Calculations required to be provided by the Permittee on site for all Inspections

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

| | |
|---|---|
| Building | Planning |
| Engineering | Public Works |
| Fire | Traffic |

PRCTI20231742

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**Building**
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CITY OF PUYALLUP STATE OF WASHINGTON

# Structural Engineering & Design, Inc.

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

Project Name : INGERSOLL RAND

Project Number : 23-1109-15

Date : 12/11/23

Street Address: 418 VALLEY AVE NW, STE B-107
City/State : PUYALLUP, WA 98371

Scope of Work : STORAGE RACK/SHELVING

ENHAO ZHANG STATE OF WASHINGTON 21022747 REGISTERED PROFESSIONAL ENGINEER

Enhao Zhang

Digitally signed by Enhao Zhang
Date: 2023.12.20 15:15:26 -08'00'

Structural

Engineering & Design Inc.

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By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**TABLE OF CONTENTS**

Structural
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**Design Data**

1) The analyses herein conforms to the requirements of the:
*2018 IBC Section 2209*
*2022 CBC 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

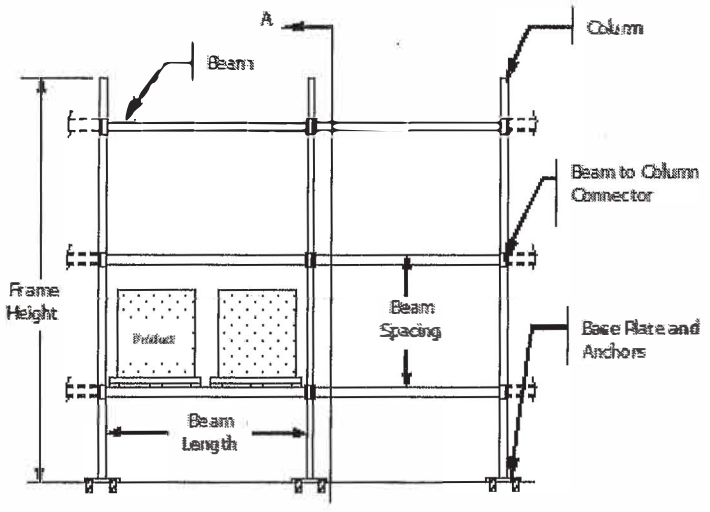
3) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein.

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.

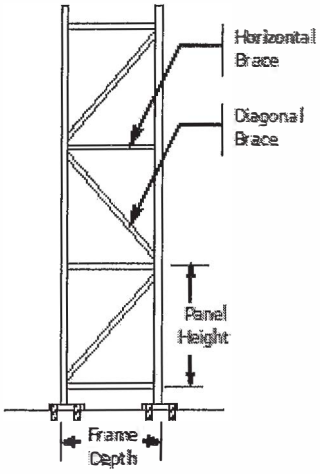
5) The existing slab on grade is 5.5" thick with minimum 7123 psi compressive strength. Allowable Soil bearing capacity is 750 psf. The design of the existing slab is by others.

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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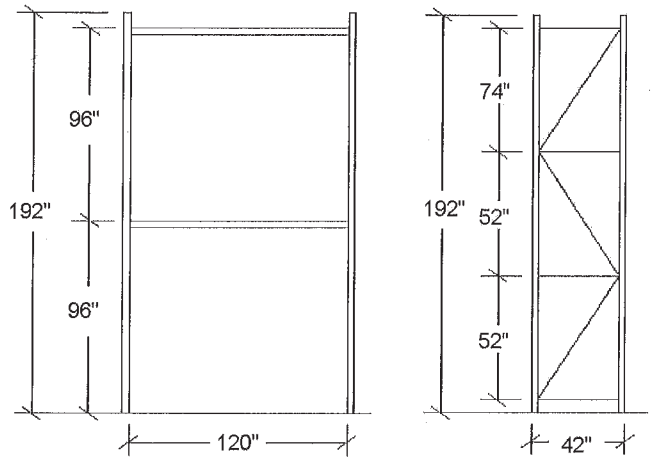
**Configuration & Summary: TYPE 1 SELECTIVE RACK**




***RACK COLUMN REACTIONS*
*ASD LOADS*
*AXIAL DL= 50 lb*
*AXIAL LL= 3,200 lb*
*SEISMIC AXIAL Ps=+/- 3,333 lb*
*BASE MOMENT= 8,000 in-lb*

| Seismic Criteria | # Bm Lvls | Frame Depth | Frame Height | # Diagonals | Beam Length | Frame Type |
|---|---|---|---|---|---|---|
| Ss=1.28, Fa=1.2 | 2 | 42 in | 192.0 in | 3 | 120 in | Single Row |

| Component | Description | | | | | STRESS |
|---|---|---|---|---|---|---|
| Column | Fy=55 ksi | INTLK LU70/3x3x14ga | | P=3250 lb, M=15157 in-lb | | 0.88-OK |
| Column & Backer | None | None | | None | | N/A |
| Beam | Fy=55 ksi | Intlk 50E 5Hx2.75Wx0.059"Thk | | Lu=120 in | Capacity: 5516 lb/pr | 0.58-OK |
| Beam Connector | Fy=55 ksi | Lvl 1: 3 pin OK | Mconn=9152 in-lb | | Mcap=12691 in-lb | 0.72-OK |
| Brace-Horizontal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.13-OK |
| Brace-Diagonal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.56-OK |
| Base Plate | Fy=36 ksi | 8x5x0.375 | | | Fixity= 8000 in-lb | 0.63-OK |
| Anchor | 2 per Base | 0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3124 lb) | | | | 0.733-OK |
| Slab & Soil | 5.5" thk x 7123 psi slab on grade. 750 psf Soil Bearing Pressure | | | | | 0.25-OK |

| Level | Load** Per Level | Beam Spcg | Brace | Story Force Transv | Story Force Longit. | Column Axial | Column Moment | Conn. Moment | Beam Connector |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 3,200 lb | 96.0 in | 52.0 in | 374 lb | 166 lb | 3,250 lb | 15,157 "# | 9,152 "# | 3 pin OK |
| 2 | 3,200 lb | 96.0 in | 52.0 in | 749 lb | 332 lb | 1,625 lb | 7,968 "# | 3,847 "# | 3 pin OK |
| | | | 74.0 in | | | | | | |

** Load defined as product weight per pair of beams Total: 1,123 lb 498 lb

**Notes**

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**Seismic Forces** Configuration: TYPE 1 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**

Ss= 1.280
S1= 0.414
Fa= 1.200
Fv= 1.850
Sds=2/3*Ss*Fa= 1.024
Sd1=2/3*S1*Fv= 0.511
Ca=0.4*2/3*Ss*Fa= 0.4096
(Transverse, Braced Frame Dir.) R= 4.0
Ip= 1.0
$P_{RF1}$= 1.0
Pallet Height=hp= 48.0 in
DL per Beam Lvl= 50 lb

## Transverse (Cross Aisle) Seismic Load

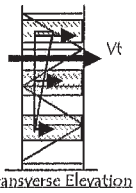
**V= Cs*Ip*Ws=Cs*Ip*(0.67*P*Prf+D)**

Cs1= Sds/R
= 0.2560
Cs2= 0.044*Sds
= 0.0451
Cs3= 0.5*S1/R
= 0.0518
Cs-max= 0.2560

**Base Shear Coeff=Cs= 0.2560**

Cs-max * Ip= 0.2560
$V_{min}$= 0.015
Eff Base Shear=Cs= 0.2560
Ws= $(0.67*PL_{RF1} * PL)$+DL (RMI 2.6.2)
= 4,388 lb

**Vtransv=Vt= 0.256 * (100 lb + 4288 lb)**
**Etransverse= 1,123 lb**
***Limit States Level Transverse seismic shear per upright***



Transverse Elevation

| Level | PRODUCT LOAD P | P*0.67*$P_{RF1}$ | DL | hi | wi*hi | Fi | Fi*(hi+hp/2) |
|---|---|---|---|---|---|---|---|
| 1 | 3,200 lb | 2,144 lb | 50 lb | 96 in | 210,624 | 374.3 lb | 44,916-# |
| 2 | 3,200 lb | 2,144 lb | 50 lb | 192 in | 421,248 | 748.7 lb | 161,719-# |
| sum: | P=6400 lb | 4,288 lb | 100 lb | W=4388 lb | 631,872 | 1,123 lb | Σ=206,635 |

## Longitudinal (Downaisle) Seismic Load

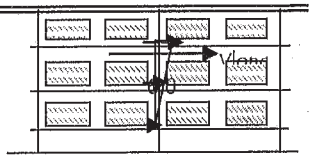
Similarly for longitudinal seismic loads, using R=6.0

Cs1=Sd1/(T*R)= 0.1135
Cs2= 0.0451
Cs3= 0.0345
Cs-max= 0.1135

Ws= $(0.67 * PL_{RF2} * P)$ + DL
= 4,388 lb
Cs=Cs-max*Ip= 0.1135

$P_{RF2}$= 1.0
(Longitudinal, Unbraced Dir.) R= 6.0
T= 0.75 sec

**Vlong= 0.1135 * (100 lb + 4288 lb)**
**Elongitudinal= 498 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 | 3,200 lb | 2,144 lb | 50 lb | 96 in | 210,624 | 166.0 lb |
| 2 | 3,200 lb | 2,144 lb | 50 lb | 192 in | 421,248 | 332.0 lb |
| sum: | | 4,288 lb | 100 lb | W=4388 lb | 631,872 | 498 lb |

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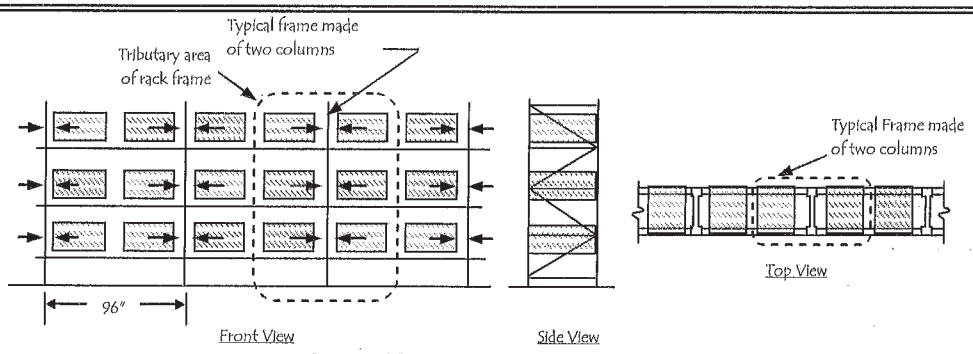
**Downaisle Seismic Loads** Configuration: TYPE 1 SELECTIVE RACK

Determine the story moments by applying portal analysis. The base plate is assumed to provide partial fixity.

