Structural Engineering & Design, Inc.

1815 Wright Ave
La Verne, CA 91750
Phone: 909.596.1351 Fax: 909.596.7186

Project Name : JOHNSON STORAGE AND MOVING

Project Number : 24-0917-10

Date : 09/20/24

Street Address: 2225 NE TOWNE CENTRE BLVD
City/State : LEE'S SUMMIT, MO 64064

Scope of Work : STORAGE RACK

STATE OF MISSOURI
ENHAO
ZHANG
Enhao Zhang
NUMBER
PE-2022045911
PROFESSIONAL ENGINEER

9/20/24

Structural

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By: Dray S   Project: ~~Johnson Storage~~   Project #: ~~24-0917-~~ 10

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Design Data

1) The analyses herein conforms to the requirements of the:
2018 IBC Section 2209

ANSI MH 16.1-2012 Specifications for the Design of Industrial Steel Storage Racks "2012 RMI Rack Design Manual"
ASCE 7-16, section 15.5.3

2) Transverse braced frame steel conforms to ASTM A570, Gr.55, with minimum strength, Fy=55 ksi
Longitudinal frame beam and connector steel conforms to ASTM A570, Gr.55, with minimum yield, Fy=55 ksi
All other steel conforms to ASTM A36, Gr. 36 with minimum yield, Fy= 36 ksi

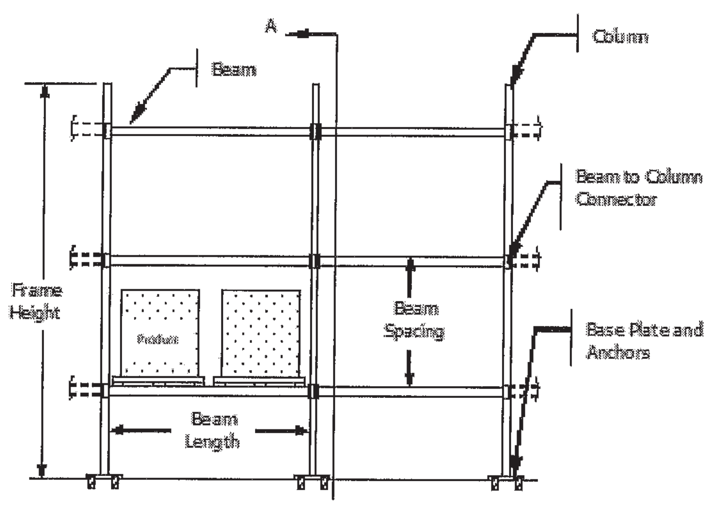
3) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein.

4) All welds shall conform to AWS procedures, utilizing E70xx electrodes or similar. All such welds shall be performed in shop, with no field welding allowed other than those supervised by a licensed deputy inspector.

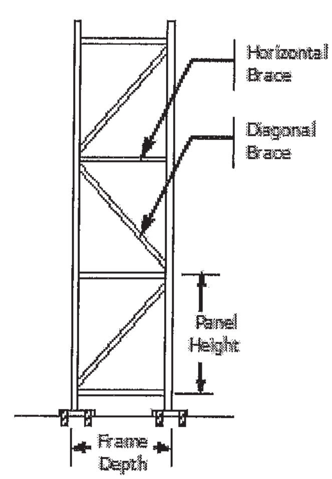
5) The existing slab on grade is 6" thick with minimum 4000 psi compressive strength. Allowable Soil bearing capacity is 750 psf. The design of the existing slab is by others.

6) Load combinations for rack components correspond to 2012 RMI Section 2.1 for ASD level load criteria

Definition of Components



Front View: Down Aisle (Longitudinal) Frame



Section A: Cross Aisle (Transverse) Frame

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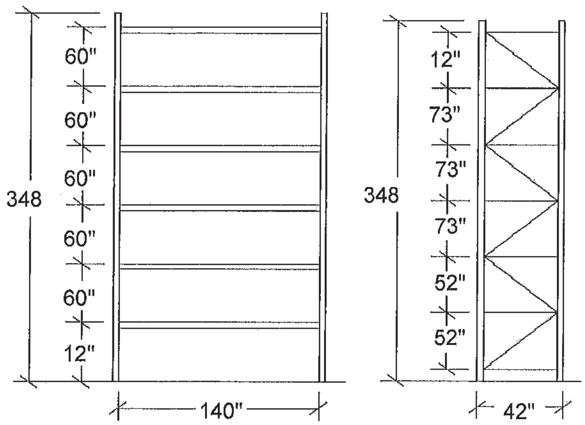
Configuration & Summary: TYPE A SELECTIVE RACK



**RACK COLUMN REACTIONS
ASD LOADS
AXIAL DL= 120 lb
AXIAL LL= 12,600 lb
SEISMIC AXIAL Ps=+/- 1,821 lb
BASE MOMENT= 0 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=0.099, Fa=1.6	6	42 in	348.0 in	6	140 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10600 lb, M=15318 in-lb		0.87-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	Intlk 50E 5Hx2.75Wx0.059"Thk	Lu=140 in	Capacity: 4309 lb/pr	0.97-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=13125 in-lb	Mcap=15230 in-lb	0.86-OK
Brace-Horizontal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.05-OK
Brace-Diagonal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.22-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.99-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=872 lb)			0.217-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.6-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
1	4,200 lb	12.0 in	52.0 in	6 lb	13 lb	12,720 lb	4,653 "#	9,794 "#	3 pin OK
2	4,200 lb	60.0 in	52.0 in	34 lb	77 lb	10,600 lb	15,318 "#	13,125 "#	3 pin OK
3	4,200 lb	60.0 in	73.0 in	61 lb	140 lb	8,480 lb	14,169 "#	11,985 "#	3 pin OK
4	4,200 lb	60.0 in	73.0 in	89 lb	204 lb	6,360 lb	12,063 "#	10,176 "#	3 pin OK
5	4,200 lb	60.0 in	73.0 in	117 lb	268 lb	4,240 lb	9,000 "#	7,697 "#	3 pin OK
6	4,200 lb	60.0 in	12.0 in	145 lb	332 lb	2,120 lb	4,979 "#	4,547 "#	3 pin OK

