11 April 2018

Hansen Architectural Systems
5500 SE Alexander ST
Hillsboro, OR 97124

SUBJ: ALUMINUM FRAMED RAILING
PICKET, CABLE AND GLASS INFILL SYSTEMS
SERIES 100, 200, 300, 350 AND 400 SERIES SYSTEMS

The Hansen Aluminum Railing System (ARS) utilizes aluminum extrusions with infills to construct building guards and rails for decks, balconies, stairs, fences and similar locations. The system is intended for interior and exterior weather exposed applications and is suitable for use in most natural environments except for highly corrosive environments. The RS may be used for residential, commercial and industrial applications. The ARS is an engineered system designed for the following criteria:

The design loading conditions are:

On Top Rail:

Concentrated load = 200 lbs any direction, any location
Uniform load = 50 plf, any perpendicular to rail

On In-fill Panels:

Concentrated load = 50# on one sf.
Distributed load = 25 psf on area of in-fill, including spaces
Wind load = 28.5 psf typical installation (higher wind loads may be allowed based on post spacing and anchorage method)

Refer to IBC Section 1607.7.1 for loading.

The ARS system will meet or exceed all requirements of the 2000, 2003, 2006, 2009, 2012, 2015 and 2018 *International Building Codes* and *International Residential Codes*, and state building codes based on these versions of the IBC, and the 2015 *Aluminum Design Manual*. Wood components and anchorage to wood are designed in accordance with the 2015 *National Design Specification for Wood Construction*.

Edward Robison, P.E.

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Typical Installations:
Refer to *Guard Posts Mounted To Wood Decks Residential Installations 42" Guard Height* report for other details and mounting requirements for mounting to wood framing in compliance with the 2018 IBC and 2018 IRC.

Surface mounted with base plates:

Residential Applications:
Rail Height 36" or 42" above finish floor.
Standard Post spacing 6' on center maximum.
Bottom rail intermediate post required over 5'.
All top rails

Commercial and Industrial Applications:
Rail Height 42" above finish floor.
Standard Post spacing 5' on center maximum.
All top rails

Core pocket /embedded posts or stainless steel stanchion mounted:

Residential Applications:
Rail Height 36" or 42" above finish floor.
Standard Post spacing 6' on center maximum, series 100
8' on center Series 200, 300, 350 and 400.
Bottom rail intermediate post required over 5'.

Commercial and Industrial Applications:
Rail Height 42" above finish floor.
Standard Post spacing 6' on center maximum, series 100
6' on center Series 200, 300, 350 and 400.

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SIGNED:
11 Apr 2018

Registered Professional Engineer/Structural
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EDWARD C.
ROBISON
Date Signed
ARIZONA U. S. A.
EXP 03/31/2020

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STATE OF MARYLAND
EDWARD C. ROBISON
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PROFESSIONAL ENGINEER

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland,
License No. 52500, Expiration Date: 04/09/2020

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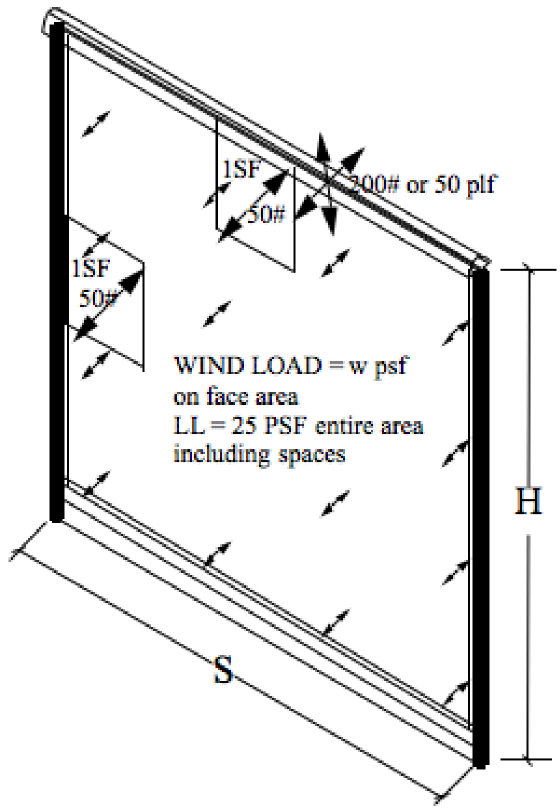
LOAD CASES:

Glass rail Dead load = 5 plf for 42” rail height or less.



Loading:
Horizontal load to top rail from in-fill:
25 psf*H/2
Post moments
M_i = 25 psf*H*S*H/2 =
= 12.5*S*H^2

For top rail loads:
M_c = 200#*H
M_u = 50plf*S*H

For wind load surface area:

M_w = w psf*H*S*H*055 =
= 0.55w*S*H^2

Solving for w :
w = M/(0.55*S*H^2)

Wind load equivalent for 42” rail height, 5’ post spacing 50 plf top rail load:
M_u = 50plf*5’*3.5’ = 875#’ = 10,500#”

w = 875/(0.55*5*3.5^2) = 26 psf

Allowable wind load adjustment for other post spacing:

w = 26*(5/S)

<u>WIND LOADING</u>

For wind load surface area is full area of guard:

Calculated in accordance with ASCE/SEI 7-05 Section 6.5.14 *Design Wind Loads on Solid Freestanding Walls and Solid Signs* **(or ASCE/SEI 7-10 Chapter 29.4)**. This section is applicable for free standing building guardrails, wind walls and balcony railings that return to building walls. Section 6.5.12.4.4 (29.6) *Parapets* may be applicable when the rail is along a roof perimeter. **Wind loads must be determined by a qualified individual for a specific installation.**

$p = q_p(GC_p) = q_zGC_f$ (ASCE 7-05 eq. 6-26 or 7-10 eq. 29.4-1)

G = 0.85 from section 6.5.8.2 (sec 26.9.4.)

$C_f = 2.5*0.8*0.6 = 1.2$ Figure 6-20 (29.4-1) with reduction for solid and end returns, will vary.

$Q_z = K_zK_{zt}K_dV^2I$ Where:

I = 1.0

K_z from Table 6-3 (29.3-1) at the height z of the railing centroid and exposure.

$K_d = 0.85$ from Table 6-4 (Table 26-6).

K_{zt} From Figure 6-4 (Fig 26.8-1) for the site topography, typically 1.0.

V = Wind speed (mph) 3 second gust, Figure 6-1 (Fig 26.5-1A) or per local authority.

Simplifying - Assuming $1.3 \le C_f \le 2.6$ (Typical limits for fence or guard with returns.)

For $C_f = 1.3$: $F = q_h*0.85*1.3 = 1.11\ q_h$

For $C_f = 2.6$: $F = q_h*0.85*2.6 = 2.21q_h$

Wind Load will vary along length of fence in accordance with ASCE 7-05 Figure 6-20 (29.4-1).

Typical exposure factors for K_z with height 0 to 15' above grade:

Exposure	B	C	D
K_z =	0.70	0.85	1.03

MINIMUM WIND LOAD TO BE USED IS 10 PSF.

Centroid of wind load acts at 0.55h on the fence.

Typical wind load range for I = 1.0 and K_{zt} = 1.0

Table 1:	**<u>Wind load in psf $C_f = 1.3$</u>**			**<u>Wind load in psf $C_f = 2.60$</u>**		
Wind Speed	B	C	D	B	C	D
V	$0.00169V^2$	$0.00205V^2$	$0.00249V^2$	$0.00337V^2$	$0.00409V^2$	$0.00495V^2$
85	12.2	14.8	17.9	24.3	29.5	35.8
90	13.7	16.6	20.2	27.3	33.1	40.1
100	16.9	20.5	24.9	33.7	36.9	49.5
110	20.5	24.8	30.1	40.7	49.5	59.9
120	24.3	29.6	35.8	48.5	58.9	71.3
130	28.6	34.7	42.0	56.9	69.1	83.7
140	33.1	40.2	48.8	66.0	80.1	97.1

Where guard ends without a return the wind forces may be as much as 1.667 times C_f=2.6 value.

When I = 0.87 is applicable (occupancy category I) multiply above loads by 0.87.

For wind loads based on ASCE 7-10 wind speeds, figures 26.5-1A, B and C, multiply the wind loads by 0.6 to convert to Allowable Stress Design loads.

For example - Exp B with $C_f = 1.3$; 7-05 wind speed = 85 mph w= 12.2 psf:

7-10 wind speed= 110mph w = 0.6*20.5 = 12.3 psf (ASD wind loads used herein)

GLASS STRENGTH FULLY TEMPERED INFILL PANELS

All glass is fully tempered glass conforming to the specifications of ANSI Z97.1, ASTM C 1048-97b and CPSC 16 CFR 1201. The average Modulus of Rupture for the glass F_r is 24,000 psi. In accordance with UBC 2406.6 or IBC 2407.1.1 glass used as structural balustrade panels shall be designed for a safety factor of 4.0. This is applicable only to structural panels (glass provides support to railing). Glass not used in guardrails may be designed for a safety factor of 2.5 in accordance with ASTM E1300-12a.

Values for the modulus of rupture, F_R, modulus of Elasticity, E and shear modulus, G for glass are typically taken as (see AAMA CW-12-84 *Structural Properties of Glass*) :

F_R = 24,000 psi.

E = 10,400 ksi. While the value of E for glass varies with the stress and load duration this value is typically used as an average value for the stress range of interest.

G = 3,800 ksi: This is rarely used when checking the deflection in glass. The shear component of the deflection tends to be very small, under 1% of the bending component and is therefore ignored.

μ = 0.22 (Typical value of Poisson's ratio for common glasses.

The safety factor of 4 is dictated by the building code (IBC 2407.1.1). It is applied to the modulus of rupture since glass does not have a yield stress.

There is no deflection limits for the glass in guards other than practical limits for the opening sizes, retention in the frames and occupant comfort. Refer to ASTM E 1300-12a for a standard method of calculating deflections but the deflection limits are concerned with glazing in windows and similar parts of the building envelope rather than a free standing guard. IBC 2403.3 applies a limit of L/175 or 3/4" for the supporting frame. From IBC Table 1604.3 footnote h similar types of construction have a limit of L/60. ICC AC 273 *Acceptance Criteria for Handrails and Guards* paragraph 4.2.4 applies a deflection limit of h/12 to the posts and L/96 to the top rail.

The shear strength of glass tracks closely to the modulus of rupture because failure under shear load will be a tensile failure with strength limited by the modulus of rupture. Thus shear loads are transformed using Mohr's circle to determine the critical tension stress to evaluate the failure load. The safety factor of 4 is applicable to this case same as the bending case. Thus the shear stress is limited based on principal stresses of 0 and 6,000 psi to 6,000/2 = 3,000 psi.
Bearing stress can be derived in a similar fashion with the principal stresses being –6,000 psi and 6,000 psi so the bearing stress = 6,000 psi.

Bending strength of glass for the given thickness:

$I = 12"*(t)^3 /12= (t)^3$ in^3/ft

$S = 12"*(t)^2 /6= 2*(t)^2$ in^3/ft

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For lites simply supported on two opposite sides the moment and deflection are calculated from basic beam theory:

$M_w = W*L^2/8$ for uniform load W and span L or

$M_p = P*L/4$ for concentrated load P and span L, highest moment P @ center

Maximum wind loads:

$W = M_a*8/L^2$ for uniform load W and span L (rail to rail distance)

Deflection can be calculated using basic beam theory:

$\Delta = (1-\nu^2)5wL^4/(384EI)$ for uniform load

For concentrated load:

$\Delta = (1-\nu^2)\ PL^3/(48EI)$

Maximum allowable deflection: Use L/60 deflection limit for infill. This will prevent glass from deflecting enough to disengage from the frame.

For uniform load (wind load)

Solving for w

$w = [t^3*1.676*10^8]/L^3$

Solving for L

$L = [(t^3*1.676*10^8)/w]^{1/3}$

Solving for t

$t = [L^3w/(1.676*10^8)]^{1/3}$

For Concentrated load

Solving for P

$P = (8.74*10^6t^3)/L^2$

Solving for L

$L = [8.74*10^6*t^3/P]^{1/2}$

Solving for t

$t = [PL^2/(8.74*10^6)]^{1/3}$

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From IBC 2407 the minimum nominal glass thickness for infill panels in guards is 1/4"

1/4" FULLY TEMPERED GLASS
Weight = 2.89 psi
t_{ave} = 0.223"

For 1/4" glass $S = 2*(0.223)^2 = 0.0995$ in^3/ft
$M_{allowable}$ = 6,000psi*0.0995 in^3/ft = 597#"/ft
For FS = 2.5 (no fall hazard, glass fence or wind screen)
M_{all} = 597"#*4/2.5 = 955"#

Moment for 36" wide lite (infill for 42" rail height) 25 psf or 50 lb load
M_w = 25psf*3'²*12"/'/8= 337.5"#
M_p = 50*36"/4 = 450"#
Moment for 42" wide lite (infill for 48" rail height) 25 psf or 50 lb load
M_w = 25psf*3.5'²*12"/'/8= 459.4"#
M_p = 50*42"/4 = 525"#

for 36" wide lite (infill for 42" rail height)
W = 597"#*8/(3'*36")= 44 psf
for 42" wide lite (infill for 48" rail height)
W = 597"#*8/(3.5'*42")= 32.5 psf

Deflection:
36" wide lite (infill for 42" rail height) 25 psf or 50 lb load
L/60 = 36/60 = 0.60
$\Delta = [(1-0.22^2)*25*36^4/0.25^3]/(9.58 \times 10^9) = 0.27"$
or $\Delta = (1-0.22^2)*50*36^3/(4.992*10^8*0.25^3) = 0.285"$

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3/8” FULLY TEMPERED GLASS

Weight = 4.75 psi
t_{ave} = 0.366”

For 3/8” glass S = 2*(0.366)^2 = 0.268 in^3/ft
$M_{allowable}$ = 6,000psi*0.268 in^3/ft = 1,607#”/ft
For FS = 2.5 (no fall hazard, glass fence or wind screen)
M_{all} = 1,607”#*4/2.5 = 2,571#”

Moment for 36” wide lite (infill for 42” rail height) 25 psf or 50 lb load
M_w = 25psf*3’^{2}*12”/’/8= 337.5”#
M_p = 50*36”/4 = 450”#
Moment for 42” wide lite (infill for 48” rail height) 25 psf or 50 lb load
M_w = 25psf*3.5’^{2}*12”/’/8= 459.4”#
M_p = 50*42”/4 = 525”#

for 36” wide lite (infill for 42” rail height)
W = 1,607”#*8/(3’*36”)= 119 psf
for 42” wide lite (infill for 48” rail height)
W = 1,607”#*8/(3.5’*42”)= 87.5 psf

Deflection:
36” wide lite (infill for 42” rail height) 25 psf or 50 lb load
L/60 = 36/60 = 0.60
$\Delta = [(1-0.22^2)*25*36^4/0.366^3]/(9.58 \times 10^9) = 0.085”$
or $\Delta = (1-0.22^2)*50*36^3/(4.992*10^8*0.366^3) = 0.090”$

Check maximum wind load based on deflection:

36” width $w = [0.366^3*1.676*10^8]/36^3 =$ 175 psf (does not control)
42” width $w = [0.366^3*1.676*10^8]/42^3 =$ 110 psf (does not control)

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LAMINATED GLASS INFILL

The 2015 and 2018 IBC require laminated glass panels where a walking surface is directly below the guard.

Glass sizes checked in this report are 1/4”, 5/16” and 7/16”

Glass is assumed to use a PVB interlayer with a shear modulus (G) of 140psi. This low value for G accounts for high exterior temperatures that may be present in the southern part of the US and Hawaii.

Effective thickness calculated according to ASTM E1300 appendix X11.

Variable	Description
H1 & H2	Glass pane thicknesses
Hv	Interlayer thickness
E	Young's Modulus
g	Shear Modulus
Hs	.5(h1+h2)+hv
Hs;1	hsh1/(h1+h2)
Hs;1	hsh2/(h1+h2)
Is	$h1(hs;2)^2+h2(hs;1)^2$
a	Minimum Pane Width
Γ	$1/(1+9.6(Eishv/(G(ahs)^2))$
hef;w	$\sqrt[3]{((h1)^3+(h2)^3+12\Gamma Is)}$
h1;ef;σ	$\sqrt{((hef;w)^3/(h1+2\Gamma hs;2))}$
h2;ef;σ	$\sqrt{((hef;w)^3/(h2+2\Gamma hs;1))}$

1/4” Laminated Glass:
Tempered+0.06”+tempered, (.102” glass + 0.06” interlayer + .102” glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.102	0.102	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.162	0.081	0.081	0.001338444	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.372604684	0.200887242	0.223453105	0.223453105

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5/16" Laminated Glass:
1/8"+0.06"+1/8", (.115" glass + 0.06" interlayer + .115" glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.115	0.115	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.175	0.0875	0.0875	0.001760938	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.345016429	0.21780446	0.242724016	0.242724016

7/16" Laminated Glass:
3/16"+0.06"+3/16", (.180" glass + 0.06" interlayer + .180" glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.18	0.18	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.24	0.12	0.12	0.005184	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.251798561	0.301209506	0.337137597	0.337137597

Glass Size, t_{ave} (in)	$t_{ef,W}$ (in)	$t_{ef,\sigma}$ (in)	I (in^4/ft)	S (in^3/ft)	W_a (psf)
1/4	0.201	0.223	0.0081206	0.099458	29
5/16	0.218	0.243	0.0103602	0.118098	37
3/8	0.301	0.337	0.0272709	0.227138	98

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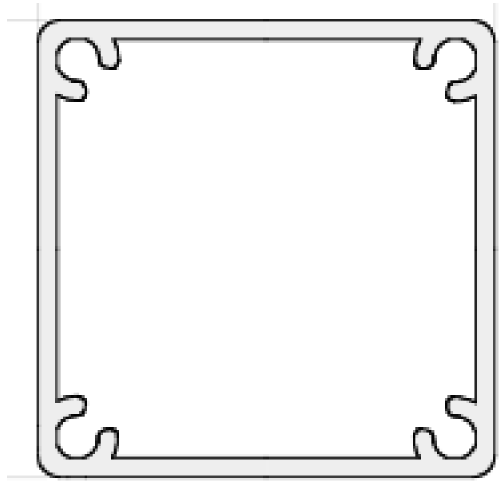
2-3/8" Square Post
6061-T6 Aluminum



4 screw post
-Area 0.995"
$I_{xx} = I_{yy} = 0.863$ in^4
S = 0.726 in^3
Z = 0.9748 in^3
r = 0.923 in
J = 1.341 in^4
k ≤ 1 for all applications

Based on 2015 ADM Chapter F

Lateral torsional buckling:
Lateral torsional buckling may occur on posts that are unrestricted in rotation at the free end. However, typical installations involve a top rail that will restrict the post from rotating and prevent lateral torsional buckling. Testing of the ARS system has never resulted in a lateral torsional buckling failure.

