JOHNSTON BURKHOLDER ASSOCIATES
consulting structural engineers

STRUCTURAL FIXTURE ANCHORAGE CALCULATIONS

FOR

Puyallup, WA

310 31st Ave SE
Store #2403

PREPARED FOR

City of Puyallup, WA

9/7/21

JBA Project #2135202403

930 CENTRAL • KANSAS CITY, MO 64105
(816) 421-4200 • WWW.JBAENGR.COM

PRCA20231436

Calculation Index

PRCA20231436

JOHNSTON BURKHOLDER ASSOCIATES
consulting structural engineers

PROJECT NO:	Sheet No:	Of:
2135202403	1	11
PROJECT NAME: #2403 - Puyallup, WA		
MADE BY: GMB	DATE: 09/07/21	
CHECKED BY:	DATE:	

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

Lateral Seismic Analysis

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Store Latitude/Longitude Coordinates (per Google):
N 47° 09' 40" 47.1611
W 122° 17' 20" 122.2889

	Braced	Down Aisle	Reference
Response Modification Factor, R =	4.0	6.0	ASCE-7, Table 15.4-1
Overstrength Factor, Omega, Ω_o =	2.0		ASCE-7, Table 15.4-1
Deflection Amplification Factor, C_d =	3.5		ASCE-7, Table 15.4-1
Detail Reference Section =	15.5.3		ASCE-7, Table 15.4-1
Occupancy Category =	II		IBC, Table 1604.5
Importance Factor, I_p =	1.0		ASCE-7 Sect. 15.5.3
0.2 Second Period Accel., S_s =	1.261 g		IBC Figs. 1613.2.1(1-8), ASCE-7 Figs. 22-1 thru 22-8
1.0 Second Period Accel., S_1 =	0.435 g		IBC Figs. 1613.2.1(1-8), ASCE-7 Figs. 22-1 thru 22-8
(Soil) Site Class =	D (Default)		IBC 1613.2.2 -> ASCE-7, Table 20.3-1
F_a =	1.200		IBC Table 1613.2.3(1), ASCE-7 Table 11.4-1
F_v =	1.865		IBC Table 1613.2.3(2), ASCE-7 Table 11.4-2 + Sect. 11.4.8
S_{MS} =	1.513 g		IBC eq. 16-36, ASCE-7 eq. 11.4-1
S_{M1} =	0.811 g		IBC eq. 16-37, ASCE-7 eq. 11.4-2
S_{DS} =	1.009 g		IBC eq. 16-38, ASCE-7 eq. 11.4-3
S_{D1} =	0.541 g		IBC eq. 16-39, ASCE-7 eq. 11.4-4
Seismic Design Category			
--based on S_{DS} =	D		IBC Table 1613.2.5(1), ASCE-7 Table 11.6-1
-- based on S_{D1} =	D		IBC Table 1613.2.5(2), ASCE-7 Table 11.6-2

Shelving Fixture

C_s =	0.252	RMI sect. 2.6.3 w/ASCE-7, Sect. 11.4.8
C_s, min =	0.044	RMI sect. 2.6.3 and ASCE-7 sect. 15.5.3
Base Shear, $V = C_s I_p W$ =	0.252 W	RMI sect. 2.6.2

Rack Fixture

	Braced	Down Aisle	
Period, T (H_{rack} ≤ 96") =	0.265	1.249	sec. - RMI sect. 2.6.3
Period, T (96" < H_{rack} ≤ 120") =	0.483	1.182	sec. - RMI sect. 2.6.3
Period, T (H_{rack} > 120") =	0.352	1.348	sec. - RMI sect. 2.6.3
Period, T (H_{rack} = 168" w/Base Isolator) =	NA	NA	sec. - RMI sect. 2.6.3 <--- Not Applicable for this project
C_s (H_{rack} ≤ 96") =	0.252	0.108	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (96" < H_{rack} ≤ 120") =	0.252	0.076	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (H_{rack} > 120") =	0.252	0.067	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (H_{rack} = 168" w/Base Isolator) =	NA	NA	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s, min =	0.044	0.044	--> RMI sect. 2.6.3 and ASCE-7 sect. 15.5.3

T_S, (S_{D1}/S_{DS}) = 0.536 sec.

T_L = 6 sec.

Base Shear:

	Braced	Down Aisle	
V (H_{rack} ≤ 96") = $C_s I_p W_s$ =	0.252	0.108	W_s --> RMI sect. 2.6.2
V (96" < H_{rack} ≤ 120") = $C_s I_p W_s$ =	0.252	0.076	W_s --> RMI sect. 2.6.2
V (H_{rack} > 120") = $C_s I_p W_s$ =	0.252	0.067	W_s --> RMI sect. 2.6.2
V (H_{rack}=168"w/Base Iso) = $C_s I_p W_s$ =	NA	NA	W_s --> RMI sect. 2.6.2

Load Combinations for LRFD Member Design (RMI, Section 2.2):

for RISA Frame analysis

DL = Dead Load
PL = Maximum load from pallets/product stored on racks
EL = Seismic Load - RMI section 2.6.6 - Vert. Distribution

LC #1: 1.4DL + 1.2PL

LC #2: 1.2DL + 1.4PL

LC #6a: $(0.9-0.2S_{DS})$DL + $(0.9-0.2S_{DS})PL_{app}$ + ρ(1.0)EL <--- PL_{app} = (0.67)PL at each shelf level; ρ = 1.3 at "Braced" frames

0.6982 DL 0.6982 PL_{app} 1.0000 EL

LC #6b: $(0.9-0.2S_{DS})$DL + $(0.9-0.2S_{DS})PL_{app}$ + ρ(1.0)EL <--- PL_{app} = (1.0)PL at top shelf only; ρ = 1.3 at "Braced" frames

0.6982 DL 0.6982 PL_{app} 1.0000 EL

LC #5: $(1.2+0.2S_{DS})$DL + $(0.85+0.2S_{DS})$PL + ρ(1.0)EL <--- ρ = 1.3 at "Braced" frames

1.4018 DL 1.0518 PL 1.0000 EL

PRCA20231436

JBA Johnston Burkholder Associates consulting structural engineers	PROJECT NO: 2135202403	Sheet No: 2	Of: 11
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	Braced	Down Aisle	
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Overstrength Factor, Omega, Ω_o =	2.0		ASCE-7, Table 15.4-1
Deflection Amplification Factor, C_d =	3.5		ASCE-7, Table 15.4-1
Detail Reference Section =	15.5.3		ASCE-7, Table 15.4-1
Occupancy Category =	II		IBC, Table 1604.5
Importance Factor, I_p =	1.5		ASCE-7 Sect. 15.5.3
0.2 Second Period Accel., S_s =	1.261 g		IBC Figs. 1613.2.1(1-8), ASCE-7 Figs. 22-1 thru 22-8
1.0 Second Period Accel., S_1 =	0.435 g		IBC Figs. 1613.2.1(1-8), ASCE-7 Figs. 22-1 thru 22-8
(Soil) Site Class =	D (Default)		IBC 1613.2.2 -> ASCE-7, Table 20.3-1
F_a =	1.20		IBC Table 1613.2.3(1), ASCE-7 Table 11.4-1
F_v =	1.87		IBC Table 1613.2.3(2), ASCE-7 Table 11.4-2 + Sect. 11.4.8
S_{MS} =	1.513 g		IBC eq. 16-36, ASCE-7 eq. 11.4-1
S_{M1} =	0.811 g		IBC eq. 16-37, ASCE-7 eq. 11.4-2
S_{DS} =	1.009 g		IBC eq. 16-38, ASCE-7 eq. 11.4-3
S_{D1} =	0.541 g		IBC eq. 16-39, ASCE-7 eq. 11.4-4
Seismic Design Category			
--based on S_{DS} =	D		IBC Table 1613.2.5(1), ASCE-7 Table 11.6-1
-- based on S_{D1} =	D		IBC Table 1613.2.5(2), ASCE-7 Table 11.6-2

Shelving Fixture

C_s =	0.252	RMI sect. 2.6.3 w/ASCE-7, Sect. 11.4.8
C_s, min =	0.044	RMI sect. 2.6.3 and ASCE-7 sect. 15.5.3
Base Shear, $V = C_s I_p W$ =	0.378 W	RMI sect. 2.6.2

