PRCTI20221624

SEIZMIC INC

MATERIAL HANDLING ENGINEERING

EST. 1985

SPECIAL PRODUCTS	*CONVEYORS*	*STORAGE RACKS*	*OTHER SERVICES*	*SHELVING*	*SPECIAL PRODUCTS*
TANK SUPPORTS	TALL SUPPORTS	SELECTIVE	SEISMIC ANALYSIS	METAL	SHUTTLES
MACHINERY	HEADER STEEL	DRIVE-IN	PERMIT AQUISITION	METAL/WOOD	VLM VRC
RACK BLDGS	ROBOTIC PLATFORMS	PUSH BACK	EGRESS PLANS	MOVABLE	CAROUSELS
SHEDS	PICK MODULES	FLOW RACK	STATE APPROVALS	GONDOLAS	FENCING SYSTEMS
WORK PLATFORMS	ROOF VERIFICATION	CANTILEVER	PRODUCT TESTING	LOCKERS	MODULAR OFFICES
FOOTINGS			TITLE 24	CATWALKS	MINI-LOAD SYSTEMS

LICENSED IN 50 STATES

THE APPROVED CONSTRUCTION PLANS, DOCUMENTS AND ALL ENGINEERING MUST BE POSTED ON THE JOB AT ALL INSPECTIONS IN A VISIBLE AND READILY ACCESSIBLE LOCATION.

FULL SIZED LEDGIBLE COLOR PLANS ARE REQUIRED TO BE PROVIDED BY THE PERMITEE ON SITE FOR INSPECTION

City of Puyallup
Building
ACCEPTED
JMontgomery
11/04/2022
11:22:28 AM
CITY OF PUYALLUP STATE OF WASHINGTON

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

ANALYSIS OF

STORAGE RACKS

FOR

Red Dot Corp. WA

2504 E. Main Avenue, Puyallup, WA

Job No. 22-2726

Approved by:

SAL E. FATEEN, P.E.

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

SEIZMIC Engineering

Digitally signed by Sal Fateen
Date: 2022.10.14 09:52:54-07'00'

EXPIRES
04/12/2024

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

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PRCTI20221624

PROJECT: Red Dot Corp. WA
FOR: Raymond West_Jack Murp
ADDRESS: 2504 E. Main Avenu
Puyallup, WA
SHEET#: 1
CALCULATED BY: ang
DATE: 10/13/2022
PN: 20220808_22

Table of Contents

Scope:

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

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

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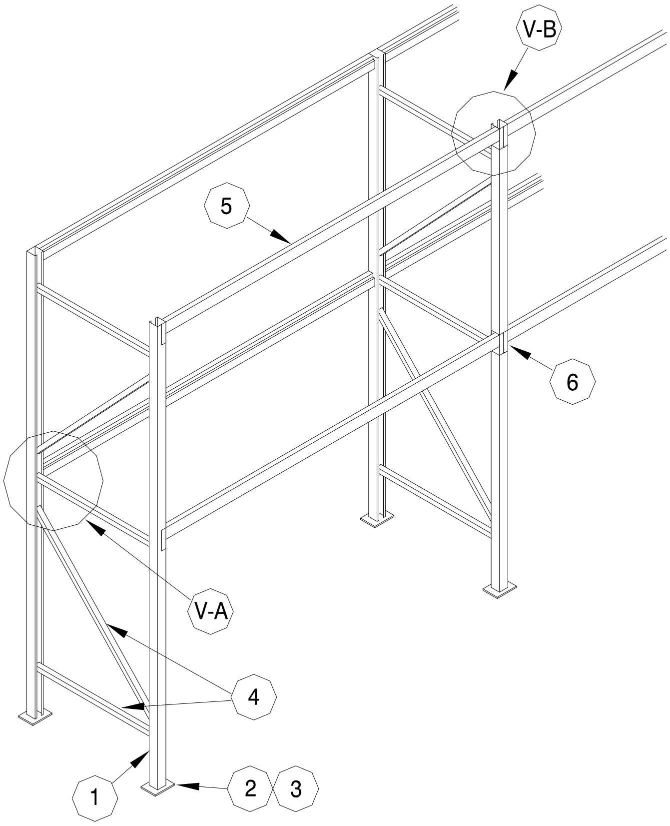
DATE: 10/13/2022
PN: 20220808_22

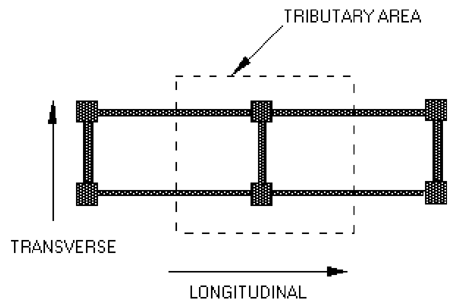
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being comon between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction 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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CALCULATED BY: ang
DATE: 10/13/2022

PN: 20220808_22

COMPONENTS AND SPECIFICATIONS Configuration 1: Type A - 12" Flue L 1.7

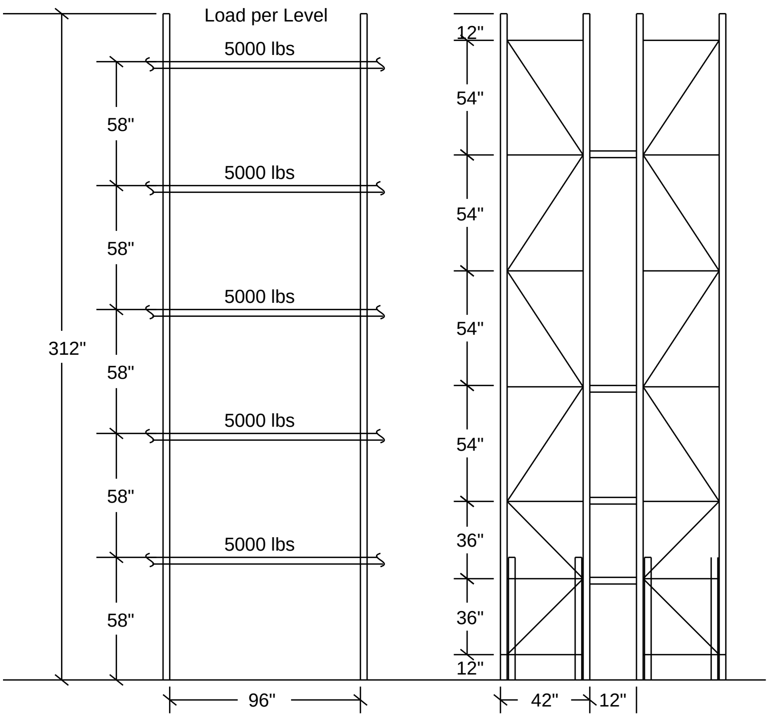
Analysis per section 2209 of the 2018 IBC

Levels: 5 Panels: 6

S_s = 1.26 F_a = 1.2 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1035 lbs.
V_{Trans} = 4355 lbs.

P_{static} = 12750 lbs.



FRONT VIEW SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 92% (level 3) **BACKER TO LEVEL 1** 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 70% (level 1) **HORIZONTAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 43% (panel 2) **DIAGONAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 73% (panel 3)	4.13 x 2.5 - .060 (SSB416M) Steel = 55 ksi Max Static Cap. = 5458 lb. Stress = 93% Max stress = 97% (level 2)	**Level 1** 4 Pin 2" cc Connector Stress = 64% **Level2** 3 Pin 2" cc Connector Stress = 68% Max stress = 68% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 8 x 8 x 0.375 in. 4 anchors/plate Moment = 0 in-lb. Stress = 28%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 44% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2359 lbs. Shear Capacity = 4999 lbs. Anchor stress = 81%

Notes:
1.5 x 1.5" x 14 GA tube brace at diagonals 1-4 and horizontals 1-3. Standard brace at all others.

This calculation summary applies only to back-to-back rows with frame ties.
Use (4) frame ties per upright. Minimum Sx is 1.0 cu. in. C3 x 3.5#.
Attach to sides of column with 1/8" thk. saddle and (2) 7/16" A449 bolts at 4" o.c.

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SHEET#: 4
CALCULATED BY: ang
DATE: 10/13/2022

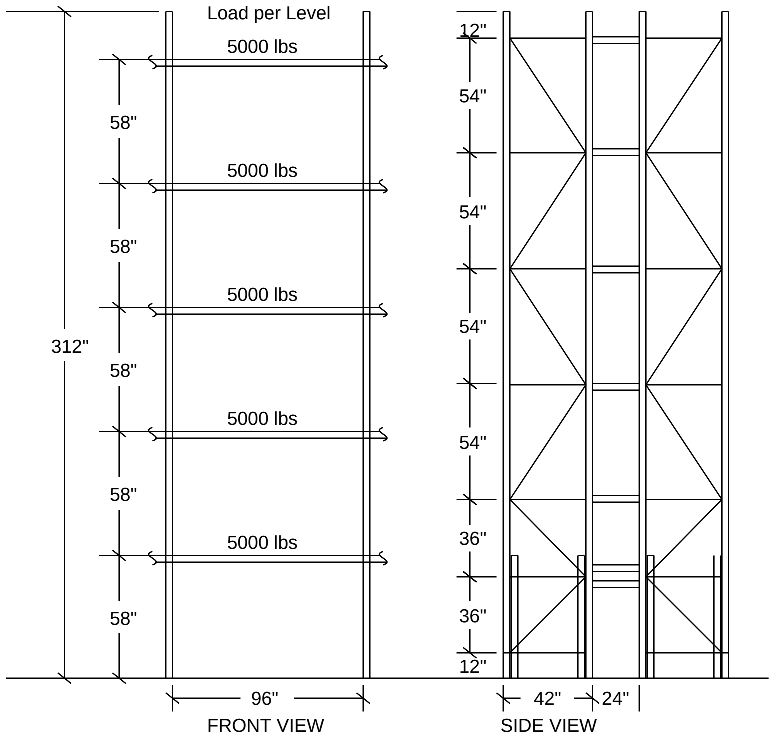
PN: 20220808_22

COMPONENTS AND SPECIFICATIONS   Configuration 2: Type A - 24" Flue   L 1.7

Analysis per section 2209 of the 2018 IBC

S_s = 1.26 F_a = 1.2 I = 1 V_{Long} = 1035 lbs. P_{static} = 12750 lbs.

