Storage Rack Design

Sonny's

Lees Summit, MO

Project Number
TD092508

These calculations review the structure being installed for structural capability.
The sealing of the drawings used in conjunction with these calculations is for the structural review only. Other information is not reviewed, nor approved.

STATE OF MISSOURI
GARRY J.
MONTS
NUMBER
PE-2004005375
PROFESSIONAL ENGINEER

9/22/2025

	Engineering Seal Data:	
	State MO **Number** 2004005375	

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Design Summary Sheet

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Project: **Sonny's**
Lees Summit, MO

Pallet Rack
4 Level

Building Code -------------------------- **2018 IBC**

Steel Design Specification --------AISI, AISC, both based on LRFD Method, & ASCE 7

Total No. of Storage Levels -------	4				
Top Storage Level --------------------	20.000	Feet	Frame Depth -----------	42	Inches
Typical Beam Span ------------------	108	Inches	Panel Spacing----------	42	Inches
Allowable Soil Pressure ------------	1500	Psf	Max. Shelf Loading --	5300	Pounds
Allowable Compressive Strength of Concrete ----------------------------	4000	Psi	Avg. Shelf Loading --	5300	Pounds
Total bay load including dead ----	21.600	Max	Bay Loading ------------	14.604	Avg.

Period of vibration-Based on Rayleigh Method-down aisle--- Kx= 1.7 kips

T= 1.824 Seconds

Seismic base shear down aisle	0.0099	Ws Ws= Total bay load + dead load
Total base shear in kips------------	0.0726	Kips per rack column---down aisle
Seismic base shear cross aisle	0.378	Kips per frame-based on default values

Column Type ----------- **C-Section IR** CIR **Face** **3** **Depth** **3** **Thickness** **0.083**

First Level Drift

Design Ratio Live load+dead load+seismic -------------- $\boxed{0.757}$ 0.169 Inches

Design Ratio Live load+dead load ---------------------------- $\boxed{0.948}$

or cross aisle

Level No.	**Beam Connectors**	Connector		**Beam**	Beam **Weld**	Weld **Capacity** in-kips	Actual **Moment** in-kips
1	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	7.41
2	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	6.23
3	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	5.76
4	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	4.85
5	No storage level	da	N/A	da	0	0	0.00
6	No storage level	da	N/A	da	0	0	0.00
7	No storage level	da	N/A	da	0	0	0.00
8	No storage level	da	N/A	da	0	0	0.00
9	No storage level	da	N/A	da	0	0	0.00
10	No storage level	da	N/A	da	0	0	0.00
11	No storage level	da	N/A	da	0	0	0.00
12	No storage level	da	N/A	da	0	0	0.00

Upright Bracing Summary

Vertical Legé é é é .	1	Inches	Yield Fy=	36	Ksi
Widthé é é é é é é	1.5	Inches	Brace type =	CWLips	
Gageé é é é é é é	0.063	Inches			
Design Ratioé é é ..	$\boxed{0.155}$				

Base Plate Summary Slab Thicknessé é . 6 Inches Ratio = $\boxed{0.502}$ OK

Base Plate Widthé .. 8 Base Plate Depthé .. 5 Base Plate Thické . 0.375

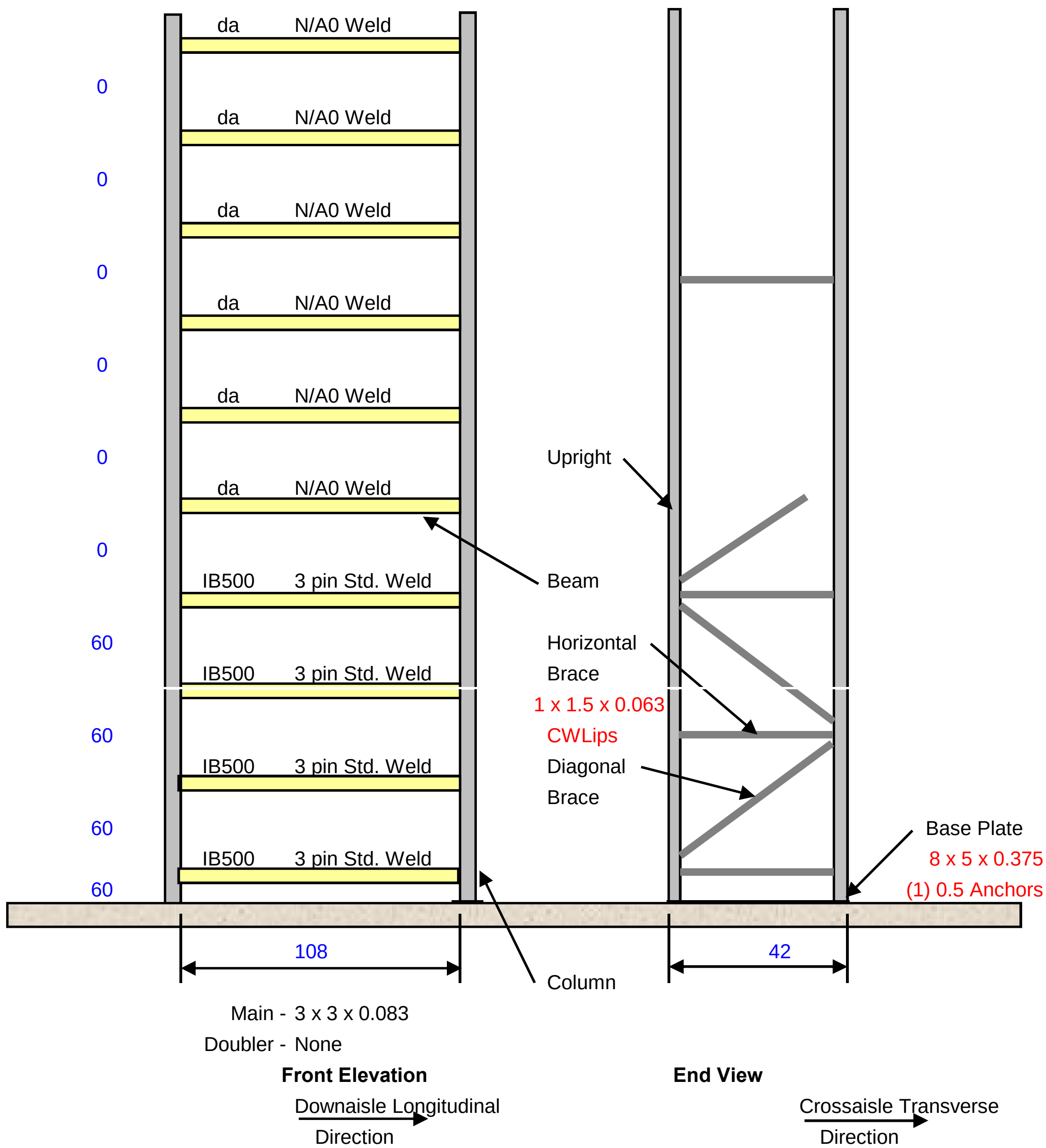
Yield Fy= 36 Ksi

Anchor Bolt Summary Redhead Trubolt ESR-2251 or equal

Anchor Bolt Descriptioné é é .. 1/2" Dia. X 2-1/2" Embd. Qty/Col.= 1

Anchor Bolt Design Ratio $\boxed{0.183}$

Date: 9/22/2025 **Pallet Rack Configuration - 4 Level** Page: ***1a***



Storage Level	Shelf Loading	Dead Load
1	5300	100
2	5300	100
3	5300	100
4	5300	100
5		
6		
7		
8		
9		
10		
11		
12		

2018 IBC

Using RMI specifications

2018 IBC Section 2209.1 Steel Storage Racks

RMI Minimum Seismic Forces modified by 2018 IBC

Down Aisle direction---Longitudinal

			Zip Code	64064		
V = CsIeW			Height	20.000	Feet	
Cs = Sd1/T/R			Ie =	**1**		
			R =	**6**	Limit State	
Cv = Sd1			Sds = 2/3 Sms			
			Sms = FaSs			
			Ss =	**0.099**		
			Fa =	**1.600**	Soil profile	**D**
			Sms =	0.158		
Cs =	**0.0099**		Sds =	0.106		

Ct	**Ta**		Calculated T in accordance with the Rayleigh Method
0.035	0.331	0.40	1.824

Cs = Sds/R			Sd1 = 2/3Sm1			
Ca = Sds / 2.5			Sm1 = FvS1			
			S1 =	**0.068**		
			Fv =	**2.400**	Soil profile	**D**
			Sm1 =	0.163		
Cs =	**0.0176**	**Max.**	Sd1 =	0.109		

Cs = .044Sds

Cs = **0.0046** **Min.**

Cs used in Base Shear Calculations

Down Aisle

Cs = **0.0099**

Date:
9/22/2025

2018 IBC

Using RMI specifications

2018 IBC Section 2209.1 Steel Storage Racks

RMI Minimum Seismic Forces modified by 2018 IBC

Cross Aisle direction---Transverse

V = CsIeW			Height	20.000	Feet	
Cs = Sd1/T/R			Ie =	**1**		
			R =	**4**	Limit State	
Cv = Sd1			Sds = 2/3 Sms			
			Sms = FaSs			
			Ss =	**0.099**		
			Fa =	**1.600**	Soil profile	**D**
			Sms =	0.158		
Cs =	**0.0685**		Sds =	0.106		

