SEIZMIC INC

MATERIAL HANDLING ENGINEERING

EST. 1985

SPECIAL PRODUCTS	*CONVEYORS*	*STORAGE RACKS*	*OTHER SERVICES*	*SHELVING*	*SPECIAL PRODUCTS*
TANK SUPPORTS	TALL SUPPORTS	SELECTIVE	SEISMIC ANALYSIS	METAL	SHUTTLES
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RACK BLDGS	ROBOTIC PLATFORMS	PUSH BACK	EGRESS PLANS	MOVABLE	CAROUSELS
SHEDS	PICK MODULES	FLOW RACK	STATE APPROVALS	GONDOLAS	FENCING SYSTEMS
WORK PLATFORMS	ROOF VERIFICATION	CANTILEVER	PRODUCT TESTING	LOCKERS	MODULAR OFFICES
FOOTINGS			TITLE 24	CATWALKS	MINI-LOAD SYSTEMS

LICENSED IN 50 STATES

ANALYSIS OF STORAGE RACKS FOR

Wallflower Merchandise

2229 NE Town Centre Blvd, Lee's Summit, MO

Job No. 24-2416

Approved by:

SAL E. FATEEN, P.E.

9/18/2024

STATE OF MISSOURI
SAL E. FATEEN
NUMBER
EN-02?5587
REGISTERED PROFESSIONAL ENGINEER

Digitally signed by Sal Fateen
Date: 2024.09.18 13:44:10-07'00'

EXPIRES
12-31-2024

1130 E. Cypress St * Covina, CA 91724 * (909) 869-0989

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Scope:

This storage system analysis is intended to determine its compliance with appropriate building codes with respect to static and seismic forces.

The storage racks are prefabricated and are to be field assembled only, with no field welding.

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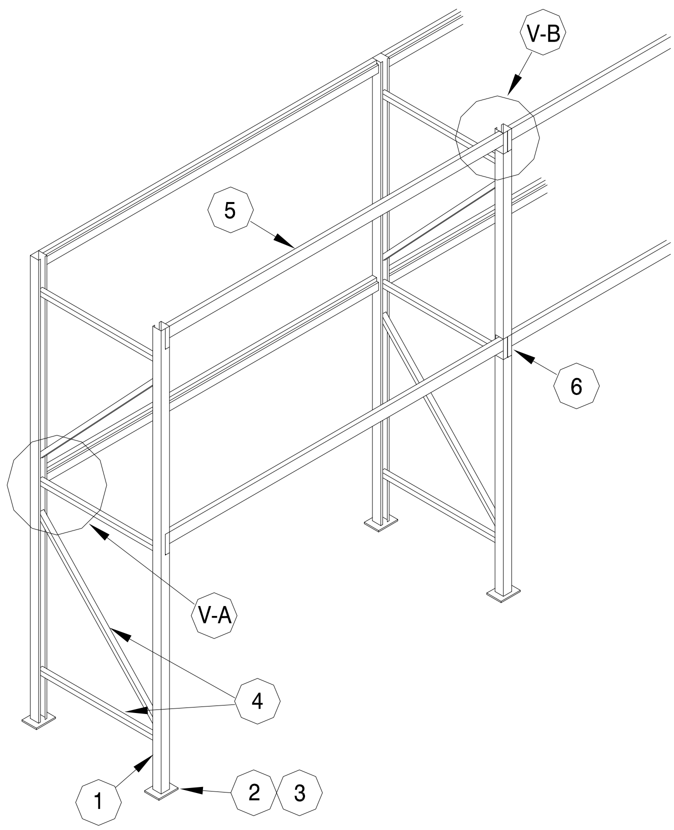
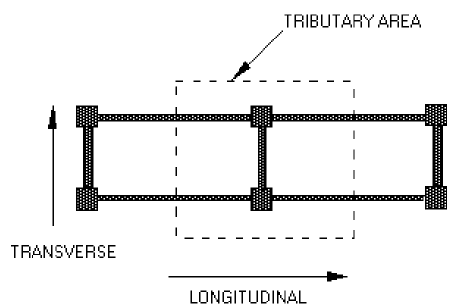
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction is maintained by the beam to column moment resisting connections, while bracing acts in the transverse direction.



CONCEPTUAL DRAWING

Some components may not be used or may vary

Legend
1. Column
2. Base Plate
3. Anchors
4. Bracing
5. Beam
6. Connector



NOTE: ACTUAL CONFIGURATION SHOWN ON COMPONENTS & SPECIFICATIONS SHEET

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PROJECT: Wallflower Merchandise
FOR: Shoppas Material Handl
ADDRESS: 2229 NE Town Centr
Lee's Summit, MO
SHEET#: 3
CALCULATED BY: aswenson
DATE: 9/18/2024
PN: 20240917_019

COMPONENTS AND SPECIFICATIONS   Configuration 1: Type B   M 1.7

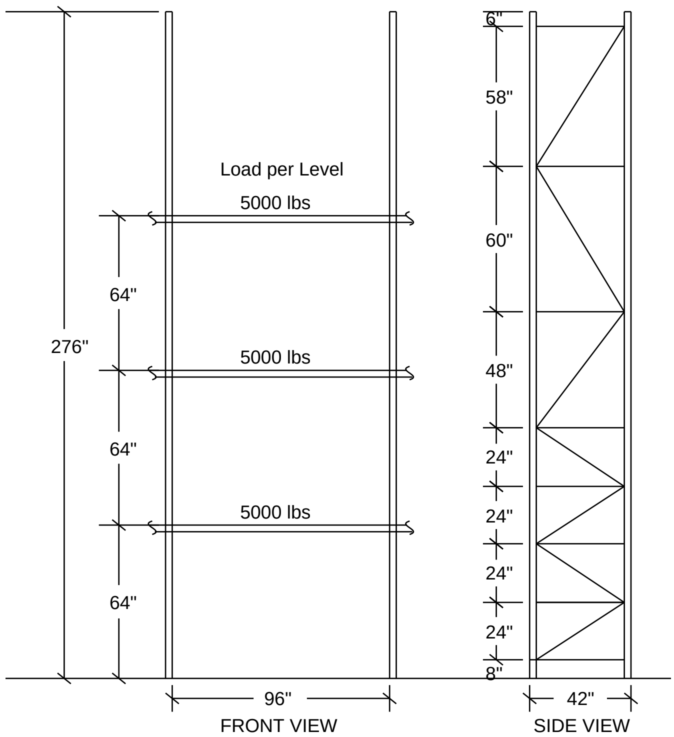
Analysis per section 2209 of the 2018 IBC

Levels: 3   Panels: 7

$S_s = 0.1$ $F_a = 1.6$ $I = 1$
$S_1 = 0.07$ $F_v = 2.4$ SDC = B

V_{Long} = 189 lbs.
V_{Trans} = 284 lbs.