**Seismic Story Forces**

Vlong= 498 lb
Vcol=Vlong/2= 249 lb
F1= 166 lb
F2= 332 lb
F3= 0 lb



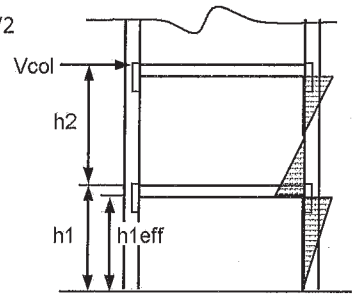

Conceptual System

**Seismic Story Moments**

Mbase-max= 8,000 in-lb *<=== Default capacity*
Mbase-v= (Vcol*h1eff)/2
= 11,579 in-lb *<=== Moment going to base*
Mbase-eff= Minimum of Mbase-max and Mbase-v
= 8,000 in-lb
M 1-1= [Vcol * h1eff]-Mbase-eff
= (249 lb * 93 in)-8000 in-lb
= 15,157 in-lb

h1-eff= h1 - beam clip height/2
= 93 in

M 2-2= [Vcol-(F1)/2] * h2
= [249 lb - 166 lb]*96 in/2
= 7,968 in-lb




Beam to Column Elevation

Mseis= (Mupper+Mlower)/2

Mseis(1-1)= (15157 in-lb + 7968 in-lb)/2
= 11,563 in-lb

Mseis(2-2)= (7968 in-lb + 0 in-lb)/2
= 3,984 in-lb

rho= 1.0000

**Summary of Forces**

| LEVEL | hi | Axial Load | Column Moment** | Mseismic** | Mend-fixity | Mconn** | Beam Connector |
|---|---|---|---|---|---|---|---|
| 1 | 96 in | 3,250 lb | 15,157 in-lb | 11,563 in-lb | 1,512 in-lb | 9,152 in-lb | 3 pin OK |
| 2 | 96 in | 1,625 lb | 7,968 in-lb | 3,984 in-lb | 1,512 in-lb | 3,847 in-lb | 3 pin OK |

Mconn= (Mseismic + Mend-fixity)*0.70*rho
Mconn-allow(3 Pin)= 12,691 in-lb

**all moments based on limit states level loading

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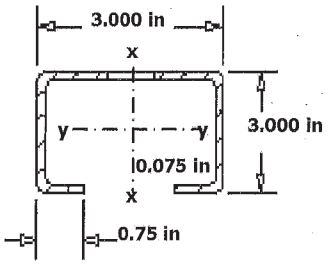
**Column (Longitudinal Loads)** **Configuration: TYPE 1 SELECTIVE RACK**

## Section Properties

Section: INTLK LU70/3x3x14ga

| | | |
|---|---|---|
| Aeff = 0.643 in^2 | Iy = 0.749 in^4 | Kx = 1.7 |
| Ix = 1.130 in^4 | Sy = 0.493 in^3 | Lx = 93.5 in |
| Sx = 0.753 in^3 | ry = 1.080 in | Ky = 1.0 |
| rx = 1.326 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= 50 lb
COLUMN PL= 3,200 lb
Mcol= 15,157 in-lb
Sds= 1.0240
1+0.105*Sds= 1.1075
1.4+0.14Sds= 1.5434
1+0.14Sds= 1.1434
0.85+0.14*Sds= 0.9934
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: 0.810264 * 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: 0.69535* *seismic moment coeff: 0.7 * Mcol*

By analysis, Load case 6 governs utilizing loads as such

**Axial Load=Pax= 1.14336*50 lb + 0.99336*0.7*3200 lb**
**= 2,282 lb**

**Moment=Mx= 0.7*rho*Mcol**
**= 0.7 * 15157 in-lb**
**= 10,610 in-lb**

## Axial Analysis

KxLx/rx = 1.7*93.5"/1.326"
= 119.9

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

Fe < Fy/2
Fn= Fe
= π^2E/(KL/r)max^2
= 20.3 ksi

Fe= π^2E/(KL/r)max^2
= 20.3ksi

Fy/2= 27.5 ksi

Pn= Aeff*Fn
= 13,029 lb

Ωc= 1.92

Pa= Pn/Ωc
= 13029 lb/1.92
= 6,786 lb

P/Pa= 0.34 > 0.15

## Bending Analysis

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

Pno= Ae*Fy
= 0.643 in^2 *55000 psi
= 35,365 lb

Pao= Pno/Ωc
= 35365lb/1.92
= 18,419 lb

Myield=My= Sx*Fy
= 0.753 in^3 * 55000 psi
= 41,415 in-lb

Max= My/Ωf
= 41415 in-lb/1.67
= 24,799 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500 ksi/(1.7*93.5 in)^2
= 13,022 lb

μx= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*2282 lb/13022 lb)]}^-1
= 0.66

## Combined Stresses

| | | | |
|---|---|---|---|
| (2282 lb/6786 lb) + (0.85*10610 in-lb)/(24799 in-lb*0.66) = | 0.88 | < 1.0, OK | (EQ C5-1) |
| (2282 lb/18419 lb) + (10610 in-lb/24799 in-lb) = | 0.55 | < 1.0, OK | (EQ C5-2) |

*** For comparison, total column stress computed for load case 5 is: 84.0%* *izing loads 2648.2208 lb Axial and M= 7957 in-lb*

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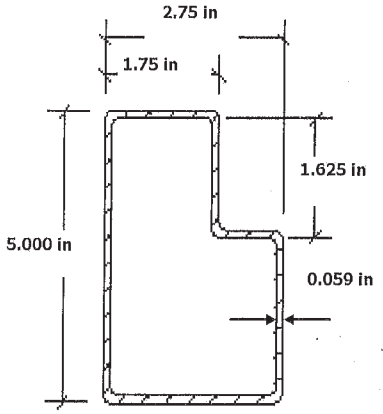
**BEAM** Configuration: TYPE 1 SELECTIVE RACK

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

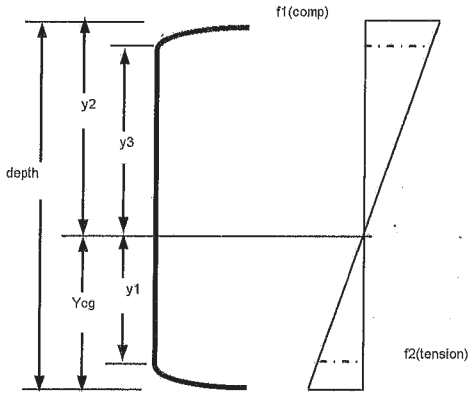
Beam= **Intlk 50E 5Hx2.75Wx0.059"Thk**

| | |
|---|---|
| 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 |

Fy

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= 31.26 in-k

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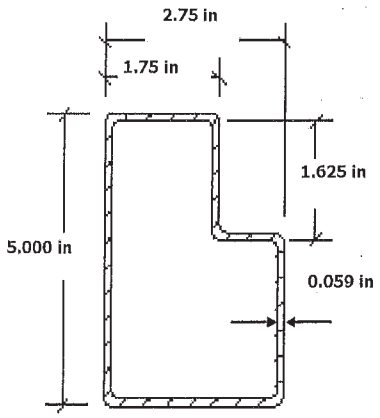
**BEAM** Configuration: TYPE 1 SELECTIVE RACK

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= 150.0
Fu=Fuv= 65 ksi L= 120 in
Fya= 58.6 ksi Beam Level= 1
P=Product Load= 3,200 lb/pair
D=Dead Load= 50 lb/pair



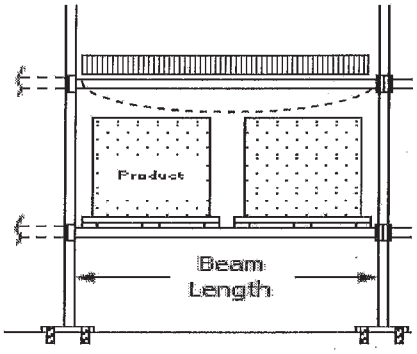

**1. Check Bending Stress Allowable Loads**

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*150*120 in)/[(6*29500 ksi*2.9155 in^3)+(3*150*120 in)]
= 0.937
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/(120in * 0.937 * 1.599)
= **5,523 lb/pair** ***allowable load based on bending stress***




Mend= W*L*(1-Rm)/8
= (5523 lb/2) * 120 in * (1-0.937)/8
= 2,610 in-lb @ 5523 lb max allowable load
= 1,512 in-lb **@ 3200 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*150*120 in)/[(5*150*120 in)+(10*29500 ksi*2.9155 in^4)]
= 0.924 in
if Dmax= L/180 *Based on L/180 Deflection Criteria*
and Dss= 5*W*L^3/(384*E*Ib)

Allowable Deflection= L/180
= 0.667 in
Deflection at imposed Load= 0.387 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*(120 in)^2*0.924)
= **5,516 lb/pair** ***allowable load based on deflection limits***