Levels: 5 Panels: 6 S_1 = 0.43 F_v = 1.87 SDC = D V_{Trans} = 4355 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 92% (level 3) **BACKER TO LEVEL 1** 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 70% (level 1) **HORIZONTAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 43% (panel 2) **DIAGONAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 73% (panel 3)	4.13 x 2.5 - .060 (SSB416M) Steel = 55 ksi Max Static Cap. = 5458 lb. Stress = 93% Max stress = 97% (level 2)	**Level 1** 4 Pin 2" cc Connector Stress = 64% **Level2** 3 Pin 2" cc Connector Stress = 68% Max stress = 68% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 8 x 8 x 0.375 in. 4 anchors/plate Moment = 0 in-lb. Stress = 28%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 44% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2359 lbs. Shear Capacity = 4999 lbs. Anchor stress = 92%

Notes:
1.5 x 1.5" x 14 GA tube brace at diagonals 1-4 and horizontals 1-3. Standard brace at all others.

This calculation summary applies only to back-to-back rows with frame ties.
Use (7) frame ties per upright. Minimum Sx is 1.0 cu. in. C3 x 3.5#.
Attach to sides of column with 1/8" thk. saddle and (2) 7/16" A449 bolts at 4" o.c.

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CALCULATED BY: ang
DATE: 10/13/2022

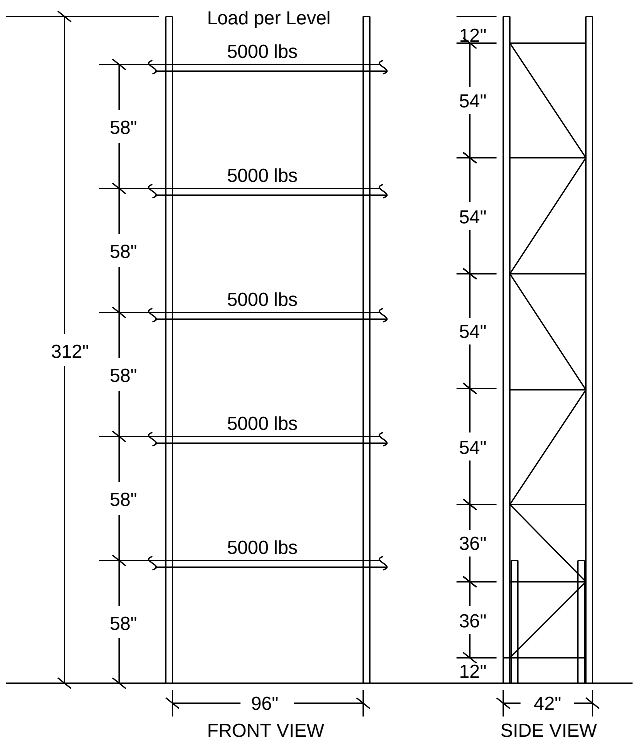
PN: 20220808_22

COMPONENTS AND SPECIFICATIONS   Configuration 3: Type A1   L 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 5   Panels: 6

$S_s = 1.26$ $F_a = 1.2$ $I = 1$ $V_{Long} = 1035$ lbs. $P_{static} = 12750$ lbs.
$S_1 = 0.43$ $F_v = 1.87$ SDC = D $V_{Trans} = 4355$ lbs. $P_{seismic} = 16838$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 92% (level 3) **BACKER TO LEVEL 1** 3 x 3 - .089 (LM25) Steel = 55000 psi Stress = 70% (level 1) **HORIZONTAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 43% (panel 2) **DIAGONAL BRACE** 1.5 x 1.5 -0.075 (Tube) Stress = 95% (panel 3)	4.13 x 2.5 - .060 (SSB416M) Steel = 55 ksi Max Static Cap. = 5458 lb. Stress = 93% Max stress = 97% (level 2)	**Level 1** 4 Pin 2" cc Connector Stress = 64% **Level2** 3 Pin 2" cc Connector Stress = 68% Max stress = 68% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 10 x 10 x 0.375 in. 6 anchors/plate Moment = 0 in-lb. Stress = 28%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 42% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2330 lbs. Shear Capacity = 4999 lbs. Anchor stress = 94%

Notes:
1.5 x 1.5" x 14 GA tube brace at diagonals 1-5 and horizontals 1-3. Standard brace at all others.

(6) 5/8" anchors per 10 x 10" base plate at 8" & 4" o/c.

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COMPONENTS AND SPECIFICATIONS Configuration 4: Type B L 1.7

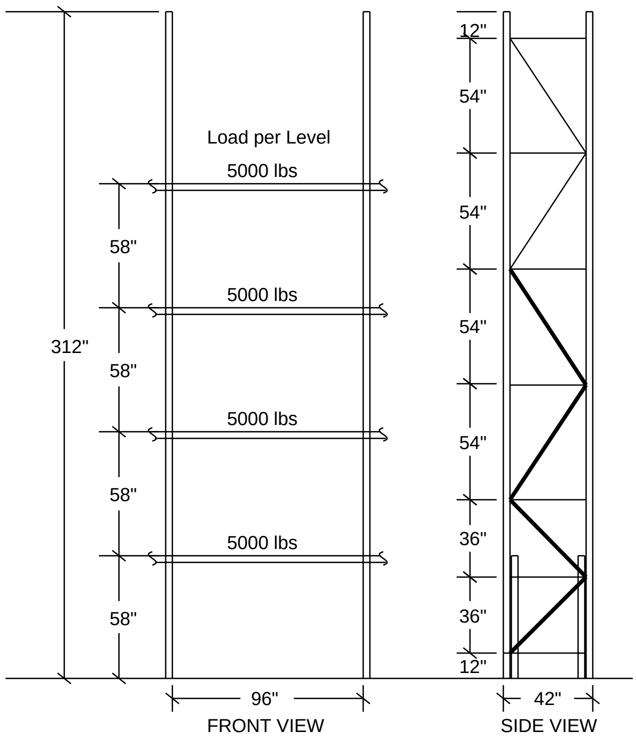
Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 6

S_s = 1.26 F_a = 1.2 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 828 lbs.
V_{Trans} = 3484 lbs.

P_{static} = 10200 lbs.
$P_{seismic}$ = 11271 lbs.



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

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 8 x 8 x 0.375 in. 4 anchors/plate Moment = 0 in-lb. Stress = 20%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 31% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.625 in. x 4.5 in. Embed. Pullout Capacity = 2359 lbs. Shear Capacity = 4999 lbs. Anchor stress = 78%

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

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Loads and Distributions: Type A - 12" Flue

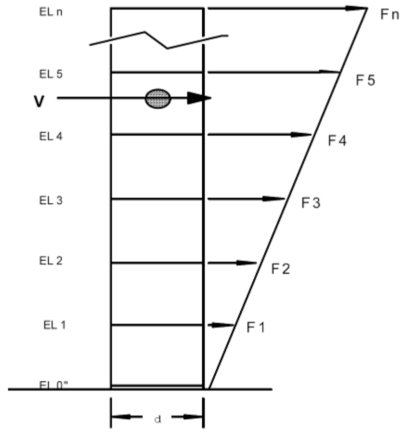
Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels:	5	SDC:	D	R_L:	6	Ss:	1.26
Pallets Wide:	2	W_{PL}:	25000	R_T:	4	S1:	0.43
Pallets Deep:	1	W_{DL}:	500 lbs	Fa:	1.2	Ip:	1
Pallet Load:	2500	Fv:	1.87	Tl:	1.5		
Total Frame Load:	25500 lbs						

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

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

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



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

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

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

$= 0.54 / (1.5 \cdot 6) \cdot 1 \cdot 17250 =$ **1035 lbs**

V_{long} need not be greater than:

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

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

$= 1.01 / 6 \cdot 1 \cdot 17250 =$ **2903.75 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 6 \cdot 1 \cdot 17250 =$ **622.44 lbs**

V_{long} shall not be less than:

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

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

$= \mathrm{Max}[0.04, 0.04, 0.03] \cdot 1 \cdot 17250 =$ **766.59 lbs**

Since: $1035 \leq 2903.75$
& $1035 \geq 622.44$
& $1035 \geq 766.59$

V_{long} = 1035 lbs

Transverse

V_{trans} need not be greater than:

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

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

$= 1.01 / 4 \cdot 1 \cdot 17250 =$ **4355.63 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 4 \cdot 1 \cdot 17250 =$ **933.66 lbs**

V_{trans} shall not be less than:

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

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

$= \mathrm{Max}[0.04, 0.05, 0.03] \cdot 1 \cdot 17250 =$ **933.66 lbs**

Since: $4355.63 \geq 933.66$
& $4355.63 \geq 933.66$

V_{trans} = 4355 lbs

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Loads and Distributions: Type A - 12" Flue (Page 2)

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

		Longitudinal			Transverse		
Level	h_x	w_x	$w_x h_x$	f_i	w_x	$w_x h_x$	f_i
1	58	2550	147900	69	2550	147900	290.33
2	116	2550	295800	138	2550	295800	580.67
3	174	2550	443700	207	2550	443700	871
4	232	2550	591600	276	2550	591600	1161.33
5	290	2550	739500	345	2550	739500	1451.67

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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	400 kip-in/rad
k_{be}	2930 kip-in/rad
k_b	138 kip-in/rad
k_{ce}	553 kip-in/rad
I_b	1.59 in^4
L	96 in
I_c	1.36 in^4
H	290 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	84 in	5 kip
2	142 in	5 kip
3	200 in	5 kip
4	258 in	5 kip
5	317 in	5 kip

Calculated T = 4.09

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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LRFD Basic Load Combinations: Type A - 12" Flue

2018 IBC& RMI / ANSI MH 16.1

V_{Trans} = **4,355 lbs**

V_{Long} = **1,035 lbs**

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

D = Dead Load · 0.5 = **250 lbs**

$M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **926,163 in-lbs**

$E_{Trans} = M_{Trans}$ / frame depth = **22,051 lbs**

$\beta = 0.7$

$\beta = 1.0$ (Uplift combination only)