P_{static} = 7650 lbs.
$P_{seismic}$ = 822 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 2.7 - 0.09 (313) Steel = 55000 psi Stress = 75% (level 1) **HORIZONTAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 8% (panel 1) **DIAGONAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 14% (panel 1)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 93% Max stress = 93% (level 1)	4 Tab 2" cc Connector (IM) Stress = 37% Max stress = 37% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 5.094 x 4.688 x 0.194 in. 1 anchors/plate Moment = 0 in-lb. Stress = 23%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 14% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 4094 lbs. Shear Capacity = 4468 lbs. Anchor stress = 2%

Notes:

Loads and Distributions: Type B

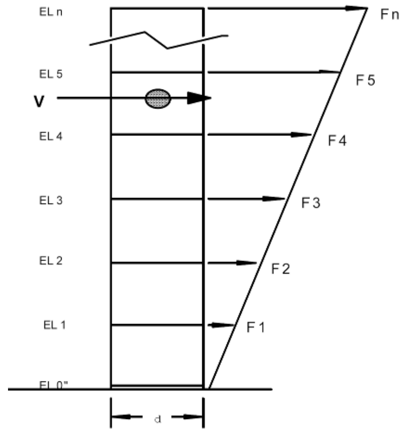
Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels:	3	SDC:	B	R_L:	6	Ss:	0.1
Pallets Wide:	2	W_{PL}:	15000	R_T:	4	S1:	0.07
Pallets Deep:	1	W_{DL}:	300 lbs	Fa:	1.6	Ip:	1
Pallet Load:	2500	Fv:	2.4	Tl:	1.5		
Total Frame Load:	15300 lbs						

$S_{DS} = 2/3 \cdot S_s \cdot F_a =$ **0.11**

$S_{D1} = 2/3 \cdot S_1 \cdot F_v =$ **0.11**

$W_s = 0.67 \cdot W_{PL} + W_{DL} =$ **10350 lbs**



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

$V_{long1} = C_s \cdot I_p \cdot W_s$

$= S_{D1} / (T_L \cdot R_L) \cdot I_P \cdot W_s$

$= 0.11 / (1.5 \cdot 6) \cdot 1 \cdot 10350 =$ **126.5 lbs**

V_{long} need not be greater than:

$V_{long2} = C_s \cdot I_p \cdot W_s$

$= S_{DS} / R_L \cdot I_P \cdot W_s$

$= 0.11 / 6 \cdot 1 \cdot 10350 =$ **189.75 lbs**

If S_1 >= 0.6, then V_{long} shall not be less than:

$V_{long3} = C_s \cdot I_p \cdot W_s$

$= 0.5 \cdot S_1 / R_L \cdot I_P \cdot W_s$

$= 0.5 \cdot 0.07 / 6 \cdot 1 \cdot 10350 =$ **60.38 lbs**

V_{long} shall not be less than:

$V_{long4} = C_s \cdot I_p \cdot W_s$

$= \text{Max}[0.044 \cdot S_{DS} , 0.03] \cdot I_P \cdot W_s$

$= \text{Max}[0.03, 0.03] \cdot 1 \cdot 10350 =$ **310.5 lbs**

Since: 126.5 < 310.5
& 310.5 > 60.38

V_{long} = 189 lbs

Transverse

V_{trans} need not be greater than:

$V_{trans1} = C_s \cdot I_p \cdot W_s$

$= S_{DS} / R_T \cdot I_P \cdot W_s$

$= 0.11 / 4 \cdot 1 \cdot 10350 =$ **284.63 lbs**

If S_1 >= 0.6, then V_{trans} shall not be less than:

$V_{trans2} = C_s \cdot I_p \cdot W_s$

$= 0.5 \cdot S_1 / R_T \cdot I_P \cdot W_s$

$= 0.5 \cdot 0.07 / 4 \cdot 1 \cdot 10350 =$ **90.56 lbs**

V_{trans} shall not be less than:

$V_{trans3} = C_s \cdot I_p \cdot W_s$

$= \text{Max}[0.044 \cdot S_{DS} , 0.5 \cdot S_1 / R_T , 0.03] \cdot I_P \cdot W_s$

$= \text{Max}[0, 0.01, 0.03] \cdot 1 \cdot 10350 =$ **310.5 lbs**

Since: 310.5 > 284.63
& 310.5 > 90.56

V_{trans} = 284 lbs

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Loads and Distributions: Type B (Page 2)

$$f_i = V \frac{W_i H_i}{\Sigma W_i H_i}$$

		Longitudinal			Transverse		
Level	h_x	w_x	$w_x h_x$	f_i	w_x	$w_x h_x$	f_i
1	64	2550	163200	31.5	2550	163200	47.33
2	128	2550	326400	63	2550	326400	94.67
3	192	2550	489600	94.5	2550	489600	142

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Fundamental Period of Vibration (Longitudinal)

Per FEMA 460 Appendix A - Development of An Analytical Model for the Displacement Based Seismic Design of Storage Racks in Their Down Aisle Direction

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{N_L} W_{pi} h_{pi}^2}{g(N_c(\frac{k_c k_{be}}{k_c + k_{be}}) + N_b\left(\frac{k_b k_{ce}}{k_b + k_{ce}}\right))}} \quad \text{(A-7)}$$

# of levels	3
min. # of bays	3
N_c	36
N_b	8
k_c	360 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	150 kip-in/rad
k_{ce}	603 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	0.98 in^4
H	192 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	93 in	5 kip
2	157 in	5 kip
3	222 in	5 kip

Where:

W_{pi} = the weight of the ith pallet supported by the storage rack

h_{pi} = the elevation of the center of gravity of the ith pallet with respect to the base of the storage rack

g = the acceleration of gravity

N_L = the number of loaded levels

k_c = the rotational stiffness of the connector

k_{be} = the flexural rotational stiffness of the beam-end

k_b = the rotational stiffness of the base plate

k_{ce} = the flexural rotational stiffness of the base upright-end

N_c = the number of beam-to-upright connections

N_b = the number of base plate connections

$$k_{be} = \frac{6EI_b}{L}$$

$$k_{ce} = \frac{4EI_c}{H}$$

$$k_b = \frac{EI_c}{H}$$

L = the clear span of the beams

H = the clear height of the upright

I_b = the moment of inertia about the bending axis of each beam

I_c = the moment of inertia of each base upright

E = the Young's modulus of the beams

Calculated T = 3.22

Since the calculated T is greater than 1.5, the more conservative value of 1.5 is used in the calculations

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FOR: Shoppas Material Handl
ADDRESS: 2229 NE Town Centr
Lee's Summit, MO
SHEET#: 7
CALCULATED BY: aswenson
DATE: 9/18/2024
PN: 20240917_019

LRFD Basic Load Combinations: Type B

2018 IBC& RMI / ANSI MH 16.1

V_{Trans} = **284 lbs**

V_{Long} = **189 lbs**

P = Product Load / 2 = **7,500 lbs**

D = Dead Load · 0.5 = **150 lbs**

$M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **42,410 in-lbs**

$E_{Trans} = M_{Trans}$ / frame depth = **1,009 lbs**

$\beta = 0.7$

$\beta = 1.0$ (Uplift combination only)