C_b=1.3 for cantilevered beam with concentrated load at free end (ADM 15 F.4.1)
$\lambda = 2.3(L_BS_C/(C_b(I_y J)^{1/2})^{1/2} = 2.3(L_b*.726/(1.3*(.863*1.341)^{1/2}))^{1/2} = 1.657\ L_B^{1/2}$
Inelastic buckling controls when $\lambda<C_c=65.7$
$65.7 = 1.657\ L_B^{1/2}$
L_b=1,572"

For L_b=42"
$\lambda = 1.657*42"^{1/2} = 10.74$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$
M_p=35ksi*.9748in^3=34,118"#
$M_{nmb}=34{,}118(1-10.74/65.7)+\pi^2*10.1*10^6*10.74*.726/65.7^3$=31,281"#
M_{nmb}/Ω = 31,281"#/1.65 = 18,958"#

Above lateral torsional buckling strengths only apply to posts installed without a top rail or some other member to restrict the top of the post against torsion.

Yielding/Rupture/Local Buckling:
Check local buckling of post wall:
b/t=1.562"/0.1"=15.62<20.8
Per ADM 15 Design Aid Table 2-19, F_c/Ω=21.2ksi (Local buckling does not apply)
Z<1.5S
$M_{np}/\Omega = ZF_y/\Omega = 0.9748in^3*21.2ksi = 20,666^{\#}$" or
$M_{nu}/1.95 = ZF_u = 0.9748in^3*38ksi/1.95 = 18,996$"# (Controls)

Bending strength of post installed with top rail:
M_a=19,000"#

Strong axis deflections:
$\Delta = PL^3/(3EI) = PL^3/(3*10,100,000psi*0.863in^4) = PL^3/26,148,900$
$P_{1"} = 26,148,900/L^3$ for 42" post height = 353# (Load for 1" deflection)
$L_{1"} = (26,148,900/P)^{1/3}$ for 250# L = 47.1" (Height for 1" deflection)
For L/12 (maximum allowable post deflection from ASTM E-985 test loads)
$P = EI/(4L^2)$: for 42" height:
$P = 10,100,000psi*0.863in^4/(4*42^2) = 1,235$# - Deflection will not control post loads

For posts directly fascia mounted with 3/8" bolts through post:
Reduced strength at bolt hole:
For loading parallel to bolt axis:
Assume 3/8" + 1/8" over size + 1/8" damage =1/2" holes both sides of post

$S_{red} = 0.6237\ in^3$
$Z_{red} = 0.7590\ in^3$

Addition of holes at base of post only affects rupture strength.

$M_{nu}/\Omega = ZF_u/\Omega = 0.7590in^3*38ksi/1.95 = 14,791$"#

For loading perpendicular to bolt axis
$I_{red}=0.8750in^4$
$S_{red}=0.7365in^3$
$Z_{red}=0.8666in^3$

$M_{nu}/\Omega = ZF_u/\Omega = 0.8666in^3*38ksi/1.95 = 16,888$"#

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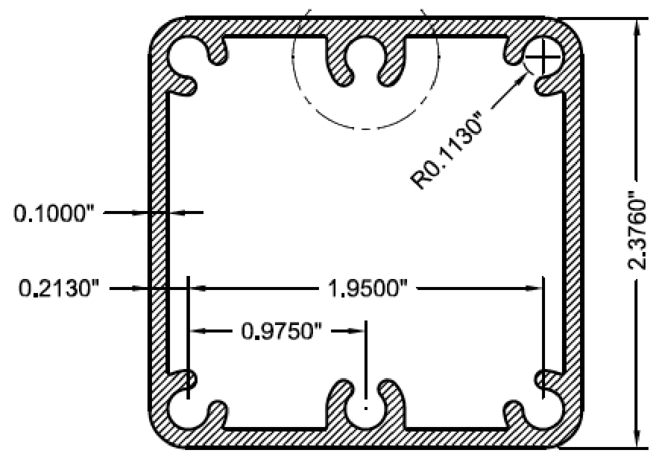
2-3/8" Square Post
6 Screw Post
Post Strength
6005-T5 or 6061-T6
-Area 1.1482"
$I_{xx} = 0.9971 \text{ in}^4$
$I_{yy} = 0.8890 \text{ in}^4$
$S_{xx} = 0.8388 \text{ in}^3$; $Z_{xx} = 0.9996 \text{ in}^3$
$S_{yy} = 0.7482 \text{ in}^3$; $Z_{yy} = 0.9011 \text{ in}^3$
$r_{xx} = 0.9319$ in
$r_{yy} = 0.8799$ in
J = 1.341 in
k ≤ 1 for all applications
Based on 2015 ADM Chapter F



Lateral torsional buckling:
Lateral torsional buckling may occur on posts that are unrestricted in rotation at the free end. However, typical installations involve a top rail that will restrict the post from rotating and prevent lateral torsional buckling. Testing of the ARS system has never resulted in a lateral torsional buckling failure.

C_b=1.3 for cantilevered beam with concentrated load at free end (ADM 15 F.4.1)
$\lambda = 2.3(L_B S_C/(C_b(I_y J)^{1/2})^{1/2} = 2.3(L_b*.8388/(1.3*(.889*1.341)^{1/2}))^{1/2} = 1.768\ L_B^{1/2}$
Inelastic buckling controls when $\lambda < C_c = 65.7$
$65.7 = 1.768\ L_B^{1/2}$
L_b=1,381" > 48" (Much higher than practical post heights)

For L_b=42"
$\lambda = 1.768*42"^{1/2} = 11.46$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2 E\lambda S_{xc}/C_c^3$
M_p=35ksi*.9996in^3=34,986"#
M_{nmb}=34,986(1-11.46/65.7)+π^2*10*10^6*11.46*.8388/65.7^3=32,229"#
M_{nmb}=32,229"#/1.65=19,533"#

Above lateral torsional buckling strengths only apply to posts installed without a top rail or some other member to restrict the top of the post against torsion.

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Yielding/Rupture/Local Buckling:

b/t = 1.95/0.1 = 19.5 < 20.8
F_c/Ω = 21.2 ksi
Z<1.5S
$M_{np}/\Omega = ZF_y/\Omega$ = 0.9996in^3*21.2ksi = 21,192$^{\#}$" or
$M_{nu}/1.95 = ZF_u$ = 0.9996in^3*38ksi/1.95 = 19,479"# (Controls)

Bending strength of post installed with top rail:
M_a=19,500"#

Strong axis deflections:
Δ = PL^3/(3EI) = PL^3/(3*10,100,000psi*0.9971in^4) = PL^3/30,212,130
$P_{1"}$ = 30,212,130/L^3 for 42" post height = 408#
$L_{1"}$ = (30,212,130/P)$^{1/3}$ for 250# L = 49 5/16"
For L/12 (maximum allowable post deflection from ASTM E-985 test loads)
P = EI/(4L^2): for 42" height:
P = 10,100,000psi*0.9971in^4/(4*42^2) = 1,427# - Deflection will not control post loads

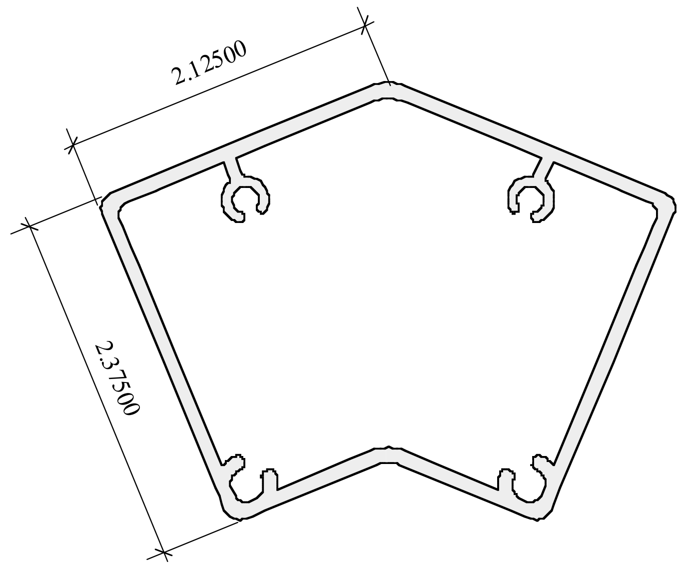
Deflection for 200# load for 42" post height:
Δ = PL^3/(3EI) = 200*42^3/(3*10,100,000psi*0.9971in^4) = 0.49"

For posts directly fascia mounted with 3/8" (7/16" dia holes) bolts through post:
Reduced strength at bolt hole:
Bending perpendicular to bolts
S_{red} = 0.6026 in^3
F_{tb} = 21 ksi at reduced section
M_{red} = 21ksi *0.6026 in^3 = 12,655"#

For bending parallel to bolts:
S_{red} = 0.564 in^3, A_f = 0.125*1.875^2 = 0.439 in^2
F_{tb} = 21 ksi at reduced section
M_{red} = 21ksi *0.564 in^3 = 11,844"#
To allow for shear stress from bolt bearing on post limit moment so that:
M/11,844 +[(T_{bolt}/0.439)/12000]$^2 \leq$ 1.0
For example if bolt tension = 2,000# the maximum allowable moment is:
M_a = {1.0-[(2000/0.439)/12000]^2}*11,844 = 10,137"#

Post 45˚ Corner

6061-T6



Post Section Properties
-Area 1.261”
$I_{xx} = 1.120$ in^4
$I_{yy} = 1.742$ in^4
$S_{xx} = 0.812$ in^3
$S_{yy} = 0.900$ in^3
$Z_{xx} = 1.127$ in^3
$Z_{yy} = 1.340$ in^3
$r_{xx} = 0.975$ in
$r_{yy} = 1.175$ in
J = 1.947 in
k = 1 for all applications

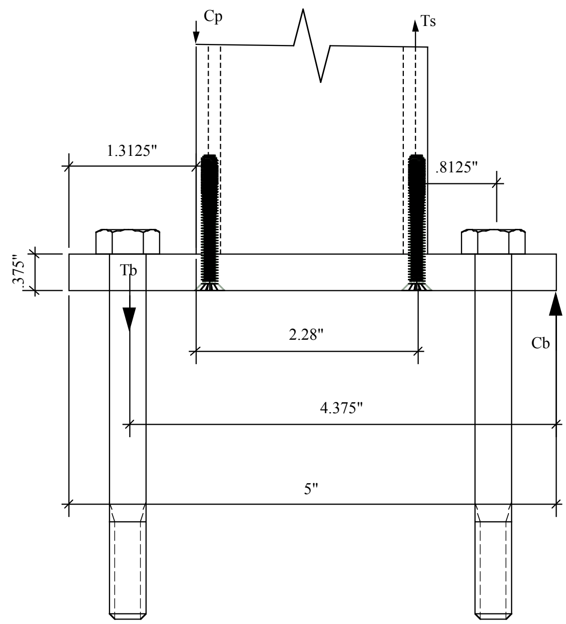
Allowable bending stress
ADM Table 2-21

Lateral torsional buckling will not be a concern for corner posts because they will be braced in multiple directions.

Yielding/Rupture/Local Buckling:
For bending about X-axis
$b/t = 1.75/0.09 = 19.4 < 20.8$
$F_c/\Omega = 21.2$ ksi
Z<1.5S
$M_{np}/\Omega = ZF_y/\Omega = 1.127\text{in}^3 * 21.2\text{ksi} = 23{,}892$#” or
$M_{nu}/1.95 = ZF_u = 1.127\text{in}^3 * 38\text{ksi}/1.95 = 21{,}962$”# (Controls)
For bending about Y-axis
$b/t = 1.812/0.09 = 20.1 < 20.8$
$F_c/\Omega = 21.2$ ksi
Z<1.5S
$M_{np}/\Omega = ZF_y/\Omega = 1.340\text{in}^3 * 21.2\text{ksi} = 28{,}408$#” or
$M_{nu}/1.95 = ZF_u = 1.340\text{in}^3 * 38\text{ksi}/1.95 = 26{,}113$”# (Controls)

Connection to base plate
Post uses standard base plate
Post anchorage methods and strengths are the same as for the square post.

For angles other than 135˚ Use the Adjustable Fastening Plates for Top Rails on either the square or 135˚ posts as needed to achieve the desired angle.



Connection to base plate

Failure modes → screw tension
→ screw shear
→ screw withdrawal

For screw withdrawal
See ADM 5.4
From testing screw engagement in slot is adequate so that failure is consistently screw rupture without withdrawal from the slot.

Base plate to post screws are AISI 4037 steel alloy fabricated in accordance with SAE J429 Grade 8 and coated with Magni 550 corrosion protection.
Refer to base plate attachment strength test report for determination of allowable screw tension strength and allowable moment on the connection.
Average failure moment = 22,226"#
Safety factor calculated in accordance with ADM 9.3.2 = 2.07

Allowable Moment on the base plate to post connection:
$M_{allowable}$ = 22,226"#/2.07 = 10,895"#

Allowable screw tension load:
T_{all} = 10,895"#/(2*2.28") = 2,389# From testing

Calculated strength:
Screw tension → F_{tU} = 0.0376 • 150 ksi = 5,640# Screw rupture on net tension area
For fracture SF = 1.6/(0.9*0.75) = 2.37 → 5,640/2.37 =2,380#

Using the calculated screw strength
M_{all} = 2 • 2,380# • 2.28" = 10,852"#

Base plate bending stress

$F_t = 24\text{ ksi} \rightarrow S_{min} = \frac{5" \bullet 3/8^2}{6} = 0.117\text{ in}^3$

Base plate allowable moment

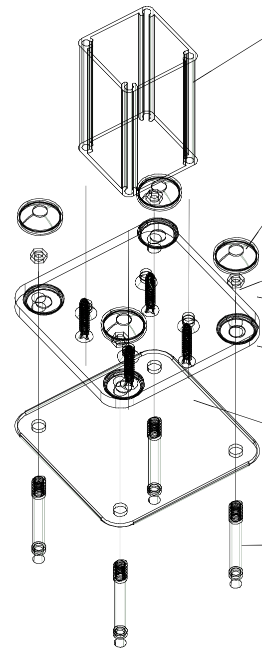
$M_{all} = 24\text{ ksi} \bullet 0.117\text{ in}^3 = 2{,}812\text{ "}^{\#}$

→ Base plate bending stress

$T_B = C$

$M = 0.8125" \bullet T_B \bullet 2$

$T_{all} = \frac{2{,}812}{2 \bullet 0.8125} = 1{,}730^{\#}$



Maximum post moment for base plate strength

$M_{all} = 2 \bullet 1{,}730 \bullet 4.375" = 15{,}142^{\#}\text{"}$

Limiting factor = screws to post

$M_{ult} = 2 \bullet 5{,}314^{\#} \bullet 2.28" = 24{,}232^{\#}\text{"}$

$M_{all} = 2 \bullet 2{,}293^{\#} \bullet 2.28" = 10{,}500^{"\#}$

Refer to *Guard Rail Post To Base Plate Screw Connection Strength* report dated 11/22/2010 by this engineer for testing results. Testing has confirmed that screws fail in tension and not pullout from the screw slot, 2015 ADM J5.4.1.2 is not applicable based on testing.

