SEIZMIC INC

MATERIAL HANDLING ENGINEERING

EST. 1985

SPECIAL PRODUCTS	*CONVEYORS*	*STORAGE RACKS*	*OTHER SERVICES*	*SHELVING*	*SPECIAL PRODUCTS*
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WORK PLATFORMS	ROOF VERIFICATION	CANTILEVER	PRODUCT TESTING	LOCKERS	MODULAR OFFICES
FOOTINGS			TITLE 24	CATWALKS	MINI-LOAD SYSTEMS

LICENSED IN 50 STATES

ANALYSIS OF
STORAGE RACKS
FOR

CED

5041 W 2400 S, Suite #20, West Valley City, UT

Job No. 24-1789

Approved by:

SAL E. FATEEN, P.E.

7/10/2024

LICENSED PROFESSIONAL ENGINEER
No. 2[illegible]28[illegible]9
SAL E. FATEEN
STATE OF UTAH

EXPIRES
03-31-2025

Digitally signed by Sal Fateen
Date: 2024.07.11 15:20:03-07'00'

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

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Scope:

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

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

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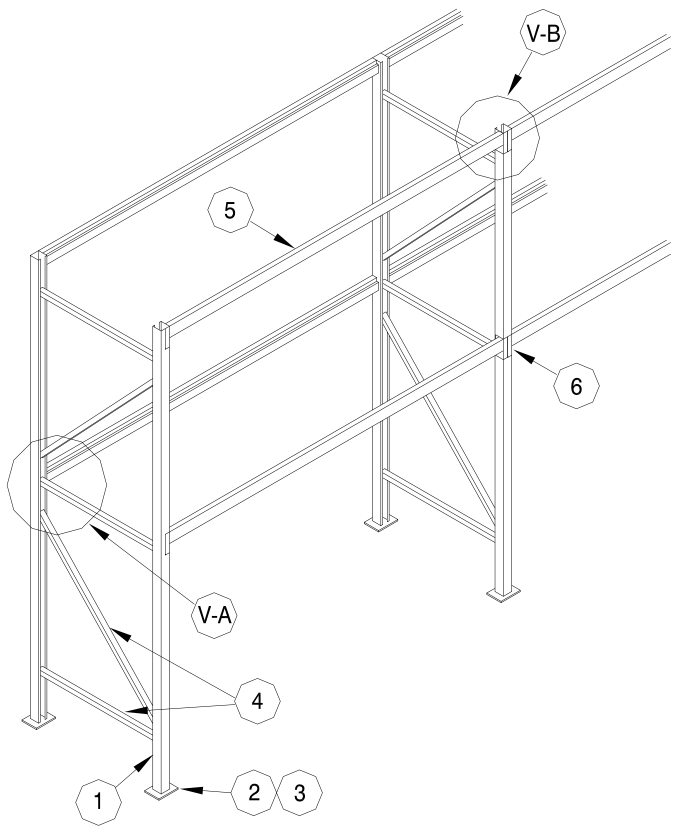
PROJECT: CED
FOR: Interlake-Mecalux_Reyn
ADDRESS: 5041 W 2400 S, Sui
West Valley City, UT
SHEET#: 2
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

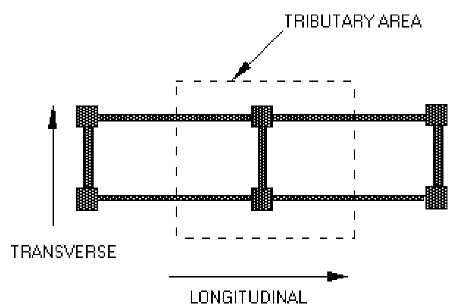
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction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



CONCEPTUAL DRAWING

Some components may not be used or may vary

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



NOTE: ACTUAL CONFIGURATION SHOWN ON COMPONENTS & SPECIFICATIONS SHEET

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PROJECT: CED
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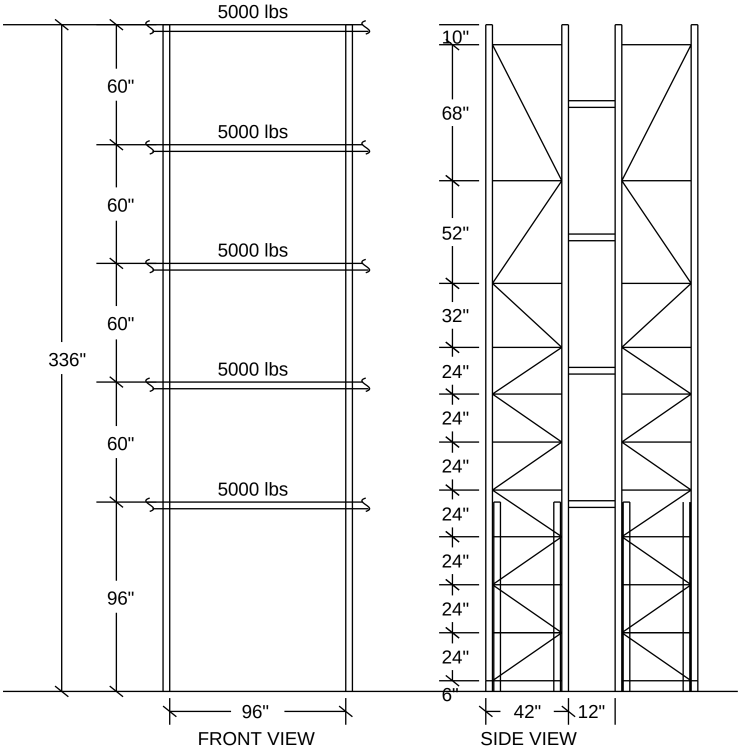
ADDRESS: 5041 W 2400 S, Sui
West Valley City, UT
SHEET#: 3
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

COMPONENTS AND SPECIFICATIONS Configuration 1: CR1 M 1.7

Analysis per section 2209 of the IBC2021

Levels: 5 Panels: 10 Load per Level

S_s = 1.29 F_a = 1.2 I = 1 V_{Long} = 1073 lbs. P_{static} = 12750 lbs.
S_1 = 0.45 F_v = 1.85 SDC = D V_{Trans} = 4441 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 83% (level 2) **BACKER TO LEVEL 1** 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 97% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 66% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 89% (panel 9)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5906 lb. Stress = 86% Max stress = 93% (level 2)	**Level 1** 5 Tab Connector (IM) Stress = 86% **Level2** 4 Tab 2" cc Connector (IM) Stress = 64% Max stress = 86% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 12 x 23 x 0.5 in. 7 anchors/plate Moment = 8500 in-lb. Stress = 8%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 58% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2817 lbs. Shear Capacity = 4999 lbs. Anchor stress = 89%

Notes:
BCR 8'
SPST 7x24 1X32 1x52 1x68
Full depth base plate 12"x46"x0.5" + STIFFENER BETWEEN COLUMNS with Sx=9.5", Sy=6". 14 anchors/plate. For each upright, 2 anchors in middle, 6 anchors on each end

Standard row spacers are required in this profile.

