SEIZMIC INC

MATERIAL HANDLING ENGINEERING

EST. 1985

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FOOTINGS			TITLE 24	CATWALKS	MINI-LOAD SYSTEMS

LICENSED IN 50 STATES

ANALYSIS OF STORAGE RACKS FOR

Zoetis

1120 NW Main Street, Lee's Summit, MO

Job No. 23-0260

Approved by:

SAL E. FATEEN, P.E.

STATE OF MISSOURI
SAL E. FATEEN
NUMBER
EN-026587
REGISTERED PROFESSIONAL ENGINEER

Digitally signed by Sal Fateen
Date: 2023.02.01 12:37:40-08'00'

EXPIRES
12/31/2024

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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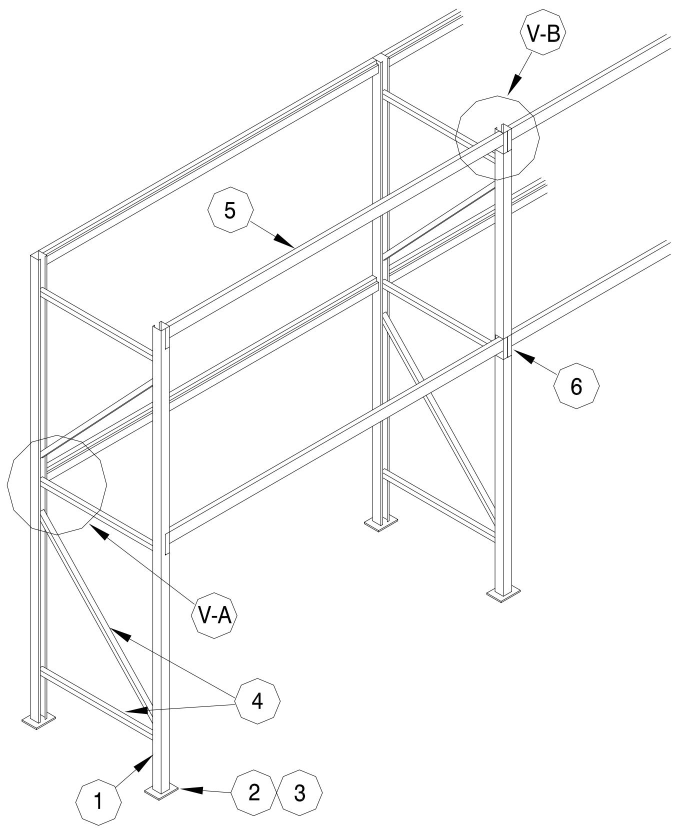
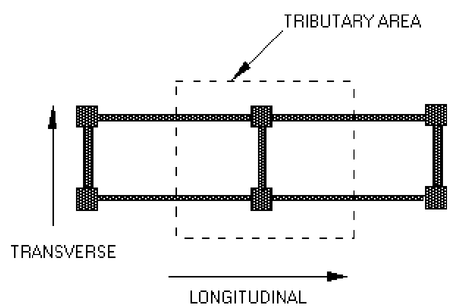
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being common between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction 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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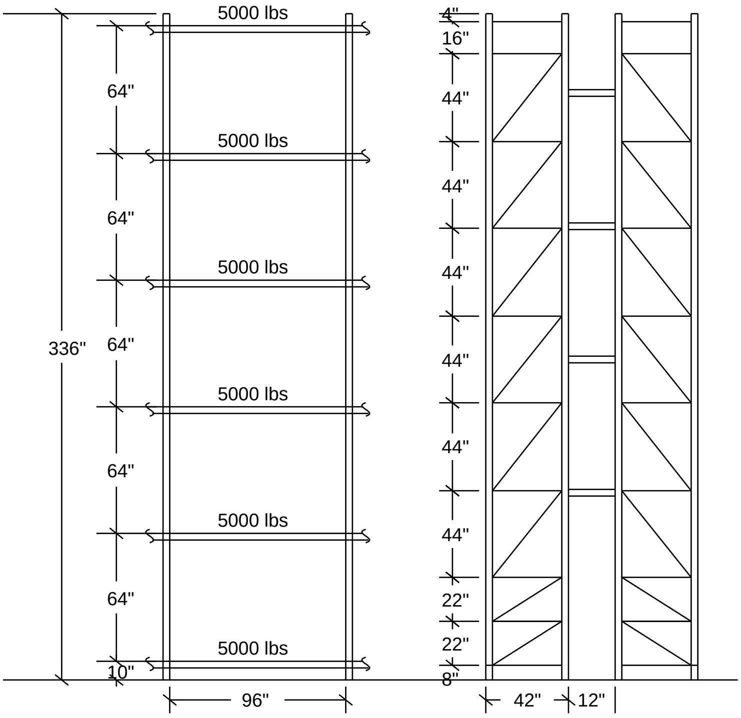
PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 3
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS Configuration 1: Type S-03 MHE 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9

S_s = 0.11 F_a = 1.6 I = 1 V_{Long} = 414 lbs. P_{static} = 15300 lbs.
S_1 = 0.07 F_v = 2.4 SDC = B V_{Trans} = 621 lbs.



FRONT VIEW SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 94% (level 3) **HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 16% (panel 1) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 36% (panel 3)	4 x 2.5 -0.063 Steel = 55 ksi Max Static Cap. = 5222 lb. Stress = 97% Max stress = 97% (level 1)	3 Pin 2" cc Connector Stress = 37% Max stress = 45% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 18%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 28% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 71%

Notes:
Bracing panel spacing adjusted from MHE's standard.

This calculation summary applies only to back-to-back rows.
Standard row spacers are required in this profile.

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PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 4
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 3   Panels: 9

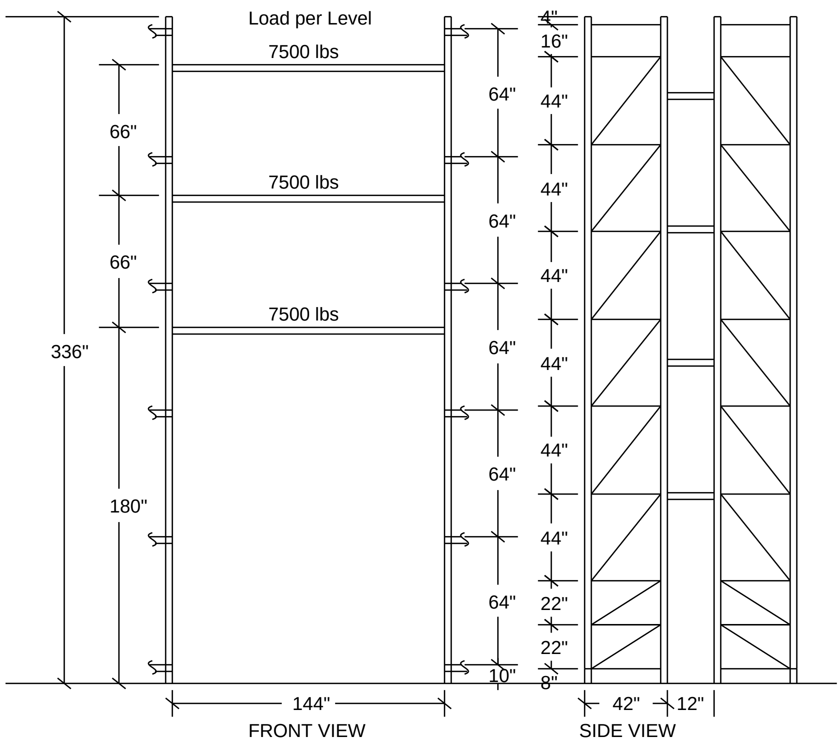
Configuration 2: Type T-01 - Tunnel
Adjacent to: Type S-03

MHE 1.7

$S_s = 0.11$ $F_a = 1.6$ $I = 1$
$S_1 = 0.07$ $F_v = 2.4$ SDC = B

$V_{Long} = 360$ lbs.
$V_{Trans} = 541$ lbs.

