



STRUCTURAL FIXTURE ANCHORAGE CALCULATIONS

FOR

Puyallup, WA

310 31st Ave SE Store #2403

PREPARED FOR

CITY OF PUYALLUP, WA



JBA PROJECT #2135202403

PRCA20231436



Calculation Index

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PRCA20231436							
	PROJECT NO:	10		Sheet No:	Of:		
JOHNSTON	213520240 PROJECT NAME:	13			1	11	
BURKHOLDER	#2403 - Pu	ıyallup, V	VA				
ASSOCIATES	MADE BY: GMB			DATE:	09/07/	21	
consulting structural engineers	CHECKED BY:			DATE:	00/01/	City of	of Puyallup L Permitting Services
							ED PERMIT Planning
Lateral Seismic Analysis	0.0.40)					Engineering	
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH1	6.3-16)					Fire Or	Traffic
		Down Aisle				itude/Longitu	
Response Modification Factor, R =			ASCE-7, Table 15.4-			es (per Goo	
Overstrength Factor, Omega, Ω_0 =			ASCE-7, Table 15.4-				7.1611
Deflection Amplification Factor, C _d =			ASCE-7, Table 15.4-		VV 12	<mark>22° 17' 20"</mark> 12	22.2889
Detail Reference Section =			ASCE-7, Table 15.4-	·1			
Occupancy Category =			IBC, Table 1604.5 ASCE-7 Sect. 15.5.3				
Importance Factor, $I_p = 0.2$ Second Period Accel., $S_s = 0.2$			IBC Figs. 1613.2.1(1		7 Fige 22-1 tl	hru 22-8	
1.0 Second Period Accel., S ₁ =			IBC Figs. 1613.2.1(1	•	-		
(Soil) Site Class =	U		IBC 1613.2.2 -> ASC	•	-	11u 22-0	
(3011) Site Class = $F_a = \frac{1}{2}$	•	,	IBC Table 1613.2.3(•			
F_v =			IBC Table 1613.2.3(2	-		+ Sect 11.4	1 8
S _{MS} =			IBC eq. 16-36, ASCE	-		7 0001. 11.1	1.0
$S_{M1} =$			IBC eq. 16-37, ASCE	-			
$S_{DS} =$			IBC eq. 16-38, ASCE	-			
S _{D1} =			IBC eq. 16-39, ASCE	· ·			
Seismic Design Category		•					
based on S _{DS} =			IBC Table 1613.2.5(1), ASCE-7	Table 11.6-1		
based on S _{D1} =			IBC Table 1613.2.5(2	-			
Shelving Fixture							
C_s =			RMI sect. 2.6.3 w/AS				
C _s , min =			RMI sect. 2.6.3 and /	ASCE-7 sec	ct. 15.5.3		
Base Shear, $V = C_s I_p W =$ Rack Fixture			RMI sect. 2.6.2				
Period, T ($H_{rack} \le 96$ ") =		Down Aisle 1 249	sec RMI sect. 2.6.3	3 T _o ($(S_{D1}/S_{DS}) = 0.$	536 sec	
Period, T (96" < $H_{rack} \le 120$ ")			sec RMI sect. 2.6.3	-	$T_1 = 6$		
Period, T (H _{rack} > 120") =			sec RMI sect. 2.6.3		-		
Period, T (H _{rack} = 168" w/Base Isolator) =	: NA	NA	sec RMI sect. 2.6.3				
$C_s (H_{rack} \le 96") =$			> min[S_{DS}/R , S_{D1}/R		w/ASCE-7,		
$C_s (96" < H_{rack} \le 120") =$			> min[S_{DS}/R , S_{D1}/R		w/ASCE-7,		
$C_s (H_{rack} > 120") =$			> min[S_{DS}/R , S_{D1}/R		w/ASCE-7,		
C _s (H _{rack} = 168" w/Base Isolator) =			\rightarrow min[S_{DS}/R , S_{D1}/R		w/ASCE-7,	Sect. 11.4.	8
C_s , min = Base Shear:	0.044	0.044	> RMI sect. 2.6.3 a	and ASCE-7	sect. 15.5.3		
	_{ack} ≤ 96") = 0	?lW. =	0.252 Down Aisle	W> R	MI sect. 2.6	2	
V (96" < H _{rac}					MI sect. 2.6		
	$\frac{1}{120} = 0$				MI sect. 2.6		
V (H _{rack} =168"w/B	ase Iso) = 0	$C_s I_p W_s =$			MI sect. 2.6.		
Load Combinations for LRFD Member I	Design (RMI,						
for RISA Frame analysis			DL = Dead Load				
LC #1 : 1.4DL + 1.2PL			PL = Maximum load	-	-		
LC #2: 1.2DL + 1.4PL	(4.0)		EL = Seismic Load -				
LC #6a : (0.9-0.2S _{DS})DL + (0.9-0.2S _{DS})P 0.6982 DL 0.6982		L < PL 1.0000		n snelf leve	ει; ρ = 1.3 at "l	≾raced" fram	nes
U.6962 DL U.6962 LC #6b: (0.9-0.2S _{DS})DL + (0.9-0.2S _{DS})P				helf only: o	= 1.3 at "Bra	ced" frames	
0.6982 DL 0.6982		1.0000	• • • • • • • • • • • • • • • • • • • •	orny, p	at bid	.54 11411105	
LC #5: (1.2+0.2S _{DS})DL + (0.85+0.2S _{DS}				nes			
1.4018 DL 1.0518		1.0000					

PRCA20231436							
1110/120231430	PROJECT NO:		Shee	et No:		Of:	
JOHNSTON	21352024	103		2		1	1
BURKHOLDER	PROJECT NAME: #2403 - P	uvallun V	VA				
	MADE BY:	у.аа.р, <u>.</u>	DAT	E:			
ASSOCIATES	GMB				09/0)7/21	
consulting structural engineers	CHECKED BY:		DAT	Έ:		Deve	City of Puyallup elopment & Permitting Se ISSUED PERMIT
Lateral Seismic Analysis			<u> </u>				Building Plannir
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16	5.3-16)					E	Fire Public Wi
·	Braced	Down Aisle			Store I		naituda
Response Modification Factor, R =	4.0	6.0	ASCE-7, Table 15.4-1			nates (per G	•
Overstrength Factor, Omega, Ω_0 =	2.0		ASCE-7, Table 15.4-1			47° 09' 40"	47.161
Deflection Amplification Factor, C _d =	3.5		ASCE-7, Table 15.4-1		W	122° 17' 20"	122.288
Detail Reference Section =	15.5.3		ASCE-7, Table 15.4-1	L			
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 (Def	ault)	IBC 1613.2.2 -> ASCE-7	7, Table 2	20.3-1		
$F_a =$	1.20		IBC Table 1613.2.3(1), A	ASCE-7	Гable 11.4	J-1	
$F_v =$	1.87		IBC Table 1613.2.3(2), A	ASCE-7	Гable 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), A	ASCE-7	Γable 11.6	i-1	
based on S_{D1} =	D		IBC Table 1613.2.5(2), A	ASCE-7	Γable 11.6	i-2	
Shelving Fixture							
$C_s =$			RMI sect. 2.6.3 w/ASCE				
C_s , min =	0.044		RMI sect. 2.6.3 and ASC	CE-7 sect	i. 15.5.3	1	
Base Shear, $V = C_s I_p W =$	0.378	W	RMI sect. 2.6.2			1	
Rack Fixture	Braced	Down Aisle					
Period, T ($H_{rack} \le 96$ ") =	0.265		sec RMI sect. 2.6.3				
Period, T (96" < H _{rack} ≤ 120") =	0.483		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 Ap	plicable fo	or this proje	ect
$C_s (H_{rack} \le 96") =$	0.252	0.108	> min[S_{DS}/R , $S_{D1}/((T)$			', Sect. 11.4	4.8
C (06" - H < 120") -	0.050	0.076	- min[C /D C ///T)	\/D\\ 1	WACCE 7		4.0

