THE APPROVED CONSTRUCTION PLANS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.
PRINT in COLOR and to SCALE.

Approval of submitted plans is not an approval of omissions or oversight by this office or noncompliance with any applicable regulations of local government. The contractor is responsible for making sure that the building complies with all applicable building codes and regulations of the local government.

SEIZMIC INC

MATERIAL HANDLING ENGINEERING

EST. 1985

REVISION:
PRCTI20240671

SPECIAL PRODUCTS	*CONVEYORS*	*STORAGE RACKS*	*OTHER SERVICES*	*SHELVING*	*SPECIAL PRODUCTS*
TANK SUPPORTS	TALL SUPPORTS	SELECTIVE	SEISMIC ANALYSIS	METAL	SHUTTLES
MACHINERY	HEADER STEEL	DRIVE-IN	PERMIT AQUISITION	METAL/WOOD	VLM VRC
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

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ANALYSIS OF
STORAGE RACKS
FOR

Red Dot Corporation

2504 E. Main Avenue, Puyallup, WA

Job No. 24-1042

Approved by:

SAL E. FATEEN, P.E.

6/4/2024

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

Digitally signed by Sal Fateen
Date: 2024.06.05 06:26:44-07'00'

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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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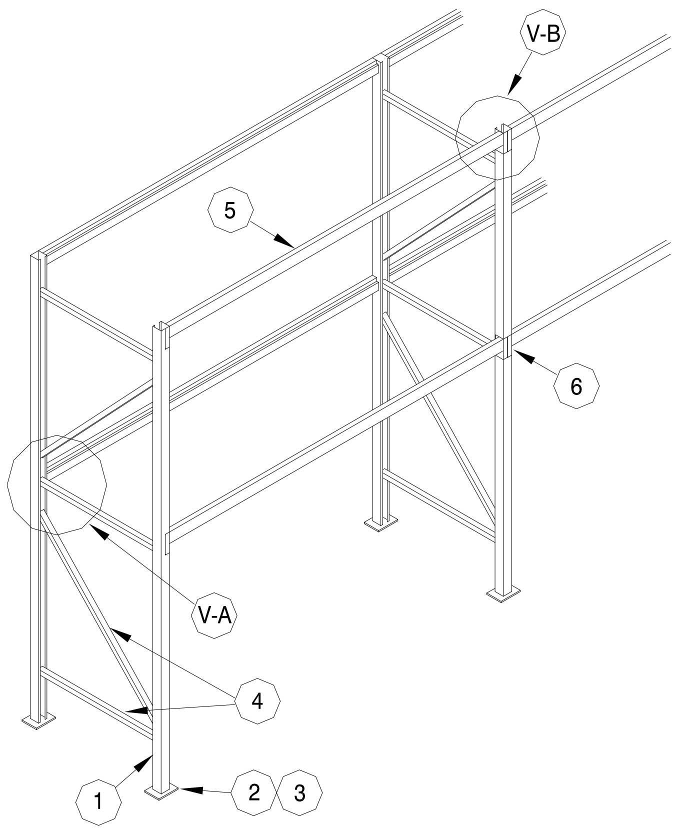
Puyallup, WA
SHEET#: 2
CALCULATED BY: ang
DATE: 6/4/2024
PN: 20240415_19

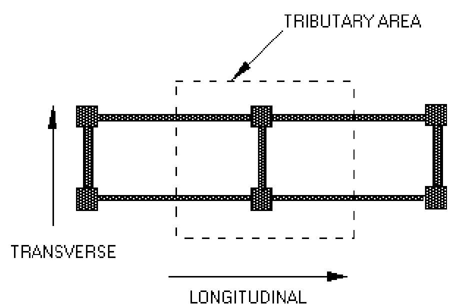
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction 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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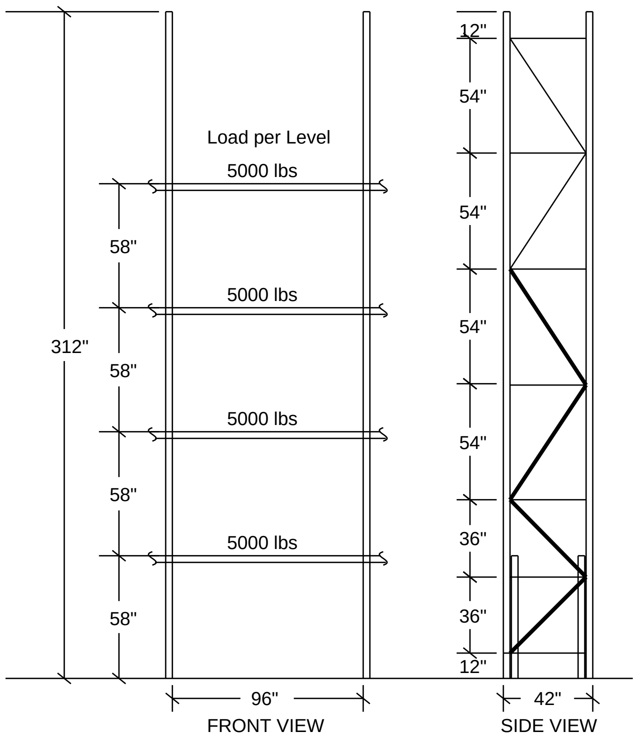
COMPONENTS AND SPECIFICATIONS Configuration 1: Type A L 1.7

