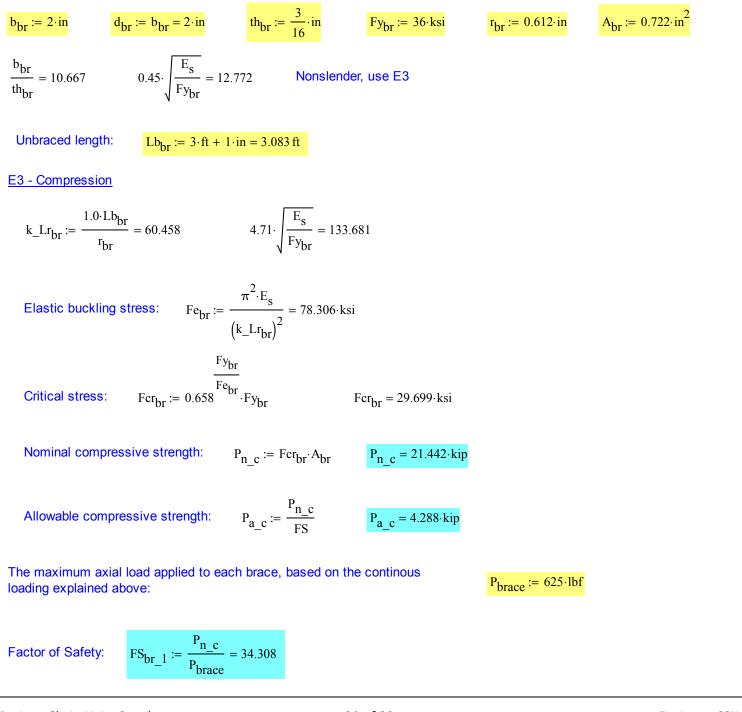
Factor of Safety:

$$FS_{M_{lcb}} := \frac{M_{nx}}{M_{legb}} = 9.216$$



### L2x2x3/16 Braces

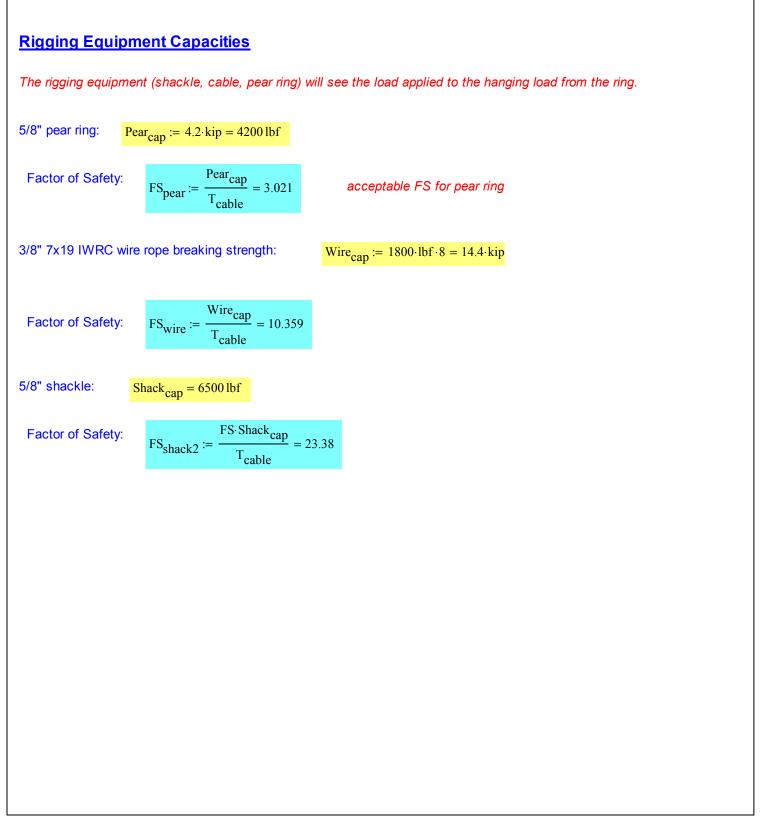
The angle braces, which are 3'-1" in length brace the uprights with the base assembly beams. Using the load assumption above, that the braces are attached to the base and will see half of the axial load applied to them.





