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**ANALYSIS OF
STORAGE RACKS
FOR**

H & R Block

1220 NW Main St., Lee's Summit, MO

Job No. 24-1680

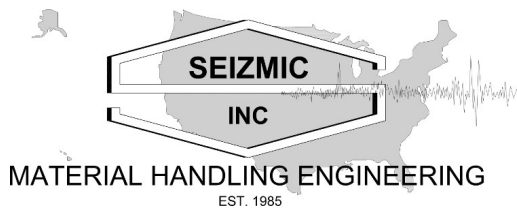
Approved by:

SAL E. FATEEN, P.E.

6/25/2024



**EXPIRES
12-31-2024**



PROJECT: H & R Block
FOR: Crown Equipment Corp_T
ADDRESS: 1220 NW Main St.
Lee's Summit, MO
SHEET#: 1
CALCULATED BY: aswenson
DATE: 6/25/2024
PN: 20240415_29

TEL: (909) 869-0989
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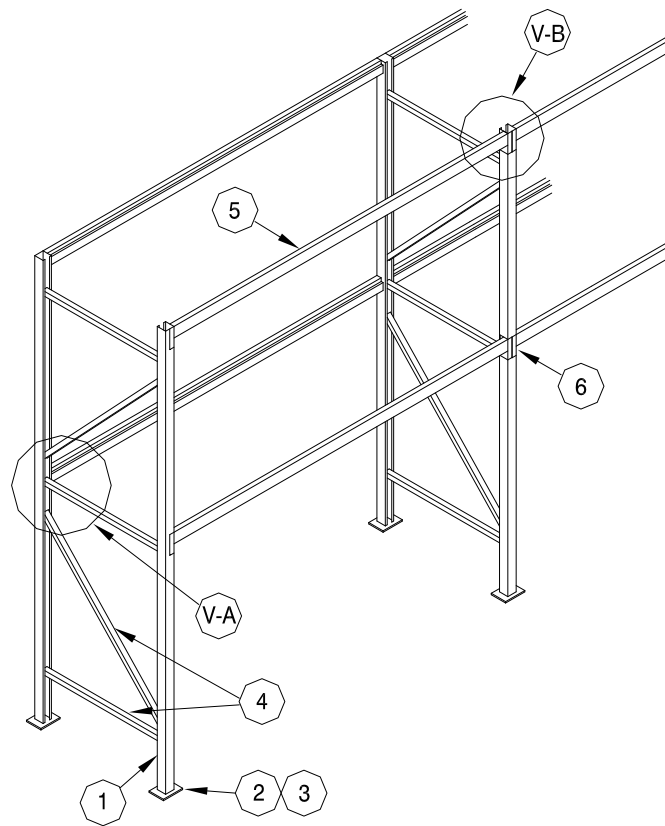
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Scope:

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.

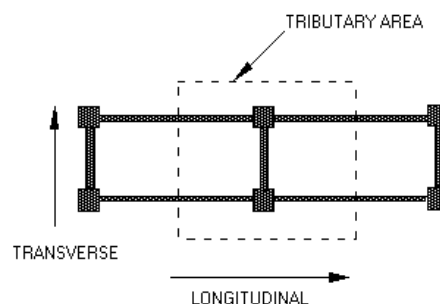
The storage racks consist of several bays, interconnected in one or both directions, with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal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

COMPONENTS AND SPECIFICATIONS

Configuration 1: Elevation A

IM 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 7

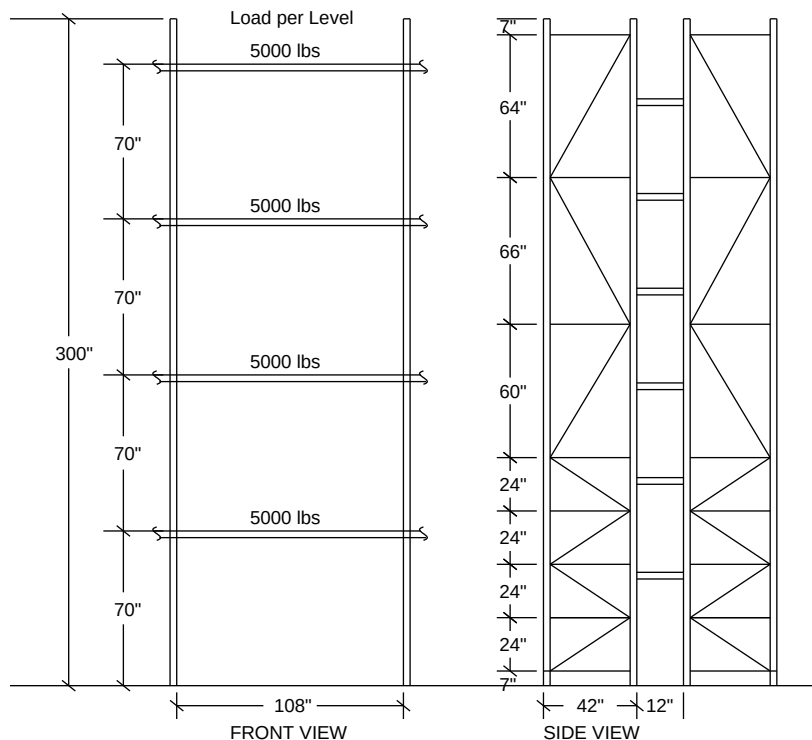
$$S = 0.11 \quad F = 1.6 \quad I = 1$$

$$S_s = 0.07 \quad F_v = 2.4 \quad SDC = B$$

$$V_{Long} = 276 \text{ lbs.}$$

$$V_{Trans} = 414 \text{ lbs.}$$

$$P_{static} = 10200 \text{ lbs.}$$



FRAME

COLUMN

3 x 3 -0.090 (19075)
Steel = 55000 psi
Stress = 93% (level 4)

HORIZONTAL BRACE

1.5 x 1.25 - .0625 (IHAFW)
Stress = 12% (panel 1)

DIAGONAL BRACE

1.5 x 1.25 - .0625 (IHAFW)
Stress = 29% (panel 5)

BEAM

4.5 x 2.75 -0.059 (45E)
Steel = 55 ksi Max Static Cap. = 5412 lb.
Stress = 94%

Max stress = 94% (level 1)

CONNECTOR

4 Tab 2" cc Connector (IM)
Stress = 47%

Max stress = 47% (level 1)

Base Plate

Steel = 36000 psi
3 x 4.5 x 0.12 in. 1 anchors/plate
Moment = 0 in-lb. Stress = 0%

Slab & Soil

Slab = 6" x 4000 psi
Sub Grade Reaction = 50 pci
Slab Bending Stress = 27% (S)

Anchors

Hilti KB1 ESR-678
0.5 in. x 3.625 in. Embed.
Pullout Capacity = 4094 lbs.
Shear Capacity = 3393 lbs.
Anchor stress = 91%

Notes:

Standard row spacers are required in this profile.