$\rho = 1$

$S_{DS} = .11$

L = Live Load = **0 lbs** S = Snow Load = **0 lbs** R = Rain Load = **0 lbs**
Lr = Live Roof Load = **0 lbs** W = Wind Load = **0 lbs**

Basic Load Combinations

1. Dead Load = 1.4 D + 1.2 P
= (1.4 · 150) + (1.2 · 7,500)= **9,210 lbs**

2. Gravity Load = 1.2 D + 1.4 P + 1.6 L + 0.5 (L_r or S or R)
= (1.2 · 150) + (1.4 · 7,500) + (1.6 · 0) + (0.5 · 0)= **10,680 lbs**

3. Snow/Rain = 1.2D + 0.85P + (0.5L or 0.5W) + 1.6(L_r or S or R)
= (1.2 · 150) + (0.85 · 7,500) + (0.5 · 0) + (1.6 · 0)= **6,555 lbs**

4. Wind Load = 1.2D + 0.85P + 0.5L + 1.0W + 0.5(L_r or S or R)
= (1.2 · 150) + (0.85 · 7,500) + (0.5 · 0) + (1.0 · 0) + (0.5 · 0)= **6,555 lbs**

5A. Seismic Load (Transverse) = $(1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Trans} + 0.2S$
= (1.2 + 0.2 · .11) · 150 + (1.2 + 0.2 · .11) · 0.7 · 7,500 + 0.5 · 0 + 1 · 1,009 + 0.2 · 0= **7,608 lbs**

5B. Seismic Load (Longitudinal) = $(1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Long} + 0.2S$
= (1.2 + 0.2 · .11) · 150 + (1.2 + 0.2 · .11) · 0.7 · 7,500 + 0.5 · 0 + 1 · 0 + 0.2 · 0= **6,598 lbs**

6. Wind Uplift = $0.9D + 0.9P_{app} + 1.0W$
= 0.9 · 150 + 0.9 · 7,500 + 1.0 · 0 = **135 lbs**

7. Seismic Uplift = $(0.9 - 0.2S_{DS})D + (0.9 - 0.2S_{DS})\beta P_{app} - \rho E_{Trans}$
= (0.9 - 0.2 · .11) · 150 + (0.9 - 0.2 · .11) · 1 · 7,500 - 1 · 1,009= **5,706 lbs**
For a single beam, D = **32 lbs** P = **2,500 lbs** I = **312 lbs**

8. Product/Live/Impact = 1.2D + 1.6L + 0.5(SorR) + 1.4P + 1.4I
(1.2 · 32) + (1.6 · 0) + (0.5 · 0) + (1.4 · 2,500) + (1.4 · 312) = **3,975 lbs**

ASD Load Combinations for Slab Analysis

1. $(1 + 0.105S'_{DS})D + 0.75((1.4 + 0.14S_{DS})\beta P + 0.7\rho E)$
= (1 + 0.105 · .11) · 150 + 0.75((1.4 + 0.14 · .11) · 0.7 · 7,500 + 0.7 · 1 · 1,009)= **6,255 lbs**

2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$
= (1 + 0.14 · .11) · 150 + (0.85 + 0.14 · .11) · 0.7 · 7,500 + 0.7 · 1 · 1,009= **5,402 lbs**

3. D + P
= 150 + 7,500 = **7,650 lbs**

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Longitudinal Analysis: Type B

This analysis is based on the Portal Method, with the point of contra flexure of the columns assumed at mid-height between beams, except for the lowest portion, where the base plate provides only partial fixity and the contra flexure is assumed to occur closer to the base (or at the base of pinned condition, where the base plate cannot carry moment).

$M_{ConnR} = M_{ConnL} = M_{Conn}$

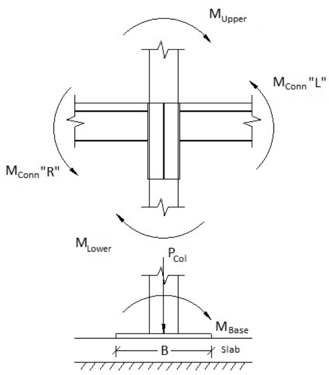
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

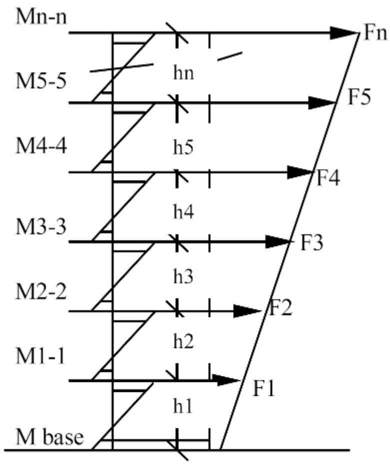
$V_{Col} = V_{Long}$ / # of columns = **95 lbs**

$M_{Base} =$ **0 in-lbs**

$M_{Lower} = ((V_{col} \cdot h_i) - M_{Base}$

(95 lbs · 62 in.) - 0 in-lbs = 5890 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	64	16	7,650	5,890	5,551	11,441
2	64	32	5,100	5,890	5,551	11,441
3	64	47	2,550	5,890	5,551	8,496

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ADDRESS: 2229 NE Town Centr
Lee's Summit, MO
SHEET#: 9
CALCULATED BY: aswenson
DATE: 9/18/2024
PN: 20240917_019

COLUMN ANALYSIS: Type B (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

Section subject to torsional or flexural-torsion buckling (Section C4.1.2)

$K_x \cdot L_x / R_x$ = 1.7 · 62 / 1.081 = **97.53**

$K_y \cdot L_y / R_y$ = 1 · 24 / 0.934 = **25.7**

KL/R_{max} = **97.53**

$r_o = (r_x^2 + r_y^2 + X_o^2)^{1/2}$ (Eq. C3.1.2.1-7)

= $(1.081^2 + 0.934^2 + -2.349^2)^{1/2}$ = **2.749 in.**

$\beta = 1 - (Xo/ro)^2$ (Eq C4.1.2-3)

= $1 - (-2.349/2.749)^2$ = **0.27**

$F_{e1} = \Pi^2 E / (KL/r)_{max}^2$ (Eq C4.1.1-1)

= $3.14^2 \cdot 29500 / 97.53^2$ = **30.609 ksi**

$F_{e2} = (1 / 2\beta)((\sigma_{ex} + \sigma_t) - (\sigma_{ex} + \sigma_t)^2 - (4\beta\sigma_{ex}\sigma_t))^{1/2})$ (Eq C4.1.2-1)

= $(1 / (2 \cdot 0.27)((30.609 + 210.409) - (30.609 + 210.409)^2$

$- (4 \cdot 0.27 \cdot 30.609 \cdot 210.409))^{1/2})$= **27.573 ksi**

where:

$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$ (Eq C3.1.2-11)

= $3.14^2 \cdot 29500 / 97.53^2$ = **30.609 ksi**

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$ (Eq C3.1.2-9)

= $1 / 0.841 \cdot 2.749^2 (11300 \cdot 0.002$

$+ (3.14^2 \cdot 29500 \cdot 1.66) / (0.8 \cdot 24)^2)$ = **210.409 ksi**

F_e = Min(F_{e1}, F_{e2}) =**27.573 ksi**

$P_n = A_{eff} \cdot F_n$ (Eq C4.1-1)

