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.

Table of Contents for Analysis Package

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.

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
JEFFREY MICHAEL REDER
ENGINEER
NO. PE77455

01/06/2015

Scott Horn, E.I.T.

01/06/2015

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^{TH} 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.

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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.

Design Load: $P_{des} := 2500 \cdot lbf$

Minimum Yield Strength: $F_y := 36 \cdot ksi$

Minimum Factor of Safety: $FS := 5.0$

Minimum Ultimate Strength: $F_u := 58 \cdot ksi$

First Load Path: Chain Motor Hoist Point

5/8" Shackle Capacity: $Shack_{cap} := 6500 \cdot lbf$ shackle capacity based on an inherent FOS=5.0

Factor of Safety: $FS_{top\_shack} := \frac{Shack_{cap} \cdot FS}{P_{des}} = 13$

WT Piece:

The motor is mounted off of the bottom flange of the W4 above, with (4) 1/2" bolts, to a 6" piece of WT. The web of the WT contains a 7/8" diameter hole.

Bearing Strength for Hole in stem of WT to support Motor:

Checking the web of the WT for supporting the motor. The shape is a W4x13 with the bottom flange removed.

AISC - J3.10:

Clear distance in the direction of the force: $l_c := 4 \cdot in - 2.625 \cdot in - \frac{0.875 \cdot in}{2} = 0.938 \cdot in$

Thickness of the connected material: $t_{pl} := 0.280 \cdot in$

Bolt diameter: $d_b := \frac{3}{4} \cdot in$

Nominal Bearing Strength: $R_{n\_b} := \min(1.2 \cdot l_c \cdot t_{pl} \cdot F_u, 2.4 \cdot d_b \cdot t_{pl} \cdot F_u)$ $R_{n\_b} = 18.27 \cdot kip$

Allowable Bearing Strength: $R_{a\_b} := \frac{R_{n\_b}}{FS}$ $R_{a\_b} = 3.654 \cdot kip$

Factor of Safety: $FS_{wt\_brg} := \frac{R_{n\_b}}{P_{des}} = 7.308$

Plate Tension Values:

AISC - J4.1:

Minimum gross area for plate with bolt hole: $A_{g\_pl} := 6 \cdot in \cdot t_{pl} = 1.68 \cdot in^2$

Minimum effective net area for plate with bolt hole: $A_{e\_pl} := \left[6 \cdot in - \left(\frac{7}{8} \cdot in + \frac{1}{16} \cdot in\right)\right] \cdot t_{pl} = 1.417 \cdot in^2$

(a) Tensile yielding: $R_{n\_ty} := F_y \cdot A_{g\_pl}$ $\quad R_{n\_ty} = 60.48 \cdot kip$

$R_{a\_ty} := \frac{R_{n\_ty}}{FS}$ $\quad R_{a\_ty} = 12.096 \cdot kip$

(b) Tensile rupture: $R_{n\_tr} := F_u \cdot A_{e\_pl}$ $\quad R_{n\_tr} = 82.215 \cdot kip$

$R_{a\_tr} := \frac{R_{n\_tr}}{FS}$ $\quad R_{a\_tr} = 16.443 \cdot kip$

Factor of Safety: $FS_{wt\_pl\_t} := \frac{\min(R_{n\_ty}, R_{n\_tr})}{P_{des}} = 24.192$

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Capacity of 1/2" Bolts

Grade 5 bolts, each measuring 1-1/4" long

Yield strength: $Fy_{bolt} := 92 \cdot ksi$

Tensile strength: $Fu_{bolt} := 120 \cdot ksi$

Bolt diameter: $d_{b1} := \frac{1}{2} \cdot in$ $A_b := \frac{d_{b1}^{\ 2} \cdot \pi}{4}$

Bolt tensile strength: $Tn_{bolt} := 0.75 \cdot Fu_{bolt} \cdot A_b$ $Tn_{bolt} = 17.671 \cdot kip$

Bolt shear strength: $Vn_{bolt} := 0.4 \cdot Fu_{bolt} \cdot A_b$ $Vn_{bolt} = 9.425 \cdot kip$

Maximum tensile load applied to each bolt: $P_{max\_bolt} := \frac{P_{des}}{4} = 625\ lbf$

Factor of Safety: $FS_{tm\_bolt} := \frac{Tn_{bolt}}{P_{max\_bolt}} = 28.274$

Prying Action in WT bolted to the W beam:

$F_{u\_wt} := 65ksi$

WT width: $b_{WT} := 4 \cdot in$ $\quad b_{pr} := \frac{b_{WT}}{2} - \left(\frac{1}{8}in + \frac{3}{4}in + \frac{9}{16}in \cdot \frac{1}{2}\right) = 0.844 \cdot in$

