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PROJECT: COSTCO WHOLESALE STORE #660 – PUYALLUP (REMODEL & ADDITIONAL RACKING THROUGHOUT THE STORE)
(STORAGE RACK BY RIDG-U-RAK)

LOCATION: 1201 39th AVENUE SW
PUYALLUP, WA 98373

PREPARED FOR: COSTCO WHOLESALE – ISSAQUAH, WA

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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 3, section 1.4 "Drawings". All documents shall bear appropriate seals and signatures in ink of contrasting color.

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This report has been prepared in Collaboration with Peter S. Higgins & Associates

Date: 08/25/2021
GKM Job Number 2108127
Sheet #0 of #23

8/25/21

ADAM MARK McLAUGHLIN
STATE OF WASHINGTON
48096
REGISTERED
PROFESSIONAL ENGINEER

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

PRCTI20220514

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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 (5.0' high units) 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. (Exceeds 2018 IBC with WA Amendments as required - Seismic Requirements)

1.2 Project History:

This is a modification to a store reviewed by Peter S. Higgins & Associates per the following history:

Original Costco Store Review: June 2000 - (Higgins Job #04384)
Remodeled: January 2008 (Higgins Job #08359)
Cooler Addition: August 2013 (Higgins Job #10647)
Remodeled and Over Freezer Racks Added: May 2014 (Higgins Job #10970)
Meat Deli Cooler Remodeled: August 2015 (Higgins Job#11458)
Remodel / Additional Racks Sales Floor: June 2019 - (Higgins Job #13218 - GKM #190692)

Existing storage racks were installed under permits as required by the local authority, and remain qualified by the review and permit. Remodeling required, new racking can be substituted for existing racks, same rack material. New racking shown in revision clouds are reviewed in 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.

1.3.3 References Continued:

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 / Existing Floor Plan EX21 (Project #98-5080 - dated August 11, 2021)

1.5 Loads:

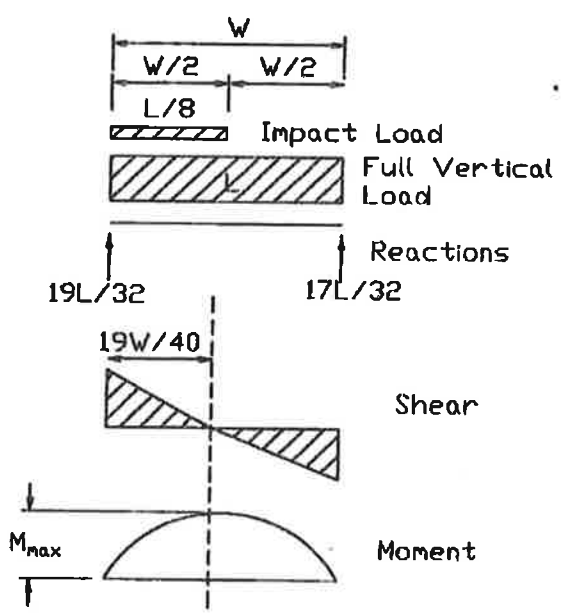
1.5.1 Vertical (Dead plus Live):

Load per pallet = 2.5 kips
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.



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

1.5.3 Seismic Continued:

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:

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

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 >> Sms_{actual} = 1.52 - OKAY

$S_{m1} = F_v S_1 =$ 1.50 >> $Sm1_{actual}$ = 0.81 - OKAY

For default soils:

F_a = 1.00 $S_s \leq 2.90$

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 = 0.44 per Sms = 1.52; Sm1 = 0.81 per ASCE 7 Hazards report TL = 6; Ts = 0.54; 1.5Ts = 0.80

