PRCTI20230388

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LICENSED IN 50 STATES

ANALYSIS OF
STORAGE RACKS
FOR

Abatix

410 Valley Ave., Ste 102, Puyallup, WA

Job No. 23-0079

Approved by:

SAL E. FATEEN, P.E.

SAL E. FATEEN
STATE OF WASHINGTON
#29558
REGISTERED
PROFESSIONAL ENGINEER

SEIZMIC Engineering

Digitally signed by Sal Fateen
Date: 2023.01.12 13:33:56-08'00'

EXPIRES
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PROJECT: Abatix
FOR: Interlake-Mecalux_Benj
ADDRESS: 410 Valley Ave., S
Puyallup, WA
SHEET#: 1
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

Table of Contents

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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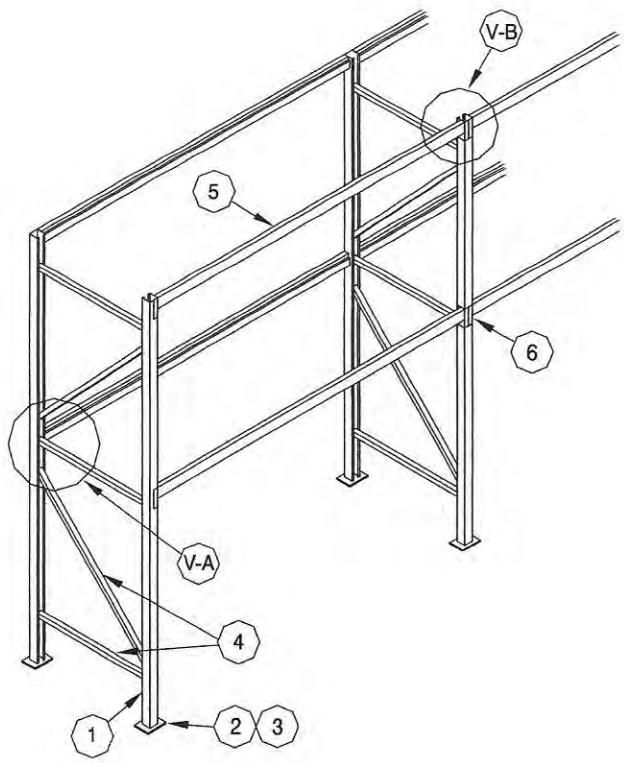
SHEET#: 2
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

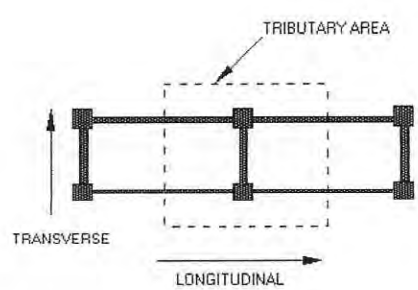
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction 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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ADDRESS: 410 Valley Ave., S
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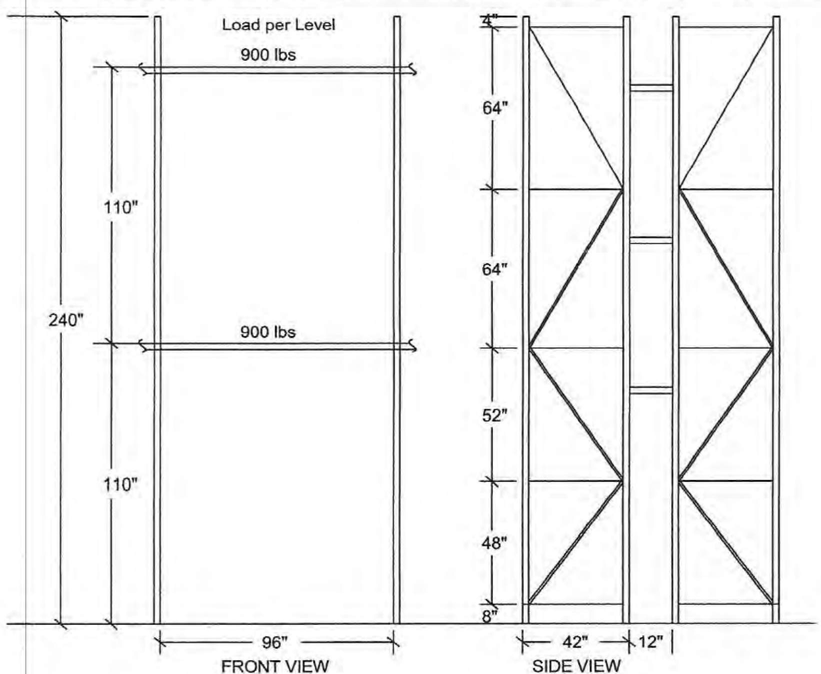
SHEET#: 3
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

COMPONENTS AND SPECIFICATIONS   Configuration 1: Green (12"RS)   M 1.7

Analysis per section 2209 of the IBC2018

Levels: 2   Panels: 4 $S_s = 1.28$ $F_a = 1.2$ $I = 1$ $S_1 = 0.44$ $F_v = 1.86$ SDC = D $V_{Long} = 85$ lbs. $V_{Trans} = 358$ lbs. $P_{static} = 1000$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 2.7 - 0.09 (313) Steel = 55000 psi Stress = 32% (level 1) **HORIZONTAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 10% (panel 1) **DIAGONAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 25% (panel 4)	**Level 1** 5.94 x 2.75 -0.059 (59E) Steel = 55 ksi Max Static Cap. = 8972 lb. Stress = 11% **Level 2** 2.75 x 2.75 -0.059 (27E) Steel = 55 ksi Max Static Cap. = 2502 lb. Stress = 39% Max stress = 39% (level 2)	**Level 1** 5 Tab Connector (IM) Stress = 5% **Level2** 3 Tab 2" cc Connector (IM) Stress = 12% Max stress = 12% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 7.75 x 5 x 0.375 in. 2 anchors/plate Moment = 2295 in-lb. Stress = 5%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 4% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 2480 lbs. Shear Capacity = 3351 lbs. Anchor stress = 27%