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

**Allowable load= 5,516 lb/pair**
**Imposed Product Load= 3,200 lb/pair**
**Beam Stress= 0.58** *Beam at Level 1*

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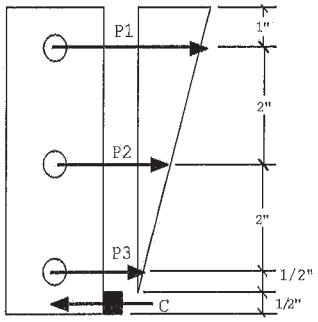
By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**3 Pin Beam to Column Connection** TYPE 1 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**
**= 9,152 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= 0.075 in Fu= 65,000 psi

Omega= 2.22 a= 2.22

Pbearing= alpha * Fu * diam * tcol/Omega
= 2.22 * 65000 psi * 0.438 in * 0.075 in/2.22
= 2,135 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= 2,135 lb
= 5,531 lb

Mconn-allow= [P1*4.5"+P1*(2.5"/4.5")*2.5"+P1*(0.5"/4.5")*0.5"]
= 2135 LB*[4.5"+(2.5"/4.5")*2.5"+ (0.5"/4.5")*0.5"]
= 12,691 in-lb > Mconn max, OK

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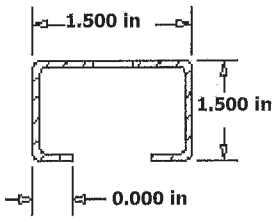
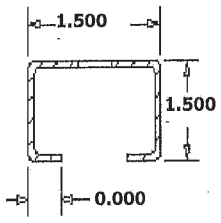
**Transverse Brace** Configuration: TYPE 1 SELECTIVE RACK

## 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= 74.0 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 + [(0.85+0.14Sds)*B*P + [0.7*rho*E]<= 1.0, ASD Method* → 0

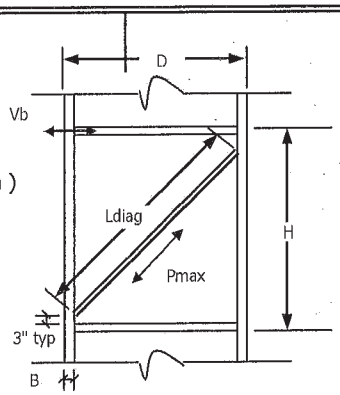
**Vtransverse= 1,123 lb**
**Vb=Vtransv*0.7*rho= 1123 lb * 0.7 * 1**
**= 786 lb**
Ldiag= [(D-B*2)^2 + (H-6")^2]^1/2
= 76.9 in

**Pmax= V*(Ldiag/D) * 0.75**
**= 1,079 lb**
***axial load on diagonal brace member***

Pn= AREA*Fn
= 0.315 in^2 * 11722 psi
= 3,692 lb

(kl/r)= (k * Ldiag)/r min
= (1 x 76.9 in /0.488 in )
= 157.6 in
Fe= pi^2*E/(kl/r)^2
= 11,722 psi

*Since Fe<Fy/2,*
Fn= Fe
= 11,722 psi



Typical Panel Configuration

Pallow= Pn/Ω
= 3692 lb /1.92
= 1,923 lb

**Pn/Pallow= 0.56 <= 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= 786 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= 0.13 <= 1.0 OK**

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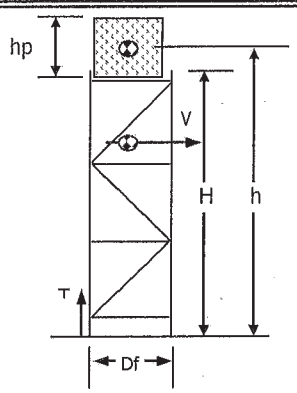
By: Bob S | Project: Ingersoll Rand | Project #: 23-1109-15

**Single Row Frame Overturning** | Configuration: TYPE 1 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= | 1,123 lb | Sds= | 1.0240 |
| DEAD LOAD PER UPRIGHT=D= | 100 lb | (0.9-0.2Sds)= | 0.6952 |
| PRODUCT LOAD PER UPRIGHT=P= | 6,400 lb | (0.9-0.2Sds)= | 0.6952 |
| Papp=P*0.67= | 4,288 lb | B= | 1.0000 |
| Wst LC1=Wst1=(0.6952*D + 0.6952*Papp*1)= | 3,050 lb | rho= | 1.0000 |
| | | Frame Depth=Df= | 42.0 in |
| Product Load Top Level, Ptop= | 3,200 lb | Htop-lvl=H= | 192.0 in |
| DL/Lvl= | 50 lb | # Levels= | 2 |
| Seismic Ovt based on E, Σ(Fi*hi)= | 139,987 in-lb | # Anchors/Base= | 2 |
| height/depth ratio= | 4.6 in | hp= | 48.0 in |
| | | h=H+hp/2= | 216.0 in |



SIDE ELEVATION

**A) Fully Loaded Rack**

**Load case 1:**

Movt= Σ(Fi*hi)*E*rho
= 139,987 in-lb

Mst= Wst1 * Df/2
= 3050 lb * 42 in/2
= 64,050 in-lb

T= (Movt-Mst)/Df
= (139987 in-lb - 64050 in-lb)/42 in
= 1,808 lb **Net Uplift per Column**

**Net Seismic Uplift= 1,808 lb**

**B) Top Level Loaded Only**

**Load case 1:**

V1=Vtop= Cs * Ip * Ptop >= 350 lb for H/D >6.0
= 0.256 * 3200 lb
= 819 lb

V1eff= 819 lb
Critical Level= 2

$V2=V_{DL}$= Cs*Ip*D
Cs*Ip= 0.2560
= 26 lb

Mst= (0.6952*D + 0.6952*Ptop*1) * 42 in/2
= 48,177 in-lb

Movt= [V1*h + V2 * H/2]*rho
= 179,405 in-lb

T= (Movt-Mst)/Df
= (179405 in-lb - 48177 in-lb)/42 in
= 3,124 lb **Net Uplift per Column**

**Net Seismic Uplift= 3,124 lb**

**Anchor**

**Check (2) 0.5" x 3.25" Embed HILTI KWIKBOLT TZ anchor(s) per base plate.**
**Special inspection is required per ESR 1917.**

| | | | | | |
|---|---|---|---|---|---|
| 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: | (904 lb/1961 lb)^1 + (280 lb/2517 lb)^1 = | 0.57 | <= 1.2 | OK |
| Top Level Loaded: | (1562 lb/1961 lb)^1 + (204 lb/2517 lb)^1 = | 0.88 | <= 1.2 | OK |

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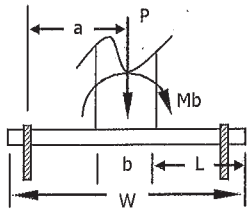
By: Bob S | Project: Ingersoll Rand | Project #: 23-1109-15

**Base Plate** Configuration: TYPE 1 SELECTIVE RACK

**Section**

Baseplate= 8x5x0.375
Eff Width=W = 8.00 in
Eff Depth=D = 5.00 in
Column Width=b = 3.00 in
Column Depth=dc = 3.00 in
L = 2.50 in
Plate Thickness=t = 0.375 in

a = 3.00 in
Anchor c.c. =2*a=d = 6.00 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= 50 lb
COLUMN PL= 3,200 lb
Base Moment= 8,000 in-lb
1+0.105*Sds= 1.1075
1.4+0.14Sds= 1.5434
B= 0.7000

**Axial=P= 1.10752 * 50 lb + 0.75 * (1.54336 * 0.7 * 3200 lb)**
**= 2,648 lb**
**Mb= Base Moment*0.75*0.7*rho**
**= 8000 in-lb * 0.75*0.7*rho**
**= 4,200 in-lb**

**Axial Load P = 2,648 lb** **Mbase=Mb = 4,200 in-lb**

Axial stress=fa = P/A = P/(D*W)
= 66 psi
Moment Stress=fb = M/S = 6*Mb/[(D*B^2]
= 78.8 psi
Moment Stress=fb1 = fb-fb2
= 29.5 psi
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2
= 103 in-lb
S-plate = (1)(t^2)/6
= 0.023 in^3/in
fb/Fb = Mtotal/[(S-plate)(Fb)]
= **0.63** **OK**
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2]
= -825 lb No Tension

M1= wL^2/2= fa*L^2/2
= 207 in-lb
Moment Stress=fb2 = 2 * fb * L/W
= 49.2 psi
M2= fb1*L^2)/2
= 92 in-lb
Mtotal = M1+M2+M3
= 402 in-lb/in
Fb = 0.75*Fy
= 27,000 psi
F'p= 0.7*F'c
= 4,986 psi OK
Tallow= 1,961 lb **OK**

**Cross Aisle Loads** *Critical load case RMI Sec 2.1, item 4: (1+0.115ds)DL + (1+0.14SDS)PL*0.75+EL*0.75 <= 1.0, ASD Method*

**Pstatic= 2,648 lb**

Movt*0.75*0.7*rho= 73,493 in-lb
Frame Depth= 42.0 in
**P=Pstatic+Pseismic= 4,398 lb**

Pseismic= Movt/Frame Depth
= 1,750 lb

b =Column Depth= 3.00 in

L =Base Plate Depth-Col Depth= 2.50 in

fa = P/A = P/(D*W)
= 110 psi

M= wL^2/2= fa*L^2/2
= 344 in-lb/in

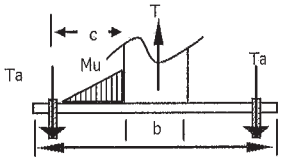
Sbase/in = (1)(t^2)/6
= 0.023 in^3/in

Fbase = 0.75*Fy
= 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.54** **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= 3,124 lb
Qty Anchor per BP= 2
Net Tension per anchor=Ta= 1,562 lb
c= 2.50 in
Mu=Moment on Baseplate due to uplift= Ta*c/2
= 1,953 in-lb
Splate= 0.117 in^3
[fb/Fb]*0.75= 0.463 **OK**