$P_{static} = 13350$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 91% (level 1 adj.) **HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 14% (panel 1) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 35% (panel 3)	6.5 x 2.5 -0.075 Steel = 55 ksi Max Static Cap. = 8598 lb. Stress = 88% Max stress = 88% (level 1)	4 Pin 2" cc Connector Stress = 19% Max stress = 19% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 16%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 24% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 71%

Notes:
Analyzed as interior tunnel bay. Adjacent to Type S-03.

Independently floor mounted column protectors required at tunnel bay.

Bracing panel spacing adjusted from MHE's standard.

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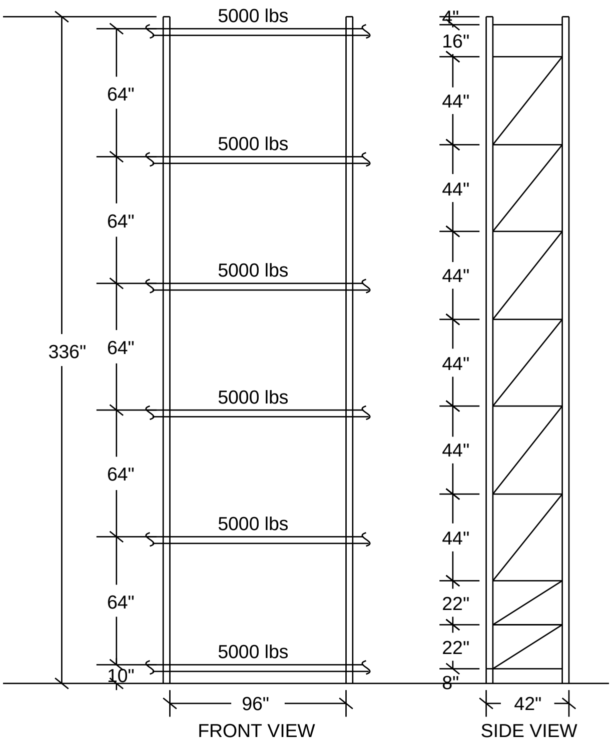
PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 5
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS   Configuration 3: Type S-03 - Single Row   MHE 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 6   Panels: 9   Load per Level

$S_s = 0.11$ $F_a = 1.6$ $I = 1$ $V_{Long} = 414$ lbs. $P_{static} = 15300$ lbs.
$S_1 = 0.07$ $F_v = 2.4$ SDC = B $V_{Trans} = 621$ lbs. $P_{seismic} = 2622$ lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 94% (level 3) **HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 16% (panel 1) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 36% (panel 3)	4 x 2.5 -0.063 Steel = 55 ksi Max Static Cap. = 5222 lb. Stress = 97% Max stress = 97% (level 1)	3 Pin 2" cc Connector Stress = 37% Max stress = 45% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 18%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 28% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 71%

Notes:
Bracing panel spacing adjusted from MHE's standard.

(2) 1/2" anchors per 7 x 6.375" base plate. Cross aisle top ties to other back-to-back rows as alternative.

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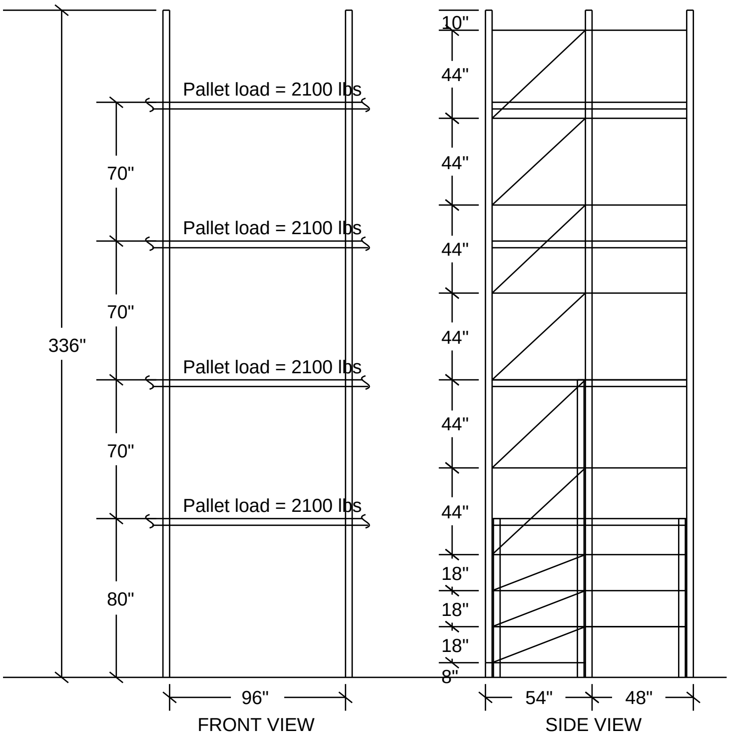
PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 6
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS Configuration 4: Type P-03 MHE 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 9

S_s = 0.11 F_a = 1.6 I = 1 V_{Long} = 462 lbs. P_{static} = 17000 lbs.
S_1 = 0.07 F_v = 2.4 SDC = B V_{Trans} = 693 lbs.



EXTERIOR FRAME		EXT. BEAM	EXT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 63% (level 2) **BACKER TO LEVEL 1** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 66% (level 1)	**HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 25% (panel 4) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 46% (panel 4)	C 4 x 4.5 Steel = 50000 psi Stress = 65% (level 1)	3 Pin 2" cc Connector Stress = 38% (level 1)
INTERIOR FRAME		**INTERIOR BEAM**	**INT. CONNECTOR**
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 78% (level 3)	**BACKER TO LEVEL 2** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static: 89% (level 1)	C 5 x 6.7 Steel = 50000 psi Stress = 75% (level 1)	3 Pin 2" cc Connector Stress = 62% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * **EXTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 20% **INTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 20%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 31% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 1%

Notes:
Analyzed as (2) pallets deep x (2) pallets wide over (1) frame, (1) space, and (1) monopost. 2,100 LBs per pallet.

Bracing panel spacing adjusted from MHE's standard.

(2) 1/2" anchors per interior 7 x 6.375" base plate.

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ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 7
CALCULATED BY: kelvira
DATE: 1/31/2023

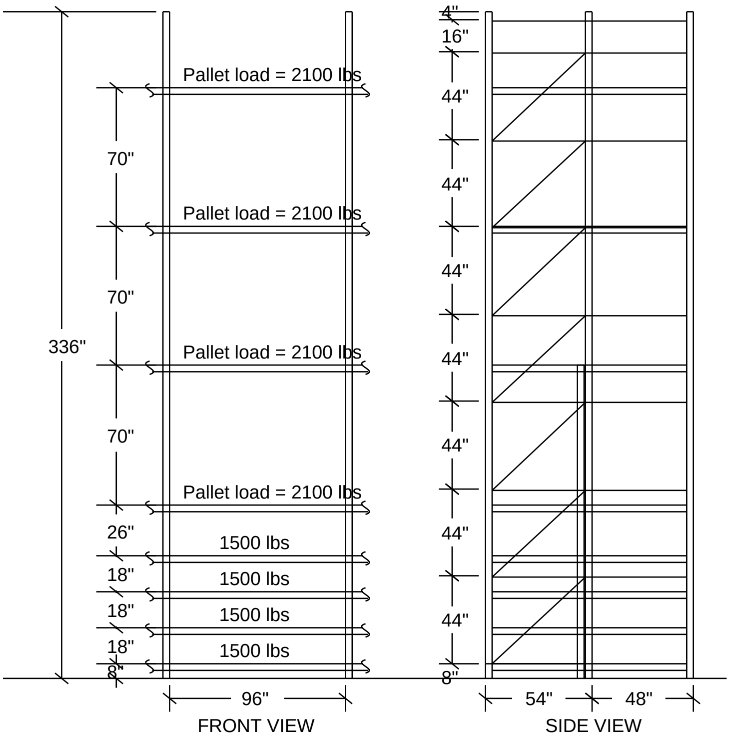
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS Configuration 5: Type C-02 - EXT MHE 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 8 Panels: 8

$S_s = 0.11$ $F_a = 1.6$ $I = 1$ $V_{Long} = 554$ lbs. $P_{static} = 20200$ lbs.
$S_1 = 0.07$ $F_v = 2.4$ SDC = B $V_{Trans} = 831$ lbs.



