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

Reviewed 4/19/2022 DLSubject to field inspectors approvals.

Analysis of Storage Racks
for

Life Science Logistics Puyallup

302 33Rd St SE, Puyallup, WA
Job No. **22-0685**

APPROVED TO PROCEEDSUBJECT TO SPECIAL INSPECTION REPORT 4/19/2022 DL

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

PRCTI20220562

EXPIRES
04-12-2022

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

SEIZMIC INC
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1130 E. CYPRESS ST, COVINA, CA 91724

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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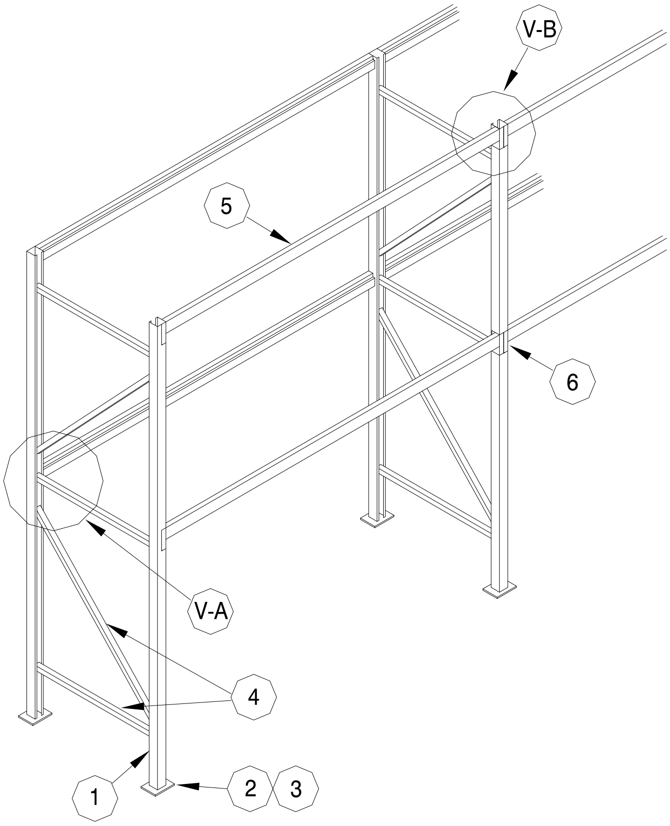
PROJECT: Life Science Logistics
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 2
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

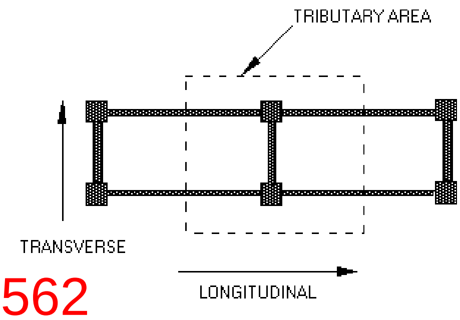
The storage racks consist of several bays, interconnected in one or both directions,with the columns of the vertical frames being comon between adjacent bays. This analysis will focus on a tributary bay to be analyzed in both the longitudinal and transverse direction. Stability in the longitudinal direction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



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NOTE: ACTUAL CONFIGURATION SHOWN ON COMPONENTS & SPECIFICATIONS SHEET

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PROJECT: Life Science Logistics
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 3
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS Configuration 1: Bay A (BTB) M 1.7

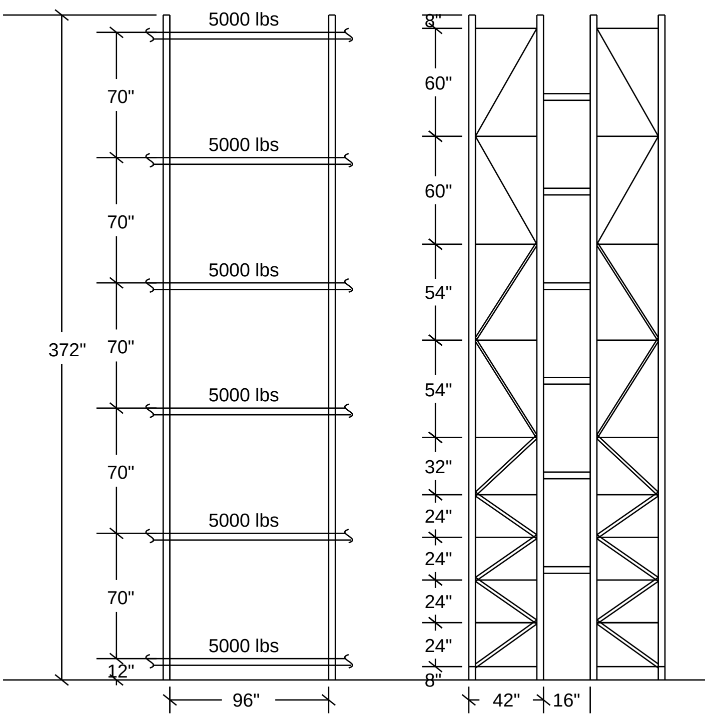
Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1242 lbs.
V_{Trans} = 4295 lbs.

P_{static} = 15300 lbs.



FRONT VIEW SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 93% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 64% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 74% (panel 8)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 93% Max stress = 93% (level 1)	4 Tab 2" cc Connector (IM) Stress = 62% Max stress = 84% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 3105 in-lb. Stress = 20%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 70% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 87%

Notes:
Special Strutting 1-7
Same as T1 Back to Back
Use 6 Face Row Spacers per frame. 4 Bolts @ 4" o.c.Conn. 1/2" dia A449 bolt. 0.375thk plate. Min row spacer Sx is 1.5cu.in.

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ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 4
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS   Configuration 2: Bay A (Single)   M 1.7

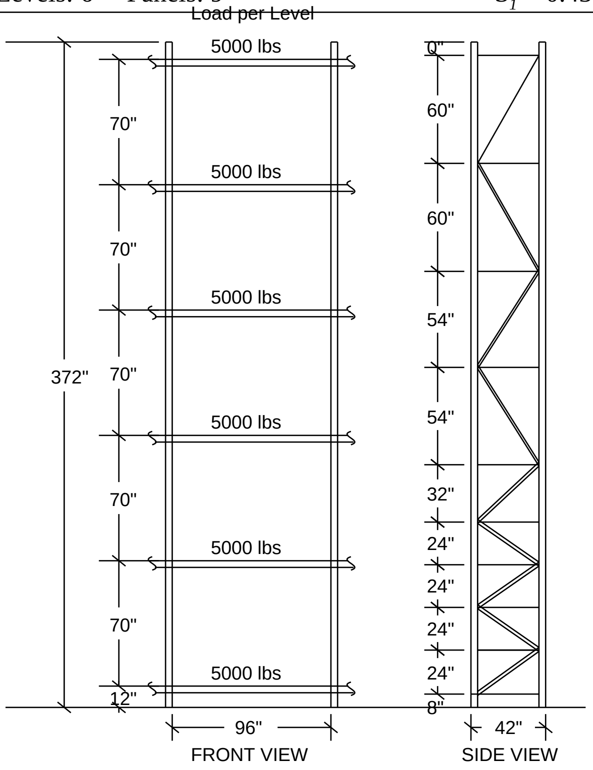
Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1242 lbs.
V_{Trans} = 4295 lbs.

P_{static} = 15300 lbs.
$P_{seismic}$ = 20009 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 93% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 64% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 74% (panel 6)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 93% Max stress = 93% (level 1)	4 Tab 2" cc Connector (IM) Stress = 62% Max stress = 84% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 3105 in-lb. Stress = 26%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 66% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 88%

Notes:
Special Strutting, Same as T1 Single Row
Double Diagonals 1-8
5/8" Anchors

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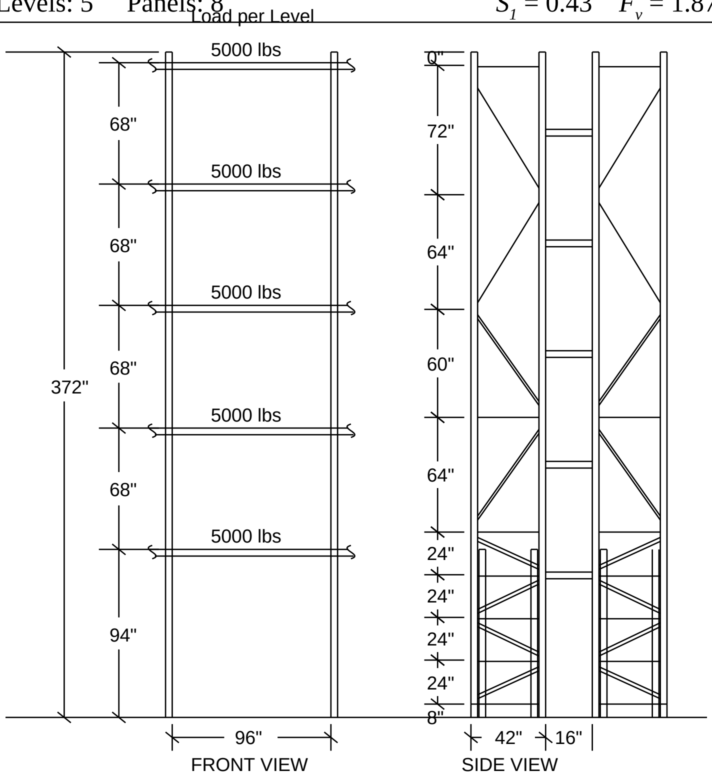
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 5
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS   Configuration 3: Bay J, G, K, E (BTB)   M   1.7

Analysis per section 2209 of the 2018 IBC

Levels: 5   Panels: 8 S_s = 1.25 F_a = 1.0 I = 1 S_1 = 0.43 F_v = 1.87 SDC = D V_{Long} = 1035 lbs. V_{Trans} = 3579 lbs. P_{static} = 12750 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 75% (level 2) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 69% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 53% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 100% (panel 7)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5906 lb. Stress = 86% Max stress = 93% (level 2)	**Level 1** 5 Tab Connector (IM) Stress = 80% **Level2** 4 Tab 2" cc Connector (IM) Stress = 69% Max stress = 80% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 12000 in-lb. Stress = 20%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 61% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 84%

Notes:
Double Diagonals 1-6
First Beam level F5M
Use 5 Face Row Spacers per frame. 4 Bolts @ 4" o.c Conn. 1/2" dia A449 bolt. 0.375thk plate. Min row spacer Sx is 1.5cu.in.