MATERIAL HANDLING ENGINEERING

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COMPONENTS AND SPECIFICATIONS

Configuration 2: Elevation B

IM 1.7

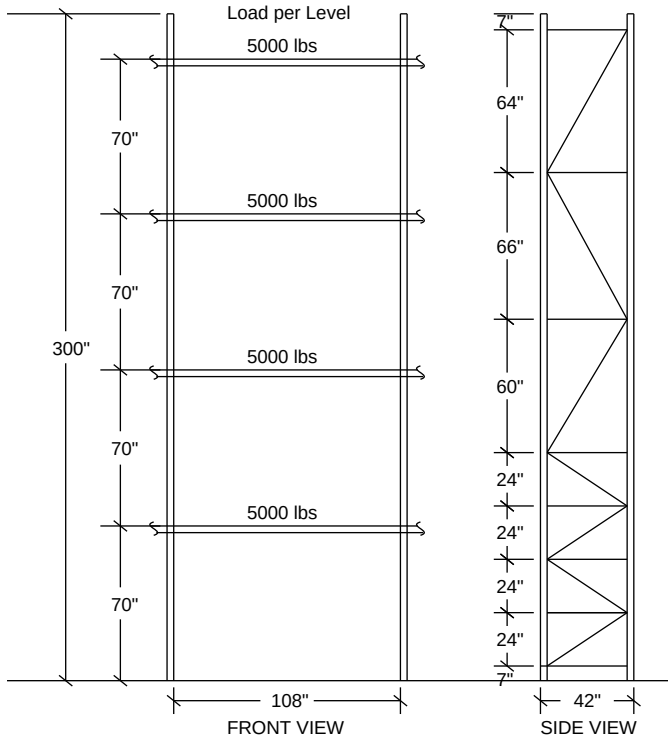
Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 7

$S = 0.11$ $F = 1.6$ $I = 1$
 $S_s = 0.07$ $F_v = 2.4$ $SDC = B$

$V_{Long} = 276$ lbs.
 $V_{Trans} = 414$ lbs.

$P_{static} = 10200$ lbs.
 $P_{seismic} = 1619$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 3 -0.090 (19075) Steel = 55000 psi Stress = 93% (level 4) HORIZONTAL BRACE 1.5 x 1.25 - .0625 (IHAFW) Stress = 12% (panel 1) DIAGONAL BRACE 1.5 x 1.25 - .0625 (IHAFW) Stress = 29% (panel 5)	4.5 x 2.75 -0.059 (45E) Steel = 55 ksi Max Static Cap. = 5412 lb. Stress = 94% Max stress = 94% (level 1)	4 Tab 2" cc Connector (IM) Stress = 47% Max stress = 47% (level 1)
Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 3 x 4.5 x 0.12 in. 1 anchors/plate Moment = 0 in-lb. Stress = 0%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 27% (S)	Hilti KB1 ESR-678 0.5 in. x 3.625 in. Embed. Pullout Capacity = 4094 lbs. Shear Capacity = 3393 lbs. Anchor stress = 91%

Notes:

Loads and Distributions: Elevation B

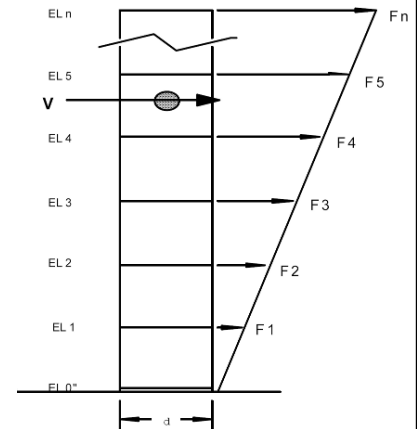
Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels: 4	SDC: B	R_L : 6	S_s : 0.11
Pallets Wide: 2	W_{PL} : 20000	R_T : 4	S_1 : 0.07
Pallets Deep: 1	W_{DL} : 400 lbs	F_a : 1.6	I_p : 1
Pallet Load: 2500	F_v : 2.4	T_l : 1.5	
Total Frame Load: 20400 lbs			

$$S_{DS} = 2/3 \cdot S_s \cdot F_a = 0.12$$

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

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



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

$$\begin{aligned} V_{long1} &= C_s \cdot I_p \cdot W_s \\ &= S_{D1} / (T_L \cdot R_L) \cdot I_p \cdot W_s \\ &= 0.11 / (1.5 \cdot 6) \cdot 1 \cdot 13800 = 168.67 \text{ lbs} \end{aligned}$$

V_{long} need not be greater than:

$$\begin{aligned} V_{long2} &= C_s \cdot I_p \cdot W_s \\ &= S_{DS} / R_L \cdot I_p \cdot W_s \\ &= 0.12 / 6 \cdot 1 \cdot 13800 = 276 \text{ lbs} \end{aligned}$$

If $S_1 \geq 0.6$, then V_{long} shall not be less than:

$$\begin{aligned} V_{long3} &= C_s \cdot I_p \cdot W_s \\ &= 0.5 \cdot S_1 / R_L \cdot I_p \cdot W_s \\ &= 0.5 \cdot 0.07 / 6 \cdot 1 \cdot 13800 = 80.5 \text{ lbs} \end{aligned}$$

V_{long} shall not be less than:

$$\begin{aligned} V_{long4} &= C_s \cdot I_p \cdot W_s \\ &= \text{Max}[0.044 \cdot S_{DS}, 0.03] \cdot I_p \cdot W_s \\ &= \text{Max}[0.03, 0.03] \cdot 1 \cdot 13800 = 414 \text{ lbs} \end{aligned}$$

Since: $168.67 < 414$
& $414 > 80.5$

$$V_{long} = 276 \text{ lbs}$$

Transverse

V_{trans} need not be greater than:

$$\begin{aligned} V_{trans1} &= C_s \cdot I_p \cdot W_s \\ &= S_{DS} / R_T \cdot I_p \cdot W_s \\ &= 0.12 / 4 \cdot 1 \cdot 13800 = 414 \text{ lbs} \end{aligned}$$

If $S_1 \geq 0.6$, then V_{trans} shall not be less than:

$$\begin{aligned} V_{trans2} &= C_s \cdot I_p \cdot W_s \\ &= 0.5 \cdot S_1 / R_T \cdot I_p \cdot W_s \\ &= 0.5 \cdot 0.07 / 4 \cdot 1 \cdot 13800 = 120.75 \text{ lbs} \end{aligned}$$

V_{trans} shall not be less than:

$$\begin{aligned} V_{trans3} &= C_s \cdot I_p \cdot W_s \\ &= \text{Max}[0.044 \cdot S_{DS}, 0.5 \cdot S_1 / R_T, 0.03] \cdot I_p \cdot W_s \\ &= \text{Max}[0.01, 0.01, 0.03] \cdot 1 \cdot 13800 = 414 \text{ lbs} \end{aligned}$$

Since: $414 \geq 120.75$
& $414 \geq 414$

$$V_{trans} = 414 \text{ lbs}$$



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DATE: 6/25/2024
PN: 20240415_29

Loads and Distributions: Elevation B (Page 2)

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

		Longitudinal			Transverse		
Level	h_x	w_x	$w_x h_x$	f_i	w_x	$w_x h_x$	f_i
1	70	2550	178500	27.6	2550	178500	41.4
2	140	2550	357000	55.2	2550	357000	82.8
3	210	2550	535500	82.8	2550	535500	124.2
4	280	2550	714000	110.4	2550	714000	165.6