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 27.573)^{1/2}$ = **1.412** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\text{^}(\lambda_c^2)) \cdot F_y$ = **23.866** (Eq C4.1-2)

Thus:

P_n = **16601 lbs**

P_a = **14111 lbs**

3 x 2.7 - 0.09	
SECTION PROPERTIES	
Depth	**2.717 in.**
Width	**3.03 in.**
t	**0.09 in.**
Radius	**0.138 in.**
Area	**0.841 $in.^2$**
AreaNet	**0.712 $in.^2$**
I_x	**0.982 $in.^4$**
S_x	**0.649 $in.^3$**
$S_{x\ Net}$	**0.59 $in.^3$**
R_x	**1.081 in.**
I_y	**0.733 $in.^4$**
S_y	**0.444 $in.^3$**
R_y	**0.934 in.**
J	**0.002 $in.^4$**
C_w	**1.66 $in.^6$**
J_x	**2.514 in.**
X_o	**-2.349 in.**
K_x	1.7
L_x	**62 in.**
K_y	1
L_y	**24 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	0.95
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

COLUMN ANALYSIS: Type B (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

Lateral-torsional buckling strength [Resistance] (Section C3.1.2)

$P_{ao} = P_{no}\phi_c =$ **32519 lbs**

Where:

$P_{no} = A_e F_y = 0.696 \cdot 55 =$ **38258 lbs**

$M_c = M_n = S_c F_c = S_{min} F_c$ (Eq C3.1.2.1-1)

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **285.758 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **202.979 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 =$ **1551.526 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **202.979 ksi**

Since: $F_e \geq 2.78 F_y$

$F_c = (S_e / S_c)$

i.e. $F_e = F_y =$ **55 ksi** (Eq C3.1.1-3)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **53.6 ksi**

$M_{nx} =$ **31607 in-lbs** $M_{ny} =$ **23782 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **28447 in-lbs** $M_{ny}\phi_b =$ **21404 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2 =$ **25736 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2 =$ **370664 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.778** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.985** (Eq C5.2.2-5)

$P_{trans} =$ **7,608 lbs** $P_{long} =$ **6,598 lbs**

$M_u = M_x =$ **5882 in-lbs** (Eq C5.2.2-2)

$P_{u\_st} = (1.2 \cdot D) + (1.4 \cdot P) =$ **10680 lbs**

$P_{u\_st} / P_a = 10680 / 14111 = 0.76$ Static Stress = **75%**

Since: $P_l / P_a \geq 0.15$

Stress1 $= P_l / P_a + M_x / (\phi_b M_{nx}) + M_y / (\phi_b M_{ny})$ (Eq C5.2.2-2)

= ((6,598 / 14111) + (5882 / 28447) + (1 / 21404)) = **67%**

Stress2 $= P_l / P_{ao} + C_{mx} M_x / (\phi_b M_{nx} \alpha_x) + C_{my} M_y / (\phi_b M_{ny} \alpha_y)$ (Eq C5.2.2-1)

= (6,598 / 32519) + (0.85 · 5882 / 28447 · 0.778)) + (0.85 · 1 / 21404 · 0.985))) = **42%**

Stress3 P_t / P_{ao} = 7,608 / 32519 =**23%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **75%**

3 x 2.7 - 0.09	
SECTION PROPERTIES	
Depth	**2.717 in.**
Width	**3.03 in.**
t	**0.09 in.**
Radius	**0.138 in.**
Area	**0.841 in.** ^{2}
AreaNet	**0.712 in.** ^{2}
I_x	**0.982 in.** ^{4}
S_x	**0.649 in.** ^{3}
$S_{x\ Net}$	**0.59 in.** ^{3}
R_x	**1.081 in.**
I_y	**0.733 in.** ^{4}
S_y	**0.444 in.** ^{3}
R_y	**0.934 in.**
J	**0.002 in.** ^{4}
C_w	**1.66 in.** ^{6}
J_x	**2.514 in.**
X_o	**-2.349 in.**
K_x	1.7
L_x	**62 in.**
K_y	1
L_y	**24 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	0.95
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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FOR: Shoppas Material Handl
ADDRESS: 2229 NE Town Centr
Lee's Summit, MO
SHEET#: 11
CALCULATED BY: aswenson
DATE: 9/18/2024
PN: 20240917_019

BEAM ANALYSIS Type B

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

Effective width w = C - 2t - 2r = 1.75 - (2 · 0.059) - (2 · 0.09) = **1.45 in.**

w/t = 1.452 / 0.059 = **24.61**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (F_y / E)^{1/2} = (1.052 / 2) \cdot 24.61 \cdot (55 / 29500)^{1/2} =$ **0.56**

λ <= 0.673: Flange is fully effective.

Check web for local buckling (B2.3)

f_1(comp) $= F_y \cdot (y_3 / y_2) = 55 * 1.98 / 2.13 =$ **51.15 ksi**

f_2(tension) $= F_y \cdot (y_1 / y_2) = 55 * 1.72 / 2.13 =$ **44.52 ksi**

$\Psi = -(f_2 / f_1) = -(44.52 / 51.15) =$ **-0.87**

Buckling coefficient $k = 4 + 2 \cdot (1 - \Psi)^3 + 2 \cdot (1 - \Psi)$
$= 4 + 2(1 - \mathbf{-0.87})^3 + 2(1 - \mathbf{-0.87}) =$ **20.83**

Flat Depth w = y1 + y3 = 1.72 + 1.98 = **3.702**

w/t = 3.702/0.059 = **62.75** **w/t < 200: OK**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (f_1 / E)^{1/2} = (1.052 / 2) \cdot 62.746 \cdot (51.15 / 29500)^{1/2} =$ **0.6**

$b1 = w \cdot (3 - \Psi) = 4 \cdot (3 - -0.87) =$ **14.33**

b2 = w/2 = **1.85**

b1 + b2 = 14.33 + 1.85 = **16.18** **Web is fully effective**

Determine effect of cold working on steel yield point (FYA) per section A7.2

Corner cross-sectional area $Lc = (\Pi / 2) \cdot (r + t / 2)$
$= (\Pi / 2) \cdot (0.09 + 0.059 / 2) =$ **0.188**

L_f = effective width = **1.452**

$C = 2 \cdot L_c / L_f + 2 \cdot L_c = 2 \cdot 0.188 / 1.452 + 2 \cdot L_c =$ **0.2054**

$m = 0.192 \cdot (F_u / F_y) - 0.068 = 0.192 \cdot (65 / 55) - 0.068 =$ **0.1589**

$B_c = 3.69 \cdot (F_u / F_y) - 0.819 \cdot (F_u / F_y)^2 - 1.79$
$= 3.69 \cdot (65 / 55) - 0.819 \cdot (65 / 55)^2 - 1.79 =$ **1.43**

