Eastern Engineering Group

3401 NW 82nd Avenue Suite 370 Doral, FL 33122
(305) 599-8133
info@easterneg.com
www.easterneg.com

STRUCTURAL CALCULATIONS

Project name:

8965 SW 63rd CT

PACHECO ARCHITECTURE, LLC.
SHOP DRAWING REVIEW

■ NO EXCEPTIONS TAKEN ☐ REJECTED
☐ NOTE COMMENTS ☐ RE-SUBMIT

CORRECTIONS OR COMMENTS MADE ON SHOP DRAWINGS DURING THIS REVIEW DO NOT RELIEVE CONTRACTOR FROM COMPLYING WITH REQUIREMENTS OF THE PERMITTED DRAWINGS AND SPECIFICATIONS INCLUDING APPLICABLE MANUFACTURER SPECIFICATIONS. THIS CHECK IS ONLY FOR GENERAL CONFORMANCE WITH DESIGN CONCEPT AND GENERAL COMPLIANCE WITH THE INFORMATION IN THE CONTRACT DOCUMENTS. CONTRACTOR IS RESPONSIBLE FOR: CONFIRMING AND CORRELATING ALL QUANTITIES AND DIMENSIONS; SELECTING FABRICATION PROCESSES AND TECHNIQUES OF CONSTRUCTION; COORDINATING HIS WORK WITH THAT OF ALL OTHER TRADES AND PERFORMING HIS WORK IN A SAFE AND SATISFACTORY MANNER.

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DATE: 08-28-26 REVIEWED BY: J. ESTORINO
ON BEHALF OF PACHECO ARCHITECTURE, LLC.

SHOP DRAWING REVIEW

Project Name
De La Concepcion Residence

ROCHELL ENGINEERING

☑ APPROVED ☐ REJECTED
☐ REVISE & RESUBMIT ☐ APPROVED AS NOTED

BY Alex Rochell, PE DATE 8/28/2026

SUBMITTAL# SPEC

This review is only for general conformance of the project and general compliance. Corrections or comments made on these drawings during this review do not relieve Contractor/Subcontractor from compliance with the requirements of the plans and specifications. Contractor is responsible for all dimensions and fabrication to be confirmed and correlated at the job site

Job No.:

26-0651

Project address:

8965 SW 63rd Ct, Pinecrest, FL 33156, USA

These calculations are strictly valid for the address above. Use in other projects is unauthorized and may constitute fraud.

THIS ITEM HAS BEEN DIGITALLY SIGNED AND SEALED BY **GONZALO A. PAZ, PE** ON THE DATE ADJACENT TO THE SEAL.

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GONZALO A. PAZ LICENSE No 60734 STATE OF FLORIDA PROFESSIONAL ENGINEER

Digitally signed by Gonzalo A Paz
Date: 2026.08.31 09:53:54-04'00'

Prepared by: Gonzalo Paz, PE
Lic. No. 60734/CAN # 26655

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DESIGN CRITERIA:

Calculations based on:

1. Florida Building Code 8th Edition 2023
2. Minimum Design Loads for Buildings and Other Structures ASCE 7-22
3. Building Code Requirements for Structural Concrete ACI 318-19
4. American Institute of Steel Construction AISC 360-16
5. Aluminum Design Manual 2020
6. American Wood Council NDS 2024

CALCULATION INDEX:

I.	Interior Cantilever Stair Design	3-33
II.	Glass Railing At Stair Design	34-54
III.	Guardrail Design	55-65
IV.	Handrail Design	66-78

Total Pages = 78

CALCULATION STATEMENT:

To the best of my knowledge, ability, belief and professional judgment, I hereby attest that the manual calculations and computer-generated calculations are in compliance with the existing governing codes.

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LICENSE
No 60734
STATE OF FLORIDA
PROFESSIONAL ENGINEER

Digitally signed by Gonzalo A Paz
Date: 2026.08.31 09:53:39-04'00'

Prepared by: Gonzalo Paz, PE
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INTERIOR CANTILEVER STAIR DESIGN

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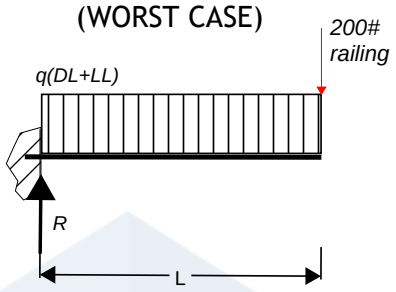
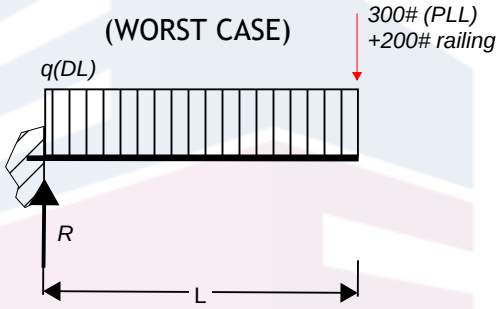
Loads:
DL= 30psf (DLsystem 15 + DLsuperimposed 15psf)
LL= 40psf or 300# conc.
Mrailing= 200#*3ft= 600#-ft= 0.6k-ft
T.S= 12in

L= 4'-1"=4.083ft

Loads Combination
1. DL+LL+Mrailing or 200#Conc.



2. DL+LL(conc.)+Mrailing or 200# Conc.



*For Tread analysis see spreadsheet printout
*Deflection Analysis

Δactual= 0.25 in
Δ Allow= L/360= 4.083ft/360 * 12 * 2= 0.27 in
Δ Allow= 0.27 in > Δ actual= 0.25 in (OK)

B. THE STRUCTURE IS DESIGNED FOR THE FOLLOWING LOADING:

	SUPER IMPOSED DEAD LOADS	LIVE LOADS
ROOF	25 PSF	30 PSF
SECOND FLR.	35 PSF	40 PSF
GROUND FLR.	35 PSF	40 PSF
GARAGE	15 PSF	50 PSF
STAIRS	15 PSF	40 PSF
TERRACE	15 PSF	60 PSF
BALCONY	15 PSF	60 PSF

TAKEN FROM

date 10.26.2021
issued
drawn M.V/M.M/R.P
checked A.R
project no. 2024
revisions
1 05-08-25 BLDG. DEPT. COMMENTS
2 09-02-25 BLDG. DEPT. COMMENTS
3 11-25-25 BLDG. DEPT. COMMENTS

NEW RESIDENCE FOR:
MR & MRS DE LA CONCEPCION
8965 SW 63RD COURT
PINECREST, FLORIDA 33156

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: 26-0651.ec6

LIC# : KW-06018068, Build:20.25.07.09 Eastern Engineering Group (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Combination #1 (DL+DLL Dist. + 200 Conc)

Code References

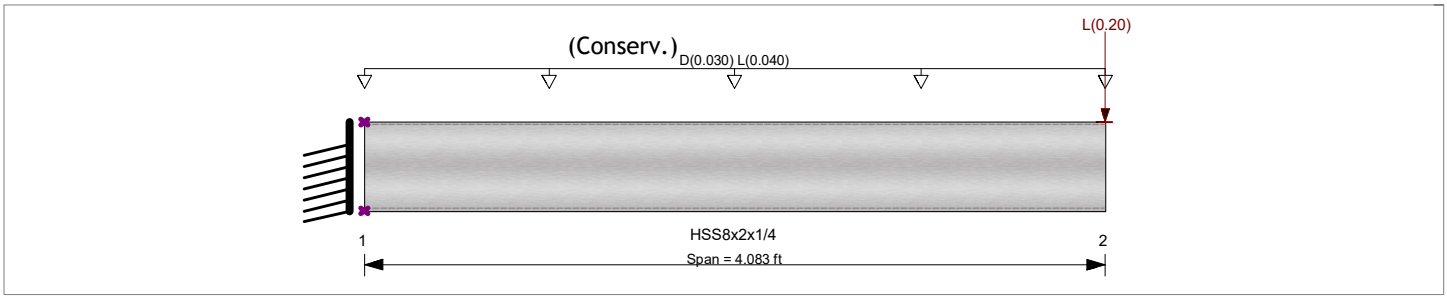
Governing Code : IBC 2018, CBC 2019
Referenced Design Standard(s) : AISC 360-22*
Load Combination Set : ASCE 7-22 / IBC 2024

*User-selected, not referenced by : IBC 2018, CBC 2019

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Minor Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.030, L = 0.040 ksf, Tributary Width = 1.0 ft
Point Load : L = 0.20 k @ 4.083 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.190 : 1	Maximum Shear Stress Ratio =	0.048 : 1
Section used for this span	HSS8x2x1/4	Section used for this span	HSS8x2x1/4
Ma : Applied	1.400 k-ft	Va : Applied	0.4858 k
Mn / Omega : Allowable	7.359 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.120 in	Ratio =	817	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.141 in	Ratio =	695	>=360.	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<360.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mny	Mny/Omega	Cb	Rm	Summary of Shear Values Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 4.08 ft	1	0.034	0.012		-0.25	0.25	12.29	7.36	1.00	1.00	0.12	16.73	10.02
+D+L													
Dsgn. L = 4.08 ft	1	0.190	0.048		-1.40	1.40	12.29	7.36	1.00	1.00	0.49	16.73	10.02
+D+0.750L													
Dsgn. L = 4.08 ft	1	0.151	0.039		-1.11	1.11	12.29	7.36	1.00	1.00	0.39	16.73	10.02
+0.60D													
Dsgn. L = 4.08 ft	1	0.020	0.007		-0.15	0.15	12.29	7.36	1.00	1.00	0.07	16.73	10.02

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+L	0.1410	4.083		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0211	4.083		0.0000	0.000
+D+L	1	0.1410	4.083		0.0000	0.000
+D+0.750L	1	0.1110	4.083		0.0000	0.000
+0.60D	1	0.0126	4.083		0.0000	0.000
L Only	1	0.1199	4.083		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: 26-0651.ec6

LIC# : KW-06018068, Build:20.25.07.09 Eastern Engineering Group (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Combination #1 (DL+DLL Dist. + 200 Conc)

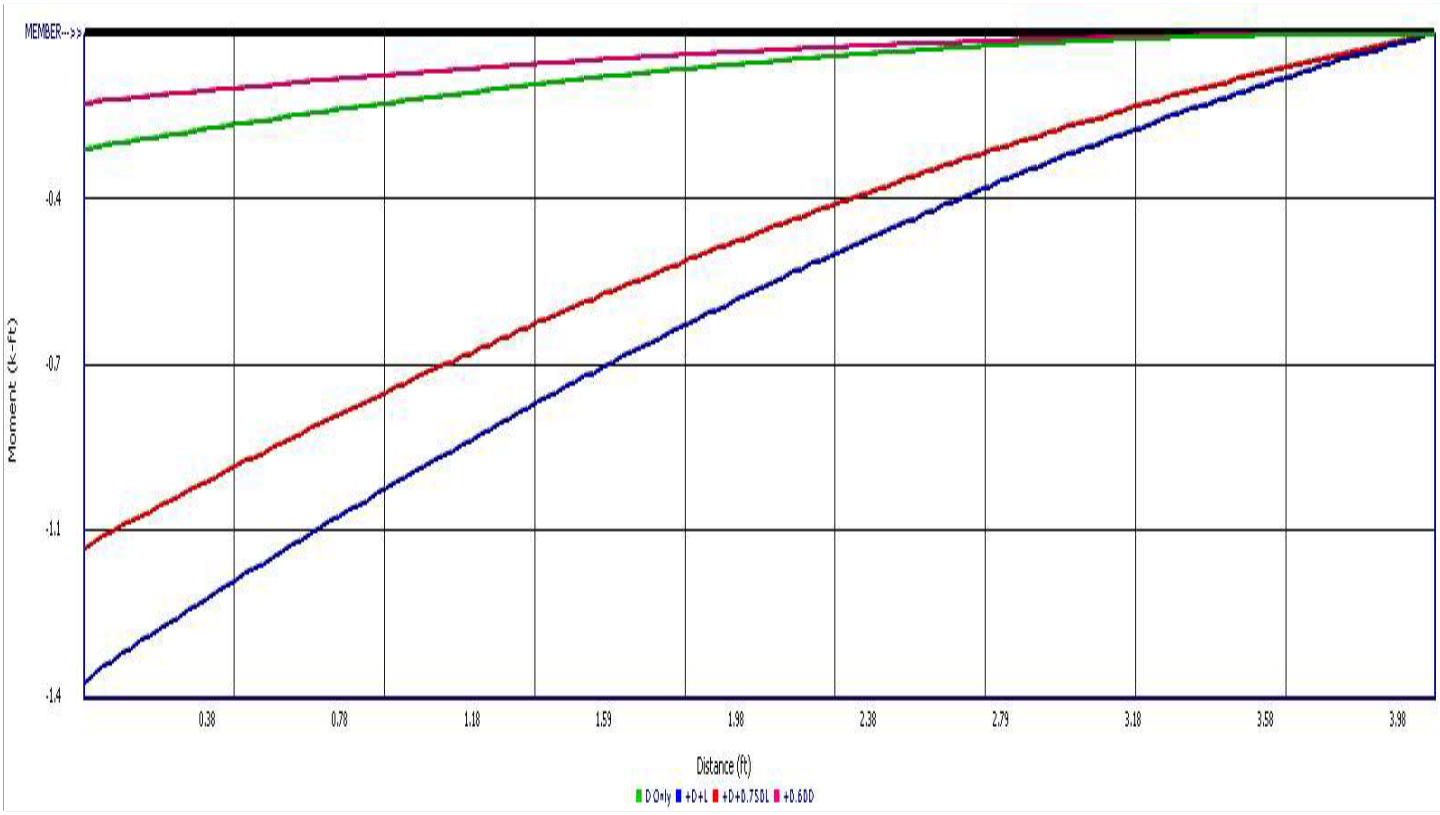
Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.486	
Max Upward from Load Combinations	0.486	
Max Upward from Load Cases	0.363	
D Only	0.122	
+D+L	0.486	
+D+0.750L	0.395	
+0.60D	0.073	
L Only	0.363	

Steel Section Properties : HSS8x2x1/4

Depth	=	8.000	in	I xx	=	28.50	in^4	J	=	9.360 in^4
				S xx		7.12	in^3	Cw	=	6.35 in^6
Width	=	2.000	in	R xx	=	2.570	in			
Wall Thick	=	0.233	in	Zx	=	9.680	in^3			
Area	=	4.300	in^2	I yy	=	2.940	in^4	C	=	0.000 in^3
Weight	=	15.620	plf	S yy	=	2.940	in^3			
				R yy	=	0.827	in			
				Zy	=	3.430	in^3			
Ycg	=	4.000	in							



Project Title:
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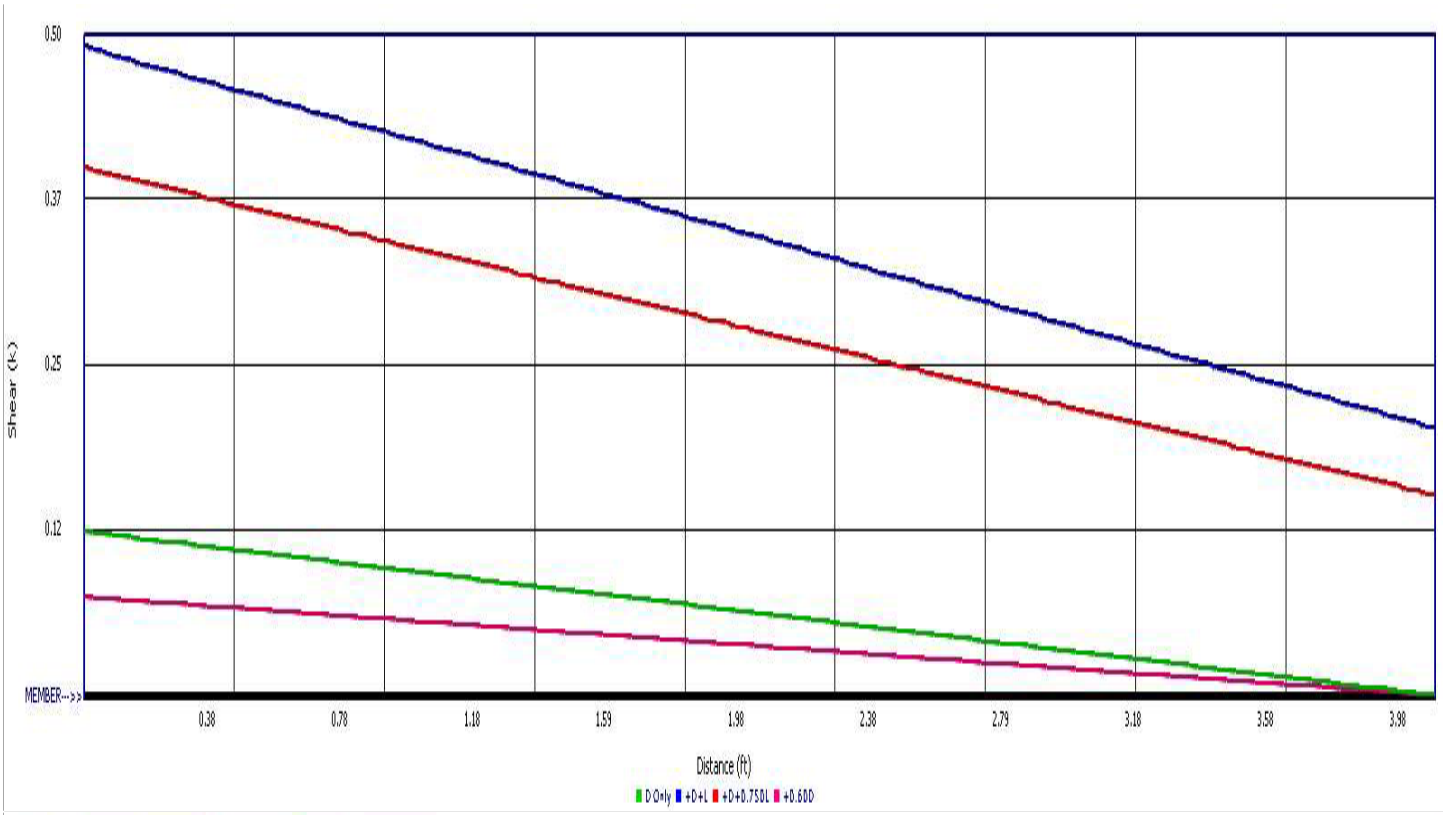
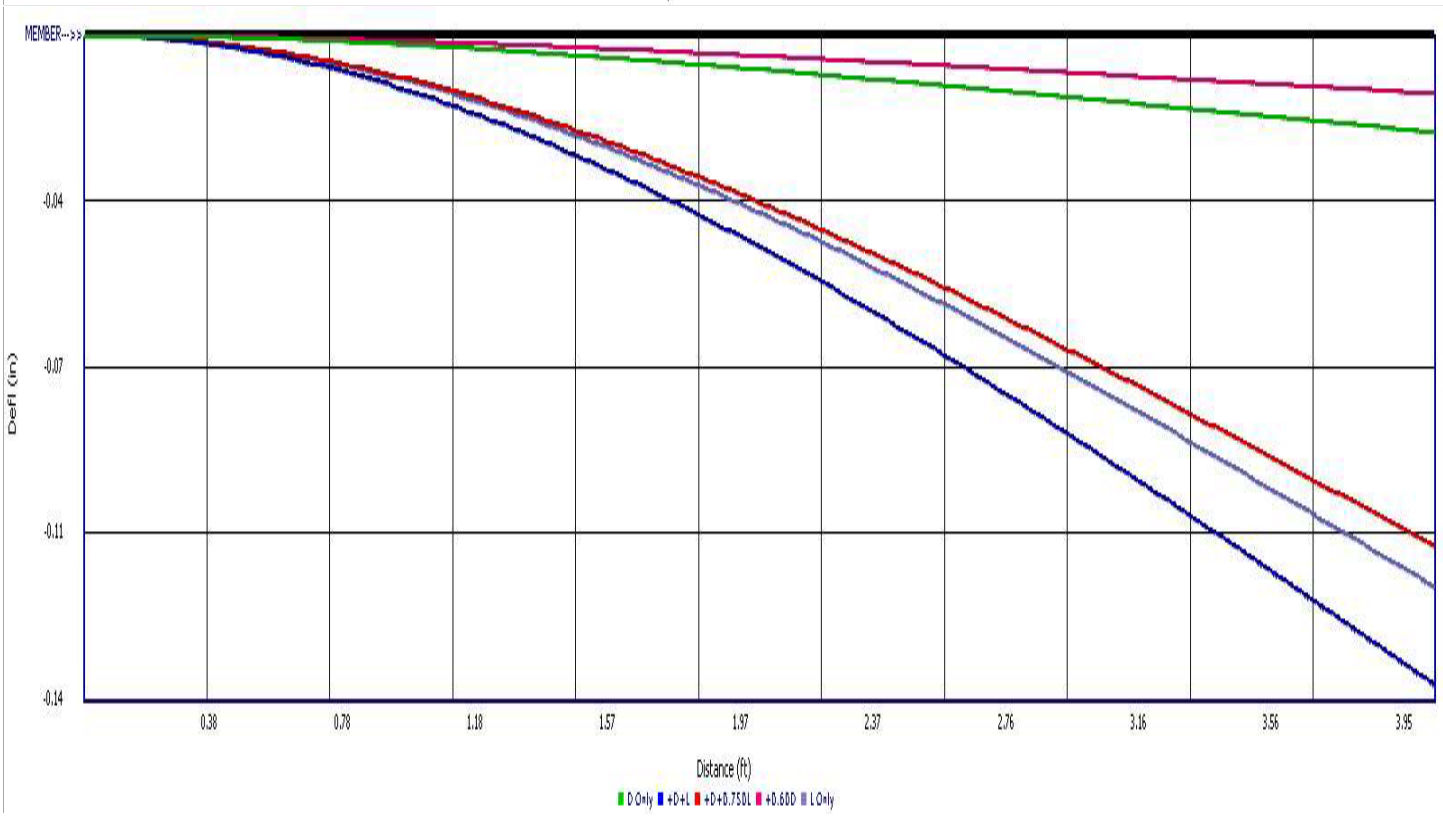
Steel Beam

Project File: 26-0651.ec6

DESCRIPTION: Combination #1 (DL+DLL Dist. + 200 Conc)





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.122	-0.250

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: 26-0651.ec6

LIC# : KW-06018068, Build:20.25.07.09 Eastern Engineering Group (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Combination #2 (DL Dist. + 300 Conc. + 200 Conc.)