$\rho = 1$

$S_{DS} = 1.01$

L = Live Load = **0 lbs** S = Snow Load = **0 lbs** R = Rain Load = **0 lbs**

Lr = Live Roof Load = **0 lbs** W = Wind Load = **0 lbs**

Basic Load Combinations

1. Dead Load $= 1.4\ D + 1.2\ P$

$= (1.4 \cdot 250) + (1.2 \cdot 12{,}500) =$ **15,350 lbs**

2. Gravity Load $= 1.2\ D + 1.4\ P + 1.6\ L + 0.5\ (L_r \text{ or } S \text{ or } R)$

$= (1.2 \cdot 250) + (1.4 \cdot 12{,}500) + (1.6 \cdot 0) + (0.5 \cdot 0) =$ **17,800 lbs**

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

$= (1.2 \cdot 250) + (0.85 \cdot 12{,}500) + (0.5 \cdot 0) + (1.6 \cdot 0) =$ **10,925 lbs**

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

$= (1.2 \cdot 250) + (0.85 \cdot 12{,}500) + (0.5 \cdot 0) + (1.0 \cdot 0) + (0.5 \cdot 0) =$ **10,925 lbs**

5A. Seismic Load (Transverse) $= (1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Trans} + 0.2S$

$= (1.2 + 0.2 \cdot 1.01) \cdot 250 + (1.2 + 0.2 \cdot 1.01) \cdot 0.7 \cdot 12{,}500 + 0.5 \cdot 0 + 1 \cdot 22{,}051 + 0.2 \cdot 0 =$ **34,669 lbs**

5B. Seismic Load (Longitudinal) $= (1.2 + 0.2S_{DS})D + (1.2 + 0.2S_{DS})\beta P + 0.5L + \rho E_{Long} + 0.2S$

$= (1.2 + 0.2 \cdot 1.01) \cdot 250 + (1.2 + 0.2 \cdot 1.01) \cdot 0.7 \cdot 12{,}500 + 0.5 \cdot 0 + 1 \cdot 0 + 0.2 \cdot 0 =$ **12,617 lbs**

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

$= 0.9 \cdot 250 + 0.9 \cdot 12{,}500 + 1.0 \cdot 0 =$ **225 lbs**

7. Seismic Uplift $= (0.9 - 0.2S_{DS})D + (0.9 - 0.2S_{DS})\beta P_{app} - \rho E_{Trans}$

$= (0.9 - 0.2 \cdot 1.01) \cdot 250 + (0.9 - 0.2 \cdot 1.01) \cdot 1 \cdot 12{,}500 - 1 \cdot 22{,}051 =$ **-13,152 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 $= 1.2D + 1.6L + 0.5(S or R) + 1.4P + 1.4I$

$(1.2 \cdot 32) + (1.6 \cdot 0) + (0.5 \cdot 0) + (1.4 \cdot 2{,}500) + (1.4 \cdot 312) =$ **3,975 lbs**

ASD Load Combinations for Slab Analysis

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

$= (1 + 0.105 \cdot 1.01) \cdot 250 + 0.75((1.4 + 0.14 \cdot 1.01) \cdot 0.7 \cdot 12{,}500 + 0.7 \cdot 1 \cdot 22{,}051) =$ **21,968 lbs**

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

$= (1 + 0.14 \cdot 1.01) \cdot 250 + (0.85 + 0.14 \cdot 1.01) \cdot 0.7 \cdot 12{,}500 + 0.7 \cdot 1 \cdot 22{,}051 =$ **24,396 lbs**

3. D + P

$= 250 + 12{,}500 =$ **12,750 lbs**

SEIZMIC INC
MATERIAL HANDLING ENGINEERING
EST. 1985

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1130 E. CYPRESS ST, COVINA, CA 91724

PRCTI20221624

PROJECT: Red Dot Corp. WA
FOR: Raymond West_Jack Murp
ADDRESS: 2504 E. Main Avenu
Puyallup, WA
SHEET#: 11
CALCULATED BY: ang
DATE: 10/13/2022
PN: 20220808_22

Longitudinal Analysis: Type A - 12" Flue

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).

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

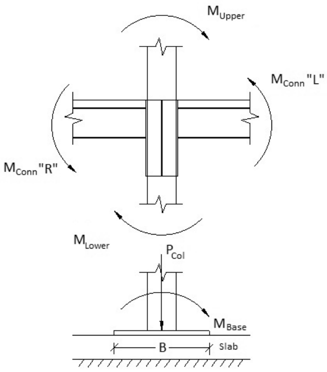
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

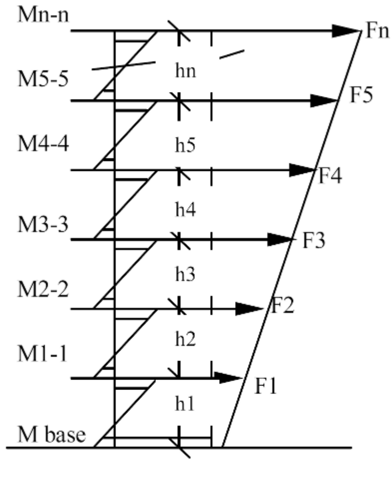
$V_{Col} = V_{Long}$ / # of columns = **518 lbs**

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

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

(518 lbs · 56 in.) - 0 in-lbs = 29008 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	58	35	12,750	29,008	5,885	34,893
2	58	69	10,200	29,008	4,760	33,768
3	58	104	7,650	29,008	4,760	33,768
4	58	138	5,100	29,008	4,760	33,768
5	58	173	2,550	29,008	4,760	19,264

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DATE: 10/13/2022
PN: 20220808_22

COLUMN WITH BACKER ANALYSIS: Type A - 12" Flue (Level 1)

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

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

$K_x \cdot L_x / R_x$ = 1.7·56 / 1.251 = **76.12**

$K_y \cdot L_y / R_y$ = 1·36 / 1.862 = **19.33**

KL/R_{max} = **76.12**

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

$= (1.251^2 + 1.862^2 + -2.819^2)^{1/2} =$ **3.602 in.**

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

$= 1 - (-2.819/3.602)^2 =$ **0.388**

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

$= 3.14^2 \cdot 29500 / 76.12^2 =$ **50.252 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)

$= (1 / (2 \cdot 0.388)((50.252 + 86.671) - (50.252 + 86.671)^2$

$- (4 \cdot 0.388 \cdot 50.252 \cdot 86.671))^{1/2}) =$ **35.347 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 76.12^2 =$ **50.252 ksi**

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

$= 1 / 1.739 \cdot 3.602^2 (11300 \cdot 0.005$

$+ (3.14^2 \cdot 29500 \cdot 5.422) / (0.8 \cdot 36)^2) =$ **86.671 ksi**

$F_e = \text{Min}(F_{e1}, F_{e2}) =$ **35.347 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 35.347)^{1/2} =$ **1.247** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\wedge(\lambda_c^2)) \cdot F_y =$ **28.676** (Eq C4.1-2)

Thus:

P_n = **42549 lbs**

P_a = **36167 lbs**

3 x 3 - .089	
SECTION PROPERTIES	
Depth	6 in.
Width	3 in.
t	0.089 in.
Radius	0.19 in.
Area	1.739 in.^{2}
AreaNet	1.484 in.^{2}
I_x	2.72 in.^{4}
S_x	1.813 in.^{3}
$S_{x\ Net}$	1.686 in.^{3}
R_x	1.251 in.
I_y	6.027 in.^{4}
S_y	1.869 in.^{3}
R_y	1.862 in.
J	0.005 in.^{4}
C_w	5.422 in.^{6}
J_x	2.966 in.
X_o	-2.819 in.
K_x	1.7
L_x	56 in.
K_y	1
L_y	36 in.
K_t	0.8
F_y	55 ksi
F_u	65 ksi
Q	1
G	11300 ksi
E	29500 ksi
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

COLUMN WITH BACKER ANALYSIS: Type A - 12" Flue (Level 1)

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

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

$P_{ao} = P_{no}\phi_c =$ **69367 lbs**

Where:

$P_{no} = A_e F_y = 1.484 \cdot 55 =$ **81609 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **227.97 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **126.148 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 =$ **4481.149 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **126.148 ksi**

Since: $0.56\ F_y < 2.78 F_y$

$F_c = (10/9)F_y(1 - (10F_y / 36F_e)) =$ **53.7 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **53.7 ksi**

$M_{nx} =$ **90516 in-lbs** $M_{ny} =$ **100346 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **81465 in-lbs** $M_{ny}\phi_b =$ **90312 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2 =$ **87368 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2 =$ **1354053 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.869** (Eq C5.2.2-4)

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

$P_{trans} =$ **34,669 lbs** $P_{long} =$ **12,617 lbs**

$M_u = M_x =$ **28980 in-lbs** (Eq C5.2.2-2)

$P_{u\_st} = (1.2 \cdot D) + (1.4 \cdot P) =$ **17800 lbs**

$P_{u\_st} / P_a = 17800 / 36167 = 0.49$ Static Stress = **49%**

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)

= ((12,617 / 36167) + (28980 / 81465) + (1 / 90312)) = **70%**

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)

= (12,617 / 69367) + (0.85 · 28980 / 81465 · 0.869)) + (0.85 · 1 / 90312 · 0.992))) = **53%**

Stress3 P_t / P_{ao} = 34,669 / 69367 =**49%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **70%**

3 x 3 - .089	
SECTION PROPERTIES	
Depth	**6 in.**
Width	**3 in.**
t	**0.089 in.**
Radius	**0.19 in.**
Area	**1.739 in.** ^{2}
AreaNet	**1.484 in.** ^{2}
I_x	**2.72 in.** ^{4}
S_x	**1.813 in.** ^{3}
$S_{x\ Net}$	**1.686 in.** ^{3}
R_x	**1.251 in.**
I_y	**6.027 in.** ^{4}
S_y	**1.869 in.** ^{3}
R_y	**1.862 in.**
J	**0.005 in.** ^{4}
C_w	**5.422 in.** ^{6}
J_x	**2.966 in.**
X_o	**-2.819 in.**
K_x	1.7
L_x	**56 in.**
K_y	1
L_y	**36 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	1
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

COLUMN ANALYSIS: Type A - 12" Flue (Level 3)

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 · 56 / 1.251 = **67.17**

$K_y \cdot L_y / R_y$ = 1 · 54 / 1.103 = **48.96**

KL/R_{max} = **67.17**

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

$= (1.251^2 + 1.103^2 + -2.819^2)^{1/2} =$ **3.275 in.**

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

$= 1 - (-2.819/3.275)^2 =$ **0.259**

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

$= 3.14^2 \cdot 29500 / 67.17^2 =$ **64.536 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)