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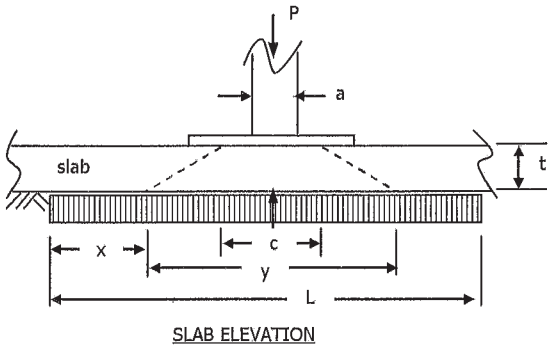
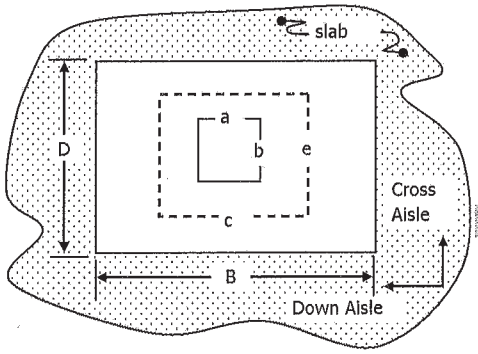
By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**Slab on Grade** Configuration: TYPE 1 SELECTIVE RACK



SLAB ELEVATION



Baseplate Plan View

Concrete
f'c= 7,123 psi
tslab=t= 5.5 in
teff= 5.5 in
phi=Ø= 0.6

Soil
fsoil= 750 psf
Movt= 176,947 in-lb
Frame depth= 42.0 in
Sds= 1.024
0.2*Sds= 0.205
λ= 0.600
β=B/D= 1.600
F'c^0.5= 84.40 psi

Base Plate
Effec. Baseplate width=B= 8.00 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= 5.50 in
midway dist face of column to edge of plate=e= 4.00 in

**Column Loads**

DEAD LOAD=D= 50 lb per column
*unfactored ASD load*
PRODUCT LOAD=P= 3,200 lb per column
*unfactored ASD load*
Papp= 2,144 lb per column
P-seismic=E= (Movt/Frame depth)
= 4,213 lb per column
*unfactored Limit State load*
B= 0.7000
rho= 1.0000
Sds= 1.0240
1.2 + 0.2*Sds= 1.4048
0. 9 - 0.20Sds= 0.6952

Load Case 1) (1.2+0.2Sds)D + (1.2+0.2Sds)*B*P+ rho*E RMI SEC 2.2 EQTN 5
= 1.4048 * 50 lb + 1.4048 * 0.7 * 3200 lb + 1 * 4213 lb
= 7,430 lb
Load Case 2) (0.9-0.2Sds)D + (0.9-0.2Sds)*B*Papp + rho*E RMI SEC 2.2 EQTN 7
= 0.6952 * 50 lb + 0.6952 * 0.7 * 2144 lb + 1 * 4213 lb
= 5,291 lb
Load Case 3) 1.2*D + 1.4*P RMI SEC 2.2 EQTN 1,2
= 1.2*50 lb + 1.4*3200 lb
= 4,540 lb
Load Case 4) 1.2*D + 1.0*P + 1.0E ACI 318-14 Sec 5.3.1
= 7,473 lb Eqtn 5.3.1e

**Effective Column Load=Pu= 7,473 lb per column**

**Puncture**

Apunct= [(c+t)+(e+t)]*2*t
= 225.50 in^2
Fpunct1= [(4/3 + 8/(3*β)] * λ *(F'c^0.5)
= 151.9 psi
Fpunct2= 2.66 * λ * (F'c^0.5)
= 134.7 psi
Fpunct eff= 134.7 psi

fv/Fv= Pu/(Apunct*Fpunct)
= **0.246** **< 1 OK**

**Slab Bending**

Pse=DL+PL+E= 7,473 lb

Asoil= (Pse*144)/(fsoil)
= 1,435 in^2
x= (L-y)/2
= 11.1 in
Fb= 5*(phi)*(f'c)^0.5
= 253.19 psi

L= (Asoil)^0.5
= 37.88 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 320.6 in-lb

y= (c*e)^0.5 + 2*t
= 15.7 in
S-slab= 1*teff^2/6
= 5.04 in^3
fb/Fb= M/(S-slab*Fb)
= **0.251** **< 1, OK**

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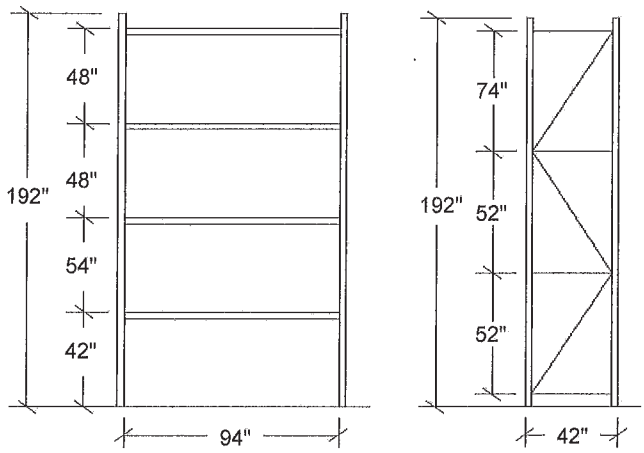
By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**Configuration & Summary: TYPE 2 SELECTIVE RACK**




***RACK COLUMN REACTIONS*
*ASD LOADS*
*AXIAL DL= 100 lb*
*AXIAL LL= 5,600 lb*
*SEISMIC AXIAL Ps=+/- 5,374 lb*
*BASE MOMENT= 8,000 in-lb*

| Seismic Criteria | # Bm Lvls | Frame Depth | Frame Height | # Diagonals | Beam Length | Frame Type |
|---|---|---|---|---|---|---|
| Ss=1.28, Fa=1.2 | 4 | 42 in | 192.0 in | 3 | 94 in | Single Row |

| Component | Description | | | | STRESS |
|---|---|---|---|---|---|
| Column | Fy=55 ksi | INTLK LU70/3x3x14ga | P=4275 lb, M=10754 in-lb | | 0.49-OK |
| Column & Backer | None | None | None | | N/A |
| Beam | Fy=55 ksi | Intlk 311/3.31"deepx2.75"x0.07" | Lu=94 in | Capacity: 3746 lb/pr | 0.75-OK |
| Beam Connector | Fy=55 ksi | Lvl 1: 2 pin OK | Mconn=8265 in-lb | Mcap=9726 in-lb | 0.85-OK |
| Brace-Horizontal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | 0.24-OK |
| Brace-Diagonal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | 0.99-OK |
| Base Plate | Fy=36 ksi | 8x5x0.375 | | Fixity= 8000 in-lb | 0.92-OK |
| Anchor | 2 per Base | 0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=2760 lb) | | | 0.733-OK |
| Slab & Soil | 5.5" thk x 7123 psi slab on grade. 750 psf Soil Bearing Pressure | | | | 0.47-OK |

| Level | Load** Per Level | Beam Spcg | Brace | Story Force Transv | Story Force Longit. | Column Axial | Column Moment | Conn. Moment | Beam Connector |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2,800 lb | 42.0 in | 52.0 in | 175 lb | 77 lb | 5,700 lb | 9,043 "# | 8,265 "# | 2 pin OK |
| 2 | 2,800 lb | 54.0 in | 52.0 in | 399 lb | 177 lb | 4,275 lb | 10,754 "# | 7,702 "# | 2 pin OK |
| 3 | 2,800 lb | 48.0 in | 74.0 in | 599 lb | 266 lb | 2,850 lb | 7,435 "# | 5,425 "# | 2 pin OK |
| 4 | 2,800 lb | 48.0 in | | 799 lb | 354 lb | 1,425 lb | 4,249 "# | 2,823 "# | 2 pin OK |

** Load defined as product weight per pair of beams Total: 1,972 lb 874 lb

**Notes**

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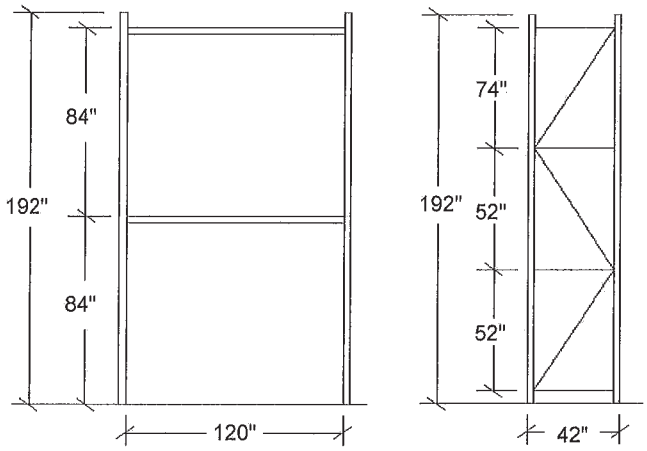
By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**Configuration & Summary: TYPE 3 SELECTIVE RACK**




