PRCTI20230908

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

Structural Engineering & Design, Inc.

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

Project Name : SHELTER LOGIC

Project Number : 23-0509-5

Date : 06/21/23

Street Address: 1601 INDUSTRIAL PKWY
City/State : PUYALLUP, WA 98371

Scope of Work : STORAGE RACK

ENHAO ZHANG
STATE OF WASHINGTON
21022747
REGISTERED
PROFESSIONAL ENGINEER

Enhao Zhang

Digitally signed by Enhao Zhang
Date: 2023.06.21 10:41:30 -07'00'

Structural

Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

TABLE OF CONTENTS

Structural
Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Design Data

1) The analyses herein conforms to the requirements of the:
2018 IBC Section 2209
2019 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

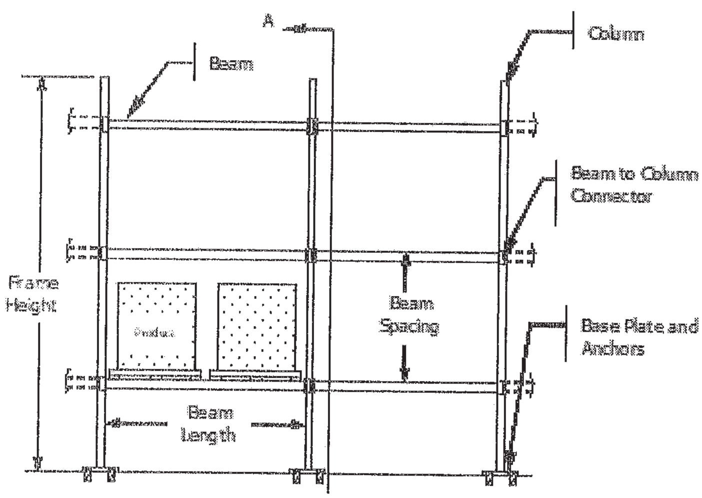
3) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein.

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.

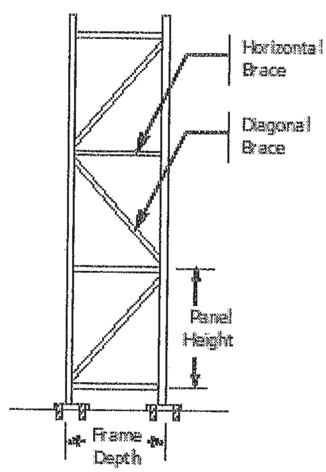
5) The existing slab on grade is 6" thick with minimum 2500 psi compressive strength. Allowable Soil bearing capacity is 750 psf. The design of the existing slab is by others.

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

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

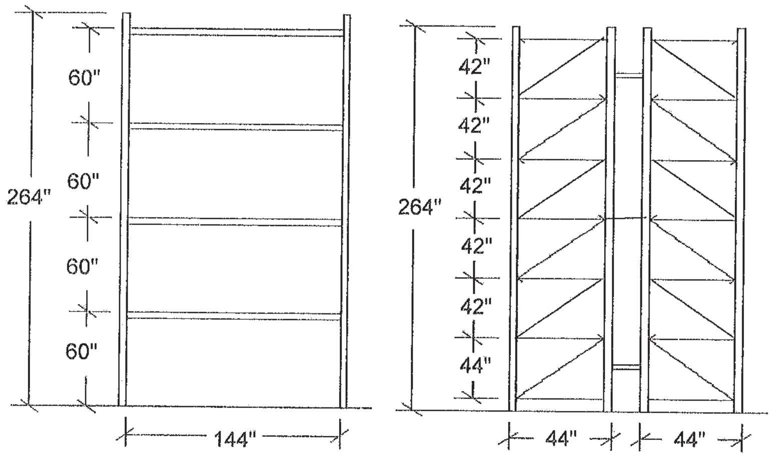
Building	Planning
Engineering	Public Works
Fire	Traffic

Structural
Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Configuration & Summary: TYPE 1 SELECTIVE RACK



**RACK COLUMN REACTIONS
ASD LOADS
AXIAL DL= 150 lb
AXIAL LL= 7,200 lb
SEISMIC AXIAL Ps=+/- 6,718 lb
BASE MOMENT= 8,000 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=1.285, Fa=1	4	44 in	264.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	UMH C3312TD 3x3x12ga	P=7350 lb, M=17907 in-lb		0.86-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	UMH SB556 5.5"x2.5"x16ga	Lu=144 in	Capacity: 4257 lb/pr	0.85-OK
Beam Connector	Fy=55 ksi	Lvl 1: 3 pin OK	Mconn=12104 in-lb	Mcap=12578 in-lb	0.96-OK
Brace-Horizontal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.7-OK
Brace-Diagonal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.96-OK
Base Plate	Fy=36 ksi	8x5x0.375		Fixity= 8000 in-lb	0.85-OK
Anchor	2 per Base	0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3380 lb)			0.833-OK
Slab & Soil	6" thk x 2500 psi slab on grade. 750 psf Soil Bearing Pressure				0.91-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
1	3,600 lb	60.0 in	44.0 in	213 lb	91 lb	7,350 lb	17,907 "#	12,104 "#	3 pin OK
2	3,600 lb	60.0 in	42.0 in	426 lb	182 lb	5,513 lb	12,272 "#	9,178 "#	3 pin OK
3	3,600 lb	60.0 in	42.0 in	639 lb	273 lb	3,675 lb	9,545 "#	6,792 "#	3 pin OK
4	3,600 lb	60.0 in	42.0 in	852 lb	364 lb	1,838 lb	5,454 "#	3,451 "#	3 pin OK
			42.0 in						
			42.0 in						
			Total:	2,131 lb	909 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: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