Analysis per section 2209 of the 2021 IBC

Levels: 4 Panels: 6

S_s = 1.26 F_a = 1.2 I = 1 V_{Long} = 828 lbs. P_{static} = 10200 lbs.
S_1 = 0.43 F_v = 1.87 SDC = D V_{Trans} = 3484 lbs. $P_{seismic}$ = 11271 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 3 - .075 (LM20) Steel = 55000 psi Stress = 75% (level 2) **BACKER TO LEVEL 1** 3 x 3 - .075 (LM20) Steel = 55000 psi Stress = 65% (level 1) **HORIZONTAL BRACE** 1.5 x 1.25 - .075 Stress = 87% (panel 2) **DIAGONAL BRACE** 1.5 x 1.25 - .075 Stress = 72% (panel 5) **1ST-4TH DIAGONAL** 1.5 x 1.5 -0.075 (Tube) Stress = 87%	4.13 x 2.5 - .060 (SSB416M) Steel = 55 ksi Max Static Cap. = 5458 lb. Stress = 93% Max stress = 97% (level 2)	**Level 1** 4 Pin 2" cc Connector Stress = 55% **Level2** 3 Pin 2" cc Connector Stress = 53% Max stress = 55% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 8 x 8 x 0.375 in. 4 anchors/plate Moment = 0 in-lb. Stress = 20%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 31% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 2003 lbs. Shear Capacity = 3351 lbs. Anchor stress = 91%

Notes:
1.5 x 1.5" x 14 GA tube brace at diagonals 1-4.

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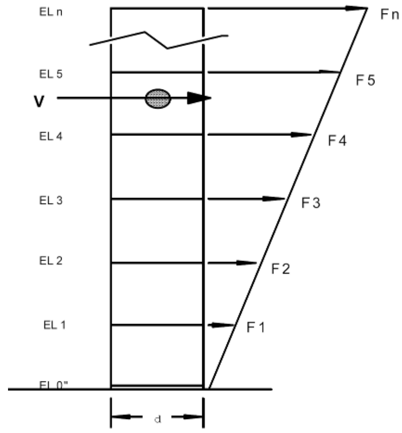
Loads and Distributions: Type A

Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2021 IBC

# of Levels:	4	SDC:	D	R_L:	6	Ss:	1.26
Pallets Wide:	2	W_{PL}:	20000	R_T:	4	S1:	0.43
Pallets Deep:	1	W_{DL}:	400 lbs	Fa:	1.2	Ip:	1
Pallet Load:	2500	Fv:	1.87	Tl:	1.5		
Total Frame Load:	20400 lbs						



$S_{DS} = 2/3 \cdot S_s \cdot F_a =$ **1.01**

$S_{D1} = 2/3 \cdot S_1 \cdot F_v =$ **0.54**

$W_s = 0.67 \cdot W_{PL} + W_{DL} =$ **13800 lbs**

Seismic Shear per RMI 2012 2.6.3:

Longitudinal

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

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

$= 0.54 / (1.5 \cdot 6) \cdot 1 \cdot 13800 =$ **828 lbs**

V_{long} need not be greater than:

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

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

$= 1.01 / 6 \cdot 1 \cdot 13800 =$ **2323 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 6 \cdot 1 \cdot 13800 =$ **497.95 lbs**

V_{long} shall not be less than:

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

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

$= \mathrm{Max}[0.04, 0.03] \cdot 1 \cdot 13800 =$ **613.27 lbs**

Since: $828 \leq 2323$
$\& \ 828 \geq 497.95$
$\& \ 828 \geq 613.27$

V_{long} = 828 lbs

Transverse

V_{trans} need not be greater than:

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

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

$= 1.01 / 4 \cdot 1 \cdot 13800 =$ **3484.5 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 4 \cdot 1 \cdot 13800 =$ **746.93 lbs**

V_{trans} shall not be less than:

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

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

$= \mathrm{Max}[0.04, 0.05, 0.03] \cdot 1 \cdot 13800 =$ **746.93 lbs**

Since: $3484.5 \geq 746.93$
$\& \ 3484.5 \geq 746.93$

V_{trans} = 3484 lbs

Loads and Distributions: Type A (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	58	2550	147900	82.8	2550	147900	348.4
2	116	2550	295800	165.6	2550	295800	696.8
3	174	2550	443700	248.4	2550	443700	1045.2
4	232	2550	591600	331.2	2550	591600	1393.6

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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

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

# of levels	4
min. # of bays	3
N_c	48
N_b	8
k_c	400 kip-in/rad
k_{be}	2930 kip-in/rad
k_b	148 kip-in/rad
k_{ce}	595 kip-in/rad
I_b	1.59 in^4
L	96 in
I_c	1.17 in^4
H	232 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	84 in	5 kip
2	142 in	5 kip
3	200 in	5 kip
4	259 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 = 3.44

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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SHEET#: 7
CALCULATED BY: ang
DATE: 6/4/2024
PN: 20240415_19

LRFD Basic Load Combinations: Type A

2021 IBC& RMI / ANSI MH 16.1

V_{Trans} = **3,484 lbs** | $M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **606,216 in-lbs** | $\beta = 0.7$

V_{Long} = **828 lbs** | $E_{Trans} = M_{Trans}$ / frame depth = **14,433 lbs** | $\beta = 1.0$ (Uplift combination only)

P = Product Load / 2 = **10,000 lbs** | $\rho = 1$

D = Dead Load $\cdot$ 0.5 = **200 lbs** | $S_{DS} = 1.01$

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
= $(1.4 \cdot 200) + (1.2 \cdot 10{,}000)$= **12,280 lbs**

2. Gravity Load = 1.2 D + 1.4 P + 1.6 L + 0.5 (L_r or S or R)
= $(1.2 \cdot 200) + (1.4 \cdot 10{,}000) + (1.6 \cdot 0) + (0.5 \cdot 0)$= **14,240 lbs**

3. Snow/Rain = 1.2D + 0.85P + (0.5L or 0.5W) + 1.6(L_r or S or R)
= $(1.2 \cdot 200) + (0.85 \cdot 10{,}000) + (0.5 \cdot 0) + (1.6 \cdot 0)$= **8,740 lbs**

4. Wind Load = 1.2D + 0.85P + 0.5L + 1.0W + 0.5(L_r or S or R)
= $(1.2 \cdot 200) + (0.85 \cdot 10{,}000) + (0.5 \cdot 0) + (1.0 \cdot 0) + (0.5 \cdot 0)$= **8,740 lbs**