$= (1 / (2 \cdot 0.259))((64.536 + 48.152) - (64.536 + 48.152)^2$

$- (4 \cdot 0.259 \cdot 64.536 \cdot 48.152))^{1/2}) =$ **29.591 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 67.17^2 =$ **64.536 ksi**

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

$= 1 / 0.869 \cdot 3.275^2 (11300 \cdot 0.002$

$+ (3.142 \cdot 29500 \cdot 2.711) / (0.8 \cdot 54)^2) =$ **48.152 ksi**

$F_e = \mathrm{Min}(F_{e1}, F_{e2}) =$ **29.591 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 29.591)^{1/2} =$ **1.363** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\text{^}(\lambda_c^2)) \cdot F_y =$ **25.264** (Eq C4.1-2)

Thus:

P_n = **17812 lbs**

P_a = **15140 lbs**

3 x 3 - .089	
SECTION PROPERTIES	
Depth	3 in.
Width	3 in.
t	0.089 in.
Radius	0.19 in.
Area	0.869 $in.^2$
AreaNet	0.742 $in.^2$
I_x	1.36 $in.^4$
S_x	0.907 $in.^3$
$S_{x\ Net}$	0.843 $in.^3$
R_x	1.251 in.
I_y	1.058 $in.^4$
S_y	0.613 $in.^3$
R_y	1.103 in.
J	0.002 $in.^4$
C_w	2.711 $in.^6$
J_x	3.096 in.
X_o	-2.819 in.
K_x	1.5
L_x	56 in.
K_y	1
L_y	54 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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COLUMN ANALYSIS: Type A - 12" Flue (Level 3)

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

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

$P_{ao} = P_{no}\phi_c =$ **32961 lbs**

Where:

$P_{no} = A_e F_y = 0.705 \cdot 55 =$ **38778 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **175.069 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **67.937 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 =$ **349.502 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **67.937 ksi**

Since: $0.56\ F_y < 2.78 F_y$

$F_c = (10/9)F_y(1 - (10F_y / 36F_e)) =$ **47.4 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **45.3 ksi**

$M_{nx} =$ **38178 in-lbs** $M_{ny} =$ **27768 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **34360 in-lbs** $M_{ny}\phi_b =$ **24992 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2 =$ **56109 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2 =$ **105607 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.877** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.935** (Eq C5.2.2-5)

$P_{trans} =$ **26,131 lbs** $P_{long} =$ **10,094 lbs**

$M_u = M_x =$ **9005 in-lbs** (Eq C5.2.2-2)

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

$P_{u\_st} / P_a = 10680 / 15140 = 0.75$ Static Stress = **74%**

Since: $P_l / P_a \geq 0.15$

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

= ((10,094 / 15140) + (9005 / 34360) + (1 / 24992)) = **92%**

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

= (10,094 / 32961) + (0.85 · 9005 / 34360 · 0.877)) + (0.85 · 1 / 24992 · 0.935))) = **56%**

Stress3 P_t / P_{ao} = 26,131 / 32961 =**79%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **92%**

3 x 3 - .089	
SECTION PROPERTIES	
Depth	**3 in.**
Width	**3 in.**
t	**0.089 in.**
Radius	**0.19 in.**
Area	**0.869 in.** ^{2}
AreaNet	**0.742 in.** ^{2}
I_x	**1.36 in.** ^{4}
S_x	**0.907 in.** ^{3}
$S_{x\ Net}$	**0.843 in.** ^{3}
R_x	**1.251 in.**
I_y	**1.058 in.** ^{4}
S_y	**0.613 in.** ^{3}
R_y	**1.103 in.**
J	**0.002 in.** ^{4}
C_w	**2.711 in.** ^{6}
J_x	**3.096 in.**
X_o	**-2.819 in.**
K_x	1.5
L_x	**56 in.**
K_y	1
L_y	**54 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	0.9
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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DATE: 10/13/2022
PN: 20220808_22

BEAM ANALYSIS Type A - 12" Flue

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

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

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

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

λ <= 0.673: Flange is fully effective.

Check web for local buckling (B2.3)

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

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

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

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

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

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

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

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

b2 = w/2 = **1.88**

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

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

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

L_f = effective width = **1.255**

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

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

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

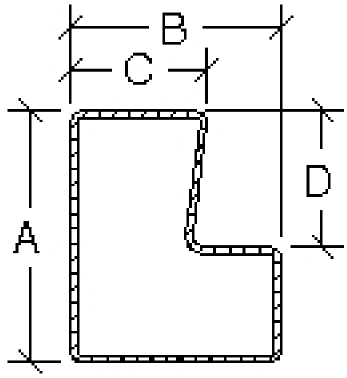
Fu/Fy = 70 / 55 = **1** **>= 1.2 = OK**

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

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

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

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



4.13 x 2.5 - .060

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

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BEAM ANALYSIS Type A - 12" Flue

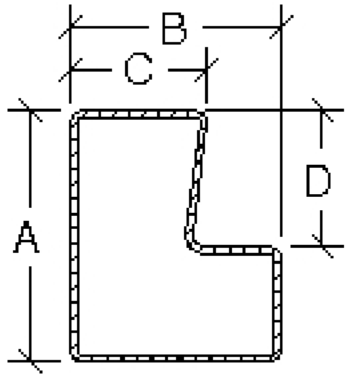
Check Allowable Tension Stress for Bottom Flange

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

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

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

$F_{ya} = F_{ya\text{-}top} = \mathbf{60.95\ ksi}$



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

Check Bending Capacity

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

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

For DL = 2% of PL:

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

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

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

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

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

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

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

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

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

4.13 x 2.5 - .060

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

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Allowable and Actual Bending Moment at Each Level

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

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	30,576	32,748	13,439	32,748	Pass
2	30,576	31,308	7,883	31,308	Pass
3	30,576	31,308	5,382	31,308	Pass
4	30,576	31,308	3,131	31,308	Pass
5	30,576	31,308	2,380	31,308	Pass

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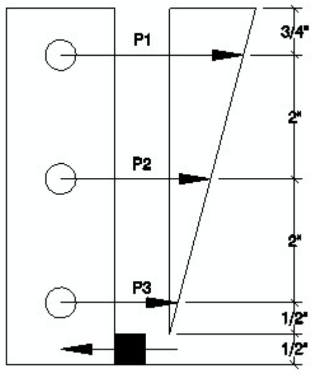
Beam to Column Analysis: Type A - 12" Flue

1. Shear Strength of Pin



Pin Diameter = **0.35 in.**

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

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

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

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

2. Bearing Strength of Pin

Column Thickness $t_c =$ **0.09 in.**

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

$m_f =$ **1.0**

$F_u =$ **65000 psi**

$P_n = C \cdot m_f \cdot d \cdot t_c \cdot F_u =$ **6074.25 lbs** AISI E3.3.1 -1

$P_{Bearing} = \phi P_n = 0.75 \cdot 6074.25 =$ **4555 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

$P_1 = \text{Min}(P_{Shear}, P_{Bearing}, P_{Clip}) =$ **3896 lbs**

$M_{Conn\text{-}Allow} = (P_1 \cdot 4.5) + (P_1 \cdot (2.5 / 4.5) \cdot 2.5) + (P_1 \cdot (0.5 / 4.5) \cdot 0.5) =$ **23159.56 in-lbs**

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BRACE ANALYSIS Type A - 12" Flue (Panel 3)

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

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

$K_x \cdot L_x / R_x = 0 \cdot 60 / 0.57 = \mathbf{104.46}$
$K_y \cdot L_y / R_y = 1 \cdot 60 / 0.57 = \mathbf{104.46}$
$KL / R_{max} = \mathbf{104.46}$

$r_o = (r_x^2 + r_y^2 + x_o^2)^{1/2}$
$= (0.57^2 + 0.57^2 + 0^2)^{1/2} = \mathbf{0.81\ in.}$ (Eq C3.1.2.1-7)

$\beta = 1 - (x_o / r_o)^2 = 1 - (0 / 0.81)^2 = \mathbf{1}$ (Eq C4.1.2-3)

$F_{e1} = \Pi^2 E / (KL / r)_{max}^2 = 3.14^2 \cdot 29500 / 104.46^2 = \mathbf{26.684\ ksi}$ (Eq C4.1.1-1)

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

$= (1 / (2 \cdot 1)((26.68 + 9334.96) - ((26.68 + 9334.96)^2$
$- (4 \cdot 1 \cdot 26.68 \cdot 9334.96))^{1/2}) = \mathbf{26.684\ ksi}$ (Eq C4.1.2-1)

where:

$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$
$= 3.14^2 \cdot 29500 / 104.46^2 = \mathbf{26.684\ ksi}$ (Eq C3.1.2-11)

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$
$= 1 / 0.41 \cdot 0.81^2 (11300 \cdot 0.2229$
$+ (3.14^2 \cdot 29500 \cdot 0) / (0.8 \cdot 60)^2) = \mathbf{9334.962\ ksi}$ (Eq C3.1.2-9)

$F_e = \text{Min(Fe1, Fe2)} = \mathbf{26.684\ ksi}$

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

$\lambda_c = (F_y / F_e)^{1/2} = (36 / 26.684)^{1/2} = \mathbf{1.162}$ (Eq C4.1-4)

Since $\lambda_c < 1.5$, $F_n = (0.658^{\wedge}(\lambda_c^2)) \cdot F_y = \mathbf{20.468\ ksi}$ (Eq C4.1-2)

Thus

$P_n = \mathbf{8,369\ lbs}$
$P_a = P_n \cdot \phi_c = \mathbf{7,114\ lbs}$

1.5 x 1.5 -0.075 (Tube)

SECTION PROPERTIES	
Depth	1.5 in.
Width	1.5 in.
t	0.075 in.
Radius	0.107 in.
Area	0.409 in^2
AreaNet	0.409 in^2
Ix	0.135 in^4
Sx	0.18 in^3
Sx net	0.18 in^3
Rx	0.574 in.
Iy	0.135 in^4
Sy	0.18 in^3
Ry	0.574 in.
J	0.223 in^4
Cw	0 in^6
Jx	0 in.
Xo	0 in.
Kx	0
Lx	60 in.
Ky	1
Ly	60 in.
Kt	0.8
Fyv	36 ksi
Fuv	42 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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BRACE ANALYSIS Type A - 12" Flue (Panel 3)

