ECLIPSE ENGINEERING INC.

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Structural Calculations

Steel Storage Racks

By Pipp Mobile Storage Systems, Inc.
PIPP P.O. #107042 S.O. #312976

BCBG Maxazria
Pioneer Place
700 SW Fifth Avenue - Space #3015
Portland, Oregon 97204

Prepared For:
Pipp Mobile Storage Systems, Inc.
2966 Wilson Drive NW
Walker, MI 49544

Please note: The calculations contained within justify the seismic resistance of the shelving racks, the fixed and mobile base supports, and the connection to the existing partition walls for both lateral and overturning forces as required by the 2010 Oregon Structural Specialty Code. These storage racks are not accessible to the general public.

STEEL ANTI-TIP CLIP AND ANTI-TIP TRACK DESIGN

Tension (Uplift) Force on each side - $T = 154.1\,lb$

Connection from Shelf to Carriage = 1/4" diameter bolt through 14 ga. steel:

Capacity of #12 screw (smaller than 1/4" diam. bolt) in 16 ga. steel (thinner than 14 ga. posts and clips) - $Z_c := 349\cdot lb$

$\text{if}\left(T < 2\cdot Z_c, \text{"(2) 1/4" Bolts are Adequate"}, \text{"No Good"}\right) = \text{"(2) 1/4" Bolts are Adequate"}$

Use 3/16" Diameter anti-tip device for connection of carriage to track

Yield Stress of Angle Steel -	$F_y := 36\cdot ksi$	
Thickness of Anti-tip Head -	$t_a := 0.090\cdot in$	
Width of Anti-tip Rod + Radius -	$b_r := 0.25\cdot in$	
Width of Anti-tip Head -	$b_a := 0.490\cdot in$	
Width of Anti-tip Flange -	$L_a := \frac{b_a - b_r}{2}$	$L_a = 0.12\cdot in$
Tension Force per Flange leg -	$T_l := 0.5\cdot T$	$T_l = 77.07\,lb$
Bending Moment on Leg -	$M_l := \frac{T_l\cdot L_a}{2}$	$M_l = 0.39\cdot ft\cdot lb$
Section Modulus of Leg -	$S_l := \frac{b_a\cdot t_a^2}{6}$	$S_l = 0.001\cdot in^3$
Bending Stress on Leg -	$f_b := \frac{M_l}{S_l}$	$f_b = 6.99\cdot ksi$
Ratio of Allowable Loads -	$\frac{f_b}{0.75\cdot F_y} = 0.259$	MUST BE LESS THAN 1.0
Width of Anti-Tip track -	$L := 5.1\cdot in$	
Thickness of Aluminum Track -	$t_t := 0.33\cdot in$	Average Thickness
Spacing of Bolts -	$S_{tb} := 22.5\cdot in$	
Section Modulus of Track -	$S_t := \frac{L\cdot t_t^2}{6}$	$S_t = 0.0926\cdot in^3$
Design Moment on Track - for continuous track section	$M := \frac{T\cdot S_{tb}}{8}$	$M = 36.13\,ft\cdot lb$
Bending Stress on Track -	$f_b := \frac{M}{S_t}$	$f_b = 4.68\cdot ksi$
Allowable Stress of Aluminum -	$F_b := 21\cdot ksi$	
Ratio of Allowable Loads -	$f_b\cdot F_b^{-1} = 0.223$	MUST BE LESS THAN 1.0

ANTI-TIP CLIP STEEL CONNECTION AND TRACK ARE ADEQUATE

Connection from Steel Racks to Wall

Seismic Analysis Procedure per ASCE-7 Section 13.3.1:

Average Roof Height - $h_r = 32\,ft$

Height of Rack Attachments - $z_b := z + h_t = 25\,ft$ At Top for fixed racks connected to walls

Seismic Base Shear Factor - $V_t := \dfrac{0.4 \cdot a_p \cdot S_{DS}}{\dfrac{R_p}{I_p}} \cdot \left(1 + 2 \cdot \dfrac{z_b}{h_r}\right)$ $V_t = 0.465$

Shear Factor Boundaries - $V_{tmin} := 0.3 \cdot S_{DS} \cdot I_p$ $V_{tmin} = 0.218$

$V_{tmax} := 1.6 \cdot S_{DS} \cdot I_p$ $V_{tmax} = 1.162$

$V_t := \text{if}\left(V_t > V_{tmax}, V_{tmax}, V_t\right)$

$V_t := \text{if}\left(V_t < V_{tmin}, V_{tmin}, V_t\right)$ $V_t = 0.465$

Seismic Coefficient - $V_t = 0.465$

Number of Shelves - $N = 8$

Weight per Shelf - $W_{tj} = 50\,lb$

Total Weight on Rack - $W_T := 0.667 \cdot 4 \cdot P_p$ $W_T = 368.15\,lb$

Seismic Force at top and bottom - $T_v := \dfrac{0.7 \cdot V_t \cdot W_T}{2}$ $T_v = 59.95\,lb$

Connection at Top:

Standard Stud Spacing - $S_{stud} := 16 \cdot in$

Width of Rack - $w = 2.5\,ft$

Number of Connection Points - on each rack $N_c := \text{floor}\left(\dfrac{w}{S_{stud}}\right)$ $N_c = 1$

Force on each connection point - $F_c := \dfrac{T_v}{N_c}$ $F_c = 59.95\,lb$

Capacity per inch of embedment - $W_s := 135 \cdot \dfrac{lb}{in}$

Required Embedment - $d_s := \dfrac{F_c}{W_s}$ $d_s = 0.444 \cdot in$

For Steel Studs:

Pullout Capacity in 20 ga studs - per Scafco $T_{20} := 84 \cdot lb$ For #10 Screw - per Scafco

MIN #10 SCREW ATTACHED TO EXISTING WALL STUD IS ADEQUATE TO RESIST SEISMIC FORCES ON SHELVING UNITS. EXPANSION BOLT IS ADEQUATE BY INSPECTION AT THE BASE

Pipp Mobile STEEL STORAGE RACK DESIGN
2009 IBC & 2010 CBC - 2208 & ASCE 7-05 - 15.5.3

Design Vertical Steel Posts at Each Corner :

$plf := lb \cdot ft^{-1}$
$psf := lb \cdot ft^{-2}$
$pcf := lb \cdot ft^{-3}$
$ksi := 1000 \cdot lb \cdot in^{-2}$
$kips := 1000 \cdot lb$

Shelving Dimensions:

Total Height of Shelving Unit -	$h_t := 9 \cdot ft$	
Width of Shelving Unit -	$w := 3 \cdot ft$	
Depth of Shelving Unit -	$d := 3 \cdot ft$	
Number of Shelves -	$N := 4$	
Vertical Shelf Spacing -	$S := 36 \cdot in$	

Shelving Loads:

Maximum Live Load on each shelf is 100 lbs:

Weight per shelf -	$W_{tj} := 100 \cdot lb$	$W_{tj} = 100\,lb$
Load in psf -	$LL_j := \dfrac{W_{tj}}{w \cdot d}$	$LL_j = 11.1111 \cdot psf$
Design Live Load on Shelf -	$LL := LL_j$	$LL = 11.1111 \cdot psf$
Dead Load on Shelf -	$DL := 2.50 \cdot psf$	

Section Properties of Double Rivet 'L' Post :

Modulus of Elasticity of Steel -	$E := 29000 \cdot ksi$	$b := 1.5 \cdot in$ $h := 1.5 \cdot in$
Steel Yield Stress -	$F_y := 33 \cdot ksi$	$r_y := 0.47 \cdot in$
Section Modulus in x and y -	$S_x := 0.04 \cdot in^3$	$r_x := 0.47 \cdot in$
Moment of Inertia in x and y -	$I_x := 0.06 \cdot in^4$	$t := 0.075 \cdot in$
Full Cross Sectional Area -	$A_p := 0.22 \cdot in^2$	$h_c := 1.42 \cdot in$ $b_c := 1.42 \cdot in$

Length of Unbraced Post -	$L_x := S = 36 \cdot in$	$L_y := S = 36 \cdot in$	$L_t := S = 36 \cdot in$
Effective Length Factor -	$K_x := 1.0$	$K_y := 1.0$	$K_t := 1.0$

Section Properties Continued:

Density of Steel -	$\rho steel := 490 \cdot pcf$	
Weight of Post -	$W_p := \rho steel \cdot A_p \cdot h_t$	$W_p = 6.74 \cdot lb$
Vertical DL on Post -	$P_d := DL \cdot w \cdot .25d \cdot N + W_p$	$P_d = 29.24\,lb$
Vertical LL on Post -	$P_l := LL \cdot w \cdot .25 \cdot d \cdot N$	$P_l = 100\,lb$
Total Vertical Load on Post -	$P_p := P_d + P_l$	$P_p = 129.24 \cdot lb$

Floor Load Calculations :

Weight of Mobile Carriage:	$W_c := 90 \cdot lb$	
Total Load on Each Unit:	$W := 4 \cdot P_p + W_c$	$W = 606.95\,lb$
Area of Each Shelf Unit:	$A_u := (w + 9in) \cdot (d + 3in)$	$A_u = 12.1875\,ft^2$
Floor Load under Shelf:	$PSF := \dfrac{W}{A_u}$	$PSF = 50 \cdot psf$

NOTE: SHELVING LIVE LOAD IS CONSISTENT WITH 75 psf REQ'D FOR RETAIL FLOOR LOADING

Find the Seismic Load using Full Design Live Load :

ASCE-7 Seismic Design Procedure:

Importance Factor -	$I_E := 1.0$	
Determine S_S and S_1 from maps -	$S_s := 0.985$	$S_1 := 0.345$
Determine the Site Class -	Class D	
Determine F_a and F_v -	$F_a := 1.106$	$F_v := 1.71$
Determine S_{MS} and S_{M1} -	$S_{MS} := F_a \cdot S_s$	$S_{M1} := F_v \cdot S_1$
	$S_{MS} = 1.0894$	$S_{M1} = 0.5899$
Determine S_{DS} and S_{DI} -	$S_{DS} := \dfrac{2}{3} \cdot S_{MS}$	$S_{DI} := \dfrac{2}{3} \cdot S_{M1}$
	$S_{DS} = 0.726$	$S_{DI} = 0.393$

Structural System - Section 15.5.3 ASCE-7:

4. Steel Storage Racks	$R := 4.0$	$\Omega_o := 2$	$C_d := 3.5$
	$R_p := R$	$a_p := 2.5$	$I_p := 1.0$

Total Vertical LL Load on Shelf -	$W_l := LL \cdot w \cdot d$	$W_l = 100\,lb$
Total Vertical DL Load on Shelf -	$W_d := DL \cdot w \cdot d + 4 \cdot \dfrac{W_p}{N}$	$W_d = 29.24\,lb$

Seismic Analysis Procedure per ASCE-7 Section 13.3.1:

Average Roof Height -	$h_r := 32.0 \cdot ft$	
Height of Rack Attachment -	$z := 16.0 \cdot ft$	(0'-0" For Ground floor)
Seismic Base Shear Factor -	$V_t := \dfrac{0.4 \cdot a_p \cdot S_{DS}}{\dfrac{R_p}{I_p}} \cdot \left(1 + 2 \cdot \dfrac{z}{h_r}\right)$	$V_t = 0.363$
Shear Factor Boundaries -	$V_{tmin} := 0.3 \cdot S_{DS} \cdot I_p$	$V_{tmin} = 0.218$
	$V_{tmax} := 1.6 \cdot S_{DS} \cdot I_p$	$V_{tmax} = 1.162$
	$V_t := \text{if}\left(V_t > V_{tmax}, V_{tmax}, V_t\right)$	
	$V_t := \text{if}\left(V_t < V_{tmin}, V_{tmin}, V_t\right)$	$V_t = 0.363$

Pipp Mobile STEEL STORAGE RACK DESIGN
2009 IBC & 2010 CBC - 2208 & ASCE 7-05 - 15.5.3

Design Vertical Steel Posts at Each Corner :

$plf := lb \cdot ft^{-1}$

$psf := lb \cdot ft^{-2}$

$pcf := lb \cdot ft^{-3}$

$ksi := 1000 \cdot lb \cdot in^{-2}$

$kips := 1000 \cdot lb$

Shelving Dimensions:

Total Height of Shelving Unit -	$h_t := 9 \cdot ft$	
Width of Shelving Unit -	$w := 2.5 \cdot ft$	
Depth of Shelving Unit -	$d := 2.5 \cdot ft$	
Number of Shelves -	$N := 8$	
Vertical Shelf Spacing -	$S := 15.43 \cdot in$	

Shelving Loads:

Maximum Live Load on each shelf is 50 lbs:

Weight per shelf -	$W_{tj} := 50 \cdot lb$	$W_{tj} = 50\,lb$
Load in psf -	$LL_j := \dfrac{W_{tj}}{w \cdot d}$	$LL_j = 8 \cdot psf$
Design Live Load on Shelf -	$LL := LL_j$	$LL = 8 \cdot psf$
Dead Load on Shelf -	$DL := 2.50 \cdot psf$	

Section Properties of Double Rivet 'L' Post :

		$b := 1.5 \cdot in$
Modulus of Elasticity of Steel -	$E := 29000 \cdot ksi$	$h := 1.5 \cdot in$
Steel Yield Stress -	$F_y := 33 \cdot ksi$	$r_y := 0.47 \cdot in$
Section Modulus in x and y -	$S_x := 0.04 \cdot in^3$	$r_x := 0.47 \cdot in$
Moment of Inertia in x and y -	$I_x := 0.06 \cdot in^4$	$t := 0.075 \cdot in$
Full Cross Sectional Area -	$A_p := 0.22 \cdot in^2$	$h_c := 1.42 \cdot in$
		$b_c := 1.42 \cdot in$

Length of Unbraced Post -	$L_x := S = 15.43 \cdot in$	$L_y := S = 15.43 \cdot in$	$L_t := S = 15.43 \cdot in$
Effective Length Factor -	$K_x := 1.0$	$K_y := 1.0$	$K_t := 1.0$

Section Properties Continued:

Density of Steel -	$\rho steel := 490 \cdot pcf$	
Weight of Post -	$W_p := \rho steel \cdot A_p \cdot h_t$	$W_p = 6.74 \cdot lb$
Vertical DL on Post -	$P_d := DL \cdot w \cdot .25 d \cdot N + W_p$	$P_d = 37.99\,lb$
Vertical LL on Post -	$P_l := LL \cdot w \cdot .25 \cdot d \cdot N$	$P_l = 100\,lb$
Total Vertical Load on Post -	$P_p := P_d + P_l$	$P_p = 137.99 \cdot lb$

Floor Load Calculations :

Weight of Mobile Carriage:	$W_c := 90 \cdot lb$	
Total Load on Each Unit:	$W := 4 \cdot P_p + W_c$	$W = 641.95\,lb$
Area of Each Shelf Unit:	$A_u := (w + 9in) \cdot (d + 3in)$	$A_u = 8.9375\,ft^2$
Floor Load under Shelf:	$PSF := \frac{W}{A_u}$	$PSF = 72 \cdot psf$

NOTE: SHELVING LIVE LOAD IS CONSISTENT WITH 75 psf REQ'D FOR RETAIL FLOOR LOADING

Find the Seismic Load using Full Design Live Load :

ASCE-7 Seismic Design Procedure:

Importance Factor -	$I_E := 1.0$		
Determine S_S and S_1 from maps -	$S_s := 0.985$		$S_1 := 0.345$
Determine the Site Class -	Class D		
Determine F_a and F_v -	$F_a := 1.106$		$F_v := 1.71$
Determine S_{MS} and S_{M1} -	$S_{MS} := F_a \cdot S_s$		$S_{M1} := F_v \cdot S_1$
	$S_{MS} = 1.0894$		$S_{M1} = 0.5899$
Determine S_{DS} and S_{DI} -	$S_{DS} := \frac{2}{3} \cdot S_{MS}$		$S_{DI} := \frac{2}{3} \cdot S_{M1}$
	$S_{DS} = 0.726$		$S_{DI} = 0.393$

Structural System - Section 15.5.3 ASCE-7:

4. Steel Storage Racks	$R := 4.0$	$\Omega_o := 2$	$C_d := 3.5$
	$R_p := R$	$a_p := 2.5$	$I_p := 1.0$
Total Vertical LL Load on Shelf -	$W_l := LL \cdot w \cdot d$		$W_l = 50\,lb$
Total Vertical DL Load on Shelf -	$W_d := DL \cdot w \cdot d + 4 \cdot \frac{W_p}{N}$		$W_d = 18.99\,lb$

