

GARY K. MUNKELT & ASSOCIATES, LLC

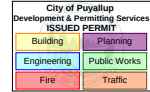
121 East Chestnut Street, Suite 101

Souderton, PA 18964

Phone: 215-855-8713

PRCTI20250452

These calculations must be on site and made available by the Permittee for all inspections.



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

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Storage Rack Bracing Checks

2 – Level Storage Racks (Merchandise Display):

Storage Rack Beam Checks

Storage Rack Upright / Post Base Stability Checks

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Storage Rack Bracing Checks

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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 in ink of contrasting color.

REPRODUCTIONS OF SIGNED COPIES ARE INVALID

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06/20/2025

9:13:58 AM



Date: 06/10/2025

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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

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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 May 07, 2025

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1.5 Loads:

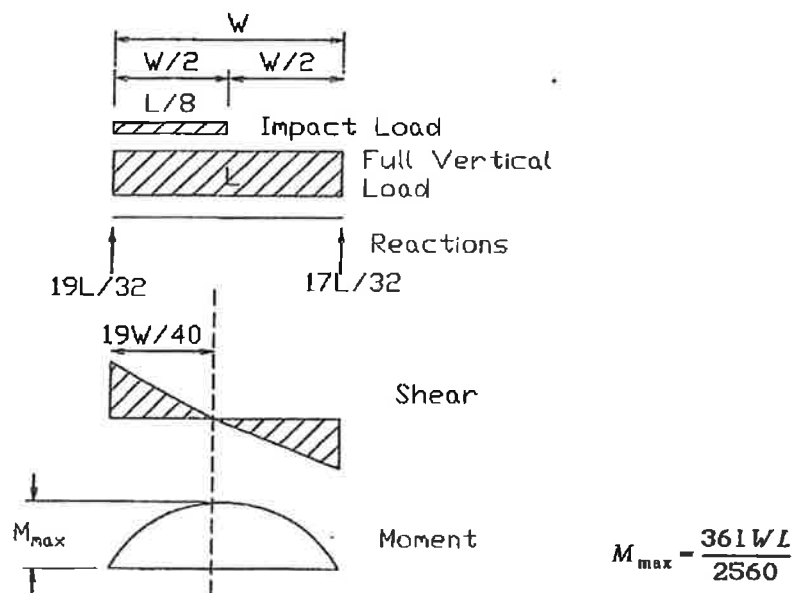
1.5.1 Vertical (Dead plus Live):

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 = 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.



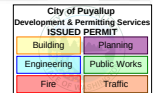
1.5.3 Seismic:

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:

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

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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:

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

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

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 Puyallup, WA are $S_s = 1.27$; $S_1 = 0.437$

$S_{ms} = 1.524$; $S_{m1} = 0.814$ per ASCE 7 Hazards

Seismic Design Category "D"

B may be conservatively taken from the following table:

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

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

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

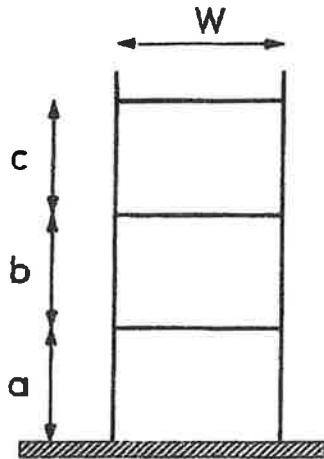
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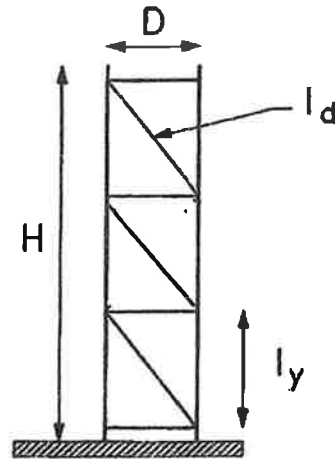
2 3 - Level Racks (Main Merchandise Area):