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PROJECT: CED
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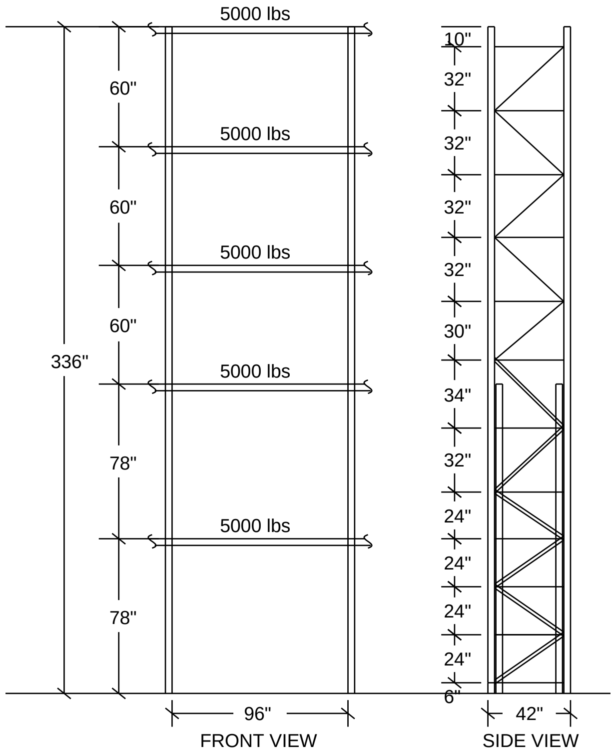
ADDRESS: 5041 W 2400 S, Sui
West Valley City, UT
SHEET#: 4
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

COMPONENTS AND SPECIFICATIONS Configuration 2: CR2 M 1.7

Analysis per section 2209 of the IBC2021

Levels: 5 Panels: 11 Load per Level

S_s = 1.29 F_a = 1.2 I = 1 V_{Long} = 1073 lbs. P_{static} = 12750 lbs.
S_1 = 0.45 F_v = 1.85 SDC = D V_{Trans} = 4441 lbs. $P_{seismic}$ = 20191 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 56% (level 3) **BACKER TO LEVEL 2** 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 72% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 57% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 80% (panel 7)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5906 lb. Stress = 86% Max stress = 86% (level 1)	5 Tab Connector (IM) Stress = 80% Max stress = 80% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 12 x 23 x 0.5 in. 7 anchors/plate Moment = 8500 in-lb. Stress = 8%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 59% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2817 lbs. Shear Capacity = 4999 lbs. Anchor stress = 90%

Notes:
BCR 14'
SPST 4X24; 7X32
Full depth base plate 12"x46"x0.5" + STIFFENER BETWEEN COLUMNS with Sx=9.5", Sy=6". 14 anchors/plate. 2 anchors in middle, 6 anchors on each end

Diagonal Braces Doubled 1 - 6

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CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the IBC2021

Levels: 4 Panels: 10 Load per Level

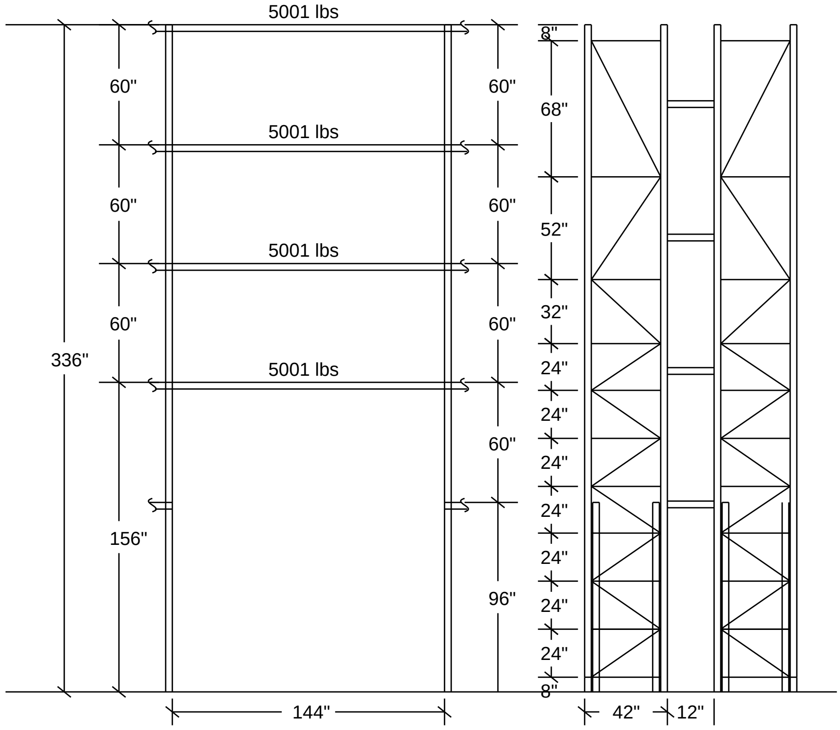
Configuration 3: CT1
Adjacent to: CR1

M 1.7

S_s = 1.29 F_a = 1.2 I = 1
S_1 = 0.45 F_v = 1.85 SDC = D

V_{Long} = 966 lbs.
V_{Trans} = 3997 lbs.

P_{static} = 11475 lbs.