Notes:
Anchors: U0074596
Welded J footplate
SPST DD1-3
Standard row spacers are required in this profile.
Diagonal Braces Doubled 1 - 3

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COMPONENTS AND SPECIFICATIONS   Configuration 2: Blue (12"RS)   M 1.7

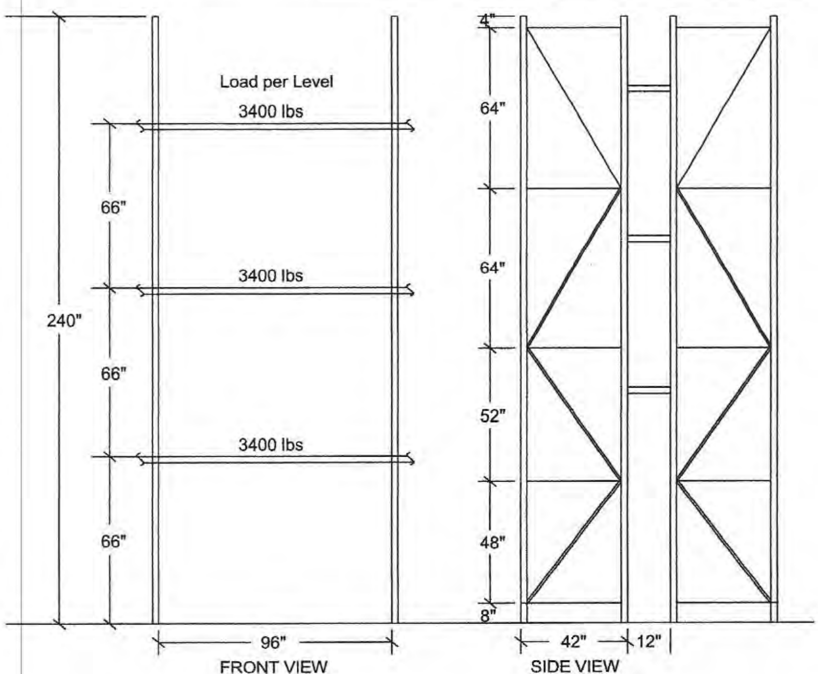
Analysis per section 2209 of the IBC2018

Levels: 3   Panels: 4

$S_s = 1.28$ $F_a = 1.2$ $I = 1$
$S_1 = 0.44$ $F_v = 1.86$ SDC = D

$V_{Long} = 435$ lbs.
$V_{Trans} = 1819$ lbs.

$P_{static} = 5250$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 2.7 - 0.09 (313) Steel = 55000 psi Stress = 81% (level 1) **HORIZONTAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 51% (panel 1) **DIAGONAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 85% (panel 1)	3.66 x 2.75 -0.059 (36E) Steel = 55 ksi Max Static Cap. = 4176 lb. Stress = 83% Max stress = 83% (level 1)	3 Tab 2" cc Connector (IM) Stress = 67% Max stress = 67% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 7.75 x 5 x 0.375 in. 2 anchors/plate Moment = 5833 in-lb. Stress = 19%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 21% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 2480 lbs. Shear Capacity = 3351 lbs. Anchor stress = 72%

Notes:
Anchors: U0074596
Welded J footplate
SPST DD1-3
Standard row spacers are required in this profile.
Diagonal Braces Doubled 1 - 3

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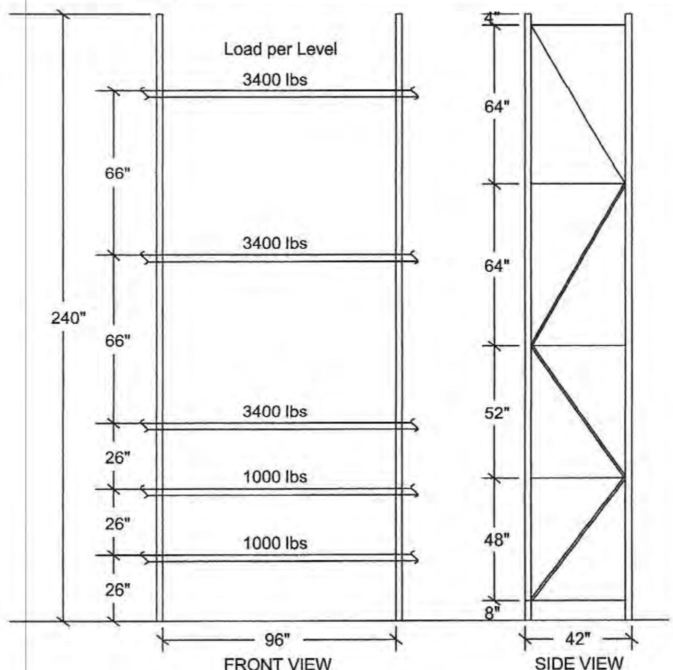
COMPONENTS AND SPECIFICATIONS   Configuration 3: Red   M 1.7

