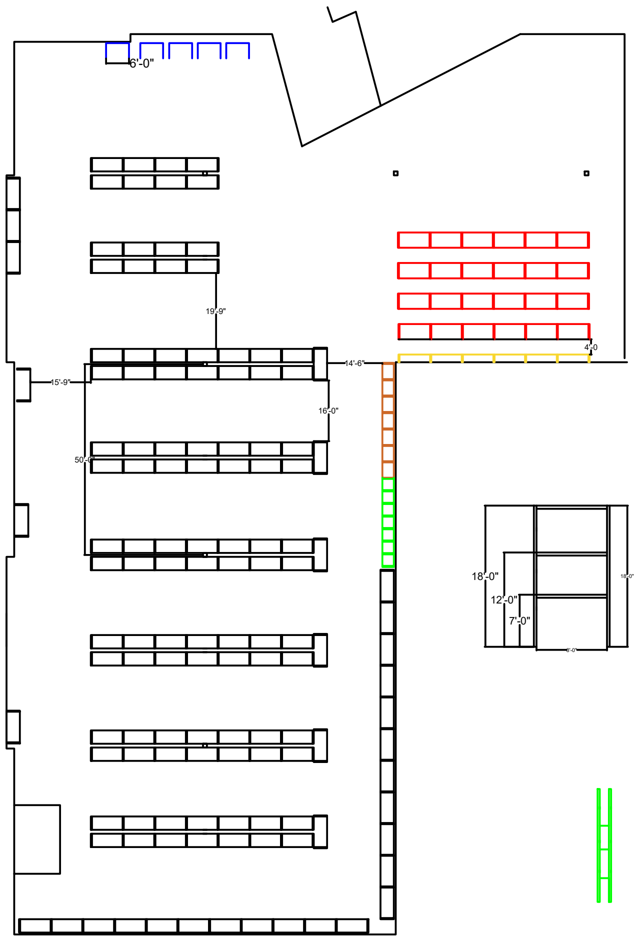
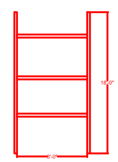


STATE OF MISSOURI
ENHAO ZHANG
NUMBER
PE-2022045911
PROFESSIONAL ENGINEER

THE P.E. CERTIFICATION PROVIDED HEREIN PERTAINS TO THE ADEQUACY OF THE STORAGE RACK STRUCTURAL COMPONENTS ONLY. ALL OTHER AREAS ARE OUTSIDE THE SCOPE OF WORK OF THIS CERTIFICATION. .



Uprights 8'x48" Teardrop
96x2.75 2100 CAP/shelf

Uprights 8'x24" Teardrop
96x2.75 2100 CAP/shelf

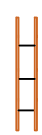
BEAM LEVELS TBD ON SITE

Uprights 18'x42" Teardrop
NEW ERA PRODUCTS
96x4" beams MECALUX
BEAMS CAP/shelf
42x46 Wire decking
(2500 lbs cap wwmaterial
handling)

Single Sided Cantilever
(MECALUX)
12'tall with COL.H23 C W
8"X18 U 12
BASE H23 W 8"X18 U 4'

48" Straight Arm H23
S5"x10 U 4'

Uprights 8'x36" Teardrop
Reel Rack 3' wide

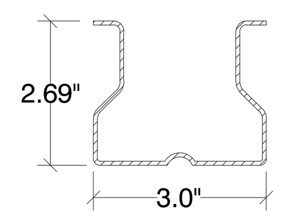


Uprights 8'x36" Teardrop
Reel Rack 4' wide

JOB NAME: BUTLER SUPPLY

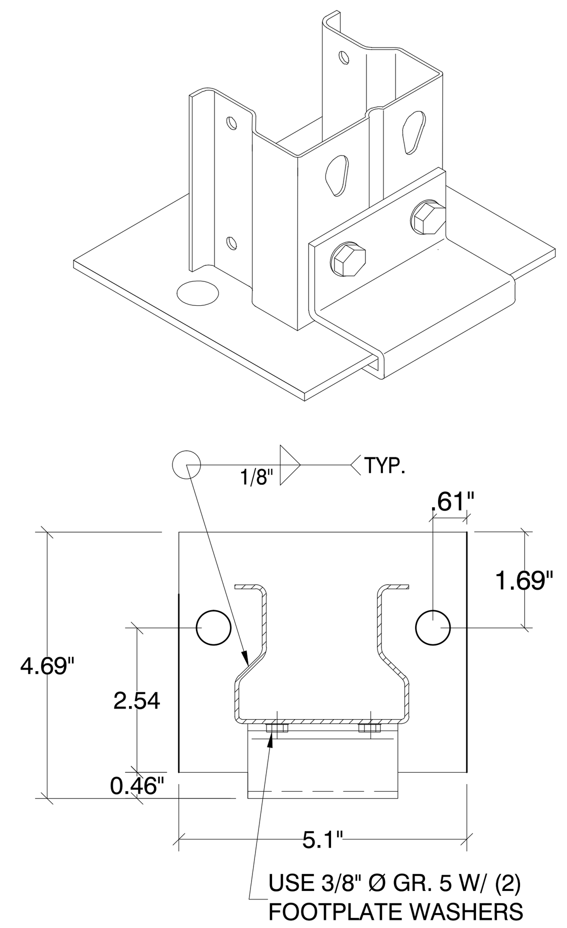
LOCATION: 2736 NE MCBAINE
DR
Lees Summit MO 64064

1B



COLUMN

1

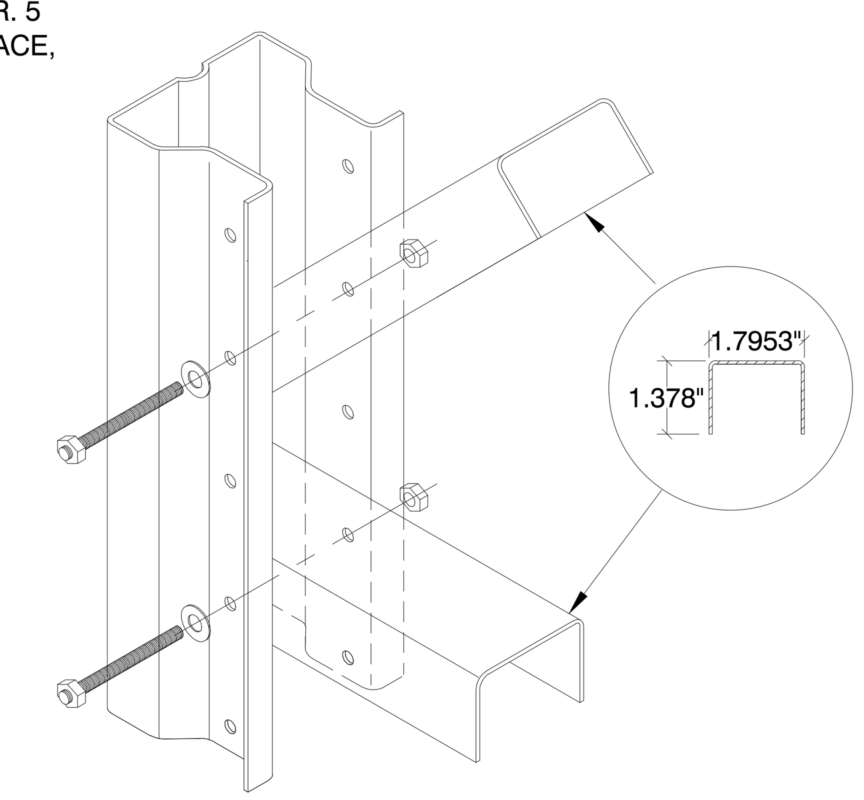


NOTES:
USE @ ALL COLUMN/BASE LOCATIONS TYP.

DESCRIPTION	BASE PLATE: 5.094" X 4.688"	DESCRIPTION	COLUMN: MCLX 314
MATERIAL	0.194" THICK PLATE	MATERIAL	14 GAGE THK STEEL
STEEL YIELD	ASTM A36, Fy=36,000 PSI	STEEL YIELD	ASTM A570, Fy=55,000 PSI

2

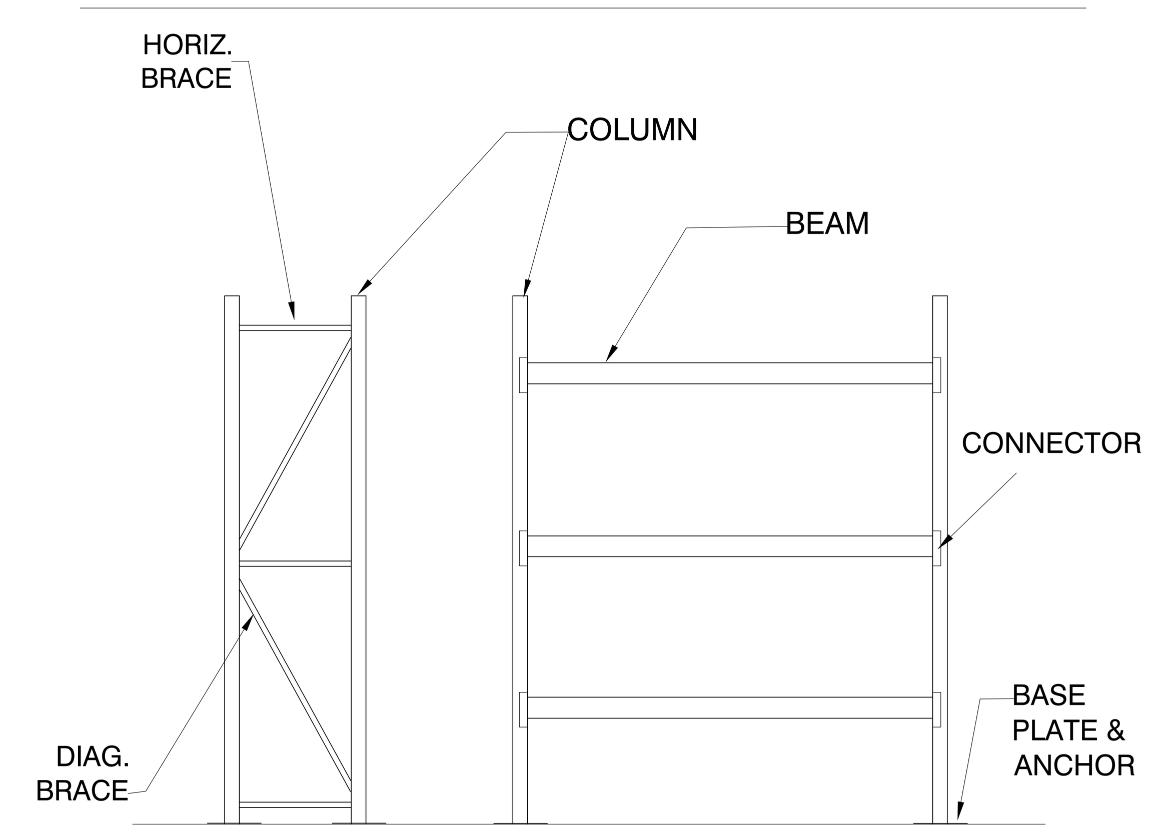
ATTACHMENT:
ATTACH W/ (4) 5/16"Ø X 3/4" GR. 5 BOLTS PER BRACE, (2) @ EA. END



DESCRIPTION	BRACING: HORIZ. & DIAG.
MATERIAL	16 GAGE STEEL (C456)
STEEL YIELD	ASTM A570, Fy=55,000 PSI

NOTES:
USE @ ALL MCLX BRACE LOCATIONS TYP.

