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PROJECT: COSTCO WHOLESALE STORE #1838 – LEE’S SUMMIT (NEW STORE / NEW RACKING AND TIRE RACK UNITS FOR TIRE SALES AREA)  
**(STORAGE & TIRE RACK BY RIDG-U-RAK)**  
LOCATION: 10 SOUTHEAST OLDHAM PARKWAY  
LEE'S SUMMIT, MO 64081

PREPARED FOR: COSTCO WHOLESALE – ISSAQUAH, WA

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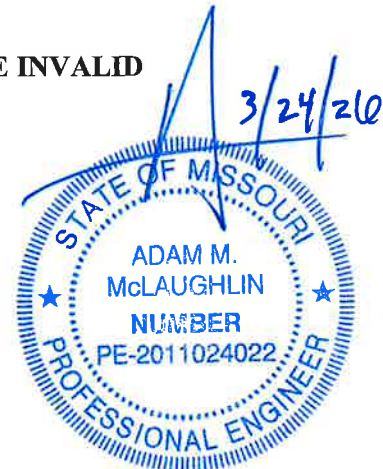
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**Notice to Building Departments**

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 2, section 1.4 “Drawings”. All documents shall bear appropriate seals and signatures per Building Code requirements.

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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 racking) is exempt from the building permit (although it easily satisfies seismic criteria).

Code Requirements - 2021 IBC with Missouri Amendments, ASCE/SEI 7, RMI/ANSI MH16.1, AISI/ANSI S100, and FEMA 460 are used for the structural design requirements.

### **1.2 Project History:**

This racking project is for a new Costco Warehouse Store #1838 - Lee's Summit, MO

This review is for the new storage rack & tire rack units as supplied by Ridg-U-Rak per drawings CW-10T & RU-9 and Costco's Floor Plan (Rack Layout) #A-101. (Dwg. # 25-5563-01 Dated 02/17/2026)

### **1.3 Design Methods:**

#### **1.3.1 Static Loads:**

Per ANSI MH16.1, this review employs LRFD direct design methods which are the only unrestrained 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

### **1.3.3 References Continued:**

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-10T, RU-9

#### **1.4.2 By Others:**

Costco Wholesale #1838 - Lee's Summit, MO

Costco's Floor Plan (Rack Layout) #A-101. (Dwg. # 25-5563-01 Dated 02/17/2026)

**1.5 Loads:**

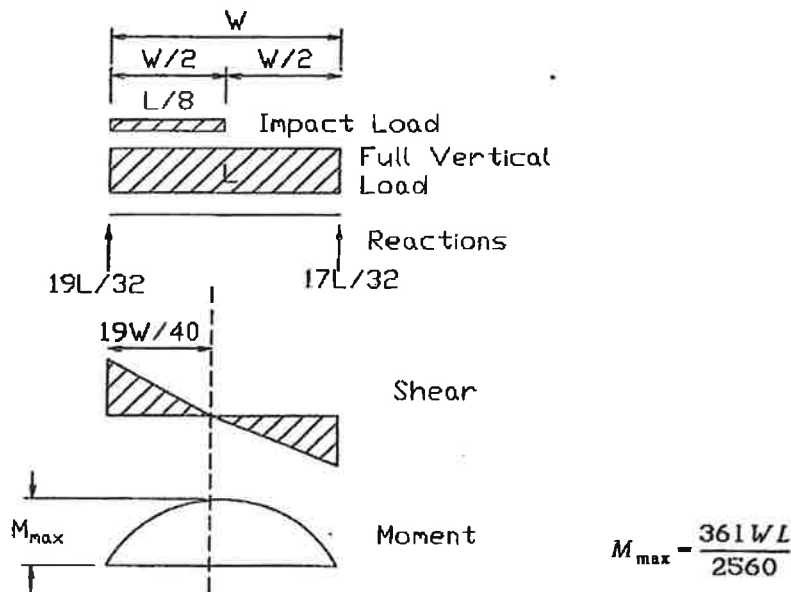
**1.5.1 Vertical (Dead plus Live):**

Load per pallet = 2.5 kips (2 pallets wide / level at 2.5 kips / pallet = 5.0 kips per 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 = Load on the beam (usually expressed in terms of a unit or pallet load). W = Width 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.



**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:

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

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**1.5.3 Seismic Continued:**

The demands are to MCE levels which include the  $I_e = 1.5$  factor for public warehouse stores.