** Load defined as product weight per pair of beams Total: 452 lb 1,034 lb

Notes

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Seismic Forces   Configuration: TYPE A SELECTIVE RACK

Lateral analysis is performed with regard to the requirements of the 2012 RMI ANSI MH 16.1-2012 Sec 2.6 & ASCE 7-16 sec 15.5.3

Transverse (Cross Aisle) Seismic Load

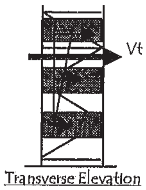
V= Cs*Ip*Ws=Cs*Ip*(0.67*P*Prf+D)

Cs1= Sds/R
= 0.0264
Cs2= 0.044*Sds
= 0.0046
Cs3= 0.5*S1/R
= 0.0085
Cs-max= 0.0264

Base Shear Coeff=Cs= 0.0264

Cs-max * Ip= 0.0264
V_{min}= 0.015
Eff Base Shear=Cs= 0.0264
Ws= (0.67*PL_{RF1} * PL)+DL (RMI 2.6.2)
= 17,124 lb

Vtransv=Vt= 0.0264 * (240 lb + 16884 lb)
Etransverse= 452 lb
Limit States Level Transverse seismic shear per upright



Ss= 0.099
S1= 0.068
Fa= 1.600
Fv= 2.400
Sds=2/3*Ss*Fa= 0.106
Sd1=2/3*S1*Fv= 0.109
Ca=0.4*2/3*Ss*Fa= 0.0422
(Transverse, Braced Frame Dir.) R= 4.0
Ip= 1.0
P_{RF1}= 1.0
Pallet Height=hp= 48.0 in
DL per Beam Lvl= 40 lb

Level	PRODUCT LOAD P	P*0.67*P_{RF1}	DL	hi	wi*hi	Fi	Fi*(hi+hp/2)
1	4,200 lb	2,814 lb	40 lb	12 in	34,248	5.6 lb	202-#
2	4,200 lb	2,814 lb	40 lb	72 in	205,488	33.5 lb	3,216-#
3	4,200 lb	2,814 lb	40 lb	132 in	376,728	61.4 lb	9,578-#
4	4,200 lb	2,814 lb	40 lb	192 in	547,968	89.3 lb	19,289-#
5	4,200 lb	2,814 lb	40 lb	252 in	719,208	117.2 lb	32,347-#
6	4,200 lb	2,814 lb	40 lb	312 in	890,448	145.1 lb	48,754-#
sum:	P=25200 lb	16,884 lb	240 lb	W=17124 lb	2,774,088	452 lb	Σ=113,386

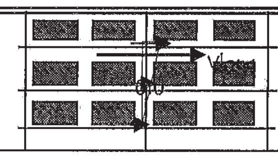
Longitudinal (Downaisle) Seismic Load

Similarly for longitudinal seismic loads, using R=6.0
Cs1=Sd1/(T*R)= 0.0604
Cs2= 0.0046
Cs3= 0.0057
Cs-max= 0.0604

Ws= (0.67 * PL_{RF2} * P) + DL
= 17,124 lb
Cs=Cs-max*Ip= 0.0604

P_{RF2}= 1.0
(Longitudinal, Unbraced Dir.) R= 6.0
T= 0.30 sec

Vlong= 0.0604 * (240 lb + 16884 lb)
Elongitudinal= 1,034 lb *Limit States Level Longit. seismic shear per upright*



Level	PRODUC LOAD P	P*0.67*P_{RF2}	DL	hi	wi*hi	Fi
1	4,200 lb	2,814 lb	40 lb	12 in	34,248	12.8 lb
2	4,200 lb	2,814 lb	40 lb	72 in	205,488	76.6 lb
3	4,200 lb	2,814 lb	40 lb	132 in	376,728	140.4 lb
4	4,200 lb	2,814 lb	40 lb	192 in	547,968	204.2 lb
5	4,200 lb	2,814 lb	40 lb	252 in	719,208	268.1 lb
6	4,200 lb	2,814 lb	40 lb	312 in	890,448	331.9 lb
sum:		16,884 lb	240 lb	W=17124 lb	2,774,088	1,034 lb

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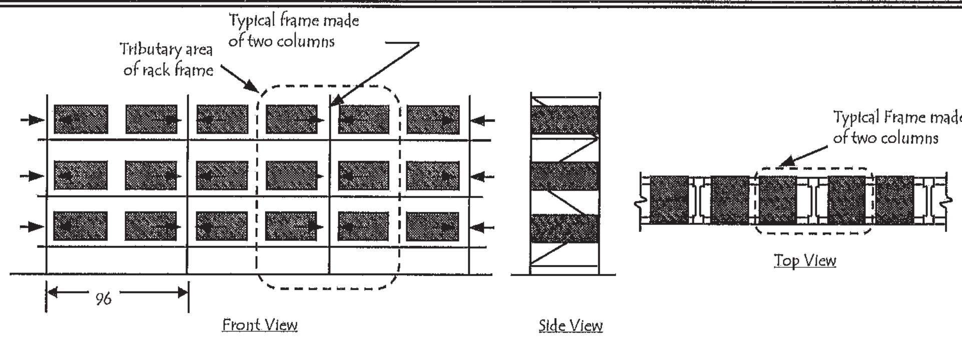
Downaisle Seismic Loads Configuration: TYPE A SELECTIVE RACK

Determine the story moments by applying portal analysis. The base plate is assumed to provide no fixity.