GENERAL RACK CONFIGURATION

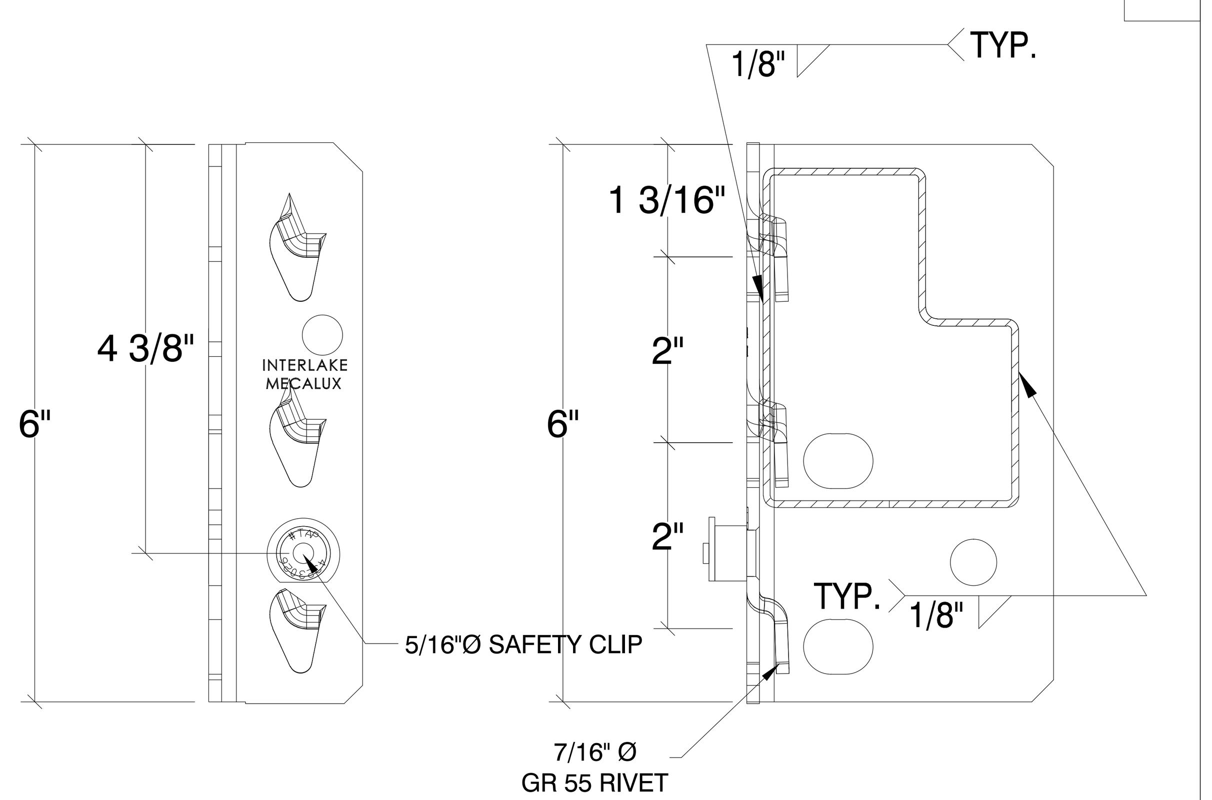


NOT A DEPICTION OF THIS PROJECT

GENERAL PROJECT NOTES

1. DESIGNED PER REQUIREMENTS OF THE 2018 IBC, ASCE 7-16 AND THE 2012 RMI RACK DESIGN MANUAL
2. SEISMIC CRITERIA Ss = 0.099, S1 = 0.068, Fa = 1.600, Fv = 2.400
   Ip = 1.0 (NO PUBLIC ACCESS), Sds = 0.106, Sd1 = 0.109, OCC. CATAG. II,
   SITE CLASS D-DEFAULT, SEISMIC DESIGN CATAG. B
3. STORAGE CAPACITY:
   TYPE 1 SELECTIVE RACK = 3000 LBS PER LEVEL TYP.
4. ANCHORS : HILTI KWIKBOLT TZ2 ESR #4266 (50 FT-LBS TORQUE), OR POWERS SD2 ESR#2502 (40 FT-LBS TORQUE)
   1/2" Ø x 3-1/4" MIN. EMBEDT.
   (1) ANCHOR PER BASE PLATE
5. PERIODIC SPECIAL INSPECTION IS REQUIRED DURING ANCHOR INSTALLATION. ANCHORS SHALL BE INSTALLED PER ICC ESR#4266.
6. EXISTING S.O.G. CONCRETE THICKNESS & COMPRESSIVE STRENGTH, 6" x 4000 PSI
7. SOIL BEARING PRESSURE 750 PSF.
8. ALL RACK INSTALLATIONS AND RACKS MANUFACTURED IN CONFORMITY WITH THIS STANDARD SHALL DISPLAY IN ONE OR MORE CONSPICUOUS LOCATIONS A PERMANENT PLAQUE EACH NOT LESS THAN 50 SQUARE INCHES IN AREA AND SHOWING THE MAXIMUM PERMISSIBLE UNIT LOAD IN CLEAR, LEGIBLE PRINT.
9. ALL BOLTS GR. 5 OR BETTER, INSTALL TO SNUG TIGHT FIT OR BETTER
10. ALL WELDING PERFORMED IN THE SHOP OF AN APPROVED FABRICATOR BY AWS CERTIFIED WELDERS USING E70XX ELECTRODE OR BETTER. NO FIELD WELDING PERFORMED. SPECIAL INSPECTION IS REQUIRED ONLY FOR ANY FIELD WELDING.
11. THE CLEAR SPACE BELOW SPRINKLERS SHALL BE A MIN. OF 18" BETWEEN TOP OF THE STORAGE AND THE CEILING SPRINKLER DEFLECTOR.
12. THE PRODUCT SHOWN ON THE DETAILS HEREIN IS ASSUMED TO BE IN GOOD, UNDAMAGED CONDITION. THE PRODUCT MUST BE FREE OF ANY DAMAGE AND/OR FABRICATION DEFICIENCIES OR IRREGULARITIES. IT IS THE RESPONSIBILITY OF THE OWNER, USER, PRODUCT PROVIDER AND/OR INSTALLER OF THE COMPONENTS TO NOTIFY SED, INC IN WRITING, OF ANY DAMAGE, DEFICIENCIES OR IRREGULARITIES.
13. IT IS THE RESPONSIBILITY OF THE OWNER AND/OR USER OF THE COMPONENTS SHOWN HEREIN TO NOTIFY SED, INC OF ANY DAMAGE OR DEFICIENCIES IN THE SYSTEM DURING USE. THIS INCLUDES SUCH OCCURRENCES SUCH AS IMPACT DAMAGE TO THE COMPONENTS FROM FORKLIFT OR HEAVY MACHINERY, IMPACT FROM DROPPED LOADS ON THE SYSTEM, DAMAGE TO SYSTEMS BY IMPROPER USE OR INSTALLATION, ETC. THE USER OF THE PRODUCT MUST NOTIFY SED, INC IN WRITING SHOULD ANY DAMAGE OR DEFICIENCY OCCUR TO THE PRODUCT SHOWN HEREIN. IT IS THE RESPONSIBILITY OF THE OWNER AND/OR USER TO MAINTAIN THE SAFETY AND PROPER USE OF THE STORAGE PRODUCT SHOWN HEREIN.
14. IT IS THE RESPONSIBILITY OF THE OWNER/USER OF THE STORAGE SYSTEM SHOWN HEREIN TO REPAIR OR REPLACE ANY COMPONENT THAT IS DAMAGED OR OTHERWISE DEFICIENT.
15. THE OWNER SHALL MAINTAIN THE STRUCTURAL INTEGRITY OF THE RACK SYSTEM BY ASSURING PROPER OPERATIONAL, HOUSE KEEPING AND MAINTENANCE PROCEDURES, BUT NOT LIMITED TO THE FOLLOWING:
   a. PROHIBIT ANY OVER LOADING OF ANY PALLET POSITIONS AND OF OVERALL RACK SYSTEMS.
   b. REGULARLY INSPECT FOR DAMAGE. IF DAMAGE IS FOUND, IMMEDIATELY UNLOAD THE AFFECTED AREA AND REPLACE OR REPAIR ANY DAMAGED COLUMNS, BEAMS, OR OTHER STRUCTURAL COMPONENTS.
   c. REQUIRE ALL PALLETS TO BE MAINTAINED IN GOOD, SAFE, OPERATING CONDITION.
   d. ENSURE THAT PALLETS ARE PROPERLY PLACED ONTO PALLET LOAD SUPPORT MEMBERS IN PROPERLY STACKED AND STABLE POSITION.
   e. REQUIRE THAT ALL GOODS STORED ON EACH PALLET TO BE PROPERLY STACKED AND STABLE.
   f. PROHIBIT DOUBLE STACKING OF ANY PALLET POSITION, INCLUDING THE TOP MOST POSITION, UNLESS THE RACK SYSTEM IS SPECIFICALLY DESIGNED FOR SUCH LOADING.
   g. THE OWNER AND/OR USER OF THE RACK SYSTEM MUST PROVIDE MEASURES TO MITIGATE DAMAGE TO THE STORAGE RACK BY USE OF IMPACT PROTECTIVE DEVICES IN AREAS WHERE FORKLIFT AND/OR HEAVY MACHINERY ARE IN USE.

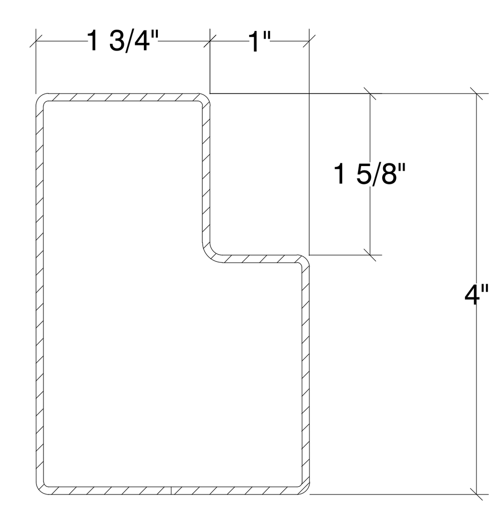
3



DESCRIPTION	3-TAB CONNECTOR
MATERIAL	7 GAGE
STEEL YIELD	ASTM A570, Fy=55,000 PSI

NOTES:
USE @ ALL BEAM TO COLUMN CONNECTION LOCATIONS TYP.

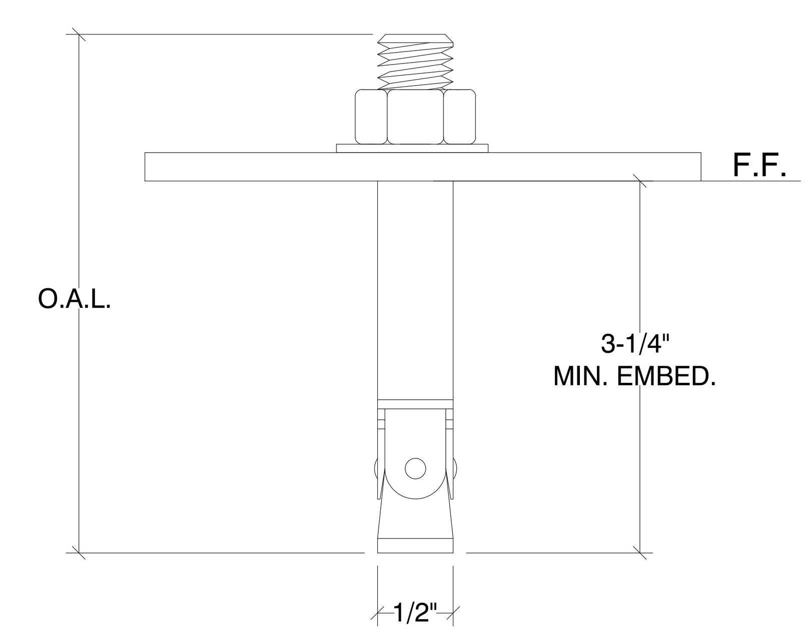
4



DESCRIPTION	4" BEAM (INTLK 40E)
MATERIAL	16 GAGE
STEEL YIELD	ASTM A570, Fy=55,000 PSI

NOTES:
USE @ ALL BEAM LOCATIONS TYP.

5



NOTES:
CONFIRM O.A.L. OF ANCHORS WITH INSTALLER TO ENSURE REQUIRED EMBEDMENT IS OBTAINED.

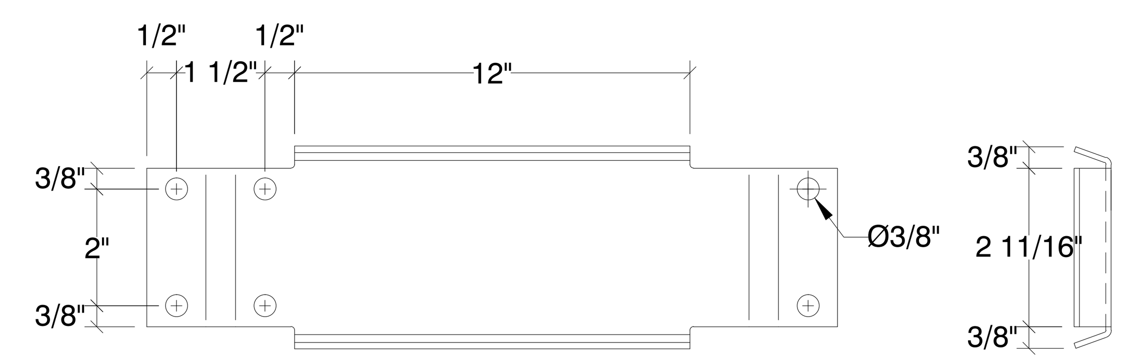
DESCRIPTION	HILTI KWIKBOLT TZ2 ANCHOR
SIZE	1/2"Ø X 3-1/4" MIN. EMBED.
ESR#	4266

NOTES:
SEE NOTE #4 ABOVE FOR ANCHOR SPECS.

SEE ATTACHED PALLET RACK KC DWG.
FOR STORAGE RACK ELEVATIONS.

DESCRIPTION	STORAGE RACK ELEVATIONS

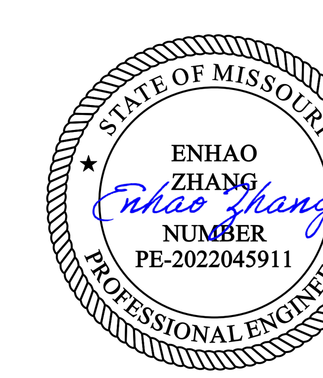
6



NOTES:
USES (3) ROW SPACERS FOR DOUBLE ROW UNITS

DESCRIPTION	STD ROW SPACER
MATERIAL	14 GAGE
STEEL YIELD	ASTM A570, Fy=55,000 PSI

NOTES:
ATTACH WITH (4) 5/16"Ø GR. 5 BOLTS, (2) @ EACH END



REVISION

REV. | DATE

STRUCTURAL ENGINEERING & DESIGN INC.
1815 WRIGHT AVE. LA VERNE, CA 91750 PHONE 909.596.1351 FAX 909.596.7186

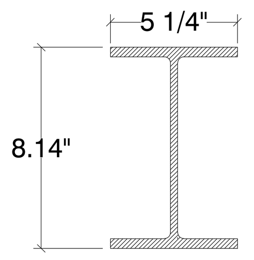
BUTLER SUPPLY
2736 NE MCBAINE DR
LEE'S SUMMIT, MO 64064