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ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 6
CALCULATED BY: tchang
DATE: 3/24/2022

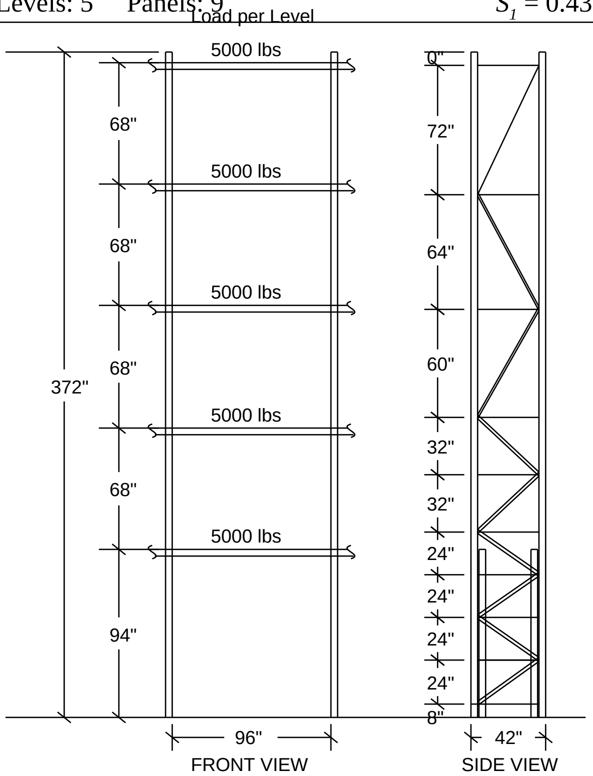
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS Configuration 4: Bay J, G, K, E (Single) M 1.7

Analysis per section 2209 of the 2018 IBC

Levels: 5 Panels: 9

S_s = 1.25 F_a = 1.0 I = 1 V_{Long} = 1035 lbs. P_{static} = 12750 lbs.
S_1 = 0.43 F_v = 1.87 SDC = D V_{Trans} = 3579 lbs. $P_{seismic}$ = 17718 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 65% (level 3) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 72% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 53% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 69% (panel 9)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5906 lb. Stress = 86% Max stress = 93% (level 2)	**Level 1** 5 Tab Connector (IM) Stress = 85% **Level2** 4 Tab 2" cc Connector (IM) Stress = 69% Max stress = 85% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 24%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 57% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 1757 lbs. Shear Capacity = 2678 lbs. Anchor stress = 97%

Notes:
1st Beam level must have F5M endplate
Special Strutting Pattern
Reinforced Both 8'
Double Diagonals 1-8

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ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 7
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Configuration 5: Bay B (Single)
Adjacent to: Bay J, G, K, E (Single)

M 1.7

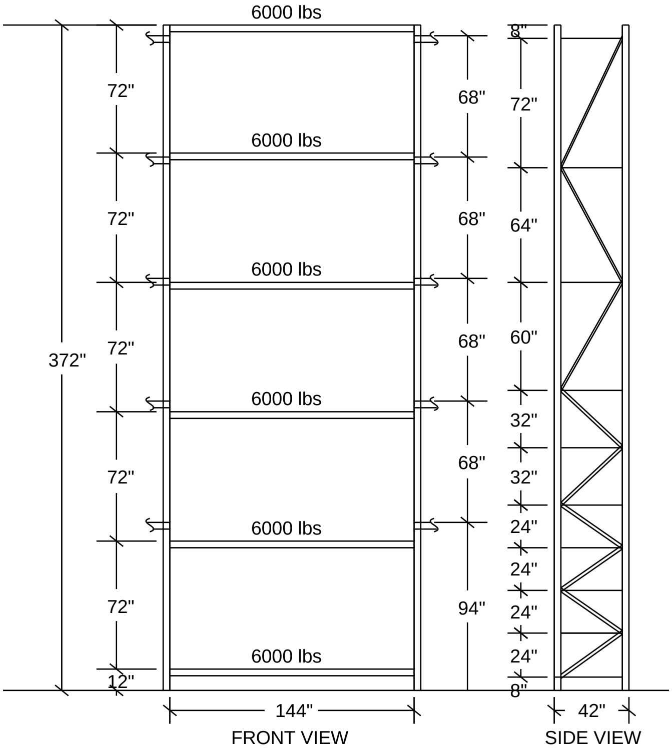
Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9 Load per Level

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1259 lbs.
V_{Trans} = 4354 lbs.

P_{static} = 15525 lbs.
$P_{seismic}$ = 21107 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 97% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 65% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 68% (panel 7)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 59% Max stress = 59% (level 1)	5 Tab Connector (IM) Stress = 42% Max stress = 42% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 3148 in-lb. Stress = 27%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 68% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 96%

Notes:
Load Reduced, Anchors Govern
Special Strutting
Double Diagonals 1-9
5/8" Anchors

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ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 8
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Configuration 6: Bay T1 (BTB)
Adjacent to: Bay A (BTB)

M 1.7

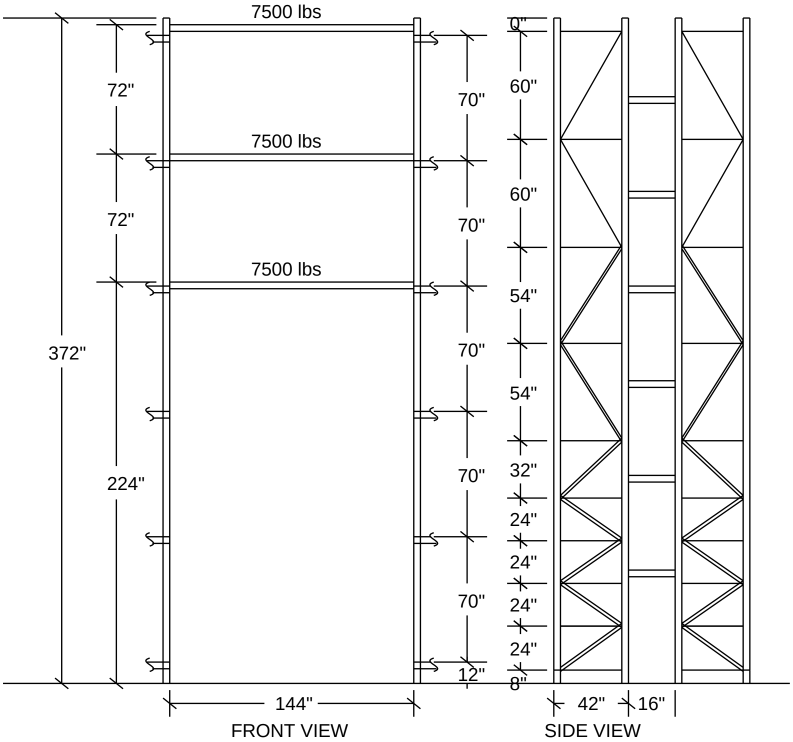
Analysis per section 2209 of the 2018 IBC

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1082 lbs.
V_{Trans} = 3742 lbs.

P_{static} = 13350 lbs.

Levels: 3 Panels: 9 Load per Level



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 86% (level 1 adj.) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 56% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 95% (panel 8)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 73% Max stress = 73% (level 1)	5 Tab Connector (IM) Stress = 39% Max stress = 39% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 8500 in-lb. Stress = 20%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 65% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 90%

Notes:
Special Strutting Pattern
Double Diagonals 1-7
Use 6 Face Row Spacers per frame. 4 Bolts @ 4" o.c.Conn. 1/2" dia A449 bolt. 0.375thk plate. Min row spacer Sx is 1.5cu.in.

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PROJECT: Life Science Logistics
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 9
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Configuration 7: Bay T1 (Single)
Adjacent to: Bay A (Single)

M 1.7

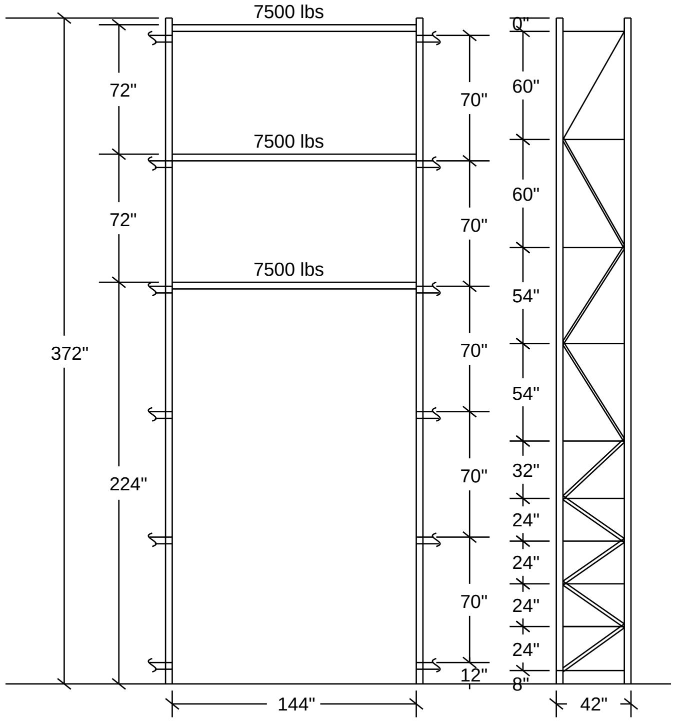
Analysis per section 2209 of the 2018 IBC

Levels: 3 Panels: 9 Load per Level

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1082 lbs.
V_{Trans} = 3742 lbs.

P_{static} = 13350 lbs.
$P_{seismic}$ = 20062 lbs.



FRONT VIEW SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 86% (level 1 adj.) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 56% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 67% (panel 6)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 73% Max stress = 73% (level 1)	5 Tab Connector (IM) Stress = 39% Max stress = 39% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 25%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 61% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 91%

Notes:
Special Strutting Pattern
Double Diagonals 1-8
5/8" Anchors

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FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 10
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 3 Panels: 9 Load per Level

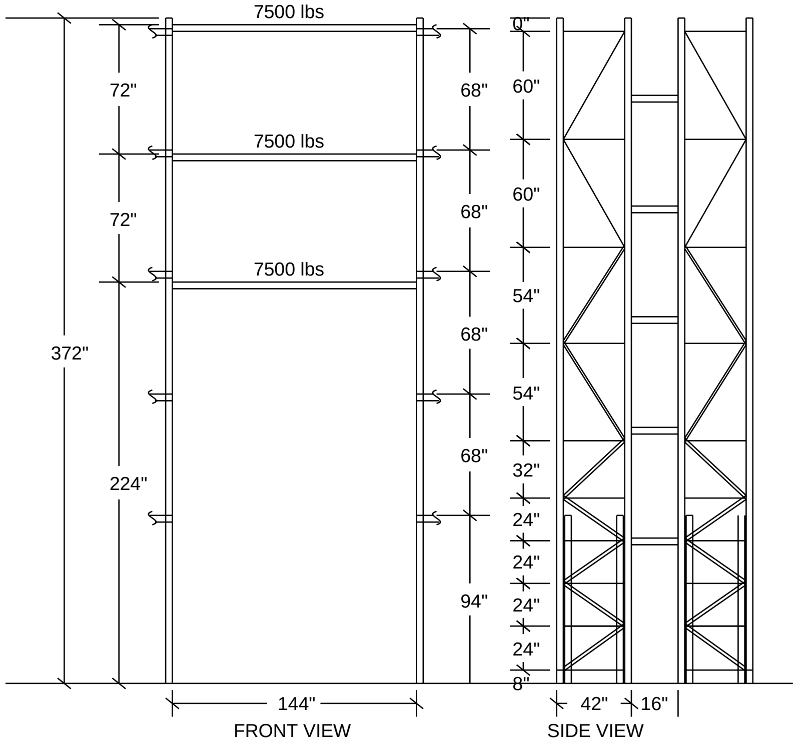
Configuration 8: Bay T1 (BTB)
Adjacent to: Bay J, G, K, E (BTB)

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 978 lbs.
V_{Trans} = 3384 lbs.