Code References

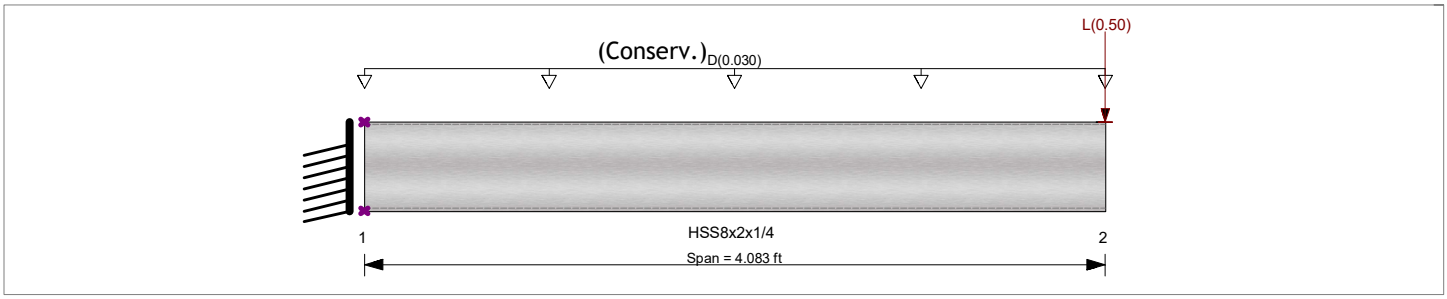
Governing Code : IBC 2018, CBC 2019
Referenced Design Standard(s) : AISC 360-22*
Load Combination Set : ASCE 7-22 / IBC 2024

*User-selected, not referenced by : IBC 2018, CBC 2019

Material Properties

Analysis Method : Allowable Strength Design
Beam Bracing : Completely Unbraced
Bending Axis : Minor Axis Bending

Fy : Steel Yield : 46.0 ksi
E: Modulus : 29,000.0 ksi



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight NOT internally calculated and added
Uniform Load : D = 0.030 ksf, Tributary Width = 1.0 ft
Point Load : L = 0.50 k @ 4.083 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.311 : 1	Maximum Shear Stress Ratio =	0.062 : 1
Section used for this span	HSS8x2x1/4	Section used for this span	HSS8x2x1/4
Ma : Applied	2.292 k-ft	Va : Applied	0.6225 k
Mn / Omega : Allowable	7.359 k-ft	Vn/Omega : Allowable	10.020 k
Load Combination	+D+L	Load Combination	+D+L
		Location of maximum on span	0.000 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.229 in	Ratio =	427	>=360	Span: 1 : L Only
Max Upward Transient Deflection	0 in	Ratio =	0	<360	n/a
Max Downward Total Deflection	0.251 in	Ratio =	391	>=360.	Span: 1 : +D+L
Max Upward Total Deflection	0 in	Ratio =	0	<360.0	n/a

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	Summary of Moment Values Mmax +	Mmax -	Ma Max	Mny	Mny/Omega	Cb	Rm	Summary of Shear Values Va Max	Vny	Vny/Omega
D Only													
Dsgn. L = 4.08 ft	1	0.034	0.012		-0.25	0.25	12.29	7.36	1.00	1.00	0.12	16.73	10.02
+D+L													
Dsgn. L = 4.08 ft	1	0.311	0.062		-2.29	2.29	12.29	7.36	1.00	1.00	0.62	16.73	10.02
+D+0.750L													
Dsgn. L = 4.08 ft	1	0.242	0.050		-1.78	1.78	12.29	7.36	1.00	1.00	0.50	16.73	10.02
+0.60D													
Dsgn. L = 4.08 ft	1	0.020	0.007		-0.15	0.15	12.29	7.36	1.00	1.00	0.07	16.73	10.02

Overall Maximum Deflections

Span	Load Combination	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
1	+D+L	0.2505	4.083		0.0000	0.000

Maximum Deflections for Load Combinations

Load Combination	Span	Max. Downward Defl	Location in Span	Span	Max. Upward Defl	Location in Span
D Only	1	0.0211	4.083		0.0000	0.000
+D+L	1	0.2505	4.083		0.0000	0.000
+D+0.750L	1	0.1932	4.083		0.0000	0.000
+0.60D	1	0.0126	4.083		0.0000	0.000
L Only	1	0.2294	4.083		0.0000	0.000

Project Title:
Engineer:
Project ID:
Project Descr:

Steel Beam

Project File: 26-0651.ec6

LIC# : KW-06018068, Build:20.25.07.09 Eastern Engineering Group (c) ENERCALC, LLC 1982-2026

DESCRIPTION: Combination #2 (DL Dist. + 300 Conc. + 200 Conc.)

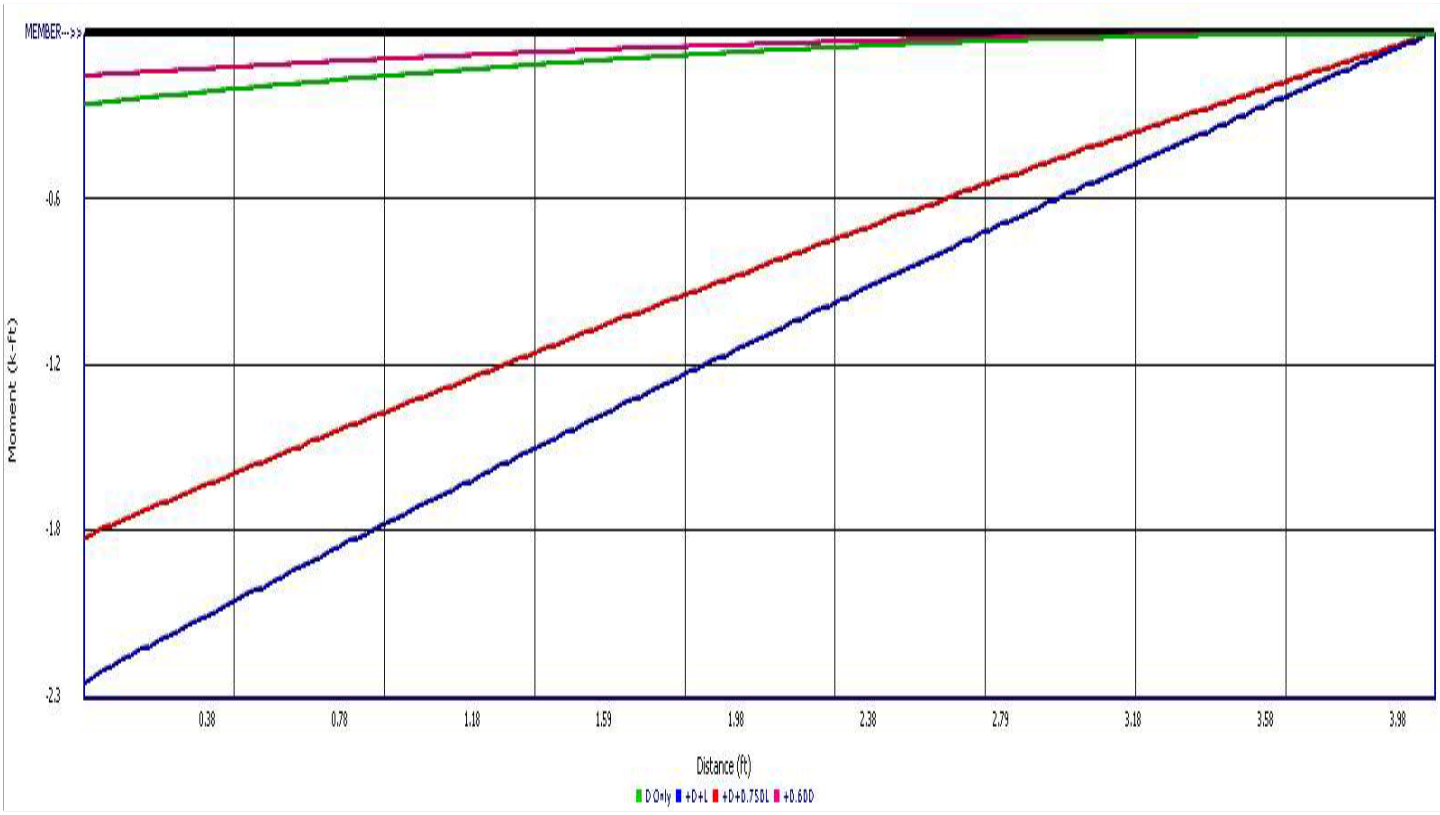
Vertical Reactions

Support notation : Far left is #' Values in KIPS

Load Combination	Support 1	Support 2
Max Upward from all Load Conditions	0.622	
Max Upward from Load Combinations	0.622	
Max Upward from Load Cases	0.500	
D Only	0.122	
+D+L	0.622	
+D+0.750L	0.497	
+0.60D	0.073	
L Only	0.500	

Steel Section Properties : HSS8x2x1/4

Depth	=	8.000	in	I xx	=	28.50	in^4	J	=	9.360 in^4
				S xx		7.12	in^3	Cw	=	6.35 in^6
Width	=	2.000	in	R xx	=	2.570	in			
Wall Thick	=	0.233	in	Zx	=	9.680	in^3			
Area	=	4.300	in^2	I yy	=	2.940	in^4	C	=	0.000 in^3
Weight	=	15.620	plf	S yy	=	2.940	in^3			
				R yy	=	0.827	in			
				Zy	=	3.430	in^3			
Ycg	=	4.000	in							



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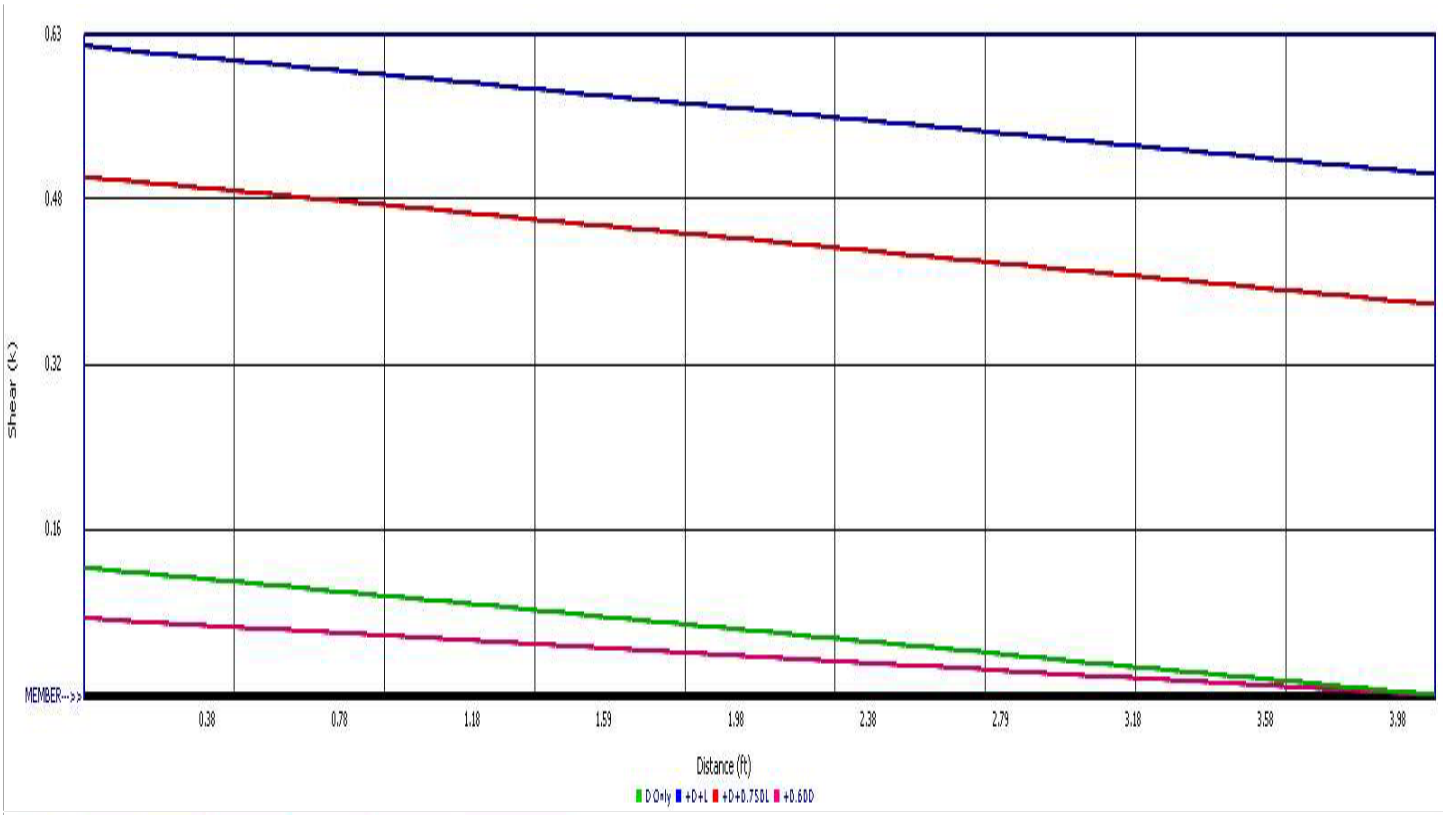
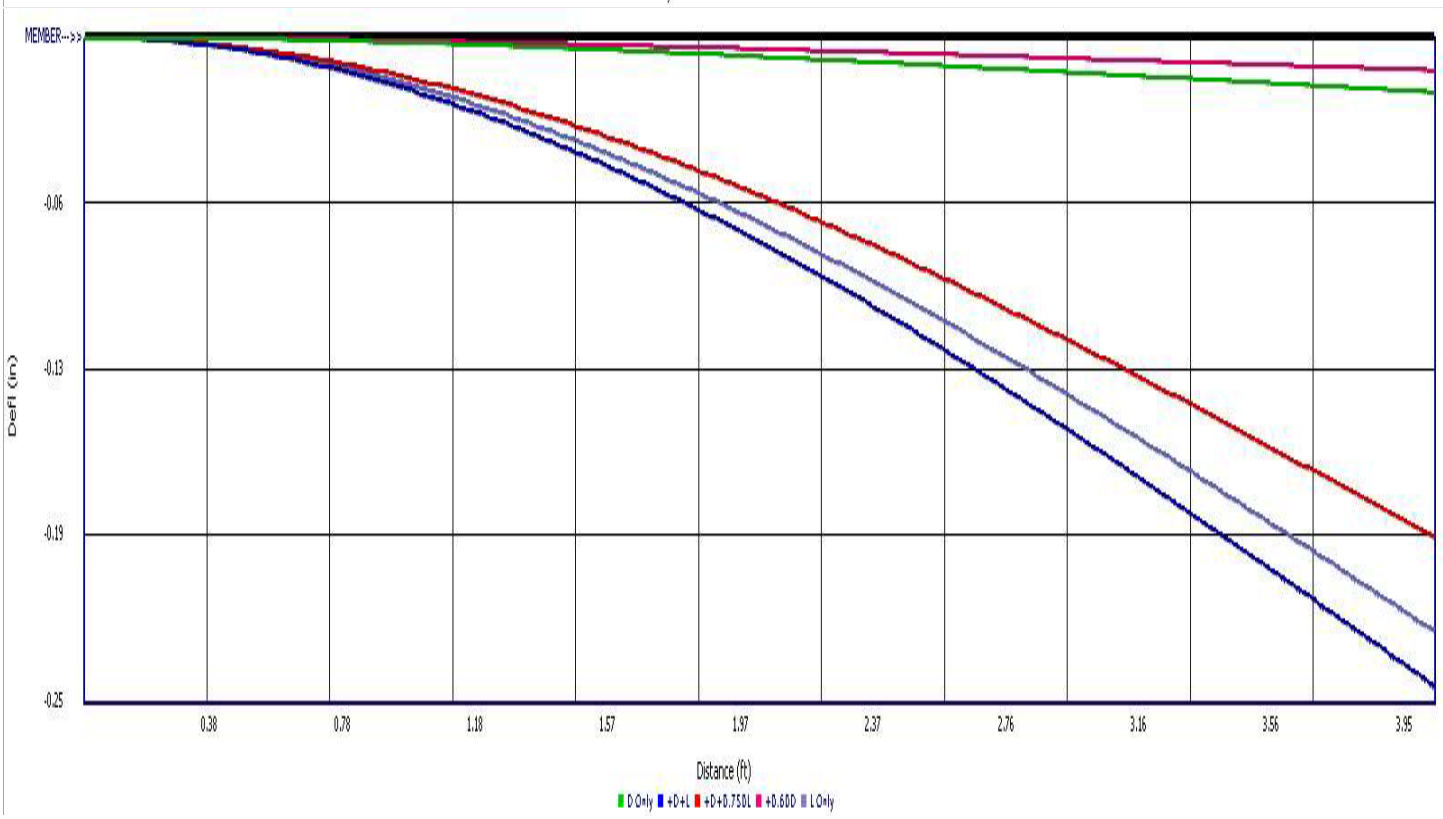
Project ID:
Project Descr:

Steel Beam

Project File: 26-0651.ec6

DESCRIPTION: Combination #2 (DL Dist. + 300 Conc. + 200 Conc.)





Beam Span Moments & Shears at Incremental Locations

Load Type/ Combination	Span Location (ft)	Span ID	Shear (k)	Moment (ft-k)
D Only	0.00	Span 1	0.122	-0.250

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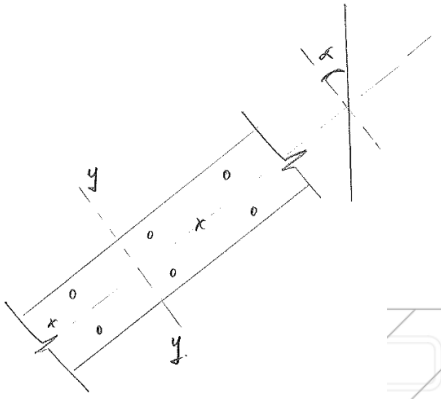
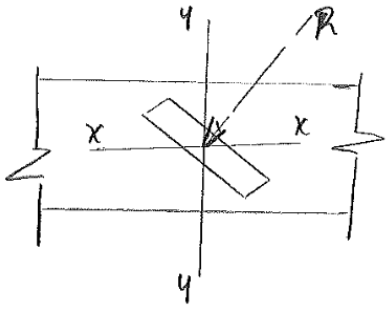
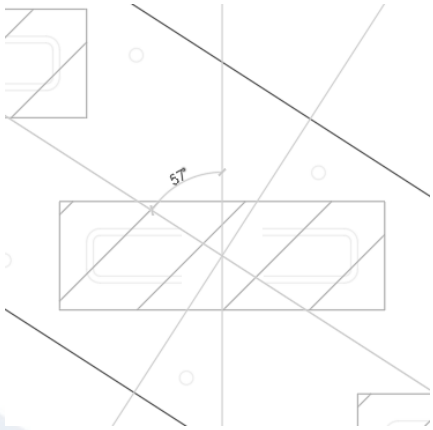
PLATE CONN. AT CONC. WALL



Loads:

Vmax= 0.6225= 622.5#
Mmax= 2.292 k-ft= 27 504#-in
α= 55^0

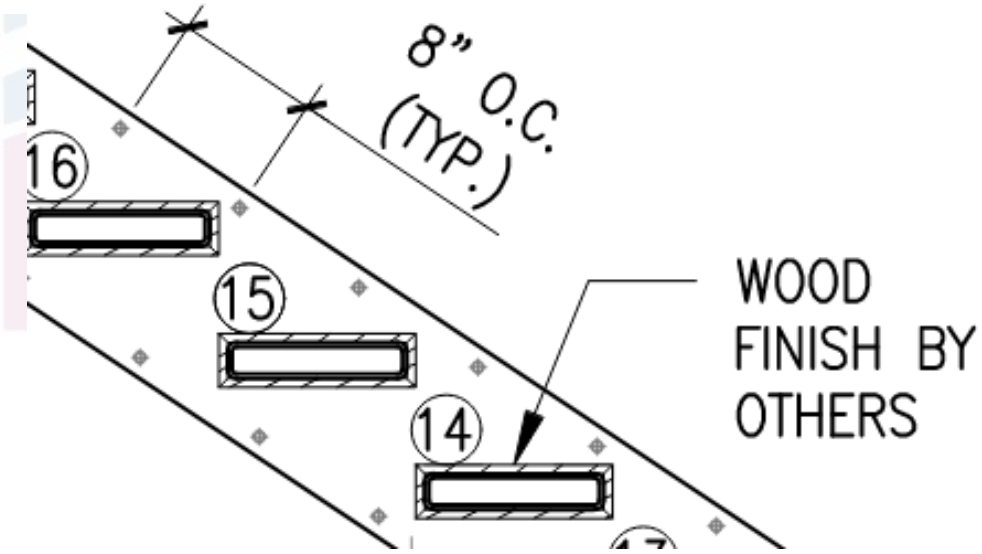




Loads Processing:

Mx=Mmax.*cos55^0
Mx= 27 504#-in* cos55^0
Mx= 15 775.64 #-in

My=Mmax.*sen55^0
My= 27 504#-in* sen55^0
My= 22 529.95#-in



ASD
Fy= 622.5#*cos55^0
Fy= 357.05#

Fx= 622.5#*sen55^0
Fx= 509.92#

Fz= 200# (railing reaction)

LRFD
Fy= 572#
Fx= 816#
Fz= 320#
Mx= 25 242#-in
My= 36 048#-in

*See next page spreadsheet printout for connection analysis

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

Specifier's comments:

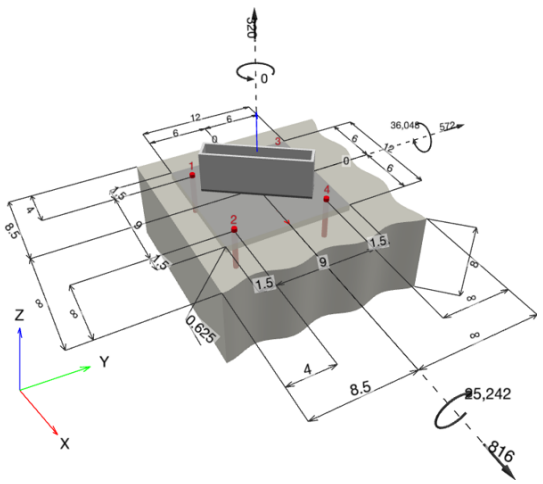
1 Anchor Design

1.1 Input data

Anchor type and diameter:	**Embed plate D1.1 GR. B 1/2**
Item number:	not available
Specification text:	∅ 1/2 IN EMBED PLATE D1.1 GR. B WITH 4 IN NOMINAL EMBEDMENT DEPTH PER TECHNICAL DATA , CAST IN PLACE INSTALLATION PER MPII
Effective embedment depth:	h_{ef} = 4.000 in.
Material:	
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Shear edge breakout verification:	Row closest to edge (Case 3 only from ACI 318-19 Fig. R.17.7.2.1b)
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.625 in.
Anchor plate[CBFEM] :	l_x x l_y x t = 12.000 in. x 12.000 in. x 0.625 in.;
Profile:	Rectangular HSS (AISC), HSS10X2X.250; (L x W x T) = 10.000 in. x 2.000 in. x 0.250 in.
Base material:	cracked concrete, 2000, f_c' = 2,000 psi; h = 8.000 in.
Reinforcement:	tension: not present, shear: not present; edge reinforcement: none or < No. 4 bar

[CBFEM] - The anchor calculation is based on a component-based Finite Element Method (CBFEM)

Geometry [in.] & Loading [lb, in.lb]

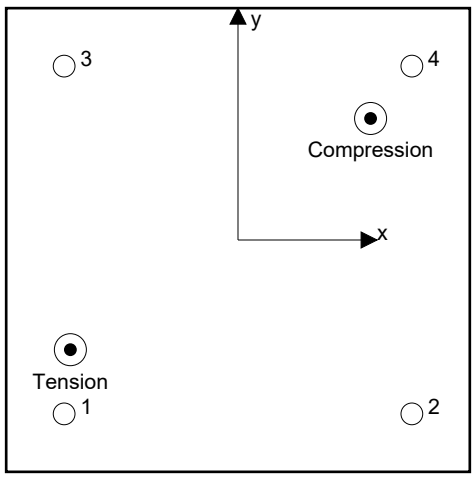


Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

1.1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 320; V_x = 816; V_y = 572; M_x = -25,242; M_y = 36,048; M_z = 0;	no	80

1.2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	3,709	260	211	152
2	83	243	200	138
3	858	243	200	137
4	-1	250	204	144

Resulting tension force in (x/y)=(-4.339/-2.839): 4,650 [lb]
Resulting compression force in (x/y)=(3.432/3.145): 4,465 [lb]

Anchor forces are calculated based on a component-based Finite Element Method (CBFEM)

1.3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	3,709	9,555	39	OK
Pullout Strength*	3,709	6,597	57	OK
Concrete Breakout Failure**	4,651	5,838	80	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	3
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

1.3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)
$\phi N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.20	65,000

Calculations

N_{sa} [lb]
12,740

Results

N_{sa} [lb]	ϕ_{steel}	ϕN_{sa} [lb]	N_{ua} [lb]
12,740	0.750	9,555	3,709

1.3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)
$N_p = 8 A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)
$\phi N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.59	1.000	2,000

Calculations

N_p [lb]
9,424

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
9,424	0.700	6,597	3,709

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	4
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

1.3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ — ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ — ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ — ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.000	2.839	1.339	4.000	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
-	24	1.000	2,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
280.00	144.00	0.679	0.818	0.900	1.000	8,586

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
8,340	0.700	5,838	4,651

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	5
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

1.4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	260	8,281	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	996	27,123	4	OK
Concrete edge failure in direction y-**	816	5,950	14	OK

* highest loaded anchor **anchor group (relevant anchors)
When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.4.1 Steel Strength

$V_{sa} = A_{se,V} f_{uta}$ ACI 318-19 Eq. (17.7.1.2a)
$\phi V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.20	65,000

Calculations

V_{sa} [lb]
12,740

Results

V_{sa} [lb]	ϕ_{steel}	ϕV_{sa} [lb]	V_{ua} [lb]
12,740	0.650	8,281	260

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 e'_N}{3 h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	4.000	0.000	0.000	4.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	-	24	1.000	2,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
361.00	144.00	1.000	1.000	0.900	1.000	8,586

Results

V_{cpg} [lb]	$\phi_{concrete}$	ϕV_{cpg} [lb]	V_{ua} [lb]
38,747	0.700	27,123	996

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.4.3 Concrete edge failure in direction y-

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V} \psi_{ed,V} \psi_{c,V} \psi_{h,V} \psi_{parallel,V} V_b$ ACI 318-19 Eq. (17.7.2.1b)

$\phi V_{cbg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)*

$A_{Vc0} = 4.5 c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e'_v}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f'_c} c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
4.000	4.000	0.000	1.000	8.000

l_e [in.]	λ_a	d_a [in.]	f'_c [psi]	$\psi_{parallel,V}$
4.000	1.000	0.500	2,000	2.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
114.00	72.00	1.000	1.000	1.000	2,684

Results

V_{cbg} [lb]	$\phi_{concrete}$	ϕV_{cbg} [lb]	V_{ua} [lb]
8,500	0.700	5,950	816

*Anchor row defined by: Anchor 1, 2; Case 3 controls

When the input edge distance is set to "infinity", edge breakout verification is not performed in that direction

1.5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.797	0.137	5/3	73	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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1.6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, ACI318, IS1946, AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- The equations presented in this report are based on imperial units. When inputs are displayed in metric units, the user should be aware that the equations remain in their imperial format.
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://viewer.joomag.com/profis-design-guide-us-en-summer-2021/0841849001625154758?short&/
- Attention! In case of compressive anchor forces a buckling check as well as the proof of the local load transfer into and within the base material (incl. punching) has to be done separately.
- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (EN1992-4, AS5216, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

1.7 Installation data

Profile: Rectangular HSS (AISC), HSS10X2X.250; (L x W x T) = 10.000 in. x 2.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 0.562 in.

Plate thickness (input): 0.625 in.

Anchor type and diameter: Embed plate D1.1 GR. B 1/2

Item number: not available

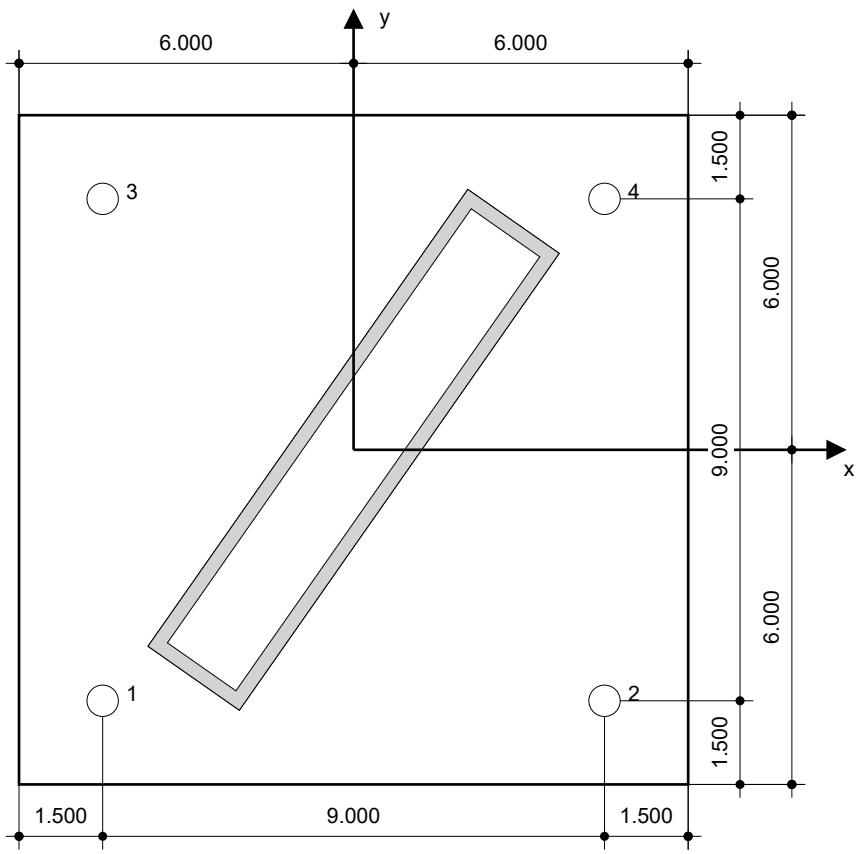
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 4.000 in.

Minimum thickness of the base material: 4.813 in.

⌀ 1/2 in Embed plate D1.1 GR. B with 4 in nominal embedment depth per Technical data , cast in place installation per MPII



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-4.500	-4.500	4.000	-	4.000	-
2	4.500	-4.500	13.000	-	4.000	-
3	-4.500	4.500	4.000	-	13.000	-
4	4.500	4.500	13.000	-	13.000	-

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2 Anchor plate design

2.1 Input data

Anchor plate:	Shape: Rectangular l_x x l_y x t = 12.000 in x 12.000 in x 0.625 in Calculation: CBFEM Material: ASTM A36; F_y = 36,000 psi; ε_{lim} = 5.00%
Anchor type and size:	Embed plate D1.1 GR. B 1/2, h_{ef} = 4.000 in
Anchor stiffness:	The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.
Design method:	AISC and LRFD-based design using component-based FEM
Stand-off installation:	e_b = 0.000 in (No stand-off); t = 0.625 in
Profile:	HSS10X2X.250; (L x W x T x FT) = 10.000 in x 2.000 in x 0.250 in x - Material: ASTM A500 Gr.B Rect; F_y = 46,000 psi; ε_{lim} = 5.00% Eccentricity x: 0.000 in Eccentricity y: 0.000 in
Base material:	Cracked concrete; 2000; $f_{c,cyl}$ = 2,000 psi; h = 8.000 in
Welds (profile to anchor plate):	Type of redistribution: Plastic Material: E70xx
Mesh size:	Number of elements on edge: 8 Min. size of element: 0.394 in Max. size of element: 1.969 in

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Company:		Page:	11
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

2.2 Summary

	Description	Profile		Anchor plate			Concrete [%]
		σ_{Ed} [psi]	ε_{Pl} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Hole bearing [%]	
1	Combination 1	14,439	0.00	13,403	0.00	1	7

2.3 Anchor plate classification

Results below are displayed for the decisive load combinations: Combination 1

Anchor tension forces	Equivalent rigid anchor plate (CBFEM)	Component-based Finite Element Method (CBFEM) anchor plate design
Anchor 1	3,033 lb	3,709 lb
Anchor 2	468 lb	83 lb
Anchor 3	1,267 lb	858 lb
Anchor 4	-1 lb	-1 lb

User accepted to consider the selected anchor plate as rigid by his/her engineering judgement. This means the anchor design guidelines can be applied.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.4 Profile/Stiffeners/Plate

Profile and stiffeners are verified at the level of the steel to concrete connection. The connection design does not replace the steel design for critical cross sections, which should be performed outside of PROFIS Engineering.

2.4.1 Equivalent stress and plastic strain

Part	Load combination	Material	f_y [psi]	ε_{lim} [%]	σ_{Ed} [psi]	ε_{Pl} [%]	Status
Plate	Combination 1	ASTM A36	36,000	5.00	13,403	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	14,439	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	9,033	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	8,698	0.00	OK
Profile	Combination 1	ASTM A500 Gr.B Rect	46,000	5.00	10,681	0.00	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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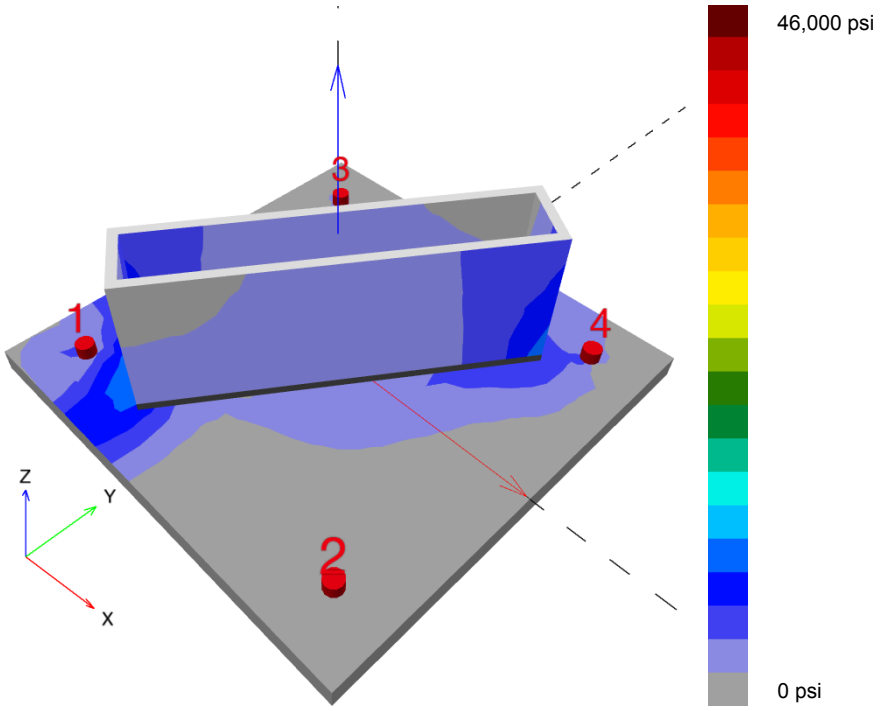
Company:		Page:	13
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

2.4.1.1 Equivalent stress

Results below are displayed for the decisive load combination: 1 - Combination 1



Input data and results must be checked for conformity with the existing conditions and for plausibility!
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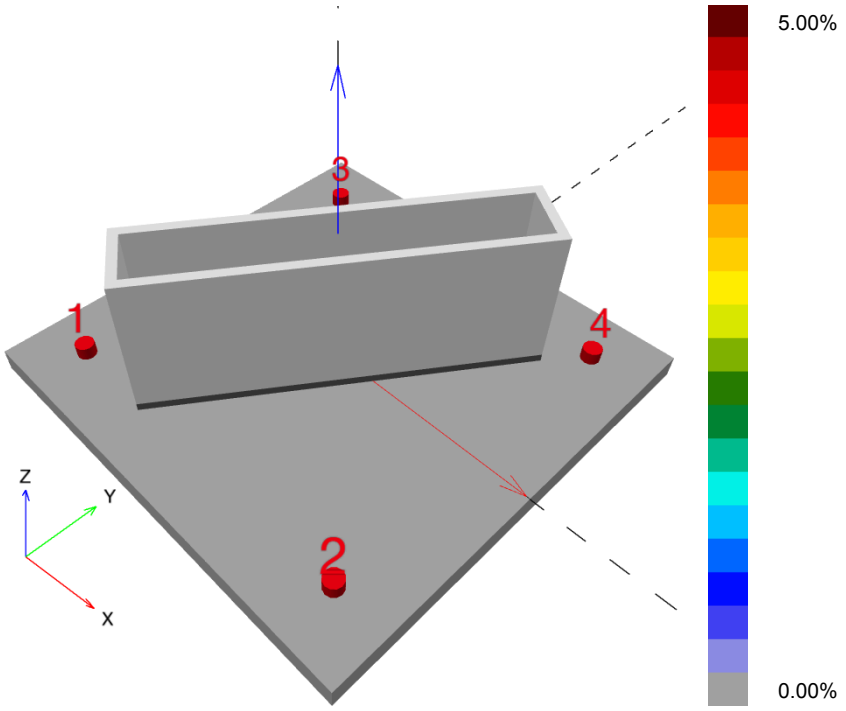
2.4.1.2 Plastic strain

Results below are displayed for the decisive load combination: 1 - Combination 1



Company:		Page:	15
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

2.4.2 Plate hole bearing resistance, AISC 360-16 Section J3

Decisive load combination: 1 - Combination 1

Equations

$R_n = \min(1.2\, l_c\, t\, F_u\, ,\ 2.4\, d\, t\, F_u)$ (AISC 360-16 J3-6a, c)

$\phi R_n = 0.75\, R_n$

$V \leq \phi R_n$

Variables

	l_c [in]	t [in]	F_u [psi]	d [in]	R_n [lb]
Anchor 1	1.568	0.625	58,000	0.500	43,514
Anchor 2	2.359	0.625	58,000	0.500	43,514
Anchor 3	1.537	0.625	58,000	0.500	43,514
Anchor 4	12.572	0.625	58,000	0.500	43,514

Results

	V [lb]	ϕR_n [lb]	Utilization [%]	Status
Anchor 1	260	32,635	1	OK
Anchor 2	243	32,635	1	OK
Anchor 3	243	32,635	1	OK
Anchor 4	250	32,635	1	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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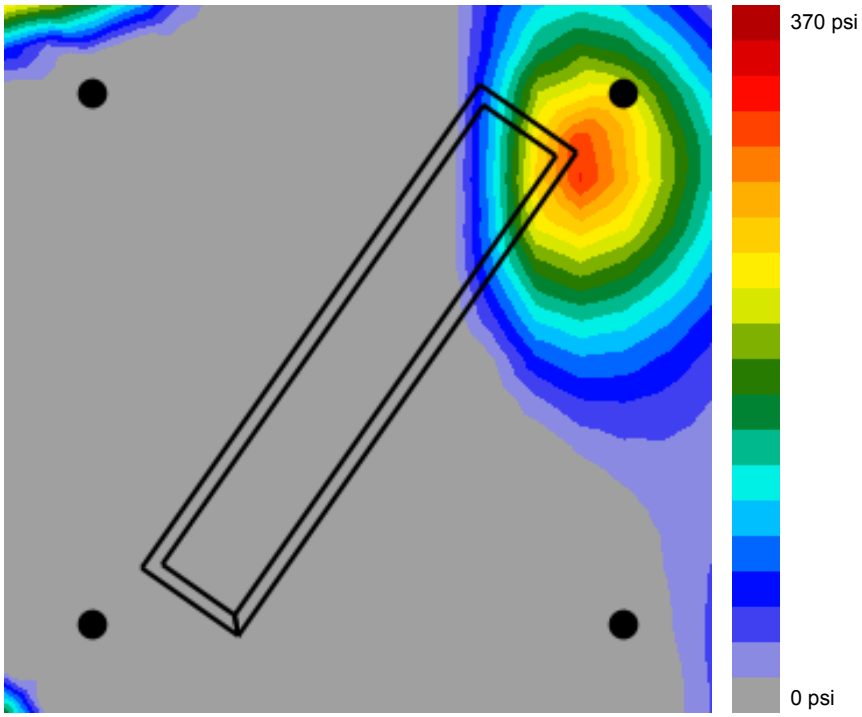
2.5 Concrete

Decisive load combination: 1 - Combination 1

2.5.1 Compression in concrete under the anchor plate



2.5.2 Concrete block compressive strength resistance check, AISC 360-16 Section J8

Equations

$F_p = \phi f_{p,max}$

$f_{p,max} = 0.85 f_c' \sqrt{(\frac{A_2}{A_1})} \le 1.7 f_c; \sqrt{(\frac{A_2}{A_1})} \le 2$

$\sigma = \frac{N}{A_1}$

Utilization $= \frac{\sigma}{F_p}$

Company:		Page:	17
Address:		Specifier:	
Phone I Fax:		E-Mail:	
Design:	Drafts_Concrete - Apr 16, 2026 (1)	Date:	6/18/2026
Fastening point:			

Variables

N [lb]	f_c' [psi]	ϕ	A_1 [in²]	A_2 [in²]
4,465	2,000	0.65	32.69	126.96

Results

Load combination	F_p [psi]	σ [psi]	Utilization [%]	Status
Combination 1	2,178	137	7	OK

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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2.6 Symbol explanation

A_1	Loaded area of concrete
A_2	Supporting area
d	Nominal diameter of the bolt
ε_{lim}	Limit plastic strain
ε_{Pl}	Plastic strain from CBFEM results
f_c	Concrete compressive strength
f_c'	Concrete compressive strength
F_u	Specified minimum tensile strength of the connected material
F_p	Concrete block design bearing strength
$f_{p,max}$	Concrete block design bearing strength maximum
f_y	Yield strength
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material
N	Resulting compression force
σ	Average stress in concrete
σ_{Ed}	Equivalent stress
ϕ	Resistance factor
ϕR_n	Factored resistance
t	Thickness of the anchor plate
V	Resultant of shear forces Vy, Vz in bolt.

2.7 Warnings

- By using the CBFEM calculation functionality of PROFIS Engineering you may act outside the applicable design codes and your specified anchor plate may not behave rigid. Please, validate the results with a professional designer and/or structural engineer to ensure suitability and adequacy for your specific jurisdiction and project requirements.
- The anchor is modeled considering stiffness values determined from load displacement curves tested in an independent laboratory. Please note that no simple replacement of the anchor is possible as the anchor stiffness has a major impact on the load distribution results.

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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3 Summary of results

Design of the anchor plate, anchors, welds and other elements are based on CBFEM (component based finite element method) and AISC.

	Load combination	Max. utilization	Status
Anchors	Combination 1	80%	OK
Anchor plate	Combination 1	38%	OK
Concrete	Combination 1	7%	OK
Profile	Combination 1	32%	OK

Fastening meets the design criteria!