5A. Seismic Load (Transverse) = $(1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Trans} + 0.2S$
= $(1.2 + 0.2 \cdot 1.01) \cdot 200 + (1.2 + 0.2 \cdot 1.01) \cdot 0.7 \cdot 10{,}000 + 0.5 \cdot 0 + 1 \cdot 14{,}433 + 0.2 \cdot 0$= **24,528 lbs**

5B. Seismic Load (Longitudinal) = $(1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Long} + 0.2S$
= $(1.2 + 0.2 \cdot 1.01) \cdot 200 + (1.2 + 0.2 \cdot 1.01) \cdot 0.7 \cdot 10{,}000 + 0.5 \cdot 0 + 1 \cdot 0 + 0.2 \cdot 0$= **10,094 lbs**

6. Wind Uplift = $0.9D + 0.9P_{app} + 1.0W$
= $0.9 \cdot 200 + 0.9 \cdot 10{,}000 + 1.0 \cdot 0$ = **180 lbs**

7. Seismic Uplift = $(0.9 - 0.2S_{DS})D + (0.9 - 0.2S_{DS})\beta P_{app} - \rho E_{Trans}$
= $(0.9 - 0.2 \cdot 1.01) \cdot 200 + (0.9 - 0.2 \cdot 1.01) \cdot 1 \cdot 10{,}000 - 1 \cdot 14{,}433$= **-7,314 lbs**
For a single beam, D = **32 lbs** P = **2,500 lbs** I = **312 lbs**
See Base Plate tension Analysis for Over-Strength factor application.

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

ASD Load Combinations for Slab Analysis

1. $(1 + 0.105S'_{DS})D + 0.75((1.4 + 0.14S_{DS})\beta P + 0.7\rho E)$
= $(1 + 0.105 \cdot 1.01) \cdot 200 + 0.75((1.4 + 0.14 \cdot 1.01) \cdot 0.7 \cdot 10{,}000 + 0.7 \cdot 1 \cdot 14{,}433)$= **15,891 lbs**
2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$
= $(1 + 0.14 \cdot 1.01) \cdot 200 + (0.85 + 0.14 \cdot 1.01) \cdot 0.7 \cdot 10{,}000 + 0.7 \cdot 1 \cdot 14{,}433$= **17,271 lbs**
3. D + P
= 200 + 10,000 = **10,200 lbs**

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DATE: 6/4/2024

PN: 20240415_19

Longitudinal Analysis: Type A

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).

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

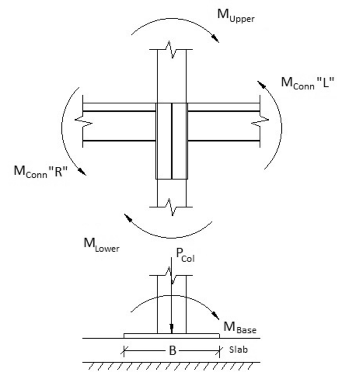
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

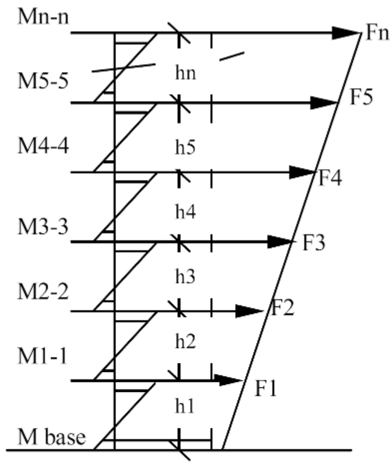
$V_{Col} = V_{Long}$ / # of columns = **414 lbs**

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

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

(414 lbs · 56 in.) - 0 in-lbs = 23184 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	58	41	10,200	23,184	5,885	29,069
2	58	83	7,650	23,184	4,760	27,944
3	58	124	5,100	23,184	4,760	27,944
4	58	166	2,550	23,184	4,760	16,352

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COLUMN WITH BACKER ANALYSIS: Type A (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$K_x \cdot L_x / R_x$ = 1.7 · 56 / 1.26 = **75.54**

$K_y \cdot L_y / R_y$ = 1 · 36 / 1.869 = **19.26**

KL/R_{max} = **75.54**

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

= $(1.26^2 + 1.869^2 + -2.838^2)^{1/2}$ = **3.624 in.**

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

= $1 - (-2.838/3.624)^2$ = **0.387**

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

= $3.14^2 \cdot 29500 / 75.54^2$ = **51.02 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)

= $(1 / (2 \cdot 0.387)((51.02 + 88.107) - (51.02 + 88.107)^2$

$- (4 \cdot 0.387 \cdot 51.02 \cdot 88.107))^{1/2})$= **35.893 ksi**

where:

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

= $3.14^2 \cdot 29500 / 75.54^2$ = **51.02 ksi**

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

= $1 / 1.473 \cdot 3.624^2 (11300 \cdot 0.003$

$+ (3.14^2 \cdot 29500 \cdot 4.773) / (0.8 \cdot 36)^2)$ = **88.107 ksi**

F_e = Min(F_{e1}, F_{e2}) =**35.893 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 35.893)^{1/2}$ = **1.238** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\hat{}(\lambda_c{}^2)) \cdot F_y$ = **28.962** (Eq C4.1-2)

Thus:

P_n = **36537 lbs**

P_a = **31057 lbs**

3 x 3 - .075	
SECTION PROPERTIES	
Depth	6 in.
Width	3 in.
t	0.074 in.
Radius	0.125 in.
Area	1.473 in.^{2}
AreaNet	1.262 in.^{2}
I_x	2.34 in.^{4}
S_x	1.56 in.^{3}
$S_{x\ Net}$	1.444 in.^{3}
R_x	1.26 in.
I_y	5.146 in.^{4}
S_y	1.597 in.^{3}
R_y	1.869 in.
J	0.003 in.^{4}
C_w	4.773 in.^{6}
J_x	2.982 in.
X_o	-2.838 in.
K_x	1.7
L_x	56 in.
K_y	1
L_y	36 in.
K_t	0.8
F_y	55 ksi
F_u	65 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