Analysis per section 2209 of the IBC2018

Levels: 5   Panels: 4 $S_s = 1.28$ $F_a = 1.2$ $I = 1$ $S_1 = 0.44$ $F_v = 1.86$ SDC = D $V_{Long} = 530$ lbs. $V_{Trans} = 2211$ lbs. $P_{static} = 6350$ lbs. $P_{seismic} = 6785$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 2.7 - 0.09 (313) Steel = 55000 psi Stress = 73% (level 4) **HORIZONTAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 62% (panel 1) **DIAGONAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 97% (panel 1)	**Level 1+** 2.75 x 2.75 -0.059 (27E) Steel = 55 ksi Max Static Cap. = 2502 lb. Stress = 43% **Level 3+** 3.66 x 2.75 -0.059 (36E) Steel = 55 ksi Max Static Cap. = 4176 lb. Stress = 83% Max stress = 83% (level 3)	3 Tab 2" cc Connector (IM) Stress = 33% Max stress = 49% (level 3)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 7.75 x 5 x 0.375 in. 2 anchors/plate Moment = 3180 in-lb. Stress = 20%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 26% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 2480 lbs. Shear Capacity = 3351 lbs. Anchor stress = 82%

Notes:
Anchors: U0074596
Welded J FP
SPST DD1-3
*Also valid for back to back rows using standard row spacers.
Diagonal Braces Doubled 1 - 3

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Loads and Distributions: Green (12"RS)

Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the IBC2018

# of Levels: 2 SDC: D R_L: 6 Ss: 1.28

Pallets Wide: 2 W_{PL}: 1800 R_T: 4 S1: 0.44

Pallets Deep: 1 W_{DL}: 200 lbs Fa: 1.2 Ip: 1

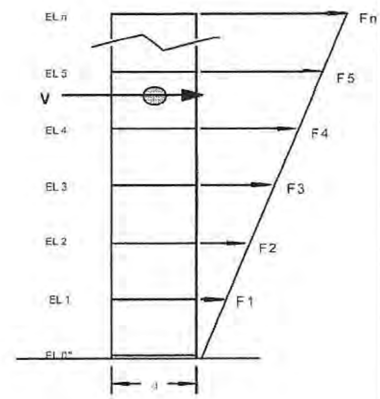
Pallet Load: 450 Fv: 1.86 Tl: 1.5

Total Frame Load: 2000 lbs

$S_{DS} = 2/3 \cdot S_s \cdot F_a =$ **1.02**

$S_{D1} = 2/3 \cdot S_1 \cdot F_v =$ **0.55**

$W_s = 0.67 \cdot W_{PL} + W_{DL} =$ **1406 lbs**



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

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

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

$= 0.55 / (1.5 \cdot 6) \cdot 1 \cdot 1406 =$ **85.92 lbs**

V_{long} need not be greater than:

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

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

$= 1.02 / 6 \cdot 1 \cdot 1406 =$ **239.02 lbs**

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

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

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

$= 0.5 \cdot 0.44 / 6 \cdot 1 \cdot 1406 =$ **51.55 lbs**

V_{long} shall not be less than:

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

$= \mathrm{Max}[0.044 \cdot S_{DS}, 0.03] \cdot I_p \cdot W_s$

$= \mathrm{Max}[0.04, 0.04, 0.03] \cdot 1 \cdot 1406 =$ **63.1 lbs**

Since: $85.92 \leq 239.02$
$\& \ 85.92 \geq 51.55$
$\& \ 85.92 \geq 63.1$

V_{long} = 85 lbs

Transverse

V_{trans} need not be greater than:

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

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

$= 1.02 / 4 \cdot 1 \cdot 1406 =$ **358.53 lbs**

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

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

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

$= 0.5 \cdot 0.44 / 4 \cdot 1 \cdot 1406 =$ **77.33 lbs**

V_{trans} shall not be less than:

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

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

$= \mathrm{Max}[0.04, 0.05, 0.03] \cdot 1 \cdot 1406 =$ **77.33 lbs**

Since: $358.53 \geq 77.33$
$\& \ 358.53 \geq 77.33$

V_{trans} = 358 lbs

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Loads and Distributions: Green (12"RS) (Page 2)

$$f_i = V\frac{W_iH_i}{\Sigma W_iH_i}$$

		Longitudinal			Transverse		
Level	h_x	w_x	w_xh_x	f_i	w_x	w_xh_x	f_i
1	110	500	55000	28.33	500	55000	119.33
2	220	500	110000	56.67	500	110000	238.67

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SHEET#: 8
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PN: 20230111_9

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

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

# of levels	2
min. # of bays	3
N_c	24
N_b	8
k_c	520 kip-in/rad
k_{be}	7763 kip-in/rad
k_b	131 kip-in/rad
k_{ce}	526 kip-in/rad
I_b	4.21 in^4
L	96 in
I_c	0.98 in^4
H	220 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	162 in	1 kip
2	273 in	1 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 = 1.57

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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CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

LRFD Basic Load Combinations: Green (12"RS)

IBC2018& RMI / ANSI MH 16.1

V_{Trans} = **358 lbs**
$M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **65,633 in-lbs**
$\beta = 0.7$

V_{Long} = **85 lbs**
$E_{Trans} = M_{Trans}$ / frame depth = **1,562 lbs**
$\beta = 1.0$ (Uplift combination only)

P = Product Load / 2 = **900 lbs**
$\rho = 1$

D = Dead Load · 0.5 = **100 lbs**
$S_{DS} = 1.02$

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 $= 1.4\,D + 1.2\,P$
$= (1.4 \cdot 100) + (1.2 \cdot 900) =$ **1,220 lbs**

2. Gravity Load $= 1.2\,D + 1.4\,P + 1.6\,L + 0.5\,(L_r \text{ or } S \text{ or } R)$
$= (1.2 \cdot 100) + (1.4 \cdot 900) + (1.6 \cdot 0) + (0.5 \cdot 0) =$ **1,380 lbs**