Rack Fixture

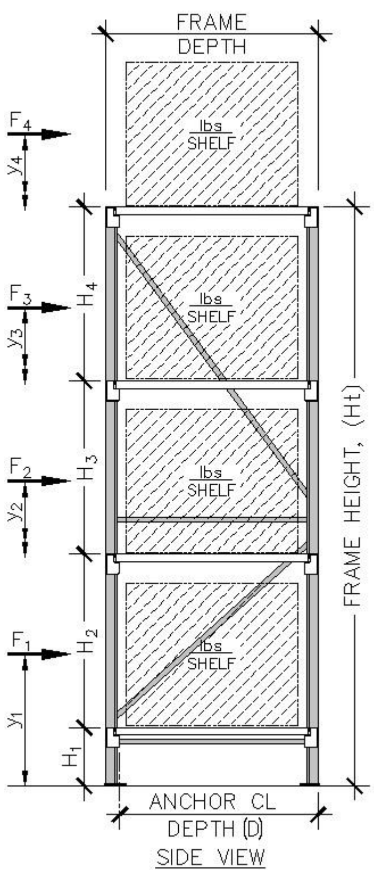
	Braced	Down Aisle	
Period, T (H_{rack} ≤ 96") =	0.265	1.249	sec. - RMI sect. 2.6.3
Period, T (96" < H_{rack} ≤ 120") =	0.483	1.182	sec. - RMI sect. 2.6.3
Period, T (H_{rack} > 120") =	0.352	1.348	sec. - RMI sect. 2.6.3
Period, T (H_{rack} = 168" w/Base Isolator) =	NA	NA	sec. - RMI sect. 2.6.3 <--- Not Applicable for this project
C_s (H_{rack} ≤ 96") =	0.252	0.108	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (96" < H_{rack} ≤ 120") =	0.252	0.076	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (H_{rack} > 120") =	0.252	0.067	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s (H_{rack} = 168" w/Base Isolator) =	NA	NA	--> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
C_s, min =	0.044	0.044	--> RMI sect. 2.6.3 and ASCE-7 sect. 15.5.3

Base Shear:

	Braced	Down Aisle	
V (H_{rack} ≤ 96") = $C_s I_p W_s$ =	0.378	0.162 W_s	--> RMI sect. 2.6.2
V (96" < H_{rack} ≤ 120") = $C_s I_p W_s$ =	0.378	0.114 W_s	--> RMI sect. 2.6.2
V (H_{rack} > 120") = $C_s I_p W_s$ =	0.378	0.100 W_s	--> RMI sect. 2.6.2
V (H_{rack}=168"w/Base Iso) = $C_s I_p W_s$ =	NA	NA W_s	--> RMI sect. 2.6.2

Racking Anchorage Design - Frame Load Diagram

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)



Base Shear, RMI, sect. 2.6.2:

$V = (C_s)(I_p)(W_s)$ C_sI_p, C_s based on frame height and I_p = 1.0 or 1.5 with Public Access

$W_s = (0.67(PL_{RF})(PL)) + DL$

PL_{RF} = 1.0 for Cross-Aisle and Down-Aisle frames

PL = (0.67)PL for RMI, sect. 2.6.9(1) & ASCE 7, 15.5.3.6(a)

(1.0)PL for RMI, sect. 2.6.9(2) & ASCE 7, 15.5.3.6(b)

Overturning Stability:

Center of Mass (CM) of Product Load (PL) is typically 20" above the shelf or (1/2)(Shelf height, H_i) when shelf height is < 40" (which is the assumed pallet height).

$F_{x = 1..n}$, is set at a Service Load level using $V = (0.7)[C_sI_pW_s]$

Load Case #1: (2/3)PL at each shelf level, RMI, sect.2.6.9(1) & ASCE 7, 15.5.3.6(a)

ω_x= (0.67)PL	$h_x = y_i$	$(0.7V)(\omega_x)(h_x)$ $(\omega_x)(h_x)$	$\Sigma\omega_x h_x$	Ovrturn'g Mom, M_{OT}	Resist'g Mom, M_{RST}
ω_4	y_4	$(\omega_4)(y_4)$	F_{x4}	$(F_{x4})(y_4)$	$\omega_4(D/2)$
ω_3	y_3	$(\omega_3)(y_3)$	F_{x3}	$(F_{x3})(y_3)$	$\omega_3(D/2)$
ω_2	y_2	$(\omega_2)(y_2)$	F_{x2}	$(F_{x2})(y_2)$	$\omega_2(D/2)$
ω_1	y_1	$(\omega_1)(y_1)$	F_{x1}	$(F_{x1})(y_1)$	$\omega_1(D/2)$
$\omega_u=DL_{frame}$	$y_u=Ht/2$	$(\omega_u)(y_u)$	F_{xu}	$(F_{xu})(y_u)$	$\omega_u(D/2)$
		$\Sigma(\omega_x)(h_x)$	$\Sigma(F_{xu}+F_x)=0.7V$	$M_{OT}=\Sigma(F)(y)$	$M_{RST}=\Sigma(\omega)(D/2)$

Load Case #2: (1.0)PL at top shelf level only, RMI, sect.2.6.9(2) & ASCE 7, 15.5.3.6(b)

ω_x= (1.0)PL	$h_x = y_i$	$(0.7V)(\omega_x)(h_x)$ $(\omega_x)(h_x)$	$\Sigma\omega_x h_x$	Ovrturn'g Mom, M_{OT}	Resist'g Mom, M_{RST}
ω_4	y_4	$(\omega_4)(y_4)$	F_{x4}	$(F_{x4})(y_4)$	$\omega_4(D/2)$
$\omega_u=DL_{frame}$	$y_u=Ht/2$	$(\omega_u)(y_u)$	F_{xu}	$(F_{xu})(y_u)$	$\omega_u(D/2)$
		$\Sigma(\omega_x)(h_x)$	$\Sigma(F_{xu}+F_x)=0.7V$	$M_{OT}=\Sigma(F)(y)$	$M_{RST}=\Sigma(\omega)(D/2)$

Factor Of Safety against Overturning at Load Case #1 & #2, $FOS_{OT} = M_{RST}/M_{OT}$:

FOS_{OT} < 1.0; Anchor Bolts required for both Shear & Tension

FOS_{OT} >= 1.0; Anchor Bolts required for Shear only, no net uplift tension at base connection

FOS_{OT} >= 1.5; Anchor Bolts required for Shear only for frames 96" tall and taller at sales floor area and for all frames taller than 48" in storage areas (non sales floor).

Anchorage Connection Design Load Combinations: RMI, section 2.2 - Strength Design

RMI LC #6: $(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})(0.67)PL - \Omega_o(EL)$, for Load Case #1 Shear, $R_{uh} = (\Omega_o)V/2$

$(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PL - \Omega_o(EL)$, for Load Case #2 Tension, $R_{uv} = [(\Omega_o M_{OT}/0.7)-(0.9-0.2S_{DS})M_{RST}]/(FrameDepth)$

Rack Frame Member Design Load Combinations: RMI, section 2.2 - Strength Design

RMI LC #1: 1.4DL + 1.2PL Redundancy factor, ρ = 1.0 <- SDC "A"/"B"/"C", RMI, sect. 2.6.2.1

RMI LC #2: 1.2DL + 1.4PL 1.3 <- SDC "D"/"E"/"F", RMI, sect. 2.6.2.1

RMI LC #5: $(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})(0.67)PL + \rho EL$, for Load Case #1

$(1.2+0.2S_{DS})DL + (0.85+0.2S_{DS})PL + \rho EL$, for Load Case #2

RMI LC #6: $(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})(0.67)PL - \rho EL$, for Load Case #1

$(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PL - \rho EL$, for Load Case #2

Rack Framing Member Design: RMI, section 6.3

Per ANSI/MH16.1, Section 6.3, effective lengths may be determined by rational methods consistent with AISI or AISC. AISC Design by Second-Order Analysis, Section C2.2a is used. Notional loads are applied to gravity load cases and K=1.0 is used since the ratio of second-order drift to first-order drift $(P-\delta) / (P-\Delta) < 1.1$.

