

# Structural Engineering # Design, Inc.

1815 Wrlght 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 Digitally signed by Enhao Zhang Date: 2023.06.21 10:41:30 -07'00'





1815 Wright Ave La Verne, CA	91750 Tel: 909.596.1351 Fax: 9	09.596.7186
By: JJM/MQZ Project: SHELTE	Company for the parties of the contract of the	Project #: 33 (FOC) F
TABLE OF CONTENTS		
Title Page	1	
Table of Contents	2	
Design Data and Definition of Components	3	
Critical Configuration	4	
Seismic Loads	5 to 6	
Column	7	
Beam and Connector	8 to 9	
Bracing	10	
Anchors	11	
Base Plate	12	
Slab on Grade	13	
Other Configurations	14 +0 15	

# City of Puyallup Development & Permitting Services ISSUED PERMIT Building Planning Engineering Public Works Fire Traffic

# 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:

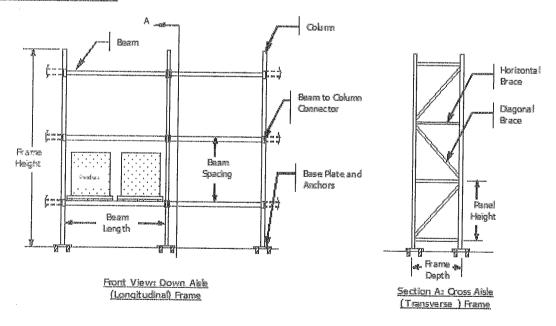
2018 IBC Section 2209

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**

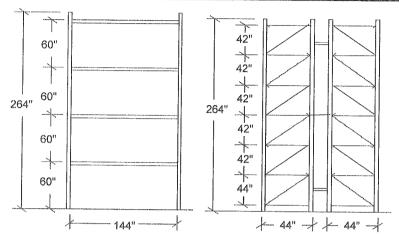


# Engineering & Design Inc.



181	Wright Ave La	Verne, CA 91750 Tel: 909.596,	1351 Fax: 909.596.7186	
By: JJM/MQZ	Duningto			AND THE POST OF THE POST OF THE PERSON OF TH

### 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

AND THE COMMENT OF THE PROPERTY OF THE PROPERT	CONTRACTOR DESCRIPTION OF THE PROPERTY OF THE	Commence in the Contract of th	Maliferina de la companya del companya del companya de la companya			
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

Comp	onent		CARROLI CONCENTRATION AND MACHINE NAME AND ADDRESS OF THE PARTY OF THE	Description						
Colu	ımn	Fy≃55 ksi	UM	IH C3312TD 3x3	x12ga	P=735	50 lb, M=17907	7 in-lb	STRESS 0.86-OK	
Column 8	& Backer	None		None		The second secon	None	THE RESIDENCE OF THE PARTY OF T	N/A	
Bea	em	Fy≕55 ksi	UMH	SB556 5.5"x2.5	5"x16ga	Lu=144 in	Capacity: 4257 lb/pr		0.85-OK	
Beam Co	nnector	Fy=55 ksi	Lvl 1:	3 pin OK	Mconn=1	2104 in-lb	A STATE OF THE PARTY OF THE PAR	2578 in-lb	0.96-OK	
Brace-Ho	orizontal	Fy=55 ksi		ACCOUNTS OF THE PARTY OF THE PA	UMH C 1.38x0.9	94x0.39x16ga	THE RESERVE THE PROPERTY OF TH	MATERIA MATERIAL MATERIAL MATERIAL PROPERTY AND PROPERTY	0.7-OK	
Brace-D	iagonal	Fy=55 ksi			UMH C 1.38x0.9	94x0.39x16ga	THE PROPERTY AND ADDRESS OF THE PROPERTY OF TH	er were die auffahren gegen der zuer zuer zu zu zu zu zeige den zeige den zu	0.96-OK	
Base	Plate	Fy=36 ksi		8x5>	<0.375	Washington and the second	Fixity= 8	000 in-lb	0.85-OK	
Anc	nor	2 per Base	0.5" x 3.25" E	mbed HILTI KWIK	BOLT TZ ESR 191	7 Inspection Rec	d (Net Seismic l	Jplift=3380 lb)	0.833-OK	
Slab 8	k Soil				n grade. 750 psf			CHARLES OF THE STREET, WAS AND ASSESSMENT OF THE STREET, WAS AND ASSESSMENT OF THE STREET, WAS A	0.91-OK	
Level	Load**			Story Force	Story Force	Column	Column	Conn.	Beam	
PROPERTY SHOPS IN SECURIOR SECURITY SEC	Per Level	Beam Spcg	Brace	Transv	Longit.	Axial	Moment	Moment	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 l			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			1,838 lb	5,454 "#	3,451 "#	3 pin OK	
			42.0 in			•	,	.,	ar print serie	
			42.0 in							

** Load defined as product weight per pair of beams	Total:	2.131 lb	909 lb.	THE RESERVE OF THE PROPERTY OF
Notes			The state of the s	THE SAME AND ADDRESS OF THE SAME ADDRESS OF THE SAME AND ADDRESS OF THE SAME AND ADDRESS OF THE SAME AND ADDRESS OF THE SAME ADDRESS OF THE SA

# Engineering & Design Inc.



1815 Wright Ave_La Verne, CA 91750 Tel; 909.596,1351 Fax: 909.596,7186						
By: JJM/MQZ	Project:	CIJELTEI	PLOCIC		Project #:	205005
Seismic Forces Configuration: T	YPE 1 SELECTI			THE CONTRACTOR OF THE CONTRACT	64-94-94 обощно высовые выполняем не полняем полняем в в полняем в полняем в полняем в полняем в полняем в полн На Сомполняем не полняем в полняем в полняем в полняем в полняем в полняем в не полняем в него не полняем в полня	
Lateral analysis is performed with regard to the r	equirements of the	2012 RMI ANSI I	MH 16.1-2012 Sec	2.6 & ASCE 7-16 sec 15.5.3	Ss	= 1.285
Transverse (Cross Aisle) Seismic L	oad			>		= 0.443
V= Cs*Ip*Ws=Cs	*Ip*(0.67*P*	Prf+D)	and the financial transport constructions and the process of the second construction of the second con	yt Vt		= 1.000
Cs1= Sds/R					Fv	= 1.857
= 0.2142		Cs-max * Ip=	0.2142		Sds=2/3*Ss*Fa	= 0.857
Cs2 = 0.044*Sds		V <sub>min</sub> =	0.015		Sd1=2/3*S1*Fv	= 0.548
= 0.0377	Eff Ba	se Shear=Cs=	0.2142	Transverse Elevation	Ca=0.4*2/3*Ss*Fa	= 0.3427
Cs3 = 0.5*S1/R		Ws=	(0.67*PL <sub>RF1</sub> *	PL)+DL (RMI 2.6.2)	(Transverse, Braced Frame Dir.) I	= 4.0
= 0.0554		Englishment and appropriate the second secon	9,948 lb		Iр	= 1.0
Cs-max= 0.2142		Vtransv=Vt=	0.2142 * (30	00 lb + 9648 lb)	$P_{RF1}$	= 1.0
Base Shear Coeff=Cs= 0.2142		Etransverse=	2,131 lb		Pallet Height=hp	= 48.0 in
LISSANSTPORTA NADOVET, O SEPTEMBERGENE PROPERTO FOR MANAPORTO TO THE STREET OF THE PROPERTOR OF THE PROPERTO		Limit States Le	vel Transverse s	eismic shear per upright	DL per Beam Lvi	= 75 lb
Level PRODUCT LOAD P	P*0.67*P <sub>RF1</sub>	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-#