Ct	**Ta**		Calculated T in accordance with the Rayleigh Method
0.035	0.331	0.40	1.824

Cs = Sds/R			Sd1 = 2/3Sm1			
Ca = Sds / 2.5			Sm1 = FvS1			
			S1 =	**0.068**		
			Fv =	**2.400**	Soil profile	**D**
			Sm1 =	0.163		
Cs =	**0.026**	**Max.**	Sd1 =	0.109		

Cs = .044Sds

Cs = **0.0046** **Min.**

Cs used in Base Shear Calculations

Cross Aisle

Cs = **0.0264**

Base Shear

Load Factors and Combinations for LRFD Method

For all rack members

1	1.2D+L+1.4P	D = dead load
2	1.2D+1.6L+.5(S or R) + 1.4P	L= live load
3	1.2D+1.6(S or R)+(.5L or .8W) + .85P	P = product load
4	1.2D+1.3W+.5L+.5(S or R) +.85P	S = snow load
5	(1.2+.2Sds)D+(.85+.2Sds)P	W = wind load
6	1.2D+1.5E+.5L+.2S+.85P	R = rain
7	.9D-(1.3W or 1.5E)+.45P	E = seismic
		I = impact

1 Warehouse Rack System Non-public

Ip = **1**

V = CsIeWs	**Cs =**	**0.010**	Longitudinal
V = CsIeWs	**Cs =**	**0.026**	Transverse

Vl =	**0.0099 Ws**	Longitudinal	Limit State
Vt =	**0.0264 Ws**	Transverse	Limit State

Rack systems **Ws = (.67*Prf*P)+D+.25L** **Prf = Paverage / Pmaximum**

<u>Force at various shelf levels</u>

Fx = (V-F1)WxHx^k / ∑Wi Hi^k	for shelves greater than 12" above floor
F1 = CsIpWs	for shelf 12" or less above floor
Fx = VWxHx^k / ∑WiHi^k	for all levels when first shelf > 12" above floor

Exponent related to the structures period

T <= .5	**k =**	**1**
T > 2.5	**k =**	**2**

If the base shear is based on the default Cs value then k shall be taken as 1

Period Longitudinal direction	1.824	seconds	k =	2
Period Transverse direction	0.397	seconds	k =	1

FUNDAMENTAL PERIOD OF VIBRATION WORKSHEET

9/22/2025

Based on the Rayleigh Method

Used base shear-Longitudinal direction only

Percent Total Load/Level contributing base shear -----------------> 67.00 % for seismic

Distribution exponent -(k) -------------------> 1

Kx -------> 1.7

T ---------> 1.824 **seconds** **E*Ix** 38527

Column Style OC

Computed Distribution exponent -(k) ---- → 1.662

Elevation	Column DL	Column PL	Column LL	Column Tot. Load	Column Cu. Load	W	W * h^k	H
60	0.050	2.65	0	2.7	10.80	1.8255	109.5	0.10
120	0.050	2.65	0	2.7	8.10	1.8255	219.1	0.20
180	0.050	2.65	0	2.7	5.40	1.8255	328.6	0.30
240	0.050	2.65	0	2.7	2.70	1.8255	438.1	0.40
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
0	0.000	0	0	0	0.00	0	0.0	0.00
240	0.2	10.6	0	10.8		7.30	1095	1

Level	Cum F	L	Pcr	ΔP	Ω	Δt	Wi*Δt^2	Fi*Δt
1	1.0000	60	36.548	1.642	1.419	2.330	9.91	0.233
2	0.9000	60	36.548	3.119	1.285	4.228	32.64	0.846
3	0.7000	60	36.548	4.268	1.173	5.577	56.78	1.673
4	0.4000	60.000	36.548	4.925	1.080	6.286	72.13	2.514
5	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
6	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
7	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
8	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
11	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
12	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.000
							171.46	5.27

g = 32.2 ft/sec^2

T = 1.824 **seconds**

$$T = 2\pi \sqrt{\Sigma W i \Delta^2 \; / \; g \, \Sigma F i \Delta i}$$

Eq 30-10

Date: **FUNDAMENTAL PERIOD OF VIBRATION WORKSHEET** Page: **6**

9/22/2025

		Ss	0.10	
Seismic Coeficients:		S1	0.068	Soil Profile Type - D
		Rd =	6	Down Aisle
Cs = 1.2*Cv/(R*T^.667)		Rc=	4	Cross Aisle
		Cs =	0.0099	Down aisle per column
Av =	0.099	Cs =	0.0264	Cross aisle per frame

Actual Down-aisle base shear per col. =	0.073	kips
Actual Cross-aisle base shear per frame =	0.386	kips

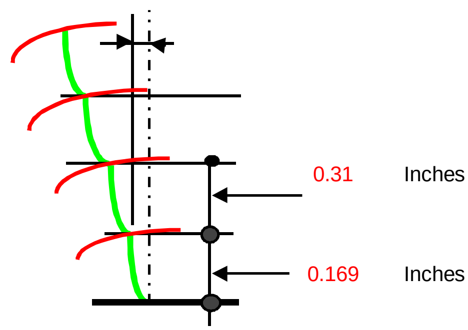
Level	Drift	P∆	Shelf Spacing
1	0.1692	1.83	60
2	0.3070	2.49	60
3	0.4049	2.19	60
4	0.4564	1.23	60
5	0.0000	0.00	0
6	0.0000	0.00	0
7	0.0000	0.00	0
8	0.0000	0.00	0
9	0.0000	0.00	0
10	0.0000	0.00	0
11	0.0000	0.00	0
12	0.0000	0.00	0

Guide Line for Drift Limit Only

.333*Column Width

Column Width

3



Rack Displacement Actual

Date: **LOADING DATA ON RACK SYSTEM** Page: **7**

9/22/2025

Special Notes:

Sonny's
Lees Summit, MO

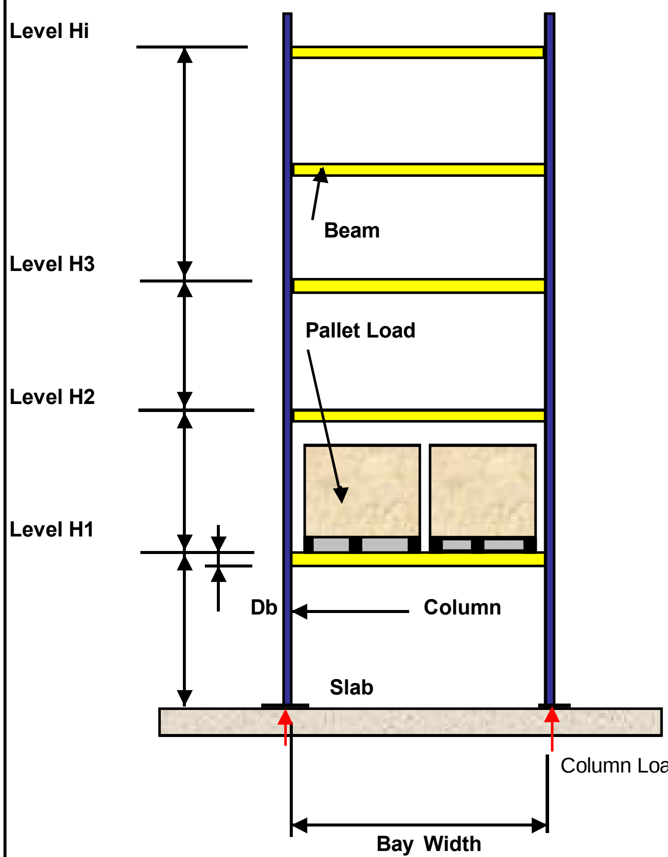
0.183

Rack Elevations and Loadings

Multiple deep-Single column load	No
Number Of Storage Levels No.=	**4**

Exterior Column Check

Shelf Load is for 1 Deep



	Level No.	Spacing	Loading *	Dead Load	Sum Shelf Load
60	1	60	5300	100	21600
120	2	60	5300	100	16200
180	3	60	5300	100	10800
240	4	60	5300	100	5400
	5				0
	6				0
	7				0
	8				0
	9				0
	10				0
	11				0
	12				0
	Totals	240	21200	400	
			10.80		

Top Storage Level = 20.00 Feet

Column Loading Pc=		10800	Pounds	Dead+Live Load
60	Pc=	10.8	Kips	Unfactored
Beam Depth	Db =	5		Inches

FRONT ELEVATION

Rack Area/Profile : **Pallet Rack**
4 Level

SLAB AND SOIL DATA

Product Type---------- **CIR**

Column: C-Section IR

Allowable Soil Pressure Qsp= **1500**

Qsp= 10.42 Psi

Shelf Loading for Seismic RMI 2.7.2

Allowable Compressive Strength Concrete

CA Ws = .67P+D Live load = 0

Fc'= **4000** Psi

DA Ws =(.67*R*P)+D+.25*L

DA Ws = 0.670 P+D

Maximum Shelf Load	PLm =	**5300**	Pounds
Average Shelf Load	PLa =	**5300**	Pounds

CA Ws = 0.670 P+D

1.0

Load coff.