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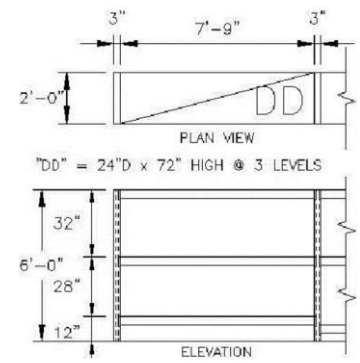
Storage Rack - Seismic Design | Rack DD | 24-72 075A

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Seismic Importance Factor (I_p) =	1.0	<--- No Public Access Allowed (Typ. at Back	
Supported on Elevated Floor (Y/N):	No	Stockroom / Grocery Storage Areas)	
Max. Weight per level (2 Pallets / shelf) =	1200 lbs/shelf		
Weight of Unit =	250 lbs	<--- Shipping weight per Manuf.	
Rack Trib width (CL-to-CL of frames) =	96 in	Total Shelf Load	
h_9 =	0 in	0 lbs	
h_8 =	0 in	0 lbs	
h_7 =	0 in	0 lbs	
h_6 =	0 in	0 lbs	
h_5 =	0 in	0 lbs	
h_4 =	0 in	0 lbs	
h_3 =	32 in	1200 lbs	
h_2 =	28 in	1200 lbs	
h_1 =	12 in	1200 lbs	
Total Shelf Height, H_t =	72 in		
Unit Height, H_u =	72 in		
Unit Base Depth, D =	24 in		



Note: Per ANSI MH16.1, Section 6.3, effective lengths may be determined by rational methods consistent with AISI or AISC. AISC Design by Second-Order Analysis, Section C2.2a is used. Notional loads are applied to gravity load cases and K=1.0 is used since the ratio of second-order drift to first-order drift (P-δ) / (P-Δ) < 1.1.

Overturning Stability (Load cases are per ASCE 7 sect. 15.5.3.3.2):

Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]
[RMI sect 2.6.2, PL_{RF} = 1.0]

Seismic (C_s)(I_p) =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
W_s = (0.67)(PL_{RF})((0.67)PL)+DL =	1866.0 lbs		
Base Shear, V = $C_s I_p W_s$ =	470.6 lbs	(Braced)	
	202.0 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads, E = 0.7)	F_9 =	0.0 lbs	@ 0 in (CM)
Note:	F_8 =	0.0 lbs	@ 0 in (CM)
(CM) = Product Center of	F_7 =	0.0 lbs	@ 0 in (CM)
Mass typically 20 inches	F_6 =	0.0 lbs	@ 0 in (CM)
above the top of shelf at	F_5 =	0.0 lbs	@ 0 in (CM)
each level.	F_4 =	0.0 lbs	@ 0 in (CM)
	F_3 =	163.7 lbs	@ 92 in (CM)
	F_2 =	99.6 lbs	@ 56 in (CM)
	F_1 =	46.2 lbs	@ 26 in (CM)
	F_u =	19.9 lbs	@ 36 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 22554 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 31944 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 1.416
NO UPLIFT - ANCHORS REQUIRED

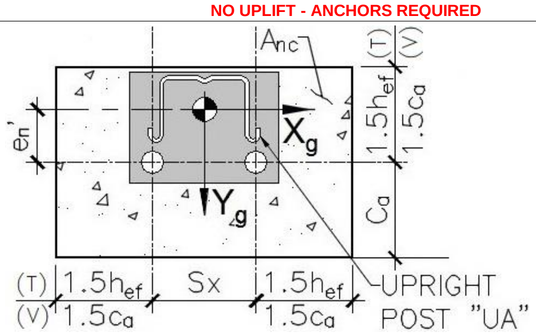
Load Case 2 [RMI sect. 2.6.8(2), PL=1.0(PL)]
[RMI 2.6.2, PL_{RF}=1.0]

Seismic (C_s)(I_p) =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
W_s = (0.67)(PL_{RF})((1)PL)+DL =	1054.0 lbs		
Base Shear, V = $C_s I_p W_s$ =	265.8 lbs	(Braced)	
	114.1 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads)	F_9 =	0.0 lbs	
	F_8 =	0.0 lbs	
	F_7 =	0.0 lbs	
	F_6 =	0.0 lbs	
	F_5 =	0.0 lbs	
	F_4 =	0.0 lbs	
	F_3 =	172.0 lbs	@ 92 in (CM)
	F_2 =	0.0 lbs	
	F_1 =	0.0 lbs	
	F_u =	14.0 lbs	@ 36 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 16333 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 17400 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 1.065
NO UPLIFT - ANCHORS REQUIRED



Anchor Design (using "Cracked Concrete" Properties)

Upright Post Type =	UA		
Try:	1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed.		
Embedment (h_{nom}) =	2.5 in		
f_c' =	2500 psi		
e_n' =	1.875 in	<-- Eccen. of Anchor	
h_{ef} =	1.75 in	1.5(h_{ef}) =	2.625 in
Conc. thickness, t =	4 in	1.5(C_a) =	4.950 in
# of Anchors, n =	1		
Sx =	0.00 in		
Sy =	0.00 in		
A_{se} =	0.176 in^2		

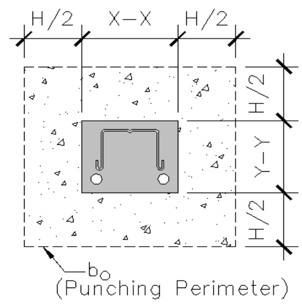
Base Reactions:

	LC #1		LC #2	
R_h =	165 lbs		93 lbs	
R_v =	0 lbs		0 lbs	
Overturning FOS =	1.416	< 1.5 ABs Reqd	1.065	< 1.5 ABs Reqd
Sliding Restraint, R_{RST}/FOS =	468lbs / 2.842 >= 1.5 OK		302lbs / 3.245 >= 1.5 OK	
Reactions (Factored Loads):	LC #1		LC #2	
Base Shear (R_{uh}) =	471 lbs		266 lbs	
Net Uplift (R_{uv}) =	0 lbs		0 lbs	
Overturning + Gravity (P_u) =	5303 lbs		2903 lbs	
Tension Allowables				
Steel Strength, ϕN_{sa} =	13309 lbs	<--ACI 318-14 Eq 17.4.1.2		
Concrete Breakout, (0.75)ϕN_{cbg} =	959 lbs	<--ACI 318-14 Eq 17.4.2.1b		
Pullout Strength, (0.75)ϕN_{pn} =	802 lbs	<--ACI 318-14 Eq 17.4.3.1		
Factored Tension Load, (N_u) =	0 lbs	(LC #1)	0 lbs	(LC #2)
max tension stress ratio (TSR) =	0.000	OK (LC#1)	0.000	OK (LC#2)
Shear Allowables				
Steel Strength, ϕV_{sa} =	4254 lbs	<--ACI 318-14 Eq 17.5.1.2c		
Concrete breakout (Yg), ϕV_{cbg} =	1078 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete breakout (Xg), ϕV_{cbg} =	782 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete pryout, ϕV_{cpg} =	1377 lbs	<--ACI 318-14 Eq 17.5.3.1b		
	LC #1		LC #2	
Factored Shear Load (V_u) : Braced =	471 lbs		266 lbs	
Down Aisle =	202 lbs		114 lbs	
Max shear stress ratio (VSR) : Braced =	0.436	OK	0.247	OK
Down Aisle =	0.258	OK	0.146	OK
Braced (TSR+VSR <= 1.2) =	0.436	<= 1.2 OK - LC #1 Controls		
Down Aisle (VSR <= 1.0) =	0.258	OK - LC #1 Controls		

USE: (1) 1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed. ICC REPORT #ESR-3889

Punching Shear Check:
(Design per ACI 318-14 section 14.5.5)

Max. Factored Vertical Load (P_u) =	5303	lbs	
Slab Concrete f'c =	2500	psi	
Slab thickness (t) =	4	in.	
Rack Post X-X =	5.00	in.	
Rack Post Y-Y =	3.75	in.	
b_o =	33.50	in.	
β =	1.33		
V_n =	22333	lbs	Table 14.5.5.1b
V_n max =	17822	lbs	Table 14.5.5.1c
ϕV_n =	10693	lbs	
$V_u/\phi V_n$ =	0.496	< 1.0 O.K.	