Seismic Analysis Procedure per ASCE-7 Section 13.3.1:

Average Roof Height -	$h_r := 32.0 \cdot ft$	
Height of Rack Attachment -	$z := 16.0 \cdot ft$	(0'-0" For Ground floor)
Seismic Base Shear Factor -	$V_t := \frac{0.4 \cdot a_p \cdot S_{DS}}{\frac{R_p}{I_p}} \cdot \left(1 + 2 \cdot \frac{z}{h_r}\right)$	$V_t = 0.363$
Shear Factor Boundaries -	$V_{tmin} := 0.3 \cdot S_{DS} \cdot I_p$	$V_{tmin} = 0.218$
	$V_{tmax} := 1.6 \cdot S_{DS} \cdot I_p$	$V_{tmax} = 1.162$
	$V_t := if\left(V_t > V_{tmax}, V_{tmax}, V_t\right)$	
	$V_t := if\left(V_t < V_{tmin}, V_{tmin}, V_t\right)$	$V_t = 0.363$

Seismic Loads Continued :

For ASD, Shear may be reduced - $V_p := \frac{V_t}{1.4} = 0.259$

Seismic DL Base Shear - $V_{td} := V_p \cdot W_d \cdot N = 39.41\,lb$

DL Force per Shelf : $F_d := V_p \cdot W_d = 4.93\,lb$

Seismic LL Base Shear - $V_{tl} := V_p \cdot W_l \cdot N = 103.75\,lb$

LL Force per Shelf : $F_l := V_p \cdot W_l = 12.97\,lb$

0.67 * LL Force per Shelf : $F_{l.67} := 0.67 \cdot V_p \cdot W_l = 8.69\,lb$

Force Distribution per ASCE-7 Section 15.5.3.3:

Operating Weight is one of Two Loading Conditions :
Condition #1: Each Shelf Loaded to 67% of Live Weight
Cumulative Heights of Shelves -

$H_1 := 0.0 \cdot S + 1.0 \cdot S + 2.0 \cdot S + 3.0 \cdot S + 4.0 \cdot S + 5.0 \cdot S + 6.0 \cdot S + 7.0 \cdot S$

$H_2 := 0$ $\quad H := H_1 + H_2$ $\quad H = 36\,ft$

Total Moment at Shelf Base - $M_t := H \cdot W_d + H \cdot 0.67 \cdot W_l$ $\quad M_t = 1889.95\,ft \cdot lb$

Vertical Distribution Factors for Each Shelf -

Total Base Shear - $V_{total} := V_{td} + 0.67 \cdot V_{tl}$ $\quad V_{total} = 108.93\,lb$

$$C_1 := \frac{W_d \cdot 0.0 \cdot S + W_l \cdot 0.67 \cdot 0.0 \cdot S}{M_t} = 0 \qquad C_2 := \frac{W_d \cdot 1.0 \cdot S + W_l \cdot 0.67 \cdot 1.0 \cdot S}{M_t} = 0.036$$

$F_1 := C_1 \cdot (V_{total}) = 0$ $\quad F_2 := C_2 \cdot (V_{total}) = 3.89\,lb$

$$C_3 := \frac{W_d \cdot 2.0 \cdot S + W_l \cdot 0.67 \cdot 2.0 \cdot S}{M_t} = 0.071 \qquad C_4 := \frac{W_d \cdot 3.0 \cdot S + W_l \cdot 0.67 \cdot 3.0 \cdot S}{M_t} = 0.107$$

$F_3 := C_3 \cdot (V_{total}) = 7.78\,lb$ $\quad F_4 := C_4 \cdot (V_{total}) = 11.67\,lb$

$$C_5 := \frac{W_d \cdot 4.0 \cdot S + W_l \cdot 0.67 \cdot 4.0 \cdot S}{M_t} = 0.143 \qquad C_6 := \frac{W_d \cdot 5.0 \cdot S + W_l \cdot 0.67 \cdot 5.0 \cdot S}{M_t} = 0.179$$

$F_5 := C_5 \cdot (V_{total}) = 15.56\,lb$ $\quad F_6 := C_6 \cdot (V_{total}) = 19.45\,lb$

$$C_7 := \frac{W_d \cdot 6.0 \cdot S + W_l \cdot 0.67 \cdot 6.0 \cdot S}{M_t} = 0.214 \qquad C_8 := \frac{W_d \cdot 7.0 \cdot S + W_l \cdot 0.67 \cdot 7.0 \cdot S}{M_t} = 0.25$$

$F_7 := C_7 \cdot (V_{total}) = 23.34\,lb$ $\quad F_8 := C_8 \cdot (V_{total}) = 27.23\,lb$

$C_1 + C_2 + C_3 + C_4 + C_5 + C_6 + C_7 + C_8 = 1$ $\quad$ Coefficients should total 1.0

Force Distribution Continued :

Condition #2: Top Shelf Only Loaded to 100% of Live Weight

Total Moment at Base of Shelf - $M_{ta} := (N-1)\cdot S\cdot W_d + (N-1)\cdot S\cdot W_l = 621\,ft\cdot lb$

Total Base Shear - $V_{total2} := V_{td} + F_l$ $V_{total2} = 52.38\,lb$

$$C_{1a} := \frac{W_d\cdot 0.0\cdot S + 0\cdot W_l\cdot 0.0\cdot S}{M_{ta}} = 0 \qquad C_{11a} := \frac{W_d\cdot(N-1)\cdot S + W_l\cdot(N-1)\cdot S}{M_{ta}} = 1$$

$$F_{1a} := C_{1a}\cdot\left(V_{total2}\right) = 0 \qquad F_{11a} := C_{11a}\cdot\left(V_{total2}\right) = 52.38\,lb$$

Condition #1 Controls for Total Base Shear

By Inspection, Force Distribution for intermediate shelves without LL are negligible.

Moment calculation for each column is based on total seismic base shear.

Column at center of rack is the worst case for this shelving rack system.

Column Design in Short Direction : $M_s := \frac{1}{4}\cdot\frac{S}{2}\cdot\left(V_{td} + V_{tl}\right) = 23.01\,ft\cdot lb$

Bending Stress on Column - $f_{bx} := M_s\cdot S_x^{-1} = 6.9\cdot ksi$

Allowable Bending Stress - $F_b := 0.6\cdot F_y = 19.8\cdot ksi$

Bending at the Base of Each Column is Adequate

Deflection of Shelving Bays - worst case is at the bottom bay

$$\Delta := \frac{\left(V_{td} + V_{tl}\right)\cdot S^3}{12\cdot E\cdot I_x} = 0.0252\cdot in \qquad \frac{S}{\Delta} = 612.5716$$

$$\Delta_t := \Delta\cdot(N-1) = 0.1763\cdot in \qquad \Delta_a := 0.05\cdot h_t = 5.4\cdot in$$

$\text{if}\left(\Delta_t < \Delta_a, \text{"Deflection is Adequate"}, \text{"No Good"}\right) = \text{"Deflection is Adequate"}$

Moment at Rivet Connection:

Shear on each rivet -

$$d_r := 0.25\cdot in \qquad V_r := \frac{M_s}{1.5\cdot in} = 184.09\,lb \qquad A_r := \frac{d_r^{\,2}\cdot 3.14}{4} = 0.0491\cdot in^2$$

Steel Stress on Rivet - $f_v := \frac{V_r}{A_r} = 3.75\cdot ksi$

Allowable Stress on Rivet - $F_{vr} := 0.4\cdot 80\cdot ksi = 32\cdot ksi$

RIVET CONNECTION IS ADEQUATE FOR MOMENT CONNECTION FROM BEAM TO POST

Seismic Uplift on Shelves :

Seismic Vertical Component: $E_v := 0.2 \cdot S_{DS} \cdot (DL + LL) \cdot w \cdot d$ $\quad E_v = 9.5323\ lb$

Vertical Dead Load of Shelf: $D := (DL + LL) \cdot w \cdot d$ $\quad D = 65.625\ lb$

Note: since the shelf LL is used to generate the seismic uplift force, it may also be used to calculate the net uplift load. For an empty shelf, only the DL would be used, but the ratio of seismic uplift will be the same.