	Biacoa	DOWN / NOIC
Period, T (H _{rack} ≤ 96") =	0.265	1.249 sec RMI sect. 2.6.3
Period, T (96" $< H_{rack} \le 120$ ") =	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
$C_s (H_{rack} \le 96") =$	0.252	0.108> min[S_{DS}/R , $S_{D1}/((T)(R))$] w/ASCE-7, Sect. 11.4.8
$C_s (96" < H_{rack} \le 120") =$	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

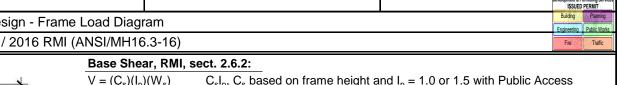
$V (H_{rack} \le 96") = C_s I_p W_s =$	0.378	0.162 W _s > RMI sect. 2.6.2
$V (96" < H_{rack} \le 120") = C_s I_p W_s =$	0.378	0.114 W _s > RMI sect. 2.6.2
$V (H_{rack} > 120") = C_s I_p W_s =$	0.378	0.100 W _s > RMI sect. 2.6.2
$V (H_{rack}=168"w/Base Iso) = C_sI_pW_s =$	NA	NA W _s > RMI sect. 2.6.2

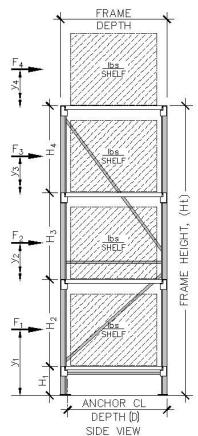
consulting structural engineers

PROJECT NO:	Sheet No:	Of:
2135202403	3	11
PROJECT NAME:	•	•
#2403 - Puyallup, WA		
MADE BY:	DATE:	
GMB	09	9/07/21
CHECKED BY:	DATE:	City of Puyallup

Racking Anchorage Design - Frame Load Diagram

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





 $V = (C_s)(I_p)(W_s)$

 $C_s I_p$, 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)

$\omega_x =$	b - v	(0.7V)(ω_x)(h_x)	Ovrturn'g	Resist'g
(0.67)PL	$h_x = y_i$	$(\omega_x)(h_x)$	∑ω _x h _x	Mom, M_OT	Mom, M_{RST}
ω_4	y ₄	$(\omega_4)(y_4)$	F _{x4}	$(F_{x4})(y_4)$	ω 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}	$(F_{x1})(y_1)$	$\omega_1(D/2)$
$\omega_u \text{=} DL_{frame}$	y _u =Ht/2	$(\omega_u)(y_u)$	F_{xu}	$(F_{xu})(y_u)$	$\omega_u(D/2)$
		$\sum (\omega^x)(\mu^x)$	$\Sigma(F_{xu}+F_{xi})=0.7V$	$M_{OT}=\Sigma(F)(y)$	$M_{RST}=\Sigma(\omega)(D/2)$

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 =	h - v	(0.7V)(ω_x)(h _x)	Ovrturn'g	Resist'g
(1.0)PL	$h_x = y_i$	$(\omega_x)(h_x)$	Σω _x h _x	Mom, M_{OT}	Mom, M _{RST}
ω_4	y ₄	$(\omega_4)(y_4)$	F _{x4}	$(F_{x4})(y_4)$	$\omega_4(D/2)$
ω_u =DL _{frame}	y _u =Ht/2	$(\omega_u)(y_u)$	F _{xu}	$(F_{xu})(y_u)$	$\omega_u(D/2)$
		$\sum (\omega_x)(h_x)$	∑(F _{xu} +F _{x4})=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: $(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})(0.67)PL - \Omega_0(EL)$, for Load Case #1

Shear, $R_{uh} = (\Omega_0)V/2$

 $(0.9-0.2S_{DS})DL + (0.9-0.2S_{DS})PL - \Omega_0(EL)$, for Load Case #2

Tension, $R_{IIV} = [(\Omega_0 M_{OT}/0.7) - (0.9 - 0.2 S_{DS}) M_{RST}]/(FrameDepth)$

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

RMI LC #1: 1.4DL + 1.2PL

Redundancy factor, $\rho = 1.0$

<- 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

 $(0.9\text{-}0.2S_{DS})DL$ + $(0.9\text{-}0.2S_{DS})PL$ - $\rho EL,$ 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 $(P-\delta) / (P-\Delta) < 1.1$.