3. Snow/Rain $= 1.2D + 0.85P + (0.5L \text{ or } 0.5W) + 1.6(L_r \text{ or } S \text{ or } R)$
$= (1.2 \cdot 100) + (0.85 \cdot 900) + (0.5 \cdot 0) + (1.6 \cdot 0) =$ **885 lbs**

4. Wind Load $= 1.2D + 0.85P + 0.5L + 1.0W + 0.5(L_r \text{ or } S \text{ or } R)$
$= (1.2 \cdot 100) + (0.85 \cdot 900) + (0.5 \cdot 0) + (1.0 \cdot 0) + (0.5 \cdot 0) =$ **885 lbs**

5A. Seismic Load (Transverse) $= (1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Trans} + 0.2S$
$= (1.2 + 0.2 \cdot 1.02) \cdot 100 + (1.2 + 0.2 \cdot 1.02) \cdot 0.7 \cdot 900 + 0.5 \cdot 0 + 1 \cdot 1{,}562 + 0.2 \cdot 0 =$ **2,587 lbs**

5B. Seismic Load (Longitudinal) $= (1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Long} + 0.2S$
$= (1.2 + 0.2 \cdot 1.02) \cdot 100 + (1.2 + 0.2 \cdot 1.02) \cdot 0.7 \cdot 900 + 0.5 \cdot 0 + 1 \cdot 0 + 0.2 \cdot 0 =$ **1,024 lbs**

6. Wind Uplift $= 0.9D + 0.9P_{app} + 1.0W$
$= 0.9 \cdot 100 + 0.9 \cdot 900 + 1.0 \cdot 0 =$ **90 lbs**

7. Seismic Uplift $= (0.9 - 0.2S_{DS})D + (0.9 - 0.2S_{DS})\beta P_{app} - \rho E_{Trans}$
$= (0.9 - 0.2 \cdot 1.02) \cdot 100 + (0.9 - 0.2 \cdot 1.02) \cdot 1 \cdot 900 - 1 \cdot 1{,}562 =$ **-866 lbs**
For a single beam, D = **32 lbs** P = **450 lbs** I = **56 lbs**
See Base Plate tension Analysis for Over-Strength factor application.

8. Product/Live/Impact $= 1.2D + 1.6L + 0.5(S \text{ or } R) + 1.4P + 1.4I$
$(1.2 \cdot 32) + (1.6 \cdot 0) + (0.5 \cdot 0) + (1.4 \cdot 450) + (1.4 \cdot 56) =$ **746 lbs**

ASD Load Combinations for Slab Analysis

1. $(1 + 0.105S'_{DS})D + 0.75((1.4 + 0.14S_{DS})\beta P + 0.7\rho E)$
$= (1 + 0.105 \cdot 1.02) \cdot 100 + 0.75((1.4 + 0.14 \cdot 1.02) \cdot 0.7 \cdot 900 + 0.7 \cdot 1 \cdot 1{,}562) =$ **1,660 lbs**

2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$
$= (1 + 0.14 \cdot 1.02) \cdot 100 + (0.85 + 0.14 \cdot 1.02) \cdot 0.7 \cdot 900 + 0.7 \cdot 1 \cdot 1{,}562 =$ **1,833 lbs**

3. $D + P$
$= 100 + 900 =$ **1,000 lbs**

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CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

Longitudinal Analysis: Green (12"RS)

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).

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

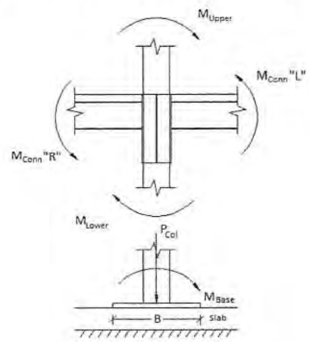
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

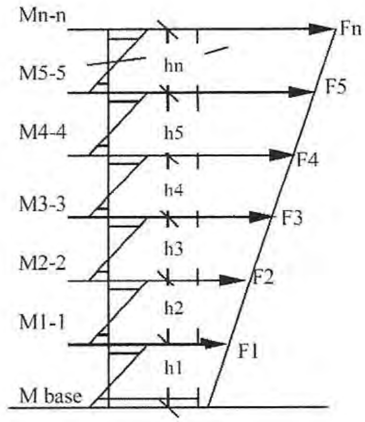
$V_{Col} = V_{Long}$ / # of columns = **43 lbs**

$M_{Base} =$ **2295 in-lbs**

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

(43 lbs · 108 in.) - 2295 in-lbs = 2349 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	110	14	1,000	2,349	645	2,994
2	110	28	500	2,349	1,413	2,587

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PN: 20230111_9

COLUMN ANALYSIS: Green (12"RS) (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2018.

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

$K_x \cdot L_x / R_x$ = 1.7 · 108 / 1.081 = **169.89**

$K_y \cdot L_y / R_y$ = 1 · 52 / 0.934 = **55.68**

KL/R_{max} = **169.89**

$r_o = (r_x^2 + r_y^2 + X_o^2)^{1/2}$ (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)

$= 3.14^2 \cdot 29500 / 169.89^2 =$ **10.088 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)

$= (1 / (2 \cdot 0.27)((10.088 + 48.04) - (10.088 + 48.04)^2$

$- (4 \cdot 0.27 \cdot 10.088 \cdot 48.04))^{1/2})=$ **8.687 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 169.89^2 =$ **10.088 ksi**

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

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

$+ (3.142 \cdot 29500 \cdot 1.66) / (0.8 \cdot 52)^2) =$ **48.04 ksi**

F_e = Min(F_{e1}, F_{e2}) =**8.687 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 8.687)^{1/2} =$ **2.516** (Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

$F_n = (0.877 / \lambda_c^2) \cdot F_y =$ **7.619** (Eq C4.1-3)

Thus:

P_n = **5380 lbs**

P_a = **4573 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	**108 in.**
K_y	1
L_y	**52 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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Puyallup, WA
SHEET#: 12
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

COLUMN ANALYSIS: Green (12"RS) (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2018.