BASE PLATE ANCHORAGE

$T_{Des} = \frac{10{,}500}{2 \bullet 4.375"} = 1{,}195^{\#}$

adjustment for concrete bearing pressure:

a = 2*1,195/(2*3000psi*4.75") = 0.087"

$T'_{Des} = \frac{10{,}500}{2 \bullet (4.375"-0.087/2)} = 1{,}206^{\#}$

For 200# top load and 42" post ht

$T_{200} = \frac{8{,}400}{2*4.375"} = 960\#$

For 42" post height the maximum live load at the top of the post is:

$P_{max} = 10{,}500^{"\#}/42" = 250^{\#}$

For 50 plf live load maximum post spacing is:

$S_{max} = 250^{\#}/50\text{ plf} = 5' = 5'0"$

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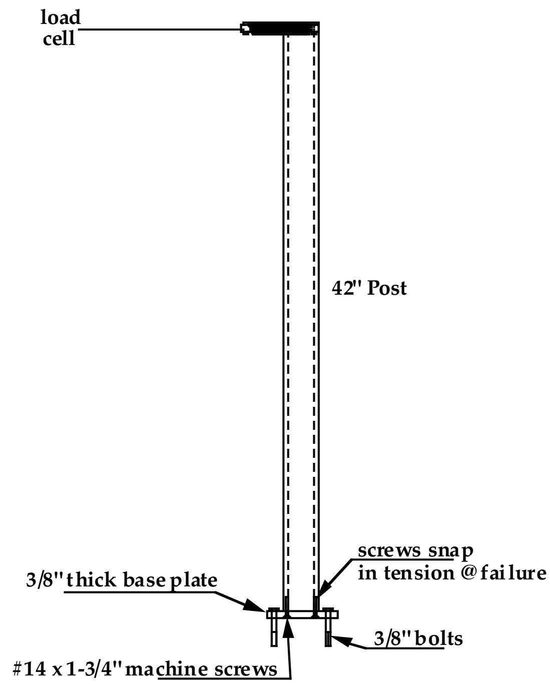
LOAD TESTS:
Connection strength from load testing post/base plate assemblies:
42” from top of base plate to centerline of load.



M_{fail} = (524.2#*42”)= 22,226”#
Based on 7 load tests performed by Edward C. Robison, P.E.
Load tests – minimum failure load at 42” post height = 524.2#, failure range = 515# to 540# (variation under 5%). The failure load based on the load tests is 8.8% below the load predicted by the calculations for screw rupture (observed failure mode) because of the prying action which occurs from the base plate bending as the load increases to failure.

From ADM 9.3.2 Tests for Determining Structural Performance:

$$SF = \frac{(1.05\alpha+1)}{M_M F_M (\alpha+1)} e^{\{-\beta_o\sqrt{[V_M^2+V_F^2+C_P V_P^2+V_Q^2]}\}}$$

Where: $M_M = 1.10$, $F_M = 1.00$, $V_M = 0.06$, $V_Q = 0.21$, $\beta_o = 3.5$, $V_F = 0.05$, $V_P = 0.0192$
MM = 1.10 selected because strength is controlled by steel screw not aluminum failure.
$C_P = (n^2-1)/(n^2-3n) = (7^2-1)/(7^2-3*7) = 1.71$; $\alpha = 0.2$
$SF = (1.05*0.2+1)/[1*1.1*(0.2+1)*e\{3.5\sqrt{[0.06^2+0.05^2+1.71*0.0146^2+0.21^2]} = 2.07$

From test strengths
$M_{allowable}$ = 22,226”#/2.07 = 10,895”#

Test	Max. Load	Failure Mode	Comments
#1	516#	Screw fracture	Powers® Double Acting Anchors with 3/8” bolts

On test the anchors were slipping at 400# load allowing the base plate deflection to increase significantly and increasing the prying forces on the screws reducing the ultimate load.

Tests 1- 5: Red Head Tru-Bolt wedge anchors, 3/8” x 3-3/4” with 2-5/8” minimum embedment.

#2	523#	Screw fracture	1 anchor slipped at 400#
#3	515#	Screw fracture	1 anchor slipped at 401#
#4	520#	Screw fracture	1 anchor slipped at 383#
#5	532#	Screw fracture	1 anchor slipped at 320#
#6	524#	Screw fracture	3/8” bolt to steel beam
#7	540#	Screw fracture	3/8” bolt to steel beam

Average failure load at screw fracture = 529.2#
Coefficient of variation = 0.0146

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RAISED BASEPLATE DESIGN AND ANCHORAGE –

Baseplates are raised up and bear on nuts installed on epoxy anchored threaded rod.

Guard rail Height: 42”

loading: 200# concentrated load or

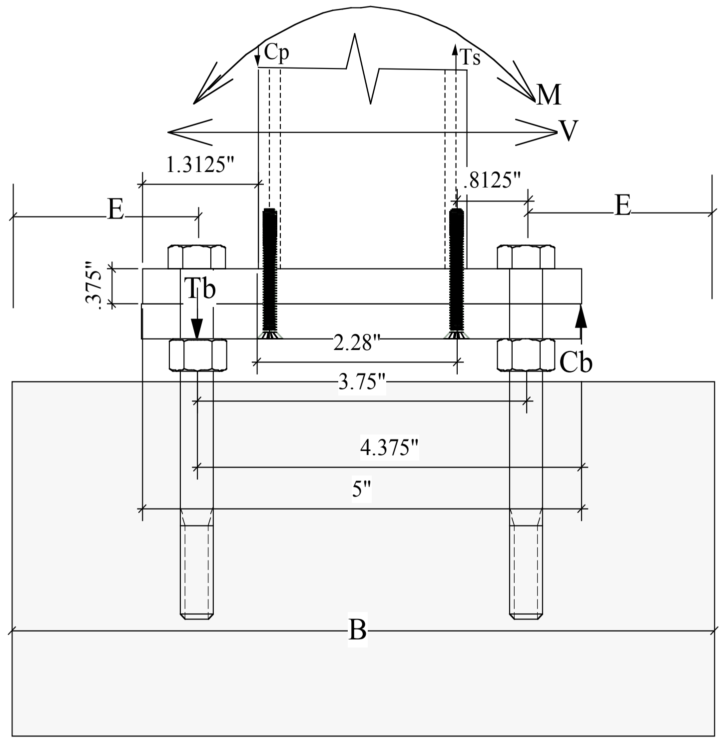
50 plf uniform load on top rail

or

25 psf distributed load on area

or

25 psf = 80 mph exp C wind load:



Design moment on posts:

M_l = 42”*200# = 8,400”#

M_l = 42”*50plf*5ft = 10,500”#

M_w = 3.5’*5’*25psf*42”/2 = 9,188”#

Design anchorage for 10,500”# moment.

Design shear = 438# (wind)

Bolt tension for typical design

T=10,500/(2*3.75)=1,400#

Anchor to concrete:

3/8” x 5” all-thread embedment depth = 3.5” and 4,000 psi concrete strength.

Hilti HIT-RE 500SD per ESR-2322, Simpson Set-XP per ESR-2508 or other adhesive capable of developing the required strength.

T = 2,700# Adjustment for anchor spacing = 3.75”

C_s@ 3.75” = 1-0.20[(5.625-3.75)/4.5] = 0.917

Adjustment for edge distance = 2-1/8”

C_e = 1-0.30[(3.375-2.125)/2.25] = 0.833

T’ = 2,700#*0.917*0.833 = 2,062#

Check base plate strength: Bending is biaxial because it sits on bearing nuts:

M = (3.75”-2.28”)/2*1,400#*2*√2 = 2,910”#

Bending stress in plate

The effective width at the post screws: 3.86”

S = 2*3.86”*0.375^2/6 = 0.181 in^3

f_b = 2,910/0.181 = 16,080 psi

Allowable = 19 ksi

Bearing on nut:

Area = $(0.8^2\text{-}0.5625^2)\pi = 1.0\ in^2$

f_B = 1,400#/1.0 = 1,400 psi - Okay

Screws to post – okay based on standard base plate design

Posts okay based on standard post design

OFFSET BASE PLATE

Offset base plate will have same allowable loads as the standard base plate.

Anchors to concrete are same as for standard base plate.

BASEPLATE MOUNTED TO WOOD – SINGLE FAMILY RESIDENCE

For 200# top load and 36" post height:

M = 200#*36" = 7,200"#

$T_{200} = \dfrac{7{,}200}{2*4.375"} = 823\#$

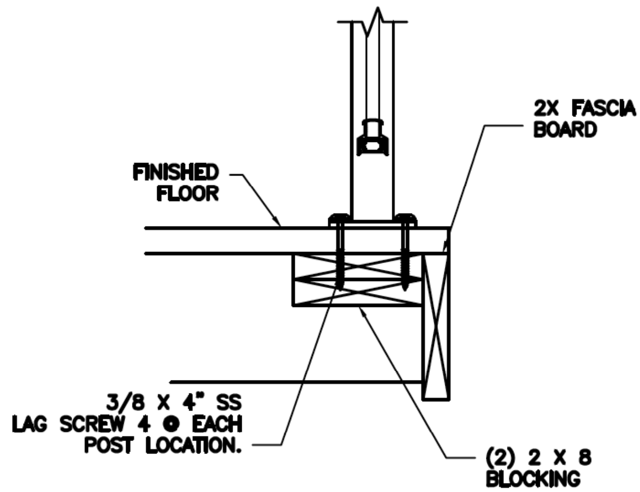
Adjustment for wood bearing:

Bearing Area Factor:

C_b = (5"+0.375)/5" = 1.075

a = 2*823/(1.075*625psi*5") = 0.49"

T = 7,200/[2*(4.375-0.49/2)] = 872#



Required embed depth:

For protected installations the minimum embedment is:

l_e = 872#/323#/in = 2.70" : +7/32" for tip = 2.92"

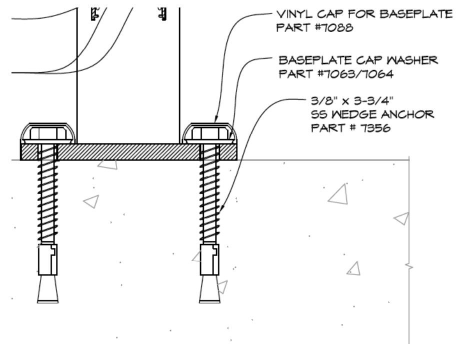
For weather exposed installations the minimum embedment is:

l_e = 872#/243#/in = 3.59" : +7/32" for tip = 3.81"

FOR WEATHER EXPOSED INSTALLATIONS USE 5" LAG SCREWS AND INCREASE BLOCKING TO 4.5" MINIMUM THICKNESS.

REFER TO *GUARD POSTS MOUNTED TO WOOD DECKS RESIDENTIAL INSTALLATIONS 42" GUARD HEIGHT* REPORT FOR OTHER DETAILS AND MOUNTING REQUIREMENTS FOR MOUNTING TO WOOD FRAMING. MAY BE USED FOR COMMERCIAL APPLICATIONS AT 4' POST SPACING.

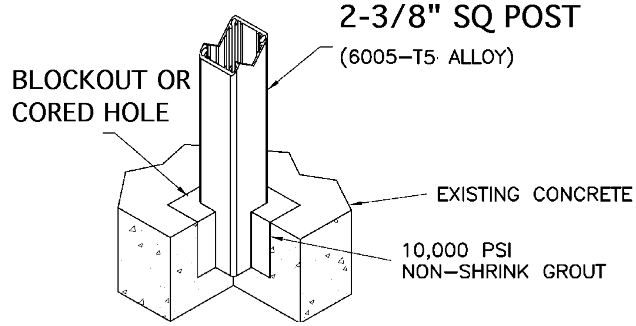
BASE PLATE MOUNTED TO UNCRACKED CONCRETE - Expansion Bolt Alternative:
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8”x3.75” concrete anchors with 3” effective embedment. Anchor strength based on ESR-2427
Minimum conditions used for the calculations:



$f'_c \geq 3{,}000$ psi; edge distance =2.25” spacing = 3.75”
h = 3.0”: embed depth
For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
$A_{Ncg} = (1.5*3*2+3.75)*(1.5*3+2.25) = 86.06\ in^2$ 2 anchors
$A_{Nco} = 9*3^2 = 81\ in^2$
$C_{a,cmin} = 1.5$” (ESR-2427 Table 3)
$C_{ac} = 5.25$” (ESR-2427 Table 3)
$\varphi_{ed,N} = 1.0$
$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)
$\varphi_{cp,N} =$ max (1.5/5.25 or 1.5*3”/5.25) = 0.857 ($c_{a,min} \leq c_{ac}$)
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$
$N_{cb} = 86.06/81*1.0*1.0*0.857*6{,}830 = 6{,}219 \leq 2*4{,}200$
based on concrete breakout strength.
Determine allowable tension load on anchor pair
$T_s = 0.65*6{,}219\#/1.6 = 2{,}526\#$
Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
$A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03$
$A_{vco} = 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$
$\varphi_{ed,V} = 1.0$ (affected by only one edge)
$\varphi_{c,V} = 1.4$ uncracked concrete
$\varphi_{h,V} = \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} = 1.225$
$V_b = [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5} = 1{,}636\#$
$V_{cb} = 43.03/40.5*1.0*1.4*1.225*1{,}636\# = 2{,}981\#$
Steel shear strength = 1,830#*2 = 3,660
Allowable shear strength
$ØV_N/1.6 = 0.70*2{,}981\#/1.6 = 1{,}304\#$
Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load:
$M_a = 2{,}526\#*4.375” = 11{,}053”\# > 10{,}500”\#$
DEVELOPS FULL BASEPLATE MOUNTING STRENGTH.
ALLOWABLE SUBSTITUTIONS: Use same size anchor and embedment
Hilti Kwik Bolt TZ in accordance with ESR-1917
Powers Power Stud+ SD2 in accordance with ESR-2502
Powers Wedge-Bolt+ in accordance with ESR-2526
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

CORE MOUNTED POSTS

Mounted in either 4"x4"x4" blockout, or 2-3/8" to 6" dia by 4" deep cored hole.
Assumed concrete strength 2,500 psi for existing concrete

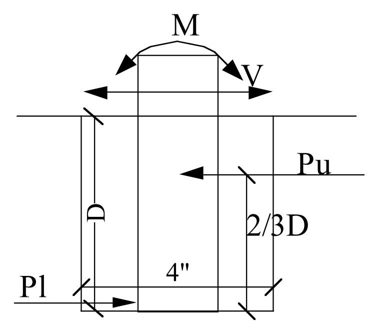


Max load – 6'•50 plf = 300#
M = 300#•42" = 12,600"#

Check grout reactions
From $\Sigma M_{PL} = 0$

$$P_U = \frac{12{,}600"\# + 300\# \cdot 3.33"}{2.67"} = 5{,}093\#$$

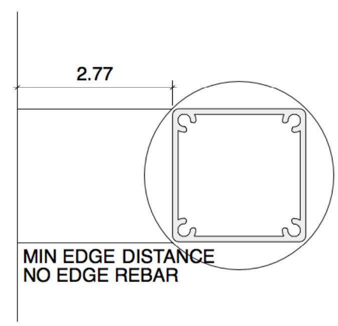
$$f_{Bmax} = \frac{5{,}093\#\cdot 2\cdot 1/0.85}{2"\cdot 2.375"} = 2{,}523 \text{ psi post to grout}$$



$f_{Bconc} = 2{,}523\cdot 2"/4" = 1{,}262$ psi grout to concrete

Core mount okay for 6' post spacing

MINIMUM EDGE DISTANCE:
When #4 or larger rebar is installed along slab edge between the post and slab edge the minimum edge distance from edge of hole to slab edge is 1-1/4".



When no rebar is present required edge distance:
Assume that embed is only near one edge and that slab thickness is greater than $1.5C_{a1}$

Design as 2-way shear:
Three sided breakout surface
Length of perpendicular break = $2.375"+3*C_{a1}$
Length of parallel breaks = $2"+1.5C_{a1}$
$b_o = 2.375"+3*C_{a1}+2*(2"+1.5C_{a1})$
$\boldsymbol{\beta} = (2.375"+3*C_{a1})/(2"+1.5C_{a1})$
$V_{n,min} = V*LF/ø = 5093\#*1.6/0.75 = 10{,}865\#$

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λ	f'c	β	αs	d	bo
1	3000	1.70922661	30	2.3923629	20.7291774

ACI Table 22.6.5.2		
vc		
Least of:	$4\lambda\sqrt{f'c}$	219.089023
	$(2+4/\beta)\lambda\sqrt{f'c}$	237.72472
	$(2+\alpha_s d/b_o)\lambda\sqrt{f'c}$	299.183169
	$v_c d b_o$	**10865.0004**

$C_{a,min}$ = 2.39" measured from the face of the post
= 2.39"+2.375"/2 = 3.58" measured from the center of the post

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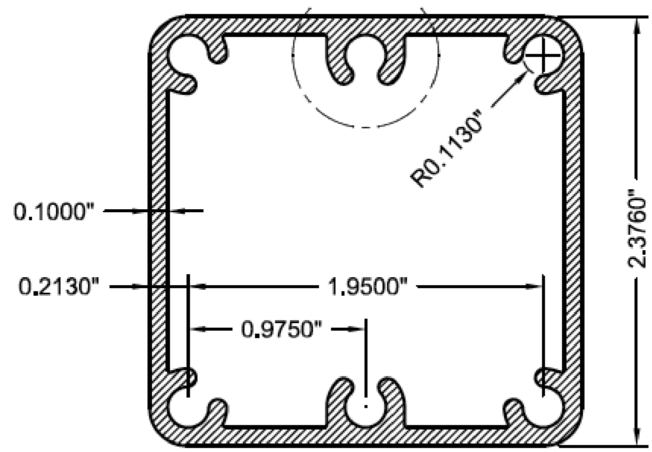
SIX SCREW POST – 2-3/8" Square



Post Strength
6005-T5 or 6061-T6
-Area 1.1482"
$I_{xx} = 0.9971$ in^4
$I_{yy} = 0.8890$ in^4
$S_{xx} = 0.8388$ in^3 ; $Z_{xx} = 0.9996$ in^3
$S_{yy} = 0.7482$ in^3; $Z_{yy} = 0.9011$ in^3
$r_{xx} = 0.9319$ in
$r_{yy} = 0.8799$ in
J = 1.341 in
k ≤ 1 for all applications
Based on 2015 ADM Chapter F

Lateral torsional buckling:
Lateral torsional buckling may occur on posts that are unrestricted in rotation at the free end. However, typical installations involve a top rail that will restrict the post from rotating and prevent lateral torsional buckling. Testing of the ARS system has never resulted in a lateral torsional buckling failure.