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

$0.6S_{ds}$ = 1.16 g > 0.6g B = 1.70

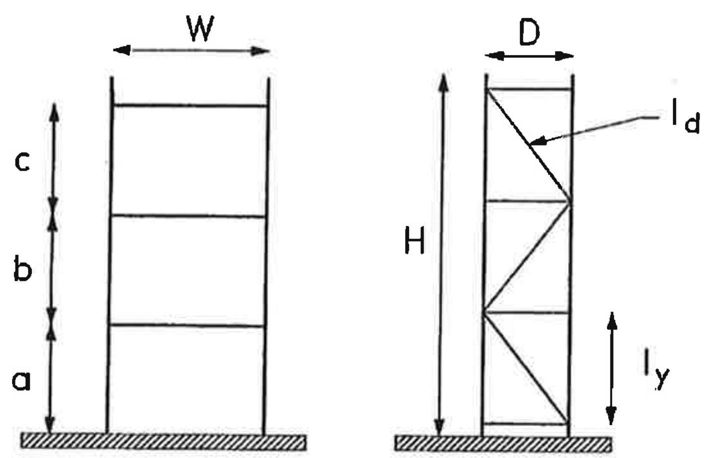
Yielding:

D = 8.6 T

T may be determined by any rational method, but the formulations of

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**

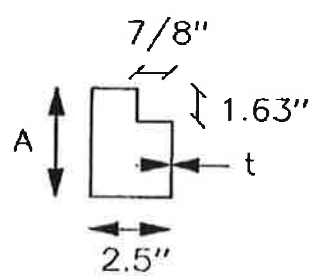
2.2 Check Beams:

2.2.1 Design Forces:

$$M \leq \frac{361\,W\,L}{2560} = \qquad S_{min} < \frac{M}{\phi F_{ye}} =$$

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**

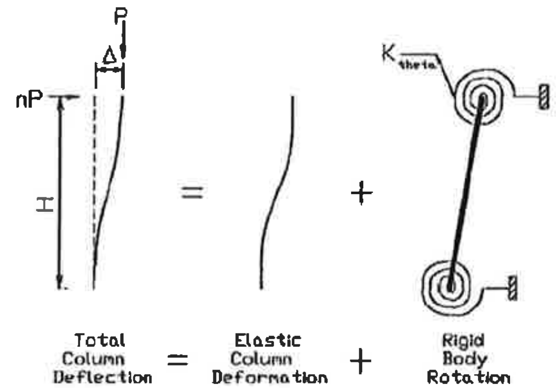
2.3 Check Posts (Dead plus Live Loads):

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

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

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

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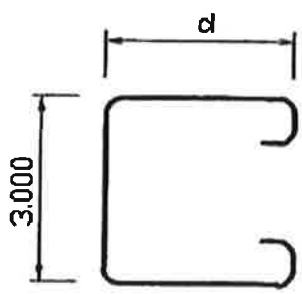
2.3.3 Post Properties (net section):