Tributary length: $p_{WT} := 2.5in = 2.5 \cdot in$

$\Omega := 5.0$ $\quad F_u = 58000\,psi$

thickness of the flange: $tf_{WT} := 0.345 \cdot in$

Required strength in each bolt: $T_{pry} := \frac{P_{des}}{4} = 625\,lbf$

The (nominal) minimum thickness required to eliminate prying action: $t_{min} := \sqrt{\frac{5.0 \cdot 4 \cdot T_{pry} \cdot b_{pr}}{p_{WT} \cdot F_{u\_wt}}}$ $\quad t_{min} = 0.255 \cdot in$

Thickness of connecting part: $t_{pry} := tf_{WT} = 0.345 \cdot in$

Prying action does not control the design

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 = "W4X13"

Weight:	$wt = 13 \cdot plf$	Moment of inertia:	$I_x = 11.3 \cdot in^4$	$I_y = 3.86 \cdot in^4$
Area:	$A = 3.83 \cdot in^2$	Section modulus:	$S_x = 5.46 \cdot in^3$	$S_y = 1.9 \cdot in^3$
Depth:	$d = 4.16 \cdot in$	Radius of gyration:	$r_x = 1.72 \cdot in$	$r_y = 1 \cdot in$
Web thickness:	$t_w = 0.28 \cdot in$	Plastic Section modulus:	$Z_x = 6.28 \cdot in^3$	$Z_y = 2.92 \cdot in^3$
Flange width:	$b_f = 4.06 \cdot in$	Effective radius of gyration:	$r_{ts} = 1.16 \cdot in$	
Flange thickness:	$t_f = 0.345 \cdot in$			
Fillet:	$k = 0.595 \cdot in$	$k_1 = 0.5 \cdot in$		
Distance between fillet:	$T = 2.97 \cdot in$	Distance between flg centriods:	$h_o = 3.82 \cdot in$	
b/2t:	$b\_t = 5.88$	Torsional constant:	$J = 0.151 \cdot in^4$	
h/t:	$h\_t = 10.6$	Warping constant:	$C_w = 14 \cdot in^6$	

Strong (x) Axis - Allowable Flexural Capacity — AISC 360-10 Section F3

Unbraced length: $L_{bx} := 3 \cdot ft$

Limiting unbraced length for limit state of yielding: $L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E_s}{F_y}} = 4.16\,ft$ (F2-5) $c := 1$ (F2-8b)

Limiting unbraced length for limit state of inelastic lateral-torsional buckling: $L_r := 1.95 \cdot r_{ts} \cdot \frac{E_s}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E_s} \cdot \frac{S_x \cdot h_o}{J \cdot c}\right)^2}} = 26.4\,ft$ (F2-6)

Yielding

$$M_{px} := F_y \cdot Z_x = 18.84 \cdot kip \cdot ft$$ (F2-1)

Lateral Torsional & Elastic Buckling

$C_b := 1.0$ per AISC 360-10 F1

When Lp <Lb < Lr: $$M_{n\_LTB} := C_b \cdot \left[\left(M_{px}\right) - \left(M_{px} - 0.7 \cdot F_y \cdot S_x\right) \frac{\left(L_{bx} - L_p\right)}{\left(L_r - L_p\right)}\right] = 19.225 \cdot kip \cdot ft$$ (F2-2)

When Lb > Lr: $$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_{bx}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_{bx}}{r_{ts}}\right)^2} = 369.246 \cdot ksi$$ (F2-4)

$$M_{n\_EB} := F_{cr} \cdot S_x = 168.007 \cdot kip \cdot ft$$ (F2-3)

Nominal & Allowable Moment

$$M_{nx\_B} := \begin{cases} M_{px} & \text{if } L_{bx} \le L_p \\ \min\left(M_{n\_LTB}, M_{px}\right) & \text{if } L_p < L_{bx} \le L_r \\ \min\left(M_{n\_EB}, M_{px}\right) & \text{otherwise} \end{cases}$$

$M_{nx\_B} = 18.84 \cdot kip \cdot ft$

$\frac{M_{nx\_B}}{FS} = 3.768 \cdot kip \cdot ft$

$$M_{nx} := \min\left(M_{nx\_B}\right) = 18.84 \cdot kip \cdot ft$$

$M_{nx} = 18.84 \cdot kip \cdot ft$

Allowable Moment about X axis: $M_{ax} := \frac{M_{nx}}{FS} = 3.768 \cdot kip \cdot ft$

$M_{ax} = 3.768 \cdot kip \cdot ft$

Strong (y) Axis - Allowable Shear Capacity

AISC 360-10 Section G2

Distance between flanges: $h := T = 2.97 \cdot in$

Web buckling coefficient: $k_v := 5$ if $\frac{h}{t_w} = 10.6$ < $2.24 \cdot \sqrt{\frac{E_s}{F_y}} = 63.576$