COLUMN WITH BACKER ANALYSIS: Type A (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$P_{ao} = P_{no}\phi_c$ = **58979 lbs**

Where:

$P_{no} = A_e F_y = 1.262 \cdot 55$ = **69388 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f$ = **229.49 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_t / \sigma_{ex}))^{1/2}) / (C_{TF} S_f)$ = **127.074 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2$ = **4446.75 ksi** (Eq 3.1.2.1-10)

$F_{e.min}$ = **127.074 ksi**

Since: $0.56\ F_y < 2.78 F_y$

$F_c = (10/9)F_y(1 - (10F_y / 36F_e))$ = **53.8 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c$ = **53.8 ksi**

M_{nx} = **77687 in-lbs** M_{ny} = **85931 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b$ = **69918 in-lbs** $M_{ny}\phi_b$ = **77338 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2$ = **75173 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2$ = **1156155 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex}))$ = **0.878** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey}))$ = **0.992** (Eq C5.2.2-5)

P_{trans} = **24,528 lbs** P_{long} = **10,094 lbs**

$M_u = M_x$ = **23184 in-lbs** (Eq C5.2.2-2)

$P_{u\_st} = (1.2 \cdot D) + (1.4 \cdot P)$ = **14240 lbs**

P_{u_st} / P_a = 14240 / 31057 = 0.46 Static Stress = **45%**

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)

= ((10,094 / 31057) + (23184 / 69918) + (1 / 77338)) = **65%**

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)

= (10,094 / 58979) + (0.85 · 23184 / 69918 · 0.878)) + (0.85 · 1 / 77338 · 0.992))) = **49%**

Stress3 P_t / P_{ao} = 24,528 / 58979 =**41%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **65%**

3 x 3 - .075	
SECTION PROPERTIES	
Depth	**6 in.**
Width	**3 in.**
t	**0.074 in.**
Radius	**0.125 in.**
Area	**1.473 in.** ^{2}
AreaNet	**1.262 in.** ^{2}
I_x	**2.34 in.** ^{4}
S_x	**1.56 in.** ^{3}
$S_{x\ Net}$	**1.444 in.** ^{3}
R_x	**1.26 in.**
I_y	**5.146 in.** ^{4}
S_y	**1.597 in.** ^{3}
R_y	**1.869 in.**
J	**0.003 in.** ^{4}
C_w	**4.773 in.** ^{6}
J_x	**2.982 in.**
X_o	**-2.838 in.**
K_x	1.7
L_x	**56 in.**
K_y	1
L_y	**36 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 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

COLUMN ANALYSIS: Type A (Level 2)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$K_x \cdot L_x / R_x$ = 1.5 · 56 / 1.26 = **66.66**

$K_y \cdot L_y / R_y$ = 1 · 36 / 1.115 = **32.29**

KL/R_{max} = **66.66**

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

= $(1.26^2 + 1.115^2 + -2.838^2)^{1/2}$ = **3.299 in.**

β = $1 - (Xo/ro)^2$ (Eq C4.1.2-3)

= $1 - (-2.838/3.299)^2$ = **0.26**

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

= $3.14^2 \cdot 29500 / 66.66^2$ = **65.53 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)

= $(1 / (2 \cdot 0.26)((65.53 + 106.323) - (65.53 + 106.323)^2$

$- (4 \cdot 0.26 \cdot 65.53 \cdot 106.323))^{1/2})$= **43.392 ksi**

where:

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

= $3.14^2 \cdot 29500 / 66.66^2$ = **65.53 ksi**

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

= $1 / 0.737 \cdot 3.299^2 (11300 \cdot 0.001$

$+ (3.14^2 \cdot 29500 \cdot 2.387) / (0.8 \cdot 36)^2)$ = **106.323 ksi**

F_e = Min(F_{e1}, F_{e2}) =**43.392 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 43.392)^{1/2}$ = **1.126** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\text{^}(\lambda_c^2)) \cdot F_y$ = **32.357** (Eq C4.1-2)

Thus:

P_n = **19144 lbs**

P_a = **16272 lbs**

3 x 3 - .075	
SECTION PROPERTIES	
Depth	**3 in.**
Width	**3 in.**
t	**0.074 in.**
Radius	**0.125 in.**
Area	**0.737 in.^{2}**
AreaNet	**0.631 in.^{2}**
I_x	**1.17 in.^{4}**
S_x	**0.78 in.^{3}**
$S_{x\ Net}$	**0.722 in.^{3}**
R_x	**1.26 in.**
I_y	**0.916 in.^{4}**
S_y	**0.532 in.^{3}**
R_y	**1.115 in.**
J	**0.001 in.^{4}**
C_w	**2.387 in.^{6}**
J_x	**3.114 in.**
X_o	**-2.838 in.**
K_x	1.5
L_x	**56 in.**
K_y	1
L_y	**36 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	0.9
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 A (Level 2)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$P_{ao} = P_{no}\phi_c =$ **27660 lbs**

Where:

$P_{no} = A_e F_y = 0.592 \cdot 55 =$ **32541 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **260.084 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **130.992 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 EdI_{yc}) / (S_f(K_y L_y)^2 =$ **791.132 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **130.992 ksi**

Since: $0.56\ F_y < 2.78F_y$

$F_c = (10/9)F_y(1 - (10F_y / 36F_e)) =$ **54 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **51.3 ksi**

$M_{nx} =$ **37038 in-lbs** $M_{ny} =$ **27276 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **33334 in-lbs** $M_{ny}\phi_b =$ **24548 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2 =$ **48277 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2 =$ **205694 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.857** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.967** (Eq C5.2.2-5)