Seismic Story Forces

Vlong= 1,034 lb
Vcol=Vlong/2= 517 lb
F1= 13 lb
F2= 77 lb
F3= 140 lb



Conceptual System

Seismic Story Moments COL

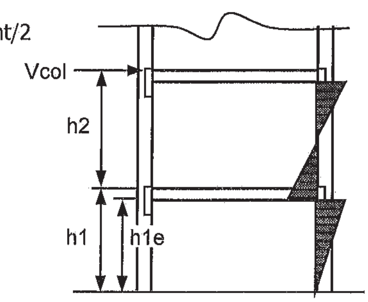
Mbase-max= 0 in-lb *<=== Default capacity*
Mbase-v= (Vcol*h1eff)/2
= 2,327 in-lb *<=== Moment going to base*
Mbase-eff= Minimum of Mbase-max and Mbase-v
= 0 in-lb *PINNED BASE ASSUMED*

h1-eff= h1 - beam clip height/2
= 9 in

M 1-1= [Vcol * h1eff]-Mbase-eff
= (517 lb * 9 in)-0 in-lb
= 4,653 in-lb

M 2-2= [Vcol-(F1)/2] * h2
= [517 lb - 38.3 lb]*60 in/2
= 15,318 in-lb

Mseis= (Mupper+Mlower)/2



Beam to Column

Mseis(1-1)= (4653 in-lb + 15318 in-lb)/2
= 9,986 in-lb

Mseis(2-2)= (15318 in-lb + 14169 in-lb)/2
= 14,744 in-lb

rho= 1.0000

Summary of Forces

LEVEL	hi	Axial Load	Column Moment**	Mseismic**	Mend-fixity	Mconn**	Beam Connector
1	12 in	12,720 lb	4,653 in-lb	9,986 in-lb	4,006 in-lb	9,794 in-lb	3 pin OK
2	60 in	10,600 lb	15,318 in-lb	14,744 in-lb	4,006 in-lb	13,125 in-lb	3 pin OK
3	60 in	8,480 lb	14,169 in-lb	13,116 in-lb	4,006 in-lb	11,985 in-lb	3 pin OK
4	60 in	6,360 lb	12,063 in-lb	10,532 in-lb	4,006 in-lb	10,176 in-lb	3 pin OK
5	60 in	4,240 lb	9,000 in-lb	6,989 in-lb	4,006 in-lb	7,697 in-lb	3 pin OK
6	60 in	2,120 lb	4,979 in-lb	2,489 in-lb	4,006 in-lb	4,547 in-lb	3 pin OK

Mconn= (Mseismic + Mend-fixity)*0.70*rho
Mconn-allow(3 Pin)= 15,230 in-lb

**all moments based on limit states level loading

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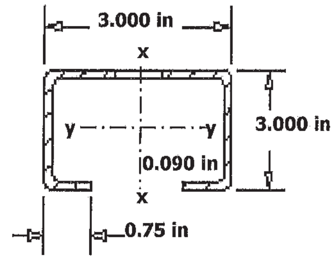
Column (Longitudinal Loads)   **Configuration: TYPE A SELECTIVE RACK**

Section Properties

Section: INTLK LU75/3x3x13ga

Aeff = 0.757 in^2	Iy = 0.871 in^4	Kx = 1.7
Ix = 1.320 in^4	Sy = 0.574 in^3	Lx = 57.5 in
Sx = 0.879 in^3	ry = 1.080 in	Ky = 1.0
rx = 1.320 in	Fy= 55 ksi	Ly = 52.0 in
Ωf= 1.67	Cmx= 0.85	Cb= 1.0
E= 29,500 ksi		



Loads   Considers loads at level 2

COLUMN DL= 100 lb
COLUMN PL= 10,500 lb
Mcol= 15,318 in-lb
Sds= 0.1056
1+0.105*Sds= 1.0111
1.4+0.14Sds= 1.4148
1+0.14Sds= 1.0148
0.85+0.14*Sds= 0.8648
B= 0.7000
rho= 1.0000

Critical load cases are: RMI Sec 2.1
*Load Case 5: : (1+0.105*Sds)D + 0.75*(1.4+0.14Sds)*B*P + 0.75*(0.7*rho*E)<= 1.0, ASD Method*
*axial load coeff: 0.7427616 * P*   *seismic moment coeff: 0.5625 * Mcol*
*Load Case 6: : (1+0.14*Sds)D + (0.85+0.14Sds)*B*P + (0.7*rho*E)<= 1.0, ASD Method*
axial load coeff: 0.60535   *seismic moment coeff: 0.7 * Mcol*

By analysis, Load case 5 governs utilizing loads as such

Axial Load=Pax= 1.011088*100 lb + 0.75 * 1.414784 * 0.7 * 10500 lb
= 7,900 lb

Moment=Mx= 0.75*0.7*rho*Mcol
= 0.525*15318 in-lb
= 8,042 in-lb

Axial Analysis

KxLx/rx = 1.7*57.5"/1.3196"
= 74.1

KyLy/ry = 1*52"/1.08"
= 48.1

Fe= π^2E/(KL/r)max^2
= 53.1ksi

Fy/2= 27.5 ksi

Pn= Aeff*Fn
= 30,846 lb

Ωc= 1.92

Fe > Fy/2
Fn= Fy(1-Fy/4Fe)
= 55 ksi*[1-55 ksi/(4*53.1 ksi)]
= 40.7 ksi
Pa= Pn/Ωc
= 30846 lb/1.92
= 16,066 lb

P/Pa= 0.49 > 0.15

Bending Analysis

Check: Pax/Pa + (Cmx*Mx)/(Max*µx) ≤ 1.0
P/Pao + Mx/Max ≤ 1.0

Pno= Ae*Fy
= 0.757 in^2 *55000 psi
= 41,635 lb

Pao= Pno/Ωc
= 41635lb/1.92
= 21,685 lb

Myield=My= Sx*Fy
= 0.879 in^3 * 55000 psi
= 48,345 in-lb

Max= My/Ωf
= 48345 in-lb/1.67
= 28,949 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500 ksi/(1.7*57.5 in)^2
= 40,222 lb

µx= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*7900 lb/40222 lb)]}^-1
= 0.62

Combined Stresses

(7900 lb/16066 lb) + (0.85*8042 in-lb)/(28949 in-lb*0.62) =	0.87	< 1.0, OK	(EQ C5-1)
(7900 lb/21685 lb) + (8042 in-lb/28949 in-lb) =	0.64	< 1.0, OK	(EQ C5-2)

*** For comparison, total column stress computed for load case 6 is: 70.0%*   *zing loads 6457.6408 lb Axial and M= 10722 in-lb*

