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FOOTINGS			TITLE 24	CATWALKS	MINI-LOAD SYSTEMS

LICENSED IN 50 STATES

ANALYSIS OF

STORAGE RACKS

FOR

Carrier Enterprise Inc.

648 NE Maguire Blvd, Lee's Summit, MO

Job No. 26-0698

Approved by:

SAL E. FATEEN, P.E.

3/9/2026

EXPIRES
12-31-2026

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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.
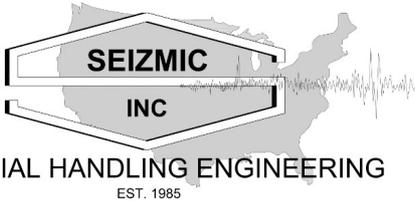
PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 2
CALCULATED BY: ateruel
DATE: 3/9/2026
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The storage racks consist of several bays, interconnected in one or both directions,with the columns 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

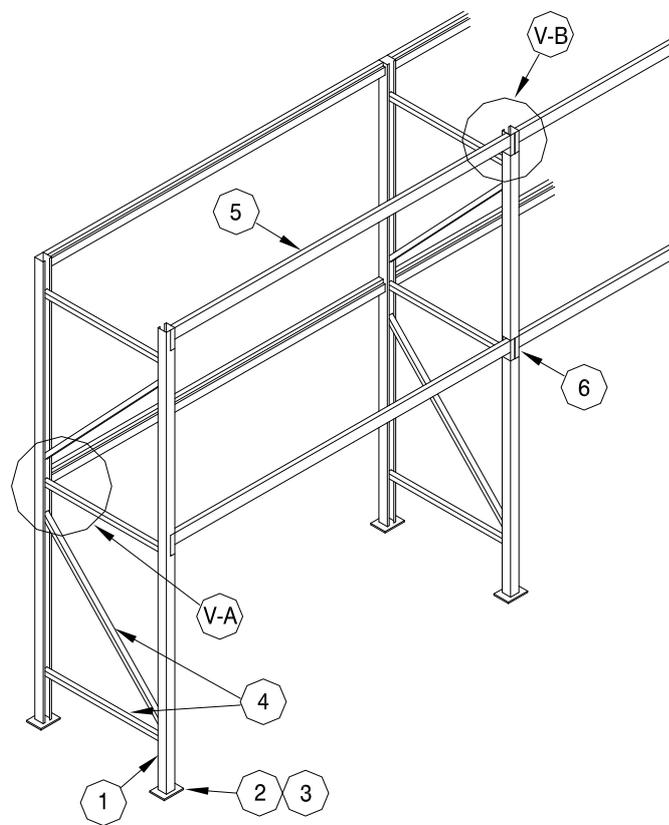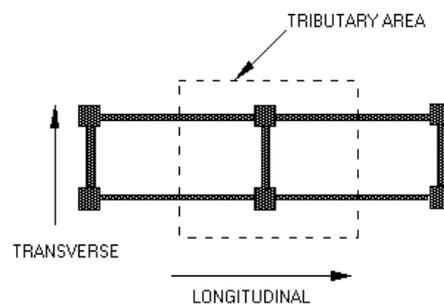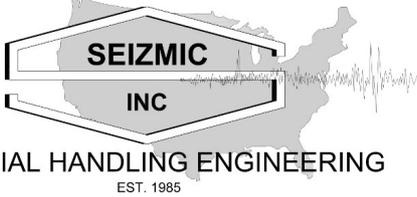**SEIZMIC INC**

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FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 3
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

COMPONENTS AND SPECIFICATIONS   Configuration 1: 42" deep single deep bays   M   1.7

Analysis per section 2209 of the 2018 IBC

Levels: 3   Panels: 3

$S_s = 0.1$ $S_{DS} = 0.11$ $I = 1$ $V_{Long} = 138$ lbs. $P_{static} = 5550$ lbs.
$S_1 = 0.07$ $S_{D1} = 0.11$ SDC = B $V_{Trans} = 207$ lbs. $P_{seismic} = 564$ lbs.

Load per Level

3600 lbs
3600 lbs
3600 lbs

60"   60"   60"   60"   192"

12"   78"   64"   32"   6"

96"
FRONT VIEW

42"
SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 3.0 x 2.72 - .075 (314) Steel = 55000 psi Stress = 66% (level 2)	3.66 x 2.75 -0.059 (36E) Steel = 55 ksi Max Static Cap. = 3916 lb. Stress = 94%	3 Tab 2" cc Connector (IM) Stress = 35%
HORIZONTAL BRACE 1.7953 x 1.378 - .0625 (C456) Stress = 6% (panel 1)	Max stress = 94% (level 1)	Max stress = 35% (level 1)
DIAGONAL BRACE 1.7953 x 1.378 - .0625 (C456) Stress = 23% (panel 2)		

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 = 17%	Slab = 6" 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:

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PN: 20260302_054

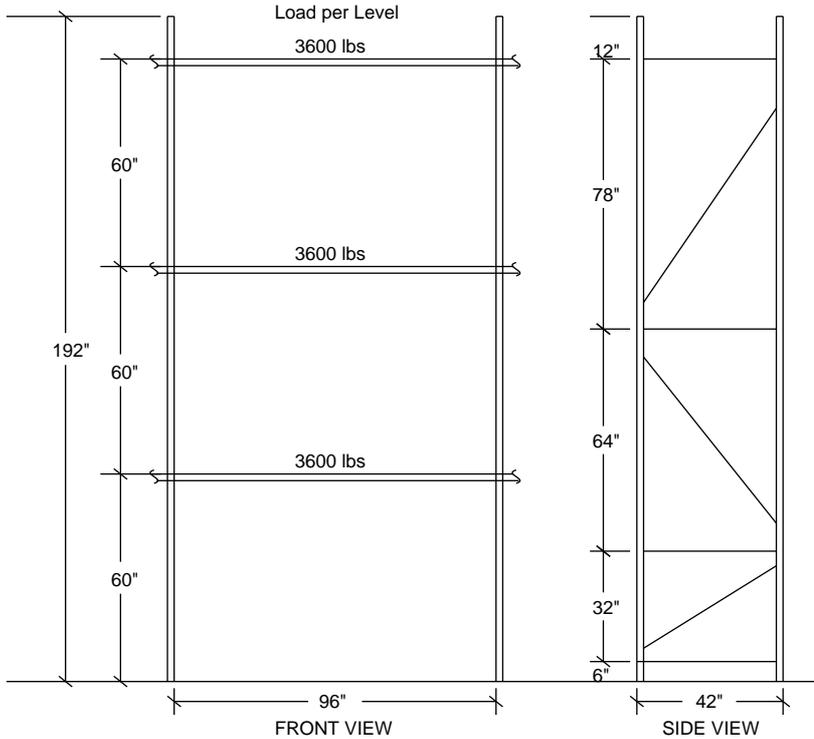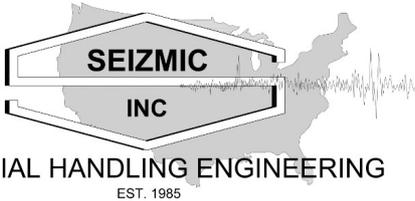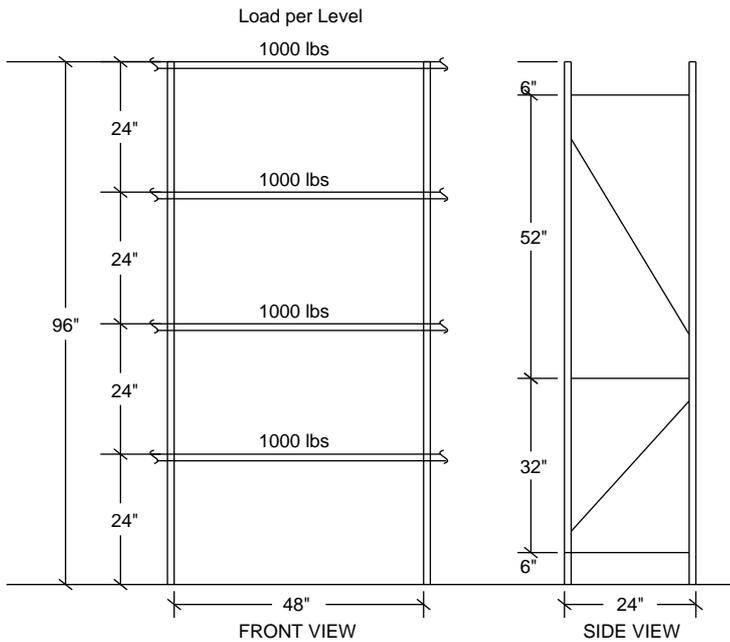| COMPONENTS AND SPECIFICATIONS	Configuration 2: 24" deep single deep bays	M 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 4   Panels: 2

$S_s = 0.1$ $S_{DS} = 0.11$ $I = 1$ $V_{Long} = 56$ lbs. $P_{static} = 2200$ lbs.
$S_1 = 0.07$ $S_{D1} = 0.11$ $SDC = B$ $V_{Trans} = 84$ lbs. $P_{seismic} = 202$ lbs.

Load per Level

FRONT VIEW — Levels loaded 1000 lbs each; heights 24" between levels, total 96"; width 48"

SIDE VIEW — 6", 52", 32", 6" dimensions; depth 24"

FRAME	BEAM	CONNECTOR
COLUMN 3.0 x 2.72 - .075 (314) Steel = 55000 psi Stress = 14% (level 1) **HORIZONTAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 1% (panel 1) **DIAGONAL BRACE** 1.7953 x 1.378 - .0625 (C456) Stress = 6% (panel 2)	2.75 x 2.75 -0.059 (27E) Steel = 55 ksi Max Static Cap. = 4816 lb. Stress = 22% Max stress = 22% (level 1)	3 Tab 2" cc Connector (IM) Stress = 5% Max stress = 5% (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 = 7%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 5% (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 = 1%

Notes:
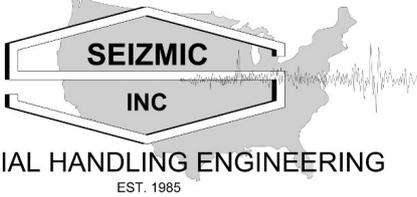
PROJECT: Carrier Enterprise Inc
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SHEET#: 5
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Loads and Distributions: 42" deep single deep bays