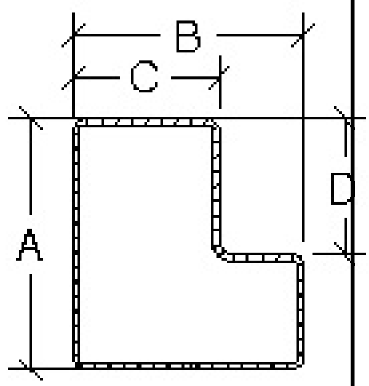
Fu/Fy = 65 / 55 = **1** **< 1.2**

r/t = 0.09 / 0.059 = **1.525** **<= 7 = OK**

$F_{yc} = B_c \cdot F_y / (r / t)^m = 1.43 \cdot 55 / (1.525)^m =$ **73**

$F_{ya\text{-}top} = C \cdot F_{yc} + (1 - C) \cdot F_y = 0.205 \cdot 73 + (1 - 0.205) \cdot 55 =$ **59**

$F_{ya\text{-}bottom} = F_{ya\text{-}top} \cdot Y_{cg} / (A - Y_{cg}) = 59 \cdot 1.87 / (4.0 - 1.87) =$ **52**



4.00 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	4.0 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.72
Y2 =	2.13
Y3 =	1.98
Ycg =	1.87
Ix =	1.55
Sx =	0.78
E =	29500
FBeam F =	360
Beam Length L =	96

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SHEET#: 12
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DATE: 9/18/2024
PN: 20240917_019

BEAM ANALYSIS Type B

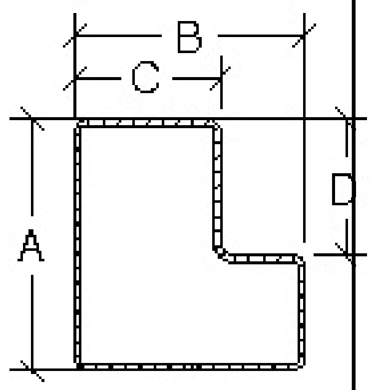
Check Allowable Tension Stress for Bottom Flange

$L_{flange\text{-}bot} = B - (2 \cdot r) - (2 \cdot t) = 2.75 - (2 \cdot 0.09) - (2 \cdot 0.059) = \mathbf{2.45}$

$C_{bottom} = 2 \cdot L_c / (L_{flange\text{-}bot} + 2 \cdot L_c) = 2 \cdot 0.188 / (2.45 + 2 \cdot 0.188) = \mathbf{0.133}$

$F_{y\text{-}bottom} = C_{bottom} \cdot F_{yc} + (1 - C_{bottom}) \cdot F_y = 0.133 \cdot 73 + (1 - 0.133) \cdot 55 = \mathbf{57.44}$

$F_{ya} = F_{ya\text{-}top} =$ **58.78 ksi**



Determine Allowable Capacity For Beam Pair (Per Section 5.2 of the RMI, PT II)

Check Bending Capacity

$M_{Center} = \phi \cdot M_n = W \cdot L \cdot \Omega \cdot R_m / 8$

$\Omega = LRFD\ Load\ Factor = (1.2 \cdot DL + 1.4 \cdot PL + 1.4 \cdot 0.125 \cdot PL) / PL$

For DL = 2% of PL:

$\Omega = 1.2 \cdot 0.02 + 1.4 + 1.4 \cdot 0.125 = \mathbf{1.6}$

$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L))$
$= 1 - ((2 \cdot 360 \cdot 96) / (6 \cdot 29500 \cdot 1.55 + 3 \cdot 360 \cdot 96)) = \mathbf{0.82}$

$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x =$ **43.5 in-kip**

$W = \phi \cdot M_n \cdot 8 \cdot (\#\text{ of Beams}) / (L \cdot R_m \cdot \Omega) = (43.5 \cdot 8 \cdot 2) / (96 \cdot 0.82 \cdot 1.6)$
= **5548** lbs/pair

Check Deflection Capacity

$\Delta_{max} = \Delta_{ss} \cdot R_d$

$\Delta_{max} = L / 180$

$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x)$
$= 1 - (4 \cdot 360 \cdot 96) / (5 \cdot 360 \cdot 96 + 10 \cdot 29500 \cdot 1.55) = \mathbf{0.78}$

$\Delta_{ss} = (5 \cdot W \cdot L^3) / (384 \cdot E \cdot I_x)$

$L / 180 = (5 \cdot W \cdot L^3 \cdot R_d) / (384 \cdot E \cdot I_x \cdot (\#\text{ of Beams}))$

$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d)$
$= (384 \cdot 29500 \cdot 1.55 \cdot 2) / (180 \cdot 5 \cdot 96^2 \cdot 0.78) \cdot 1000 =$ **5430** lbs/pair

4.00 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	4.0 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.72
Y2 =	2.13
Y3 =	1.98
Ycg =	1.87
Ix =	1.55
Sx =	0.78
E =	29500
FBeam F =	360
Beam Length L =	96

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Allowable and Actual Bending Moment at Each Level

$M_{static} = Wl^2 / 8$ $M_{allow,static} = W_{allow,static} \cdot l^2 / 8$ $M_{seismic} = M_{conn}$ $M_{allow,seismic} = S_x \cdot F_b$

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	30,576	32,580	4,753	32,580	Pass
2	30,576	32,580	3,286	32,580	Pass
3	30,576	32,580	2,779	32,580	Pass

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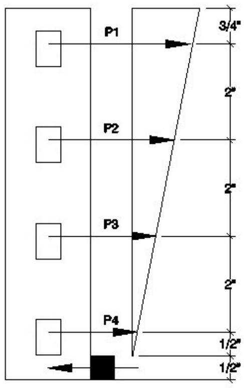
Beam to Column Analysis: Type B

1. Shear Strength of Tab



Height of the Tab h = **0.6 in.**
Thickness of the Tab t_t = **0.135 in.**

$F_y =$ **55000 psi**

$C_v =$ **1.0**

$V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = \mathbf{2673\ lbs}$ AISC G2-1

$P_{Shear} = \phi \cdot V_n = 0.9 \cdot 2673 = \mathbf{2405\ lbs}$

2. Bearing Strength of Tab

Thickness of the column t_c = **0.09 in.**

$A_{pb} = h \cdot t_c = \mathbf{0.05\ in.}$

$R_n = 1.8 \cdot F_y \cdot A_{pb} = \mathbf{5346\ lbs}$ AISC J7 -1

$P_{Bearing} = \phi \cdot R_n = 0.75 \cdot 5346 = \mathbf{4009\ lbs}$

3. Moment Strength of Bracket

Edge Dist. = **1 in.**

$T_{Clip} =$ **0.179 in.**

$S_{Clip} =$ **0.127 in.^{3}**

$M_n = S_c \cdot F_y = \mathbf{6985\ in\text{-}lbs}$ AISI C3.1.1 -1

$M_{Strength} = \phi M_n = 0.9 \cdot M_n = 0.9 \cdot S_{Clip} \cdot F_y = \mathbf{6286.5\ in\text{-}lbs}$