***RACK COLUMN REACTIONS*
*ASD LOADS*
*AXIAL DL= 50 lb*
*AXIAL LL= 4,000 lb*
*SEISMIC AXIAL Ps=+/- 3,690 lb*
*BASE MOMENT= 8,000 in-lb*

| Seismic Criteria | # Bm Lvls | Frame Depth | Frame Height | # Diagonals | Beam Length | Frame Type |
|---|---|---|---|---|---|---|
| Ss=1.28, Fa=1.2 | 2 | 42 in | 192.0 in | 3 | 120 in | Single Row |

| Component | Description | | | | STRESS |
|---|---|---|---|---|---|
| Column | Fy=55 ksi | INTLK LU70/3x3x14ga | P=4050 lb, M=17110 in-lb | | 0.92-OK |
| Column & Backer | None | None | None | | N/A |
| Beam | Fy=55 ksi | Intlk 50E 5Hx2.75Wx0.059"Thk | Lu=120 in | Capacity: 5516 lb/pr | 0.73-OK |
| Beam Connector | Fy=55 ksi | Lvl 1: 3 pin OK | Mconn=10349 in-lb | Mcap=12691 in-lb | 0.82-OK |
| Brace-Horizontal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | 0.17-OK |
| Brace-Diagonal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | 0.7-OK |
| Base Plate | Fy=36 ksi | 8x5x0.375 | | Fixity= 8000 in-lb | 0.71-OK |
| Anchor | 2 per Base | 0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3306 lb) | | | 0.783-OK |
| Slab & Soil | 5.5" thk x 7123 psi slab on grade. 750 psf Soil Bearing Pressure | | | | 0.33-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 | 84.0 in | 52.0 in | 466 lb | 207 lb | 4,050 lb | 17,110 "# | 10,349 "# | 3 pin OK |
| 2 | 4,000 lb | 84.0 in | 52.0 in | 932 lb | 413 lb | 2,025 lb | 8,679 "# | 4,361 "# | 3 pin OK |
| | | | 74.0 in | | | | | | |

** Load defined as product weight per pair of beams Total: 1,398 lb 620 lb

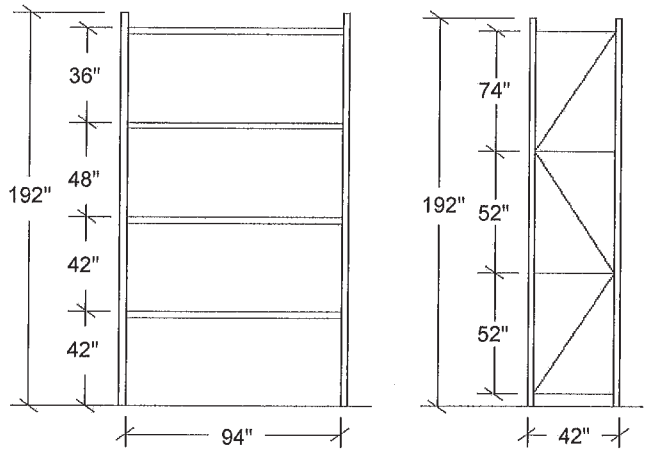
**Notes**

Structural
Engineering & Design Inc.

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

By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**Configuration & Summary: TYPE 4 SELECTIVE RACK**




***RACK COLUMN REACTIONS*
*ASD LOADS*
*AXIAL DL= 100 lb*
*AXIAL LL= 5,600 lb*
*SEISMIC AXIAL Ps=+/- 4,838 lb*
*BASE MOMENT= 8,000 in-lb*

| Seismic Criteria | # Bm Lvls | Frame Depth | Frame Height | # Diagonals | Beam Length | Frame Type |
|---|---|---|---|---|---|---|
| Ss=1.28, Fa=1.2 | 4 | 42 in | 192.0 in | 3 | 94 in | Single Row |

| Component | Description | | | | | STRESS |
|---|---|---|---|---|---|---|
| Column | Fy=55 ksi | INTLK LU70/3x3x14ga | | P=5700 lb, M=9043 in-lb | | 0.49-OK |
| Column & Backer | None | None | | None | | N/A |
| Beam | Fy=55 ksi | Intlk 311/3.31"deepx2.75"x0.07" | | Lu=94 in | Capacity: 3746 lb/pr | 0.75-OK |
| Beam Connector | Fy=55 ksi | Lvl 1: 2 pin OK | Mconn=7396 in-lb | | Mcap=9726 in-lb | 0.76-OK |
| Brace-Horizontal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.24-OK |
| Brace-Diagonal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.99-OK |
| Base Plate | Fy=36 ksi | 8x5x0.375 | | | Fixity= 8000 in-lb | 0.89-OK |
| Anchor | 2 per Base | 0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=2336 lb) | | | | 0.625-OK |
| Slab & Soil | 5.5" thk x 7123 psi slab on grade. 750 psf Soil Bearing Pressure | | | | | 0.44-OK |

| Level | Load** Per Level | Beam Spcg | Brace | Story Force Transv | Story Force Longit. | Column Axial | Column Moment | Conn. Moment | Beam Connector |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2,800 lb | 42.0 in | 52.0 in | 194 lb | 86 lb | 5,700 lb | 9,043 "# | 7,396 "# | 2 pin OK |
| 2 | 2,800 lb | 42.0 in | 52.0 in | 389 lb | 172 lb | 4,275 lb | 8,272 "# | 6,816 "# | 2 pin OK |
| 3 | 2,800 lb | 48.0 in | 74.0 in | 611 lb | 271 lb | 2,850 lb | 7,386 "# | 5,007 "# | 2 pin OK |
| 4 | 2,800 lb | 36.0 in | | 778 lb | 345 lb | 1,425 lb | 3,102 "# | 2,421 "# | 2 pin OK |

** Load defined as product weight per pair of beams Total: 1,972 lb 874 lb

**Notes**

Structural
Engineering & Design Inc.

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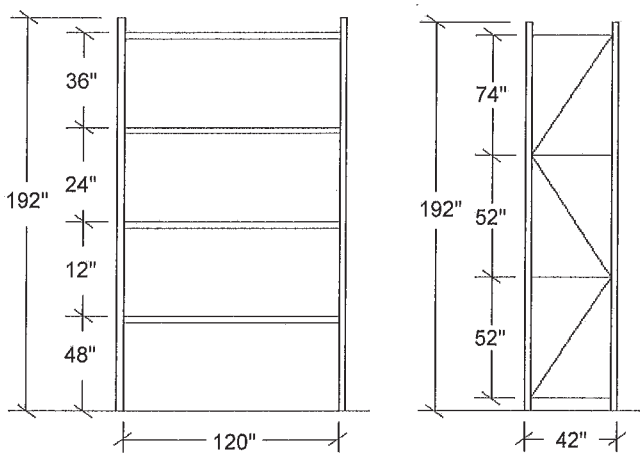
By: Bob S Project: Ingersoll Rand Project #: 23-1109-15

**Configuration & Summary: TYPE 5 SELECTIVE RACK**




***RACK COLUMN REACTIONS*
*ASD LOADS*
*AXIAL DL= 100 lb*
*AXIAL LL= 5,600 lb*
*SEISMIC AXIAL Ps=+/- 3,558 lb*
*BASE MOMENT= 8,000 in-lb*

| Seismic Criteria | # Bm Lvls | Frame Depth | Frame Height | # Diagonals | Beam Length | Frame Type |
|---|---|---|---|---|---|---|
| Ss=1.28, Fa=1.2 | 4 | 42 in | 192.0 in | 3 | 120 in | Single Row |

| Component | Description | | | | | STRESS |
|---|---|---|---|---|---|---|
| Column | Fy=55 ksi | INTLK LU70/3x3x14ga | | P=5700 lb, M=11665 in-lb | | 0.58-OK |
| Column & Backer | None | None | | None | | N/A |
| Beam | Fy=55 ksi | Intlk 50E 5Hx2.75Wx0.059"Thk | | Lu=120 in | Capacity: 5516 lb/pr | 0.51-OK |
| Beam Connector | Fy=55 ksi | Lvl 1: 3 pin OK | Mconn=5785 in-lb | | Mcap=12691 in-lb | 0.46-OK |
| Brace-Horizontal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.24-OK |
| Brace-Diagonal | Fy=55 ksi | Intlk 1-1/2x1-1/2x14ga | | | | 0.99-OK |
| Base Plate | Fy=36 ksi | 8x5x0.375 | | | Fixity= 8000 in-lb | 0.88-OK |
| Anchor | 2 per Base | 0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=1488 lb) | | | | 0.375-OK |
| Slab & Soil | 5.5" thk x 7123 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 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 2,800 lb | 48.0 in | 52.0 in | 303 lb | 135 lb | 5,700 lb | 11,665 "# | 5,785 "# | 3 pin OK |
| 2 | 2,800 lb | 12.0 in | 52.0 in | 379 lb | 168 lb | 4,275 lb | 2,219 "# | 2,903 "# | 3 pin OK |
| 3 | 2,800 lb | 24.0 in | 74.0 in | 531 lb | 235 lb | 2,850 lb | 3,428 "# | 3,185 "# | 3 pin OK |
| 4 | 2,800 lb | 36.0 in | | 759 lb | 336 lb | 1,425 lb | 3,025 "# | 1,985 "# | 3 pin OK |

** Load defined as product weight per pair of beams Total: 1,972 lb 874 lb

**Notes**

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

# SHELVING ANALYSIS

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Design Data** Configuration: Type A Shelving: 87" H x 48" W x 24" D

1) The analyses of the light duty storage fixtures conforms to the requirements of the 2018 IBC and ASCE 7-16

2) All steel conforms to ASTM A36, Gr. 36 with minimum yield, Fy= 36 ksi unless otherwise noted on the plans or analysis herein.

3) Anchor bolts shall be provided by installer per ICC reference on the 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 above grade floor is designed by others

6) All Hardawre shall conform to Grade 5 properties or better, installed to snug tight fit.

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Engineering & Design Inc.