$P_{trans} =$ **17,193 lbs** $P_{long} =$ **7,570 lbs**

$M_u = M_x =$ **9605 in-lbs** (Eq C5.2.2-2)

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

$P_{u\_st} / P_a = 10680 / 16272 = 0.66$ Static Stress = **65%**

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)

= ((7,570 / 16272) + (9605 / 33334) + (1 / 24548)) = **75%**

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)

= (7,570 / 27660) + (0.85 · 9605 / 33334 · 0.857)) + (0.85 · 1 / 24548 · 0.967))) = **55%**

Stress3 P_t / P_{ao} = 17,193 / 27660 =**62%**

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

3 x 3 - .075	
SECTION PROPERTIES	
Depth	**3 in.**
Width	**3 in.**
t	**0.074 in.**
Radius	**0.125 in.**
Area	**0.737 in. ^{2}**
AreaNet	**0.631 in. ^{2}**
I_x	**1.17 in. ^{4}**
S_x	**0.78 in. ^{3}**
$S_{x\ Net}$	**0.722 in. ^{3}**
R_x	**1.26 in.**
I_y	**0.916 in. ^{4}**
S_y	**0.532 in. ^{3}**
R_y	**1.115 in.**
J	**0.001 in. ^{4}**
C_w	**2.387 in. ^{6}**
J_x	**3.114 in.**
X_o	**-2.838 in.**
K_x	1.5
L_x	**56 in.**
K_y	1
L_y	**36 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	0.9
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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DATE: 6/4/2024
PN: 20240415_19

BEAM ANALYSIS Type A

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

Effective width w = C - 2t - 2r = 1.625 - (2 · 0.06) - (2 · 0.125) = **1.26 in.**

w/t = 1.255 / 0.06 = **20.99**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (F_y / E)^{1/2} = (1.052 / 2) \cdot 20.993 \cdot (55 / 29500)^{1/2} =$ **0.48**

λ <= 0.673: Flange is fully effective.

Check web for local buckling (B2.3)

f_1(comp) $= F_y \cdot (y_3 / y_2) = 55 * 1.99 / 2.18 =$ **50.34 ksi**

f_2(tension) $= F_y \cdot (y_1 / y_2) = 55 * 1.76 / 2.18 =$ **44.43 ksi**

$\Psi = -(f_2 / f_1) = -(44.43 / 50.34) =$ **-0.88**

Buckling coefficient $k = 4 + 2 \cdot (1 - \Psi)^3 + 2 \cdot (1 - \Psi)$
$= 4 + 2(1 - \mathbf{-0.88})^3 + 2(1 - \mathbf{-0.88}) =$ **21.11**

Flat Depth w = y1 + y3 = 1.76 + 1.99 = **3.755**

w/t = 3.755/0.06 = **62.8** **w/t < 200: OK**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (f_1 / E)^{1/2} = (1.052 / 2) \cdot 62.799 \cdot (50.34 / 29500)^{1/2} =$
0.59

b1 = w · (3 - Ψ) = 4 · (3 - -0.88) = **14.58**

b2 = w/2 = **1.88**

b1 + b2 = 14.58 + 1.88 = **16.46** **Web is fully effective**

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

Corner cross-sectional area $Lc = (\Pi / 2) \cdot (r + t / 2)$
$= (\Pi / 2) \cdot (0.125 + 0.06 / 2) =$ **0.243**

L_f = effective width = **1.255**

$C = 2 \cdot L_c / L_f + 2 \cdot L_c = 2 \cdot 0.243 / 1.255 + 2 \cdot L_c =$ **0.2793**

$m = 0.192 \cdot (F_u / F_y) - 0.068 = 0.192 \cdot (70 / 55) - 0.068 =$ **0.1764**

$B_c = 3.69 \cdot (F_u / F_y) - 0.819 \cdot (F_u / F_y)^2 - 1.79$
$= 3.69 \cdot (70 / 55) - 0.819 \cdot (70 / 55)^2 - 1.79 =$ **1.58**

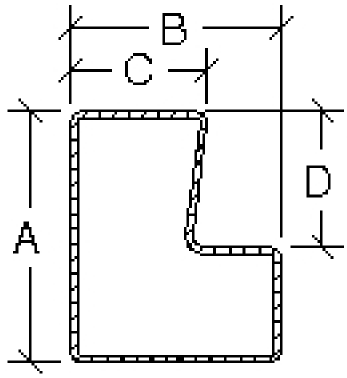
Fu/Fy = 70 / 55 = **1** **>= 1.2 = OK**

r/t = 0.125 / 0.06 = **2.09** **<= 7 = OK**

$F_{yc} = B_c \cdot F_y / (r / t)^m = 1.58 \cdot 55 / (2.09)^m =$ **76**

$F_{ya\text{-}top} = C \cdot F_{yc} + (1 - C) \cdot F_y = 0.279 \cdot 76 + (1 - 0.279) \cdot 55 =$ **61**

$F_{ya\text{-}bottom} = F_{ya\text{-}top} \cdot Y_{cg} / (A - Y_{cg}) = 61 \cdot 1.95 / (4.125 - 1.95) =$ **54**



4.13 x 2.5 - .060

Top flange width C =	1.625 in.
Bottom width B =	2.5 in.
Web depth A =	4.125 in.
Beam thickness t =	0.06 in.
Radius r =	0.125 in.
Fy =	55
Fu =	70
Y1 =	1.76
Y2 =	2.18
Y3 =	1.99
Ycg =	1.95
Ix =	1.59
Sx =	0.73
E =	29500
FBeam F =	300
Beam Length L =	96

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PN: 20240415_19

BEAM ANALYSIS Type A

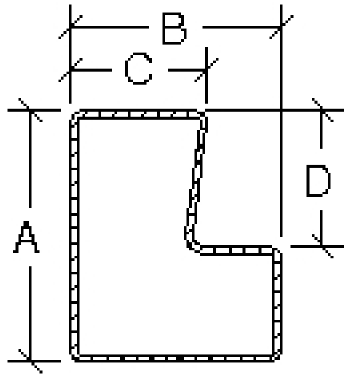
Check Allowable Tension Stress for Bottom Flange