Web Shear Coefficient:

$$C_v := \begin{cases} 1.0 \text{ if } \frac{h}{t_w} \le 1.10 \sqrt{\frac{k_v \cdot E_s}{F_y}} & \text{(G2-3)} \\ \frac{1.10 \cdot \sqrt{\frac{k_v \cdot E_s}{F_y}}}{\frac{h}{t_w}} \text{ if } 1.10 \sqrt{\frac{k_v \cdot E_s}{F_y}} < \frac{h}{t_w} \le 1.37 \sqrt{\frac{k_v \cdot E_s}{F_y}} & \text{(G2-4)} \\ \frac{1.51 \cdot E_s \cdot k_v}{\left(\frac{h}{t_w}\right)^2 \cdot F_y} \text{ otherwise} & \text{(G2-5)} \end{cases}$$

$C_v = 1.0$

Shear Area: $A_w := d \cdot t_w = 1.165 \cdot in^2$

Nominal & Allowable Shear

$V_{ny} := 0.6 \cdot F_y \cdot A_w \cdot C_v = 25.16 \cdot kip$ (G2-1)

Allowable Y axis Shear: $V_{ay} := \frac{V_{ny}}{FS} = 5.032 \cdot kip$

Allowable Capacity Summary for W4x13:

Strong axis bending: $M_{ax} = 3.768 \cdot kip \cdot ft$

Weak axis bending $M_{ay} := 0$ Weak axis shear: $V_{ax} := 0$

Strong axis shear: $V_{ay} = 5.032 \cdot kip$

Compression
: $C_a := 0$

Member Check

Strong shear: $V_y := P_{des} = 2500\, lbf$

Factor of Safety: $FS_{V\_W4} := \frac{V_{ny}}{V_y} = 10.064$

Strong moment: $M_x := \frac{P_{des} \cdot 3 \cdot ft}{4} = 1.875 \cdot kip \cdot ft$

Factor of Safety: $FS_{Mx\_W4} := \frac{M_{nx}}{M_x} = 10.048$

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Prying Action in bottom flange the W beam:

WT width: $b_{WT} := 4.06 \cdot in$ $b_{pr} = 0.844 \cdot in$

Tributary length: $p_{WT} = 0.208\,ft$

thickness of the flange: $tf_{WT} := 0.345 \cdot in$

Required strength in each bolt: $T_{pry} := \frac{P_{des}}{4} = 625\,lbf$

The (nominal) minimum thickness required to eliminate prying action: $t_{min} := \sqrt{\frac{5.0 \cdot 4 \cdot T_{pry} \cdot b_{pr}}{p_{WT} \cdot F_u}}$ $t_{min} = 0.27 \cdot in$

Thickness of connecting part: $t_{pry} := tf_{WT} = 0.345 \cdot in$

Prying action does not control the design

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. $Lb_{upr} := L_{tot\_upr} = 3.625\,ft$

Section Properties for SHAPE_NAME = "HSS3X3X1/4"

Depth:	$h = 3\cdot in$	Moment of inertia:	$I_x = 3.02\cdot in^4$	$I_y = 3.02\cdot in^4$
Width:	$b = 3\cdot in$	Section modulus:	$S_x = 2.01\cdot in^3$	$S_y = 2.01\cdot in^3$
Design wall thickness:	$t = 0.233\cdot in$	Radius of gyration:	$r_x = 1.11\cdot in$	$r_y = 1.11\cdot in$
Weight:	$wt = 8.81\cdot plf$	Plastic Section modulus:	$Z_x = 2.48\cdot in^3$	$Z_y = 2.48\cdot in^3$
Area:	$A = 2.44\cdot in^2$	Torsional constant:	$J = 5.08\cdot in^4$	
b/2t:	$b\_t = 9.88$	Warping constant:	$C = 3.52\cdot in^3$	
h/t:	$h\_t = 9.88$			

Allowable Compression Capacity

Unbraced length for x,y axis: $L_x := 3.625 \cdot ft$ $L_y := 3.625 \cdot ft$

Column slenderness ratio: $klr_x := \frac{1.0 \cdot L_x}{r_x} = 39.189$ $klr_y := \frac{2.0 \cdot L_y}{r_y} = 78.378$ $klr := \max(klr_x, klr_y) = 78.378$

Euler buckling stress: $F_e := \frac{\pi^2 \cdot E_s}{klr^2} = 46.591 \cdot ksi$ (E3-4)

Critical compression stress:

$$F_{cr} := \begin{cases} 0.658^{\frac{F_y}{F_e}} \cdot F_y & \text{if } klr < 4.71\sqrt{\frac{E_s}{F_y}} \quad \text{(E3-2)} \\ 0.877 \cdot F_e & \text{otherwise} \quad \text{(E3-3)} \end{cases}$$

$F_{cr} = 26.053 \cdot ksi$

Allowable compression stress: $F_{ac} := \frac{F_{cr}}{FS} = 5.211 \cdot ksi$

Allowable compression capacity: $C_a := F_{ac} \cdot A = 12.714 \cdot kip$

Allowable Capacity Summary for prof = "HSS3X3X1/4"