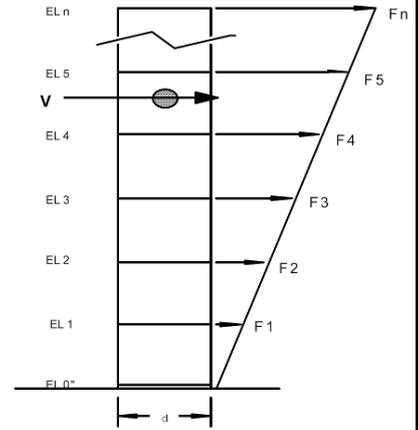### Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels:	3	SDC:	B	R_L:	6	Ss:	0.1
Pallets Wide:	2	W_{PL}:	10800	R_T:	4	S1:	0.07
Pallets Deep:	1	W_{DL}:	300 lbs	Fa:	1.6	Ip:	1
Pallet Load:	1800	Fv:	2.4	Tl:	1.5		

Total Frame Load: 11100 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} = $ **7536 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 7536 = $ **92.11 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 7536 = $ **138.16 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 7536 = $ **43.96 lbs**

V_{long} shall not be less than:

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

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

$= Max[0.03, 0.03] \cdot 1 \cdot 7536 = $ **226.08 lbs**

Since: 92.11 < 226.08
& 226.08 > 43.96

$V_{long} = $ **138 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 7536 = $ **207.24 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 7536 = $ **65.94 lbs**

V_{trans} shall not be less than:

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

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

$= Max[0, 0.01, 0.03] \cdot 1 \cdot 7536 = $ **226.08 lbs**

Since: 226.08 > 207.24
& 226.08 > 65.94

$V_{trans} = $ **207 lbs**
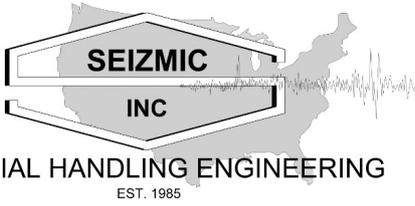
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		Lee's Summit, MO
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Loads and Distributions: 42" deep single deep bays (Page 2)

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

Level	h_x	Longitudinal			Transverse		
		w_x	$w_x h_x$	f_i	w_x	$w_x h_x$	f_i
1	60	1850	111000	23	1850	111000	34.5
2	120	1850	222000	46	1850	222000	69
3	180	1850	333000	69	1850	333000	103.5
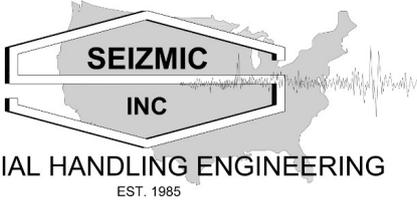
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	FOR:	Wiese USA_Curt Trifile
	ADDRESS:	648 NE Maguire Blv
		Lee's Summit, MO
	SHEET#:	7
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Fundamental Period of Vibration (Longitudinal)

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

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

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

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

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

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

# of levels	3
min. # of bays	3
N_c	36
N_b	8
k_c	150 kip-in/rad
k_{be}	2297 kip-in/rad
k_b	135 kip-in/rad
k_{ce}	543 kip-in/rad
I_b	1.25 in^4
L	96 in
I_c	0.83 in^4
H	180 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	87 in	3 kip
2	147 in	3 kip
3	208 in	3 kip

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 = 3.73

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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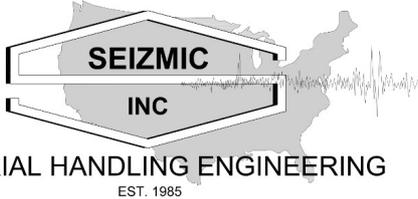**ADDRESS:** 648 NE Maguire Blv
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CALCULATED BY: ateruel
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LRFD Basic Load Combinations: 42" deep single deep bays
2018 IBC& RMI / ANSI MH 16.1

V_{Trans} = **207 lbs**

V_{Long} = **138 lbs**

P = Product Load / 2 = **5,400 lbs**

D = Dead Load · 0.5 = **150 lbs**

M_{Trans} = $\Sigma(f_{Trans} \cdot h_x)$ = **28,980 in-lbs**

E_{Trans} = M_{Trans} / frame depth = **690 lbs**

β = 0.7

β = 1.0 (Uplift combination only)

ρ = 1

S_{DS} = .11

L = Live Load = **0 lbs** S = Snow Load = **0 lbs** R = Rain Load = **0 lbs**
L_r = 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 · 5,400)= **6,690 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 · 5,400) + (1.6 · 0) + (0.5 · 0)= **7,739 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 · 5,400) + (0.5 · 0) + (1.6 · 0)= **4,770 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 · 5,400) + (0.5 · 0) + (1.0 · 0) + (0.5 · 0)= **4,770 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 · 5,400 + 0.5 · 0 + 1 · 690 + 0.2 · 0= **5,492 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 · 5,400 + 0.5 · 0 + 1 · 0 + 0.2 · 0= **4,802 lbs**

