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PRCTI20241266

GARY K. MUNKELT & ASSOCIATES, LLC

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Souderton, PA 18964
Phone: 215-855-8713

PROJECT: COSTCO WHOLESALE STORE #660 (REMODEL – ADD-NEW RACK IN MULTIPLE AREAS)
(STORAGE RACK BY RIDG-U-RAK)
LOCATION: 1201 39TH STREET
PUYALLUP, WA 98373

PREPARED FOR: COSTCO WHOLESALE – ISSAQUAH, WA

TABLE OF CONTENTS

Notice to Building Departments

If this calculation is submitted for building permit approval, it shall contain all sheets of calculations as listed in the table of contents and shall be accompanied by all drawings listed in Reference section on sheet 2, section 1.4 "Drawings". All documents shall bear appropriate seals and signatures in ink of contrasting color.

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No Changes to Calculations
Revision Dated: 09/25/2024
Date: 04/29/2024
GKM Job Number 2404131-R2
Sheet #0 of #23

City of Puyallup
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ADAM MARK McLAUGHLIN
STATE OF WASHINGTON
48096
REGISTERED
PROFESSIONAL ENGINEER

Adam McLaughlin

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DN: C=US, S=Pennsylvania, L=North Wales, O="Gary K. Munkelt and Associates, LLC", CN=Adam McLaughlin, E=adam.mclaughlin@gkmassoc.com
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DATE SIGNED: 9/26/2024

Calculations required to be provided by the Permittee on site for all Inspections

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1 Reference Data:

1.1 Scope of Work and Codes:

This calculation reviews the installation of the storage racks for structural adequacy. The sealing of drawings is for the structural review of the storage racks only. Other information is not reviewed, nor approved. Note - The single level racking is exempt from the building permit (although it easily satisfies seismic criteria).

Code Requirements - IBC, ASCE/SEI 7, RMI/ANSI MH16.1, AISI/ANSI S100, and FEMA 460 are used for the design. (Meets or Exceeds 2018 IBC)

1.2 Project History:

This is a modification to a store that would have originally been reviewed by Peter S. Higgins & Associates.

Existing storage racks would have been installed under permits as required by the local authority, and remain qualified by the review and permit. Additional new racking as shown in clouds is reviewed under this report.

1.3 Design Methods:

1.3.1 Static Loads:

Per ANSI MH16.1, this review employs LRFD direct design methods which are the only unrestricted design method in AISC 360 & AISI S100. This report uses notional loads to determine second order displacements, and is found in AISC 360, Section C "Design For Stability" or AISI S100 Appendix 2.

1.3.2 Seismic Loads:

For seismic loads, FEMA 460 6.5.1 is employed (as permitted by ANSI MH16.1.2.6.2).

1.3.3 References:

In addition to the above documents, the reviewer is referred to the following references for assistance with these methods, and the connector values used in the design as required by both AISC 360 and FEMA 460.

General Design Methods:

Displacement Based Design for Storage Racks
Higgins, P,
ASCE/SEI Conference, Long Beach, CA (Invited paper)
ASCE/SEI Proceedings, 2007

Shake Table Results for Typical Racks:

Recent Shake Table Studies of Full Scale Storage Racks:
Filiatrault, A. , Wanitkorkul, A., Higgins, P.
ASCE/SEI Conference, Long Beach, CA (Invited paper, presented by Higgins)
ASCE/SEI Proceedings, 2007

Connector Values for Bolted Connector Racks:

Experimental Stiffness and Seismic Response of Pallet-Type Steel Storage Rack Connectors
Filiatrault, A. , Wanitkorkul, A., Higgins, P.,
ASCE J. Pract. Period Struct.Des. Constr. (11(3), 161-170 (2006)

Connector Values for Rivet Connector Racks:

Experimental Stiffness and Seismic Response of Pallet-Type Steel Storage Rack Tear Drop Connectors
Filiatrault, A. , Wanitkorkul, A., Higgins, P., Courtwright, J., ASCE J. Pract. Period Struct. Des. Constr. (12(4), 210-215 (2007)

General Seismic Force Transverse Rack Behavior:

Experimental Seismic Response of Base Isolated Pallet-Type Storage Racks
Filiatrault, A. , Wanitkorkul, A., Higgins, P., Courtwright, J., Michael, A.
Earthquake Spectra 24:3 pp 617-639 August 2008.

Pallet Sliding Effects:

An Investigation - The Sliding of Pallets on Storage Racks Subject to Earthquake
Degee, H. , DeNoel, V.
FEM Research Project: RFS-PR-03114, Universite de Liege, 2006

These publications are widely available, and may be found in any large library, or easily borrowed from any leading University library.

1.4 Drawings:

1.4.1 By Gary K. Munkelt & Associates, LLC:

CW-10, RU-9

1.4.2 By Others:

Costco Wholesale #660 - Puyallup, WA
Proposed Floor Plan RP 1 (#98-5080) - dated Nov. 18, 2022

1.5 Loads:

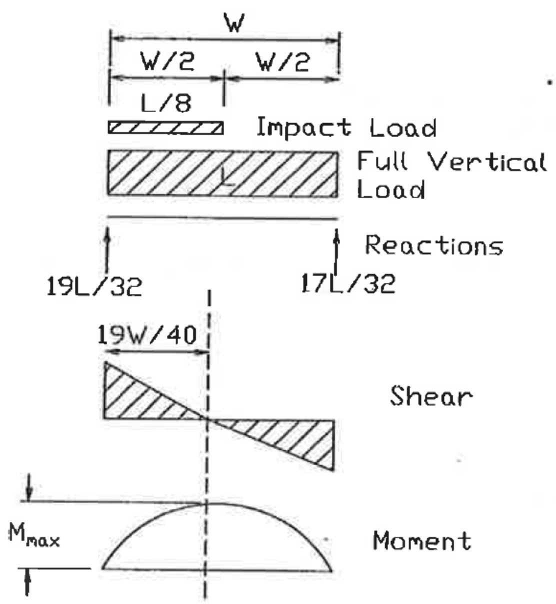
1.5.1 Vertical (Dead plus Live):