2.1 Components and Geometry:



W' = 105 "
a = 60 "
b = 60 "
c = 60 "

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



H = 180 "
D = 34 "
ly = 48 "
ld = 54 " (Bottom two diagonals, 61" top diagonal)

Upright Type - **UF-H-33I**

2.2 Check Beams:

2.2.1 Design Forces:

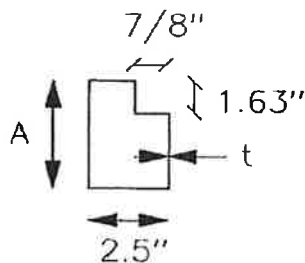
$$M \leq \frac{361 W L}{2560} =$$

51.8 inch kips factored

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

0.96 in³

2.2.2 Beam Properties:



RB-S-500

A = 5.00 in

t = 0.075 in

S_x = 1.40 in³

I_x = 3.51 in⁴

OK

OK



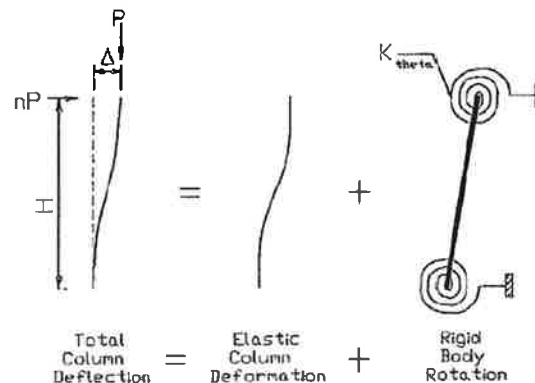
2.3 Check Posts (Dead plus Live Loads):

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:

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

K_{θ} 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:

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

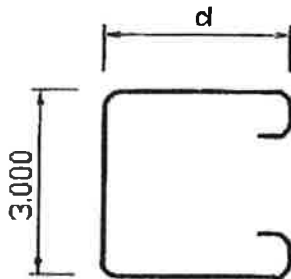
K_{conn} from tests or published literature

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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

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2.3.4 Post Demand, Capacity and Combined Stress Check Continued:

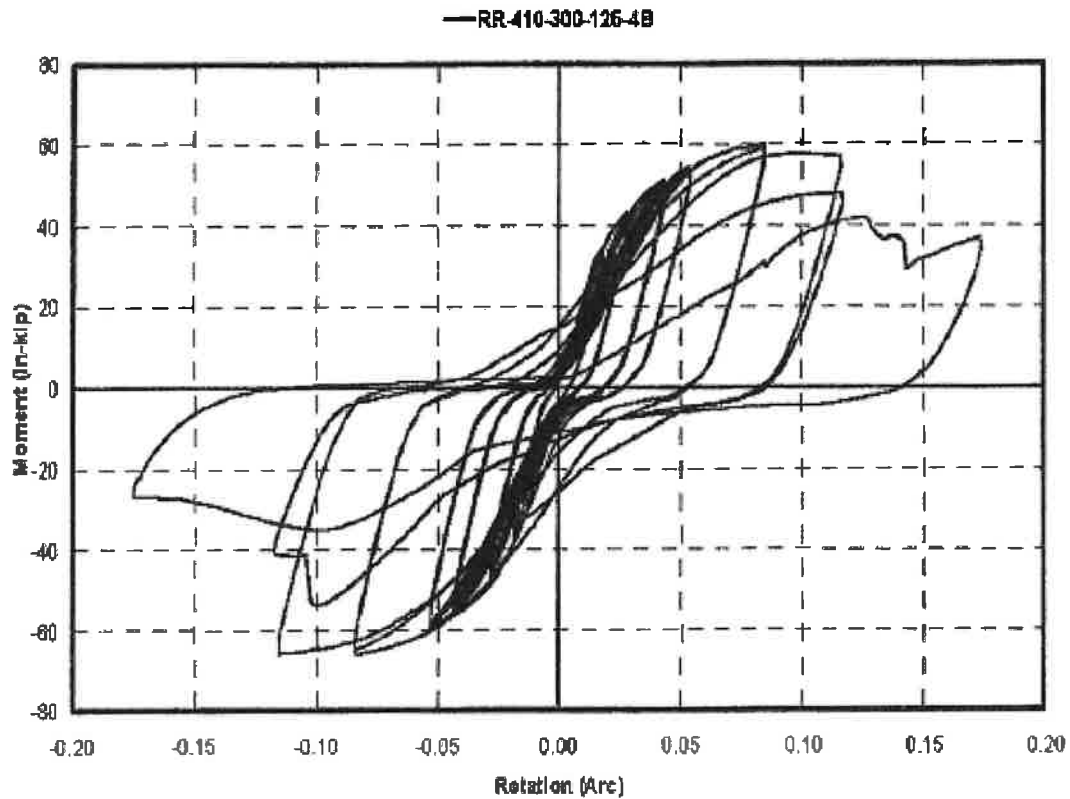
Combined Stress = 0.32

2.4 Longitudinal Seismic:

2.4.1 Connection Properties:

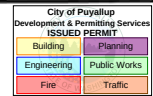
The following are test results from Ridg-U-Rak for the beam end connector stiffness based on RMI MH16.1 specifications for testing of beam end loading:

Combined Strength - Moment vs Rotation



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 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

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2.4.2 Determine Period of Structure Continued:

N_c = number of beam end connectors in the run of rack

R = response reduction factor from RMI

S_1 = Site seismic coefficient from ASCE7/RMI

S_{d1} = Design spectral response acceleration parameter = $S_{m1}(DBE/MCE)$

S_{ds} = Design spectral response acceleration parameter = $S_{ms}(DBE/MCE)$

S_{m1} = Site seismic coefficient (= $F_v S_1$)

S_{ms} = Site seismic coefficient (= $F_a S_s$)

S_s = Site seismic coefficient from ASCE7/RMI

T = structure period as computed at right

V_b = base shear for the entire storage rack as computed below

W = Total rack weight

W_{pi} = Weight at a given position in a run of rack

Defined Expressions

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

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

$$k_{bu} = \frac{k_c k_{ba}}{k_a + k_{ba}}$$

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

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 $I_e = 1.5$ 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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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

$$\text{Yields } T = 1.95 \text{ sec}$$

2.4.3 Displacement / Rotational Demand:

First order displacement demand:

$$D = 8.6T = 16.8 \text{ inches}$$

Second order effects:

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

Where:

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

Yielding:

$$D_{tot} = 20.8 \text{ inches}$$

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2.4.3 Displacement / Rotational Demand Continued:

Rotational Demand:

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

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:

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

For:

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

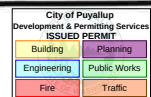
2.5.2 Displacement Demand:

Second order deflections are clearly negligible, yielding

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

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2.5.3 Displacement Capacity:

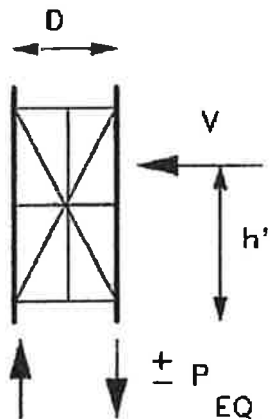
While the uprights and beams will deflect under lateral load, this displacement can be easily accommodated by pallet sliding. Movement of the pallets may included in the displacement capacity per ANSI MH16.1 section 2.6.2 Commentary.

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.