PRCA20231436						_	
JOHNSTON	PROJECT NO:	402		SHEET NO:	1	OF:	11
BURKHOLDER	PROJECT NAME:	2135202403 4 11 PROJECT NAME:					11
ASSOCIATES	#2403 - I	403 - Puyallup, WA					
consulting structural engineers	MADE BY:			DATE:	00/6		
930 CENTRAL · KANSAS CITY, MO 64105	GMB CHECKED BY:			DATE:	09/0	07/21	City of Davidles
816.421.4200 · WWW.JBAENGR.COM	CHECKED BY:			DATE:			City of Puyallup Development & Permitting Services ISSUED PERMIT
Storage Rack - Seismic Design	Rack DD		24-72 075	A			Building Planning
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH	16 3-16)						Engineering Public Works Fire Traffic
Seismic Importance Factor (I_p) = 1.0		Access Allowed (Tvp. at Back				THE COLUMN
Supported on Elevated Floor (Y/N): No	Stockroom / Gr	rocery Storage A					
Max. Weight per level (2 Pallets / shelf) = 1200 lbs/she Weight of Unit = 250 lbs		veight per Manuf.			3"	7'-9"	3"
Rack Trib width (CL-to-CL of frames) = 96 in	11 0	Total Shelf Loa	<u>d</u>		-	/ -3	
$h_9 = 0$ in $h_8 = 0$ in		0 lbs 0 lbs		2'-0"			
$h_7 = 0$ in $h_6 = 0$ in		0 lbs 0 lbs				PLAN VIEW	
$h_5 = 0$ in		0 lbs		"DD"	= 24"D x 72"		VELS
$h_4 = 0 \text{ in} \\ h_3 = 32 \text{ in}$		0 lbs 1200 lbs		1	-		
$h_2 = 28 \text{ in}$		1200 lbs		33	2"		>
h ₁ = 12 in Total Shelf Height, H ₁ = 72 in		1200 lbs		6'-0"	+		
Unit Height, $H_u = 72$ in				28	В"		>
Unit Base Depth, D = 24 in Note: Per ANSI MH16.1, Section 6.3, effective lengths m.	ay be determined b	y rational method	ds consistent with AISI or AISC.	T 13	2"		
AISC Design by Second-Order Analysis, Section C2.2a is is used since the ratio of second-order drift to first-order d			gravity load cases and K=1.0		1	ELEVATION	1
Overturning Stability (Load cases are per ASCE 7 sect.	, , , , ,	.1.]			
Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]	13.3.3.3.2).		Load Case 2 [RMI sect. 2.6.8(2), PL=1.0(PL)]			
[RMI sect 2.6.2, $PL_{RF} = 1.0$] Seismic $(C_s)(I_p) = 0.252 \text{ W}_s$ (Braced)			[RMI 2.6 Seismic $(C_s)(I_p)$ =	5.2, PL _{RF} =1.0]	W _s (Braced)		
0.108 W _s (Down A	sle)			0.108	W _s (Down Aisle	e)	
$W_s = (0.67)(PL_{RF})((0.67)PL) + DL = 1866.0 \text{ lbs}$ Base Shear, $V = C_s I_p W_s = 470.6 \text{ lbs}$ (Braced)			$W_s = (0.67)(PL_{RF})((1)PL)+DL =$ Base Shear, $V = C_sI_0W_s =$		(Braced)		
202.0 lbs (Down Aisle	ı			114.1 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)			Horizontal forces / level, F (Service Loads) F ₉ =		ect 2.6.6)		
Note: F ₈ = 0.0 lbs @ 0 in (CM)			F ₈ =	0.0 lbs			
(CM) = Product Center of F_7 = 0.0 lbs @ 0 in (CM) Mass typically 20 inches F_6 = 0.0 lbs @ 0 in (CM)			F ₇ = F ₆ =				
above the top of shelf at $F_5 = 0.0 \text{ lbs}$ @ 0 in (CM)			F ₅ =	0.0 lbs			
each level. $F_4 = 0.0$ lbs @ 0 in (CM) $F_3 = 163.7$ lbs @ 92 in (CM))		F ₄ = F ₃ =		@ 92 in (CM)		
$F_2 = 99.6 \text{ lbs}$ @ 56 in (CM) $F_1 = 46.2 \text{ lbs}$ @ 26 in (CM)			F ₂ = F ₁ =				
F _u = 19.9 lbs @ 36 in (CM	,		F _u =	14.0 lbs	@ 36 in (CM)		
Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$ $M_{OT} = 22554 \text{ in-lbs}$			Calculate Overturning Moment Mort =	(Service), M _{OT} = 16333 in-lbs	= Σf _i h _i		
Calculate Resisting Moment (Service), M _{RST}			Calculate Resisting Moment (S	ervice), M _{RST}			
$M_{RST} = 31944 \text{ in-lbs}$ Factor of Safety, FOS _{OT} = $M_{RST}/M_{OT} = 1.416$			Factor of Safety, FOS	17400 in-lbs $S_{OT} = M_{RST}/M_{OT} = 0$	1.065		
NO UPLIFT - ANCHORS REQUIRED			NO UPLIF	T - ANCHORS	REQUIRED		
Anc ES			Base Reactions:	LC #1		LC #2	
			$R_h = R_v =$			93 lbs 0 lbs	
- P X 4 15 15.			Overturning FOS = Sliding Restraint, R _{RST} /FOS =		< 1.5 ABs Reqd	1.065 302lbs / 3.24	< 1.5 ABs Reqd
			Reactions (Factored Loads):		>= 1.5 OK	LC #2	3 >= 1.3 OK
4 4 Yyg 4			Base Shear $(R_{uh}) =$ Net Uplift $(R_{uv}) =$			266 lbs 0 lbs	
4 '.g 4 O			Overturning + Gravity (P _u) =			2903 lbs	
(7) 1 51			Tension Allowables Steel Strength, φN _{sa} =	13309 lbs	<aci 318-14<="" td=""><td>Ea 17.4.1.2</td><td></td></aci>	Ea 17.4.1.2	
(T) 1.5h _{ef} Sx 1.5h _{ef} UPRIGHT (V) 1.5ca 1.5ca POST "L	1 1 11	(Concrete Breakout, (0.75)	959 lbs	<aci 318-14<="" td=""><td>Eq 17.4.2.1b</td><td></td></aci>	Eq 17.4.2.1b	
(v) ¹ 1.5c _a 1 1.5c _a 1 POST "L	IA		Pullout Strength, $(0.75)\phi N_{pn} =$ Factored Tension Load, $(N_u) =$		<aci 318-14<br="">(LC #1)</aci>	Eq 17.4.3.1 0 lbs	(LC #2)
Anchor Design (using "Cracked Concrete" Properties)			max tension stress ratio (TSR) =	0.000	OK (LC#1)	0.000	OK (LC#2)
Upright Post Type = UA Try: 1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed.			$\begin{tabular}{ll} Shear Allowables \\ Steel Strength, $\phi V_{sa} = $$ \end{tabular}$		<aci 318-14<="" td=""><td>•</td><td></td></aci>	•	
Embedment $(h_{non}) = 2.5 \text{ in}$ $f_c' = 2500 \text{ psi}$			Concrete breakout (Yg), $\phi V_{cbq} =$ Concrete breakout (Xg), $\phi V_{cbq} =$		<aci 318-14<br=""><aci 318-14<="" td=""><td></td><td></td></aci></aci>		
en' = 1.875 in < Eccen. of Anchor			Concrete prediction (Ag), $\psi V_{cbq} =$	1377 lbs	<aci 318-14<="" td=""><td>Eq 17.5.3.1b</td><td></td></aci>	Eq 17.5.3.1b	
$h_{ef} = 1.75 \text{ in}$ $1.5(h_{ef}) = 2.625 \text{ in}$ Conc. thickness, t = 4 in $1.5(C_a) = 4.950 \text{ in}$		Factored S	hear Load (V _u): Braced =	LC #1 471 lbs		LC #2 266 lbs	
# of Anchors, n = 1			Down Aisle =	202 lbs	OK	114 lbs	OK
Sx = 0.00 in Sy = 0.00 in		ıvıax snear stre	ess ratio (VSR) : Braced = Down Aisle =		OK OK	0.247 0.146	OK OK
A _{se} = 0.176 in^2			Braced (TSR+VSR <= 1.2) =	0.436	<= 1.2 OK - LC		
USE: (1) 1/2"Ø DeWalt Screw Bolt	₊ Anchor - 2	1/2" embe	Down Aisle (VSR <= 1.0) = d_ ICC REPORT #FSI		OK - LC #1 Co	1111015	
JOE. (1) 112 DETTAIL OCIEW DOIL	. 7.1.01101 - 2	.,		. 5555			

PRCA20231436			
JOHNSTON	PROJECT NO:	SHEET NO:	OF:
BURKHOLDER	2135202403 PROJECT NAME:	5	11
ASSOCIATES	#2403 - Puyallup, WA		
consulting structural engineers	MADE BY:	DATE:	_
930 CENTRAL · KANSAS CITY, MO 64105	GMB CHECKED BY:	09/0	07/21 City of Puyallup
816.421.4200 · WWW.JBAENGR.COM	CHECKED BY:	DATE:	Development & Permitting Services ISSUED PERMIT
Storage Rack - Seismic Design	Rack DD 24-72 0	75A	Building Planning
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH16	6.3-16)		Engineering Public Works Fire Traffic
$\begin{array}{c} \textbf{Punching Shear Check:} \\ (\text{Design per ACI 318-14 section 14.5.5}) \\ \text{Max. Factored Vertical Load } (P_{\text{u}}) = & 5303 \text{ lbs} \\ \text{Slab Concrete } f \text{c} = & 2500 \text{ psi} \\ \text{Slab thickness } (t) = & 4 \text{ in.} \\ \text{Rack Post X-X} = & 5.00 \text{ in.} \\ \text{Rack Post Y-Y} = & 3.75 \text{ in.} \\ b_{\text{o}} = & 33.50 \text{ in.} \\ \beta = & 1.33 \\ V_{\text{n}} = & 22333 \text{ lbs Table 14.5.} \\ V_{\text{n}} \text{max} = & 17822 \text{ lbs Table 14.5.} \\ \phi V_{\text{n}} = & 10693 \text{ lbs} \\ V_{\text{v}}/\phi V_{\text{n}} = & 0.496 < 1.0 \text{ O.K.} \\ \\ \textbf{Slab tension based on Soil bearing area check:} \\ Allowable soil bearing = & 500 \\ \text{Max. Service Vertical Load } (P) = & 3135 \\ \text{Area reqd. for bearing } (A_{\text{reqd}}) = & 6.27 \\ \text{"b" distance} = & 30.05 \\ \text{Slab thickness } (t) = & 4.00 \\ \text{S} = & (1^m)(t)^2/6 = & 2.67 \\ \phi M_{\text{nt}} \text{ (tension allowable)} = f_1^* 7.5^* [(f'_{\text{c}})^{1/2}]^* S = & 600.00 \\ \end{array}$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	Prical Load (ASD) - RMI, sect 2.1 - I 0.105Sds)DL+(3/4)[(1.4+0.14Sds)PL S _{DS} = DL = (Frame Wt/2) = PL = Σ(Shelf Load h ₁ - h ₉)/2 = EL = M _{OT, LO#1} / ((0.7)(D)) P = 3135 lbs < At Each Prical Load (LRFD) - RMI, sect 2.2 - 2+0.2SDS)DL + (1.2+0.2Sds)PL + pE P _U = 5303 lbs < At Each Prical AT ONE END, FREE TO DEFLECT ROTATE AT OTHER—UNIFORMLY DISTI	LC#4: +(0.7)pEL] = 1.009 (lp=1) = 125 lbs = 1800 lbs = 2004 lbs h Post -LC#5: :I. h Post VERTICALLY BUT NOT RIBUTED LOAD - 8/3 wI -
$M_u = wL^2/3 = (w_u)[(b-min(X-X,Y-Y))/2)^2]/3 = 338.50$	in-lb/in - Defl. End M1 = 170 in-lb/in < 1.0 O.K.	Δmax. (at deflected end) .	$ = \frac{w!^4}{24E!} $ $ = \frac{w(!^4 - x^2)^2}{24E!} $
Rack FOS Overturning with Resistance from Effective Weight of Slab on Width of Single Rack = Slab thickness (t) = Modulus of Rupture, f, = 7.5°SQRT(f'c) = Concrete Slab Section Modulus, S = b(t)²/6 = Allowable Concrete Slab Bending Moment, M _{all} /FS = S°t/1.5 = Effective Cantilever Span Length (l _c) at M _{all} = Total Length of Slab (l _c + Width of Single Rack) = Trib. Width of Slab = Trib width of Rack = Weight of Concrete Slab at Rack (P _{conc}) = Resisting Moment - Concrete Slab at Rack, M _{RST(slab)} = P _{conc} * l _c /2 = Load Combination #1: M _{RST(Rack)} + M _{RST(slab)} = Total Overturning FOS =	24 in 4.0 in 375.0 psi 32.0 in³/ft 666.7 ft*lbs/ft 5.2 ft 7.2 ft 8.0 ft 2865.6 lbs 123174 in*lbs 22554 in*lbs 155118 in*lbs 6.878 OK	RACK WIDTH(I TIVE VERED SPAN (Lc)	SLAB THICKNESS (t)
	16333 in*lbs 140574 in*lbs	<u></u>	
Total Overturning FOS =	8.607 OK		