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type A Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 200 lb |
| 2 | 42.00 in | 200 lb |
| 3 | 42.00 in | 200 lb |

**Load Reactions**

| | |
|---|---|
| Axial column DL= | 20 lb |
| Axial column LL= | 300 lb |
| Axial column seismic load=+/- | 536 lb |
| Net Seismic Uplift per footplate= | 1,035 lb |

**Configuration**

| | |
|---|---|
| # of Levels= | 3 |
| Height= | 87.00 in |
| Width= | 48.00 in |
| Depth= | 24.00 in |

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 200 lb per shelf level

**Component Summary**

| | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.14** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.44** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.78** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.94** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.03** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type B Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 160 lb |
| 2 | 28.00 in | 160 lb |
| 3 | 28.00 in | 160 lb |
| 4 | 28.00 in | 160 lb |

**Load Reactions**

Axial column DL= 25 lb
Axial column LL= 320 lb
Axial column seismic load=+/- 546 lb
Net Seismic Uplift per footplate= 1,040 lb

**Configuration**

# of Levels= 4
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 160 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.12** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.35** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.80** | **OK** |
| **X Brace** | **1" x 13ga strap w/(1) 0.25 in diam bolt per end (Grade 5 Bolt)** | **0.18** | **OK** |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.95** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.17** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.04** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type C Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 120 lb |
| 2 | 21.00 in | 120 lb |
| 3 | 21.00 in | 120 lb |
| 4 | 21.00 in | 120 lb |
| 5 | 21.00 in | 120 lb |

**Load Reactions**

Axial column DL= 30 lb
Axial column LL= 300 lb
Axial column seismic load=+/- 509 lb
Net Seismic Uplift per footplate= 963 lb

**Configuration**

# of Levels= 5
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 120 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.26** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.74** | **OK** |
| **X Brace** | **1" x 13ga strap w/(1) 0.25 in diam bolt per end (Grade 5 Bolt)** | **0.17** | **OK** |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.88** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.03** | **OK** |

**Notes :**

**Ensure to install anchors at locations of minimum 5" concrete thickness**

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type C1 Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 120 lb |
| 2 | 21.00 in | 120 lb |
| 3 | 21.00 in | 120 lb |
| 4 | 21.00 in | 120 lb |
| 5 | 21.00 in | 120 lb |

**Load Reactions**

Axial column DL= 30 lb
Axial column LL= 300 lb
Axial column seismic load=+/- 509 lb
Net Seismic Uplift per footplate= 963 lb

**Configuration**

# of Levels= 5
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 120 lb per shelf level

**Component Summary**

| | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.26** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.74** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.88** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.03** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

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Engineering & Design Inc.

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type D Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 100 lb |
| 2 | 17.00 in | 100 lb |
| 3 | 17.00 in | 100 lb |
| 4 | 17.00 in | 100 lb |
| 5 | 17.00 in | 100 lb |
| 6 | 16.00 in | 100 lb |

**Load Reactions**

Axial column DL= 35 lb
Axial column LL= 300 lb
Axial column seismic load=+/- 512 lb
Net Seismic Uplift per footplate= 966 lb

**Configuration**

# of Levels= 6
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 100 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.22** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.75** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.88** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.03** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

By: Bz Eng: Mqz  Project: INGERSOLL RAND  Project #: 23-1109-15

**Summary of Results**  **Configuration: Type D1 Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 100 lb |
| 2 | 17.00 in | 100 lb |
| 3 | 17.00 in | 100 lb |
| 4 | 17.00 in | 100 lb |
| 5 | 17.00 in | 100 lb |
| 6 | 16.00 in | 100 lb |

**Load Reactions**

Axial column DL= 35 lb
Axial column LL= 300 lb
Axial column seismic load=+/- 512 lb
Net Seismic Uplift per footplate= 966 lb

**Configuration**

# of Levels= 6
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 100 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.22** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.75** | **OK** |
| **X Brace** | **1" x 13ga strap w/(1) 0.25 in diam bolt per end (Grade 5 Bolt)** | **0.17** | **OK** |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.88** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.03** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type E Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 90 lb |
| 2 | 14.00 in | 90 lb |
| 3 | 14.00 in | 90 lb |
| 4 | 14.00 in | 90 lb |
| 5 | 14.00 in | 90 lb |
| 6 | 14.00 in | 90 lb |
| 7 | 14.00 in | 90 lb |

**Load Reactions**

Axial column DL= 40 lb
Axial column LL= 315 lb
Axial column seismic load=+/- 535 lb
Net Seismic Uplift per footplate= 1,004 lb

**Configuration**

# of Levels= 7
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 90 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.20** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.78** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.93** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.17** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.04** | **OK** |

**Notes :**

Ensure to install anchors at locations of minimum 5" concrete thickness

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type F Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 75 lb |
| 2 | 12.00 in | 75 lb |
| 3 | 12.00 in | 75 lb |
| 4 | 12.00 in | 75 lb |
| 5 | 12.00 in | 75 lb |
| 6 | 12.00 in | 75 lb |
| 7 | 12.00 in | 75 lb |
| 8 | 12.00 in | 75 lb |

**Load Reactions**

Axial column DL= 45 lb
Axial column LL= 300 lb
Axial column seismic load=+/- 519 lb
Net Seismic Uplift per footplate= 972 lb

**Configuration**

# of Levels= 8
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 75 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.16** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.76** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.89** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.16** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.04** | **OK** |

**Notes :**

**Ensure to install anchors at locations of minimum 5" concrete thickness**

Structural
Engineering & Design Inc.

1815 Wright Ave., La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.593.8561

**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Summary of Results** **Configuration: Type G Shelving: 87" H x 48" W x 24" D**

| Load Level | Elevation | Load |
|---|---|---|
| 1 | 3.00 in | 70 lb |
| 2 | 10.50 in | 70 lb |
| 3 | 10.50 in | 70 lb |
| 4 | 10.50 in | 70 lb |
| 5 | 10.50 in | 70 lb |
| 6 | 10.50 in | 70 lb |
| 7 | 10.50 in | 70 lb |
| 8 | 10.50 in | 70 lb |
| 9 | 10.50 in | 70 lb |

**Load Reactions**

Axial column DL= 50 lb
Axial column LL= 315 lb
Axial column seismic load=+/- 546 lb
Net Seismic Uplift per footplate= 1,022 lb

**Configuration**

# of Levels= 9
Height= 87.00 in
Width= 48.00 in
Depth= 24.00 in

Seismic Coeff: Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860

Steel Fy= 33,000 psi
Product Load= 70 lb per shelf level

| Component Summary | | | |
|---|---|---|---|
| **Column** | **Rousseau T Post 14ga** | **0.11** | **OK** |
| **Shelf** | **20 ga Shelf** | **0.15** | **OK** |
| **Steel Panel** | **24 ga Back Panel with (1) 1/4" diam bolt at 18" o.c., 24 ga Side Panel with weld or bolt at 12" o.c.** | **0.80** | **OK** |
| | | | |
| **Anchor**** | **(2)3/8" x 2-1/2" Embed HILTI KB TZ2 (ICC ESR 4266) per footplate. Special Inspec. Reqd During Install** | **0.94** | **OK** |
| **Footplate** | **5.5 in * 4 in * 0.25 in Thk [with (1) 1/4"Diam Gr5 M.B. to Post]** | **0.17** | **OK** |
| **Slab** | **5.5 in thick x 7123 psi slab/750 psf soil** | **0.04** | **OK** |

**Notes :**

**Ensure to install anchors at locations of minimum 5" concrete thickness**

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Seismic Forces** Configuration: Type A Shelving: 87" H x 48" W x 24" D

**V1= $S_{DS}$ *W/R**
**V2= [0.4*ap*$S_{DS}$*Ws*(1+2*z/h)/(Rp/Ip)]**
**V3= 0.044*Sds**
**V4= 0.5*S1/R**

$V_{minimum}$= 0.015

V1= 0.4096
V2= 0.1638
V3= 0.0451
V4= 0.0000
Seismic Coeff=Cs= 0.4096
(Either Direction)

W= DL + LL*0.67
= 442 lb

Cs * Ip= 0.4096

V= Cs * W
= 181 lb

# Load Levels= 3
Ss= 1.280
S1= 0.440
Fa= 1.200
Fv= 1.860
Sds= 1.024
Sd1= 0.546
Rp= 2.5
ap= 1.0
Ip= 1.00
R=Rp= 2.50
z/h= 0.00
*Grnd Floor Install*
Depth=D= 24.0 in

Structura

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**Seismic Forces** Configuration: Type A Shelving: 87" H x 48" W x 24" D

**Transverse Distribution**

| Level | LL | DL | hi | wi*hi | Fi | Fi*hi |
|---|---|---|---|---|---|---|
| 1 | 200 lb | 10 lb | 3 in | 630 | 4.0 lb | 12 in-lb |
| 2 | 200 lb | 10 lb | 45 in | 9,450 | 60.3 lb | 2,715 in-lb |
| 3 | 200 lb | 10 lb | 87 in | 18,270 | 116.6 lb | 10,148 in-lb |
| Sum: | 600 lb | 40 lb | | 28,350 | 181 lb | 12,875 in-# = Movt |

**Transverse Column Loads**

Axial DL per post= 20 lb
Axial LL per post= 300 lb

Movt= 12,875 in-lb

**All lateral loads taken by the closed steel panels or X bracing**

Pseismic= Movt/D
= 536 lb

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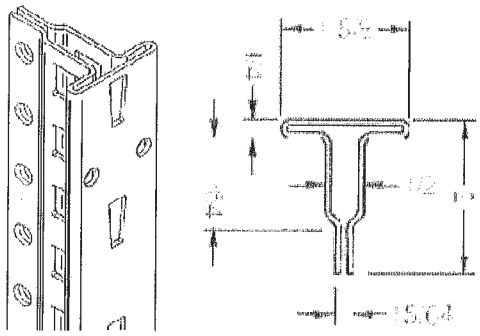
**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Rear Double "T" Post** Configuration: Type A Shelving: 87" H x 48" W x 24" D