# Second Load Path: Eyebolt Assembly Point The loads applied to the bolts and plates are caused by the hanging load through the system of shackles, cables, and the pear ring. Vertical distance to load from top: $l_{\text{vert hg}} := 3 \cdot \text{ft}$ $l_{\text{horz}_hg} := \frac{3 \cdot ft}{2} = 1.5 \text{ ft}$ Horizontal distance to load from end at top: $l_{cable} := \sqrt{l_{vert hg}^2 + l_{horz hg}^2} = 3.354 \, \text{ft}$ Length of cable: $P_{total_{hg}} := P_{des} = 2500 \, lbf$ Total load being hung: $P_{half\_total} := \frac{P_{total\_hg}}{2} = 1250 \, lbf$ Total load applied to each of the 2 cables: This is also the vertical component of the load at the top of the upright P<sub>half</sub> total Tension in each cable: T<sub>cable</sub> ≔ $= 1390.059 \, lbf$ cable sin lvert\_hg $P_{horz\_top} := T_{cable} \cdot cos \left( \frac{l_{cable}}{l_{vert\_bg}} \right) = 608.083 \text{ lbf}$ at the top of the cable Horizontal component:







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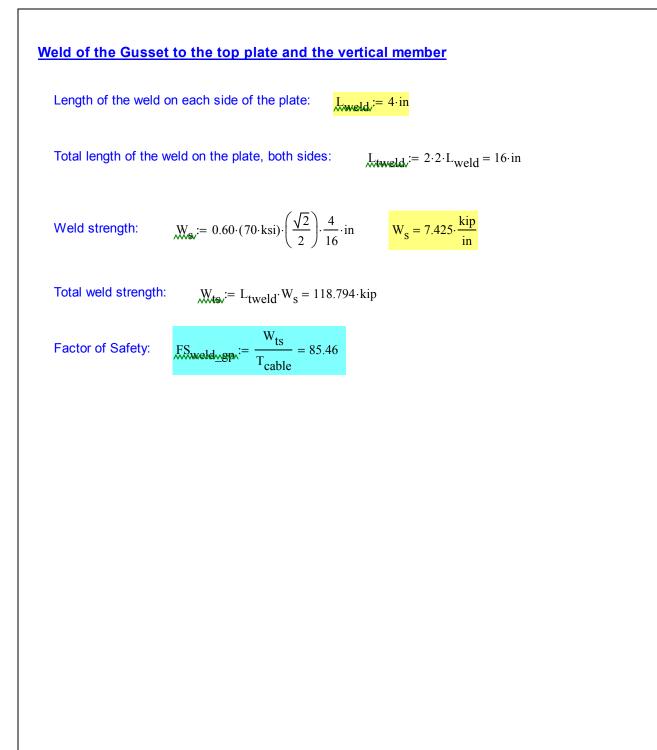
# **Gusset Plates**

A triangular plate is located at the top of the uprights between the columns and W4 beam. The contain a 13/16" diameter hole in the center of the plate, that support a system created by 5/8" shackle, 3/8" diameter 7x19 IWRC, and a 5/8" pear ring. The plate legs are each 4" long.