Compression: $C_a = 12.714 \cdot kip$

Member Check

Compression: $Comp := \frac{P_{des}}{2} = 1250\,lbf$ *Total load over both uprights*

Member Checks

Factor of Safety: $FS_{C\_hss} := \frac{F_{cr} \cdot A}{Comp} = 50.855$

Bolt at the column base:

Capacity of 1/2" Bolts

Yield strength: $Fy_{bolt} := 92 \cdot ksi$

Tensile strength: $Fu_{bolt} := 120 \cdot ksi$

Bolt diameter: $d_{b1} := \frac{1}{2} \cdot in$ $\quad A_b := \frac{d_{b1}{}^2 \cdot \pi}{4}$

Bolt tensile strength: $Tn_{bolt} := 0.75 \cdot Fu_{bolt} \cdot A_b$ $\quad Tn_{bolt} = 17.671 \cdot kip$

Bolt shear strength: $Vn_{bolt} := 0.4 \cdot Fu_{bolt} \cdot A_b$ $\quad Vn_{bolt} = 9.425 \cdot kip$

Maximum tensile load applied to each bolt: $P_{max\_bolt} := \frac{P_{des}}{2} = 1250\,lbf$

Factor of Safety: $FS_{tm\_bolt} := \frac{Tn_{bolt}}{P_{max\_bolt}} = 14.137$

Through-Bolting to HSS - AISC (7-15)

The bolt at the base is through-bolted to the HSS upright.

Number of fasteners: $n_f := 1$ $\quad$ Fastener diameter: $d_f := \frac{1}{2} \cdot in$ $\quad$ Design thickness of HSS: $t_d := 0.25 \cdot in \cdot 0.93$

Nominal strength: $R_{n\_tb} := 1.8 \cdot n_f \cdot F_y \cdot d_f \cdot t_d$ $\quad R_{n\_tb} = 7.533 \cdot kip$

Factor of Safety: $FS_{tb} := \frac{R_{n\_tb}}{P_{max\_bolt}} = 6.026$

Weld at the base of the HSS upright

The HSS is secured to the base with a weld to a L2x2x3/16 angle in addition to the through-bolt.

Length of the weld on each side of the angles: $L_{weld} := 3 \cdot in$

Total length of the weld on the plate, both sides: $L_{tweld} := 2 \cdot 2 \cdot L_{weld} = 12 \cdot in$

Weld strength: $W_s := 0.60 \cdot (70 \cdot ksi) \cdot \left(\frac{\sqrt{2}}{2}\right) \cdot \frac{4}{16} \cdot in$ $\quad W_s = 7.425 \cdot \frac{kip}{in}$

Total weld strength: $W_{ts} := L_{weld} \cdot W_s = 22.274 \cdot kip$

Load applied to the weld: $P_{weld\_b} := P_{max\_bolt} = 1250\, lbf$

Factor of Safety: $FS_{weld\_gp} := \frac{W_{ts}}{P_{weld\_b}} = 17.819$

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:	$r_x = 1.18 \cdot in$	$r_y = 0.398 \cdot in$
Flange width:	$b_f = 1.41 \cdot in$	Plastic Section modulus:	$Z_x = 1.32 \cdot in^3$	$Z_y = 0.399 \cdot in^3$
Flange thickness:	$t_f = 0.273 \cdot in$	Torsional constant:	$J = 0.027 \cdot in^4$	
Fillet:	$k = 0.688 \cdot in$	Warping constant:	$C_w = 0.307 \cdot in^6$	
Distance between fillet:	$T = 1.625 \cdot in$	Polar radius of gyration: (about shear center)	$r_o = 1.53 \cdot in$	
Effective radius of gyration:	$r_{ts} = 0.469 \cdot in$	Distance to centroid: (from web)	$x_{bar} = 0.437 \cdot in$	
Distance between flg centriods:	$h_o = 2.73 \cdot in$			
Shear center:	$e_o = 0.461 \cdot in$			
Flexural Constant:	$H = 0.655$			

Strong Axis Flexural Capacity

AISC 360-05 Section F2

Unbraced length: $L_{bx} := \frac{5ft}{2} - 10.5in = 1.625\,ft$

Limiting unbraced length for limit state of yielding: $L_p := 1.76 \cdot r_y \cdot \sqrt{\frac{E_s}{F_y}} = 1.66\,ft$ (F2-5) $c := \frac{h_o}{2} \cdot \sqrt{\frac{I_y}{C_w}}$ (F2-8b)

Limiting unbraced length for limit state of inelastic lateral-torsional buckling: $L_r := 1.95 \cdot r_{ts} \cdot \frac{E_s}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E_s} \cdot \frac{S_x \cdot h_o}{J \cdot c}\right)^2}} = 12.3\,ft$ (F2-6)