EXTERIOR FRAME		EXT. BEAM	EXT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 74% (level 6)	**HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 29% (panel 1) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 60% (panel 1)	C 4 x 4.5 Steel = 50000 psi Stress = 65% (level 6)	3 Pin 2" cc Connector Stress = 25% (level 6)

INTERIOR FRAME		INTERIOR BEAM	INT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 78% (level 7)	**BACKER TO LEVEL 6** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress: 70% (level 6)	C 5 x 6.7 Steel = 50000 psi Stress = 75% (level 5)	3 Pin 2" cc Connector Stress = 39% (level 6)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * **EXTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 24% **INTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 24%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 37% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 2%

Notes:
Levels 1-4: Carton flow, 1,500 LBs per level. 3" x 16 GA beam.
Levels 5-8: Pushback, 2,100 LBs per pallet.

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FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 8
CALCULATED BY: kelvira
DATE: 1/31/2023

PN: 20220622_4

COMPONENTS AND SPECIFICATIONS   Configuration 6: Type C-02 - INT   MHE 1.7

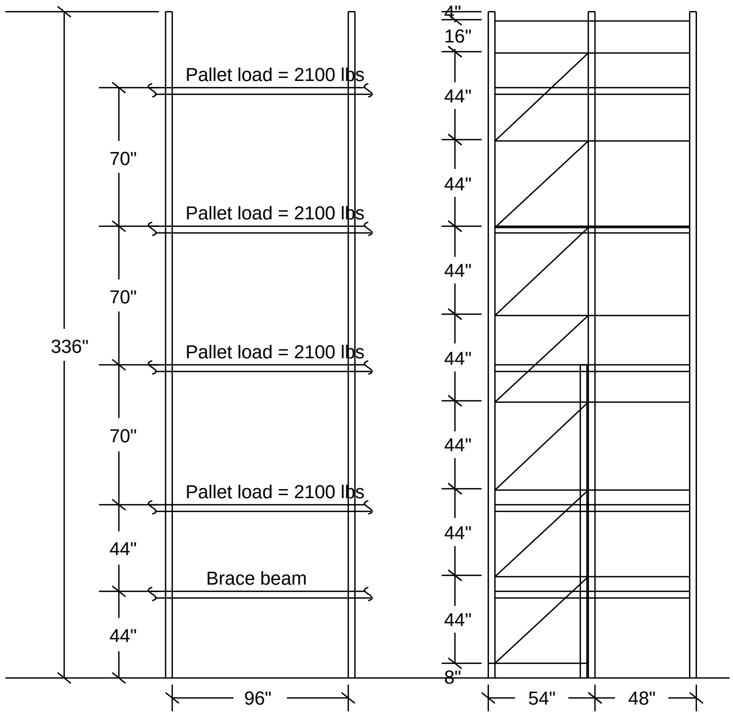
Analysis per section 2209 of the 2018 IBC

Levels: 5   Panels: 8

S_s = 0.11 F_a = 1.6 I = 1
S_1 = 0.07 F_v = 2.4 SDC = B

V_{Long} = 465 lbs.
V_{Trans} = 697 lbs.

P_{static} = 17050 lbs.



FRONT VIEW   SIDE VIEW

EXTERIOR FRAME		EXT. BEAM	EXT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 63% (level 3)	**HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 26% (panel 1) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 54% (panel 1)	C 4 x 4.5 Steel = 50000 psi Stress = 65% (level 2)	3 Pin 2" cc Connector Stress = 23% (level 2)

INTERIOR FRAME		INTERIOR BEAM	INT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 78% (level 4)	**BACKER TO LEVEL 3** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static: 60% (level 3)	C 5 x 6.7 Steel = 50000 psi Stress = 75% (level 2)	3 Pin 2" cc Connector Stress = 36% (level 3)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * **EXTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 20% **INTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 20%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 31% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 1%

Notes:
Level 1: Interior column 1.5" x 16 GA brace beam. No load.
Levels 2-5: Pushback, 2,100 LBs per pallet.

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PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 9
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 3 Panels: 9

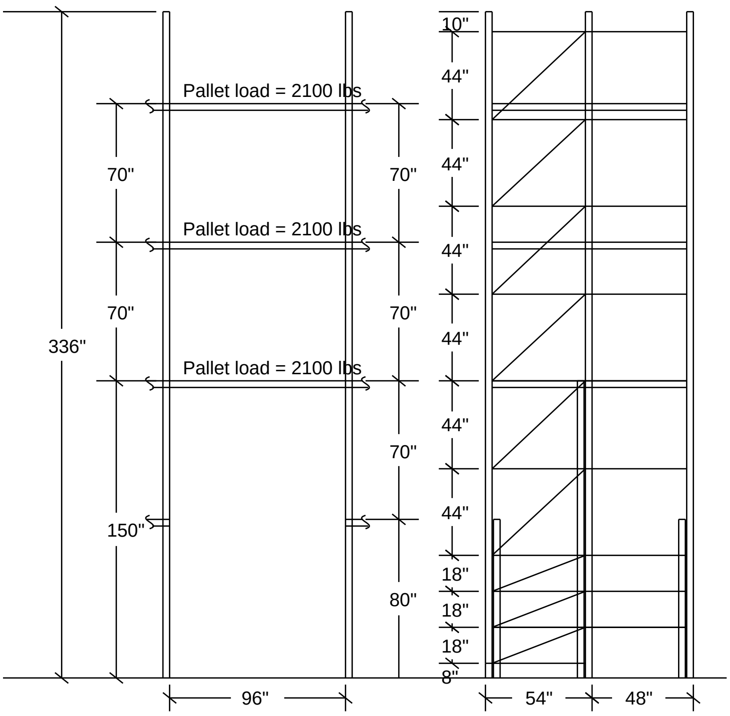
Configuration 7: Type T-02
Adjacent to: Type P-03

MHE 1.7

$S_s = 0.11$ $F_a = 1.6$ $I = 1$
$S_1 = 0.07$ $F_v = 2.4$ SDC = B

$V_{Long} = 404$ lbs.
$V_{Trans} = 606$ lbs.

$P_{static} = 14875$ lbs.