Sul	m: P=14400 lb	9,648 lb	300 lb	W=9948 lb	1,492,200	2,131 lb	Σ=434,724
Longitudinal (Do	<u>wnaisle) Seismic</u>	Load					_ ,
Similarly for longitudinal se	elsmic loads, using R=6.0	Ws=	0.67 * PL <sub>RF</sub>	<sub>2</sub> * P) + DL	P <sub>RF2</sub> =	= 1.0	
Cs1=Sd1/(T*R)	)= 0.0914	= 5	9,948 lb	(Longitud	inal, Unbraced Dir.) R=		tanament programme
Cs2	2= 0.0377	Cs=Cs-max*Ip=	0.0914		T=	= 1.00 sec	
Cs3	3= 0.0369	Viong=	0.0914 * (	300 lb + 9648 lb)	and the second of the second s		
Cs-max	(= 0.0914	Elongitudinal= !	909 lb	Limit States Level Longit, sei	smic shear per upright		minus of tables promise of the state of the
Level	PRODUC LOAD P	P*0.67*P <sub>RF2</sub>	DL	hi	wi*hi	Fl	Front View
1	3,600 lb	2,412 lb	75 lb	60 in	149,220	90.9 lb	ACTIVITIES AND THE PARTY OF THE
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	

W=9948 lb

1,492,200

9,648 lb

300 lb

909 lb

# Engineering & Design Inc.



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

JJM/MQZ

Project: SHELTER LOCK

Project #: 23<u>-0500-</u>1

**Downaisle Seismic Loads** 

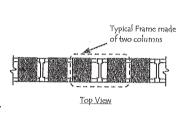
Configuration: TYPE 1 SELECTIVE RACK

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 lbF2 = 182 lbF3= 273 lb

Typical frame made of two columns Tributary area of rack frame Front View Side View Conceptual System



Seismic Story Moments

COL

Mbase-max= 8,000 in-lb

<=== Default capacity

h1-eff= h1 - beam clip height/2

=57 in

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

Mseis= (Mupper+Mlower)/2

M 2-2= [Vcol-(F1)/2] \* h2

= [455 lb - 90.9 lb]\*60 in/2

= 12,272 in-lb

Beam to Column

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

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

= 15,089 in-lb

= 10,908 in-lb

rho= 1.0000

Vcol

h2

h1

And the state of t	77.74 V. 10.014 MURA (F. F. MARKA)		Sumr	nary of Forces		The second secon	manuschi, Grigory Benedit zu derd gestabettet (4 heb Annehe 1 zeh verenze 2004 dat det en eus zu er
LEVEL	hi	Axial Load	Column Moment**	Mselsmic**	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 ln	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