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

Transverse (Cross Aisle) Seismic Load

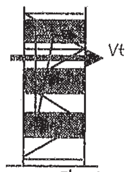
V= Cs*Ip*Ws=Cs*Ip*(0.67*P*Prf+D)

Cs1= Sds/R
= 0.2142
Cs2= 0.044*Sds
= 0.0377
Cs3= 0.5*S1/R
= 0.0554
Cs-max= 0.2142

Base Shear Coeff=Cs= **0.2142**

Cs-max * Ip= 0.2142
V_{min}= 0.015
Eff Base Shear=Cs= 0.2142
Ws= (0.67*PL_{RF1} * PL)+DL (RMI 2.6.2)
= 9,948 lb

Vtransv=Vt= 0.2142 * (300 lb + 9648 lb)
Etransverse= 2,131 lb
Limit States Level Transverse seismic shear per upright



Transverse Elevation

Ss= 1.285
S1= 0.443
Fa= 1.000
Fv= 1.857
Sds=2/3*Ss*Fa= 0.857
Sd1=2/3*S1*Fv= 0.548
Ca=0.4*2/3*Ss*Fa= 0.3427
(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	3,600 lb	2,412 lb	75 lb	60 in	149,220	213.1 lb	17,900-#
2	3,600 lb	2,412 lb	75 lb	120 in	298,440	426.2 lb	61,373-#
3	3,600 lb	2,412 lb	75 lb	180 in	447,660	639.3 lb	130,417-#
4	3,600 lb	2,412 lb	75 lb	240 in	596,880	852.4 lb	225,034-#
sum:	P=14400 lb	9,648 lb	300 lb	W=9948 lb	1,492,200	2,131 lb	Σ=434,724

Longitudinal (Downaisle) Seismic Load

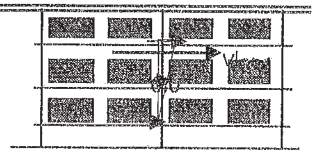
Similarly for longitudinal seismic loads, using R=6.0

Cs1=Sd1/(T*R)= 0.0914
Cs2= 0.0377
Cs3= 0.0369
Cs-max= 0.0914

Ws= (0.67 * PL_{RF2} * P) + DL
= 9,948 lb
Cs=Cs-max*Ip= 0.0914

P_{RF2}= 1.0
(Longitudinal, Unbraced Dir.) R= 6.0
T= 1.00 sec

Vlong= 0.0914 * (300 lb + 9648 lb)
Elongitudinal= 909 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	3,600 lb	2,412 lb	75 lb	60 in	149,220	90.9 lb
2	3,600 lb	2,412 lb	75 lb	120 in	298,440	181.8 lb
3	3,600 lb	2,412 lb	75 lb	180 in	447,660	272.7 lb
4	3,600 lb	2,412 lb	75 lb	240 in	596,880	363.6 lb
sum:		9,648 lb	300 lb	W=9948 lb	1,492,200	909 lb

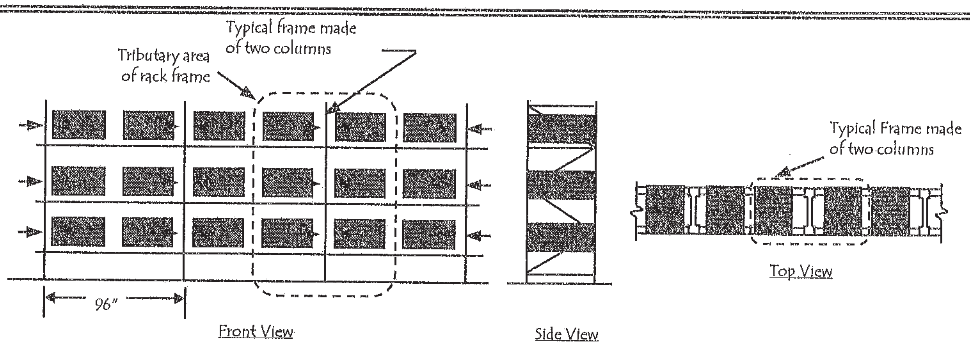
Downaisle Seismic Loads

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= 909 lb
Vcol=Vlong/2= 455 lb
F1= 91 lb
F2= 182 lb
F3= 273 lb



Conceptual System

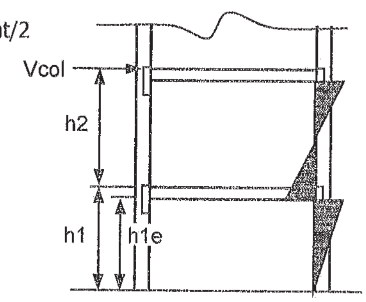
Seismic Story Moments

COL

Mbase-max= 8,000 in-lb <=== *Default capacity*
Mbase-v= (Vcol*h1eff)/2
= 12,953 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
= (455 lb * 57 in)-8000 in-lb
= 17,907 in-lb

h1-eff= h1 - beam clip height/2
= 57 in

M 2-2= [Vcol-(F1)/2] * h2
= [455 lb - 90.9 lb]*60 in/2
= 12,272 in-lb



Beam to Column

Mseis= (Mupper+Mlower)/2

Mseis(1-1)= (17907 in-lb + 12272 in-lb)/2
= 15,089 in-lb

Mseis(2-2)= (12272 in-lb + 9545 in-lb)/2
= 10,908 in-lb

rho= 1.0000

Summary of Forces

LEVEL	hi	Axial Load	Column Moment**	Mseismic**	Mend-fixity	Mconn**	Beam Connector
1	60 in	7,350 lb	17,907 in-lb	15,089 in-lb	2,203 in-lb	12,104 in-lb	3 pin OK
2	60 in	5,513 lb	12,272 in-lb	10,908 in-lb	2,203 in-lb	9,178 in-lb	3 pin OK
3	60 in	3,675 lb	9,545 in-lb	7,499 in-lb	2,203 in-lb	6,792 in-lb	3 pin OK
4	60 in	1,838 lb	5,454 in-lb	2,727 in-lb	2,203 in-lb	3,451 in-lb	3 pin OK