Given:

$$S_{ms} = F_a S_s = 2.90 \gg S_{ms \text{ act}} = 0.160 \text{ OKAY}$$

$$S_{m1} = F_v S_1 = 1.50 \gg S_{m1 \text{ act}} = 0.164 \text{ OKAY}$$

For default soils:

$$F_a = 1.20 \quad S_s \leq 2.42$$

$$F_v = 1.50 \quad S_1 \leq 1.00$$

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 Lee's Summit are  $S_s = 0.100$ ;  $S_1 = 0.068$ ;

$S_{ms} = 0.160$ ;  $S_{m1} = 0.164$ ;  $S_{ds} = 0.107$ ;  $S_{d1} = 0.109$

per ASCE7 - Hazards Report - Actual SDC - "B"

B may be conservatively taken from the following table:

$0.6S_{ds}$	Damping	B	$0.6S_{ds} = 1.1616 \text{ g} > 0.6\text{g}$	B = 1.70
< 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		

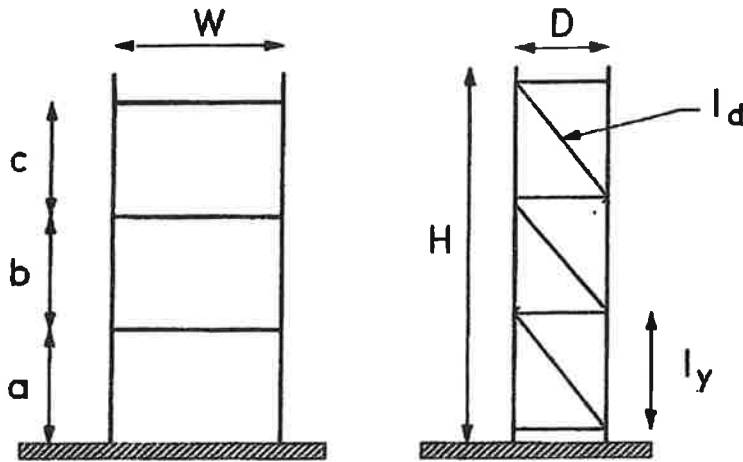
Yielding:

$$\mathbf{D = 8.6 T}$$

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 "  
 a = 60 "  
 b = 60 "  
 c = 60 "

H = 180 "  
 D = 34 "  
 l<sub>y</sub> = 48 "  
 l<sub>d</sub> = 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:**

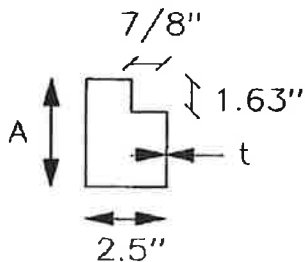
$$M \leq \frac{361 WL}{2560} =$$

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

51.8 inch kips factored

0.96 in<sup>3</sup>

**2.2.2 Beam Properties:**



**RB-S-500**

A = 5.00 in

t = 0.075 in

S<sub>x</sub> = 1.40 in<sup>3</sup>

I<sub>x</sub> = 3.51 in<sup>4</sup>

**OK**

**OK**

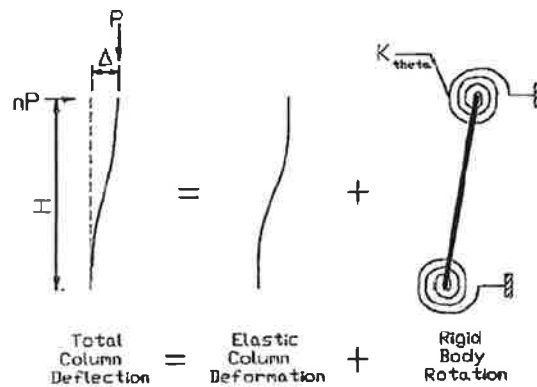
**2.3 Check Posts (Dead plus Live Loads):**

**2.3.1 Vertical Load:**

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

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:

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

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

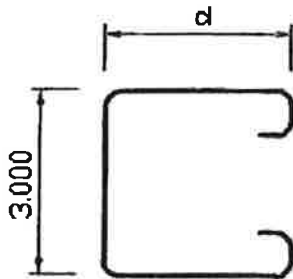
$$K_{theta} = \left( \left( \frac{1}{K_{beam}} \right) + \left( \frac{1}{K_{conn}} \right) \right)^{-1}$$

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 in<sup>2</sup>  
S<sub>x</sub> = 0.98 in<sup>3</sup>  
r<sub>x</sub> = 1.19 in  
S<sub>y</sub> = 0.64 in<sup>3</sup>  
r<sub>y</sub> = 0.95 in  
F<sub>y</sub> = 55.0 ksi