PRCA20231436		
JOHNSTON	PROJECT NO:	SHEET NO: OF:
	2135202403	6 11
BURKHOLDER	PROJECT NAME: #2403 - Puyallup, WA	
ASSOCIATES	MADE BY:	DATE:
consulting structural engineers	GMB	09/07/21
930 CENTRAL · KANSAS CITY, MO 64105 816.421.4200 · WWW.JBAENGR.COM	CHECKED BY:	DATE: City of Poyanup Development & Permitting Services
810.421.4200 · W W W.JBAENGR.COM		ISSUED PERMIT
Storage Rack - Seismic Design	Rack O 44-72 07	5A Engineering Public Works
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH1	6.3-16)	Fire Traffic
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)	3" -, ^, 3"
Max. Weight per level (2 Pallets / shelf) = 1200 lbs/shelf Weight of Unit = 250 lbs	< Shipping weight per Manuf.	7'-9"
Rack Trib width (CL-to-CL of frames) = 96 in	Total Shelf Load	
$h_9= 0$ in $h_8= 0$ in	0 lbs 0 lbs	3'-8"
$h_7 = 0$ in	0 lbs	0 1
$h_6 = 0$ in $h_5 = 0$ in	0 lbs 0 lbs	PLAN VIEW
$h_4 = 0$ in	0 lbs	"O" = 44"D x 72" HIGH @ 3 LEVELS
$h_3 = 32 \text{ in}$ $h_2 = 28 \text{ in}$	1200 lbs 1200 lbs	
$h_1 = 12 \text{ in}$	1200 lbs	32" — COMBO LOAD STOP BEAM (X)
Total Shelf Height, $H_t=72$ in Unit Height, $H_u=72$ in		6'-0" STANDARD LOAD
Unit Base Depth, D = 44 in		28" STOP BEAM (L)
, , ,	be determined by rational methods consistent with AISI or AIS ed. Notional loads are applied to gravity load cases and K=1.	12"
is used since the ratio of second-order drift to first-order drift	$(P-\delta) / (P-\Delta) < 1.1.$	₹ ELEVATION J
Overturning Stability (Load cases are per ASCE 7 sect. 1		
Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)] [RMI sect 2.6.2, PL _{RF} = 1.0]	Load Case 2 [RMI sect. 2.6.	8(2), PL=1.0(PL)] 2.6.2, PL _{RF} =1.0]
Seismic $(C_s)(I_p) = 0.252 W_s$ (Braced)	Seismic (C _s)(I _s	$) = 0.252 W_s (Braced)$
$0.108~W_s$ (Down Aisl $W_s = (0.67)(PL_{RF})((0.67)PL)+DL = 1866.0$ lbs	e) $W_s = (0.67)(PL_{RF})((1)PL)+D$	$0.108 \text{ W}_{\text{s}} \text{ (Down Aisle)}$ L = 1054.0 lbs
Base Shear, $V = C_s I_p W_s = 470.6 \text{ lbs}$ (Braced)	Base Shear, $V = C_s I_p V$	s = 265.8 lbs (Braced)
202.0 lbs (Down Aisle) Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)	Horizontal forces / Jeve	114.1 lbs (Down Aisle) I, $F_x = C_{vx}V$ (RMI sect 2.6.6)
(Service Loads, E = 0.7) $F_9 = 0.0$ lbs @ 0 in (CM)	(Service Loads) F	g= 0.0 lbs
Note: $F_8 = 0.0 \text{ lbs}$ @ 0 in (CM) (CM) = Product Center of $F_7 = 0.0 \text{ lbs}$ @ 0 in (CM)		f ₈ = 0.0 lbs f ₇ = 0.0 lbs
Mass typically 20 inches $F_6 = 0.0 \text{ lbs}$ @ 0 in (CM)		7 = 0.0 lbs
above the top of shelf at $F_5 = 0.0 \text{ lbs}$ @ 0 in (CM) each level. $F_4 = 0.0 \text{ lbs}$ @ 0 in (CM)		f ₅ = 0.0 lbs 4 = 0.0 lbs
$F_3 = 163.7 \text{ lbs}$ @ 92 in (CM)	F	3 = 172.0 lbs @ 92 in (CM)
$F_2 = 99.6 \text{ lbs}$ @ 56 in (CM) $F_1 = 46.2 \text{ lbs}$ @ 26 in (CM)		2 = 0.0 lbs 1 = 0.0 lbs
$F_u = 19.9 \text{ lbs}$ @ 36 in (CM)	F	u = 14.0 lbs @ 36 in (CM)
Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$ $M_{OT} = 22554 \text{ in-lbs}$	Calculate Overturning Mom	ent (Service), $M_{OT} = \Sigma f_i h_i$ $T_i = 16333 in-lbs$
Calculate Resisting Moment (Service), M _{RST}	Calculate Resisting Momen	(Service), M _{RST}
$M_{RST} = 58564 \text{ in-lbs}$ Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT} = 2.597$		$_{T}$ = 31900 in-lbs OS _{OT} = M _{RST} /M _{OT} = 1.953
NO UPLIFT - NO ANCHORS REQUIRED		FT - NO ANCHORS REQUIRED
Anc T	Base Reactions:	LC #1 LC #2
		h = 165 lbs 93 lbs v = 0 lbs 0 lbs
	Overturning FO	S = 2.597 >= 1.5 1.953 >= 1.5
5 Xg → -: -:	Sliding Restraint, R _{RST} /FO Reactions (Factored Load	S = 361lbs / 2.194 >= 1.5 OK 225lbs / 2.414 >= 1.5 OK ls): LC #1 LC #2
	Base Shear (R _{ul}) = 471 lbs 266 lbs
4 4 1 Y _g 4 3 3	Net Uplift (R_u Overturning + Gravity (P	
	Tension Allowables	
(T) 1.5 h_{ef} Sx 1.5 h_{ef} UPRIGHT	Steel Strength, ϕN_c Concrete Breakout, (0.75) ϕN_c	
(V) 1.5ca 1 1.5ca POST "UA	Pullout Strength, (0.75)φN	_{on} = 802 lbs <aci 17.4.3.1<="" 318-14="" eq="" td=""></aci>
Anchor Design (using "Cracked Concrete" Properties)	Factored Tension Load, (N max tension stress ratio (TSR	. , , , , , , , , , , , , , , , , , , ,
Upright Post Type = UA	Shear Allowables	<u> </u>
Try: 1/2"Ø DeWalt Screw Bolt+ Anchor - 2 1/2" embed. Embedment (h _{nom}) = 2.5 in	Steel Strength, ϕV Concrete breakout (Yg), ϕV_c	
f _c ' = 2500 psi	Concrete breakout (Xg), ϕV_{cl}	g = 782 lbs <aci 17.5.2.1b<="" 318-14="" eq="" td=""></aci>
$e_n' = 1.875 \text{ in} < \text{ Eccen. of Anchor}$ $h_{ef} = 1.75 \text{ in} = 1.5(h_{ef}) = 2.625 \text{ in}$	Concrete pryout, φV _c	_{sq} = 1377 lbs <aci 17.5.3.1b<br="" 318-14="" eq="">LC #1 LC #2</aci>
Conc. thickness, $t = \frac{4 \text{ in}}{1.5(C_a)} = 4.950 \text{ in}$	Factored Shear Load (V _u): Brace	d = 471 lbs 266 lbs
# of Anchors, n =1 Sx =0.00 in	Down Aisl Max shear stress ratio (VSR): Brace	
Sy = 0.00 in	Down Aisl	e = 0.258 OK 0.146 OK
A _{se} = 0.176 in^2	Braced (TSR+VSR <= 1.2 Down Aisle (VSR <= 1.0	,
USE: NO UPLIFT - (1) 1/2"Ø DeWalt	•	, = 0.250 OK - 20 #1 OUIIIOIS
3321 113 31 211 1 (1) 112 8 BOTTAIL		

