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 : 24-1119-12

Date : 11/21/24

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

Scope of Work : STORAGE RACKS

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

Enhao Zhang
Digitally signed by Enhao Zhang
Date: 2024.11.22 11:41:16 -08'00'

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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

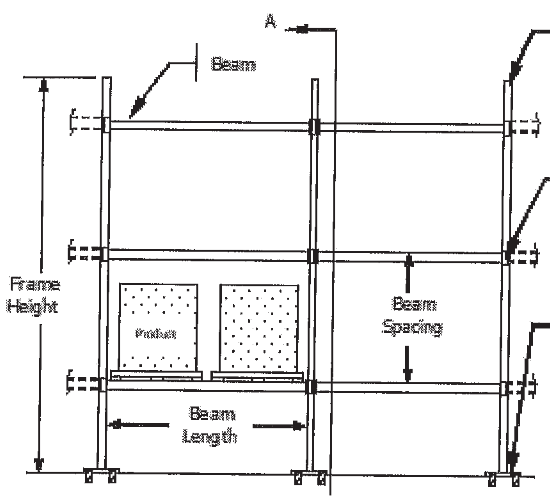
3) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein.

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.

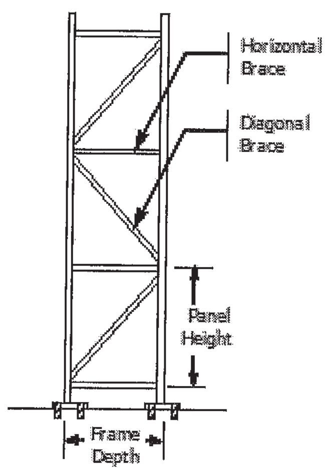
5) The existing slab on grade is 6" thick with minimum 4000 psi compressive strength. Allowable Soil bearing capacity is 750 psf. The design of the existing slab is by others.

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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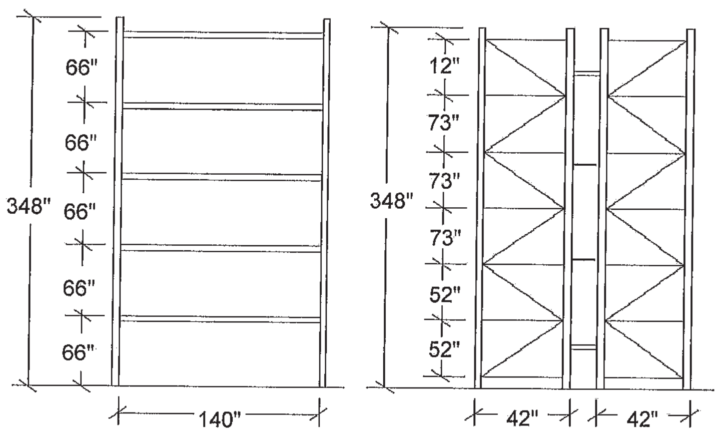
Configuration & Summary: TYPE A SELECTIVE RACK



***RACK COLUMN REACTIONS*
ASD LOADS
AXIAL DL= 188 lb
AXIAL LL= 10,000 lb
SEISMIC AXIAL Ps=+/- 1,565 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	5	42 in	348.0 in	6	140 in	

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10188 lb, M=14333 in-lb		0.91-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	0.93-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=10140 in-lb	Mcap=15230 in-lb	0.67-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			0.18-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.97-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=1066 lb)			0.517-OK
Slab & Soil		6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure			0.46-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	66.0 in	52.0 in	24 lb	30 lb	10,188 lb	14,333 "#	10,140 "#	3 pin OK
2	4,000 lb	66.0 in	52.0 in	49 lb	61 lb	8,150 lb	7,008 "#	7,225 "#	3 pin OK
3	4,000 lb	66.0 in	73.0 in	73 lb	91 lb	6,113 lb	6,006 "#	6,349 "#	3 pin OK
4	4,000 lb	66.0 in	73.0 in	97 lb	121 lb	4,075 lb	4,505 "#	5,123 "#	3 pin OK
5	4,000 lb	66.0 in	73.0 in	121 lb	152 lb	2,038 lb	2,503 "#	3,547 "#	3 pin OK
			12.0 in						
Total:				364 lb	455 lb				

** Load defined as product weight per pair of beams

Notes	Std row spacers

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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

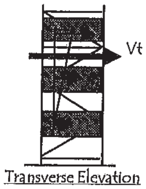
V= Cs*Ip*Ws=Cs*Ip*(0.67*P*Prf+D)

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)
= 13,775 lb

Vtransv=Vt= 0.0264 * (375 lb + 13400 lb)
Etransverse= 364 lb
Limit States Level Transverse seismic shear per upright