Yielding

$M_p := F_y \cdot Z_x = 3.96 \cdot kip \cdot ft$ (F2-1)

Lateral Torsional Buckling

$C_b := 1.0$ per AISC 360-05 F1

When Lp <Lb < Lr: $M_{n\_LTB} := C_b \cdot \left[(M_p) - (M_p - 0.7 \cdot F_y \cdot S_x) \frac{(L_{bx} - L_p)}{(L_r - L_p)}\right] = 3.965 \cdot kip \cdot ft$ (F2-2)

When Lb > Lr: $F_{cr} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_{bx}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_o} \cdot \left(\frac{L_{bx}}{r_{ts}}\right)^2} = 251.12 \cdot ksi$ (F2-4)

$M_{n\_EB} := F_{cr} \cdot S_x = 23.019 \cdot kip \cdot ft$ (F2-3)

Nominal & Allowable Moment

$$M_{nx} := \begin{cases} M_p & \text{if } L_{bx} \le L_p \\ \min(M_{n\_LTB}, M_p) & \text{if } L_p < L_{bx} \le L_r \\ \min(M_{n\_EB}, M_p) & \text{otherwise} \end{cases}$$

$M_{nx} = 3.96 \cdot kip \cdot ft$

Allowable Moment about X axis: $M_{ax} := \frac{M_{nx}}{FS} = 0.792 \cdot kip \cdot ft$

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Strong Axis Shear Capacity

AISC 360-05 Section G2

Distance between flanges: $h := T = 1.625 \cdot in$

Web buckling coeffiecient: $k_v := 5$ if $\frac{h}{t_w} = 9.6$ < 260 per G.2.1.b.i Verify h/tw < 260 for unstiffened webs

Web Shear Coefficient:

$$C_v := \begin{cases} 1.0 & \text{if } \frac{h}{t_w} \le 1.10\sqrt{\frac{k_v \cdot E_s}{F_y}} \\ \frac{1.10 \cdot \sqrt{\frac{k_v \cdot E_s}{F_y}}}{\frac{h}{t_w}} & \text{if } 1.10\sqrt{\frac{k_v \cdot E_s}{F_y}} < \frac{h}{t_w} \le 1.37\sqrt{\frac{k_v \cdot E_s}{F_y}} \\ \frac{1.51 \cdot E_s \cdot k_v}{\left(\frac{h}{t_w}\right)^2 \cdot F_y} & \text{otherwise} \end{cases}$$

(G2-3)

(G2-4)

(G2-5)

$C_v = 1.0$

Shear Area: $A_w := d \cdot t_w = 0.51 \cdot in^2$

Nominal & Allowable Shear

$V_{ny} := 0.6 \cdot F_y \cdot A_w \cdot C_v = 11.016 \cdot kip$ (G2-1)

Allowable Y axis Shear: $V_{ay} := \frac{V_{ny}}{FS} = 2.203 \cdot kip$

Allowable Capacity Summary for prof = "C3X4.1"

Strong axis bending: $M_{ax} = 0.792 \cdot kip \cdot ft$

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: $V_y := \frac{P_{des}}{2} = 1250\,lbf$

Maximum axial force from the upright column

Factor of Safety: $FS_{V\_bab} := \frac{V_{ny}}{V_y} = 8.813$

Strong moment: $M_x := \frac{0.5 \cdot P_{des}}{5 \cdot ft} \cdot \frac{5 \cdot ft}{2} \cdot \frac{5 \cdot ft}{4} = 0.781 \cdot kip \cdot ft$

assuming that the the load from the column is continuously supported along the bottom from existing steel

Factor of Safety: $FS_{bab} := \frac{M_{nx}}{M_x} = 5.069$

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:

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

Interaction:

$$INT_{M\_lcb} := \frac{M_{legb}}{M_{ax}} = 0.543$$

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.

$b_{br} := 2 \cdot in$ $\quad d_{br} := b_{br} = 2 \cdot in$ $\quad th_{br} := \frac{3}{16} \cdot in$ $\quad Fy_{br} := 36 \cdot ksi$ $\quad r_{br} := 0.612 \cdot in$ $\quad A_{br} := 0.722 \cdot in^2$

$\frac{b_{br}}{th_{br}} = 10.667$ $\quad 0.45 \cdot \sqrt{\frac{E_s}{Fy_{br}}} = 12.772$ $\quad$ Nonslender, use E3

Unbraced length: $Lb_{br} := 3 \cdot ft + 1 \cdot in = 3.083\,ft$

E3 - Compression

$k\_Lr_{br} := \frac{1.0 \cdot Lb_{br}}{r_{br}} = 60.458$ $\quad 4.71 \cdot \sqrt{\frac{E_s}{Fy_{br}}} = 133.681$

Elastic buckling stress: $Fe_{br} := \frac{\pi^2 \cdot E_s}{\left(k\_Lr_{br}\right)^2} = 78.306 \cdot ksi$