C_b=1.3 for cantilevered beam with concentrated load at free end (ADM 15 F.4.1)
$\lambda = 2.3(L_B S_C/(C_b(I_y J)^{1/2})^{1/2} = 2.3(L_b*.8388/(1.3*(.889*1.341)^{1/2}))^{1/2} = 1.768\ L_B^{1/2}$
Inelastic buckling controls when $\lambda < C_c = 65.7$
$65.7 = 1.768\ L_B^{1/2}$
L_b=1,381" > 48" (Much higher than practical post heights)

For L_b=42"
$\lambda = 1.768*42"^{1/2} = 11.46$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2 E\lambda S_{xc}/C_c^3$
M_p=35ksi*.9996in^3=34,986"#
$M_{nmb}=34{,}986(1-11.46/65.7)+\pi^2*10*10^6*11.46*.8388/65.7^3=32{,}229$"#
M_{nmb}=32,229"#/1.65=19,533"#

Above lateral torsional buckling strengths only apply to posts installed without a top rail or some other member to restrict the top of the post against torsion.

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Yielding/Rupture/Local Buckling:

b/t = 1.95/0.1 = 19.5 < 20.8
F_c/Ω = 21.2 ksi
Z<1.5S
$M_{np}/\Omega = ZF_y/\Omega$ = 0.9996in^3*21.2ksi = 21,192$^{\#"}$ or
$M_{nu}/1.95 = ZF_u$ = 0.9996in^3*38ksi/1.95 = 19,479"# (Controls)
Weak axis bending = 0.9011in^3*38ksi/1.95 = 17,560"# (Controls for weak axis bending)

Bending strength of post installed with top rail:
M_a = 19,500"#

Strong axis deflections:
$\Delta = PL^3/(3EI) = PL^3/(3*10,100,000psi*0.9971in^4) = PL^3/30,212,130$
$P_{1"} = 30,212,130/L^3$ for 42" post height = 408#
$L_{1"} = (30,212,130/P)^{1/3}$ for 250# L = 49 5/16"
For L/12 (maximum allowable post deflection from ASTM E-985 test loads)
$P = EI/(4L^2)$: for 42" height:
P = 10,100,000psi*0.9971in^4/(4*42^2) = 1,427# - Deflection will not control post loads

Deflection for 200# load for 42" post height:
$\Delta = PL^3/(3EI) = 200*42^3/(3*10,100,000psi*0.9971in^4) = 0.49"$

For posts directly fascia mounted with 3/8" (7/16" dia holes) bolts through post:
Reduced strength at bolt hole:
Bending perpendicular to bolts
S_{red} = 0.6026 in^3
F_{tb} = 21 ksi at reduced section
M_{red} = 21ksi *0.6026 in^3 = 12,655"#

For bending parallel to bolts:
S_{red} = 0.564 in^3, $A_f = 0.125*1.875^2 = 0.439$ in^2
F_{tb} = 21 ksi at reduced section
M_{red} = 21ksi *0.564 in^3 = 11,844"#
To allow for shear stress from bolt bearing on post limit moment so that:
$M/11,844 + [(T_{bolt}/0.439)/12000]^2 \leq 1.0$
For example if bolt tension = 2,000# the maximum allowable moment is:
$M_a = \{1.0-[(2000/0.439)/12000]^2\}*11,844 = 10,137"\#$

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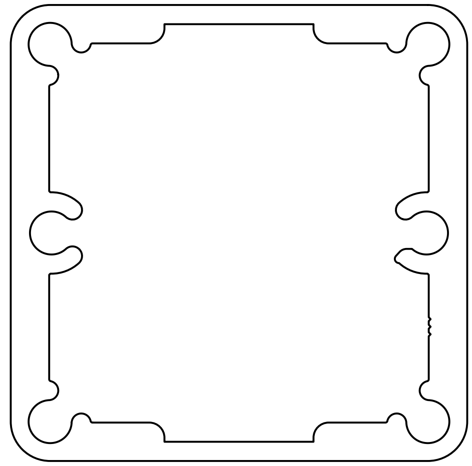
Heavy Post
6061-T6 Aluminum



Heavy posts are typically used for cable rail corner and end posts that receive high cable loading.

-Area 1.4927 in^2
$I_{xx} = 1.0757 in^4$
$I_{yy} = 1.2643 in^4$
$S_x = 0.88888\ in^3$
$S_y = 1.0062\ in^3$
$Z_x = 1.131\ in^3$
$Z_y = 1.347\ in^3$
J = 2.34 in
$k \leq 1$ for all applications

Allowable bending stress ADM Table 2-19

For thick wall post, lateral torsional buckling and local buckling do not control.

Yielding/Rupture Strength
$F_y/\Omega = 35ksi/1.65 = 21.2ksi$
$F_u/\Omega = 38ksi/1.95 = 19.5ksi$ (Controls)

$M_{all}(x) = ZF_{tu}/k_t = 1.131*19.5ksi/1 = 22{,}055"\#$
$M_{all}(y) = ZF_{tu}/k_t = 1.347*19.5ksi/1 = 26{,}267"\#$

$\Delta = PL^3/(3EI) = PL^3/(3*10{,}100{,}000psi*1.0757in^4) = PL^3/32{,}593{,}710$
$P_{1"} = 32{,}593{,}710/L^3$ for 42" post height = 440#
$L_{1"} = (32{,}593{,}710/P)^{1/3}$ for 250# L = 50.7"
For L/12 (maximum allowable post deflection from ASTM E-985 test loads)
$P = EI/(4L^2)$: for 42" height:
$P = 10{,}100{,}000psi*1.0757in^4/(4*42^2) = 1{,}540\#$ - Deflection will not control post loads

Deflection for 200# load for 42" post height:
$\Delta = PL^3/(3EI) = 200*42^3/(3*10{,}100{,}000psi*1.0757in^4) = 0.45"$

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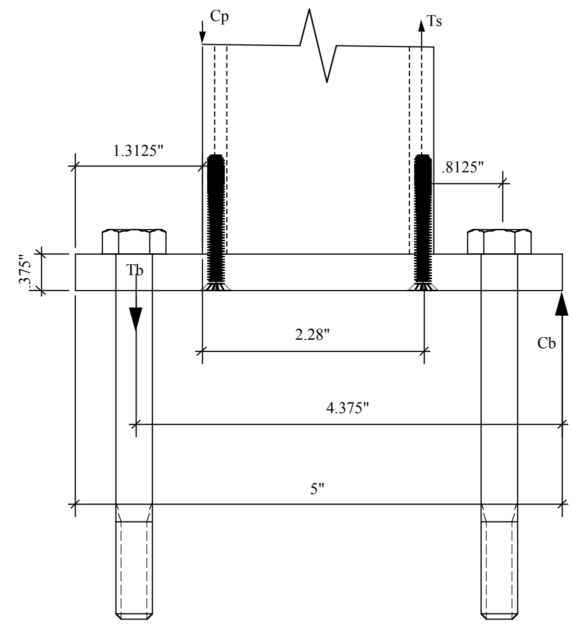
SIX SCREW CONNECTION TO BASE PLATE



Screws are the same as for the standard 4 screw connection.

Screw embedment length into the screw slots is adequate to develop the full screw tension strength.

Use same screw tension strength as used for the four screw connection:

T_a = 2,293# per screw
V_a = 917# per screw

V_{des} = 6*917 = 5,502#
limiting shear load on post so that screw shear stress doesn’t reduce the allowable tension:
$V_{0.2}$ = 0.2*5,502# = 1,100#

Base plate thickness and strength same as for standard post.

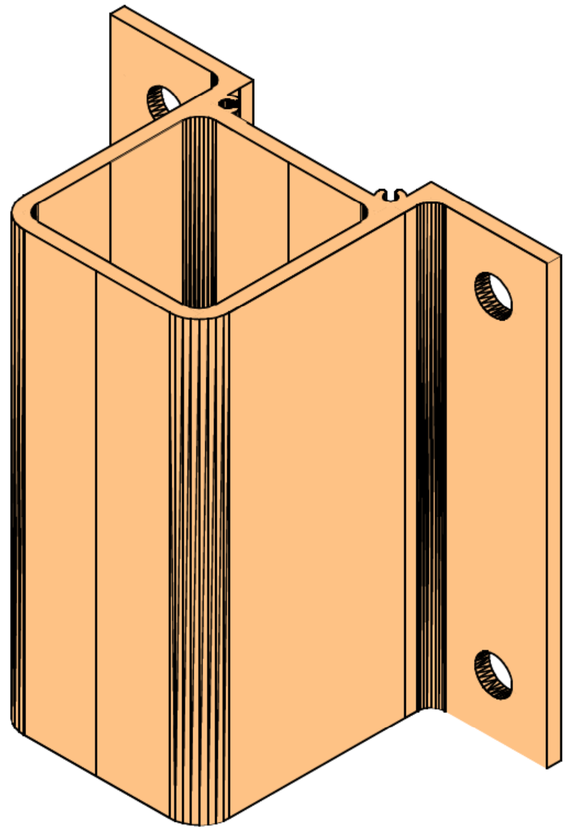
Allowable moment on the posts based on screw tension strength:

Strong axis bending -
M_{base} = 3 screws*2,293#*2.28” = 15,684”# < 19,479”#
Doesn’t develop full post strength.

Weak axis bending -
M_{base} = 2 screws*2,293#*2.28”+ 2 screws*0.5*2,293#*2.28”/2 = 13,070”# ≤ 17,560”#”#
6 screw connection won’t develop the full post strength for weak axis bending.

LIMITING POST MOMENTS FOR SIX SCREW CONNECTION:
STRONG AXIS BENDING M_A = 15,684”# = 1,307’#
WEAK AXIS BENDING M_A = 13,070”# = 1,089’#

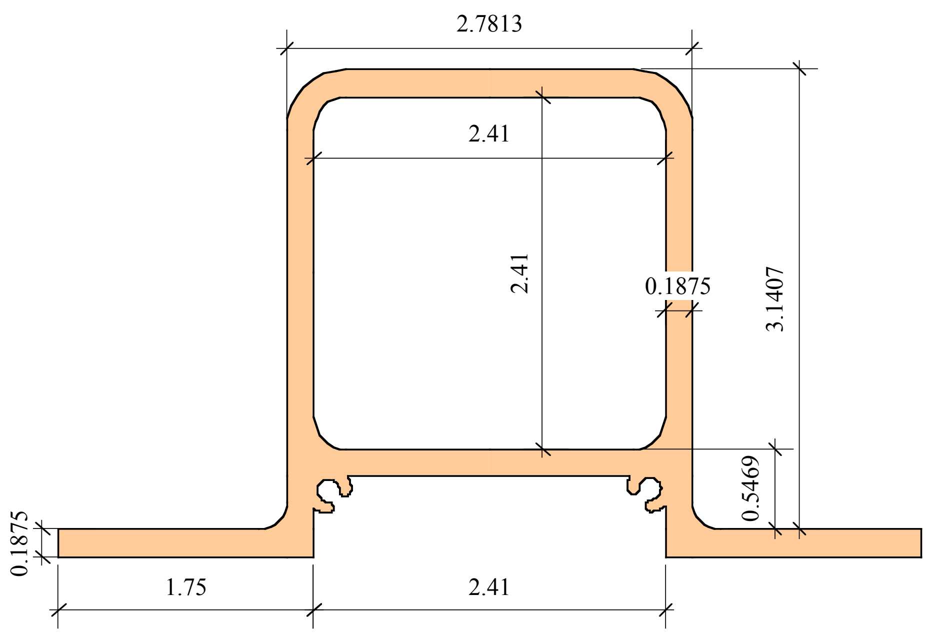
FASCIA BRACKET



Allowable stresses
ADM Table 2-24 6063-T6 Aluminum

Ft = 15 ksi, uniform tension
Ft = 20 ksi, flat element bending
F_B = 31 ksi
Fc = 20 ksi, flat element bending

Section Properties
Area: 2.78 sq in
Perim: 28.99 in
I_{xx}: 3.913 in^4
I_{yy}: 5.453 in^4
C_{xx}: 1.975 in/1.353 in
C_{yy}: 2.954 in
S_{xx}: 1.981 in^3 front
S_{xx}: 2.892 in^3
S_{yy}: 1.846 in^3



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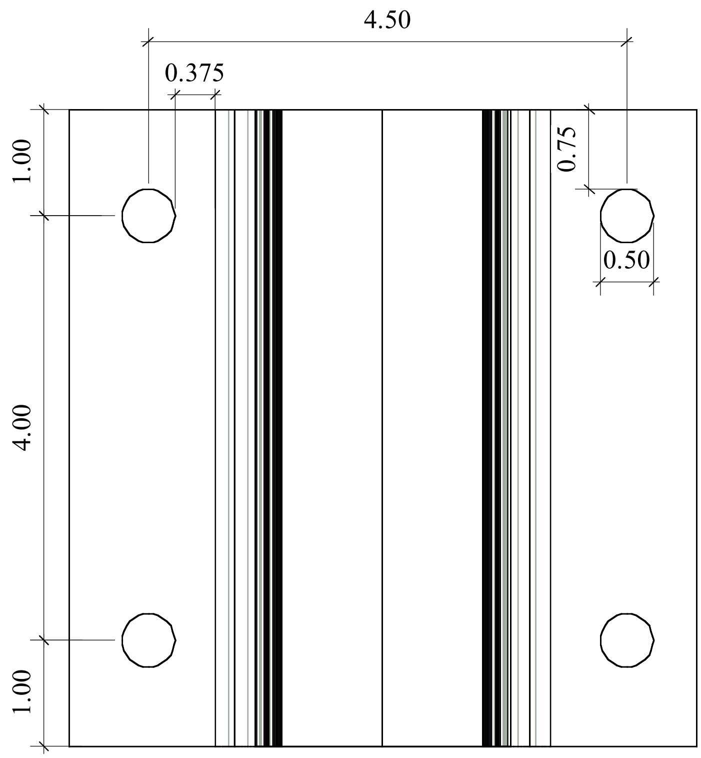
Allowable moment on bracket:

$M_a = F_t*S$

M_{axx} = 15 ksi*1.981 in^3 = 29,175"# - Outward moment

M_{ayy} = 15 ksi*1.846 in^3 = 27,690"# - Sidewise moment



Flange bending strength
Determine maximum allowable bolt load:

Tributary flange
b_f= 8t = 8*0.1875 = 1.5" each side of hole
b_t =1.5"+1"+0.5"+1.75" = 4.75"

S = 4.75"*0.1875^2/6=0.0278 in^3
M_{af} = 0.0278 in^3*20 ksi = 557"#

Allowable bolt tension
T = M_{af}/0.375 = 1,485#
3/8" bolt standard washer

For Heavy washer
T=M_{af}/0.1875= 2,971#

Typical Installation – Post load = 250# at 42" AFF – Top hole is 3" below finish floor

T_{up} = [250#*(42"+ 7")/5"]/2 bolts = 1,225# tension

T_{bot} = [250#(42"+3")/5"]/2 bolts = 1,125# tension

For centerline holes:
T = [250#*(42"+ 5")/3"]/2 bolts = 1,958# tension

For lag screws into beam face:
- 3/8" lag screw – withdrawal strength per 2015 NDS Table 12.2A

Wood species – G ≥ 0.43 – W = 243#/in

Adjustments – Cd = 1.33, Cm = 0.75 (where weather exposed)

No other adjustments required.