$C =$ **2.15**

$d =$ Edge Dist. / 2 = **0.5 in.**

$M_{Strength} = c \cdot d \cdot P_{Clip}$

$P_{Clip} = M_{Strength} / (c \cdot d) = \mathbf{5837\ lbs}$

Minimum Value of P1 Governs

$P_1 = \text{Min}(P_{Shear}, P_{Bearing}, P_{Clip}) = \mathbf{2405\ lbs}$

$M_{Conn\text{-}Allow} = (P_1 \cdot 6.5) + (P_1 \cdot (4.5 / 6.5) \cdot 4.5) + (P_1 \cdot (2.5 / 6.5) \cdot 2.5) + (P_1 \cdot (0.5 / 6.5) \cdot 0.5) = \mathbf{25530\ in\text{-}lbs}$

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PN: 20240917_019

BRACE ANALYSIS Type B (Panel 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

Section subject to torsional or flexural-torsion buckling (Section C4.1.2)

$K_x \cdot L_x / R_x$ = $0 \cdot 41 / 0.736$ = **55.7**

$K_y \cdot L_y / R_y$ = $1 \cdot 41 / 0.448$ = **91.48**

KL/R_{max} = **91.48**

$r_o = (r_x^2 + r_y^2 + X_o^2)^{1/2}$ (Eq. C3.1.2.1-7)

$= (0.736^2 + 0.448^2 + -0.995^2)^{1/2} =$ **1.317 in.**

$\beta = 1 - (Xo/ro)^2$ (Eq C4.1.2-3)

$= 1 - (-0.995/1.317)^2 =$ **0.428**

$F_{e1} = \Pi^2 E / (KL/r)_{max}^2$ (Eq C4.1.1-1)

$= 3.14^2 \cdot 29500 / 91.48^2 =$ **34.793 ksi**

$F_{e2} = (1 / 2\beta)((\sigma_{ex} + \sigma_t) - (\sigma_{ex} + \sigma_t)^2 - (4\beta\sigma_{ex}\sigma_t))^{1/2})$ (Eq C4.1.2-1)

$= (1 / (2 \cdot 0.428)((93.849 + 17.474) - (93.849 + 17.474)^2$

$- (4 \cdot 0.428 \cdot 93.849 \cdot 17.474))^{1/2}) =$ **15.677 ksi**

where:

$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$ (Eq C3.1.2-11)

$= 3.14^2 \cdot 29500 / 55.7^2 =$ **93.849 ksi**

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$ (Eq C3.1.2-9)

$= 1 / 0.257 \cdot 1.317^2 (11300 \cdot 0$

$+ (3.142 \cdot 29500 \cdot 0.025) / (0.8 \cdot 41)^2) =$ **17.474 ksi**

$F_e = \text{Min}(F_{e1}, F_{e2}) =$ **15.677 ksi**

$P_n = A_{eff} \cdot F_n$ (Eq C4.1-1)

$\lambda_c = (F_y / F_e)^{1/2} = (50 / 15.677)^{1/2} =$ **1.786** (Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

$F_n = (0.877 / \lambda_c^2) \cdot F_y =$ **13.749** (Eq C4.1-3)

Thus:

P_n = **3527 lbs**

P_a = **2998 lbs**

1.7953 x 1.378 - .0625	
SECTION PROPERTIES	
Depth	**1.795 in.**
Width	**1.378 in.**
t	**0.06 in.**
Radius	**0.118 in.**
Area	**0.257 in.^{2}**
AreaNet	**0.257 in.^{2}**
I_x	**0.139 in.^{4}**
S_x	**0.156 in.^{3}**
$S_{x\ Net}$	**0.156 in.^{3}**
R_x	**0.736 in.**
I_y	**0.052 in.^{4}**
S_y	**0.056 in.^{3}**
R_y	**0.448 in.**
J	**0 in.^{4}**
C_w	**0.025 in.^{6}**
J_x	**1.337 in.**
X_o	**-0.995 in.**
K_x	0
L_x	**41 in.**
K_y	1
L_y	**41 in.**
K_t	0.8
F_y	**50 ksi**
F_u	**60 ksi**
Q	1
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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BRACE ANALYSIS Type B (Panel 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

Lateral-torsional buckling strength [Resistance] (Section C3.1.2)

$P_{ao} = P_{no}\phi_c =$ **10905 lbs**

Where:

$P_{no} = A_e F_y = 0.257 \cdot 50 =$ **12830 lbs**

$M_c = M_n = S_c F_c = S_{min} F_c$ (Eq C3.1.2.1-1)

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **87.641 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **17.846 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f(K_y L_y)^2 =$ **102.588 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **17.846 ksi**

Since: $F_e \leq 0.56F_y$

$F_c = F_e =$ **17.846 ksi** (Eq C3.1.2.1-3)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **17.8 ksi**

$M_{nx} =$ **2778 in-lbs** $M_{ny} =$ **993 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **2500 in-lbs** $M_{ny}\phi_b =$ **893 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2 =$ **24075 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2 =$ **8919 lbs** (Eq C5.2.2-7)

$P_a =$ **2998 lbs**

$V_{Trans} =$ **284 lbs**

$V_{Trans(new)} = 284 \cdot 1.3 =$ **370 lbs**

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} =$ **41.01 in.**

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D =$ **414 lbs**

Brace Stress = $V_{Diag} / P_a =$ **14%**

1.7953 x 1.378 - .0625	
SECTION PROPERTIES	
Depth	**1.795 in.**
Width	**1.378 in.**
t	**0.06 in.**
Radius	**0.118 in.**
Area	**0.257 in.^{2}**
AreaNet	**0.257 in.^{2}**
I_x	**0.139 in.^{4}**
S_x	**0.156 in.^{3}**
$S_{x\ Net}$	**0.156 in.^{3}**
R_x	**0.736 in.**
I_y	**0.052 in.^{4}**
S_y	**0.056 in.^{3}**
R_y	**0.448 in.**
J	**0 in.^{4}**
C_w	**0.025 in.^{6}**
J_x	**1.337 in.**
X_o	**-0.995 in.**
K_x	0
L_x	**41 in.**
K_y	1
L_y	**41 in.**
K_t	0.8
F_y	**50 ksi**
F_u	**60 ksi**
Q	1
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 Type B

Assumed cracked concrete application

Anchor Type	**0.5" dia., 3.25 hef, 5.5" min, slab**				
ICC Report Number	**ESR-4266**			$1.5 \cdot h_{ef}$	= **4.875 in.**
Slab Thickness (h)	= **7 in.**	C_{a1}	= 12	use$C_{a1,adj}$	= **4.875 in.**
Min. Slab Thickness (h)	= **5.5 in.**	C_{a2}	= 12	use$C_{a2,adj}$	= **4.875 in.**
Concrete Strength (f_c)	= **4000 psi**				
Diameter (d_a)	= **0.5 in.**			$3 \cdot h_{ef}$	= **9.75 in.**
Nominal Embedment (h_{nom})	= **3.75 in.**				
Effective Embedment (h_{ef})	= **3.25 in.**	S_1	= **0 in.**	Use $S_{1,adj}$	= **0 in.**
Number of Anchors (n)	= **1**	S_2	= **0 in.**	Use $S_{2,adj}$	= **0 in.**
e`N	= **0**				
e`V	= **0**				