Input data and results must be checked for conformity with the existing conditions and for plausibility!
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Weld Design For Tube

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$b := 8.00$ Width of Tube Steel (in)

$d := 2.00$ Depth of Tube Steel (in)

Load Data:

$M_{weld} := 2.292$ Moment (Kips-ft)

$V_{weld} := 0.6225$ Shear (Kips)

$P_{weld} := 0.200$ Tension or Compression (Kips)

$T_{weld} := 0.00$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $F_v = 21.00$ ksi

$$t_e := \left| \begin{array}{l} x \leftarrow 0.00001 \\ \text{while } \sqrt{\left[\frac{P_{weld}}{2(b+d)\cdot x} + \frac{M_{weld}\cdot 12}{\left(b\cdot d + \frac{d^2}{3}\right)\cdot x}\right]^2 + \left[\frac{1.5V_{weld}}{2(b+d)\cdot x} + \frac{T_{weld}\cdot 12\cdot\sqrt{\left(\frac{b}{2}\right)^2+\left(\frac{d}{2}\right)^2}}{\frac{(b+d)^3}{6}\cdot x}\right]^2} \geq F_v \\ x \leftarrow x + 0.01 \end{array} \right.$$

$t_e = 0.08$ in

$S_{weld} := \left(b\cdot d + \frac{d^2}{3}\right)\cdot t_e$ $S_{weld} = 1.39$ in^3

$A_{weld} := 2\cdot(d+b)\cdot t_e$ $A_{weld} = 1.60$ in^2

$J_{weld} := \frac{(b+d)^3}{6}\cdot t_e$ $J_{weld} = 13.33$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$

$f_b = 19.83$ ksi

Check Shear Stress :

$$f_v := \frac{1.5 V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot \sqrt{\left(\frac{b}{2}\right)^2 + \left(\frac{d}{2}\right)^2}}{J_{weld}}$$

$f_v = 0.58$ ksi

Check Tension Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$

$f_a = 0.12$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^2}$$

$f_{weld} = 19.97$ ksi

$$STRESS := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{cases}$$

STRESS = "OK"

$$W_{weld} := \begin{cases} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{cases}$$

$W_{weld} = \text{"3/16"}$

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GLASS RAILING AT STAIR DESIGN

Glass Effective Laminated Thickness

Laminate Glass Data:

$E := 10400ksi$ Young Modulus of Elasticity of Glass E MPa
For Soda Lime Glass: E=10 400 Ksi (71705.5 MPa)

$h_v := 0.06in$ Interlayer Thickness (in)

$h_1 := 0.219 \cdot in$ Glass Ply 1 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$h_2 := 0.219 \cdot in$ Glass Ply 2 Minimum Thickness (in) (See ASTM E1300-09a Table 4)

$a := 40in$ Length Scale (Smallest in-Plane Dimension of the Laminate Plate (in)

$G := 3830psi$ Interlayer Storage Shear Modulus G=3830 psi (For Wind 3s/50 Celcius)
For Exterior: SentryGlass 1,640 psi (11.3 MPa) PVB 70 psi (0.48 MPa)
For Interior: SentryGlass 3830 psi (26.4 MPa) PVB 140 psi (0.96 MPa)

Then

$$h_s := 0.5 \cdot (h_1 + h_2) + h_v \qquad h_s = 0.279 \cdot in$$

$$h_{s.1} := \frac{h_s \cdot h_1}{h_1 + h_2} \qquad h_{s.1} = 0.140 \cdot in$$

$$h_{s.2} := \frac{h_s \cdot h_2}{h_1 + h_2} \qquad h_{s.2} = 0.140 \cdot in$$

$$I_s := h_1 \cdot h_{s.2}^2 + h_2 \cdot h_{s.2}^2 \qquad I_s = 0.009 \cdot in \cdot in \cdot in$$

$$\Gamma := \frac{1}{1 + \frac{9.6 \cdot E \cdot I_s \cdot h_v}{G \cdot h_s^2 \cdot a^2}} \qquad \Gamma = 0.903$$

Laminate Effective Thickness For Deflection

$$h_{efw} := \sqrt[3]{h_1^3 + h_2^3 + 12 \cdot \Gamma \cdot I_s} \qquad h_{efw} = 0.484 \cdot in$$

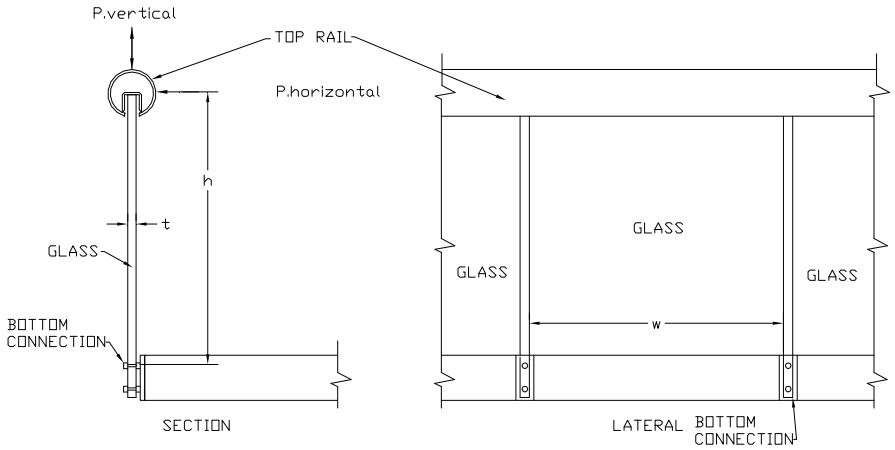
Laminate Effective Thickness For Bending Stress

$$h_{1.ef\sigma} := \sqrt{\frac{h_{efw}^3}{\max(h_1, h_2) + 2 \cdot \Gamma \cdot \max(h_{s.1}, h_{s.2})}} \qquad h_{1.ef\sigma} = 0.491 \cdot in$$

TABLE 4 Nominal and Minimum Glass Thicknesses

Nominal Thickness or Designation, mm (in.)	Minimum Thickness, mm (in.)
2.5 (3⁄32)	2.16 (0.085)
2.7 (lami)	2.59 (0.102)
3.0 (1⁄8)	2.92 (0.115)
4.0 (5⁄32)	3.78 (0.149)
5.0 (3⁄16)	4.57 (0.180)
6.0 (1⁄4)	5.56 (0.219)
8.0 (5⁄16)	7.42 (0.292)
10.0 (3⁄8)	9.02 (0.355)
12.0 (1⁄2)	11.91 (0.469)
16.0 (5⁄8)	15.09 (0.595)
19.0 (3⁄4)	18.26 (0.719)
22.0 (7⁄8)	21.44 (0.844)
25.0 (1)	24.61 (0.969)

Glass Railing Design w/ Standoff Support



Loads Data:

$P_{200} := 200.00$ Concentrated Load (lbs)

$q_{50} := 50.00$ Uniform Load (plf)

$q_{wind} := 5.00$ Uniform Distributed Load (psf)

Glass Data:

$E := 10400000.00$ Modulus of Elasticity of Glass (psi)

$Mr_{flexure} := 24000.00$ Modulus of Rupture of Glass in Flexure For Live Loads (psi)

$F_{b.wind} := 9600.00$ Allowable Stress in Flexure For Wind Loads Per ASTM E1300 (psi)

$Mr_{shear} := 12000.00$ Modulus of Rigity of glass in Shear (psi)

$SF := 4.00$ Safety Factor

Geometric Glass Railing Data:

$h := 44.0$ Height of Glass Lite Pannel in Cantilever (in)

$t_s := 0.491$ Effective Thickness of Glass Pannel for Stress (in) Per ASTM E1300 Appendix X9

$t_d := 0.484$ Effective Thickness of Glass Pannel for Deflection (in) Per ASTM E1300 Appendix X9

$w_{min} := 40.0$ **Minimum** Width of Glass Pannel (in)

$w_{max} := 60.00$ **Maximum** Width of Glass Pannel (in)

Then

$$F_{b.live} := \frac{Mr_{flexure}}{SF}$$ $F_{b.live} = 6000.00$ psi

$$F_v := \frac{Mr_{shear}}{SF}$$ $F_v = 3000.00$ psi

$$L := \begin{cases} h \\ w_{min} \text{ if } w_{min} < h \end{cases}$$ $L = 40.00$ in

$$S_x := \frac{L \cdot t_s^2}{6}$$ $S_x = 1.61$ in^3

$$I_x := \frac{L \cdot t_s^3}{12}$$ $I_x = 0.39$ in^4

$$A := L \cdot t_s$$ $A = 19.64$ in^2

Actual Glass Moment in Length (L):

Concentrated Load = 200 lbs.

$$M_{200} := P_{200} \cdot h$$ $M_{200} = 8800.00$ lbs – in

Uniform Load = 50 plf

$$M_{50} := q_{50} \cdot \frac{L}{12} \cdot h$$ $M_{50} = 7333.33$ lbs – in

Uniform Distributed Wind Load

$$M_{wind} := \frac{q_{wind}}{144} \cdot L \cdot \frac{h^2}{2}$$ $M_{wind} = 1344.44$ lbs – in

$$M_{Glass.live} := \max(M_{200}, M_{50})$$ $M_{Glass.live} = 8800.00$ lbs – in

$$M_{Glass.wind} := M_{wind}$$ $M_{Glass.wind} = 1344.44$ lbs – in

$$M_{Glass} := \max(M_{Glass.live}, M_{Glass.wind})$$ $M_{Glass} = 8800.00$ lbs – in

Actual Glass Shear in Length (L):

Concentrated Load = 200 lbs.

$$V_{200} := P_{200}$$ $V_{200} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50} := q_{50} \cdot \frac{L}{12}$$ $V_{50} = 166.67$ lbs

Uniform Distributed Wind Load

$$V_{wind} := \frac{q_{wind}}{144} \cdot L \cdot h$$ $V_{wind} = 61.11$ lbs

$$V_{Glass} := \max\left(V_{200}, V_{50}, V_{wind}\right)$$ $V_{Glass} = 200.00$ lbs

Section Required:

Bending Design:

Section Modulus Required

$$Sx_r := \max\left(\frac{M_{Glass.live}}{F_{b.live}}, \frac{M_{Glass.wind}}{F_{b.wind}}\right)$$ $Sx_r = 1.47$ in^3

Shear Design:

Area Required

$$A_r := \frac{V_{Glass}}{F_v}$$ $A_r = 0.07$ in^2

Section Provided:

$$BENDING_{glass} := \text{if}\left(Sx_r \geq \min\left(S_x\right), \text{"N.G"}, \text{"OK"}\right)$$ $BENDING_{glass} = \text{"OK"}$

$$SHEAR_{glass} := \text{if}\left(A_r \geq A, \text{"N.G"}, \text{"OK"}\right)$$ $SHEAR_{glass} = \text{"OK"}$

Check Deflection:

$$\Delta_{allow} := \frac{h}{30}$$ $\Delta_{allow} = 1.47$ in

$$I_d := \frac{L \cdot t_d^{\,3}}{12}$$ $I_d = 0.38$ in^4

$$\Delta_{200} := \frac{P_{200} \cdot h^3}{3E \cdot I_d}$$ $\Delta_{200} = 1.44$ in

$$\Delta_{wind} := \frac{\left(\frac{q_{wind}}{144}\right) L \cdot h^4}{8E \cdot I_d}$$ $\Delta_{wind} = 0.17$ in

$$DEFLECTION_{glass} := \begin{cases} \text{"N.G."} \\ \text{"O.K."} \quad \text{if } \max(\Delta_{200}, \Delta_{wind}) \le \Delta_{allow} \end{cases}$$ $DEFLECTION_{glass} = \text{"O.K."}$

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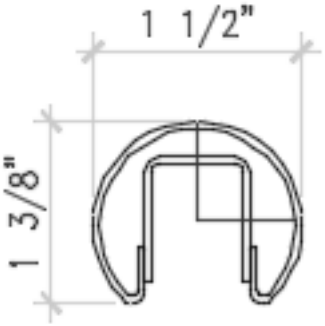
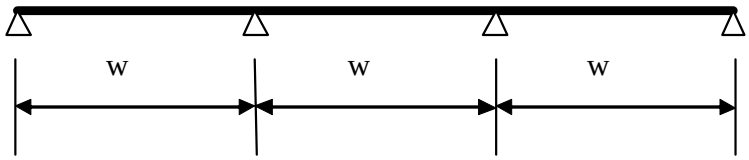
SECTION PROPERTIES



TOP RAIL

Area = 0.4003 in^2
Ixx = 0.0518 in^4
Iyy = 0.0765 in^4
Jz = 0.1283 in^4
Pxy = 0.0000 in^4
rxx = 0.3596 in
ryy = 0.4372 in
Sxx max = 0.0861 in^3
Sxx min = 0.0743 in^3
Syy max = 0.1020 in^3
Syy min = 0.1020 in^3



Top rail to provide redistribution of load between glass panels & to remain in place in case that one of the glass planes breaks

$F_{b.top} := 34800.00$ Top Railing Allowable Bending Stress (psi)

$F_{v.top} := 23200.00$ Top Railing Allowable Shear Stress (psi)

$Sx_{top} := 0.0743$ Top Railing Inertia Modulus for Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.1020$ Top Railing Inertia Modulus for Horizontal Loads $\left(in^3\right)$

$A_{top} := 0.4003$ Top Railing Area $\left(in^2\right)$

Actual Moment:

Concentrated Load = 200 lbs.

$$M_{200.top} := \frac{P_{200} \cdot w_{max}}{5}$$

$M_{200.top} = 2400.00$ lb – in

Uniform Load = 50 plf

$$M_{50.top} := 0.1012 \cdot \frac{q_{50}}{12} \cdot w_{max}^{2}$$

$M_{50.top} = 1518.00$ lb – in

$M_{actual.top} := \max\left(M_{200.top}, M_{50.top}\right)$

$M_{actual.top} = 2400.00$ lb – in

Actual Shear:

Concentrated Load = 200 lbs.

$$V_{200.top} := P_{200}$$

$V_{200.top} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{50.top} := 0.6 \cdot \frac{q_{50}}{12} \cdot w_{max}$$

$V_{50.top} = 150.00$ lbs

$$V_{actual.top} := \max\left(V_{200.top}, V_{50.top}\right)$$

$V_{actual.top} = 200.00$ lbs

Section Required:

Bending Design:

Section Modulus Required

$$Shr := \frac{M_{actual.top}}{F_{b.top}}$$

$Shr = 0.07$ in^3

Shear Design:

Area Required

$$Ahr := \frac{1.5 \cdot V_{actual.top}}{F_{v.top}}$$

$Ahr = 0.01$ in^2

Section Provided:

$$BENDING_{top} := \text{if}\left(Shr \geq \min\left(Sx_{top}, Sy_{top}\right), \text{"N.G"}, \text{"OK"}\right)$$

$BENDING_{top} = \text{"OK"}$

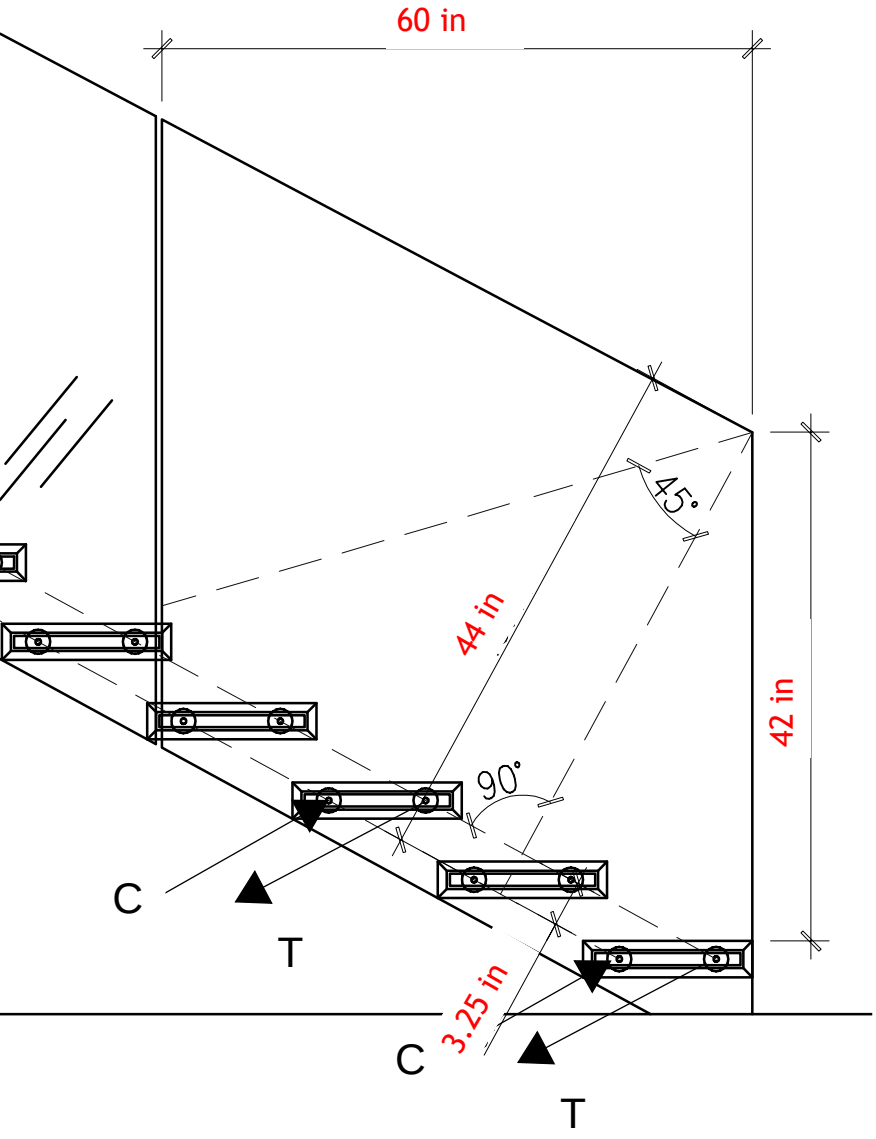
$$SHEAR_{top} := \text{if}\left(Ahr \geq A_{top}, \text{"N.G"}, \text{"OK"}\right)$$

$SHEAR_{top} = \text{"OK"}$

Stair Railing:

Loads:

- 200# any direction any location. (worst case scenario)
-50#/ft any direction any location.



Stand-offs two at each treads therefore there will be at least 4 pairs of stand-offs for the 200 pounds (AS A CONSERVATIVE ANALYSIS FOR DESIGN POURPOSES WE WILL ASSUME ONLY 2 PAIR OF STAND-OFF TO TAKE THE 200# ACTION), then the equivalent load per pair of stand-offs for the 50#/ft = 100# (assuming stand-off pairs spaced at 24" o.c.) therefore 200# will be the worst case scenario.

T per stand-off = 200# * (44 in) / (3.25 in * 2 pairs)

T per stand-off = 1353.84 #

Shear per stand-off = System Weight = 10 psf * 4ft * 5 ft / 2 stand-offs

Shear per stand-off = 120#

S.S Bolt Threaded Tension & Shear Stress Design 14th

Data:

$T_{bolt} := 1.43$ Tension in Anchor (kips) (Conservative) $\Omega := 2.25$ ASD Factor=2.00

$V_{bolt} := 0.120$ Shear in Anchor (kips) (Conservative)

$F_{nv} := 33.75$ Nominal Shear Strength (ksi) Table J3.2 AISC 16.1-120 .....=0.45*Fu

$F_{nt} := 53.25$ Nominal Tensile Strength (ksi) Table J3.2 AISC 16.1-120.....=0.75*Fu

$D := 0.375$ Screw Diameter (in) $n := 16.00$ Threads per in..... (AISC-Table 7-17 page 7-81)

$$F_v := \frac{F_{nv}}{\Omega}$$ $F_v = 15.000$ ksi

$$F_t := \frac{F_{nt}}{\Omega}$$ $F_t = 23.667$ ksi

$$A_{bolt} := 0.7854 \cdot \left(D - \frac{0.9743}{n} \right)^2$$ $A_{bolt} = 0.077$ in^2

Check Shear

$$f_v := \frac{V_{bolt}}{A_{bolt}}$$ $f_v = 1.549$ ksi

$V_{allow} := F_v \cdot A_{bolt}$ $V_{allow} = 1.162$ kips

$SHEAR := \text{if}\left(f_v \le F_v, \text{"OK"}, \text{"NG"} \right)$ SHEAR = "OK"

Check Tension

$$f_t := \frac{T_{bolt}}{A_{bolt}}$$ $f_t = 18.454$ ksi

$$F'_{nt} := \min\left(F_t, \max\left(0, 1.3F_{nt} - \Omega \cdot \frac{F_{nt}}{F_{nv}} \cdot f_v \right) \right)$$ $F'_{nt} = 23.667$ ksi

$T_{allow} := F'_{nt} \cdot A_{bolt}$ $T_{allow} = 1.834$ kips

$TENSION := \text{if}\left(T_{bolt} \le T_{allow}, \text{"OK"}, \text{"NG"} \right)$ TENSION = "OK"

Minimum Thread Length

$$TL_{min} := \frac{2 \cdot T_{bolt}}{F_v \cdot \pi \cdot \left(D - \frac{0.9743}{n} \right)}$$ $TL_{min} = 0.193$ in

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Minimum Thread Length Design

.

*for 3/8" Diam Bolt A307 Ttable= Available Tensile Strength * Gross Bolt Area=2.475kips

.

Available Tensile Strength =22.5kips (see attached table 7-2)

Gross Bolt Area=0.11 (see attached table 7-17)

.