Storage Rack - Seismic Design Rack O	PRCA20231436				
### BURKHOLDER ASSOCIATES ### 2403 - Puyallup, WA ### 2403 - Puyallup		OHNSTON			
#2403 - Puyallup, WA GMB GOENTRAL - KANSA CITY, MO 64105 816-421-4200 - WWW.JBAENGR.COM Storage Rack - Seismic Design Rack O 44-72 075A Second Se					11
### Structural engineers SMB			#2403 - Puyallup, WA		
Storage Rack - Seismic Design Rack O					7/04
Storage Rack - Seismic Design Rack O 44-72 075A					City of Puyallup
Big 2018 ASCE 7-16 / 2016 RMI (ANSI/MH16.3-16)	816.421.4200 · WV	WW.JBAENGR.COM			ISSUED PERMIT
Punching Shear Check: (Design per ACJ 318-14 section 14.5.5) Max. Factored Vertical Load (P.) = 4119 lbs Slab thorocrete fre = 250 psi Slab Concrete free free free free free free free	Storage Rack - Seis	mic Design	Rack O 44-72 075	A	Engineering Public Works
Design per ACJ 318-14 section 14.5.5	IBC 2018 / ASCE 7-	16 / 2016 RMI (ANSI/MH1	6.3-16)		Fire Traffic
	Punching Shear Check: (Design per ACI 318-14 section 1- Max. Factored Vertical I Slab Con Slab thic Rack Rack Slab tension based on Soil bear Max. Are	4.5.5) Load (P_u) = 4119 lbs Increte f'c = 2500 psi Iskness (t) = 4 in. Post X-X = 5.00 in. Post Y-Y = 3.75 in. b_0 = 33.50 in. $β$ = 1.33 V_n = 22333 lbs Table 14.5 V_n max = 17822 lbs Table 14.5 V_n = 10693 lbs $V_u/ΦV_n$ = 0.385 < 1.0 O.K. Fing area check: Allowable soil bearing = 500 Service Vertical Load (P) = 2719 a reqd. for bearing (A_{reqd}) = 5.44 "b" distance = 27.98 Slab thickness (t) = 4.00 $S = (1^n)(0)^2/6 = 2.67$ cowable) = $f_1^*7.5^*[(f_0^*)^{1/2}]^*S = 600.00$ Find bearing, $w_u = P_u / A_{reqd} = 5.26$ $V_u/Φ/M_{ret} = 0.429$ Alistance from Effective Weight of Slab on Width of Single Rack = Slab thickness (t) = 0.429 Alistance from Effective Weight of Slab on Width of Single Rack = 9 Slab Bending Moment, $M_{all}/FS = S^*f_1/1.5 = 0.000$ Fifective Cantilever Span Length (I_0) at $M_{all} = 0.000$ Correte Slab Section Modulus, $S = b(t)^2/6 = 0.000$ Fifective Cantilever Span Length (I_0) at $M_{all} = 0.0000$ Alistance from Effective Weight of Single Rack = 100000000000000000000000000000000000	Max Vertic: P = (1+0.10	DAT ONE END, FREE TO DEFLECT TATE AT OTHER—Uniform Load DAT ONE END, FREE TO DEFLECT TATE AT OTHER—Uniform Load M M max. (at deflected end) M M AX. (at deflected end)	+(0.7)pEL] = 1.009 (lp=1) = 125 lbs = 1800 lbs = 1093 lbs = 1093 lbs = 1093 lbs Post LC#5: L In Post VERTICALLY BUT NOT RIBUTED LOAD - \$\frac{8}{3}\text{si}\$ - \$\frac{\text{si}}{6}(12 - 3\text{si})\$ - \$\frac{\text{si}}{6}(2 - 3\text{si})\$ - \$\frac{\text{si}}{24}\text{El} - \$\frac{\text{si}}{6}(12 - 3\text{si})\$ - \$\frac{\text{si}}{24}\text{El} - \$\frac{\text{si}}{6}(12 - 3\text{si})\$ - \$\frac{\text{si}}{24}\text{El}