CLIENT PALLET RACK KC
DATE 06/30/2025
DRWN BY DAS ENG. BY BOB
MFG. MECALUX
TYPE STORAGE RACK

JOB NO.
25-0625-3

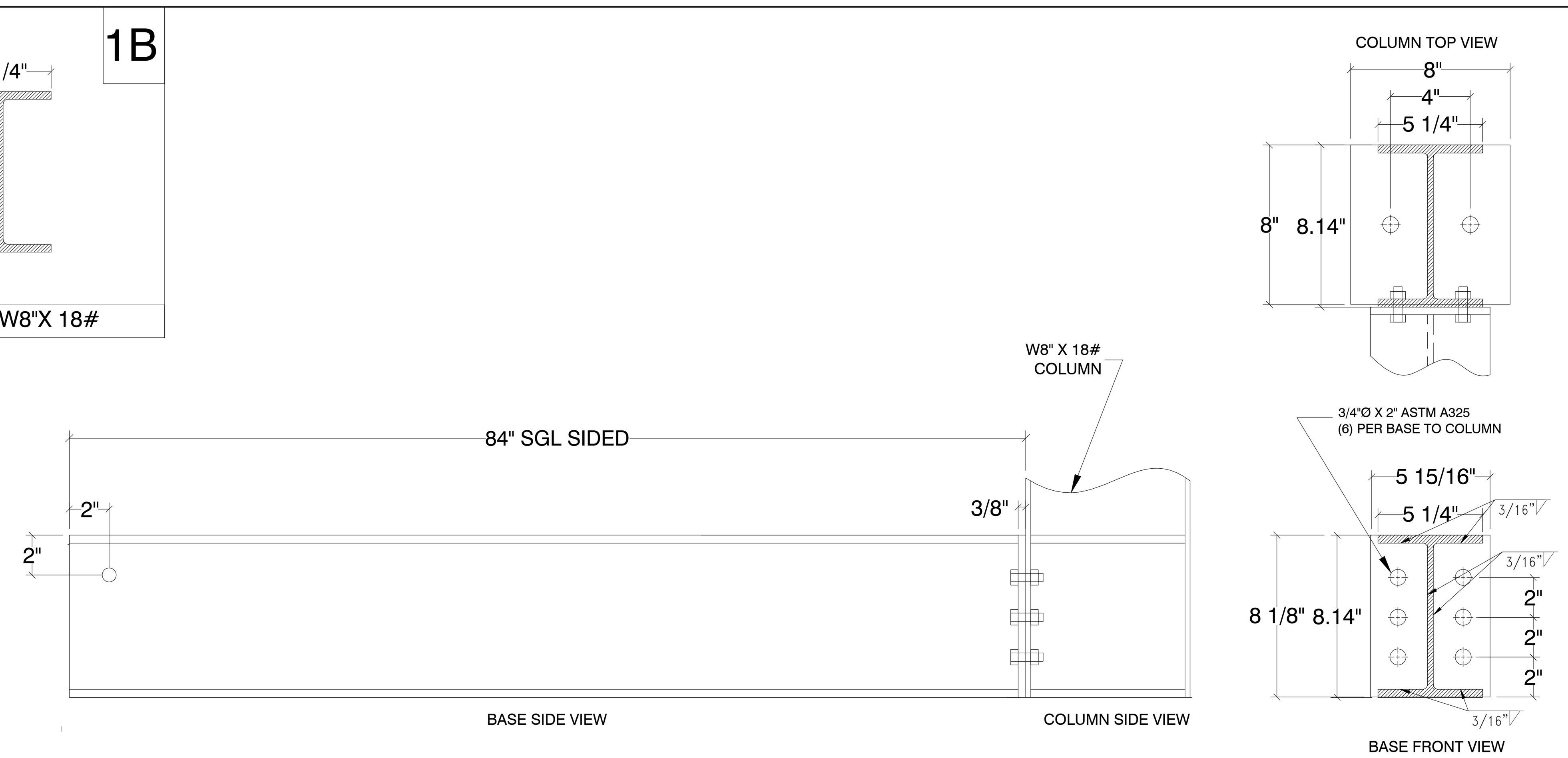
SHEET NO.
SED 1 OF 2

1B



COLUMN W8"X 18#

1

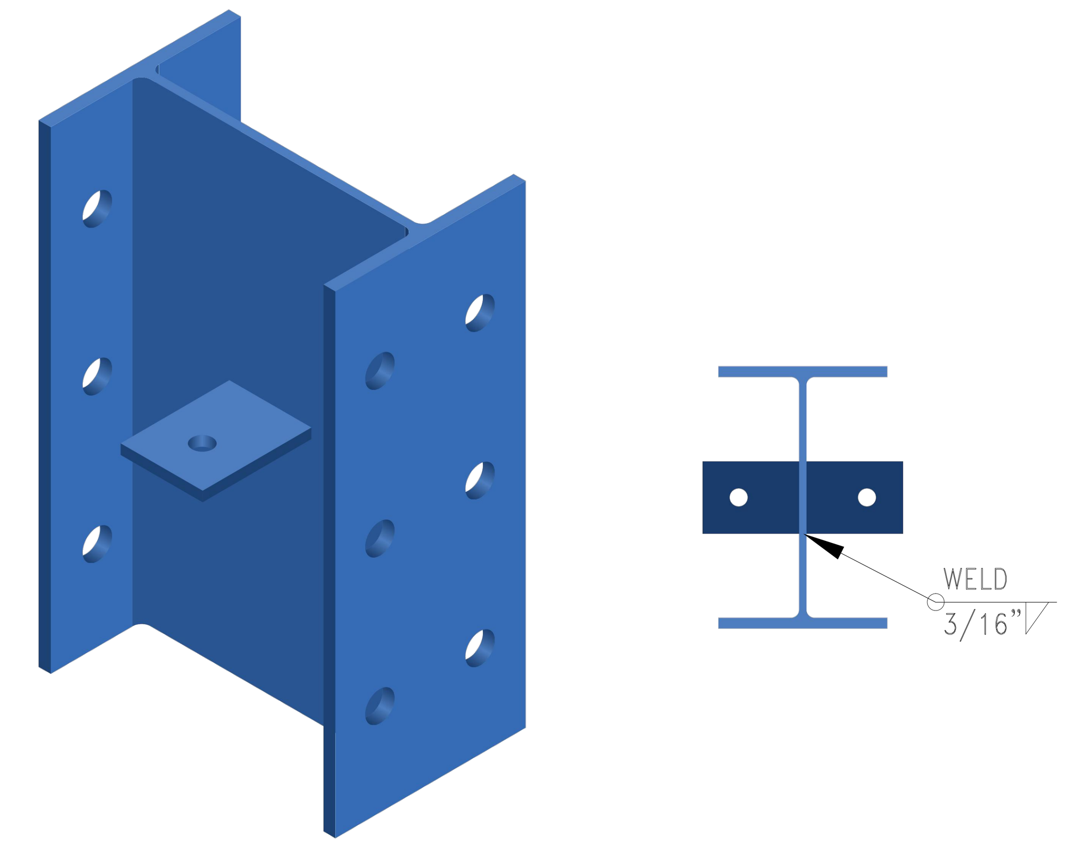


DESCRIPTION	BASE: STRUCTURAL
MATERIAL	W8" X 18#
STEEL YIELD	ASTM A572, Fy=36,000 PSI

DESCRIPTION	COLUMN: STRUCTURAL
MATERIAL	W8" X 18#
STEEL YIELD	ASTM A572, Fy=36,000 PSI

NOTES:
USE @ ALL SGL SIDED STRUCTURAL BASE TO COLUMN LOCATIONS.
ATTACH W/ 3/4"Ø ASTM A325 BOLTS COLUMN TO BASE.

2



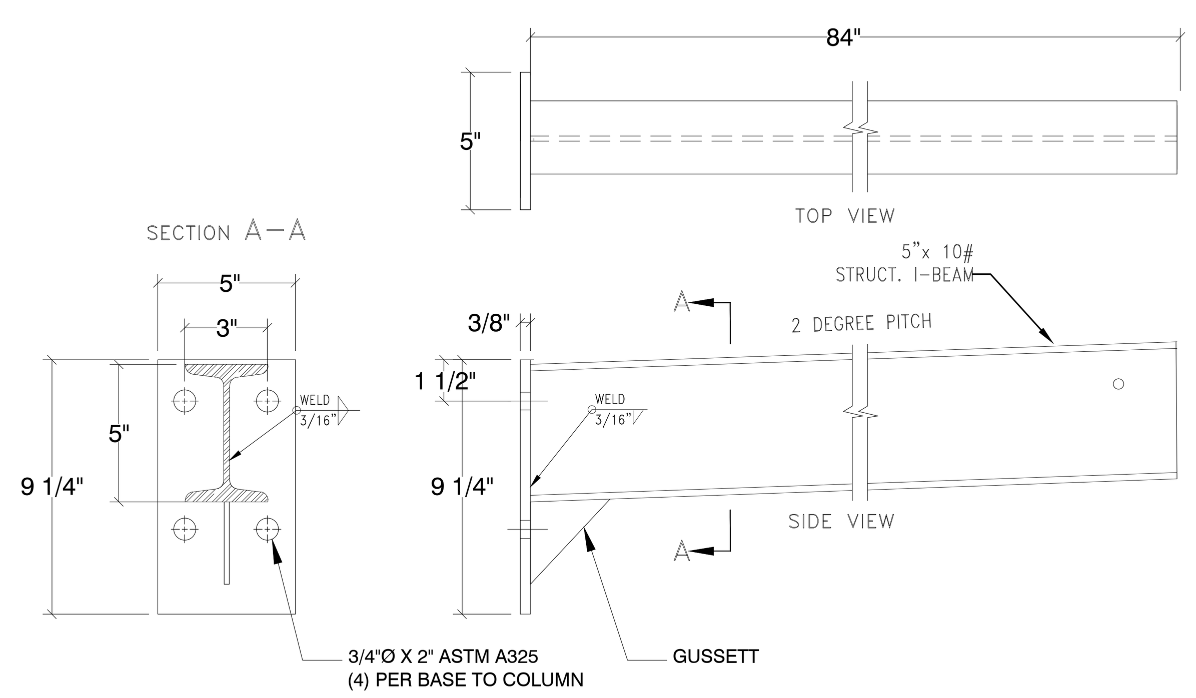
DESCRIPTION	BRACE CLIP
MATERIAL	3/16" THK
STEEL YIELD	ASTM A36, Fy=36,000 PSI

NOTES:
USE @ ALL BRACE TO COLUMN CONNECTION LOCATIONS TYP.

GENERAL PROJECT NOTES

1. DESIGNED PER REQUIREMENTS OF THE 2018 IBC, ASCE 7-16, SECTION 15.5.3, & 2012 RMI RACK DESIGN MANUAL
2. SEISMIC CRITERIA Ss = 0.099, S1 = 0.068, Fa = 1.600, Fv = 2.400, Ip = 1.0 (NO PUBLIC ACCESS), Sds = 0.106, Sd1 = 0.109, OCCUPANCY CATAG. II SITE CLASS D-DEFAULT, SEISMIC DESIGN CATAG. B
3. STORAGE CAPACITY: TYPE 2 SGL-SIDED CANTILEVER = 1000# UNIFORM LOAD PER ARM LEVEL TYP.
4. ANCHORS: HILTI KWIKBOLT TZ2 ESR#4266 (110 FT-LBS TORQUE), OR POWERS SD2 ESR#2502 (110 FT-LBS TORQUE) 3/4"Ø X 4-3/4" MIN. EMBED. (4) ANCHORS PER BASE PLATE. (2) @ TOE, (2) @ COLUMN.
5. PERIODIC SPECIAL INSPECTION IS REQUIRED DURING ANCHOR INSTALLATION. ANCHORS SHALL BE INSTALLED PER ICC ESR#4266.
6. EXISTING S.O.G. CONCRETE THICKNESS & COMPRESSIVE STRENGTH, 6" X 4000 PSI
7. SOIL BEARING PRESSURE 750 PSF.
8. ALL RACK INSTALLATIONS AND RACKS MANUFACTURED IN CONFORMITY WITH THIS STANDARD SHALL DISPLAY IN ONE OR MORE CONSPICUOUS LOCATIONS A PERMANENT PLAQUE EACH NOT LESS THAN 150 SQUARE INCHES IN AREA AND SHOWING THE MAXIMUM PERMISSIBLE UNIT LOAD IN CLEAR, LEGIBLE PRINT.
9. ALL BOLTS ASTM A325 OR BETTER, INSTALL TO SNUG TIGHT FIT OR BETTER
10. ALL WELDING PERFORMED IN THE SHOP OF AN APPROVED FABRICATOR BY AWS CERTIFIED WELDERS USING E70XX ELECTRODE OR BETTER. FIELD WELDS, IF ANY, SHALL BE PROVIDED UNDER THE SUPERVISION OF A LICENSED DEPUTY INSPECTOR. SPECIAL INSPECTION IS REQUIRED FOR ALL STRUCTURAL WELDS EXCEPT FOR WELDING DONE IN AN APPROVED FABRICATOR'S SHOP.
11. THE CLEAR SPACE BELOW SPRINKLERS SHALL BE A MIN. OF 18" BETWEEN TOP OF THE STORAGE AND THE CEILING SPRINKLER DEFLECTOR.
12. THE PRODUCT SHOWN ON THE DETAILS HEREIN IS ASSUMED TO BE IN GOOD, UNDAMAGED CONDITION. THE PRODUCT MUST BE FREE OF ANY DAMAGE AND/OR FABRICATION DEFICIENCIES OR IRREGULARITIES. IT IS THE RESPONSIBILITY OF THE OWNER, USER, PRODUCT PROVIDER AND/OR INSTALLER OF THE COMPONENTS TO NOTIFY SED, INC IN WRITING, OF ANY DAMAGE, DEFICIENCIES OR IRREGULARITIES.
13. IT IS THE RESPONSIBILITY OF THE OWNER AND/OR USER OF THE COMPONENTS SHOWN HEREIN TO NOTIFY SED, INC OF ANY DAMAGE OR DEFICIENCIES IN THE SYSTEM DURING USE.THIS INCLUDES SUCH OCCURRENCES SUCH AS IMPACT DAMAGE TO THE COMPONENTS FROM FORKLIFT OR HEAVY MACHINERY, IMPACT FROM DROPPED LOADS ON THE SYSTEM, DAMAGE TO SYSTEMS BY IMPROPER USE OR INSTALLATION, ETC. THE USER OF THE PRODUCT MUST NOTIFY SED, INC IN WRITING SHOULD ANY DAMAGE OR DEFICIENCY OCCUR TO THE PRODUCT SHOWN HEREIN. IT IS THE RESPONSIBILITY OF THE OWNER AND/OR USER TO MAINTAIN THE SAFETY AND PROPER USE OF THE STORAGE PRODUCT SHOWN HEREIN.
14. IT IS THE RESPONSIBILITY OF THE OWNER/USER OF THE STORAGE SYSTEM SHOWN HEREIN TO REPAIR OR REPLACE ANY COMPONENT THAT IS DAMAGED OR OTHERWISE DEFICIENT.
15. THE OWNER SHALL MAINTAIN THE STRUCTURAL INTEGRITY OF THE RACK SYSTEM BY ASSURING PROPER OPERATIONAL, HOUSE KEEPING AND MAINTENANCE PROCEDURES, BUT NOT LIMITED TO THE FOLLOWING:
   A. PROHIBIT ANY OVER LOADING OF ANY PALLET POSITIONS AND OF OVERALL RACK SYSTEMS.
   B. REGULARLY INSPECT FOR DAMAGE. IF DAMAGE IS FOUND, IMMEDIATELY UNLOAD THE AFFECTED AREA AND REPLACE OR REPAIR ANY DAMAGED COLUMNS, BEAMS, OR OTHER STRUCTURAL COMPONENTS.
   C. REQUIRE ALL PALLETS TO BE MAINTAINED IN GOOD, SAFE, OPERATING CONDITION.
   D. ENSURE THAT PALLETS ARE PROPERLY PLACED ONTO PALLET LOAD SUPPORT MEMBERS IN PROPERLY STACKED AND STABLE POSITION.
   E. REQUIRE THAT ALL GOODS STORED ON EACH PALLET TO BE PROPERLY STACKED AND STABLE.
   F. PROHIBIT DOUBLE STACKING OF ANY PALLET POSITION, INCLUDING THE TOP MOST POSITION
   G. THE OWNER AND/OR USER OF THE RACK SYSTEM MUST PROVIDE MEASURES TO MITIGATE DAMAGE TO THE STORAGE RACK BY USE OF IMPACT PROTECTIVE DEVICES IN AREAS WHERE FORKLIFT AND/OR HEAVY MACHINERY ARE IN USE.
   H. ENSURE THAT THE RACKS ARE NOT MODIFIED OR REARRANGED IN A MANNER NOT WITHIN THE ORIGINAL DESIGN CONFIGURATION.