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

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

$P_{ao} = P_{no}\phi_c = 14{,}720 \cdot 0.85 = \mathbf{12{,}512\ lbs.}$

Where $P_{no} = A_eF_y = 0.41 \cdot 36 = \mathbf{14{,}720\ lbs.}$

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

$F_e = C_br_oA(\sigma_{ey}\sigma_t)^{1/2} / S_f = 921.5$ ksi

$F_e = C_sA\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF}S_f) = 921.5$ ksi (Eq C3.1.2.1-4)

$F_e = (C_b\Pi^2EdI_{yc}) / (S_f(K_yL_y)^2 = 90.97$ ksi (Eq C3.1.2.1-10)

$F_{e,min} = \mathbf{90.97\ ksi}$ (Eq C3.1.2.1-14)

Since, $0.56F_y < F_e < 2.78F_y$

$F_c = (10 / 9)F_y(1 - (10F_y / (36F_e)) = \mathbf{35.6\ ksi}$ (Eq C3.1.2.1-2)

reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = \mathbf{35.6\ ksi}$

$M_{nx} = \mathbf{6{,}404\ in\text{-}lbs}$ $M_{ny} = \mathbf{6{,}404\ in\text{-}lbs}$ $M_c = M_{n,min}$

$M_{nx}\phi_b = \mathbf{5{,}764\ in\text{-}lbs}$ $M_{ny}\phi_b = \mathbf{5{,}764\ in\text{-}lbs}$

$P_{Ex} = \Pi^2EI_x / (K_xL_x)^2 = \mathbf{10{,}910\ lbs}$ (Eq C5.2.2-6)

$P_{Ey} = \Pi^2EI_y / (K_yL_y)^2 = \mathbf{10{,}910\ lbs}$ (Eq C5.2.2-7)

Max $P_a = \mathbf{8{,}369\ lbs}$

$V_{Trans} = \mathbf{3{,}130\ lbs}$

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

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = \mathbf{5216\ lbs.}$

Brace Stress = $V_{Diag} / P_a = \mathbf{73\%}$

1.5 x 1.5 -0.075 (Tube)

SECTION PROPERTIES	
Depth	1.5 in.
Width	1.5 in.
t	0.075 in.
Radius	0.107 in.
Area	0.409 in^2
AreaNet	0.409 in^2
Ix	0.135 in^4
Sx	0.18 in^3
Sx net	0.18 in^3
Rx	0.574 in.
Iy	0.135 in^4
Sy	0.18 in^3
Ry	0.574 in.
J	0.223 in^4
Cw	0 in^6
Jx	0 in.
Xo	0 in.
Kx	0
Lx	60 in.
Ky	1
Ly	60 in.
Kt	0.8
Fyv	36 ksi
Fuv	42 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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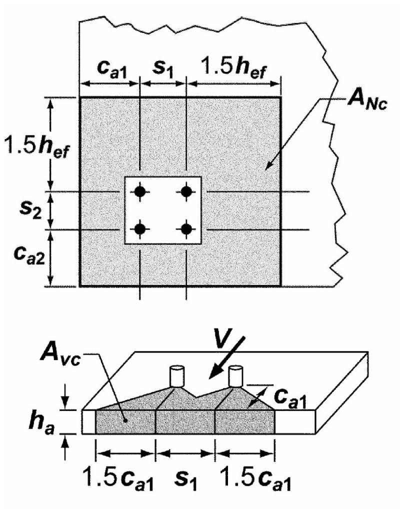
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 Type A - 12" Flue

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)	= **7 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})	= Hef	S_1	= **6 in.**	Use $S_{1,adj}$	= **6 in.**
Number of Anchors (n)	= **4**	S_2	= **6 in.**	Use $S_{2,adj}$	= **6 in.**
e`N	= **0**				
e`V	= **0**				

From ICC ESR Report

A_{se}	= **0.164 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 = **3145 lbs**	0.75	**2359 lbs**
Shear Capacity = **6665 lbs**	0.75	**4999 lbs**

ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Type A - 12" Flue

Steel Strength	17.4.1
ϕ = **0.75**	17.3.3.a i
$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 4 \cdot 0.164 \cdot 106700 =$ **52,496 lbs**	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
ϕ = **0.65**	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) =$ **324 sq.in.**	
$A_{Nco} = 9h_{ef}^2 =$ **144 sq.in.**	
Check if $A_{Nco} \geq A_{Nc}$ $A_{Nc}/A_{Nco} =$ **2.25**	
$\Psi_{ec,N} =$ **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**	
$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} =$ **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
$0.65 \cdot (324/144) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 8601 =$ **12,579 lbs**	
Pullout Strength ϕN_{pn}	17.4.3
ϕ=**0.65**	17.3.3 c ii Category 1-B
$\Psi_{cp} =$ **1**	17.4.3.6
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} =$ **32,884 lbs**	17.4.3.1

Steel Strength (ϕN_{sa}) = **52,496 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **12,579 lbs**

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

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Type A - 12" Flue

Steel Strength ϕV_{sa} V_{sa}=10,255 / Anchor -- per report	17.5.1
ϕ = **0.65**	17.3.3. Condition a ii
$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 4 \cdot 10{,}255 =$ **26,663 lbs**	17.5.1.2a
Concrete Breakout Strength ϕV_{cbg}	17.5.2
ϕ = **0.7**	17.3.3 ci-B
$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **294 sq.in.**	
$A_{Vco} = 3C_{a1}h_a =$ **252 sq.in.**	
Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1.167**	
$\Psi_{ec,V}$ = **1**	17.5.2.5
$\Psi_{ed,V}$ = **0.9**	17.5.2.6
$\Psi_{C,V}$ = **1**	17.5.2.7
$\Psi_{h,V}$ = **1.604**	17.5.2.8
d_a = **0.625 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
$0.7 \cdot (294/252) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.604 \cdot 16{,}713 =$ **39,396 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} = **19,352 lbs**	
$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 19{,}352 =$ **27,093 lbs**	

Steel Strength (ϕV_{sa}) = **26,663 lbs**

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **39,396 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **27,093 lbs**

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OVERTURNING ANALYSIS Configuration1 Type A - 12" Flue

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**

$V_{Trans} = (1 \cdot 0.2525 \cdot 1 \cdot ((0.67 \cdot 16{,}750) + 500)) =$ **2,959 lbs**

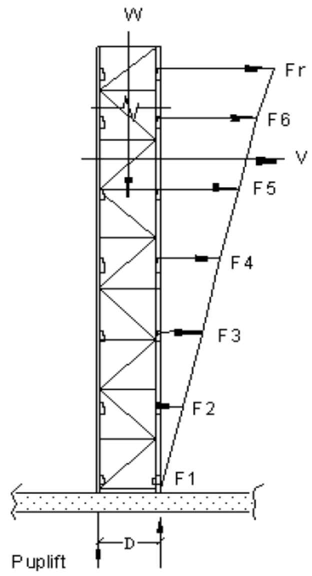
$M_{ovt} = V_{Trans} \cdot Ht = 2{,}959 \cdot 239 =$ **707,201 in-lbs**

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

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

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (707{,}201 - 362{,}250) / 42 =$ **8,213 lbs**

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (707{,}201 + 362{,}250) / 42 =$ **25,463 lbs**



CROSS AISLE ELEVATION

TOP SHELF LOADED

Shear = **1,388 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 1{,}388 \cdot (290 + ((58 - 10) / 2)) =$ **435,832 in-lbs**

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

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (435{,}832 - 115{,}500) / 42 =$ **7,626 lbs**

SIDE LOADS ON TOP SHELF

Top loaded shelf level (H) = **290 in.**

$M_{ovt} = 1.6 \cdot 350 \cdot H = 1.6 \cdot 350 \cdot 290 =$ **162kip**

$P_{uplift} = M_{ovt}/d = 162 / 42 =$ **3,866 lbs**

ANCHORS

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

Pull Out Capacity per Anchor (T_{Anchor}): **2,359 lbs**

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

$P_{Resist} = 0.95 \cdot T_{Anchor} \cdot \#Anchors \cdot Row\ Spacer\ Stress = 0.95 \cdot 2{,}359 \cdot 4 \cdot 0.776 =$ **6,953 lbs**

COMBINED STRESS

Fully Loaded	= ((6,953 / 4) / 2,359) + ((2,959 / 8) / 4,999)	= **0.811**
Top Shelf Loaded	= ((6,953 / 4) / 2,359) + ((1,388 / 8) / 4,999)	= **0.772**
Seismic UpLift Critical (LC#7B)	= (6,953 / 4) / 2,359	= **0.737**
Side Load Top Shelf	= ((3,866 / 2) / 4) / 2,359	= **0.205**

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SIDE ROW SPACER ANALYSIS

ROW SPACER CONNECTION CHECK

Per AISI E3 Bolted Connection

Spacer S_x	1 cubic in
F_{nt}	81 kip
$P_{additional}$	4.188 kip
Spacer length L	12 in
# of spacers n	4
Bolt diameter d	0.438 in
Column thickness tc	0.089 in
Bracket thickness tb	0.125 in
Plate width b	3 in
F_y (column)	55 ksi
F_u (column)	65 ksi
Connection pattern	2 bolts @ 4" o.c.
Bolt type	A449
Bolt spacing s	4 in
F_y (plate)	36 ksi
F_y (Row Spacer)	50 Ksi

E3.1 SHEAR, SPACING AND EDGE DISTANCE

$P_{n1} = t \cdot e \cdot F_u = 0.089 \cdot 1 \cdot 65 = \mathbf{5.79\ kip}$

$P_{a1} = 0.7 \cdot P_{n1} = 0.7 \cdot 5.79 = \mathbf{4.05\ kip}$

E3.2 RUPTURE IN NET SECTION (SHEAR LAG)

$P_{n2} = A_n \cdot F_t = (s - d) \cdot t \cdot F_t = (4 - 0.438) \cdot 0.089 \cdot 27.828 = \mathbf{8.82\ kip}$

$P_{a2} = 0.55 \cdot P_{n2} = 0.55 \cdot 8.82 = \mathbf{4.85\ kip}$

E3.3 BEARING

$P_{n3} = c \cdot m_f \cdot d \cdot t \cdot F_u = 3 \cdot 1 \cdot 0.438 \cdot 0.089 \cdot 65 = \mathbf{7.59\ kip}$