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

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

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

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

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

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

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 $T = 4.31$

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

# of levels	4	
min. # of bays	3	
N _c	48	
N _b	8	
k _c	360 kip-in/rad	
k _{be}	3412 kip-in/rad	
k _b	147 kip-in/rad	
k _{ce}	588 kip-in/rad	
I _b	2.08 in ⁴	
L	108 in	
I _c	1.4 in ⁴	
H	280 in	
E	29500 ksi	
Level	h _{pi}	W _{pi}
1	102 in	5 kip
2	172 in	5 kip
3	242 in	5 kip
4	313 in	5 kip

LRFD Basic Load Combinations: Elevation B

2018 IBC & RMI / ANSI MH 16.1

$$\begin{aligned} V_{\text{Trans}} &= 414 \text{ lbs} & M_{\text{Trans}} &= \Sigma(f_{\text{Trans}} \cdot h_x) = 86,940 \text{ in-lbs} & \beta &= 0.7 \\ V_{\text{Long}} &= 276 \text{ lbs} & E_{\text{Trans}} &= M_{\text{Trans}} / \text{frame depth} = 2,070 \text{ lbs} & \beta &= 1.0 \text{ (Uplift combination only)} \\ P &= \text{Product Load} / 2 = 10,000 \text{ lbs} & & & \rho &= 1 \\ D &= \text{Dead Load} \cdot 0.5 = 200 \text{ lbs} & & & S_{\text{DS}} &= .12 \end{aligned}$$

$$\begin{aligned} L &= \text{Live Load} = 0 \text{ lbs} & S &= \text{Snow Load} = 0 \text{ lbs} & R &= \text{Rain Load} = 0 \text{ lbs} \\ L_r &= \text{Live Roof Load} = 0 \text{ lbs} & W &= \text{Wind Load} = 0 \text{ lbs} \end{aligned}$$

Basic Load Combinations

1. Dead Load $= 1.4 D + 1.2 P$
 $= (1.4 \cdot 200) + (1.2 \cdot 10,000) = 12,280 \text{ lbs}$
2. Gravity Load $= 1.2 D + 1.4 P + 1.6 L + 0.5 (L_r \text{ or } S \text{ or } R)$
 $= (1.2 \cdot 200) + (1.4 \cdot 10,000) + (1.6 \cdot 0) + (0.5 \cdot 0) = 14,240 \text{ lbs}$
3. Snow/Rain $= 1.2 D + 0.85 P + (0.5 L \text{ or } 0.5 W) + 1.6 (L_r \text{ or } S \text{ or } R)$
 $= (1.2 \cdot 200) + (0.85 \cdot 10,000) + (0.5 \cdot 0) + (1.6 \cdot 0) = 8,740 \text{ lbs}$
4. Wind Load $= 1.2 D + 0.85 P + 0.5 L + 1.0 W + 0.5 (L_r \text{ or } S \text{ or } R)$
 $= (1.2 \cdot 200) + (0.85 \cdot 10,000) + (0.5 \cdot 0) + (1.0 \cdot 0) + (0.5 \cdot 0) = 8,740 \text{ lbs}$
- 5A. Seismic Load (Transverse) $= (1.2 + 0.2 S_{\text{DS}}) D + (1.2 + 0.2 S_{\text{DS}}) \beta P + 0.5 L + \rho E_{\text{Trans}} + 0.2 S$
 $= (1.2 + 0.2 \cdot .12) \cdot 200 + (1.2 + 0.2 \cdot .12) \cdot 0.7 \cdot 10,000 + 0.5 \cdot 0 + 1 \cdot 2,070 + 0.2 \cdot 0 = 10,882 \text{ lbs}$
- 5B. Seismic Load (Longitudinal) $= (1.2 + 0.2 S_{\text{DS}}) D + (1.2 + 0.2 S_{\text{DS}}) \beta P + 0.5 L + \rho E_{\text{Long}} + 0.2 S$
 $= (1.2 + 0.2 \cdot .12) \cdot 200 + (1.2 + 0.2 \cdot .12) \cdot 0.7 \cdot 10,000 + 0.5 \cdot 0 + 1 \cdot 0 + 0.2 \cdot 0 = 8,812 \text{ lbs}$
6. Wind Uplift $= 0.9 D + 0.9 P_{\text{app}} + 1.0 W$
 $= 0.9 \cdot 200 + 0.9 \cdot 10,000 + 1.0 \cdot 0 = 180 \text{ lbs}$
7. Seismic Uplift $= (0.9 - 0.2 S_{\text{DS}}) D + (0.9 - 0.2 S_{\text{DS}}) \beta P_{\text{app}} - \rho E_{\text{Trans}}$
 $= (0.9 - 0.2 \cdot .12) \cdot 200 + (0.9 - 0.2 \cdot .12) \cdot 1 \cdot 10,000 - 1 \cdot 2,070 = 6,865 \text{ lbs}$
 For a single beam, $D = 36 \text{ lbs}$ $P = 2,500 \text{ lbs}$ $I = 312 \text{ lbs}$
8. Product/Live/Impact $= 1.2 D + 1.6 L + 0.5 (S \text{ or } R) + 1.4 P + 1.4 I$
 $= (1.2 \cdot 36) + (1.6 \cdot 0) + (0.5 \cdot 0) + (1.4 \cdot 2,500) + (1.4 \cdot 312) = 3,980 \text{ lbs}$

ASD Load Combinations for Slab Analysis

1. $(1 + 0.105 S'_{\text{DS}}) D + 0.75 ((1.4 + 0.14 S_{\text{DS}}) \beta P + 0.7 \rho E)$
 $= (1 + 0.105 \cdot .12) \cdot 200 + 0.75 ((1.4 + 0.14 \cdot .12) \cdot 0.7 \cdot 10,000 + 0.7 \cdot 1 \cdot 2,070) = 8,727 \text{ lbs}$
2. $(1 + 0.14 S_{\text{DS}}) D + (0.85 + 0.14 S_{\text{DS}}) \beta P + 0.7 \rho E$
 $= (1 + 0.14 \cdot .12) \cdot 200 + (0.85 + 0.14 \cdot .12) \cdot 0.7 \cdot 10,000 + 0.7 \cdot 1 \cdot 2,070 = 7,719 \text{ lbs}$
3. $D + P$
 $= 200 + 10,000 = 10,200 \text{ lbs}$

Longitudinal Analysis: Elevation B

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

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

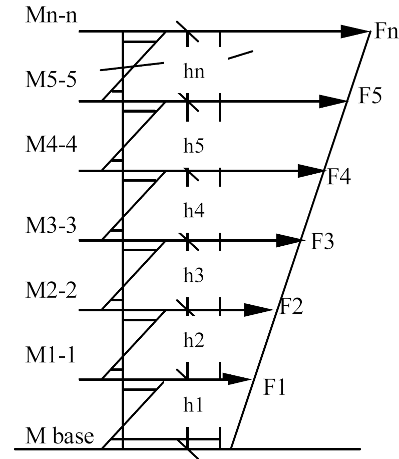
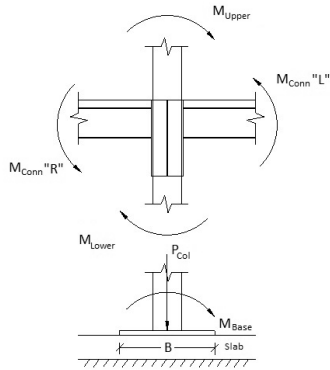
$$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$$

$$V_{Col} = V_{Long} / \# \text{ of columns} = 138 \text{ lbs}$$

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

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

$$(138 \text{ lbs} \cdot 68 \text{ in.}) - 0 \text{ in-lbs} = 9384 \text{ in-lbs}$$



FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	70	14	10,200	9,384	5,487	14,871
2	70	28	7,650	9,384	5,487	14,871
3	70	41	5,100	9,384	5,487	14,871
4	70	55	2,550	9,384	5,487	10,179



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COLUMN ANALYSIS: Elevation B (Level 4)

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

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

$$K_x \cdot L_x / R_x = 1.5 \cdot 68 / 1.256 = 81.23$$

$$K_y \cdot L_y / R_y = 1 \cdot 66 / 1.11 = 59.49$$

$$KL/R_{max} = 81.23$$

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

$$= (1.256^2 + 1.11^2 + -2.835^2)^{1/2} = 3.293 \text{ in.}$$

$$\beta = 1 - (X_o/r_o)^2 \quad (\text{Eq C4.1.2-3})$$

$$= 1 - (-2.835/3.293)^2 = 0.259$$

$$F_{el} = \pi^2 E / (KL/r)_{max}^2 \quad (\text{Eq C4.1.1-1})$$

$$= 3.14^2 \cdot 29500 / 81.23^2 = 44.126 \text{ ksi}$$

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

$$= (1 / (2 \cdot 0.259))((44.126 + 33.014) - ((44.126 + 33.014)^2 - (4 \cdot 0.259 \cdot 44.126 \cdot 33.014))^{1/2}) = 20.263 \text{ ksi}$$

where:

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

$$= 3.14^2 \cdot 29500 / 81.23^2 = 44.126 \text{ ksi}$$

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

$$= 1 / 0.885 \cdot 3.293^2 (11300 \cdot 0.002 + (3.142 \cdot 29500 \cdot 2.775) / (0.8 \cdot 66)^2) = 33.014 \text{ ksi}$$

$$F_c = \text{Min}(F_{el}, F_{e2}) = 20.263 \text{ ksi}$$

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

$$\lambda_c = (F_y / F_c)^{1/2} = (55 / 20.263)^{1/2} = 1.648 \quad (\text{Eq C4.1-4})$$

Since $\lambda_c \geq 1.5$:

$$F_n = (0.877 / \lambda_c^2) \cdot F_y = 17.771 \quad (\text{Eq C4.1-3})$$

Thus:

$$P_n = 12256 \text{ lbs}$$

$$P_a = 10418 \text{ lbs}$$

3 x 3 -0.090	
SECTION PROPERTIES	
Depth	3 in.
Width	3 in.
t	0.09 in.
Radius	0.125 in.
Area	0.885 in. ²
AreaNet	0.702 in. ²
I _x	1.396 in. ⁴
S _x	0.931 in. ³
S _{x Net}	0.762 in. ³
R _x	1.256 in.
I _y	1.09 in. ⁴
S _y	0.632 in. ³
R _y	1.11 in.
J	0.002 in. ⁴
C _w	2.775 in. ⁶
J _x	3.108 in.
X _o	-2.835 in.
K _x	1.5
L _x	68 in.
K _y	1
L _y	66 in.
K _t	0.8
F _y	55 ksi
F _u	65 ksi
Q	0.95
G	11300 ksi
E	29500 ksi
C _{mx}	0.85
C _s	-1
C _b	1
C _{tf}	1
Phi _b	0.9
Phi _c	0.85

COLUMN ANALYSIS: Elevation B (Level 4)

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

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

$$P_{ao} = P_{no} \phi_c = 32243 \text{ lbs}$$

Where:

$$P_{no} = A_c F_y = 0.69 \cdot 55 = 37933 \text{ lbs}$$

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

$$F_c = C_b r_o A (\sigma_{cy} \sigma_t)^{1/2} / S_f = 119.569 \text{ ksi}$$

$$F_c = C_s A \sigma_{cx} (j + C_s (j^2 + r_o^2 (\sigma_c / \sigma_{cx}))^{1/2}) / (C_{TF} S_f) = 46.504 \text{ ksi} \quad (\text{Eq 3.1.2.1-4})$$

$$F_c = (C_b \pi^2 E d I_{yc}) / (S_f (K_y L_y)^2) = 234.818 \text{ ksi} \quad (\text{Eq 3.1.2.1-10})$$

$$F_{c,min} = 46.504 \text{ ksi}$$

Since: $0.56 F_y < 2.78 F_y$

$$F_c = (10/9) F_y (1 - (10 F_y / 36 F_c)) = 41 \text{ ksi} \quad (\text{Eq C3.1.2.1-2})$$

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = 40.2 \text{ ksi}$

$$M_{nx} = 30612 \text{ in-lbs} \quad M_{ny} = 25422 \text{ in-lbs} \quad M_c = M_{n,min}$$

$$M_{nx} \phi_b = 27551 \text{ in-lbs} \quad M_{ny} \phi_b = 22880 \text{ in-lbs}$$

$$P_{Ex} = \pi^2 E I_x / (K_x L_x)^2 = 39066 \text{ lbs} \quad (\text{Eq C5.2.2-6})$$

$$P_{Ey} = \pi^2 E I_y / (K_y L_y)^2 = 72848 \text{ lbs} \quad (\text{Eq C5.2.2-7})$$

$$\alpha_x = (1 - (\phi_c P / P_{Ex})) = 0.951 \quad (\text{Eq C5.2.2-4})$$

$$\alpha_y = (1 - (\phi_c P / P_{Ey})) = 0.974 \quad (\text{Eq C5.2.2-5})$$

$$P_{trans} = 10,882 \text{ lbs} \quad P_{long} = 8,812 \text{ lbs}$$

$$M_u = M_x = 0 \text{ in-lbs} \quad (\text{Eq C5.2.2-2})$$

$$P_{u,st} = (1.2 \cdot D) + (1.4 \cdot P) = 3560 \text{ lbs}$$

$$P_{u,st} / P_a = 3560 / 10418 = 0.94 \quad \text{Static Stress} = 93\%$$

$$\text{Since: } P_i / P_a \geq 0.15$$

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

$$= ((8,812 / 10418) + (0 / 27551) + (1 / 22880)) = 84\%$$

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

$$= (8,812 / 32243) + (0.85 \cdot 0 / 27551 \cdot 0.951) + (0.85 \cdot 1 / 22880 \cdot 0.974) = 27\%$$

$$\text{Stress3} \quad P_i / P_{ao} = 10,882 / 32243 = 33\%$$

$$\text{Column Stress} = \text{Max}(\text{Stress1}, \text{Stress2}, \text{Stress3}, \text{Static}) = 93\%$$

3 x 3 -0.090	
SECTION PROPERTIES	
Depth	3 in.
Width	3 in.
t	0.09 in.
Radius	0.125 in.
Area	0.885 in. ²
AreaNet	0.702 in. ²
I _x	1.396 in. ⁴
S _x	0.931 in. ³
S _{x Net}	0.762 in. ³
R _x	1.256 in.
I _y	1.09 in. ⁴
S _y	0.632 in. ³
R _y	1.11 in.
J	0.002 in. ⁴
C _w	2.775 in. ⁶
J _x	3.108 in.
X _o	-2.835 in.
K _x	1.5
L _x	68 in.
K _y	1
L _y	66 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

BEAM ANALYSIS Elevation B

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

$$\text{Effective width } w = C - 2t - 2r = 1.75 - (2 \cdot 0.059) - (2 \cdot 0.09) = \mathbf{1.45 \text{ in.}}$$

$$w/t = 1.452 / 0.059 = \mathbf{24.61}$$

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

$\lambda \leq \mathbf{0.673}$: Flange is fully effective.