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

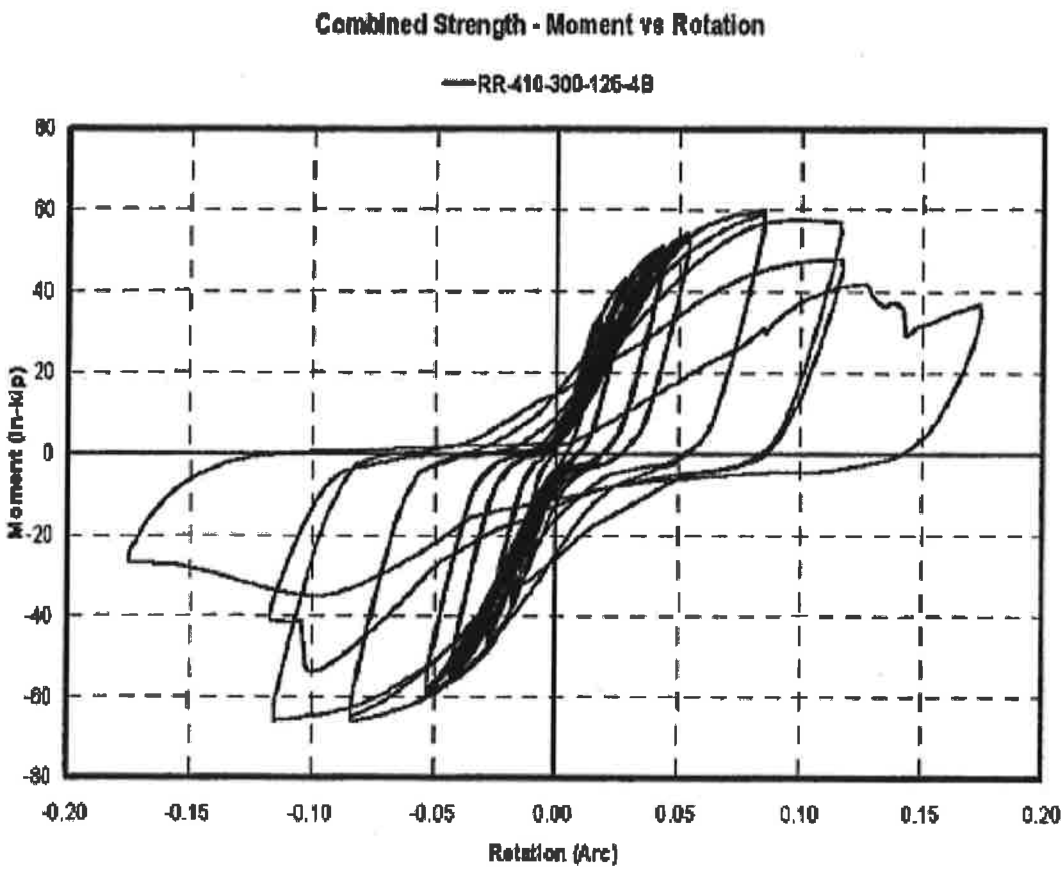
2.3.4 Post Demand, Capacity and Combined Stress Check Continued:

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

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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 minimum 8 bays long. For this configuration:

No. Bays	8		g	386	$in^2\ sec^2$
No. Level	3		E	29000	ksi
L	108	inches C/C	Nc	48	
W	5	kips / level	Nb	9	
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	81.8	
			K2	37572	

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

Level	Wpi	hpi	Wpi*hpi	$Wpi*hpi^2$
1	20	84	1680	141120
2	20	144	2880	414720
3	20	204	4080	832320
Sum or Max	60	204	8640	1388160

Yields T = 1.94 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:

$$\alpha = \frac{\sum_{i=1}^{N_L} W_{pi} h_{pi} \left(\frac{k_c + k_{be}}{k_c k_{be}} \right)}{\left(N_c + N_b \left(\frac{k_b k_{ce}}{k_c k_{be}} \right) \left(\frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} = 0.23$$

Yielding:

D_{tot} = **20.6 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:

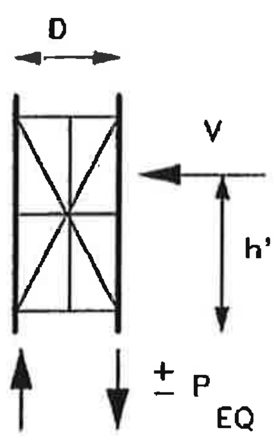
While the uprights and beams will deflect under lateral load, this displacement can be easily accommodated by pallet sliding. Movement of the pallets may included in the displacement capacity per ANSI MH16.1 section 2.6.2 Commentary.

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 $\alpha = 0.6S_s = 1.7$, 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.

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 $<$ $\phi P_n =$ 37.7 kips **OK**

2.5.6 Stability:

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

2.5.6 Stability Continued:

NOTE:
Based on above analysis, the seismic ground motion coefficients for Puyallup, WA are much less than noted above, therefore anchoring is acceptable for Puyallup, WA.

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") h_{nom} = 5.0"
$\psi_{ed,}$N	1	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).