181	<u> 5 Wright Ave_La Verne, CA 91</u>	750 Tel: 909.596.135	51 Fax: 909,596.	7186	
By: JJM/MQZ	Project: SHELTER L	3616		Project #	: 23.0500.5
Column (Longitudinal Loads)	Configuration: TYPE 1 SELE	CTIVE RACK			
THE RELATION BY AND A STATE OF THE STATE OF	Antoning of the PA (And And And And And And And And And And	Annual Control of the		er turning visignet i 1970 februar skilver en men en e	menteprotegic (CO) (A) fall til stadd verke kan som kinningsstad man penasi person per som på ykjed prek i Versom som kritisk til det å forsvetke mense av mansatt som som kjent i sidner pelyde (en de såd formås som ve
Section Properties					
Cashiama HMH COOK OTTO O		CON HACK & Branches and the contract of the Villa contract courts and authorized by the contract courts and the contract courts and the contract courts are contract courts.	n y maa keelineelineelineelineelineelineelineeli	an in Changain an Amhaigh y Chùin Changain an Ann ann an Aire a Tha Changain an Aire ann an	3.000 in
Section: UMH C3312TD 3x	=				
Aeff = $0.761 \text{ in}^2$ Ix = $1.221 \text{ in}^4$	$Iy = 0.692 \text{ in}^4$		x = 1.7	<del></del>	
$Sx = 0.817 \text{ in}^{-2}$	$Sy = 0.397 \text{ in}^3$		x = 57.3  in		0.108 in
rx = 1.267  in	ry = 0.954 in		xy = 1.0	3.000 in	0.100 111
$\Omega f = 1.67$	Fy= 55 ksi Cmx= 0.85		y = 44.0  in		
E = 29,500  ksi	CITIX= 0.85	(	Cb= 1.0		
E 29,300 KSI					
.oads Considers loads	The second secon	WIN SOME LIBERTY AND THE STATE OF THE STATE		**************************************	10201
COLUMN DL= 150 lb	Critical load cases are: RMI Sec		de all historia sunting per historia de la produce at antinente y le 120 de 190 de la presidente de la produce de la presidente de la presiden		от то жили то то стой от доли в стой у 14 - 20 доля в стой об и и то то стой от то то то то то то то то то то В стой стой стой стой стой от 14 стой у 14 - 20 доля, бы в невы и в нем и нем и нем трит в 4-14 и туст у наридур В стой стой стой стой стой от то стой у 14 - 20 доля в нем нем и нем и нем и нем трит в 4-14 и туст у наридура
COLUMN PL= 7,200 lb	Load Case 5: : (1+0.105*Sds)L	) + 0.75*(1.4+0.14Sds	s)*B*P + 0.75*(0.)	7*rho*E)<= 1	1.0, ASD Method
Mcol= 17,906 in-lb	axial load coeff: 0,7979674	5 * P	seismic moment co	peff: 0.5625	* Mcol
Sds= 0.8567	Load Case 6: : (1+0.14*Sds)D	+ (0.85+0.14Sds)*B*F	7 + (0.7*rho*E)<=	= 1.0, ASD Me	rthod
1+0.105*Sds= 1.0900	axial load coeff: 0.67896		seismic moment co	peff: 0.7 * M	col
1.4+0.14Sds= 1.5199	By analysis, Load case 6 govern	s utilizing loads as such	า	THE PERSONNEL PRODUCTION OF THE PERSONNEL PROPERTY OF THE PERSONNEL PR	og grange et spergen et som men men men med et det ein å bespekt grang zuer zu den angestate filmmen men
1+0.14Sds= 1.1199					
0.85+0.14*Sds== 0.9699	Axial Load=Pax= 1.119938*15	0 lb + 0.969938*0.7*7200	lb Moment=N	4x= 0.7*rhc	o*Mcol
B= 0.7000	= 5,056 lb			= 0.7 * 1	7906 in-lb
rho= 1.0000				= 12,534	in-lb
xial Analysis			ом и принципального регультатуру в дом общений принципального принципального принципального принципального при	Commence of the state of the st	consumes ( (DPS) PANAPAGES of Amelianism and Associate A
KxLx/rx = 1.7*57.25"/1.267	, ,, ,		Fe > Fy/2		and Change of the Professional Angelogical Section 2015 and the Company of the Co
= 76.8	= 46.1	F	n= Fy(1-Fy/4Fe)		
5 <sup>m</sup> A C2 C ( ) **			= 55 ksi*[1-55 k	(4*49.3 ks	i)]
Fe= π^2E/(KL/r)max^	2 Fy/2= 27.5 ksi		= 39.7 ksi		
= 49.3ksi		F	Pa= Pn/Ωc		
Pn= Aeff*Fn	Ωc= 1.92		= 30192 lb/1.92		
= 30,192 lb P/Pa= 0.32	0.45		= 15,725 lb		
P/Pa= 0.32 ending Analysis	> 0.15				
Check: Pax/Pa + (Cmx*M	x)/(Max*ux) < 1.0	ind date that a contracting property is decreased in the contraction of the decreased annual contraction of the	graph Colombia (Communication Colombia) (Colombia) (Col	NAMES OF PERSONS AND A RESPONSIVE TO A STREET OF PARTY.	ent ya financia nichola di ya ni ili andola kitakili yiki ni wasar nak kekaya ya mari ya yiki 2 Milankin miliki asa ya wa osoba ya ya Nika kitakiki kitaki kitaki kitaki a nika kitaya miliki kitaki 20 Jili wa ciki
P/Pao + Mx/Max s	***				
Pno= Ae*Fy		Pao= Pno/Ωc	Mvield=1	∕ly= Sx*Fy	
= 0.761 in^2 *55000	) psi	= 41855lb/1.92	,		^3 * 55000 psi
= 41,855  lb		= 21,799 lb		= 44,935 is	
Max== My/Ωf		Down # ADET ////			
= 44935 in-lb/1.67		Pcr= π^2EI/(KL)max			
= 26,907 in-lb			/(1.7*57.25 in)^2		
$\mu x = \{1/[1-(\Omega c^*P/Pcr)]\}$	Δ.1	= 37,531 lb			
$= \{1/[1-(1.92*5056)]\}$					
== 0.74	m/2/221(n) 15T				
ombined Stresses					
то не при	The state of the s	Address of the Second Sec	(CPAP) di Sifter Milatinos (Common Craste). And Crastel Lide (Si Heliconnice America America (CPAP) His March Common II de Sprins Charles (Sprins And Pressure America America America America (CPAP) His March Common II de Sprins Charles (Sprins And Pressure America America America America (CPAP)	(AMAII) менін жеретінік класацы үліді (деремі) дінділеді. 1904 ж. Б. Санай консказацы (4,49 сіді (4,46 сіді (4,46 сіді)	epaignate mana chuidheadh ag ann an ann a manadh a dhann a' dhèirthe chann ann an ann an ann an ann ann ann an
	(0.85*12534 in-lb)/(26907 in-lb*0		< 1.0, OK	(EQ C5-1	.)
(5056 lb	)/21799 lb) + (12534 in-lb/26907 ir	n-lb) = 0.70	< 1.0, OK	(EQ C5-2	!)
XX Engagnesian telefore	deman abusan arms 1 1 2 1				
rui cultivarisori, total co	lumn stress computed for load case	e 5 is: 80.0%	g loads 5908.8.	58665 lb Axia	l and M≈ 9400 in-lb

# 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 #: 23-0500. REAM Configuration: TYPE 1 SELECTIVE RACK 5 DETERMINE ALLOWABLE MOMENT CAPACITY 2.50 in A) Check compression flange for local buckling (B2.1) 1.63 in W = c - 2\*t - 2\*r= 1.625 in - 2\*0.059 in - 2\*0.059 in = 1.389 in1.625 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 5.500 in < 0.673, Flange is fully effective 0.059 in 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/f1Eq. B2.3-5 Beam= UMH SB556 5.5"x2.5"x16ga = -2.004 $Ix = 3.219 in^4$  $k = 4 + 2*(1-Y)^3 + 2*(1-Y)$ Ea. B2.3-4  $Sx = 1.125 \text{ in}^3$ = 64.22Ycg= 3.630 in flat depth=w= y1+y3 t= 0.059 in = 5.264 inw/t= 89.22033898 OK Bend Radius=r= 0.059 in  $l=lambda= [1.052/(k)^0.5] * (w/t) * (f1/E)^0.5$ Fy=Fyv= 55.00 ksi = [1.052/(64.22)^0.5] \* 5.264 \* (51.53/29500)^0.5 Fu=Fuv= 65.00 ksi = 0.49< 0.673 E= 29500 ksi be=w= 5.264 in b2 = be/2Eq B2.3-2 top flange=b= 1.625 in b1 = be(3-Y)= 2.63 inbottom flange= 2.500 in = 1.052Web depth= 5.500 in b1+b2=3.682 in > 1.752 in, Web is fully effective Determine effect of cold working on steel yield point (Fya) per section A7.2 f1(comp) 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 inm = 0.192\*(Fu/Fy) - 0.068depth (EQ A7.2-4) = 0.1590Bc= 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) $^m$ f2(tension) (EQ A7.2-2) = 78.485 ksiThus, Fya-top= 58.92 ksi (tension stress at top) Fya-bottom= Fya\*Ycg/(depth -Ycg) y1= Ycg-t-r= 3.512 in = 114.37 ksi(tension stress at bottom) y2= depth-Ycg= 1.870 in Check allowable tension stress for bottom flange y3= y2-t-r= 1.752 in Lflange-bot=Lfb= Lbottom - 2\*r\*-2\*t = 2.264 inCbottom=Cb= 2\*Lc/(Lfb+2\*Lc) = 0.109Fv-bottom=Fyb= Cb\*Fyc + (1-Cb)\*Fyf = 57.57 ksi Fya= (Fya-top)\*(Fyb/Fya-bottom) = 29.66 ksiif F = 0.95Then F\*Mn=F\*Fya\*Sx=[ 31.69 in-k