Check web for local buckling (B2.3)

$$f_1(\text{comp}) = F_y \cdot (y_3 / y_2) = 55 \cdot 2.22 / 2.37 = \mathbf{51.54 \text{ ksi}}$$

$$f_2(\text{tension}) = F_y \cdot (y_1 / y_2) = 55 \cdot 1.98 / 2.37 = \mathbf{45.93 \text{ ksi}}$$

$$\Psi = -(f_2 / f_1) = -(45.93 / 51.54) = \mathbf{-0.89}$$

$$\text{Buckling coefficient } k = 4 + 2 \cdot (1 - \Psi)^3 + 2 \cdot (1 - \Psi)$$

$$= 4 + 2(1 - \mathbf{-0.89})^3 + 2(1 - \mathbf{-0.89}) = \mathbf{21.31}$$

$$\text{Flat Depth } w = y_1 + y_3 = 1.98 + 2.22 = \mathbf{4.202}$$

$$w/t = 4.202 / 0.059 = \mathbf{71.22} \quad \mathbf{w/t < 200: OK}$$

$$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (f_1 / E)^{1/2} = (1.052 / 2) \cdot 71.22 \cdot (51.54 / 29500)^{1/2} = \mathbf{0.68}$$

$$b_1 = w \cdot (3 - \Psi) = 4 \cdot (3 - \mathbf{-0.89}) = \mathbf{16.35}$$

$$b_2 = w/2 = \mathbf{2.1}$$

$$b_1 + b_2 = 16.35 + 2.1 = \mathbf{18.45} \quad \mathbf{\text{Web is fully effective}}$$

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

$$\text{Corner cross-sectional area } L_c = (\Pi / 2) \cdot (r + t / 2)$$

$$= (\Pi / 2) \cdot (0.09 + 0.059 / 2) = \mathbf{0.188}$$

$$L_f = \text{effective width} = \mathbf{1.452}$$

$$C = 2 \cdot L_c / L_f + 2 \cdot L_c = 2 \cdot 0.188 / 1.452 + 2 \cdot L_c = \mathbf{0.2054}$$

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

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

$$= 3.69 \cdot (65 / 55) - 0.819 \cdot (65 / 55)^2 - 1.79 = \mathbf{1.43}$$

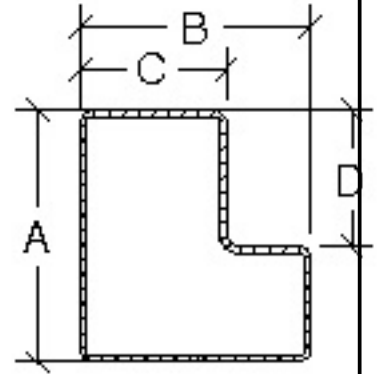
$$F_u / F_y = 65 / 55 = \mathbf{1} \quad \mathbf{< 1.2}$$

$$r/t = 0.09 / 0.059 = \mathbf{1.525} \quad \mathbf{\leq 7 = OK}$$

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

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

$$F_{ya-bottom} = F_{ya-top} \cdot Y_{cg} / (A - Y_{cg}) = 59 \cdot 2.13 / (4.5 - 2.13) = \mathbf{53}$$



4.5 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	4.5 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.98
Y2 =	2.37
Y3 =	2.22
Ycg =	2.13
Ix =	2.08
Sx =	0.88
E =	29500
FBeam F =	360
Beam Length L =	108

BEAM ANALYSIS Elevation B

Check Allowable Tension Stress for Bottom Flange

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

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

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

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

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

Check Bending Capacity

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

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

For DL = 2% of PL:

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

$$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L)) \\ = 1 - ((2 \cdot 360 \cdot 108) / (6 \cdot 29500 \cdot 2.08 + 3 \cdot 360 \cdot 108)) = \mathbf{0.84}$$

$$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x = \mathbf{49.04 \text{ in-kip}}$$

$$W = \phi \cdot M_n \cdot 8 \cdot (\# \text{ of Beams}) / (L \cdot R_m \cdot \Omega) = (49.04 \cdot 8 \cdot 2) / (108 \cdot 0.84 \cdot 1.6) \\ = \mathbf{5412 \text{ lbs/pair}}$$

Check Deflection Capacity

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

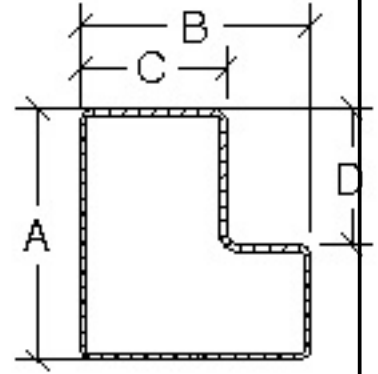
$$\Delta_{\text{max}} = L / 180$$

$$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x) \\ = 1 - (4 \cdot 360 \cdot 108) / (5 \cdot 360 \cdot 108 + 10 \cdot 29500 \cdot 2.08) = \mathbf{0.81}$$

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

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

$$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d) \\ = (384 \cdot 29500 \cdot 2.08 \cdot 2) / (180 \cdot 5 \cdot 108^2 \cdot 0.81) \cdot 1000 = \mathbf{5564 \text{ lbs/pair}}$$



4.5 x 2.75 -0.059

Top flange width C =	1.75 in.
Bottom width B =	2.75 in.
Web depth A =	4.5 in.
Beam thickness t =	0.059 in.
Radius r =	0.09 in.
Fy =	55
Fu =	65
Y1 =	1.98
Y2 =	2.37
Y3 =	2.22
Ycg =	2.13
Ix =	2.08
Sx =	0.88
E =	29500
FBeam F =	360
Beam Length L =	108



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1130 E. CYPRESS ST, COVINA, CA 91724

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	34,398	36,531	6,056	36,531	Pass
2	34,398	36,531	4,193	36,531	Pass
3	34,398	36,531	3,227	36,531	Pass
4	34,398	36,531	2,744	36,531	Pass

Beam to Column Analysis: Elevation B

1. Shear Strength of Tab

Height of the Tab $h = 0.6$ in.

Thickness of the Tab $t_t = 0.135$ in.