W' = 243#/in*1.6 = 389 #/in – where protected from weather

W' = 243#/in*1.6*0.7 = 272#/in – where weather exposed

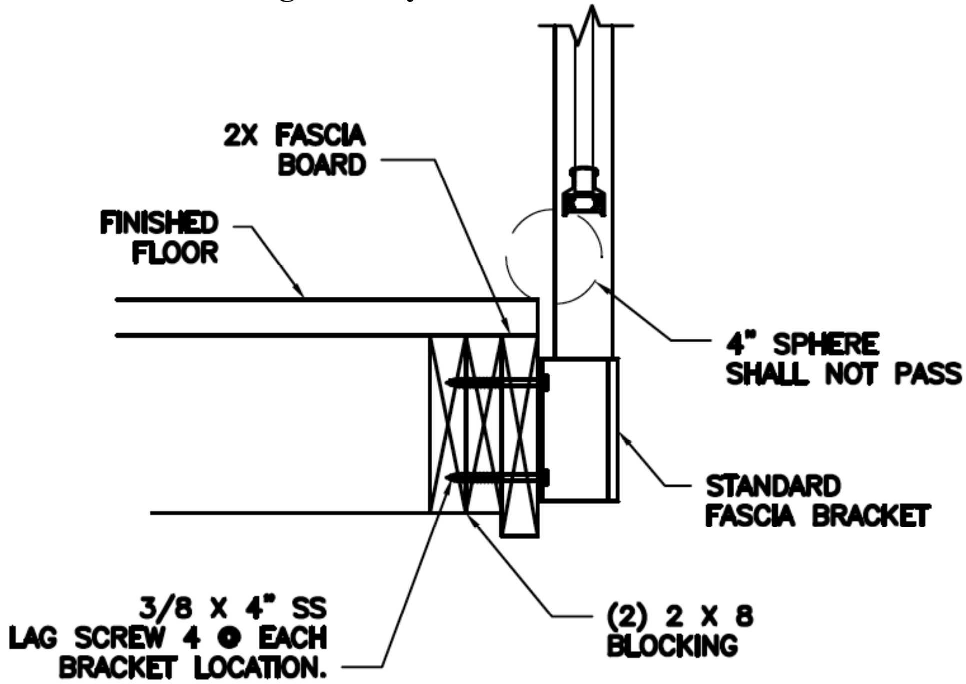
For protected installations the minimum embedment is:

l_e = 1,225#/389#/in = 3.15" : +7/32" for tip = 3.37"

For weather exposed installations the minimum embedment is:

l_e = 1,225#/272#/in = 4.50" : +7/32" for tip = 4.72" requires 5-1/2" screw

Fascia Brackets- Single Family Residence installations to wood deck:



Typical Installation – Post load = 200# at 36" AFF – Top hole is 3" below finish floor

T_{up} = [200#*(36"+ 7")/5"]/2 bolts = 860# tension

T_{bot} = [200#(36"+3")/5"]/2 bolts = 780# tension

For protected installations the minimum embedment is:

l_e = 860#/323#/in = 2.66" : +7/32" for tip = 2.88"

For weather exposed installations the minimum embedment is:

l_e = 860#/243#/in = 3.54" : +7/32" for tip = 3.76"

4" lag screws are acceptable for installation with 36" guard height on residential decks. Backing may be either built-up 2x lumber or solid beams.

Typical Installation – Post load = 200# at 42" AFF – Top hole is 3" below finish floor

T_{up} = [200#*(42"+ 7")/5"]/2 bolts = 980# tension

T_{bot} = [200#(42"+3")/5"]/2 bolts = 900# tension

For protected installations the minimum embedment is:

l_e = 980#/323#/in = 3.03" : +7/32" for tip = 3.25"

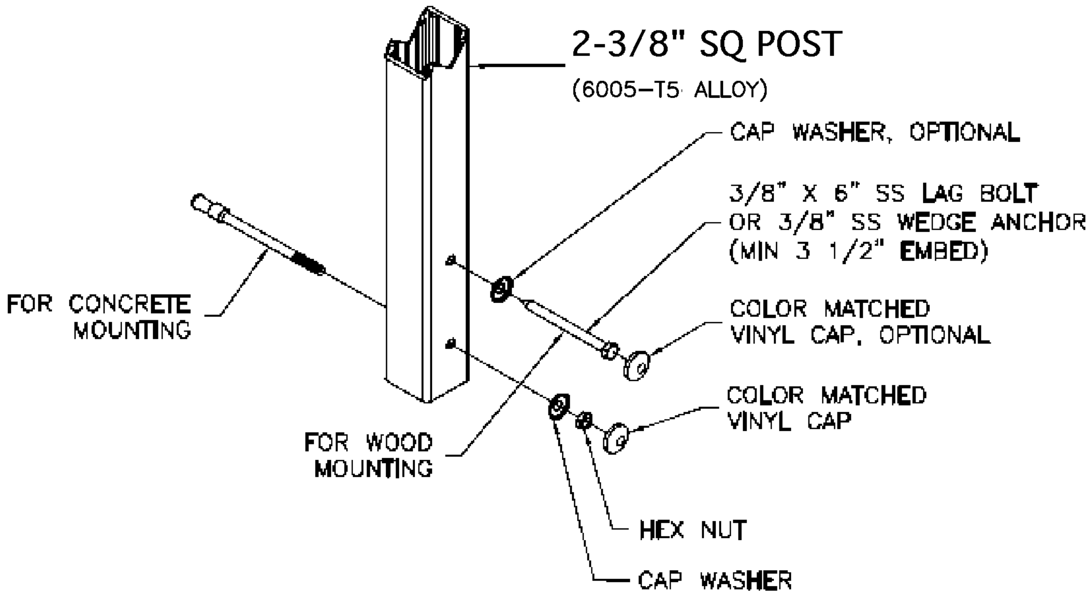
For weather exposed installations the minimum embedment is:

l_e = 980#/243#/in = 4.03" : +7/32" for tip = 4.25"

5" lag screws are required for installation with 42" guard height on residential decks. Backing may be either built-up 2x lumber or solid beams.

FASCIA MOUNTED POST

Commercial application – Load = 200# or 50 plf any direction on top rail



For 42” rail height and 4’ on center post spacing:

P = 200# or 50plf*4 = 200#

M_{deck} = 42”*200plf = 8,400”#

Load from glass infill lites:

Wind = 25 psf

M_{deck} = 3.5’*25psf*42”/2*4’o.c. = 7,350”#

DL = 4’*(3 psf*3’+3.5plf)+10# = 60# each post (vertical load)

Typical anchor to wood: 3/8” lag screw. Withdrawal strength of the lags from *National Design Specification For Wood Construction* (NDS) Table 11.2A.

For Doug-Fir Larch or equal, G = 0.50

W = 305 #/in of thread penetration.

C_D = 1.6 for guardrail live loads or wind loads.

C_m = 1.0 for weather protected supports (lags into wood not subjected to wetting).

$T_b = WC_DC_ml_m$ = total withdrawal load in lbs per lag

W’= WC_DC_m =305#/”*1.6*1.0 = 488#/in

Lag screw design strength – 3/8” x 5” lag, l_m = 5”-2.375”-7/32” = 2.4”

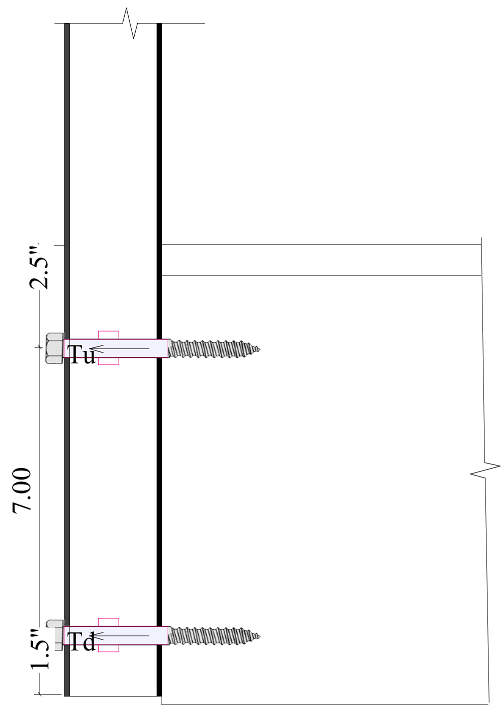
T_b = 488*2.4” = 1,171#

Z_{ll} = 220# per lag, (horizontal load) NDS Table 12K

Z'_{ll} = 220#*1.6*1.0 = 352#

Z_T = 140# per lag, (vertical load)

Z_T = 140#*1.6*1.0 = 224#



Anchors to be minimum of 7" center to center and post shall extend 1-1/2" below bottom anchor.

From ∑M about end
M = (8.5"*T+1.5"*1.5/8.5*T) = 8.76"T
Allowable post moment

M_a=972#*8.76" = 8,515"#

For 3/8" lag screw okay for 36" rail height

For 3/8" carriage bolts:
Allowable load per bolt = 0.11 in^2*20 ksi = 2,200#
For bearing on 2" square bearing plate – area = 3.8 in^2

P_b = 3.8 in^2*1.19*405*1.33 = 2,436#

M_a = 2,200#*8.76" = 19,272"# (exceeds post strength)

For vertical load lag capacity is:

2 lags*187# = 374#/post for live load
2 lags#140# = 280#
D + L = 200/374+60/280 = 0.75<1.0 okay

For corner posts:

For interior and exterior corners there is four lags, two each way. Two lags will act in withdrawal and two will be in shear: Okay be inference from running posts.

POST STRENGTH AT BOLT HOLE:
Directly mounted posts require 7/16" diameter hole through post reducing the post strength at the hole.
S_h = 0.726-2*(7/16*0.125)*(2.255/2)^2 = 0.588 in^3
M_{ared} = 19,000*0.588 = 11,172"#

Maximum moment calculated at the centerline of the top hole must not exceed 11,172"# = 931'#

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STANCHION MOUNT

2"x1-1/2"x 1/8" A500 steel tube

Stanchion Strength

F_{yc} = 45 ksi

Z_{yy} = 0.543 in^3

M_n = 0.543 in^3 * 45 ksi = 24,435#"

M_s = $øM_n$/1.6 = 0.9*24,435/1.6 = 13,745#"

Equivalent post top load

42" post height

V = 13,745"#/42" = 327#

Post may be attached to stanchion with screws or by grouting.

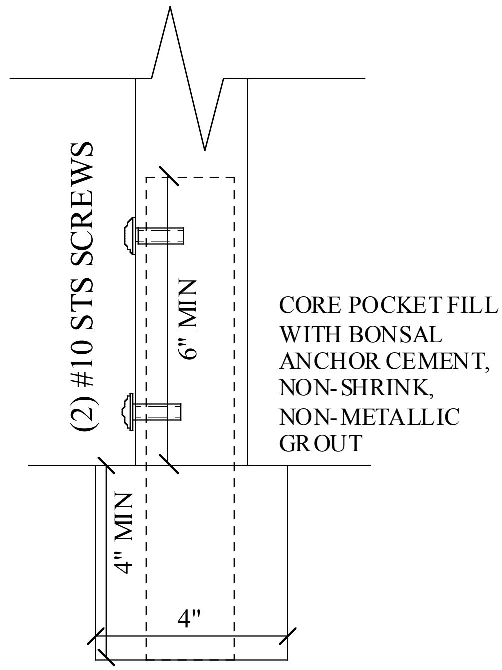
Grout bond strength to stanchion:

$A_{surface}\sqrt{f'c}$ = 7"*4"*$\sqrt{8{,}000}$ psi = 2,500#

(ignores mechanical bond)

for 200# maximum uplift the safety factor against pulling out:

SF = 2,500#/200# = 12.5 > 3.0 therefore okay.



Bearing strength on grout:

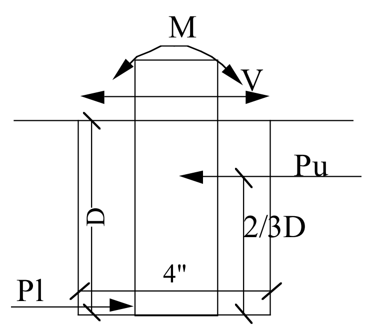
From ∑M about base of stanchion = 0

$$P_u = \frac{M+V*D}{2/3D} =$$

For: M = 10,500"#, V = 250lb, D = 4"

$$P_u = \frac{10{,}500+250*4}{2/3*4} = 4{,}312\#$$

$$f_{Bmax} = \frac{P_u*2}{D*1.5"*0.85} = \frac{4{,}312*2}{4"*1.5"*0.85} = 1{,}691 \text{ psi}$$



For: M = 12,600"#, V = 300lb, D = 4"

$$P_u = \frac{12{,}600+300*4}{2/3*4} = 5{,}175\#$$

$$f_{Bmax} = \frac{P_u*2}{D*1.5"*0.85} = 2{,}029 \text{ psi}$$

Post bearing load on top of stanchion for M = 12,600#":

B = 12,600/6" = 2,100#

For 26 ksi allowable bearing pressure, A = 2.1/26 = 0.081", b = 0.081/1.5" = 0.054"

HSS 2”x1-1/2”x 1/8” powder coated A500 steel tube stanchion:
Stanchion Strength
F_y = 46 ksi
Z_{yy} = 0.475 in^3
M_n = 0.475 in^3 *46 ksi = 21,850#”
M_s = øM_n/1.6 = 0.9*21,850/1.6 = 12,291#”
Equivalent post top load
42” post height
V = 12,291”#/42” = 293#

May be welded to a steel base plate with fillet weld all around.

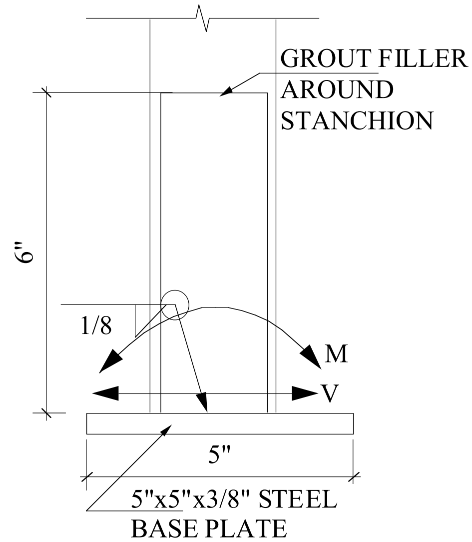
Aluminum Tube Stanchion
2” x 1.5” x ¼” 6061-T6 Aluminum Tube
F_{cb} = 21 ksi From ADM Table 2-22
S_{yy} = 0.719 in^3
M_a = 0.719 in^3 *21 ksi = 15,099#”
Equivalent post top load
42” post height
V = 15,099”#/42” = 360#

Strength of weld affected aluminum stanchion when welded to base plate:
F_{cbw} = 9 ksi
S_{yy} = 0.719 in^3
M_a = 0.719 in^3 *9 ksi = 6,471#”
Equivalent post top load
42” post height
V = 6,471”#/42” = 154#

Because of strength reduction from weld effected metal the aluminum stanchion welded to a base plate typically requires a topping slab to be poured in place over the base plate with a minimum thickness of 2” above the base plate so that the maximum bending moment occurs outside of the weld effected zone.

When welded to base plate limit the maximum moment on the weld effected zone to 6,471”#.

STANCHION MOUNT – ON BASE PLATE



2"x1-1/2"x 1/8" A304 1/4 hard Stainless steel tube
or A500 steel tube powder coated

Stanchion Strength
$F_{yc} = 50$ ksi
$Z_{yy} = 0.543$ in^3
Reserve strength method from SEI ASCE8-02 section 3.3.1.1 procedure II.
where $d_c/t = (2*2/3) /0.125 = 10.67 < \lambda_1$
$\lambda_1 = 1.1/\sqrt{(F_{yc}/E_o)} = 1.1/\sqrt{(50/28*10^3)} = 26$
$M_n = 0.543$ in^3 * 50 ksi = 27,148#"
$M_s = øM_n/1.6 = 0.9*27,148/1.6 = 15,270$#"
Equivalent post top load
42" post height
V = 15,270"#/42" = 363#

Weld to base plate : 1/8" fillet weld all around – develops full wall thickness.
Check weld strength SEI/ASCE 8-02 section 5.2.2: transverse loaded fillet weld:

$øP_n = øtLF_{ua}$, Use Z for tL
$P_n = 0.55*0.362*80$ ksi
$P_n = 15,928$
Ps = 15,928/1.2 = 13,273#"

Grout bond strength to stanchion:

$A_{surface}\sqrt{f'c}$ = 7"*6"*$\sqrt{10,000}$ psi = 4,200# (ignores mechanical bond)

for 200# maximum uplift the safety factor against pulling out:

SF = 4,200#/200# = 21 > 3.0 therefore okay.

Bond strength to post is similar.