$P_{a3} = 0.6 \cdot P_{n3} = 0.6 \cdot 7.59 = \mathbf{4.56\ kip}$

E3.4 SHEAR AND TENSION IN BOLTS

$P_{n4} = A_b \cdot F_{nv} = 0.15 \cdot 47 = \mathbf{7.07\ kip}$

$P_{a4} = 0.65 \cdot P_{n4} = 0.65 \cdot 7.07 = \mathbf{4.59\ kip}$

$P_a = \mathrm{Min}(P_{a1}, P_{a2}, P_{a3}, P_{a4})$

$= \mathrm{Min}(4.05, 4.85, 4.56, 4.59) = \mathbf{4.05\ kip}$

CONNECTION MOMENT CAPACITY

Moment Arm Lm = **4 in.**

$M_{a1} = P_a \cdot L \cdot 2 = 4.05 \cdot 4 \cdot 2 = \mathbf{32.4\ kip\text{-}in}$

ROW SPACER MEMBER CHECK

$M_n = F_y \cdot S_x = 50 \cdot 1 = \mathbf{50\ kip\text{-}in}$

$M_{a2} = 0.9 \cdot M_n = 0.9 \cdot 50 = \mathbf{45\ kip\text{-}in}$

Moment per Row Spacer M_r $= 2 \cdot P_{additional} \cdot L / n = 2 \cdot 4.188 \cdot 12 / 4 = \mathbf{25.13\ kip\text{-}in}$

Row Spacer Capacity M_a $= \mathrm{Min}\ (M_{a1}, M_{a2}) = \mathrm{Min}(32.4, 45) = \mathbf{32.4\ kip\text{-}in}$

Stress $= M_r / M_a = 25.13 / 32.4 = \mathbf{77\%}$

Base Plate Analysis: Type A - 12" Flue

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} = **34669 lbs**

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

$P/A = P_{col}/(D \cdot B) = 34669 / (8 \cdot 8) =$ **542 psi**

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

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

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

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

$= (2.5^2 / 2) \cdot (542 + 0 + 0.67 \cdot 0) =$ **1692.82 in-lbs**

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

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 1692.82 / (0.19 \cdot 32,400) =$ **0.28**

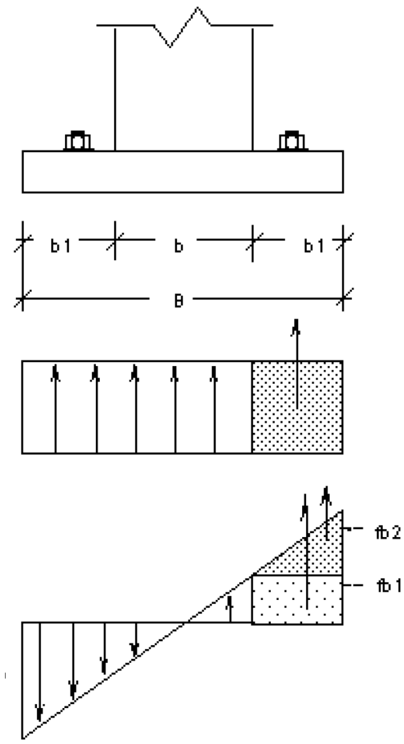

Plate width B =	8 in.
Plate depth D =	8 in.
Plate thickness t =	0.38 in.
Column width b =	3 in.
Column depth d =	3 in.
b1 =	2.5 in.
S_x =	6 in.
S_y =	6 in.
T_{Total} =	9,436 lbs.

Base Plate Tension analysis

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

$L_w = (S_x - b) / 2 =$ **1.5 in.**

$L_d = (S_y - b) / 2 =$ **1.5 in.**

Moment Arm (L) = $\text{Max}(L_w, L_d) =$ **1.5 in.**

$M_{anchor} = T_{Total} / 2 \cdot L =$ **7077 in-lbs**

$S = D \cdot t^2/6 =$ **0.19**in^3

$M_{baseplate} = S \cdot F_y =$ **6,750 in-lbs**

$\phi M_{baseplate} = 0.9 \cdot M_n =$ **6,075 in-lbs**

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

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

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Equation for Maximum Considered Earthquake Base Rotation

Per RMI 2012 Commentary 2.6.4

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

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

# of levels	5
min. # of bays	3
N_c	60
N_b	8
k_c	400 kip-in/rad
k_{be}	2930 kip-in/rad
k_b	138 kip-in/rad
k_{ce}	553 kip-in/rad
I_b	1.59 in^4
L	96 in
I_c	1.36 in^4
H	290 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	84 in	5 kip
2	142 in	5 kip
3	200 in	5 kip
4	258 in	5 kip
5	317 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

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

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 = 1.97

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= 0.78

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}=28,308 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) = **34,670 lbs**

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

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **4.129 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **25.646 in.**

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

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

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

P_{max} / P_a = **0.35**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **24,396 lbs**

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

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

$d,req'd$ = $(P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **4.129 in.**

b = $(E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **25.646 in.**

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

P_a = P_n / Ω = **55,366 lbs**

P_{max} / P_a = **0.44**

Base Plate

Width B	8 in.
Depth W	8 in.

Frame

Frame depth d	42 in.

Concrete

Thickness t	7 in.
f'c	4,000 psi
ϕ	0.6
Ω	3
λ	1
k_s	50 pci
r_1	4 in
E_c	3,604,997 psi

Cantilever Analysis

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	31
CALCULATED BY	ang
DATE	10/13/2022

CANTILEVER RACK TYPE C ANALYSIS:

PROJECT SCOPE

THE SUPPORT STRUCTURE SHALL BE ANALYZED WITH RESPECT TO THE REQUIREMENTS SET FORTH IN THE 2018 IBC. THE STRUCTURAL COMPONENTS SHALL BE ANALYZED WITH RESPECT TO THE MOST CRITICAL CONFIGURATIONS OF LOADS RESULTING FROM STATIC AND LATERAL FORCES.

SPECIFICATION

COLUMN STEEL- 50,000 PSI

ARM STEEL- 50,000 PSI

BASE STEEL- 50,000 PSI

BOLTS- A449 OR SIMILAR

SLAB- 7 IN * 4000 PSI

SUBGRADE MODULUS- 50 PCI

SEISMIC FORMULA- Cs x W

CONFIGURATION

LEVELS= 5

ARM LENGTH= 42 IN

wLL/ARM= 1,750 LB

wDL/ARM= 27 LB

SUMMARY OF RESULTS

ARM	COLUMN	LONGITUDINAL BRACE
TYPE= S4x7.7#	TYPE= W10X22	TYPE= 2.5" x 7 GA
STRESS= 41%/OK	STRESS (S)= 42%/OK	STRESS= 54%
	STRESS (D)= 33%/OK	CONNECTION = BOLTED
# BOLTS= 4	BASE	0.75"Ø A449 BOLT
BOLT TYPE= A449	TYPE= W10X22	STRESS= 56%
STRESS= 19%/OK	STRESS (S)= 67%	ANCHORS

SLAB AND SOIL		ANCHORS (cont.)
SLAB THICK= 7.0 IN		Simpson Strong Bolt 2 (ICC ESR-3037)
CONCRETE f'c= 4,000 PSI		(6) 0.625"x3.375" EMBED PER BASE
SUBGRADE MODULUS = 50 PCI	STRESS= 0.1	STRESS (S)= 7%/OK
		STRESS (D)= 15%/OK

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	32
CALCULATED BY	ang
DATE	10/13/2022

LOADS AND DISTRIBUTION:

SEISMIC ANALYSIS PER SECTION 1613 OF THE 2018 IBC

V = Cs x W
= S_{DS} x W / (R/I)
S_{DS} = S_{MS} x 2/3
S_{MS} = Fa x Ss
Ss= 1.26
Fa = 1.20
I= 1.00
R= 3
V-coeff.= (2/3) x Fa x Ss x Ie / (R x 1.4)
= 0.240
wLL/ARM= 1,750 LB
wDL/ARM= 27 LB

OF LEVELS= 5

Vtotal(sgl.) = 0.24 x 1199 LB x 5 LVLS
= 1,439 LB

LEVEL	Hx	Wx	Hx Wx	Fi	Fi * Hx
H1	29 IN	1,777 LB	51,532 IN-LB	121 LB	3,509 IN-LB
H2	49 IN	1,777 LB	87,071 IN-LB	204 LB	10,017 IN-LB
H3	69 IN	1,777 LB	122,610 IN-LB	288 LB	19,863 IN-LB
H4	89 IN	1,777 LB	158,149 IN-LB	371 LB	33,046 IN-LB
H5	109 IN	1,777 LB	193,688 IN-LB	455 LB	49,568 IN-LB
		∑Fn=V/Frame=	613,048 IN-LB	1,439 LB	116,002 IN-LB

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	33
CALCULATED BY	ang
DATE	10/13/2022

ARM ANALYSIS:

1) CHECK BENDING

wLL+wDL=W= 1,777 LB

Marm= W * L / 2
= 1776.95 LB * 42 IN / 2
= 37,316 IN-LB

Fb= [(2/3) - (Fy*L/r(T)^2/(1.53X10^6*Cb)]*Fy < 0.6*Fy
= [(2/3)-(50 KSI*(16.8)^2*1)/(1.53X10^6*1)]*50 KSI
= 32,869 PSI
Fb-eff= 30,000 PSI

fb= M/S
= 37315.95 IN-LB/3.03 IN^3
= 12,315 PSI

fb/Fb= 0.41 ≤ 1.0 OK

ARM: S4x7.7#
A= 4.0 IN
B= 2.66 IN
Iy= 6.050 IN^4
Sx= 3.030 IN^3
Fy= 50,000 PSI
Lmax= 42.0 IN
Lunsup= 42.0 IN
r(T)= 2.50

2) CHECK DEFLECTION

DEFLECTION= WL^3/8EI
= 0.080 IN

ALLOWABLE DEFLECTION= L/180
= **0.233 IN** **OK**

3) CHECK CONNECTION

CHECK EFFECT OF ARM MOMENT ON BOLTED CONNECTION

BOLT DIAM= 0.750 IN
# BOLT= 4
Ft= 42,000 PSI
Fu= 65,000 PSI
BEARING PLATE
THICK.=tmin= 0.500 IN

P1 IS MINIMUM OF TENSION AND BEARING CAPACITY

TENSION CAP= Ft*BOLT AREA*2
= 42000 PSI*(0.75 IN)^2*(∏/4)*2
= 37,110 LB
BEARING CAP= Fu*BEARING AREA*2
= 65000 PSI*0.75 IN*0.5 IN *2
= 48,750 LB
P1= 37,110 LB