Transverse Elevation

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= 75 lb

Level	PRODUCT LOAD P	P*0.67*P_{RF1}	DL	hi	wi*hi	Fi	Fi*(hi+hp/2)
1	4,000 lb	2,680 lb	75 lb	66 in	181,830	24.3 lb	2,187-#
2	4,000 lb	2,680 lb	75 lb	132 in	363,660	48.5 lb	7,566-#
3	4,000 lb	2,680 lb	75 lb	198 in	545,490	72.8 lb	16,162-#
4	4,000 lb	2,680 lb	75 lb	264 in	727,320	97.1 lb	27,965-#
5	4,000 lb	2,680 lb	75 lb	330 in	909,150	121.3 lb	42,940-#
sum:	P=20000 lb	13,400 lb	375 lb	W=13775 lb	2,727,450	364 lb	Σ=96,820

Longitudinal (Downaisle) Seismic Load

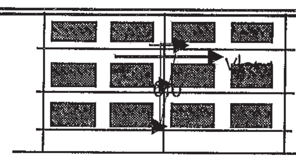
Similarly for longitudinal seismic loads, using R=6.0

Cs1=Sd1/(T*R)= 0.0330
Cs2= 0.0046
Cs3= 0.0057
Cs-max= 0.0330

Ws= (0.67 * PL_{RF2} * P) + DL
= 13,775 lb
Cs=Cs-max*Ip= 0.0330

P_{RF2}= 1.0
(Longitudinal, Unbraced Dir.) R= 6.0
T= 0.55 sec

Vlong= 0.033 * (375 lb + 13400 lb)
Elongitudinal= 455 lb ***Limit States Level Longit. seismic shear per upright***


Front View

Level	PRODUC LOAD P	P*0.67*P_{RF2}	DL	hi	wi*hi	Fi
1	4,000 lb	2,680 lb	75 lb	66 in	181,830	30.3 lb
2	4,000 lb	2,680 lb	75 lb	132 in	363,660	60.7 lb
3	4,000 lb	2,680 lb	75 lb	198 in	545,490	91.0 lb
4	4,000 lb	2,680 lb	75 lb	264 in	727,320	121.3 lb
5	4,000 lb	2,680 lb	75 lb	330 in	909,150	151.7 lb
sum:		13,400 lb	375 lb	W=13775 lb	2,727,450	455 lb

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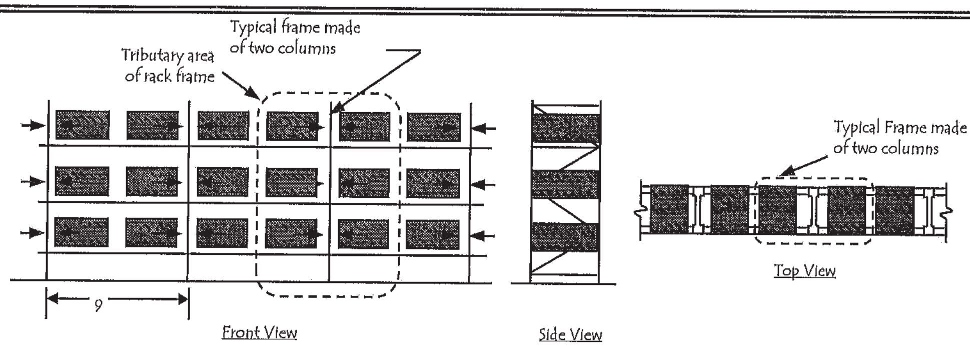
Downaisle Seismic Loads Configuration: TYPE A SELECTIVE RACK

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

Seismic Story Forces

Vlong= 455 lb
Vcol=Vlong/2= 228 lb
F1= 30 lb
F2= 61 lb
F3= 91 lb



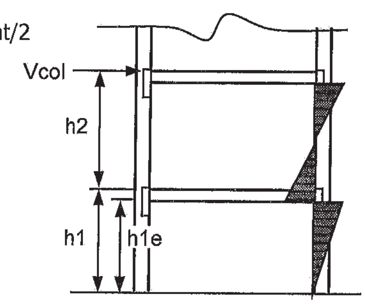
Conceptual System

Seismic Story Moments

COL

Mbase-max= 0 in-lb <=== *Default capacity*
Mbase-v= (Vcol*h1eff)/2
= 7,166 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
= 63 in