Load per pallet = 2.5 kips (2 Pallets wide at 2.5 kips / pallet = 5.0 kips / level)
Design beams for 25% impact

1.5.2 Impact Loads of Machine Loaded (Pallet / Selective) Racks:

Conservatively assume beams are simply supported UNO. Note: rack nomenclature is somewhat different than standard structural notation. Here L = **L**oad on the beam (usually expressed in terms of a unit or pallet load). W = **W**idth of the bay (or span of beam).

Take 25% impact of a single pallet as a UDL on 1/2 of span. There are two beams supporting each level (one front, one rear). Accordingly, each two pallet wide beam supports one pallet load on the full span, plus 1/8 of a pallet load on half of the span.



$$M_{max} = \frac{361\,WL}{2560}$$

1.5.3 Seismic:

ASCE 7 - section 15.5.3 per ANSI MH16.1, employing displacement based design per section 2.6.2. The target displacement of the mass centroid per FEMA 460, section 6.5.1:

$$D = \frac{g\,S_{m1}\,T}{4\,\pi^2\,B}$$

1.5.3 Seismic Continued:

The demands are to MCE levels which include the Ie = 1.5 factor for public warehouse stores.
Given:

$S_{ms} = F_a S_s =$ 2.90 >> $S_{ms\ act}$ = 1.524 OKAY

$S_{m1} = F_v S_1 =$ 1.50 >> $S_{m1\ act}$ = 0.814 OKAY

For default soils:

F_a = 1.20 $S_s \leq 2.42$

F_v = 1.50 $S_1 \leq 1.00$

Note - The seismic coefficients for S_s & S_1 are based on high seismic areas. (Actual seismic Spectral Response Accelerations for this store located in Puyallup, WA are S_s = 1.27; S_1 = 0.437
Sms = 1.524; Sm1 = 0.814 per ASCE 7 Hazards
Seismic Design Category "D"

B may be conservatively taken from the following table:

$0.6S_{ds}$	Damping	B
< 0.1g	5%	1.00
0.2g	10%	1.20
0.3g	15%	1.35
0.4g	20%	1.50
0.5g	25%	1.60
>0.6g	30%	1.70

$0.6S_{ds} =$ 1.1616 g > 0.6g B = 1.70

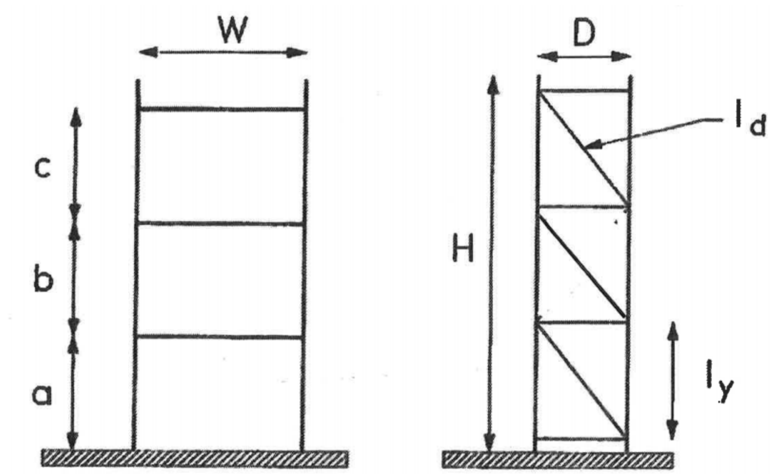
Yielding:

D = 8.6 T

T may be determined by any rational method, but the formulations of FEMA 460 are used for this design.

2 3 - Level Racks (Main Merchandise Area):

2.1 Components and Geometry:



W' =	105 "	H =	180 "
a =	60 "	D =	34 "
b =	60 "	l_y =	48 "
c =	60 "	l_d =	54 " (Bottom two diagonals, 61" top diagonal)

Beam Type - **RB-S-500** Upright Type - **UF-H-33I**

2.2 Check Beams:

2.2.1 Design Forces:

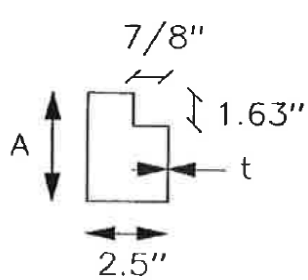
$$M \le \frac{361\,W\,L}{2560} =$$

$$S_{min} < \frac{M}{\phi F_{ye}} =$$

51.8 inch kips factored

0.96 in^3

2.2.2 Beam Properties:



RB-S-500

A = 5.00 in

t = 0.075 in

S_x = 1.40 in^3 **OK**

I_x = 3.51 in^4 **OK**

2.3 Check Posts (Dead plus Live Loads):

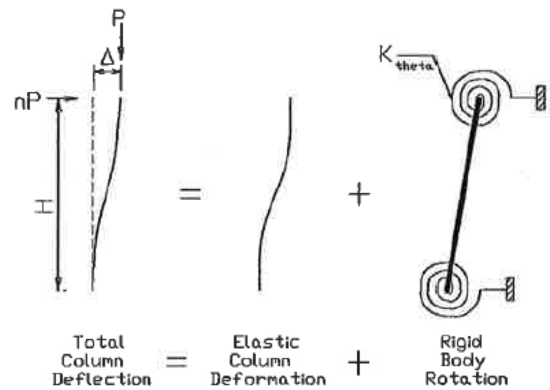
2.3.1 Vertical Load:

P < (# of Levels)(L) / 2 =

7.5 kips

10.5 kips factored

2.3.2 Second Order Effects:



Primary Notional Moment (base fixity assumption verified below)

$$M_{primary} = \eta P \frac{H}{2}$$

Deflection

$$\Delta = \eta P \left(\frac{H^3}{12EI_c} + \frac{H^2}{2K_\theta} \right)$$

Second Order Notional Moment

$$M_{P-\Delta} = \frac{P\Delta}{2} = \frac{\eta P^2}{2} \left(\frac{H^3}{12EI_c} + \frac{H^2}{2K_\theta} \right)$$

Define:

$$s = \frac{M_{P\Delta}}{M_{primary}} = PH \left(\frac{H}{12EI_c} + \frac{1}{2K_\theta} \right)$$

To acceptable accuracy:

$$M_{notional} = M_{primary} \left(1 + \sum_{i=1}^{20} s^i \right) = \eta \frac{PH}{2} \left(1 + \sum_{i=1}^{20} s^i \right)$$

K_{theta} comprised of the beam end stiffness in series with the connector stiffness yielding:

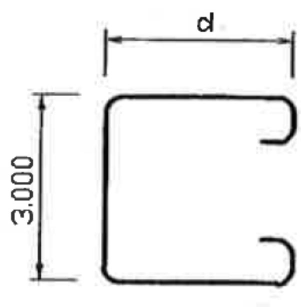
$$K_{theta} = \left(\left(\frac{1}{K_{beam}} \right) + \left(\frac{1}{K_{conn}} \right) \right)^{-1}$$

With:

$$K_{beam} = \frac{6EI}{L_{beam}}$$

K_{conn} from tests or published literature

2.3.3 Post Properties (net section):



UF-H-33

d =	2.75	in
t =	0.125	in
As =	0.99	in2
Sx =	0.98	in3
rx =	1.19	in
Sy =	0.64	in3
ry =	0.95	in
Fy =	55.0	ksi

2.3.4 Post Demand, Capacity and Combined Stress Check:

Capacity

L_x =	60	in	
L_y =	48	in	
Kl/r_x =	50		
Kl/r_y =	51		
F_n =	44.8	ksi	
ϕP_n =	37.7	kips	
ϕM_{nx} =	48.5	inch kips	
ϕM_{ny} =	31.7	inch kips	

Stability

K_{conn} =	3000	in-kip / rad	
I_{beam} =	3.51	in4	RB-S-500
L_{beam} =	102	in	
K_{beam} =	5988	in-kip / rad	
K_{theta} =	1999	in-kip / rad	
s =	0.24		

Demand

n =	0.005	
P =	10.5	kips
H =	60	inch
M_{not} =	2.1	in-kip

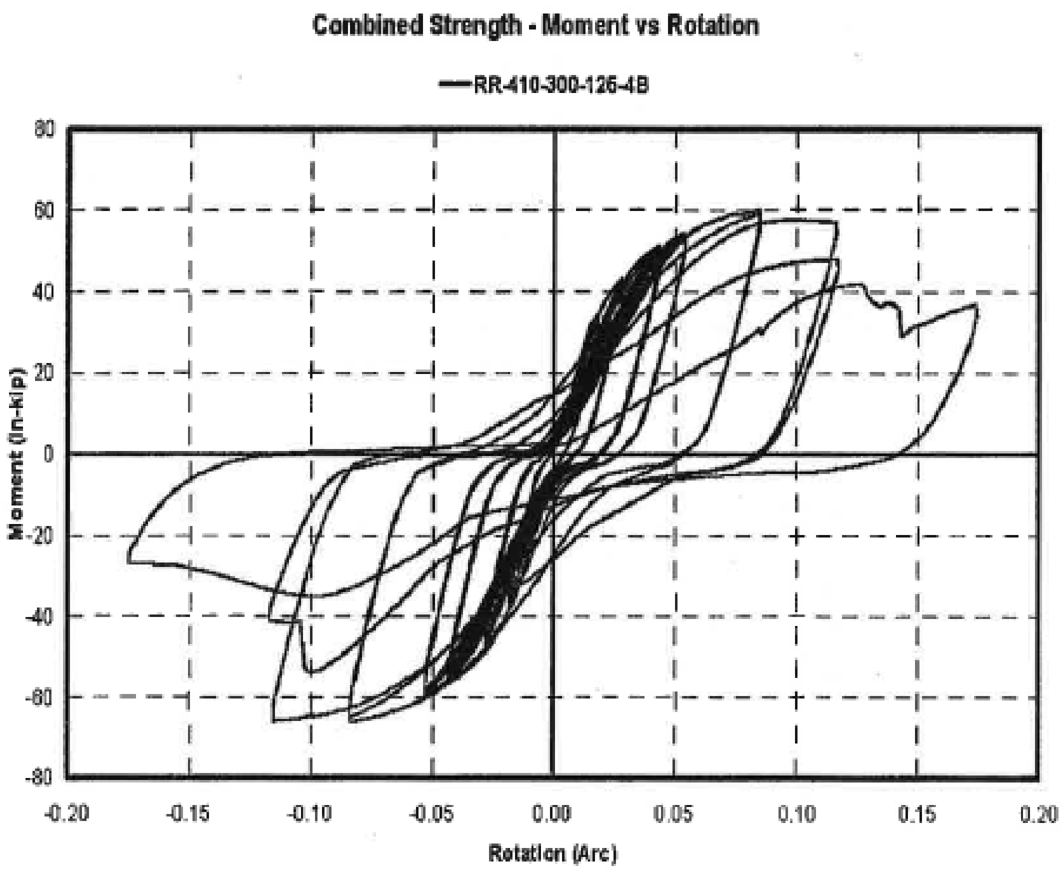
2.3.4 Post Demand, Capacity and Combined Stress Check Continued:

Combined Stress = 0.32

2.4 Longitudinal Seismic:

2.4.1 Connection Properties:

The following are test results from Ridg-U-Rak for the beam end connector stiffness based on RMI MH16.1 specifications for testing of beam end loading:



Secant stiffness at 0.095 rad > 500 in-kips per radian
Maximum rotational capacity > 0.12 radians

2.4.2 Determine Period of Structure:

FEMA 460:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{N} W_{pi} h_{pi}^2}{g\left(N_c\left(\frac{k_c k_{be}}{k_c + k_{be}}\right) + N_b\left(\frac{k_b k_{ce}}{k_b + k_{ce}}\right)\right)}}$$

Nomenclature (used throughout)

Cd = displacement ampification factor from ASCE 7
DBE = Design Basis Earthquake
DBE/MCE = ratio of earthquake accellerations
Delta, tot Delta = horizontal displacement of a mass centroid, top mass centroid
Fa = site coefficient for short period per ASCE 7./NEHRP
Fv = site coefficient for 1 sec period per ASCE 7./NEHRP
g = accelleration due to gravity
Hcol = clear height of upright between levels - conservatively use spacing between beams
hpi = height of Wpi centroid above the base
I = structure importance factor
Ib = moment of inertia of beam
Ic = moment of inertia of column
K1 = constant as defined below
K2 = constant as defined below
kb = secant rotational stiffness of the base plate at the moment under consideration
kbe = end rotational stiffness of the beam (6EIb/L)
kbu = constant as defined below
kc = secant rotational stiffness of the connector at the moment under consideration
kce = end rotational stiffnes of column (6EIc/Hcol) - FEMA 460 USES 4EIc/Hcol
kct = tangent rotational stiffness of the connector at the moment under consideration
ku = constant as defined below
L = bay width
Mc = moment in beam end connector
MCE = Maximum considered earthquake
Nb = number of base plates in the run of rack

2.4.2 Determine Period of Structure Continued:

Nc = number of beam end connectors in the run of rack
R = response reduction factor from RMI
S1 = Site seismic coefficient from ASCE7/RMI
Sd1 = Design spectral response accelleration parameter =Sm1(DBE/MCE)
Sds = Design spectral response accelleration parameter =Sms(DBE/MCE)
Sm1 = Site seismic coefficient (=FvS1)
Sms = Site seismic coefficient (=FaSs)
Ss = Site seismic coefficient from ASCE7/RMI
T = structure period as computed at right
Vb = base shear for the entire storage rack as computed below
W = Total rack weight
Wpi = Weight at a given position in a run of rack
Defined Expressions

$$K1 = N_c + N_b\left(\frac{k_b k_{ce}}{k_c k_{be}}\right)\left(\frac{k_c + k_{be}}{k_b + k_{ce}}\right) \qquad K2 = N_c k_{bu} + N_b k_{bu}$$

$$k_{bu} = \frac{k_c k_{be}}{k_e + k_{be}} \qquad k_u = \frac{k_b k_{ce}}{k_b + k_{ce}}$$

Substitution of these expressions into the period equation above yields:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{N} W_{pi} h_{pi}^2}{g(N_c k_{bu} + N_b k_u)}}$$

This formulation is a general solution for racks with a variable importance factor. The Ie = 1.5 is included in the displacement demand computed above.[While present in the generalize formulation nomenclature, The ASCE 7 C_d amplification factors are not used for the analysis].

The solution is an iterative procedure where the connector stiffness is converged with the rotational angle. For this structure, the upper bound period displacement demand does not exceed the capacity of the connector, therefore the following will be used to calculate the period in the longitudinal (down aisle) direction. The connector properties are shown in the above testing chart for a combined strength (Moment vs. Rotation) per pair of connectors .

2.4.2 Determine Period of Structure Continued:

The typical row in a Costco is between 8 to 10 bays long. For this configuration:

No. Bays	10		g	386	$in^2\ sec^2$
No. Level	3		E	29000	ksi
L	108	inches C/C	Nc	60	
W	5	kips / level	Nb	11	
Ib	3.5	in^4 (RB-S-500)	Kbe	5639	
Ic	1.4	in^4	Kce	4060	
Kc	500	in-kips/rad from above	Kbu	459	
Kb	3000	in-kips/rad	Ku	1725	
			K1	101.3	
			K2	46534	

For a rack with a constant load per level and spacing between beams:

Level	Wpi	hpi	Wpi*hpi	Wpi*hpi^2
1	25	84	2100	176400
2	25	144	3600	518400
3	25	204.0	5100	1040400
Sum or Max	75	204.0	10800	1735200

Yields T = 1.95 sec

2.4.3 Displacement / Rotational Demand:

First order displacement demand:

D = 8.6T = 16.8 inches

Second order effects:

$$D_{tot} = D(1+\alpha)$$

Where:

$$\alpha = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left(\frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left(N_c + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}} \right) \left(\frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} = 0.2321$$

Yielding:

D_{tot} = 20.8 inches

2.4.3 Displacement / Rotational Demand Continued:

Rotational Demand:

$\theta_{max} = D_{tot} / h_{pi}$ max. = 0.10 radian

2.4.4 Connector Capacity:

The stiffness corresponds to the rotational demand, and is within the capacity of the connector.