Mconn= (Mseismic + Mend-fixity)*0.70*rho
Mconn-allow(3 Pin)= 12,578 in-lb

**all moments based on limit states level loading

Structural
Engineering & Design Inc.

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

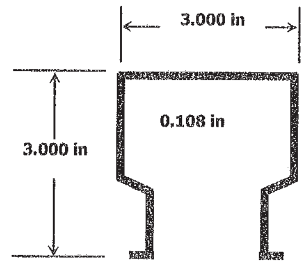
By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

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

Section Properties

Section: UMH C3312TD 3x3x12ga

Aeff = 0.761 in^2	Iy = 0.692 in^4	Kx = 1.7
Ix = 1.221 in^4	Sy = 0.397 in^3	Lx = 57.3 in
Sx = 0.817 in^3	ry = 0.954 in	Ky = 1.0
rx = 1.267 in	Fy= 55 ksi	Ly = 44.0 in
Ωf= 1.67	Cmx= 0.85	Cb= 1.0
E= 29,500 ksi		



Loads **Considers loads at level 1**

COLUMN DL= 150 lb
COLUMN PL= 7,200 lb
Mcol= 17,906 in-lb
Sds= 0.8567
1+0.105*Sds= 1.0900
1.4+0.14Sds= 1.5199
1+0.14Sds= 1.1199
0.85+0.14*Sds= 0.9699
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: 0.79796745 * 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: 0.67896 *seismic moment coeff: 0.7 * Mcol*

By analysis, Load case 6 governs utilizing loads as such

Axial Load=Pax= **1.119938*150 lb + 0.969938*0.7*7200 lb**
= 5,056 lb

Moment=Mx= 0.7*rho*Mcol
= 0.7 * 17906 in-lb
= 12,534 in-lb

Axial Analysis

KxLx/rx = 1.7*57.25"/1.267"
= 76.8

KyLy/ry = 1*44"/0.954"
= 46.1

Fe > Fy/2
Fn= Fy(1-Fy/4Fe)
= 55 ksi*[1-55 ksi/(4*49.3 ksi)]
= 39.7 ksi

Fe= π^2E/(KL/r)max^2
= 49.3ksi

Fy/2= 27.5 ksi

Pa= Pn/Ωc
= 30192 lb/1.92
= 15,725 lb

Pn= Aeff*Fn
= 30,192 lb

Ωc= 1.92

P/Pa= 0.32 > 0.15

Bending Analysis

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

Pno= Ae*Fy
= 0.761 in^2 *55000 psi
= 41,855 lb

Pao= Pno/Ωc
= 41855lb/1.92
= 21,799 lb

Myield=My= Sx*Fy
= 0.817 in^3 * 55000 psi
= 44,935 in-lb

Max= My/Ωf
= 44935 in-lb/1.67
= 26,907 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500 ksi/(1.7*57.25 in)^2
= 37,531 lb

μx= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*5056 lb/37531 lb)]}^-1
= 0.74

Combined Stresses

(5056 lb/15725 lb) + (0.85*12534 in-lb)/(26907 in-lb*0.74) =	0.86	< 1.0, OK	(EQ C5-1)
(5056 lb/21799 lb) + (12534 in-lb/26907 in-lb) =	0.70	< 1.0, OK	(EQ C5-2)

*** For comparison, total column stress computed for load case 5 is: 80.0% g loads 5908.858665 lb Axial and M= 9400 in-lb*

Structural
Engineering & Design Inc.

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

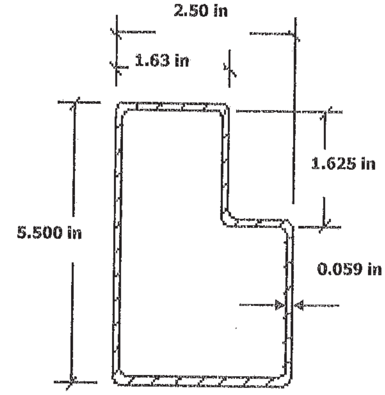
Building	Planning
Engineering	Public Works
Fire	Traffic

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

By: JJM/MQZ Project: SHELTER LOGIC Project #: 23-0509

BEAM Configuration: TYPE 1 SELECTIVE RACK 5

DETERMINE ALLOWABLE MOMENT CAPACITY



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

w= c - 2*t -2*r
= 1.625 in - 2*0.059 in - 2*0.059 in
= 1.389 in
w/t= 23.54
l=lambda= [1.052/(k)^0.5] * (w/t) * (Fy/E)^0.5 Eq. B2.1-4
= [1.052/(4)^0.5] * 23.54 * (55/29500)^0.5
= 0.535 < 0.673, Flange is fully effective Eq. B2.1-1