From ICC ESR Report

A_{se} = **0.099 sq.in.**

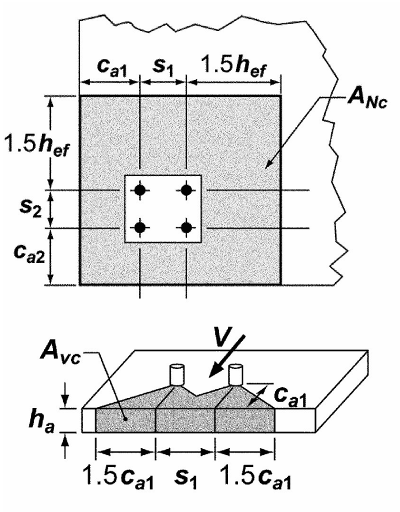
$f`_{uta}$ = **114000 psi**

S_{min} = **2 in.**

C_{min} = **2.25 in.**

C_{ae} = **8 in.**

$N_{p,cr}$ = **9999 lbs**



	$\phi_{Seismic}$	Adj. Strength
Tension Capacity = **4094 lbs**	1	**4094 lbs**
Shear Capacity = **4468 lbs**	1	**4468 lbs**

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ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Type B

Steel Strength 17.4.1

ϕ = **0.75** 17.3.3.a i

$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 1 \cdot 0.099 \cdot 114000 =$ **8,465 lbs** 17.4.1.2

Concrete Breakout Strength ϕN_{cbg} 17.4.2

ϕ = **0.65** 17.3.3 c ii Category 1-B

$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) =$ **95.063 sq.in.**

$A_{Nco} = 9h_{ef}^2 =$ **95.063 sq.in.**

Check if $A_{Nco} \geq A_{Nc}$ $A_{Nc}/A_{Nco} =$ **1**

$\Psi_{ec,N} =$ **1** 17.4.2.4

$\Psi_{ed,N} =$ **1** 17.4.2.5

$\Psi_{C,N} =$ **1** 17.4.2.6

$K_c =$ **17**

$\lambda_a =$ **1**

$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} =$ **6299 lbs** 17.4.2.2 d

$\Psi_{cp,N} =$ **1** 17.4.2.7

$\phi N_{cbg} = \phi(A_{Nc}/A_{Nco})(\Psi_{ec,N})(\Psi_{ed,N})(\Psi_{C,N})(\Psi_{cp,N})(N_b)$ 17.4.2.1

$0.65 \cdot (95.063/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299 =$ **4,094 lbs**

Pullout Strength ϕN_{pn} 17.4.3

ϕ=**0.65** 17.3.3 c ii Category 1-B

$\Psi_{cp} =$ **1** 17.4.3.6

$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} =$ **8,221 lbs** 17.4.3.1

Steel Strength (ϕN_{sa}) = **8,465 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **4,094 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **8,221 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Type B

Steel Strength ϕV_{sa} V_{sa}=6,875 / Anchor -- per report	17.5.1
ϕ = **0.65**	17.3.3. Condition a ii
$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 1 \cdot 6{,}875 =$ **4,469 lbs**	17.5.1.2a
Concrete Breakout Strength ϕV_{cbg}	17.5.2
ϕ = **0.7**	17.3.3 ci-B
$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **252 sq.in.**	
$A_{Vco} = 3C_{a1}h_a =$ **252 sq.in.**	
Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1**	
$\Psi_{ec,V}$ = **1**	17.5.2.5
$\Psi_{ed,V}$ = **0.9**	17.5.2.6
$\Psi_{C,V}$ = **1**	17.5.2.7
$\Psi_{h,V}$ = **1.604**	17.5.2.8
d_a = **0.5 in.**	17.5.2.2
L_e = **1 in.**	17.2.6 d
λ_a = **1**	
The smaller of $7(L_e / d_a)^{0.2}(d_a)^{0.5}\lambda_a(f_c)^{0.5}ca1^{1.5}$ and $9\lambda_a(f_c)^{0.5}ca1^{1.5}$ = **14,948 lbs**	17.5.2.2 a, 17.5.2.2 b
$\phi V_{cbg} = \phi(A_{Vc}/A_{Vco})(\Psi_{ec,V})(\Psi_{ed,V})(\Psi_{C,V})(\Psi_{h,V})(V_b)$	17.5.2.1
$0.7 \cdot (252/252) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.604 \cdot 14{,}948 =$ **15,101 lbs**	
Pryout Strength ϕV_{cpg}	17.5.3
ϕ= **0.7**	17.3.3 Ci-B
K_{cp} = **2**	17.5.3.1
N_{cbg} = **6,298 lbs**	
$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 6{,}298 =$ **8,817 lbs**	

Steel Strength (ϕV_{sa}) = **4,469 lbs**

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **15,101 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **8,817 lbs**

SEIZMIC INC
MATERIAL HANDLING ENGINEERING
EST. 1985

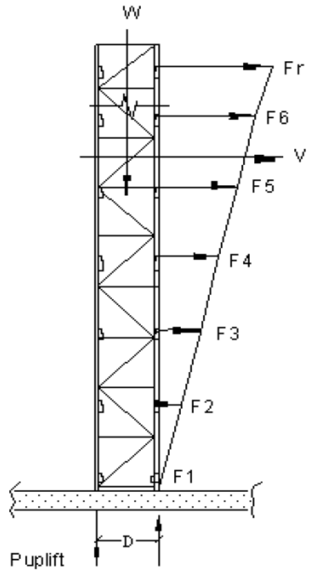
TEL:(909)869-0989
1130 E. CYPRESS ST, COVINA, CA 91724

PROJECT: Wallflower Merchandise
FOR: Shoppas Material Handl
ADDRESS: 2229 NE Town Centr
Lee's Summit, MO
SHEET#: 20
CALCULATED BY: aswenson
DATE: 9/18/2024
PN: 20240917_019

OVERTURNING ANALYSIS Configuration1 Type B

Per RMI Sec 2.6.9 and ASCE7-16. Sec 15.5.3.6. Weight of rack with all levels loaded to 67% capacity, & with only top level loaded

FULLY LOADED



CROSS AISLE ELEVATION

W_{pl} = **15,000 lbs** W_{dl} = **300 lbs**

$W_{pl} \cdot 67\% = 15{,}000 \cdot 0.67 =$ **10,050 lbs**

$V_{Trans} = (1 \cdot 0.0275 \cdot 1 \cdot ((0.67 \cdot 10{,}050) + 300)) =$ **193 lbs**

$M_{ovt} = V_{Trans} \cdot Ht = 193 \cdot 179 =$ **34,547 in-lbs**

$M_{st} = ((W_{pl} \cdot 0.67) + W_{dl}) \cdot d \cdot Factor$

$= ((15{,}000 \cdot 0.67) + 300) \cdot 42 \cdot 0.5 =$ **217,350 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (34{,}547 - 217{,}350) / 42 =$ **-4,352 lbs**