**2.3.4 Post Demand, Capacity and Combined Stress Check:**

**Capacity**

L<sub>x</sub> = 60 in  
L<sub>y</sub> = 48 in  
Kl/r<sub>x</sub> = 50  
Kl/r<sub>y</sub> = 51  
F<sub>n</sub> = 44.8 ksi  
φP<sub>n</sub> = 37.7 kips  
φM<sub>nx</sub> = 48.5 inch kips  
φM<sub>ny</sub> = 31.7 inch kips

**Stability**

K<sub>conn</sub> = 3000 in-kip / rad  
I<sub>beam</sub> = 3.51 in<sup>4</sup> RB-S-500  
L<sub>beam</sub> = 105 in  
K<sub>beam</sub> = 5817 in-kip / rad  
K<sub>theta</sub> = 1979 in-kip / rad  
s = 0.24

**Demand**

n = 0.005  
P = 10.5 kips  
H = 60 inch  
M<sub>not</sub> = 2.1 in-kip

**2.3.4 Post Demand, Capacity and Combined Stress Check Continued:**

Combined Stress = 0.32

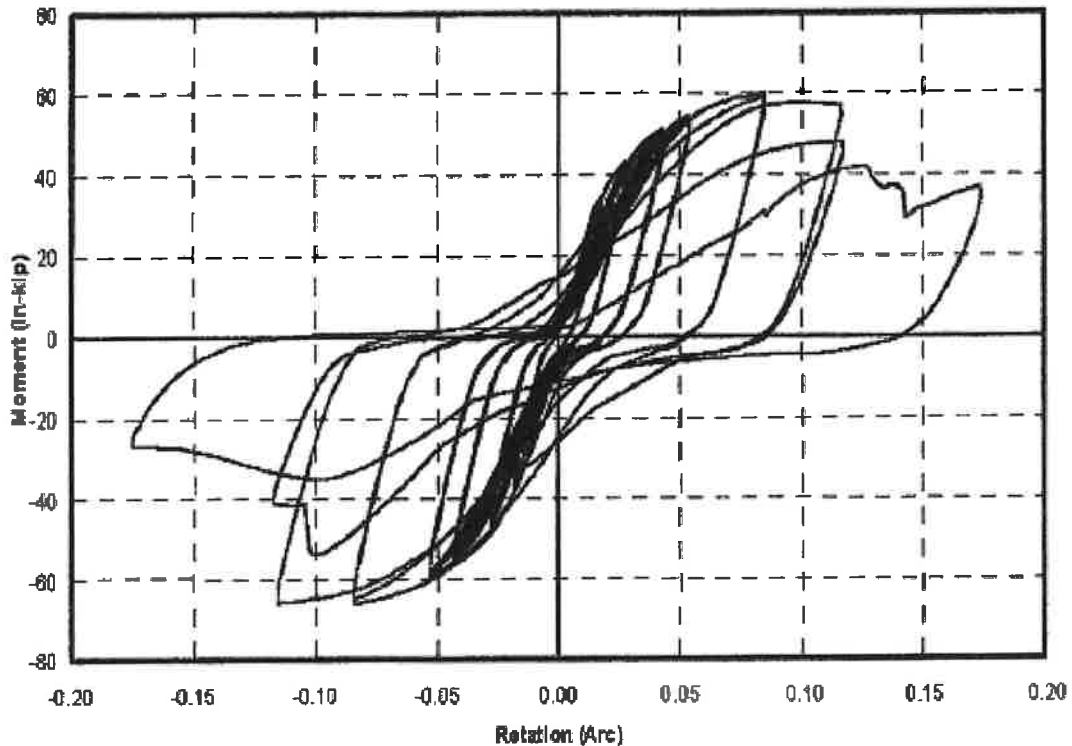
**2.4 Longitudinal Seismic:**

**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:

**Combined Strength - Moment vs Rotation**

—RR-410-300-126-4B



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:

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

Nomenclature (used throughout)

Cd = displacement amplification factor from ASCE 7

DBE = Design Basis Earthquake

DBE/MCE = ratio of earthquake accelerations

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 = acceleration 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 stiffness 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 acceleration parameter = Sml(DBE/MCE)
- Sds = Design spectral response acceleration parameter = Sms(DBE/MCE)
- Sml = 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

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

$$k_{bu} = \frac{k_c k_{ba}}{k_c + k_{ba}} \qquad k_u = \frac{k_b k_{ca}}{k_b + k_{ca}}$$

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 Cd 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 .