3

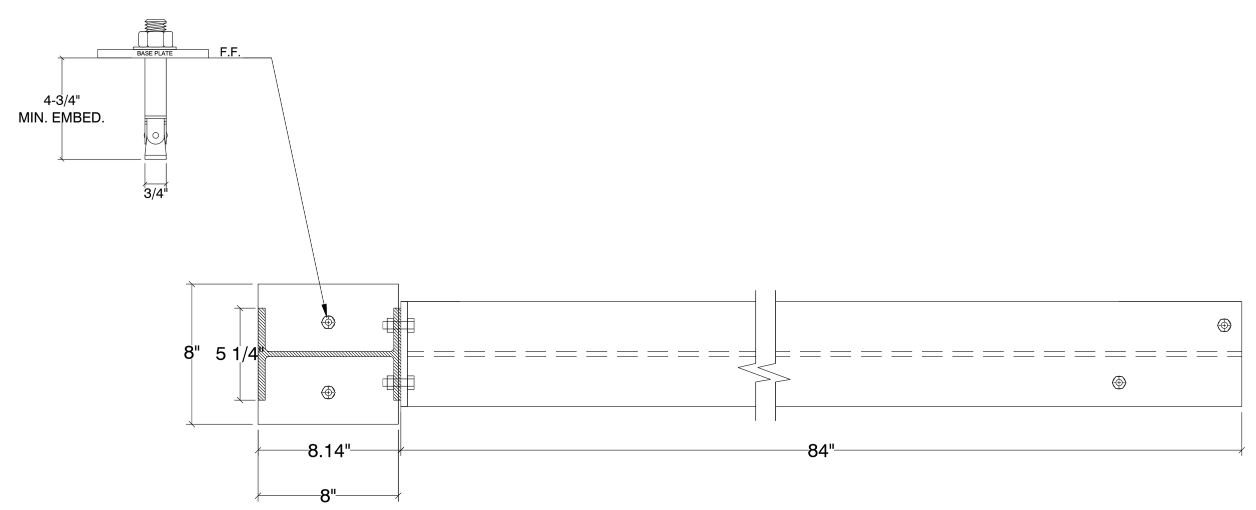


NOTES:
USE @ ALL ARM LOCATIONS TYP.
ALL LOADS TO BE EVENLY DISTRIBUTED OVER ALL ARMS. NO TIP LOADING ALLOWED.

DESCRIPTION	4" STRUCTURAL ARM
MATERIAL	S5" X 10#
STEEL YIELD	ASTM A572, Fy=50,000 PSI

NOTES:
ATTACH W/ (4) 3/4" DIA. ASTM A325 BOLTS.

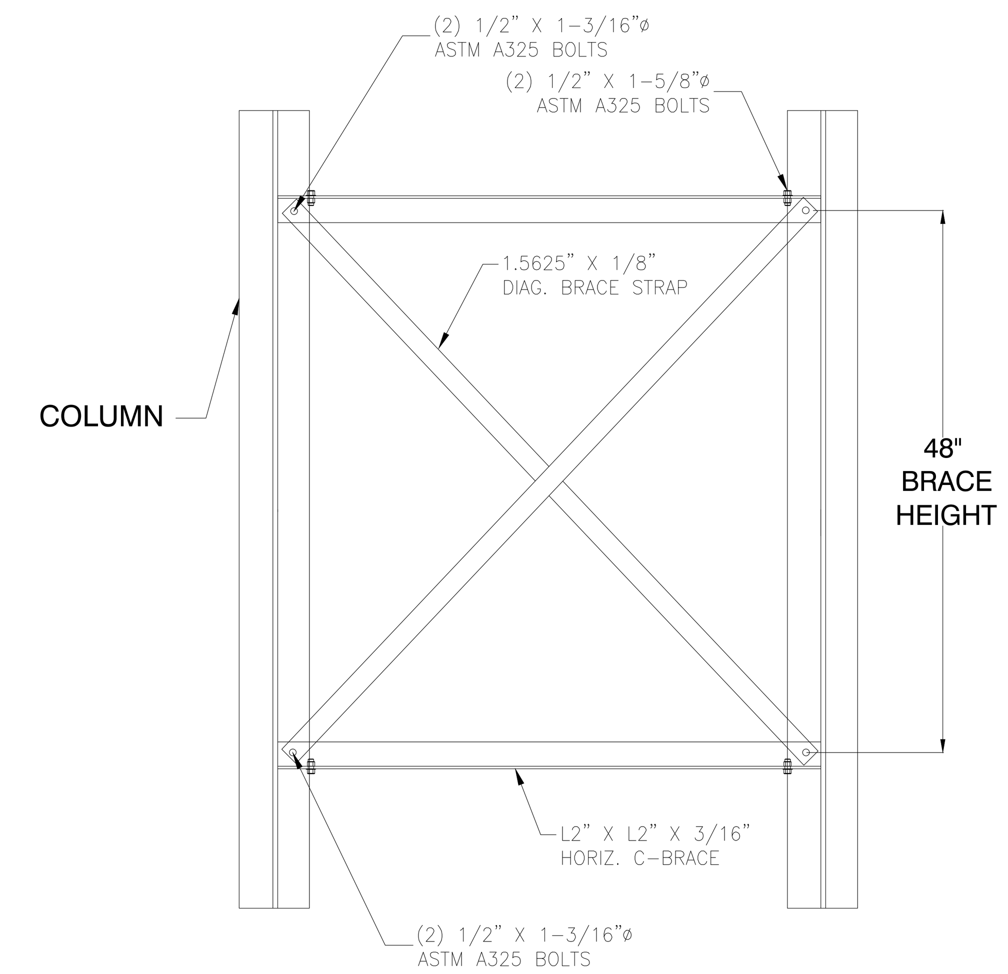
4



SEE NOTE # 4 ANCHOR DETAILS AND PLACEMENT

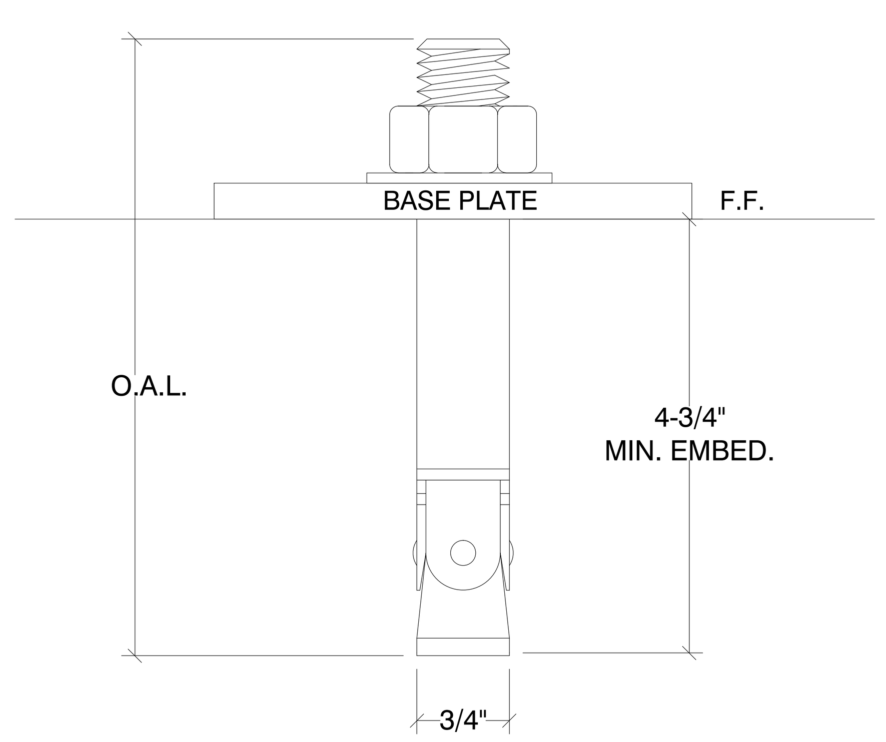
DESCRIPTION	ANCHOR PLAN

5



DESCRIPTION	BRACE TO COLUMN CONNECTION

6

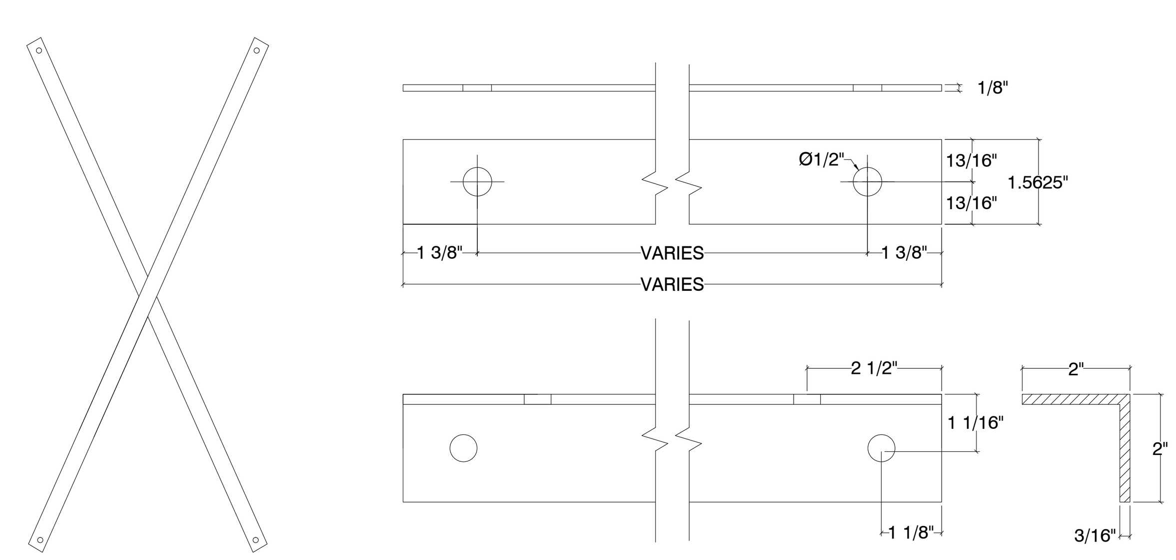


NOTES:
CONFIRM O.A.L. OF ANCHORS WITH INSTALLER TO ENSURE REQUIRED EMBEDMENT IS OBTAINED.

DESCRIPTION	HILTI KWIKBOLT TZ2 ANCHOR
SIZE	3/4"Ø x 4-3/4" MIN. EMBED.
ESR#	4266

NOTE:
SEE NOTE #4 ABOVE FOR ANCHOR SPECS.

7



NOTES:
USE @ ALL BRACE LOCATIONS TYP.

DESCRIPTION	HORIZ. C-BRACE
MATERIAL	L2" X L2" X 3/16"
STEEL YIELD	ASTM A36, Fy=36,000 PSI

DESCRIPTION	DIAG. BRACE
MATERIAL	1.5625" X 1/8" STRAP
STEEL YIELD	ASTM A36, Fy=36,000 PSI

SEE ATTACHED PALLET RACK KC DWG.
FOR CANTI. RACK ELEVATIONS

NOTE: CANTILEVER RACK ELEVATIONS

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

REVISION | REV. | DATE

STRUCTURAL ENGINEERING & DESIGN INC.
1815 WRIGHT AVE. LA VERNE, CA 91750 PHONE 909.596.1351 FAX 909.596.7186

BUTLER SUPPLY
2736 NE MCBAINE DR
LEE'S SUMMIT, MO 64064

CLIENT PALLET RACK KC
DATE 06/30/2025
DRAWN BY DAS ENG. BY BOB
MFG. MECALUX
TYPE HD CANTILEVER RACK

JOB NO. 25-0625-3

SHEET NO. SED 2 OF 2

Structural Engineering & Design, Inc.

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

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

Project Name : BUTLER SUPPLY

Project Number : 25-0625-3

Date : 07/01/25

Street Address: 2736 NE MCBAINE DR
City/State : LEES SUMMIT, MO 64064

Scope of Work : STORAGE RACK

Structural

Engineering & Design Inc.

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

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

TABLE OF CONTENTS

Structural
Engineering & Design Inc.

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

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

Design Data

1) The analyses herein conforms to the requirements of the:
2021 IBC Section 2209

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

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

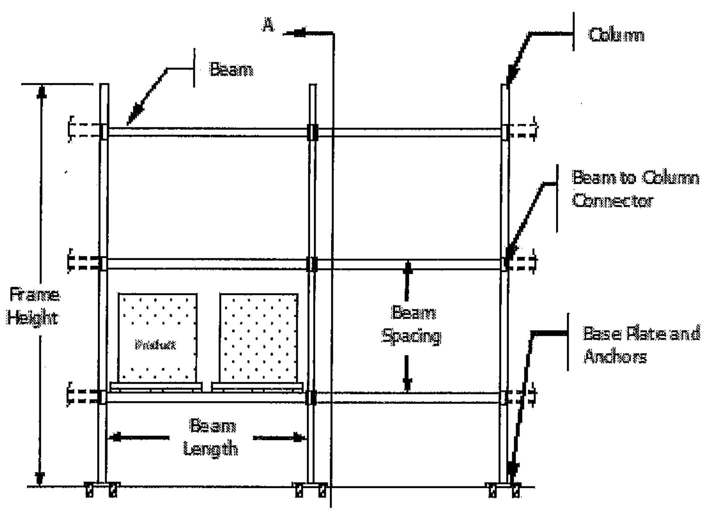
3) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein.

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

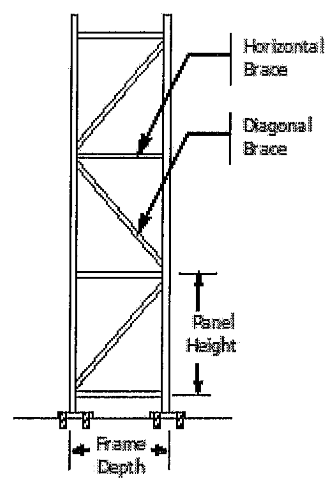
5) The existing slab on grade is 6" thick with minimum 4000 psi compressive strength. Allowable Soil bearing capacity is 750 psf. The design of the existing slab is by others.

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

Definition of Components



Front View: Down Aisle (Longitudinal) Frame



Section A: Cross Aisle (Transverse) Frame

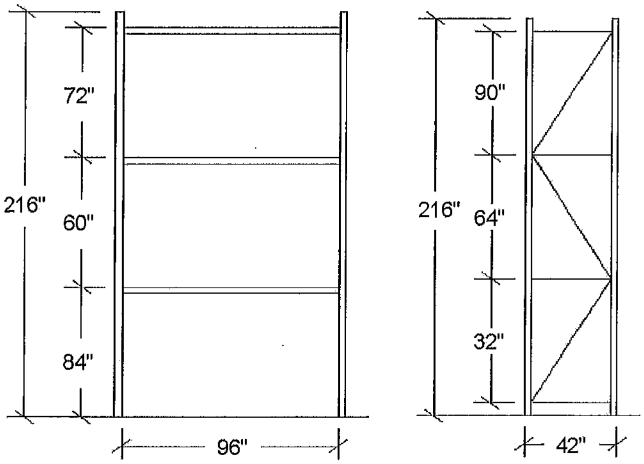
Structural

Engineering & Design Inc.