Critical stress: $Fcr_{br} := 0.658^{\frac{Fy_{br}}{Fe_{br}}} \cdot Fy_{br}$ $\quad Fcr_{br} = 29.699 \cdot ksi$

Nominal compressive strength: $P_{n\_c} := Fcr_{br} \cdot A_{br}$ $\quad P_{n\_c} = 21.442 \cdot kip$

Allowable compressive strength: $P_{a\_c} := \frac{P_{n\_c}}{FS}$ $\quad P_{a\_c} = 4.288 \cdot kip$

The maximum axial load applied to each brace, based on the continous loading explained above: $P_{brace} := 625 \cdot lbf$

Factor of Safety: $FS_{br\_1} := \frac{P_{n\_c}}{P_{brace}} = 34.308$

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: $l_{vert\_hg} := 3 \cdot ft$

Horizontal distance to load from end at top: $l_{horz\_hg} := \frac{3 \cdot ft}{2} = 1.5\,ft$

Length of cable: $l_{cable} := \sqrt{l_{vert\_hg}^{\,2} + l_{horz\_hg}^{\,2}} = 3.354\,ft$

Total load being hung: $P_{total\_hg} := P_{des} = 2500\,lbf$

Total load applied to each of the 2 cables: $P_{half\_total} := \frac{P_{total\_hg}}{2} = 1250\,lbf$ *This is also the vertical component of the load at the top of the upright*

Tension in each cable: $T_{cable} := \frac{P_{half\_total}}{\sin\left(\frac{l_{cable}}{l_{vert\_hg}}\right)} = 1390.059\,lbf$

Horizontal component: $P_{horz\_top} := T_{cable} \cdot \cos\left(\frac{l_{cable}}{l_{vert\_hg}}\right) = 608.083\,lbf$ *at the top of the cable*

Rigging Equipment Capacities

The rigging equipment (shackle, cable, pear ring) will see the load applied to the hanging load from the ring.

5/8" pear ring: $Pear_{cap} := 4.2 \cdot kip = 4200\,lbf$

Factor of Safety: $FS_{pear} := \frac{Pear_{cap}}{T_{cable}} = 3.021$ *acceptable FS for pear ring*

3/8" 7x19 IWRC wire rope breaking strength: $Wire_{cap} := 1800 \cdot lbf \cdot 8 = 14.4 \cdot kip$

Factor of Safety: $FS_{wire} := \frac{Wire_{cap}}{T_{cable}} = 10.359$

5/8" shackle: $Shack_{cap} = 6500\,lbf$

Factor of Safety: $FS_{shack2} := \frac{FS \cdot Shack_{cap}}{T_{cable}} = 23.38$

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:

Length of the third side of the plate: $l_{3gp} := \sqrt{(4\cdot in)^2 + (4\cdot in)^2} = 5.657\cdot in$

If each side of the plate is 4", depth of the plate in the 45 degree direction: $d_{gp} := \sqrt{(4\cdot in)^2 - \left(\frac{l_{3gp}}{2}\right)^2} = 2.828\cdot in$

AISC - J3.10:

Clear distance in the direction of the force: $l_{c\_gp} := \frac{d_{gp}}{2} - \frac{\frac{13}{16}\cdot in}{2} = 1.008\cdot in$

Thickness of the connected material: $t_{gp} := \frac{3}{8}\cdot in$

Bolt diameter: $d_{b\_gp} := \frac{13}{16}\cdot in - \frac{1}{16}\cdot in = 0.75\cdot in$

Nominal Bearing Strength: $R_{n\_b} := \min(1.2\cdot l_{c\_gp}\cdot t_{gp}\cdot F_u, 2.4\cdot d_{b\_gp}\cdot t_{gp}\cdot F_u)$ $R_{n\_b} = 26.308\cdot kip$

Allowable Bearing Strength: $R_{a\_b} := \frac{R_{n\_b}}{FS}$ $R_{a\_b} = 5.262\cdot kip$

Factor of Safety: $FS_{gp\_brg} := \frac{R_{n\_b}}{T_{cable}} = 18.926$

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Weld of the Gusset to the top plate and the vertical member

Length of the weld on each side of the plate: $L_{weld} := 4 \cdot in$

Total length of the weld on the plate, both sides: $L_{tweld} := 2 \cdot 2 \cdot L_{weld} = 16 \cdot in$

Weld strength: $W_s := 0.60 \cdot (70 \cdot ksi) \cdot \left(\frac{\sqrt{2}}{2}\right) \cdot \frac{4}{16} \cdot in$ $W_s = 7.425 \cdot \frac{kip}{in}$

Total weld strength: $W_{ts} := L_{tweld} \cdot W_s = 118.794 \cdot kip$

Factor of Safety: $FS_{weld\_gp} := \frac{W_{ts}}{T_{cable}} = 85.46$

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:

Clear distance in the direction of the force: $l_c := \frac{3}{4} \cdot in - \frac{1}{2} \cdot \frac{9}{16} \cdot in = 0.469 \cdot in$

Thickness of the connected material: $t_{pl} := \frac{3}{8} \cdot in$

Bolt diameter: $d_b := 0.375 \cdot in$

Nominal Bearing Strength: $R_{n\_b} := \min\left(1.2 \cdot l_c \cdot t_{pl} \cdot F_u, 2.4 \cdot d_b \cdot t_{pl} \cdot F_u\right)$ $R_{n\_b} = 12.234 \cdot kip$

Allowable Bearing Strength: $R_{a\_b} := \frac{R_{n\_b}}{2.00}$ $R_{a\_b} = 6.117 \cdot kip$

Horizontal load at the top of the cable: $P_{horz\_top} = 608.083\ lbf$ *calculated above*

Horizontal load in each bolt in the palte: $P_{h\_eab} := \frac{P_{horz\_top}}{4} = 152.021\ lbf$

Factor of Safety: $FS_{tp\_brg} := \frac{R_{n\_b}}{P_{h\_eab}} = 80.478$

Capacity of 3/8" Bolts

Grade 5

Bolt diameter: $d_{b2} := \frac{3}{8} \cdot in$ $\qquad A_{b2} := \frac{d_{b2}^{\,2} \cdot \pi}{4}$

Bolt tensile strength: $Ta_{bolt2} := 0.75 \cdot Fu_{bolt} \cdot A_{b2}$ $\qquad Ta_{bolt2} = 9.94 \cdot kip$

Factor of Safety: $FS_{t\_b2} := \frac{Ta_{bolt2}}{P_{half\_total}} = 7.952$

Bolt shear strength: $Va_{bolt2} := 0.4 \cdot Fu_{bolt} \cdot A_{b2}$ $\qquad Va_{bolt2} = 5.301 \cdot kip$

Factor of Safety: $FS_{v\_b2} := \frac{Va_{bolt2}}{P_{h\_eab}} = 34.873$

W4 top beam

The W4x13 crossbar will receive compression from the top plate.

Allowable Compression Capacity & Compression Stress **AISC 360-10 Section E3**

$\lambda_{fc} := \frac{b_f}{2 \cdot t_f} = 2.582$ $\lambda_{rf\_c} := 0.56 \cdot \sqrt{\frac{E_s}{F_y}} = 15.894$ **Flange is slender**

$\lambda_{wc} := \frac{d - 2 \cdot t_f}{t_w} = 14.435$ $\lambda_{rw\_c} := 1.49 \cdot \sqrt{\frac{E_s}{F_y}} = 42.29$ **Web in non slender**

Unbraced length for X,Y axis: $L_x := 3ft$ $L_y := 3ft$

Column Slenderness ratio: $klr_x := \frac{1.0 \cdot L_x}{r_x} = 30.508$ $klr_y := \frac{1.0 \cdot L_y}{r_y} = 90.452$ $klr := \max(klr_x, klr_y) = 90.452$

Euler Buckling Stress: $F_e := \frac{\pi^2 \cdot E_s}{klr^2} = 34.983 \cdot ksi$ (E3-4) $Q := 1.415 - 0.74 \cdot \frac{b_f}{2 \cdot t_f} \cdot \sqrt{\frac{F_y}{E_s}} = 1.262$ *Qs only*

Critical Compression Stress:

$$F_{cr} := \begin{cases} Q \cdot (0.658)^{\frac{Q \cdot F_y}{F_e}} \cdot F_y & \text{if } klr \le 4.71 \sqrt{\frac{E_s}{Q \cdot F_y}} \\ 0.877 \cdot F_e & \text{otherwise} \end{cases}$$

(E3-2) $F_{cr} = 26.377 \cdot ksi$

(E3-3)

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

Factor of Safety: $FS_{comp\_cb} := \frac{C_n}{P_{horz\_top}} = 52.053$

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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): $P_{n\_c} = 21442.341\ lbf$

Factor of Safety: $FS_{br\_R} := \frac{P_{n\_c}}{P_{braceR}} = 42.885$

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.

Length of the vertical where brace is located on upright: $l_{v\_br} := 3.04 \cdot ft$ Horizontal: $l_{h\_br} := 1.54 \cdot ft$

Length of the brace using those dimensions: $l_{br} := \sqrt{l_{v\_br}^2 + l_{h\_br}^2} = 3.408\ ft$

Vertical component of the angle brace: $P_{vert\_br} := P_{braceR} \cdot \sin\left(\frac{l_{br}}{l_{v\_br}}\right)$ $P_{vert\_br} = 450.266\ lbf$

Moment applied to the weld: $M_{weld\_top} := P_{vert\_br} \cdot 1 \cdot in$ $M_{weld\_top} = 0.45 \cdot kip \cdot in$ Size of weld: $S_w := \frac{1}{4} \cdot in$