FRONT VIEW SIDE VIEW

EXTERIOR FRAME		EXT. BEAM	EXT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 63% (level 1) **BACKER TO LEVEL 1** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Stress = 57% (level 1)	**HORIZONTAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 22% (panel 4) **DIAGONAL BRACE** 1.50 x 1.25 x 0.375 -0.063 Stress = 43% (panel 4)	C 4 x 4.5 Steel = 50000 psi Stress = 65% (level 1)	3 Pin 2" cc Connector Stress = 22% (level 1)

INTERIOR FRAME		INTERIOR BEAM	INT. CONNECTOR
COLUMN 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static = 78% (level 2)	**BACKER TO LEVEL 2** 3.0 x 3.0 x 0.6875 -0.1050 Steel = 55000 psi Static: 77% (level 1)	C 5 x 6.7 Steel = 50000 psi Stress = 75% (level 1)	3 Pin 2" cc Connector Stress = 34% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * **EXTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 18% **INTERIOR** 7 x 6.375 x 0.375 in. 2 anchors/plate Moment = 0 in-lb. Stress = 18%	Slab = 7" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 27% (S)	Hilti Kwik Bolt TZ 2 (KB-TZ2) ESR-4266 0.5 in. x 3.75 in. Embed. Pullout Capacity = 3097 lbs. Shear Capacity = 4468 lbs. Anchor stress = 1%

Notes:
Analyzed as (2) pallets deep x (2) pallets wide over (1) frame, (1) space, and (1) monopost. 2,100 LBs per pallet.
Bracing panel spacing adjusted from MHE's standard.
Adjacent to Type P-03, also valid for Type C-02

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PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 10
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

Loads and Distributions: Type S-03

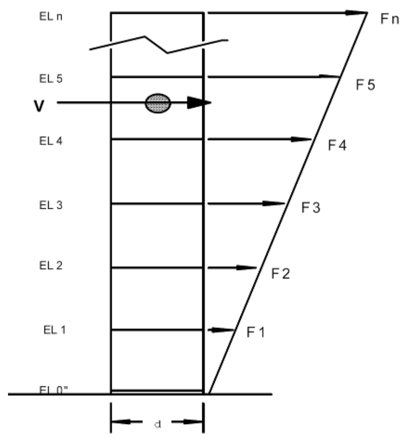
Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels:	6	SDC:	B	R_L:	6	Ss:	0.11
Pallets Wide:	2	W_{PL}:	30000	R_T:	4	S1:	0.07
Pallets Deep:	1	W_{DL}:	600 lbs	Fa:	1.6	Ip:	1
Pallet Load:	2500	Fv:	2.4	Tl:	1.5		
Total Frame Load:	30600 lbs						

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

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

$W_s = 0.67 \cdot W_{PL} + W_{DL} =$ **20700 lbs**



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

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

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

$= 0.11 / (1.5 \cdot 6) \cdot 1 \cdot 20700 =$ **253 lbs**

V_{long} need not be greater than:

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

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

$= 0.12 / 6 \cdot 1 \cdot 20700 =$ **414 lbs**

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

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

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

$= 0.5 \cdot 0.07 / 6 \cdot 1 \cdot 20700 =$ **115.58 lbs**

V_{long} shall not be less than:

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

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

$= \mathrm{Max}[0.01, 0.01, 0.03] \cdot 1 \cdot 20700 =$ **621 lbs**

Since: 253 < 621
& 621 > 115.58

V_{long} = 414 lbs

Transverse

V_{trans} need not be greater than:

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

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

$= 0.12 / 4 \cdot 1 \cdot 20700 =$ **621 lbs**

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

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

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

$= 0.5 \cdot 0.07 / 4 \cdot 1 \cdot 20700 =$ **173.36 lbs**

V_{trans} shall not be less than:

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

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

$= \mathrm{Max}[0.01, 0.01, 0.03] \cdot 1 \cdot 20700 =$ **621 lbs**

Since: 621 ≥ 173.36
& 621 ≥ 621

V_{trans} = 621 lbs

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Loads and Distributions: Type S-03 (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	10	2550	25500	17.91	2550	25500	26.62
2	74	2550	188700	29.02	2550	188700	43.55
3	138	2550	351900	54.12	2550	351900	81.21
4	202	2550	515100	79.22	2550	515100	118.88
5	266	2550	678300	104.32	2550	678300	156.54
6	330	2550	841500	129.41	2550	841500	194.2

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	6
min. # of bays	3
N_c	72
N_b	8
k_c	300 kip-in/rad
k_{be}	2863 kip-in/rad
k_b	140 kip-in/rad
k_{ce}	560 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	1.57 in^4
H	330 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	38 in	5 kip
2	102 in	5 kip
3	166 in	5 kip
4	230 in	5 kip
5	294 in	5 kip
6	360 in	5 kip

Calculated T = 4.86

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 S-03

2018 IBC& RMI / ANSI MH 16.1

V_{Trans} = **621 lbs**
M_{Trans} = $\Sigma(f_{Trans} \cdot h_x)$ = **144,435 in-lbs**
β = 0.7

V_{Long} = **414 lbs**
E_{Trans} = M_{Trans} / frame depth = **3,438 lbs**
β = 1.0 (Uplift combination only)

P = Product Load / 2 = **15,000 lbs**
ρ = 1

D = Dead Load · 0.5 = **300 lbs**
S_{DS} = .12

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 · 300) + (1.2 · 15,000)= **18,420 lbs**

2. Gravity Load = 1.2 D + 1.4 P + 1.6 L + 0.5 (L_r or S or R)
= (1.2 · 300) + (1.4 · 15,000) + (1.6 · 0) + (0.5 · 0)= **21,360 lbs**

3. Snow/Rain = 1.2D + 0.85P + (0.5L or 0.5W) + 1.6(L_r or S or R)
= (1.2 · 300) + (0.85 · 15,000) + (0.5 · 0) + (1.6 · 0)= **13,110 lbs**

4. Wind Load = 1.2D + 0.85P + 0.5L + 1.0W + 0.5(L_r or S or R)
= (1.2 · 300) + (0.85 · 15,000) + (0.5 · 0) + (1.0 · 0) + (0.5 · 0)= **13,110 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 · .12) · 300 + (1.2 + 0.2 · .12) · 0.7 · 15,000 + 0.5 · 0 + 1 · 3,438 + 0.2 · 0= **16,658 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 · .12) · 300 + (1.2 + 0.2 · .12) · 0.7 · 15,000 + 0.5 · 0 + 1 · 0 + 0.2 · 0= **13,219 lbs**

6. Wind Uplift = $0.9D + 0.9P_{app} + 1.0W$
= 0.9 · 300 + 0.9 · 15,000 + 1.0 · 0 = **270 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 · .12) · 300 + (0.9 - 0.2 · .12) · 1 · 15,000 - 1 · 3,438= **9,963 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 · .12) · 300 + 0.75((1.4 + 0.14 · .12) · 0.7 · 15,000 + 0.7 · 1 · 3,438)= **13,266 lbs**

2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$
= (1 + 0.14 · .12) · 300 + (0.85 + 0.14 · .12) · 0.7 · 15,000 + 0.7 · 1 · 3,438= **11,813 lbs**

3. D + P
= 300 + 15,000 = **15,300 lbs**

Longitudinal Analysis: Type S-03

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).

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

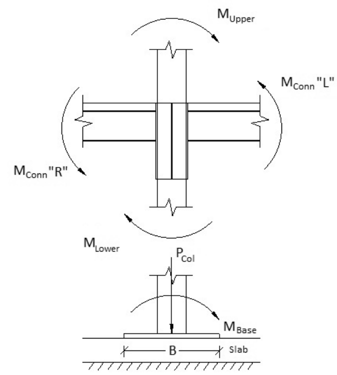
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

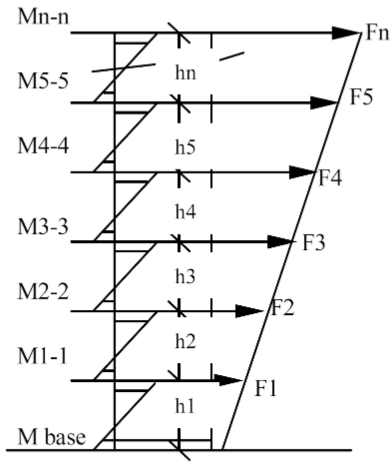
$V_{Col} = V_{Long}$ / # of columns = **207 lbs**

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

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

(207 lbs · 8 in.) - 0 in-lbs = 1656 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	10	9	15,300	1,656	4,843	12,088
2	64	15	12,750	12,834	4,843	17,677
3	64	27	10,200	12,834	4,843	17,677
4	64	40	7,650	12,834	4,843	17,677
5	64	52	5,100	12,834	4,843	17,677
6	64	65	2,550	12,834	4,843	11,260