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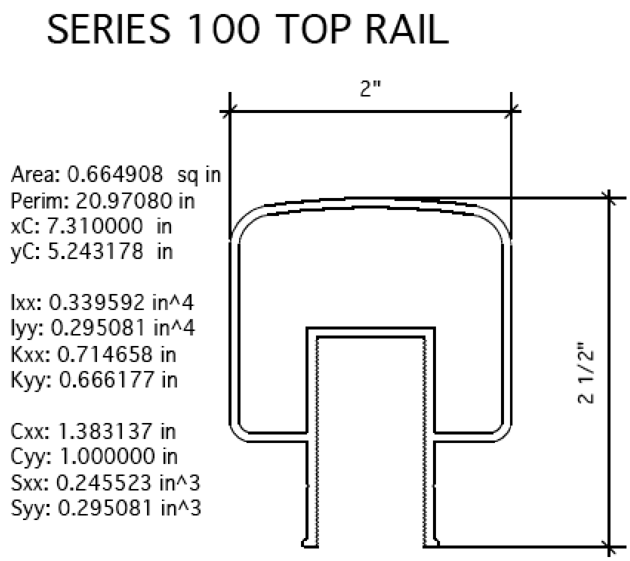
Series 100 Top Rail



Butts into post

Alloy 6063 – T6 Aluminum

Allowable Stress
ADM Table 2-21
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
$J=0.2359in^4$
$\lambda = 2.3(L_BS_C/(C_b(I_y J)^{1/2})^{1/2} = 2.3(L_b*.2455/(1*(.2951*.2359)^{1/2})^{1/2} = 2.219\ L_B^{1/2}$
Inelastic buckling controls when $\lambda<C_c=78$
$78 = 2.219\ L_B^{1/2}$
$L_b=1{,}236"$

For $L_b=60"$, $\lambda=17.19$
$Z_x=0.3880$ in^3
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$
$M_p=30ksi*.3880in^3=11{,}640"\#$
$M_{nmb}=11{,}640(1-17.19/78)+\pi^2*10*10^6*17.19*.2455/78^3 = 9{,}952"\#$
$M_{nmb}/\Omega=9{,}952"\#/1.65 = 6{,}032"\#$

Check local buckling about strong axis:
$R_b/t=2.5"/0.065" = 38.46>31.2$
$F_c/\Omega = 18.5-.593*38.46^{1/2}=14.82ksi$
$M_a=14.82ksi*0.2455in^3=3{,}638"\#$ (Controls)

Check local buckling about weak axis:
$b/t = 1.186"/0.065" = 18.25 < 22.8$
$F_c/\Omega = 15.2ksi$ (local buckling does not control)
$M_a=(F_c/\Omega)*Z_y=15.2ksi*0.3915in^3=5{,}951"\#$ (Controls)

Find maximum top rail span:
$L_{max}=3{,}638"\#*4/200\# = 72"$ For single span condition
$L_{max}=3{,}638"\#*(64/13)/200\# = 89"$ For two span condition

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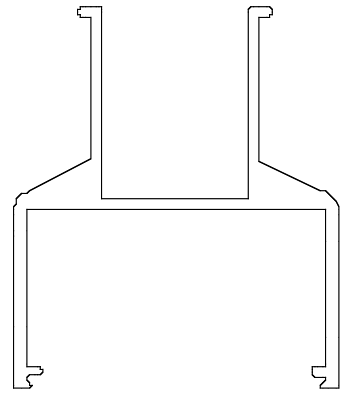
SERIES 100 BOTTOM RAIL



Rail Properties:
6063-T6 Aluminum
$I_{xx} = 0.102$ in^4, $S_{xx} = 0.101$ in^3
$I_{yy} = 0.164$ in^4, $S_{yy} = 0.193$ in^3
$r_{xx} = 0.476$”, $r_{yy} = 0.603$”

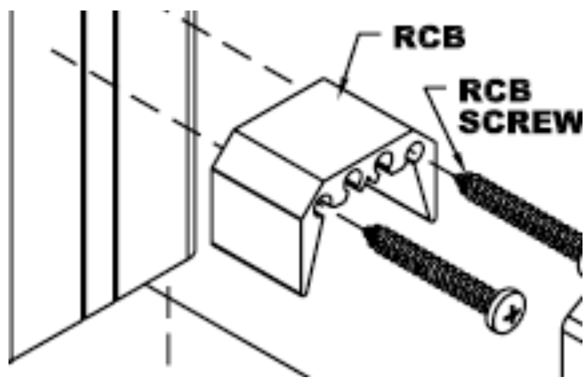
b/t = .807”/.07” = 11.5>7.3
Fc/Ω=19-0.53*11.5 = 12.9ksi
Allowable Moments ➜ Horiz.= 0.193in³ ·12.9 ksi =2,490”#
Maximum allowable load for 72” o.c. post spacing
W = 2,490”#*8/(67.625”²) = 4.36 pli = 52.27 plf
P = 2,490”#*4/67.625” = 147#
Max span for 50 plf load = $(8*2{,}490/(50/12))^{1/2}$ = 69” clear span

Rail fasteners -Bottom rail connection block to post #10x1.5” 55 PHP SMS Screw
Check shear @ post (6005-T5 or 6061-T6)

2x F_{upost}x dia screw x Post thickness x SF

$$V = 2 \cdot 38 \text{ ksi} \cdot 0.1697” \cdot 0.10” \cdot \frac{1}{3\text{ (FS)}} =$$

V = 430#/screw



Since minimum of 2 screws used for each
Allowable load = 2· 430# = 860#

Rail Connection to RCB

2 screws each end
#8 Tek screw to 6063-T6

$$V = 2 \cdot 30 \text{ ksi} \cdot 0.1309” \cdot 0.07” \cdot \frac{1}{3\text{ (FS)}} = 183\#$$

V_{All} = 2*183 = 366#

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Intermediate post used to provide additional support to bottom rail.
1.4" square 0.1" wall thickness
Acts in compression only.
Secured to rail with two #8 tek screws
Shear strength of screws:

#8 Tek screw to 6063-T6

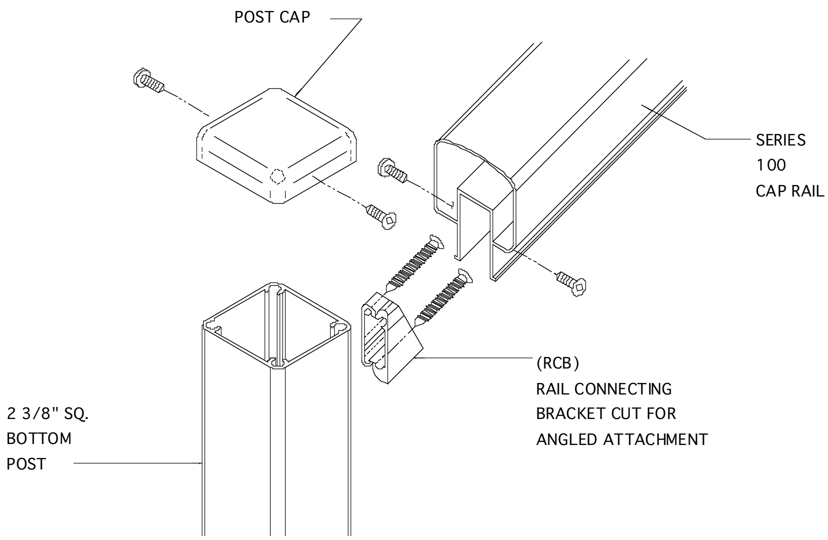
$$V = 2 \cdot 30\text{ ksi} \cdot 0.1309" \cdot 0.07" \cdot \frac{1}{3\text{ (FS)}} = 183\#$$

$V_{All} = 2*183 = 366\#$

Top rail connection to post face:
Use RCB attached to post with 2 #10 screws same as bottom rail.
To 6061-T6 or 6005-T5

$$V = 2 \cdot 38\text{ ksi} \cdot 0.1697" \cdot 0.10" \cdot \frac{1}{3\text{ (FS)}} = 430\#/\text{screw}$$

Since minimum of 2 screws used for each
Allowable load =
$2 \cdot 430\# = 860\#$



The connection block can be cut square for use in horizontal rail applications or angled for use in sloped applications such as along stairs or ramps.

Connection of rail to RCB is with (2) #8 Tek screw to 6063-T6

$$V = 2 \cdot 30\text{ ksi} \cdot 0.1309" \cdot 0.07" \cdot \frac{1}{3\text{ (FS)}} = 183\#$$

$V_{tot} = 2*183\# = 366\# \geq 200\#$ okay

Intermediate post fitting

Used for intermediate posts along stairways
Fitting locks into top of post using structural silicone.

Maximum load on fitting is 300#
6' post spacing * 50 plf = 300#

Shear resisted by direct bearing between fitting and post
area = 2.175"*0.1875 = 0.408 in^2
Bearing pressure = 300#/.408 = 736 psi

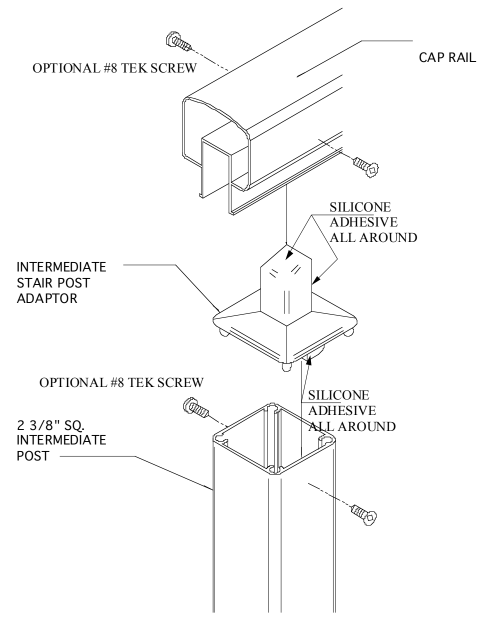
Moment of fitting to post:
This is an intermediate post with rotation of top rail restrained at rail ends.
Moment of fitting is created by eccentricity between bottom of top rail and top of post:

e = 0.425"

M = 300# * (0.425") = 127.5#"

Moment on fitting is resisted by tearing in silicone
Silicone tear strength: From Dow Corning, (silicone manufacturer), CRL 95C Silicone is the same product as the Dow Corning 995 Silicone Structural Glazing Sealant, from Dow Corning product information sheet

Tear strength $\geq$ 49 ppi
Peel strength $\geq$ 40 ppi
Ult. tension adhesion $\geq$ 170 psi
Tensile strength $\geq$ 48 psi @ 25% elongation
Tensile strength $\geq$ 75 psi @ 50% elongation

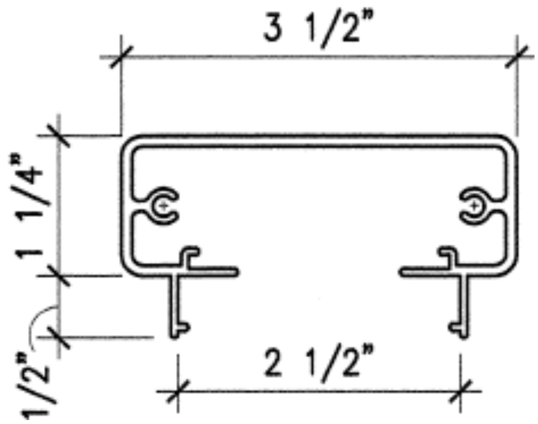


Moment capacity:
I_x=2.175"*2.175"^{2}+2*2.175^3/12+2*2.175*(2.175"/2)^2
I_x=17.15in^4/in
M_a=49ppi*17.15in^4/in/2.175"=386"#

SF = 386#"/127.5#" = 3 > 2.0 okay

Option #8 Tek screws:
Shear strength = V= 2·38 ksi ·0.1309" · 0.07" · $\frac{1}{3\ (FS)}$ = 232#
Added moment capacity = 232#*2.375" = 551#"

Series 200 Top rail



Area: 0.887 sq in

I_{xx}: 0.254 in^4
I_{yy}: 1.529 in^4
r_{xx}: 0.536 in
r_{yy}: 1.313 in
C_{xx}: 1.194 in
C_{yy}: 1.750 in
S_{xx}: 0.213 in^3 bottom
S_{xx}: 0.412 in^3 top
Z_{xx}: 0.421 in^3
S_{yy}: 0.874 in^3
J=0.001661in^4

6063-T6 Aluminum alloy
For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625"

Calculate lateral torsional buckling strength per ADM F.4.2.1
r_{ye}=((.254$^{.5}$/.874)*(0+.038*.001661*67.625^2)$^{1/2}$)$^{1/2}$=0.557in
λ=67.625"/(.557) = 121.4
C_c=78.4 for 6063-T6
F_c/Ω = 60414/121.4^2 = 4.10 ksi (limiting strength for horizontal loading)

Check for local buckling of top element under vertical loading:
b/t = 3.125"/.094" = 33.24
F_c/Ω = 19-.17*33.24 = 13.3 ksi (limiting strength for vertical loading)

Allowable Moments → Horiz.= 0.874in^3 ·4.10 ksi = 3,583#" = 299#'
Vertical load = 0.457in^3 · 13.3 ksi = 6,078#" top compression
or = 0.421in^3 ·15.2 ksi = 6,399#" controls vertical- bottom tension

Maximum allowable load for 72" o.c. post spacing - vertical
W = 3,583"#*8/(67.625"^{2}) = 6.268 pli = 75.2 plf
P = 3,583"#*4/67.625" = 212#

For horizontal loading:
Δ_{max} = 200*72^3/(48*10x10^6*1.529in^4) = 0.102"

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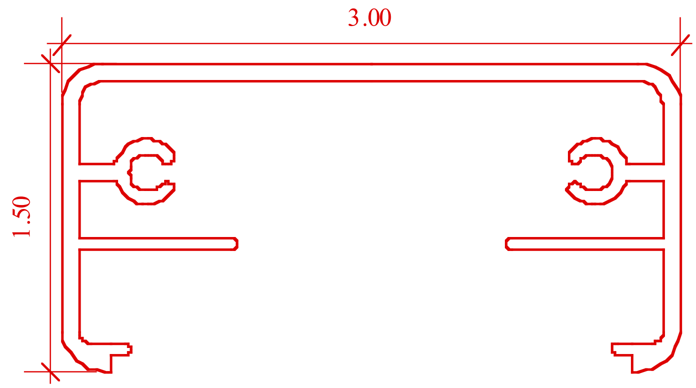
Series 200X Top rail
Area: 0.744 sq in
Perim: 18.466 in
I_{xx}: 0.1325 in^4
I_{yy}: 0.8512 in^4
r_{xx}: 0.4626 in
r_{yy}: 0.5660 in
$C_{y,t}$: 0.545 in
$C_{y,b}$: 0.954 in
S_{xx}: 0.139 in^3 bottom S_{xx}: 0.243 in^3 top
S_{yy}: 0.566 in^3
Z_{xx}: 0.246 in^3
J = 0.0008104 in^4



6063-T6 Aluminum alloy
For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625"

Calculate lateral torsional buckling strength per ADM F.4.2.1
r_{ye}=((.1235$^{.5}$/.243)*(0+.038*.0008104*67.625^2)$^{1/2}$)$^{1/2}$=0.737in
λ=67.625"/(.737) = 91.76
C_c=78.4 for 6063-T6
F_c/Ω = 60414/91.76^2 = 7.18 ksi (limiting strength for horizontal loading)

Check for local buckling of top element under vertical loading:
b/t = 2.571"/.074" = 34.74
F_c/Ω = 19-.17*34.74 = 13.1 ksi (limiting strength for vertical loading)

Allowable Moments ➔ Horiz.= 0.566in³ ▪7.18 ksi = 4,064#" = 339#'
Vertical load = 0.243in³ ▪ 13.1 ksi = 3,183#" top compression
or = 0.246in³ ▪15.2 ksi = 3,739#" controls vertical- bottom tension

Maximum allowable load for 72" o.c. post spacing - vertical
W = 3,183"#*8/(67.625"^{2}) = 5.568 pli = 66.8 plf
P = 3,183"#*4/67.625" = 188# (Load share with bottom rail needed for 6' spans)

Maximum allowable load for 72" o.c. post spacing - horizontal
W = 4,064"#*8/(67.625"^{2}) = 7.11 pli = 85.3 plf
P = 4,064"#*4/67.625" = 240#

For horizontal loading:
Δ_{max} = 200*72^3/(48*10x10^6*0.8512in^4) = 0.182"

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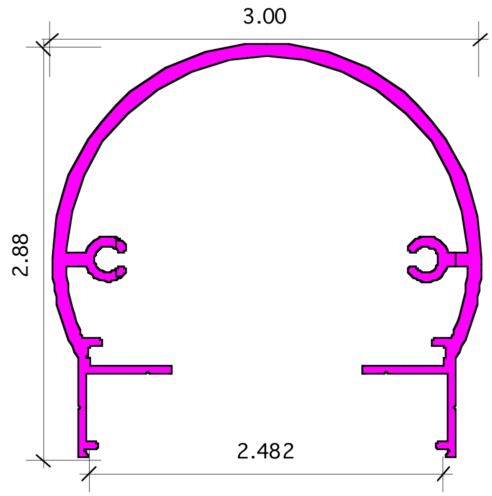
Series 300 Top Rail



Area: 0.881 sq in
Perim: 21.29 in
I_{xx}: 0.581 in^4 I_{yy}: 1.07 in^4
r_{xx}: 0.400 in r_{yy}: 1.15 in
$C_{xx,b}$: 1.444 in $C_{xx,t}$: 1.438 in
$S_{xx,t}$: 0.404 in^3 S_{yy}: 0.662 in^3
Z_{xx}: 0.575 in^3 Z_{yy}: 0.864 in^3
J = 0.0005419 in^4

Allowable stresses ADM Table 2-21

6063-T6 Aluminum alloy
For 72” on center posts; L = 72”-2.375”-1”x2 = 67.625”

Calculate lateral torsional buckling strength per ADM F.4.2.1
r_{ye}=((1.07$^{.5}$/.404)*(0+.038*.0005419*67.625^2)$^{1/2}$)$^{1/2}$=0.886 in
λ=67.625”/(.886”) = 76.33
C_c=78.4 for 6063-T6
M_p=0.864in^3*15.2ksi = 13,133”#
M_{nmb}=13,133*(1-76.33/78.4)+π^2*10.1*10^6*76.33*.662/78.4^3 = 10,799”#

Check for local buckling of top curved element under vertical loading:
R_b/t = 1.5”/.086” = 17.44 < 31.2 Local buckling does not control

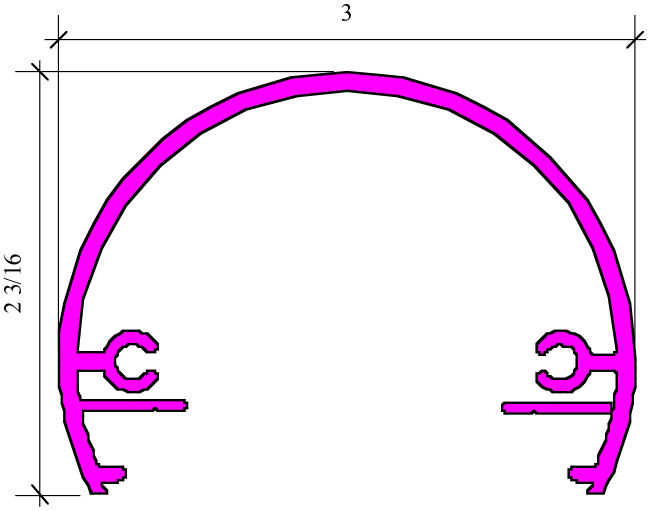
Allowable Moments ➜ Horiz.= 10,799”#/1.65 = 6,545”#
Vertical = 0.575in³ ⋅15.2 ksi = 8,740#” controls vertical- bottom tension

Maximum allowable load for 72” o.c. post spacing
W = 6,545”#*8/(67.625”^{2}) = 11.45 pli = 137.4 plf
P = 6,545”#*4/67.625” = 387# (Load share with bottom rail needed for 6’ spans)

Rail to post connection: Direct bearing for downward forces and horizontal forces:
For uplift connected by (2) #10 Tek screws each post:
2x F_{upost}x dia screw x Post thickness / SF (ADM 5.4.3)
V= 2⋅30 ksi ⋅0.1379” ⋅ 0.09”/3= 325#/screw

For horizontal loading:
Δ_{max} = 200*72^3/(48*10x10^6*1.07in^4) = 0.145”

Top rail 300X



Wall thickness t = 0.09375" min.
Allowable stresses ADM Table 2-24

line 11
$F_{Cb} \rightarrow L/\,r_y = \dfrac{(72 - 2\ 3/8" - 2.1")}{1.137} = 59.4$

Based on 72" max post spacing
$F_{Cb} = 16.7 - 0.073(59.4) = 12.36$ ksi
$M_{all\ horiz} = 12.36^{ksi} \bullet (0.656) = 8{,}111"^{\#}$
Vertical loads shared with bottom rail
For vertical load → bottom in tension top comp.