NOTE: Racks are acceptable under longitudinal, down aisle direction under seismic consideration:

2.5 Transverse Seismic:

Machine Loaded Racks - 34" minimum upright depth governs by inspection.

2.5.1 Determine Period:

The upright / posts are braced using horizontal & diagonal channels in the transverse direction, and may be reasonably considered as a cantilever off the floor slab.

2.5.1 Determine Period in Transverse Direction Continued:

Determine Period - Ref. Roark's Formulas for Stress & Strain, 7th ed., Table 16.7, Case 3b:

$$T = \frac{2\pi}{3.52}\sqrt{\frac{W l^3}{E I g}}$$

For:

D =	34	in (depth of upright)
I < $0.8[2(D/2)^2 As]$ =	458	in^4
W =	15.0	kips (bay load)
l =	180	in (top beam level)
T =	0.23	sec

2.5.2 Displacement Demand:

Second order deflections are clearly negligible, yielding

D_{tot} = 2.0 inches

2.5.3 Displacement Capacity:

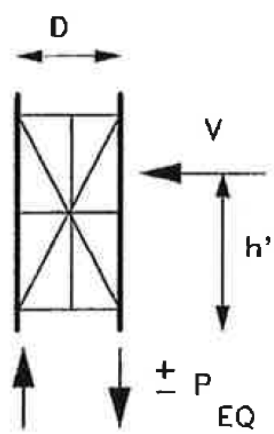
While the uprights and beams will deflect under lateral load, this displacement can be easily accommodated by pallet sliding. Movement of the pallets may included in the displacement capacity per ANSI MH16.1 section 2.6.2 Commentary.

Per the above reference, Degee paper, cited above, the maximum (and very rare) pallet coefficient of friction on the steel is less than 0.6. For $\mu / \alpha < 0.4$, the pallets do not stick. Conservatively taking $\alpha = 0.6Ss = 1.7$, this ratio will not exceed $0.6 / 1.7 = 0.36 < 0.40$, so the pallets will not stick, and the dynamic coefficient of friction may be used.

2.5.4 Stability:

ANSI MH16.1 and FEMA 460 are silent on the shear demand to the frame when pallet movement only is used to satisfy the displacement demand. Since sliding forces cannot exceed the coefficient of friction on the dynamically active fraction of the load. The COF for wood on steel (Plastic pallets have a lower value) is typically 0.2 - 0.3. Conservatively use 0.3 to develop base shear.

V = 0.3(15.0)(2/3) = 3.0 kips (Load per Bay) This will further conservatively be applied in the triangular force distribution on the upright.



$$P_{eq} = \frac{Vh'}{D}$$

h' =	160	inches
OTM = Vh' =	480	inch kips
RM = WD/2 =	255	inch kips (W = 15.0 kips / bay)
Net OTM =	225	inch kips
P_{eq} = OTM / D =	14.1	kips

Note: Bracing shown is schematic - see section 2.1 for actual geometry.

2.5.5 Post Capacity:

$P + P_{eq}$ = 21.6 kips < ɸPn = 37.7 kips **OK**

2.5.6 Stability:

$T_{anchors}$ = Net OTM / D = 6.6 kips

2.5.6 Stability Continued:

NOTE:
Based on above analysis, the tension forces in the anchoring for the project site located in Puyallup, WA would be much less than the above analysis. Anchoring acceptable as shown:

2.5.7 Anchorage:

Steel strength of anchor in tension will not govern:

Concrete Anchor Breakout Strength per ACI 318, Chapter 17:

Concrete Pull-out does not govern on external threaded screw anchors:

Units: pounds, inches UNO

Analysis based on no edge distance issues:

Symbol	Units	Description
Anc	in^2	Breakout area of anchor group
Anco	in^2	Breakout area of one anchor
h_{ef} =	inches	Minimum effective embedment
f'c =	psi	Concrete strength used per RMI minimum or project site concrete strength
δ_a =	none	Modification for light weight concrete
$\phi_{sa}nV_{saeq}$	Lbs.	Total Shear resistance of the 1/2 steel anchors
ϕcbVcbg =	Lbs.	Shear Capacity for the (2) 1/2 steel anchors (no edge distance issues)
Basic f'c	psi	Basic Concrete Strength after 28 days for pullout
Category	none	Per ACI 318 Section 17.3.3
Cone Cap	Lbs.	Breakout capacity of concrete for bolt group
	none	ACI 318 - Section 17.4.2.2
K_{cr} & K_{uncr}	none	
N_b	Lbs.	ACI 318 - Section 17.4.2.2
$N_{p,cr}$	Lbs.	Report pullout capacity in cracked concrete
$N_{p,uncr}$	Lbs.	Report pullout capacity in uncracked concrete
phi	none	Combined strength reduction under seismic per ACI 318 - 17.2.3.4.4 & (0.75*0.65) = 0.49
phiN_{cbg}	Lbs.	Capacity of bolt group (bolts or concrete) ACI 318 - Section 17.4.2.1
S1	inches	Spacing between two anchors in one row
S2	inches	Spacing between rows of two anchors