### 135020203 Storage Rack - Kansas Cert Pro Model 105 105	PRCA20231436					
### ASSOCIATES Onsulting structural engineers 99 (Christian Kansas Christ World) 18						4.4
### ASSOCIATES 90 CRATEAL - KANASA CITY, MO 64105 \$16.04.12.04 - WWW.HIARDRCOM Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ 22-90 OGP Storage Rack - Seismic Design Rack OGP\$ Storage Rack - Seismic Rack Rack Rack Rack Rack Rack Rack Rac				8	3	11
Some continues Section Continues C			۱۸/۸			
Storage Rack - Selsmin Lose Lose Line Lose Lose Lose Lose Lose Lose Lose Los			VV /\	DATE:		
93 CENTRAL: KANAS CITY, MO 64105 Storage Rack - Selsmin Design Rack OOPS Storage Rack - Selsmin Design Rack OOPS Sacrote impairance Start (1) Success of life and the self in the self					09/07/21	
Storage Rack - Seismic Design Rack OGPS 22-90 OGP				DATE:		
Screen Company Compa	810.421.4200 * W W W.JBAENGR.COM					ISSUED PERMIT
Secretary Secretary Company	Storage Rack - Seismic Design	Rack OGP5	22-90 OG	>		
March Property P	IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH1	6.3-16)				
March Property P			f (Typ. at Back		13" o' n1"	13"
Content Cont	Supported on Elevated Floor (Y/N): No	Stockroom / Grocery Storage	Areas)		1 4 8 - 0 <u>7</u>	— ▶ 4
Reads. The worth (CL-to-CL of framewal) Part Oin		< Shipping weight per Manu	ıf.	-		
	Rack Trib width (CL-to-CL of frames) = 96.5 in	Total Shelf Lo		22"	1 00	DE >
Note: Per ANSI MB Boss Depth, 10 - 10 to						353
No. 22 is 130 bit	$h_7 = 0$ in	0 lbs		"06		I S
No. 22 in 120 lbs				-	51 5 - 22 DA30 IT @ 5 ELVE	
Total Sheel Health, H ₁ = 90 in	$h_4 = 22 \text{ in}$	120 lbs		22	2"	
Treat Shelf Height, H. 90 to Unit Heig					+	
Wide Pop ANS INTERS Depth Depth D	$h_1 = 4$ in			22	2"	
Note: Per ANSI MH 10, Science 53, effective legister may be determined by rational methods consistent with ASI or AISC. AISC Design by Second-Order Analysis, Section C2.2 as used. Notices loads are applied to gravity load cases and Nr.1				7'-6"	+	
ASS C Design by Second Order Analysis, Section C22 as used. Notional loads are applied to gravity load cases and K=1.0	Unit Base Depth, D = 22 in			. 22	2"	8
Lead Case 1 [Riff seez 2.0 A(1) - PL-A5 (PR-1)		-			<u>,</u>	
Load Case 1 (RMI sect_2 2.6 R(1) = PLB SF(PL)			to gravity road caooc and re-rio	20	1	
RMM Sect 2c, Plus = 1.01 Selection (CC, Cl.) Selection (CC	Overturning Stability (Load cases are per ASCE 7 sect. 1	5.5.3.3.2):		4	" ELEVATION	-
Selamin (Co,life)	Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]					
Color Colo					W。(Braced)	
Base Shear, V = C ₁ k/W ₁ = 7.07 lbs (Braced) 5.50 kb (Down Asie) Horizontal forces / level, F ₁ = C ₂ k (PMI sect 2.6.6) (Gervice Loads, E = 0.7) F ₂ = 0.0 lbs © loin (CM) State (CM) = Product Center of F ₁ = 0.0 lbs © loin (CM) Mass spically 20 inches F ₁ = 0.0 lbs © loin (CM) Mass spically 20 inches F ₁ = 0.0 lbs © loin (CM) Mass spically 20 inches F ₁ = 1.0 lbs © loin (CM) F ₂ = 0.0 lbs © loin (CM) F ₃ = 0.0 lbs © loin (CM) F ₄ = 1.51 lbs © 79 in (CM) F ₅ = 7.1 hs © 35 in (CM) F ₇ = 7.0 lbs © loin (CM) F ₈ = 2.24 lbs © loin (CM) F ₉ = 2.	0.108 W _s (Down Aisl	e)	, 5, ,	0.108		
Horizontal forces / level, F, = C_A / CMM set 2.66					(Braced)	
	50.8 lbs (Down Aisle)			30.4 lbs	(Down Aisle)	
Note Fig. 0.0 lbs 0.0 in (CM) Fig. 0.0 lbs 0.0 lbs Fig. 0.0 lbs Fig. 0.0 lbs Fig. 0.0 lbs Fi					et 2.6.6)	
Mass typically 20 Inches	<u>Note:</u> $F_8 = 0.0 \text{ lbs}$ @ 0 in (CM)		, , ,	0.0 lbs		
Second part F F 22.4 Ibs 6 110 in (CM) F F 2 24.4 Ibs 6 110 in (CM) F F F 11.6 Ibs 6 57 in (CM) F F F 7 11.6 Ibs 6 57 in (CM) F F 7 11.6 Ibs 6 57 in (CM) F F 7 11.6 Ibs F F F F F F F F F			· ·			
F ₂ = 11.6 lbs	above the top of shelf at $F_5 = 22.4 \text{ lbs}$ @ 110 in (CM)				@ 110 in (CM)	
F _y = 1.7 t bs	4					
Calculate Overturning Moment (Service), Morg = \$2.9 it lbs	$F_2 = 7.1 \text{ lbs}$ @ 35 in (CM)			0.0 lbs		
Calculate Overturning Moment (Service), Moy = 2fh, Moy = 2711 in-lbs Calculate Resisting Moment (Service), Masy = 6822 in-lbs Factor of Safety, FOSqr = Masyr Moy = 1.160 NO UPLIFT - ANCHORS REQUIRED POST Moy = 6821 in-lbs Masy = 6822 in-lbs Masyr = 6					@ 45 in (CM)	
Calculate Resisting Moment (Service), M _{RST} 3520 in-bs Factor of Safety, FOS _{OT} = M _{RST} M _{OT} = 1.160 NO UPLIFT - ANCHORS REQUIRED UPLIFT - ANCHORS REQUIRED UPLIFT - ANCHORS REQUIRED Base Reactions: Lc #1 LC #2 Calculate Resisting Moment (Service), M _{RST} 3520 in-bs Factor of Safety, FOS _{OT} = M _{RST} M _{OT} = 0.850	Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$		Calculate Overturning Moment	(Service), M _{OT} = 3		
Ractor of Safety, FOSon = Magn/Mory = 1.160 No UPLIFT - ANCHORS REQUIRED						
Base Reactions: LC #1 LC #2 S S S S S S S S S	$M_{RST} = 6622 \text{ in-lbs}$		M _{RST} =	3520 in-lbs		
Base Reactions:						
R _B	UPRIGHT					
Overturning FOS = 1.160 <1.5 Abs Reqd 0.850 <1.5 Abs Reqd Silding Restraint, R _{RST} /FOS = 24lbs / 2.983 = 1.5 OK Reactions (Factored Loads): LC #1 LC #2 LC #2 LC #2 LC #2 LC #2 LC #2 LC #3 LC #4 LC #4 LC #2 LC #4	POST					
Sliding Restraint, R _{RST} /FOS = 124lbs / 2.983 >= 1.5 OK Reactions (Factored Loads): L.C #1 LC #2 LC #1 LC #2	"OGP"		$R_v =$	0 lbs	28 lbs	;
Reactions (Factored Loads): LC #1						
Net Uplift (R _{uv}) = 0 lbs 426 lbs	6		Reactions (Factored Loads):	LC #1	LC #2	!
Overturning + Gravity (P _u) = 1280 lbs 746 lbs Tension Allowables Tension Allowables Steel Strength, \$\phi_{Na} = 5675 lbs 5675 lbs \$\cdot - ACI 318-14 Eq 17.4.1.2 \$\cdot - ACI 318-14 Eq 17.4.2.1b \$\cdot - ACI 318-14 Eq 17.5.1.2c \$\cdot - ACI 318-14 Eq 17.5.1.2c \$\cdot - ACI 318-14 Eq 17.5.1.2c \$\cdot - ACI 318-14 Eq 17.5.2.1b \$\cdot - ACI 318-14 Eq 17.5.3.1b \$\cdo						
Steel Strength, \(\phi\)\(\phi_{\text{case}}\) Steel Strength, \(\phi\)\(\phi_{\te			Overturning + Gravity (P _u) =			
Concrete Breakout, $(0.75) \phi N_{con} = 636 \text{lbs}$			-	5675 lbs	<aci 17.4.1.2<="" 318-14="" eq="" td=""><td></td></aci>	
			Concrete Breakout, (0.75)φN _{cbq} =	636 lbs -	<aci 17.4.2.1i<="" 318-14="" eq="" td=""><td></td></aci>	
	1.000		•		· ·	s (LC #2)
			max tension stress ratio (TSR) =			, ,
	, -		-	1449 lbs ·	<aci 17.5.1.20<="" 318-14="" eq="" td=""><td>3</td></aci>	3
$\begin{array}{c ccccccccccccccccccccccccccccccccccc$	Embedment (h _{nom}) = 2 in		Concrete breakout (Yg), φV _{cbq} =	1440 lbs ·	<aci 17.5.2.1<="" 318-14="" eq="" td=""><td>0</td></aci>	0
$ \begin{array}{c ccccccccccccccccccccccccccccccccccc$						
# of Anchors, n = 1	$h_{ef} = 1.33 \text{ in}$ 1.5(h_{ef}) = 1.995 in			LC #1	LC #2	!
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		Factored	, -,			
$A_{50} = 0.094 \text{ in}^2$ Braced (TSR+VSR <= 1.2) = 1.049 <= 1.2 OK - LC #2 Controls Down Aisle (VSR <= 1.0) = 0.056 OK - LC #1 Controls	Sx = 0.00 in	Max shear s	tress ratio (VSR) : Braced =	0.130	OK 0.077	OK
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			Down Aisle (VSR <= 1.0) =	0.056		
	USE: (1) 3/8"Ø DeWalt Screw Bolt+	Anchor - 2" embed.	ICC REPORT #ESR-38	889		