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

$P_{ao} = P_{no}\phi_c =$ **33017 lbs**

Where:

$P_{no} = A_e F_y = 0.706 \cdot 55 =$ **38844 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **78.386 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_t / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **52.145 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 EdI_{yc}) / (S_f(K_y L_y)^2 =$ **330.503 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **52.145 ksi**

Since: $0.56\ F_y < 2.78F_y$

$F_c = (10/9)F_y(1 - (10F_y / 36F_e)) =$ **43.2 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **42.3 ksi**

$M_{nx} =$ **24944 in-lbs** $M_{ny} =$ **18768 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **22449 in-lbs** $M_{ny}\phi_b =$ **16891 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2 =$ **8481 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2 =$ **78958 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.891** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.988** (Eq C5.2.2-5)

$P_{trans} =$ **2,587 lbs** $P_{long} =$ **1,024 lbs**

$M_u = M_x =$ **2344 in-lbs** (Eq C5.2.2-2)

$P_{u\_st} = (1.2 \cdot D) + (1.4 \cdot P) =$ **1380 lbs**

$P_{u\_st} / P_a = 1380 / 4573 = 0.3$ Static Stress = **30%**

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)

$= ((1{,}024 / 4573) + (2344 / 22449) + (1 / 16891)) =$ **32%**

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)

$= (1{,}024 / 33017) + (0.85 \cdot 2344 / 22449 \cdot 0.891)) + (0.85 \cdot 1 / 16891 \cdot 0.988))) =$ **13%**

Stress3 $P_t / P_{ao} = 2{,}587 / 33017 =$ **7%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **32%**

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	**108 in.**
K_y	1
L_y	**52 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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CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

BEAM ANALYSIS Green (12"RS)

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)

$f_1(\text{comp}) = F_y \cdot (y_3 / y_2) = 55 * 1.38 / 1.53 =$ **49.64 ksi**

$f_2(\text{tension}) = F_y \cdot (y_1 / y_2) = 55 * 1.07 / 1.53 =$ **38.62 ksi**

$\Psi = -(f_2 / f_1) = -(38.62 / 49.64) =$ **-0.78**

Buckling coefficient $k = 4 + 2 \cdot (1 - \Psi)^3 + 2 \cdot (1 - \Psi)$
$= 4 + 2(1 - \mathbf{-0.78})^3 + 2(1 - \mathbf{-0.78}) =$ **18.8**

Flat Depth w = y1 + y3 = 1.07 + 1.38 = **2.452**

w/t = 2.452/0.059 = **41.56** **w/t < 200: OK**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (f_1 / E)^{1/2} = (1.052 / 2) \cdot 41.559 \cdot (49.64 / 29500)^{1/2} =$
0.41

$b1 = w \cdot (3 - \Psi) = 2 \cdot (3 - -0.78) =$ **9.26**

b2 = w/2 = **1.23**

b1 + b2 = 9.26 + 9.26 = **10.49** **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**

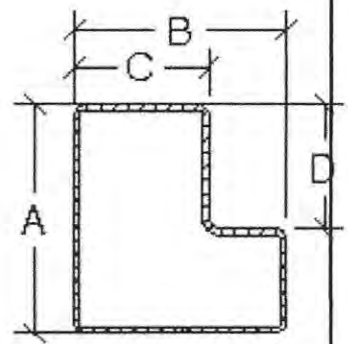
Fu/Fy = 65 / 55 = **1** **< 1.2**

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**

$F_{ya\text{-}bottom} = F_{ya\text{-}top} \cdot Y_{cg} / (A - Y_{cg}) = 59 \cdot 1.22 / (2.75 - 1.22) =$ **47**



2.75 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	2.75 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.07
Y2 =	1.53
Y3 =	1.38
Ycg =	1.22
Ix =	0.65
Sx =	0.41
E =	29500
FBeam F =	230
Beam Length L =	96

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DATE: 1/11/2023
PN: 20230111_9

BEAM ANALYSIS Green (12"RS)

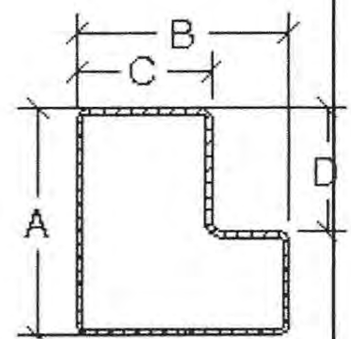
Check Allowable Tension Stress for Bottom Flange

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

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

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

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



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:

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

$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L))$
$= 1 - ((2 \cdot 230 \cdot 96) / (6 \cdot 29500 \cdot 0.65 + 3 \cdot 230 \cdot 96)) = \mathbf{0.76}$

$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x = \mathbf{22.92\ in\text{-}kip}$

$W = \phi \cdot M_n \cdot 8 \cdot (\#\ of\ Beams) / (L \cdot R_m \cdot \Omega) = (22.92 \cdot 8 \cdot 2) / (96 \cdot 0.76 \cdot 1.6)$
$= \mathbf{3162}$ lbs/pair

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x)$
$= 1 - (4 \cdot 230 \cdot 96) / (5 \cdot 230 \cdot 96 + 10 \cdot 29500 \cdot 0.65) = \mathbf{0.71}$

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

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

$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d)$
$= (384 \cdot 29500 \cdot 0.65 \cdot 2) / (180 \cdot 5 \cdot 96^2 \cdot 0.71) \cdot 1000 = \mathbf{2502}$ lbs/pair