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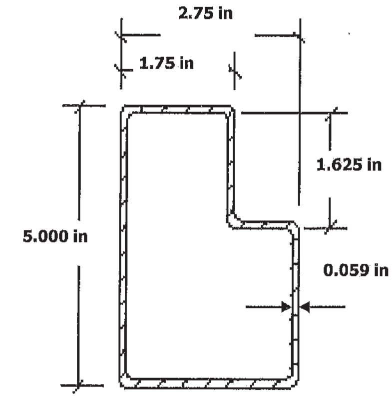
BEAM Configuration: TYPE A SELECTIVE RACK

DETERMINE ALLOWABLE MOMENT CAPACITY



A) Check compression flange for local buckling (B2.1)

w= c - 2*t -2*r
= 1.75 in - 2*0.059 in - 2*0.059 in
= 1.514 in
w/t= 25.66
l=lambda= [1.052/(k)^0.5] * (w/t) * (Fy/E)^0.5 Eq. B2.1-4
= [1.052/(4)^0.5] * 25.66 * (55/29500)^0.5
= 0.583 < 0.673, Flange is fully effective Eq. B2.1-1

B) check web for local buckling per section b2.3

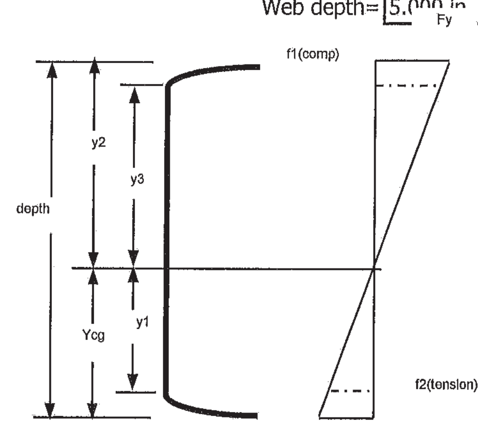
f1(comp)= Fy*(y3/y2)= 51.18 ksi
f2(tension)= Fy*(y1/y2)= 102.95 ksi
Y= f2/f1 Eq. B2.3-5
= -2.012
k= 4 + 2*(1-Y)^3 + 2*(1-Y) Eq. B2.3-4
= 64.67
flat depth=w= y1+y3
= 4.764 in w/t= 80.74576271 OK
l=lambda= [1.052/(k)^0.5] * (w/t) * (f1/E)^0.5
= [1.052/(64.67)^0.5] * 4.764 * (51.18/29500)^0.5
= 0.44 < 0.673
be=w= 4.764 in b2= be/2 Eq B2.3-2
b1= be(3-Y) = 2.38 in
= 0.951
b1+b2= 3.331 in > 1.582 in, Web is fully effective

Beam= **Intlk 50E 5Hx2.75Wx0.059"Thk**

Ix=	2.916 in^4
Sx=	1.114 in^3
Ycg=	3.300 in
t=	0.059 in
Bend Radius=r=	0.059 in
Fy=Fyv=	55.00 ksi
Fu=Fuv=	65.00 ksi
E=	29500 ksi
top flange=b=	1.750 in
bottom flange=	2.750 in
Web depth=	5.000 in

Determine effect of cold working on steel yield point (Fya) per section A7.2

Fya= C*Fyc + (1-C)*Fy (EQ A7.2-1)
Lcorner=Lc= (p/2) * (r + t/2)
0.139 in C= 2*Lc/(Lf+2*Lc)
Lflange-top=Lf= 1.514 in = 0.155 in
m= 0.192*(Fu/Fy) - 0.068 (EQ A7.2-4)
= 0.1590
Bc= 3.69*(Fu/Fy) - 0.819*(Fu/Fy)^2 - 1.79 (EQ A7.2-3)
= 1.427
since fu/Fv= 1.18 < 1.2
and r/t= 1 < 7 OK
then Fyc= Bc * Fy/(R/t)^m (EQ A7.2-2)
= 78.485 ksi
Thus, Fya-top= 58.64 ksi (tension stress at top)
Fya-bottom= Fya*Ycg/(depth -Ycg)
= 113.84 ksi (tension stress at bottom)



y1= Ycg-t-r= 3.182 in
y2= depth-Ycg= 1.700 in
y3= y2-t-r= 1.582 in

Check allowable tension stress for bottom flange

Lflange-bot=Lfb= Lbottom - 2*r*-2*t
= 2.514 in
Cbottom=Cb= 2*Lc/(Lfb+2*Lc)
= 0.100
Fy-bottom=Fyb= Cb*Fyc + (1-Cb)*Fyf
= 57.34 ksi
Fya= (Fya-top)*(Fyb/Fya-bottom)
= 29.54 ksi
if F= 0.95 Then F*Mn=F*Fya*Sx= $\boxed{\text{31.26 in-k}}$

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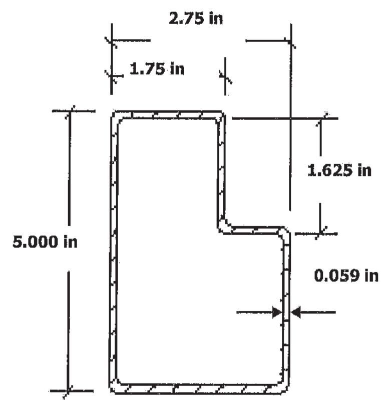
BEAM   Configuration: TYPE A SELECTIVE RACK

RMI Section 5.2, PT II

Section

Beam= Intlk 50E 5Hx2.75Wx0.059"Thk
Ix=Ib= 2.916 in^4
Sx= 1.114 in^3
t= 0.059 in   E= 29500 ksi
Fy=Fyv= 55 ksi   F= 240.0
Fu=Fuv= 65 ksi   L= 140 in
Fya= 58.6 ksi   Beam Level= 1
P=Product Load= 4,200 lb/pair
D=Dead Load= 40 lb/pair

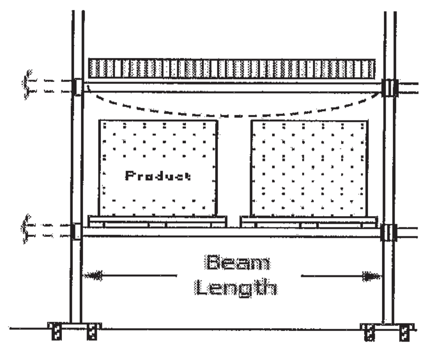


1. Check Bending Stress Allowable Loads

Mcenter=F*Mn= W*L*W*Rm/8

W=LRFD Load Factor= 1.2*D + 1.4*P+1.4*(0.125)*P	***RMI 2.2, item 8***
FOR DL=2% of PL,	