FRONT VIEW
SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 80% (level 1 adj.) **BACKER TO LEVEL 1** 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 86% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 60% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 (C715) Stress = 88% (panel 9)	6.56 x 2.75 -0.059 (65E) Steel = 55 ksi Max Static Cap. = 7414 lb. Stress = 68% Max stress = 68% (level 1)	5 Tab Connector (IM) Stress = 41% Max stress = 41% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 12 x 23 x 0.5 in. 7 anchors/plate Moment = 8500 in-lb. Stress = 7%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 54% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2817 lbs. Shear Capacity = 4999 lbs. Anchor stress = 83%

Notes:
BCR 8'
SPST 7X24; 1X32 1x52 1x68
Full depth base plate 12"x46"x0.5" + STIFFENER BETWEEN COLUMNS with Sx=9.5", Sy=6". 14 anchors/plate. For each upright, 2 anchors in middle, 6 anchors on each end
Standard row spacers are required in this profile.

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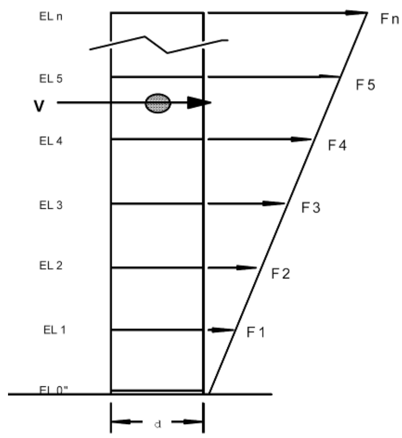
PROJECT: CED
FOR: Interlake-Mecalux_Reyn
ADDRESS: 5041 W 2400 S, Sui
West Valley City, UT
SHEET#: 6
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

Loads and Distributions: CR1

Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the IBC2021



# of Levels:	5	SDC:	D	R_L:	6	Ss:	1.29
Pallets Wide:	2	W_{PL}:	25000	R_T:	4	S1:	0.45
Pallets Deep:	1	W_{DL}:	500 lbs	Fa:	1.2	Ip:	1
Pallet Load:	2500	Fv:	1.85	Tl:	1.5		
Total Frame Load:	25500 lbs						

$S_{DS} = 2/3 \cdot S_s \cdot F_a =$ **1.03**

$S_{D1} = 2/3 \cdot S_1 \cdot F_v =$ **0.56**

$W_s = 0.67 \cdot W_{PL} + W_{DL} =$ **17250 lbs**

Seismic Shear per RMI 2012 2.6.3:

Longitudinal

$V_{long1} = C_s \cdot I_p \cdot W_s$
$= S_{D1} / (T_L \cdot R_L) \cdot I_P \cdot W_s$
$= 0.56 / (1.5 \cdot 6) \cdot 1 \cdot 17250 =$ **1073.33 lbs**

V_{long} need not be greater than:

$V_{long2} = C_s \cdot I_p \cdot W_s$
$= S_{DS} / R_L \cdot I_P \cdot W_s$
$= 1.03 / 6 \cdot 1 \cdot 17250 =$ **2961.25 lbs**

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

$V_{long3} = C_s \cdot I_p \cdot W_s$
$= 0.5 \cdot S_1 / R_L \cdot I_P \cdot W_s$
$= 0.5 \cdot 0.45 / 6 \cdot 1 \cdot 17250 =$ **649.75 lbs**

V_{long} shall not be less than:

$V_{long4} = C_s \cdot I_p \cdot W_s$
$= Max[0.044 \cdot S_{DS}, 0.03] \cdot I_P \cdot W_s$
$= Max[0.05, 0.03] \cdot 1 \cdot 17250 =$ **781.77 lbs**

Since: $1073.33 \leq 2961.25$
& $1073.33 \geq 649.75$
& $1073.33 \geq 781.77$

V_{long} = 1073 lbs

Transverse

V_{trans} need not be greater than:

$V_{trans1} = C_s \cdot I_p \cdot W_s$
$= S_{DS} / R_T \cdot I_P \cdot W_s$
$= 1.03 / 4 \cdot 1 \cdot 17250 =$ **4441.88 lbs**

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

$V_{trans2} = C_s \cdot I_p \cdot W_s$
$= 0.5 \cdot S_1 / R_T \cdot I_P \cdot W_s$
$= 0.5 \cdot 0.45 / 4 \cdot 1 \cdot 17250 =$ **974.62 lbs**

V_{trans} shall not be less than:

$V_{trans3} = C_s \cdot I_p \cdot W_s$
$= Max[0.044 \cdot S_{DS}, 0.5 \cdot S_1 / R_T, 0.03] \cdot I_P \cdot W_s$
$= Max[0.05, 0.06, 0.03] \cdot 1 \cdot 17250 =$ **974.62 lbs**

Since: $4441.88 \geq 974.62$
& $4441.88 \geq 974.62$

V_{trans} = 4441 lbs

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Loads and Distributions: CR1 (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	96	2550	244800	95.38	2550	244800	394.76
2	156	2550	397800	154.99	2550	397800	641.48
3	216	2550	550800	214.6	2550	550800	888.2
4	276	2550	703800	274.21	2550	703800	1134.92
5	336	2550	856800	333.82	2550	856800	1381.64

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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

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

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

# of levels	5
min. # of bays	3
N_c	60
N_b	8
k_c	520 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	177 kip-in/rad
k_{ce}	711 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	2.03 in^4
H	336 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	123 in	5 kip
2	183 in	5 kip
3	243 in	5 kip
4	303 in	5 kip
5	382 in	5 kip

Calculated T = 4.43

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

LRFD Basic Load Combinations: CR1

IBC2021& RMI / ANSI MH 16.1

V_{Trans} = **4,441 lbs**

V_{Long} = **1,073 lbs**

P = Product Load / 2 = **12,500 lbs**

D = Dead Load · 0.5 = **250 lbs**

$M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **1,107,289 in-lbs**

$E_{Trans} = M_{Trans}$ / frame depth = **26,364 lbs**

β = 0.7

β = 1.0 (Uplift combination only)

ρ = 1

S_{DS} = 1.03

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 · 250) + (1.2 · 12,500)= **15,350 lbs**