2.5.7 Anchorage Continued:

$\psi_{c,}N$	none	Per ACI 318 - Section 17.4.2.6 (post installed, K_{cr} = 17)
$\psi_{ec,}N$	none	Per ACI 318 - Section 17.4.2.4 (concentric load)
$\psi_{ed,}N$	none	Per ACI 318 - Section 17.4.2.5 (distance form edges)
δ_a =	1.0	Modification for light weight concrete

Variables		Basis f'c =	2500
Concrete		**Anchors**	1/2" diameter - Hilti KH-EZ Screw Anchors referenced
f'c =	3000	$N_{p,uncr}$ =	N/A With screw type anchors
Computed		$N_{p,cr}$ =	N/A With screw type anchors
Anco =	144	Category	1
Anc =	216	K_{uncr} =	24
Nb =	7449	K_{cr} =	17
$\psi_{c,}N$ =	1.4	Phi =	0.49
$\psi_{ec,}N$ =	1.0	h_{ef} =	4.0 (Conservatively use 4")
$\psi_{ed,}N$	1.0	S1 =	6.0
$phiN_{cbg}$	7626	S2 =	0.0

Based on the above analysis, maximum base shear can be resisted by the available shear capacity for the anchor group (2 anchors per group / plate).

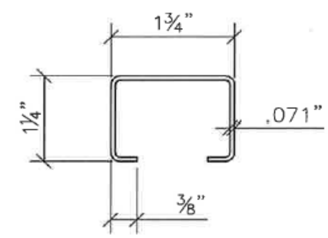
** For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum effective embedment of 4". (5" Nominal Embedment to produce 4" effective embedment)

2.5.8 Brace:

2.5.8.1 Design Forces:

$P_{br} \leq$ 5.7 kips (Bottom two diagonals)

2.5.8.2 Brace Capacity:



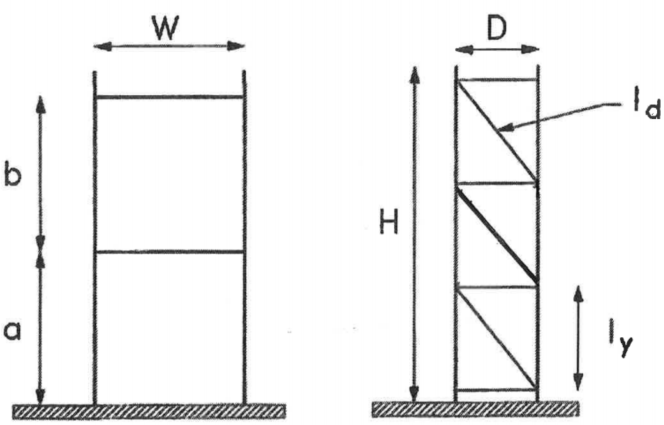
A_{net} =	0.32	in2
r_{min} =	0.46	in
ϕ=	0.85	(AISI - Compression elements)
k =	0.80	
t =	14ga	

2.5.8.2 Brace Capacity Continued:

L	$\frac{kl}{r}$	F_n	ϕP_n	
(in)		(ksi)	(kips)	
54	95	26.6	7.2	**OK**

3 2 - Level Racks (Merchandise Display):

3.1 Components and Geometry:



W' = 105 "
a - b = 96 " max

Beam Type - **RB-S-500**

H = 180 " (120" for the 2 Level Racks with same loading)
D = 34 "
l_y = 48 "
l_d = 54 " (Bottom two diagonals, 61" top diagonal)
Upright Type - **UF-H-33I**

3.2 Check Beams:

OK per section 2.2.

3.3 Check Posts (Dead plus Live Loads):

3.3.1 Vertical Load:

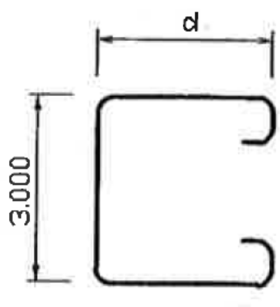
$$P < \frac{(\#\text{ of Levels})(L)}{2} =$$

5.0 kips
7.0 kips factored

3.3.2 Second Order Effects:

Refer to section 2.3.2 for derivation.

3.3.3 Post Properties (net section):



UF-H-33

d =	2.75	in
t =	0.125	in
As =	0.99	in2
Sx =	0.98	in3
rx =	1.19	in
Sy =	0.64	in3
ry =	0.95	in
Fy =	55	ksi

3.3.4 Post Demand, Capacity and Combined Stress Check:

Capacity

L_x =	96	in	
L_y =	48	in	
Kl/r_x =	81		
Kl/r_y =	51		
F_n =	32.6	ksi	
ϕP_n =	27.4	kips	
ϕM_{nx} =	48.5	inch kips	
ϕM_{ny} =	31.7	inch kips	

Stability

K_{conn} =	3000	in-kip / rad	
I_{beam} =	3.51	in4	RB-S-500
L_{beam} =	102	in	
K_{beam} =	5988	in-kip / rad	
K_{theta} =	1999	in-kip / rad	
s =	0.30		