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**2.4.2 Determine Period of Structure Continued:**

The typical row in a Costco new facility is between 8 & 10 bays long. For this configuration use:

No. Bays	10		g	386	in <sup>2</sup> sec <sup>2</sup>
No. Level	3		E	29000	ksi
L	108	inches C/C	Nc	60	
W	5	kips / level	Nb	11	
Ib	3.5	in <sup>4</sup> (RB-S-500)	Kbe	5639	
Ic	1.4	in <sup>4</sup>	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 <sup>2</sup>
1	25	84	2100	176400
2	25	144	3600	518400
3	25	204	5100	1040400
Sum or Max	75	204	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:

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

Yielding:

**D<sub>tot</sub> = 20.8 inches**

**2.4.3 Displacement / Rotational Demand Continued:**

Rotational Demand:

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

**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:

$$T = \frac{2\pi}{3.52} \sqrt{\frac{Wl^3}{EIg}}$$

For:

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

**2.5.2 Displacement Demand:**

Second order deflections are clearly negligible, yielding

$$D_{\text{tot}} = 2.0 \text{ inches}$$

**2.5.3 Displacement Capacity:**

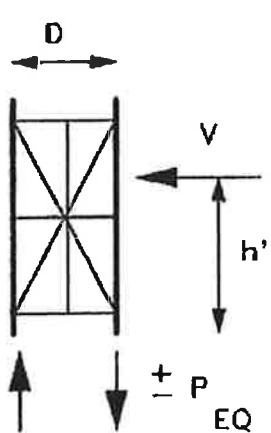
While the uprights and beams will deflect under lateral load, this displacement can be easily accommodated by pallet sliding. Movement of the pallets may included in the displacement capacity per ANSI MH16.1 section 2.6.2 Commentary.

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

**2.5.6 Stability:**

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

Very conservative - 67% fully loaded and top load conditions check and will not exceed fully loaded case noted above:

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**2.5.6 Stability Continued:**

**NOTE:**

Based on above analysis, for a project site located in Lee's Summit, MO the seismic forces will not produce uplift nor tension in the anchors and is 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 <sup>2</sup>	Breakout area of anchor group
Anco	in <sup>2</sup>	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
$\delta_a$	none	Modification for light weight concrete
$\phi_{sa} n V_{saeq}$	Lbs.	Total Shear resistance of the 1/2 steel anchors
$\phi_{cb} V_{cbg}$	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. none	Breakout capacity of concrete for bolt group 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
$\phi_i$	none	Combined strength reduction under seismic per ACI 318 - 17.2.3.4.4 & (0.75*0.65) = 0.49
$\phi_i N_{cbg}$	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

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**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 from edges)
$\delta_a =$	1.0	Modification for light weight concrete	
Variables		Basis $f'_c = 2500$	
<b>Concrete</b>		<b>Anchors</b> 1/2" diameter - Hilti KH-EZ Screw Anchors referenced	
$f'_c =$	3000	$N_{p,uncr} =$ N/A With screw type anchors	
<b>Computed</b>		$N_{p,cr} =$ 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
$\phi N_{cbg}$	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).

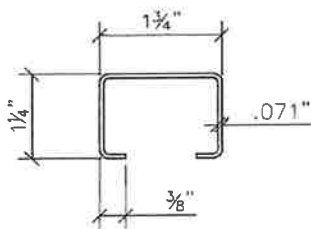
**\*\*** For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum nominal embedment of 5" for screw type anchors. (Nominal embedment = Top of Concrete Slab to Bottom Tip of Anchor)

**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:**



$A_{net} = 0.32$  in<sup>2</sup>  
 $r_{min} = 0.46$  in  
 $\phi = 0.85$  (AISI - Compression elements)  
 $k = 0.80$   
 $t = 14ga$

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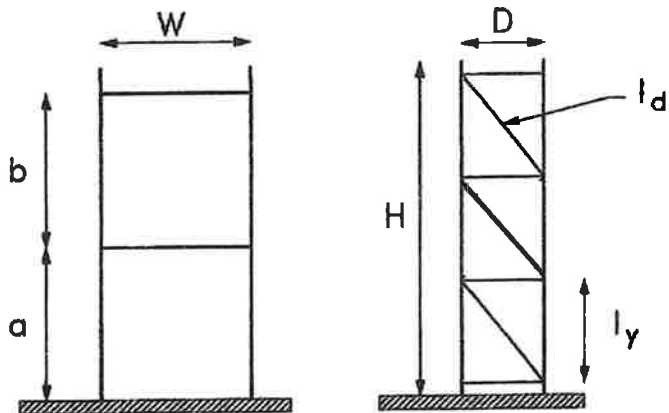
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**2.5.8.2 Brace Capacity Continued:**