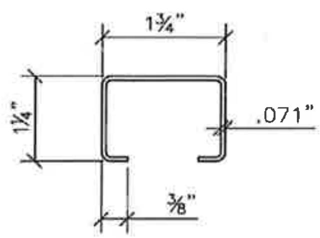
** For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum effective embedment of 4" as shown above (5" minimum nominal embedment)

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:



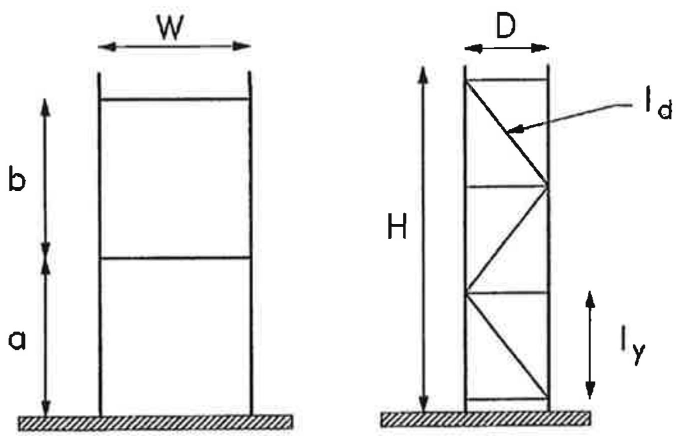
A_{net} =	0.32 in2
r_{min} =	0.46 in
ϕ=	0.85 (AISI - Compression elements)
k =	0.80
t =	14ga

2.5.8.2 Brace Capacity Continued:

L (in)	$\frac{kl}{r}$	F_n (ksi)	ϕP_n (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:

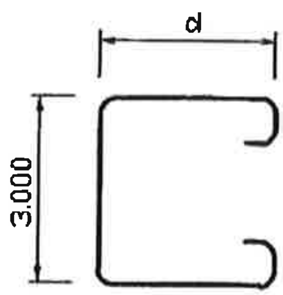
$$P < \frac{(\#\text{ of Levels})(L)}{2} =$$