$(P_{uplift} \leq 0)$ = no uplift

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (34{,}547 + 217{,}350) / 42 =$ **5,997 lbs**

TOP SHELF LOADED

Shear = **145 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 145 \cdot (192 + ((64 - 10) / 2)) =$ **31,755 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (5{,}000 + 300) \cdot (42 \cdot 0.5) =$ **111,300 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (31{,}755 - 111{,}300) / 42 =$ **-1,893 lbs**

$(P_{uplift} \leq 0)$ = no uplift

ANCHORS

No. of Anchors (#Anchors): **1**

Pull Out Capacity per Anchor (T_{Anchor}): **4,094 lbs**

Shear Capacity per Anchor: **4,468 lbs**

COMBINED STRESS

Fully Loaded	= ((0 / 1) / 4,094) + ((193 / 2) / 4,468)	= **0.022**
Top Shelf Loaded	= ((0 / 1) / 4,094) + ((145 / 2) / 4,468)	= **0.016**
Seismic UpLift Critical (LC#7B)	= (0 / 1) / 4,094	= **0**

Base Plate Analysis: Type B

The base plate will be analyzed with the rectangular stress resulting from the vertical load P, combined with the triangular stresses resulting from the moment Mb (if any). Three criteria are used in determining Mb:
1. Moment capacity of the base plate
2. Moment capacity of the anchor bolts
3. $V_{col} \cdot h/2$ (full fixity)
Mb is the smallest value obtained from these three criteria.

F_y = **36000 psi**

P_{col} = **10680 lbs**

M_{Base} = **0 in-lbs**

$P/A = P_{col}/(D \cdot B) = 10680 / (4.69 \cdot 5.09) =$ **447 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 0 / (4.69 \cdot 5.09^2 / 6) =$ **0 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 0 \cdot (2 \cdot 1.03/5.09) =$ **0 psi**

$f_{b1} = f_b - f_{b2} = 0 - 0 =$ **0 psi**

$M_b = wb_1^2 / 2 = (b_1^2 / 2) \cdot (f_a + f_{b1} + 0.67 \cdot f_{b2})$

$= (1.03^2 / 2) \cdot (447 + 0 + 0.67 \cdot 0) =$ **238.15 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.03sq.in.**

$F_{Base} = 0.9 \cdot F_y =$ **32,400 psi**

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 238.15 / (0.03 \cdot 32,400) =$ **0.23**

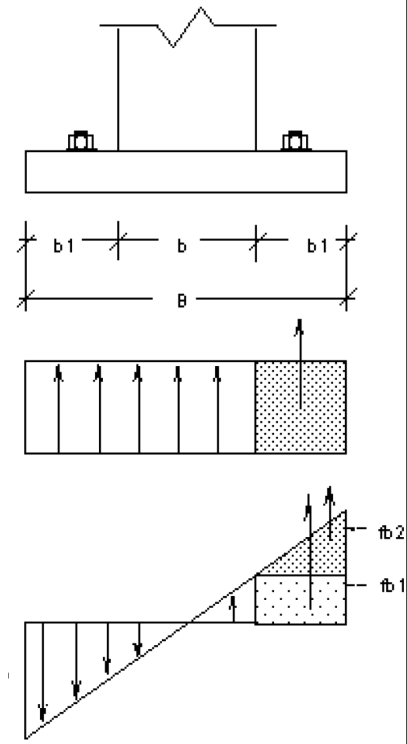


Plate width B =	5.09 in.
Plate depth D =	4.69 in.
Plate thickness t =	0.19 in.
Column width b =	3.03 in.
Column depth d =	2.72 in.
b1 =	1.03 in.

Equation for Maximum Considered Earthquake Base Rotation

Per RMI 2012 Commentary 2.6.4

$$\alpha_S = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left(\frac{k_c + k_{be}}{k_c k_{be}}\right)}{\left(N_c + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}}\right)\right)\left(\frac{k_c + k_{be}}{k_b + k_{ce}}\right)}$$

α_s - the first iteration of the second order amplification term computed using W_{pi} from section 2.6.4 of the Commentary

# of levels	3
min. # of bays	3
N_c	36
N_b	8
k_c	360 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	150 kip-in/rad
k_{ce}	603 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	0.98 in^4
H	192 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	93 in	5 kip
2	157 in	5 kip
3	222 in	5 kip

Where:

W_{pi} = the weight of the ith pallet supported by the storage rack
h_{pi} = the elevation of the center of gravity of the ith pallet with respect to the base of the storage rack
N_L = the number of loaded levels
k_c = the rotational stiffness of the connector
k_{be} = the flexural rotational stiffness of the beam-end
k_b = the rotational stiffness of the base plate
k_{ce} = the flexural rotational stiffness of the base upright-end
N_c = the number of beam-to-upright connections
N_b = the number of base plate connections

$$k_{be} = \frac{6EI_b}{L} \quad k_{ce} = \frac{4EI_c}{H} \quad k_b = \frac{EI_c}{H}$$

L = the clear span of the beams
H = the clear height of the upright
I_b = the moment of inertia about the bending axis of each beam
I_c = the moment of inertia of each base upright
E = the Young's modulus of the beams

α_s = 1.33

Per RMI 2012 7.1.3

$$\theta_b = \frac{C_d(1+\alpha_S)M_b}{k_b}$$

C_d= the deflection amplification factor per section 2.6.6
M_b= the base moment from analysis
Θ_b= 0.56

Per RMI 2012 2.6.6,

in unbraced direction, seismic separation for rack structure is 0.05 h_{total}. Therefore

$\tan\Theta_{max}$=0.5 Θ_{max}=2.862 rad Θ_b ok

Maximum moment in base plate

M_{max}= if one anchor, then 0 OR (# of anchors / 2) * anchor pull out capacity * spacing of anchor(Sx)

M_{max}=0 kip-in $\geq M_b$ OK

SLAB AND SOIL ANALYSIS (LRFD)

Slab/Soil analysis based on Empirical Method - FEMA 460 Appendix D

P_{max} = Gravity_Load (see Basic Load Combinations) = **10,680 lbs**

f'_t = $7.5 \cdot (f'_c)^{1/2}$ = **474 psi**

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **2.353 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **16.825 in.**

$b,req'd$ = $1.5 \cdot b$ = **25 in.**

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **157,467 lbs**

P_a = $\phi \cdot P_a$ = **94,480 lbs**

P_{max} / P_a = **0.11**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **7,650 lbs**

f'_t = $7.5 \cdot (f'_c)^{1/2}$ = **474 psi**

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **157,467 lbs**

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **2.353 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **16.825 in.**

$b,req'd$ = $1.5 \cdot b$ = **25 in.**

P_a = P_n / Ω = **52,489 lbs**

P_{max} / P_a = **0.15**

Base Plate

Width B	5.094 in.
Depth W	4.688 in.

Frame

Frame depth d	42 in.

Concrete

Thickness t	7 in.
f'c	4,000 psi
ϕ	0.6
Ω	3
λ	1
k_s	50 pci
r_1	2.44 in
E_c	3,604,997 psi