Fu=60ksi (table 2-6 see attached)

Ltable=0.25" Nut Thread Length for Non High-Strength Bolts (see attached table 7-20)

Minimum Base Thickness for Bolt Connection Design AISC

Load Data:

$T_{bolt} := 1.472$ Tension in Anchor (kips) conservative

Bolt Data:

$D := 0.375$ Bolt Diameter (in)

$n := 16.00$ Threads per in..... (AISC-ASD Table 4-147)

Base Material Data:

$Fv_{base} := 18.40$ Base Material Allowable Shear Strength (ksi)

$Fu_{base} := 58.00$ Base Material Tensile Strength (ksi) by Table 2-3 or 2-4 page 2-39 or 2-40

AISC Equivalent Bolts Thread Length Data:

$T_{table} := 2.475$ Maximum Allowable Tension in Anchor by Table 7-2 page 7-23

$Fu_{table} := 60.00$ Maximun Tensile Strength (ksi) by Table 2-5 page 2-41

$L_{table} := 0.25$ Nut Thread Length (in) For High-Strength Bolts.....Table 7-15 page 7-80
For Non High-Strength Bolts.....Table 7-18 page 7-83

Minimum Thread Length (by Calculations)

$$TL_{calc} := \frac{2 \cdot T_{bolt}}{Fv_{base} \cdot \pi \cdot \left(D - \frac{0.9743}{n} \right)}$$

$TL_{calc} = 0.162$ in

Minimum Thread Length (by Equivalent AISC Standard)

$$TL_{equivalent} := L_{table} \cdot \frac{T_{bolt}}{T_{table}} \cdot \frac{Fu_{table}}{Fu_{base}}$$

$TL_{equivalent} = 0.154$ in

Minimum Thread Length

$TL_{min} := \max(TL_{calc}, TL_{equivalent})$

$TL_{min} = 0.162$ in

USE MINIMUM 1/4"
CONSERVATIVE

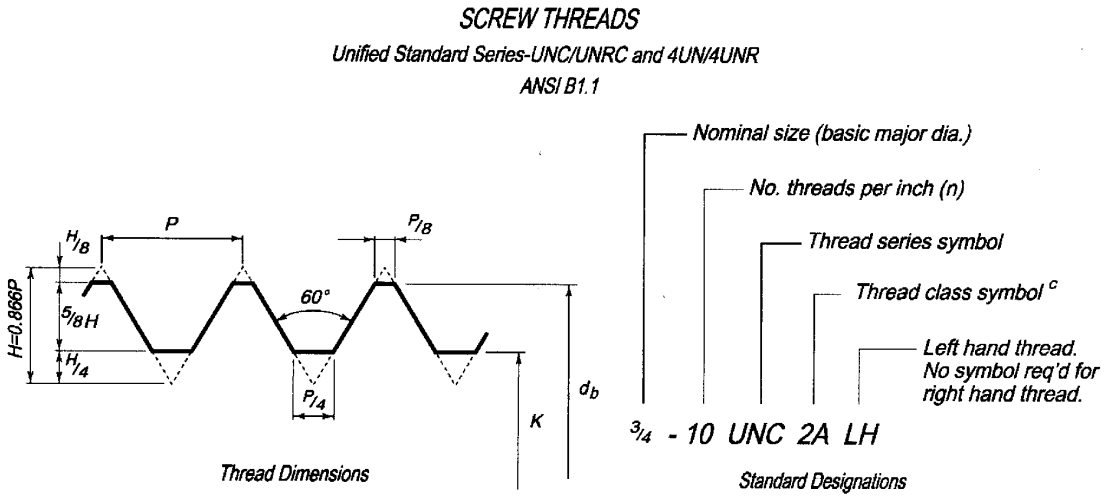
Table 7–2
Available Tensile Strength of Bolts, kips

Nominal Bolt Diameter d_b, in.			5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}			0.307		0.442		0.601		0.785	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	13.8	20.7	19.9	29.8	27.1	40.6	35.3	53.0
A490	56.5	84.8	17.3	26.0	25.0	37.4	34.0	51.0	44.4	66.6
A307	22.5	33.8	6.90	10.4	9.94	14.9	13.5	20.3	17.7	26.5

Nominal Bolt Diameter d_b, in.			1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}			0.994		1.23		1.48		1.77	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	44.7	67.1	55.2	82.8	66.8	100	79.5	119
A490	56.5	84.8	56.2	84.2	69.3	104	83.9	126	99.8	150
A307	22.5	33.8	22.4	33.5	27.6	41.4	33.4	50.1	39.8	59.6

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

Table 7–17
Threading Dimensions for High-Strength and Non-High-Strength Bolts



Diameter		Area			
Bolt Diameter d_b, in.	Min. Root K, in.	Gross Bolt Area, in.^{2}	Min. Root Area, in.^{2}	Net Tensile Area[a], in.^{2}	Threads per inch, n[b]
1/4	0.196	0.0490	0.0301	0.0320	20
3/8	0.307	0.110	0.0742	0.0780	16
1/2	0.417	0.196	0.136	0.142	13
5/8	0.527	0.307	0.218	0.226	11
3/4	0.642	0.442	0.323	0.334	10
7/8	0.755	0.601	0.447	0.462	9
1	0.865	0.785	0.587	0.606	8
1 1/8	0.970	0.994	0.740	0.763	7
1 1/4	1.10	1.23	0.942	0.969	7
1 3/8	1.19	1.49	1.12	1.16	6
1 1/2	1.32	1.77	1.37	1.41	6
1 3/4	1.53	2.41	1.85	1.90	5
2	1.76	3.14	2.43	2.50	4.5
2 1/4	2.01	3.98	3.17	3.25	4.5
2 1/2	2.23	4.91	3.90	4.00	4
2 3/4	2.48	5.94	4.83	4.93	4
3	2.73	7.07	5.85	5.97	4
3 1/4	2.98	8.30	6.97	7.10	4
3 1/2	3.23	9.62	8.19	8.33	4
3 3/4	3.48	11.0	9.51	9.66	4
4	3.73	12.6	10.9	11.1	4

[a] Net tensile area $= 0.7854 \times \left(d_b - \frac{0.9743}{n}\right)^2$

[b] For diameters listed, thread series is UNC (coarse). For larger diameters, thread series is 4UN.

[c] 2A denotes Class 2A fit applicable to external threads;
2B denotes corresponding Class 2B fit for internal threads.

Table 2–6
Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts: Conventional	High-Strength Bolts: Twist-Off-Type Tension-Control[d]	Common Bolts	Nuts	Washers	Direct-Tension-Indicator Washers	Threaded Rods	Shear Stud Connectors	Anchor Rods: Hooked	Anchor Rods: Headed	Anchor Rods: Threaded & Nutted
A108		—	65	0.375 to 0.75, incl.								■			
A325[d]		—	105	over 1 to 1.5 incl.	■										
		—	120	0.5 to 1, incl.	■										
A490		—	150	0.5 to 1.5	■										
F1852		—	105	1.125		■									
		—	120	0.5 to 1, incl.		■									
A194 Gr. 2H		—	—	0.25 to 4				▒							
A563		—	—	0.25 to 4				■							
F436[b]		—	—	0.25 to 4					■						
F959		—	—	0.5 to 1.5						■					
A36		36	58-80	to 10							■		▒		▒
A193 Gr. B7[e]		—	100	over 4 to 7							▒		▒		▒
		—	115	over 2.5 to 4							▒		▒		▒
		—	125	2.5 and under							▒		▒		▒
A307	Gr. A	—	60	0.25 to 4			■							▒	
	Gr. C	—	58-80	0.25 to 4							▒		▒		▒
A354 Gr. BD		—	140	2.5 to 4 incl.							▒		▒	▒	▒
		—	150	0.25 to 2.5, incl.							▒		▒	▒	▒
A449		—	90	1.75 to 3 incl.	[c]						▒		▒	▒	▒
		—	105	1.125 to 1.5, incl.	[c]						▒		▒	▒	▒
		—	120	0.25 to 1, incl.	[c]						▒		▒	▒	▒
A572	Gr. 42	42	60	to 6							▒		▒		▒
	Gr. 50	50	65	to 4							▒		▒		▒
	Gr. 55	55	70	to 2							▒		▒		▒
	Gr. 60	60	75	to 1.25							▒		▒		▒
	Gr. 65	65	80	to 1.25							▒		▒		▒
A588		42	63	Over 5 to 8, incl.							▒		▒		▒
		46	67	Over 4 to 5, incl.							▒		▒		▒
		50	70	4 and under							▒		▒		▒
A687		105	150 max.	0.625 to 3							▒		▒		▒
F1554	Gr. 36	36	58-80	0.25 to 4									■	■	■
	Gr. 55	55	75-95	0.25 to 4									▒	▒	▒
	Gr. 105	105	125-150	0.25 to 3									▒	▒	▒

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

— Indicates that a value is not specified in the material specification.

[a] Minimum unless a range is shown or maximum (max.) is indicated.

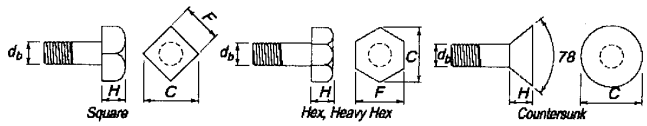
[b] Special washer requirements may apply per RCSC Specification Table 6.1 for some steel-to-steel bolting applications and per Part 14 for anchor-rod applications.

[c] See AISC Specification Section A3.3 for limitations on use of ASTM A449 bolts.

[d] When atmospheric corrosion resistance is desired, Type 3 can be specified.

[e] For anchor rods with temperature and corrosion resistance characteristics.

Table 7–20
Dimensions of Non-High-Strength Bolts, in.



Bolts Dia d_b, in.		Square			Hex			Heavy Hex			Countersunk		Min, Thrd. Length, in.	
		F, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*C*, in.	*H*, in.	*L* ≤ 6 in.	*L* > 6 in.
Bolts	1/4	3/8	1/2	3/16	7/16	1/2	3/16	–	–	–	1/2	1/8	3/4	1
	3/8	9/16	13/16	1/4	9/16	5/8	1/4	–	–	–	11/16	3/16	1	1 1/4
	1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
	5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	1 1/8	1 1/8	5/16	1 1/2	1 3/4
	3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
	7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
	1	1 1/2	2 1/8	11/16	1 1/2	1 3/4	1 1/16	1 5/8	1 7/8	1 1/16	1 13/16	1/2	2 1/4	2 1/2
	1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
	1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
	1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	11/16	3	3 1/4
	1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
	1 3/4	–	–	–	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	–	–	3 3/4	4
	2	–	–	–	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	–	–	4 1/4	4 1/2
	2 1/4	–	–	–	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	–	–	4 3/4	5
	2 1/2	–	–	–	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	–	–	5 1/4	5 1/2
	2 3/4	–	–	–	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	–	–	5 3/4	6
	3	–	–	–	4 1/2	5 3/16	2	4 5/8	5 5/16	2	–	–	6	6 1/2
	3 1/4	–	–	–	4 7/8	5 5/8	2 3/16	–	–	–	–	–	6	7
	3 1/2	–	–	–	5 1/4	6 1/16	2 5/16	–	–	–	–	–	6	7 1/2
	3 3/4	–	–	–	5 5/8	6 1/2	2 1/2	–	–	–	–	–	6	8
	4	–	–	–	6	6 15/16	2 11/16	–	–	–	–	–	6	8 1/2

Notes:
For high-strength bolt and nut dimensions, refer to Table 7–15.
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.
Minimum thread length = $2d_b$ + 1/4-in. for bolts up to 6 in. long, and $2d_b$ + 1/2-in. for bolts longer than 6 in.

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www.easterneg.com

PROJ. NAME: _____

PROJ No.: _____ DATE: _____

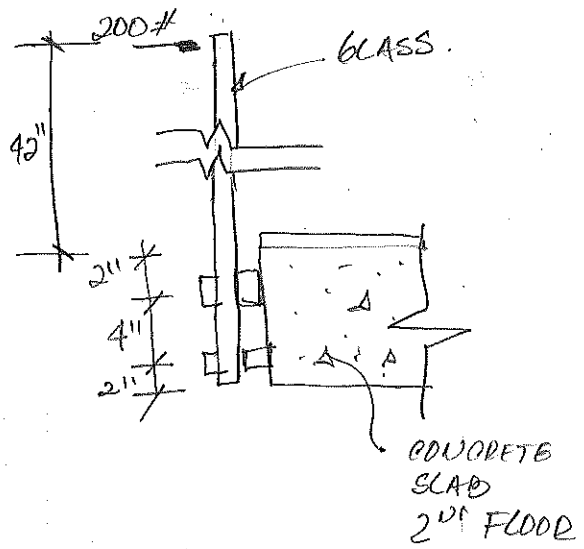
DESIGNED BY: _____ PAGE: _____

REVIEWED BY: _____

CONNECTION TO CONCRETE SLAB

Load

P= 200#



Load processing

$T = \frac{P \times L}{d} = \frac{200\# \ (42"+2"+4)}{4}$

T= 2400#

@ Asuming min (2) pair of stand-off per panel

$T_{bolt} = \frac{2400\#}{2 \text{ bolt}} = 1200\ \#$ < 1922.46# =Tallow (OK)

$V \text{ per bolt} = \frac{200\#}{2} = 100\ \#$ < 524.16# = Vallow (OK)

@. Use Ø 3/8" Threaded Rod C6t Every Epoxy w/3-3/8" min embedment, 4" min spacing & 2" min edge dist

Tbolt = 2910#x0.7379x0.8953
Tbolt = 1922.46#

Sbolt = 1300#x0.5166x0.7805
Sbolt = 524.16#

COMBINED

1200#/1922.46# + 150#/524.16#<1.00 (Conserv.)

0.91< 1.00 (OK)

PERFORMANCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod Ultimate Tension and Shear Loads[1,2,3] Installed in Solid Concrete

THREADED ROD DIA. (in.)	EMBEDMENT IN CONCRETE (in.)	MAX. CLAMPING FORCE AFTER PROPER CURE (ft./lbs.)	ULTIMATE TENSION (lbs.)			ULTIMATE SHEAR (lbs.)
			3,000 PSI CONCRETE	5,000 PSI CONCRETE	7,000 PSI CONCRETE	3,000 PSI CONCRETE & HIGHER
3/8	1-1/2	9	3,160	3,785	4,405	N/A
	3-3/8		11,640	12,315	12,985	5,200
1/2	2	16	6,075	7,015	7,950	N/A
	4-1/2		20,005	23,305	26,605	11,420
5/8	2-1/2	47	8,570	9,995	11,420	N/A
	5-5/8		24,905	29,015	33,125	18,300
3/4	3	70	12,030	13,570	15,105	N/A
	6-3/4		36,645	42,695	48,740	25,720
7/8	3-1/2	90	15,005	17,335	19,660	N/A
	7-7/8		55,575	70,338	85,100	32,120
1	4	110	17,735	20,390	23,045	N/A
	9		62,250	73,850	85,450	38,520
1-1/4	5	370	34,695	36,935	39,170	N/A
	11-1/4		77,815	90,655	103,495	65,080
1-1/2	13	450	101,085	117,765	134,445	N/A

1 Allowable working loads for the single installations under static loading should not exceed 25% capacity of the Ultimate Load. To calculate the Allowable Load of the anchor rod, divide the Ultimate Load by 4.
2 Performance values are based on the use of high strength threaded rod (ASTM A193 Gr. B7). The use of lower strength rods will result in lower ultimate tension and shear loads.
3 Linear interpolation may be used for intermediate spacing and edge distances.

PERFORMANCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod Allowable Tension Loads[1] Installed in Solid Concrete

THREADED ROD DIA in.	EMBEDMENT IN CONCRETE in.	ALLOWABLE TENSION LOAD BASED ON CONCRETE STRENGTH (lbs.)			ALLOWABLE TENSION LOAD BASED ON STEEL STRENGTH (lbs.)		
		3,000 psi concrete	5,000 psi concrete	7,000 psi concrete	ASTM A307	ASTM A193 GRADE B7	ASTM F593 AISI 304 SS
3/8	1-1/2	790	945	1,100	2,080	4,340	3,995
	3-3/8	2,910	3,080	3,245	2,080	4,340	3,995
1/2	2	1,520	1,755	1,990	3,730	7,780	7,155
	4-1/2	5,000	5,825	6,650	3,730	7,780	7,155
5/8	2-1/2	2,145	2,500	2,855	5,870	12,230	11,250
	5-5/8	6,225	7,255	8,280	5,870	12,230	11,250
3/4	3	3,010	3,395	3,775	8,490	17,690	14,860
	6-3/4	9,160	10,675	12,185	8,490	17,690	14,860
7/8	3-1/2	3,750	4,335	4,915	11,600	25,510	20,835
	7-7/8	13,895	17,585	21,275	11,600	25,510	20,835
1	4	4,435	5,100	5,760	15,180	31,620	26,560
	9	15,565	18,465	21,365	15,180	31,620	26,560
1-1/4	5	8,675	9,235	9,795	23,800	49,580	34,670
	11-1/4	19,455	22,665	25,875	23,800	49,580	34,670
1-1/2	13	25,270	29,440	33,610	33,720	70,250	47,770

1 Use lower value of either bond or steel strength for allowable tension load.

PERFORMANCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod Allowable Shear Loads[1] Installed in Solid Concrete

THREADED ROD DIA. (in.)	EMBEDMENT IN CONCRETE (in.)	ALLOWABLE SHEAR LOAD BASED ON CONCRETE STRENGTH (lbs.)	ALLOWABLE SHEAR LOAD BASED ON STEEL STRENGTH (lbs.)		
		3,000 psi concrete & higher	ASTM A307	ASTM A193 GRADE B7	ASTM F593 AISI 304 SS
3/8	1-1/2	N/A	1,040	2,170	1,995
	3-3/8	1,300	1,040	2,170	1,995
1/2	2	N/A	1,870	3,895	3,585
	4-1/2	2,855	1,870	3,895	3,585
5/8	2-1/2	N/A	2,940	6,125	5,635
	5-5/8	4,575	2,940	6,125	5,635
3/4	3	N/A	4,250	8,855	7,440
	6-3/4	6,430	4,250	8,855	7,440
7/8	3-1/2	N/A	5,800	12,760	10,730
	7-7/8	8,030	5,800	12,760	10,730
1	4	N/A	7,590	15,810	13,285
	9	9,630	7,590	15,810	13,285
1-1/4	5	N/A	11,900	24,790	18,840
	11-1/4	16,270	11,900	24,790	18,840

1 Use lower value of either concrete or steel strength for allowable shear load.

PERFORMANCE TABLE

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For the Most Demanding Jobs

Rebar Ultimate Tension Loads[1,2,3] Installed in Solid Concrete

REINFORCING BAR	EMBEDMENT IN CONCRETE (in.)	ULTIMATE TENSION (lbs.)			ULTIMATE YIELD STRENGTH GRADE 60 REBAR (lbs.)	ULTIMATE TENSILE STRENGTH GRADE 60 REBAR (lbs.)
		3,000 psi concrete	5,000 psi concrete	7,000 psi concrete		
#3	1-1/2	3,160	3,785	4,405	6,600	9,900
	3-3/8	11,640	12,315	12,985		
#4	2	6,075	7,015	7,950	12,000	18,000
	4-1/2	20,005	23,305	26,605		
#5	2-1/2	8,570	9,995	11,420	18,600	27,900
	5-5/8	24,905	29,015	33,125		
#6	3	12,030	13,570	15,105	26,400	39,600
	6-3/4	36,645	42,695	48,740		
#7	3-1/2	15,005	17,335	19,660	36,000	54,000
	7-7/8	55,575	70,338	85,100		
#8	4	17,735	20,390	23,045	47,400	71,100
	9	62,250	73,850	85,450		
#10	5	34,695	36,935	39,170	79,200	114,300
	11-1/4	77,815	90,655	103,495		
#11	13	101,085	117,764	134,443	93,600	140,400

1 Allowable working loads for the single installation under static loading should not exceed 25% capacity of the Ultimate Load. To calculate the Allowable Load of the anchor, divide the ultimate load by 4.
2 Performance values are based on the use of ASTM A615 Grade 60 reinforcing bar. The use of lower strength rebar will result in lower ultimate tension loads
3 SHEAR DATA: Provided the distance from the rebar to the edge of the concrete member exceeds 1.25 times the embedment depth of the rebar, calculate the ultimate shear load for the rebar anchorage as 60% of the ultimate tensile strength of the rebar.

PERFORMANCE REFERENCE TABLE

C6+
For the Most Demanding Jobs

Threaded Rod and Rebar Installation in Solid Concrete Edge/Spacing Distance Load Factor Summary[1,2]

LOAD FACTOR		DISTANCE FROM EDGE OF CONCRETE	
Critical Edge Distance—Tension			
100% Tension Load	→	1.25 x Anchor Embedment	4.21"
Minimum Edge Distance—Tension			
70% Tension Load	→	0.50 x Anchor Embedment	1.68"
Critical Edge Distance—Shear			
100% Shear Load	→	1.25 x Anchor Embedment	4.21"
Minimum Edge Distance—Shear			
30% Shear Load	→	0.30 x Anchor Embedment	1.01"

LOAD FACTOR		DISTANCE FROM ANOTHER ANCHOR	
Critical Spacing—Tension			
100% Tension Load	→	1.50 x Anchor Embedment (o	5.06"
Minimum Spacing—Tension			
75% Tension Load	→	0.75 x Anchor Embedment	2.53"
Critical Spacing—Shear			
100% Shear Load	→	1.50 x Anchor Embedment	5.06"
Minimum Spacing—Shear			
30% Shear Load	→	0.50 x Anchor Embedment	1.68"

1 Use linear interpolation
2 Anchors are affected by the product of tension and sh

Tension- E.D factor	
4.210	1
2.000	0.7379
1.680	0.7000

Shear- E.D factor	
4.210	1
2.000	0.5166
1.010	0.3000

Tension- Spacing	
5.060	1
4.000	0.8953
2.530	0.7500

Shear- Spacing	
5.060	1
4.000	0.7805
1.680	0.3000

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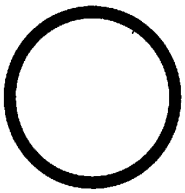


HSS2.375–
HSS1.660

Table 1–13 (continued)
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*D*/*t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS2.375×0.250	0.233	5.68	1.57	10.2	0.910	0.766	0.762	1.07	1.82	1.53
×0.218	0.203	5.03	1.39	11.7	0.824	0.694	0.771	0.960	1.65	1.39
×0.188	0.174	4.40	1.20	13.6	0.733	0.617	0.781	0.845	1.47	1.23
×0.154	0.143	3.66	1.00	16.6	0.627	0.528	0.791	0.713	1.25	1.06
×0.125	0.116	3.01	0.823	20.5	0.527	0.443	0.800	0.592	1.05	0.887
HSS1.900×0.188	0.174	3.44	0.943	10.9	0.355	0.374	0.613	0.520	0.710	0.747
×0.145	0.135	2.72	0.749	14.1	0.293	0.309	0.626	0.421	0.586	0.617
×0.120	0.111	2.28	0.624	17.1	0.251	0.264	0.634	0.356	0.501	0.527
HSS1.660×0.140	0.130	2.27	0.625	12.8	0.184	0.222	0.543	0.305	0.368	0.444

Guardrail Design (Top and Post)

Allowable Stresses for **Aluminum**				
	6063-T6 (psi)	6063-T5 (psi)	6061-T6 (psi)	6005-T5 (psi)
Fb	15000	9500	19000	19000
(Tubes & Shapes)	4800	4800	9000	8000
Fb	18000	11500	24000	24000
(Round & Oval)	5500	5500	10500	9000
Fv	8500	5500	12000	12000
Tubes, Shapes & Round	2800	2800	5000	4500
E	10100 ksi	10100 ksi	10100 ksi	10100 ksi

White bars apply to nonwelded members and to welded members at loc. fathers than 1.0 in from a weld
Shadded bars apply to within 1.0 in of a weld

Allowable Stress for Aluminum Welding:
Aluminum Filler Alloy 5356 Fv=7000 psi

Allowable Stress for Steel
Strong and Weak Axis Allowables Stress for Tube **Steel** Shape:
Fb=23760 psi, Fv=14400 psi for Fy=36 ksi E=29000 Ksi
Fb=33000 psi, Fv=20000 psi for Fy=50 ksi E=29000 Ks

Allowable Stress for Steel Welding:
Welding: E-70XX Fv=21000 psi

Allowable Stress for Stainless Steel
Strong and Weak Axis Allowables Stress for Tube Steel Shape:
Fb=16216 psi, Fv=9189 psi for (SS 304) Fy=30 ksi E=28000 Ksi
Fb=34730 psi, Fv=20838 psi for (S32304 Duplex SS) Fy=58 ksi E=28000 Ksi
Fb=62870 psi, Fv=37720 psi for (S17400-H1150) Fy=105 ksi E=28000 Ksi

Allowable Stress for Steel Welding:
Welding: E318 or E309 Fv=19200 psi

Allowable Stress for Wood
Strong and Weak Axis Allowables Stress for Lumber No.2 Non Dense (2"-4" thick):
Fb=1350 psi, Fv=175 psi for 2"-4" Wide E=1400 Ksi
Fb=1150 psi, Fv=175 psi for 5"-6" Wide E=1400 Ksi

DESIGN INFORMATION:

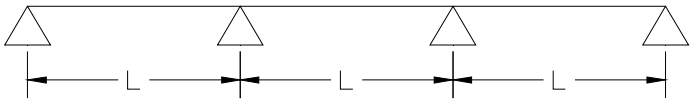
Florida Building Code 2023
AISC Manual of Steel Construction 16th Ed, NDS-ASD for Wood Construction 2024, ASCE 07-22
Structural Stainless Steel, Aluminum Design Manual 2020

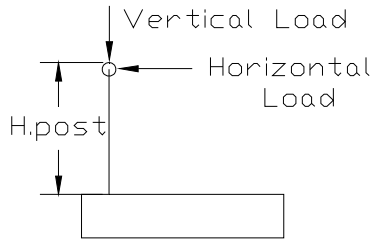
DESIGN LOADS:

Handrail Design Loads: Concentrated Load = 200 lbs. applied at any point in any direction.
Uniform Load = 50 plf applied in any direction (other than dwelling units).