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

By: Bob S   Project: Butler Supply   Project #: 25-0625-3

Configuration & Summary: TYPE 1 SELECTIVE RACK



***RACK COLUMN REACTIONS*
UNFACTORED LOAD
AXIAL D= 75 lb
AXIAL P= 4,500 lb
AXIAL S= lb
SEISMIC AXIAL Ps=+/- 441 lb
BASE MOMENT= 0 in-lb

Seismic Criteria	# Bm Lvls	Frame Depth	Frame Height	# Diagonals	Beam Length	Frame Type
Ss=0.099, Fa=1.6	3	42 in	216.0 in	3	96 in	Single Row

Component	Description				STRESS
Column	Fy=55 ksi	Mecalux 314 3.0"x2.69"x0.070"	P=4575 lb, M=4415 in-lb		0.85-OK
Column & Backer	None	None	None		N/A
Beam	Fy=55 ksi	Intlk 40E 4Hx2.75Wx0.059"Thk	Lu=96 in	Capacity: 5164 lb/pr	0.58-OK
Beam Connector	Fy=55 ksl	Lvl 1: 3 Tab OK	Mconn=3521 in-lb	Mcap=8828 in-lb	0.4-OK
Brace-Horizontal	Fy=50 ksi	Mclx C456 Sgl 1.7953x1.378x16ga(U31x)			0.04-OK
Brace-Diagonal	Fy=50 ksi	Mclx C456 Sgl 1.7953x1.378x16ga(U31x)			0.39-OK
Base Plate	Fy=36 ksi	5.09x4.66x0.194	Fixity= 0 in-lb		0.52-OK
Anchor	1 per Base	0.5" x 3.25" Embed HILTI TZ2 ESR 4266 Inspection Reqd (Net Seismic Uplift=0 lb)			0.025-OK
Slab & Soil		6" thk x 4000 psi slab on grade. 750 psf Soil Bearing Pressure			0.27-OK

Level	Load** Per Level	Beam Spcg	Brace	Story Force Transv	Story Force Longit.	Column Axial	Column Moment	Conn. Moment	Beam Connector
1	3,000 lb	84.0 in	32.0 in	31 lb	21 lb	4,575 lb	4,415 "#	3,521 "#	3 Tab OK
2	3,000 lb	60.0 in	64.0 in	53 lb	35 lb	3,050 lb	1,326 "#	2,310 "#	3 Tab OK
3	3,000 lb	72.0 in	90.0 in	79 lb	53 lb	1,525 lb	954 "#	1,846 "#	3 Tab OK
			Total:	163 lb	109 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: Bob S   Project: Butler Supply   Project #: 25-0625-3

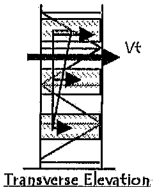
Seismic Forces   Configuration: TYPE 1 SELECTIVE RACK

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= 0.099
S1= 0.068
Fa= 1.600
Fv= 2.400
Sds=2/3*Ss*Fa= 0.106
Sd1=2/3*S1*Fv= 0.109

Transverse (Cross Aisle) Seismic Load

V= Cs*Ip*Ws=Cs*Ip*(0.67*P*Prf+D+0.2S)



Transverse Elevation

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

Base Shear Coeff=Cs= 0.0264

Cs-max * Ip= 0.0264
V_{min}= 0.015
Eff Base Shear=Cs= 0.0264
Ws= (0.67*PL_{RF1} * PL)+DL+0.2S
= 6,180 lb

Vtransv=Vt= 0.0264 * (150 lb + 6030 lb + 0.2
Etransverse= 163 lb
Limit States Level Transverse seismic shear per upright

(Transverse, Braced Frame Dir.) R= 4.0
Ip= 1.0
P_{RF1}= 1.0
Pallet Height=hp= 0.0 in
DL per Beam Lvl= 50 lb

Level	PRODUCT LOAD P	P*0.67*P_{RF1}	DL	hi	wi*hi	Fi	Fi*(hi+hp/2)
1	3,000 lb	2,010 lb	50 lb	84 in	173,040	30.8 lb	2,587-#
2	3,000 lb	2,010 lb	50 lb	144 in	296,640	52.9 lb	7,618-#
3	3,000 lb	2,010 lb	50 lb	216 in	444,960	79.3 lb	17,129-#
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
			0 lb		0		
sum:	P=9000 lb	6,030 lb	150 lb	W=6180 lb	914,640	163 lb	Σ=27,334

Longitudinal (Downaisle) Seismic Load

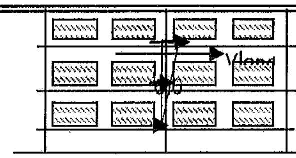
Similarly for longitudinal seismic loads, using R=6.0

Cs1= 0.0176
Cs2= 0.0046
Cs3= 0.0057
Cs-max= 0.0176

Ws= (0.67 * PL_{RF2} * P) + DL+0.2S
= 6,180 lb
Cs=Cs-max*Ip= 0.0176

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

Vlong= 0.0176 * (150 lb + 6030 lb + 0.2 * 0)
Elongitudinal= 109 lb   ***Limit States Level Longit. seismic shear per upright***


Front View

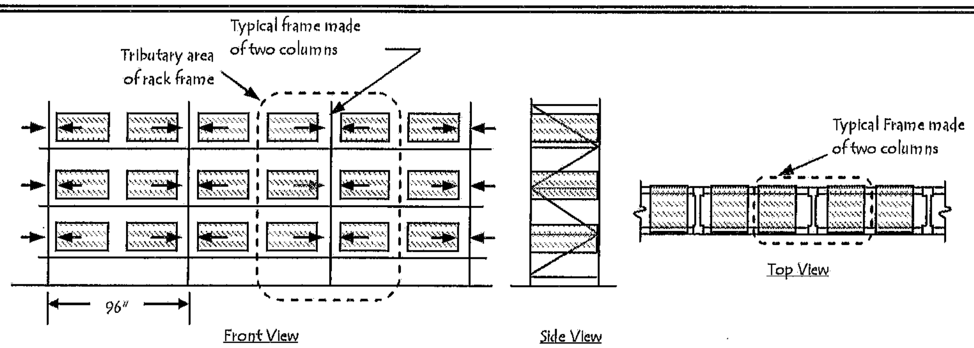
Level	PRODUC LOAD P	P*0.67*P_{RF2}	DL	hi	wi*hi	Fi
1	3,000 lb	2,010 lb	50 lb	84 in	173,040	20.6 lb
2	3,000 lb	2,010 lb	50 lb	144 in	296,640	35.4 lb
3	3,000 lb	2,010 lb	50 lb	216 in	444,960	53.0 lb
					0	
					0	
					0	
					0	
					0	
					0	
					0	
					0	
					0	
					0	
					0	
					0	
sum:		6,030 lb	150 lb	W=6180 lb	914,640	109 lb

Downaisle Seismic Loads Configuration: TYPE 1 SELECTIVE RACK

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

Seismic Story Forces

Vlong= 109 lb
Vcol=Vlong/2= 55 lb
F1= 21 lb
F2= 35 lb
F3= 53 lb



Conceptual System

Seismic Story Moments COl

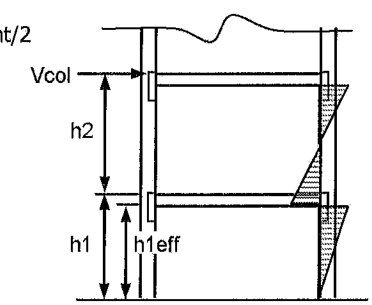
Mbase-max= 0 in-lb *<=== Default capacity*
Mbase-v= (Vcol*h1eff)/2
= 2,207 in-lb *<=== Moment going to base*
Mbase-eff= Minimum of Mbase-max and Mbase-v
= 0 in-lb *PINNED BASE ASSUMED*

h1-eff= h1 - beam clip height/2
= 81 in

M 1-1= [Vcol * h1eff]-Mbase-eff
= (55 lb * 81 in)-0 in-lb
= 4,415 in-lb

M 2-2= [Vcol-(F1)/2] * h2
= [55 lb - 17.7 lb]*60 in/2
= 1,326 in-lb

Mseis= (Mupper+Mlower)/2



Beam to Column Elevation

Mseis(1-1)= (4415 in-lb + 1326 in-lb)/2
= 2,870 in-lb

Mseis(2-2)= (1326 in-lb + 954 in-lb)/2
= 1,140 in-lb

rho= 1.0000

Summary of Forces

LEVEL	hi	Axial Load	Column Moment**	Mseismic**	Mend-fixity	Mconn**	Beam Connector
1	84 in	4,575 lb	4,415 in-lb	2,870 in-lb	2,160 in-lb	3,521 in-lb	3 Tab OK
2	60 in	3,050 lb	1,326 in-lb	1,140 in-lb	2,160 in-lb	2,310 in-lb	3 Tab OK
3	72 in	1,525 lb	954 in-lb	477 in-lb	2,160 in-lb	1,846 in-lb	3 Tab OK

Mconn= (Mseismic + Mend-fixity)*0.70*rho
Mconn-allow(3 Pin)= 8,828 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

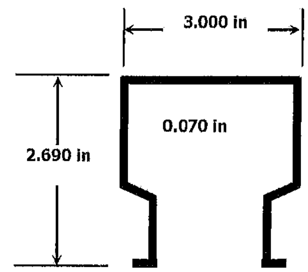
By: Bob S | Project: Butler Supply | Project #: 25-0625-3

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

Section Properties

Section: Mecalux 314 3.0"x2.69"x0.070"

Aeff = 0.538 in^2	Iy = 0.464 in^4	Kx = 1.7
Ix = 0.765 in^4	Sy = 0.307 in^3	Lx = 82.0 in
Sx = 0.510 in^3	ry = 0.928 in	Ky = 1.0
rx = 1.190 in	Fy= 55 ksi	Ly = 32.0 in
Ωf= 1.67	Cmx= 0.85	Cb= 1.0
E= 29,500 ksi		



Loads **Considers loads at level 1**

COLUMN DL= 75 lb
COLUMN PL= 4,500 lb
Mcol= 4,414 in-lb
Sds= 0.1056
1+0.105*Sds= 1.0111
1.4+0.14Sds= 1.4148
1+0.14Sds= 1.0148
0.85+0.14*Sds= 0.8648
B= 0.7000
rho= 1.0000

Critical load cases are: RMI Sec 2.1
*Load Case 5: : (1+0.105*Sds)D + 0.75*(1.4+0.14Sds)*B*P + 0.75*(0.7*rho*E)<= 1.0, ASD Method*
*axial load coeff: 0.7427616 * P* *seismic moment coeff: 0.5625 * Mcol*
*Load Case 6: : (1+0.104*Sds)D + (0.85+0.14Sds)*B*P + (0.7*rho*E)<= 1.0, ASD Method*
axial load coeff: 0.60535 *seismic moment coeff: 0.7 * Mcol*

By analysis, Load case 5 governs utilizing loads as such

Axial Load=Pax= 1.011088*75 lb + 0.75 * 1.414784 * 0.7 * 4500 lb
= **3,418 lb**

Moment=Mx= **0.75*0.7*rho*Mcol**
= **0.525*4414 in-lb**
= **2,317 in-lb**

Axial Analysis

KxLx/rx = 1.7*82"/1.19"
= 117.1

KyLy/ry = 1*32"/0.9284"
= 34.5

Fe < Fy/2
Fn= Fe
= π^2E/(KL/r)max^2
= 21.2 ksi

Fe= π^2E/(KL/r)max^2
= 21.2ksi

Fy/2= 27.5 ksi

Pa= Pn/Ωc
= 11413 lb/1.92
= 5,944 lb

Pn= Aeff*Fn
= 11,413 lb

Ωc= 1.92

P/Pa= 0.58 > 0.15

Bending Analysis

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

Pno= Ae*Fy
= 0.538 in^2 *55000 psi
= 29,585 lb

Pao= Pno/Ωc
= 29585lb/1.92
= 15,409 lb

Myield=My= Sx*Fy
= 0.51 in^3 * 55000 psi
= 28,050 in-lb

Max= My/Ωf
= 28050 in-lb/1.67
= 16,796 in-lb

Pcr= π^2EI/(KL)max^2
= π^2*29500 ksi/(1.7*82 in)^2
= 11,462 lb

μx= {1/[1-(Ωc*P/Pcr)]}^-1
= {1/[1-(1.92*3418 lb/11462 lb)]}^-1
= 0.43

Combined Stresses

(3418 lb/5944 lb) + (0.85*2317 in-lb)/(16796 in-lb*0.43) =	0.85	< 1.0, OK	(EQ C5-1)
(3418 lb/15409 lb) + (2317 in-lb/16796 in-lb) =	0.36	< 1.0, OK	(EQ C5-2)

*** For comparison, total column stress computed for load case 6 is: 77.0%* *izing loads 2800.1784 lb Axial and M= 3089 in-lb*

Structural

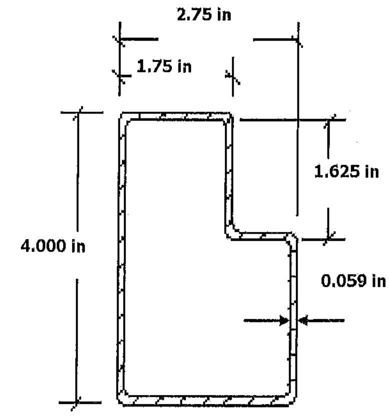
Engineering & Design Inc.