# Engineering & Design Inc.



1815 Wright Ave La Verne, CA 91750 Tel: 909,596,1351 Fax: 909,596,7186

RMI 2.2, item 8

JJM/MQZ Project: Project #: SHELTERLAGIC 22.000BEAM 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

Beam Level= 1 P=Product Load= 3,600 lb/pair D=Dead Load= 75 lb/pair

E= 29500 ksi

F= 150.0

L= 144 in

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 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)]

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)

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)

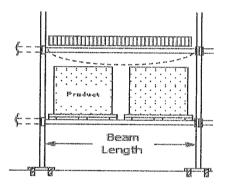
= 4,257 lb/pair allowable load based on deflection limits

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



Allowable Deflection= L/180

Deflection at imposed Load= 0.677 in

= 0.800 in

# Engineering & Design Inc.

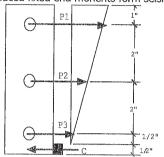


1815 Wright Ave. La Verne, CA 91750 Tel: 909 596 1351 Fax: 909 596 7186

By: JJM/MQZ Project: Project #: SHELTER LOGIC 23-0509---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 monents 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-lbLoad 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= 0.50 in

 $= 0.127 \text{ in}^3 * 0.66 * \text{Fy}$ 

= 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

# 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-£

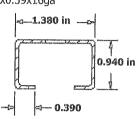
Transverse Brace

Configuration: TYPE 1 SELECTIVE RACK

### Section Properties

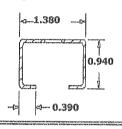
Diagonal Member= UMH C 1.38x0.94x0.39x16ga

ember= UMH C 1.38; Area= 0.214 in^2 r min= 0.356 in Fy= 55,000 psi K= 1.0 Ωc= 1.92



Horizontal Member= UMH C 1.38x0.94x0.39x16qa

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+[(0.85+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

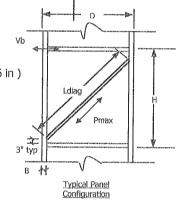
= 0.214 in^2 \* 12803 psi

= 2,740 lb

(ki/r)= (k \* Ldiag)/r min=  $(1 \times 53.7 in /0.356 in )$ = 150.8 inFe=  $pi^2*E/(kl/r)^2$ = 12,803 psi