Beam to Column

M 1-1= [Vcol * h1eff]-Mbase-eff
= (228 lb * 63 in)-0 in-lb
= 14,333 in-lb

M 2-2= [Vcol-(F1)/2] * h2
= [228 lb - 30.4 lb]*66 in/2
= 7,008 in-lb

Mseis= (Mupper+Mlower)/2

Mseis(1-1)= (14333 in-lb + 7008 in-lb)/2
= 10,670 in-lb

Mseis(2-2)= (7008 in-lb + 6006 in-lb)/2
= 6,507 in-lb

rho= 1.0000

Summary of Forces

LEVEL	hi	Axial Load	Column Moment**	Mseismic**	Mend-fixity	Mconn**	Beam Connector
1	66 in	10,188 lb	14,333 in-lb	10,670 in-lb	3,815 in-lb	10,140 in-lb	3 pin OK
2	66 in	8,150 lb	7,008 in-lb	6,507 in-lb	3,815 in-lb	7,225 in-lb	3 pin OK
3	66 in	6,113 lb	6,006 in-lb	5,255 in-lb	3,815 in-lb	6,349 in-lb	3 pin OK
4	66 in	4,075 lb	4,505 in-lb	3,504 in-lb	3,815 in-lb	5,123 in-lb	3 pin OK
5	66 in	2,038 lb	2,503 in-lb	1,252 in-lb	3,815 in-lb	3,547 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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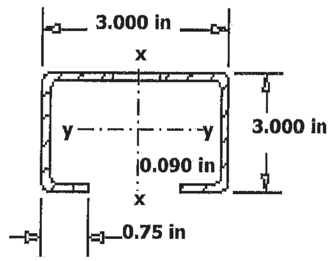
Column (Longitudinal Loads) **Configuration: TYPE A SELECTIVE RACK**

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 = 63.5 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 1

COLUMN DL= 187 lb
COLUMN PL= 10,000 lb
Mcol= 14,332 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*187 lb + 0.75 * 1.414784 * 0.7 * 10000 lb
= 7,617 lb

Moment=Mx= 0.75*0.7*rho*Mcol
= 0.525*14332 in-lb
= 7,524 in-lb

Axial Analysis

KxLx/rx = 1.7*63.5"/1.3196"
= 81.8

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

Fe= п^2E/(KL/r)max^2
= 43.5ksi

Fy/2= 27.5 ksi

Pn= Aeff*Fn
= 28,477 lb

Ωc= 1.92

Fe > Fy/2
Fn= Fy(1-Fy/4Fe)
= 55 ksi*[1-55 ksi/(4*43.5 ksi)]
= 37.6 ksi
Pa= Pn/Ωc
= 28477 lb/1.92
= 14,832 lb

P/Pa= 0.51 > 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= п^2EI/(KL)max^2
= п^2*29500 ksi/(1.7*63.5 in)^2
= 32,980 lb

µx= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*7617 lb/32980 lb)]}^-1
= 0.56

Combined Stresses

(7617 lb/14832 lb) + (0.85*7524 in-lb)/(28949 in-lb*0.56) =	0.91	< 1.0, OK	(EQ C5-1)
(7617 lb/21685 lb) + (7524 in-lb/28949 in-lb) =	0.61	< 1.0, OK	(EQ C5-2)

*** For comparison, total column stress computed for load case 6 is: 88.0%* — *g loads 6243.252608 lb Axial and M= 10032 in-lb*

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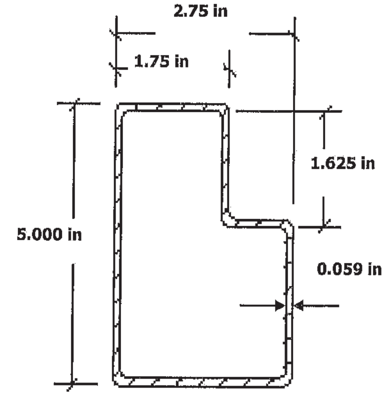
BEAM   Configuration: TYPE A SELECTIVE RACK

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

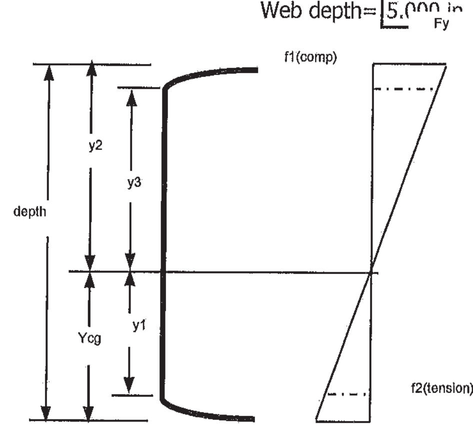
f1(comp)= Fy*(y3/y2)=   51.18 ksi
f2(tension)= Fy*(y1/y2)=   102.95 ksi
Y= f2/f1   Eq. B2.3-5
= -2.012
k= 4 + 2*(1-Y)^3 + 2*(1-Y)   Eq. B2.3-4
= 64.67
flat depth=w= y1+y3
= 4.764 in   w/t= 80.74576271   OK
l=lambda= [1.052/(k)^0.5] * (w/t) * (f1/E)^0.5
= [1.052/(64.67)^0.5] * 4.764 * (51.18/29500)^0.5
= 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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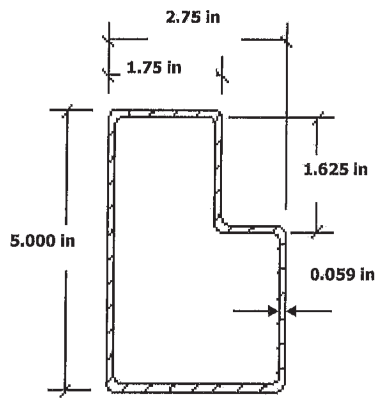
BEAM | Configuration: TYPE A SELECTIVE RACK