2. Gravity Load = 1.2 D + 1.4 P + 1.6 L + 0.5 (L_r or S or R)

= (1.2 · 250) + (1.4 · 12,500) + (1.6 · 0) + (0.5 · 0)= **17,800 lbs**

3. Snow/Rain = 1.2D + 0.85P + (0.5L or 0.5W) + 1.6(L_r or S or R)

= (1.2 · 250) + (0.85 · 12,500) + (0.5 · 0) + (1.6 · 0)= **10,925 lbs**

4. Wind Load = 1.2D + 0.85P + 0.5L + 1.0W + 0.5(L_r or S or R)

= (1.2 · 250) + (0.85 · 12,500) + (0.5 · 0) + (1.0 · 0) + (0.5 · 0)= **10,925 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 · 1.03) · 250 + (1.2 + 0.2 · 1.03) · 0.7 · 12,500 + 0.5 · 0 + 1 · 26,364 + 0.2 · 0= **39,018 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 · 1.03) · 250 + (1.2 + 0.2 · 1.03) · 0.7 · 12,500 + 0.5 · 0 + 1 · 0 + 0.2 · 0= **12,653 lbs**

6. Wind Uplift = $0.9D + 0.9P_{app} + 1.0W$

= 0.9 · 250 + 0.9 · 12,500 + 1.0 · 0 = **225 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 · 1.03) · 250 + (0.9 - 0.2 · 1.03) · 1 · 12,500 - 1 · 26,364= **-17,515 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 · 32) + (1.6 · 0) + (0.5 · 0) + (1.4 · 2,500) + (1.4 · 312) = **3,975 lbs**

ASD Load Combinations for Slab Analysis

1. $(1 + 0.105S'_{DS})D + 0.75((1.4 + 0.14S_{DS})\beta P + 0.7\rho E)$

= (1 + 0.105 · 1.03) · 250 + 0.75((1.4 + 0.14 · 1.03) · 0.7 · 12,500 + 0.7 · 1 · 26,364)= **24,251 lbs**

2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$

= (1 + 0.14 · 1.03) · 250 + (0.85 + 0.14 · 1.03) · 0.7 · 12,500 + 0.7 · 1 · 26,364= **27,440 lbs**

3. D + P

= 250 + 12,500 = **12,750 lbs**

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Longitudinal Analysis: CR1

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).

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

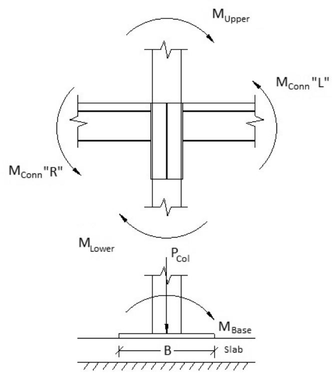
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

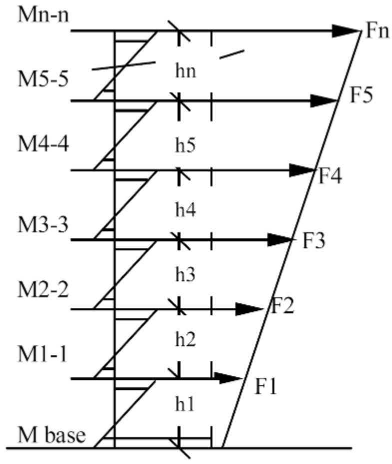
$V_{Col} = V_{Long}$ / # of columns = **537 lbs**

$M_{Base} =$ **8500 in-lbs**

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

(537 lbs · 94 in.) - 8500 in-lbs = 41978 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	96	48	12,750	41,978	7,147	39,459
2	60	77	10,200	22,646	5,551	28,197
3	60	107	7,650	22,646	5,551	28,197
4	60	137	5,100	22,646	5,551	28,197
5	60	167	2,550	22,646	5,551	16,874

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COLUMN WITH BACKER ANALYSIS: CR1 (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

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

$K_x \cdot L_x / R_x$ $= 1.7 \cdot 94 / 1.489$ $=$ **107.32**

$K_y \cdot L_y / R_y$ $= 1 \cdot 24 / 1.652$ $=$ **14.53**

KL/R_{max} $=$ **107.32**

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

$= (1.489^2 + 1.652^2 + -2.124^2)^{1/2} =$ **3.075 in.**

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

$= 1 - (-2.124/3.075)^2 =$ **0.523**

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

$= 3.14^2 \cdot 29500 / 107.32^2 =$ **25.279 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)

$= (1 / (2 \cdot 0.523)((25.279 + 250.763) - (25.279 + 250.763)^2$

$- (4 \cdot 0.523 \cdot 25.279 \cdot 250.763))^{1/2}) =$ **24.061 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 107.32^2 =$ **25.279 ksi**

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

$= 1 / 1.829 \cdot 3.075^2 (11300 \cdot 0.005$

$+ (3.142 \cdot 29500 \cdot 5.421) / (0.8 \cdot 24)^2) =$ **250.763 ksi**

$F_e = \text{Min}(F_{e1}, F_{e2}) =$ **24.061 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 24.061)^{1/2} =$ **1.512** (Eq C4.1-4)

Since $\lambda_c \geq 1.5$:

$F_n = (0.877 / \lambda_c^2) \cdot F_y =$ **21.101** (Eq C4.1-3)

Thus:

$P_n =$ **30660 lbs**

$P_a =$ **26061 lbs**

3.98 x 2.72 - .089	
SECTION PROPERTIES	
Depth	**5.438 in.**
Width	**3.984 in.**
t	**0.089 in.**
Radius	**0.089 in.**
Area	**1.829 in.^{2}**
AreaNet	**1.453 in.^{2}**
I_x	**4.054 in.^{4}**
S_x	**2.035 in.^{3}**
$S_{x\ Net}$	**1.743 in.^{3}**
R_x	**1.489 in.**
I_y	**4.99 in.^{4}**
S_y	**1.595 in.^{3}**
R_y	**1.652 in.**
J	**0.005 in.^{4}**
C_w	**5.421 in.^{6}**
J_x	**2.463 in.**
X_o	**-2.124 in.**
K_x	1.7
L_x	**94 in.**
K_y	1
L_y	**24 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

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ADDRESS: 5041 W 2400 S, Sui
West Valley City, UT
SHEET#: 12
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

COLUMN WITH BACKER ANALYSIS: CR1 (Level 1)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

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

$P_{ao} = P_{no}\phi_c$ = **67927 lbs**

Where:

$P_{no} = A_e F_y$ = 1.453 · 55 = **79915 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f$ = **219.991 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f)$ = **171.045 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2$ = **6740.893 ksi** (Eq 3.1.2.1-10)