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ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 15
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COLUMN ANALYSIS: Type S-03 (Level 3)

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

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

$K_x \cdot L_x / R_x$ = $1.5 \cdot 62 / 1.251$ = **74.37**

$K_y \cdot L_y / R_y$ = $1 \cdot 44 / 1.088$ = **40.44**

KL/R_{max} = **74.37**

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

$= (1.251^2 + 1.088^2 + -2.761^2)^{1/2} =$ **3.22 in.**

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

$= 1 - (-2.761/3.22)^2 =$ **0.265**

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

$= 3.14^2 \cdot 29500 / 74.37^2 =$ **52.641 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)

$= (1 / (2 \cdot 0.265)((52.641 + 66.795) - (52.641 + 66.795)^2$

$- (4 \cdot 0.265 \cdot 52.641 \cdot 66.795))^{1/2}) =$ **31.664 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 74.37^2 =$ **52.641 ksi**

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

$= 1 / 1.002 \cdot 3.22^2 (11300 \cdot 0.004$

$+ (3.14^2 \cdot 29500 \cdot 2.776) / (0.8 \cdot 44)^2) =$ **66.795 ksi**

$F_e = \mathrm{Min}(F_{e1}, F_{e2}) =$ **31.664 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 31.664)^{1/2} =$ **1.318** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658\text{^}(\lambda_c^2)) \cdot F_y =$ **26.584** (Eq C4.1-2)

Thus:

P_n = **19310 lbs**

P_a = **16413 lbs**

3.0 x 3.0 x 0.6875 -0.1050	
SECTION PROPERTIES	
Depth	**3 in.**
Width	**3 in.**
t	**0.105 in.**
Radius	**0.188 in.**
Area	**1.002 in.^{2}**
AreaNet	**0.726 in.^{2}**
I_x	**1.567 in.^{4}**
S_x	**1.045 in.^{3}**
$S_{x\ Net}$	**0.816 in.^{3}**
R_x	**1.251 in.**
I_y	**1.186 in.^{4}**
S_y	**0.679 in.^{3}**
R_y	**1.088 in.**
J	**0.004 in.^{4}**
C_w	**2.776 in.^{6}**
J_x	**3.058 in.**
X_o	**-2.761 in.**
K_x	1.5
L_x	**62 in.**
K_y	1
L_y	**44 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**70 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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Lee's Summit, MO
SHEET#: 16
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

COLUMN ANALYSIS: Type S-03 (Level 3)

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

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

$P_{ao} = P_{no}\phi_c =$ **33959 lbs**

Where:

$P_{no} = A_e F_y = 0.726 \cdot 55 =$ **39952 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **183.147 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_t / \sigma_{ex}))^{1/2}) / (C_{TF}S_f) =$ **85.14 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 EdI_{yc}) / (S_f(K_y L_y)^2 =$ **512.19 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **85.14 ksi**

Since: $0.56\ F_y < 2.78F_y$

$F_c = (10/9)F_y(1 -(10F_y / 36F_e)) =$ **50.1 ksi** (Eq C3.1.2.1-2)

Reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c =$ **50.1 ksi**

$M_{nx} =$ **40861 in-lbs** $M_{ny} =$ **34022 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **36775 in-lbs** $M_{ny}\phi_b =$ **30620 in-lbs**

$P_{Ex} = \Pi^2\ EI_x / (K_x L_x)^2 =$ **52750 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2\ EI_y / (K_y L_y)^2 =$ **178361 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.855** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.957** (Eq C5.2.2-5)

$P_{trans} =$ **16,658 lbs** $P_{long} =$ **13,219 lbs**

$M_u = M_x =$ **5122 in-lbs** (Eq C5.2.2-2)

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

$P_{u\_st} / P_a = 14240 / 16413 = 0.87$ Static Stress = **87%**

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)

= ((13,219 / 16413) + (5122 / 36775) + (1 / 30620)) = **94%**

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)

= (13,219 / 33959) + (0.85 · 5122 / 36775 · 0.855)) + (0.85 · 1 / 30620 · 0.957))) = **52%**

Stress3 P_t / P_{ao} = 16,658 / 33959 =**49%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **94%**

3.0 x 3.0 x 0.6875 -0.1050	
SECTION PROPERTIES	
Depth	**3 in.**
Width	**3 in.**
t	**0.105 in.**
Radius	**0.188 in.**
Area	**1.002 in. ^{2}**
AreaNet	**0.726 in. ^{2}**
I_x	**1.567 in. ^{4}**
S_x	**1.045 in. ^{3}**
$S_{x\ Net}$	**0.816 in. ^{3}**
R_x	**1.251 in.**
I_y	**1.186 in. ^{4}**
S_y	**0.679 in. ^{3}**
R_y	**1.088 in.**
J	**0.004 in. ^{4}**
C_w	**2.776 in. ^{6}**
J_x	**3.058 in.**
X_o	**-2.761 in.**
K_x	1.5
L_x	**62 in.**
K_y	1
L_y	**44 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**70 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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BEAM ANALYSIS Type S-03

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

Effective width w = C - 2t - 2r = 1.5 - (2 · 0.063) - (2 · 0.125) = **1.12 in.**

w/t = 1.124 / 0.063 = **17.84**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (F_y / E)^{1/2} = (1.052 / 2) \cdot 17.841 \cdot (55 / 29500)^{1/2} = \mathbf{0.41}$

λ <= 0.673: Flange is fully effective.

Check web for local buckling (B2.3)

f_1(comp) $= F_y \cdot (y_3 / y_2) = 55 * 1.97 / 2.16 = \mathbf{50.22\ ksi}$

f_2(tension) $= F_y \cdot (y_1 / y_2) = 55 * 1.65 / 2.16 = \mathbf{42.0\ ksi}$

$\Psi = -(f_2 / f_1) = -(42.0 / 50.22) = \mathbf{-0.84}$

Buckling coefficient $k = 4 + 2 \cdot (1 - \Psi)^3 + 2 \cdot (1 - \Psi)$
$= 4 + 2(1 - \mathbf{-0.84})^3 + 2(1 - \mathbf{-0.84}) = \mathbf{20.06}$

Flat Depth w = y1 + y3 = 1.65 + 1.97 = **3.624**

w/t = 3.624/0.063 = **57.52** **w/t < 200: OK**

$\lambda = (1.052 / k^{1/2}) \cdot (w/t) \cdot (f_1 / E)^{1/2} = (1.052 / 2) \cdot 57.524 \cdot (50.22 / 29500)^{1/2} =$
0.56

$b1 = w \cdot (3 - \Psi) = 4 \cdot (3 - -0.84) = \mathbf{13.9}$

b2 = w/2 = **1.81**

b1 + b2 = 13.9 + 13.9 = **15.72** **Web is fully effective**

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

Corner cross-sectional area $Lc = (\Pi / 2) \cdot (r + t / 2)$
$= (\Pi / 2) \cdot (0.125 + 0.063 / 2) = \mathbf{0.246}$

L_f = effective width = **1.124**

$C = 2 \cdot L_c / L_f + 2 \cdot L_c = 2 \cdot 0.246 / 1.124 + 2 \cdot L_c = \mathbf{0.3043}$

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

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

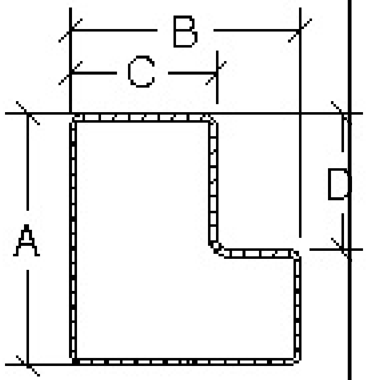
Fu/Fy = 70 / 55 = **1** **>= 1.2 = OK**

r/t = 0.125 / 0.063 = **1.984** **<= 7 = OK**

$F_{yc} = B_c \cdot F_y / (r / t)^m = 1.58 \cdot 55 / (1.984)^m = \mathbf{77}$

$F_{ya\text{-}top} = C \cdot F_{yc} + (1 - C) \cdot F_y = 0.304 \cdot 77 + (1 - 0.304) \cdot 55 = \mathbf{62}$