Pav / Pm Ratio Ral = 1.000 100% Ratio

RMI Section 2.7.2

Down aisle cofficient	0.67
Cross aisle coffecient	0.67

Date: **RACK CONFIGURATION---LOAD /LEVEL/SEISMIC DATA** Page: ***8***

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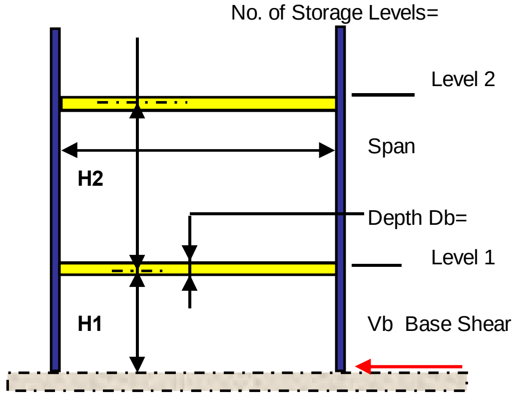
Enter Seismic Code **2018 IBC**

Avg.Top Shelf Load= **5300** Pounds

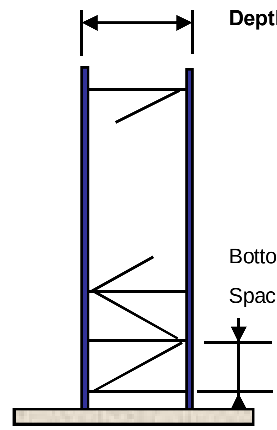
No. of Storage Levels= **4**



Span **108** Inches

Depth Db= 5 Inches

Vb Base Shear



Depth

42

42

Bottom Panel

Spacing Inches

Front Elevation

Longitudinal Direction

End Elevation

Transverse Direction

Load-Level-Seismic Values

Down aisle seismic loading

Level	**Spacing Inches**	**Loading Pounds**	**Dead Load**		**Ratio**	**Vi / Level**	**Sum**
				Vb=	0.0099	Wp	
				Vb =	0.145	kips/frame	
				Vb=	0.073	Kips/Column	
1	60	3551	100	14604	0.033	0.002	0.073
2	60	3551	100	10953	0.133	0.010	0.070
3	60	3551	100	7302	0.300	0.022	0.061
4	60	3551	100	3651	0.533	0.039	0.039
5	0	0	0	0	0.000	0.000	0.000
6	0	0	0	0	0.000	0.000	0.000
7	0	0	0	0	0.000	0.000	0.000
8	0	0	0	0	0.000	0.000	0.000
9	0	0	0	0	0.000	0.000	0.000
10	0	0	0	0	0.000	0.000	0.000
11	0	0	0	0	0.000	0.000	0.000
12	0	0	0	0	0.000	0.000	0.000
Totals	240	14204	400		1.000	0.073	Kips
Loads:	7.30	Kips	0.20	Kips			
Total D+P+L	14.60	Kips--Avg.					

Summary of Data

Summation of Rack Load = 21.60 Kips D+P

Total Base Shear/Column = 0.073 Kips

Total Base Shear/Frame = 0.145 KIps

Seismic down aisle loading

14.60 kips

Seismic cross aisle loading

14.60 kips

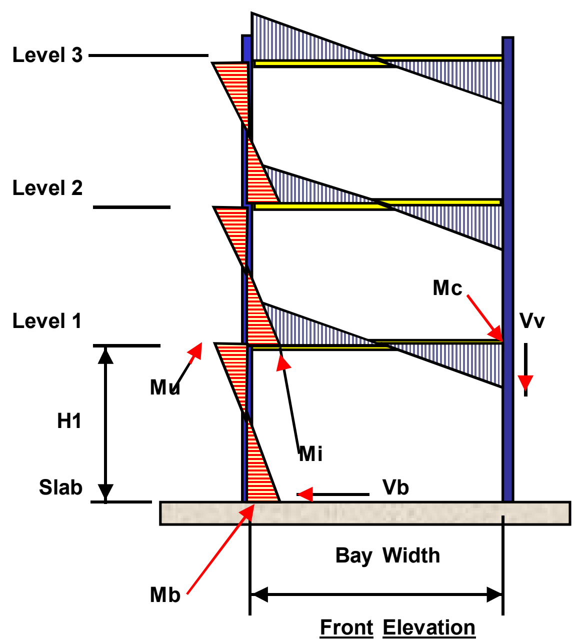
LRFD Design Loadings

Load Combinations		**Frame**	**Column**	**Ratio**	**Load Factor**
Axial Loading	1.2D + 1.4P	30.16	15.08	2.065	1
Axial plus seismic	1.2D+.85P+E	18.50	9.25	1.267	

Date: 9/22/2025

SHEAR / MOMENTS IN COLUMNS AND BEAMS

Moment Diagram of Rack Structure



108 In

Base Fixity
PinnedRb= 1
Fixed Rb= 0.5

Base Fixity .... Rb= 1
Base Shear Vb = 0.073 Kips

Moment at Base Mb= (H1 * Vb) * Rb
Moment at base Mb= 0.00 In-Kips

Column Load Pmax= 7.30 Kips

Mu= Moment Above Beam Level-Kips
Mi= Moment Below Beam Level-Kips
Mc= Connection Moment -Kips
Vv= Vertical Shear-End Connection- Kips

PORTAL MOMENT DISTRIBUTIONS

Level	Spacing	Mi	Mu	Mc	Column M facotored
12	0	0.00	0.00	0.00	0.00
11	0	0.00	0.00	0.00	0.00
10	0	0.00	0.00	0.00	0.00
9	0	0.00	0.00	0.00	0.00
8	0	0.00	0.00	0.00	0.00
7	0	0.00	0.00	0.00	0.00
6	0	0.00	0.00	0.00	0.00
5	0	0.00	0.00	0.00	0.00
4	60	1.16	0.00	0.58	1.16
3	60	1.82	1.16	1.49	1.82
2	60	2.11	1.82	1.96	2.11
1	57.50	4.17	2.11	3.14	4.17

Totals	237.50	**Inches**
	19.79	**Feet**

COLUMN DESIGN CHECK--COMBINED AXIAL AND SEISMIC

Date:
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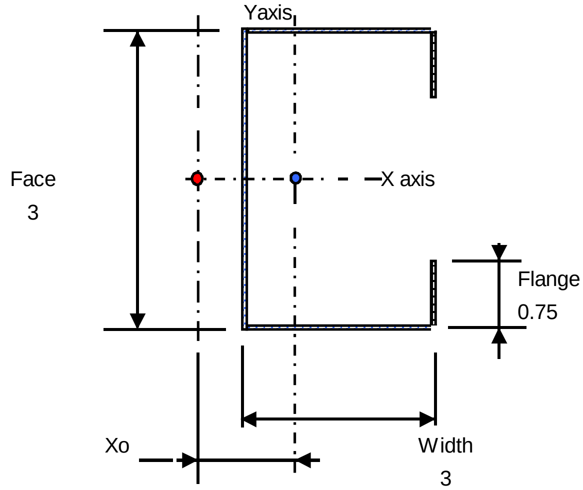
Description: 4 Level

Column Description: special Column

Fy = 55 Ksi

E = 29500



Column Cross Section

Column Section Properties

Column=

	T =	0.0830	In.
Gross Area	Ag=	0.824	In^2
Net Area	An=	0.598	In^3
	Ix =	1.306	In.^4
	Sx =	0.871	In.^3
	Rx =	1.259	In.
	Iy =	1.024	In.^4
	Min. Sy =	0.595	In.^3
	Ry =	1.115	In.

Torsional Properties

M=	1.611	Xo =	-2.849
Cw =	2.749		
Ro =	3.308	J =	0.0018917
β =	0.258	Sx eff =	0.585 In.^3

Critical Buckling Values --- Current Edition AISI LRFD

σex = (π^2E(KxlX/Rx)^2)	Eq. C3.1.2-7	48.305
σey =(π^2E(KyLy/Ry)^2)	Eq. C3.1.2-8	279.312
σet =(1/ARo^2)*(GJ+π^2Cw/(KtLt)^2)	Eq. C3.1.2-9	109.389
Fe =(1/2β)*((σex+σt)-((σex+σt)^2-4$\beta\sigma$exσt)^.5	Eq. C4.2-1	35.583

If First Beam Level Is Near Floor Use Second Level For Column Design

Lx =	57.500	Inches	Ly =	36	Inches	Lt =	36	
Kx =	1.7	Inches	Ky =	1	Inches	Kt =	0.8	Inches

Concentrically Loaded Compression Members Section C4

Fe = 35.583

Fy = 55

For λ < 1.5 Fn = Fy * .658^(λ^2) — λ = 1.243

For λ > 1.5 Fn = Fy * .877/(λ^2) — Φ = 0.85

Fn= 28.801 Ksi — ΦPn= 15.90 Kips

Frame Capacity - Vertical Load Only = 23,003 Pounds

Lateral Buckling Strength

Mn = Sx eff (Mc / Sx) — Φb = 0.9 — My = Sx * Fy

Me = Cb Ro A(σeyσt)^.5 — Cb =1.00 — My = 47.884

Mc = Crit. Moment — Me = 476.373

Mc = 47.884

For Me > 2.78*My Mc = My

For .56*My< Me < 2.78*My Mc = My(1-10*My/(36*Me)) — Mn = 32.155

For Me < .56*My Mc = Me — ΦMn = 28.939 in-kips

Date:
9/22/2025

COLUMN DESIGN CHECK--CONT'D

Section C5 Combined Axial Load and Bending

Design Ratio Dr = Pu / Oc Pno + CmMux / ObMnx O nx <= 1.0 Cm = .85
σex = (1- Pu / Pe) Φc= .85 Φb=.9