P_{static} = 12075 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 67% (level 1 adj.) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 68% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 50% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 96% (panel 8)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 73% Max stress = 73% (level 1)	5 Tab Connector (IM) Stress = 32% Max stress = 32% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 8500 in-lb. Stress = 19%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 60% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 94%

Notes:
Special Strutting Pattern
Double Diagonals 1-7
Reinforced 19'
Use 5 Face Row Spacers per frame. 4 Bolts @ 4" o.c. Conn. 1/2" dia A449 bolt. 0.375thk plate. Min row spacer Sx is 1.5cu.in.

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PROJECT: Life Science Logistics
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 11
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 3 Panels: 9 Load per Level

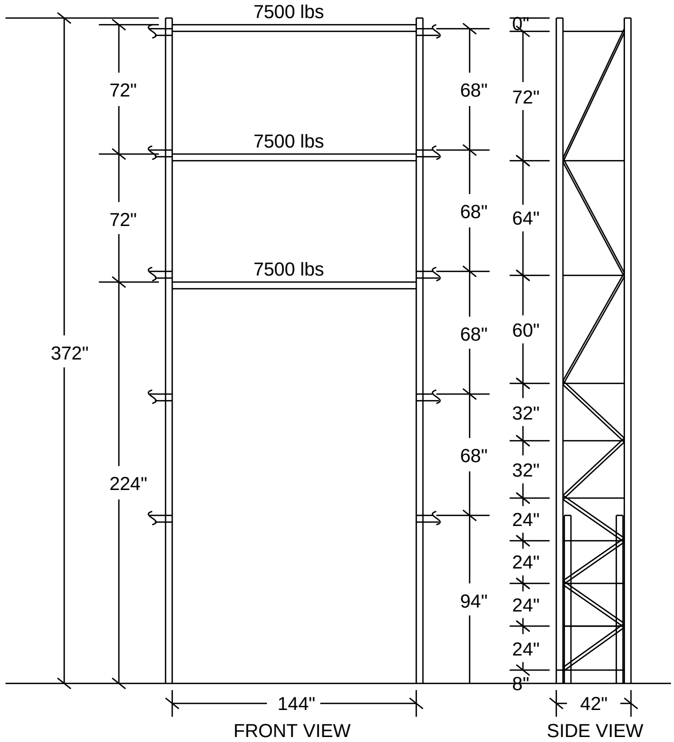
Configuration 9: Bay T1 (Single)
Adjacent to: Bay J, G, K, E (Single)

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 978 lbs.
V_{Trans} = 3384 lbs.

P_{static} = 12075 lbs.
$P_{seismic}$ = 19234 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 68% (level 1 adj.) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 68% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 50% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 76% (panel 7)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 73% Max stress = 73% (level 1)	5 Tab Connector (IM) Stress = 32% Max stress = 32% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 23%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 56% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 86%

Notes:
Special Strutting Pattern
Double Diagonals 1-9
5/8" Anchors
Reinforced 19'

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EST. 1985

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1130 E. CYPRESS ST, COVINA, CA 91724

PROJECT: Life Science Logistics
FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 12
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 9

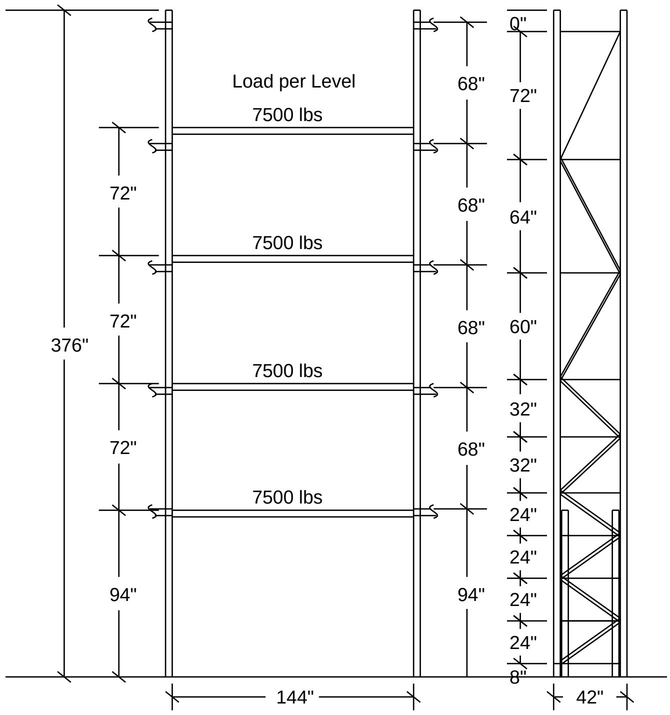
Configuration 10: Bay H and I (Single)
Adjacent to: Bay J, G, K, E (Single)

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1132 lbs.
V_{Trans} = 3916 lbs.

P_{static} = 13975 lbs.
$P_{seismic}$ = 18198 lbs.



FRONT VIEW SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 66% (level 1) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 80% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 58% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 62% (panel 7)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 10312 lb. Stress = 73% Max stress = 73% (level 1)	5 Tab Connector (IM) Stress = 90% Max stress = 90% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi * 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 25%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 59% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 1757 lbs. Shear Capacity = 2678 lbs. Anchor stress = 94%

Notes:
Special Strutting, Same as J, G, K, E Single Row
Double Diagonals 1-8

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Puyallup, WA
SHEET#: 13
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS   Configuration 11: Bay C (BTB)   M 1.7

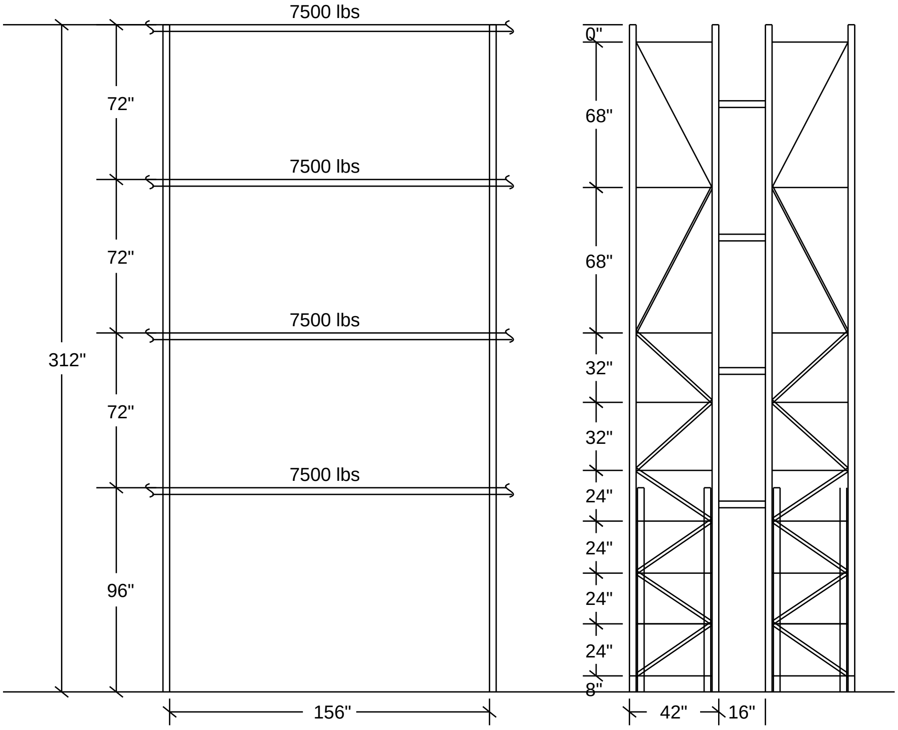
Analysis per section 2209 of the 2018 IBC

Levels: 4 Panels: 8 Load per Level

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1230 lbs.
V_{Trans} = 4253 lbs.

P_{static} = 15200 lbs.



FRONT VIEW   SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 74% (level 3) **BACKER TO LEVEL 1** 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 87% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 63% (panel 4) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 88% (panel 8)	6.56 x 2.75 - .084 (65Q) Steel = 55 ksi Max Static Cap. = 9044 lb. Stress = 84% Max stress = 84% (level 1)	5 Tab Connector (IM) Stress = 97% Max stress = 97% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 12000 in-lb. Stress = 21%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 66% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 89%

Notes:
Special Strutting
Double Diagonals 1-7
Reinforced 8'
Use 4 Face Row Spacers per frame. 4 Bolts @ 4" o.c. Conn. 1/2" dia A449 bolts, 0.375thk plate. Min row spacer Sx is 1.5cu.in.

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DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS   Configuration 12: Bay F   M   1.7

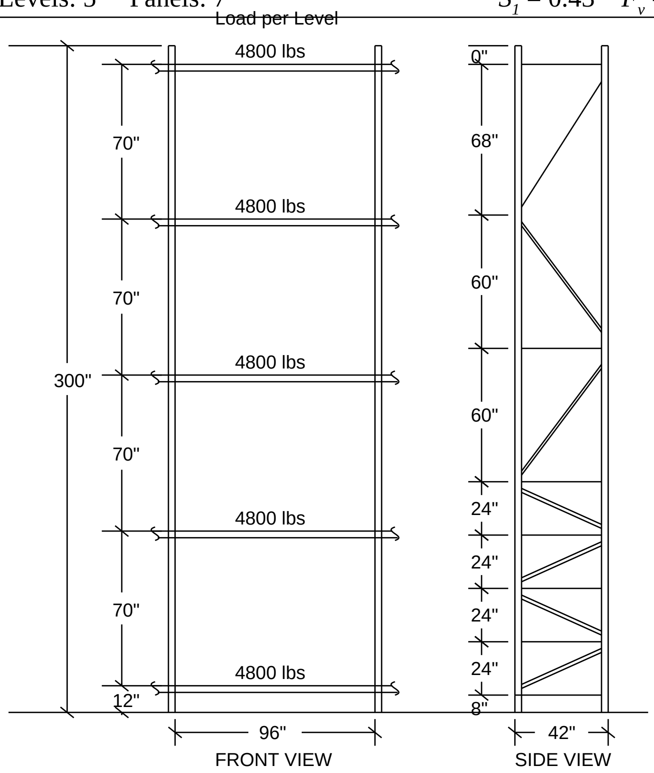
Analysis per section 2209 of the 2018 IBC

Levels: 5   Panels: 7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 994 lbs.
V_{Trans} = 3440 lbs.