6. Wind Uplift
= $0.9D + 0.9P_{app} + 1.0W$
= 0.9 · 150 + 0.9 · 5,400 + 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 · 5,400 - 1 · 690= **4,182 lbs**
For a single beam, D = **32 lbs** P = **1,800 lbs** I = **225 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 · 1,800) + (1.4 · 225) = **2,873 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 · 5,400 + 0.7 · 1 · 690)= **4,526 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 · 5,400 + 0.7 · 1 · 690= **3,906 lbs**

3. $D + P$
= 150 + 5,400 = **5,550 lbs**
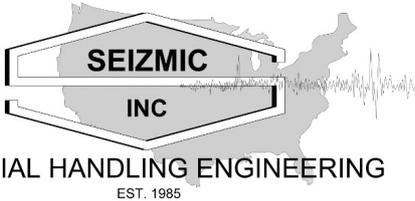
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CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

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Longitudinal Analysis: 42" deep single deep bays

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).

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

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

$$V_{Col} = V_{Long} / \text{\# of columns} = \textbf{69 lbs}$$

$$M_{Base} = \textbf{0 in-lbs}$$

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

$$\textbf{(69 lbs} \cdot \textbf{58 in.) - 0 in-lbs = 4002 in-lbs}$$
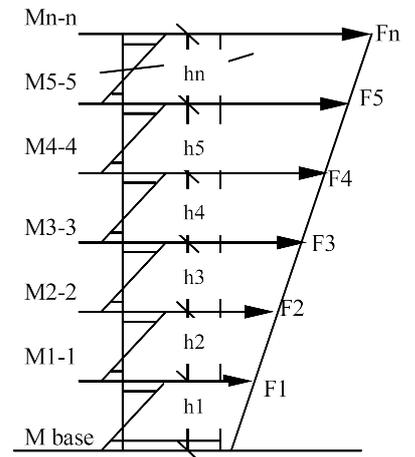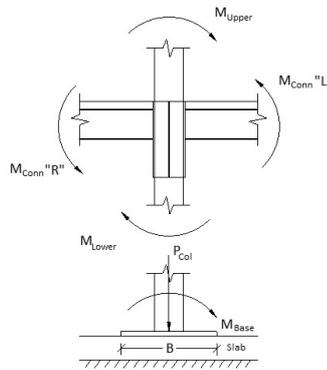
FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	60	12	5,550	4,002	2,400	6,402
2	60	23	3,700	4,002	2,400	6,402
3	60	35	1,850	4,002	2,400	4,401
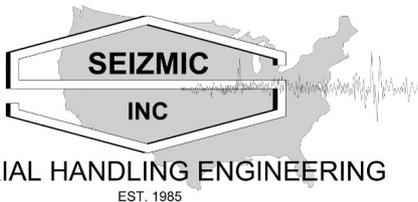
PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
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Lee's Summit, MO
SHEET#: 10
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

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COLUMN ANALYSIS: 42" deep single deep bays (Level 2)
Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

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

$K_x \cdot L_x / R_x$ = 1.5·58 / 1.103 = **78.9**

$K_y \cdot L_y / R_y$ = 1·64 / 0.945 = **67.75**

KL/R_{max} = **78.9**

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

$= (1.103^2 + 0.945^2 + -2.348^2)^{1/2} = $ **2.76 in.**

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

$= 1 - (-2.348/2.76)^2 = $ **0.277**

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

$= 3.14^2 \cdot 29500 / 78.9^2 = $ **46.773 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)

$= (1 / (2 \cdot 0.277)((46.773 + 37.022) - (46.773 + 37.022)^2$

$- (4 \cdot 0.277 \cdot 46.773 \cdot 37.022))^{1/2}) = $ **22.308 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 78.9^2 = $ **46.773 ksi**

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

$= 1 / 0.682 \cdot 2.76^2 (11300 \cdot 0.001$

$+ (3.142 \cdot 29500 \cdot 1.314) / (0.8 \cdot 58)^2) = $ **37.022 ksi**

$F_e = Min(F_{e1}, F_{e2}) = $ **22.308 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 22.308)^{1/2} = $ **1.57** (Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

$F_n = (0.877 / \lambda_c^2) \cdot F_y = $ **19.564** (Eq C4.1-3)

Thus:

P_n = **10800 lbs**

P_a = **9180 lbs**

3.0 x 2.72 - .075	
SECTION PROPERTIES	
Depth	2.717 in.
Width	3 in.
t	0.075 in.
Radius	0.14 in.
Area	0.682 in.^{2}
AreaNet	0.575 in.^{2}
I_x	0.829 in.^{4}
S_x	0.553 in.^{3}
$S_{x\,Net}$	0.504 in.^{3}
R_x	1.103 in.
I_y	0.609 in.^{4}
S_y	0.364 in.^{3}
R_y	0.945 in.
J	0.001 in.^{4}
C_w	1.314 in.^{6}
J_x	2.547 in.
X_o	-2.348 in.
K_x	1.5
L_x	58 in.
K_y	1
L_y	64 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: 42" deep single deep bays (Level 2)
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 = $ **25807 lbs**

Where:

$P_{no} = A_e F_y = 0.552 \cdot 55 = $ **30361 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 = $ **141.708 ksi**

$F_e = C_s A \sigma_{ex} (j + C_s (j^2 + r_o^2(\sigma_c / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = $ **57.184 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 = $ **212.622 ksi** (Eq 3.1.2.1-10)

$F_{e.min} = $ **57.184 ksi**

Since: $0.56\ F_y < 2.78 F_y$

$F_c = (10/9)F_y (1 - (10F_y / 36F_e)) = $ **44.8 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = $ **42.9 ksi**

$M_{nx} = $ **21608 in-lbs** $\qquad M_{ny} = $ **15602 in-lbs** $\qquad M_c = M_{n,min}$

$M_{nx}\phi_b = $ **19447 in-lbs** $\qquad M_{ny}\phi_b = $ **14042 in-lbs**

$P_{Ex} = \Pi^2 E I_x / (K_x L_x)^2 = $ **31904 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 E I_y / (K_y L_y)^2 = $ **43267 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) = $ **0.913** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) = $ **0.936** (Eq C5.2.2-5)

$P_{trans} = $ **5,492 lbs** $\qquad P_{long} = $ **4,802 lbs**

$M_u = M_x = $ **1382 in-lbs** (Eq C5.2.2-2)

$P_{u_st} = (1.2 \cdot D) + (1.4 \cdot P) = $ **5160 lbs**

$P_{u\_st} / P_a = 5160 / 9180 = 0.67 \quad$ Static Stress = **66%**

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)

$= ((4,802 / 9180) + (1382 / 19447) + (1 / 14042)) = $ **59%**

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

$= (4,802 / 25807) + (0.85 \cdot 1382 / 19447 \cdot 0.913)) + (0.85 \cdot 1 / 14042 \cdot 0.936))) = $ **25%**

$Stress3 \quad P_t / P_{ao} = 5,492 / 25807 = $ **21%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **66%**

3.0 x 2.72 - .075	
SECTION PROPERTIES	
Depth	2.717 in.
Width	3 in.
t	0.075 in.
Radius	0.14 in.
Area	0.682 in. ^{2}
AreaNet	0.575 in. ^{2}
I_x	0.829 in. ^{4}
S_x	0.553 in. ^{3}
$S_{x\ Net}$	0.504 in. ^{3}
R_x	1.103 in.
I_y	0.609 in. ^{4}
S_y	0.364 in. ^{3}
R_y	0.945 in.
J	0.001 in. ^{4}
C_w	1.314 in. ^{6}
J_x	2.547 in.
X_o	-2.348 in.
K_x	1.5
L_x	58 in.
K_y	1
L_y	64 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

PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 12
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

SEIZMIC INC

MATERIAL HANDLING ENGINEERING
EST. 1985

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

BEAM ANALYSIS   42" deep single deep bays

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**

λ = (1.052 / k$^{1/2}$) · (w/t) · $(F_y / E)^{1/2}$ = (1.052 / 2) · 24.61 · (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 * 1.82 / 1.96 = **50.83 ksi**

f_2(tension) = $F_y \cdot (y_1 / y_2)$ = 55 * 1.55 / 1.96 = **43.32 ksi**

Ψ = - (f_2 / f_1) = -(43.32 / 50.83) = **-0.85**

Buckling coefficient k = 4 + 2 · (1 - Ψ)^3 + 2 · (1 - Ψ)
= 4 + 2(1 - **-0.85**)^3 + 2(1 - **-0.85**) = **20.41**

Flat Depth w = y1 + y3 = 1.55 + 1.82 = **3.362**

w/t   =   3.362/0.059 = **56.98**   **w/t < 200: OK**

λ = (1.052 / k$^{1/2}$) · (w/t) · $(f_1 / E)^{1/2}$ = (1.052 / 2) · 56.983 · (50.83 / 29500)$^{1/2}$ = **0.55**

b1 = w · (3 - Ψ) = 3 · (3 - -0.85) = **12.95**

b2 = w/2 = **1.68**

b1 + b2 = 12.95 + 1.68 = **14.63**   **Web is fully effective**

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

Corner cross-sectional area Lc = (Π / 2) · (r + t / 2)
 = (Π / 2) · (0.09 + 0.059 / 2) = **0.188**

L_f = effective width = **1.452**

C = 2 · L_c / L_f + 2 · L_c = 2 · 0.188 / 1.452 + 2 · L_c = **0.2054**

m = 0.192 · (F_u / F_y) - 0.068 = 0.192 · (65 / 55) - 0.068 = **0.1589**

B_c = 3.69 · (F_u / F_y) - 0.819 · $(F_u / F_y)^2$ - 1.79
 = 3.69 · (65 / 55) - 0.819 · (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 · 55 / (1.525)^m = **73**

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

$F_{ya\text{-}bottom}$ = $F_{ya\text{-}top} \cdot Y_{cg} / (A - Y_{cg})$ = 59 · 1.7 / (3.66 - 1.7) = **51**

3.66 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	3.66 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.55
Y2 =	1.96
Y3 =	1.82
Ycg =	1.7
Ix =	1.25
Sx =	0.63
E =	29500
FBeam F =	150
Beam Length L =	96

PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 13
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

SEIZMIC INC

MATERIAL HANDLING ENGINEERING
EST. 1985

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

BEAM ANALYSIS   42" deep single deep bays

Check Allowable Tension Stress for Bottom Flange

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

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

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

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

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:

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

$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L))$
$= 1 - ((2 \cdot 150 \cdot 96) / (6 \cdot 29500 \cdot 1.25 + 3 \cdot 150 \cdot 96)) = \mathbf{0.89}$

$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x = \mathbf{35.43 \ in\text{-}kip}$

$W = \phi \cdot M_n \cdot 8 \cdot (\# \ of \ Beams) / (L \cdot R_m \cdot \Omega) = (35.43 \cdot 8 \cdot 2) / (96 \cdot 0.89 \cdot 1.6)$
$= \mathbf{4146} \ lbs/pair$

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x)$
$= 1 - (4 \cdot 150 \cdot 96) / (5 \cdot 150 \cdot 96 + 10 \cdot 29500 \cdot 1.25) = \mathbf{0.87}$

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

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

$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d)$
$= (384 \cdot 29500 \cdot 1.25 \cdot 2) / (180 \cdot 5 \cdot 96^2 \cdot 0.87) \cdot 1000 = \mathbf{3916} \ lbs/pair$

3.66 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	3.66 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.55
Y2 =	1.96
Y3 =	1.82
Ycg =	1.7
Ix =	1.25
Sx =	0.63
E =	29500
FBeam F =	150
Beam Length L =	96

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PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 14
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

Allowable and Actual Bending Moment at Each Level

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

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	22,176	23,496	2,547	23,496	Pass
2	22,176	23,496	1,546	23,496	Pass
3	22,176	23,496	1,201	23,496	Pass

PROJECT: Carrier Enterprise Inc
FOR: Wiese USA_Curt Trifile
ADDRESS: 648 NE Maguire Blv
Lee's Summit, MO
SHEET#: 15
CALCULATED BY: ateruel
DATE: 3/9/2026
PN: 20260302_054

SEIZMIC INC

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

Beam to Column Analysis: 42" deep single deep bays

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 = $0.6 \cdot F_y \cdot A_w \cdot C_v$ = **2673 lbs**   AISC G2-1

P_{Shear} = $\phi \cdot V_n = 0.9 \cdot 2673$ = **2405 lbs**
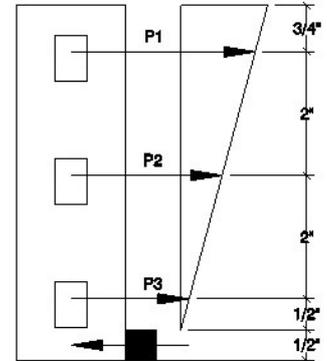
2. Bearing Strength of Tab

Thickness of the column t_c = **0.08 in.**

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

R_n = $1.8 \cdot F_y \cdot A_{pb}$ = **4455 lbs**   AISC J7 -1

$P_{Bearing}$ = $\phi \cdot R_n = 0.75 \cdot 4455$ = **3341 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 = **1.67**

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

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

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

Minimum Value of P1 Governs

P_1 = $Min(P_{Shear}, P_{Bearing}, P_{Clip})$ = **2405 lbs**

$M_{Conn-Allow}$ = $(P_1 \cdot 4.5) + (P_1 \cdot (2.5 / 4.5) \cdot 2.5) + (P_1 \cdot (0.5 / 4.5) \cdot 0.5)$ = **14296.39 in-lbs**
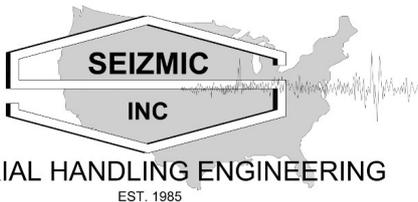
SEIZMIC	**PROJECT:** Carrier Enterprise Inc
INC	**FOR:** Wiese USA_Curt Trifile
MATERIAL HANDLING ENGINEERING	**ADDRESS:** 648 NE Maguire Blv
EST. 1985	Lee's Summit, MO
	SHEET#: 16
	CALCULATED BY: ateruel
	DATE: 3/9/2026
TEL:(909)869-0989	
1130 E. CYPRESS ST, COVINA, CA 91724	**PN:** 20260302_054

BRACE ANALYSIS 42" deep single deep bays (Panel 2)
Analyzed per RMI, AISI 2012 (LRFD) and the 2018 IBC.