W= 1.599



Rm= 1 - [(2*F*L)/(6*E*Ib + 3*F*L)]
1 - (2*240*140 in)/[(6*29500 ksi*2.9155 in^3)+(3*240*140 in)]
= 0.891
if F= 0.95
Then F*Mn=F*Fya*Sx= 62.06 in-k
Thus, allowable load
per beam pair=W= F*Mn*8*(# of beams)/(L*Rm*W)
= 62.06 in-k * 8 * 2/(140in * 0.891 * 1.599)
= 4,979 lb/pair ***allowable load based on bending stress***

Mend= W*L*(1-Rm)/8
= (4979 lb/2) * 140 in * (1-0.891)/8
= 4,749 in-lb   @ 4979 lb max allowable load
= 4,006 in-lb   **@ 4200 lb imposed product load**

2. Check Deflection Stress Allowable Loads

Dmax= Dss*Rd
Rd= 1 - (4*F*L)/(5*F*L + 10*E*Ib)
= 1 - (4*240*140 in)/[(5*240*140 in)+(10*29500 ksi*2.9155 in^4)]
= 0.869 in
if Dmax= L/180   *Based on L/180 Deflection Criteria*
and Dss= 5*W*L^3/(384*E*Ib)

Allowable Deflection= L/180
= 0.778 in
Deflection at imposed Load= 0.758 in

L/180= 5*W*L^3*Rd/(384*E*Ib*# of beams)
solving for W yields,
W= 384*E*I*2/(180*5*L^2*Rd)
= 384*2.9155 in^4*2/[180*5*(140 in)^2*0.869)
= 4,309 lb/pair ***allowable load based on deflection limits***

Thus, based on the least capacity of item 1 and 2 above:

Allowable load=	**4,309 lb/pair**
Imposed Product Load=	**4,200 lb/pair**
Beam Stress=	**0.97**

Beam at Level 1

8.2

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1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

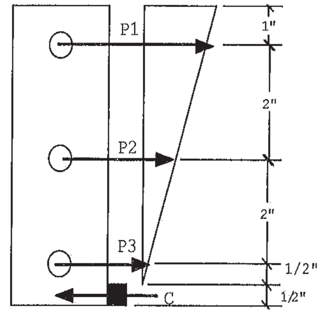
By Dray S | Project: ~~Johnson Storage~~ | Project #: ~~24-0917-~~ 10

3 Pin Beam to Column Connection TYPE A SELECTIVE RACK

The beam end moments shown herein show the result of the maximum induced fixed end moments form seismic + static loads and the code mandated minimum value of 1.5%(DL+PL)

Mconn max= (Mseismic + Mend-fixity)*0.70*Rho
= 13,125 in-lb Load at level 2

rho= 1.0000



Connector Type= 3 Pin

Shear Capacity of Pin

Pin Diam= 0.44 in Fy= 55,000 psi

Ashear= (0.438 in)^2 * Pi/4
= 0.1507 in^2

Pshear= 0.4 * Fy * Ashear
= 0.4 * 55000 psi * 0.1507in^2
= 3,315 lb

Bearing Capacity of Pin

tcol= 0.090 in Fu= 65,000 psi

Omega= 2.22 a= 2.22

Pbearing= alpha * Fu * diam * tcol/Omega
= 2.22 * 65000 psi * 0.438 in * 0.09 in/2.22
= 2,562 lb < 3315 lb

Moment Capacity of Bracket

Edge Distance=E= 1.00 in Pin Spacing= 2.0 in Fy= 55,000 psi

C= P1+P2+P3 tclip= 0.18 in Sclip= 0.127 in^3
= P1+P1*(2.5"/4.5")+P1*(0.5"/4.5")
= 1.667 * P1

Mcap= Sclip * Fbending C*d= Mcap = 1.667 d= E/2
= 0.127 in^3 * 0.66 * Fy = 0.50 in
= 4,610 in-lb

Pclip= Mcap/(1.667 * d)
= 4610.1 in-lb/(1.667 * 0.5 in) Thus, P1= 2,562 lb
= 5,531 lb

Mconn-allow= [P1*4.5"+P1*(2.5"/4.5")*2.5"+P1*(0.5"/4.5")*0.5"]
= 2562 LB*[4.5"+(2.5"/4.5")*2.5"+ (0.5"/4.5")*0.5"]
= 15,230 in-lb > Mconn max, OK

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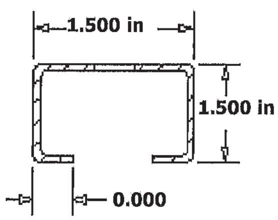
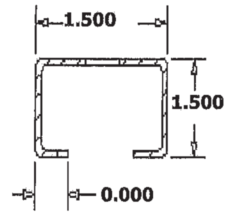
Transverse Brace Configuration: TYPE A SELECTIVE RACK

Section Properties

Diagonal Member= Intlk 1-1/2x1-1/2x14ga
Area= 0.315 in^2
r min= 0.488 in
Fy= 55,000 psi
K= 1.0
Ωc= 1.92



Horizontal Member= Intlk 1-1/2x1-1/2x14ga
Area= 0.315 in^2
r min= 0.488 in
Fy= 55,000 psi
K= 1.0



Frame Dimensions

Bottom Panel Height=H= 73.0 in
Frame Depth=D= 42.0 in
Column Width=B= 3.0 in

Clear Depth=D-B*2= 36.0 in
X Brace= NO
rho= 1.00

Diagonal Member

*Load Case 6: : ~~(1+0.104*Sds)D +~~ 0 [(0.85+0.14Sds)*B*P + [0.7*rho*E]<= 1.0, ASD Method*

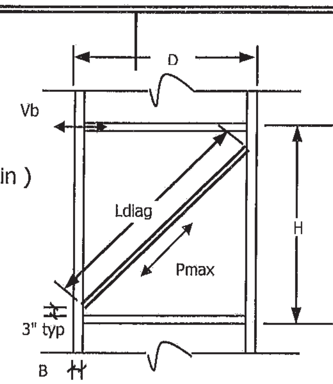
Vtransverse= 452 lb
Vb=Vtransv*0.7*rho= 452 lb * 0.7 * 1
= 316 lb
Ldiag= [(D-B*2)^2 + (H-6")^2]^1/2
= 76.1 in