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
Fy=Fyv= 55 ksi
Fu=Fuv= 65 ksi
Fya= 58.6 ksi

E= 29500 ksi
F= 240.0
L= 140 in
Beam Level= 1
P=Product Load= 4,000 lb/pair
D=Dead Load= 75 lb/pair

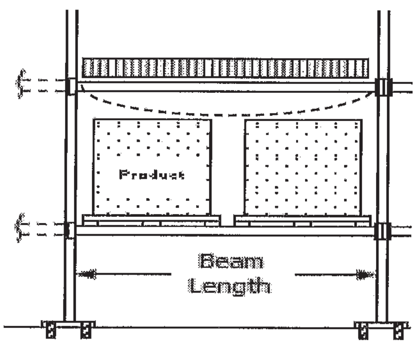


1. Check Bending Stress Allowable Loads

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
= 3,815 in-lb **@ 4000 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= 0.722 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=	**4,000 lb/pair**
Beam Stress=	**0.93**

Beam at Level 1

8.2

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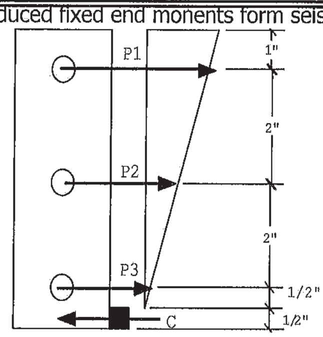
The beam end moments shown herein show the result of the maximum induced fixed end monents form seismic + static loads and the code mandated minimum value of 1.5%(DL+PL)

Mconn max= (Mseismic + Mend-fixity)*0.70*Rho
= 10,140 in-lb Load at level 1

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

Structural

Engineering & Design Inc.

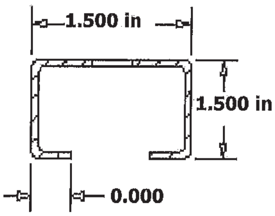
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: Dray S   Project: ~~Johnson Storage~~   Project #: ~~24-1119-~~ 12

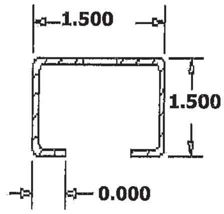
Transverse Brace   Configuration: TYPE A SELECTIVE RACK

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*

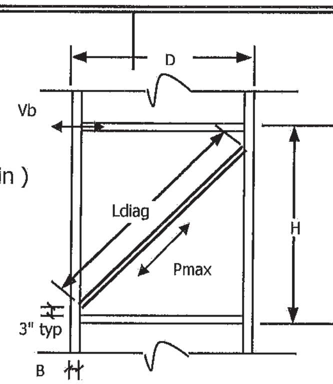
Vtransverse= 364 lb
Vb=Vtransv*0.7*rho= 364 lb * 0.7 * 1
= 255 lb
Ldiag= [(D-B*2)^2 + (H-6")^2]^1/2
= 76.1 in

Pmax= V*(Ldiag/D) * 0.75
= 346 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= 0.18 <= 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= 255 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= 0.04 <= 1.0 OK

Structural

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

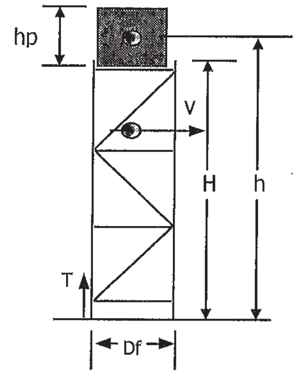
By: Dray S | Project: ~~Johnson Storage~~ | Project #: ~~24-1119-12~~

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



Vtrans=V=E=Qe= 364 lb
DEAD LOAD PER UPRIGHT=D= 375 lb
PRODUCT LOAD PER UPRIGHT=P= 20,000 lb
Papp=P*0.67= 13,400 lb
Wst LC1=Wst1=(0.87888*D + 0.87888*Papp*1)= 12,106 lb