L	$\frac{kl}{r}$	$F_n$	$\phi P_n$	
(in)	r	(ksi)	(kips)	
54	95	26.6	7.2	<b><u>OK</u></b>

**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:**

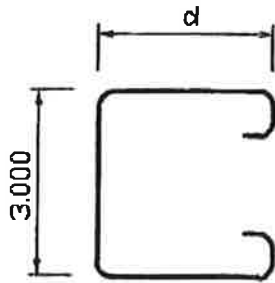
$$P < \frac{(\# \text{ of Levels})(L)}{2} =$$

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 in <sup>2</sup>
Sx =	0.98 in <sup>3</sup>
rx =	1.19 in
Sy =	0.64 in <sup>3</sup>
ry =	0.95 in
Fy =	55 ksi

**3.3.4 Post Demand, Capacity and Combined Stress Check:**

**Capacity**

L <sub>x</sub> =	96 in
L <sub>y</sub> =	48 in
Kl/r <sub>x</sub> =	81
Kl/r <sub>y</sub> =	51
F <sub>n</sub> =	32.6 ksi
φP <sub>n</sub> =	27.4 kips
φM <sub>nx</sub> =	48.5 inch kips
φM <sub>ny</sub> =	31.7 inch kips

**Stability**

K <sub>conn</sub> =	3000 in-kip / rad
I <sub>beam</sub> =	3.51 in <sup>4</sup> RB-S-500
L <sub>beam</sub> =	105 in
K <sub>beam</sub> =	5817 in-kip / rad
K <sub>theta</sub> =	1979 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**

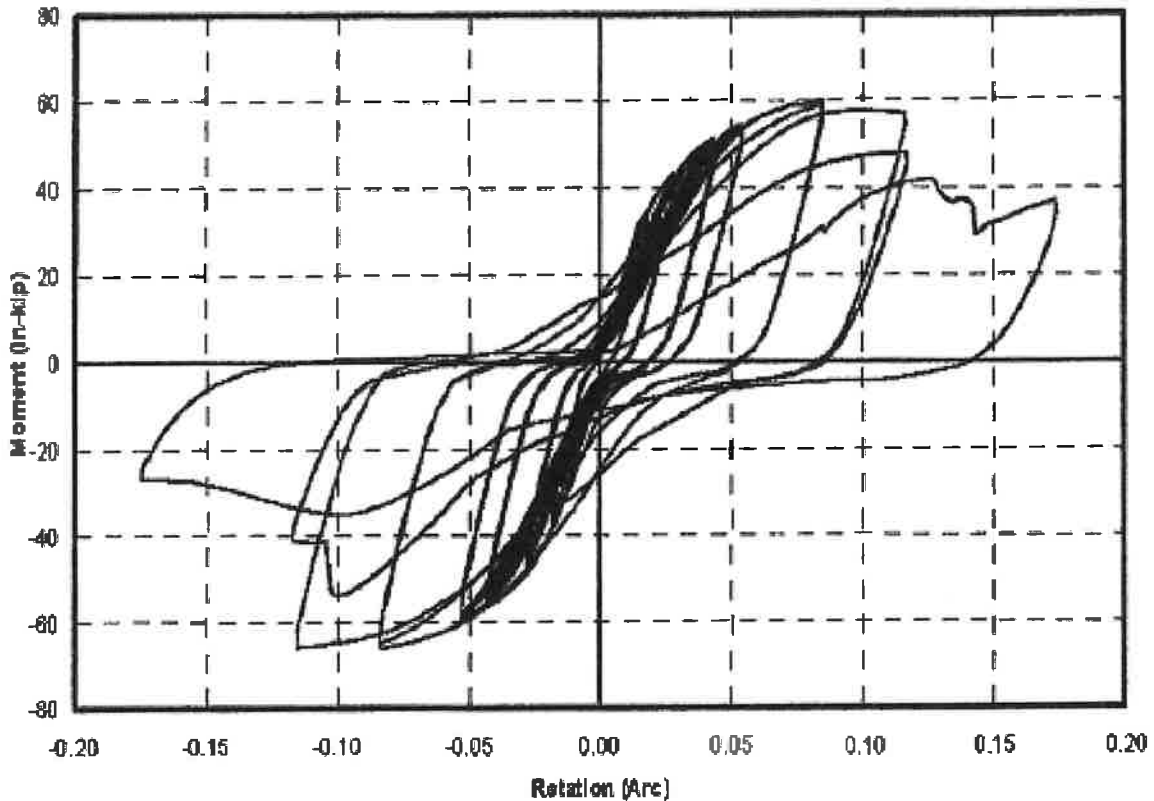
**3.4 Longitudinal Seismic:**

**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:

**Combined Strength - Moment vs Rotation**

—RR-410-300-125-4B



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:

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

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

$$K2 = N_c k_{bu} + N_b k_{bu}$$

$$k_{bu} = \frac{k_c k_{be}}{k_c + k_{be}}$$

$$k_u = \frac{k_b k_{ce}}{k_b + k_{ce}}$$

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  $I_e = 1.5$  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 .

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**3.4.2 Determine Period of Structure Continued:**

The typical row in a Costco new facility is between 8 & 10 bays long. For this configuration use:

No. Bays	10			g	386	in <sup>2</sup> sec <sup>2</sup>
No. Level	2			E	29000	ksi
L	108	inches C/C		Nc	40	
W	5	kips / level		Nb	11	
Ib	3.5	in <sup>4</sup> (RB-S-500)		Kbe	5639	
Ic	1.4	in <sup>4</sup>		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 <sup>2</sup>
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:

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

Yielding:

**D<sub>tot</sub> = 23.0 inches**

Rotational Demand:

**θ<sub>max</sub> = D<sub>tot</sub> / h<sub>pi</sub> 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:

$$T = \frac{2\pi}{3.52} \sqrt{\frac{Wl^3}{EIg}}$$

For:

D = 34 in (depth of upright)

$I < 0.8[2(D/2)^2 A_s] = 458 \text{ in}^4$  (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

$$D_{\text{tot}} = 1.6 \text{ inches}$$

#### **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:**

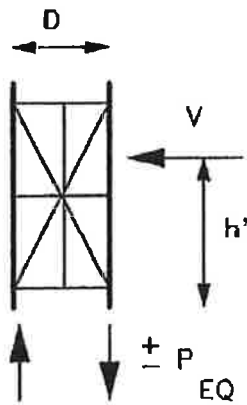
Per the above reference Degee paper cited above, the maximum (and very rare) pallet coefficient of friction on the steel is less than 0.6. For  $\mu / \alpha < 0.4$ , the pallets do not stick. Conservatively taking  $\alpha = 0.6Ss = 1.7$ , this ratio will not exceed  $0.6 / 1.7 = 0.36 < 0.40$ , so the 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  $< \phi P_n = 27.4$  kips **OK**

**3.5.6 Stability:**

$T_{bolt} = \text{Net OTM} / D = 5.6$  kips

Very conservative - 67% fully loaded and top loa conditions check and will not exceed fully loaddec case noted above:

**3.5.6 Stability Continued:**

**NOTE:**

Based on above analysis, for a project site located in Lee's Summit, MO the seismic forces will not produce uplift nor tension in the anchors and is acceptable as shown:

**3.5.7 Anchorage:**

For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum embedment of 5" for screw type anchors. (Minimum embedment = Top of Concrete Slab to Bottom Tip of Anchor) is acceptable.