Fn= Fe = 12,803 psi



Pallow=  $Pn/\Omega$ 

= 2740 lb /1.92

= 1,427 lb

Pn/Pallow=

0.96

<= 1.0 OK

### Horizontal brace

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

(k|/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

Pn= AREA\*Fn

= 0.214in^2\*19058 psi = 4.078 lb Pallow= Pn/Ωc

= 4078 lb /1.92= 2,124 lb

Fy/2= 27,500 psi

Pn/Pallow=

0.70

<= 1.0 OK



By: JJM/MQZ	Phone Land	CA 91750 Tel: 909.596.1351 F FER-LOGIC	THE RESIDENCE OF THE PARTY OF T	oject#: 22 OFOO F
Single Row Frame Overturning	Configuration: TYP	E 1 SELECTIVE RACK	котивия в Нейг Киртинг нам этогоройной о Наколичных на снецианий, я дентубующей для откож всехода. На берейную забат станов «Э.С. в «Выней еней вызычных не усторующей становальной должных надажений всехода все На берейную забат станов «Э.С. в «Выней еней вызычных не усторующей становальной деней всехода в подажений все	and a second
Loads				ат мандарындагы жастыр тарап кайра жашан жасыр ашашындарын дарын дарында жасыр жасыр жасыр жасыр үнөө үнөө үнө Мөм меруин на барын барын кастаттарын Бөйбөгөг магынан катырынан кеңине кеңине барын барын менен жасыр кеңине кеңи
Critical Load case(s):	SAMANY CONTRACTOR CONTRACTOR OF GRASH ASSAULT AND ANALYSIS ASSAULT	A report of the first of the fi	облика жануу түйүндө төбүйү да анын араанда байуун байта анын анын аттуун төбө өзөбөлүктөн жаналуу төгөөрүү ба Кашанын түйөө байын байша анын даша файн балдын байтаа такка жаналатын үгүн бүйүнүн байчан аймануу байтаатын б Өсөө	A MONEY
t) RMI Sec 2.2, item 7: (0.9-0.2Sds)D -	+ (0.9-0.20Sds)*B*Papp	- E*rho	hp	A CONTRACTOR OF THE PROPERTY O
			-	*   *   *
Minney No. W.		Sds= 0.856	•	
vtrans=v=e= DEAD LOAD PER UPRIGHT	Qe= 2,131 lb	(0.9-0.2Sds) = 0.728		<b>*</b>
PRODUCT LOAD PER UPRIGHT		(0.9-0.2Sds) = 0.728		
	.67= 9,648 lb	B= 1.000	O/NLCCLOP 0CC17,6:-99888578537	H h
Vst LC1=Wst1=(0.72866*D + 0.72866*Papp		rho= 1.000	J	
,	1, 7,21015	Frame Depth=Df= 44.0 ii	n	TA
Product Load Top Level, Pi	top= 3,600 lb	Htop-lvl=H= 240.0		
	/Lvl= 75 lb	# Levels= 4	""	- Di milio
Seismic Ovt based on E, $\Sigma$ (Fi*		# Anchors/Base= 2		Df Df
height/depth ra		hp= 48.0 ir	1	SIDE ELEVATION
) Fully Loaded Rack		h=H+hp/2= 264.0		
oad case 1:		NET COLD BE ANTICOPY OF THE METERS OF THE STATE OF THE ST	PT refer teles comments. Précéde des courses vant échies y écons le represent accessiva plantaire de Product teles comments. Précéde des courses vant échies y écons l'arran accessing à d'Augénées de d'Années de la comment de l'accessing de la comment de	THE CONTROL OF THE PROPERTY OF THE CONTROL OF THE C
Movt= $\Sigma(Fi*hi)*E*rho$		Wst1 * Df/2	T= (Movt-Mst	
= 295,596 in-lb		7248 lb * 44 in/2 159,456 in-lb	= (295596 ir = 3,094 lb	n-lb - 159456 in-lb)/44 in
тер стем с то образува, е доката закраже <sup>н</sup> иста в образува с да не в му услово бы москот годо водом в на точ част в му				
		Net Seismi	c Uplift= 3,094 lb	PPENNENS STEEL
		Net Seismi	c Uplift= 3,094 lb	
pad case 1:		Net Seismi		
oad case 1: ☼ V1=Vtop= Cs * Ip * Ptop >= 35	50 lb for H/D >6.0	Net Seismi	Movt [V1*h + V	
oad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb	50 lb for H/D >6.0	Net Seismi	Movt= [V1*h + V = 211,287 in	-lb
oad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb			Movt= [V1*h + V = 211,287 in T= (Movt-Mst)	i-lb I/Df
Dad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb	Critical Level= 4	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
<b>Dad case 1:</b> ☐ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb		1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst)	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D = 64 lb	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D = 64 lb Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Dad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Dad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1: □ V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D = 64 lb Mst= (0.72866*D + 0.7286	Critical Level= 4 Cs*Ip= (	4	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  □ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286  = 62,519 in-lb	Critical Level= 4 Cs*Ip= ( 56*Ptop*1) * 44 in/2	Net Seismi	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in	:-lb )/Df :-lb - 62519 in-lb)/44 in
Pad case 1:  ○ V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286  = 62,519 in-lb	Critical Level= 2 Cs*Ip= ( 56*Ptop*1) * 44 in/2	Net Seismi	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	:-lb )/Df :-lb - 62519 in-lb)/44 in
Dad case 1:  (2) V1=Vtop= Cs * Ip * Ptop >= 35	Critical Level= 4 Cs*Ip= ( 66*Ptop*1) * 44 in/2	Net Seismi	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	:-lb )/Df :-lb - 62519 in-lb)/44 in
Dad case 1:  O V1=Vtop= Cs * Ip * Ptop >= 35 = 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D = 64 lb Mst= (0.72866*D + 0.7286 = 62,519 in-lb  Uchor  Discharge (2) 0.5" x 3.25" Embed HILTI Ecial Inspection is required per ES Pullout Capacity=Tca	Critical Level= 4 Cs*Ip= ( 66*Ptop*1) * 44 in/2  KWIKBOLT TZ anchoi R 1917. pp= 1,961 lb	Net Seismi	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	:-lb )/Df :-lb - 62519 in-lb)/44 in
Dad case 1:  (2) V1=Vtop= Cs * Ip * Ptop >= 35  = 0.2142 * 3600 lb  = 771 lb  V1eff= 771 lb  V2=V <sub>DL</sub> = Cs*Ip*D  = 64 lb  Mst= (0.72866*D + 0.7286  = 62,519 in-lb  Minor are are are are are are are are are ar	Critical Level= 4 Cs*Ip= ( 66*Ptop*1) * 44 in/2  KWIKBOLT TZ anchor R 1917. ap= 1,961 lb ap= 2,517 lb	Net Seismic  (s) per base plate.  L.A. City Jurisdiction? NO Phi= 1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	-lb )/Df -lb - 62519 in-lb)/44 in Net Uplift per Colur
= 0.2142 * 3600 lb = 771 lb V1eff= 771 lb V2=V <sub>DL</sub> = Cs*Ip*D = 64 lb Mst= (0.72866*D + 0.7286 = 62,519 in-lb mchor mck (2) 0.5" x 3.25" Embed HILTI pecial inspection is required per ES Pullout Capacity=Tca	Critical Level= 4 Cs*Ip= ( 66*Ptop*1) * 44 in/2  KWIKBOLT TZ anchor R 1917. ap= 1,961 lb ap= 2,517 lb (1547 lb/1961 lb)^1	Net Seismic  (s) per base plate.  L.A. City Jurisdiction? NO Phi= 1 + (532 lb/2517 lb)^1 = 1	Movt= [V1*h + V = 211,287 in T= (Movt-Mst) = (211287 in = 3,381 lb	p*Phi= 1,961 lb p*Phi= 2,517 lb