2.75 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	2.75 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.07
Y2 =	1.53
Y3 =	1.38
Ycg =	1.22
Ix =	0.65
Sx =	0.41
E =	29500
FBeam F =	230
Beam Length L =	96

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SHEET#: 15
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

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	6,000	53,832	1,110	53,832	Pass
2	6,000	15,012	908	15,012	Pass

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CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

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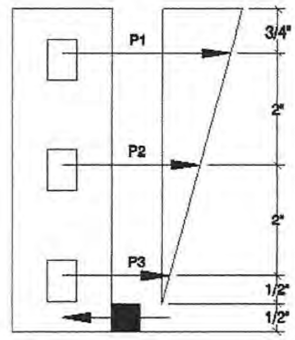
Beam to Column Analysis: Green (12"RS)

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

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

$C =$ **1.67**

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

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

$P_{Clip} = M_{Strength} / (c \cdot d) = \mathbf{7542\ lbs}$

Minimum Value of P1 Governs

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

$M_{Conn\text{-}Allow} = (P_1 \cdot 4.5) + (P_1 \cdot (2.5 / 4.5) \cdot 2.5) + (P_1 \cdot (0.5 / 4.5) \cdot 0.5) = \mathbf{14296.39\ in\text{-}lbs}$

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DATE: 1/11/2023
PN: 20230111_9

BRACE ANALYSIS Green (12"RS) (Panel 4)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2018.

Section Subject to Torsional or Flexural-Torsion Buckling
(Section C4.1.2)

$K_x \cdot L_x / R_x = 0 \cdot 69 / 0.74 = \mathbf{93.74}$
$K_y \cdot L_y / R_y = 1 \cdot 69 / 0.45 = \mathbf{153.95}$
$KL / R_{max} = \mathbf{153.95}$

$r_o = (r_x^2 + r_y^2 + x_o^2)^{1/2}$
$= (0.74^2 + 0.45^2 + -1^2)^{1/2} = \mathbf{1.32\ in.}$ (Eq C3.1.2.1-7)

$\beta = 1 - (x_o / r_o)^2 = 1 - (-1 / 1.32)^2 = \mathbf{0.43}$ (Eq C4.1.2-3)

$F_{e1} = \Pi^2 E / (KL / r)_{max}{}^2 = 3.14^2 \cdot 29500 / 153.95^2 = \mathbf{12.285\ ksi}$ (Eq C4.1.1-1)

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

$= (1 / (2 \cdot 0.43)((33.14 + 11.1) - ((33.14 + 11.1)^2$
$- (4 \cdot 0.43 \cdot 33.14 \cdot 11.1))^{1/2}) = \mathbf{9.121\ ksi}$ (Eq C4.1.2-1)

where:

$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$
$= 3.14^2 \cdot 29500 / 153.95^2 = \mathbf{33.136\ ksi}$ (Eq C3.1.2-11)

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$
$= 1 / 0.26 \cdot 1.32^2 (11300 \cdot 0.0003$
$+ (3.14^2 \cdot 29500 \cdot 0.03) / (0.8 \cdot 69)^2) = \mathbf{11.1\ ksi}$ (Eq C3.1.2-9)

$F_e = \mathrm{Min}(Fe1, Fe2) = \mathbf{9.121\ ksi}$

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

$\lambda_c = (F_y / F_e)^{1/2} = (50 / 9.121)^{1/2} = \mathbf{2.341}$
Since $\lambda_c >= 1.5$, $F_n = (0.877 / (\lambda_c^2)) \cdot F_y = \mathbf{7.999}$ (Eq C4.1-4)

Thus (Eq C4.1-3)

$P_n = \mathbf{2{,}053\ lbs}$
$P_a = P_n \cdot \phi_c = \mathbf{1{,}745\ lbs}$

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
Ix	0.139 in^4
Sx	0.156 in^3
Sx net	0.156 in^3
Rx	0.736 in.
Iy	0.052 in^4
Sy	0.056 in^3
Ry	0.448 in.
J	0 in^4
Cw	0.025 in^6
Jx	1.337 in.
Xo	-0.995 in.
Kx	0
Lx	69 in.
Ky	1
Ly	69 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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DATE: 1/11/2023
PN: 20230111_9

BRACE ANALYSIS Green (12"RS) (Panel 4)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2018.

Lateral-Torsional Buckling Strength [Resistance] (Section C3.1.2)

$P_{ao} = P_{no}\phi_c = 12{,}830 \cdot 0.85 = \mathbf{10{,}906\ lbs.}$

Where $P_{no} = A_e F_y = 0.26 \cdot 50 = \mathbf{12{,}830\ lbs.}$

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f = 41.51$ ksi

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = 11$ ksi (Eq C3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 = 36.22$ ksi (Eq C3.1.2.1-10)

$F_{e,min} = \mathbf{11\ ksi}$ (Eq C3.1.2.1-14)

Since, $F_e <= 0.56F_y$

$F_c = F_e = \mathbf{11\ ksi}$ (Eq C3.1.2.1-3)

reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = \mathbf{11\ ksi}$

$M_{nx} = \mathbf{1{,}717\ in\text{-}lbs}$ $M_{ny} = \mathbf{614\ in\text{-}lbs}$ $M_c = M_{n,min}$

$M_{nx}\phi_b = \mathbf{1{,}545\ in\text{-}lbs}$ $M_{ny}\phi_b = \mathbf{552\ in\text{-}lbs}$

$P_{Ex} = \Pi^2 E I_x / (K_x L_x)^2 = \mathbf{8{,}500\ lbs}$ (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 E I_y / (K_y L_y)^2 = \mathbf{3{,}149\ lbs}$ (Eq C5.2.2-7)