## Net Section Properties

Column= Rousseau T Post 14ga
Aeff = 0.297 in^2
Ix (downaisle) = 0.057 in^4
Sx (downaisle) = 0.070 in^3
rx (downaisle) = 0.438 in
Iy (crossaisle) = 0.081 in^4
Sy (crossaisle) = 0.063 in^3
ry (crossaisle) = 0.525 in
Fy= 33 ksi
Axial DL= 20 lb
Axial LL= 300 lb
Pseismic= 536 lb

Ωf= 1.67
E= 29,500 ksi
Cb= 1.0
Cmx= 0.85
Kx = 1.0
Lx = 42.0 in
Ky = 1.0
Ly = 42.0 in



**Loads** Load Case: (Fully Loaded)

**Axial=P= DL+0.75LL+0.75*0.7*Pseismic**
**= 526 lb** **Moment=My= 0 in-lb**

## Axial Analysis

KxLx/rx = 1*42"/0.438"
= 95.9

KyLy/ry = 1*42"/0.525"
= 80.0

Fe > Fy/2
Fn= Fy(1-Fy/4Fe)
= 33 ksi*[1-33 ksi/(4*31.7 ksi)]
= 24.4 ksi

Fe= π^2E/(KL/r)max^2
= 31.7ksi

Fy/2= 16.5 ksi

Pn= Aeff*Fn
= 7,247 lb

Ωc= 1.92

Pa= Pn/Ωc
= 7247 lb/1.92
= 3,775 lb

P/Pa= 0.14 < 0.15

## Bending Analysis

Check: P/Pa + My/May ≤ 1.0

Pno= Ae*Fy
= 0.297 in^2 *33000 psi
= 9,801 lb

Pao= Pno/Ωc
= 9801lb/1.92
= 5,105 lb

May= My/Ωf
= 2079 in-lb/1.67
= 1,245 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500000 psi/(1*42 in)^2
= 9,408 lb

Myield=My= Sy*Fy
= 0.063 in^3 * 33000 psi
= 2,079 in-lb

μ= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*526 lb/9408 lb)]}^-1
= 0.89

## Combined Stresses

(526 lb/3775 lb) + (0 in-lb/1245 in-lb) = 0.10 < 1.0, OK

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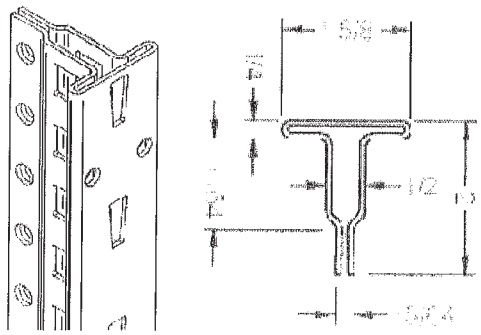
**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Front Box Post** Configuration: Type A Shelving: 87" H x 48" W x 24" D

## Net Section Properties

Column= Rousseau T Post 14ga
Aeff = 0.297 in^2
Ix (downaisle) = 0.057 in^4
Sx (downaisle) = 0.070 in^3
rx (downaisle) = 0.438 in
Iy (crossaisle) = 0.081 in^4
Sy (crossaisle) = 0.063 in^3
ry (crossaisle) = 0.5 in
Fy= 33 ksi
Axial DL= 20 lb
Axial LL= 300 lb
Pseismic= 536 lb

Ωf= 1.67
E= 29,500 ksi
Cb= 1.0
Cmx= 0.85
Kx = 1.0
Lx = 0.0 in
Ky = 1.0
Ly = 42.0 in



**Loads** Load Case: (Fully Loaded)

**Axial=P= DL+0.75LL+0.75*0.7*Pseismic**
**526 lb** **Moment=Mx= 0 in-lb**

## Axial Analysis

KxLx/rx = 1*0"/0.438"
= 0.0

KyLy/ry = 1*42"/0.525"
= 80.0

Fe > Fy/2
Fn= Fy(1-Fy/4Fe)
= 33 ksi*[1-33 ksi/(4*45.5 ksi)]
= 27.0 ksi

Fe= π^2E/(KL/r)max^2
= 45.5ksi

Fy/2= 16.5 ksi

Pn= Aeff*Fn
= 8,024 lb

Ωc= 1.92

Pa= Pn/Ωc
= 8024 lb/1.92
= 4,179 lb

P/Pa= 0.13 <= 0.15

## Bending Analysis

Check: P/Pa + My/May ≤ 1.0

Pno= Ae*Fy
= 0.297 in^2 *33000 psi
= 9,801 lb

Pao= Pno/Ωc
= 9801lb/1.92
= 5,105 lb

May= My/Ωf
= 2310 in-lb/1.67
= 1,383 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500000 psi/(1*42 in)^2
= 13,369 lb

Myield=My= Sx*Fy
= 0.07 in^3 * 33000 psi
= 2,310 in-lb

μ= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*526 lb/13369 lb)]}^-1
= 0.92

## Combined Stresses

(526 lb/4179 lb) + (0 in-lb/1383 in-lb) = 0.10 < 1.0, OK (EQ C5-3)

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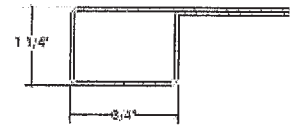
**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

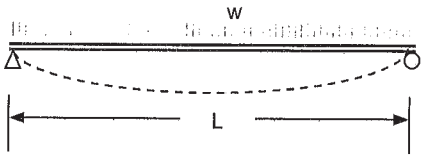
**Shelf** Configuration: Type A Shelving: 87" H x 48" W x 24" D

It is assumed that the downaisle lips will carry 2/3 of the total shelf load, with the remainder carried by the transverse lips

Beam Type= 20 ga Shelf
Ix= 0.0350 in^4
Sx= 0.046 in^3
Fy-beam= 33,000 psi
Shelf Span=L= 48 in
Max Shelf LL= 200 lb
Load=w=LL*(2/3)/(2*L)= 16.5 plf




20 GAGE SH SHELF




Beam Diagram

**Check Beam Bending**

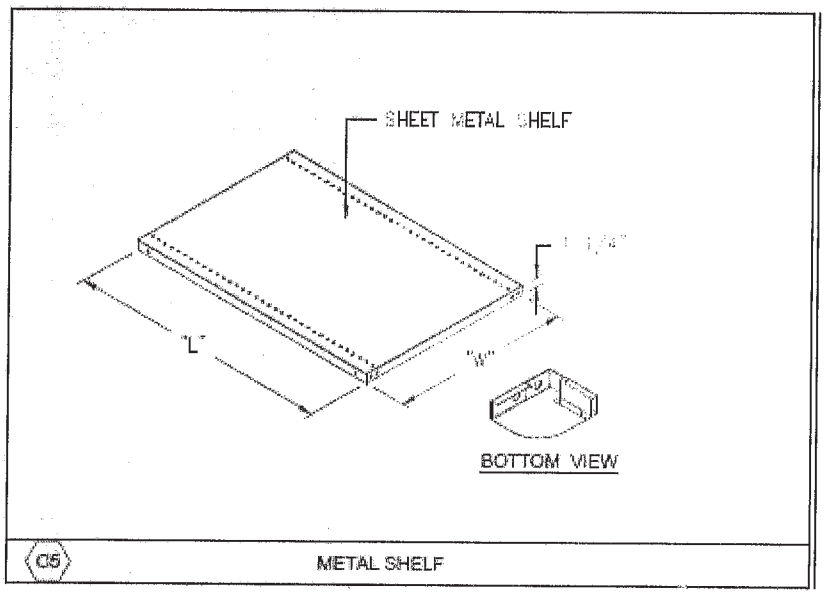
M= w * L^2/8
= 396 in-lb

fb= M/Sy
= 8,609 psi

Fb= 0.6 * Fy
19,800 psi
**fb/Fb= 0.44 OK**

**Check Beam Deflection**

E= 29,500,000 psi
D= 5 * w * L^4/(384 * E * Ix)
= 0.0950 in
Dallow= L/180
= **0.27 in** **OK**




SR22 12GA TYP.

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By: Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

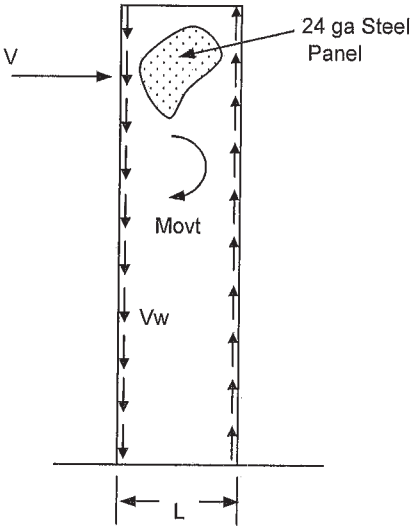
## Closed Back Panel — Configuration: Type A Shelving: 87" H x 48" W x 24" D

The closed back and side panels will act as a steel diaphragm transferring the lateral loads to the floor.

Shear per Frame=V= 181 lb
L= 48.0 in
Height= 87.0 in
Movt= Σ(Fi*hi)*1.15*0.7
= 29,613 in-#
Omega= 2.0
Bolt Frequency= 12 in o.c.
# Bolt/Post=N= 9
Seismic Load per bolt=Vb= (Movt/D)/N
= 69 lb

Check Bolt

Fv= 28,000 psi
Fu= 65,000 psi
Bolt Diam= 0.25 in
tmin= 0.0300 in
Bolt Shear Capacity= Bolt Area * Fv/Omega
= **687 lb** **OK**
Bolt Bearing Capacity= 1.2 * Bolt Diam *tmin * Fu /Omega
= **293 lb** **OK**
**Bolt Stress= 0.24** **OK**




Front Elevation

## Closed Side Panel

The closed back and side panels will act as a steel diaphragm transferring the lateral loads to the floor.