Guardrail System Loads: Concentrated Load = 200 lbs. applied at any point in any direction at the top of the guardrail (other than dwelling units include a load of 100 lbs applied vertically downward at the top of guardrail).
Concentrated horizontal load of 200 lbs applied on a 1 sq ft. area at any point in the system.

Handrail Design:





General Data:

$L := 48.00$ Railing Span (in)

$H_{post} := 36.0$ Post Height (in)

General Loads:

$q_{wind} := 5.0$ Wind Pressure (psf)

$P_{200} := 200.00$ Single Concentrated Load (lbs) .....Railing P=200.00, Fence P=0.00

$q_{50} := 50.00$ Uniform Distributed Load (plf).....Railing q=50.00, Fence q=0.00

$P_{50} := 50.00$ Single Concentrated Load (lbs) .....Railing P=50.00, Fence P=0.00

Top Railing Data:

$Fb_{TR} := 10500.0$ psi

$Fv_{TR} := 5000.00$ psi

$Sx_{TR} := 0.309$ Vertical Loads in^3

$A_{TR} := 0.309$ in^2

$Sy_{TR} := 0.28$ Horizontal Loads in^3

Post Data:

$Fb_{PT} := 25200.0$ psi

$Fv_{PT} := 16800.0$ psi

$Sx_{PT} := 0.309$ in^3

$A_{PT} := 0.749$ in^2

Top Railing Design:

Actual Moment:

Concentrated Load = 200 lbs.

$$M_{TR.200} := \frac{P_{200} \cdot L}{4}$$

$M_{TR.200} = 2400.00$ in − lb

Uniform Load = 50 plf

$$M_{TR.50} := \left(\frac{q_{50}}{12 \cdot 8}\right) \cdot L^2$$

$M_{TR.50} = 1.2 \times 10^3$ in − lb

For Wind Pressure

$$M_{TR.wind} := \left(\frac{q_{wind}}{144}\right) \cdot \frac{H_{post}}{2} \cdot L^2$$

$M_{TR.wind} = 1.44 \times 1$ in − lb

$$M_{TR} := \max\left(M_{TR.200}, M_{TR.50}, M_{TR.wind}\right)$$

$M_{TR} = 2400.00$ in − lb

Actual Shear:

Concentrated Load = 200 lbs.

$$V_{TR.200} := P_{200} \text{ lbs}$$

$V_{TR.200} = 200.00$ lbs

Uniform Load = 50 plf

$$V_{TR.50} := 0.6 \cdot \left(\frac{q_{50}}{12}\right) \cdot L$$

$V_{TR.50} = 120.00$ lbs

For Wind Pressure

$$V_{TR.wind} := 0.6 \cdot \left(\frac{q_{wind}}{144}\right) \cdot \frac{H_{post}}{2} \cdot L$$

$V_{TR.wind} = 18.00$ lbs

$$V_{TR} := \max\left(V_{TR.200}, V_{TR.50}, V_{TR.wind}\right)$$

$V_{TR} = 200.00$ lbs

Section Required:

Bending Design: Section Modulus Required

$$S_{TR.min} := \frac{M_{TR}}{Fb_{TR}}$$

$S_{TR.min} = 0.23$ in^3

Shear Design: Area Required

$$A_{TR.min} := \frac{1.5 \cdot V_{TR}}{Fv_{TR}}$$

$A_{TR.min} = 0.06$ in^2

Section Provided:

$$BENDING_{top} := \text{if}\left(S_{TR.min} \geq \min\left(Sx_{TR}, Sy_{TR}\right), \text{"N.G"}, \text{"OK"}\right)$$

$BENDING_{top} = \text{"OK"}$

$$SHEAR_{top} := \text{if}\left(A_{TR.min} \geq A_{TR}, \text{"N.G"}, \text{"OK"}\right)$$

$SHEAR_{top} = \text{"OK"}$

Post Design:

Actual Reaction:

Concentrated Load = 200 lbs.

$$R_{PT.200} := P_{200}$$ $R_{PT.200} = 200.00$ lbs

Uniform Load = 50 plf

$$R_{PT.50} := \left(\frac{q_{50}}{12}\right)\cdot L$$ $R_{PT.50} = 200.00$ lbs

For Wind Pressure

$$R_{PT.wind} := \left[\left(\frac{q_{wind}}{144}\right)\right]\cdot H_{post}\cdot L$$ $R_{PT.wind} = 60.00$ lbs

$$R_{PT} := \max\left(R_{PT.200}, R_{PT.50}, R_{PT.wind}\right)$$ $R_{PT} = 200.00$ lbs

Actual Moment:

$$M_{PT} := \max\left(\max\left(R_{PT.200}, R_{PT.50}\right)\cdot H_{post}, R_{PT.wind}\cdot\frac{H_{post}}{2}\right)$$ $M_{PT} = 7200.00$ in - lb

Actual Shear:

$$V_{PT} := R_{PT}$$ $V_{PT} = 200.00$ lbs.

Section Required:

Bending Design: Section Modulus Required:

$$S_{PT.min} := \frac{M_{PT}}{Fb_{PT}}$$ $S_{PT.min} = 0.29$ in^3

Shear Design: Area Required:

$$A_{PT.min} := \frac{1.5\cdot V_{PT}}{Fv_{PT}}$$ $A_{PT.min} = 0.02$ in^2

Section Provided:

$$BENDING_{post} := \text{if}\left(S_{PT.min} \geq Sx_{PT}, \text{"N.G"}, \text{"OK"}\right)$$ $BENDING_{post} = \text{"OK"}$

$$SHEAR_{post} := \text{if}\left(A_{PT.min} \geq A_{PT}, \text{"N.G"}, \text{"OK"}\right)$$ $SHEAR_{post} = \text{"OK"}$

Eastern
Engineering Group

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(305) 599-8133
info@easterneg.com
www.easterneg.com

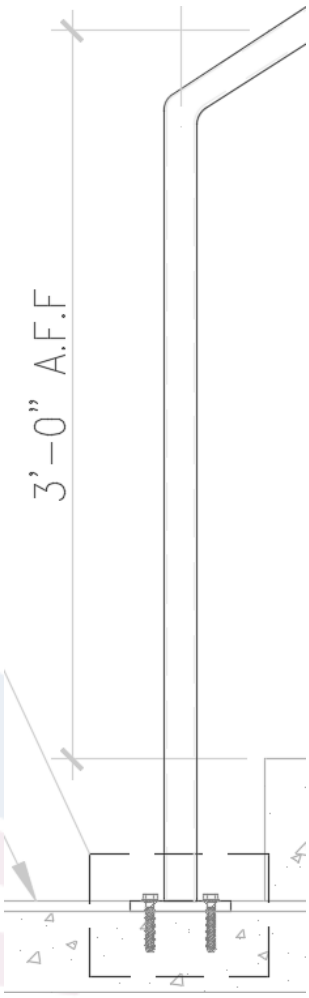
POST CONNECTION

Loads:

M= 7200 #.in
V= 200#

Load processing:

Tbolt = 928# (See next sheet printout)
Sbolt = 200#/ 4 bolt = 50# using 100#



Use Ø3/8" Flat head screw bolt + by dewalt w/ 2-1/2" min. embedment, 3" min. spacing & 4" min. edge dist.

Tallow = 1415# * 1 * 0.80 = 1132# > Tbolt (OK)
Sallow=1185# * 1 * 0.63 = 746.55# > Sbolt (OK)

Combined:
Tbolt/Tallow + Sbolt/Sallow < 1
0.95 < 1 (OK)

CHECK PLATE

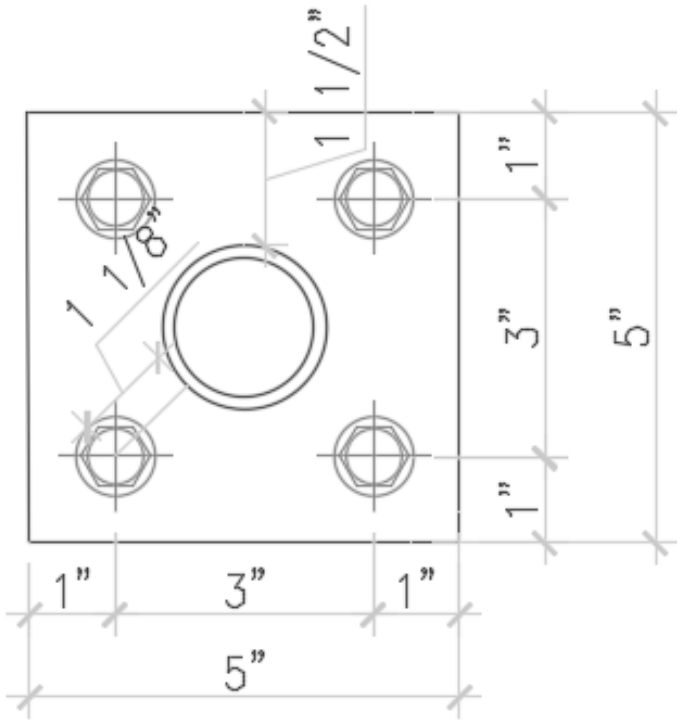
Loads:
Tbolt = 928#
ecc = 1.125 in

MdueTension = 1044#.in
Mallow = (36000psi*1.5 (2.25in*(t)²)/6)÷1.67
tmin = 0.29 in. use 3/8" thk.

Comp = 1856#
ecc = 1.5in

MdueComp = 2784 #.in
Mallow = (36000psi*1.5 (5in*(t)²)/6)÷1.67
tmin = 0.32 in use 3/8" thk.

use a 5x5x3/8 steel plate



Connection Force Resultant from Statics Analysis

Data Input:

$n := 2.0$ Number of Anchor Bolts in Tension due to Bending Only

$d := 4.0$ Distance from Tension Force to Outermost Compression Fiber (in)

$h := 5.0$ High plate (in)

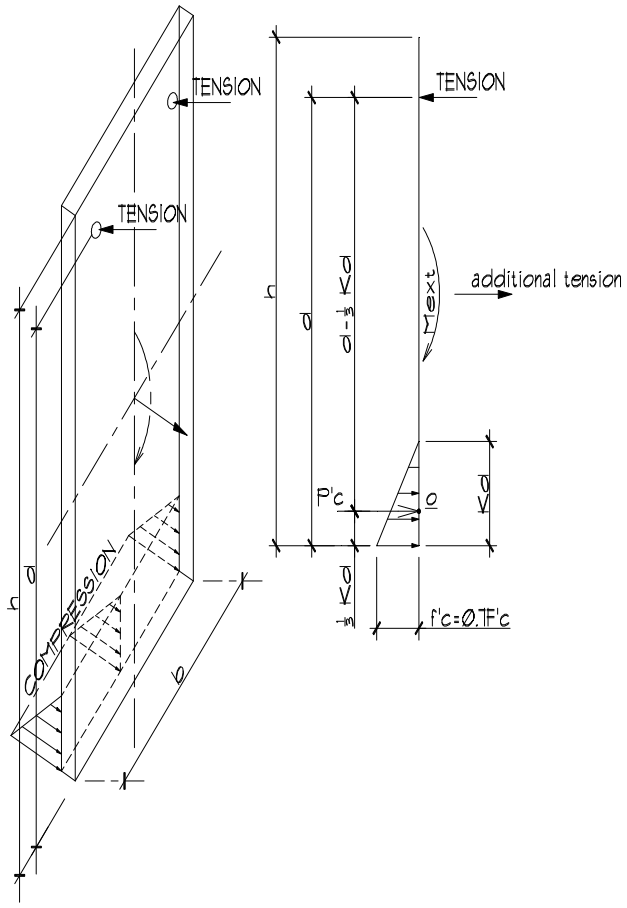
$b := 5.0$ Width of Compression Area (Post Section) (in)

$f'_c := 3000.0$ Maximum Concrete Compressive Strength (psi)

$M_{ext} := 7200.0$ External Cantilever End Moment (lbs-in)

$P_{ext} := 0.0$ Additional Tension (lbs-in)

BASE PLATE WITH EXTERNAL MOMENT



Analysis:

Compression Force:

$$C_{force} = \left(\frac{1}{2}\right)\cdot 0.7\cdot f'_c\cdot kd\cdot b$$

Sum of Moments about Compression Force:

$$\left[0 = C_{force}\cdot\left[d - \left(\frac{1}{3}\right)\cdot kd\right] - M_{ext} + P_{ext}\cdot\left(d - \frac{h}{2}\right)\right]$$

Calculation of kd:

$x := 1$ (Required Iteration Initial Guess)

Given

$$-\frac{1}{6}\cdot 0.7\cdot f'_c\cdot x^2\cdot b + \left(\frac{1}{2}\right)\cdot(x)\cdot 0.7\cdot f'_c\cdot b\cdot d - M_{ext} + P_{ext}\cdot\left(d - \frac{h}{2}\right) = 0$$

$kd := Find(x)$

Height of Compression Prism: $kd = 0.3533$ in

Tension in Bolts:

$$T_{bolt} := if\left[kd > 0, \frac{\left(\frac{1}{2}\right)\cdot 0.7\cdot f'_c\cdot kd\cdot b}{n} + \frac{P_{ext}}{2\cdot n}, \frac{M_{ext}}{n(2.d - h)} + \frac{P_{ext}}{2\cdot n}\right]$$

Tension per Bolt: $T_{bolt} = 927.3$ lbs

Allowable Load Capacities for Screw-Bolt+ in Normal-Weight Concrete[1,2,3,4,5]

Nominal Anchor Diameter in.	Minimum Nominal Embedment Depth in. (mm)	Minimum Concrete Compressive Strength									
		f'c = 2,500 psi (17.3 MPa)		f'c = 3,000 psi (20.7 MPa)		f'c = 4,000 psi (27.6 MPa)		f'c = 6,000 psi (41.4 MPa)		f'c = 8,000 psi (55.2 MPa)	
		Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)	Tension lbs (kN)	Shear lbs (kN)
1/4	1 (25)	330 (1.5)	415 (1.8)	350 (1.6)	440 (2.0)	385 (1.7)	480 (2.1)	430 (1.9)	520 (2.3)	430 (1.9)	520 (2.3)
	1-5/8 (41)	710 (3.2)	415 (1.8)	750 (3.3)	440 (2.0)	815 (3.6)	480 (2.1)	815 (3.6)	520 (2.3)	815 (3.6)	520 (2.3)
	2-1/2 (64)	915 (4.1)	505 (2.2)	965 (4.3)	535 (2.4)	1,050 (4.7)	585 (2.6)	1,070 (4.8)	635 (2.8)	1,070 (4.8)	635 (2.8)
3/8	1-1/2 (38)	660 (2.9)	890 (4.0)	720 (3.2)	975 (4.3)	835 (3.7)	1,125 (5.0)	1,020 (4.5)	1,375 (6.1)	1,020 (4.5)	1,590 (7.1)
	2 (51)	920 (4.1)	1,080 (4.8)	1,005 (4.5)	1,185 (5.3)	1,160 (5.2)	1,365 (6.1)	1,180 (5.2)	1,585 (7.1)	1,365 (6.1)	1,585 (7.1)
	2-1/2 (64)	1,295 (5.8)	1,080 (4.8)	1,415 (6.3)	1,185 (5.3)	1,600 (7.1)	1,365 (6.1)	1,615 (7.2)	1,585 (7.1)	1,855 (8.3)	1,585 (7.1)
	3-1/4 (83)	1,855 (8.3)	1,580 (7.0)	2,035 (9.1)	1,735 (7.7)	2,265 (10.1)	2,000 (8.9)	2,265 (10.1)	2,140 (9.5)	2,590 (11.5)	2,140 (9.5)
	4-1/2 (114)	2,725 (12.1)	1,580 (7.0)	2,985 (13.3)	1,735 (7.7)	3,450 (15.3)	2,000 (8.9)	3,770 (16.8)	2,140 (9.5)	3,770 (16.8)	2,140 (9.5)
1/2	1-3/4 (44)	710 (3.2)	1,495 (6.7)	780 (3.5)	1,640 (7.3)	900 (4.0)	1,895 (8.4)	1,100 (4.9)	2,320 (10.3)	1,100 (4.9)	2,675 (11.9)
	2-1/2 (64)	1,670 (7.4)	2,010 (8.9)	1,830 (8.1)	2,200 (9.8)	2,115 (9.4)	2,540 (11.3)	2,115 (9.4)	2,885 (12.8)	2,115 (9.4)	2,885 (12.8)
	3 (76)	2,140 (9.5)	2,010 (8.9)	2,345 (10.4)	2,200 (9.8)	2,690 (11.9)	2,540 (11.3)	2,690 (11.9)	2,885 (12.8)	2,690 (11.9)	2,885 (12.8)
	4-1/4 (108)	3,315 (14.7)	2,350 (10.5)	3,630 (16.1)	2,575 (11.5)	4,120 (18.3)	2,970 (13.2)	4,120 (18.3)	3,380 (15.0)	4,120 (18.3)	3,380 (15.0)
	5-1/2 (140)	3,935 (17.5)	2,350 (10.5)	4,310 (19.2)	2,575 (11.5)	4,975 (22.1)	2,970 (13.2)	5,330 (23.7)	3,380 (15.0)	5,330 (23.7)	3,380 (15.0)
5/8	2-1/2 (64)	1,435 (6.4)	2,655 (11.8)	1,570 (7.0)	2,910 (12.9)	1,815 (8.1)	3,355 (14.9)	2,220 (9.9)	4,110 (18.3)	2,220 (9.9)	4,295 (19.1)
	3-1/4 (83)	2,440 (10.9)	3,015 (13.4)	2,670 (11.9)	3,305 (14.7)	3,085 (13.7)	3,815 (17.0)	3,085 (13.7)	4,295 (19.1)	3,085 (13.7)	4,295 (19.1)
	4 (102)	2,940 (13.1)	3,015 (13.4)	3,225 (14.3)	3,305 (14.7)	3,720 (16.5)	3,815 (16.9)	3,830 (17.0)	4,295 (19.1)	4,150 (18.5)	4,295 (19.1)
	5 (127)	3,615 (16.1)	3,420 (15.2)	3,960 (17.6)	3,745 (16.7)	4,570 (20.3)	4,325 (19.2)	4,825 (21.5)	4,870 (21.7)	5,570 (24.8)	4,870 (21.7)
	6-1/4 (159)	5,130 (22.8)	3,420 (15.2)	5,620 (25.0)	3,745 (16.7)	6,490 (28.9)	4,325 (19.2)	7,945 (35.3)	4,870 (21.7)	7,945 (35.3)	4,870 (21.7)
3/4	2-1/2 (64)	1,510 (6.7)	2,905 (12.9)	1,655 (7.4)	3,180 (14.1)	1,910 (8.5)	3,675 (16.3)	2,340 (10.4)	4,500 (20.0)	2,340 (10.4)	5,195 (23.1)
	4-1/4 (108)	2,975 (13.2)	4,265 (19.0)	3,260 (14.5)	4,670 (20.8)	3,765 (16.7)	5,395 (24.0)	4,435 (19.7)	6,070 (27.0)	5,125 (22.8)	6,070 (27.0)
	5 (127)	4,755 (21.2)	4,265 (19.0)	5,210 (23.2)	4,670 (20.8)	6,015 (26.8)	5,395 (24.0)	7,365 (32.8)	6,070 (27.0)	7,365 (32.8)	6,070 (27.0)
	6-1/4 (159)	5,125 (22.8)	4,265 (19.0)	5,615 (25.0)	4,670 (20.8)	6,480 (28.8)	5,395 (24.0)	7,940 (35.3)	6,070 (27.0)	7,940 (35.3)	6,070 (27.0)

1. Tabulated load values are for anchors installed in uncracked concrete. Concrete compressive strength must be at the specified minimum at the time of installation.
2. Allowable load capacities are calculated using an applied safety factor of 4.0 to average ultimate load capacities.
3. Allowable load capacities must be multiplied by reduction factors when anchor spacing or edge distances are less than critical distances.
4. Linear interpolation may be used to determine allowable loads for intermediate embedments and compressive strengths.
5. For lightweight concrete multiply tabulated allowable load values by a reduction factor of 0.60.