Max $P_a = \mathbf{2{,}053\ lbs}$

$V_{Trans} = \mathbf{233\ lbs}$

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} = \mathbf{69.04\ in.}$

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = \mathbf{440.03\ lbs.}$

Brace Stress = $V_{Diag} / P_a = \mathbf{25\%}$

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
Ix	0.139 in^4
Sx	0.156 in^3
Sx net	0.156 in^3
Rx	0.736 in.
Iy	0.052 in^4
Sy	0.056 in^3
Ry	0.448 in.
J	0 in^4
Cw	0.025 in^6
Jx	1.337 in.
Xo	-0.995 in.
Kx	0
Lx	69 in.
Ky	1
Ly	69 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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PROJECT: Abatix
FOR: Interlake-Mecalux_Benj
ADDRESS: 410 Valley Ave., S
Puyallup, WA
SHEET#: 19
CALCULATED BY: kelvira
DATE: 1/11/2023
PN: 20230111_9

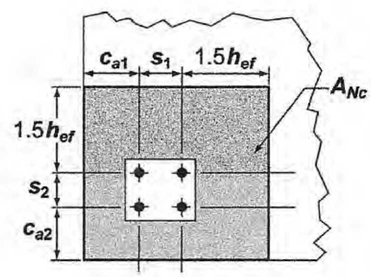
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 Green (12"RS)

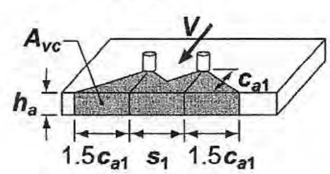
Assumed cracked concrete application

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)	= **6 in.**	C_{a1}	= 12	$useC_{a1,adj}$	= **4.875 in.**
Min. Slab Thickness (h)	= **5.5 in.**	C_{a2}	= 12	$useC_{a2,adj}$	= **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})	= Hef	S_1	= **6 in.**	Use $S_{1,adj}$	= **6 in.**
Number of Anchors (n)	= **2**	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 = **3307 lbs**	0.75	**2480 lbs**
Shear Capacity = **4468 lbs**	0.75	**3351 lbs**

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DATE: 1/11/2023
PN: 20230111_9

ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Green (12"RS)

Steel Strength	17.4.1
$\phi = \mathbf{0.75}$	17.3.3.a i
$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 2 \cdot 0.099 \cdot 114000 = \mathbf{16{,}929\ lbs}$	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
$\phi = \mathbf{0.65}$	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) = \mathbf{153.563\ sq.in.}$	
$A_{Nco} = 9h_{ef}^2 = \mathbf{95.063\ sq.in.}$	
Check if $A_{Nco} \geq A_{Nc}$ $\quad A_{Nc}/A_{Nco} = \mathbf{1.615}$	
$\Psi_{ec,N} = \mathbf{1}$	17.4.2.4
$\Psi_{ed,N} = \mathbf{1}$	17.4.2.5
$\Psi_{C,N} = \mathbf{1}$	17.4.2.6
$K_c = \mathbf{17}$	
$\lambda_a = \mathbf{1}$	
$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} = \mathbf{6299\ lbs}$	17.4.2.2 d
$\Psi_{cp,N} = \mathbf{1}$	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
$0.65 \cdot (153.563/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299 = \mathbf{6{,}614\ lbs}$	
Pullout Strength ϕN_{pn}	17.4.3
$\phi = \mathbf{0.65}$	17.3.3 c ii Category 1-B
$\Psi_{cp} = \mathbf{1}$	17.4.3.6
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} = \mathbf{16{,}442\ lbs}$	17.4.3.1

Steel Strength (ϕN_{sa}) = **16,929 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **6,614 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **16,442 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Green (12"RS)

Steel Strength ϕV_{sa} V_{sa}=6,875 / Anchor -- per report	17.5.1
ϕ = **0.65**	17.3.3. Condition a ii
$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 2 \cdot 6{,}875 =$ **8,938 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.**	
$A_{Vco} = 3C_{a1}h_a =$ **216 sq.in.**	
Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1.167**	
$\Psi_{ec,V} = 1$	17.5.2.5
$\Psi_{ed,V} =$ **0.9**	17.5.2.6
$\Psi_{C,V} = 1$	17.5.2.7
$\Psi_{h,V} = 1.732$	17.5.2.8
d_a = **0.5 in.**	17.5.2.2
L_e = **1 in.**	17.2.6 d
$\lambda_a = 1$	
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
$0.7 \cdot (252/216) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.732 \cdot 14{,}948 =$ **19,030 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} = **10,175 lbs**	
$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 10{,}175 =$ **14,245 lbs**	
Steel Strength (ϕV_{sa}) = **8,938 lbs**	
Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **19,030 lbs**	
Embedment Strength - Pryout Strength (ϕV_{cpg}) = **14,245 lbs**	

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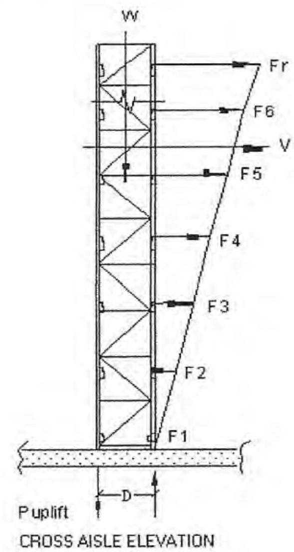
OVERTURNING ANALYSIS Configuration1 Green (12"RS)

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



W_{pl} = **1,800 lbs** W_{dl} = **200 lbs**

$W_{pl} \cdot 67\% = 1{,}800 \cdot 0.67 =$ **1,206 lbs**

$V_{Trans} = (1 \cdot 0.255 \cdot 1 \cdot ((0.67 \cdot 1{,}206) + 200)) =$ **257 lbs**