Demand

n =	0.005	
P =	7.0	kips

3.3.4 Post Demand, Capacity and Combined Stress Check Continued:

Demand Continued

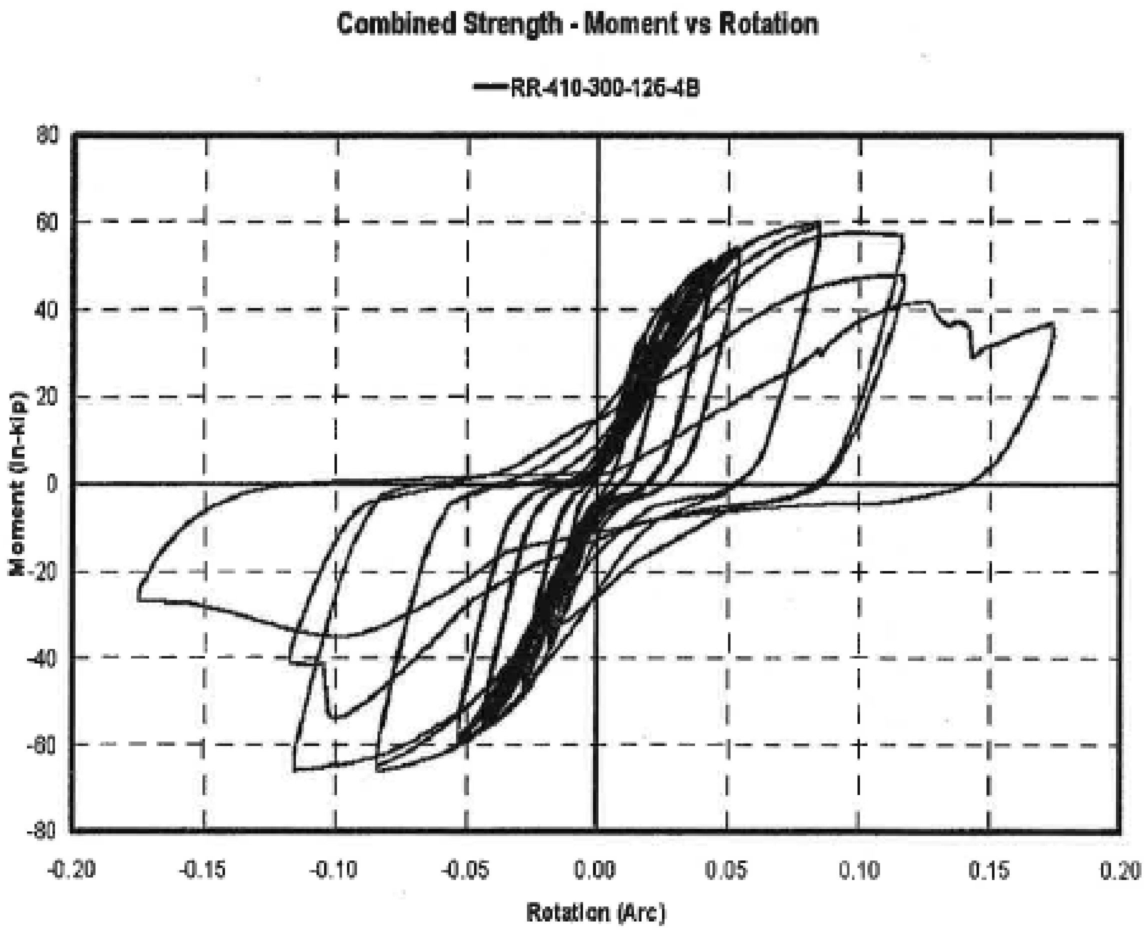
H = 96 inch
M_{not} = 2.31 in-kip

Combined Stress = 0.30

3.4 Longitudinal Seismic:

3.4.1 Connection Properties:

The following are test results from Ridg-U-Rak for the beam end connector stiffness based on RMI MH16.1 specifications for testing of beam end loading:



Secant stiffness at 0.095 rad > 500 in-kips per radian
Maximum rotational capacity > 0.12 radians

3.4.1 Connection Properties Continued:

Note - For the Merchandise Display, 2 Level Racks & 3 Level Racks, the RB-S-500 beams have the same beam end connectors, therefore have the same Combined Strength - Moment vs Rotation chart as shown on the following page:

For the nomenclature for determining the period of the structure refer to section 2.4.2:

3.4.2 Determine Period of Structure:

FEMA 460:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{N} W_{pi} h_{pi}^2}{g\left(N_c\left(\frac{k_c k_{be}}{k_c + k_{be}}\right) + N_b\left(\frac{k_b k_{ce}}{k_b + k_{ce}}\right)\right)}}$$

$$K1 = N_c + N_b\left(\frac{k_b k_{ce}}{k_c k_{be}}\right)\left(\frac{k_c + k_{be}}{k_b + k_{ce}}\right) \qquad K2 = N_c k_{bu} + N_b k_{bu}$$

$$k_{bu} = \frac{k_c k_{be}}{k_e + k_{be}} \qquad k_u = \frac{k_b k_{ce}}{k_b + k_{ce}}$$

Substitution of these expressions into the period equation above yields:

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{N} W_{pi} h_{pi}^2}{g(N_c k_{bu} + N_b k_u)}}$$

This formulation is a general solution for racks with a variable importance factor. The Ie = 1.5 is included in the displacement demand computed above.[While present in the generalize formulation nomenclature, The ASCE 7 C_d amplification factors are not used for the analysis].

The solution is an iterative procedure where the connector stiffness is converged with the rotational angle. For this structure, the upper bound period displacement demand does not exceed the capacity of the connector, therefore the following will be used to calculate the period in the longitudinal (down aisle) direction. The connector properties are shown in the above testing chart for a combined strength (Moment vs. Rotation) per pair of connectors .