P_{static} = 12250 lbs.
$P_{seismic}$ = 13477 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 94% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 51% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 56% (panel 5)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 90% Max stress = 90% (level 1)	4 Tab 2" cc Connector (IM) Stress = 51% Max stress = 66% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 2485 in-lb. Stress = 14%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 50% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 98%

Notes:
Double Diagonals 1-6
Standard Row Spacers for Back to Back

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CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS Configuration 13: Bay B (Symmetrical) M 1.7

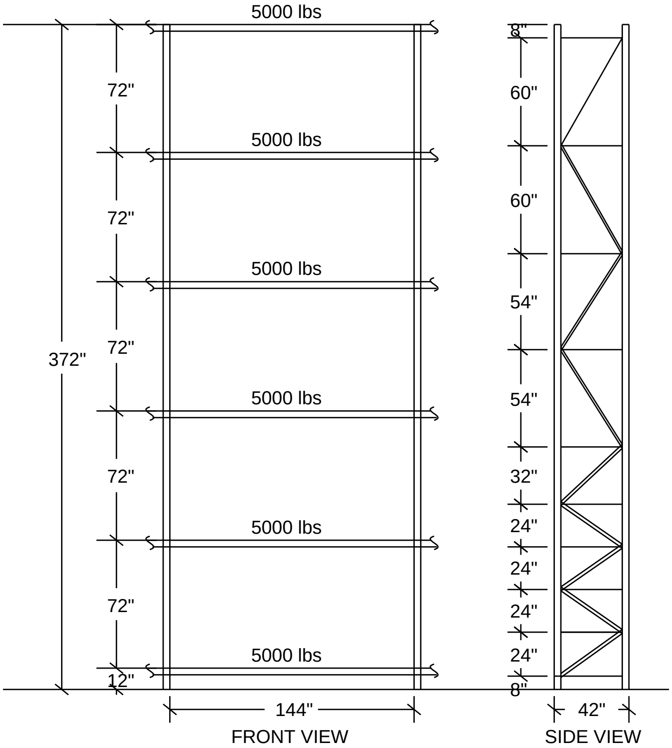
Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9 Load per Level

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1242 lbs.
V_{Trans} = 4295 lbs.

P_{static} = 15300 lbs.
$P_{seismic}$ = 20564 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 94% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 64% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 58% (panel 6)	5.94 x 2.75 -0.059 (59E) Steel = 55 ksi Max Static Cap. = 6274 lb. Stress = 81% Max stress = 81% (level 1)	5 Tab Connector (IM) Stress = 44% Max stress = 59% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 3105 in-lb. Stress = 26%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 67% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 93%

Notes:

Special Strutting
Double Diagonals 1-8
5/8" Anchors

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CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 5 Panels: 9

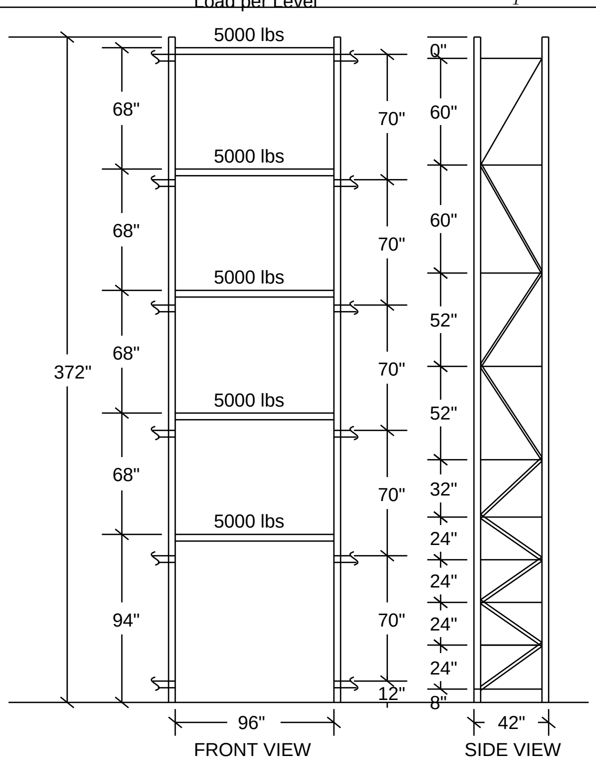
Configuration 14: Bay T2 (Adj To A) Single
Adjacent to: Bay A (Single)

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1138 lbs.
V_{Trans} = 3937 lbs.

P_{static} = 14025 lbs.
$P_{seismic}$ = 18852 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 87% (level 1 adj.) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 59% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 56% (panel 6)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 93% Max stress = 93% (level 1)	4 Tab 2" cc Connector (IM) Stress = 51% Max stress = 51% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 25%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 61% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 85%

Notes:

Special Strutting
Double Diagonals 1-8
5/8" Anchors

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SHEET#: 17
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 5 Panels: 9 Load per Level

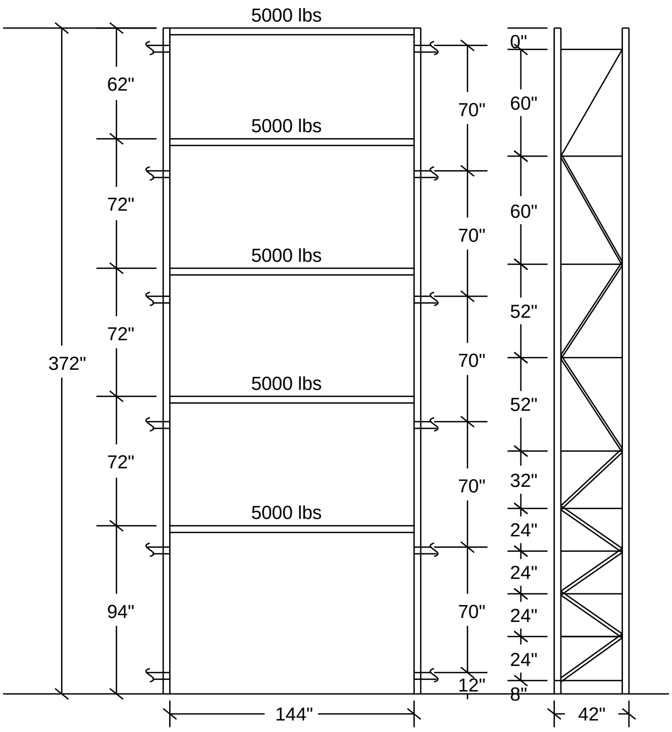
Configuration 15: Bay T3 (Adj To A) Single
Adjacent to: Bay A (Single)

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1138 lbs.
V_{Trans} = 3937 lbs.

P_{static} = 14025 lbs.
$P_{seismic}$ = 19107 lbs.



FRONT VIEW
SIDE VIEW

FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 87% (level 1 adj.) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 59% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 56% (panel 6)	5.94 x 2.75 -0.059 (59E) Steel = 55 ksi Max Static Cap. = 6274 lb. Stress = 81% Max stress = 81% (level 1)	5 Tab Connector (IM) Stress = 36% Max stress = 36% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 8500 in-lb. Stress = 25%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 62% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 87%

Notes:

Special Strutting
Double Diagonals 1-8
5/8" Anchors

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CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Configuration 16: Bay B (Sym) Single
Adjacent to: Bay A (Single)

M 1.7

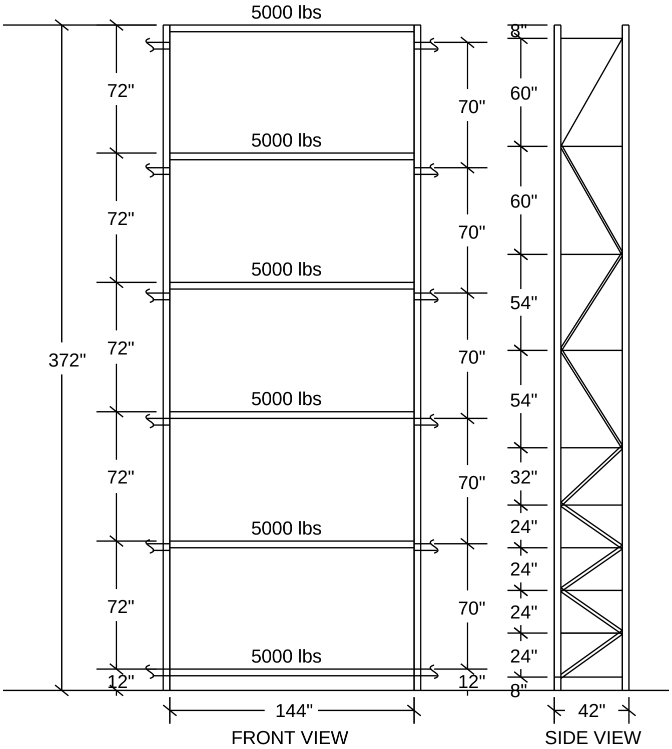
Analysis per section 2209 of the 2018 IBC

Levels: 6 Panels: 9 Load per Level

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 1242 lbs.
V_{Trans} = 4295 lbs.

P_{static} = 15300 lbs.
$P_{seismic}$ = 20287 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 4 x 2.68 - .105 (4B102) Steel = 55000 psi Stress = 94% (level 1) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 64% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 57% (panel 6)	5.94 x 2.75 -0.059 (59E) Steel = 55 ksi Max Static Cap. = 6274 lb. Stress = 81% Max stress = 81% (level 1)	5 Tab Connector (IM) Stress = 44% Max stress = 57% (level 2)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 12 x 12 x 0.375 in. 8 anchors/plate Moment = 3105 in-lb. Stress = 26%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 66% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.625 in. x 4.438 in. Embed. Pullout Capacity = 2104 lbs. Shear Capacity = 3943 lbs. Anchor stress = 91%

Notes:

Special Strutting
Double Diagonals 1-8
5/8" Anchors

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SHEET#: 19
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COMPONENTS AND SPECIFICATIONS

Analysis per section 2209 of the 2018 IBC

Levels: 2 Panels: 7

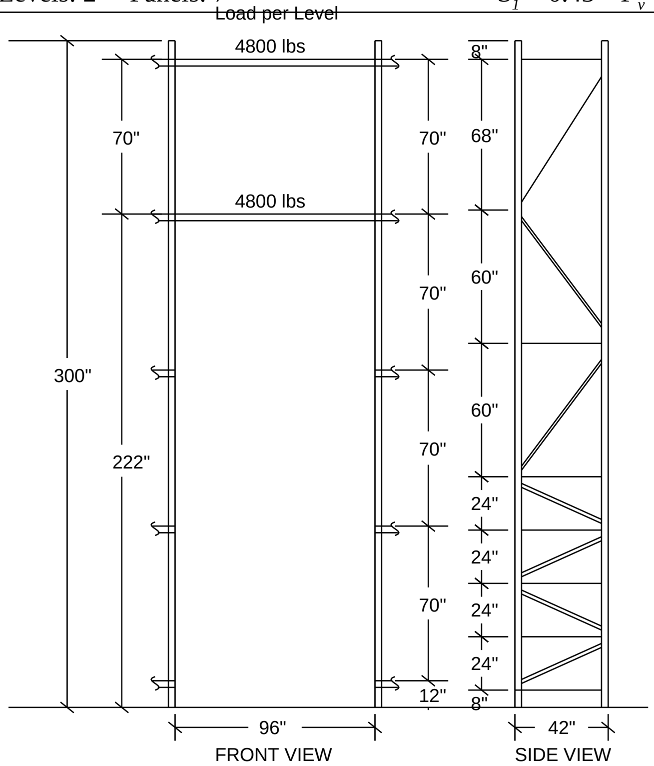
Configuration 17: BayT4 (Adj to F)
Adjacent to: Bay F

M 1.7

S_s = 1.25 F_a = 1.0 I = 1
S_1 = 0.43 F_v = 1.87 SDC = D

V_{Long} = 696 lbs.
V_{Trans} = 2408 lbs.