LOAD ADJUSTMENT FACTORS FOR NORMAL-WEIGHT CONCRETE

Edge Distance Reduction Factors - Tension (F_{NC})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Min. Edge Distance c_{min} (in)		1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4
Edge Distance (inches)	1-1/2	1.00	0.77	0.64	0.85	0.74	0.67	0.59	0.55	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	1.00	0.83	0.67	0.93	0.79	0.71	0.62	0.57	0.87	0.71	0.65	0.58	0.54	0.73	0.65	0.60	0.56	0.53	0.73	0.59	0.56	0.53
	2	1.00	0.88	0.71	1.00	0.84	0.76	0.65	0.59	0.94	0.76	0.68	0.60	0.56	0.78	0.68	0.63	0.58	0.54	0.78	0.61	0.58	0.54
	2-1/4	1.00	0.94	0.75	1.00	0.89	0.80	0.68	0.61	1.00	0.80	0.71	0.63	0.57	0.82	0.71	0.65	0.60	0.56	0.82	0.63	0.60	0.56
	2-1/2	1.00	1.00	0.78	1.00	0.95	0.84	0.71	0.63	1.00	0.84	0.74	0.65	0.59	0.87	0.75	0.68	0.62	0.57	0.87	0.66	0.62	0.57
	2-3/4	1.00	1.00	0.82	1.00	1.00	0.88	0.74	0.65	1.00	0.88	0.77	0.67	0.61	0.91	0.78	0.70	0.64	0.59	0.91	0.68	0.64	0.59
	3	1.00	1.00	0.86	1.00	1.00	0.92	0.77	0.67	1.00	0.92	0.81	0.69	0.62	0.96	0.81	0.73	0.66	0.60	0.96	0.70	0.66	0.60
	3-1/2	1.00	1.00	0.93	1.00	1.00	1.00	0.83	0.71	1.00	1.00	0.87	0.74	0.65	1.00	0.87	0.78	0.69	0.63	1.00	0.75	0.69	0.63
	4	1.00	1.00	1.00	1.00	1.00	1.00	0.88	0.75	1.00	1.00	0.94	0.78	0.69	1.00	0.94	0.83	0.73	0.66	1.00	0.79	0.73	0.66
	4-1/2	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.79	1.00	1.00	1.00	0.82	0.72	1.00	1.00	0.88	0.77	0.69	1.00	0.84	0.77	0.69
	5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.84	1.00	1.00	1.00	0.87	0.75	1.00	1.00	0.93	0.81	0.72	1.00	0.89	0.81	0.72
	5-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.88	1.00	1.00	1.00	0.91	0.79	1.00	1.00	0.98	0.85	0.75	1.00	0.93	0.85	0.75
	6	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.96	0.82	1.00	1.00	1.00	0.89	0.78	1.00	0.98	0.89	0.78
	6-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	1.00	1.00	1.00	1.00	0.85	1.00	1.00	1.00	0.92	0.81	1.00	1.00	0.92	0.81
	7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.88	1.00	1.00	1.00	0.96	0.84	1.00	1.00	0.96	0.84
	7-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.92	1.00	1.00	1.00	1.00	0.87	1.00	1.00	1.00	0.87
	8	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.90
	8-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	1.00	1.00	0.93	1.00	1.00	1.00	0.93
	9	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.96
	9-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00	0.99
	10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Spacing Reduction Factors - Tension (F_{NS})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Minimum Spacing s_{min} (in)		1-1/2	1-1/2	1-1/2	2	2	2	2	2	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	3	3	3	3
Spacing Distance (inches)	1-1/2	0.89	0.73	0.66	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	0.94	0.77	0.68	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2	1.00	0.80	0.70	0.88	0.77	0.71	0.67	0.63	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/4	1.00	0.83	0.72	0.93	0.80	0.74	0.69	0.64	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/2	1.00	0.86	0.74	0.97	0.83	0.76	0.70	0.65	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-3/4	1.00	0.89	0.76	1.00	0.86	0.78	0.72	0.66	0.92	0.78	0.74	0.67	0.64	0.80	0.73	0.69	0.65	0.63	-	-	-	-
	3	1.00	0.92	0.78	1.00	0.89	0.80	0.74	0.67	0.95	0.80	0.75	0.68	0.65	0.83	0.74	0.70	0.66	0.64	0.83	0.69	0.66	0.64
	3-1/2	1.00	0.95	0.82	1.00	0.92	0.83	0.77	0.70	1.00	0.85	0.79	0.71	0.67	0.88	0.78	0.73	0.68	0.65	0.88	0.71	0.68	0.65
	4	1.00	1.00	0.86	1.00	1.00	0.89	0.80	0.72	1.00	0.89	0.82	0.73	0.68	0.92	0.81	0.75	0.70	0.67	0.93	0.74	0.71	0.67
	4-1/2	1.00	1.00	0.90	1.00	1.00	0.93	0.83	0.74	1.00	0.93	0.86	0.75	0.70	0.97	0.85	0.78	0.72	0.68	0.97	0.76	0.73	0.69
	5	1.00	1.00	0.94	1.00	1.00	0.98	0.86	0.76	1.00	0.98	0.89	0.78	0.72	1.00	0.88	0.81	0.75	0.70	1.00	0.79	0.75	0.70
	5-1/2	1.00	1.00	0.97	1.00	1.00	1.00	0.89	0.78	1.00	1.00	0.93	0.80	0.74	1.00	0.92	0.83	0.77	0.72	1.00	0.81	0.77	0.72
	6	1.00	1.00	1.00	1.00	1.00	1.00	0.93	0.81	1.00	1.00	0.96	0.82	0.75	1.00	0.95	0.86	0.79	0.73	1.00	0.84	0.79	0.73
	6-1/2	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.83	1.00	1.00	1.00	0.85	0.77	1.00	0.98	0.89	0.81	0.75	1.00	0.86	0.81	0.75
	7	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.85	1.00	1.00	1.00	0.87	0.79	1.00	1.00	0.91	0.83	0.76	1.00	0.89	0.83	0.77
	7-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.87	1.00	1.00	1.00	0.90	0.81	1.00	1.00	0.94	0.85	0.78	1.00	0.91	0.85	0.78
	8	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.92	0.83	1.00	1.00	0.97	0.87	0.80	1.00	0.94	0.87	0.80
	8-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.94	0.84	1.00	1.00	0.99	0.89	0.81	1.00	0.96	0.89	0.81
	9	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.97	0.86	1.00	1.00	1.00	0.91	0.83	1.00	0.99	0.91	0.83
	9-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.99	0.88	1.00	1.00	1.00	0.93	0.84	1.00	1.00	0.93	0.85
	10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.95	0.86	1.00	1.00	0.95	0.86
	10-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.91	1.00	1.00	1.00	0.97	0.88	1.00	1.00	0.97	0.88
	11	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.93	1.00	1.00	1.00	0.99	0.89	1.00	1.00	0.99	0.89
	11-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.91	1.00	1.00	1.00	0.91
	12	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.93
	13	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.96
	14	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00	0.99
	15	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

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Edge Distance Reduction Factors - Shear (F_{VC})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Min. Edge Distance c_{min}(in)		1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4
Edge Distance (inches)	1-1/2	0.58	0.63	0.59	0.40	0.37	0.39	0.31	0.32	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	0.68	0.73	0.69	0.46	0.43	0.45	0.36	0.38	0.35	0.31	0.36	0.30	0.31	0.27	0.26	0.32	0.25	0.26	0.26	0.22	0.22	0.23
	2	0.78	0.84	0.78	0.53	0.49	0.52	0.41	0.43	0.41	0.35	0.41	0.35	0.36	0.30	0.29	0.37	0.29	0.30	0.30	0.25	0.26	0.27
	2-1/4	0.87	0.94	0.88	0.59	0.55	0.58	0.46	0.48	0.46	0.40	0.46	0.39	0.40	0.34	0.33	0.41	0.32	0.33	0.33	0.28	0.29	0.30
	2-1/2	0.97	1.00	0.98	0.66	0.61	0.64	0.51	0.54	0.51	0.44	0.51	0.43	0.45	0.38	0.36	0.46	0.36	0.37	0.37	0.31	0.32	0.33
	2-3/4	1.00	1.00	1.00	0.73	0.67	0.71	0.56	0.59	0.56	0.49	0.56	0.48	0.49	0.42	0.40	0.51	0.40	0.41	0.41	0.34	0.35	0.37
	3	1.00	1.00	1.00	0.79	0.73	0.77	0.61	0.64	0.61	0.53	0.61	0.52	0.54	0.46	0.44	0.55	0.43	0.45	0.44	0.38	0.39	0.40
	3-1/2	1.00	1.00	1.00	0.92	0.85	0.90	0.72	0.75	0.71	0.62	0.72	0.61	0.63	0.53	0.51	0.64	0.50	0.52	0.52	0.44	0.45	0.47
	4	1.00	1.00	1.00	1.00	0.97	1.00	0.82	0.86	0.81	0.71	0.82	0.69	0.72	0.61	0.58	0.74	0.57	0.59	0.59	0.50	0.51	0.53
	4-1/2	1.00	1.00	1.00	1.00	1.00	1.00	0.92	0.97	0.91	0.80	0.92	0.78	0.81	0.68	0.66	0.83	0.65	0.67	0.67	0.56	0.58	0.60
	5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.89	1.00	0.87	0.90	0.76	0.73	0.92	0.72	0.74	0.74	0.63	0.64	0.66
	5-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	0.95	0.99	0.84	0.80	1.00	0.79	0.82	0.82	0.69	0.71	0.73
	6	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.88	1.00	0.86	0.89	0.89	0.75	0.77	0.80
	6-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	0.95	1.00	0.93	0.97	0.96	0.81	0.84	0.86
	7	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.88	0.90	0.93
	7-1/2	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.96	1.00
	8	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Spacing Reduction Factors - Shear (F_{VS})

Diameter (in)		1/4			3/8					1/2					5/8					3/4			
Nominal Embedment h_{nom} (in)		1	1-5/8	2-1/2	1-1/2	2	2-1/2	3-1/4	4-1/2	1-3/4	2-1/2	3	4-1/4	5-1/2	2-1/2	3-1/4	4	5	6-1/4	2-1/2	4-1/4	5	6-1/4
Minimum Spacing s_{min} (in)		1-1/2	1-1/2	1-1/2	2	2	2	2	2	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	2-3/4	3	3	3	3
Spacing Distance (inches)	1-1/2	0.60	0.60	0.60	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1-3/4	0.61	0.62	0.61	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2	0.63	0.64	0.63	0.59	0.58	0.59	0.57	0.57	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/4	0.65	0.66	0.65	0.60	0.59	0.60	0.58	0.58	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-1/2	0.66	0.67	0.66	0.61	0.60	0.61	0.59	0.59	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2-3/4	0.68	0.69	0.68	0.62	0.61	0.62	0.59	0.60	0.59	0.58	0.59	0.58	0.58	0.57	0.57	0.58	0.57	0.57	-	-	-	-
	3	0.69	0.71	0.70	0.63	0.62	0.63	0.60	0.61	0.60	0.59	0.60	0.59	0.59	0.58	0.57	0.59	0.57	0.57	0.57	0.56	0.56	0.57
	3-1/2	0.73	0.74	0.73	0.65	0.64	0.65	0.62	0.63	0.62	0.60	0.62	0.60	0.60	0.59	0.59	0.61	0.58	0.59	0.59	0.57	0.57	0.58
	4	0.76	0.78	0.76	0.68	0.66	0.67	0.64	0.64	0.64	0.62	0.64	0.62	0.62	0.60	0.60	0.62	0.60	0.60	0.60	0.58	0.59	0.59
	4-1/2	0.79	0.81	0.79	0.70	0.68	0.69	0.65	0.66	0.65	0.63	0.65	0.63	0.63	0.61	0.61	0.64	0.61	0.61	0.61	0.59	0.60	0.60
	5	0.82	0.85	0.83	0.72	0.70	0.71	0.67	0.68	0.67	0.65	0.67	0.64	0.65	0.63	0.62	0.65	0.62	0.62	0.62	0.60	0.61	0.61
	5-1/2	0.86	0.88	0.86	0.74	0.72	0.74	0.69	0.70	0.69	0.66	0.69	0.66	0.66	0.64	0.63	0.67	0.63	0.64	0.64	0.61	0.62	0.62
	6	0.89	0.92	0.89	0.76	0.74	0.76	0.70	0.71	0.70	0.68	0.70	0.67	0.68	0.65	0.65	0.68	0.64	0.65	0.65	0.63	0.63	0.63
	6-1/2	0.92	0.95	0.92	0.79	0.76	0.78	0.72	0.73	0.72	0.69	0.72	0.69	0.69	0.66	0.66	0.70	0.66	0.66	0.66	0.64	0.64	0.64
	7	0.95	0.99	0.96	0.81	0.78	0.80	0.74	0.75	0.74	0.71	0.74	0.70	0.71	0.68	0.67	0.71	0.67	0.67	0.67	0.65	0.65	0.66
	7-1/2	0.99	1.00	0.99	0.83	0.80	0.82	0.76	0.77	0.75	0.72	0.76	0.72	0.72	0.69	0.68	0.73	0.68	0.69	0.69	0.66	0.66	0.67
	8	1.00	1.00	1.00	0.85	0.82	0.84	0.77	0.79	0.77	0.74	0.77	0.73	0.74	0.70	0.69	0.75	0.69	0.70	0.70	0.67	0.67	0.68
	9	1.00	1.00	1.00	0.90	0.87	0.89	0.81	0.82	0.80	0.77	0.81	0.76	0.77	0.73	0.72	0.78	0.72	0.72	0.72	0.69	0.69	0.70
	10	1.00	1.00	1.00	0.94	0.91	0.93	0.84	0.86	0.84	0.80	0.84	0.79	0.80	0.75	0.74	0.81	0.74	0.75	0.75	0.71	0.71	0.72
	11	1.00	1.00	1.00	0.98	0.95	0.97	0.87	0.89	0.87	0.82	0.87	0.82	0.83	0.78	0.77	0.84	0.76	0.77	0.77	0.73	0.74	0.74
	12	1.00	1.00	1.00	1.00	0.99	1.00	0.91	0.93	0.91	0.85	0.91	0.85	0.86	0.80	0.79	0.87	0.79	0.80	0.80	0.75	0.76	0.77
	13	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.96	0.94	0.88	0.94	0.88	0.89	0.83	0.82	0.90	0.81	0.82	0.82	0.77	0.78	0.79
	14	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	0.97	0.91	0.98	0.90	0.92	0.85	0.84	0.93	0.84	0.85	0.85	0.79	0.80	0.81
	15	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	1.00	0.93	0.95	0.88	0.86	0.96	0.86	0.87	0.87	0.81	0.82	0.83
	16	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	0.96	0.98	0.91	0.89	0.99	0.88	0.90	0.90	0.83	0.84	0.85
	17	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	0.93	0.91	1.00	0.91	0.92	0.92	0.86	0.86	0.88
	18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.94	1.00	0.93	0.95	0.94	0.88	0.89	0.90
	19	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.96	1.00	0.95	0.97	0.97	0.90	0.91	0.92
	20	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	0.98	1.00	0.99	0.92	0.93	0.94
	21	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.95	0.97
	22	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.97	0.99
	23	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	0.99	1.00
	24	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

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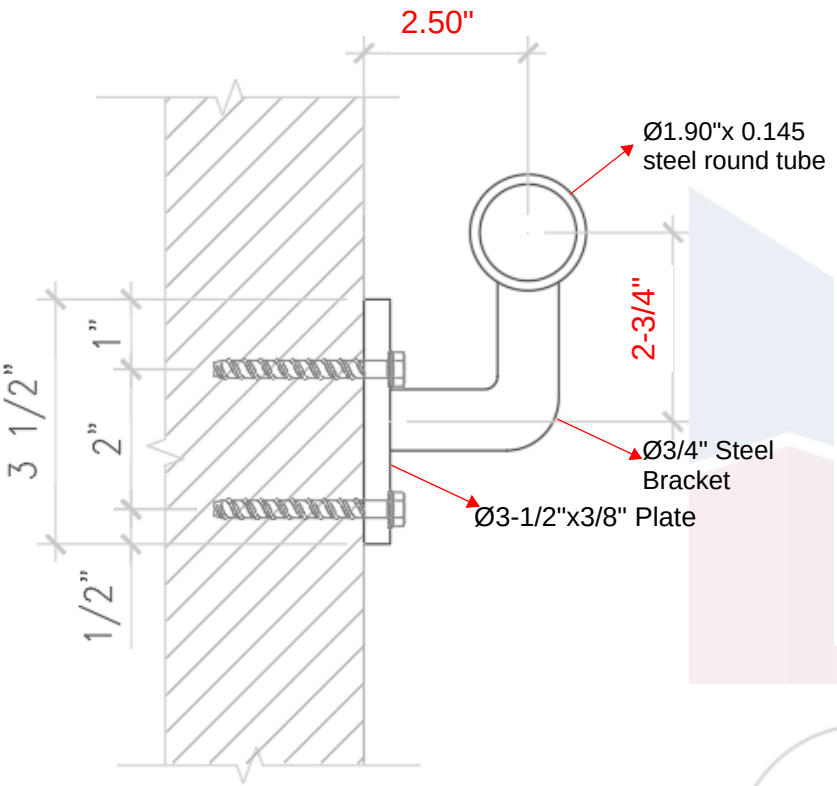
SECTION PROPERTIES



BRACKET: Steel Fy= 36 ksi

Area = 0.4418 in^2
Ixx = 0.0155 in^4
Iyy = 0.0155 in^4
Jz = 0.0311 in^4
Pxy = 0.0000 in^4
rxx = 0.1875 in
ryy = 0.1875 in
Sxx max = 0.0414 in^3
Sxx min = 0.0414 in^3
Syy max = 0.0414 in^3
Syy min = 0.0414 in^3

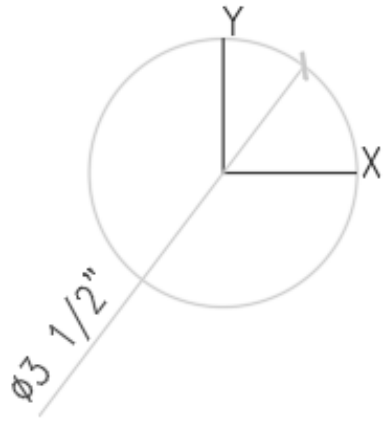
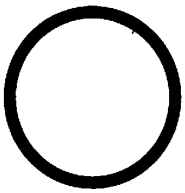


PLATE: Steel Fy= 36 KSI

Area = 9.6211 in^2
Ixx = 7.3662 in^4
Iyy = 7.3662 in^4
Jz = 14.7324 in^4
Pxy = -0.0000 in^4
rxx = 0.8750 in
ryy = 0.8750 in
Sxx max = 4.2092 in^3
Sxx min = 4.2092 in^3
Syy max = 4.2092 in^3
Syy min = 4.2092 in^3



HSS2.375–
HSS1.660

Table 1–13 (continued)
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*D*/*t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS2.375×0.250	0.233	5.68	1.57	10.2	0.910	0.766	0.762	1.07	1.82	1.53
×0.218	0.203	5.03	1.39	11.7	0.824	0.694	0.771	0.960	1.65	1.39
×0.188	0.174	4.40	1.20	13.6	0.733	0.617	0.781	0.845	1.47	1.23
×0.154	0.143	3.66	1.00	16.6	0.627	0.528	0.791	0.713	1.25	1.06
×0.125	0.116	3.01	0.823	20.5	0.527	0.443	0.800	0.592	1.05	0.887
HSS1.900×0.188	0.174	3.44	0.943	10.9	0.355	0.374	0.613	0.520	0.710	0.747
×0.145	0.135	2.72	0.749	14.1	0.293	0.309	0.626	0.421	0.586	0.617
×0.120	0.111	2.28	0.624	17.1	0.251	0.264	0.634	0.356	0.501	0.527
HSS1.660×0.140	0.130	2.27	0.625	12.8	0.184	0.222	0.543	0.305	0.368	0.444

Handrail & Circular Design Bracket Screws Located at the Center

Allowable Stresses for **Aluminum**				
	6063-T6 (psi)	6063-T5 (psi)	6061-T6 (psi)	6005-T5 (psi)
Fb	15000	9500	19000	19000
(Tubes & Shapes)	4800	4800	9000	8000
Fb	18000	11500	24000	24000
(Round & Oval)	5500	5500	10500	9000
Fv	8500	5500	12000	12000
Tubes, Shapes & Round	2800	2800	5000	4500
E	10100 ksi	10100 ksi	10100 ksi	10100 ksi

White bars apply to nonwelded members and to welded members at loc. fathers than 1.0 in from a weld
Shadded bars apply to within 1.0 in of a weld

Allowable Stress for Aluminum Welding:
Aluminum Filler Alloy 5356 Fv=7000 psi

Allowable Stress for Steel
Strong and Weak Axis Allowables Stress for Tube **Steel** Shape:
Fb=23760 psi, Fv=14400 psi for Fy=36 ksi E=29000 Ksi
Fb=33000 psi, Fv=20000 psi for Fy=50 ksi E=29000 Ks

Allowable Stress for Steel Welding:
Welding: E-70XX Fv=21000 psi

Allowable Stress for Stainless Steel
Strong and Weak Axis Allowables Stress for Tube Steel Shape:
Fb=16216 psi, Fv=9189 psi for (SS 304) Fy=30 ksi E=28000 Ksi
Fb=34730 psi, Fv=20838 psi for (S32304 Duplex SS) Fy=58 ksi E=28000 Ksi
Fb=62870 psi, Fv=37720 psi for (S17400-H1150) Fy=105 ksi E=28000 Ksi

Allowable Stress for Steel Welding:
Welding: E318 or E309 Fv=19200 psi

Allowable Stress for Wood
Strong and Weak Axis Allowables Stress for Lumber No.2 Non Dense (2"-4" thick):
Fb=1350 psi, Fv=175 psi for 2"-4" Wide E=1400 Ksi
Fb=1150 psi, Fv=175 psi for 5"-6" Wide E=1400 Ksi

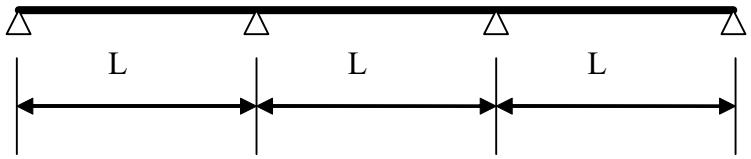
DESIGN INFORMATION:

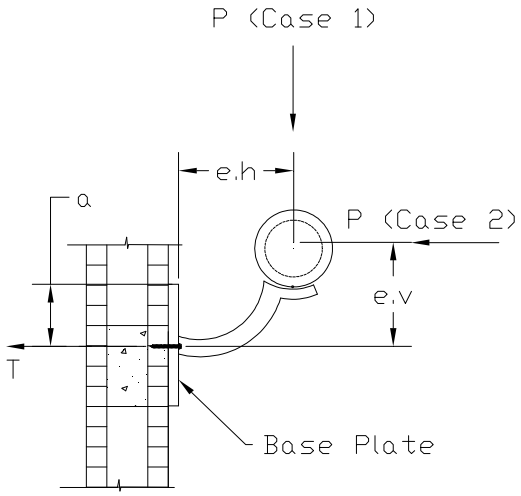
Florida Building Code 2023
Design Manual 2020, Aluminum Design Loads for Building and Others Structures ASCE/SEI 7-22.