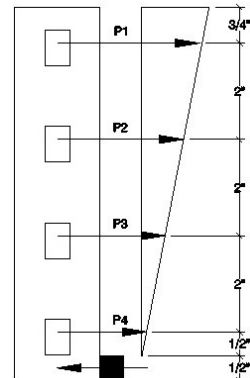
$$F_y = 55000 \text{ psi}$$

$$C_v = 1.0$$

$$V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = 2673 \text{ lbs}$$

AISC G2-1

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



2. Bearing Strength of Tab

Thickness of the column $t_c = 0.09$ in.

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

$$R_n = 1.8 \cdot F_y \cdot A_{pb} = 5346 \text{ lbs}$$

AISC J7 -1

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

3. Moment Strength of Bracket

Edge Dist. = 1 in.

$$T_{\text{Clip}} = 0.179 \text{ in.}$$

$$S_{\text{Clip}} = 0.127 \text{ in.}^3$$

$$M_n = S_c \cdot F_y = 6985 \text{ in-lbs}$$

AISI C3.1.1 -1

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

$$C = 2.15$$

$$d = \text{Edge Dist.} / 2 = 0.5 \text{ in.}$$

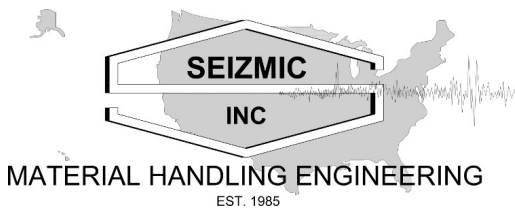
$$M_{\text{Strength}} = c \cdot d \cdot P_{\text{Clip}}$$

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

Minimum Value of P1 Governs

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

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



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BRACE ANALYSIS Elevation B (Panel 5)

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

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

$$K_x \cdot L_x / R_x = 1 \cdot 64 / 0.605 = 105.86$$

$$K_y \cdot L_y / R_y = 1 \cdot 64 / 0.457 = 139.92$$

$$KL/R_{max} = 139.92$$

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

$$= (0.605^2 + 0.457^2 + (-1.149)^2)^{1/2} = 1.377 \text{ in.}$$

$$\beta = 1 - (X_o/r_o)^2 \quad (\text{Eq C4.1.2-3})$$

$$= 1 - (-1.149/1.377)^2 = 0.303$$

$$F_{el} = \pi^2 E / (KL/r)_{max}^2 \quad (\text{Eq C4.1.1-1})$$

$$= 3.14^2 \cdot 29500 / 139.92^2 = 14.871 \text{ ksi}$$

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

$$= (1 / (2 \cdot 0.303))((25.984 + 11.928) - (25.984 + 11.928)^2 - (4 \cdot 0.303 \cdot 25.984 \cdot 11.928))^{1/2} = 8.794 \text{ ksi}$$

where:

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

$$= 3.14^2 \cdot 29500 / 105.86^2 = 25.984 \text{ ksi}$$

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

$$= 1 / 0.255 \cdot 1.377^2 (11300 \cdot 0 + (3.142 \cdot 29500 \cdot 0.033) / (0.8 \cdot 64)^2) = 11.928 \text{ ksi}$$

$$F_c = \text{Min}(F_{el}, F_{e2}) = 8.794 \text{ ksi}$$

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

$$\lambda_c = (F_y / F_c)^{1/2} = (50 / 8.794)^{1/2} = 2.384 \quad (\text{Eq C4.1-4})$$

Since $\lambda_c \geq 1.5$:

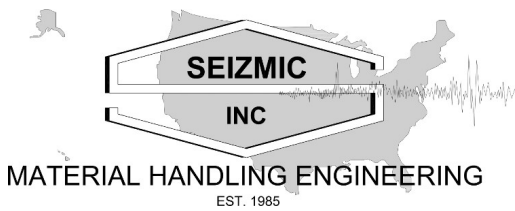
$$F_n = (0.877 / \lambda_c^2) \cdot F_y = 7.712 \quad (\text{Eq C4.1-3})$$

Thus:

$$P_n = 1963 \text{ lbs}$$

$$P_a = 1668 \text{ lbs}$$

1.5 x 1.25 - .0625	
SECTION PROPERTIES	
Depth	1.5 in.
Width	1.25 in.
t	0.06 in.
Radius	0.125 in.
Area	0.255 in. ²
AreaNet	0.255 in. ²
I _x	0.093 in. ⁴
S _x	0.124 in. ³
S _{xNet}	0.124 in. ³
R _x	0.605 in.
I _y	0.053 in. ⁴
S _y	0.073 in. ³
R _y	0.457 in.
J	0 in. ⁴
C _w	0.033 in. ⁶
J _x	1.301 in.
X _o	-1.149 in.
K _x	1
L _x	64 in.
K _y	1
L _y	64 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



PROJECT: H & R Block
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BRACE ANALYSIS Elevation B (Panel 5)

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

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

$$P_{ao} = P_{no} \phi_c = 10820 \text{ lbs}$$

Where:

$$P_{no} = A_c F_y = 0.255 \cdot 50 = 12730 \text{ lbs}$$

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

$$F_c = C_b r_o A (\sigma_{cy} \sigma_t)^{1/2} / S_f = 49.729 \text{ ksi}$$

$$F_c = C_s A \sigma_{ex} (j + C_s (j^2 + r_o^2 (\sigma_c / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = 15.989 \text{ ksi} \quad (\text{Eq 3.1.2.1-4})$$

$$F_c = (C_b \pi^2 E d I_{yc}) / (S_f (K_y L_y)^2) = 45.794 \text{ ksi} \quad (\text{Eq 3.1.2.1-10})$$

$$F_{c,min} = 15.989 \text{ ksi}$$

Since: $F_c \leq 0.56 F_y$

$$F_c = F_c = 15.989 \text{ ksi} \quad (\text{Eq C3.1.2.1-3})$$

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = 16 \text{ ksi}$

$$M_{nx} = 1985 \text{ in-lbs} \quad M_{ny} = 1168 \text{ in-lbs} \quad M_c = M_{n,min}$$

$$M_{nx} \phi_b = 1787 \text{ in-lbs} \quad M_{ny} \phi_b = 1051 \text{ in-lbs}$$

$$P_{Ex} = \pi^2 E I_x / (K_x L_x)^2 = 6617 \text{ lbs} \quad (\text{Eq C5.2.2-6})$$

$$P_{Ey} = \pi^2 E I_y / (K_y L_y)^2 = 3788 \text{ lbs} \quad (\text{Eq C5.2.2-7})$$

$$P_a = 1668 \text{ lbs}$$

$$V_{Trans} = 270 \text{ lbs}$$

$$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} = 64.9 \text{ in.}$$

$$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = 488 \text{ lbs}$$

$$\text{Brace Stress} = V_{Diag} / P_a = 29\%$$

1.5 x 1.25 - .0625	
SECTION PROPERTIES	
Depth	1.5 in.
Width	1.25 in.
t	0.06 in.
Radius	0.125 in.
Area	0.255 in. ²
AreaNet	0.255 in. ²
I _x	0.093 in. ⁴
S _x	0.124 in. ³
S _{x Net}	0.124 in. ³
R _x	0.605 in.
I _y	0.053 in. ⁴
S _y	0.073 in. ³
R _y	0.457 in.
J	0 in. ⁴
C _w	0.033 in. ⁶
J _x	1.301 in.
X _o	-1.149 in.
K _x	1
L _x	64 in.
K _y	1
L _y	64 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