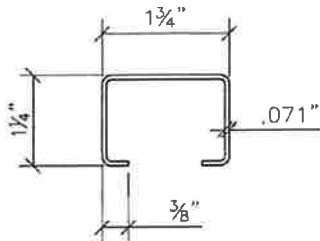
**3.5.8 Brace:**

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

**3.5.8.1 Design Forces:**

$$P_{br} \leq 3.8 \text{ kips}$$

**2.5.8.2 Brace Capacity:**



$$A_{net} = 0.32 \text{ in}^2$$

$$r_{min} = 0.46 \text{ in}$$

$$\phi = 0.85 \text{ (AISI - Compression)}$$

$$k = 0.80$$

$$t = 14ga$$

L	$\frac{kl}{r}$	$F_n$	$\phi P_n$
(in)	r	(ksi)	(kips)
54	95	26.6	7.2

**OK**

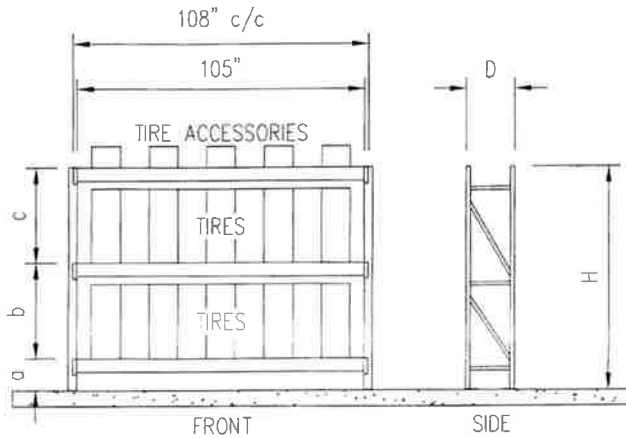
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**4 Tire Racks:**

**4.1 Components and Geometry:**



W' =	105 "	H =	84 "
a =	12 "	D =	18 "
b =	36 "	l <sub>y</sub> =	32 "
c =	36 "	l <sub>d</sub> =	28 " (Bottom diagonals, 31" top diagonal)
Beam Type -	<b><u>RB-S-500</u></b>	Upright Type -	<b><u>UF-S-31</u></b>

**4.2 Check Beams:**

**4.2.1 Design Forces:**

The maximum load on a tire rack beam level is approximately 400 lbs. All rack beams are manufactured using the standard 5" high beam at 105" in length for the Pallet Racks & the Tire Racks, therefore by inspection, referring to section 2.2, beams are acceptable for the Tire Rack Units.

**4.3 Check Posts (Dead plus Live Loads):**

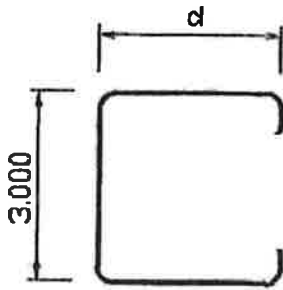
**4.3.1 Vertical Load At Critical Section:**

$P < \frac{(\# \text{ of Levels})(L)}{2} =$	# of Levels above critical section = 2 (base beam not included)
0.4 kips	Based on ≈ 12.5 pcf of gross area -
0.56 kips factored	conservatively use 400 lbs / beam level max.

**4.3.2 Second Order Effects:**

Refer to section 2.3.2 for notional loading formulas to determine second order effects.

**4.3.3 Post Properties (net section):**



**UF-S-31**

d = 1.65 in  
t = 0.077 in  
As = 0.426 in<sup>2</sup>  
S<sub>x</sub> = 0.38 in<sup>3</sup>  
r<sub>x</sub> = 1.152 in  
S<sub>y</sub> = 0.183 in<sup>3</sup>  
r<sub>y</sub> = 0.648 in  
F<sub>y</sub> = 55.0 ksi

**4.3.4 Post Demand, Capacity and Combined Stress Check:**

**Capacity**

L<sub>x</sub> = 36 in  
L<sub>y</sub> = 32 in  
Kl/r<sub>x</sub> = 31  
Kl/r<sub>y</sub> = 49  
F<sub>n</sub> = 42.8 ksi  
φP<sub>n</sub> = 15.5 kips  
φM<sub>nx</sub> = 18.7 inch kips  
φM<sub>ny</sub> = 9.1 inch kips

**Stability**

K<sub>conn</sub> = 3000 in-kip / rad  
I<sub>beam</sub> = 3.51 in<sup>4</sup> RB-S-500  
L<sub>beam</sub> = 105 in  
K<sub>beam</sub> = 5817 in-kip / rad  
K<sub>theta</sub> = 1979 in-kip / rad  
s = 0.01

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**4.3.4 Post Demand, Capacity and Combined Stress Check Continued:**

**Demand**

n = 0.005  
 P = 15.5 kips  
 H = 36.0 inch  
 $M_{not} = 0.05$  in-kip

**Combined Stress = 0.04**

**4.4 Longitudinal Seismic:**

**4.4.1 Connection Properties:**

Referring to the above section 2.4.1 for testing of beam end connectors.