Pe = π^2EIb / (Kb Lb)^2		First Beam Level				H1=	57.5
Kb =	1.7		Column Unfact	Fact	Fact	H2=	60
Pe =	39.79	P1 =	10.80	9.25	15.08	M1 =	4.17
		P2 =	8.10	6.94	11.31	M2 =	2.11

Nominal	Axial Strength	ΦPn =	15.90	Kips	from previous page			
Full Dead+Live Load	Pu unfactored	Puf =	10.80	Kips	1.2D+1.4L	Pu =	15.08	Kips
(1.2+.2Sds)D+(.85+.2Sds)P	Factored	Pu =	9.48	Kips				
Nominal Flexu	Flexural Strength	ΦMnx =	28.94	In-Kips				
Required	Mux unfactored	Muxf=	4.17	In-Kips	Magnification factor =		1.116	
Seismic moment factored		Mux =	4.17	In-kips	Factor=	1.000		
Total factored moment			4.17	In-kips				
1.2D+.85L+E	Pu / Φc Pn	=	0.596		Mf * Mu/(ΦMn)=		0.161	

Dr = design ratio Pu / ΦPn + CmxMx / ΦbMnx α

Dr = $\boxed{0.757}$ **O.K.**

Design Check For Dead Load + Live Load Only

Unfactored	Column Load	Pc =	10.80	Kips			
1.2D+1.4W	Factored Column Load	Pu =	15.08	Kips	Cross Aisle	11.13	Kips
Down Aisle					Control P=	15.08	Kips
Nominal Axial Strength		ΦPn=	15.90	Kips	Φc =	0.85	

Design Ratio Pu /Φc Pn = $\boxed{0.948}$ **Ok**

DATE: 9/22/2025

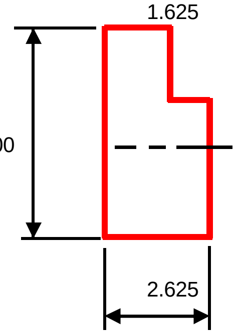
BEAM DESIGN WITH PARTIAL END FIXITY

Page: 11

Beam =	**Special**	**Span =**	**108** Inches	T =	0.063	In.
YIELD Fy=	55	Ksi		Inside Rad	0.125	In.
From Cold Working	61.21	Ksi		Area =	0.919	In^2
Fu =	70.00	Ksi				

SECTION PROPERTIES

Sx =	1.057	In.^3
Ix =	2.767	In.^4
E * Ix =	81626.5	Ksi
ΦbMn=	.90*Sx*Fy	
ΦbMn=	58.21	in-kips



6172 stress
6972 deflection

Req. Cap. 5300

Gr= 5.60

Mer= 4.274

RMI impact addition	0.125		
Load factor =	(1.4 + 1.4 * RMI addition) * PL		
Load factor =	1.575	1.2D+1.4L =	1.424
Beam weight =	30.12		

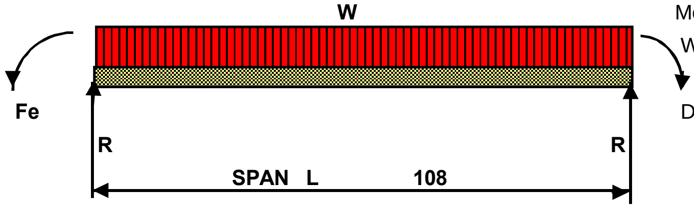
END CONNECTOR SPRING CONSTANT

G = 2 * E*Ix/(Fe*L) + 1

Me = W*L/(12*G)

W = 8 * (Phi*Mn/L.F. + Me)/L-1.2D

Fe =	330	In-Kips/Ra	G =	5.581
Me_max =	6	In-Kips		
	Unfactored	Upper limit		



Me =	4.98	at stress limit	
W =	3.086	Kips/Beam	
		Factored	
D =	0.531	IN.	Deflection

Wt = 6172 Per Pair of Beams — **Stress limit**

Equation for Defl. Cap when limit is L/180:

W = (384*E*Ix)*(L/180 + Me* L^2/(8*E*Ix))/(5*L^3)

Deflection Limit

Span divided by----> 180 Di = 0.600 In. Deflection Limit

Wd = Beam Load Based on Deflection Limit = L/Deflection limit

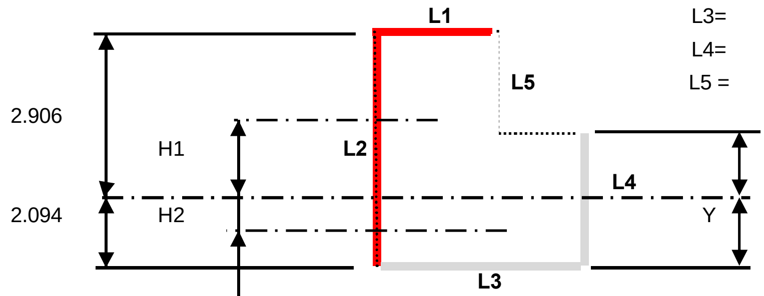
Wd = 3.486 Kips Me = 5.621 at defl. limit

Wt = 6972 Per Pair of Beams — **Deflection Limit**

Date: 9/22/2025

Beam Weld
Plastic Section Modulus

L1=	0	Inches	Offset
L2=	5	Inches	1.625
L3=	0	Inches	
L4=	3.375	Inches	
L5 =	0	Inches	
	8.375	Inches	
	1.2813		



Center of Area

Weld Pattern

At = Ab
At= (L2-Y) + L1 + (L4-Y)+(L5-Y)
Ab= 2Y +L3

Y = (L1 + L2 +L4+L5 - L3) / 4

Y= 2.0938 Inches

H1= 1.2045 Inches   At= 4.188
H2= 1.0469 Inches   Ab= 4.188

Z= H1*At + H2*Ab   Z= 9.428 Inches^3

Steel Thickness= 0.063 Inches
Zt= 0.594 Inches^3

Plastic Moment Capacity

Φ= 0.7   Mu= 41.58 In-Kips
Fu= 70 Ksi
ΦMn= 29.10 In-Kips

Φ= 0.55   Vn= 27.700
ΦVn= 15.235 Kips

Design Ratio Check

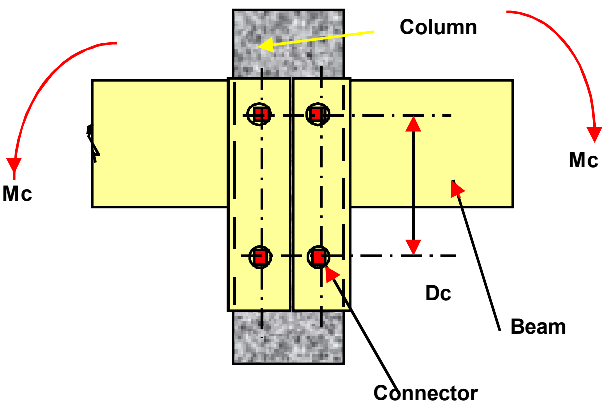
Mfac= 7.414 In-Kips   D+L+E
Vfact= 1.350 Kips   Shelf Loadé é é é   5400 Pounds

Ratio=Mfac/FMn + Vfac/FVn <1.03

Ratio= **0.343** Ok

Date: **BEAM TO COLUMN CONNECTION CHECK** Page: ***13***

9/22/2025 4



Beam to Column Connection

Moment at Beam to Column Connection

Level No.	Factored Moment Mc In-Kips	Factored Beam Shear Vi Kips
1	3.14	1.156
2	1.96	1.156
3	1.49	1.156
4	0.58	1.156
5	0.00	0.000
6	0.00	0.000
7	0.00	0.000
8	0.00	0.000
9	0.00	0.000
10	0.00	0.000
11	0.00	0
12	0.00	0