Product Load Top Level, Ptop= 4,000 lb
DL/Lvl= 75 lb
Seismic Ovt based on E, Σ(Fi*hi)= 65,749 in-lb
height/depth ratio= 7.9 in

Sds= 0.1056
(0.9-0.2Sds)= 0.8789
(0.9-0.2Sds)= 0.8789
B= 1.0000
rho= 1.0000

Frame Depth=Df= 42.0 in
Htop-lvl=H= 330.0 in
# Levels= 5
# Anchors/Base= 2
hp= 48.0 in
h=H+hp/2= 354.0 in

A) Fully Loaded Rack

Load case 1:

Movt= Σ(Fi*hi)*E*rho
= 65,749 in-lb

Mst= Wst1 * Df/2
= 12106 lb * 42 in/2
= 254,226 in-lb

T= (Movt-Mst)/Df
= (65749 in-lb - 254226 in-lb)/42 in
= -4,488 lb **No Uplift**

Net Seismic Uplift= -4,488 lb

B) Top Level Loaded Only

Load case 1:

V1=Vtop= Cs * Ip * Ptop >= 350 lb for H/D >6.0
= 0.0264 * 4000 lb
= 106 lb
V1eff= 350 lb
$V2=V_{DL}$= Cs*Ip*D
= 10 lb
Mst= (0.87888*D + 0.87888*Ptop*1) * 42 in/2
= 80,747 in-lb

Critical Level= 5
Cs*Ip= 0.0264

Movt= [V1*h + V2 * H/2]*rho
= 125,534 in-lb
T= (Movt-Mst)/Df
= (125534 in-lb - 80747 in-lb)/42 in
= 1,066 lb **Net Uplift per Column**

Net Seismic Uplift= 1,066 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= 970 lb
Shear Capacity=Vcap= 1,250 lb

L.A. City Jurisdiction? NO
Phi= 1

Tcap*Phi= 970 lb
Vcap*Phi= 1,250 lb

Fully Loaded:	(91 lb/1250 lb)^1 =	0.07	<= 1.2 OK
Top Level Loaded:	(533 lb/970 lb)^1 + (87 lb/1250 lb)^1 =	0.62	<= 1.2 OK

Structural

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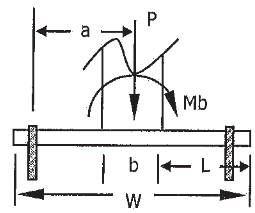
By: Dray S Project: ~~Johnson Storage~~ Project #: ~~24-1119-~~ 12

Base Plate Configuration: TYPE A SELECTIVE RACK

Section

Baseplate= 7.75x5x0.375
Eff Width=W = 7.75 in
Eff Depth=D = 5.00 in
Column Width=b = 3.00 in
Column Depth=dc = 3.00 in
L = 2.38 in
Plate Thickness=t = 0.375 in

a = 2.88 in
Anchor c.c. =2*a=d = 5.75 in
N=# Anchor/Base= 2
Fy = 36,000 psi



Downaisle Elevation

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= 188 lb
COLUMN PL= 10,000 lb
Base Moment= 0 in-lb
1+0.105*Sds= 1.0111
1.4+0.14Sds= 1.4148
B= 0.7000

Axial=P= 1.011088 * 187.5 lb + 0.75 * (1.414784 * 0.7 * 10000 lb)
= 7,617 lb
Mb= Base Moment*0.75*0.7*rho
= 0 in-lb * 0.75*0.7*rho
= 0 in-lb

Axial Load P = 7,617 lb **Mbase=Mb = 0 in-lb**

Effe
Effe

Axial stress=fa = P/A = P/(D*W)
= 197 psi
Moment Stress=fb = M/S = 6*Mb/[(D*B^2]
= 0.0 psi
Moment Stress=fb1 = fb-fb2
= 0.0 psi
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2
= 0 in-lb
S-plate = (1)(t^2)/6
= 0.023 in^3/in
fb/Fb = Mtotal/[(S-plate)(Fb)]
= **0.88** **OK**
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2]
= -6,053 lb No Tension

M1= wL^2/2= fa*L^2/2
= 554 in-lb
Moment Stress=fb2 = 2 * fb * L/W
= 0.0 psi
M2= fb1*L^2)/2
= 0 in-lb
Mtotal = M1+M2+M3
= 554 in-lb/in
Fb = 0.75*Fy
= 27,000 psi
F'p= 0.7*F'c
= 2,800 psi OK
Tallow= 970 lb **OK**

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= 7,617 lb

Movt*0.75*0.7*rho= 34,518 in-lb
Frame Depth= 42.0 in
P=Pstatic+Pseismic= 8,439 lb