P_{static} = 8575 lbs.
$P_{seismic}$ = 11497 lbs.



FRAME	BEAM	CONNECTOR
COLUMN 3.98 x 2.72 - .089 (4B101) Steel = 55000 psi Stress = 70% (level 1 adj.) **HORIZONTAL BRACE** 2.756 x 1.378 - .06 Stress = 36% (panel 1) **DIAGONAL BRACE** 2.756 x 1.378 - .06 Stress = 54% (panel 7)	4.00 x 2.75 -0.059 (40E) Steel = 55 ksi Max Static Cap. = 5430 lb. Stress = 90% Max stress = 90% (level 1)	4 Tab 2" cc Connector (IM) Stress = 32% Max stress = 32% (level 1)

Base Plate	Slab & Soil	Anchors
Steel = 36000 psi 7.874 x 7.874 x 0.394 in. 4 anchors/plate Moment = 8500 in-lb. Stress = 12%	Slab = 6" x 4000 psi Sub Grade Reaction = 50 pci Slab Bending Stress = 37% (S)	Hilti Kwik Bolt TZ (KB-TZ) ESR-1917 0.5 in. x 3.625 in. Embed. Pullout Capacity = 2068 lbs. Shear Capacity = 2678 lbs. Anchor stress = 82%

Notes:
Double Diagonals 1-6
Standard Row Spacers for Back to Back

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CALCULATED BY: tchang
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PN: 20220323_29

Loads and Distributions: Bay A (BTB)

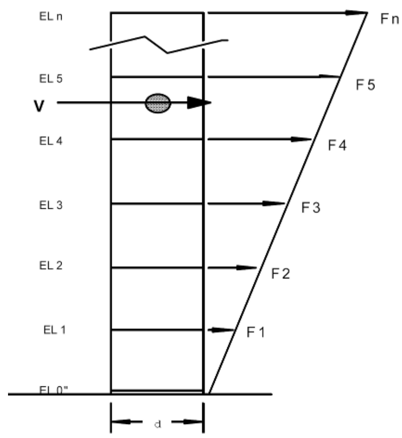
Determines seismic base shear per Section 2.6 of the RMI & Section 2209, of the 2018 IBC

# of Levels:	6	SDC:	D	R_L:	6	Ss:	1.25
Pallets Wide:	2	W_{PL}:	30000	R_T:	4	S1:	0.43
Pallets Deep:	1	W_{DL}:	600 lbs	Fa:	1	Ip:	1
Pallet Load:	2500	Fv:	1.87	Tl:	1.5		
Total Frame Load:	30600 lbs						

$S_{DS} = 2/3 \cdot S_s \cdot F_a =$ **0.83**

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

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



Seismic Shear per RMI 2012 2.6.3:

Longitudinal

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

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

$= 0.54 / (1.5 \cdot 6) \cdot 1 \cdot 20700 =$ **1242 lbs**

V_{long} need not be greater than:

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

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

$= 0.83 / 6 \cdot 1 \cdot 20700 =$ **2863.5 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 6 \cdot 1 \cdot 20700 =$ **741.75 lbs**

V_{long} shall not be less than:

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

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

$= \text{Max}[0.04, 0.04, 0.03] \cdot 1 \cdot 20700 =$ **755.96 lbs**

Since: $1242 \leq 2863.5$
$\& \; 1242 \geq 741.75$
$\& \; 1242 \geq 755.96$

V_{long} = 1242 lbs

Transverse

V_{trans} need not be greater than:

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

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

$= 0.83 / 4 \cdot 1 \cdot 20700 =$ **4295.25 lbs**

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

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

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

$= 0.5 \cdot 0.43 / 4 \cdot 1 \cdot 20700 =$ **1112.63 lbs**

V_{trans} shall not be less than:

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

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

$= \text{Max}[0.04, 0.05, 0.03] \cdot 1 \cdot 20700 =$ **1112.63 lbs**

Since: $4295.25 \geq 1112.63$
$\& \; 4295.25 \geq 1112.63$

V_{trans} = 4295 lbs

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Loads and Distributions: Bay A (BTB) (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	12	2550	30600	120.89	2550	30600	181.09
2	82	2550	209100	82.82	2550	209100	303.91
3	152	2550	387600	153.52	2550	387600	563.35
4	222	2550	566100	224.22	2550	566100	822.78
5	292	2550	744600	294.92	2550	744600	1082.22
6	362	2550	923100	365.62	2550	923100	1341.65

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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

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

# of levels	6
min. # of bays	3
N_c	72
N_b	8
k_c	360 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	192 kip-in/rad
k_{ce}	769 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	2.36 in^4
H	362 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	44 in	5 kip
2	114 in	5 kip
3	184 in	5 kip
4	254 in	5 kip
5	324 in	5 kip
6	395 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
g = the acceleration of gravity
N_L = the number of loaded levels
k_c = the rotational stiffness of the connector
k_{be} = the flexural rotational stiffness of the beam-end
k_b = the rotational stiffness of the base plate
k_{ce} = the flexural rotational stiffness of the base upright-end
N_c = the number of beam-to-upright connections
N_b = the number of base plate connections

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

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

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

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

Calculated T = 4.92

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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CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

LRFD Basic Load Combinations: Bay A (BTB)

2018 IBC& RMI / ANSI MH 16.1

V_{Trans} = **4,295 lbs**
V_{Long} = **1,242 lbs**
P = Product Load / 2 = **15,000 lbs**
D = Dead Load · 0.5 = **300 lbs**

$M_{Trans} = \Sigma(f_{Trans} \cdot h_x)$ = **1,097,067 in-lbs**
$E_{Trans} = M_{Trans}$ / frame depth = **26,120 lbs**

β = 0.7
β = 1.0 (Uplift combination only)
ρ = 1
S_{DS} = .83

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 · .83) · 300 + (1.2 + 0.2 · .83) · 0.7 · 15,000 + 0.5 · 0 + 1 · 26,120 + 0.2 · 0= **40,873 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 · .83) · 300 + (1.2 + 0.2 · .83) · 0.7 · 15,000 + 0.5 · 0 + 1 · 0 + 0.2 · 0= **14,752 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 · .83) · 300 + (0.9 - 0.2 · .83) · 1 · 15,000 - 1 · 26,120= **-14,890 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 · .83) · 300 + 0.75((1.4 + 0.14 · .83) · 0.7 · 15,000 + 0.7 · 1 · 26,120)= **25,979 lbs**
2. $(1 + 0.14S_{DS})D + (0.85 + 0.14S_{DS})\beta P + 0.7\rho E$
= (1 + 0.14 · .83) · 300 + (0.85 + 0.14 · .83) · 0.7 · 15,000 + 0.7 · 1 · 26,120= **28,764 lbs**
3. D + P
= 300 + 15,000 = **15,300 lbs**

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FOR: Malin Systems_Tim McWi
ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 24
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

Longitudinal Analysis: Bay A (BTB)

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).

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

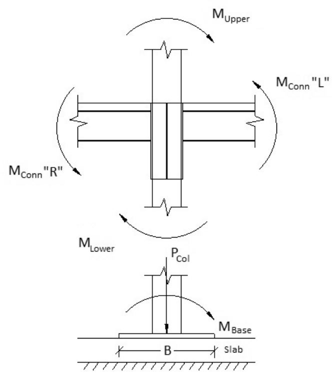
$M_{Conn} = ((M_{Upper} + M_{Lower}) / 2) + M_{Ends}$

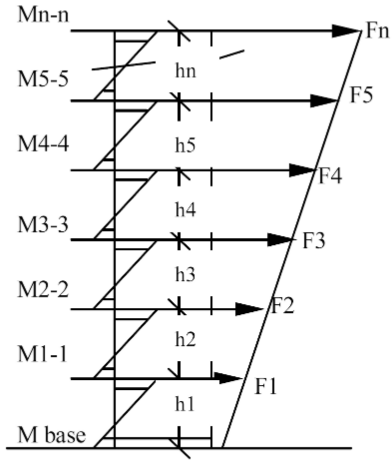
$V_{Col} = V_{Long}$ / # of columns = **621 lbs**

$M_{Base} =$ **3105 in-lbs**

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

(621 lbs · 10 in.) - 3105 in-lbs = 3105 in-lbs





FRONT ELEVATION

Levels	h_i	f_i	Axial Load	Moment	Beam End Moment	Connector Moment
1	12	60	15,300	3,105	5,551	26,665
2	70	41	12,750	39,123	5,551	44,674
3	70	77	10,200	39,123	5,551	44,674
4	70	112	7,650	39,123	5,551	44,674
5	70	147	5,100	39,123	5,551	44,674
6	70	183	2,550	39,123	5,551	25,112

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ADDRESS: 302 33Rd St SE
Puyallup, WA
SHEET#: 25
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

COLUMN ANALYSIS: Bay A (BTB) (Level 1)

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

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

$K_x \cdot L_x / R_x$ = $1.7 \cdot 10 / 1.474$ = **11.53**

$K_y \cdot L_y / R_y$ = $1 \cdot 8 / 0.908$ = **8.81**

KL/R_{max} = **11.53**

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

$= (1.474^2 + 0.908^2 + -1.983^2)^{1/2} =$ **2.632 in.**

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

$= 1 - (-1.983/2.632)^2 =$ **0.433**

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

$= 3.14^2 \cdot 29500 / 11.53^2 =$ **2188.567 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)

$= (1 / (2 \cdot 0.433)((2188.567 + 2880.291) - (2188.567 + 2880.291)^2$

$- (4 \cdot 0.433 \cdot 2188.567 \cdot 2880.291))^{1/2})=$ **1414.315 ksi**

where:

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

$= 3.14^2 \cdot 29500 / 11.53^2 =$ **2188.567 ksi**

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

$= 1 / 1.087 \cdot 2.632^2 (11300 \cdot 0.004$

$+ (3.142 \cdot 29500 \cdot 3.045) / (0.8 \cdot 8)^2) =$ **2880.291 ksi**

$F_e = \mathrm{Min}(F_{e1}, F_{e2}) =$ **1414.315 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (55 / 1414.315)^{1/2} =$ **0.197** (Eq C4.1-4)

Since $\lambda_c < 1.5$:

$F_n = (0.658^{\wedge}(\lambda_c^2)) \cdot F_y =$ **54.112** (Eq C4.1-2)

Thus:

P_n = **50675 lbs**

P_a = **43074 lbs**

4 x 2.68 - .105	
SECTION PROPERTIES	
Depth	**2.717 in.**
Width	**4 in.**
t	**0.105 in.**
Radius	**0.138 in.**
Area	**1.087 in.^{2}**
AreaNet	**0.937 in.^{2}**
I_x	**2.361 in.^{4}**
S_x	**1.176 in.^{3}**
$S_{x\ Net}$	**1.065 in.^{3}**
R_x	**1.474 in.**
I_y	**0.896 in.^{4}**
S_y	**0.506 in.^{3}**
R_y	**0.908 in.**
J	**0.004 in.^{4}**
C_w	**3.045 in.^{6}**
J_x	**2.434 in.**
X_o	**-1.983 in.**
K_x	1.7
L_x	**10 in.**
K_y	1
L_y	**8 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	1
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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Puyallup, WA
SHEET#: 26
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

COLUMN ANALYSIS: Bay A (BTB) (Level 1)

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

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

$P_{ao} = P_{no}\phi_c =$ **43781 lbs**

Where:

$P_{no} = A_e F_y = 0.937 \cdot 55 =$ **51507 lbs**

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f =$ **6107.981 ksi**

$F_e = C_s A\sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_t / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) =$ **2921.53 ksi** (Eq 3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 =$ **9421.378 ksi** (Eq 3.1.2.1-10)

$F_{e.min} =$ **2921.53 ksi**

Since: $F_e \geq 2.78 F_y$

$F_c = (S_e / S_c)$

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

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

$M_{nx} =$ **58564 in-lbs** $M_{ny} =$ **27851 in-lbs** $M_c = M_{n,min}$

$M_{nx}\phi_b =$ **52707 in-lbs** $M_{ny}\phi_b =$ **25066 in-lbs**

$P_{Ex} = \Pi^2 EI_x / (K_x L_x)^2 =$ **2378591 lbs** (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 EI_y / (K_y L_y)^2 =$ **4077511 lbs** (Eq C5.2.2-7)

$\alpha_x = (1 - (\phi_c P / P_{ex})) =$ **0.994** (Eq C5.2.2-4)

$\alpha_y = (1 - (\phi_c P / P_{ey})) =$ **0.997** (Eq C5.2.2-5)

$P_{trans} =$ **40,873 lbs** $P_{long} =$ **14,752 lbs**

$M_u = M_x =$ **3105 in-lbs** (Eq C5.2.2-2)

$P_{u\_st} = (1.2 \cdot D) + (1.4 \cdot P) =$ **21360 lbs**

$P_{u\_st} / P_a = 21360 / 43074 = 0.62$ Static Stress = **61%**

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)

= ((14,752 / 43074) + (3105 / 52707) + (1 / 25066)) = **40%**

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)

= (14,752 / 43781) + (0.85 · 3105 / 52707 · 0.994)) + (0.85 · 1 / 25066 · 0.997))) = **38%**

Stress3 P_t / P_{ao} = 40,873 / 43781 =**93%**

Column Stress = Max(Stress1, Stress2, Stress3, Static) = **93%**

4 x 2.68 - .105	
SECTION PROPERTIES	
Depth	**2.717 in.**
Width	**4 in.**
t	**0.105 in.**
Radius	**0.138 in.**
Area	**1.087 in. ^{2}**
AreaNet	**0.937 in. ^{2}**
I_x	**2.361 in. ^{4}**
S_x	**1.176 in. ^{3}**
$S_{x\ Net}$	**1.065 in. ^{3}**
R_x	**1.474 in.**
I_y	**0.896 in. ^{4}**
S_y	**0.506 in. ^{3}**
R_y	**0.908 in.**
J	**0.004 in. ^{4}**
C_w	**3.045 in. ^{6}**
J_x	**2.434 in.**
X_o	**-1.983 in.**
K_x	1.7
L_x	**10 in.**
K_y	1
L_y	**8 in.**
K_t	0.8
F_y	**55 ksi**
F_u	**65 ksi**
Q	1
G	**11300 ksi**
E	**29500 ksi**
C_{mx}	0.85
C_s	-1
C_b	1
C_{tf}	1
Phi_b	0.9
Phi_c	0.85

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DATE: 3/24/2022
PN: 20220323_29

BEAM ANALYSIS Bay A (BTB)

Determine allowable bending moment per AISI

Check compression flange for local buckling (B2.1)

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

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

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

λ <= 0.673: Flange is fully effective.

Check web for local buckling (B2.3)

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

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

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

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

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

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

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

b1 = w · (3 - Ψ) = 4 · (3 - -0.87) = **14.33**

b2 = w/2 = **1.85**

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

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

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

L_f = effective width = **1.452**

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

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

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

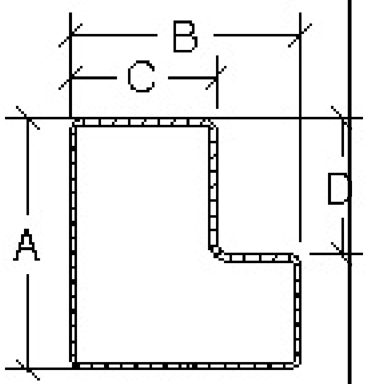
Fu/Fy = 65 / 55 = **1** **< 1.2**

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

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

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

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



4.00 x 2.75 -0.059

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

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SHEET#: 28
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

BEAM ANALYSIS Bay A (BTB)

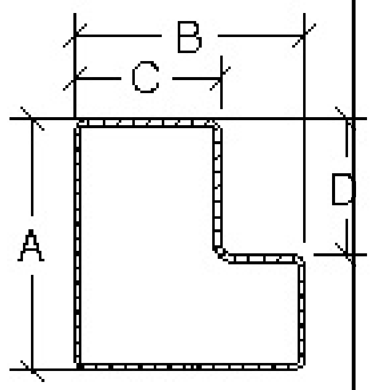
Check Allowable Tension Stress for Bottom Flange

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

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

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

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



4.00 x 2.75 -0.059

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

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

Check Bending Capacity

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

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

For DL = 2% of PL:

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

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

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

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

Check Deflection Capacity

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

$\Delta_{max} = L / 180$

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

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

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

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

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Allowable and Actual Bending Moment at Each Level

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

Level	M_{static}	$M_{allow,static}$	$M_{seismic}$	$M_{allow,seismic}$	Result
1	30,576	32,580	7,928	32,580	Pass
2	30,576	32,580	10,803	32,580	Pass
3	30,576	32,580	8,735	32,580	Pass
4	30,576	32,580	5,430	32,580	Pass
5	30,576	32,580	3,122	32,580	Pass
6	30,576	32,580	2,776	32,580	Pass

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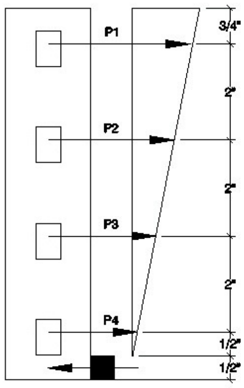
SHEET#: 30
CALCULATED BY: tchang
DATE: 3/24/2022
PN: 20220323_29

Beam to Column Analysis: Bay A (BTB)

1. Shear Strength of Tab



Height of the Tab h = **0.6 in.**

Thickness of the Tab t_t = **0.135 in.**

$F_y =$ **55000 psi**

$C_v =$ **1.0**

$V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = \mathbf{2673\ lbs}$ AISC G2-1

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

2. Bearing Strength of Tab

Thickness of the column t_c = **0.11 in.**

$A_{pb} = h \cdot t_c = \mathbf{0.06\ in.}$

$R_n = 1.8 \cdot F_y \cdot A_{pb} = \mathbf{6237\ lbs}$ AISC J7 -1

$P_{Bearing} = \phi \cdot R_n = 0.75 \cdot 6237 = \mathbf{4677\ lbs}$

3. Moment Strength of Bracket

Edge Dist. = **1 in.**

$T_{Clip} =$ **0.179 in.**

$S_{Clip} =$ **0.127 in.^{3}**

$M_n = S_c \cdot F_y = \mathbf{6985\ in\text{-}lbs}$ AISI C3.1.1 -1

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

$C =$ **2.15**

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

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

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

Minimum Value of P1 Governs

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

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

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BRACE ANALYSIS Bay A (BTB) (Panel 8)

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 · 65 / 1.1 = **59.36**
$K_y \cdot L_y / R_y$ = 1 · 65 / 0.44 = **148.84**
KL / R_{max} = **148.84**

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

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

$F_{e1} = \Pi^2 E / (KL / r)_{max}^2 = 3.14^2 \cdot 29500 / 148.84^2$ = **13.142 ksi** (Eq C4.1.1-1)

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

= $(1 / (2 \cdot 0.65)((82.63 + 14.24) - ((82.63 + 14.24)^2$
$- (4 \cdot 0.65 \cdot 82.63 \cdot 14.24))^{1/2})$ = **13.342 ksi** (Eq C4.1.2-1)

where:
$\sigma_{ex} = \Pi^2 E / (K_x L_x / R_x)^2$
= $3.14^2 \cdot 29500 / 148.84^2$ = **82.627 ksi** (Eq C3.1.2-11)

$\sigma_t = 1 / A r_o^2 (GJ + (\Pi^2 E C_w) / (K_t L_t)^2)$
= $1 / 0.32 \cdot 1.46^2 (11300 \cdot 0.0004$
$+ (3.14^2 \cdot 29500 \cdot 0.07) / (0.8 \cdot 65)^2)$ = **14.237 ksi** (Eq C3.1.2-9)

F_e = Min(Fe1, Fe2) = **13.142 ksi**

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

$\lambda_c = (F_y / F_e)^{1/2} = (50 / 13.142)^{1/2}$ = **1.951** (Eq C4.1-4)
Since λ_c >= 1.5, $F_n = (0.877 / (\lambda_c^2)) \cdot F_y$ = **11.526** (Eq C4.1-3)