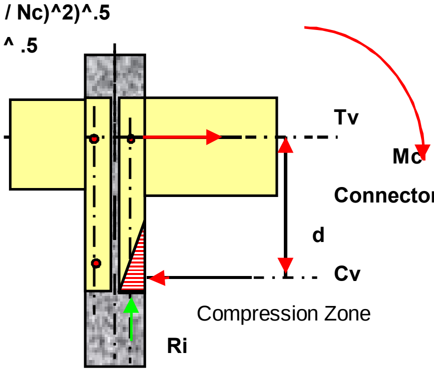
Mca= d * (Fva ^2 - (Ri / Nc)^2)^.5

Fva = (Vh ^2 + Vi ^2) ^ .5

Vh = Mc / d

Vi = Ri / Nc

Nc = Number Connectors



Allowable Shear on Connectors

Fv = .4Fu

Fv = **20** Ksi

Fva = Ab * Fv

Ab = Shear Area of Connector

Connector Discription: **Stud**

Ab = 0.155 In. ^2

Fva = 3.10 Kips Allowable Shear

Beam Type Includes fixed end moment

Offset Ω = **1**

Combined Forces on Connection Point

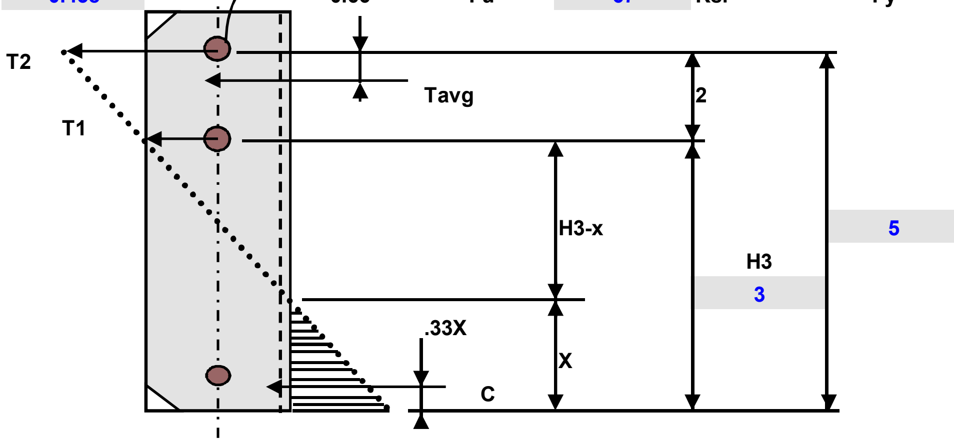
Level	End Conn	Connectors Moment	Shelf Loading		Beam No.	Weld Description	Factored Ma	Mc End Mom.
1	3P	18.19	5400	5	IB500	Std.	29.1	7.41
2	3P	18.19	5400	5	IB500	Std.	29.1	6.23
3	3P	18.19	5400	5	IB500	Std.	29.1	5.76
4	3P	18.19	5400	5	IB500	Std.	29.1	4.85
5	da	0	0	.	da	0	0	0.00
6	da	0	0	.	da	0	0	0.00
7	da	0	0	.	da	0	0	0.00
8	da	0	0	.	da	0	0	0.00
9	da	0	0	.	da	0	0	0.00
10	da	0	0	.	da	0	0	0.00
11	da	0	0	.	da	0	0	0.00
12	da	0	0	.	da	0	0	0.00

Connectors:	**SC**	Standard	**MC**	Seismic Connector	**SP**	Special
Welds:	**S**	Standard	**C**	Weld	**W**	Weld All Around

Page: **14**

Stud Connector 3 Pin

		Studs	**Ae=**	**0.151**	**In^2**		2
Dia. Stud=	**0.438**	**0.66**	**Fu=**	**67**	**Ksi**	**Fy =**	**50**



<u>Connector Analysis</u>

C= T2 + T1

C=Tavg

Σ Ma=0=.67XC - ((H3-X) + 1.34)Tavg

.67XC= ((H3-X) + 1.34)Tavg

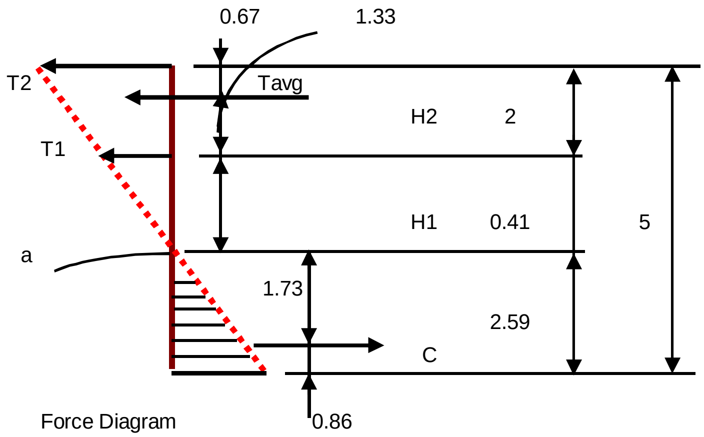
.67X=H3-X+1.34

x= 2.59 Inches

T2max = .4FuAe

T2max= 4.04 Kips

T1=H1T2 / (H1+H2) = 0.68 Kips



Force Diagram

<u>Determine Moment Capacity of Connector--Based on Stud Shear Capacity</u>

C= T2+T1 = 4.72 Kips

Ma= (T2+T1)*Dist. + C*Dist

Ma= 18.15 Inch-Kips ← Connector Capacity

Mas= 24.20 Inch-Kips ← Connector Capacity---Seismic

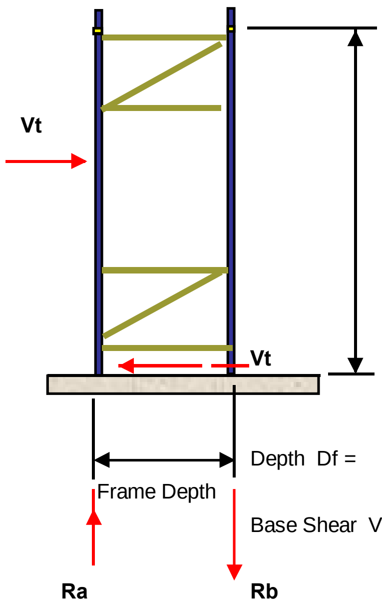
Bearing Consideration

Column Thickness		**Mas**		F= (d*Col. Tk.*1.67*Ft)	Actual Thickness	
0.07	Inches	15.34	In-Kips		**0.083**	
0.075	Inches	16.44	In-Kips		3.036	
0.083	Inches	18.19	In-Kips		18.19	in-kips
0.1	Inches	21.92	In-Kips			
0.126	Inches	24.20	In-Kips			
0.135	Inches	24.20	In-Kips			

Date: 5

9/22/2025

All Shelves Loaded



Vt = Total Shear

Rack Hgt.

overturning based on

heigth to load mass center

Depth Df = 42

Base Shear Vt = 0.38

SEISMIC FORCES HORIZONTAL

Level No.	Vi * Hi	Seismic Shelf Load	Moment	Due to OT
	In-Kips	Kips	In-Kips	Comp
1	3.18	3.551	77.20	12.7
2	10.90	3.551	74.02	10.0
3	23.16	3.551	63.12	7.0
4	39.96	3.551	39.96	3.7
5	0.00	0.000	0.00	0.0
6	0.00	0.000	0.00	0.0
7	0.00	0.000	0.00	0.0
8	0.00	0.000	0.00	0.0
9	0.00	0.000	0.00	0.0
10	0.00	0.000	0.00	0.0
11	0.00	0.000	0.00	0.0
12	0.00	0.000	0.00	0.0
	77.20			

Seismic Reaction Due to Overturning

Rs = Mot / Frame Depth Df

Rs = 1.88 Kips

Fac Rs= 1.88 Kips

Reaction Due to Pallet Loads + Dead Load

Rp = Pallet+Dead Load / 2 (2 Columns/Frame)

Rp = 9.25 Kips

Fac Ue = 7.37 Kips - factored Uplift

Resulting Reactions Combining Seismic + Pallet + Dead Load

Ra = Rp - Rs Uplift Side If Uplift Exists Rs Must be > Rp for Tension

Rb = Rp + Rs Compression Side of Upright

Factored Values:

Rb = 11.133 Kips

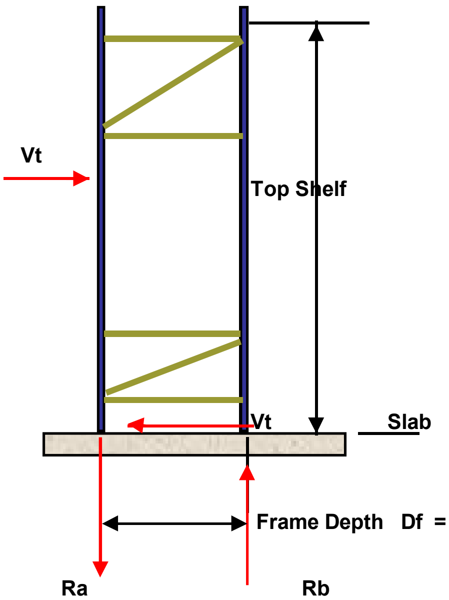
Ra = 7.367 Kips No Uplift

Vc = 0.189 kips

Loading /inch of height

Seismic 0.008

DL+PLé é é é é é é é é é é 0.039



Top Shelf Loaded Only

Top Load	Lt =	5.4	Kips
Top Shelf	Ot=	34.22	In-Kips

V = 0.14 kips

42
Inches

Seismic Reaction Due to Overturning
Rs = Mot / Frame Depth Df
Rs = 0.855 Kips

Reaction Due to Pallet Loads + Dead Loads
Rp = Pallet+Dead Load / 2 (2 Columns/Frame)
Rp = 2.313 Kips

Fac E = 0.86 Kips

Resulting Reactions Combining Seismic + Pallet + Dead Load

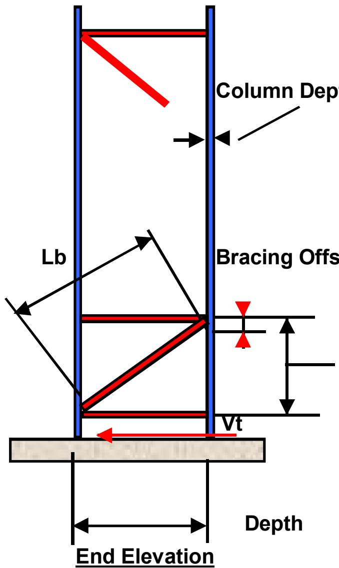
Ra = Rp - Rs Uplift Side Frame Rs Must be > Rp for Tension In Anchor Bolt
Rb =Rp + Rs Compression Side of Frame