Section modulus of the weld: $S_{w\_top} := 2 \cdot in \cdot (2 \cdot in + S_w)$ $S_{w\_top} = 4.5 \cdot \frac{in^3}{in}$

Total length of weld: $L_{w\_top} := 2 \cdot 2 \cdot in = 4 \cdot in$ Ultimate weld stress: $f_{u\_weld} := 70 \cdot ksi$

Effective throat of the fillet weld: $E_w := S_w \frac{\sqrt{2}}{2}$ $E_w = 0.177 \cdot in$

Nominal weld stress: $f_{weld} := f_{u\_weld} \cdot E_w$ $f_{weld} = 12.374 \cdot \frac{kip}{in}$

Nominal moment capacity of the weld: $M_{weld} := f_{weld} \cdot S_{w\_top}$ $M_{weld} = 55.685 \cdot kip \cdot in$

Factor of Safety: $FS_{w\_top} := \frac{M_{weld}}{M_{weld\_top}} = 123.671$

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

Horizontal component of the angle brace: $P_{horz\_br} := \left| P_{braceR} \cdot \cos\left(\frac{l_{br}}{l_{h\_br}}\right) \right| = 299.427\ \text{lbf}$

Moment applied to the weld: $M_{weld\_bott} := P_{horz\_br} \cdot 2 \cdot in$ $\quad M_{weld\_bott} = 0.599 \cdot kip \cdot in$

Total length of weld: $L_{w\_bott} := 3 \cdot in + 2 \cdot in = 5 \cdot in$ $\quad$ Ultimate weld stress: $f_{u\_weld} = 70 \cdot ksi$

Effective throat of the fillet weld: $E_w = 0.177 \cdot in$ $\quad$ Size of weld: $S_w = 0.25 \cdot in$

Section modulus of the weld: $S_{w\_bott} := \frac{4 \cdot (2 \cdot in - 0.5 \cdot S_w) \cdot (3 \cdot in - 0.5 \cdot S_w) + (3 \cdot in - 0.5 \cdot S_w)^2}{6}$ $\quad S_{w\_bott} = 4.971 \cdot \frac{in^3}{in}$

Nominal weld stress: $f_{weld} = 12.374 \cdot \frac{kip}{in}$

Nominal moment capacity of the weld: $M_{weld\_b} := f_{weld} \cdot S_{w\_bott}$ $\quad M_{weld\_b} = 61.517 \cdot kip \cdot in$

Factor of Safety: $FS_{w\_bott} := \frac{M_{weld\_b}}{M_{weld\_bott}} = 102.725$

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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:

Clear distance in the direction of the force: $l_c := \frac{2 \cdot in}{2} - \frac{\frac{7}{16} \cdot in}{2} = 0.781 \cdot in$

Thickness of the connected material: $t_{ang} := \frac{3}{16} \cdot in$

Bolt diameter: $d_b = 0.375 \cdot in$

Nominal Bearing Strength: $R_{n\_b} := \min\left(1.2 \cdot l_c \cdot t_{ang} \cdot F_u, 2.4 \cdot d_b \cdot t_{ang} \cdot F_u\right)$ $R_{n\_b} = 9.787 \cdot kip$

Factor of Safety: $FS_{ang\_brg} := \frac{R_{n\_b}}{\max\left(P_{vert\_br}, P_{horz\_br}\right)} = 21.737$

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

Uprights

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

Total length: $L_{tot\_upr} = 3.625\, ft$

Plastic section modulus for HSS: $Z_x := 2.48 \cdot in^3$

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: $M_{nx} := F_y \cdot Z_x = 89.28 \cdot kip \cdot in$ (F7-1)

Factor of Safety: $FS_{mx\_upr} := \dfrac{M_{nx}}{M_{upr}} = 10.629$

Weld from upright to the bottom of the top plate has the RISA moment applied to it.

Moment applied to the weld: $M_{upr} = 0.7 \cdot kip \cdot ft$

Section modulus of the weld: $S_{w\_upr} := \frac{(3 \cdot in)^2}{3}$ $S_{w\_upr} = 3 \cdot \frac{in^3}{in}$

Total length of weld: $L_{w\_upr} := 2 \cdot 3 \cdot in$

Ultimate weld stress: $f_{u\_weld} = 70 \cdot ksi$

Effective throat of the fillet weld: $E_w = 0.177 \cdot in$

Size of weld: $S_w = 0.25 \cdot in$

Nominal weld stress: $f_{weld} = 12.374 \cdot \frac{kip}{in}$

Nominal moment capacity of the weld: $M_{weld\_upr} := f_{weld} \cdot S_{w\_upr}$ $M_{weld\_upr} = 37.123 \cdot kip \cdot in$

Factor of Safety: $FS_{w\_upr} := \frac{M_{weld\_upr}}{M_{upr}} = 4.419$

acceptable for this fictional redundancy based load path