Net Uplift Load on Shelf: $F_u := E_v - 0.6 \cdot D$ $\quad F_u = -29.8427\ lb$

Note: This uplift load is for the full shelf. Each shelf will be connected at each corner.

Number of Shelf Connections: $N_c := 4$

Uplift Force per Corner: $F_{uc} := \dfrac{F_u}{N_c}$ $\quad F_{uc} = -7.4607\ lb$

NOTE: Since the uplift force is negative, a mechanical connection is not required.

Find Allowable Axial Load for Column :

Allowable Buckling Stresses -

$$\sigma_{ex.x} := \frac{\pi^2 \cdot E}{\left(\frac{K_x \cdot L_x}{r_x}\right)^2} = 265.56 \cdot ksi \qquad \sigma_{ex} := \sigma_{ex.x} = 265.56 \cdot ksi$$

Distance from Shear Center to CL of Web via X-axis

$$e_c := \frac{t \cdot h_c^{\,2} \cdot b_c^{\,2}}{4 \cdot I_x} \qquad e_c = 1.2706 \cdot in$$

Distance From CL Web to Centroid -

$$x_c := 0.649 \cdot in - 0.5 \cdot t \qquad x_c = 0.6115 \cdot in$$

Distance From Shear Center to Centroid -

$$x_o := x_c + e_c \qquad x_o = 1.8821 \cdot in$$

Polar Radius of Gyration -

$$r_o := \sqrt{r_x^{\,2} + r_y^{\,2} + x_o^{\,2}} \qquad r_o = 1.996 \cdot in$$

Torsion Constant -

$$J := \frac{1}{3} \cdot \left(2 \cdot b \cdot t^3 + h \cdot t^3\right) \qquad J = 0.00063 \cdot in^4$$

Warping Constant -

$$C_w := \frac{t \cdot b^3 \cdot h^2}{12} \cdot \left(\frac{3 \cdot b \cdot t + 2 \cdot h \cdot t}{6 \cdot b \cdot t + h \cdot t}\right) \qquad C_w = 0.0339 \cdot in^6$$

Shear Modulus -

$$G := 11300 \cdot ksi$$

$$\sigma_t := \frac{1}{A_p \cdot r_o^{\,2}} \cdot \left[G \cdot J + \frac{\pi^2 \cdot E \cdot C_w}{\left(K_t \cdot L_t\right)^2}\right] \qquad \sigma_t = 54.6557 \cdot ksi$$

$$\beta := 1 - \left(\frac{x_o}{r_o}\right)^2 \qquad \beta = 0.1109$$

$$F_{et} := \frac{1}{2 \cdot \beta} \cdot \left[\left(\sigma_{ex} + \sigma_t\right) - \sqrt{\left(\sigma_{ex} + \sigma_t\right)^2 - 4 \cdot \beta \cdot \sigma_{ex} \cdot \sigma_t}\right] \qquad F_{et} = 46.0616 \cdot ksi$$

Elastic Flexural Buckling Stress -

$$F_e := \mathrm{if}\left(F_{et} < \sigma_{ex}, F_{et}, \sigma_{ex}\right) \qquad F_e = 46.0616 \cdot ksi$$

Allowable Compressive Stress -

$$F_n := \mathrm{if}\left[F_e > \frac{F_y}{2}, F_y \cdot \left(1 - \frac{F_y}{4 \cdot F_e}\right), F_e\right] \qquad F_n = 27.0894 \cdot ksi$$

Factor of Safety for Axial Comp. -

$$\Omega_o := 1.92$$

Find Effective Area -

Determine the Effective Width of Flange -

Flat width of Flange - $w_f := b - 0.5 \cdot t$ $w_f = 1.4625 \cdot in$

Flange Plate Buckling Coefficient - $k_f := 0.43$

Flange Slenderness Factor - $\lambda_f := \dfrac{1.052}{\sqrt{k_f}} \cdot \dfrac{w_f}{t} \cdot \sqrt{\dfrac{F_n}{E}}$ $\lambda_f = 0.9561$

$\rho_f := \left(1 - \dfrac{0.22}{\lambda_f}\right) \cdot \dfrac{1}{\lambda_f}$ $\rho_f = 0.8052$

Effective Flange Width - $b_e := if(\lambda_f > 0.673, \rho_f \cdot w_f, w_f)$ $b_e = 1.1777 \cdot in$

Determine Effective Width of Web -

Flat width of Web - $w_w := h - t$ $w_w = 1.425 \cdot in$

Web Plate Buckling Coefficient - $k_w := 0.43$

Web Slenderness Factor - $\lambda_w := \dfrac{1.052}{\sqrt{k_w}} \cdot \dfrac{w_w}{t} \cdot \sqrt{\dfrac{F_n}{E}}$ $\lambda_w = 0.9316$

$\rho_w := \left(1 - \dfrac{0.22}{\lambda_w}\right) \cdot \dfrac{1}{\lambda_w}$ $\rho_w = 0.8199$

Effective Web Width - $h_e := if(\lambda_w > 0.673, \rho_w \cdot w_w, w_w)$ $h_e = 1.1684 \cdot in$

Effective Column Area - $A_e := t \cdot (h_e + b_e)$ $A_e = 0.176 \cdot in^2$

Nominal Column Capacity - $P_n := A_e \cdot F_n$ $P_n = 4766\,lb$

Allowable Column Capacity - $P_a := \dfrac{P_n}{\Omega_o}$ $P_a = 2483\,lb$

Check Combined Stresses -

$P_{crx} := \dfrac{\pi^2 \cdot E \cdot I_x}{(K_x \cdot L_x)^2}$ $P_{crx} = 7.21 \times 10^4\,lb$

$P_{cr} := P_{crx}$ $P_{cr} = 72130.2\,lb$

Magnification Factor - $\alpha := 1 - \left(\dfrac{\Omega_o \cdot P_p}{P_{cr}}\right)$ $\alpha = 0.996$ $C_m := 0.85$

Combined Stress: $\dfrac{P_p}{P_a} + \dfrac{C_m \cdot f_{bx}}{F_b \cdot \alpha} = 0.353$ MUST BE LESS THAN 1.0

Final Design: **'L' POSTS WITH BEAM BRACKET ARE ADEQUATE FOR REQD COMBINED AXIAL AND BENDING LOADS**

NOTE: P_p is the total vertical load on post, not 67% live load, so the design is conservative

STEEL STORAGE RACK DESIGN - cont'd

Find Overturning Forces :

Total Height of Shelving Unit - $H_t := h_t = 9\,ft$ Width of Shelving Unit - $w = 2.5\,ft$

Depth of Shelving Unit - $d = 2.5\,ft$ WORST CASE

Number of Shelves - $N = 8$ Vertical Shelf Spacing - $S = 15.43\cdot in$

Height to Top Shelf Center of G - $h_{top} := H_t$ $h_{top} = 9\,ft$

Height to Shelf Center of G - $h_c := \frac{(N+1)}{2}\cdot S$ $h_c = 5.7862\cdot ft$

From Vertical Distribution of Seismic Force previously calculated -

Controlling Load Cases -

Weight of Rack and 67% of LL - $W := (W_d + 0.67\cdot W_l)\cdot N$ $W = 419.95\,lb$

Seismic Rack and 67% of LL - $V := V_{td} + 0.67\cdot V_{tl}$ $V = 108.93\,lb$

$M_a := F_1\cdot 0.0\cdot S + F_2\cdot 1.0\cdot S + F_3\cdot 2.0\cdot S + F_4\cdot 3.0\cdot S + F_5\cdot 4.0\cdot S + F_6\cdot 5.0\cdot S + F_7\cdot 6.0\cdot S + F_8\cdot 7.0\cdot S$

$M_b := 0$

Overturning Rack and 67% of LL - $M := M_a + M_b = 700.32\,ft\cdot lb$

Weight of Rack and 100% Top Shelf - $W_a := W_d\cdot N + W_l$ $W_a = 201.95\,lb$

Seismic Rack and 100% Top Shelf - $V_a := V_{td} + F_l$ $V_a = 52.38\,lb$

Overturning Rack and 100% Top Shelf - $M_a := V_{td}\cdot h_c + F_l\cdot h_{top}$ $M_a = 344.78\,ft\cdot lb$