Pseismic= Movt/Frame Depth
= 822 lb

b =Column Depth= 3.00 in

L =Base Plate Depth-Col Depth= 2.38 in

fa = P/A = P/(D*W)
= 218 psi

M= wL^2/2= fa*L^2/2
= 614 in-lb/in

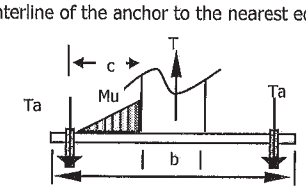
Sbase/in = (1)(t^2)/6
= 0.023 in^3/in

Fbase = 0.75*Fy
= 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.97** **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= 1,066 lb
Qty Anchor per BP= 2
Net Tension per anchor=Ta= 533 lb
c= 2.38 in
Mu=Moment on Baseplate due to uplift= Ta*c/2
= 633 in-lb
Splate= 0.117 in^3
[fb/Fb]*0.75= 0.15 **OK**

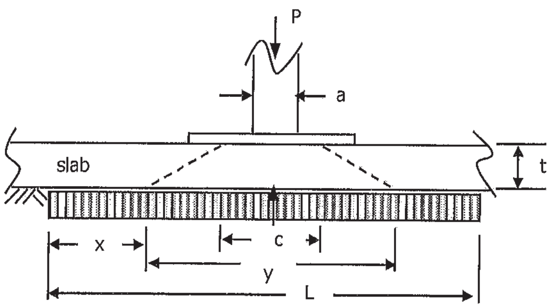
Structural

Engineering & Design Inc.

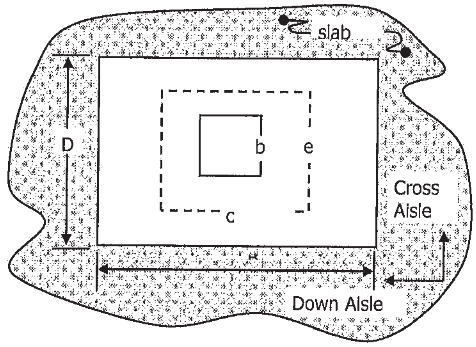
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: Dray S   Project: ~~Johnson Storage~~   Project #: ~~24-1119-~~ 12

Slab on Grade   Configuration: TYPE A 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= 65,749 in-lb
Frame depth= 42.0 in
Sds= 0.106
0.2*Sds= 0.021
λ= 0.600
β=B/D= 1.550
F'c^0.5= 63.20 psi

Base Plate

Effec. Baseplate width=B= 7.75 in   width=a= 3.00 in
Effec. Baseplate Depth=D= 5.00 in   depth=b= 3.00 in
midway dist face of column to edge of plate=c= 5.38 in
midway dist face of column to edge of plate=e= 4.00 in

Column Loads

DEAD LOAD=D= 188 lb per column
unfactored ASD load
PRODUCT LOAD=P= 10,000 lb per column
unfactored ASD load
Papp= 6,700 lb per column
P-seismic=E= (Movt/Frame depth)
= 1,565 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 * 188 lb + 1.22112 * 0.7 * 10000 lb + 1 * 1565 lb
= 10,342 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 * 188 lb + 0.87888 * 0.7 * 6700 lb + 1 * 1565 lb
= 5,852 lb
Load Case 3) 1.2*D + 1.4*P   RMI SEC 2.2 EQTN 1,2
= 1.2*188 lb + 1.4*10000 lb
= 14,225 lb
Load Case 4) 1.2*D + 1.0*P + 1.0E   ACI 318-14 Sec 5.3.1 Eqtn 5.3.1e
= 11,791 lb

Effective Column Load=Pu= 14,225 lb per column

Puncture

Apunct= [(c+t)+(e+t)]*2*t
= 256.50 in^2
Fpunct1= [(4/3 + 8/(3*β)] * λ *(F'c^0.5)
= 115.8 psi
Fpunct2= 2.66 * λ * (F'c^0.5)
= 100.9 psi
Fpunct eff= 100.9 psi

fv/Fv= Pu/(Apunct*Fpunct)
= **0.550** **< 1 OK**

Slab Bending

Pse=DL+PL+E= 14,225 lb

Asoil= (Pse*144)/(fsoil)
= 2,731 in^2
x= (L-y)/2
= 17.8 in
Fb= 5*(phi)*(f'c)^0.5
= 189.74 psi

L= (Asoil)^0.5
= 52.26 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 826.1 in-lb

y= (c*e)^0.5 + 2*t
= 16.6 in
S-slab= 1*teff^2/6
= 6.0 in^3
fb/Fb= M/(S-slab*Fb)
= **0.726** **< 1, OK**

Structural

Engineering & Design Inc.