Shear per Frame=V= 181 lb
Frame Depth=d= 24.0 in
Height= 87.0 in
Movt= 29,613 in-#
Omega= 2.0
Weld Frequency= 12 in o.c.
# Bolt/Post=N= 8
Seismic Load per bolt=Vb= (Movt/D)/N
= 154 lb

Check Weld on Formed Steel

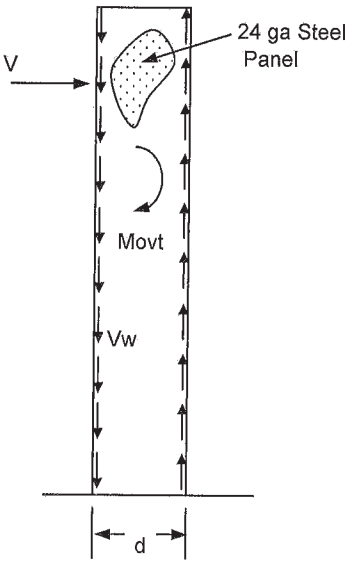
Lweld= 0.375 in

Fu= 65 ksi

tmin= 0.024 in

Weld Capacity= [1-(0.01*L/t)]* L * Fu * 1.0/2.5
= 197 lb

**Weld Stress= 0.78** **OK**




Side Elevation

# Structura Engineering & Design Inc.

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

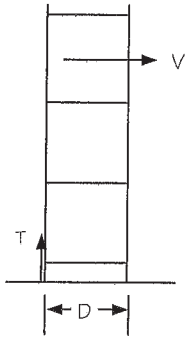
**Anchors** Configuration: Type A Shelving: 87" H x 48" W x 24" D

## Loads

Check load case: 0.9D + 0.9*0.67LL + V

| | |
|---|---|
| Type= | Sgl Row Units |
| Vtrans=V*n= | 181 lb |
| DL/Frame*n= | 40 lb |
| LL/Frame*n= | 600 lb |
| Wst=0.9*(DL+ 0.67LL)total= | 398 lb |
| LL @ TOP= | 200 lb |
| DL/Lvl= | 10 lb |
| DL*0.9= | 9 lb |
| Lateral Ovt Forces=Σ(Fi*hi)*1.15*n= | 14,806 in-lb |

| | |
|---|---|
| Frame Depth*n=D= | 24.0 in |
| Htop-lvl= | 87.0 in |
| # Levels= | 3 |
| # Anchors/Post= | 2 |
| n= | 1 |



SIDE ELEVATION

## 1) Fully Loaded rack

Vtrans= 181 lb

Movt= Σ(Fi*hi)
= 29,613 in-lb

Mst= Wst * D/2
= 398 lb * 24 in/2
= 4,776 in-lb

**Uplift=T= (Movt-Mst)/D**
**= (29613 in-lb - 4776 in-lb)/24 in**
**= 1,035 lb**

## 2) Top Level Loaded Only

Critical Level= 3
Vtop= 0.4096 * (200 lb + 10 lb)
= 86 lb

Mst= 0.6*(DL-total + LL-top) *D/2
= (200 lb * 0.67 + 10 lb*0.6) * 24 in/2
= 1,896 in-lb

Hgt @ Lvl 3= 87.0 in
Movt= Vtop*Htop*1.15
= 86 lb * 87 in * 1.15
= 17,208 in-lb

**Uplift=T= (Movt-Mst)/D**
**= (17208 in-lb - 1896 in-lb)/24 in**
**= 638 lb**

## Anchor

**Check (2) 3/8" x 2-1/2" Embed HILTI KB TZ2 anchor(s) per footplate**
**Special inspection is required per ICC ESR 4266 in SDC C,D&E**

Net Uplift1= 1,035 lb
Net Uplift2= 638 lb

Pullout Capacity=Tcap= 500 lb
Shear Capacity=Vcap= 500 lb

Phi= 1.00

Tcap*Phi= 500 lb
Vcap*Phi= 500 lb

| | | | |
|---|---|---|---|
| Fully Loaded: | (518 lb/500 lb)^1 + (45 lb/500 lb)^1 = | 1.13 | <= 1.2 OK |
| Top Level Loaded: | (319 lb/500 lb)^1 + (22 lb/500 lb)^1 = | 0.68 | <= 1.2 OK |

## Check Attachment of Post to Base Plate

Bolt Diam= 0.25 in
# Bolt=N= 1
Fv= 21,000 psi
Fu-steel= 58,000 psi
t-min= 0.150 in

Bolt Shear= Uplift*0.525
= 543 lb

Shear Capacity= Bolt Area * N * Fv
= 1,031 lb **OK**
Bearing Capacity= Bolt Diam * t-min * 1.2 * Fu
= 2,610 lb **OK**

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**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

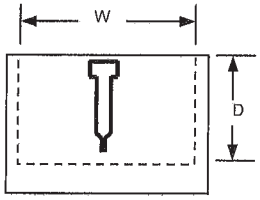
**Base Plate** Configuration: Type A Shelving: 87" H x 48" W x 24" D

**Section**

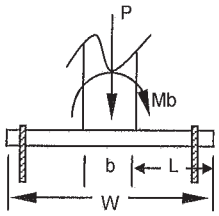
Actual base plate size is 5.5 in x 4 in x 0.25 in thk

Eff Width=W = 5.50 in
Eff Depth=D = 4.00 in
Plate Thickness=t = 0.250 in

Column Width=b = 1.625 in
Column depth=b = 2.000 in
L = 2.00 in
Fy = 36,000 psi



Base Plate Plan




**Cross Aisle Loads**

Axial DL= 20 lb
Axial L= 300 lb
Pseismic= 536 lb

DL+0.75LL + 0.75*0.7*Pseismic= 526 lb

L = Base Plate Depth-Col Depth
= 1.94 in

fa = P/A = P/(D*W)
= 24 psi

M= wL^2/2= fa*L^2/2
= 45 in-lb/in

Sbase/in = (1)(t^2)/6
= 0.01 in^3/in

Fbase = 0.75*Fy
= 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.16** **OK**

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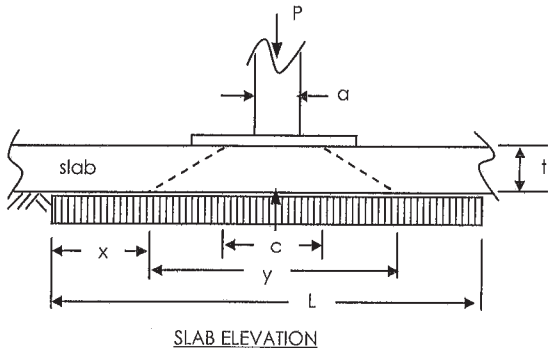
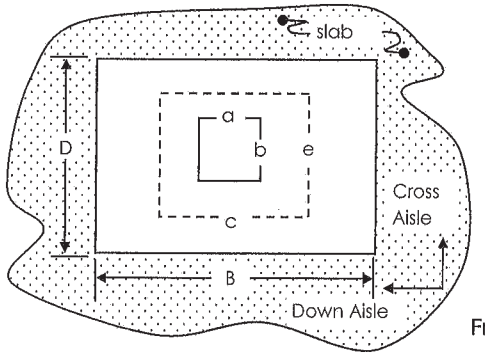
**By:** Bz Eng: Mqz **Project:** INGERSOLL RAND **Project #:** 23-1109-15

**Slab on Grade** Configuration: Type A Shelving: 87" H x 48" W x 24" D




SLAB ELEVATION




Baseplate Plan View

Concrete
f'c= 7,123 psi
tslab=t= 5.5 in
phi=Ø= 0.60

Soil
fsoil= 750 psf
Movt= 12,875 in-lb
Frame depth= 24.0 in

Base Plate
B= 5.50 in
D= 4.00 in

width=a= 1.63 in
depth=b= 2.00 in

eff. baseplate width=c= 3.56 in
eff. baseplate depth=e= 3.00 in

## Load Case 1: Product + Seismic

Product DL= 30 lb
Product LL= 300 lb

P-seismic=E= (Movt/Frame depth)
= 536 lb

(Strength Design Loads)

**Puncture**

Pu= 1.2DL + 1.0LL + 1.0*E
= 872 lb
Apunct= [(c+t)+(e+t)]*2*t
= 193.19 in^2

Fpunct= 2.66*sqrt(f'c)*phi
= 134.7 psi
fv/Fv= Pu/(Apunct*Fpunct)
= **0.03** **< 1.0 OK**

**Slab Bending**

Asoil= (P*144)/(fsoil)
= 168 in^2
x= (L-y)/2
= 0.0 in
Fb= 5*(phi)*(f'c)^0.5
= 253.19 psi

L= (Asoil)^0.5
= 12.96 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 0 in-lb

y= (c*e)^0.5 + t*2
= 14.3 in
S-slab= 1*t^2/6
= 5.04 in^3
fb/Fb= M/(S-slab*Fb)
= **0.00** **< 1.0 OK**

## Load Case 2: Static Loads

PDL= 30 lb PLL= 300 lb

**Puncture**

Pu= 1.2*PDL + 1.6*PLL
= 516 lb
Apunct= [(c+t)+(e+t)]*2*t
= 193 in^2

Fpunct= 2.66*sqrt(f'c)*phi
= 134.7 psi
fv/Fv= Pu/(Apunct*Fpunct)
= **0.02** **< 1.0 OK**

**Slab Bending**

Asoil= (Pu*144)/(fsoil)
= 99 in^2
x= (L-y)/2
= 0.0 in
Fb= 5*(phi)*(f'c)^0.5
= 253.19 psi

L= (Asoil)^0.5
= 9.95 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 0 in-lb

y= (c*e)^0.5 + t*2
= 14.3 in
S-slab= 1*t^2/6
= 5.04 in^3
fb/Fb= M/(S-slab*Fb)
= **0.00** **< 1.0, OK**