$F_{e.min}$ = **171.045 ksi**

Since: $F_e \geq 2.78F_y$

$F_c = (S_e / S_c)$

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

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c$ = **55 ksi**

M_{nx} = **95842 in-lbs** M_{ny} = **87735 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b$ = **86258 in-lbs** $M_{ny}\phi_b$ = **78962 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2$ = **46224 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2$ = **2522566 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex}))$ = **0.751** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey}))$ = **0.995** (Eq C5.2.2-5)

P_{trans} = **39,018 lbs** P_{long} = **12,653 lbs**

$M_u = M_x$ = **41946 in-lbs** (Eq C5.2.2-2)

P_{u_st} = (1.2 · D) + (1.4 · P) = **17800 lbs**

P_{u_st} / P_a = 17800 / 26061 = 0.68 Static Stress = **68%**

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)

= ((12,653 / 26061) + (41946 / 86258) + (1 / 78962)) = **97%**

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)

= (12,653 / 67927) + (0.85 · 41946 / 86258 · 0.751)) + (0.85 · 1 / 78962 · 0.995))) = **73%**

Stress3 P_t / P_{ao} = 39,018 / 67927 =**56%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **97%**

3.98 x 2.72 - .089	
SECTION PROPERTIES	
Depth	**5.438 in.**
Width	**3.984 in.**
t	**0.089 in.**
Radius	**0.089 in.**
Area	**1.829 in. ^{2}**
AreaNet	**1.453 in. ^{2}**
I_x	**4.054 in. ^{4}**
S_x	**2.035 in. ^{3}**
$S_{x\ Net}$	**1.743 in. ^{3}**
R_x	**1.489 in.**
I_y	**4.99 in. ^{4}**
S_y	**1.595 in. ^{3}**
R_y	**1.652 in.**
J	**0.005 in. ^{4}**
C_w	**5.421 in. ^{6}**
J_x	**2.463 in.**
X_o	**-2.124 in.**
K_x	1.7
L_x	**94 in.**
K_y	1
L_y	**24 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

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COLUMN ANALYSIS: CR1 (Level 2)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

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

$K_x \cdot L_x / R_x$ = 1.5 · 58 / 1.489 = **58.43**

$K_y \cdot L_y / R_y$ = 1 · 24 / 0.939 = **25.57**

KL/R_{max} = **58.43**

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

$= (1.489^2 + 0.939^2 + -2.124^2)^{1/2} =$ **2.758 in.**

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

$= 1 - (-2.124/2.758)^2 =$ **0.407**

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

$= 3.14^2 \cdot 29500 / 58.43^2 =$ **85.285 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)

$= (1 / (2 \cdot 0.407)((85.285 + 311.691) - (85.285 + 311.691)^2$

$- (4 \cdot 0.407 \cdot 85.285 \cdot 311.691))^{1/2}) =$ **72.329 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 58.43^2 =$ **85.285 ksi**

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

$= 1 / 0.914 \cdot 2.758^2(11300 \cdot 0.002$

$+ (3.142 \cdot 29500 \cdot 2.711) / (0.8 \cdot 24)^2) =$ **311.691 ksi**

F_e = Min(F_{e1}, F_{e2}) = **72.329 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 72.329)^{1/2} =$ **0.872** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\text{^}(\lambda_c^2)) \cdot F_y =$ **40.007** (Eq C4.1-2)

Thus:

P_n = **26882 lbs**

P_a = **22850 lbs**

3.98 x 2.72 - .089	
SECTION PROPERTIES	
Depth	**2.719 in.**
Width	**3.984 in.**
t	**0.089 in.**
Radius	**0.089 in.**
Area	**0.914 in.^{2}**
AreaNet	**0.727 in.^{2}**
I_x	**2.027 in.^{4}**
S_x	**1.018 in.^{3}**
$S_{x\,Net}$	**0.871 in.^{3}**
R_x	**1.489 in.**
I_y	**0.805 in.^{4}**
S_y	**0.455 in.^{3}**
R_y	**0.939 in.**
J	**0.002 in.^{4}**
C_w	**2.711 in.^{6}**
J_x	**2.55 in.**
X_o	**-2.124 in.**
K_x	1.5
L_x	**58 in.**
K_y	1
L_y	**24 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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West Valley City, UT
SHEET#: 14
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

COLUMN ANALYSIS: CR1 (Level 2)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

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

$P_{ao} = P_{no}\phi_c =$ **31413 lbs**

Where:

$P_{no} = A_e F_y = 0.672 \cdot 55 =$ **36956 lbs**

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

$F_e = C_b r_o A (\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **404.077 ksi**

$F_e = C_s A \sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **253.449 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 =$ **1087.892 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **253.449 ksi**

Since: $F_e \geq 2.78 F_y$

$F_c = (S_e / S_c)$

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

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **52.2 ksi**

$M_{nx} =$ **45481 in-lbs** $M_{ny} =$ **23766 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **40933 in-lbs** $M_{ny}\phi_b =$ **21389 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2 =$ **77975 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2 =$ **407109 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.882** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.977** (Eq C5.2.2-5)

$P_{trans} =$ **26,336 lbs** $P_{long} =$ **10,123 lbs**

$M_u = M_x =$ **13239 in-lbs** (Eq C5.2.2-2)

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

$P_{u\_st} / P_a = 14240 / 22850 = 0.62$ Static Stress = **62%**

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)

= ((10,123 / 22850) + (13239 / 40933) + (1 / 21389)) = **76%**

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)

= (10,123 / 31413) + (0.85 · 13239 / 40933 · 0.882)) + (0.85 · 1 / 21389 · 0.977))) = **63%**

Stress3 P_t / P_{ao} = 26,336 / 31413 =**83%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **83%**

3.98 x 2.72 - .089	
SECTION PROPERTIES	
Depth	**2.719 in.**
Width	**3.984 in.**
t	**0.089 in.**
Radius	**0.089 in.**
Area	**0.914 in.** ^{2}
AreaNet	**0.727 in.** ^{2}
I_x	**2.027 in.** ^{4}
S_x	**1.018 in.** ^{3}
$S_{x\ Net}$	**0.871 in.** ^{3}
R_x	**1.489 in.**
I_y	**0.805 in.** ^{4}
S_y	**0.455 in.** ^{3}
R_y	**0.939 in.**
J	**0.002 in.** ^{4}
C_w	**2.711 in.** ^{6}
J_x	**2.55 in.**
X_o	**-2.124 in.**
K_x	1.5
L_x	**58 in.**
K_y	1
L_y	**24 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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SHEET#: 15
CALCULATED BY: banjarjian
DATE: 7/10/2024
PN: 20240710_9