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

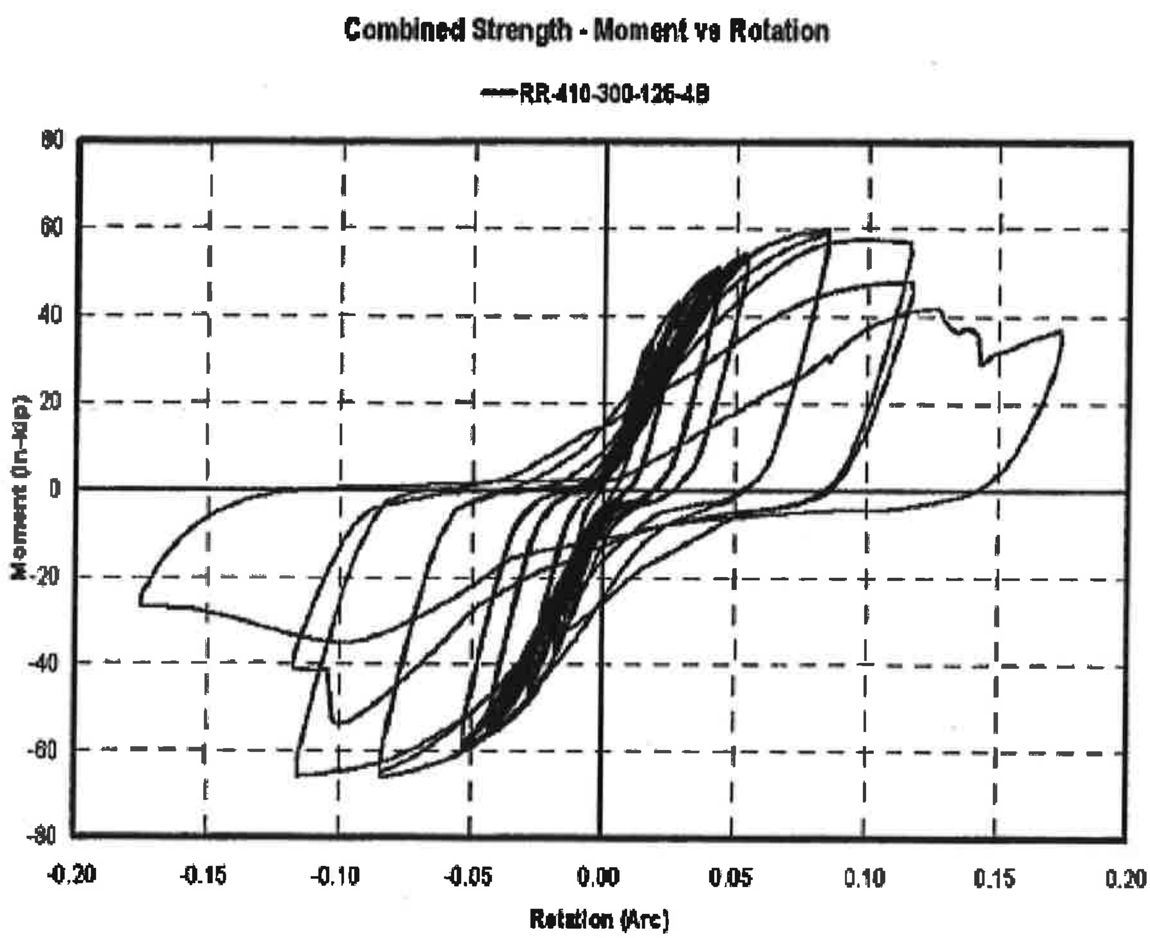
M_{not} = 2.31 in-kip

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 minimum 8 bays long. For this configuration:

No. Bays	8		g	386	in^2 sec^2
No. Level	2		E	29000	ksi
L	108	inches C/C	Nc	32	
W	5	kips / level	Nb	9	
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	58.9	
			K2	27069.3	

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

Level	Wpi	hpi	Wpi*hpi	$Wpi*hpi^2$
1	20	134	2680	359120
2	20	204	4080	832320
Sum or Max	40	204	6760	1191440

Yields T = 2.12 sec

3.4.3 Displacement / Rotational Demand:

First order displacement demand:

D = 8.6T = 18.3 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} = 22.9 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:

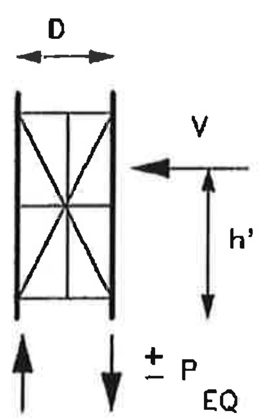
Per the above reference Degee paper cited above, the maximum (and very rare) pallet coefficient of friction on the steel is less than 0.6. For $\mu / \alpha < 0.4$, the pallets do not stick. Conservatively taking $\alpha = 0.6Ss = 1.7$, this ratio will not exceed $0.6 / 1.7 = 0.36 < 0.40$, so the pallets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

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 < ϕPn = 27.4 kips **<u>OK</u>**

3.5.6 Stability:

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

3.5.6 Stability Continued:

NOTE:
Based on above analysis, the seismic ground motion coefficients for Puyallup, WA are much less than noted above, therefore anchoring is acceptable for Puyallup, WA.

3.5.7 Anchorage:

Refer to section 2.5.7 for calculations for determining the available capacity of the anchors. Based on the above uplift force use minimum 1/2" diameter ICC approved external thread screw anchors with a minimum embedment of 4.0" (5.0" nominal embedment) into concrete slab for above high seismic zones.

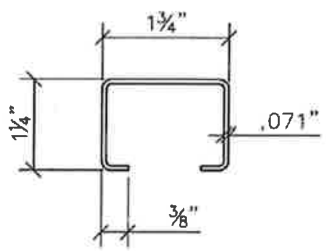
3.5.8 Brace:

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**