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

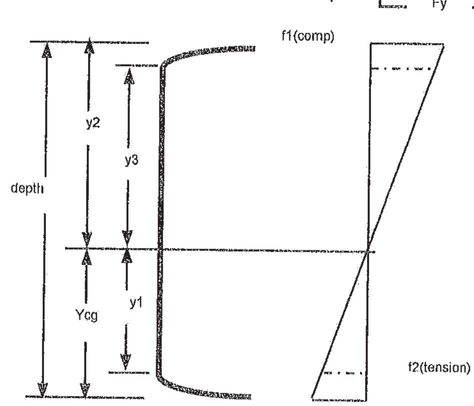
f1(comp)= Fy*(y3/y2)= 51.53 ksi
f2(tension)= Fy*(y1/y2)= 103.29 ksi
Y= f2/f1 Eq. B2.3-5
= -2.004
k= 4 + 2*(1-Y)^3 + 2*(1-Y) Eq. B2.3-4
= 64.22
flat depth=w= y1+y3
= 5.264 in w/t= 89.22033898 OK
l=lambda= [1.052/(k)^0.5] * (w/t) * (f1/E)^0.5
= [1.052/(64.22)^0.5] * 5.264 * (51.53/29500)^0.5
= 0.49 < 0.673
be=w= 5.264 in b2= be/2 Eq B2.3-2
b1= be(3-Y) = 2.63 in
= 1.052
b1+b2= 3.682 in > 1.752 in, Web is fully effective

Beam= **UMH SB556 5.5"x2.5"x16ga**

Ix=	3.219 in^4
Sx=	1.125 in^3
Ycg=	3.630 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=	1.625 in
bottom flange=	2.500 in
Web depth=	5.500 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= 1.389 in = 0.167 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= 58.92 ksi (tension stress at top)
Fya-bottom= Fya*Ycg/(depth -Ycg)
= 114.37 ksi (tension stress at bottom)



y1= Ycg-t-r= 3.512 in
y2= depth-Ycg= 1.870 in
y3= y2-t-r= 1.752 in

Check allowable tension stress for bottom flange

Lflange-bot=Lfb= Lbottom - 2*r*-2*t
= 2.264 in
Cbottom=Cb= 2*Lc/(Lfb+2*Lc)
= 0.109
Fy-bottom=Fyb= Cb*Fyc + (1-Cb)*Fyf
= 57.57 ksi
Fya= (Fya-top)*(Fyb/Fya-bottom)
= 29.66 ksi
if F= 0.95 Then F*Mn=F*Fya*Sx= $\boxed{\text{31.69 in-k}}$

Structural Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509~~

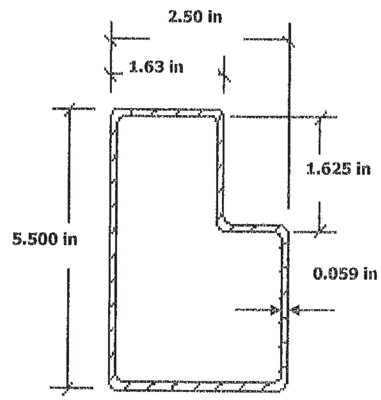
BEAM Configuration: TYPE 1 SELECTIVE RACK 5

RMI Section 5.2, PT II

Section

Beam= UMH SB556 5.5"x2.5"x16ga
Ix=Ib= 3.219 in^4
Sx= 1.125 in^3
t= 0.059 in
Fy=Fyv= 55 ksi
Fu=Fuv= 65 ksi
Fya= 58.9 ksi

E= 29500 ksi
F= 150.0
L= 144 in
Beam Level= 1
P=Product Load= 3,600 lb/pair
D=Dead Load= 75 lb/pair



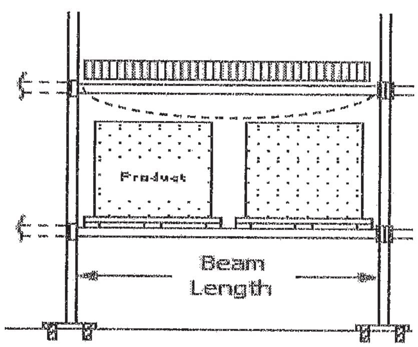
1. Check Bending Stress Allowable Loads

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*150*144 in)/[(6*29500 ksi*3.219 in^3)+(3*150*144 in)]
= 0.932
if F= 0.95
Then F*Mn=F*Fya*Sx= 62.97 in-k
Thus, allowable load
per beam pair=W= F*Mn*8*(# of beams)/(L*Rm*W)
= 62.97 in-k * 8 * 2/(144in * 0.932 * 1.599)
= 4,695 lb/pair ***allowable load based on bending stress***



Mend= W*L*(1-Rm)/8
= (4695 lb/2) * 144 in * (1-0.932)/8
= 2,873 in-lb @ 4695 lb max allowable load
= 2,203 in-lb **@ 3600 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*144 in)/[(5*150*144 in)+(10*29500 ksi*3.219 in^4)]
= 0.918 in
if Dmax= L/180 *Based on L/180 Deflection Criteria*
and Dss= 5*W*L^3/(384*E*Ib)

Allowable Deflection= L/180
= 0.800 in
Deflection at imposed Load= 0.677 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*3.219 in^4*2/[180*5*(144 in)^2*0.918)
= 4,257 lb/pair ***allowable load based on deflection limits***

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

Allowable load= 4,257 lb/pair
Imposed Product Load= 3,600 lb/pair
Beam Stress= 0.85 *Beam at Level 1*

8.2

Structural Engineering & Design Inc.