Max Vertical Load (ASD) - RMI, sect 2.1 - LC#4:
P = (1+0.105Sds)DL+(3/4)[(1.4+0.14Sds)PL+(0.7)ρEL]

S_{DS} =	1.009	(Ip=1)
DL = (Frame Wt/2) =	125	lbs
PL = Σ(Shelf Load h_1 - h_9)/2 =	1800	lbs
EL = $M_{OT, LC\#1}$ / ((0.7)(D)) =	2004	lbs

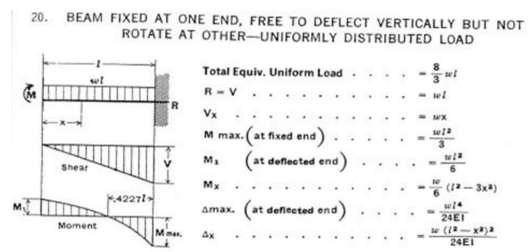
P = 3135 lbs <--- At Each Post

Max Vertical Load (LRFD) - RMI, sect 2.2 - LC#5:
P = (1.2+0.2SDS)DL + (1.2+0.2Sds)PL + ρEL

P_u = 5303 lbs <--- At Each Post

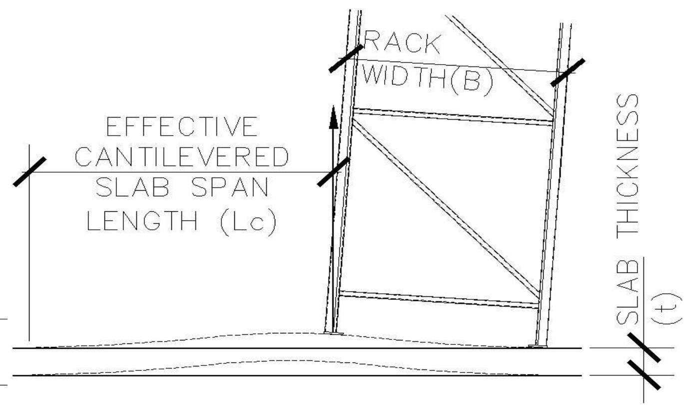
Slab tension based on Soil bearing area check:

Allowable soil bearing =	500	psf
Max. Service Vertical Load (P) =	3135	lbs
Area reqd. for bearing (A_{reqd}) =	6.27	ft^2
"b" distance =	30.05	in
Slab thickness (t) =	4.00	in
S = (1")(t)^2/6 =	2.67	in^3/in
ϕM_{nt} (tension allowable) = f_t*7.5*[(f'c)$^{1/2}$]*S =	600.00	in-lb/in
Factored uniform bearing, w_u = P_u / A_{reqd} =	5.87	lb/in/in
M_u = wL^2/3 = (w_u)[(b-min(X-X,Y-Y))/2)^2] / 3 =	338.50	in-lb/in - Defl. End M1 = 170 in-lb/in
$M_u/\phi M_{nt}$ =	0.564	< 1.0 O.K.



Rack FOS Overturning with Resistance from Effective Weight of Slab on Grade:

Width of Single Rack =	24	in
Slab thickness (t) =	4.0	in
Modulus of Rupture, f_r = 7.5*SQRT(f'c) =	375.0	psi
Concrete Slab Section Modulus, S = b(t)^2/6 =	32.0	in^3/ft
Allowable Concrete Slab Bending Moment, M_{all}/FS = S*f_r/1.5 =	666.7	ft*lbs/ft
Effective Cantilever Span Length (l_c) at M_{all} =	5.2	ft
Total Length of Slab (l_c + Width of Single Rack) =	7.2	ft
Trib. Width of Slab = Trib width of Rack =	8.0	ft
Weight of Concrete Slab at Rack (P_{conc}) =	2865.6	lbs
Resisting Moment - Concrete Slab at Rack, $M_{RST(slab)}$ = P_{conc} * l_c/2 =	123174	in*lbs
Load Combination #1: M_{OT} =	22554	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	155118	in*lbs
Total Overturning FOS =	6.878	OK
Load Combination #2: M_{OT} =	16333	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	140574	in*lbs
Total Overturning FOS =	8.607	OK



PRCA20231436

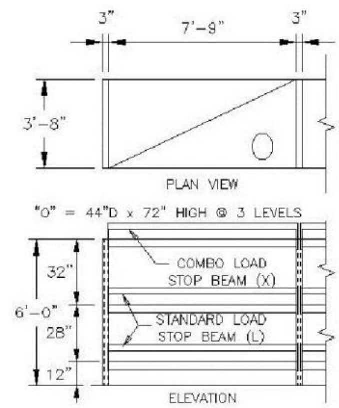
JOHNSTON BURKHOLDER ASSOCIATES consulting structural engineers 930 CENTRAL · KANSAS CITY, MO 64105 816.421.4200 · WWW.JBAENGR.COM	PROJECT NO: 2135202403	SHEET NO: 6	OF: 11
	PROJECT NAME: #2403 - Puyallup, WA		
	MADE BY: GMB	DATE: 09/07/21	
	CHECKED BY:	DATE:	

City of Puyallup
Development & Permitting Services
ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

Storage Rack - Seismic Design | Rack O | 44-72 075A

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Seismic Importance Factor (I_p) =	1.0	<--- No Public Access Allowed (Typ. at Back Stockroom / Grocery Storage Areas)
Supported on Elevated Floor (Y/N):	No	
Max. Weight per level (2 Pallets / shelf) =	1200 lbs/shelf	
Weight of Unit =	250 lbs	<--- Shipping weight per Manuf.
Rack Trib width (CL-to-CL of frames) =	96 in	**Total Shelf Load**
h_9 =	0 in	0 lbs
h_8 =	0 in	0 lbs
h_7 =	0 in	0 lbs
h_6 =	0 in	0 lbs
h_5 =	0 in	0 lbs
h_4 =	0 in	0 lbs
h_3 =	32 in	1200 lbs
h_2 =	28 in	1200 lbs
h_1 =	12 in	1200 lbs
Total Shelf Height, H_t =	72 in	
Unit Height, H_u =	72 in	
Unit Base Depth, D =	44 in	



Note: Per ANSI MH16.1, Section 6.3, effective lengths may be determined by rational methods consistent with AISI or AISC. AISC Design by Second-Order Analysis, Section C2.2a is used. Notional loads are applied to gravity load cases and K=1.0 is used since the ratio of second-order drift to first-order drift (P-δ) / (P-Δ) < 1.1.

Overturning Stability (Load cases are per ASCE 7 sect. 15.5.3.3.2):

Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]
[RMI sect 2.6.2, PL_{RF} = 1.0]

Seismic $(C_s)(I_p)$ =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
$W_s = (0.67)(PL_{RF})((0.67)PL)+DL$ =	1866.0 lbs		
Base Shear, $V = C_sI_pW_s$ =	470.6 lbs	(Braced)	
	202.0 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads, E = 0.7)	F_9 =	0.0 lbs	@ 0 in (CM)
Note:	F_8 =	0.0 lbs	@ 0 in (CM)
(CM) = Product Center of	F_7 =	0.0 lbs	@ 0 in (CM)
Mass typically 20 inches	F_6 =	0.0 lbs	@ 0 in (CM)
above the top of shelf at	F_5 =	0.0 lbs	@ 0 in (CM)
each level.	F_4 =	0.0 lbs	@ 0 in (CM)
	F_3 =	163.7 lbs	@ 92 in (CM)
	F_2 =	99.6 lbs	@ 56 in (CM)
	F_1 =	46.2 lbs	@ 26 in (CM)
	F_u =	19.9 lbs	@ 36 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 22554 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 58564 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 2.597
NO UPLIFT - NO ANCHORS REQUIRED

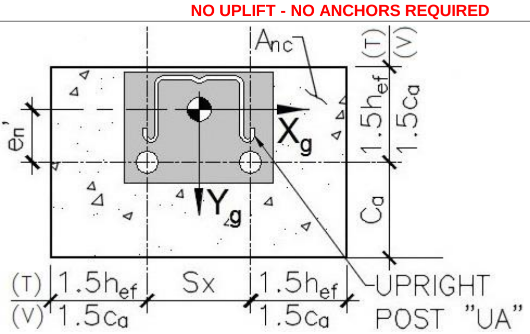
Load Case 2 [RMI sect. 2.6.8(2), PL=1.0(PL)]
[RMI sect 2.6.2, PL_{RF}=1.0]

Seismic $(C_s)(I_p)$ =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
$W_s = (0.67)(PL_{RF})((1)PL)+DL$ =	1054.0 lbs		
Base Shear, $V = C_sI_pW_s$ =	265.8 lbs	(Braced)	
	114.1 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads)	F_9 =	0.0 lbs	
	F_8 =	0.0 lbs	
	F_7 =	0.0 lbs	
	F_6 =	0.0 lbs	
	F_5 =	0.0 lbs	
	F_4 =	0.0 lbs	
	F_3 =	172.0 lbs	@ 92 in (CM)
	F_2 =	0.0 lbs	
	F_1 =	0.0 lbs	
	F_u =	14.0 lbs	@ 36 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 16333 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 31900 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 1.953
NO UPLIFT - NO ANCHORS REQUIRED



Anchor Design (using "Cracked Concrete" Properties)