$F_b = 18$ ksi line 3
$F_c = 18$ ksi line 16.1

$M_{all\ vert} = (0.309in^4) \bullet 18\text{ ksi} = 5{,}562"^{\#}$

Allowable loads

Horizontal → uniform → $W = \dfrac{8{,}111 \bullet 8}{72^2} = 12.5\ \#/in = W = 150$ plf

$P_H = \dfrac{4 \bullet 8{,}111}{72} = 451\ \#$

Vertical → $W = \dfrac{5{,}562 \bullet 8}{72^2} = 5.6\ \#/in = 103$ plf (Top rail alone)

$P = \dfrac{5{,}562 \bullet 4}{72} = 309\ \#$

For horizontal loading:
$\Delta_{max} = 200*72^3/(48*10x10^6*0.984in^4) = 0.158"$

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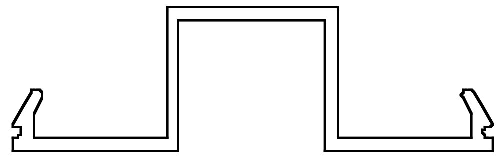
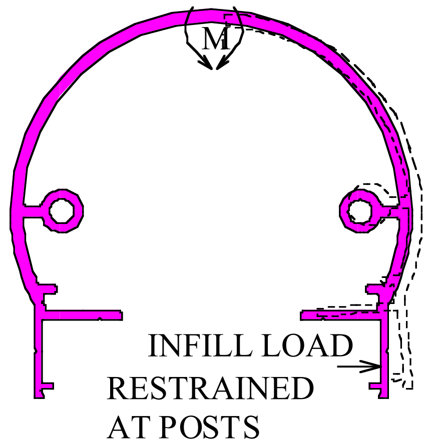


Insert channel for glass – 6063-T6

$I_{yy} = 0.156\ in^4$ $I_{xx} = 0.023\ in^4$

$S_{yy} = 0.125\ in^3$ $S_{xx} = 0.049\ in^4$



Insert compression locks into top rail

Horizontal forces transferred between insert and top rail by direct bearing on locking tabs.

Bearing area = 1/8" width

Allowable bearing load will be controlled by spreading of top rail

Check significance of circumferential stress:

R/t = 3"/0.09375 = 32 > 5 therefore can assume plane bending and error will be minimal

M = 2.08"*W

$M_{all} = S*F_b$

F_b = 20 ksi for flat element bending in own plane, ADM Table 2-21

$S = 12"/ft*(0.094)^2/6 = 0.0177\ in^3$

$W_{all} = M_{all}/2.08" = (S* F_b)/2.08" = (0.0177\ in^3*20\ ksi)/2.08" = 170$ plf

For 36" panel height – 1/2 will be tributary to top rail:

Maximum live load = 170 plf/(3'/2) = 113 psf.

Check deflection:

$\Delta = WL^3/(3EI)$

$I = 12"*0.09375^3/12 = .000824\ in^4$

$\Delta = 170plf*2.08"^3/(3*10.1x10^6*.000824) = 0.06"$

The required deflection to cause the infill to disengage: 0.05"

Reduce allowable load to limit total deflection:

0.05/0.06*113 plf = 94 plf

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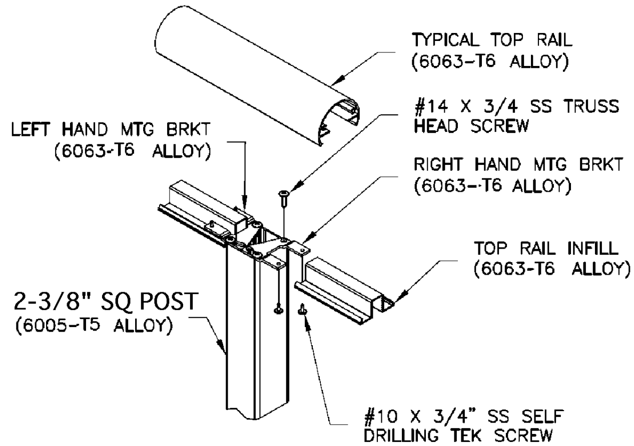
Top rail connection to post:

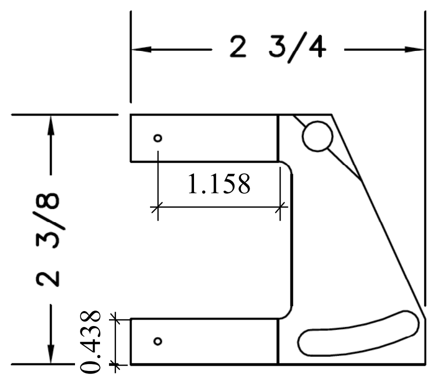


For Vertical loads top rail is restrained by (2) #10 tek screws each side.
Connection of bracket to post is with (2) #14 screws so is stronger.

For horizontal loads the top rail directly bears on side of post.

Tek screw strength: Check shear @ rail (6063-T6)
2x F_{urail}x dia screw x Rail thickness x SF
$V = 2 \cdot 30\text{ ksi} \cdot 0.1379" \cdot 0.09" \cdot \dfrac{1}{3\text{ (FS)}} = 325\#/\text{screw}$

Since minimum of 2 screws used for each
Allowable load = 2· 325# = 650#



LHB

Post bearing strength

$V_{all} = A_{bearing} * F_B$
$A_{bearing} = 0.09" * 2.25" = 0.2025\text{ in}^2$
$F_B = 21\text{ ksi}$
$V_{all} = 0.2025\text{ in}^2 * 21\text{ ksi} = 4.25\text{ k}$

Bracket tab bending strength
Vertical uplift force
For 5052-H32 aluminum stamping 1/8" thick
$F_b = 18$ ksi – ADM Table 2-09
$S = 0.438" * (.125)^3/12 = 0.00007\text{ in}^3$
$M_a = 18\text{ ksi} * 0.00007 = 126"\#$
$P_a = M_a/l = 126"\#/1.158" = 109\#$
Uplift limited by bracket strength:

$Up_{all} = 2*109 = 218\#$ per bracket

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RAIL SPLICES:

Splice plate strength:

Vertical load will be direct bearing from rail/plate to post no bending or shear in plate.

Horizontal load will be transferred by shear in the fasteners.

Rail to splice plates:

#8 Tek screw strength: Check shear @ rail (6063-T6)

2x F_{urail}x dia screw x rail thickness x SF

$$V = 2 \cdot 30\text{ ksi} \cdot 0.1379" \cdot 0.09" \cdot \frac{1}{3\text{ (FS)}} = 325\#/\text{screw};\ \text{for two screws} = 650\#$$

or $F_{urplate}$x dia screw x plate thickness x SF

$$V = 38\text{ ksi} \cdot 0.1379" \cdot 0.125" \cdot \frac{1}{3\text{ (FS)}} = 218\#/\text{screw};\ \text{for two screws} = 436\#$$

Post to splice plate:

Screws into post screw chase so screw to post connection will not control.

splice plate screw shear strength

2x F_{uplate}x dia screw x plate thickness x SF

$$V = 2 \cdot 38\text{ ksi} \cdot 0.1379" \cdot 0.125" \cdot \frac{1}{3\text{ (FS)}} = 416\#/\text{screw};\ \text{for two screws} = 832\#$$

Check moment from horizontal load:

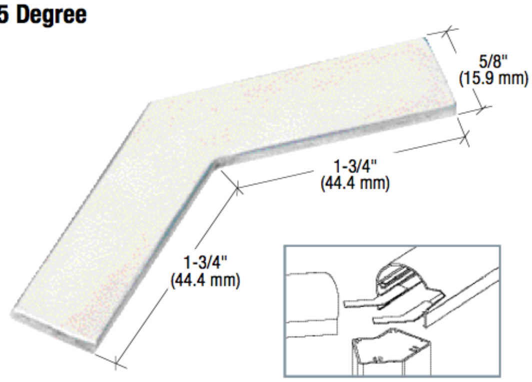
M = P*0.75". For 200# maximum load from a single rail on to splice plates

M = 0.75*200 = 150#"

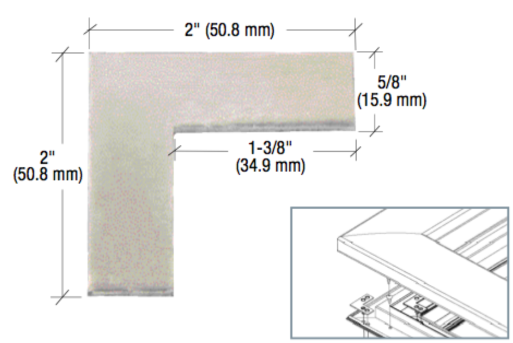
$S = 0.125*(0.625)^2/6 = 0.008\text{ in}^3$

$f_b = 150\#"/(0.008*2) = 9{,}216\text{ psi}$

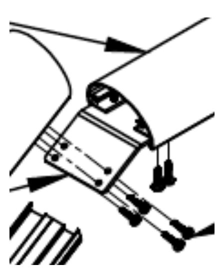
45 Degree



90 Degree



For corner brackets screw strength and bending strength will be the same.



Single full width bar may be used instead of the two ⅝" bars.

May be used to create vertical miters and splice rail sections.

May be used with #10 tek screws.

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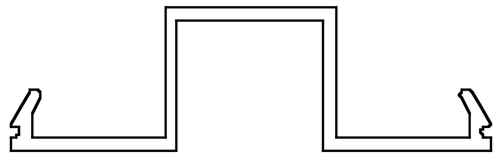
Insert channel for glass – 6063-T6

$I_{yy} = 0.156$ in^4 $I_{xx} = 0.023$ in^4

$S_{yy} = 0.125$ in^3 $S_{xx} = 0.049$ in^4



Insert compression locks into top rail

Horizontal forces transferred between insert and top rail by direct bearing on locking tabs.

Bearing area = 1/8" width

Allowable bearing load will be controlled by spreading of top rail

$M = 2.08"*W$

$M_{all} = S*F_b$

F_b = 20 ksi for flat element bending in own plane, ADM Table 2-24

$S = 12"/ft*(0.094)^2/6 = 0.0177$ in^3

$W_{all} = M_{all}/2.08" = (S* F_b)/2.08" = (0.0177$ in^3$*20$ ksi$)/2.08" = 170$ plf

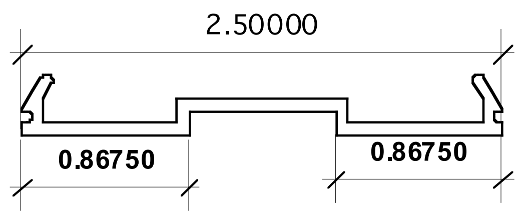
For 36" panel height – 1/2 will be tributary to top rail:

Maximum wind load = 170 plf/(3'/2) = 113 psf.

Insert channel for picket infill – 6063-T6

$I_{yy} = 0.144$ in^4 $I_{xx} = 0.0013$in^4

$S_{yy} = 0.115$ in^3 $S_{xx} = 0.0057$ in^4



Insert compression locks into top rail

Horizontal forces transferred between insert and top rail by direct bearing on locking tabs.

Bearing area = 1/8" width

Allowable bearing load will be controlled by spreading of top rail

$M = 2.08"*W$

$M_{all} = S*F_b$

F_b = 20 ksi for flat element bending in own plane, ADM Table 2-24

$S = 12"/ft*(0.094)^2/6 = 0.0177$ in^3

$W_{all} = M_{all}/2.08" = (S* F_b)/2.08" = (0.0177$ in^3$*20$ ksi$)/2.08" = 170$ plf

For 36" panel height – 1/2 will be tributary to top rail:

Maximum live load = 170 plf/(3'/2) = 113 psf.

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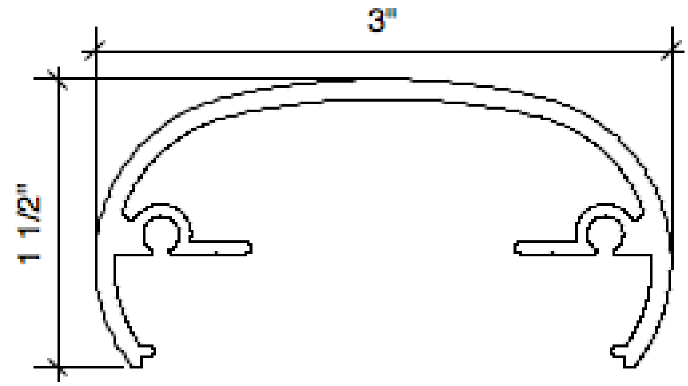
Top Rail Series 320



$I_{xx} = 0.118\ in^4$
$I_{yy} = 0.796\ in^4$
$S_{xx,bot} = 0.129\ in^3$
$S_{xx,top}=0.201\ in^3$
$Z_{xx}= 0.244\ in^3$
$S_{yy} = 0.531\ in^3$
$Z_{yy} = 0.669 in^3$
$J = 0.001730\ in^4$

Allowable stresses ADM Table 2-21
6063-T6 Aluminum

For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625"

Calculate lateral torsional buckling strength per ADM F.4.2.1
$r_{ye}=((.118^{.5}/.129)*(0+.038*.00173*67.625^2)^{1/2})^{1/2}=0.907$ in
$\lambda=67.625"/(.907") = 74.6$
$C_c=78.4$ for 6063-T6
$M_p=0.669in^3*25ksi = 16,725"\#$
$M_{nmb}= 16,725"\#(1-74.6/78.4)+\pi^2*10.1*10^6*74.6*0.531/78.4^3=9,005"\#$

Check for local buckling of top curved element under vertical loading:
$R_b/t = 3.687"/0.1" = 36.87 > 31.2$ Local buckling controls
$F_c/\Omega = 18.5-.593*36.87^{1/2} = 14.90$ ksi

Allowable Moments ➔ Horiz.= 9,005"# (Inelastic lateral torsional buckling)
Vertical = $0.244in^3 \cdot 15.2$ ksi = 3,709"# (Yielding)
Vertical = $0.201in^3*14.9ksi$ = 2,995"# (Local Buckling)

Maximum allowable load for 72" o.c. post spacing
W = 2,995"#*8/(67.625"^{2}) = 5.24 pli = 62.9 plf
P = 2,995"#*4/67.625" = 177# (Load share with bottom rail needed for 6' spans)

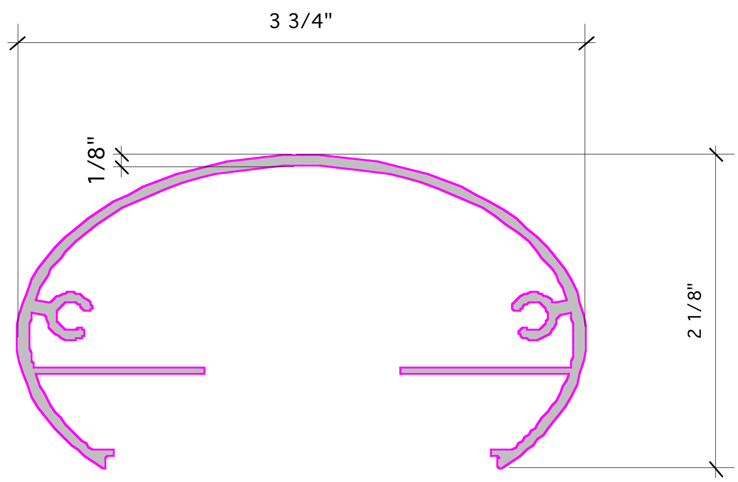
For horizontal loading:
$\Delta_{max} = 200*72^3/(48*10x10^6*0.796in^4) = 0.195"$

Top Rail Series 350

Area: 0.725 sq in
Perim: 21.338 in

Ixx: 0.263 in^4
Iyy: 1.398 in^4
rxx: 0.602 in
ryy: 1.389 in

Cxx: 1.128 in
Cyy: 1.875 in
Sxx: 0.233 in^3
Syy: 0.737 in^3



Allowable stresses ADM Table 2-22

6063-T6 Aluminum alloy
For 72” on center posts; L = 72”-2.375”-1”x2 = 67.625”