$L_{flange\text{-}bot} = B - (2 \cdot r) - (2 \cdot t) = 2.5 - (2 \cdot 0.125) - (2 \cdot 0.06) = \mathbf{2.13}$

$C_{bottom} = 2 \cdot L_c / (L_{flange\text{-}bot} + 2 \cdot L_c) = 2 \cdot 0.243 / (2.13 + 2 \cdot 0.243) = \mathbf{0.186}$

$F_{y\text{-}bottom} = C_{bottom} \cdot F_{yc} + (1 - C_{bottom}) \cdot F_y = 0.186 \cdot 76 + (1 - 0.186) \cdot 55 = \mathbf{58.96}$

$F_{ya} = F_{ya\text{-}top} =$ **60.95 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:

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

$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L))$
$= 1 - ((2 \cdot 300 \cdot 96) / (6 \cdot 29500 \cdot 1.59 + 3 \cdot 300 \cdot 96)) = \mathbf{0.84}$

$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x =$ **42.22 in-kip**

$W = \phi \cdot M_n \cdot 8 \cdot (\#\ of\ Beams) / (L \cdot R_m \cdot \Omega) = (42.22 \cdot 8 \cdot 2) / (96 \cdot 0.84 \cdot 1.6)$
= **5218** lbs/pair

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x)$
$= 1 - (4 \cdot 300 \cdot 96) / (5 \cdot 300 \cdot 96 + 10 \cdot 29500 \cdot 1.59) = \mathbf{0.81}$

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

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

$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d)$
$= (384 \cdot 29500 \cdot 1.59 \cdot 2) / (180 \cdot 5 \cdot 96^2 \cdot 0.81) \cdot 1000 =$ **5346** lbs/pair

4.13 x 2.5 - .060

Top flange width C =	1.625 in.
Bottom width B =	2.5 in.
Web depth A =	4.125 in.
Beam thickness t =	0.06 in.
Radius r =	0.125 in.
Fy =	55
Fu =	70
Y1 =	1.76
Y2 =	2.18
Y3 =	1.99
Ycg =	1.95
Ix =	1.59
Sx =	0.73
E =	29500
FBeam F =	300
Beam Length L =	96

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	32,748	11,140	32,748	Pass
2	30,576	31,308	5,982	31,308	Pass
3	30,576	31,308	3,581	31,308	Pass
4	30,576	31,308	2,380	31,308	Pass

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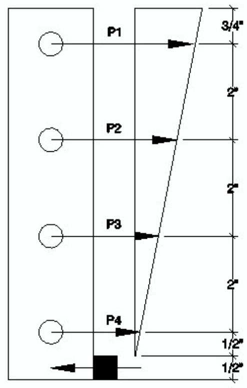
Beam to Column Analysis: Type A

1. Shear Strength of Pin



Pin Diameter = **0.35 in.**

$F_n =$ $F_{nv} =$ **54000 psi** AISI Table E3.4-1

$A_b =$ $d^2 \cdot \Pi / 4 =$ **0.1 in.**

$P_n =$ $A_b \cdot F_n =$ **5195.41 lbs** AISI Table E3.4-1

$P_{Shear} =$ $\phi P_n = 0.75 \cdot P_n =$ **3896 lbs**

2. Bearing Strength of Pin

Column Thickness $t_c =$ **0.07 in.**

Since $d / t_c < 10$ $C = 3$

$m_f =$ **1.0**

$F_u =$ **65000 psi**

$P_n =$ $C \cdot m_f \cdot d \cdot t_c \cdot F_u =$ **5050.5 lbs** AISI E3.3.1 -1

$P_{Bearing} =$ $\phi P_n = 0.75 \cdot 5050.5 =$ **3787 lbs**

3. Moment Strength of Bracket

Edge Dist. = **1 in.**

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

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

$M_n =$ $S_c \cdot F_y =$ **6985 in-lbs** AISI C3.1.1 -1

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

$C =$ **2.15**

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

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

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

Minimum Value of P1 Governs

$P_1 =$ $Min(P_{Shear}, P_{Bearing}, P_{Clip}) =$ **3787 lbs**

$M_{Conn\text{-}Allow} =$ $(P_1 \cdot 6.5) + (P_1 \cdot (4.5 / 6.5) \cdot 4.5) + (P_1 \cdot (2.5 / 6.5) \cdot 2.5) + (P_1 \cdot (0.5 / 6.5) \cdot 0.5) =$ **40200.46 in-lbs**

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PN: 20240415_19

BRACE ANALYSIS Type A (Panel 5)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$K_x \cdot L_x / R_x$ = 1 · 60 / 0.614 = **97.66**

$K_y \cdot L_y / R_y$ = 1 · 60 / 0.404 = **148.63**

KL/R_{max} = **148.63**

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

$= (0.614^2 + 0.404^2 + -0.917^2)^{1/2} =$ **1.175 in.**

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

$= 1 - (-0.917/1.175)^2 =$ **0.391**

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

$= 3.14^2 \cdot 29500 / 148.63^2 =$ **13.181 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)

$= (1 / (2 \cdot 0.391)((30.53 + 17.818) - (30.53 + 17.818)^2$

$- (4 \cdot 0.391 \cdot 30.53 \cdot 17.818))^{1/2}) =$ **12.519 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 97.66^2 =$ **30.53 ksi**

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

$= 1 / 0.279 \cdot 1.175^2(11300 \cdot 0.001$

$+ (3.142 \cdot 29500 \cdot 0.015) / (0.8 \cdot 60)^2) =$ **17.818 ksi**

F_e = Min(F_{e1}, F_{e2}) =**12.519 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (36 / 12.519)^{1/2} =$ **1.696** (Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