Upright Post Type =	UA		
Try:	1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed.		
Embedment (h_{nom}) =	2.5 in		
f_c' =	2500 psi		
e_n' =	1.875 in	<-- Eccen. of Anchor	
h_{ef} =	1.75 in	$1.5(h_{ef})$ =	2.625 in
Conc. thickness, t =	4 in	$1.5(C_a)$ =	4.950 in
# of Anchors, n =	1		
Sx =	0.00 in		
Sy =	0.00 in		
A_{se} =	0.176 in^2		

Base Reactions:

	LC #1		LC #2	
R_h =	165 lbs		93 lbs	
R_v =	0 lbs		0 lbs	
Overturning FOS =	2.597	>= 1.5	1.953	>= 1.5
Sliding Restraint, R_{RST}/FOS =	361lbs / 2.194 >= 1.5 OK		225lbs / 2.414 >= 1.5 OK	
Reactions (Factored Loads):	LC #1		LC #2	
Base Shear (R_{uh}) =	471 lbs		266 lbs	
Net Uplift (R_{uv}) =	0 lbs		0 lbs	
Overturning + Gravity (P_u) =	4119 lbs		2045 lbs	
Tension Allowables				
Steel Strength, ϕN_{sa} =	13309 lbs	<--ACI 318-14 Eq 17.4.1.2		
Concrete Breakout, $(0.75)\phi N_{cbg}$ =	959 lbs	<--ACI 318-14 Eq 17.4.2.1b		
Pullout Strength, $(0.75)\phi N_{pn}$ =	802 lbs	<--ACI 318-14 Eq 17.4.3.1		
Factored Tension Load, (N_u) =	0 lbs	(LC #1)	0 lbs	(LC #2)
max tension stress ratio (TSR) =	0.000	OK (LC#1)	0.000	OK (LC#2)
Shear Allowables				
Steel Strength, ϕV_{sa} =	4254 lbs	<--ACI 318-14 Eq 17.5.1.2c		
Concrete breakout (Yg), ϕV_{cbg} =	1078 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete breakout (Xg), ϕV_{cbg} =	782 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete pryout, ϕV_{cpg} =	1377 lbs	<--ACI 318-14 Eq 17.5.3.1b		
	LC #1		LC #2	
Factored Shear Load (V_u) : Braced =	471 lbs		266 lbs	
Down Aisle =	202 lbs		114 lbs	
Max shear stress ratio (VSR) : Braced =	0.436	OK	0.247	OK
Down Aisle =	0.258	OK	0.146	OK
Braced (TSR+VSR <= 1.2) =	0.436	<= 1.2 OK - LC #1 Controls		
Down Aisle (VSR <= 1.0) =	0.258	OK - LC #1 Controls		

USE: NO UPLIFT - (1) 1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed.

PRCA20231436

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PROJECT NO:	SHEET NO:	OF:
2135202403	7	11
PROJECT NAME: #2403 - Puyallup, WA		
MADE BY: GMB	DATE: 09/07/21	
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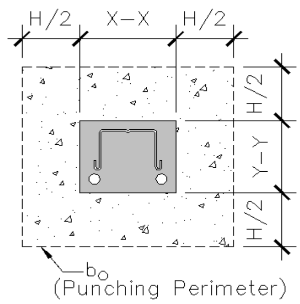
City of Puyallup
Development & Permitting Services
ISSUED PERMIT
Building | Planning
Engineering | Public Works
Fire | Traffic

Storage Rack - Seismic Design — Rack O — 44-72 075A

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Punching Shear Check:
(Design per ACI 318-14 section 14.5.5)

Max. Factored Vertical Load (P_u) =	4119	lbs	
Slab Concrete f'c =	2500	psi	
Slab thickness (t) =	4	in.	
Rack Post X-X =	5.00	in.	
Rack Post Y-Y =	3.75	in.	
b_o =	33.50	in.	
β =	1.33		
V_n =	22333	lbs	Table 14.5.5.1b
V_n max =	17822	lbs	Table 14.5.5.1c
ϕV_n =	10693	lbs	
$V_u/\phi V_n$ =	0.385	< 1.0 O.K.	



Max Vertical Load (ASD) - RMI, sect 2.1 - LC#4:
P = (1+0.105Sds)DL+(3/4)[(1.4+0.14Sds)PL+(0.7)ρEL]

S_{DS} =	1.009	(Ip=1)
DL = (Frame Wt/2) =	125	lbs
PL = Σ(Shelf Load h_1 - h_9)/2 =	1800	lbs
EL = $M_{OT,\ LC\#1}$ / ((0.7)(D)) =	1093	lbs

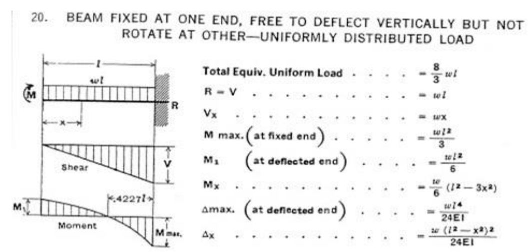
P = 2719 lbs <--- At Each Post

Max Vertical Load (LRFD) - RMI, sect 2.2 - LC#5:
P = (1.2+0.2SDS)DL + (1.2+0.2Sds)PL + ρEL

P_u = 4119 lbs <--- At Each Post

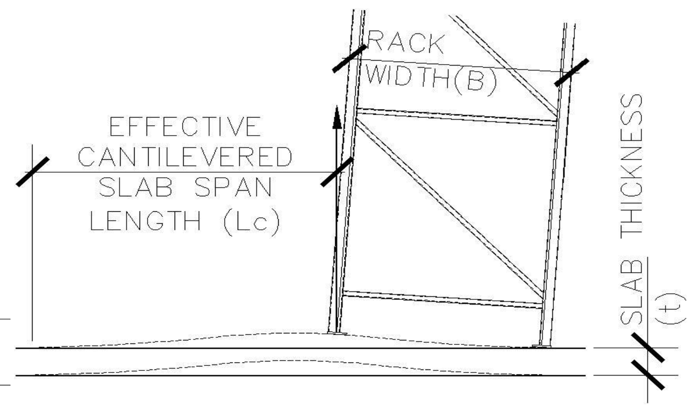
Slab tension based on Soil bearing area check:

Allowable soil bearing =	500	psf
Max. Service Vertical Load (P) =	2719	lbs
Area reqd. for bearing (A_{reqd}) =	5.44	ft^2
"b" distance =	27.98	in
Slab thickness (t) =	4.00	in
S = (1")$(t)^2$/6 =	2.67	in^3/in
ϕM_{nt} (tension allowable) = f_t*7.5*$[(f'_c)^{1/2}]$*S =	600.00	in-lb/in
Factored uniform bearing, w_u = P_u / A_{reqd} =	5.26	lb/in/in
M_u = $wL^2/3$ = $(w_u)[(b-\min(X\text{-}X,Y\text{-}Y))/2)^2]$ / 3 =	257.42	in-lb/in - Defl. End M1 = 129 in-lb/in
$M_u/\phi M_{nt}$ =	0.429	< 1.0 O.K.



Rack FOS Overturning with Resistance from Effective Weight of Slab on Grade:

Width of Single Rack =	44	in
Slab thickness (t) =	4.0	in
Modulus of Rupture, f_r = 7.5*SQRT(f'c) =	375.0	psi
Concrete Slab Section Modulus, S = $b(t)^2/6$ =	32.0	in^3/ft
Allowable Concrete Slab Bending Moment, M_{all}/FS = S*f_r/1.5 =	666.7	ft*lbs/ft
Effective Cantilever Span Length (l_c) at M_{all} =	5.2	ft
Total Length of Slab (l_c + Width of Single Rack) =	8.8	ft
Trib. Width of Slab = Trib width of Rack =	8.0	ft
Weight of Concrete Slab at Rack (P_{conc}) =	3532.3	lbs
Resisting Moment - Concrete Slab at Rack, $M_{RST(slab)}$ = P_{conc} * l_c/2 =	187153	in*lbs
Load Combination #1: M_{OT} =	22554	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	245717	in*lbs
Total Overturning FOS =	10.895	OK
Load Combination #2: M_{OT} =	16333	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	219053	in*lbs
Total Overturning FOS =	13.411	OK



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PROJECT NO:	SHEET NO:	OF:
2135202403	8	11
PROJECT NAME: #2403 - Puyallup, WA		
MADE BY: GMB	DATE: 09/07/21	
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City of Puyallup Development & Permitting Services ISSUED PERMIT
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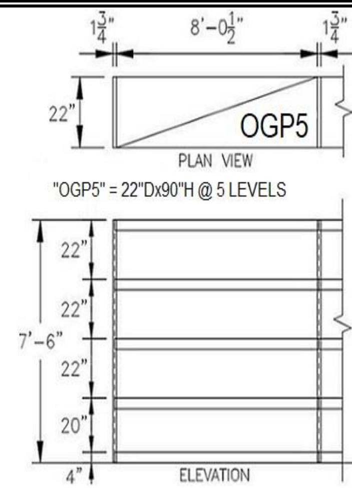
Storage Rack - Seismic Design — Rack OGP5 — 22-90 OGP