Seeplake	man 191110011119	~ == 001911 #	110.				Fire Traffic
See Place   See	By: JJM/A			91750 Tel: 909,596	6.1351 Fax: 909.5	AND THE RESIDENCE AND THE PROPERTY OF THE PROP	отник меньки этимерене Мень или из стаким и прудоборующих на техника простига объекти и постига простига и пос Станительный и постига и постига и постига и прудоборующих на принценения постига постига постига и постига и п
Baseplates 8x5x0.375 ET Width = W - 7.00 in	PPECEP SILVER VON PERENDEN AL RECORDE TRATE I LES COMPENSATIONS DE CENTRE DE CONTRACTAMENTAL DE PLANTET DE CONTRACTAMENT DE COMPENSATION DE L'ARTICLE SERVICIA DE L'ARTICLE SE CONTRACTAMENT DE CONTRACTAMENT DE L'ARTICLE SE CONTRACTAMENT DE CONT	A E AND THE	Troject. SHELTER	<del>LOGIC</del>		Project #:	-23-0509-E
Baseplatine   BicSt0.3/5   Ef Width=W = 7.00 in   Anchor C.c. = 2*a=d = 5.00 in   N=# Anchor/Base= 2   Donnaise   Ecention   Donnaise   Donnaise   Ecention   Donnaise   Ecention   Donnaise   Ecention   Donnaise   Ecention   Donnaise   Ecention   Donnaise   Donnaise   Ecention   Donnaise   Donnaise   Ecention   Donnaise	Base Plate	Configuration: TYP	E 1 SELECTIVE RACK	pik ok projektimoru uma atauki (lipikim assama 1974) kalekakennya kepakikian Prinsi Charlesta Assama Assama ka kipiki kepaban assama 1974 kalekakennya kepakikian	ACCUMANTAL AND THE SECOND COMMENTS WANTED THE SECOND SECON	THE STATE OF THE S	
Eff Width=W = 7.00 in	Section				A CONTRACTOR OF THE CONTRACTOR		b and the second of the second
### PROPID—D = 5.00 in				nd 34			
Column Width-b = 3,00 in			a =	2.50 in			NA NA
Column Depth=edc = 3.00 in			Anchor c.c. $=2*a=d=$	5.00 in		1 /	A MID
L = 2.00 in   December   Decemb			N=# Anchor/Base=	2			b la 1 lsl
Potential Principles	•		Fy =	36,000 psi			
COLUMN PL= 7,200 lb							W
COLLIMN DL= 150 lb							
COLUMN PLE - 7,200 lb		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	THE MAN CONCRETE AND ADDRESS OF THE PARTY OF
Base Momente 8,000 in-lb					lb + 0.75 * (1.51	9938 * 0.7 * 720	0 lb)
1+0.105*Sds= 1.0900				•			
1.4+0.14Sds		'					
Axial stress=fia = P/A = P/(D*W)		-1000			*0.7*rho		
Axial stress=fa = P/A = P/(D*W)		A forest and a second service of the second service of the second service of the second second service of the second seco				The second secon	E
Moment Stress=fb			Axiai Load P =	THE RESERVE AND ADDRESS OF THE PARTY OF THE			Ef
Moment Stress=fb = M/S = 6*Mb/[(0*B^2]						2	
Moment Stress=fb1 = fb-fb2			^21				
Moment Stress=fb1 = fb-fb2  ### ### ### ### ### ### ### ### ### #			دا		·		
### ### ##############################					•		
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2					**		
= 78 ln-lb S-plate = (1)(t/2)/6 S-plate = (1)(t/2)/		,	$L = (1/3)*fb2*L^2$		,-		
S-plate = (1)(t^2)/6							
= 0.023 in^3/in	S-plate =	(1)(t^2)/6			,		
fb/Fb = Mtotal/[(S-plate)(Fb)]	WARD.	0.023 in^3/in			•		
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2] Tallow= 1,961 lb OK  Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2] Tallow= 1,961 lb OK  Tallow= 1,961 lb OK  Tallow= 1,961 lb OK  Tallow= 1,961 lb OK  Check uplift load on Baseplate  Check uplift load on Baseplate  Check uplift forces on baseplate with 2 or more anchors per RMI 7.2.  When the base plate configuration consists of two anchor boths located on either side of the column and a net uplift force on 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 be column be plate and 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 be column and a net uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column be column be column be column and a net uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column be column and a net uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column be column and a net uplift force on one anchor times 1/2 the distance from the centerline of the anchor to the nearest edge of the rack column Uplift per Column: 3,380 lb Qty Anchor per BP= 2  Net Tension per anchor: Ta= 1,690 lb C 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 =	Mtotal/[(S-plate)(Fb	)]		·		
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2] Tallow= 1,961 lb OK  OSS Aisle Loads  Other load case RMI Sec 2.1, Rem 4: (1+0.1586)(pl.4-0.75+pl.4-0.75 <-1.0, ASD Method  Other load case RMI Sec 2.1, Rem 4: (1+0.1586)(pl.4-0.75+pl.4-0.75 <-1.0, ASD Method  Other load case RMI Sec 2.1, Rem 4: (1+0.1586)(pl.4-0.75+pl.4-0.75 <-1.0, ASD Method  Other load on Baseplate  Check uplift load on Baseplate with 2 or more anchors per RMI 7.2.  When the base plate configuration consists of two anchor boths 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  I = Base Plate Depth-Col Depth= 2.00 in  I = P/A = P/(D*W)  = 270 psi  Sbase/in = (1)(t^2)/6  = 0.023 in^3/in  Fbase = 0.75*Fy  = 0.023 in^3/in  Fbase = 0.75*Fy  = 0.085  OK  Mu=Moment on Baseplate due to uplift= Ta*c/2  = 1,690 in-lb  Splate= 0.117 in^3						OK	
= -2,784 lb No Tension Obs Aisle Loads			.46)(a))/[(d)*N/2]				
Pstatic= 5,909 lb  Movt*0.75*0.7*rho= 155,188 in-lb Frame Depth= 44.0 in  Pseismic= Movt/Frame Depth Frame Depth= 44.0 in  Pseismic= Movt/Frame Depth  B = 3,527 lb  Check uplift force on baseplate with 2 or more anchors per RMI 7.2.  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  M = WL^2/2 = fa*L^2/2 = 539 in-lb/in  Sbase/in = (1)(t^2)/6		•					
Movt*0.75*0.7*rho= 155,188 in-lib Frame Depth= 44.0 in  Pseismic= 9,436 lb  b = Column Depth= 2.00 in  fa = P/A = P/(D*W) = 270 psi  Sbase/in = (1)(t^2)/6 = 0.023 in^3/in  fb/Fb = M/[(S-plate)(Fb)] = 0.85  OK  Pseismic= Movt/Frame Depth  Pseismic= Movt/Frame Depth = 3,527 lb  Pseismic= Movt/Frame Depth = 3,527 lb  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 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	ross Aisie Loads	Critical load case RMI Sec 2.1, ite	m 4: (1+0.115ds)DL + (1+0.145D5)PL*0.	75+EL*0.75 <= 1.0, ASD Method	THE REAL PROPERTY AND PERSONS ASSESSMENT OF THE PERSONS ASSESSMENT OF		21. WARRING BY 1844 (1) TO 1844 (1)
Movt*0.75*0.7*rho= 155,188 in-lb Frame Depth= 44.0 in  Pseismic= Movt/Frame Depth  B = 3,527 lb  Multiplication and an et 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  Multiplication  B = P/A = P/(D*W)  B = 270 psi  Sbase/in = (1)(t^2)/6  B = 0.023 in^3/in  Fbase = 0.75*Fy  B = 27,000 psi  Fbase = 0.75*Fy  B = 27,000 psi  Mu=Moment on Baseplate due to uplift= Ta*c/2  B = 1,690 in-lb  Splate= 0.117 in^3	Plenderski	r' ana ii-					
Movt*0.75*0.7*rho= 155,188 in-lb Frame Depth= 44.0 in  Pseismic= Movt/Frame Depth Frame Depth= 44.0 in  Pseismic= Movt/Frame Depth  B = 3,527 lb  Pseismic= Movt/Frame Depth  B = 3,527 lb  B = Column Depth= 3.00 in  C = Base Plate Depth-Col Depth= 2.00 in  Frame Depth= 44.0 in  B = P/A = P/(D*W)  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	RASPERTIC	5,909 10			II .		
Frame Depth= 44.0 in  Pstatic+Pseismic= 9,436 lb  D = Column Depth= 3.00 in  L = Base Plate Depth-Col Depth= 2.00 in  fa = P/A = P/(D*W)	Mov##0 75*0 7*rbo	155 199 in lb	Danierale	March/Europe B. H			
### Pstatic+Pseismic= 9,436 lb    b = Column Depth= 3.00 in     L = Base Plate Depth-Col Depth= 2.00 in     fa = P/A = P/(D*W)							
b = Column Depth= 3.00 in  L = Base Plate Depth-Col Depth= 2.00 in  fa = P/A = P/(D*W)			٦ -	3,327 ID	to the uplift force of	n one anchor times I	1/2 the distance from
L =Base Plate Depth-Col Depth= 2.00 in $fa = P/A = P/(D*W)$ $= 270 \text{ psi}$ $Sbase/in = (1)(t^2)/6$ $= 0.023 \text{ in}^3/\text{in}$ $fb/Fb = M/[(S-plate)(Fb)]$ $= 0.85$ $M= wL^2/2 = fa*L^2/2$ $= 539 \text{ in}-lb/\text{in}$ $W= wL^2/2 = fa*L^2/2$ $= 539 \text{ in}-lb/\text{in}$ $Uplift per Column= 3,380 \text{ ib}$ $Qty Anchor per BP= 2$ $= 27,000 \text{ psi}$ $Mu=Moment on Baseplate due to uplift= Ta*c/2$ $= 1,690 \text{ in}-lb$ $Splate= 0.117 \text{ in}^3$	MAKE THE PROPERTY OF THE PROPE	TO CONCLUDE AND ADDRESS OF THE PARTY OF THE	! = 3.00 in		une centerline of the a	nchor to the nearest ed	dge of the rack column'
fa = P/A = P/(D*W) = 270 psi  Sbase/in = (1)(t^2)/6 = 0.023 in^3/in  Fbase = 0.75*Fy = 0.023 in^3/in  Fbase = 0.75*Fy = 27,000 psi  Fbase = 0.75*Fy = 27,000 psi  M= wL^2/2= fa*L^2/2 = 539 in-lb/in  Uplift per Column= 3,380 lb Qty Anchor per BP= 2 Net Tension per anchor=Ta= 1,690 lb		D GOIGHT DOPGIT	5,00 11		- c	* 4	
fa = P/A = P/(D*W) = 270 psi  Sbase/in = (1)(t^2)/6 = 0.023 in^3/in  Fbase = 0.75*Fy = 0.023 in^3/in  Fbase = 0.75*Fy = 27,000 psi  Fbase = 0.75*Fy = 27,000 psi  M= wL^2/2= fa*L^2/2 = 539 in-lb/in  Uplift per Column= 3,380 lb Qty Anchor per BP= 2 Net Tension per anchor=Ta= 1,690 lb	L ≕Base Pla	te Depth-Col Depth=	: 2.00 in		To 1 Mu	Ta Ta	
= 270 psi			MIOO III		" 11	11	
= 270 psi	fa =	P/A = P/(D*W)	M= 1	wL^2/2= fa*L^2/2		b   •	
Sbase/in = $(1)(t^2)/6$ Fbase = $0.75*Fy$ Qty Anchor per BP = $2$ Net Tension per anchor=Ta = $1,690$ lb c = $2.00$ in Mu=Moment on Baseplate due to uplift = $1,690$ in-lb Splate = $0.117$ in^3						Floration	
Sbase/in = (1)(t^2)/6			·	-,			n= 3 380 lb
= 0.023 in^3/in = 27,000 psi 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	Sbase/in =	(1)(t^2)/6	Fbase = (	).75*Fy			
fb/Fb = M/[(S-plate)(Fb)]  = 0.85 OK  C = 2.00 in  Mu=Moment on Baseplate due to uplift= Ta*c/2  = 1,690 in-lb  Splate= 0.117 in^3	kinese Quarte	0.023 in^3/in		•	Net Ter		
fb/Fb = M/[(S-plate)(Fb)] = <b>0.85 OK</b> Mu=Moment on Baseplate due to uplift= Ta*c/2 = 1,690 in-lb Splate= 0.117 in^3				-			
= <b>0.85 OK</b> = 1,690 in-lb Splate= 0.117 in^3	fb/Fb =	M/[(S-plate)(Fb)]		!	Mu=Moment on Ba		
Splate= 0.117 in^3	ence anna	0.85	ОК				
I TOTAL CONT. TO LOCAL CONT. C						Splate	
	i-i-i-cura cura dun contigue son manor e como e con delar uno que participado e i describanciamente. Característica dun acontigo de la como esta con esta con la sura cura que participado de describa de constituciones.	именном и и помери предпадат помени помени добо и добо и и помени помени помени помени помени помени помени по С поменя с том и помени пом	actives of the section of the sectio	10-000-00-00-00-0-0-0-0-0-0-0-0-0-0-0-0	[fb/Fb]*0.75=		ОК