$F_{ya\text{-}bottom} = F_{ya\text{-}top} \cdot Y_{cg} / (A - Y_{cg}) = 62 \cdot 1.84 / (4.0 - 1.84) = \mathbf{52}$



4 x 2.5 -0.063

Top flange width C =	1.5 in.
Bottom width B =	4.0 in.
Web depth A =	4.0 in.
Beam thickness t =	0.063 in.
Radius r =	0.125 in.
Fy =	55
Fu =	70
Y1 =	1.65
Y2 =	2.16
Y3 =	1.97
Ycg =	1.84
Ix =	1.55
Sx =	0.72
E =	29500
FBeam F =	300
Beam Length L =	96

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Lee's Summit, MO
SHEET#: 18
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

BEAM ANALYSIS Type S-03

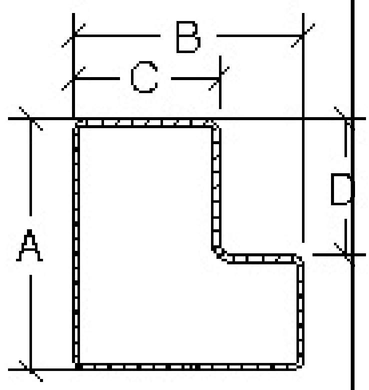
Check Allowable Tension Stress for Bottom Flange

$L_{flange\text{-}bot} = B - (2 \cdot r) - (2 \cdot t) = 4.0 - (2 \cdot 0.125) - (2 \cdot 0.063) = \mathbf{3.62}$

$C_{bottom} = 2 \cdot L_c / (L_{flange\text{-}bot} + 2 \cdot L_c) = 2 \cdot 0.246 / (3.62 + 2 \cdot 0.246) = \mathbf{0.119}$

$F_{y\text{-}bottom} = C_{bottom} \cdot F_{yc} + (1 - C_{bottom}) \cdot F_y = 0.119 \cdot 77 + (1 - 0.119) \cdot 55 = \mathbf{57.63}$

$F_{ya} = F_{ya\text{-}top} = \mathbf{61.69\ ksi}$



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:

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

$R_m = 1 - ((2 \cdot F \cdot L) / (6 \cdot E \cdot I_x + 3 \cdot F \cdot L))$
$= 1 - ((2 \cdot 300 \cdot 96) / (6 \cdot 29500 \cdot 1.55 + 3 \cdot 300 \cdot 96)) = \mathbf{0.84}$

$\phi \cdot M_n = \phi \cdot F_{ya} \cdot S_x = \mathbf{42.12\ in\text{-}kip}$

$W = \phi \cdot M_n \cdot 8 \cdot (\#\ of\ Beams) / (L \cdot R_m \cdot \Omega) = (42.12 \cdot 8 \cdot 2) / (96 \cdot 0.84 \cdot 1.6)$
$= \mathbf{5222}$ lbs/pair

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

$R_d = 1 - (4 \cdot F \cdot L) / (5 \cdot F \cdot L + 10 \cdot E \cdot I_x)$
$= 1 - (4 \cdot 300 \cdot 96) / (5 \cdot 300 \cdot 96 + 10 \cdot 29500 \cdot 1.55) = \mathbf{0.81}$

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

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

$W = (384 \cdot E \cdot I_x \cdot 2) / (180 \cdot 5 \cdot L^2 \cdot R_d)$
$= (384 \cdot 29500 \cdot 1.55 \cdot 2) / (180 \cdot 5 \cdot 96^2 \cdot 0.81) \cdot 1000 = \mathbf{5246}$ lbs/pair

4 x 2.5 -0.063

Top flange width C =	1.5 in.
Bottom width B =	4.0 in.
Web depth A =	4.0 in.
Beam thickness t =	0.063 in.
Radius r =	0.125 in.
Fy =	55
Fu =	70
Y1 =	1.65
Y2 =	2.16
Y3 =	1.97
Ycg =	1.84
Ix =	1.55
Sx =	0.72
E =	29500
FBeam F =	300
Beam Length L =	96

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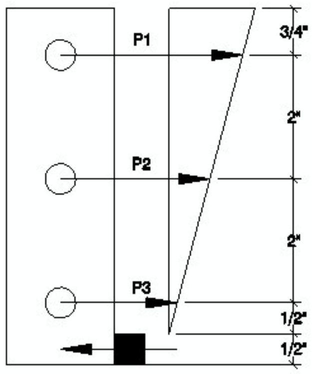
Allowable and Actual Bending Moment at Each Level

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

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	30,576	31,332	4,348	31,332	Pass
2	30,576	31,332	5,215	31,332	Pass
3	30,576	31,332	4,550	31,332	Pass
4	30,576	31,332	3,483	31,332	Pass
5	30,576	31,332	2,635	31,332	Pass
6	30,576	31,332	2,422	31,332	Pass

Beam to Column Analysis: Type S-03

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 = **0.11 in.**

Since $d / t_c < 10$ C = 3

$m_f =$ **1.0**

$F_u =$ **65000 psi**

$P_n =$ $C \cdot m_f \cdot d \cdot t_c \cdot F_u$ = **7166.25 lbs** AISI E3.3.1 -1

$P_{Bearing} =$ $\phi P_n = 0.75 \cdot 7166.25$ = **5374 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**

BRACE ANALYSIS Type S-03 (Panel 3)

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

1.50 x 1.25 x 0.375 - 0.063	
SECTION PROPERTIES	
Depth	1.5 in.
Width	1.5 in.
t	0.063 in.
Radius	0.1 in.
Area	0.269 in^2
AreaNet	0.269 in^2
Ix	0.099 in^4
Sx	0.131 in^3
Sx net	0.131 in^3
Rx	0.605 in.
Iy	0.057 in^4
Sy	0.078 in^3
Ry	0.458 in.
J	0 in^4
Cw	0.035 in^6
Jx	1.299 in.
Xo	-1.149 in.
Kx	0
Lx	52 in.
Ky	1
Ly	52 in.
Kt	0.8
Fyv	55 ksi
Fuv	70 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)

$K_x \cdot L_x / R_x = 0 \cdot 52 / 0.61 = \mathbf{85.94}$
$K_y \cdot L_y / R_y = 1 \cdot 52 / 0.46 = \mathbf{113.44}$
$KL / R_{max} = \mathbf{113.44}$

$r_o = (r_x^2 + r_y^2 + x_o^2)^{1/2}$
$= (0.61^2 + 0.46^2 + -1.15^2)^{1/2} = \mathbf{1.38\ in.}$ (Eq C3.1.2.1-7)

$\beta = 1 - (x_o / r_o)^2 = 1 - (-1.15 / 1.38)^2 = \mathbf{0.3}$ (Eq C4.1.2-3)

$F_{e1} = \Pi^2 E / (KL / r)_{max}^2 = 3.14^2 \cdot 29500 / 113.44^2 = \mathbf{22.626\ ksi}$ (Eq C4.1.1-1)

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

$= (1 / (2 \cdot 0.3)((39.42 + 16.28) - ((39.42 + 16.28)^2 - (4 \cdot 0.3 \cdot 39.42 \cdot 16.28))^{1/2}) = \mathbf{12.355\ ksi}$ (Eq C4.1.2-1)

where:

$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$
$= 3.14^2 \cdot 29500 / 113.44^2 = \mathbf{39.425\ ksi}$ (Eq C3.1.2-11)