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Seismic Importance Factor (I_p) =	1.0	<--- No Public Access Allowed (Typ. at Back Stockroom / Grocery Storage Areas)	
Supported on Elevated Floor (Y/N):	No		
Max. Weight per level (2 Pallets / shelf) =	120 lbs/shelf		
Weight of Unit =	200 lbs	<--- Shipping weight per Manuf.	
Rack Trib width (CL-to-CL of frames) =	96.5 in		Total Shelf Load
h_9 =	0 in		0 lbs
h_8 =	0 in		0 lbs
h_7 =	0 in		0 lbs
h_6 =	0 in		0 lbs
h_5 =	22 in		120 lbs
h_4 =	22 in		120 lbs
h_3 =	22 in		120 lbs
h_2 =	20 in		120 lbs
h_1 =	4 in		120 lbs
Total Shelf Height, H_t =	90 in		
Unit Height, H_u =	90 in		
Unit Base Depth, D =	22 in		



Note: Per ANSI MH16.1, Section 6.3, effective lengths may be determined by rational methods consistent with AISI or AISC. AISC Design by Second-Order Analysis, Section C2.2a is used. Notional loads are applied to gravity load cases and K=1.0 is used since the ratio of second-order drift to first-order drift (P-δ) / (P-Δ) < 1.1.

Overturning Stability (Load cases are per ASCE 7 sect. 15.5.3.3.2):

Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]
[RMI sect 2.6.2, PL_{RF} = 1.0]

Seismic $(C_s)(I_p)$ =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
$W_s = (0.67)(PL_{RF})((0.67)PL)+DL$ =	469.3 lbs		
Base Shear, $V = C_sI_pW_s$ =	118.4 lbs	(Braced)	
	50.8 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads, E = 0.7)	F_9 =	0.0 lbs	@ 0 in (CM)
Note:	F_8 =	0.0 lbs	@ 0 in (CM)
(CM) = Product Center of	F_7 =	0.0 lbs	@ 0 in (CM)
Mass typically 20 inches	F_6 =	0.0 lbs	@ 0 in (CM)
above the top of shelf at	F_5 =	22.4 lbs	@ 110 in (CM)
each level.	F_4 =	16.1 lbs	@ 79 in (CM)
	F_3 =	11.6 lbs	@ 57 in (CM)
	F_2 =	7.1 lbs	@ 35 in (CM)
	F_1 =	2.9 lbs	@ 14 in (CM)
	F_u =	22.8 lbs	@ 45 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 5711 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 6622 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 1.160
NO UPLIFT - ANCHORS REQUIRED

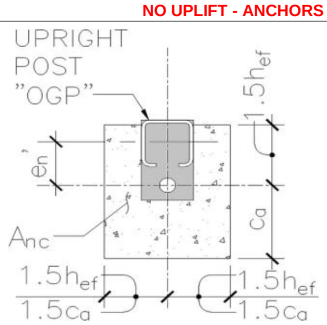
Load Case 2 [RMI sect. 2.6.8(2), PL=1.0(PL)]
[RMI 2.6.2, PL_{RF}=1.0]

Seismic $(C_s)(I_p)$ =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
$W_s = (0.67)(PL_{RF})((1)PL)+DL$ =	280.4 lbs		
Base Shear, $V = C_sI_pW_s$ =	70.7 lbs	(Braced)	
	30.4 lbs	(Down Aisle)	

Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)

(Service Loads)	F_9 =	0.0 lbs	
	F_8 =	0.0 lbs	
	F_7 =	0.0 lbs	
	F_6 =	0.0 lbs	
	F_5 =	29.4 lbs	@ 110 in (CM)
	F_4 =	0.0 lbs	
	F_3 =	0.0 lbs	
	F_2 =	0.0 lbs	
	F_1 =	0.0 lbs	
	F_u =	20.1 lbs	@ 45 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 4141 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 3520 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 0.850
UPLIFT - ANCHORS REQUIRED



Anchor Design (using "Cracked Concrete" Properties)

Upright Post Type =	OGP		
Try:	3/8"Ø DeWalt Screw Bolt+ Anchor - 2" embed.		
Embedment (h_{nom}) =	2 in		
f_c' =	2500 psi		
e_n' =	1.875 in	<-- Eccen. of Anchor	
h_{ef} =	1.33 in	$1.5(h_{ef})$ =	1.995 in
Conc. thickness, t =	4 in	$1.5(C_a)$ =	7.500 in
# of Anchors, n =	1		
Sx =	0.00 in		
Sy =	0.00 in		
A_{se} =	0.094 in^2		

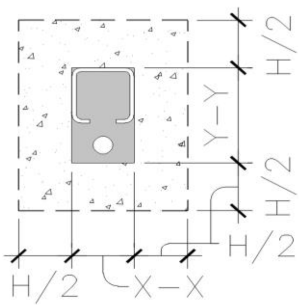
Base Reactions:

	LC #1		LC #2	
R_h =	41 lbs		25 lbs	
R_v =	0 lbs		28 lbs	
Overturning FOS =	1.160	< 1.5 ABs Reqd	0.850	< 1.5 ABs Reqd
Sliding Restraint, R_{RST}/FOS =	124lbs / 2.983 >= 1.5 OK		82lbs / 3.317 >= 1.5 OK	
Reactions (Factored Loads):	LC #1		LC #2	
Base Shear (R_{uh}) =	118 lbs		71 lbs	
Net Uplift (R_{uv}) =	0 lbs		426 lbs	
Overturning + Gravity (P_u) =	1280 lbs		746 lbs	
Tension Allowables				
Steel Strength, ϕN_{sa} =	5675 lbs	<--ACI 318-14 Eq 17.4.1.2		
Concrete Breakout, $(0.75)\phi N_{cbg}$ =	636 lbs	<--ACI 318-14 Eq 17.4.2.1b		
Pullout Strength, $(0.75)\phi N_{pn}$ =	439 lbs	<--ACI 318-14 Eq 17.4.3.1		
Factored Tension Load, (N_u) =	0 lbs	(LC #1)	426 lbs	(LC #2)
max tension stress ratio (TSR) =	0.000	OK (LC#1)	0.971	OK (LC#2)
Shear Allowables				
Steel Strength, ϕV_{sa} =	1449 lbs	<--ACI 318-14 Eq 17.5.1.2c		
Concrete breakout (Yg), ϕV_{cbg} =	1440 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete breakout (Xg), ϕV_{cbg} =	1152 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete pryout, ϕV_{cpg} =	913 lbs	<--ACI 318-14 Eq 17.5.3.1b		
	LC #1		LC #2	
Factored Shear Load (V_u) : Braced =	118 lbs		71 lbs	
Down Aisle =	51 lbs		30 lbs	
Max shear stress ratio (VSR) : Braced =	0.130	OK	0.077	OK
Down Aisle =	0.056	OK	0.033	OK
Braced (TSR+VSR <= 1.2) =	1.049	<= 1.2 OK - LC #2 Controls		
Down Aisle (VSR <= 1.0) =	0.056	OK - LC #1 Controls		

USE: (1) 3/8"Ø DeWalt Screw Bolt+ Anchor - 2" embed. ICC REPORT #ESR-3889

Punching Shear Check:
(Design per ACI 318-14 section 14.5.5)

Max. Factored Vertical Load (P_u) =	1280 lbs	
Slab Concrete f'c =	2500 psi	
Slab thickness (t) =	4 in.	
Rack Post X-X =	1.75 in.	
Rack Post Y-Y =	3.00 in.	
b_o =	25.50 in.	
β =	1.71	
V_n =	14733 lbs	Table 14.5.5.1b
V_n max =	13566 lbs	Table 14.5.5.1c
ϕV_n =	8140 lbs	
$V_u/\phi V_n$ =	0.157	< 1.0 O.K.