Controlling Weight - $W_c := \text{if}(W > W_a, W, W_a)$ $W_c = 419.95\,lb$

Controlling Shear - $V_c := \text{if}(V > V_a, V, V_a)$ $V_c = 108.93\,lb$

Controlling Moment - $M_{ot} := \text{if}(M > M_a, M, M_a)$ $M_{ot} = 700.32\,ft\cdot lb$

Tension Force on Column Anchor - per side of shelving unit $T := \frac{M_{ot}}{d} - 0.60\cdot\frac{W_c}{2}$ $T = 154.14\,lb$

$T := \text{if}(T < 0\cdot lb, 0\cdot lb, T)$ $T = 154.14\,lb$

Shear Force on Column Anchor - $V := \frac{V_c}{2}$ $V = 54.46\,lb$

USE: HILTI KWIK BOLT TZ ANCHOR (or equivalent) -
USE 3/8"φ x 2" embed installed per the requirements of Hilti

Allowable Tension Force - $T_a := 1006\cdot lb$ For 2500psi Concrete

Allowable Shear Force - $V_a := 999\cdot lb$ Per ASCE 13.4.2 $\rho := 1.3$

Combined Loading - $\left(\frac{\rho\cdot T}{T_a}\right) + \left(\frac{\rho\cdot V}{V_a}\right) = 0.27$ MUST BE LESS THAN 1.00

Seismic Loads Continued :

For ASD, Shear may be reduced - $V_p := \frac{V_t}{1.4} = 0.259$

Seismic DL Base Shear - $V_{td} := V_p \cdot W_d \cdot N = 30.33\,lb$

DL Force per Shelf : $F_d := V_p \cdot W_d = 7.58\,lb$

Seismic LL Base Shear - $V_{tl} := V_p \cdot W_l \cdot N = 103.75\,lb$

LL Force per Shelf : $F_l := V_p \cdot W_l = 25.94\,lb$

0.67 * LL Force per Shelf : $F_{l.67} := 0.67 \cdot V_p \cdot W_l = 17.38\,lb$

Force Distribution per ASCE-7 Section 15.5.3.3:

Operating Weight is one of Two Loading Conditions :
Condition #1: Each Shelf Loaded to 67% of Live Weight
Cumulative Heights of Shelves -

$H_1 := 0.0 \cdot S + 1.0 \cdot S + 2.0 \cdot S + 3.0 \cdot S$

$H_2 := 0$ $\quad H := H_1 + H_2 \quad H = 18\,ft$

Total Moment at Shelf Base - $M_t := H \cdot W_d + H \cdot 0.67 \cdot W_l \quad M_t = 1732.27\,ft \cdot lb$

Vertical Distribution Factors for Each Shelf -

Total Base Shear - $V_{total} := V_{td} + 0.67 \cdot V_{tl} \quad V_{total} = 99.85\,lb$

$$C_1 := \frac{W_d \cdot 0.0 \cdot S + W_l \cdot 0.67 \cdot 0.0 \cdot S}{M_t} = 0 \qquad C_2 := \frac{W_d \cdot 1.0 \cdot S + W_l \cdot 0.67 \cdot 1.0 \cdot S}{M_t} = 0.167$$

$$F_1 := C_1 \cdot (V_{total}) = 0 \qquad F_2 := C_2 \cdot (V_{total}) = 16.64\,lb$$

$$C_3 := \frac{W_d \cdot 2.0 \cdot S + W_l \cdot 0.67 \cdot 2.0 \cdot S}{M_t} = 0.333 \qquad C_4 := \frac{W_d \cdot 3.0 \cdot S + W_l \cdot 0.67 \cdot 3.0 \cdot S}{M_t} = 0.5$$

$$F_3 := C_3 \cdot (V_{total}) = 33.28\,lb \qquad F_4 := C_4 \cdot (V_{total}) = 49.92\,lb$$

$C_1 + C_2 + C_3 + C_4 = 1$ Coefficients should total 1.0

Force Distribution Continued :

Condition #2: Top Shelf Only Loaded to 100% of Live Weight

Total Moment at Base of Shelf - $M_{ta} := (N-1)\cdot S\cdot W_d + (N-1)\cdot S\cdot W_l = 1163\,ft\cdot lb$

Total Base Shear - $V_{total2} := V_{td} + F_l$ $\quad V_{total2} = 56.27\,lb$

$$C_{1a} := \frac{W_d\cdot 0.0\cdot S + 0\cdot W_l\cdot 0.0\cdot S}{M_{ta}} = 0 \qquad C_{11a} := \frac{W_d\cdot(N-1)\cdot S + W_l\cdot(N-1)\cdot S}{M_{ta}} = 1$$

$$F_{1a} := C_{1a}\cdot\left(V_{total2}\right) = 0 \qquad F_{11a} := C_{11a}\cdot\left(V_{total2}\right) = 56.27\,lb$$

Condition #1 Controls for Total Base Shear

By Inspection, Force Distribution for intermediate shelves without LL are negligible.

Moment calculation for each column is based on total seismic base shear.

Column at center of rack is the worst case for this shelving rack system.

Column Design in Short Direction : $M_s := \frac{1}{4}\cdot\frac{S}{2}\cdot\left(V_{td} + V_{tl}\right) = 50.28\,ft\cdot lb$

Bending Stress on Column - $f_{bx} := M_s\cdot S_x^{-1} = 15.08\cdot ksi$

Allowable Bending Stress - $F_b := 0.6\cdot F_y = 19.8\cdot ksi$

Bending at the Base of Each Column is Adequate

Deflection of Shelving Bays - worst case is at the bottom bay

$$\Delta := \frac{\left(V_{td} + V_{tl}\right)\cdot S^3}{12\cdot E\cdot I_x} = 0.2996\cdot in \qquad \frac{S}{\Delta} = 120.1531$$

$$\Delta_t := \Delta\cdot(N-1) = 0.8989\cdot in \qquad \Delta_a := 0.05\cdot h_t = 5.4\cdot in$$

$\text{if}\left(\Delta_t < \Delta_a, \text{"Deflection is Adequate"}, \text{"No Good"}\right) = \text{"Deflection is Adequate"}$

Moment at Rivet Connection:

Shear on each rivet -

$$d_r := 0.25\cdot in \qquad V_r := \frac{M_s}{1.5\cdot in} = 402.26\,lb \qquad A_r := \frac{d_r^{\,2}\cdot 3.14}{4} = 0.0491\cdot in^2$$

Steel Stress on Rivet - $f_v := \frac{V_r}{A_r} = 8.2\cdot ksi$

Allowable Stress on Rivet - $F_{vr} := 0.4\cdot 80\cdot ksi = 32\cdot ksi$

RIVET CONNECTION IS ADEQUATE FOR MOMENT CONNECTION FROM BEAM TO POST

Seismic Uplift on Shelves :

Seismic Vertical Component: $E_v := 0.2 \cdot S_{DS} \cdot (DL + LL) \cdot w \cdot d$ $E_v = 17.7937\,lb$

Vertical Dead Load of Shelf: $D := (DL + LL) \cdot w \cdot d$ $D = 122.5\,lb$

Note: since the shelf LL is used to generate the seismic uplift force, it may also be used to calculate the net uplift load. For an empty shelf, only the DL would be used, but the ratio of seismic uplift will be the same.

Net Uplift Load on Shelf: $F_u := E_v - 0.6 \cdot D$ $F_u = -55.7063\,lb$

Note: This uplift load is for the full shelf. Each shelf will be connected at each corner.

Number of Shelf Connections: $N_c := 4$

Uplift Force per Corner: $F_{uc} := \dfrac{F_u}{N_c}$ $F_{uc} = -13.9266\,lb$

NOTE: Since the uplift force is negative, a mechanical connection is not required.