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$
$= 1 / 0.27 \cdot 1.38^2 (11300 \cdot 0.0004 + (3.14^2 \cdot 29500 \cdot 0.04) / (0.8 \cdot 52)^2) = \mathbf{16.279\ ksi}$ (Eq C3.1.2-9)

$F_e = \mathrm{Min}(Fe1, Fe2) = \mathbf{12.355\ ksi}$

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 12.355)^{1/2} = \mathbf{2.11}$ (Eq C4.1-4)
Since $\lambda_c >= 1.5$, $F_n = (0.877 / (\lambda_c^2)) \cdot F_y = \mathbf{10.835}$ (Eq C4.1-3)

Thus
$P_n = \mathbf{2{,}917\ lbs}$
$P_a = P_n \cdot \phi_c = \mathbf{2{,}479\ lbs}$

BRACE ANALYSIS Type S-03 (Panel 3)

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

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

$P_{ao} = P_{no}\phi_c = 14{,}806 \cdot 0.85 = \mathbf{12{,}585\ lbs.}$

Where $P_{no} = A_e F_y = 0.27 \cdot 55 = \mathbf{14{,}806\ lbs.}$

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f = 71.47$ ksi (Eq C3.1.2.1-4)

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = 22.02$ ksi (Eq C3.1.2.1-10)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2) = 69.45$ ksi (Eq C3.1.2.1-14)

$F_{e,min} = \mathbf{22.02\ ksi}$

Since, $F_e <= 0.56F_y$

$F_c = F_e = \mathbf{22.02\ ksi}$ (Eq C3.1.2.1-3)

reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = \mathbf{22\ ksi}$

$M_{nx} = \mathbf{2{,}891\ in\text{-}lbs}$ $M_{ny} = \mathbf{1{,}705\ in\text{-}lbs}$ $M_c = M_{n,min}$

$M_{nx}\phi_b = \mathbf{2{,}602\ in\text{-}lbs}$ $M_{ny}\phi_b = \mathbf{1{,}535\ in\text{-}lbs}$

$P_{Ex} = \Pi^2 E I_x / (K_x L_x)^2 = \mathbf{10{,}606\ lbs}$ (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 E I_y / (K_y L_y)^2 = \mathbf{6{,}084\ lbs}$ (Eq C5.2.2-7)

Max $P_a = \mathbf{2{,}917\ lbs}$

$V_{Trans} = \mathbf{605\ lbs}$

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} = \mathbf{52.35\ in.}$

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = \mathbf{880.38\ lbs.}$

Brace Stress = $V_{Diag} / P_a = \mathbf{36\%}$

1.50 x 1.25 x 0.375 - 0.063

SECTION PROPERTIES	
Depth	1.5 in.
Width	1.5 in.
t	0.063 in.
Radius	0.1 in.
Area	0.269 in^2
AreaNet	0.269 in^2
Ix	0.099 in^4
Sx	0.131 in^3
Sx net	0.131 in^3
Rx	0.605 in.
Iy	0.057 in^4
Sy	0.078 in^3
Ry	0.458 in.
J	0 in^4
Cw	0.035 in^6
Jx	1.299 in.
Xo	-1.149 in.
Kx	0
Lx	52 in.
Ky	1
Ly	52 in.
Kt	0.8
Fyv	55 ksi
Fuv	70 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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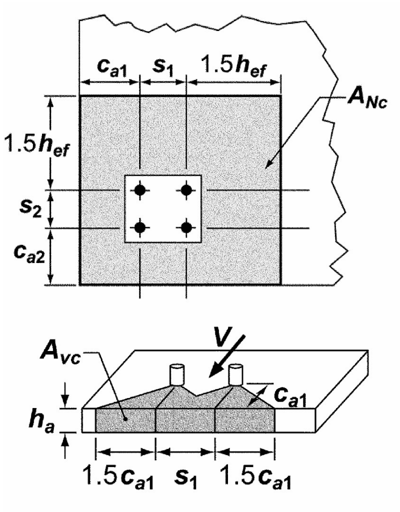
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 Type S-03

Assumed cracked concrete application

Anchor Type	**0.5" dia., 3.25 hef, 5.5" min, slab**				
ICC Report Number	**ESR-4266**			$1.5 \cdot h_{ef}$	= **4.875 in.**
Slab Thickness (h)	= **7 in.**	C_{a1}	= 12	use$C_{a1,adj}$	= **4.875 in.**
Min. Slab Thickness (h)	= **5.5 in.**	C_{a2}	= 12	use$C_{a2,adj}$	= **4.875 in.**
Concrete Strength (f_c)	= **4000 psi**				
Diameter (d_a)	= **0.5 in.**			$3 \cdot h_{ef}$	= **9.75 in.**
Nominal Embedment (h_{nom})	= **3.75 in.**				
Effective Embedment (h_{ef})	= Hef	S_1	= **5 in.**	Use $S_{1,adj}$	= **5 in.**
Number of Anchors (n)	= **2**	S_2	= **0 in.**	Use $S_{2,adj}$	= **0 in.**
e`N	= **0**				
e`V	= **0**				

From ICC ESR Report

A_{se}	= **0.099 sq.in.**
$f`_{uta}$	= **114000 psi**
S_{min}	= **2 in.**
C_{min}	= **2.25 in.**
C_{ae}	= **8 in.**
$N_{p,cr}$	= **9999 lbs**



	$\phi_{Seismic}$	Adj. Strength
Tension Capacity = **3097 lbs**	1	**3097 lbs**
Shear Capacity = **4468 lbs**	1	**4468 lbs**

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ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Type S-03

Steel Strength 17.4.1

ϕ = **0.75** 17.3.3.a i

$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 2 \cdot 0.099 \cdot 114000 =$ **16,929 lbs** 17.4.1.2

Concrete Breakout Strength ϕN_{cbg} 17.4.2

ϕ = **0.65** 17.3.3 c ii Category 1-B

$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) =$ **143.813 sq.in.**

$A_{Nco} = 9h_{ef}^2 =$ **95.063 sq.in.**

Check if $A_{Nco} \geq A_{Nc}$ $A_{Nc}/A_{Nco} =$ **1.513**

$\Psi_{ec,N}$ = **1** 17.4.2.4

$\Psi_{ed,N}$ = **1** 17.4.2.5

$\Psi_{C,N}$ = **1** 17.4.2.6

K_c = **17**

λ_a = **1**

$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} =$ **6299 lbs** 17.4.2.2 d

$\Psi_{cp,N}$ = **1** 17.4.2.7

$\phi N_{cbg} = \phi(A_{Nc}/A_{Nco})(\Psi_{ec,N})(\Psi_{ed,N})(\Psi_{C,N})(\Psi_{cp,N})(N_b)$ 17.4.2.1

$0.65 \cdot (143.813/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299 =$ **6,194 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

$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} =$ **16,442 lbs** 17.4.3.1

Steel Strength (ϕN_{sa}) = **16,929 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **6,194 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **16,442 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Type S-03

Steel Strength ϕV_{sa} V_{sa}=6,875 / Anchor -- per report	17.5.1
ϕ = **0.65**	17.3.3. Condition a ii
$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 2 \cdot 6{,}875 =$ **8,938 lbs**	17.5.1.2a
Concrete Breakout Strength ϕV_{cbg}	17.5.2
ϕ = **0.7**	17.3.3 ci-B
$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **287 sq.in.**	
$A_{Vco} = 3C_{a1}h_a =$ **252 sq.in.**	
Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1.139**	
$\Psi_{ec,V}$ = **1**	17.5.2.5
$\Psi_{ed,V}$ = **0.9**	17.5.2.6
$\Psi_{C,V}$ = **1**	17.5.2.7
$\Psi_{h,V}$ = **1.604**	17.5.2.8
d_a = **0.5 in.**	17.5.2.2
L_e = **1 in.**	17.2.6 d
λ_a = **1**	
The smaller of $7(L_e / d_a)^{0.2}(d_a)^{0.5}\lambda_a(f_c)^{0.5}ca1^{1.5}$ and $9\lambda_a(f_c)^{0.5}ca1^{1.5}$ = **14,948 lbs**	17.5.2.2 a, 17.5.2.2 b
$\phi V_{cbg} = \phi(A_{Vc}/A_{Vco})(\Psi_{ec,V})(\Psi_{ed,V})(\Psi_{C,V})(\Psi_{h,V})(V_b)$	17.5.2.1
$0.7 \cdot (287/252) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.604 \cdot 14{,}948 =$ **17,199 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} = **9,529 lbs**	
$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 9{,}529 =$ **13,341 lbs**	