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



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

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

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

2.5.5 Post Capacity:

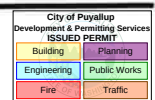
$$P + P_{eq} = 21.6 \text{ kips} < \phi P_n = 37.7 \text{ kips} \quad \underline{\text{OK}}$$

2.5.6 Stability:

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

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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
$\phi_{sa} n V_{saeq}$	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.	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
ϕ	none	Combined strength reduction under seismic per ACI 318 - 17.2.3.4.4 & $(0.75 \times 0.65) = 0.49$
ϕN_{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:

ψ_c, N	none	Per ACI 318 - Section 17.4.2.6 (post installed, $K_{cr} = 17$)
ψ_{ec}, N	none	Per ACI 318 - Section 17.4.2.4 (concentric load)
ψ_{ed}, N	none	Per ACI 318 - Section 17.4.2.5 (distance form edges)
$\delta_a =$	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	$S1 = 6.0$
$\phi N_{cbg} =$	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).

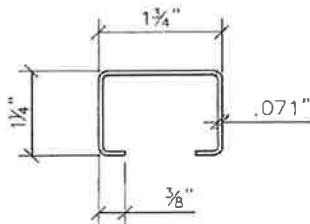
For High Seismic Areas use 1/2" diameter ICC approved external thread (Screw Anchors) with minimum effective embedment of 4". (5" Nominal Embedment to produce 4" effective embedment)

2.5.8 Brace:

2.5.8.1 Design Forces:

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

2.5.8.2 Brace Capacity:



$$\begin{aligned} A_{net} &= 0.32 \text{ in}^2 \\ r_{min} &= 0.46 \text{ in} \\ \phi &= 0.85 \text{ (AISI - Compression elements)} \\ k &= 0.80 \\ t &= 14ga \end{aligned}$$

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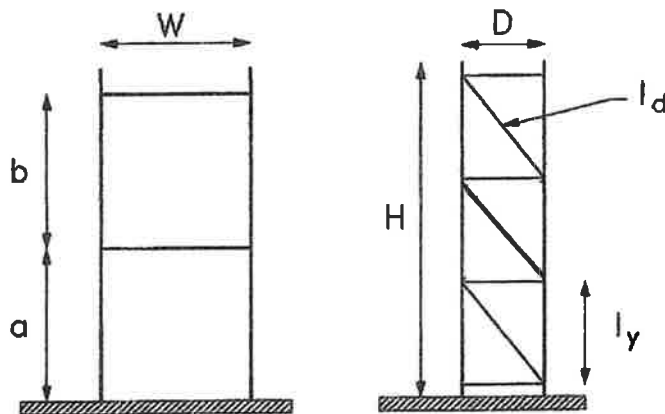


2.5.8.2 Brace Capacity Continued:

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

3 2 - Level Racks (Merchandise Display):

3.1 Components and Geometry:



$$W' = 105 \text{ "}$$

$$a - b = 96 \text{ " max}$$

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

$$H = 180 \text{ " (120" for the 2 Level Racks with same loading)}$$

$$D = 34 \text{ "}$$

$$I_y = 48 \text{ "}$$

$$I_d = 54 \text{ " (Bottom two diagonals, 61" top diagonal)}$$

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

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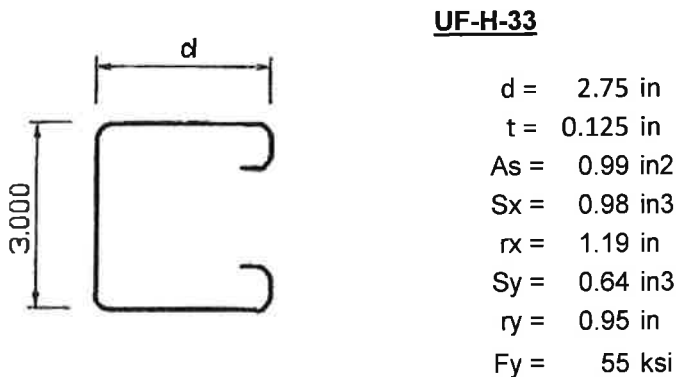
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3.3.2 Second Order Effects:

Refer to section 2.3.2 for derivation.