#### **Bearing Strength for Bolt Holes:**

 $l_{3gn} := \sqrt{(4 \cdot in)^2 + (4 \cdot in)^2} = 5.657 \cdot in$ Length of the third side of the plate: If each side of the plate is 4", depth of the plate in the 45 degree direction:  $d_{gp} := \sqrt{(4 \cdot in)^2 - (\frac{l_{3gp}}{2})^2} = 2.828 \cdot in$ AISC - J3.10:  $l_{c_gp} := \frac{d_{gp}}{2} - \frac{\frac{13}{16} \cdot in}{2} = 1.008 \cdot in$ Clear distance in the direction of the force:  $t_{gp} := \frac{3}{8} \cdot in$ Thickness of the connected material:  $d_{b_gp} := \frac{13}{16} \cdot in - \frac{1}{16} \cdot in = 0.75 \cdot in$ Bolt diameter: Nominal Bearing Strength:  $\underset{\text{MNMM}}{\text{R}} \coloneqq \min \left( 1.2 \cdot l_{c \text{ gp}} \cdot t_{\text{gp}} \cdot F_{u}, 2.4 \cdot d_{b \text{ gp}} \cdot t_{\text{gp}} \cdot F_{u} \right)$  $R_{n b} = 26.308 \cdot kip$  $\underset{\text{Reality}}{\text{R}} = \frac{\text{R}_{n\_b}}{\text{FS}} \qquad \qquad \text{R}_{a\_b} = 5.262 \cdot \text{kip}$ Allowable Bearing Strength:  $FS_{gp\_brg} := \frac{R_{n\_b}}{T_{cable}} = 18.926$ Factor of Safety:







# Plate Analysis at the top on each side of the Uprights

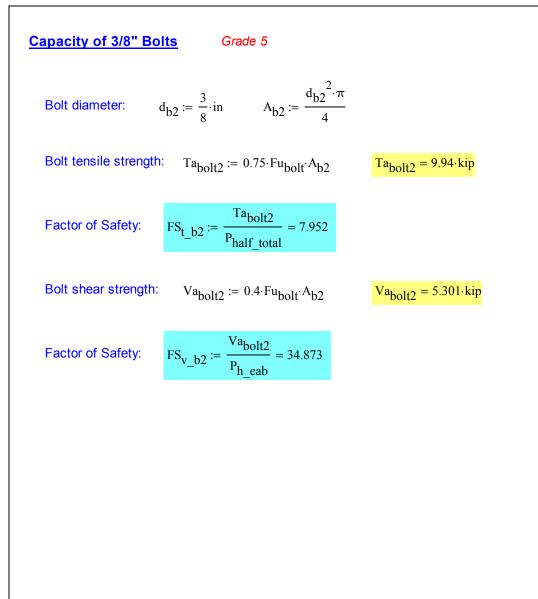
A 10" x 4" x 3/8" thick plate sits at the top of each HSS upright and below the W4 beam above.

#### **Bearing Strength for Bolt Holes:**

#### AISC - J3.10:

 $l_{\text{Mex}} = \frac{3}{4} \cdot \text{in} - \frac{1}{2} \cdot \frac{9}{16} \cdot \text{in} = 0.469 \cdot \text{in}$ Clear distance in the direction of the force:  $t_{\text{pla}} := \frac{3}{8} \cdot \text{in}$ Thickness of the connected material: Bolt diameter: du:= 0.375.in Nominal Bearing Strength:  $\mathbf{R}_{n b} := \min(1.2 \cdot \mathbf{l}_{c} \cdot \mathbf{t}_{pl} \cdot \mathbf{F}_{u}, 2.4 \cdot \mathbf{d}_{b} \cdot \mathbf{t}_{pl} \cdot \mathbf{F}_{u}) \qquad \mathbf{R}_{n b} = 12.234 \cdot \mathrm{kip}$  $\mathbf{R}_{a\_b} := \frac{\mathbf{R}_{n\_b}}{2.00} \qquad \qquad \mathbf{R}_{a\_b} = 6.117 \cdot \mathrm{kip}$ Allowable Bearing Strength: P<sub>horz top</sub> = 608.083 lbf *calculated above* Horizontal load at the top of the cable:  $P_{h\_eab} := \frac{P_{horz\_top}}{4} = 152.021 \text{ lbf}$ Horizontal load in each bolt in the palte:  $FS_{tp\_brg} \coloneqq \frac{R_{n\_b}}{P_{h eab}} = 80.478$ Factor of Safety:







# W4 top beam

The W4x13 crossbar will receive compression from the top plate.