Factored Values:

Rb =	3.17	Kips	
Ra =	1.46	Kips	No Uplift

CHECK DIAGONAL BRACING IN FRAME

Bottom Diagonal Brace in Frame



Lb = Diagonal Brace Length

Lb = ((Depth-2*Cd)^2 + (Panel Spacing-(2*Bo))^2)^.5

Column Depth Cd= 3

Depth-2*Cd Lh = 36

Panel Spacing-2*Bo Lv = 30

Lb = (Lh^2 + Lv^2)^.5

Bracing Offset Bo= 6 Lb= 46.9 Inches

Panel Spacing 42

Total Horizontal Base Shear

Vt = 0.378 **Kips**

Depth 42

Maximum Compression in Diagonal

Cmax =(Lb / Lh)* Vt*1.05------- Frame geometry multiplier

Load Distribution to Columns

Cmax = 0.52 Kips Factored

Date:	**C Section Axial Load Capacity**	Page:	**17a**
9/22/2025	Frame Bracing Member		

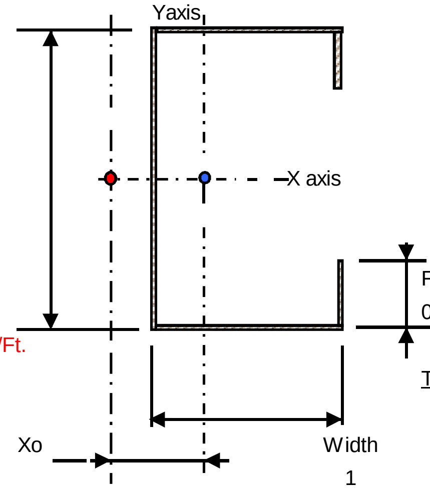
Un-braced length 46.9 inches — Special

Q value= 1

Column Description: No Holes Column

Fy = 36 Ksi

E = 29500



Face
1.5

Section Wgt/Ft.
0.82

Column Cross Section

Column Section Properties — Hole Pat

	Column=	Special		0
	T =	0.0630	In.	
Gross Area	Ag=	0.24	In^2	
Net Area	An=	0.24	In^3	
	Ix =	0.084	In.^4	
	Sx =	0.112	In.^3	
	Rx =	0.59	In.	
	Iy =	0.033	In.^4	
Min.	Sy =	0.056	In.^3	
	Ry =	0.372	In.	

Torsional Properties 2478

M=	0.533	Xo =	-0.909	
Cw =	0.023			
Ro =	1.146	J =	0.000318	
β =	0.37	Sx eff =	0.112	In.^3

Critical Buckling Values --- Current Edition 1996 AISI LRFD

σex = (π^2E(KxlX/Rx)^2)	Eq. C3.1.2-7	46.154
σey =(π^2E(KyLy/Ry)^2)	Eq. C3.1.2-8	18.346
σet =(1/ARo^2)*(GJ+π^2Cw/(KtLt)^2)	Eq. C3.1.2-9	26.355
Fe =(1/2β)*((σex+σt)-((σex+σt)^2-4β σexσt)^.5	Eq. C4.2-1	18.53

Lx =	46.86	Inches	Ly =	46.86	Inches	Lt =	46.86	
Kx =	1	Inches	Ky =	1	Inches	Kt =	0.8	Inches

Concentrically Loaded Compression Members Section C4

Fe = 18.346

Fy = 36

For λ < 1.5 Fn = Fy * .658^(λ^2) — λ = 1.401

For λ > 1.5 Fn = Fy * .877/(λ^2) — Φ = 0.85

Fn= 15.835 Ksi

ΦPn= 3.24 Kips

Channel Capacity Pmax = **3.33** kips Cmax = 0.52 kips

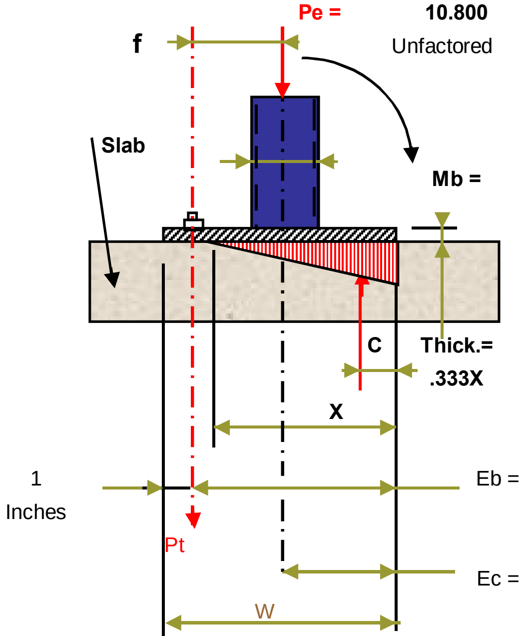
Ratio = 0.155

Pe = **10.800** Kips
Unfactored

Fc = 4000 psi

Bolt Data

Dia. =	**0.5**
Qty. =	**1**



Mb = **0.000** In-Kips
Unfactored

Special
Thick.= **0.375** **Inches**

Eb = 7 Inches

Ec = 4 Inches

Base Plate Width = 8 Inches
Base Plate Depth = 5 Inches

Solve For Tension In Anchor Bolt

Pt = -Pe[(W/2-X/3-e)/(W/2-X/3+f)]

Pt = 0.00 Kips

If Negative No Tension In Anchor Bolt

Check Maximum Stress on Slab

F1= Pmax/ Base Plate Area

F1 = 0.270 Ksi

Solve cubic equation for x value

X^3+K1X^2+K2X+K3 = 0 Blodgett(base plate)

K1 = 3(e-W/2)

K2 = 6nAs/B(f + e)

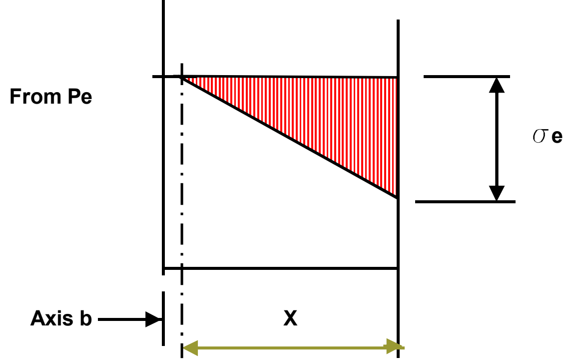
K3 = K2(W/2 + f)

X = **0** **Inches**

Solve for concrete stress

σe = 2(Pe + Pt)/ XB

Pe =	10.800
Pt =	0.000
σe =	0.270



Pressure On Slab From Base Plate

σmax=	0.270	Ksi
Actual	**Ok**	
Fall =	2.399	Ksi
Allowable		

Base Plate Data

Width =	8
Depth =	5
Thickness	0.375

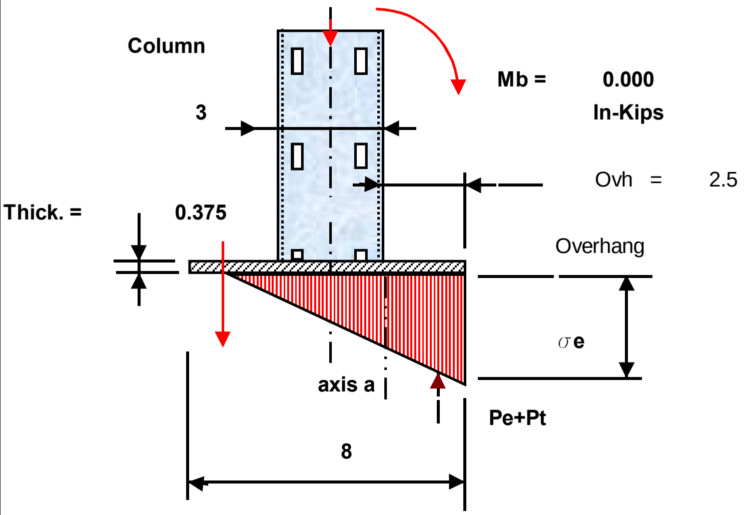
Check Bearing Plate for Full Load

LRFD	Pp = 1.7 Fc' Ae	Φ =	0.6		Effective Depth =		5	inches
					Ae =	40.00	in^2	effective area
RMI 7.2	Fc' =	4000	psi		Pu =	15.08	kips	1.2D+1.4P+.25L per column
	Pp =	272.000	kips		Pu =	11.13	kips	1.2D+.85P+E per column
	ΦPp =	163.200	kips		Pmax =	15.08	kips	
		Ok						

CHECK BASE PLATE THICKNESS

Post Style
C

Pmax = 10.800 Kips



Base Plate Loading

Check Bending In Base Plate

Moment from tension side

M = PL/2

Pt =	0.0000	kips

If Pt is negative then no tension
therefore Pt = 0

Mbp = Pt C

Mbp =	0.000	in-kips
C =	1.5	inches
Mbp =	0.000	In-Kips

Moment from compression side

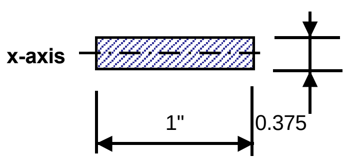
M = W L^2/3

W =	0.1350	Pli
L =	1.5	inches
Mbp =	0.101	in-kips

Design Moment

Mmax =	0.101	in-kips

Properties Of Base Plate



Section Modulus Sx= 1" * T^2 /6

Sx= 0.0234 In.^3

Allowable Bending Stress Fb= .75 * Fy AISC

Fy =	**36**	Ksi
Fb =	**27**	Ksi

Allowable Bending Moment

Mall = Sx * Fb

	Mall =	0.633	In-Kips
Mbp=	Mactual =	0.101	In-Kips

Design Ratio Rd = Mactual/Mallowable

Design Ratio Rd = 0.160 <= 1.000 OK

Date:	CHECK ANCHOR BOLTS FOR SHEAR/TENSION	Page:	20
9/22/2025			10

Check Bolt For Combined Shear And Pull-Out

Uplift Value From Overturning Section Fot= 1.458 Negative Uplift Occurs
Tension Value From Base Plate Section Ftb= 0.000 Negative No Tension