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

$K_x \cdot L_x / R_x$ = 1·69 / 0.736 = **93.74**

$K_y \cdot L_y / R_y$ = 1·69 / 0.448 = **153.95**

KL/R_{max} = **153.95**

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.**	
β = $1 - (Xo/ro)^2$	(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)
= $3.14^2 \cdot 29500 / 153.95^2$ = **12.285 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)
= $(1 / (2 \cdot 0.428)((33.136 + 11.1) - (33.136 + 11.1)^2$	
$- (4 \cdot 0.428 \cdot 33.136 \cdot 11.1))^{1/2})$ = **9.121 ksi**	

where:

σ_{ex} = $\Pi^2 E / (K_x L_x / R_x)^2$	(Eq C3.1.2-11)
= $3.14^2 \cdot 29500 / 93.74^2$ = **33.136 ksi**	
σ_t = $1 / Ar_o{}^2(GJ + (\Pi^2 EC_w) / (K_t L_t)^2)$	(Eq C3.1.2-9)
= $1 / 0.257 \cdot 1.317^2(11300 \cdot 0$	
$+ (3.142 \cdot 29500 \cdot 0.025) / (0.8 \cdot 69)^2)$ = **11.1 ksi**	

F_e = Min(F_{e1}, F_{e2}) = **9.121 ksi**	
P_n = $A_{eff} \cdot F_n$	(Eq C4.1-1)
λ_c = $(F_y / F_e)^{1/2}$ = $(50 / 9.121)^{1/2}$ = **2.341**	(Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

F_n = $(0.877 / \lambda_c{}^2) \cdot F_y$ = **7.999**	(Eq C4.1-3)

Thus:

P_n = **2052 lbs**

P_a = **1744 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	1
L_x	69 in.
K_y	1
L_y	69 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

BRACE ANALYSIS 42" deep single deep bays (Panel 2)
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 = $ **41.506 ksi**

$F_e = C_s A \sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_c / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = $ **10.997 ksi**   (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2) = $ **36.221 ksi**   (Eq 3.1.2.1-10)

$F_{e.min} = $ **10.997 ksi**

Since: $F_e \leq 0.56 F_y$

$F_c = F_e = $ **10.997 ksi**   (Eq C3.1.2.1-3)

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

$M_{nx} = $ **1717 in-lbs** $M_{ny} = $ **613 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b = $ **1545 in-lbs** $M_{ny}\phi_b = $ **552 in-lbs**

$P_{Ex} = \Pi^2 E I_x / (K_x L_x)^2 = $ **8500 lbs**   (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 E I_y / (K_y L_y)^2 = $ **3149 lbs**   (Eq C5.2.2-7)

$P_a = $ **1744 lbs**

$V_{Trans} = $ **161 lbs**

$V_{Trans(new)} = 161 \cdot 1.3 = $ **210 lbs**

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

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = $ **397 lbs**

Brace Stress = $V_{Diag} / P_a = $ **23%**

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	1
L_x	69 in.
K_y	1
L_y	69 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

POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 42" deep single deep bays

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	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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SHEET#: 19
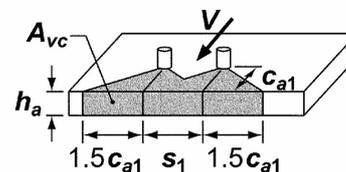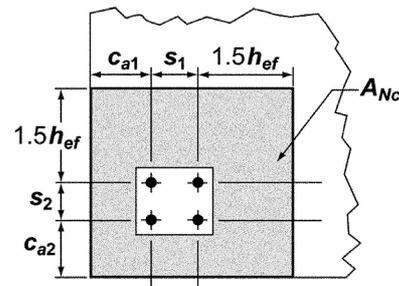**CALCULATED BY:** ateruel
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ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 42" deep single deep bays

Steel Strength   17.4.1

$\phi = 0.75$ 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

$\phi = 0.65$ 17.3.3 c ii Category 1-B

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

$A_{Nco} = 9 h_{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

$\phi = 0.65$ 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 $(\phi N_{sa}) = $ **8,465 lbs**

Embedment Strength - Concrete Breakout Strength $(\phi N_{cbg}) = $ **4,094 lbs**

Embedment Strength - Pullout Strength $(\phi N_{pn}) = $ **8,221 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 42" deep single deep bays

Steel Strength ϕV_{sa} V_{sa}=6,875 / Anchor -- per report   17.5.1

$\phi = 0.65$ 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

$\phi = 0.7$ 17.3.3 ci-B

$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **216 sq.in.**

$A_{Vco} = 3C_{a1}h_a =$ **216 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

$\Psi_{h,V} = 1.732$ 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

$0.7 \cdot (216/216) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.732 \cdot 14,948 =$ **16,311 lbs**

Pryout Strength ϕV_{cpg} 17.5.3

$\phi = 0.7$ 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 $(\phi V_{sa}) =$ **4,469 lbs**

Embedment Strength - Concrete Breakout Strength $(\phi V_{cbg}) =$ **16,311 lbs**

Embedment Strength - Pryout Strength $(\phi V_{cpg}) =$ **8,817 lbs**

OVERTURNING ANALYSIS Configuration1 42" deep single deep bays

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} = **10,800 lbs** W_{dl} = **300 lbs**

$W_{pl} \cdot 67\% = 10,800 \cdot 0.67 =$ **7,236 lbs**

$V_{Trans} = (1 \cdot 0.0275 \cdot 1 \cdot ((0.67 \cdot 7,236) + 300)) =$ **141 lbs**
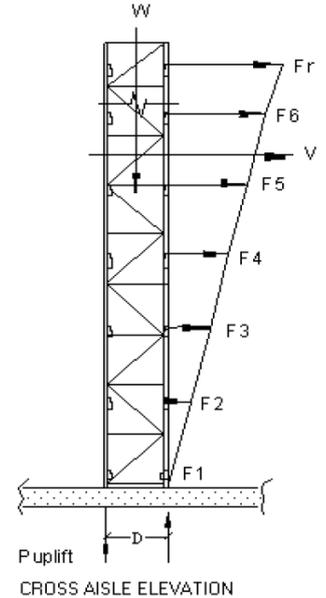
$M_{ovt} = V_{Trans} \cdot Ht = 141 \cdot 168 =$ **23,688 in-lbs**