 $L_{x} := 3 ft$ 

Allowable Compression Capacity & Compression Stress

AISC 360-10 Section E3

$\lambda_{\text{fc}} \coloneqq \frac{b_{\text{f}}}{2 \cdot t_{\text{f}}} = 2.582$	$\lambda_{\text{rf\_c}} \coloneqq 0.56 \cdot \sqrt{\frac{\text{E}_{\text{s}}}{\text{F}_{\text{y}}}} = 15.894$	Flange is slender
$\lambda_{\rm WC} := \frac{d - 2 \cdot t_{\rm f}}{t_{\rm W}} = 14.435$	$\lambda_{\text{rw}\_c} := 1.49 \cdot \sqrt{\frac{\text{E}_{\text{s}}}{\text{F}_{\text{y}}}} = 42.29$	Web in non slender

Unbraced length for X,Y axis:

Column Slenderness ratio:

$$\lim_{x \to \infty} \frac{1.0 \cdot L_x}{r_x} = 30.508 \qquad \lim_{x \to \infty} \frac{1.0 \cdot L_y}{r_y} = 90.452 \qquad \lim_{x \to \infty} \frac{1.0 \cdot L_y}{r_y} = 90.452$$

Euler Buckling Stress:  $F_{\text{Nev}} \coloneqq \frac{\pi^2 \cdot E_s}{k lr^2} = 34.983 \cdot ksi \quad \text{(E3-4)} \qquad Q \coloneqq 1.415 - 0.74 \cdot \frac{b_f}{2 \cdot t_f} \cdot \sqrt{\frac{F_y}{E_s}} = 1.262 \qquad \text{Qs}$ 

Critical Compression Stress: 
$$F_{\text{work}} := \begin{bmatrix} \frac{Q \cdot F_y}{F_e} \\ Q \cdot (0.658) & F_y & \text{if } klr \le 4.71 \sqrt{\frac{E_s}{Q \cdot F_y}} \end{bmatrix}$$
 (E3-2)  $F_{cr} = 26.377 \cdot ksi$   
0.877  $\cdot F_e$  otherwise

(E3-3)

Nominal Compression:  $C_n := F_{cr} \cdot A = 31.653 \cdot kip$ 

Factor of Safety:

 $FS_{comp\_cb} \coloneqq \frac{C_n}{P_{horz\_top}} = 52.053$ 



# Third Load Path: Lateral Load applied parallel to the angle braces

Applying a lateral load to the system of 10% of the total load being supported (10% of 2500# = 250#) will result in load being applied to the angle braces (L2x2x3/16). A RISA model was made to account for these loads.

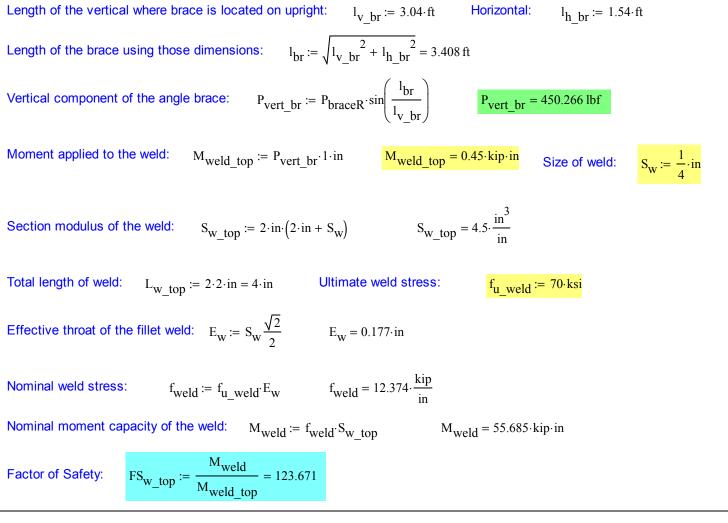
The maximum axial load applied to each brace, from RISA:

 $P_{braceR} := 500 \cdot lbf$ 

Nominal compression for L2x2 (calculated above):  $P_{n,c} = 21442.341 \text{ lbf}$ 

Factor of Safety:  $FS_{br_R} := \frac{P_{n_c}}{P_{c_c}} = 42.885$ 

Weld from angle piece at the top of the brace to the HSS upright has a moment applied to it from the vertical component of the angle brace.