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

By: **Bob S** Project: **Butler Supply** Project #: **25-0625-3**

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

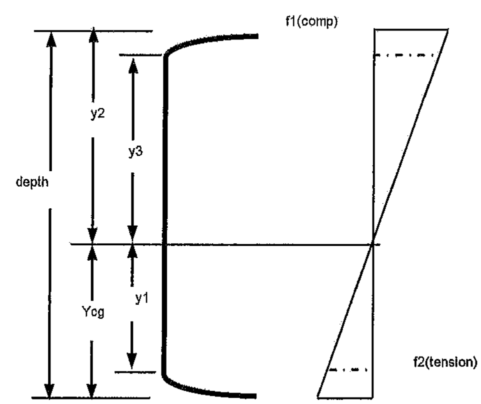
f1(comp)= Fy*(y3/y2)= 50.23 ksi
f2(tension)= Fy*(y1/y2)= 101.99 ksi
Y= f2/f1 Eq. B2.3-5
= -2.03
k= 4 + 2*(1-Y)^3 + 2*(1-Y) Eq. B2.3-4
= 65.70
flat depth=w= y1+y3
= 3.764 in w/t= 63.79661017 OK
l=lambda= [1.052/(k)^0.5] * (w/t) * (f1/E)^0.5
= [1.052/(65.7)^0.5] * 3.764 * (50.23/29500)^0.5
= 0.342 < 0.673
be=w= 3.764 in b2= be/2 Eq B2.3-2
b1= be(3-Y) = 1.88 in
= 0.748
b1+b2= 2.628 in > 1.242 in, Web is fully effective

Beam= **Intlk 40E 4Hx2.75Wx0.059"Thk**

Ix=	1.667 in^4
Sx=	0.783 in^3
Ycg=	2.640 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=	4.000 in

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

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



y1= Ycg-t-r= 2.522 in
y2= depth-Ycg= 1.360 in
y3= y2-t-r= 1.242 in

Check allowable tension stress for bottom flange

Lflange-bot=Lfb= Lbottom - 2*r*-2*t
= 2.514 in
Cbottom=Cb= 2*Lc/(Lfb+2*Lc)
= 0.100
Fy-bottom=Fyb= Cb*Fyc + (1-Cb)*Fyf
= 57.34 ksi
Fya= (Fya-top)*(Fyb/Fya-bottom)
= 29.54 ksi
if F= 0.95 Then F*Mn=F*Fya*Sx= $\boxed{\text{21.96 in-k}}$

Structural

Engineering & Design Inc.

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

By: **Bob S** Project: **Butler Supply** Project #: **25-0625-3**

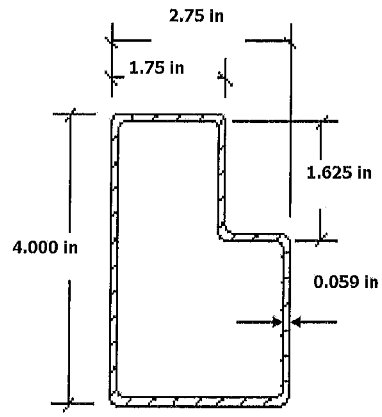
BEAM Configuration: TYPE 1 SELECTIVE RACK

RMI Section 5.2, PT II

Section

Beam= Intlk 40E 4Hx2.75Wx0.059"Thk
Ix=Ib= 1.667 in^4
Sx= 0.783 in^3
t= 0.059 in
Fy=Fyv= 55 ksi
Fu=Fuv= 65 ksi
Fya= 58.6 ksi

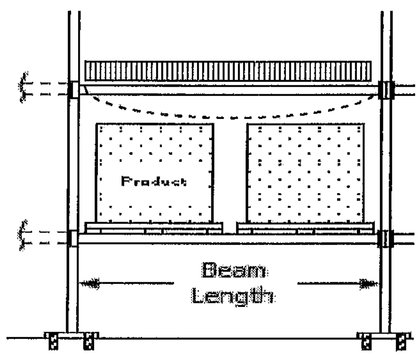
E= 29500 ksi
F= 225.0
L= 96 in
Beam Level= 1
P=Product Load= 3,000 lb/pair
D=Dead Load= 50 lb/pair



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*225*96 in)/[(6*29500 ksi*1.667 in^3)+(3*225*96 in)]
= 0.88
if F= 0.95
Then F*Mn=F*Fya*Sx= 43.59 in-k
Thus, allowable load
per beam pair=W= F*Mn*8*(# of beams)/(L*Rm*W)
= 43.59 in-k * 8 * 2/(96in * 0.88 * 1.599)
= 5,164 lb/pair ***allowable load based on bending stress***

Mend= W*L*(1-Rm)/8
= (5164 lb/2) * 96 in * (1-0.88)/8
= 3,718 in-lb @ 5164 lb max allowable load
= 2,160 in-lb **@ 3000 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*225*96 in)/[(5*225*96 in)+(10*29500 ksi*1.667 in^4)]
= 0.856 in
if Dmax= L/180 *Based on L/180 Deflection Criteria*
and Dss= 5*W*L^3/(384*E*Ib)

Allowable Deflection= L/180
= 0.533 in
Deflection at imposed Load= 0.310 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*1.667 in^4*2/[180*5*(96 in)^2*0.856)
= 5,319 lb/pair ***allowable load based on deflection limits***

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

Allowable load= 5,164 lb/pair
Imposed Product Load= 3,000 lb/pair
Beam Stress= 0.58 *Beam at Level 1*

Structural Engineering & Design Inc.

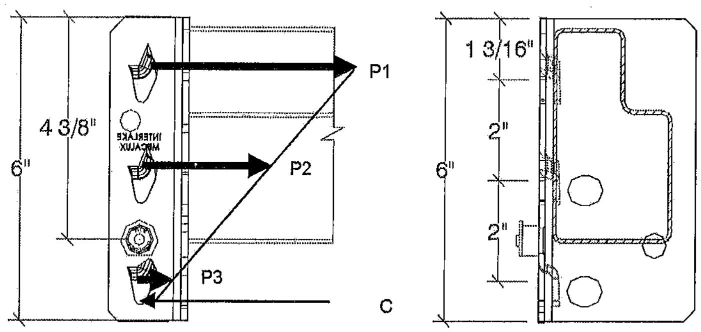
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: **Bob S** Project: **Butler Supply** Project #: **25-0625-3**

3 Tab Beam to Column Connection **Configuration: TYPE 1 SELECTIVE RACK**

Mconn max= (Mseismic + Mend-fixity)*0.70*Rho
= 3,521 in-lb **Load at level 1**

Connector Type= 3 Tab



Shear Capacity of Tab

Tab Length= 0.50 in Fy= 55,000 psi

Ashear= 0.5 in * 0.135 in
= 0.0675 in^2

Pshear= 0.4 * Fy * Ashear
= 0.4 * 55000 psi * 0.0675in^2
= 1,485 lb

Bearing Capacity of Tab

tcol= 0.070 in Fu= 65,000 psi Bearing Length= 0.5000 in

Omega= 2.22 a= 2.22

Pbearing= alpha * Fu * tab length * tcol/Omega
= 2.22 * 65000 psi * 0.5 in * 0.07 in/2.22
= 2,275 lb > 1485 lb

Moment Capacity of Bracket

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

C= P1+P2+P3
= P1+P1*(2.5"/4.5")+P1*(0.5"/4.5")
= 1.667 * P1

tclip= 0.135 in Sclip= 0.183 in^3

Mcap= Sclip * Fbending
= 0.1832 in^3 * 0.66 * Fy
= 6,650 in-lb

C*d= Mcap = 1.667

d= E /2
= 0.50 in

Pclip= Mcap/(1.667 * d)
= 6650.16 in-lb/(1.667 * 0.5 in)
= 7,979 lb

Thus, P1= 1,485 lb

Mconn-allow= [P1*4.5"+P1*(2.5"/4.5")*2.5"+P1*(0.5"/4.5")*0.5"]
= 1485 LB*[4.5"+(2.5"/4.5")*2.5"+ (0.5"/4.5")*0.5"]
= 8,828 in-lb > Mconn max, OK

Stress= 0.4

Structural

Engineering & Design Inc.

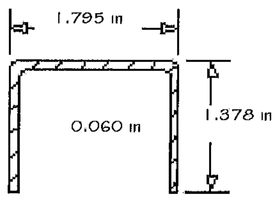
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

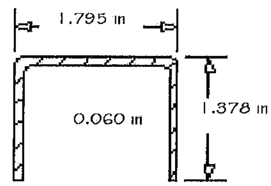
Transverse Brace Configuration: TYPE 1 SELECTIVE RACK

Section Properties

Diagonal Member= Mclx C456 Sgl 1.7953x1.378x16ga(U31x)
Area= 0.259 in^2
r min= 0.449 in
Fy= 50,000 psi
K= 1.0
Ωc= 1.92



Horizontal Member= Mclx C456 Sgl 1.7953x1.378x16ga(U31x)
Area= 0.259 in^2
r min= 0.449 in
Fy= 50,000 psi
K= 1.0



Frame Dimensions

Bottom Panel Height=H= 90.0 in
Frame Depth=D= 42.0 in
Column Width=B= 2.7 in

Clear Depth=D-B*2= 36.6 in
X Brace= NO
rho= 1.30

Diagonal Member

0

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

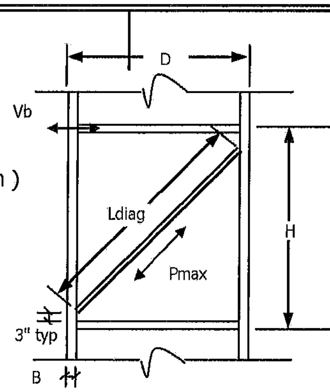
Vtransverse= 163 lb
Vb=Vtransv*0.7*rho= 163 lb * 0.7 * 1.3
= 148 lb
Ldiag= [(D-B*2)^2 + (H-6")^2]^1/2
= 91.6 in

Pmax= V*(Ldiag/D)
= 371 lb
axial load on diagonal brace member

Pn= AREA*Fn
= 0.259 in^2 * 6996 psi
= 1,810 lb

(kl/r)= (k * Ldiag)/r min
= (1 x 91.6 in /0.449 in)
= 204.0 in
Fe= pi^2*E/(kl/r)^2
= 6,996 psi

Since Fe<Fy/2,
Fn= Fe
= 6,996 psi



Typical Panel Configuration

Pallow= Pn/Ω
= 1810 lb /1.92
= 943 lb

Pn/Pallow= 0.39 <= 1.0 OK

Horizontal brace

Vb=Vtransv*0.7*rho= 148 lb

(kl/r)= (k * Lhoriz)/r min
= (1 x 42 in) /0.449 in
= 93.5 in
Since Fe>Fy/2, Fn=Fy*(1-fy/4fe)
= 31,233 psi

Fe= pi^2*E/(kl/r)^2
= 33,304 psi

Pn= AREA*Fn
= 0.259in^2*31233 psi
= 8,080 lb

Fy/2= 25,000 psi

Pallow= Pn/Ωc
= 8080 lb /1.92
= 4,208 lb

Pn/Pallow= 0.04 <= 1.0 OK

Structural Engineering & Design Inc.

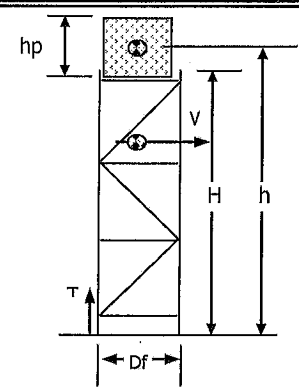
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

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



SIDE ELEVATION

		Sds=	0.1056
Vtrans=V=E=Qe=	163 lb	(0.9-0.2Sds)=	0.8789
DEAD LOAD PER UPRIGHT=D=	150 lb	(0.9-0.2Sds)=	0.8789
PRODUCT LOAD PER UPRIGHT=P=	9,000 lb	B=	1.0000
Papp=P*0.67=	6,030 lb	rho=	1.0000
Wst LC1=Wst1=(0.87888*D + 0.87888*Papp*1)=	5,431 lb		
		Frame Depth=Df=	42.0 in
Product Load Top Level, Ptop=	3,000 lb	Htop-lvl=H=	216.0 in
DL/Lvl=	50 lb	# Levels=	3
Seismic Ovt based on E, Σ(Fi*hi)=	18,546 in-lb	# Anchors/Base=	1
height/depth ratio=	5.1 in	hp=	.0 in
		h=H+hp/2=	216.0 in

A) Fully Loaded Rack

Load case 1:

Movt= Σ(Fi*hi)*E*rho
= 18,546 in-lb

Mst= Wst1 * Df/2
= 5431 lb * 42 in/2
= 114,051 in-lb

T= (Movt-Mst)/Df
= (18546 in-lb - 114051 in-lb)/42 in
= -2,274 lb **No Uplift**

Net Seismic Uplift= -4,548 lb ***Strength Level***

B) Top Level Loaded Only

Load case 1:

V1=Vtop= Cs * Ip * Ptop >= 350 lb for H/D >6.0
= 0.0264 * 3000 lb
= 79 lb

V1eff= 79 lb

Critical Level= 3

Cs*Ip= 0.0264

$V2=V_{DL}$= Cs*Ip*D
= 4 lb

Mst= (0.87888*D + 0.87888*Ptop*1) * 42 in/2
= 58,138 in-lb

Movt= [V1*h + V2 * H/2]*rho
= 17,535 in-lb

T= (Movt-Mst)/Df
= (17535 in-lb - 58138 in-lb)/42 in
= -967 lb **No Uplift**

Net Seismic Uplift= -1,934 lb ***Strength Level***

Anchor

Check (1) 0.5" x 3.25" Embed HILTI TZ2 anchor(s) per base plate.
Special inspection is required per ESR 4266.

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:	(81 lb/2517 lb)^1 =	0.03	<= 1.2 OK
Top Level Loaded:	(-1934 lb/1961 lb)^1 + (39 lb/2517 lb)^1 =	0.02	<= 1.2 OK

Structural

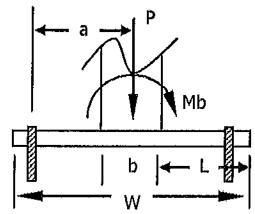
Engineering & **D**esign **I**nc.