Max Vertical Load (ASD) - RMI, sect 2.1 - LC#4:
P = (1+0.105Sds)DL+(3/4)[(1.4+0.14Sds)PL+(0.7)ρEL]

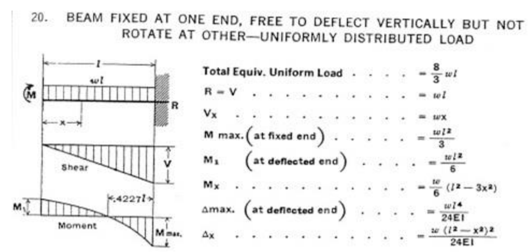
S_{DS} =	1.009	(Ip=1)
DL = (Frame Wt/2) =	100	lbs
PL = Σ(Shelf Load h_1 - h_9)/2 =	300	lbs
EL = $M_{OT, LC\#1}$ / ((0.7)(D)) =	554	lbs
P =	710 lbs	<--- At Each Post

Max Vertical Load (LRFD) - RMI, sect 2.2 - LC#5:
P = (1.2+0.2SDS)DL + (1.2+0.2Sds)PL + ρEL

P_u = 1280 lbs <--- At Each Post

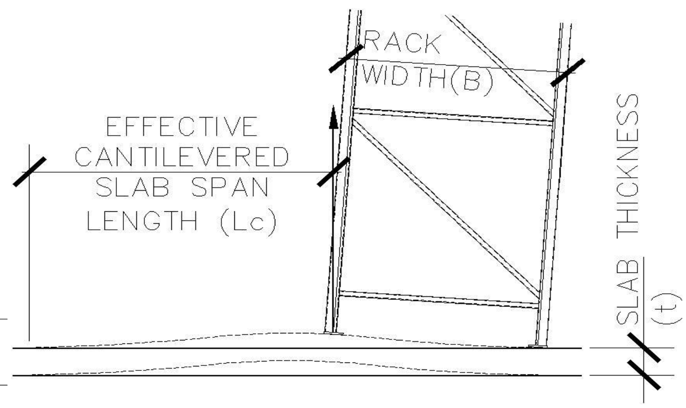
Slab tension based on Soil bearing area check:

Allowable soil bearing =	500	psf
Max. Service Vertical Load (P) =	710	lbs
Area reqd. for bearing (A_{reqd}) =	1.42	ft^2
"b" distance =	14.30	in
Slab thickness (t) =	4.00	in
$S = (1")(t)^2/6$ =	2.67	in^3/in
ϕM_{nt} (tension allowable) = $f_r*7.5*[(f'_c)^{1/2}]*S$ =	600.00	in-lb/in
Factored uniform bearing, $w_u = P_u / A_{reqd}$ =	6.26	lb/in/in
$M_u = wL^2/3 = (w_u)[(b-min(X-X,Y-Y))/2)^2] / 3$ =	82.16	in-lb/in - Defl. End M1 = 42 in-lb/in
$M_u/\phi M_{nt}$ =	0.137	< 1.0 O.K.



Rack FOS Overturning with Resistance from Effective Weight of Slab on Grade:

Width of Single Rack =	22	in
Slab thickness (t) =	4.0	in
Modulus of Rupture, f_r = 7.5*SQRT(f'c) =	375.0	psi
Concrete Slab Section Modulus, S = $b(t)^2/6$ =	32.0	in^3/ft
Allowable Concrete Slab Bending Moment, M_{all}/FS = S*f_r/1.5 =	666.7	ft*lbs/ft
Effective Cantilever Span Length (l_c) at M_{all} =	5.2	ft
Total Length of Slab (l_c + Width of Single Rack) =	7.0	ft
Trib. Width of Slab = Trib width of Rack =	8.0	ft
Weight of Concrete Slab at Rack (P_{conc}) =	2813.5	lbs
Resisting Moment - Concrete Slab at Rack, $M_{RST(slab)}$ = P_{conc} * l_c/2 =	118122	in*lbs
Load Combination #1: M_{OT} =	5711	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	124744	in*lbs
Total Overturning FOS =	21.843	OK
Load Combination #2: M_{OT} =	4141	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	121642	in*lbs
Total Overturning FOS =	29.377	OK



PRCA20231436

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PROJECT NO:	SHEET NO:	OF:
2135202403	10	11

PROJECT NAME: #2403 - Puyallup, WA

MADE BY: GMB | DATE: 09/07/21

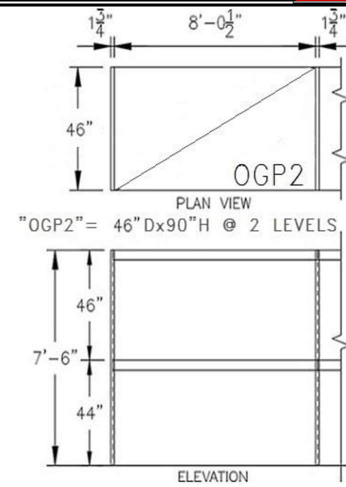
CHECKED BY: | DATE:

City of Puyallup Development & Permitting Services ISSUED PERMIT
Building | Planning | Engineering | Public Works | Fire | Traffic

Storage Rack - Seismic Design | Rack OGP2 (46-90 OGP) | 46-90 OGP

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Seismic Importance Factor (I_p) =	1.0	<--- No Public Access Allowed (Typ. at Back Stockroom / Grocery Storage Areas)	
Supported on Elevated Floor (Y/N):	No		
Max. Weight per level (2 Pallets / shelf) =	600 lbs/shelf		
Weight of Unit =	100 lbs	<--- Shipping weight per Manuf.	
Rack Trib width (CL-to-CL of frames) =	96.5 in	Total Shelf Load	
h_9 =	0 in	0 lbs	
h_8 =	0 in	0 lbs	
h_7 =	0 in	0 lbs	
h_6 =	0 in	0 lbs	
h_5 =	0 in	0 lbs	
h_4 =	0 in	0 lbs	
h_3 =	0 in	0 lbs	
h_2 =	46 in	600 lbs	
h_1 =	44 in	600 lbs	
Total Shelf Height, H_t =	90 in		
Unit Height, H_u =	90 in		
Unit Base Depth, D =	46 in		



Note: Per ANSI MH16.1, Section 6.3, effective lengths may be determined by rational methods consistent with AISI or AISC. AISC Design by Second-Order Analysis, Section C2.2a is used. Notional loads are applied to gravity load cases and K=1.0 is used since the ratio of second-order drift to first-order drift (P-δ) / (P-Δ) < 1.1.

Overturning Stability (Load cases are per ASCE 7 sect. 15.5.3.3.2):

Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]
[RMI sect 2.6.2, PL_{RF} = 1.0]

Seismic (C_s)(I_p) =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
W_s = (0.67)(PL_{RF})((0.67)PL)+DL =	638.7 lbs		
Base Shear, V = $C_s I_p W_s$ =	161.1 lbs	(Braced)	
	69.1 lbs	(Down Aisle)	

Horizontal forces / level, F_x = C_{vx}V (RMI sect 2.6.6)

(Service Loads, E = 0.7)	F_9 =	0.0 lbs	@ 0 in (CM)
Note:	F_8 =	0.0 lbs	@ 0 in (CM)
(CM) = Product Center of	F_7 =	0.0 lbs	@ 0 in (CM)
Mass typically 20 inches	F_6 =	0.0 lbs	@ 0 in (CM)
above the top of shelf at	F_5 =	0.0 lbs	@ 0 in (CM)
each level.	F_4 =	0.0 lbs	@ 0 in (CM)
	F_3 =	0.0 lbs	@ 0 in (CM)
	F_2 =	65.9 lbs	@ 110 in (CM)
	F_1 =	40.1 lbs	@ 67 in (CM)
	F_u =	6.7 lbs	@ 45 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 10241 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 20792 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 2.030
NO UPLIFT - NO ANCHORS REQUIRED

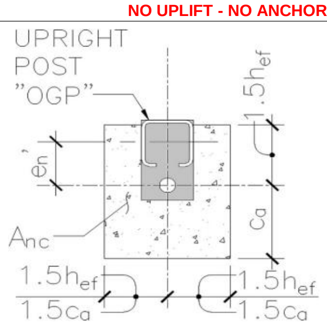
Load Case 2 [RMI sect. 2.6.8(2), PL=1.0(PL)]
[RMI 2.6.2, PL_{RF}=1.0]

Seismic (C_s)(I_p) =	0.252	W_s (Braced)	
	0.108	W_s (Down Aisle)	
W_s = (0.67)(PL_{RF})((1)PL)+DL =	502.0 lbs		
Base Shear, V = $C_s I_p W_s$ =	126.6 lbs	(Braced)	
	54.3 lbs	(Down Aisle)	

Horizontal forces / level, F_x = C_{vx}V (RMI sect 2.6.6)