PRCA20231436			
JOHNSTON BURKHOLDER	PROJECT NO: 2135202403 PROJECT NAME:	SHEET NO:	OF: 11
JDA ASSOCIATES	#2403 - Puyallup, WA		
consulting structural engineers	MADE BY:	DATE:	07/21
930 CENTRAL · KANSAS CITY, MO 64105	CHECKED BY:	DATE:	City of Puyallup
816.421.4200 · WWW.JBAENGR.COM			Development & Permitting Services ISSUED PERMIT
Storage Rack - Seismic Design	Rack OGP5	22-90 OGP	Building Planning
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH1	6.3-16)		Fire Traffic
$\begin{array}{llllllllllllllllllllllllllllllllllll$		Max Vertical Load (LRFD) - RMI, sect 2.2 P = (1.2+0.2SDS)DL + (1.2+0.2Sds)PL + pl P _u = 1280 lbs < At Eac	LC#4: L+(0.7)pEL] = 1.009 (lp=1) = 100 lbs = 300 lbs = 554 lbs ch Post - LC#5: EL
9	0 psf	20. BEAM FIXED AT ONE END, FREE TO DEFLECT ROTATE AT OTHER—UNIFORMLY DIS	VERTICALLY BUT NOT TRIBUTED LOAD
Area reqd. for bearing (A_{reqd}) = 1.4 "b" distance = 14.3	ou in	Total Equiv. Uniform Load . R = V	
$\begin{array}{lll} S = (1")(t)^2/6 = & 2.6 \\ \phi M_{nt} \mbox{ (tension allowable)} = f_t^*7.5^*[(f_c)^{1/2}]^*S = & 600.0 \\ \mbox{Factored uniform bearing, } w_u = P_u \mbox{ / } A_{teqd} = & 6.2 \\ M_u = wL^2/3 = (w_u)[(b-min(X-X,Y-Y))/2)^2] \mbox{/ } 3 = & 82.1 \\ \end{array}$	10 in 17 in ³ /in 10 in-lb/in 16 lb/in/in 6 in-lb/in - Defl. End M1 = 42 in-lb/in 17 < 1.0 O.K.	M max. (at fixed end) Shear Ma (at deflected end) Ma Ad2271 Amax. (at deflected end) Moment Ax	$w_1 = \frac{w_1 x}{3}$ $w_2 = \frac{w_1 x}{6}$ $w_3 = \frac{w_4 x}{6}$ $w_4 = \frac{w_4 x}{2}$
Rack FOS Overturning with Resistance from Effective Weight of Slab o Width of Single Rack Slab thickness (t)	= 22 in	RACK	
$\label{eq:modulus} \mbox{Modulus of Rupture, } f_t = 7.5^* \mbox{SQRT}(f'c) \\ \mbox{Concrete Slab Section Modulus, } S = b(t)^2/6 \\ \mbox{Allowable Concrete Slab Bending Moment, } M_{all}/FS = S^*f_t/1.5 \\ $	= 375.0 psi = 32.0 in ³ /ft = 666.7 ft*lbs/ft	EFFECTIVE AND THE	B) (B
Effective Cantilever Span Length (I _c) at M _{all} Total Length of Slab (I _c + Width of Single Rack) Trib. Width of Slab = Trib width of Rack Weight of Concrete Slab at Rack (P _{conc}) Resisting Moment - Concrete Slab at Rack, M _{RST(slab)} = P _{conc} * I _o /2 Load Combination #1: M _{RST(Rack)} + M _{RST(slab)}	= 7.0 ft = 8.0 ft = 2813.5 lbs = 118122 in*lbs = 5711 in*lbs	INTILEVERED SLAB SPAN ENGTH (Lc)	AB THICKNESS
Total Overturning FOS Load Combination #2: M _{OT}	= 21.843 OK		- 1S (E)
Load Combination #2: M _{OT} M _{RST(Rack)} + M _{RST(slab)} Total Overturning FOS	= 121642 in*lbs		

PRCA20231436							
JOHNSTON	PROJECT NO:			SHEET NO:	•	OF:	
	2135202			1	0		11
BURKHOLDER	PROJECT NAME						
ASSOCIATES		Puyallup, WA					
consulting structural engineers	MADE BY:			DATE:	00/0	7/04	
930 CENTRAL · KANSAS CITY, MO 64105	GMB				09/0	7/21	
816.421.4200 · WWW.JBAENGR.COM	CHECKED BY:			DATE:			City of Puyallup Development & Permitting Services
							ISSUED PERMIT Building Planning
Storage Rack - Seismic Design	Rack OGP2 (46-9	90 OGP)	46-90 OGF)			Engineering Public Works
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/	MH16.3-16)						Fire Traffic
`		Access Allowed (Typ. at Ba	ck		43"	0' 01"	
	1.0	Grocery Storage Areas)	•		13"	8'-0 <u>1</u> "	1 ³ / ₄ "
Max. Weight per level (2 Pallets / shelf) = 600 lbs	s/shelf				T		
S		weight per Manuf.			1		
,	i.5 in) in	Total Shelf Load 0 lbs			46"		
) in	0 lbs					_ \
The state of the s) in	0 lbs			<u> </u>	OGP.	2
The state of the s) in) in	0 lbs 0 lbs		"00P	PI 2"= 46"Dx90	LAN VIEW	EVELS
$h_4 = 0$) in	0 lbs		UGFZ	Z = 46 DX90	∏ ₩ Z LE	.VELS
G .) in 6 in	0 lbs		1	1		
	4 in	600 lbs 600 lbs			46"		
	0 in				1		
	0 in			7'-6	·" -		
Unit Base Depth, D = 4 Note: Per ANSI MH16.1, Section 6.3, effective length	6 in ths mav be determined	by rational methods consiste	ent with AISI or AISC.	1	1		
AISC Design by Second-Order Analysis, Section C2	.2a is used. Notional lo	ads are applied to gravity loa			44"		8
is used since the ratio of second-order drift to first-or	der drift $(P-\delta) / (P-\Delta) < 0$	1.1.		•	1 1		
Overturning Stability (Load cases are per ASCE 7	sect. 15.5.3.3.2):			W-1	El	LEVATION	-
Load Case 1 [RMI sect. 2.6.8(1) - PL=0.67(PL)]		Load Cas	se 2 [RMI sect. 2.6.8(2)	. ,-			
[RMI sect 2.6.2, $PL_{RF} = 1.0$] Seismic $(C_s)(I_p) = 0.252 W_s$ (Br	aced)		Seismic $(C_s)(I_p) =$	2, PL _{RF} =1.0] 0.252	W _s (Braced)		
0.108 W _s (Do	,		(-5/(-р/		W _s (Down Aisle)	
$W_s = (0.67)(PL_{RF})((0.67)PL) + DL = 638.7 \text{ lbs}$	-n		67)(PL _{RF})((1)PL)+DL =	502.0 lbs	(DI)		
Base Shear, $V = C_s I_p W_s = 161.1$ lbs (Brace 69.1 lbs (Down	,	Bas	e Shear, $V = C_s I_p W_s =$	126.6 lbs 54.3 lbs	(Braced) (Down Aisle)		
Horizontal forces / level, $F_x = C_{vx}V$ (RMI sect 2.6.6)	,	Hori	izontal forces / level, F,		,		
(Service Loads, E = 0.7) $F_9 = 0.0 \text{ lbs}$ @ 0 in	. ,		(Service Loads) F ₉ =	0.0 lbs			
Note: $F_8 = 0.0 \text{ lbs}$ @ 0 in (CM) = Product Center of $F_7 = 0.0 \text{ lbs}$ @ 0 in	. ,		F ₈ =	0.0 lbs 0.0 lbs			
Mass typically 20 inches $F_6 = 0.0 \text{ lbs}$ @ 0 in	. ,		F ₆ =	0.0 lbs			
above the top of shelf at $F_5 = 0.0 \text{ lbs}$ @ 0 in	` '		F ₅ =	0.0 lbs			
each level. $F_4 = 0.0 \text{lbs} @ 0 \text{in}$ $F_3 = 0.0 \text{lbs} @ 0 \text{in}$			F ₄ = F ₃ =	0.0 lbs 0.0 lbs			
	in (CM)		F ₂ =	83.0 lbs	@ 110 in (CM)		
	n (CM)		F ₁ =	0.0 lbs	@ 45 i= (OM)		
$F_u = 6.7$ lbs @ 45 i Calculate Overturning Moment (Service), $M_{OT} = \Sigma f_i h_i$	n (CM)	Calculate	F _u = e Overturning Moment (5.7 lbs (Service), Mor =	@ 45 in (CM) : Σf.h.		
$M_{OT} = 10241 \text{ in-lbs}$			M _{OT} =	9381 in-lbs			
Calculate Resisting Moment (Service), M_{RST} $M_{RST} = 20792$ in-lbs		Calculate	Resisting Moment (Se	ervice), M _{RST} 16100 in-lbs			
Factor of Safety, $FOS_{OT} = M_{RST}/M_{OT} = 2.030$			Factor of Safety, FOS		1.716		
NO UPLIFT - NO ANCHORS REQU	IRED		NO UPLIFT -	NO ANCHORS	REQUIRED		
UPRIGHT		Base R	Reactions:	LC #1		LC #2	
POST 6			R _h =	56 lbs		44 lbs	
OGP T			R _v = Overturning FOS =	0 lbs 2.030	>= 1.5	0 lbs 1.716	>= 1.5
		Sliding I	Restraint, R _{RST} /FOS =			114lbs / 2.567	
6. 6.		Reaction	ons (Factored Loads):	LC #1		LC #2	
			Base Shear (R _{uh}) = Net Uplift (R _{III}) =	161 lbs 0 lbs		127 lbs 0 lbs	
Anc		Overtu	rning + Gravity (P _u) =	1528 lbs		1056 lbs	
			Allowables	F075 !!	AC1045 : : :	- 47 4 4 -	
1.5h _{ef}			Steel Strength, $\phi N_{sa} =$ Breakout, $(0.75)\phi N_{cbg} =$	5675 lbs 636 lbs	<aci 318-14="" e<="" td=""><td></td><td></td></aci>		
1.5ca 1 1.5ca			Strength, $(0.75)\phi N_{pn} =$	439 lbs	<aci 318-14="" e<="" td=""><td></td><td></td></aci>		
			Tension Load, (N _u) =	0 lbs	(LC #1)	0 lbs	(LC #2)
Anchor Design (using "Cracked Concrete" Properties) Upright Post Type = OGP		max tensio Shear All	on stress ratio (TSR) =	0.000	OK (LC#1)	0.000	OK (LC#2)
Try: 3/8"Ø DeWalt Screw Bolt+ Anchor - 2" embed.			Steel Strength, ϕV_{sa} =	1449 lbs	<aci 318-14="" e<="" td=""><td>•</td><td></td></aci>	•	
Embedment (h _{nom}) = 2 in			breakout (Yg), $\phi V_{cbq} =$	1440 lbs	<aci 318-14="" e<="" td=""><td></td><td></td></aci>		
f _c ' = 2500 psi e _n ' = 1.875 in < Eccen. of Anchor			breakout (Xg), $\phi V_{cbq} =$ oncrete pryout, $\phi V_{cpq} =$	1152 lbs 913 lbs	<aci 318-14="" e<="" td=""><td></td><td></td></aci>		
$h_{ef} = 1.33 \text{ in}$ 1.5(h_{ef}) = 1.995 i			_	LC #1	3. 2. 0 . 7 .	LC #2	
Conc. thickness, $t = \frac{4 \text{ in}}{1.5(C_a)} = 7.500 \text{ i}$	n	Factored Shear Load	,	161 lbs		127 lbs	_
# of Anchors, n = 1 Sx = 0.00 in		Max shear stress ratio (V	Down Aisle = /SR): Braced =	69 lbs 0.176	OK	54 lbs 0.139	OK
Sy = 0.00 in		,	Down Aisle =	0.076	OK	0.060	OK
A _{se} = 0.094 in^2			I (TSR+VSR <= 1.2) =	0.176	<= 1.2 OK - LC		
IJCE: NO LIDI IET 14) 2/0"G D-	Walt Care D-		n Aisle (VSR <= 1.0) =	0.076	OK - LC #1 Con	แบเร	
USE: NO UPLIFT - (1) 3/8"Ø De	Walt Sciew BO	ALT ALICHOI - 2 em	ibeu.				