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

By: Bob S Project: Butler Supply Project #: 25-0625-3

Base Plate Configuration: TYPE 1 SELECTIVE RACK

Section



Downaisle Elevation

Baseplate= 5.09x4.66x0.194
Eff Width=W = 5.09 in a = 1.55 in
Eff Depth=D = 4.66 in Anchor c.c. =2*a=d = 3.09 in
Column Width=b = 3.00 in N=# Anchor/Base= 1
Column Depth=dc = 2.69 in Fy = 36,000 psi
L = 1.05 in rho= 1
Plate Thickness=t = 0.194 in

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

COLUMN DL= 75 lb **Axial=P= 1.011088 * 75 lb + 0.75 * (1.414784 * 0.7 * 4500 lb)**
COLUMN PL= 4,500 lb **= 3,418 lb**
Base Moment= 0 in-lb **Mb= Base Moment*0.75*0.7*rho**
1+0.105*Sds= 1.0111 **= 0 in-lb * 0.75*0.7*rho**
1.4+0.14Sds= 1.4148 **= 0 in-lb**
B= 0.7000 **Axial Load P = 3,418 lb** **Mbase=Mb = 0 in-lb**

Axial stress=fa = P/A = P/(D*W)
= 144 psi
M1= wL^2/2= fa*L^2/2
= 79 in-lb
Moment Stress=fb = M/S = 6*Mb/[(D*B^2]
= 0.0 psi
Moment Stress=fb2 = 2 * fb * L/W
= 0.0 psi
Moment Stress=fb1 = fb-fb2
= 0.0 psi
M2= fb1*L^2)/2
= 0 in-lb
M3 = (1/2)*fb2*L*(2/3)*L = (1/3)*fb2*L^2
= 0 in-lb
Mtotal = M1+M2+M3
= 79 in-lb/in
S-plate = (1)(t^2)/6
= 0.006 in^3/in
Fb = 0.75*Fy
= 27,000 psi
fb/Fb = Mtotal/[(S-plate)(Fb)]
= **0.46** **OK**
F'p= 0.7*F'c
= 2,800 psi OK
Tanchor = (Mb-(PLapp*0.75*0.46)(a))/[(d)*N/2]
= -5,431 lb No Tension
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= 3,418 lb

Movt*0.75*0.7*rho= 18,655 in-lb
Frame Depth= 42.0 in
Pseismic= Movt/Frame Depth
= 444 lb

P=Pstatic+Pseismic= 3,862 lb

b =Column Depth= 2.69 in

L =Base Plate Depth-Col Depth= 1.05 in

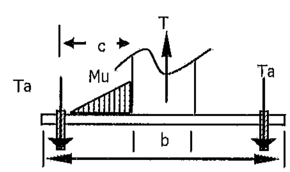
fa = P/A = P/(D*W)
= 163 psi
M= wL^2/2= fa*L^2/2
= 89 in-lb/in

Sbase/in = (1)(t^2)/6
= 0.006 in^3/in
Fbase = 0.75*Fy
= 27,000 psi

fb/Fb = M/[(S-plate)(Fb)]
= **0.52** **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= 0 lb
Qty Anchor per BP= 1
Net Tension per anchor=Ta= 0 lb
c= 1.05 in
Mu=Moment on Baseplate due to uplift= Ta*c/2
= in-lb
Splate= 0.029 in^3
[fb/Fb]*0.75= 0 **OK**

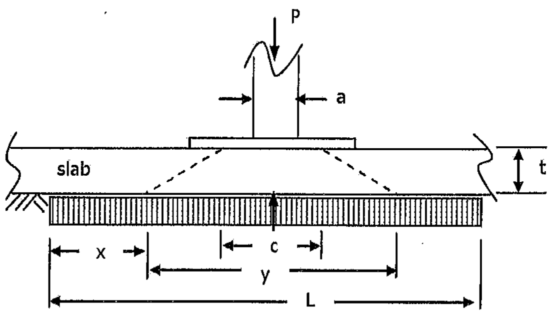
Structural

Engineering & Design Inc.

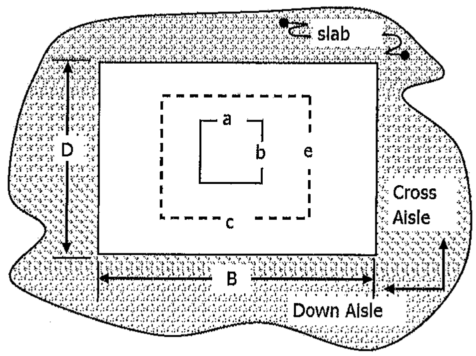
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

Slab on Grade Configuration: TYPE 1 SELECTIVE RACK



SLAB ELEVATION



Baseplate Plan View

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

Soil
fsoil= 750 psf
Movt= 18,546 in-lb
Frame depth= 42.0 in
Sds= 0.106
0.2*Sds= 0.021
λ= 0.600
β=B/D= 1.092
F'c^0.5= 63.2

Base Plate
Effec. Baseplate width=B= 5.09 in
Effec. Baseplate Depth=D= 4.66 in

width=a= 3.00 in
depth=b= 2.69 in
midway dist face of column to edge of plate=c= 4.05 in
midway dist face of column to edge of plate=e= 3.68 in

Column Loads

DEAD LOAD=D= 75 lb per column
unfactored ASD load
PRODUCT LOAD=P= 4,500 lb per column
unfactored ASD load
Papp= 3,015 lb per column
P-seismic=E= (Movt/Frame depth)
= 441 lb per column
unfactored Limit State load
B= 0.7000
rho= 1.3000
Sds= 0.1056
1.2 + 0.2*Sds= 1.2211
0. 9 - 0.20Sds= 0.8789

Load Case 1) (1.2+0.2Sds)D + (1.2+0.2Sds)*B*P+ rho*E — RMI SEC 2.2 EQTN 5
= 1.22112 * 75 lb + 1.22112 * 0.7 * 4500 lb + 1.3 * 441 lb
= 4,511 lb
Load Case 2) (0.9-0.2Sds)D + (0.9-0.2Sds)*B*Papp + rho*E — RMI SEC 2.2 EQTN 7
= 0.87888 * 75 lb + 0.87888 * 0.7 * 3015 lb + 1.3 * 441 lb
= 2,494 lb
Load Case 3) 1.2*D + 1.4*P — RMI SEC 2.2 EQTN 1,2
= 1.2*75 lb + 1.4*4500 lb
= 6,390 lb
Load Case 4) 1.2*D + 1.0*P + 1.0E — ACI 318-14 Sec 5.3.1
= 5,031 lb — Eqtn 5.3.1e

Effective Column Load=Pu= 6,390 lb per column

Puncture

Apunct= [(c+t)+(e+t)]*2*t
= 236.64 in^2
Fpunct1= [(4/3 + 8/(3*β)] * λ *(F'c^0.5)
= 143.1 psi
Fpunct2= 2.66 * λ * (F'c^0.5)
= 100.9 psi
Fpunct eff= 100.9 psi

fv/Fv= Pu/(Apunct*Fpunct)
= **0.268** **< 1 OK**

Slab Bending

Pse=DL+PL+E= 6,390 lb

Asoil= (Pse*144)/(fsoil)
= 1,227 in^2
x= (L-y)/2
= 9.6 in
Fb= 5*(phi)*(f'c)^0.5
= 189.74 psi

L= (Asoil)^0.5
= 35.03 in
M= w*x^2/2
= (fsoil*x^2)/(144*2)
= 239.3 in-lb

y= (c*e)^0.5 + 2*t
= 15.9 in
S-slab= 1*teff^2/6
= 6.0 in^3
fb/Fb= M/(S-slab*Fb)
= **0.210** **< 1, OK**

Structural
Engineering & Design Inc.

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

Cantilever Rack
Analysis

Structural
Engineering & Design Inc.

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

By: Bob S Project: Butler Supply Project #: 25-0625-3

Design Data **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

1) The analysis presented herein conforms to the requirements of the 2021 IBC and ASCE 7-16

2) W & S Shape steel conforms to ASTM A572, Gr. 50, with minimum yield, Fy= 50 ksi
   Formed steel conforms to ASTM A570, Gr. 50, with minimum yield, Fy= 50 ksi
   Other steel conforms to ASTM A36, Gr. 36, with minimum yield, Fy= 36 ksi

3) Bolts shall conform to ASTM A325-N unless noted otherwise on the plans or calculations.

4) Anchor bolts shall be provided by installer per ICC reference on plans and calculations herein. Installer must provide any special inspection as called out on plans or calculations, or as required by the ICC report indicated herein.

5) 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.

6) 6 in Thk x 4000 psi slab with 750 psf soil bearing pressure

Seismic Design Coefficient

Ss= 0.0990
S1= 0.0680
Fa= 1.6000
Fv= 2.4000
Sds= 0.1057
Sd1= 0.1089

Structural

Engineering & Design Inc.

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

By: Bob S Project: Butler Supply Project #: 25-0625-3

Summary of Results **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

144 IN H
48 IN ARM AND BASE LENGTH
48 IN BRACE HEIGHT
48 IN COLUMN SPACING

Rack Dimensions & Loads

Arm	Elev	Unif. Arm Load
h1	36 in	2,400 lb
h2	72 in	2,400 lb
h3	108 in	2,400 lb

Seismic Coeff Ss= 0.099
Seismic Coeff Fa= 1.600

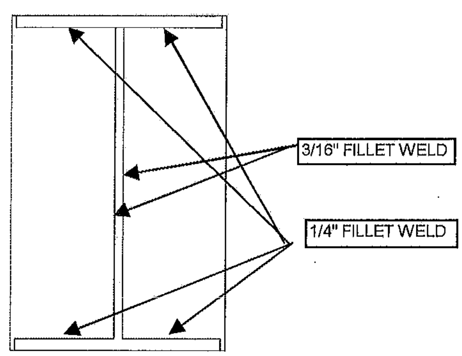
Summary of Components & Results

Column	Fy=50 ksi	W8x18	0.46	**OK**
Base	Fy=50 ksi	W8x18	0.38	**OK**
Arm	Fy=50 ksi	S5x10	0.40	**OK**
Arm Connection		(4) 3/4" Diam Bolts, Grade 5 or better	0.44	**OK**
Brace	Fy=36 ksi	X Brace: 1.575x0.984x105 Strut L2x2x3/16	0.02	**OK**
X Brace Connection	Bolted	(1) 0.5 in diam Bolt per end	0.03	**OK**
Anchors	ESR 4266	0.75 in diam x 4.75 in embed HILTI TZ2 with inspection 4 per base	0.09	**OK**
Slab & Soil		6 in thick x 4000 psi with 750 psf soil at grade	0.22	**OK**

Notes

ALL LOADS SHOWN ARE PER ARM, UNIFORM LOAD, NO TIP LOADS, NO DROPPED LOADS, NO IMPACT LOADS

OMEGA = 2.0



Structural

Engineering & Design Inc.

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

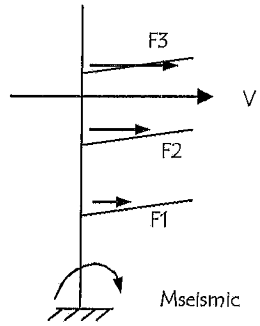
By: Bob S　　Project: Butler Supply　　Project #: 25-0625-3

Seismic Loads　　**Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

Seismic Eqtn:	Transverse	Longitudinal
Cs1=	Sds*I/R	
=	0.042	0.033
Cs2=	0.044*Sds*I	
=	0.005	0.005
Cs3=	0.5*S1*I/R	
=	0.014	0.010

Rtrans= 2.5
Rlong= 3.25
# Sides=n= 1
# Arms= 3

Ss= 0.099
S1= 0.068
Fa= 1.600
Fv= 2.400
Sds= 0.106
Sd1= 0.109
I= 1.0
phi= 0.67
** DL per arm= 127.0 lb



Side Elevation

Vtrans= 0.0423
Vlong= 0.0325
Σ Arm LL= 7,200 lb
Σ Arm LL*phi= 4,824 lb
Σ Arm DL= 381 lb

$W1=W_{arm,gravity}$= 7,581 lb
$W2=W_{arm,seismic}$= 5,205 lb

*** weight of arm plus trib weight of column per level*

V-transv= 0.0423 * 5205 lb
= 220 lb/frame
tranv shear per upright

V-long= 0.0325 * 5205 lb
= 169 lb/frame
longit shear per upright

Lateral Force Distribution

			Transverse Force				**Longitudinal Force**
Level	**W1 (DL+LL)*n**	**W2 (DL+LL*phi)**	**hx**	**wxhx**	**Fi**	**Fi * hi**	**Fi**
1	2,527 lb	1,735 lb	36 in	62460	37 lb	1,332 in-lb	28 lb
2	2,527 lb	1,735 lb	72 in	124920	73 lb	5,279 in-lb	56 lb
3	2,527 lb	1,735 lb	108 in	187380	110 lb	11,880 in-lb	85 lb
			Σ=	374,760	220 lb	18,491 in-lb	

Summary of Frame Loads

Mseismic= 18,491 in-lb
Mwind= 24,019 in-lb
Pcol= 7,581 lb　　DL+LL

Marm-max= (LL-max+DL) * (Arm Length/2)
= (2400 lb + 31 lb) * (48 in/2)
= 58,344 in-lb
Marm-total= Σ Marm
= 175,032 in-lb

Structural

Engineering & Design Inc.

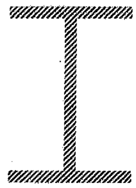
1815 Wright Ave La Verne, CA 91750 Tel: 909.596.1351 Fax: 909.596.7186

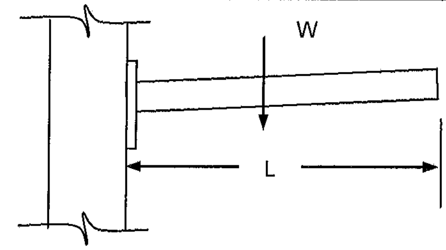
By: Bob S Project: Butler Supply Project #: 25-0625-3

Arm **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

Analysis is based on a uniformly loaded arm with no tip or impact loads considered.

Arm Type= S5x10
Ix= 12.3000 in^4
Sx= 4.920 in^3
Fy= 50,000 psi
L= 48 in





Check Arm Bending

LL-max= 2,400 lb

DL= 31 lb

M= W * L/2
= 58,344 in-lb

fb= M/Sy
= 11,859 psi

Fb= 0.6 * Fy
30,000 psi

fb/Fb= 0.40 OK

Check Arm Deflection

E= 29,000,000 psi

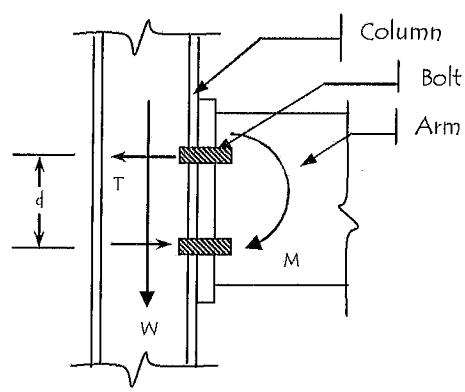
D= w * L^4/(8 * E * Ix)
= 0.0973 in

Dallow= L/180
= **0.27 in OK**

Check Bolts For Imposed Arm Loads

Spacing=d= 4.0 in
Bolt diameter= 0.75 in
Ft= 44,000 psi
Fv= 21,000 psi
tmin= 0.188 in
Fu= 58,000 psi
# Bolts in shear=Ns= 4
# Bolts in tension=Nt= 2

M= 58,344 in-lb
T= M/d
= 14,586 lb
W= LL+DL
= 2,431 lb
W/Bolt= 608 lb
T/Bolt= 7,293 lb



Arm to Column Connection

Shear Capacity= Bolt Area * Fv
= [(0.75 in)^2 * pi/4] * 21000 psi
= 9,278 lb

Bearing Capacity= Bolt Diam * tmin * Fu * 1.2
= 9,788 lb

Tension Capacity= Bolt Area * Ft
= 19,439 lb

Combined stresses: (608 lb/9278 lb) + (7293 lb/19439 lb)= 0.44 OK

Structural
Engineering & Design Inc.