Find Allowable Axial Load for Column :

Allowable Buckling Stresses -

$$\sigma_{ex.x} := \frac{\pi^2 \cdot E}{\left(\frac{K_x \cdot L_x}{r_x}\right)^2} = 48.79 \cdot ksi \qquad \sigma_{ex} := \sigma_{ex.x} = 48.79 \cdot ksi$$

Distance from Shear Center to CL of Web via X-axis $\quad e_c := \dfrac{t \cdot h_c^2 \cdot b_c^2}{4 \cdot I_x} \quad e_c = 1.2706 \cdot in$

Distance From CL Web to Centroid - $\quad x_c := 0.649 \cdot in - 0.5 \cdot t \quad x_c = 0.6115 \cdot in$

Distance From Shear Center to Centroid - $\quad x_o := x_c + e_c \quad x_o = 1.8821 \cdot in$

Polar Radius of Gyration - $\quad r_o := \sqrt{r_x^2 + r_y^2 + x_o^2} \quad r_o = 1.996 \cdot in$

Torsion Constant - $\quad J := \dfrac{1}{3} \cdot \left(2 \cdot b \cdot t^3 + h \cdot t^3\right) \quad J = 0.00063 \cdot in^4$

Warping Constant - $\quad C_w := \dfrac{t \cdot b^3 \cdot h^2}{12} \cdot \left(\dfrac{3 \cdot b \cdot t + 2 \cdot h \cdot t}{6 \cdot b \cdot t + h \cdot t}\right) \quad C_w = 0.0339 \cdot in^6$

Shear Modulus - $\quad G := 11300 \cdot ksi$

$$\sigma_t := \frac{1}{A_p \cdot r_o^2} \cdot \left[G \cdot J + \frac{\pi^2 \cdot E \cdot C_w}{\left(K_t \cdot L_t\right)^2}\right] \qquad \sigma_t = 16.7003 \cdot ksi$$

$$\beta := 1 - \left(\frac{x_o}{r_o}\right)^2 \qquad \beta = 0.1109$$

$$F_{et} := \frac{1}{2 \cdot \beta} \cdot \left[\left(\sigma_{ex} + \sigma_t\right) - \sqrt{\left(\sigma_{ex} + \sigma_t\right)^2 - 4 \cdot \beta \cdot \sigma_{ex} \cdot \sigma_t}\right] \qquad F_{et} = 12.7151 \cdot ksi$$

Elastic Flexural Buckling Stress - $\quad F_e := \text{if}\left(F_{et} < \sigma_{ex}, F_{et}, \sigma_{ex}\right) \quad F_e = 12.7151 \cdot ksi$

Allowable Compressive Stress - $\quad F_n := \text{if}\left[F_e > \dfrac{F_y}{2}, F_y \cdot \left(1 - \dfrac{F_y}{4 \cdot F_e}\right), F_e\right] \quad F_n = 12.7151 \cdot ksi$

Factor of Safety for Axial Comp. - $\quad \Omega_o := 1.92$

Find Effective Area -

Determine the Effective Width of Flange -

Flat width of Flange - $w_f := b - 0.5 \cdot t$ $\quad w_f = 1.4625 \cdot in$

Flange Plate Buckling Coefficient - $k_f := 0.43$

Flange Slenderness Factor - $\lambda_f := \frac{1.052}{\sqrt{k_f}} \cdot \frac{w_f}{t} \cdot \sqrt{\frac{F_n}{E}}$ $\quad \lambda_f = 0.6551$

$\rho_f := \left(1 - \frac{0.22}{\lambda_f}\right) \cdot \frac{1}{\lambda_f}$ $\quad \rho_f = 1.0139$

Effective Flange Width - $b_e := \text{if}\left(\lambda_f > 0.673, \rho_f \cdot w_f, w_f\right)$ $\quad b_e = 1.4625 \cdot in$

Determine Effective Width of Web -

Flat width of Web - $w_w := h - t$ $\quad w_w = 1.425 \cdot in$

Web Plate Buckling Coefficient - $k_w := 0.43$

Web Slenderness Factor - $\lambda_w := \frac{1.052}{\sqrt{k_w}} \cdot \frac{w_w}{t} \cdot \sqrt{\frac{F_n}{E}}$ $\quad \lambda_w = 0.6383$

$\rho_w := \left(1 - \frac{0.22}{\lambda_w}\right) \cdot \frac{1}{\lambda_w}$ $\quad \rho_w = 1.0267$

Effective Web Width - $h_e := \text{if}\left(\lambda_w > 0.673, \rho_w \cdot w_w, w_w\right)$ $\quad h_e = 1.425 \cdot in$

Effective Column Area - $A_e := t \cdot \left(h_e + b_e\right)$ $\quad A_e = 0.2166 \cdot in^2$

Nominal Column Capacity - $P_n := A_e \cdot F_n$ $\quad P_n = 2754\,lb$

Allowable Column Capacity - $P_a := \frac{P_n}{\Omega_o}$ $\quad P_a = 1434\,lb$

Check Combined Stresses -

$P_{crx} := \frac{\pi^2 \cdot E \cdot I_x}{\left(K_x \cdot L_x\right)^2}$ $\quad P_{crx} = 1.33 \times 10^4\,lb$

$P_{cr} := P_{crx}$ $\quad P_{cr} = 13250.86\,lb$

Magnification Factor - $\alpha := 1 - \left(\frac{\Omega_o \cdot P_p}{P_{cr}}\right)$ $\quad \alpha = 0.981$ $\quad C_m := 0.85$

Combined Stress: $\frac{P_p}{P_a} + \frac{C_m \cdot f_{bx}}{F_b \cdot \alpha} = 0.75$ MUST BE LESS THAN 1.0

Final Design: **'L' POSTS WITH BEAM BRACKET ARE ADEQUATE FOR REQD COMBINED AXIAL AND BENDING LOADS**

NOTE: P_p is the total vertical load on post, not 67% live load, so the design is conservative

STEEL STORAGE RACK DESIGN - cont'd

Find Overturning Forces :

Total Height of Shelving Unit - $H_t := h_t = 9\,ft$ Width of Shelving Unit - $w = 3\,ft$

Depth of Shelving Unit - $d = 3\,ft$ WORST CASE

Number of Shelves - $N = 4$ Vertical Shelf Spacing - $S = 36 \cdot in$

Height to Top Shelf Center of G - $h_{top} := H_t$ $h_{top} = 9\,ft$

Height to Shelf Center of G - $h_c := \frac{(N+1)}{2} \cdot S$ $h_c = 7.5 \cdot ft$

From Vertical Distribution of Seismic Force previously calculated -

Controlling Load Cases -

Weight of Rack and 67% of LL - $W := \left(W_d + 0.67 \cdot W_l\right) \cdot N$ $W = 384.95\,lb$

Seismic Rack and 67% of LL - $V := V_{td} + 0.67 \cdot V_{tl}$ $V = 99.85\,lb$

$M_a := F_1 \cdot 0.0 \cdot S + F_2 \cdot 1.0 \cdot S + F_3 \cdot 2.0 \cdot S + F_4 \cdot 3.0 \cdot S$

$M_b := 0$

Overturning Rack and 67% of LL - $M := M_a + M_b = 698.95\,ft \cdot lb$

Weight of Rack and 100% Top Shelf - $W_a := W_d \cdot N + W_l$ $W_a = 216.95\,lb$

Seismic Rack and 100% Top Shelf - $V_a := V_{td} + F_l$ $V_a = 56.27\,lb$

Overturning Rack and 100% Top Shelf - $M_a := V_{td} \cdot h_c + F_l \cdot h_{top}$ $M_a = 460.96\,ft \cdot lb$

Controlling Weight - $W_c := \text{if}\left(W > W_a, W, W_a\right)$ $W_c = 384.95\,lb$

Controlling Shear - $V_c := \text{if}\left(V > V_a, V, V_a\right)$ $V_c = 99.85\,lb$

Controlling Moment - $M_{ot} := \text{if}\left(M > M_a, M, M_a\right)$ $M_{ot} = 698.95\,ft \cdot lb$

Tension Force on Column Anchor - per side of shelving unit $T := \frac{M_{ot}}{d} - 0.60 \cdot \frac{W_c}{2}$ $T = 117.5\,lb$

$T := \text{if}(T < 0 \cdot lb, 0 \cdot lb, T)$ $T = 117.5\,lb$

Shear Force on Column Anchor - $V := \frac{V_c}{2}$ $V = 49.92\,lb$

USE: HILTI KWIK BOLT TZ ANCHOR (or equivalent) -
USE 3/8"ϕ x 2" embed installed per the requirements of Hilti

Allowable Tension Force - $T_a := 1006 \cdot lb$ For 2500psi Concrete

Allowable Shear Force - $V_a := 999 \cdot lb$ Per ASCE 13.4.2 $\rho := 1.3$

Combined Loading - $\left(\frac{\rho \cdot T}{T_a}\right) + \left(\frac{\rho \cdot V}{V_a}\right) = 0.217$ MUST BE LESS THAN 1.00