Mcap= P1 * D * 1.33
= 197,920 IN-LB **OK**

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	34
CALCULATED BY	ang
DATE	10/13/2022

COLUMN ANALYSIS: SINGLE SIDED LOADED

1) STATIC LOADS

Pmax= # LVLS x ARM LOAD
= 8,885 LB

Mstatic= ∑ARM MOMENTS
= # LVLS x W x (L/2+A/2)
= 5 x 1776.95 LB x (42 IN/2 + 10.2 IN/2)
= 231,892 IN-LB

(kl/r)x = 1.5 x 109 IN / 4.27 IN
= 38.29
(kl/r)y = 1 x 60 IN / 1.33 IN
= 45.11
(kl/r)max= 45.11
Cc = (2π^2E/Fy)^0.5
= 107.9
SINCE (KL/r)max < Cc, USE EQTN E2-1

Fa = [1-((kl/r)^2/2Cc^2)]Fy / 5/3 + 3(kl/r)/8Cc - (kl/r)^3/8Cc^3
= 25,151 PSI

fa= P/AREA
= 1,369 PSI

fa/Fa= 0.05 < 0.15

fbx= M/S
= 9,995 PSI

Fbx= 0.6Fy
= 30,000 PSI

F'ex= (12*π^2*E)/(23*(KL/rx)^2)
= 101,852 PSI

fb/Fb= 0.33 (1-fa/F'e)= 1.00

COLUMN

TYPE= W10X22
A= 10.20 IN
B= 5.75 IN
Area= 6.49 IN^2
Ix= 118.00 IN^4
Sx= 23.20 IN^3
rx= 4.27 IN
Iy= 11.40 IN^4
Sy= 3.97 IN^3
ry= 1.33 IN
Fy= 50,000 PSI
Kx= 1.5
Ky= 1.0
Lx= 109 IN
Ly= 60 IN
Cm= 0.85

CHECK INTERACTION EQTN(S):
(H1-3): **fa/Fa + fb/Fb= 0.39 < 1.0 OK**

2) CHECK EFFECT OF SEISMIC LOAD

Mseis-sgl= 116,002 IN-LB <=== SEE LOADS & DIST SHEET

fb= (Mstatic+Mseismic)/Sx
= (231892 IN-LB + 116002 IN-LB)/23.2 IN^3
= 14,995 PSI

CHECK INTERACTION EQTN(S):
(H1-3): **fa/Fa + fb/Fb= 0.55 < 1.0 OK**

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	35
CALCULATED BY	ang
DATE	10/13/2022

COLUMN ANALYSIS: DOUBLE SIDED LOADED

1) STATIC LOADS

Pmax= # LVLS x ARM LOAD
= 17,770 LB

Mstatic= ∑ARM MOMENTS
= # LVLS x W x (L/2+A/2)
= 5 x 1776.95 LB x (42 IN/2 + 10.2 IN/2)
= 0 IN-LB

(kl/r)x = 1.5 x 109 IN / 4.27 IN
= 38.29
(kl/r)y = 1 x 60 IN / 1.33 IN
= 45.11
(kl/r)max= 45.11
Cc = (2π^2E/Fy)^0.5
= 107.9
SINCE (KL/r)max < Cc, USE EQTN E2-1

Fa = [1-((kl/r)^2/2Cc^2)]Fy / (5/3 + 3(kl/r)/8Cc - (kl/r)^3/8Cc^3)
= 25,151 PSI

fa= P/AREA
= 2,738 PSI

fa/Fa= 0.11 < 0.15

fbx= M/S
= 0 PSI

Fbx= 0.6Fy
= 30,000 PSI

F'ex= (12*π^2*E)/(23*(KL/rx)^2)
= 101,852 PSI

fb/Fb= 0.00 (1-fa/F'e)= 1.00

COLUMN
TYPE= W10X22
A= 10.20 IN
B= 5.75 IN
Area= 6.49 IN^2
Ix= 118.00 IN^4
Sx= 23.20 IN^3
rx= 4.27 IN
Iy= 11.40 IN^4
Sy= 3.97 IN^3
ry= 1.33 IN
Fy= 50,000 PSI
Kx= 1.5
Ky= 1.0
Lx= 109 IN
Ly= 60 IN
Cm= 0.85

CHECK INTERACTION EQTN(S):
(H1-3): **fa/Fa + fb/Fb= 0.11 < 1.0 OK**

2) CHECK EFFECT OF SEISMIC LOAD

Mseis-dbl= 232,005 IN-LB <=== SEE LOADS & DIST SHEET

fb= (Mstatic+Mseismic)/Sx
= (0 IN-LB + 232005 IN-LB)/23.2 IN^3
= 10,000 PSI

CHECK INTERACTION EQTN(S):
(H1-3): **fa/Fa + fb/Fb= 0.44 < 1.0 OK**

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	36
CALCULATED BY	ang
DATE	10/13/2022

OVERTURNING: SINGLE SIDED LOADED

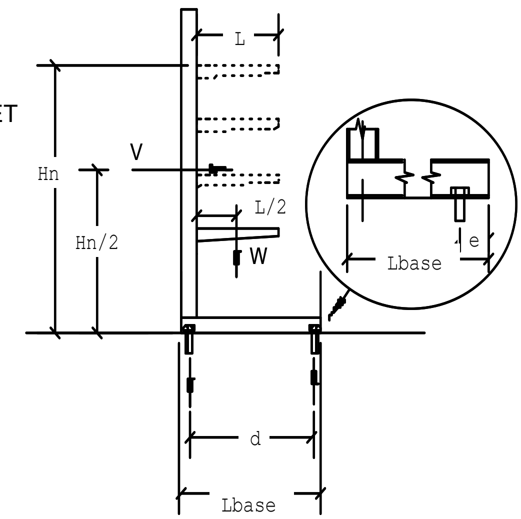
1) CHECK FULLY LOADED CASE

V= 1,439 LB

Movt= ∑Fn*Hn <=== SEE LOADS & DIST SHEET
= 116,002 IN-LB

Mst= (wDL + wLL) * L/2
= 5998 LB * 48.2 IN/2
= 144,552 IN-LB

Puplift= (Movt - 0.9*Mst)/d
= (116002 - 0.9*144552)/48.2 IN
= -292 LB <=== NO UPLIFT



SIDE VIEW

V= 0.24 * 1776.95 LE <=== SEE LOADS & DIST SHEET
= 426 LB

Movt= V * Hn * 1.15
= 426 LB * 109 IN * 1.15
= 53,399 IN-LB

Mst= (WDL*.85 + WLL) * L/2
= 1773 LB * 48.2 IN/2
= 42,727 IN-LB

Puplift= (Movt-Mst)/d
= (53399 - 42727)/48.2 IN
= **221 LB**

∑DL=5*26.95 LB 135 LB
∑LL= 0.67 x 5*1750 LB 5,863 LB
Lbase= 52 IN
e= 2.0 IN
d= 48.2 IN
L= 42 IN
Hn= 109.0 IN

TOTAL # OF ANCHORS/BASE= 6
OF ANCHORS PER END= 2

ANCHOR PULLOUT CAPACITY= 1,700 LB
ANCHOR SHEAR CAPACITY= 2,700 LB

CHECK COMBINED STRESS:

(0 LB/1700 LB) + (240 LB/2700 LB) = 0.09 ≤ 1.2 OK

USE (6) Simpson Strong Bolt 2 (ICC ESR-3037) PER BASE

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	37
CALCULATED BY	ang
DATE	10/13/2022

OVERTURNING: SINGLE SIDED LOADED

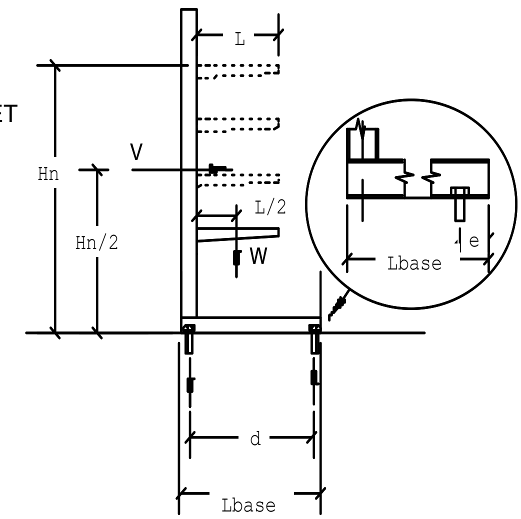
1) IMPACT LOAD

V= 1,439 LB

Movt= ∑Fn*Hn <=== SEE LOADS & DIST SHEET
= 116,002 IN-LB

Mst= (wDL + wLL) * L/2
= 5998 LB * 48.2 IN/2
= 144,552 IN-LB

Puplift= (Movt - 0.9*Mst)/d
= (116002 - 0.9*144552)/48.2 IN
= -292 LB <=== NO UPLIFT



SIDE VIEW

V= 0.24 * 1776.95 LE <=== SEE LOADS & DIST SHEET
= 426 LB

Movt= V * Hn * 1.15
= 426 LB * 109 IN * 1.15
= 53,399 IN-LB

Mst= (WDL*.85 + WLL) * L/2
= 1773 LB * 48.2 IN/2
= 42,727 IN-LB

Puplift= (Movt-Mst)/d
= (53399 - 42727)/48.2 IN
= **221 LB**

∑DL=5*26.95 LB 135 LB
∑LL= 0.67 x 5*1750 LB 5,863 LB
Lbase= 52 IN
e= 2.0 IN
d= 48.2 IN
L= 42 IN
Hn= 109.0 IN

TOTAL # OF ANCHORS/BASE= 6
OF ANCHORS PER END= 2

ANCHOR PULLOUT CAPACITY= 1,700 LB
ANCHOR SHEAR CAPACITY= 2,700 LB

CHECK COMBINED STRESS:

(0 LB/1700 LB) + (426 LB/2700 LB) = 0.16 ≤ 1.2 OK

USE (6) Simpson Strong Bolt 2 (ICC ESR-3037) PER BASE

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SEIZMIC INC.
MATERIAL HANDLING ENGINEERING
TEL: (909) 869-0989
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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	38
CALCULATED BY	ang
DATE	10/13/2022