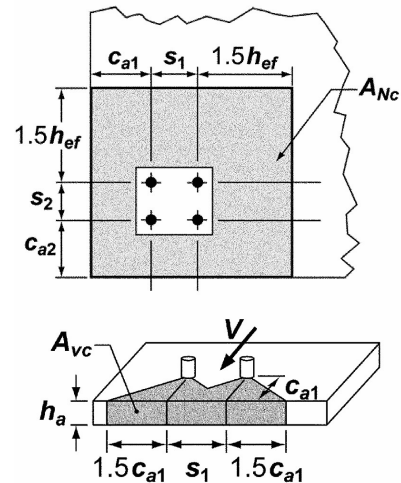
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 2 Elevation B

Assumed cracked concrete application

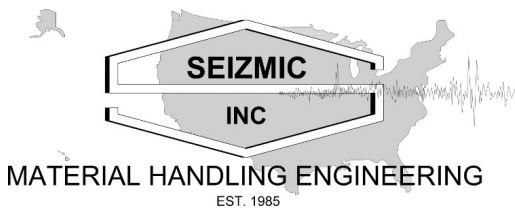
Anchor Type	0.5" dia., 3.25 hef, 6" min, slab		
ICC Report Number	ESR-678	$1.5 \cdot h_{ef}$	= 4.875 in.
Slab Thickness (h)	= 6 in.	$C_{a1} = 12$	use $C_{a1,adj} = 4.875$ in.
Min. Slab Thickness (h)	= 6 in.	$C_{a2} = 12$	use $C_{a2,adj} = 4.875$ in.
Concrete Strength (f_c)	= 4000 psi		
Diameter (d_a)	= 0.5 in.	$3 \cdot h_{ef}$	= 9.75 in.
Nominal Embedment (h_{nom})	= 3.625 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_{sc}	= 0.103 sq.in.
f_{uta}	= 105900 psi
S_{min}	= 4 in.
C_{min}	= 4 in.
C_{ae}	= 10 in.
$N_{p,cr}$	= 9999 lbs



	$\phi_{Seismic}$	Adj. Strength
Tension Capacity = 4094 lbs	1	4094 lbs
Shear Capacity = 3393 lbs	1	3393 lbs

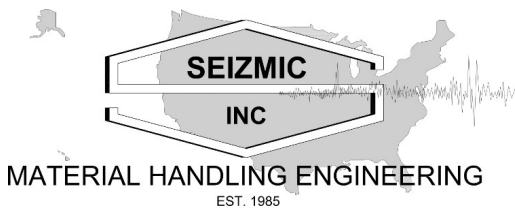


PROJECT: H & R Block
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PN: 20240415_29

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1130 E. CYPRESS ST, COVINA, CA 91724

ANCHOR ANALYSIS - TENSION STRENGTH Configuration 2 Elevation B

Steel Strength	17.4.1
$\phi = 0.75$	17.3.3.a i
$\phi N_{sa} = \phi n A_{sc} f_{uta} = 0.75 \cdot 1 \cdot 0.103 \cdot 105900 = 8,181 \text{ lbs}$	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
$\phi = 0.65$	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1,adj} + S_{1,adj} + 1.5h_{ef}) \cdot (C_{a2,adj} + S_{2,adj} + 1.5h_{ef}) = 95.063 \text{ sq.in.}$	
$A_{Nco} = 9h_{ef}^2 = 95.063 \text{ sq.in.}$	
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
$K_c = 17$	
$\lambda_a = 1$	
$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} = 6299 \text{ lbs}$	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
$0.65 \cdot (95.063/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299 = 4,094 \text{ lbs}$	
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
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} = 8,221 \text{ lbs}$	17.4.3.1
Steel Strength (ϕN_{sa}) = 8,181 lbs	
Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = 4,094 lbs	
Embedment Strength - Pullout Strength (ϕN_{pn}) = 8,221 lbs	



MATERIAL HANDLING ENGINEERING
EST. 1985

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PROJECT: H & R Block
FOR: Crown Equipment Corp_T
ADDRESS: 1220 NW Main St.
Lee's Summit, MO
SHEET#: 20
CALCULATED BY: aswenson
DATE: 6/25/2024
PN: 20240415_29

ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 2 Elevation B

Steel Strength ϕV_{sa} $V_{sa} = 5,220$ / Anchor -- per report

17.5.1

$\phi = 0.65$

17.3.3. Condition a ii

$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 1 \cdot 5,220 = 3,393$ lbs

17.5.1.2a

Concrete Breakout Strength ϕV_{cbg}

17.5.2

$\phi = 0.7$

17.3.3 ci-B

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

$A_{Vco} = 3C_{a1}h_a = 216$ sq.in.

Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} = 1$

$\Psi_{cc,V} = 1$

17.5.2.5

$\Psi_{cd,V} = 0.9$

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

$L_c = 1$ in.

17.2.6 d

$\lambda_a = 1$

The smaller of $7(L_c / d_a)^{0.2}(d_a)^{0.5}\lambda_a(f_c)^{0.5}ca1^{1.5}$ and $9\lambda_a(f_c)^{0.5}ca1^{1.5} = 14,948$ lbs

17.5.2.2 a, 17.5.2.2 b

$\phi V_{cbg} = \phi(A_{Vc}/A_{Vco})(\Psi_{cc,V})(\Psi_{cd,V})(\Psi_{C,V})(\Psi_{h,V})(V_b)$

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 (ϕV_{sa}) = 3,393 lbs

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = 16,311 lbs

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

OVERTURNING ANALYSIS Configuration2 Elevation B

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

FULLY LOADED

$$W_{pl} = 20,000 \text{ lbs} \quad W_{dl} = 400 \text{ lbs}$$

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

$$V_{Trans} = (1 \cdot 0.03 \cdot 1 \cdot ((0.67 \cdot 13,400) + 400)) = 281 \text{ lbs}$$

$$M_{ovt} = V_{Trans} \cdot Ht = 281 \cdot 242 = 68,002 \text{ in-lbs}$$

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

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

$$P_{uplift} = 1 \cdot (M_{ovt} - M_{st}) / d = (68,002 - 289,800) / 42 = -5,280 \text{ lbs}$$

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

$$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (68,002 + 289,800) / 42 = 8,519 \text{ lbs}$$

TOP SHELF LOADED

$$\text{Shear} = 162 \text{ lbs}$$

$$M_{ovt} = V_{Top} \cdot Ht = 162 \cdot (280 + ((70 - 10) / 2)) = 50,220 \text{ in-lbs}$$

$$M_{st} = (1 + W_{dl}) \cdot d = (5,000 + 400) \cdot (42 \cdot 0.5) = 113,400 \text{ in-lbs}$$

$$P_{uplift} = 1 \cdot (M_{ovt} - M_{st}) / d = (50,220 - 113,400) / 42 = -1,504 \text{ lbs}$$