Thus
P_n = **3,657 lbs**
$P_a = P_n \cdot \phi_c$ = **3,108 lbs**

2.756 x 1.378 - .06	
SECTION PROPERTIES	
Depth	2.756 in.
Width	1.378 in.
t	0.06 in.
Radius	0.09 in.
Area	0.317 in^2
AreaNet	0.317 in^2
Ix	0.381 in^4
Sx	0.276 in^3
Sx net	0.276 in^3
Rx	1.095 in.
Iy	0.061 in^4
Sy	0.06 in^3
Ry	0.437 in.
J	0 in^4
Cw	0.074 in^6
Jx	1.615 in.
Xo	-0.862 in.
Kx	0
Lx	65 in.
Ky	1
Ly	65 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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SHEET#: 32
CALCULATED BY: tchang
DATE: 3/24/2022

PN: 20220323_29

BRACE ANALYSIS Bay A (BTB) (Panel 8)

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

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

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

Where $P_{no} = A_e F_y = 0.32 \cdot 50 = \mathbf{15{,}865\ lbs.}$

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

$F_e = C_b r_o A(\sigma_{ey}\sigma_t)^{1/2} / S_f = 57.56$ ksi

$F_e = C_s A \sigma_{ex}(j + C_s(j^2 + r_o^2(\sigma_e / \sigma_{ex}))^{1/2}) / (C_{TF} S_f) = 10.45$ ksi (Eq C3.1.2.1-4)

$F_e = (C_b \Pi^2 E d I_{yc}) / (S_f (K_y L_y)^2 = 41.62$ ksi (Eq C3.1.2.1-10)

$F_{e,min} = \mathbf{10.45\ ksi}$ (Eq C3.1.2.1-14)

Since, $F_e <= 0.56 F_y$

$F_c = F_e = \mathbf{10.45\ ksi}$ (Eq C3.1.2.1-3)

reduced $F_{c,eff} = 1 - ((1 - Q) / 2) \cdot (F_c / F_y)^Q \cdot F_c = \mathbf{10.4\ ksi}$

$M_{nx} = \mathbf{2{,}871\ in\text{-}lbs}$ $M_{ny} = \mathbf{627\ in\text{-}lbs}$ $M_c = M_{n,min}$

$M_{nx}\phi_b = \mathbf{2{,}584\ in\text{-}lbs}$ $M_{ny}\phi_b = \mathbf{564\ in\text{-}lbs}$

$P_{Ex} = \Pi^2 E I_x / (K_x L_x)^2 = \mathbf{26{,}221\ lbs}$ (Eq C5.2.2-6)

$P_{Ey} = \Pi^2 E I_y / (K_y L_y)^2 = \mathbf{4{,}169\ lbs}$ (Eq C5.2.2-7)

Max $P_a = \mathbf{3{,}657\ lbs}$

$V_{Trans} = \mathbf{1{,}289\ lbs}$

$L_{Diag} = ((L - 6)^2 + (D - 2B)^2)^{1/2} = \mathbf{65.69\ in.}$

$V_{Diag} = (V_{Trans} \cdot L_{Diag}) / D = \mathbf{2314.72\ lbs.}$

Brace Stress = $V_{Diag} / P_a = \mathbf{74\%}$

2.756 x 1.378 - .06

SECTION PROPERTIES	
Depth	2.756 in.
Width	1.378 in.
t	0.06 in.
Radius	0.09 in.
Area	0.317 in^2
AreaNet	0.317 in^2
Ix	0.381 in^4
Sx	0.276 in^3
Sx net	0.276 in^3
Rx	1.095 in.
Iy	0.061 in^4
Sy	0.06 in^3
Ry	0.437 in.
J	0 in^4
Cw	0.074 in^6
Jx	1.615 in.
Xo	-0.862 in.
Kx	0
Lx	65 in.
Ky	1
Ly	65 in.
Kt	0.8
Fyv	50 ksi
Fuv	60 ksi
Q	1
G	11300 ksi
E	29500 ksi
Cmx	0.85
Cs	-1
Cb	1
Ctf	1
Phib	0.9
Phic	0.85

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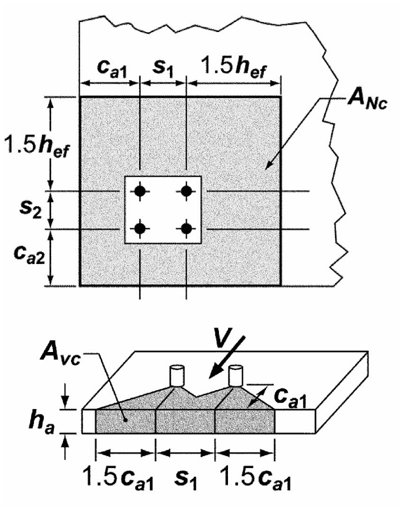
POST-INSTALLED ANCHOR ANALYSIS PER ACI 318-14, CHAPTER 17 Configuration 1 Bay A (BTB)

Assumed cracked concrete application

Anchor Type	**1/2" dia, 3.25" hef, 6" min. slab**				
ICC Report Number	**ESR-1917**			$1.5 \cdot h_{ef}$	= **4.875 in.**
Slab Thickness (h)	= **6 in.**	C_{a1}	= 12	$useC_{a1,adj}$	= **4.875 in.**
Min. Slab Thickness (h)	= **6 in.**	C_{a2}	= 12	$useC_{a2,adj}$	= **4.875 in.**
Concrete Strength (f_c)	= **4000 psi**				
Diameter (d_a)	= **0.5 in.**			$3 \cdot h_{ef}$	= **9.75 in.**
Nominal Embedment (h_{nom})	= **3.625 in.**				
Effective Embedment (h_{ef})	= Hef	S_1	= **6.25 in.**	Use $S_{1,adj}$	= **6.25 in.**
Number of Anchors (n)	= **4**	S_2	= **6.25 in.**	Use $S_{2,adj}$	= **6.25 in.**
e\`N	= **0**				
e\`V	= **0**				

From ICC ESR Report

A_{se}	= **0.101 sq.in.**
f_{uta}	= **106000 psi**
S_{min}	= **2.375 in.**
C_{min}	= **2.375 in.**
C_{ae}	= **7.5 in.**
$N_{p,cr}$	= **4915 lbs**



	$\phi_{Seismic}$	Adj. Strength
Tension Capacity = **2757 lbs**	0.75	**2068 lbs**
Shear Capacity = **3571 lbs**	0.75	**2678 lbs**

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ANCHOR ANALYSIS - TENSION STRENGTH Configuration 1 Bay A (BTB)

Steel Strength	17.4.1
$\phi = \mathbf{0.75}$	17.3.3.a i
$\phi N_{sa} = \phi n A_{se} f_{uta} = 0.75 \cdot 4 \cdot 0.101 \cdot 106000 = \mathbf{32{,}118\ lbs}$	17.4.1.2
Concrete Breakout Strength ϕN_{cbg}	17.4.2
$\phi = \mathbf{0.65}$	17.3.3 c ii Category 1-B
$A_{Nc} = (C_{a1.adj} + S_{1.adj} + 1.5h_{ef}) \cdot (C_{a2.adj} + S_{2.adj} + 1.5h_{ef}) = \mathbf{256\ sq.in.}$	
$A_{Nco} = 9h_{ef}^{2} = \mathbf{95.063\ sq.in.}$	
Check if $A_{Nco} \geq A_{Nc}$ $A_{Nc}/A_{Nco} = \mathbf{2.693}$	
$\Psi_{ec,N} = \mathbf{1}$	17.4.2.4
$\Psi_{ed,N} = \mathbf{1}$	17.4.2.5
$\Psi_{C,N} = \mathbf{1}$	17.4.2.6
$K_c = \mathbf{17}$	
$\lambda_a = \mathbf{1}$	
$N_b = K_c \lambda_a (f_c)^{0.5} (h_{ef})^{1.5} = \mathbf{6299\ lbs}$	17.4.2.2 d
$\Psi_{cp,N} = \mathbf{1}$	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
$0.65 \cdot (256/95.063) \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 6299 = \mathbf{11{,}026\ lbs}$	
Pullout Strength ϕN_{pn}	17.4.3
$\phi = \mathbf{0.65}$	17.3.3 c ii Category 1-B
$\Psi_{cp} = \mathbf{1}$	17.4.3.6
$\phi N_{pn} = \phi \Psi_{cp} N_{p,cr} (f_c/2500)^{0.5} = \mathbf{16{,}164\ lbs}$	17.4.3.1

Steel Strength (ϕN_{sa}) = **32,118 lbs**

Embedment Strength - Concrete Breakout Strength (ϕN_{cbg}) = **11,026 lbs**

Embedment Strength - Pullout Strength (ϕN_{pn}) = **16,164 lbs**

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ANCHOR ANALYSIS - SHEAR STRENGTH Configuration 1 Bay A (BTB)

Steel Strength ϕV_{sa} V_{sa}=5,495 / Anchor -- per report — 17.5.1

ϕ = **0.65** — 17.3.3. Condition a ii

$\phi V_{sa} = \phi n \cdot V_{sa} = 0.65 \cdot 4 \cdot 5{,}495 =$ **14,287 lbs** — 17.5.1.2a

Concrete Breakout Strength ϕV_{cbg} — 17.5.2

ϕ = **0.7** — 17.3.3 ci-B

$A_{Vc} = (1.5C_{a1} + S_{1.adj} + 1.5C_{a1})h_a =$ **253.5 sq.in.**

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

Check if $A_{Vco} \geq A_{Vc}$ $A_{Vc}/A_{Vco} =$ **1.174**

$\Psi_{ec,V}$ = **1** — 17.5.2.5

$\Psi_{ed,V}$ = **0.9** — 17.5.2.6

$\Psi_{C,V}$ = **1** — 17.5.2.7

$\Psi_{h,V}$ = **1.732** — 17.5.2.8

d_a = **0.5 in.** — 17.5.2.2

L_e = **1 in.** — 17.2.6 d

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

$0.7 \cdot (253.5/216) \cdot 1 \cdot 0.9 \cdot 1 \cdot 1.732 \cdot 14{,}948 =$ **38,287 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} = **16,963 lbs**

$\phi V_{cpg} = \phi K_{cp} N_{cbg} = 0.7 \cdot 2 \cdot 16{,}963 =$ **23,748 lbs**

Steel Strength (ϕV_{sa}) = **14,287 lbs**

Embedment Strength - Concrete Breakout Strength (ϕV_{cbg}) = **38,287 lbs**

Embedment Strength - Pryout Strength (ϕV_{cpg}) = **23,748 lbs**

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OVERTURNING ANALYSIS Configuration1 Bay A (BTB)

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**

$V_{Trans} = (1 \cdot 0.2075 \cdot 1 \cdot ((0.67 \cdot 20,100) + 600)) =$ **2,918 lbs**