City of Puyallup
Development & Permitting Services
ISSUED PERMIT

Building	Planning
Engineering	Public Works
Fire	Traffic

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

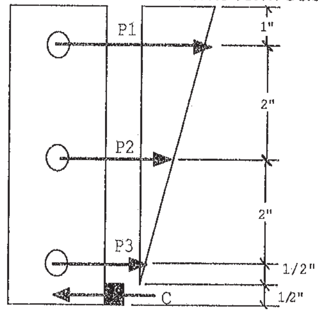
By: JJM/MQZ Project: SHELTER LOGIC Project #: 23-0509-5

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
= 12,104 in-lb Load at level 1

rho= 1.0000



Connector Type= 3 Pin

Shear Capacity of Pin

Pin Diam= 0.35 in Fy= 55,000 psi

Ashear= (0.35 in)^2 * Pi/4
= 0.0962 in^2

Pshear= 0.4 * Fy * Ashear
= 0.4 * 55000 psi * 0.0962in^2
= 2,116 lb

Bearing Capacity of Pin

tcol= 0.108 in Fu= 65,000 psi

Omega= 2.22 a= 2.22

Pbearing= alpha * Fu * diam * tcol/Omega
= 2.22 * 65000 psi * 0.35 in * 0.108 in/2.22
= 2,457 lb > 2116 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= 0.14 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= 2,116 lb
= 5,531 lb

Mconn-allow= [P1*4.5"+P1*(2.5"/4.5")*2.5"+P1*(0.5"/4.5")*0.5"]
= 2116 LB*[4.5"+(2.5"/4.5")*2.5"+ (0.5"/4.5")*0.5"]
= 12,578 in-lb > Mconn max, OK

Structural

Engineering & Design Inc.

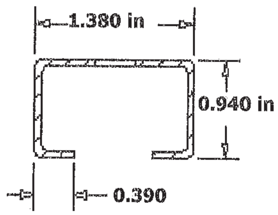
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

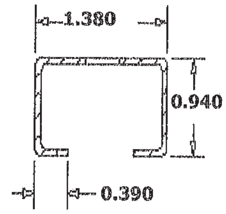
Transverse Brace Configuration: TYPE 1 SELECTIVE RACK

Section Properties

Diagonal Member= UMH C 1.38x0.94x0.39x16ga
Area= 0.214 in^2
r min= 0.356 in
Fy= 55,000 psi
K= 1.0
Ωc= 1.92



Horizontal Member= UMH C 1.38x0.94x0.39x16ga
Area= 0.214 in^2
r min= 0.356 in
Fy= 55,000 psi
K= 1.0



Frame Dimensions

Bottom Panel Height=H= 44.0 in
Frame Depth=D= 44.0 in
Column Width=B= 3.0 in
Clear Depth=D-B*2= 38.0 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*

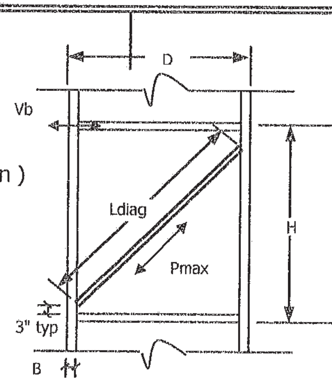
Vtransverse= 2,131 lb
Vb=Vtransv*0.7*rho= 2131 lb * 0.7 * 1
= 1,492 lb
Ldiag= [(D-B*2)^2 + (H-6")^2]^1/2
= 53.7 in

Pmax= V*(Ldiag/D) * 0.75
= 1,365 lb
axial load on diagonal brace member

Pn= AREA*Fn
= 0.214 in^2 * 12803 psi
= 2,740 lb

(kl/r)= (k * Ldiag)/r min
= (1 x 53.7 in /0.356 in)
= 150.8 in
Fe= pi^2*E/(kl/r)^2
= 12,803 psi

Since Fe<Fy/2,
Fn= Fe
= 12,803 psi



Typical Panel Configuration

Pallow= Pn/Ω
= 2740 lb /1.92
= 1,427 lb

Pn/Pallow= 0.96 <= 1.0 OK

Horizontal brace

Vb=Vtransv*0.7*rho= 1,492 lb

(kl/r)= (k * Lhoriz)/r min
= (1 x 44 in) /0.356 in
= 123.6 in
Since Fe<Fy/2, Fn=Fe
= 19,058 psi

Fe= pi^2*E/(kl/r)^2
= 19,058 psi

Fy/2= 27,500 psi

Pn= AREA*Fn
= 0.214in^2*19058 psi
= 4,078 lb

Pallow= Pn/Ωc
= 4078 lb /1.92
= 2,124 lb

Pn/Pallow= 0.70 <= 1.0 OK

Structural
Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Single Row Frame Overturning Configuration: TYPE 1 SELECTIVE RACK

Loads

Critical Load case(s):

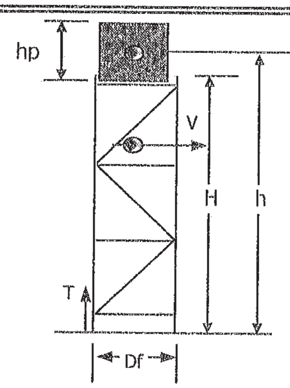
1) RMI Sec 2.2, item 7: (0.9-0.2Sds)D + (0.9-0.20Sds)*B*Papp - E*rho

Vtrans=V=E=Qe= 2,131 lb
DEAD LOAD PER UPRIGHT=D= 300 lb
PRODUCT LOAD PER UPRIGHT=P= 14,400 lb
Papp=P*0.67= 9,648 lb
Wst LC1=Wst1=(0.72866*D + 0.72866*Papp*1)= 7,248 lb