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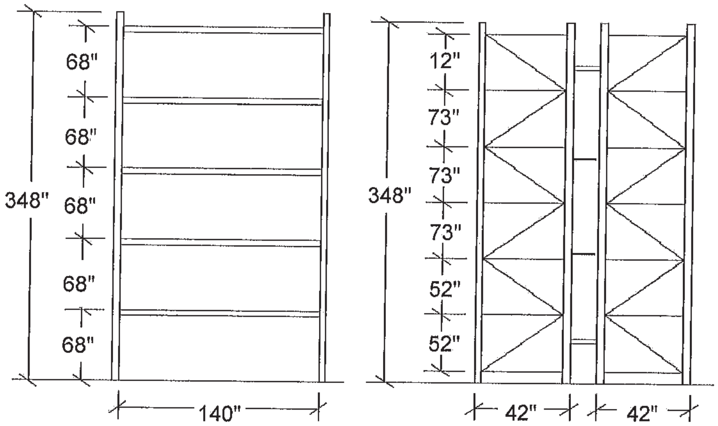
By: Dray S Project: Johnson Storage Project #: 24-1119- 12

Configuration & Summary: TYPE B SELECTIVE RACK



***RACK COLUMN REACTIONS*
ASD LOADS
AXIAL DL= 188 lb
AXIAL LL= 10,000 lb
SEISMIC AXIAL Ps=+/- 1,608 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	5	42 in	348.0 in	6	140 in	

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10188 lb, M=14788 in-lb		0.96-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	0.93-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=10373 in-lb	Mcap=15230 in-lb	0.68-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			0.18-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.97-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=1150 lb)			0.55-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.46-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	68.0 in	52.0 in	24 lb	30 lb	10,188 lb	14,788 "#	10,373 "#	3 pin OK
2	4,000 lb	68.0 in	52.0 in	49 lb	61 lb	8,150 lb	7,220 "#	7,363 "#	3 pin OK
3	4,000 lb	68.0 in	73.0 in	73 lb	91 lb	6,113 lb	6,188 "#	6,461 "#	3 pin OK
4	4,000 lb	68.0 in	73.0 in	97 lb	121 lb	4,075 lb	4,641 "#	5,197 "#	3 pin OK
5	4,000 lb	68.0 in	73.0 in	121 lb	152 lb	2,038 lb	2,579 "#	3,573 "#	3 pin OK
			12.0 in						
			Total:	364 lb	455 lb				

** Load defined as product weight per pair of beams

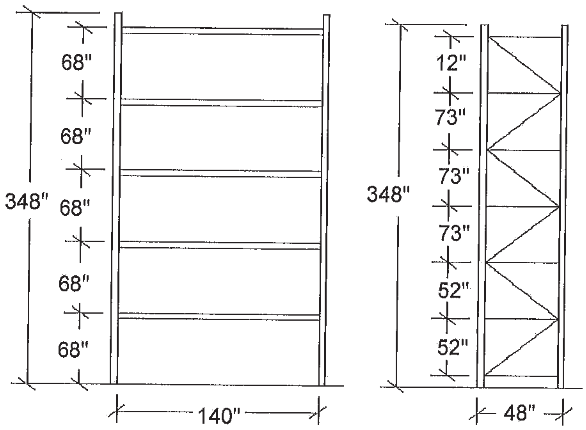
Notes	Std row spacers

Structural
Engineering & Design Inc.

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

By: Dray S Project: Johnson Storage Project #: 24-1119-12

Configuration & Summary: TYPE C SELECTIVE RACK



***RACK COLUMN REACTIONS*
ASD LOADS
AXIAL DL= 188 lb
AXIAL LL= 10,000 lb
SEISMIC AXIAL Ps=+/- 1,407 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	5	48 in	348.0 in	6	140 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10188 lb, M=14788 in-lb		0.96-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	0.93-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=10373 in-lb	Mcap=15230 in-lb	0.68-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			0.05-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.96-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=766 lb)			0.383-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.43-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	68.0 in	52.0 in	24 lb	30 lb	10,188 lb	14,788 "#	10,373 "#	3 pin OK
2	4,000 lb	68.0 in	52.0 in	49 lb	61 lb	8,150 lb	7,220 "#	7,363 "#	3 pin OK
3	4,000 lb	68.0 in	73.0 in	73 lb	91 lb	6,113 lb	6,188 "#	6,461 "#	3 pin OK
4	4,000 lb	68.0 in	73.0 in	97 lb	121 lb	4,075 lb	4,641 "#	5,197 "#	3 pin OK
5	4,000 lb	68.0 in	73.0 in	121 lb	152 lb	2,038 lb	2,579 "#	3,573 "#	3 pin OK
			12.0 in						

** Load defined as product weight per pair of beams Total: 364 lb 455 lb

Notes

Structural

Engineering & Design Inc.