TO SECURE THE TOTAL SECURITY FOR A PUBLISH A CONTROL OF THE TOTAL SECURITY ASSESSMENT OF THE TOTAL		Ave La Verne.	CA 91750 Tel:	909.596.1351 Fax: 909.596.7	186
Ву;	JJM/MQZ	Project: SHCL:	TER LOGIC	TO COMMAND WINDOWS STREET, MAKE THE STRE	Project #: 22 OCOO F
Slab on Grade	Co	nfiguration: TYPE 1	SELECTIVE RAG	CK	
Effec. Baseplate width=B=	P P P P P P P P P P P P P P P P P P P	t t width=a= 3.00 in	D	b e Cros Aisle  Down Aisle  Baseplate Plan View	
Effec. Baseplate Depth=D=	5.00 in	depth=b= 3.00 in			$\beta = B/D = 1.400$
Column Loads				nn to edge of plate=c= 5.00 in nn to edge of plate=e= 4.00 in	F'c^0.5= 50.00 psi
PRODUCT LOAD=P=  Papp= P-seismic=E= =  B= rho= Sds= 1.2 + 0.2*Sds= 0.9 - 0.20Sds=  Puncture	0.7287		Load Cas  Load Cas  Load Cas	e 1) (1.2+0.2Sds)D + (1.2+0.2S = 1.37134 * 150 lb + 1.37134 = 13,835 lb	* 0.7 * 7200 lb + 1 * 6718 lb s)*B*Papp + rho*E RMI SEC 2.2 EQTN 7
= Fpunct1= = Fpunct2=	[(c+t)+(e+t)]*2*t 252.0 in^2 [(4/3 + 8/(3* $\beta$ )] * $\lambda$ *(F'c 97.1 psi 2.66 * $\lambda$ * (F'c^0.5) 79.8 psi 79.8 psi	^0.5)		fv/Fv= Pu/(Apunct = <b>0.701</b>	*Fpunct) < 1 OK
= X= = Fb=	14,098 ib  (Pse*144)/(fsoil) 2,707 in^2 (L-y)/2 17.8 in 5*(phi)*(f'c)^0.5 150. psi	L= (Asoil)^ = 52.03 in M= w*x^2/ = (fsoil*x/ = 823.1 in	2 ^2)/(144*2)	y= (c*e)^0.5 = 16.5 in S-slab= 1*teff^2/6 = 6.0 in^3 fb/Fb= M/(S-slab*) = <b>0.915</b>	