Product Load Top Level, Ptop= 3,600 lb
DL/Lvl= 75 lb
Seismic Ovt based on E, Σ(Fi*hi)= 295,596 in-lb
height/depth ratio= 5.5 in

Sds= 0.8567
(0.9-0.2Sds)= 0.7287
(0.9-0.2Sds)= 0.7287
B= 1.0000
rho= 1.0000

Frame Depth=Df= 44.0 in
Htop-lvl=H= 240.0 in
# Levels= 4
# Anchors/Base= 2
hp= 48.0 in
h=H+hp/2= 264.0 in



SIDE ELEVATION

A) Fully Loaded Rack

Load case 1:

Movt= Σ(Fi*hi)*E*rho
= 295,596 in-lb

Mst= Wst1 * Df/2
= 7248 lb * 44 in/2
= 159,456 in-lb

T= (Movt-Mst)/Df
= (295596 in-lb - 159456 in-lb)/44 in
= 3,094 lb **Net Uplift per Column**

Net Seismic Uplift= 3,094 lb

B) Top Level Loaded Only

Load case 1:

V1=Vtop= Cs * Ip * Ptop >= 350 lb for H/D >6.0
= 0.2142 * 3600 lb
= 771 lb
V1eff= 771 lb
$V2=V_{DL}$= Cs*Ip*D
= 64 lb
Mst= (0.72866*D + 0.72866*Ptop*1) * 44 in/2
= 62,519 in-lb

Critical Level= 4
Cs*Ip= 0.2142

Movt= [V1*h + V2 * H/2]*rho
= 211,287 in-lb
T= (Movt-Mst)/Df
= (211287 in-lb - 62519 in-lb)/44 in
= 3,381 lb **Net Uplift per Column**

Net Seismic Uplift= 3,381 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
Shear Capacity=Vcap= 2,517 lb

L.A. City Jurisdiction? NO
Phi= 1

Tcap*Phi= 1,961 lb
Vcap*Phi= 2,517 lb

Fully Loaded:	(1547 lb/1961 lb)^1 + (532 lb/2517 lb)^1 =	1.00	<= 1.2	OK
Top Level Loaded:	(1690 lb/1961 lb)^1 + (192 lb/2517 lb)^1 =	0.94	<= 1.2	OK

Structural

Engineering & Design Inc.

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

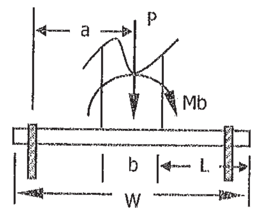
By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Base Plate Configuration: TYPE 1 SELECTIVE RACK

Section

Baseplate= 8x5x0.375
Eff Width=W = 7.00 in
Eff Depth=D = 5.00 in
Column Width=b = 3.00 in
Column Depth=dc = 3.00 in
L = 2.00 in
Plate Thickness=t = 0.375 in

a = 2.50 in
Anchor c.c. =2*a=d = 5.00 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= 150 lb
COLUMN PL= 7,200 lb
Base Moment= 8,000 in-lb
1+0.105*Sds= 1.0900
1.4+0.14Sds= 1.5199
B= 0.7000

Axial=P= 1.0899535 * 150 lb + 0.75 * (1.519938 * 0.7 * 7200 lb)
= 5,909 lb
Mb= Base Moment*0.75*0.7*rho
= 8000 in-lb * 0.75*0.7*rho
= 4,200 in-lb

Axial Load P = 5,909 lb	**Mbase=Mb = 4,200 in-lb**

Effe
Effe

Axial stress=fa = P/A = P/(D*W)
= 169 psi
Moment Stress=fb = M/S = 6*Mb/[(D*B^2]
= 102.9 psi
Moment Stress=fb1 = fb-fb2
= 44.1 psi
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2
= 78 in-lb
S-plate = (1)(t^2)/6
= 0.023 in^3/in
fb/Fb = Mtotal/[(S-plate)(Fb)]
= **0.80** **OK**
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2]
= -2,784 lb No Tension

M1= wL^2/2= fa*L^2/2
= 338 in-lb
Moment Stress=fb2 = 2 * fb * L/W
= 58.8 psi
M2= fb1*L^2)/2
= 88 in-lb
Mtotal = M1+M2+M3
= 504 in-lb/in
Fb = 0.75*Fy
= 27,000 psi
F'p= 0.7*F'c
= 1,750 psi OK
Tallow= 1,961 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= 5,909 lb

Movt*0.75*0.7*rho= 155,188 in-lb
Frame Depth= 44.0 in

P=Pstatic+Pseismic= 9,436 lb

Pseismic= Movt/Frame Depth
= 3,527 lb

b =Column Depth= 3.00 in

L =Base Plate Depth-Col Depth= 2.00 in

fa = P/A = P/(D*W)
= 270 psi

M= wL^2/2= fa*L^2/2
= 539 in-lb/in

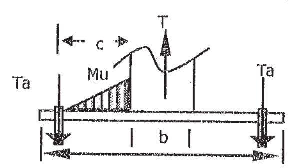
Sbase/in = (1)(t^2)/6
= 0.023 in^3/in

Fbase = 0.75*Fy
= 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.85** **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= 3,380 lb
Qty Anchor per BP= 2
Net Tension per anchor=Ta= 1,690 lb
c= 2.00 in
Mu=Moment on Baseplate due to uplift= Ta*c/2
= 1,690 in-lb
Splate= 0.117 in^3
[fb/Fb]*0.75= 0.401 **OK**

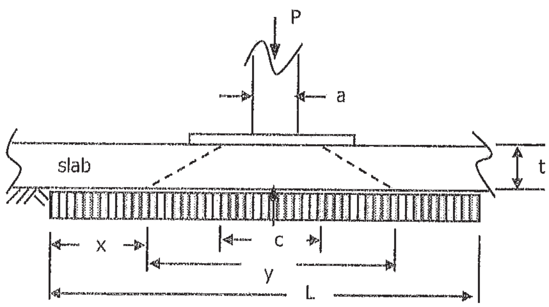
Structural

Engineering & Design Inc.