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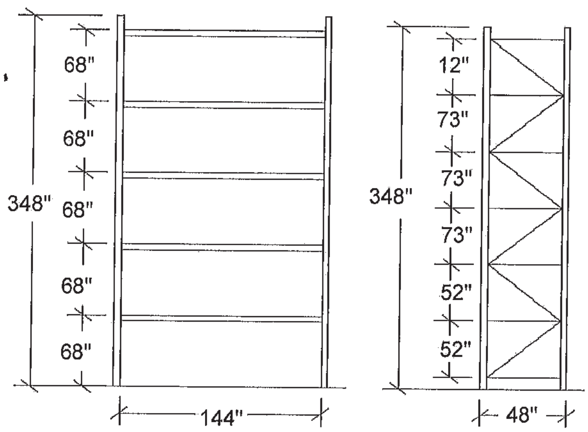
By: Dray S Project: ~~Johnson Storage~~ Project #: ~~24-1119-~~ 12

Configuration & Summary: TYPE D SELECTIVE RACK



**RACK COLUMN REACTIONS	
ASD LOADS	
AXIAL DL=	188 lb
AXIAL LL=	10,000 lb
SEISMIC AXIAL Ps=+/-	1,407 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	5	48 in	348.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	INTLK LU75/3x3x13ga	P=10188 lb, M=14788 in-lb		0.96-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=10550 in-lb	Mcap=15230 in-lb	0.69-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			0.17-OK
Base Plate	Fy=36 ksi	7.75x5x0.375	Fixity= 0 in-lb		0.96-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=766 lb)			0.383-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.43-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	68.0 in	52.0 in	24 lb	30 lb	10,188 lb	14,788 "#	10,550 "#	3 pin OK
2	4,000 lb	68.0 in	52.0 in	49 lb	61 lb	8,150 lb	7,220 "#	7,540 "#	3 pin OK
3	4,000 lb	68.0 in	73.0 in	73 lb	91 lb	6,113 lb	6,188 "#	6,638 "#	3 pin OK
4	4,000 lb	68.0 in	73.0 in	97 lb	121 lb	4,075 lb	4,641 "#	5,375 "#	3 pin OK
5	4,000 lb	68.0 in	73.0 in	121 lb	152 lb	2,038 lb	2,579 "#	3,750 "#	3 pin OK
			12.0 in						
			Total:	364 lb	455 lb				

** Load defined as product weight per pair of beams

Notes

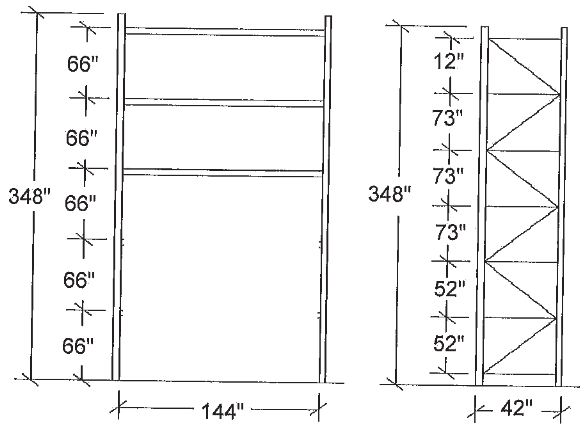
Structural

Engineering & Design Inc.

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

By: Dray S Project: Johnson Storage Project #: 24-1119-12

Configuration & Summary: TYPE T INTERIOR SELECTIVE RACK



***RACK COLUMN REACTIONS*
ASD LOADS
AXIAL DL= 188 lb
AXIAL LL= 8,000 lb
SEISMIC AXIAL Ps=+/- 1,328 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=8188 lb, M=11529 in-lb		0.69-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 3: 3 pin OK	Mconn=6125 in-lb	Mcap=15230 in-lb	0.4-OK
Brace-Horizontal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.03-OK
Brace-Diagonal	Fy=55 ksi	Intlk 1-1/2x1-1/2x14ga			0.14-OK
Base Plate	Fy=36 ksi	7.75x5x0.375		Fixity= 0 in-lb	0.79-OK
Anchor	2 per Base	0.5" x 3.25" Embed POWERS SD1 ESR 2818 No Inspection (Net Seismic Uplift=1066 lb)			0.517-OK
Slab & Soil	6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure				0.36-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
3	4,000 lb	66.0 in	73.0 in	65 lb	81 lb	6,113 lb	5,351 "#	6,125 "#	3 pin OK
4	4,000 lb	66.0 in	73.0 in	87 lb	108 lb	4,075 lb	4,013 "#	5,032 "#	3 pin OK
5	4,000 lb	66.0 in	73.0 in	108 lb	135 lb	2,038 lb	2,229 "#	3,628 "#	3 pin OK
			12.0 in						
** Load defined as product weight per pair of beams			Total:	293 lb	366 lb				

Notes

Interior tunnel