Pmax= V*(Ldiag/D) * 0.75
= 430 lb
axial load on diagonal brace member

Pn= AREA*Fn
= 0.315 in^2 * 11979 psi
= 3,773 lb

(kl/r)= (k * Ldiag)/r min
= (1 x 76.1 in /0.488 in)
= 155.9 in
Fe= pi^2*E/(kl/r)^2
= 11,979 psi

Since Fe<Fy/2,
Fn= Fe
= 11,979 psi



Typical Panel Configuration

Pallow= Pn/Ω
= 3773 lb /1.92
= 1,965 lb

Pn/Pallow= 0.22 <= 1.0 OK

Check End Weld
Lweld= 3.0 in
Fu= 65 ksi
tmin= 0.075 in
Weld Capacity= 0.75 * tmin * L * Fu/2.5
= 4,388 lb OK

Horizontal brace

Vb=Vtransv*0.7*rho= 316 lb

(kl/r)= (k * Lhoriz)/r min
= (1 x 42 in) /0.488 in
= 86.1 in
Since Fe>Fy/2, Fn=Fy*(1-fy/4fe)
= 35,745 psi

Fe= pi^2*E/(kl/r)^2
= 39,275 psi

Pn= AREA*Fn
= 0.315in^2*35745 psi
= 11,260 lb

Fy/2= 27,500 psi

Pallow= Pn/Ωc
= 11260 lb /1.92
= 5,864 lb

Pn/Pallow= 0.05 <= 1.0 OK

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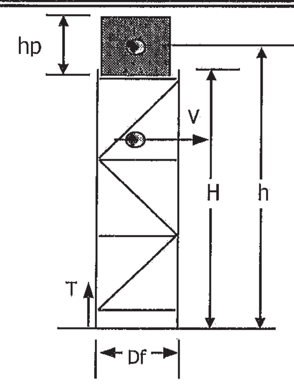
By: Dray S   Project: ~~Johnson Storage~~   Project #: ~~24-0917-10~~

Single Row Frame Overturning   Configuration: TYPE A SELECTIVE RACK

Loads

Critical Load case(s):
1) RMI Sec 2.2, item 7: (0.9-0.2Sds)D + (0.9-0.20Sds)*B*Papp - E*rho



SIDE ELEVATION

		Sds=	0.1056
Vtrans=V=E=Qe=	452 lb	(0.9-0.2Sds)=	0.8789
DEAD LOAD PER UPRIGHT=D=	240 lb	(0.9-0.2Sds)=	0.8789
PRODUCT LOAD PER UPRIGHT=P=	25,200 lb	B=	1.0000
Papp=P*0.67=	16,884 lb	rho=	1.0000
Wst LC1=Wst1=(0.87888*D + 0.87888*Papp*1)=	15,049 lb		
		Frame Depth=Df=	42.0 in
Product Load Top Level, Ptop=	4,200 lb	Htop-lvl=H=	312.0 in
DL/Lvl=	40 lb	# Levels=	6
Seismic Ovt based on E, Σ(Fi*hi)=	76,494 in-lb	# Anchors/Base=	2
height/depth ratio=	7.4 in	hp=	48.0 in
		h=H+hp/2=	336.0 in

A) Fully Loaded Rack

Load case 1:

Movt= Σ(Fi*hi)*E*rho
= 76,494 in-lb

Mst= Wst1 * Df/2
= 15049 lb * 42 in/2
= 316,029 in-lb

T= (Movt-Mst)/Df
= (76494 in-lb - 316029 in-lb)/42 in
= -5,703 lb   **No Uplift**

Net Seismic Uplift= -5,703 lb

B) Top Level Loaded Only

Load case 1:

V1=Vtop= Cs * Ip * Ptop >= 350 lb for H/D >6.0
= 0.0264 * 4200 lb
= 111 lb
V1eff= 350 lb
V2=V_{DL}= Cs*Ip*D
= 6 lb
Mst= (0.87888*D + 0.87888*Ptop*1) * 42 in/2
= 81,947 in-lb

Critical Level= 6
Cs*Ip= 0.0264

Movt= [V1*h + V2 * H/2]*rho
= 118,588 in-lb
T= (Movt-Mst)/Df
= (118588 in-lb - 81947 in-lb)/42 in
= 872 lb   **Net Uplift per Column**

Net Seismic Uplift= 872 lb

Anchor

Check (2) 0.5" x 3.25" Embed POWERS SD1 anchor(s) per base plate.
Special inspection is notrequired per ESR 2818.

Pullout Capacity=Tcap=	1,961 lb	L.A. City Jurisdiction? NO		Tcap*Phi=	1,961 lb
Shear Capacity=Vcap=	2,517 lb	Phi= 1		Vcap*Phi=	2,517 lb
Fully Loaded:		(113 lb/2517 lb)^1 =	0.04	<= 1.2	OK
Top Level Loaded:		(436 lb/1961 lb)^1 + (87 lb/2517 lb)^1 =	0.26	<= 1.2	OK

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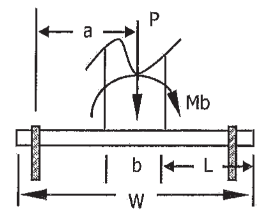
Base Plate Configuration: TYPE A SELECTIVE RACK

Section



Downaisle Elevation

Baseplate= 7.75x5x0.375
Eff Width=W = 7.15 in   a = 2.58 in
Eff Depth=D = 5.00 in   Anchor c.c. =2*a=d = 5.15 in
Column Width=b = 3.00 in   N=# Anchor/Base= 2
Column Depth=dc = 3.00 in   Fy = 36,000 psi
L = 2.08 in
Plate Thickness=t = 0.375 in

Down Aisle Loads *Load Case 5: : (1+0.105*Sds)D + 0.75*[(1.4+0.14Sds)*B*P + 0.75*[0.7*rho*E]<= 1.0, ASD Method*