$M_{ovt} = V_{Trans} \cdot Ht = 257 \cdot 236 =$ **60,652 in-lbs**

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

$= ((1{,}800 \cdot 0.67) + 200) \cdot 42 \cdot 0.5 =$ **29,526 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (60{,}652 - 29{,}526) / 42 =$ **741 lbs**

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (60{,}652 + 29{,}526) / 42 =$ **2,147 lbs**

TOP SHELF LOADED

Shear = **280 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 280 \cdot (220 + ((110 - 10) / 2)) =$ **75,600 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (900 + 200) \cdot (42 \cdot 0.5) =$ **23,100 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (75{,}600 - 23{,}100) / 42 =$ **1,250 lbs**

ANCHORS

No. of Anchors (#Anchors): **2**

Pull Out Capacity per Anchor (T_{Anchor}): **2,480 lbs**

Shear Capacity per Anchor: **3,351 lbs**

COMBINED STRESS

Fully Loaded	= ((741 / 2) / 2,480) + ((257 / 4) / 3,351)	= **0.169**
Top Shelf Loaded	= ((1,250 / 2) / 2,480) + ((280 / 4) / 3,351)	= **0.273**
Seismic UpLift Critical (LC#7B)	= (867 / 2) / 2,480	= **0.175**

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SHEET#: 23
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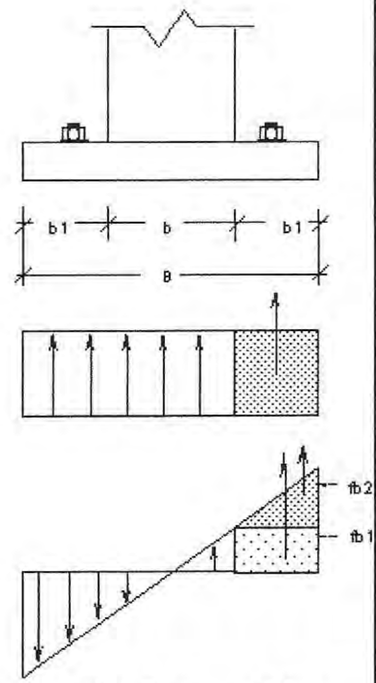
PN: 20230111_9

Base Plate Analysis: Green (12"RS)

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} = **2587 lbs**

M_{Base} = **2295 in-lbs**

$P/A = P_{col}/(D \cdot B) = 2587 / (5 \cdot 7.75) =$ **67 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 2295 / (5 \cdot 7.75^2 / 6) =$ **45.85 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 45.85 \cdot (2 \cdot 2.36/7.75) =$ **27.93 psi**

$f_{b1} = f_b - f_{b2} = 45.85 - 27.93 =$ **17.93 psi**

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

$= (2.36^2 / 2) \cdot (67 + 17.93 + 0.67 \cdot 27.93) =$ **287.68 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.18sq.in.**

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 287.68 / (0.18 \cdot 32,400) =$ **0.05**

Plate width B =	7.75 in.
Plate depth D =	5 in.
Plate thickness t =	0.38 in.
Column width b =	3.03 in.
Column depth d =	2.72 in.
b1 =	2.36 in.
S_x =	6 in.
S_y =	0 in.
T_{Total} =	4,960 lbs.
T_c =	0.35 in.

Base Plate Tension analysis

per ACI318-14 17.2.3.4.3 (b), ductile yield of base plate

Moment Arm (L) = $(S_x - b) / 2 =$ **1.49 in.**

$M_{anchor} = T_{Total} / 2 \cdot L =$ **3682.8 in-lbs**

$S = D \cdot t^2/6 =$ **0.102in³**

$M_{baseplate} = S \cdot F_y =$ **3,683 in-lbs**

$\phi M_{baseplate} = 0.9 \cdot M_n =$ **3,315 in-lbs**

$\phi M_{baseplate} < M_{anchor}$, Base plate will yield first.

Since the base plate will yield before anchor getting full tension capacity, over-strength factor is not applicable.

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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

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

# of levels	2
min. # of bays	3
N_c	24
N_b	8
k_c	520 kip-in/rad
k_{be}	7763 kip-in/rad
k_b	131 kip-in/rad
k_{ce}	526 kip-in/rad
I_b	4.21 in^4
L	96 in
I_c	0.98 in^4
H	220 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	162 in	1 kip
2	273 in	1 kip

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

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 = 0.29

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 = 0.36

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}=14,880 kip-in $\geq M_b$ OK

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SLAB AND SOIL ANALYSIS (LRFD)

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

P_{max} = Gravity_Load (see Basic Load Combinations) = **2,588 lbs**

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

$d{,}req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **1.145 in.**

$b = (E_c \cdot d{,}req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **9.801 in.**

$b{,}req'd = 1.5 \cdot b =$ **15 in.**

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **118,416 lbs**

$P_a = \phi \cdot P_a =$ **71,049 lbs**

$P_{max} / P_a =$ **0.04**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **1,834 lbs**

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

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **118,416 lbs**

$d{,}req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **1.145 in.**

$b = (E_c \cdot d{,}req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **9.801 in.**

$b{,}req'd = 1.5 \cdot b =$ **15 in.**

$P_a = P_n / \Omega =$ **39,472 lbs**

$P_{max} / P_a =$ **0.05**

Base Plate

Width B	7.75 in.
Depth W	5 in.

Frame

Frame depth d	42 in.

Concrete

Thickness t	6 in.
f'c	4,000 psi
ϕ	0.6
Ω	3
λ	1
k_s	50 pci
r_1	3.11 in
E_c	3,604,997 psi