STEEL ANTI-TIP CLIP AND ANTI-TIP TRACK DESIGN

Tension (Uplift) Force on each side - $T = 117.5\,lb$

Connection from Shelf to Carriage = 1/4" diameter bolt through 14 ga. steel:

Capacity of #12 screw (smaller than 1/4" diam. bolt) in 16 ga. steel (thinner than 14 ga. posts and clips) - $Z_c := 349\cdot lb$

$\text{if}\left(T < 2\cdot Z_c, \text{"(2) 1/4" Bolts are Adequate"}, \text{"No Good"}\right) = \text{"(2) 1/4" Bolts are Adequate'}$

Use 3/16" Diameter anti-tip device for connection of carriage to track

Yield Stress of Angle Steel - $F_y := 36\cdot ksi$

Thickness of Anti-tip Head - $t_a := 0.090\cdot in$

Width of Anti-tip Rod + Radius - $b_r := 0.25\cdot in$

Width of Anti-tip Head - $b_a := 0.490\cdot in$

Width of Anti-tip Flange - $L_a := \dfrac{b_a - b_r}{2}$ $\quad L_a = 0.12\cdot in$

Tension Force per Flange leg - $T_l := 0.5\cdot T$ $\quad T_l = 58.75\,lb$

Bending Moment on Leg - $M_l := \dfrac{T_l\cdot L_a}{2}$ $\quad M_l = 0.29\cdot ft\cdot lb$

Section Modulus of Leg - $S_l := \dfrac{b_a\cdot t_a^{2}}{6}$ $\quad S_l = 0.001\cdot in^3$

Bending Stress on Leg - $f_b := \dfrac{M_l}{S_l}$ $\quad f_b = 5.33\cdot ksi$

Ratio of Allowable Loads - $\dfrac{f_b}{0.75\cdot F_y} = 0.197$ $\quad$ MUST BE LESS THAN 1.0

Width of Anti-Tip track - $L := 5.1\cdot in$

Thickness of Aluminum Track - $t_t := 0.33\cdot in$ $\quad$ Average Thickness

Spacing of Bolts - $S_{tb} := 22.5\cdot in$

Section Modulus of Track - $S_t := \dfrac{L\cdot t_t^{2}}{6}$ $\quad S_t = 0.0926\cdot in^3$

Design Moment on Track - for continuous track section $M := \dfrac{T\cdot S_{tb}}{8}$ $\quad M = 27.54\,ft\cdot lb$

Bending Stress on Track - $f_b := \dfrac{M}{S_t}$ $\quad f_b = 3.57\cdot ksi$

Allowable Stress of Aluminum - $F_b := 21\cdot ksi$

Ratio of Allowable Loads - $f_b\cdot F_b^{-1} = 0.17$ $\quad$ MUST BE LESS THAN 1.0

ANTI-TIP CLIP STEEL CONNECTION AND TRACK ARE ADEQUATE

FIXED BEAM DESIGN: Double Hanger Bar Beam

Design criteria:

Steel Yield Stress - $F_y = 36 \cdot ksi$

Modulus of Elasticity - $E = 2.9 \times 10^4 \cdot ksi$

Width of Rack - $w = 3 ft$

Depth of Rack - $d = 3 ft$

Live Load per shelf - $w_{ll} := \dfrac{W_{tj}}{2 \cdot \max(w, d)} = 16.6667 \cdot plf$

Live Load on Shelves - $LL = 11.1111 \cdot psf$

Dead Load on Shelves - $w_{dl} := 0.80 \cdot plf$

Minimum Dist Load Req'd - $w_{tl} := w_{dl} + w_{ll} = 17.4667 \cdot plf$

Point Loads on Beam - $P := \dfrac{W_{tj}}{2} = 50 lb$

Moments for Each Load -

$$M_w := \frac{w_{tl} \cdot \max(w, d)^2}{8} = 19.65 ft \cdot lb \qquad M_p := \frac{w_{dl} \cdot \max(w, d)^2}{8} + \left(\frac{P \cdot \max(w, d)}{4}\right) = 38.4 ft \cdot lb$$

Maximum Design Moment - $M := \max(M_p, M_w) = 38.4 ft \cdot lb$

Shear for Each Load -

$$V_w := \frac{w_{tl} \cdot \max(w, d)}{2} = 26.2 lb \qquad V_p := \frac{w_{dl} \cdot \max(w, d)}{2} + \frac{P}{2} = 26.2 lb$$

Maximum Design Shear - $V := \max(V_p, V_w) = 26.2 lb$

Allowable Shear Stress - $F_v := 0.4 \cdot F_y = 14.4 \cdot ksi$

Allowable Bending Stress - $F_b := 0.66 \cdot F_y = 23.76 \cdot ksi$

Section Properties: Hanger Bar Beam

$A := 0.233 \cdot in^2$ $\qquad S := 0.066 \cdot in^3$ $\qquad I := 0.041 \cdot in^4$

Actual Shear Stress -

$f_v := \dfrac{V}{A} = 0.112 \cdot ksi \qquad \dfrac{f_v}{F_v} = 0.008$ OK

Actual Bending Stress -

$f_b := \dfrac{M}{S} = 6.982 \cdot ksi \qquad \dfrac{f_b}{F_b} = 0.294$ OK

Deflections for Each -

$$\Delta_w := \frac{5 \cdot w_{tl} \cdot \max(w, d)^4}{384 \cdot E \cdot I} = 0.027 \cdot in \qquad \Delta_p := \frac{5 \cdot w_{dl} \cdot \max(w, d)^4}{384 \cdot E \cdot I} + \left(\frac{P \cdot \max(w, d)^3}{48 \cdot E \cdot I}\right) = 0.042 \cdot in$$

Total Load Deflection - $\Delta := \max(\Delta_p, \Delta_w) = 0.042 \cdot in \qquad \dfrac{\max(w, d)}{\Delta} = 855$ OK

Hanger Bar Beam is Adequate

FIXED BEAM DESIGN: Double Rivet Std. Profile Beam

Design criteria:

Steel Yield Stress - $F_y = 36 \cdot ksi$

Modulus of Elasticity - $E = 2.9 \times 10^4 \cdot ksi$

Width of Rack - $w = 3ft$

Depth of Rack - $d = 3ft$

Total Load per Bar - $P_{tl} := V = 26.2 ft \cdot plf$

Live Load on Shelves - $LL = 11.1111 \cdot psf$

Dead Load on Shelves - $w_{dl} := \dfrac{DL \cdot \max(w, d)}{2} = 3.75 \cdot plf$

Distance from End of Shelf to Point Load - $a_{tl} := \dfrac{\min(w, d)}{3} = 1 ft$

Total Moments - $M := \dfrac{w_{dl} \cdot \min(w, d)^2}{12} + \left(\dfrac{P_{tl} \cdot a_{tl}}{2}\right) = 15.912 ft \cdot lb$

Lateral Moment from Post - $M_s = 50.2831 ft \cdot lb$ page 14 of original calcs

Total Shear - $V := \dfrac{w_{dl} \cdot \min(w, d)}{2} + \dfrac{2 \cdot P_{tl}}{2} = 31.825 lb$

Allowable Shear Stress - $F_v = 14.4 \cdot ksi$

Allowable Bending Stress - $F_b = 23.76 \cdot ksi$

Section Properties: Double Rivet Standard Profile Beam

$A := 0.264 \cdot in^2$ $S := 0.126 \cdot in^3$ $I := 0.211 \cdot in^4$

Actual Shear Stress -

$f_v := \dfrac{V}{A} = 0.121 \cdot ksi$ $\dfrac{f_v}{F_v} = 0.008$ OK

Actual Bending Stress -

$f_b := \dfrac{M + M_s}{S} = 6.304 \cdot ksi$ $\dfrac{f_b}{F_b} = 0.265$ OK

Total Load Deflection -

$$\Delta := \frac{w_{dl} \cdot \min(w, d)^4}{384 \cdot E \cdot I} + \left[\frac{P_{tl} \cdot a_{tl} \cdot \left[3 \cdot \left(\min(w, d)^2\right) - 4 \cdot a_{tl}^2\right]}{96 \cdot E \cdot I}\right] = 0.002 \cdot in$$