COLUMN DL= 120 lb   **Axial=P= 1.011088 * 120 lb + 0.75 * (1.414784 * 0.7 * 12600 lb)**
COLUMN PL= 12,600 lb   **= 9,480 lb**
Base Moment= 0 in-lb   **Mb= Base Moment*0.75*0.7*rho**
1+0.105*Sds= 1.0111   **= 0 in-lb * 0.75*0.7*rho**
1.4+0.14Sds= 1.4148   **= 0 in-lb**
B= 0.7000   **Axial Load P = 9,480 lb   Mbase=Mb = 0 in-lb**

Axial stress=fa = P/A = P/(D*W)   M1= wL^2/2= fa*L^2/2
= 265 psi   = 571 in-lb
Moment Stress=fb = M/S = 6*Mb/[(D*B^2]   Moment Stress=fb2 = 2 * fb * L/W
= 0.0 psi   = 0.0 psi
Moment Stress=fb1 = fb-fb2   M2= fb1*L^2)/2
= 0.0 psi   = 0 in-lb
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2   Mtotal = M1+M2+M3
= 0 in-lb   = 571 in-lb/in
S-plate = (1)(t^2)/6   Fb = 0.75*Fy
= 0.023 in^3/in   = 27,000 psi
fb/Fb = Mtotal/[(S-plate)(Fb)]   F'p= 0.7*F'c
= **0.90** **OK**   = 2,800 psi OK
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2]   Tallow= 1,961 lb **OK**
= -7,525 lb No Tension

Cross Aisle Loads *Critical load case RMI Sec 2.1, item 4: (1+0.11Sds)DL + (1+0.14SDS)PL*0.75+EL*0.75 <= 1.0, ASD Method*

Pstatic= 9,480 lb

Movt*0.75*0.7*rho= 40,159 in-lb   Pseismic= Movt/Frame Depth
Frame Depth= 42.0 in   = 956 lb
P=Pstatic+Pseismic= 10,436 lb
b =Column Depth= 3.00 in

L =Base Plate Depth-Col Depth= 2.08 in

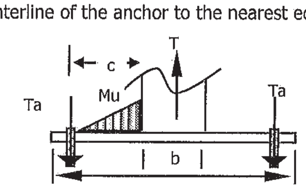
fa = P/A = P/(D*W)   M= wL^2/2= fa*L^2/2
= 292 psi   = 628 in-lb/in

Sbase/in = (1)(t^2)/6   Fbase = 0.75*Fy
= 0.023 in^3/in   = 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.99** **OK**

Check uplift load on Baseplate

Check uplift forces on baseplate with 2 or more anchors per RMI 7.2.2.
"When the base plate configuration consists of two anchor bolts located on either side of the column and a net uplift force exists, the minimum base plate thickness shall be determined based on a design bending moment in the plate equal to the uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column"



Elevation

Uplift per Column= 872 lb
Qty Anchor per BP= 2
Net Tension per anchor=Ta= 436 lb
c= 2.08 in
Mu=Moment on Baseplate due to uplift= Ta*c/2
= 452 in-lb
Splate= 0.117 in^3
[fb/Fb]*0.75= 0.107 **OK**

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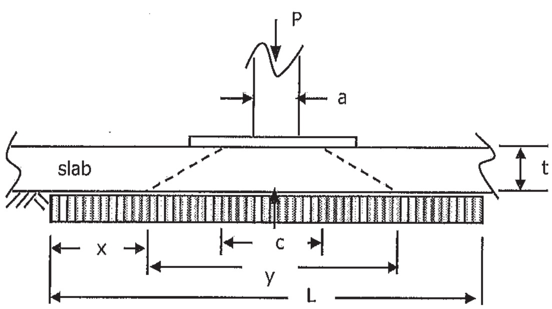
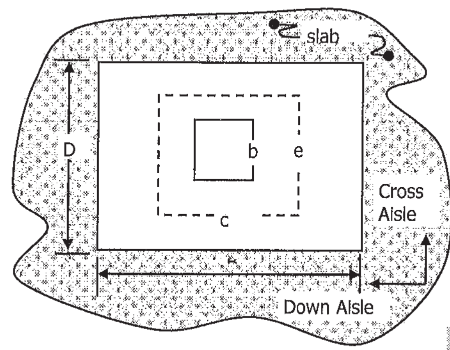
By: Project: Project #:

Slab on Grade Configuration: TYPE 1 SELECTIVE RACK



SLAB ELEVATION



Baseplate Plan View

Concrete
f'c= 4,000 psi
tslab=t= 6.0 in
teff= 6.0 in
phi=Ø= 0.6

Soil
fsoil= 750 psf
Movt= 76,494 in-lb
Frame depth= 42.0 in
Sds= 0.106
0.2*Sds= 0.021
λ= 0.600
β=B/Δ= 1.430
F'c^0.5= 63.20 psi
ks= 50.00 psi
R1= 5.0 in
Ec= 3,443,668 psi
u= 0.15

Base Plate
Effec. Baseplate width=B= 7.15 in
Effec. Baseplate Depth=D= 5.00 in
width=a= 3.00 in
depth=b= 3.00 in
midway dist face of column to edge of plate=c= 5.08 in
midway dist face of column to edge of plate=e= 4.00 in

Column Loads

DEAD LOAD=D= 120 lb per column
unfactored ASD load
PRODUCT LOAD=P= 12,600 lb per column
unfactored ASD load
Papp= 8,442 lb per column
P-seismic=E= (Movt/Frame depth)
= 1,821 lb per column
unfactored Limit State load
B= 0.7000
rho= 1.0000
Sds= 0.1056
1.2 + 0.2*Sds= 1.2211
0. 9 - 0.20Sds= 0.8789

Load Case 1) (1.2+0.2Sds)D + (1.2+0.2Sds)*B*P+ rho*E RMI SEC 2.2 EQTN 5
= 1.22112 * 120 lb + 1.22112 * 0.7 * 12600 lb + 1 * 1821 lb
= 12,738 lb
Load Case 2) (0.9-0.2Sds)D + (0.9-0.2Sds)*B*Papp + rho*E RMI SEC 2.2 EQTN 7
= 0.87888 * 120 lb + 0.87888 * 0.7 * 8442 lb + 1 * 1821 lb
= 7,120 lb
Load Case 3) 1.2*D + 1.4*P RMI SEC 2.2 EQTN 1,2
= 1.2*120 lb + 1.4*12600 lb
= 17,784 lb
Load Case 4) 1.2*D + 1.0*P + 1.0E ACI 318-11 Sec 9.2.1, Eqtn 9-5
= 14,565 lb