(Service Loads)	F_9 =	0.0 lbs	
	F_8 =	0.0 lbs	
	F_7 =	0.0 lbs	
	F_6 =	0.0 lbs	
	F_5 =	0.0 lbs	
	F_4 =	0.0 lbs	
	F_3 =	0.0 lbs	
	F_2 =	83.0 lbs	@ 110 in (CM)
	F_1 =	0.0 lbs	
	F_u =	5.7 lbs	@ 45 in (CM)

Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$
M_{OT} = 9381 in-lbs
Calculate Resisting Moment (Service), M_{RST}
M_{RST} = 16100 in-lbs
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT}$ = 1.716
NO UPLIFT - NO ANCHORS REQUIRED



Anchor Design (using "Cracked Concrete" Properties)

Upright Post Type =	OGP		
Try:	3/8"Ø DeWalt Screw Bolt+ Anchor - 2" embed.		
Embedment (h_{nom}) =	2 in		
f_c' =	2500 psi		
e_n' =	1.875 in	<-- Eccen. of Anchor	
h_{ef} =	1.33 in	1.5(h_{ef}) =	1.995 in
Conc. thickness, t =	4 in	1.5(C_a) =	7.500 in
# of Anchors, n =	1		
Sx =	0.00 in		
Sy =	0.00 in		
A_{se} =	0.094 in^2		

Base Reactions:

	LC #1		LC #2	
R_h =	56 lbs		44 lbs	
R_v =	0 lbs		0 lbs	
Overturning FOS =	2.030	>= 1.5	1.716	>= 1.5
Sliding Restraint, R_{RST}/FOS =	135lbs / 2.403 >= 1.5 OK		114lbs / 2.567 >= 1.5 OK	
Reactions (Factored Loads):	LC #1		LC #2	
Base Shear (R_{uh}) =	161 lbs		127 lbs	
Net Uplift (R_{uv}) =	0 lbs		0 lbs	
Overturning + Gravity (P_u) =	1528 lbs		1056 lbs	
Tension Allowables				
Steel Strength, ϕN_{sa} =	5675 lbs	<--ACI 318-14 Eq 17.4.1.2		
Concrete Breakout, (0.75)ϕN_{cbg} =	636 lbs	<--ACI 318-14 Eq 17.4.2.1b		
Pullout Strength, (0.75)ϕN_{pn} =	439 lbs	<--ACI 318-14 Eq 17.4.3.1		
Factored Tension Load, (N_u) =	0 lbs	(LC #1)	0 lbs	(LC #2)
max tension stress ratio (TSR) =	0.000	OK (LC#1)	0.000	OK (LC#2)
Shear Allowables				
Steel Strength, ϕV_{sa} =	1449 lbs	<--ACI 318-14 Eq 17.5.1.2c		
Concrete breakout (Yg), ϕV_{cbg} =	1440 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete breakout (Xg), ϕV_{cbg} =	1152 lbs	<--ACI 318-14 Eq 17.5.2.1b		
Concrete pryout, ϕV_{cpg} =	913 lbs	<--ACI 318-14 Eq 17.5.3.1b		
	LC #1		LC #2	
Factored Shear Load (V_u) : Braced =	161 lbs		127 lbs	
Down Aisle =	69 lbs		54 lbs	
Max shear stress ratio (VSR) : Braced =	0.176	OK	0.139	OK
Down Aisle =	0.076	OK	0.060	OK
Braced (TSR+VSR <= 1.2) =	0.176	<= 1.2 OK - LC #1 Controls		
Down Aisle (VSR <= 1.0) =	0.076	OK - LC #1 Controls		

USE: NO UPLIFT - (1) 3/8"Ø DeWalt Screw Bolt+ Anchor - 2" embed.

PRCA20231436

JOHNSTON BURKHOLDER ASSOCIATES consulting structural engineers 930 CENTRAL · KANSAS CITY, MO 64105 816.421.4200 · WWW.JBAENGR.COM	PROJECT NO: 2135202403	SHEET NO: 11	OF: 11
	PROJECT NAME: #2403 - Puyallup, WA		
	MADE BY: GMB	DATE: 09/07/21	
	CHECKED BY:	DATE:	

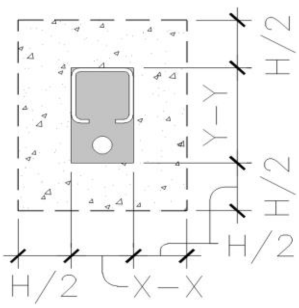
City of Puyallup Development & Permitting Services ISSUED PERMIT — Building | Planning | Engineering | Public Works | Fire | Traffic

Storage Rack - Seismic Design   Rack OGP2 (46-90 OGP)   46-90 OGP

IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)

Punching Shear Check:
(Design per ACI 318-14 section 14.5.5)

Max. Factored Vertical Load (P_u) =	1528	lbs	
Slab Concrete f'c =	2500	psi	
Slab thickness (t) =	4	in.	
Rack Post X-X =	1.75	in.	
Rack Post Y-Y =	3.00	in.	
b_o =	25.50	in.	
β =	1.71		
V_n =	14733	lbs	Table 14.5.5.1b
V_n max =	13566	lbs	Table 14.5.5.1c
ϕV_n =	8140	lbs	
$V_u/\phi V_n$ =	0.188	< 1.0 O.K.	



Max Vertical Load (ASD) - RMI, sect 2.1 - LC#4:
P = (1+0.105Sds)DL+(3/4)[(1.4+0.14Sds)PL+(0.7)ρEL]

S_{DS} =	1.009	(Ip=1)
DL = (Frame Wt/2) =	50	lbs
PL = Σ(Shelf Load h_1 - h_9)/2 =	600	lbs
EL = $M_{OT,\ LC\#1}$ / ((0.7)(D)) =	475	lbs

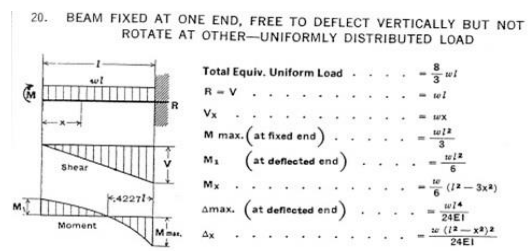
P = 966 lbs <--- At Each Post

Max Vertical Load (LRFD) - RMI, sect 2.2 - LC#5:
P = (1.2+0.2SDS)DL + (1.2+0.2Sds)PL + ρEL

P_u = 1528 lbs <--- At Each Post

Slab tension based on Soil bearing area check:

Allowable soil bearing =	500	psf
Max. Service Vertical Load (P) =	966	lbs
Area reqd. for bearing (A_{reqd}) =	1.93	ft^2
"b" distance =	16.68	in
Slab thickness (t) =	4.00	in
$S = (1")(t)^2/6$ =	2.67	in^3/in
ϕM_{nt} (tension allowable) = f_t*7.5*[(f'_c)$^{1/2}$]*S =	600.00	in-lb/in
Factored uniform bearing, $w_u = P_u / A_{reqd}$ =	5.49	lb/in/in
$M_u = wL^2/3 = (w_u)[(b-min(X-X,Y-Y))/2)^2] / 3$ =	102.02	in-lb/in - Defl. End M1 = 52 in-lb/in
$M_u/\phi M_{nt}$ =	0.170	< 1.0 O.K.



Rack FOS Overturning with Resistance from Effective Weight of Slab on Grade:

Width of Single Rack =	46	in
Slab thickness (t) =	4.0	in
Modulus of Rupture, $f_r = 7.5*SQRT(f'c)$ =	375.0	psi
Concrete Slab Section Modulus, $S = b(t)^2/6$ =	32.0	in^3/ft
Allowable Concrete Slab Bending Moment, $M_{all}/FS = S*f_r/1.5$ =	666.7	ft*lbs/ft
Effective Cantilever Span Length (l_c) at M_{all} =	5.2	ft
Total Length of Slab (l_c + Width of Single Rack) =	9.0	ft
Trib. Width of Slab = Trib width of Rack =	8.0	ft
Weight of Concrete Slab at Rack (P_{conc}) =	3617.7	lbs
Resisting Moment - Concrete Slab at Rack, $M_{RST(slab)} = P_{conc} * l_c/2$ =	195296	in*lbs
Load Combination #1: M_{OT} =	10241	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	216088	in*lbs
Total Overturning FOS =	21.101	OK
Load Combination #2: M_{OT} =	9381	in*lbs
$M_{RST(Rack)}$ + $M_{RST(slab)}$ =	211396	in*lbs
Total Overturning FOS =	22.535	OK