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

$= ((10,800 \cdot 0.67) + 300) \cdot 42 \cdot 0.5 =$ **158,256 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (23,688 - 158,256) / 42 =$ **-3,204 lbs**

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

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (23,688 + 158,256) / 42 =$ **4,332 lbs**

TOP SHELF LOADED

Shear = **107 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 107 \cdot (180 + ((60 - 10) / 2)) =$ **21,935 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (3,600 + 300) \cdot (42 \cdot 0.5) =$ **81,900 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (21,935 - 81,900) / 42 =$ **-1,427 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	$= ((0 / 1) / 4,094) + ((141 / 2) / 4,468)$	= **0.016**
Top Shelf Loaded	$= ((0 / 1) / 4,094) + ((107 / 2) / 4,468)$	= **0.012**
Seismic UpLift Critical (LC#7B)	$= (0 / 1) / 4,094$	= **0**

		PROJECT:	Carrier Enterprise Inc
SEIZMIC		**FOR:**	Wiese USA_Curt Trifile
INC		**ADDRESS:**	648 NE Maguire Blv
			Lee's Summit, MO
MATERIAL HANDLING ENGINEERING		**SHEET#:**	22
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Base Plate Analysis: 42" deep single deep bays

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} = **7739 lbs**

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

$P/A = P_{col}/(D \cdot B) = 7739 / (4.69 \cdot 5.09) =$ **324 psi**

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

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 0 \cdot (2 \cdot 1.05/5.09) =$ **0 psi**

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

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

$\quad = (1.05^2 / 2) \cdot (324 + 0 + 0.67 \cdot 0) =$ **177.62 in-lbs**

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

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

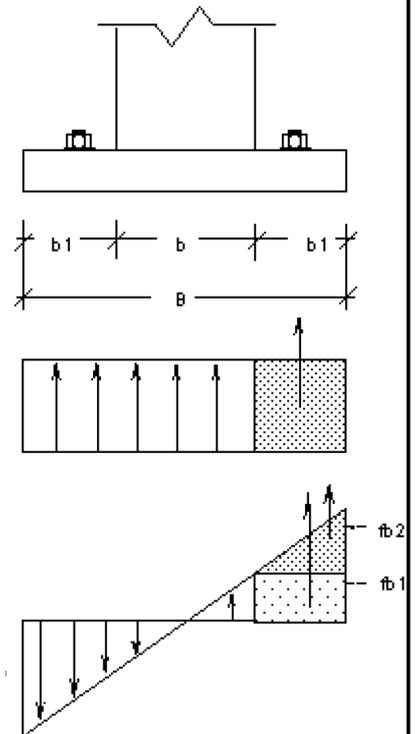$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 177.62 / (0.03 \cdot 32,400) =$ **0.17**

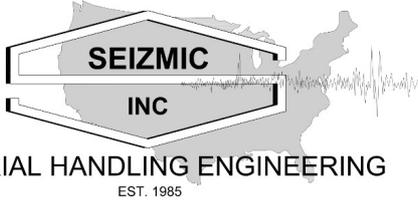|	
Plate width B =	5.09 in.
Plate depth D =	4.69 in.
Plate thickness t =	0.19 in.
Column width b =	3 in.
Column depth d =	2.72 in.
b1 =	1.05 in.

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PROJECT: Carrier Enterprise Inc
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SHEET#: 23
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Equation for Maximum Considered Earthquake Base Rotation

Per RMI 2012 Commentary 2.6.4

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

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

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

# of levels	3
min. # of bays	3
N_c	36
N_b	8
k_c	150 kip-in/rad
k_{be}	2297 kip-in/rad
k_b	135 kip-in/rad
k_{ce}	543 kip-in/rad
I_b	1.25 in⁴
L	96 in
I_c	0.83 in⁴
H	180 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	87 in	3 kip
2	147 in	3 kip
3	208 in	3 kip

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

$\alpha_s = 0.83$

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 = 0.49

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 $\qquad$ Θ_{max}=2.862 rad $\quad$ 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 $\quad \geq M_b$ OK

PROJECT: Carrier Enterprise Inc
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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) = **7,740 lbs**

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

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **2.004 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **14.911 in.**

$b,req'd$ = $1.5 \cdot b$ = **22 in.**

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **115,690 lbs**

P_a = $\phi \cdot P_a$ = **69,414 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)
= **5,550 lbs**

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

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **115,690 lbs**

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **2.004 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **14.911 in.**

$b,req'd$ = $1.5 \cdot b$ = **22 in.**

P_a = P_n / Ω = **38,563 lbs**

P_{max} / P_a = **0.14**

Base Plate

Width B	5.094 in.
Depth W	4.688 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	2.44 in
E_c	3,604,997 psi