Steel Strength (ϕV_{sa}) = **8,938 lbs**

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **17,199 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **13,341 lbs**

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PROJECT: Zoetis
FOR: Material Handling Exch
ADDRESS: 1120 NW Main Stree
Lee's Summit, MO
SHEET#: 26
CALCULATED BY: kelvira
DATE: 1/31/2023
PN: 20220622_4

OVERTURNING ANALYSIS Configuration1 Type S-03

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} = **30,000 lbs** W_{dl} = **600 lbs**

$W_{pl} \cdot 67\% = 30{,}000 \cdot 0.67 =$ **20,100 lbs**

$V_{Trans} = (1 \cdot 0.03 \cdot 1 \cdot ((0.67 \cdot 20{,}100) + 600)) =$ **422 lbs**

$M_{ovt} = V_{Trans} \cdot Ht = 422 \cdot 261 =$ **110,142 in-lbs**

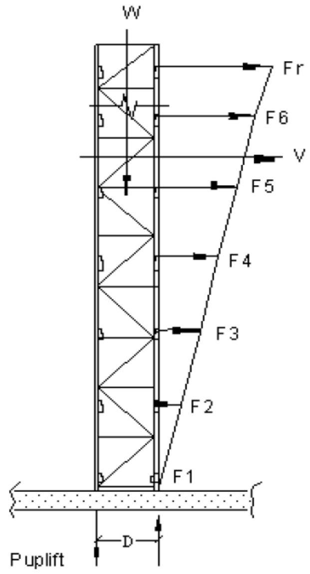
$M_{st} = ((W_{pl} \cdot 0.67) + W_{dl}) \cdot d \cdot Factor$

$= ((30{,}000 \cdot 0.67) + 600) \cdot 42 \cdot 0.5 =$ **434,700 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (110{,}142 - 434{,}700) / 42 =$ **-7,727 lbs**

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

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (110{,}142 + 434{,}700) / 42 =$ **12,972 lbs**



CROSS AISLE ELEVATION

TOP SHELF LOADED

Shear = **168 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 168 \cdot (330 + ((64 - 10) / 2)) =$ **59,976 in-lbs**

$M_{st} = (l + W_{dl}) \cdot d = (5{,}000 + 600) \cdot (42 \cdot 0.5) =$ **117,600 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (59{,}976 - 117{,}600) / 42 =$ **-1,372 lbs**

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

SIDE LOADS ON TOP SHELF

Top loaded shelf level (H) = **330 in.**

$M_{ovt} = 1.6 \cdot 350 \cdot H = 1.6 \cdot 350 \cdot 330 =$ **184kip**

$P_{uplift} = M_{ovt}/d = 184 / 42 =$ **4,400 lbs**

ANCHORS

No. of Anchors (#Anchors): **2**

Pull Out Capacity per Anchor (T_{Anchor}): **3,097 lbs**

Shear Capacity per Anchor: **4,468 lbs**

COMBINED STRESS

Fully Loaded	= ((0 / 2) / 3,097) + ((422 / 4) / 4,468)	= **0.024**
Top Shelf Loaded	= ((0 / 2) / 3,097) + ((168 / 4) / 4,468)	= **0.009**
Seismic UpLift Critical (LC#7B)	= (0 / 2) / 3,097	= **0**
Side Load Top Shelf	= **0.71**	

Base Plate Analysis: Type S-03

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} = **21360 lbs**

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

$P/A = P_{col}/(D \cdot B) = 21360 / (6.38 \cdot 7) =$ **479 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 0 / (6.38 \cdot 7^2 / 6) =$ **0 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 0 \cdot (2 \cdot 2/7) =$ **0 psi**

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

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

$= (2^2 / 2) \cdot (479 + 0 + 0.67 \cdot 0) =$ **957.31 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.16sq.in.**

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 957.31 / (0.16 \cdot 32,400) =$ **0.18**

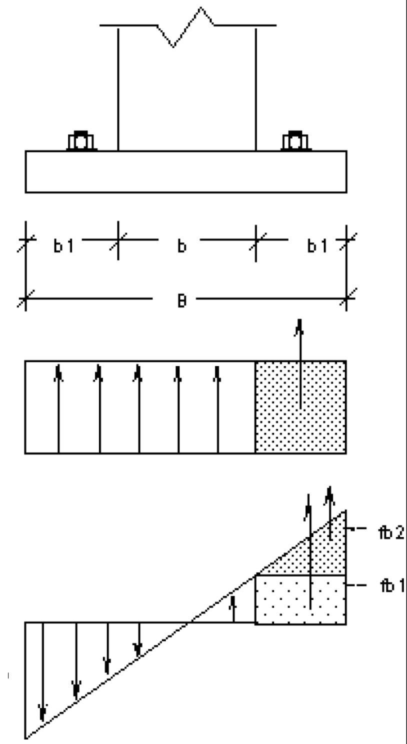


Plate width B =	7 in.
Plate depth D =	6.38 in.
Plate thickness t =	0.38 in.
Column width b =	3 in.
Column depth d =	3 in.
b1 =	2 in.
S_x =	5 in.
S_y =	0 in.
T_{Total} =	6,194 lbs.
T_e =	0.28 in.

Base Plate Tension analysis

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

Moment Arm (L) = $(S_x - b) / 2 =$ **1 in.**

$M_{anchor} = T_{Total} / 2 \cdot L =$ **3097 in-lbs**

$S = D \cdot t^2/6 =$ **0.086**in^3

$M_{baseplate} = S \cdot F_y =$ **3,097 in-lbs**

$\phi M_{baseplate} = 0.9 \cdot M_n =$ **2,787 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	6
min. # of bays	3
N_c	72
N_b	8
k_c	300 kip-in/rad
k_{be}	2863 kip-in/rad
k_b	140 kip-in/rad
k_{ce}	560 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	1.57 in^4
H	330 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	38 in	5 kip
2	102 in	5 kip
3	166 in	5 kip
4	230 in	5 kip
5	294 in	5 kip
6	360 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 = 1.97

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= 0.77

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}=15,485 kip-in $\geq M_b$ OK

SLAB AND SOIL ANALYSIS (LRFD)

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

P_{max} = Gravity_Load (see Basic Load Combinations) = **21,360 lbs**

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

$d,req'd$ $= (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **3.277 in.**

b $= (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **21.566 in.**

$b,req'd$ $= 1.5 \cdot b$ = **32 in.**

P_n $= 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **162,439 lbs**

P_a $= \phi \cdot P_a$ = **97,463 lbs**

P_{max} / P_a = **0.22**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **15,300 lbs**

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

P_n $= 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2$ = **162,439 lbs**

$d,req'd$ $= (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2}$ = **3.277 in.**

b $= (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4}$ = **21.566 in.**

$b,req'd$ $= 1.5 \cdot b$ = **32 in.**

P_a $= P_n / \Omega$ = **54,146 lbs**

P_{max} / P_a = **0.28**

Base Plate	
Width B	7 in.
Depth W	6.375 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	3.34 in
E_c	3,604,997 psi