3.4.2 Determine Period of Structure Continued:

The typical row in a Costco is between 8 and 10 bays long. For this configuration:

No. Bays	10		g	386	$in^2\ sec^2$
No. Level	2		E	29000	ksi
L	108	inches C/C	Nc	40	
W	5	kips / level	Nb	11	
Ib	3.5	in^4 (RB-S-500)	Kbe	5639	
Ic	1.4	in^4	Kce	2538	
Kc	500	in-kips/rad from above	Kbu	459	
Kb	3000	in-kips/rad	Ku	1375	
			K1	72.9	
			K2	33492.9	

For a rack with a constant load per level and spacing between beams:

Level	Wpi	hpi	Wpi*hpi	$Wpi*hpi^2$
1	25	134	3350	448900
2	25	204	5100	1040400
Sum or Max	50	204	8450	1489300

Yields T = 2.13 sec

3.4.3 Displacement / Rotational Demand:

First order displacement demand:

D = 8.6T = 18.4 inches

Second order effects:

$$D_{tot} = D(1+\alpha)$$

Where:

$$\alpha = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left(\frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left(N_c + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}} \right) \left(\frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} = 0.25$$

Yielding:

D_{tot} = 23.0 inches

Rotational Demand:

$\theta_{max} = D_{tot} / h_{pi}$ max. = 0.11 radian

3.4.4 Connector Capacity:

The stiffness corresponds to the rotational demand, and is within the capacity of the connector.

NOTE: Racks are acceptable under longitudinal, down aisle direction under seismic consideration:

3.5 Transverse Seismic:

Machine Loaded Racks - 34" minimum upright depth governs by inspection.

3.5.1 Determine Period:

The upright / posts are braced using horizontal & diagonal channels in the transverse direction, and may be reasonably considered as a cantilever off the floor slab.

Determine Period - Ref. Roark's Formulas for Stress & Strain, 7th ed., Table 16.7, Case 3b:

$$T = \frac{2\pi}{3.52}\sqrt{\frac{W l^3}{E I g}}$$

For:

D =	34	in (depth of upright)	
I < $0.8[2(D/2)^2 As]$ =	458	in^4	(moment of inertia)
W =	10.0	kips (bay load)	
l = H =	180	in (top beam level)	
T =	0.19	sec	

3.5.2 Displacement Demand:

Second order deflections are clearly negligible, yielding

D_{tot} = 1.6 inches

3.5.3 Displacement Capacity:

While the uprights and beams will deflect under lateral load, this displacement can be easily accommodated by pallet sliding. Movement of the pallets may included in the displacement capacity per ANSI MH16.1 section 2.6.2 Commentary.

3.5.3 Displacement Capacity:

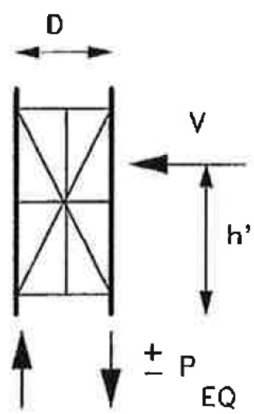
Per the above reference Degee paper cited above, the maximum (and very rare) pallet coefficient of friction on the steel is less than 0.6. For $\mu / \alpha < 0.4$, the pallets do not stick. Conservatively taking $\alpha = 0.6S_s = 1.7$, this ratio will not exceed 0.6 /1.7 = 0.36 < 0.40, so the pallets will not stick, and the dynamic coefficient of friction may be used.

3.5.4 Stability:

ANSI MH16.1 and FEMA 460 are silent on the shear demand to the frame when pallet movement only is used to satisfy the displacement demand. Since sliding forces cannot exceed the coefficient of friction on the dynamically active fraction of the load. The COF for wood on steel (Plastic pallets have a lower value) is typically 0.2 - 0.3. Conservatively use 0.3 to develop base shear.

V = 0.3(10.0)(2/3) = 2.0 kips (10.0 kips per Bay) This will further conservatively be applied in the triangular force distribution on the upright.

3.5.4 Stability Continued:



$$P_{eq} = \frac{Vh'}{D}$$

Use h' = Uses top beam level of rack

h' =	180	inches
OTM = Vh' =	360	inch kips
RM = WD/2 =	170	inch kips(W = 10.0 kips / bay)
Net OTM =	190	inch kips
P_{eq} = OTM / D =	10.6	kips

Note: Bracing shown is schematic - see section 3.1 for actual geometry.

3.5.5 Post Capacity:

$P + P_{eq}$ = 15.6 kips < ϕP_n = 27.4 kips **OK**

3.5.6 Stability:

T_{bolt} = Net OTM / D = 5.6 kips

3.5.6 Stability Continued:

NOTE:
Based on above analysis, the tension forces in the anchoring for the project site located in Puyallup, WA would be much less than the above analysis. Anchoring acceptable as shown:

3.5.7 Anchorage:

Refer to section 2.5.7 for calculations for determining the available capacity of the anchors.
For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum effective embedment of 4". (5" Nominal Embedment to produce 4" effective embedment)

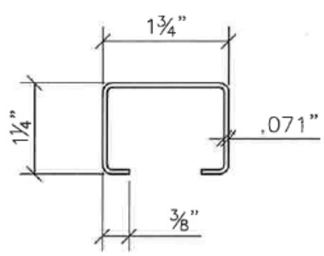
3.5.8 Brace:

Horizontal & diagonal braces are the same section, therefore diagonal brace will govern design.

3.5.8.1 Design Forces:

$P_{br} \leq$ 3.8 kips

2.5.8.2 Brace Capacity:



A_{net} = 0.32 in2
r_{min} = 0.46 in
ϕ= 0.85 (AISI - Compression)
k = 0.80
t = 14ga

L (in)	$\frac{kl}{r}$	F_n (ksi)	ϕP_n (kips)	
54	95	26.6	7.2	**OK**