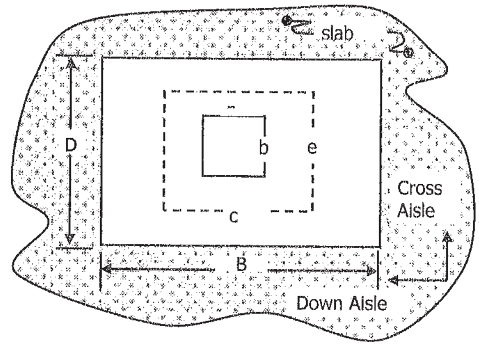
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Slab on Grade Configuration: TYPE 1 SELECTIVE RACK



SLAB ELEVATION



Baseplate Plan View

Concrete
f'c= 2,500 psi
tslab=t= 6.0 in
teff= 6.0 in
phi=Ø= 0.6

Soil
fsoil= 750 psf
Movt= 295,596 in-lb
Frame depth= 44.0 in
Sds= 0.857
0.2*Sds= 0.171
λ= 0.600
β=B/D= 1.400
F'c^0.5= 50.00 psi

Base Plate
Effec. Baseplate width=B= 7.00 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= 5.00 in
midway dist face of column to edge of plate=e= 4.00 in

Column Loads

DEAD LOAD=D= 150 lb per column
unfactored ASD load
PRODUCT LOAD=P= 7,200 lb per column
unfactored ASD load
Papp= 4,824 lb per column
P-seismic=E= (Movt/Frame depth)
= 6,718 lb per column
unfactored Limit State load
B= 0.7000
rho= 1.0000
Sds= 0.8567
1.2 + 0.2*Sds= 1.3713
0. 9 - 0.20Sds= 0.7287

Load Case 1) (1.2+0.2Sds)D + (1.2+0.2Sds)*B*P+ rho*E RMI SEC 2.2 EQTN 5
= 1.37134 * 150 lb + 1.37134 * 0.7 * 7200 lb + 1 * 6718 lb
= 13,835 lb
Load Case 2) (0.9-0.2Sds)D + (0.9-0.2Sds)*B*Papp + rho*E RMI SEC 2.2 EQTN 7
= 0.72866 * 150 lb + 0.72866 * 0.7 * 4824 lb + 1 * 6718 lb
= 9,288 lb
Load Case 3) 1.2*D + 1.4*P RMI SEC 2.2 EQTN 1,2
= 1.2*150 lb + 1.4*7200 lb
= 10,260 lb
Load Case 4) 1.2*D + 1.0*P + 1.0E ACI 318-14 Sec 5.3.1
= 14,098 lb Eqtn 5.3.1e

Effective Column Load=Pu= 14,098 lb per column

Puncture

Apunct= [(c+t)+(e+t)]*2*t
= 252.0 in^2
Fpunct1= [(4/3 + 8/(3*β)] * λ *(F'c^0.5)
= 97.1 psi
Fpunct2= 2.66 * λ * (F'c^0.5)
= 79.8 psi
Fpunct eff= 79.8 psi

fv/Fv= Pu/(Apunct*Fpunct)
= **0.701** **< 1 OK**

Slab Bending

Pse=DL+PL+E= 14,098 lb

Asoil= (Pse*144)/(fsoil)
= 2,707 in^2
x= (L-y)/2
= 17.8 in
Fb= 5*(phi)*(f'c)^0.5
= 150. psi

L= (Asoil)^0.5
= 52.03 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 823.1 in-lb

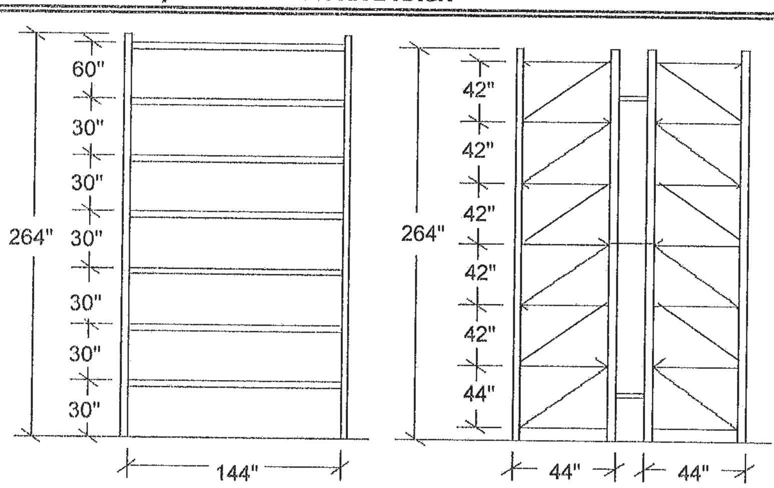
y= (c*e)^0.5 + 2*t
= 16.5 in
S-slab= 1*teff^2/6
= 6.0 in^3
fb/Fb= M/(S-slab*Fb)
= **0.915** **< 1, OK**