$\dfrac{\min(w, d)}{\Delta} = 18036$ OK

Double Rivet Standard Profile Beam is Adequate

Connection from Steel Racks to Wall

Seismic Analysis Procedure per ASCE-7 Section 13.3.1:

Average Roof Height - $h_r = 32\,ft$

Height of Rack Attachments - $z_b := z + h_t = 25\,ft$ At Top for fixed racks connected to walls

Seismic Base Shear Factor - $V_t := \dfrac{0.4 \cdot a_p \cdot S_{DS}}{\dfrac{R_p}{I_p}} \cdot \left(1 + 2 \cdot \dfrac{z_b}{h_r}\right)$ $V_t = 0.465$

Shear Factor Boundaries - $V_{tmin} := 0.3 \cdot S_{DS} \cdot I_p$ $V_{tmin} = 0.218$

$V_{tmax} := 1.6 \cdot S_{DS} \cdot I_p$ $V_{tmax} = 1.162$

$V_t := \text{if}\left(V_t > V_{tmax}, V_{tmax}, V_t\right)$

$V_t := \text{if}\left(V_t < V_{tmin}, V_{tmin}, V_t\right)$ $V_t = 0.465$

Seismic Coefficient - $V_t = 0.465$

Number of Shelves - $N = 4$

Weight per Shelf - $W_{tj} = 100\,lb$

Total Weight on Rack - $W_T := 0.667 \cdot 4 \cdot P_p$ $W_T = 344.81\,lb$

Seismic Force at top and bottom - $T_v := \dfrac{0.7 \cdot V_t \cdot W_T}{2}$ $T_v = 56.15\,lb$

Connection at Top:

Standard Stud Spacing - $S_{stud} := 16 \cdot in$

Width of Rack - $w = 3\,ft$

Number of Connection Points - on each rack $N_c := \text{floor}\left(\dfrac{w}{S_{stud}}\right)$ $N_c = 2$

Force on each connection point - $F_c := \dfrac{T_v}{N_c}$ $F_c = 28.07\,lb$

Capacity per inch of embedment - $W_s := 135 \cdot \dfrac{lb}{in}$

Required Embedment - $d_s := \dfrac{F_c}{W_s}$ $d_s = 0.208 \cdot in$

For Steel Studs:

Pullout Capacity in 20 ga studs - per Scafco $T_{20} := 84 \cdot lb$ For #10 Screw - per Scafco

MIN #10 SCREW ATTACHED TO EXISTING WALL STUD IS ADEQUATE TO RESIST SEISMIC FORCES ON SHELVING UNITS. EXPANSION BOLT IS ADEQUATE BY INSPECTION AT THE BASE

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 45.517592
Longitude = -122.67785599999999
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.05 deg grid spacing

Period (sec)	Sa (g)
0.2	0.985 (Ss, Site Class B)
1.0	0.345 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 45.517592
Longitude = -122.67785599999999
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.106 ,Fv = 1.71

Period (sec)	Sa (g)
0.2	1.089 (SMs, Site Class D)
1.0	0.590 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 45.517592
Longitude = -122.67785599999999
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.106 ,Fv = 1.71

Period (sec)	Sa (g)
0.2	0.726 (SDs, Site Class D)
1.0	0.394 (SD1, Site Class D)

SCAFCO® Steel Stud Manufacturing Co.

Fasteners (Screws and Welds)

Screw Table Notes

1. Screw spacing and edge distance shall not be less than 3 x D. (D = Nominal screw diameter)
2. The allowable screw values are based on the steel properties of the members being connected, per AISI section E4.
3. When connecting materials of different metal thicknesses or yield strength, the lowest applicable values should be used.
4. The nominal strength of the screw must be at least 3.75 times the allowable loads.
5. Values include a 3.0 factor of safety.
6. Applied loads may be multiplied by 0.75 for seismic or wind loading, per AISI A 5.1.3.
7. Penetration of screws through joined materials should not be less than 3 exposed threads. Screws should be installed and tightened in accordance with screw manufacturer's recommendations.

Allowable Loads for Screw Connections (lbs/screw)

Steel	Thickness	Steel Properties		No. 12 Dia. = 0.216 (in)		No. 10 Dia. = 0.190 (in)		No. 8 Dia, = 0.164 (in)		No. 6 Dia. = 0.138 (in)	
Mils	Design (in)	Fy (ksi)	Fu (ksi)	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout
18	0.0188	33	45					66	39	60	33
27	0.0283	33	45					121	59	111	50
30	0.0312	33	45			151	76	141	65	129	55
33	0.0346	33	45			177	84	164	72	151	61
43	0.0451	33	45	280	124	263	109	244	94	224	79
54	0.0566	33	45	394	156	370	137	344	118		
68	0.0713	33	45	557	156	523	173				

Weld Table Notes

1. Weld capacities based on AISI, section E2.
2. When connecting materials of different metal thickness or tensile strength (Fu), the lowest applicable values should be used.
3. Values include a 2.5 factor of safety.
4. Based on the minimum allowance load for fillet or flare groove welds, longitudinal or transverse loads.
5. Allowable loads based on E60xx electrodes
6. For material less than or equal to .1242" thick, drawings show nominal weld size. For such material, the effective throat of the weld shall not be less than the thickness of the thinnest connected part.

Allowable Loads For Fillet Welds And Flare Groove Welds

Mil	Design Thickness in.	Steel Properties Yield ksi	Tensile ksi	E60XX Electrodes lbs/in
43	0.0451	33	45	609
54	0.0566	33	45	764
68	0.0713	33	45	963
97	0.1017	33	45	1373
118	0.1242	33	45	1677
54	0.0566	50	65	1104
68	0.0713	50	65	1390
97	0.1017	50	65	1983
118	0.1242	50	65	2422

TABLE 9—KB-TZ CARBON AND STAINLESS STEEL ALLOWABLE SEISMIC TENSION (ASD), NORMAL-WEIGHT CRACKED CONCRETE, CONDITION B (pounds)[1,2,3]

Nominal Anchor Diameter	Embedment Depth h_{ef} (in.)	Concrete Compressive Strength[2]							
		f'c = 2,500 psi		f'c = 3,000 psi		f'c = 4,000 psi		f'c = 6,000 psi	
		Carbon steel	Stainless steel	Carbon steel	Stainless steel	Carbon steel	Stainless steel	Carbon steel	Stainless steel
3/8	2	1,006	1,037	1,102	1,136	1,273	1,312	1,559	1,607
1/2	2	1,065	1,212	1,167	1,328	1,348	1,533	1,651	1,878
	3 1/4	2,178	2,207	2,386	2,418	2,755	2,792	3,375	3,419
5/8	3 1/8	2,081	2,081	2,280	2,280	2,632	2,632	3,224	3,224
	4	3,014	2,588	3,301	2,835	3,812	3,274	4,669	4,010
3/4	3 3/4	2,736	3,594	2,997	3,937	3,460	4,546	4,238	5,568
	4 3/4	3,900	3,900	4,272	4,272	4,933	4,933	6,042	6,042

For SI: 1 lbf = 4.45 N, 1 psi = 0.00689 MPa For pound-inch units: 1 mm = 0.03937 inches

[1]Values are for single anchors with no edge distance or spacing reduction. For other cases, calculation of R_d as per ACI 318-05 and conversion to ASD in accordance with Section 4.2.1 Eq. (5) is required.

[2]Values are for normal weight concrete. For sand-lightweight concrete, multiply values by 0.60.

[3]Condition B applies where supplementary reinforcement in conformance with ACI 318-05 Section D.4.4 is not provided, or where pullout or pryout strength governs. For cases where the presence of supplementary reinforcement can be verified, the strength reduction factors associated with Condition A may be used.

TABLE 10—KB-TZ CARBON AND STAINLESS STEEL ALLOWABLE SEISMIC SHEAR LOAD (ASD), (pounds)[1]

Nominal Anchor Diameter	Allowable Steel Capacity, Seismic Shear	
	Carbon Steel	Stainless Steel
3/8	999	1,252
1/2	2,839	3,049
5/8	4,678	5,245
3/4	6,313	6,477

For **SI:** 1 lbf = 4.45 N

[1]Values are for single anchors with no edge distance or spacing reduction due to concrete failure.