BEAM ANALYSIS CR1

Determine allowable bending moment per AISI

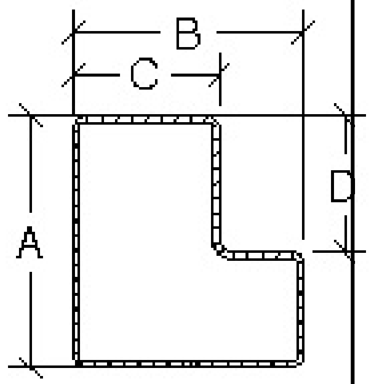
Check compression flange for local buckling (B2.1)

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

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

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

λ <= 0.673: Flange is fully effective.



Check web for local buckling (B2.3)

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

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

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

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

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

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

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

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

b2 = w/2 = **1.85**

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

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

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

L_f = effective width = **1.452**

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

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

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

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

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

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

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

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

4.00 x 2.75 -0.059

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

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DATE: 7/10/2024
PN: 20240710_9

BEAM ANALYSIS CR1

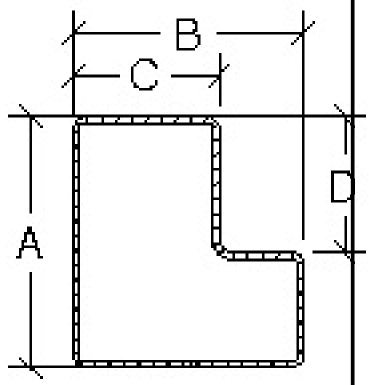
Check Allowable Tension Stress for Bottom Flange

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

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

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

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



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

Check Bending Capacity

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

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

For DL = 2% of PL:

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

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

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

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

4.00 x 2.75 -0.059

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

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

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

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

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

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

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Allowable and Actual Bending Moment at Each Level

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

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	30,576	35,436	17,370	35,436	Pass
2	30,576	32,580	8,233	32,580	Pass
3	30,576	32,580	5,461	32,580	Pass
4	30,576	32,580	3,313	32,580	Pass
5	30,576	32,580	2,776	32,580	Pass

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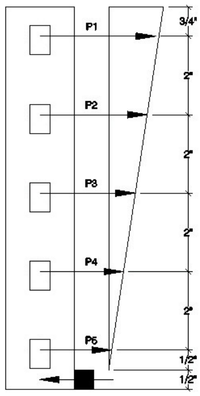
Beam to Column Analysis: CR1

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 = **0.09 in.**

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

$R_n = 1.8 \cdot F_y \cdot A_{pb} =$ **5310.36 lbs** AISC J7 -1

$P_{Bearing} = \phi \cdot R_n = 0.75 \cdot 5310.36 =$ **3982 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**

$C =$ **2.65**

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

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

$P_{Clip} = M_{Strength} / (c \cdot d) =$ **4749 lbs**

Minimum Value of P1 Governs

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

$M_{Conn\text{-}Allow} = (P_1 \cdot 8.5) + (P_1 \cdot (6.5 / 8.5) \cdot 6.5) + (P_1 \cdot (4.5 / 8.5) \cdot 4.5) + (P_1 \cdot (2.5 / 8.5) \cdot 2.5) + (P_1 \cdot (0.5 / 8.5) \cdot 0.5) =$
$=$ **39965.44 in-lbs**

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SHEET#: 19
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DATE: 7/10/2024
PN: 20240710_9

BRACE ANALYSIS CR1 (Panel 9)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

2.756 x 1.378 - .06

SECTION PROPERTIES	
Depth	2.756 in.
Width	1.378 in.
t	0.06 in.
Radius	0.09 in.
Area	0.317 in^2
AreaNet	0.317 in^2
Ix	0.381 in^4
Sx	0.276 in^3
Sx net	0.276 in^3
Rx	1.095 in.
Iy	0.061 in^4
Sy	0.06 in^3
Ry	0.437 in.
J	0 in^4
Cw	0.074 in^6
Jx	1.615 in.
Xo	-0.862 in.
Kx	0
Lx	59 in.
Ky	1
Ly	59 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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

$K_x \cdot L_x / R_x = 0 \cdot 59 / 1.1 = \mathbf{53.88}$
$K_y \cdot L_y / R_y = 1 \cdot 59 / 0.44 = \mathbf{135.1}$
$KL / R_{max} = \mathbf{135.1}$

$r_o = (r_x^2 + r_y^2 + x_o^2)^{1/2}$
$= (1.1^2 + 0.44^2 + -0.86^2)^{1/2} = \mathbf{1.46\ in.}$ (Eq C3.1.2.1-7)

$\beta = 1 - (x_o / r_o)^2 = 1 - (-0.86 / 1.46)^2 = \mathbf{0.65}$ (Eq C4.1.2-3)

$F_{e1} = \Pi^2 E / (KL / r)_{max}^2 = 3.14^2 \cdot 29500 / 135.1^2 = \mathbf{15.951\ ksi}$ (Eq C4.1.1-1)

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

$= (1 / (2 \cdot 0.65)((100.29 + 15.85) - ((100.29 + 15.85)^2$
$- (4 \cdot 0.65 \cdot 100.29 \cdot 15.85))^{1/2}) = \mathbf{14.941\ ksi}$ (Eq C4.1.2-1)

where:
$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$
$= 3.14^2 \cdot 29500 / 135.1^2 = \mathbf{100.287\ ksi}$ (Eq C3.1.2-11)

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$
$= 1 / 0.32 \cdot 1.46^2 (11300 \cdot 0.0004$
$+ (3.14^2 \cdot 29500 \cdot 0.07) / (0.8 \cdot 59)^2) = \mathbf{15.853\ ksi}$ (Eq C3.1.2-9)

$F_e = \text{Min}(Fe1, Fe2) = \mathbf{14.941\ ksi}$

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

$\lambda_c = (F_y / F_e)^{1/2} = (50 / 14.941)^{1/2} = \mathbf{1.829}$
Since $\lambda_c >= 1.5$, $F_n = (0.877 / (\lambda_c^2)) \cdot F_y = \mathbf{13.104}$ (Eq C4.1-4)

Thus (Eq C4.1-3)
$P_n = \mathbf{4,158\ lbs}$
$P_a = P_n \cdot \phi_c = \mathbf{3,534\ lbs}$

BRACE ANALYSIS CR1 (Panel 9)

Analyzed per RMI, AISI 2012 (LRFD) and the IBC2021.