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

ANCHORS

No. of Anchors (#Anchors): 1

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

Shear Capacity per Anchor: 3,393 lbs

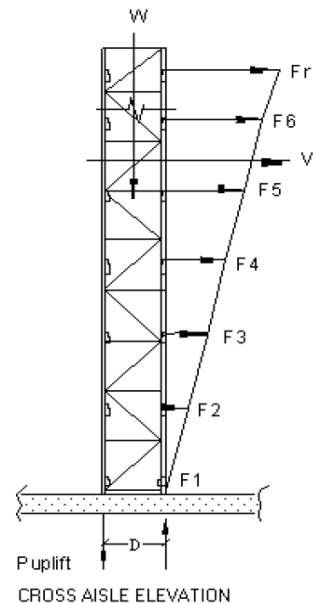
COMBINED STRESS

$$\text{Fully Loaded} = ((0 / 1) / 4,094) + ((281 / 2) / 3,393) = 0.041$$

$$\text{Top Shelf Loaded} = ((0 / 1) / 4,094) + ((162 / 2) / 3,393) = 0.024$$

$$\text{Seismic UpLift Critical (LC\#7B)} = (0 / 1) / 4,094 = 0$$

$$\text{Side Load Top Shelf} = (3,733 / 1) / 4,094 = 0.912$$



SIDE LOADS ON TOP SHELF

Top loaded shelf level (H) = 280 in.

$$M_{ovt} = 1.6 \cdot 350 \cdot H = 1.6 \cdot 350 \cdot 280 = 156 \text{ kip}$$

$$P_{uplift} = M_{ovt} / d = 156 / 42 = 3,733 \text{ lbs}$$

Base Plate Analysis: Elevation B

The base plate will be analyzed with the rectangular stress resulting from the vertical load P, combined with the triangular stresses resulting from the moment Mb (if any). Three criteria are used in determining Mb:

1. Moment capacity of the base plate
2. Moment capacity of the anchor bolts
3. $V_{col} \cdot h/2$ (full fixity)

Mb is the smallest value obtained from these three criteria.

$$F_y = 36000 \text{ psi}$$

$$P_{col} = 14240 \text{ lbs}$$

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

$$P/A = P_{col} / (D \cdot B) = 14240 / (4.5 \cdot 3) = 1,055 \text{ psi}$$

$$f_b = M_{Base} / (D \cdot B^2 / 6) = 0 / (4.5 \cdot 3^2 / 6) = 0 \text{ psi}$$

$$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 0 \cdot (2 \cdot 0 / 3) = 0 \text{ psi}$$

$$f_{b1} = f_b - f_{b2} = 0 - 0 = 0 \text{ psi}$$

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

$$= (0^2 / 2) \cdot (1,055 + 0 + 0.67 \cdot 0) = 0 \text{ in-lbs}$$

$$S_{Base} = (B \cdot t^2) / 6 = 0.01 \text{ sq.in.}$$

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

$$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 0 / (0.01 \cdot 32,400) = 0$$

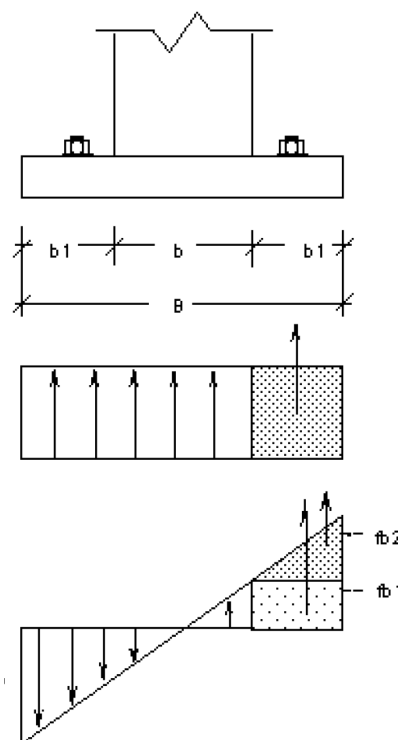


Plate width B = 3 in.
Plate depth D = 4.5 in.
Plate thickness t = 0.12 in.
Column width b = 3 in.
Column depth d = 3 in.
b1 = 0 in.

Equation for Maximum Considered Earthquake Base Rotation

Per RMI 2012 Commentary 2.6.4

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

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

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

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

$$\alpha_s = 1.88$$

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
 $\theta_b = 0.71$

Per RMI 2012 2.6.6,

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

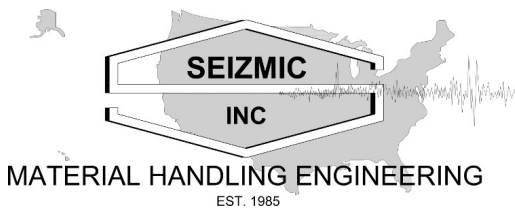
$$\tan \theta_{max} = 0.5 \quad \theta_{max} = 2.862 \text{ rad} \quad \theta_b \text{ ok}$$

Maximum moment in base plate

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

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

# of levels	4	
min. # of bays	3	
N _c	48	
N _b	8	
k _c	360 kip-in/rad	
k _{bc}	3412 kip-in/rad	
k _b	147 kip-in/rad	
k _{cc}	588 kip-in/rad	
I _b	2.08 in ⁴	
L	108 in	
I _c	1.4 in ⁴	
H	280 in	
E	29500 ksi	
Level	h _{pi}	W _{pi}
1	102 in	5 kip
2	172 in	5 kip
3	242 in	5 kip
4	313 in	5 kip



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SLAB AND SOIL ANALYSIS (LRFD)

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

$$P_{max} = \text{Gravity_Load (see Basic Load Combinations)} = 14,240 \text{ lbs}$$

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

$$d, req'd = (P_{max} / (\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} = 2.747 \text{ in.}$$

$$b = (E_c \cdot d, req'd^3 / (12 \cdot (1 - \mu^2) \cdot K_s))^{1/4} = 18.893 \text{ in.}$$

$$b, req'd = 1.5 \cdot b = 28 \text{ in.}$$

$$P_n = 1.72[(K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 = 113,220 \text{ lbs}$$

$$P_a = \phi \cdot P_n = 67,932 \text{ lbs}$$

$$P_{max} / P_a = 0.21$$

Base Plate	
Width B	3 in.
Depth W	4.5 in.

Frame	
Frame depth d	42 in.

SLAB AND SOIL ANALYSIS (ASD)

$$P_{max} = \text{MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)} \\ = 10,200 \text{ lbs}$$

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

$$P_n = 1.72[(K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 = 113,220 \text{ lbs}$$

$$d, req'd = (P_{max} / (\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} = 2.747 \text{ in.}$$

$$b = (E_c \cdot d, req'd^3 / (12 \cdot (1 - \mu^2) \cdot K_s))^{1/4} = 18.893 \text{ in.}$$

$$b, req'd = 1.5 \cdot b = 28 \text{ in.}$$

$$P_a = P_n / \Omega = 37,740 \text{ lbs}$$

$$P_{max} / P_a = 0.27$$

Concrete	
Thickness t	6 in.
f'_c	4,000 psi
ϕ	0.6
Ω	3
λ	1
K_s	50 pci
r_1	1.84 in
E_c	3,604,997 psi