Structural
Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~23-0509-5~~

Configuration & Summary: TYPE 2 SELECTIVE RACK



**RACK COLUMN REACTIONS
ASD LOADS
AXIAL DL= 263 lb
AXIAL LL= 6,975 lb
SEISMIC AXIAL Ps=+/- 6,597 lb
BASE MOMENT= 8,000 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=1.285, Fa=1	7	44 in	264.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	UMH C3313TD 3x3x13ga	P=7238 lb, M=6088 in-lb		0.48-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	UMH SB556 5.5"x2.5"x16ga	Lu=144 in	Capacity: 4257 lb/pr	0.7-OK
Beam Connector	Fy=55 ksi	Lvl 4: 3 pin OK	Mconn=5349 in-lb	Mcap=12174 in-lb	0.44-OK
Brace-Horizontal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.7-OK
Brace-Diagonal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.95-OK
Base Plate	Fy=36 ksi	8x5x0.375		Fixity= 6089 in-lb	0.84-OK
Anchor	2 per Base	0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3002 lb)			0.817-OK
Slab & Soil	6" thk x 2500 psi slab on grade. 750 psf Soil Bearing Pressure				0.89-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
1	650 lb	30.0 in	44.0 in	21 lb	9 lb	7,238 lb	6,088 "#	4,754 "#	3 pin OK
2	650 lb	30.0 in	42.0 in	42 lb	18 lb	6,875 lb	6,698 "#	4,919 "#	3 pin OK
3	650 lb	30.0 in	42.0 in	64 lb	27 lb	6,513 lb	6,562 "#	4,801 "#	3 pin OK
4	3,000 lb	30.0 in	42.0 in	346 lb	148 lb	6,150 lb	6,359 "#	5,349 "#	3 pin OK
5	3,000 lb	30.0 in	42.0 in	432 lb	184 lb	4,613 lb	5,252 "#	4,478 "#	3 pin OK
6	3,000 lb	30.0 in	42.0 in	518 lb	221 lb	3,075 lb	3,870 "#	4,187 "#	3 pin OK
7	3,000 lb	60.0 in		691 lb	295 lb	1,538 lb	4,422 "#	2,833 "#	3 pin OK

** Load defined as product weight per pair of beams Total: 2,114 lb 902 lb

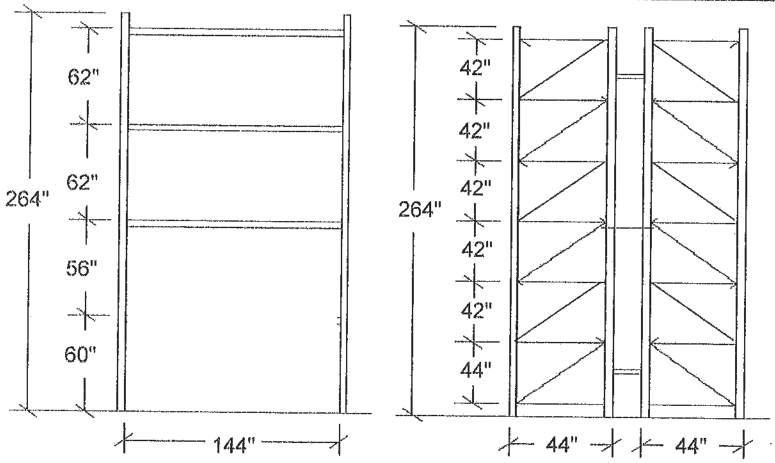
Notes
2.5" BEAM OK @ LEVELS 1-3

Structural
Engineering & Design Inc.

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

By: JJM/MQZ Project: ~~SHELTER LOGIC~~ Project #: ~~PRELIM~~

Configuration & Summary: TYPE T SELECTIVE RACK



***RACK COLUMN REACTIONS*

ASD LOADS
AXIAL DL= 200 lb
AXIAL LL= 6,300 lb
SEISMIC AXIAL Ps=+/- 6,033 lb
BASE MOMENT= 8,000 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=1.285, Fa=1	4	44 in	264.0 in	6	144 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	UMH C3312TD 3x3x12ga	P=6500 lb, M=15028 in-lb		0.72-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	UMH SB556 5.5"x2.5"x16ga	Lu=144 in	Capacity: 4257 lb/pr	0.85-OK
Beam Connector	Fy=55 ksi	Lvl 2: 3 pin OK	Mconn=8525 in-lb	Mcap=12578 in-lb	0.68-OK
Brace-Horizontal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.62-OK
Brace-Diagonal	Fy=55 ksi	UMH C 1.38x0.94x0.39x16ga			0.85-OK
Base Plate	Fy=36 ksi	8x5x0.375		Fixity= 8000 in-lb	0.76-OK
Anchor	2 per Base	0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3332 lb)			0.775-OK
Slab & Soil		6" thk x 2500 psi slab on grade. 750 psf Soil Bearing Pressure			0.77-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
2	3,600 lb	56.0 in	42.0 in	389 lb	166 lb	5,550 lb	10,688 "#	8,525 "#	3 pin OK
3	3,600 lb	62.0 in	42.0 in	597 lb	255 lb	3,700 lb	9,263 "#	6,645 "#	3 pin OK
4	3,600 lb	62.0 in	42.0 in	804 lb	343 lb	1,850 lb	5,318 "#	3,403 "#	3 pin OK
			42.0 in						
			42.0 in						
		Total:		1,894 lb	808 lb				

** Load defined as product weight per pair of beams

Notes