Effective Column Load=Pu= 17,784 lb per column

f't= 7.5(f'c)^0.5= 474.341649
d,req= {Pu*FS/[1.72*((ks*R1/Ec)*10^4+3.6)*f't*beta]}^0.5 Factor of Safety = 3
= 3.887977496 beta = 1 for d < 7.0 in
b= (Ec*d,req^3/(12*(1-u^2)*ks))^0.25 beta = 0.85 for d > 7.0 in
= 24.23712605
b,req=1.5*b= 36.35568908
If depth of frame < 2 * b,req,
Pu_new = 2*Pu_old= 35568
If depth of frame > or = 2* b,req
Pu remains as of before
Pn= 1.72*[(ks*R1/Ec)*10^4+3.6]*f't*t^2
= 1.72 * [(50 * 5 / 3443668)*10^4 + 3.6)] * 474.341649025257 * 6^2
= 127059.084
Pa=Pn/FS= 42353.028
Pu/Pa= **0.60** **OK**

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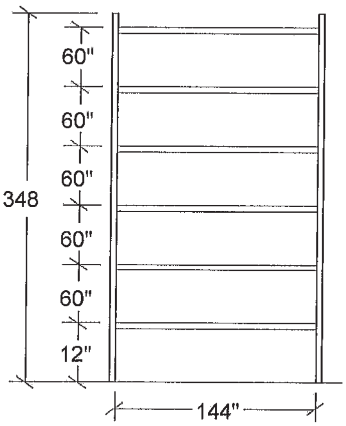
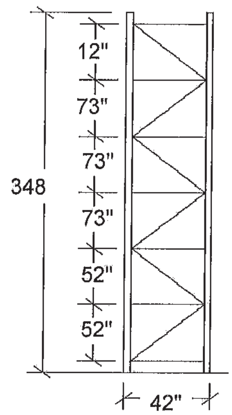
Configuration & Summary: TYPE A1 SELECTIVE RACK





***RACK COLUMN REACTIONS*

ASD LOADS

AXIAL DL= 120 lb

AXIAL LL= 12,000 lb

SEISMIC AXIAL Ps=+/- 1,737 lb

BASE MOMENT= 0 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=0.099, Fa=1.6	6	42 in	348.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10100 lb, M=14607 in-lb		0.82-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	Intlk 50E 5Hx2.75Wx0.059"Thk	Lu=144 in	Capacity: 4097 lb/pr	0.98-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=12689 in-lb	Mcap=15230 in-lb	0.83-OK
Brace-Horizontal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.05-OK
Brace-Diagonal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.21-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.95-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=960 lb)			0.233-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.56-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
1	4,000 lb	12.0 in	52.0 in	5 lb	12 lb	12,120 lb	4,437 "#	9,513 "#	3 pin OK
2	4,000 lb	60.0 in	52.0 in	32 lb	73 lb	10,100 lb	14,607 "#	12,689 "#	3 pin OK
3	4,000 lb	60.0 in	73.0 in	59 lb	134 lb	8,080 lb	13,512 "#	11,603 "#	3 pin OK
4	4,000 lb	60.0 in	73.0 in	85 lb	195 lb	6,060 lb	11,504 "#	9,877 "#	3 pin OK
5	4,000 lb	60.0 in	73.0 in	112 lb	256 lb	4,040 lb	8,582 "#	7,513 "#	3 pin OK
6	4,000 lb	60.0 in	12.0 in	138 lb	317 lb	2,020 lb	4,748 "#	4,509 "#	3 pin OK

** Load defined as product weight per pair of beams Total: 431 lb 986 lb

Notes

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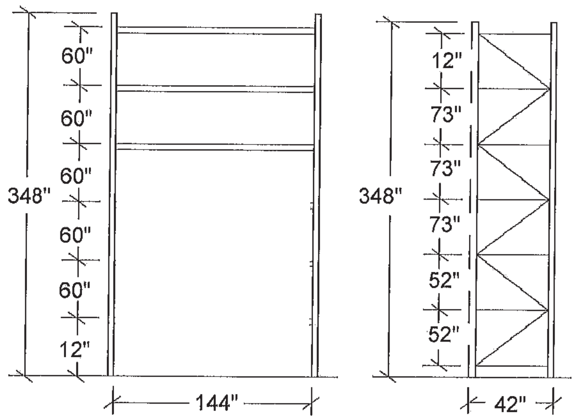
Configuration & Summary: TYPE T INTERIOR TUNNEL SELECTIVE RACK



***RACK COLUMN REACTIONS*

ASD LOADS
AXIAL DL= 120 lb
AXIAL LL= 9,150 lb
SEISMIC AXIAL Ps=+/- 1,405 lb
BASE MOMENT= 0 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=0.099, Fa=1.6	3	42 in	348.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=8200 lb, M=11243 in-lb		0.63-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	Intlk 50E 5Hx2.75Wx0.059"Thk	Lu=144 in	Capacity: 4097 lb/pr	0.98-OK
Beam Connector	Fy=55 ksi	Lvl 4: 3 pin OK	Mconn=8856 in-lb	Mcap=15230 in-lb	0.58-OK
Brace-Horizontal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.04-OK
Brace-Diagonal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.16-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.73-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=960 lb)			0.233-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.41-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
4	4,000 lb	60.0 in	73.0 in	73 lb	166 lb	6,060 lb	9,831 "#	8,856 "#	3 pin OK
5	4,000 lb	60.0 in	73.0 in	96 lb	219 lb	4,040 lb	7,335 "#	6,835 "#	3 pin OK
6	4,000 lb	60.0 in	12.0 in	118 lb	271 lb	2,020 lb	4,058 "#	4,268 "#	3 pin OK
Total:				330 lb	755 lb				

** Load defined as product weight per pair of beams

Notes

Interior tunnel