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

$P_{ao} = P_{no}\phi_c = 15{,}865 \cdot 0.85 =$ **13,485 lbs.**

Where $P_{no} = A_eF_y = 0.32 \cdot 50 =$ **15,865 lbs.**

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

$F_e = C_br_oA(\sigma_{ey}\sigma_t)^{1/2} / S_f = 66.91$ ksi

$F_e = C_sA\sigma_{ex}(j + C_s(j^2 + r_o{}^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF}S_f) = 11.66$ ksi (Eq C3.1.2.1-4)

$F_e = (C_b\Pi^2EdI_{yc}) / (S_f(K_yL_y)^2 = 50.51$ ksi (Eq C3.1.2.1-10)

$F_{e,min} =$ **11.66 ksi** (Eq C3.1.2.1-14)

Since, $F_e <= 0.56F_y$

$F_c = F_e =$ **11.66 ksi** (Eq C3.1.2.1-3)

reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **11.7 ksi**

$M_{nx} =$ **3,230 in-lbs** $M_{ny} =$ **706 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **2,907 in-lbs** $M_{ny}\phi_b =$ **635 in-lbs**

$P_{Ex} = \Pi^2EI_x / (K_xL_x)^2 =$ **31,825 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2EI_y / (K_yL_y)^2 =$ **5,060 lbs** (Eq C5.2.2-7)

Max $P_a =$ **4,158 lbs**

$V_{Trans} =$ **1,948 lbs**

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} =$ **59.2 in.**

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D =$ **3153.55 lbs.**

Brace Stress = $V_{Diag} / P_a =$ **89%**

2.756 x 1.378 - .06

SECTION PROPERTIES	
Depth	2.756 in.
Width	1.378 in.
t	0.06 in.
Radius	0.09 in.
Area	0.317 in^2
AreaNet	0.317 in^2
Ix	0.381 in^4
Sx	0.276 in^3
Sx net	0.276 in^3
Rx	1.095 in.
Iy	0.061 in^4
Sy	0.06 in^3
Ry	0.437 in.
J	0 in^4
Cw	0.074 in^6
Jx	1.615 in.
Xo	-0.862 in.
Kx	0
Lx	59 in.
Ky	1
Ly	59 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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PN: 20240710_9

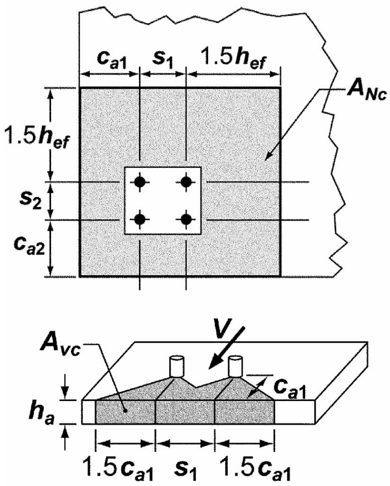
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-19(ACI 318-14), CHAPTER 17 Configuration 1 CR1

Assumed cracked concrete application

Anchor Type	**0.625" dia., 4 hef, 6" min, slab**				
ICC Report Number	**ESR-4266**			$1.5 \cdot h_{ef}$	= **6 in.**
Slab Thickness (h)	= **6 in.**	C_{a1}	= 12	use$C_{a1,adj}$	= **6 in.**
Min. Slab Thickness (h)	= **6 in.**	C_{a2}	= 12	use$C_{a2,adj}$	= **6 in.**
Concrete Strength (f_c)	= **4000 psi**				
Diameter (d_a)	= **0.625 in.**			$3 \cdot h_{ef}$	= **12 in.**
Nominal Embedment (h_{nom})	= **4.5 in.**				
Effective Embedment (h_{ef})	= **4 in.**	S_1	= **9.5 in.**	Use $S_{1,adj}$	= **9.5 in.**
Number of Anchors (n)	= **7**	S_2	= **6 in.**	Use $S_{2,adj}$	= **6 in.**
e\`N	= **0**				
e\`V	= **0**				

From ICC ESR Report

A_{se}	= **0.164 sq.in.**
$f`_{uta}$	= **106700 psi**
S_{min}	= **2.25 in.**
C_{min}	= **2.75 in.**
C_{ae}	= **9 in.**
$N_{p,cr}$	= **9999 lbs**



	$\phi_{Seismic}$	Adj. Strength
Tension Capacity = **3756 lbs**	0.75	**2817 lbs**
Shear Capacity = **6665 lbs**	0.75	**4999 lbs**

ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 CR1

Steel Strength	17.4.1
ϕ = **0.75**	17.3.3.a i
$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 7 \cdot 0.164 \cdot 106700 =$ **91,869 lbs**	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
ϕ = **0.65**	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) =$ **387 sq.in.**	
$A_{Nco} = 9h_{ef}^2 =$ **144 sq.in.**	
Check if $A_{Nco} \geq A_{Nc}$ $A_{Nc}/A_{Nco} =$ **2.688**	
$\Psi_{ec,N}$ = **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**	
$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} =$ **8601 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
$0.65 \cdot (387/144) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 8601 =$ **26,294 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
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} =$ **57,548 lbs**	17.4.3.1

Steel Strength (ϕN_{sa}) = **91,869 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **26,294 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **57,548 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 CR1

Steel Strength ϕV_{sa} V_{sa}=10,255 / Anchor -- per report — 17.5.1

ϕ = **0.65** — 17.3.3. Condition a ii

$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 7 \cdot 10{,}255 =$ **46,660 lbs** — 17.5.1.2a

Concrete Breakout Strength ϕV_{cbg} — 17.5.2

ϕ = **0.7** — 17.3.3 ci-B

$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **273 sq.in.**

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

Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1.264**

$\Psi_{ec,V}$ = **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 = **0.625 in.** — 17.5.2.2