OVERTURNING: DOUBLE SIDED LOADED

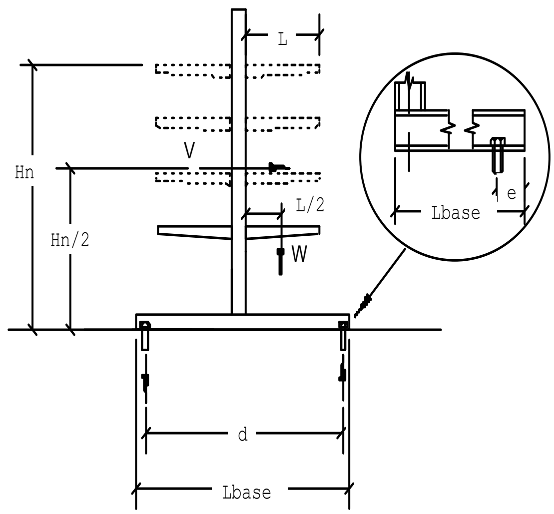
1) CHECK FULLY LOADED CASE

V= 2,879 LB

Movt= ∑Fn*Hn <=== SEE LOADS & DIST SHEET
= 232,005 IN-LB

Mst= (wDL + wLL) * L/2
= 23720 LB * 90.2 IN/2
= 1,069,772 IN-LB

Puplift= (Movt - 0.9*Mst)/d
= (232005 - 0.9*1069772)/90.2 IN
= -8,102 LB <=== NO UPLIFT



SIDE VIEW

V= 0.24 * 3553.9 LB <=== SEE LOADS & DIST SHEET
= 853 LB

Movt= V * Hn * 1.15
= 853 LB * 109 IN * 1.15
= 106,924 IN-LB

Mst= (WDL*.85 + WLL) * L/2
= 3546 LB * 90.2 IN/2
= 85,454 IN-LB

Puplift= (Movt-Mst)/d
= (106924 - 85454)/90.2 IN
= **238 LB**

∑DL=* LB 270 LB
∑LL= 0.67 x * LB 23,450 LB
Lbase= 94 IN
e= 2.0 IN
d= 90.2 IN
L= 42 IN
Hn= 109.0 IN

11725

TOTAL # OF ANCHORS/BASE= 6
OF ANCHORS PER END= 2

ANCHOR PULLOUT CAPACITY= 1,700 LB
ANCHOR SHEAR CAPACITY= 2,700 LB

CHECK COMBINED STRESS:

(0 LB/1700 LB) + (480 LB/2700 LB) = 0.18 ≤ 1.2 OK

USE (6) Simpson Strong Bolt 2 (ICC ESR-3037) PER BASE

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	39
CALCULATED BY	ang
DATE	10/13/2022

OVERTURNING: DOUBLE SIDED LOADED

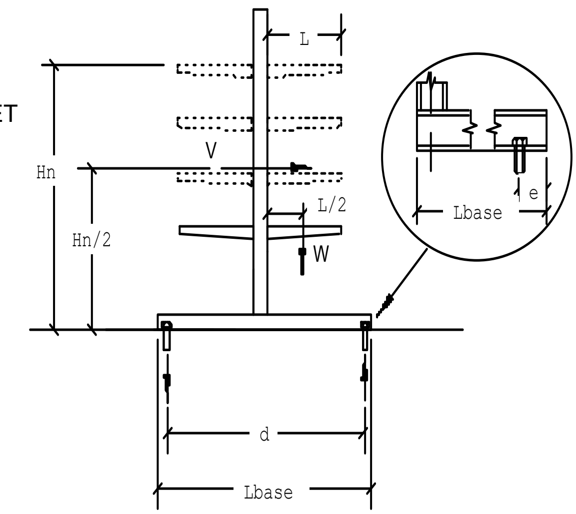
1) IMPACT LOAD

V= 2,879 LB

Movt= ∑Fn*Hn <=== SEE LOADS & DIST SHEET
= 232,005 IN-LB

Mst= (wDL + wLL) * L/2
= 23720 LB * 90.2 IN/2
= 1,069,772 IN-LB

Puplift= (Movt - 0.9*Mst)/d
= (232005 - 0.9*1069772)/90.2 IN
= 0 LB <=== NO UPLIFT



SIDE VIEW

V= 0.24 * 3553.9 LB <=== SEE LOADS & DIST SHEET
= 853 LB

Movt= V * Hn * 1.15
= 853 LB * 109 IN * 1.15
= 106,924 IN-LB

Mst= (WDL*.85 + WLL) * L/2
= 0 LB * 90.2 IN/2
= 85,454 IN-LB

Puplift= (Movt-Mst)/d
= (106924 - 85454)/90.2 IN
= **238 LB**

∑DL=* LB 270 LB
∑LL= 0.67 x * LB 23,450 LB
Lbase= 94 IN
e= 2.0 IN
d= 90.2 IN
L= 42 IN
Hn= 109.0 IN

TOTAL # OF ANCHORS/BASE= 6
OF ANCHORS PER END= 2

ANCHOR PULLOUT CAPACITY= 1,700 LB
ANCHOR SHEAR CAPACITY= 2,700 LB

CHECK COMBINED STRESS:

(0 LB/1700 LB) + (853 LB/2700 LB) = 0.32 ≤ 1.2 OK

USE (6) Simpson Strong Bolt 2 (ICC ESR-3037) PER BASE

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	40
CALCULATED BY	ang
DATE	10/13/2022

BASE ANALYSIS:

1) CHECK STATIC LOADS

Mbase= 231,892 IN-LB

fb= Mbase/Sx
= 231891.975 IN-LB/23.2 IN^3
= 9,995 PSI

Fb= 0.6*Fy
= 30,000 PSI

fb/Fb = 0.33 ≤ 1.0 OK

2) CHECK COMBINED STATIC AND SEISMIC LOADS

Mbase= 463,897 IN-LB

fb= Mbase/Sx
= 463896.895 IN-LB/23.2 IN^3
= 19,996 PSI

Fb= 0.6*Fy
= 30,000 PSI

fb/Fb = 0.67 ≤ 1.0 OK

BASE
TYPE= W10X22
A= 10.20 IN
B= 5.75 IN
t= 0.240 IN
Sx= 23.20 IN^3
Fy= 50,000 PSI

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	41
CALCULATED BY	ang
DATE	10/13/2022

DOWNAISLE BRACE:

V= V/FRAME*1.5
= 4,318 LB
Ldiag= [Y^2 + D^2]^0.5
= 77.0 IN

Vdiag= V*Ldiag/D
= 4318 LB * 77 IN/48 IN
= 6,927 LB <=== TENSION (T) ON DIAGONAL

Tallow= AREA * 0.6 * Fy * 1.33
= 0.4475 IN^2 * 0.6 * 36000 PSI * 1.33
= 12,888 LB

T/Ta= 0.54 ≤ 1.0 OK

Y= 60 IN
D= 48 IN

BRACE: 2.5" x 7 GA
BRACE AREA= 0.448 IN^2
Fy= 36,000 PSI

CHECK WELD CONNECTION

CHECK WELD CAPACITY OF 2.5 IN LONG X 0.1875 IN THICK WELD
Vallow= 0.3 * Fv * WELD THICKNESS * LENGTH * .707
= 0.3 * 70000 PSI * 0.1875 IN * 2.5 IN * 0.7071
= 6,961 LB > Vmax, OK

WELD CONNECTION
Fv= 70,000 PSI
WELD THICKNESS= 0.188 IN
WELD LENGTH= 2.5 IN

CHECK BOLT CONNECTION

CAPACITY= BOLT AREA * Fv * #BOLTS * 1.33
= (0.75 IN)^2 *(π/4)*21000 PSI*1*1.33
= 12,370 LB

STRESS= 6927 LB/12370 LB
= **0.56 ≤ 1.0 OK**

BRACE CONNECTION
CONNECTION TYPE= BOLTED
BOLT DIAM= 0.750 IN
Fv= 21,000 PSI
#BOLT/END= 1.000

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PROJECT	Red Dot Corp. WA
FOR	Raymond Handling Concepts
SHEET NO.	42
CALCULATED BY	ang
DATE	10/13/2022

Slab & Soil Analysis:

The slab is checked for puncture stress. If no puncture occurs the slab is assumed to distribute the load over a larger area of the soil hence, actins as a footing.

Pmax = **8,885 lb**

BASE PLATE
W = 5.8 in
D = 17.4 in

A1 = 100.1 in

CONCRETE
t = 7.0 in
f'c = 4,000 psi

SOIL
ks = 50 pci
Ø= 0.6

a) Puncture:

Fpunct = 2.66 x (F'c^0.5)
= 2.66 x (4000 psi)^0.5
= 168 psi
Apunct = [(Weff.+t/2)+(Deff.+t/2)] x 2 x t
= [(2.875 in + 7 in/2) + (8.7 in + 7 in/2)] x 2 x 7 in
= 422 in^2

fv/Fv = P/(Apunct x Fpunct)
= 8885 lb/[422 in^2 x 168 psi x 0.6]
= **0.21** **≤ 1.0 OK**

b) Bearing:

ØBn = 0.85 x Ø x f'c x A1
= 204,102 lb

Pu / ØBn = 8885 lb / 204102 lb
= **0.04** **≤ 1.0 OK**

c) Slab:

Pn = $1.72\ [(k_s\ R_1/E_c)10^4+3.6]f'_t\ d^2$ <= Emperical method formula: "Load Carrying Capacity for Concrete Slabs on Grade." Journal of Structural Engineering, ASCE, January 1997.
= 158,286 LB

k_s = 50 PCI modulus of subgrade reaction
R_1 = 2.875 one-half the width of base plate
E_c = 4,000,000 PSI modulus of elasticity of concrete
f'_t = 7.5 (f'c)^0.5 tensile strength in flexture of concrete
= 474 PSI

b = $[E_c d^3/(12(1-\mu^2)k_s)]^{0.25}$
= 39.11 IN <=== radius of relative stiffness
1.5*b = 58.66 IN <=== radius of relative curvature

F.S. = 0.6 factor of safety

Pa = Pn x F.S.
= **94,972 LB**

Pmax / Pa = **0.09** **≤ 1.0 OK**