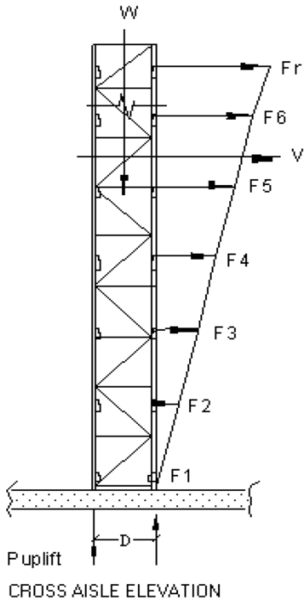
$M_{ovt} = V_{Trans} \cdot Ht = 2,918 \cdot 288 =$ **840,384 in-lbs**

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

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

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (840,384 - 434,700) / 42 =$ **9,659 lbs**

$P_{MaxDown} = 1 \cdot (M_{ovt} + M_{st}) / d = (840,384 + 434,700) / 42 =$ **30,359 lbs**



TOP SHELF LOADED

Shear = **1,162 lbs**

$M_{ovt} = V_{Top} \cdot Ht = 1,162 \cdot (362 + ((70 - 10) / 2)) =$ **455,504 in-lbs**

$M_{st} = (1 + W_{dl}) \cdot d = (5,000 + 600) \cdot (42 \cdot 0.5) =$ **117,600 in-lbs**

$P_{uplift} = 1 \cdot (M_{ovt} - M_{st})/d = (455,504 - 117,600) / 42 =$ **8,045 lbs**

SIDE LOADS ON TOP SHELF

Top loaded shelf level (H) = **362 in.**

$M_{ovt} = 1.6 \cdot 350 \cdot H = 1.6 \cdot 350 \cdot 362 =$ **202kip**

$P_{uplift} = M_{ovt}/d = 202 / 42 =$ **4,826 lbs**

ANCHORS

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

Pull Out Capacity per Anchor (T_{Anchor}): **2,068 lbs**

Shear Capacity per Anchor: **2,678 lbs**

$P_{Resist} = 0.95 \cdot T_{Anchor} \cdot \#Anchors \cdot Row\ Spacer\ Stress = 0.95 \cdot 2,068 \cdot 4 \cdot 0.772 =$ **6,064 lbs**

COMBINED STRESS

Fully Loaded	= ((6,064 / 4) / 2,068) + ((2,918 / 8) / 2,678)	= **0.869**
Top Shelf Loaded	= ((6,064 / 4) / 2,068) + ((1,162 / 8) / 2,678)	= **0.787**
Seismic UpLift Critical (LC#7B)	= (6,064 / 4) / 2,068	= **0.733**
Side Load Top Shelf	= ((4,826 / 2) / 4) / 2,068	= **0.292**

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FACE ROW SPACER ANALYSIS

Spacer S_x	1.5 cubic in
F_{nt}	81 kip
$P_{additional}$	7.032 kip
Spacer length L	16 in
# of spacers n	6
Bolt diameter d	0.5 in
Column thickness tc	0.105 in
Plate thickness tb	0.375 in
Plate width b	4 in
F_y (column)	55 ksi
F_u (column)	65 ksi
Connection pattern	4 bolts @ 4" o.c.
Bolt type	A449
Bolt spacing s	4 in
F_y (plate)	36 ksi
F_u (plate)	65 ksi
Spacer depth	3 in
F_y (Row Spacer)	36 Ksi

ROW SPACER CONNECTION CHECK

Per AISI E3 Bolted Connection

E3.4 Shear and Tension in Bolts

A_b $= \pi \cdot d^2 / 4 = \pi \cdot 0.5^2 / 4 = \mathbf{0.196\ sq\ in}$

P_n $= A_b \cdot F_{nt} = 0.196 \cdot 81 = \mathbf{15.9\ kip}$

P_{al} $= 0.75 \cdot P_n = 0.75 \cdot 15.9 = \mathbf{11.93\ kip}$

ROW SPACER PLATE CHECK

S $= 1/6 \cdot b \cdot tp^2 = 1/6 \cdot 4 \cdot 0.375^2 = \mathbf{0.094\ cu\ in}$

M $= 1.5 \cdot S \cdot F_y = 1.5 \cdot 0.094 \cdot 36 = \mathbf{5.06\ kip\text{-}in}$

a $= (s-d)/2 = (4 - 3) / 2 = \mathbf{0.5\ in.}$

ϕM $= 0.9 \cdot M = 0.9 \cdot 5.06 = \mathbf{4.56\ kip\text{-}in}$

R $= \phi M / a = 4.56 / 0.5 = \mathbf{9.11\ kip}$

P_a $= MIN(P_{al}, R) = MIN(11.93, 9.11) = \mathbf{9.11\ kip}$

CONNECTION MOMENT CAPACITY

Moment Arm

Lm $= s - a/2 = 4 - 0.5/2 = \mathbf{3.75\ in}$

M_{al} $= L_m \cdot P_a \cdot 2 = 3.75 \cdot 9.11 \cdot 2 = \mathbf{68.34\ kip\text{-}in}$

ROW SPACER MEMBER CHECK

M_n $= F_y \cdot S_x = 36 \cdot 1.5 = \mathbf{54\ kip\text{-}in}$

M_{a2} $= 0.9 \cdot M_n = \mathbf{48.6\ kip\text{-}in}$

Moment Per Row Spacer

M_r $= 2 \cdot P_{additional} \cdot L/n = 2 \cdot 7.032 \cdot 16 / 6 = \mathbf{37.5\ kip\text{-}in}$

Row Spacer Capacity

M_a $= Min(M_{a1}, M_{a2}) = Min(68.34, 48.6) = \mathbf{48.6\ kip\text{-}in}$

Stress $= M_r / M_a = 37.5 / 48.6 = \mathbf{77\%}$

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DATE: 3/24/2022

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Base Plate Analysis: Bay A (BTB)

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} = **40873 lbs**

M_{Base} = **3105 in-lbs**

$P/A = P_{col}/(D \cdot B) = 40873 / (7.87 \cdot 7.87) =$ **659 psi**

$f_b = M_{Base} / (D \cdot B^2 / 6) = 3105 / (7.87 \cdot 7.87^2 / 6) =$ **38.16 psi**

$f_{b2} = f_b \cdot (2 \cdot b_1 / B) = 38.16 \cdot (2 \cdot 1.94/7.87) =$ **18.78 psi**

$f_{b1} = f_b - f_{b2} = 38.16 - 18.78 =$ **19.39 psi**

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

$= (1.94^2 / 2) \cdot (659 + 19.39 + 0.67 \cdot 18.78) =$ **1296.58 in-lbs**

$S_{Base} = (B \cdot t^2) / 6 =$ **0.2sq.in.**

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

$f_b / F_b = M_b / (S_{Base} \cdot F_{Base}) = 1296.58 / (0.2 \cdot 32,400) =$ **0.2**

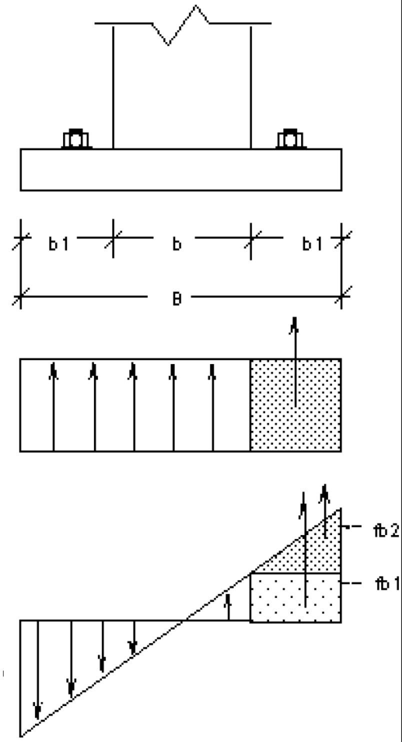

Plate width B =	7.87 in.
Plate depth D =	7.87 in.
Plate thickness t =	0.39 in.
Column width b =	4 in.
Column depth d =	2.72 in.
b1 =	1.94 in.
S_x =	6.25 in.
S_y =	6.25 in.
T_{Total} =	8,272 lbs.

Base Plate Tension analysis

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

$L_w = (S_x - b) / 2 =$ **1.13 in.**

$L_d = (S_y - b) / 2 =$ **1.77 in.**

Moment Arm (L) = $Max(L_w, L_d) =$ **1.77 in.**

$M_{anchor} = T_{Total} / 2 \cdot L =$ **7306.24 in-lbs**

$S = B \cdot t^2/6 =$ **0.2in³**

$M_{baseplate} = S \cdot F_y =$ **7,334 in-lbs**

$\phi M_{baseplate} = 0.9 \cdot M_n =$ **6,601 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

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

# of levels	6
min. # of bays	3
N_c	72
N_b	8
k_c	360 kip-in/rad
k_{be}	2861 kip-in/rad
k_b	192 kip-in/rad
k_{ce}	769 kip-in/rad
I_b	1.55 in^4
L	96 in
I_c	2.36 in^4
H	362 in
E	29500 ksi

Level	h_{pi}	W_{pi}
1	44 in	5 kip
2	114 in	5 kip
3	184 in	5 kip
4	254 in	5 kip
5	324 in	5 kip
6	395 in	5 kip

α_s = 1.59

Per RMI 2012 7.1.3

$$\theta_b = \frac{C_d(1+\alpha_S)M_b}{k_b}$$

C_d= the deflection amplification factor per section 2.6.6
M_b= the base moment from analysis
Θ_b= 0.49

Per RMI 2012 2.6.6,

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

$\tan\Theta_{max}$=0.5 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}=25,850 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) = **40,873 lbs**

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

$d,req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **4.488 in.**

$b = (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **27.301 in.**

$b,req'd = 1.5 \cdot b =$ **41 in.**

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **121,775 lbs**

$P_a = \phi \cdot P_a =$ **73,065 lbs**

$P_{max} / P_a =$ **0.56**

SLAB AND SOIL ANALYSIS (ASD)

P_{max} = MAX(ASD Load Combo 1, ASD Load Combo 2, ASD Load Combo 3)
= **28,764 lbs**

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

$P_n = 1.72[(k_s \cdot r_1 / E_c) \cdot 10^4 + 3.6] \cdot f'_t \cdot t^2 =$ **121,775 lbs**

$d,req'd = (P_{max}/(\phi \cdot 1.72 \cdot ((K_s \cdot r_1 / E_c) \cdot 10^4 + 3.6) \cdot f'_t))^{1/2} =$ **4.488 in.**

$b = (E_c \cdot d,req'd^3 / (12 \cdot (1 - \mu^2) \cdot k_s))^{1/4} =$ **27.301 in.**

$b,req'd = 1.5 \cdot b =$ **41 in.**

$P_a = P_n / \Omega =$ **40,592 lbs**

$P_{max} / P_a =$ **0.71**

Base Plate

Width B	7.874 in.
Depth W	7.874 in.

Frame

Frame depth d	42 in.

Concrete

Thickness t	6 in.
f'c	4,000 psi
ϕ	0.6
Ω	3
λ	1
k_s	50 pci
r_1	3.94 in
E_c	3,604,997 psi