**4.4.2 Determine Period of Structure:**

Referring to the above section 2.4.2 for procedure on calculating the period of the structure with the following:

The typical row in a Costco is minimum 2 bays long. For this configuration:

No. Bays	2		g	386	$\text{in}^2 \text{sec}^2$
No. Level	2		E	29000	ksi
L	108	inches C/C	Nc	8	
W	0.4	kips / level	Nb	3	
Ib	3.5	$\text{in}^4$ (RB-S-500)	Kbe	5639	
Ic	0.746	$\text{in}^4$	Kce	3606	
Kc	500	in-kips/rad from above	Kbu	459	
Kb	3000	in-kips/rad	Ku	1638	
			K1	18.7	
			K2	8587	

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

Level	Wpi	hpi	Wpi*hpi	Wpi*hpi <sup>2</sup>
1	0.4	63	25.2	1587.6
2	0.4	99	39.6	3920.4
Sum or Max	0.8	99	64.8	5508
	Yields T =	0.26 sec		

**4.4.3 Displacement / Rotational Demand:**

First order displacement demand:

$$D = 8.6T = 2.2 \text{ inches}$$

Second order effects:

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

Where:

$$\alpha = \frac{\sum_{i=1}^{N_r} W_{pi} h_{pi} \left( \frac{k_c + k_{bo}}{k_c k_{bo}} \right)}{\left( N_c + N_b \left( \frac{k_b k_{ce}}{k_c k_{be}} \right) \left( \frac{k_c + k_{be}}{k_b + k_{ce}} \right) \right)} = 0.01$$

Yielding:

$$D_{tot} = 2.2 \text{ inches}$$

**Rotational Demand:**

$$\theta_{max} = D_{tot} / h_{pi \text{ max.}} = 0.02 \text{ radian}$$

**4.4.4 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:**

**4.5 Transverse Seismic:**

Hand Loaded Racks - 18" minimum upright depth:

**4.5.1 Determine Period:**

Using the above equations from section 2.5.1 for calculating the period of the structure in the cross aisle direction is approximately 0.06 sec, conservatively well within the capacity noted above.

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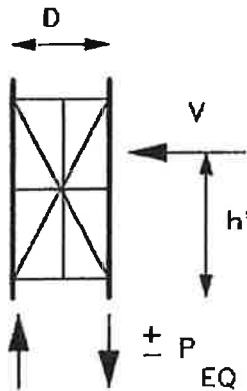
**4.5.2 Stability:**

Conservatively use the maximum  $Sds = 1.56$  &  $le = 1.0$  to determine forces on system:

$$C_s = Sds / R = 4 = 0.39 \qquad W = 1.2 \text{ kips}$$

$$V_{base} = (C_s)(le) W_e = 0.31 \qquad W_e = W(2/3) = 0.8 \text{ kips}$$

For a unit that has a beam at the base, use the RMI specification for triangular force distribution, section 2.6.7-RMI 12 to calculate the OTM:



Using  $p = 1.3$  for redundancy

For maximum load to critical section of post, between the base beam and the 2nd beam up with redundancy.

$$OTM_{critical} = 22.4 \text{ in-kips maximum}$$

$$P_{eq \text{ critical section}} = OTM / D = 1.2 \text{ kips (D = 18")}$$

$$P_{vertical} = 0.6 \text{ kips}$$

$$P_{total} = 1.8 \text{ kips}$$

The distance from floor to top of 1st beam level is 12" & does not govern:

**4.5.3 Post Capacity:**

$$P + P_{eq} = 1.8 \text{ kips} < \phi P_n = 15.5 \text{ kips} \quad \underline{\text{OK}}$$

**4.5.4 Stability:**

Check uplift using 67% Full load & the triangular force distribution per the RMI specification:

NOTE: The top beam level has conservatively been checked for 400 lbs, actual loading is less and are loaded with tire accessories and not with tires, therefore loading top level only does not govern. Use  $(0.9-0.2(Sds))P$  as restraining force.

$$OTM_{base} = 19.9 \text{ in-kips} \qquad 67\%OTM_{base} = 13.3 \text{ in-kips}$$

$$RM = 6.4 \text{ in-kips} \qquad 67\%RM = 4.3 \text{ in-kips}$$

$$67\%OTM_{net} = 9.1 \text{ in-kips}$$

$$P_{uplift} = 0.51 \text{ kips maximum}$$

$$T_{bolt} = \text{Net OTM} / D = P_{uplift} = 0.51 \text{ kips / anchor}$$

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**4.5.5 Anchorage For Tire Racks:**

Conservatively use (1) 1/2" dia. Hilti KH-EZ Screw Anchor per base plate with the 5" minimum embedment in to the concrete slab (Minimum embedment = Top of Concrete Slab to Bottom Tip of Anchor)

For low seismic, 3/8" diameter screw anchor is acceptable with 3 1/4" minimum embedment.

**4.5.6 Bracing:**

By inspection, The bracing is the same construction as the standard bracing noted above for the Pallet Racks, therefore the bracing is acceptable for the Tire Rack Units.