PRCA20231436			
JOHNSTON	PROJECT NO:	SHEET NO: OF:	4.4
BURKHOLDER	2135202403 PROJECT NAME:	11	11
JDA ASSOCIATES	#2403 - Puyallup, WA		
consulting structural engineers	MADE BY:	DATE:	
930 CENTRAL · KANSAS CITY, MO 64105	GMB	09/07/21	
816.421.4200 · WWW.JBAENGR.COM	CHECKED BY:	DATE:	City of Puyallup Development & Permitting Services ISSUED PERMIT
Storage Rack - Seismic Design Rack C	I DGP2 (46-90 OGP) 46-90 OG	 ;P	Building Planning
IBC 2018 / ASCE 7-16 / 2016 RMI (ANSI/MH10		<u>'</u>	Engineering Public Works Fire Traffic
IBC 2016 / ASCE 7-10 / 2016 KIVII (ANSI/IVII 110	0.3-10)		File A M STATILL
$\begin{array}{llllllllllllllllllllllllllllllllllll$	P = (1+0.1 P = (1+0.1) P = (1+0.1)	cal Load (LRFD) - RMI, sect 2.2 - LC#5: .2SDS)DL + (1.2+0.2Sds)PL + pEL	1.009 (Ip=1) 50 lbs 600 lbs 475 lbs
Slab tension based on Soil bearing area check:	20. BEAM FIX	ED AT ONE END, FREE TO DEFLECT VERTICA	
$\begin{array}{ll} \phi M_{nl} \; (tension \; allowable) = f_{*}^{*} 7.5^{*} [(f'_{c})^{1/2}]^{*} S = & 600.00 \\ Factored \; uniform \; bearing, \; w_{u} = P_{u} / A_{reqd} = & 5.49 \\ M_{u} = w L^{2} / 3 = (w_{u}) [(b-min(X-X,Y-Y))/2)^{2}] / 3 = & 102.02 \\ \end{array}$	lbs ft² in	TOTALE AT OTHER—UNIFORMLY DISTRIBUTED Total Equiv. Uniform Load	8 wl wx
Rack FOS Overturning with Resistance from Effective Weight of Slab on		MPAGE!	H I
$\label{eq:width of Single Rack = Slab thickness (t) = Slab thickness (t) = Modulus of Rupture, f_r = 7.5 ^{\circ} \mathrm{SQRT}(f^c) = 100 \mathrm{Modulus} of Rupture, f_r = 7.5 ^{\circ} \mathrm{SQRT}(f^c) = 100 \mathrm{Modulus} of Rupture, f_r = 7.5 ^{\circ} \mathrm{SQRT}(f^c) = 100 \mathrm{Modulus} of Slab Section Modulus, S = b(t)^{2}/6 = 100 \mathrm{Modulus} of Slab Bending Moment, M_{all} = 100 \mathrm{Modulus} of Slab Effective Cantilever Span Length (l_c) at M_{all} = 100 \mathrm{Modulus} of Slab Length of Slab (l_c + W) width of Single Rack) = 100 \mathrm{Modulus} of Slab (l_c + W) width of Single Rack) = 100 \mathrm{Modulus} of Slab (l_c + W) width of Single Rack) = 100 \mathrm{Modulus} of Slab at Rack (l_c + W) width of Slab = 100 \mathrm{Modulus} of Slab at Rack, 100 \mathrm{Modulus} of Slab at Rack, 100 \mathrm{Modulus} of Slab at Rack, 100 \mathrm{Modulus} of Slab at Rack 100 \mathrm{Modulus} of Slab at Rack, 100 \mathrm{Modulus} of Slab at Rack 100 \mathrm{Modulus} of Slab at $	4.0 in 375.0 psi 32.0 in³/ft 666.7 ft*lbs/ft 5.2 ft 9.0 ft 8.0 ft 3617.7 lbs 195296 in*lbs 10241 in*lbs 216088 in*lbs	VE ERED AN (Lc)	AB THICKNESS
$\frac{\text{Total Overturning FOS}}{\text{Load Combination #2:}}$			
$M_{RST(Rack)} + M_{RST(slab)} =$ Total Overturning FOS =			
			,