Longitudnal----------	Base Shear Per Colunm	Vlt =	0.073	Kips
Transverse -----------	Base ShearPer Column	Vtt =	0.378	Kips

Allowable Values For Expansion Anchors Manfac: Redhead Trubolt ESR-2251

	Concrete	fc' =	4000	Psi
			LRFD	
Vallowable	Allow. Shear	ΦFas=	2067	Pounds
Tallowable	Allow. Tension	ΦFt=	1221	Pounds

Resulting Combined Load Check
Fr= (Tactual / Tallowable) + (Vactual / Vallowable) <= 1.0

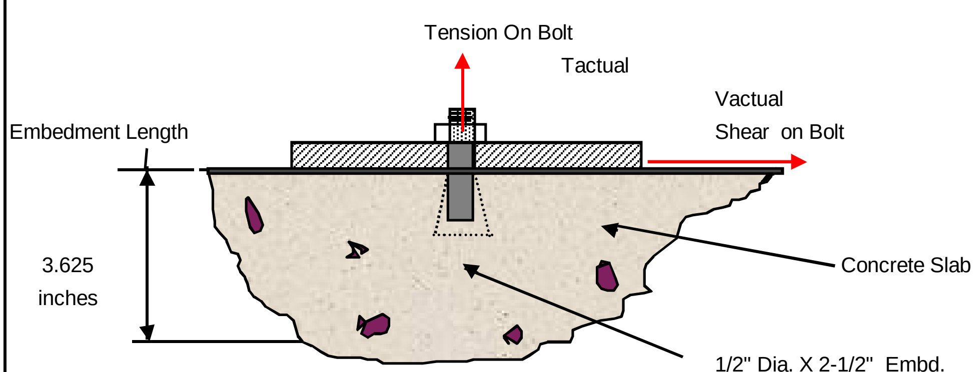
Vactual= Vtc / No. of Anchor Bolts Per Column — No. of Bolts 1 Per column

Vactual	Vtc=	0.378	Kips

Tactual = Ftb / No. of Anchor Bolts Per Column

Tactual	Ttb=	0.000	Kips

T Ratio =	0.000			
V Ratio =	0.183			
Fr=	0.183	<=	1	OK



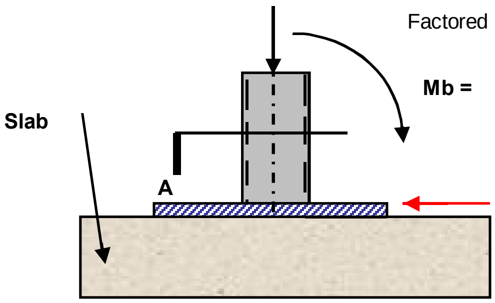
Date: 9/22/2025

Column Welds To Base Plate

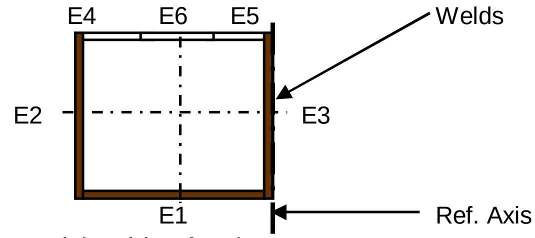
Pfac= **9.25** Kips
Factored

Mb = **0.00** In-Kips
Factored

0.109 Kips
Factored
Base Shear



Weld pattern on column



Aisle Side of Column
Section--A

	Weld Length	Section Dimensions	Weld C.G.
E1	3	3	1.5
E2	3	3	3
E3	3	3	0
E4	0	0	0
E5	0	0	0
E6	0	0	0

Base Plate Width = **8**
Base Plate Depth = **5**

Allowable Weld Stress
Eq. **E2.4-1 E2.4-2**

Pna=.4125 * Tm * Lw * Fu Longitudinal
Pnb=.6 * Tm * Lw * Fu Transverse

Pna= 25.99 Longitudinal
Pnb= 37.8 Transverse 42.00

Mc= Sx * Fu
Mc= 73.50 In-Kips
ϕ = .6
ϕ**Mc=** 44.10 In-Kips

Section Modulus of Weld Pattern

Fillet Weld Size Tw=	0.1	Inches
Weld Fu=	70	Ksi
Sx=	1.05	In^3
Lw=	9	Length of weld
Aw=	0.90	In^2
Liong=	3	Inches
Ltran=	6	Inches
Ac=	0.900	In^2

Fa= Pmax / Ac Fa= 10.28 Ksi If Fb > Fa If Fb<Fa Fv = 0
Fb= Mmax / Sx Fb= 0.00 Ksi Ft= -10.28 Ksi

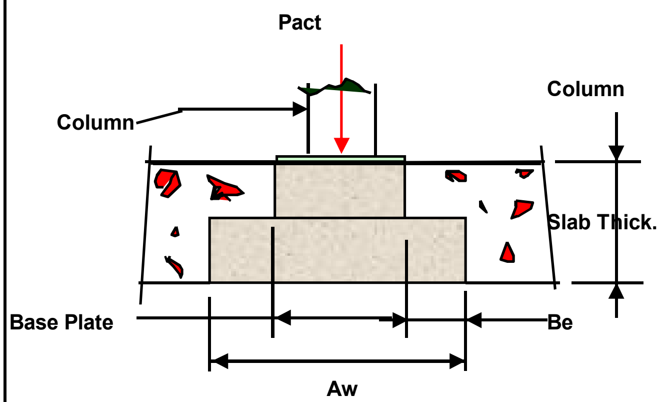
Fv= Vs / Aw Fv= 0.12 Ksi

Design Ratio Check

Desing Ratio Dr= Ft / Pnb + Fv / Pna = 1.03

Dr one= -0.245
Dr two= 0.005
Dr= $\boxed{\textbf{0.000}}$ **Ok**

Date	BASE PLATE SLAB/SOIL ANALYSIS	Page:	22
9/22/2025			

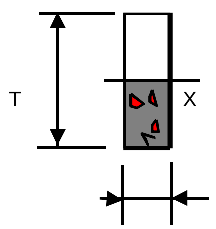


Loading=	**10.80**	**Kips**	Worst
	unfactored		Case
Fc' =	**4000**	**Psi**	
Qs =	**1500**	**Psf**	
Thick. =	**6**	**In.**	

FOOTER PROFILE UNDER COLUNM

TOP VIEW OF FOOTER

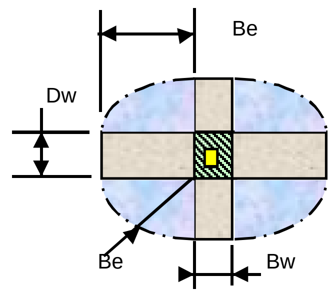
Slab Section



Sx = 1 * T^2 / 6

Sx = 6.000 In.^3

1 In.



Enter Base Plate Size Dw= 8 Bw= 5

Effective Area Ae = (2*Be+Bw)*Dw+(2*Be)*Bw+p*Be^2

Be = $\sqrt{8*Sx*Fct / Qs}$ Beam fixed one end pinned other

Fct =	101.2	Psi	Fct=1.6 $\sqrt{Fc'}$	Seismic Increase	1
Be=	21.59	In.		Yes =	1.333
				No =	1

Affective Area Ae=	2065.6 In.^2	14.3 Ft.^2			
Maximum Column Load Pmax =		21517	Pounds	21.5	Kips
Allowable					
Actual Loading on Column Pmax =		10800	Pounds	10.80	Kips
	Ratio =	0.502	Ok		

Date: 9/22/2025

Design Summary Sheet

Page: **23**

Project: **Sonny's**
Lees Summit, MO

Pallet Rack "S"
3 Level

Building Code ------------------------- **2018 IBC**

Steel Design Specification --------AISI, AISC, both based on LRFD Method, & ASCE 7

Total No. of Storage Levels -------	3				
Top Storage Level -------------------	20.000	Feet	Frame Depth -----------	42	Inches
Typical Beam Span ------------------	108	Inches	Panel Spacing----------	42	Inches
Allowable Soil Pressure ------------	1500	Psf	Max. Shelf Loading --	5300	Pounds
Allowable Compressive Strength of Concrete ----------------------------	4000	Psi	Avg. Shelf Loading --	5300	Pounds
Total bay load including dead ----	16.200	Max	Bay Loading ------------	10.953	Avg.