DESIGN LOADS:

Handrail Design Loads: Concentrated Load = 200 lbs. applied at any point in any direction.
Uniform Load = 50 plf applied in any direction (other than dwelling units).
Guardrail System Loads: Concentrated Load = 200 lbs. applied at any point in any direction at the top of the guardrail (other than dwelling units include a load of 100 lbs applied vertically downward at the top of guardrail).
Concentrated horizontal load of 200 lbs applied on a 1 sq ft. area at any point in the system.

Handrail Bracket Design:





General Data:

$L := 48.00$ Railing Span (in)

$e_H := 2.50$ Horizontal Eccentricity of Loads (in)

$e_V := 2.75$ Vertical Eccentricity of Loads (in)

Handrail Data:

$Fb_{bend.top} := 25200.0$ psi

$Fv_{shear.top} := 16800.0$ psi

$Sx_{top} := 0.309$ For Vertical Loads $\left(in^3\right)$

$Sy_{top} := 0.309$ For Horizontal Loads $\left(in^3\right)$

$A_{top} := 0.749$

Bracket Data:

$Fb_{bend.bracket} := 21600.0$ psi

$Fv_{shear.bracket} := 14400.0$ psi

$Sx_{bracket} := 0.041$ in^3

$A_{bracket} := 0.601$ in^2

Base Plate Data:

$D_{plate} := 3.50$ Plate Diam. (in)

$Sx_{plate} := 4.20$ in^3

$A_{plate} := 9.62$ in^2

Allowable Support Stress Data:

$Fp_{supp} := 2100.00$

Allowable compressive strength of support (psi)
0.35*f'c.....for Full Area of Concrete Support
0.35*f'c*SQR(A2/A1) < or = 0.7*f'c.....for less than Full Area of Concrete Support
0.7*f'm.....for Full Area Concrete Block Support
Fc.....for Wood Lumber or Plywood

Handrail Design:

Actual Moment:

Concentrated Load = 200 lbs.

$$Mpmax := \frac{200 \cdot L}{5}$$

Mpmax = 1920.00 in − lb

Uniform Load = 50 plf

$$M\omega max := 0.1012 \cdot \left(\frac{50}{12}\right) \cdot L^2$$

Mωmax = 971.52 in − lb

$$Mmax := \max(M\omega max, Mpmax)$$

Mmax = 1920.00 in − lb

Actual Shear:

Concentrated Load = 200 lbs.

$$Vpmax := 200\,lbs$$

Vpmax = 200.00 lbs

Uniform Load = 50 plf

$$V\omega max := 0.6 \cdot \left(\frac{50}{12}\right) \cdot L$$

Vωmax = 120 lbs

$$Vmax := \max(V\omega max, Vpmax)$$

Vmax = 200.00 lbs

Section Required:

Bending Design:

Section Modulus Required

$$Shr := \frac{Mmax}{Fb_{bend.top}}$$

Shr = 0.08 in^3

Shear Design:

Area Required

$$Ahr := \frac{1.5 \cdot Vmax}{Fv_{shear.top}}$$

Ahr = 0.02 in^2

Section Provided:

$$BENDING_{top} := if\left(Shr \geq \min\left(Sx_{top}, Sy_{top}\right), "N.G" , "OK"\right)$$

$BENDING_{top}$ = "OK"

$$SHEAR_{top} := if\left(Ahr \geq A_{top}, "N.G" , "OK"\right)$$

$SHEAR_{top}$ = "OK"

Bracket Design:

Actual Reaction:

Concentrated Load = 200 lbs.

$Rpmax := 200.00$ lbs

Uniform Load = 50 plf

$$R\omega max := \left[\left(\frac{50}{12}\right)\right]\cdot L$$

$R\omega max = 200.00$ lbs

$$Rmax := \max(R\omega max, Rpmax)$$

$Rmax = 200.00$ lbs

Actual Moment:

$$Mbracket := Rmax\cdot\max(e_H, e_V)$$

$Mbracket = 550.00$ in - lb

Actual Shear (For Vertical Load):

$$Vbracket := Rmax$$

$Vbracket = 200.00$ lbs.

Actual Axial (For Horizontal Load):

$$Nbracket := Rmax$$

$Nbracket = 200.00$ lbs.

Section Required:

Bending Design:

Section Modulus Required:

$$Sbracketreq := \frac{Mbracket}{Fb_{bend.bracket}}$$

$Sbracketreq = 0.03$ in^3

Shear Design:

Area Required:

$$Abracketreq := \frac{1.5\cdot Vbracket}{Fv_{shear.bracket}}$$

$Abracketreq = 0.02$ in^2

Section Provided:

$BENDING_{bracket} := \text{if}\left(Sbracketreq \geq Sx_{bracket}, \text{"N.G"}, \text{"OK"}\right)$

$BENDING_{bracket} = \text{"OK"}$

$SHEAR_{bracket} := \text{if}\left(Abracketreq \geq A_{bracket}, \text{"N.G"}, \text{"OK"}\right)$

$SHEAR_{bracket} = \text{"OK"}$

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PROJ. NAME: _____

PROJ No.: _____ DATE: _____

DESIGNED BY: M.E.P PAGE: _____

REVIEWED BY: _____

HANDRAIL CONNECTION DESIGN

(CMU CONN.)

* Loads

LL = 200# (P)

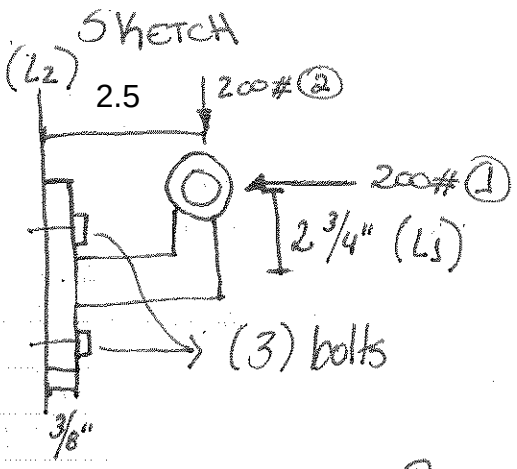
* Loads processing

Case (1)

Mact = P.L1

Mact = 200# * 2.75 in

Mact = 550 #.in (Worst Case)



Tperbolt = T1

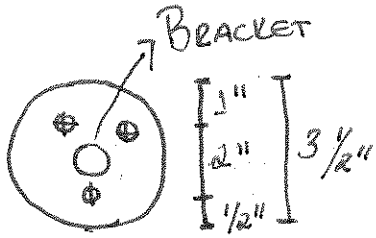
(see next sheet)

Tperbolt= 223.6#

Tperbolt = 224#

Vperbolt = 200#/3 bolts

Vperbolt = 67#



FRONT VIEW

Case (2)

Mact = P.L2

Mact = 200# * 2.5 in

Mact = 500 #.in

Combined

Tact / Tallow + Vact / Vallow =< 1

0.75 =< 1 (OK)

* Using Ø 1/4" Screw bolts w/2-1/2" min. emb., 2" min spacing and 2" min edge distance

Tallow = 605# * 1 * 0.72 = 435.6# (OK)

Sallow = 505# * 1 * 0.54 = 272.7# (OK)

Connection Force Resultant from Statics Analysis

Connection to CMU

Data Input:

$n := 1.00$ Number of Anchor Bolts in Tension due to Bending Only

$d := 2.50$ Distance from Tension Force to Outermost Compression Fiber (in)

$h := 3.50$ High plate (in)

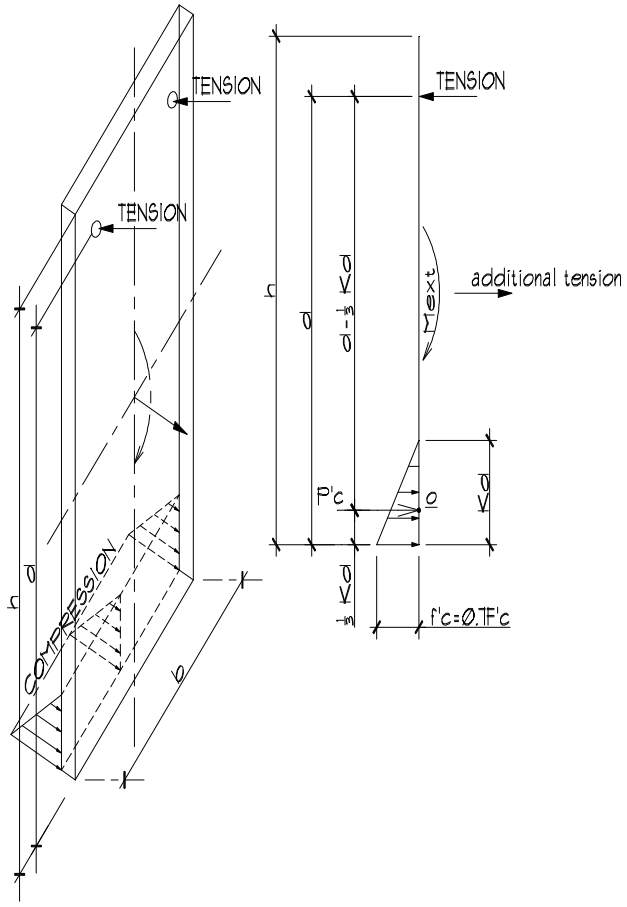
$b := 3.50$ Width of Compression Area (Post Section) (in)

$f'_c := 1500.00$ Maximum CMU Compressive Strength (psi)

$M_{ext} := 550.00$ External Cantilever End Moment (lbs-in)

$P_{ext} := 0.0$ Additional Tension (lbs-in)

BASE PLATE WITH EXTERNAL MOMENT



Analysis:

Compression Force:

$$C_{force} = \left(\frac{1}{2}\right)\cdot 0.7\cdot f'_c\cdot kd\cdot b$$

Sum of Moments about Compression Force:

$$\left[0 = C_{force}\cdot\left[d - \left(\frac{1}{3}\right)\cdot kd\right] - M_{ext} + P_{ext}\cdot\left(d - \frac{h}{2}\right)\right]$$

Calculation of kd:

$x := 1$ (Required Iteration Initial Guess)

Given

$$-\frac{1}{6}\cdot 0.7\cdot f'_c\cdot x^2\cdot b + \left(\frac{1}{2}\right)\cdot(x)\cdot 0.7\cdot f'_c\cdot b\cdot d - M_{ext} + P_{ext}\cdot\left(d - \frac{h}{2}\right) = 0$$

$kd := Find(x)$

Height of Compression Prism: $kd = 0.1217$ in

Tension in Bolts:

$$T_{bolt} := if\left[kd > 0, \frac{\left(\frac{1}{2}\right)\cdot 0.7\cdot f'_c\cdot kd\cdot b}{n} + \frac{P_{ext}}{2\cdot n}, \frac{M_{ext}}{n(2.d - h)} + \frac{P_{ext}}{2\cdot n}\right]$$

Tension per Bolt: $T_{bolt} = 223.6$ lbs

Allowable Screw-Bolt+ Tension and Shear Load Capacities Installed into the face of Grout-Filled Concrete Masonry Units[1,2,3,4,5,6,7,8,9]

CODE LISTED
ICC-ES ESR-4042

Tension Load

Anchor Diameter, d in.	Minimum Embedment h_{nom} in. (mm)	Allowable Load at c_{cr} and s_{cr} lbs (kN)	Spacing Distance, s			Edge or End Distance, c_2 or c_1 (see Illustration of Screw-Bolt+ Installed into Grouted Concrete Masonry Wall detail)		
			Critical Distance, s_{cr} in. (mm)	Minimum Distance, s_{min} in. (mm)	Allowable Load Factor at s_{min}	Critical Distance, c_{cr} in. (mm)	Minimum Distance, c_{min} in. (mm)	Allowable Load Factor at c_{min}
1/4	2-1/2 (64)	605 (2.7)	4 (102)	2 (51)	1.00 (no reduction)	3-3/4 (95)	1-1/4 (32)	0.60
3/8	2 (51)	450 (2.0)	6 (152)	3 (76)	1.00 (no reduction)	6 (152)	1-1/2 (38)	0.70
	3-1/4 (83)	1,085 (4.8)						
1/2	2-1/2 (64)	610 (2.7)	8 (203)	4 (102)	1.00 (no reduction)	8 (203)	2-5/8 (67)	0.75
	4-1/4 (108)	1,190 (5.3)						
5/8	3-1/4 (83)	880 (3.9)	10 (254)	4 (102)	1.00 (no reduction)	10 (254)	3-3/8 (88)	0.90
	5 (127)	1,270 (5.6)						
3/4	4 (102)	1,150 (5.1)	12 (305)	4 (102)	1.00 (no reduction)	12 (305)	4 (102)	1.00 (no reduction)
	6-1/4 (159)	1,355 (6.0)						

Shear Load

Anchor Diameter, d in.	Minimum Embedment h_{nom} in. (mm)	Allowable Load at c_{cr} and s_{cr} Direction 1 & 2 lbs[9] (kN)	Allowable Load at c_{cr} and s_{cr} Direction 3 & 4 lbs[9] (kN)	Spacing Distance, s			Edge or End Distance, c_2 or c_1 (see Illustration of Screw-Bolt+ Installed into Grouted Concrete Masonry Wall)			
				Critical Distance, s_{cr} in. (mm)	Minimum Distance, s_{min} in. (mm)	Allowable Load Factor at s_{min}	Critical Distance, c_{cr} in. (mm)	Minimum Distance, c_{min} in. (mm)	Allowable Load Factor at c_{min}	
									Load Perpendicular to Edge or End (Direction 1 & 2)[9]	Load Perpendicular to Edge or End (Direction 3 & 4)[9]
1/4	2-1/2 (64)	505 (2.2)	505 (2.2)	4 (102)	2 (51)	1.00 (no reduction)	3-3/4 (95)	1-1/4 (32)	0.35	1.00 (no reduction)
3/8	2 (51)	815 (3.6)	815 (3.6)	6 (152)	3 (76)	1.00 (no reduction)	6 (152)	1-1/2 (38)	0.27	1.00 (no reduction)
	3-1/4 (83)	935 (4.2)	935 (4.2)							
1/2	2-1/2 (64)	1,380 (6.1)	1,380 (6.1)	8 (203)	4 (102)	1.00 (no reduction)	8 (203)	2-5/8 (67)	0.20	1.00 (no reduction)
	4-1/4 (108)	2,180 (9.7)	2,180 (9.7)							
5/8	3-1/4 (83)	2,090 (9.3)	2,225 (9.9)	10 (254)	4 (102)	1.00 (no reduction)	10 (254)	3-3/8 (86)	0.23	1.00 (no reduction)
	5 (127)	2,640 (11.7)	2,640 (11.7)							
3/4	4 (102)	2,800 (12.5)	3,330 (14.8)	12 (305)	4 (102)	1.00 (no reduction)	12 (305)	4 (102)	0.25	1.00 (no reduction)
	6-1/4 (159)	3,100 (13.8)	3,685 (16.4)							

For SI: 1 inch = 25.4 mm; 1 lbs = 0.0044 kN, 1 psi = 0.006894 MPa.

1. All values are for anchors installed in fully grouted concrete masonry wall construction with materials meeting minimum compressive strength, f'm, of 1,500 psi (10.3 MPa). Concrete masonry units must be light-, medium, or normal-weight conforming to ASTM C90. Allowable loads are based on a safety factor of 5.0.
2. Anchors may be installed in any location in the face of the masonry wall (cell, web, bed joint) except within 1-1/4-inch from the of the vertical mortar joint (head joint), center-to-center, provided the minimum edge and end distances are maintained. Anchors may not be placed in the head joint unless the vertical joint is mortared full-depth.
3. A maximum of two anchors may be installed in a single masonry cell in accordance with the spacing and edge or end distance requirements. Embedment is measured from the outside surface of the concrete masonry unit to the embedded end of the anchor. See the figure for Illustration of Screw-Bolt+ Anchors Installed into Grouted Concrete Masonry Wall.
4. The critical spacing distance, s_{cr}, is the anchor spacing where full load values in the table may be used. The minimum spacing distance, s_{min}, is the minimum anchor spacing for which values are available and installation is permitted. Spacing distance is measured from the centerline to centerline between two anchors.
5. The critical edge or end distance, c_{cr}, is the distance where full load values in the table may be used. The minimum edge or end distance, c_{min}, is the minimum distance for which values are available and installation is permitted. Edge or end distance is measured from anchor centerline to the closest unrestrained edge.
6. The tabulated values are applicable for anchors installed into the ends of grout-filled concrete masonry units (e.g. wall opening) where minimum edge distances are maintained.
7. Load values for anchors installed less than s_{cr} and c_{cr} must be multiplied by the appropriate load reduction factor based on actual spacing (s) or edge distance (c). Load factors are multiplicative; both spacing and edge reduction factors must be considered.
8. Linear interpolation of load values between minimum spacing (s_{min}) and critical spacing (s_{cr}) and between minimum edge or end distance (c_{min}) and critical edge or end distance (c_{cr}) is permitted.
9. See the figure for Direction of Shear Loading in Relation to Edge and End of Masonry Wall figure for illustration of shear load directions.

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PROJ. NAME:
PROJ No.: DATE:
DESIGNED BY: M.E.P PAGE:
REVIEWED BY:

Plate design

Tbolt = 290.3# (Conservative)
ecc = 3.5 inch (Conservative)

Macr = Tperbolt * ecc * #bolt
Macr = 290.3# * 3.5 inch * 3 bolt
Macr = 3048.2 #.in

$$fb = \frac{Macr}{Sx}$$

$$fb = \frac{3048.2 \ \#.in}{0.5104 \ in^3}$$

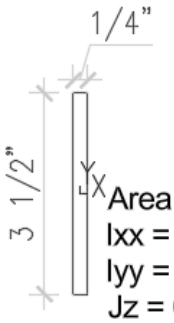
Fb = 21600 psi

fb = 5972.17 #/in2 (psi) Steel

fb < Fb

5972.14 #/in2 < 21600 #/in2

OK!



Area = 0.8750 in^2
Ixx = 0.8932 in^4
Iyy = 0.0046 in^4
Jz = 0.8978 in^4
Pxy = -0.0000 in^4
rxx = 1.0104 in
ryy = 0.0722 in
Sxx max = 0.5104 in^3
Sxx min = 0.5104 in^3
Syy max = 0.0365 in^3
Syy min = 0.0365 in^3

Project Name: **Date:**

Weld Design For Round Pipe

MATERIALS:

Electrodes:
E 70 XX ----Fu= 70 Ksi
E 60 XX ----Fu= 60 Ksi

Weld and Section Data:

$F_u := 70.00$ Ultimate Tensile Strength (ksi)

$r := 0.375$ Pipe Radious (in)

Load Data:

$M_{weld} := 0.045$ Moment (Kips-ft)

$V_{weld} := 0.200$ Shear (Kips)

$P_{weld} := 0.00$ Tension or Compression (Kips)

$T_{weld} := 0.00$ Torsional Moment (kips-ft)

Check Weld Section :

$F_v := 0.3 \cdot F_u$ $F_v = 21.00$ ksi

$t_e :=$ $x \leftarrow 0.0625$

while $\sqrt{\left(\frac{P_{weld}}{2\pi \cdot r \cdot x} + \frac{M_{weld} \cdot 12}{\pi \cdot r^2 \cdot x}\right)^2 + \left(\frac{2V_{weld}}{2\pi \cdot r \cdot x} + \frac{T_{weld} \cdot 12 \cdot r}{2 \cdot \pi \cdot r^3 \cdot x}\right)^2} \geq F_v$

$x \leftarrow x + 0.01$

$t_e = 0.06$ in

$S_{weld} := \pi \cdot r^2 \cdot t_e$ $S_{weld} = 0.03$ in^3

$A_{weld} := 2\pi \cdot r \cdot t_e$ $A_{weld} = 0.15$ in^2

$J_{weld} := 2 \cdot \pi \cdot r^3 \cdot t_e$ $J_{weld} = 0.02$ in^3

Check Bending Stress :

$$f_b := \frac{M_{weld} \cdot (12)}{S_{weld}}$$

$f_b = 19.56$ ksi

Check Shear Stress :

$$f_v := \frac{2V_{weld}}{A_{weld}} + \frac{T_{weld} \cdot 12 \cdot r}{J_{weld}}$$

$f_v = 2.72$ ksi

Check Axial Stress :

$$f_a := \frac{P_{weld}}{A_{weld}}$$

$f_a = 0.00$ Ksi

Check Combined Stress :

$$f_{weld} := \sqrt{(f_b + f_a)^2 + f_v^2}$$

$f_{weld} = 19.74$ ksi

$$STRESS := \begin{cases} \text{"N.G."} \\ \text{"OK"} \quad \text{if } F_v \geq f_{weld} \end{cases}$$

STRESS = "OK"

$$W_{weld} := \begin{cases} \text{"3/16"} \quad \text{if } \frac{t_e}{0.707} \leq 0.1875 \\ \text{" 1/4"} \quad \text{if } 0.1875 < \frac{t_e}{0.707} \leq 0.25 \\ \text{"3/8"} \quad \text{if } 0.25 < \frac{t_e}{0.707} \leq 0.375 \\ \text{"1/2"} \quad \text{if } 0.375 < \frac{t_e}{0.707} \leq 0.50 \\ \text{"5/8"} \quad \text{if } 0.50 < \frac{t_e}{0.707} \leq 0.625 \\ \text{"3/4"} \quad \text{if } 0.625 < \frac{t_e}{0.707} \leq 0.750 \\ \text{"1"} \quad \text{if } 0.75 < \frac{t_e}{0.707} \leq 1.00 \end{cases}$$

$W_{weld} = \text{"3/16"}$