Weld from angle piece at the bottom of the brace to the base assembly channel has a moment applied to it from the horizontal component of the angle brace. The weld is on the top and side of the angle piece to the base channel  $P_{horz\_br} := \left| P_{braceR} \cdot \cos\left(\frac{l_{br}}{l_{h\_br}}\right) \right| = 299.427 \, lbf$ Horizontal component of the angle brace: Moment applied to the weld:  $M_{weld bott} := P_{horz br} \cdot 2 \cdot in$   $M_{weld bott} = 0.599 \cdot kip \cdot in$  $L_{w \text{ bott}} := 3 \cdot in + 2 \cdot in = 5 \cdot in$  Ultimate weld stress:  $f_{u \text{ weld}} = 70 \cdot ksi$ Total length of weld: Effective throat of the fillet weld:  $E_w = 0.177 \cdot in$  Size of weld:  $S_w = 0.25 \cdot in$  $S_{w\_bott} := \frac{4 \cdot (2 \cdot in - 0.5 \cdot S_w) \cdot (3 \cdot in - 0.5 \cdot S_w) + (3 \cdot in - 0.5 \cdot S_w)^2}{6} \qquad S_{w\_bott} = 4.971 \cdot \frac{in^3}{in}$ Section modulus of the weld:  $f_{weld} = 12.374 \cdot \frac{kip}{ip}$ Nominal weld stress: Nominal moment capacity of the weld:  $M_{weld\_b} := f_{weld} \cdot S_{w\_bott}$   $M_{weld\_b} = 61.517 \cdot kip \cdot in$  $FS_{w\_bott} := \frac{M_{weld\_b}}{M_{weld\_bott}} = 102.725$ Factor of Safety:



# Minimum Bearing Strength for Bolt Holes: The brace connections on the uprights and base beams are made with L2x2x3/16 angle pieces. AISC - J3.10: $\lim_{n \to \infty} = \frac{2 \cdot in}{2} - \frac{\frac{7}{16} \cdot in}{2} = 0.781 \cdot in$ Clear distance in the direction of the force: $t_{ang} := \frac{3}{16} \cdot in$ Thickness of the connected material: Bolt diameter: $d_b = 0.375 \cdot in$ Nominal Bearing Strength: $R_{n b} = \min(1.2 \cdot l_c \cdot t_{ang} \cdot F_u, 2.4 \cdot d_b \cdot t_{ang} \cdot F_u) \qquad R_{n b} = 9.787 \cdot kip$ $FS_{ang\_brg} := \frac{R_{n\_b}}{max(P_{vert\_br}, P_{horz\_br})} = 21.737$ Factor of Safety:



# Fourth Load Path: Lateral Load applied parallel to the Crossbar

Applying a lateral load to the system of 10% of the total load being supported (10% of 2500# = 250#) will result in load being applied to the frame formed by the (2) uprights and the crossbar. A RISA model was made to account for these loads.

Maximum moment in each upright (from RISA):

 $M_{upr} := 0.7 \cdot kip \cdot ft$ 

# <u>Uprights</u>

The uprights will have a load applied at the top, assuming fully unbraced.

Total length:  $L_{tot upr} = 3.625 \text{ ft}$ 

Plastic section modulus for HSS:  $Z_{\text{MM}} = 2.48 \cdot \text{in}^3$ 

#### Strong Axis Allowable Flexural Capacity

Section is compact. The combined section capacity is based on yielding of the angle leg at the extreme fiber.

Nominal & Allowable Moment

Nominal moment capacity:  $M_{VX} := F_V \cdot Z_X = 89.28 \cdot \text{kip} \cdot \text{in}$  (F7-1)

Factor of Safety:

$$FS_{mx\_upr} := \frac{M_{nx}}{M_{upr}} = 10.629$$