Period of vibration-Based on Rayleigh Method-down aisle--- Kx= 1.7 kips
T= 2.819 Seconds

Seismic base shear down aisle	0.0064	Ws Ws= Total bay load + dead load
Total base shear in kips------------	0.0352	Kips per rack column---down aisle
Seismic base shear cross aisle	0.284	Kips per frame-based on default values

Column Type ----------- **Dbl C section**   **Face** **2.95**   **Thickness** **See Cal.**
DBLC   **Depth** **6**   First Level Drift

Design Ratio Live load+dead load+seismic -------------- 0.943   0.484 Inches
Design Ratio Live load+dead load ---------------------------- 0.675

or cross aisle

Level No.	Beam Connectors	Connector		Beam	Beam Weld	Weld Capacity in-kips	Actual Moment in-kips
1	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	6.82
2	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	5.00
3	3 Pin Connector	3P	3 pin	IB500	Std.	29.1	4.56
4	No storage level	da	N/A	da	0	0	0.00
5	No storage level	da	N/A	da	0	0	0.00
6	No storage level	da	N/A	da	0	0	0.00
7	No storage level	da	N/A	da	0	0	0.00
8	No storage level	da	N/A	da	0	0	0.00
9	No storage level	da	N/A	da	0	0	0.00
10	No storage level	da	N/A	da	0	0	0.00
11	No storage level	da	N/A	da	0	0	0.00
12	No storage level	da	N/A	da	0	0	0.00

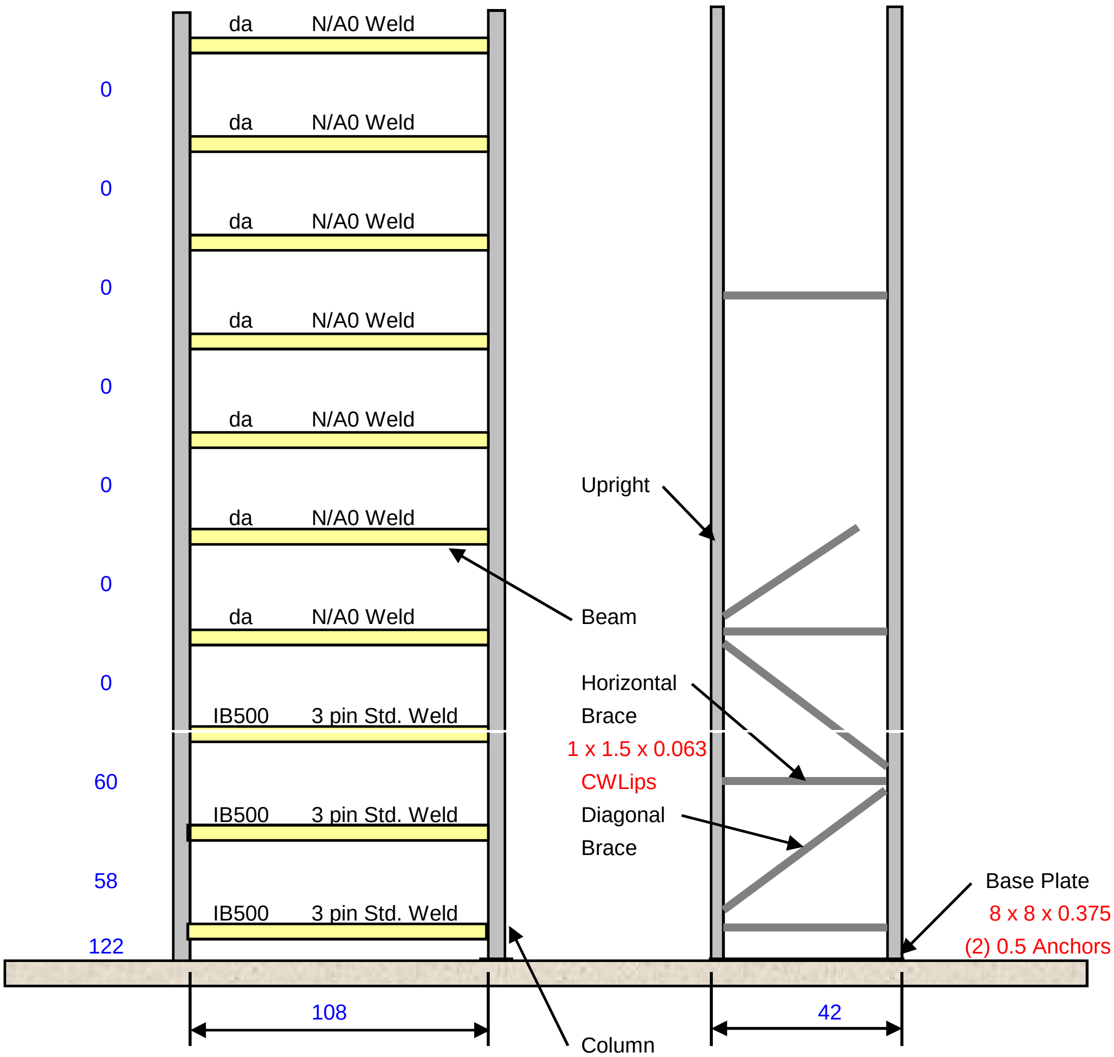
Upright Bracing Summary

Vertical Legé é é é .	1	Inches	Yield Fy=	36	Ksi
Widthé é é é é é é	1.5	Inches	Brace type =	CWLips	
Gageé é é é é é é	0.063	Inches			
Design Ratioé é é ..	0.113				

Base Plate Summary   Slab Thicknessé é . 6 Inches   Ratio = 0.350 OK
Base Plate Widthé .. 8   Base Plate Depthé .. 8   Base Plate Thické . 0.375
Yield Fy= 36 Ksi

Anchor Bolt Summary   Redhead Trubolt ESR-2251 or equal
Anchor Bolt Descriptioné é é .. 1/2" Dia. X 2-1/2" Embd.   Qty/Col.= 2
Anchor Bolt Design Ratio 0.069

Date: 9/22/2025 **Pallet Rack "S" Configuration - 3 Level** Page: ***23a***



Main - 3 x 3 x 0.083
Doubler - 3 x 3 x 0.083 x 126

Front Elevation **End View**

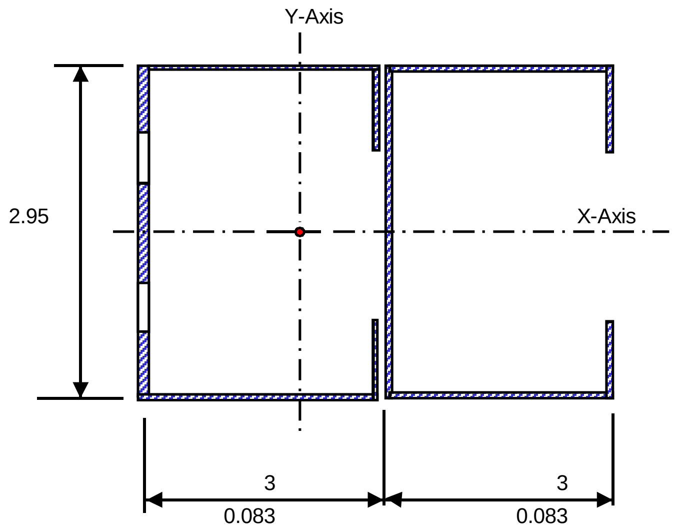
Downaisle Longitudinal Direction

Crossaisle Transverse Direction

Storage Level	Shelf Loading	Dead Load
1	5300	100
2	5300	100
3	5300	100
4		
5		
6		
7		
8		
9		
10		
11		
12		

DOUBLE COLUMN ANALYSIS AND DESIGN

Date: 9/22/2025



Column Cross Section 6

Yield of Steel	Fy=	55	Ksi
Punching	**2**		

Combined Section Properties

A gross =	1.652	In^2
Ix gross=	2.508	In^4
Sx gross=	1.700	In^3
Rx gross=	1.232	In
Ae =	1.502	In^4
Eix=	73986	

Lx=	119.5
Kx=	1.7
Ly=	36
Ky=	1
Lt=	36
Kt=	0.8

Critical Buckling Values **LRFD .... Latest Edition AISI**

Eq. C4.1-1) Fe = 3.14^2 * E / (KL/R)^2

Static Frame Capacity

Fe= 10.71 Ksi

Pframe= 17,645 Pounds

Φc = .85 Pn= AeFn

Eq. C4-4	lambda_crit =		(Fy/Fe)^.5	
Eq. C4-3	lambda_crit =		2.266	
Eq C4-2/Eq. C4-3		Fn =	9.39	Ksi
		Ae =	1.50	

Mn= Se * Fy

ΦbMn= 72.53 In-Kips

Eq C4-1	Pn=	14.11	Kips
	ΦcPn=	11.99	Kips
	ΦbMn=	72.53	In-Kips

Pu =	7.11	kips
Mu =	4.21	in-kips
Pu_stat=	11.31	kips
Puca=	8.44	Kips
Pud =	11.31	

Design Check for Dead + Live Load Only 1.2D + 1.4W

Pu/ΦcPn= 0.943 **O.K.**

Eq C5-1 Pu/ΦcPn + Cmx Mux/ΦbMnxσnx <= 1.0

Down Aisle Controls

Pu/ΦcPn= 0.593

Pe=π^2*e*Ib/(KbLb)^2

Pe= 17.69 Ksi

CmxMux/ΦbMnxσnx= 0.082

σnx=(1-Pu/Pe)

Design Check Ratio= 0.675 **O.K.**

σnx= 0.598