L_e = **1.25 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 $9\lambda_a(f_c)^{0.5}ca1^{1.5}$ = **16,713 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

$0.7 \cdot (273/216) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.732 \cdot 16{,}713 =$ **92,197 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} = **40,452 lbs**

$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 40{,}452 =$ **56,633 lbs**

Steel Strength (ϕV_{sa}) = **46,660 lbs**

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **92,197 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **56,633 lbs**

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OVERTURNING ANALYSIS Configuration1 CR1

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} = **25,000 lbs** W_{dl} = **500 lbs**

$W_{pl} \cdot 67\% = 25{,}000 \cdot 0.67 =$ **16,750 lbs**

$V_{Trans} = (1 \cdot 0.2575 \cdot 1 \cdot ((0.67 \cdot 16{,}750) + 500)) =$ **3,018 lbs**

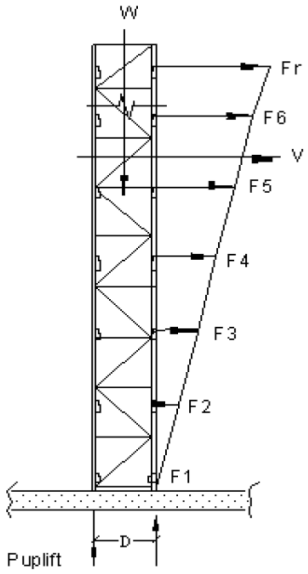
$M_{ovt} = V_{Trans} \cdot Ht = 3{,}018 \cdot 282 =$ **851,076 in-lbs**

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

$= ((25{,}000 \cdot 0.67) + 500) \cdot 42 \cdot 0.5 =$ **362,250 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (851{,}076 - 362{,}250) / 42 =$ **11,638 lbs**

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (851{,}076 + 362{,}250) / 42 =$ **28,888 lbs**



CROSS AISLE ELEVATION

TOP SHELF LOADED

Shear = **1,416 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 1{,}416 \cdot (336 + ((60 - 10) / 2)) =$ **511,176 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (5{,}000 + 500) \cdot (42 \cdot 0.5) =$ **115,500 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (511{,}176 - 115{,}500) / 42 =$ **9,420 lbs**

SIDE LOADS ON TOP SHELF

Top loaded shelf level (H) = **336 in.**

$M_{ovt} = 1.6 \cdot 350 \cdot H = 1.6 \cdot 350 \cdot 336 =$ **188kip**

$P_{uplift} = M_{ovt}/d = 188 / 42 =$ **4,480 lbs**

ANCHORS

No. of Anchors (#Anchors): **7**

Pull Out Capacity per Anchor (T_{Anchor}): **2,817 lbs**

Shear Capacity per Anchor: **4,999 lbs**

COMBINED STRESS

Fully Loaded	= ((11,638 / 7) / 2,817) + ((3,018 / 14) / 4,999)	= **0.633**
Top Shelf Loaded	= ((9,420 / 7) / 2,817) + ((1,416 / 14) / 4,999)	= **0.498**
Seismic UpLift Critical (LC#7B)	= (17,516 / 7) / 2,817	= **0.888**
Side Load Top Shelf	= **0.227**	

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Base Plate Analysis: CR1

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} = **39018 lbs**

M_{Base} = **8500 in-lbs**

$P/A = P_{col}/(D \cdot B) = 39018 / (23 \cdot 12) =$ **141 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 8500 / (23 \cdot 12^2 / 6) =$ **15.4 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 15.4 \cdot (2 \cdot 4.01/12) =$ **10.29 psi**

$f_{b1} = f_b - f_{b2} = 15.4 - 10.29 =$ **5.11 psi**

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

$= (4.01^2 / 2) \cdot (141 + 5.11 + 0.67 \cdot 10.29) =$ **1231.63 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.5sq.in.**

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 1231.63 / (0.5 \cdot 32,400) =$ **0.08**

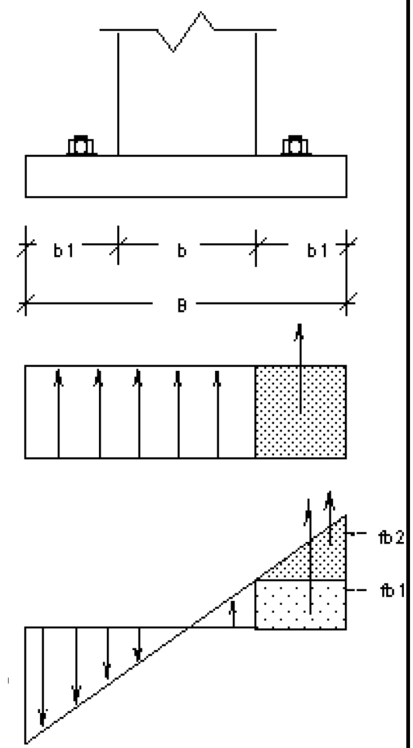

Plate width B =	12 in.
Plate depth D =	23 in.
Plate thickness t =	0.5 in.
Column width b =	3.98 in.
Column depth d =	2.72 in.
b1 =	4.01 in.

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Equation for Maximum Considered Earthquake Base Rotation

Per RMI 2012 Commentary 2.6.4

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

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

# of levels	5
min. # of bays	3
N_c	60
N_b	8
k_c	520 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	177 kip-in/rad
k_{ce}	711 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	2.03 in^4
H	336 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	123 in	5 kip
2	183 in	5 kip
3	243 in	5 kip
4	303 in	5 kip
5	382 in	5 kip

Where:

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

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

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

α_s = 1.92

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= 0.6

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}=80,285 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) = **39,018 lbs**

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

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **4.095 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **25.491 in.**

$b,req'd$ = $1.5 \cdot b$ = **38 in.**

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **139,575 lbs**

P_a = $\phi \cdot P_a$ = **83,745 lbs**

P_{max} / P_a = **0.47**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **27,440 lbs**

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

P_n = $1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **139,575 lbs**

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **4.095 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **25.491 in.**

$b,req'd$ = $1.5 \cdot b$ = **38 in.**

P_a = P_n / Ω = **46,525 lbs**

P_{max} / P_a = **0.59**

Base Plate

Width B	12 in.
Depth W	23 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	8.31 in
E_c	3,604,997 psi