3.3.3 Post Properties (net section):



3.3.4 Post Demand, Capacity and Combined Stress Check:

Capacity

$$\begin{aligned} L_x &= 96 \text{ in} \\ L_y &= 48 \text{ in} \\ K/r_x &= 81 \\ K/r_y &= 51 \\ F_n &= 32.6 \text{ ksi} \\ \phi P_n &= 27.4 \text{ kips} \\ \phi M_{nx} &= 48.5 \text{ inch kips} \\ \phi M_{ny} &= 31.7 \text{ inch kips} \end{aligned}$$

Stability

$$\begin{aligned} K_{\text{conn}} &= 3000 \text{ in-kip / rad} \\ I_{\text{beam}} &= 3.51 \text{ in}^4 \quad \text{RB-S-500} \\ L_{\text{beam}} &= 102 \text{ in} \\ K_{\text{beam}} &= 5988 \text{ in-kip / rad} \\ K_{\text{theta}} &= 1999 \text{ in-kip / rad} \\ s &= 0.30 \end{aligned}$$

Demand

$$\begin{aligned} n &= 0.005 \\ P &= 7.0 \text{ kips} \end{aligned}$$

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3.3.4 Post Demand, Capacity and Combined Stress Check Continued:

Demand Continued

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

$$\text{Combined Stress} = 0.30$$

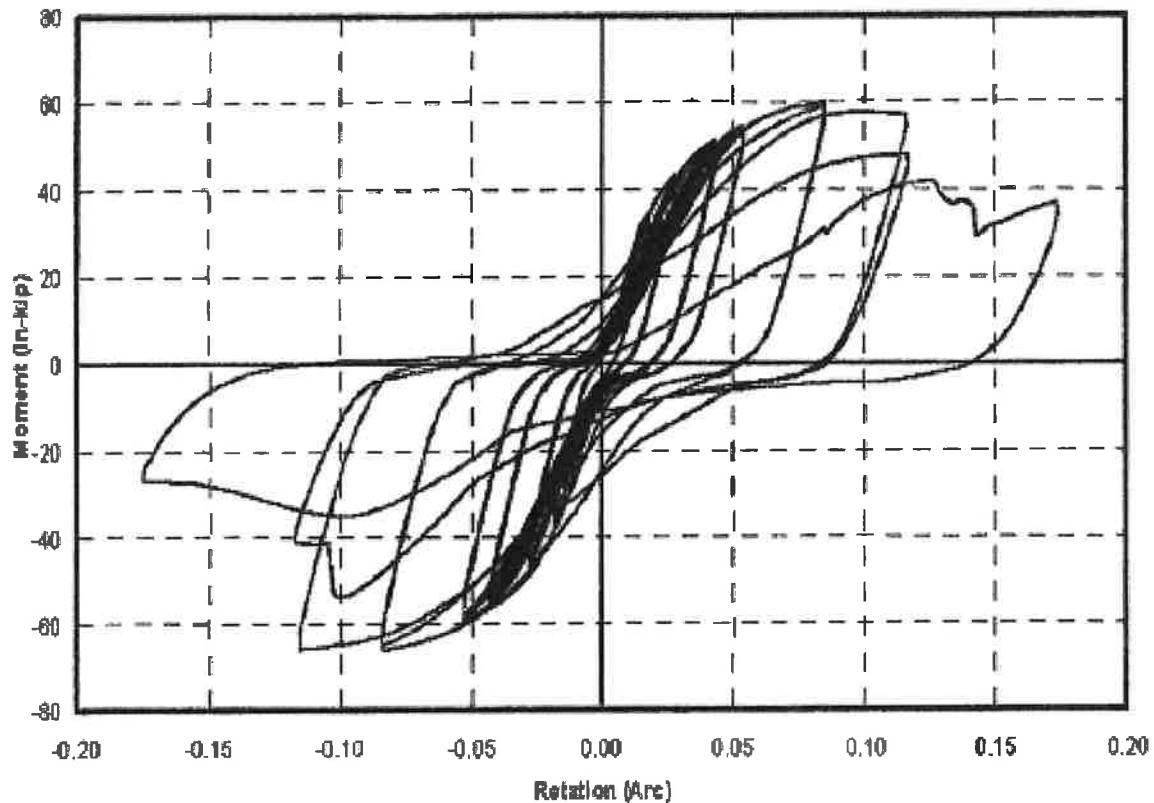
3.4 Longitudinal Seismic:

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-410-300-126-4B



Secant stiffness at 0.095 rad > 500 in-kips per radian

Maximum rotational capacity > 0.12 radians

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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:

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

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

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

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

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

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 $I_e = 1.5$ 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 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

$$\text{Yields } T = 2.13 \text{ sec}$$

3.4.3 Displacement / Rotational Demand:

First order displacement demand:

$$D = 8.6T = 18.4 \text{ inches}$$

Second order effects:

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

Where:

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

Yielding:

$$D_{tot} = 23.0 \text{ inches}$$

Rotational Demand:

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

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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:

$$\begin{aligned} D &= 34 \text{ in (depth of upright)} \\ I &< 0.8[2(D/2)^2 A_s] = 458 \text{ in}^4 \quad (\text{moment of inertia}) \\ W &= 10.0 \text{ kips (bay load)} \\ l = H &= 180 \text{ in (top beam level)} \\ T &= 0.19 \text{ sec} \end{aligned}$$

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.

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3.5.3 Displacement Capacity:

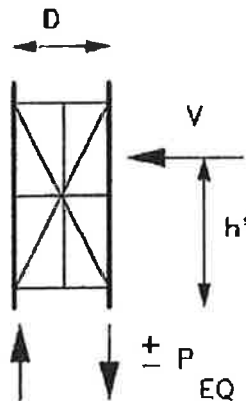
Per the above reference Degee paper cited above, the maximum (and very rare) pallet coefficient of friction on the steel is less than 0.6. For $\mu / \alpha < 0.4$, the pallets do not stick. Conservatively taking $\alpha = 0.6S_s = 1.7$, this ratio will not exceed $0.6 / 1.7 = 0.36 < 0.40$, so the pallets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

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

3.5.4 Stability Continued:



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

Use $h' =$ Uses top beam level of rack

$$h' = 180 \text{ inches}$$

$$OTM = Vh' = 360 \text{ inch kips}$$

$$RM = WD/2 = 170 \text{ inch kips (} W = 10.0 \text{ kips / bay)}$$

$$\text{Net OTM} = 190 \text{ inch kips}$$

$$P_{eq} = OTM / D = 10.6 \text{ kips}$$

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

3.5.5 Post Capacity:

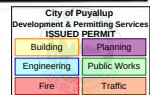
$$P + P_{eq} = 15.6 \text{ kips} < \phi P_n = 27.4 \text{ kips} \quad \underline{\text{OK}}$$

3.5.6 Stability:

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

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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)

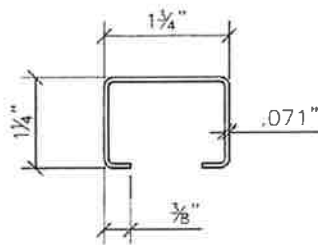
3.5.8 Brace:

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

3.5.8.1 Design Forces:

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

2.5.8.2 Brace Capacity:



$$\begin{aligned} A_{net} &= 0.32 \text{ in}^2 \\ r_{min} &= 0.46 \text{ in} \\ \phi &= 0.85 \text{ (AISI - Compression)} \\ k &= 0.80 \\ t &= 14ga \end{aligned}$$

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

OK