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

By: Bob S Project: Butler Supply Project #: 25-0625-3

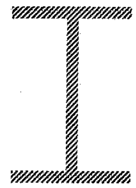
Column Load Case: DL+LL **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

Section

Section= W8x18
Area= 5.26 in^2
Ix= 61.90 in^4
Sx= 15.20 in^3
rx= 3.43 in

Iy= 7.97 in^4
Sy= 3.04 in^3
ry= 1.23 in

Fy= 50,000 psi
Kx= 2.4
Ky= 1.0
Lx= 108 in
Ly= 48 in
Cm= 1.00



Loads (Single Face Rack-Full Static Arm Loads)

Check Load Case 2 ASCE 2.4.1: : DL + LL <= 1.0, ASD Method

COLUMN DL= 381 lb
COLUMN PL= 7,200 lb
Mstatic= 175,032 in-lb

Pmax= 7,581 lb **Mstatic= 175,032 in-lb**

Combined Sress

(kl/r)x = (2.4*108 in/3.43 in)
= 75.57

(kl/r)y = (1*48 in/1.23 in)
= 39.02

(kl/r)max= 75.57

Cc = (2π^2E/Fy)^0.5
= 107.0

SINCE (KL/r)max < Cc, USE EQTN E2-1

Fa = [1-((kl/r)^2/2Cc^2)]Fy / (5/3 + 3(kl/r)/8Cc - (kl/r)^3/8Cc^3)
= 19,884 psi

fa= P/AREA
= 1,441 psi
fa/Fa= 0.07 < 0.15

fbx= M/S
= 11,515 psi

Fbx= 0.6*Fy
= 30,000 psi

F'ex= (12*π^2*E)/(23*(KL/r)^2)
= 26,150 psi

fb/Fb= 0.38 (1-fa/F'e)>0= 1.00

(H1-3): **fa/Fa + fb/Fb= 0.46 <= 1 OK**

Check Base Bending

Section= W8x18
Sx= 15.20 in^3

Fy= 50,000 psi
Fbx= 0.6*Fy
= 30,000 psi

Mbase-static= 175,032 in-lb fb/Fb= 0.38 **OK**

By: Bob S Project: Butler Supply Project #: 25-0625-3

Column Load Case: DL+LL+ Lateral Load **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

Section

Section= W8x18
Area= 5.26 in^2
Ix= 61.90 in^4 Iy= 7.97 in^4
Sx= 15.20 in^3 Sy= 3.04 in^3
rx= 3.43 in ry= 1.23 in
Stress Increase= 1.00

Fy= 50,000 psi
Kx= 2.4
Ky= 1.0
Lx= 108 in
Ly= 48 in
Cm= 1.00

Loads (Single Face Rack-Full Arm Loads plus lateral loads)

*Check Load Case 6b ASCE 12.4.2.3: : (1+0.10*Sds)D + 0.75L + 0.525E<= 1.0, ASD Method*

COLUMN DL= 381 lb
COLUMN PL= 7,200 lb
Mstatic= 175,032 in-lb
Mseismic= 18,491 in-lb
Sds= 0.1057
1+0.10*Sds= 1.0100

Pmax= 1.01 * 381 lb + 0.75 * 7200 lb
= 5,785 lb

M= 0.75*175032 in-lb + 18491 in-lb * 0.525
= 140,982 in-lb
(Mstatic + Mwind)

Combined Sress

(kl/r)x = (2.4*108 in/3.43 in)
= 75.57
Cc = (2π^2E/Fy)^0.5
= 107.0

(kl/r)y = (1*48 in/1.23 in)
= 39.02
(kl/r)max= 75.57

SINCE (KL/r)max < Cc, USE EQTN E2-1

Fa = [1-((kl/r)^2/2Cc^2)]Fy / 5/3 + 3(kl/r)/8Cc - (kl/r)^3/8Cc^3
= 19,884 psi

fa= P/AREA
= 1,100 psi
fa/Fa= 0.06 < 0.15

fbx= M/S
= 9,275 psi

Fbx= 0.6*Fy
= 30,000 psi

F'ex= (12*π^2*E)/(23*(KL/r)^2)
= 26,150 psi

fb/Fb= 0.31

(1-fa/F'e)= 1.00

(H1-3): **fa/Fa + fb/Fb= 0.36 <= 1 OK**

Check Base Bending

Section= W8x18
Sx= 15.20 in^3

Fy= 50,000 psi
Fbx= 0.6*Fy
= 30,000 psi

Mbase-lateral= 140,982 in-lb fb/Fb= 0.31 **OK**

Structural

Engineering & Design Inc.

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

By: Bob S Project: Butler Supply Project #: 25-0625-3

Anchors **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**

*Check Load Case 7 ASCE 12.4.2.3: : (0.9-0.20*Sds)DL + (0.9-0.20*Sds)Papp + E*

Overturning Forces

Seismic Forces

Seismic Ovt Moment=Mseismic= 18,491 in-lb

Rack DL= 381 lb *load per upright*

LLeff=Total LL*0.67=Papp= 4,824 lb *load per upright*

Mstabilizing=Mst= [0.879*(381 lb+4824 lb)]*56.14 in/2

= 128,426 in- lb

T-seismic= [Mseismic - Mst]/d * Omega

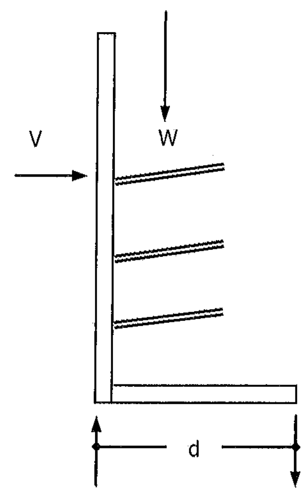
= 0 lb

net seismic uplift

Seismic shear= 220 lb/frame

Seismic Tension per anchor= 0 lb

Seismic Shear per anchor= 55 lb



Side Elevation

Base depth= 48.0 in

Column depth= 8.1 in

d= 56.1 in

0.9-0.20*Sds= 0.879

Check Anchors

Anchor Type= 0.75 in diam x 4.75 embed HILTI TZ2 Per ICBO ESR 4266

Anchor Inspection Required? Yes

Anchor/base= 4 Tallow= 1,961 lb Stress Increase= 1

Anchor/end= 2 Vallow= 2,517 lb Eqtn Exponent= 1

	Tension/anchr	Shear/anchr	Tallow	Vallow	Interactn Eq.	Status
Seismic	0 lb	55 lb	1,961 lb	2,517 lb	0.022	**OK**

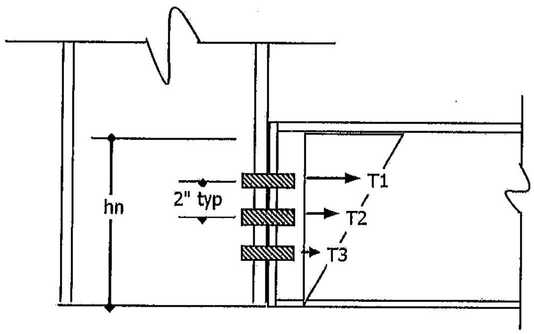
Structural

Engineering & Design Inc.

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

By: Bob S | Project: Butler Supply | Project #: 25-0625-3

Base to Column Connection | **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**



Base to Column Elevation

h1= 6.0 in
h2= 4.0 in
h3= 2.0 in

M_D= 2,232 in-lb
M_P= 172,800 in-lb
M_E= 18,491 in-lb
β= 0.7
p= 1.3
Sds= 0.106
Omega= 2

Bolt Diam= 0.750 in
Ft= 19.40 kip (Grade 5 or better)

T1= Ft * 2
= 38.80 kip

T2= T1 * h2/h1
= 25.87 kip

T3= T1 * h3/h1
= 12.93 kip

Moment Capacity=Mcap= T1 * h1 + T2*h2 + T3*h3 + T4*h4 + T5*h5 + T6*h6
= 362 in-kip

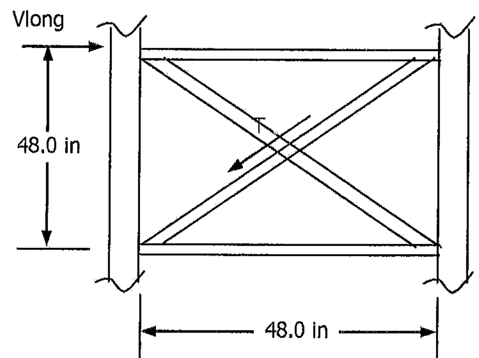
Load Case 1 (ANSI MH16.3-2016 Sec 2.1)	D+P=	175,032 in-lb
Load Case 5	(1+0.105*Sds)*D+0.75*[1.4+0.14*Sds)*β*P+L+S+0.7*p*E]=	143,227 in-lb
Load Case 6	(1+0.14*Sds)*D+(0.85+0.14*Sds)*β*P+0.7*p*E=	123,697 in-lb
Load Case 7	(0.6-0.14*Sds)*D+(0.6-0.14*Sds)*Papp+0.7*E*Omega=	-51,167 in-lb

M_demand= 175.0 in-kip **OK**

Check Longitudinal Brace

Brace Type= X Brace
Brace= 1.575x0.984x105
Area=An= 0.337 in^2

V-long= 169 lb
T= 0.525*Vlong*Ldiag/Lh
= 126 lb



Brace Panel Elevation

Ft = 0.6 * Fy
= 21,600 psi

ft= T/An
= 374 psi

ft/Ft= 0.02 < 1.0, OK

Bay Width=Lh= 48.0 in
Brace Panel Ht=Lv= 48.0 in
Ldiag= 68.0 in
Fy= 36,000 psi

Check Brace Connection

Connection= Bolted
Fv-bolt= 21,000 psi
Bolts= 1
Bolt Diam= 0.5

Bolt Capacity= Bolt Area * Fv-bolt
= 4,123 lb OK

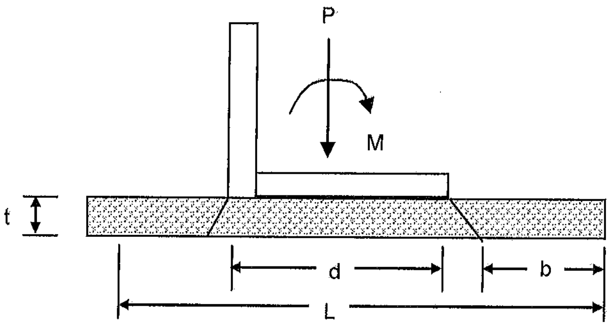
Structural

Engineering & Design Inc.

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

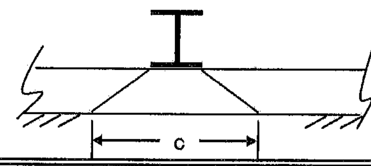
By: Bob S   Project: Butler Supply   Project #: 25-0625-3

Footing   Load case: DL+ LL   **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**



Thickness=t= 6.0 in
f'c= 4,000 psi
f-soil= 750 psf
base depth=d= 56.14 in

Base width=W= 5.25 in
B= 3
Stress Increase=a= 1.00
phi= 0.65
c= 17.3 in



Check Puncture

Pgravity= 7,581 lb

Mbase= 175,032 in-lb

Povt= Mbase/(d*0.66)
= 4,724 lb

P= 1.4 * (Pgravity + Povt)
= 17,227 lb

Fp= [(4/3) + (8/3)/B] * (f'c)^0.5 < 2.66 * (f'c)^0.5 * a
= 140.5 psi

Punct Area=A= (d + t) + (W + t) * 2 * t
= 880.7 in^2

Puncture Stress= (P/A)/Fp
= 0.14   **OK**

Check Bending

Areq= P/fsoil
= 3308 in^2
= 191.7 in x 17.3 in

b= 191.7 in - (56.14 in + 2 * 6 in) >= 0
= 123.56 in

Mslab= fsoil * c * b^2/(2*144)
= 4,763 in-lb

S= c * t^2/6
103.5 in^3

Fb= 5 * phi * (f'c)^0.5 * a
= 205.5 psi

fb/Fb= (Mslab/S)/Fb
= 0.22   **OK**

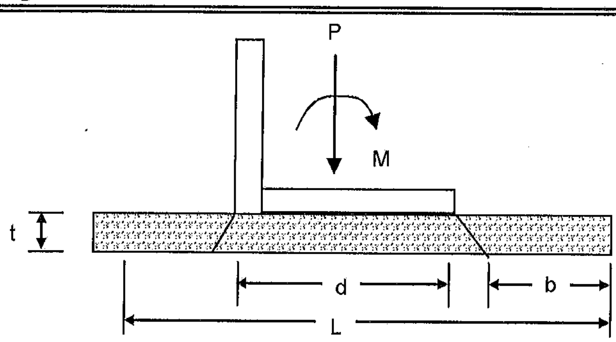
Structural

Engineering & Design Inc.

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

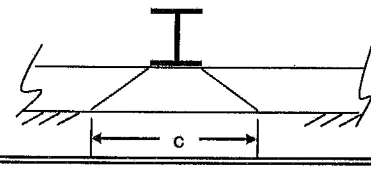
By: Bob S Project: Butler Supply Project #: 25-0625-3

Footing Load case: DL+ LL + Lateral **Configuration: TYPE 2 SGL-SIDED CANTILEVER RACK**



Thickness=t= 6.0 in
f'c= 4,000 psi
f-soil= 750 psf
base depth=d= 56.14 in

Base width=W= 5.25 in
B= 3
Stress Increase=a= 1.00
phi= 0.65
c= 17.3 in



Check Puncture

Pgravity= 5,785 lb

Mbase= 140,982 in-lb

Povt= Mbase/(d*0.66)
= 3,805 lb

P= 1.2*Pgravity + Povt
= 10,351 lb

Fp= [(4/3) + (8/3)/B] * (f'c)^0.5 < 2.66 * (f'c)^0.5 * a
= 140.5 psi

Punct Area=A= (d + t) + (W + t) * 2 * t
= 880.7 in^2

Puncture Stress= (P/A)/Fp
= 0.08 **OK**

Check Bending

Areq= P/fsoil
= 1987 in^2
= 115.2 in x 17.3 in

b= 115.2 in - (56.14 in + 2 * 6 in) >= 0
= 47.06 in

Mslab= fsoil * c * b^2/(2*144)
= 691 in-lb

S= c * t^2/6
103.5 in^3

Fb= 5 * phi * (f'c)^0.5 * a
= 205.5 psi

fb/Fb= (Mslab/S)/Fb
= 0.03 **OK**