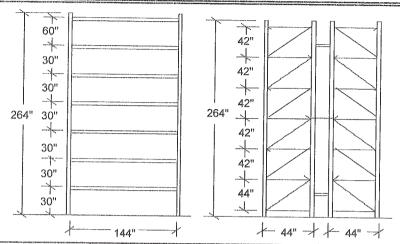
# Engineering & Design Inc.



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

By: JJM/MQZ Project: SHIFLTER 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	# 8m Lvis	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

Comp	onent			Description						
Colu	ımn	Fy=55 ksi	UM	H C3313TD 3x3	x13ga	P=72	P=7238 lb, M=6088 in-lb			
Column 8	& Backer	None		None	The second section of the sect	And in the second service of the second seco	None	0.48-OK N/A		
Bea	am	Fy=55 ksi	UMF	SB556 5.5"x2.5	"x16ga	Lu=144 in	Capacity: 4257 lb/pr		0.7-OK	
Beam Co	nnector	Fy=55 ksi	Lvl 4:	3 pin OK	Mconn≕5	349 in-lb	Mcap=12	0.44-OK		
Brace-Ho	orizontal	Fy≕55 ksi		TO BETWEEN THE COMMENT OF STREET, THE STREET, STREET, STREET, STREET, STREET, STREET, STREET, STREET, STREET,	UMH C 1.38x0.9	94x0.39x16ga	The state of the s	AND DESCRIPTION OF THE PARTY OF	0.7-OK	
Brace-D	iagonal	Fy≃55 ksi			UMH C 1.38x0.9		Michigan Company of the Company of t	ereigner, accepter Label of the copy and copy of the constitution of	0.95-OK	
Base	Plate	Fy=36 ksi		8x5x0.375   Fixity= 6089 in-lb						
Ancl	hor	2 per Base	0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic Uplift=3002 lb)						0.84-OK 0.817-OK	
Slab 8	k Soil	diam's and a second	6" thk x 2500 psi slab on grade. 750 psf Soil Bearing Pressure				essure	California de la constantia della constantia della consta	0.89-OK	
Level	Load**			Story Force Story Force Column Column Conn.					Beam	
Districtive of the section of the se	Per Level	Beam Spcg	Brace	Transv	Longit.	Axial	Moment	Moment	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	, , , , , , , , , , , , , , , , , , , ,					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	1,020 10 0,202 11 1,170 11					3 pin OK	
7	3,000 lb	60.0 in		691 lb	295 lb	1,538 lb	4,422 "#	2,833 "#	3 pin OK	

				1			
** Load defined as product weight per pair of beams	Total:	2,114 lb	902 lb	TANNA CALLA LINEAR CONTROL PROCESSION PROPERTY OF THE CONTROL OF T			
Notes	THE RESIDENCE AND ADDRESS OF THE PERSON	ALTERNATION TO THE PARTY OF THE		The final design and the contract of the contr			
2.5" BEAM OK @ LEVELS 1-3							
PETAL MARKA CHI PROGRAMMA AND							

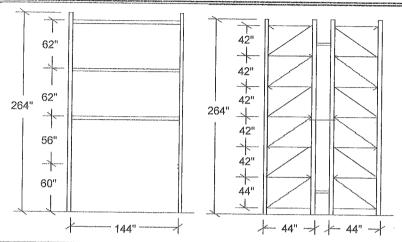
# Engineering & Design Inc.



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	# 8m 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

Comp	onent	The second secon	Description							
Colu	ımn	Fy=55 ksi	UN	TH C3312TD 3x3	x12ga	00 lb, M=15028	STRESS 0.72-OK			
Column 8	& Backer	None		None	None	THE CONTRACT OF THE PROPERTY O				
Bea	am	Fy=55 ksi	UMI	SB556 5.5"x2.5	Capacity:	4257 lb/pr	N/A 0.85-0K			
Beam Co	onnector	Fy=55 ksi	Lvi 2:	3 pin OK	Mconn=8	525 in-lb	ACTION AND DESCRIPTION OF THE PARTY.	Mcap=12578 in-lb		
Brace-Ho	orizontal	Fy=55 ksi	A STATE OF THE PROPERTY OF T	UMH C 1.38x0.94x0.39x16ga						
Brace-D	iagonal	Fy=55 ksi		UMH C 1.38x0.94x0.39x16ga						
Base	Plate	Fy=36 ksi		8x5x0.375 Fixity= 8000 in-lb						
Anc	hor	2 per Base	0.5" x 3.25" Embed HILTI KWIKBOLT TZ ESR 1917 Inspection Reqd (Net Seismic				d (Net Seismic U	Jplift=3332 lb)	0.76-OK 0.775-OK	
Slab 8	Slab & Soil			6" thk x 2500 psi slab on grade. 750 psf Soil Bearing Pressure						
Level	Load**			Story Force Story Force Column Column Con					0.77-OK Beam	
	Per Level	Beam Spcg	Brace						Connector	
							Charles Company of the Company of th	According to the Control of the Cont	Mirrord and a control of the control	
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	4 3,600 lb 62.0 in 42.0 in 804 lb		804 lb	343 lb	1,850 lb	5,318 "#	3,403 "#	3 pin OK		
			42.0 in				•	•		
			42.0 in							

** Load defined as product weight per pair of beams		Total:	1,894 lb	808 lb	(1995年) 中国中国中国中国中国中国中国中国中国中国中国中国中国中国中国中国中国中国中国
Notes			***************************************	A STATE OF THE PARTY OF THE PAR	With the Control of t
THE RESIDENCE AND ADDRESS OF THE PERSON OF T					
					· · · · · · · · · · · · · · · · · · ·