$F_n = (0.877 / \lambda_c^2) \cdot F_y =$ **10.979** (Eq C4.1-3)

Thus:

P_n = **3064 lbs**

P_a = **2604 lbs**

1.5 x 1.25 - .075	
SECTION PROPERTIES	
Depth	**1.5 in.**
Width	**1.25 in.**
t	**0.075 in.**
Radius	**0.112 in.**
Area	**0.279 in.^{2}**
AreaNet	**0.279 in.^{2}**
I_x	**0.105 in.^{4}**
S_x	**0.141 in.^{3}**
$S_{x\ Net}$	**0.141 in.^{3}**
R_x	**0.614 in.**
I_y	**0.046 in.^{4}**
S_y	**0.056 in.^{3}**
R_y	**0.404 in.**
J	**0.001 in.^{4}**
C_w	**0.015 in.^{6}**
J_x	**1.183 in.**
X_o	**-0.917 in.**
K_x	1
L_x	**60 in.**
K_y	1
L_y	**60 in.**
K_t	0.8
F_y	**36 ksi**
F_u	**42 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

BRACE ANALYSIS Type A (Panel 5)

Analyzed per RMI, AISI 2012 (LRFD) and the 2021 IBC.

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

$P_{ao} = P_{no}\phi_c$ = **8540 lbs**

Where:

$P_{no} = A_e F_y = 0.279 \cdot 36$ = **10047 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f$ = **54.46 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f)$ = **18.331 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2$ = **39.287 ksi** (Eq 3.1.2.1-10)

$F_{e.min}$ = **18.331 ksi**

Since: $F_e \leq 0.56F_y$

$F_c = F_e$ = **18.331 ksi** (Eq C3.1.2.1-3)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c$ = **18.3 ksi**

M_{nx} = **2571 in-lbs** M_{ny} = **1019 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b$ = **2314 in-lbs** $M_{ny}\phi_b$ = **917 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2$ = **8524 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2$ = **3679 lbs** (Eq C5.2.2-7)

P_a = **2604 lbs**

V_{Trans} = **871 lbs**

$V_{Trans(new)} = 871 \cdot 1.3$ = **1132 lbs**

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2}$ = **60 in.**

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D$ = **1887 lbs**

Brace Stress = V_{Diag} / P_a = **72%**

1.5 x 1.25 - .075	
SECTION PROPERTIES	
Depth	**1.5 in.**
Width	**1.25 in.**
t	**0.075 in.**
Radius	**0.112 in.**
Area	**0.279 in.^{2}**
AreaNet	**0.279 in.^{2}**
I_x	**0.105 in.^{4}**
S_x	**0.141 in.^{3}**
$S_{x\ Net}$	**0.141 in.^{3}**
R_x	**0.614 in.**
I_y	**0.046 in.^{4}**
S_y	**0.056 in.^{3}**
R_y	**0.404 in.**
J	**0.001 in.^{4}**
C_w	**0.015 in.^{6}**
J_x	**1.183 in.**
X_o	**-0.917 in.**
K_x	1
L_x	**60 in.**
K_y	1
L_y	**60 in.**
K_t	0.8
F_y	**36 ksi**
F_u	**42 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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PN: 20240415_19

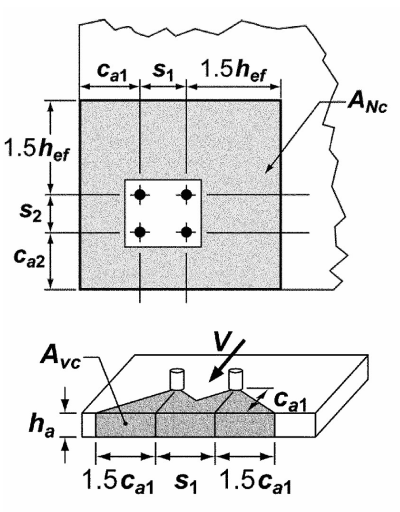
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-19(ACI 318-14), CHAPTER 17 Configuration 1 Type A

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	$useC_{a1,adj}$	= **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})	= **3.25 in.**	S_1	= **6 in.**	Use $S_{1,adj}$	= **6 in.**
Number of Anchors (n)	= **4**	S_2	= **6 in.**	Use $S_{2,adj}$	= **6 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 = **2671 lbs**	0.75	**2003 lbs**
Shear Capacity = **4468 lbs**	0.75	**3351 lbs**

ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Type A

Steel Strength	17.4.1
ϕ = **0.75**	17.3.3.a i
$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 4 \cdot 0.099 \cdot 114000$ = **33,858 lbs**	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
ϕ = **0.65**	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef})$ = **248.063 sq.in.**	
$A_{Nco} = 9h_{ef}^2$ = **95.063 sq.in.**	
Check if $A_{Nco} \geq A_{Nc}$ A_{Nc}/A_{Nco} = **2.609**	
$\Psi_{ec,N}$ = **1**	17.4.2.4
$\Psi_{ed,N}$ = **1**	17.4.2.5
$\Psi_{C,N}$ = **1**	17.4.2.6
K_c = **17**	
λ_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
$0.65 \cdot (248.063/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299$ = **10,684 lbs**	
Pullout Strength ϕN_{pn}	17.4.3
ϕ=**0.65**	17.3.3 c ii Category 1-B
Ψ_{cp} = **1**	17.4.3.6
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5}$ = **32,884 lbs**	17.4.3.1

Steel Strength (ϕN_{sa}) = **33,858 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **10,684 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **32,884 lbs**

ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Type A

Steel Strength ϕV_{sa} V_{sa}=6,875 / Anchor -- per report	17.5.1
ϕ = **0.65**	17.3.3. Condition a ii
$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 4 \cdot 6{,}875 =$ **17,875 lbs**	17.5.1.2a
Concrete Breakout Strength ϕV_{cbg}	17.5.2
ϕ = **0.7**	17.3.3 ci-B
$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **294 sq.in.**	
$A_{Vco} = 3C_{a1}h_a =$ **252 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
$\Psi_{h,V}$ = **1.604**	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
$0.7 \cdot (294/252) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.604 \cdot 14{,}948 =$ **35,237 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} = **16,437 lbs**	
$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 16{,}437 =$ **23,012 lbs**	

Steel Strength (ϕV_{sa}) = **17,875 lbs**

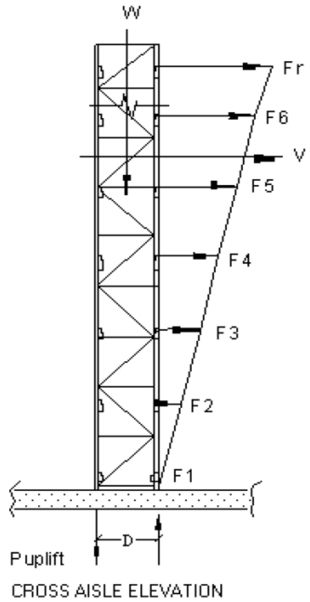
Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **35,237 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **23,012 lbs**

OVERTURNING ANALYSIS Configuration1 Type A

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} = **20,000 lbs** W_{dl} = **400 lbs**

$W_{pl} \cdot 67\% = 20{,}000 \cdot 0.67 =$ **13,400 lbs**

$V_{Trans} = (1 \cdot 0.2525 \cdot 1 \cdot ((0.67 \cdot 13{,}400) + 400)) =$ **2,367 lbs**

$M_{ovt} = V_{Trans} \cdot Ht = 2{,}367 \cdot 200 =$ **473,400 in-lbs**

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

$= ((20{,}000 \cdot 0.67) + 400) \cdot 42 \cdot 0.5 =$ **289,800 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (473{,}400 - 289{,}800) / 42 =$ **4,371 lbs**

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (473{,}400 + 289{,}800) / 42 =$ **18,171 lbs**

TOP SHELF LOADED

Shear = **1,363 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 1{,}363 \cdot (232 + ((58 - 10) / 2)) =$ **348,928 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (5{,}000 + 400) \cdot (42 \cdot 0.5) =$ **113,400 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (348{,}928 - 113{,}400) / 42 =$ **5,607 lbs**

ANCHORS

No. of Anchors (#Anchors): **4**

Pull Out Capacity per Anchor (T_{Anchor}): **2,003 lbs**

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

COMBINED STRESS

Fully Loaded	= ((4,371 / 4) / 2,003) + ((2,367 / 8) / 3,351)	= **0.634**
Top Shelf Loaded	= ((5,607 / 4) / 2,003) + ((1,363 / 8) / 3,351)	= **0.751**
Seismic UpLift Critical (LC#7B)	= (7,314 / 4) / 2,003	= **0.913**

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PN: 20240415_19

Base Plate Analysis: Type A

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} = **24528 lbs**

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

$P/A = P_{col}/(D \cdot B) = 24528 / (8 \cdot 8) =$ **383 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 0 / (8 \cdot 8^2 / 6) =$ **0 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 0 \cdot (2 \cdot 2.5/8) =$ **0 psi**

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

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

$= (2.5^2 / 2) \cdot (383 + 0 + 0.67 \cdot 0) =$ **1197.66 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.19sq.in.**

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 1197.66 / (0.19 \cdot 32,400) =$ **0.2**

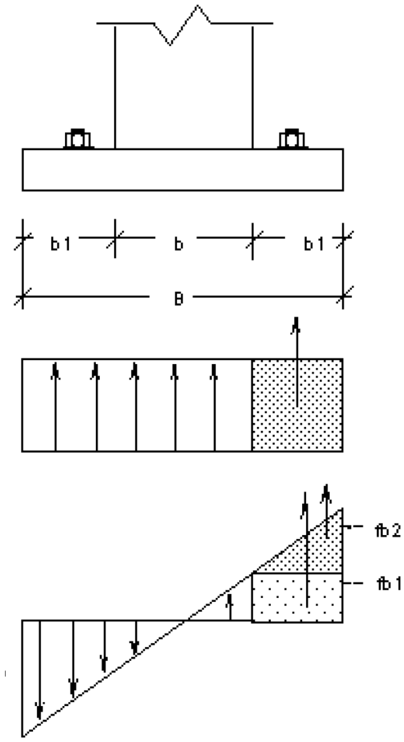

Plate width B =	8 in.
Plate depth D =	8 in.
Plate thickness t =	0.38 in.
Column width b =	3 in.
Column depth d =	3 in.
b1 =	2.5 in.

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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

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

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

# of levels	4
min. # of bays	3
N_c	48
N_b	8
k_c	400 kip-in/rad
k_{be}	2930 kip-in/rad
k_b	148 kip-in/rad
k_{ce}	595 kip-in/rad
I_b	1.59 in^4
L	96 in
I_c	1.17 in^4
H	232 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	84 in	5 kip
2	142 in	5 kip
3	200 in	5 kip
4	259 in	5 kip

α_s = 1.54

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= 0.62

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}=24,036 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) = **24,528 lbs**

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

$d,req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **3.473 in.**

$b = (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **22.525 in.**

$b,req'd = 1.5 \cdot b =$ **34 in.**

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **166,098 lbs**

$P_a = \phi \cdot P_a =$ **99,659 lbs**

$P_{max} / P_a =$ **0.25**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **17,272 lbs**

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

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **166,098 lbs**

$d,req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **3.473 in.**

$b = (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **22.525 in.**

$b,req'd = 1.5 \cdot b =$ **34 in.**

$P_a = P_n / \Omega =$ **55,366 lbs**

$P_{max} / P_a =$ **0.31**

Base Plate	
Width B	8 in.
Depth W	8 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	4 in
E_c	3,604,997 psi