Calculate lateral torsional buckling strength per ADM F.4.2.1
$r_{ye}=((.263^{.5}/.737)*(0+.038*.0008041*67.625^{2})^{1/2})^{1/2}=0.907$ in
λ=67.625”/(.907”) = 74.6
C_c=78.4 for 6063-T6
$F_c/\Omega = 60414/74.6^2 = 10.86$ ksi (limiting strength for horizontal loading)

Check for local buckling of top curved element under vertical loading:
R_b/t = 2.5”/.07” = 35.7 > 31.2 Local buckling controls
$F_c/\Omega = 18.5-.593*35.7^{.5} = 15.0$ ksi

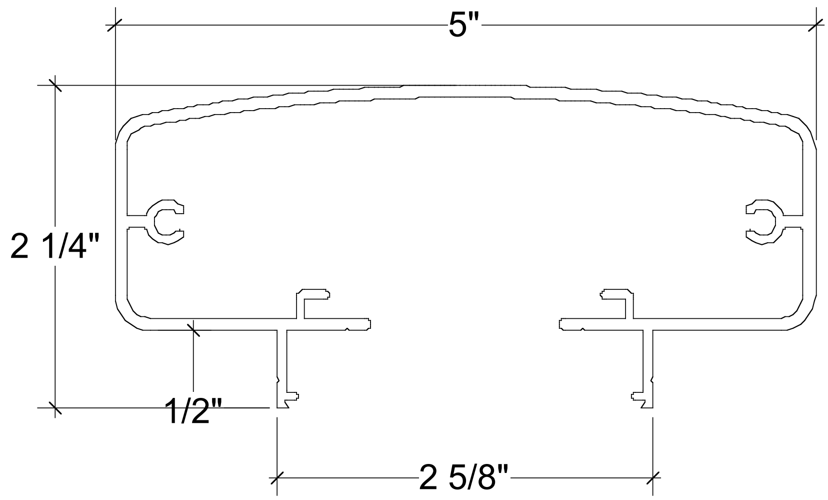
Allowable Moments → Horiz.= 0.737in^3*10.86ksi = 8,004”#
Vertical = 0.282in^3·15.0 ksi = 4,230”#
Vertical = 0.3584in^3*15.2ksi = 5,448”#

Maximum allowable load for 72” o.c. post spacing
W = 4,230”#*8/(67.625”^{2}) = 7.40 pli = 88.8 plf
P = 4,230”#*4/67.625” = 250# (Load share with bottom rail needed for 6’ spans)

For horizontal loading:
$\Delta_{max} = 200*72^3/(48*10x10^6*1.398in^4) = 0.111$”

Series 400 Top rail

I_{xx}: 0.611 in^4
I_{yy}: 3.736 in^4
r_{xx}: 0.717 in
r_{yy}: 1.774 in
C_{xx}: 1.358 in
C_{yy}: 2.50 in
S_{xx}: 0.450 in^3 bottom
S_{xx}: 0.399 in^3 top
S_{yy}: 1.494 in^3



6063-T6 Aluminum alloy
For 72" on center posts;
L = 72"-2.375"-1"x2 = 67.625"

Calculate lateral torsional buckling strength per ADM F.4.2.1
r_{ye}=((.611$^{.5}$/1.494)*(0+.038*.00219*67.625^{2})$^{1/2}$)$^{1/2}$=0.568 in
λ=67.625"/(.568") = 119
C_c=78.4 for 6063-T6
F_c/Ω = 60414/119 ^{2} = 4.266 ksi (limiting strength for horizontal loading)

Check for local buckling of top curved element under vertical loading:
R_b/t = 12"/.087" = 138 > 31.2 Local buckling controls
F_c/Ω = 18.5-0.593*119$^{1/2}$ = 12.03ksi

Allowable Moments ➜ Horiz.= 1.494in^3* 4.266 ksi = 6,373"#
Vertical = 0.399in^3·12.03 ksi = 4,800"#
Vertical = 0.772in^3*15.2ksi = 11,734"#

Maximum allowable load for 72" o.c. post spacing
W = 4,800"#*8/(67.625"^{2}) = 8.40 pli = 101 plf
P = 4,800"#*4/67.625" = 284#

For horizontal loading:
Δ_{max} = 200*72^{3}/(48*10x10^{6}*3.736in^4) = 0.042"

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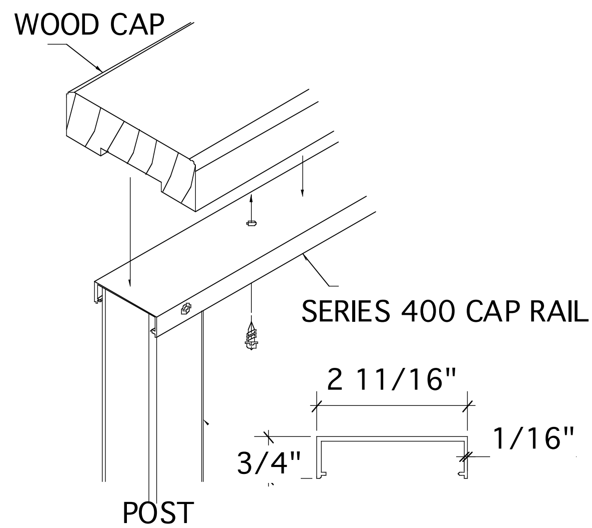
SERIES 400 TOP RAIL
COMPOSITE MATERIAL OR
Alloy 6063 – T6 Aluminum



I_{xx}: 0.0138 in^4; I_{yy}: 0.265 in^4
C_{xx}: 0.573 in; C_{yy}: 1.344 in
S_{xx}: 0.024 in^3; S_{yy}: 0.197 in^3

Wood
2"x4" nominal
I_{xx}: 0.984 in^4; I_{yy}: 5.359 in^4
C_{xx}: 0.75 in; C_{yy}: 1.75 in
S_{xx}: 1.313 in^3; S_{yy}: 3.063 in^3

For wood use allowable stress from NDS Table 4A for lowest strength wood that may be used:
F_b = 725 psi (mixed maple #1), C_D =1.6, C_F = 1.5
F'_b = 725*1.6*1.5 = 1,740 psi
F'_b = 725*1.6*1.5*1.1 = 1,914 psi for flat use (vertical loading)
Composite action between aluminum and wood:
$n = E_a/E_w$ = 10.1/1.1 = 9.18

Composite Shape Section Properties
Effective properties adjusted for E=10.1*10^{3}ksi
I_{xx}=0.2763in^4 I_{yy}=0.8484in^4

Allowable Stress for aluminum: ADM Table 2-21
F_T = 15.2 ksi
$F_C \rightarrow$ 6' span
Rail is braced by wood At 16" o.c. and legs have stiffeners therefore
F_c = 15.2 ksi

Vertical loading: $M_{a,x}$=1,914psi*0.2763in^4/1.0427"*9.18=4,656"# (Wood failure)
$M_{a,x}$=15.2ksi*0.2763in^4/1.2073"=3,479"# (Aluminum failure controls)

Horizontal loading: $M_{a,y}$=1,740psi*0.8484in^4/1.75"*9.18=7,744"# (Wood failure controls)
$M_{a,x}$=15.2ksi*0.8484in^4/1.3434"=9,599"# (Aluminum failure)

Maximum allowable load for 72" o.c. post spacing
W = 3,479"#*8/(67.625"^{2}) = 6.09 pli = 73 plf
P = 3,479"#*4/67.625" = 206#

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Connection between aluminum and wood needs to be able to resist transverse shear for vertical loading.

V=200#/2=100# (Midspan 200# concentrated load)

v=VQ/I

Q=YA'=.6338"*.26406in^2=0.1674in^3
v=100#*0.1674in^3/0.2763in^4=60.59pli=727.0plf

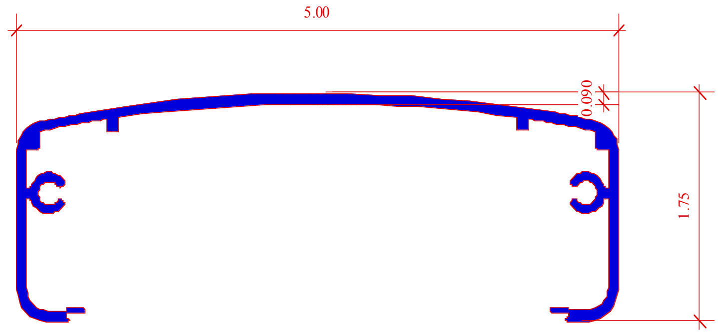
Use #6 Wood Screws (Larger screws do not appreciably increase shear strength due to limited penetration and will increase probability of splitting)
Z'=1.6*76#=122# each
Aluminum bearing = 2*.138"*.062"*30ksi/3 = 171#

Screw spacing to create composite bending at service loading = 122#/60.59pli => 2" O.C staggered

Adhesive strength to create composite bending in lieu of screws = 60.59pli/2.6875" = 22.5psi

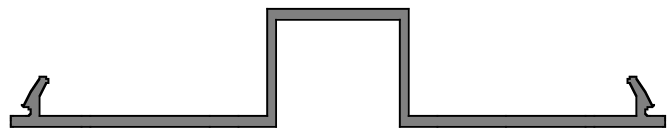
COMPOSITES: Composite materials, plastic lumber or similar may be used provided that the size and strength is comparable to the wood.

Series 500 Top rail



Area: 0.854 sq in Perim: 20.44 in
I_{xx}: 0.262 in^4 I_{yy}: 3.204 in^4
K_{xx}: 0.553 in K_{yy}: 1.936 in
C_{xx}: 1.184 in C_{yy}: 2.497 in
S_{xx}: 0.221 in^3 S_{yy}: 1.283 in^3
Z_{xx}: 0.405 in^3 Z_{yy}: 1.593 in^3
J: 0.001801 in^4

Infill Piece



Area: 0.410 sq in Perim: 12.145 in
I_{xx}: 0.028 in^4 I_{yy}: 0.553 in^4
K_{xx}: 0.261 in K_{yy}: 1.161 in
C_{xx}: 0.534 in C_{yy}: 2.061 in
S_{xx}: 0.052 in^3 S_{yy}: 0.268 in^3

6063-T6 Aluminum alloy
Determine Maximum Post Spacing: -
Horizontal load ADM 3.4.15
If designed as a curved element, R_b/t = 12.5"/.086" = 145
$F_c/\Omega = 18.5 - .593*145^{1/2} = 11.4$ksi

Calculate lateral torsional buckling strength per ADM F.4.2.1
$r_{ye}=((.262^{.5}/1.283)*(0+.038*.001801*67.625^2)^{1/2})^{1/2}=0.472$ in
λ=67.625"/(.472") = 143
C_c=78.4 for 6063-T6
$F_c/\Omega = 60414/143^2 = 2.95$ksi (limiting strength for horizontal loading)

Allowable Moments → Horiz.= 1.283in^3* 2.95 ksi = 3,785"#
Vertical = 0.221in^3·11.4 ksi = 2,519"#

Maximum allowable load for 72" o.c. post spacing
W = 2,519"#*8/(67.625"^{2}) = 4.41 pli = 52.9 plf
P = 2,519"#*4/67.625" = 150# (Load share with bottom rail required)

For horizontal loading:
$\Delta_{max} = 200*72^3/(48*10x10^6*(3.204+0.553in^4) = 0.041"$

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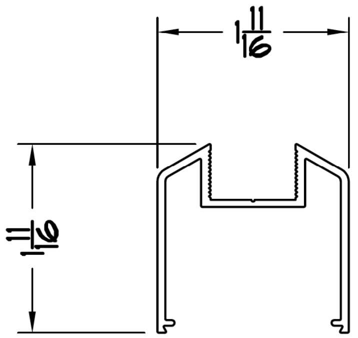
Glass Infill Bottom Rail
6063-T6

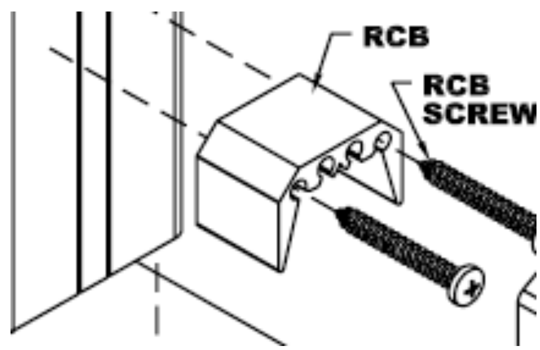


Area: 0.3923 sq in
Perim: 11.648 in

Ixx: 0.0869 in^4
Iyy: 0.172 in^4
Kxx: 0.472 in
Kyy: 0.662 in
Cxx: 1.0133 in
Cyy: 0.8435 in
Sxx: 0.0857 in^3 Bottom
Sxx: 0.129 in^3 Top
Syy: 0.204 in^3

b/t = 1.397"/0.07" = 19.96
F_c/Ω = 155/19.96 = 7.77 ksi

Allowable Moments → Horiz.= 0.204in^3*7.77 ksi =1,585"#
Maximum allowable load for 72" o.c. post spacing
W = 1,585"#*8/(67.625"^{2}) = 2.77 pli = 33.3 plf
P = 1,585"#*4/67.625" = 94#
Max span for 50 plf load = $(8*1{,}585/(50/12))^{1/2}$ = 55" clear span
Rail fasteners -Bottom rail connection block to post
#10×1.5" 55 PHP SMS Screw

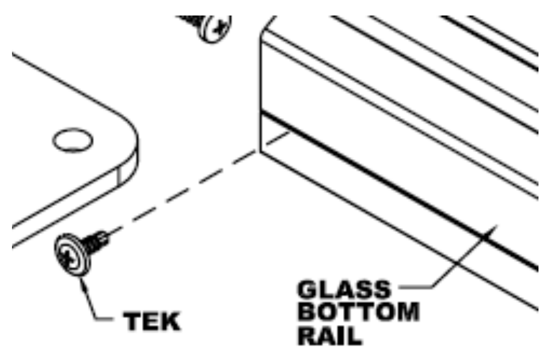


Check shear @ post (6005-T5)
2x F_{upost}x dia screw x Post thickness x SF
$V = 2 \cdot 38\text{ ksi} \cdot 0.1697" \cdot 0.10" \cdot \frac{1}{3\,(FS)} =$
V = 430#/screw
Since minimum of 2 screws used for each
Allowable load = 2· 430# = 860#

Rail Connection to RCB

2 screws each en
#8 Tek screw to 6063-T6

$2*30_{ksi} \cdot 0.1309" \cdot 0.07" \cdot \frac{1}{3}$ = 232#/screw



Allowable tension = 2*232 = 464#
OK

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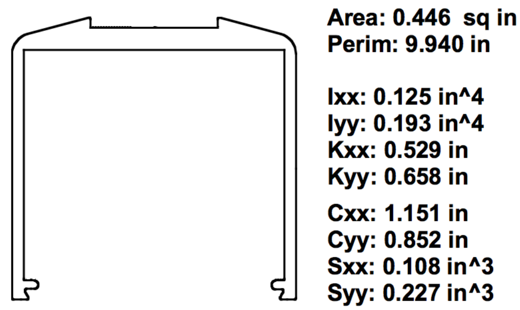
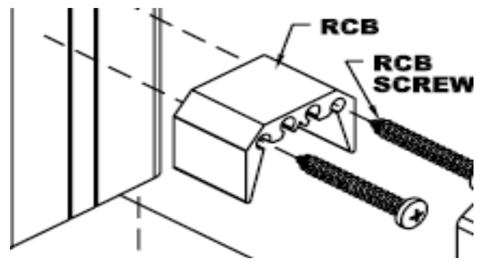
Picket bottom rail



Bottom rail strength
6063-T6 Aluminum alloy
For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625"

Calculate lateral torsional buckling strength per ADM F.4.2.1
$J=0.001752in^4$
$r_{ye}=((.125^{.5}/.227)*(0+.038*.001752*67.625^2)^{1/2})^{1/2}=0.927$ in
$\lambda=67.625"/(.927") = 73.0$
$C_c=78.4$ for 6063-T6
$F_c/\Omega = 15.2$ksi

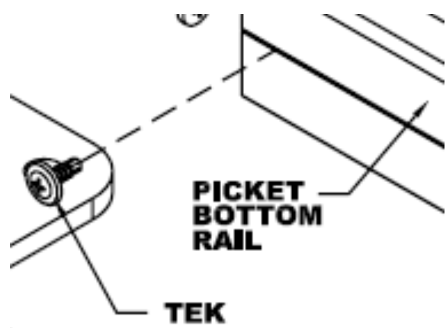


Check local buckling of vertical legs:
b/t = 1.5"/.07" = 21.4>12.6
$F_c/\Omega = 155/21.4 = 7.24$ksi

Allowable Moments ➔ Horiz.= $0.227in^3*7.24$ ksi = 1,643"#
Vertical = $0.108in^3 \cdot 15.2$ ksi = 1,642"#

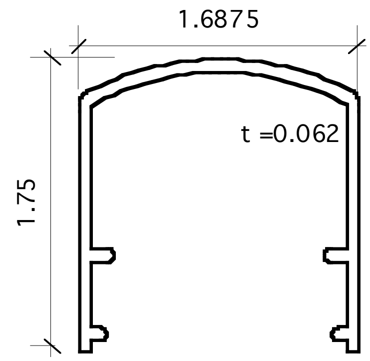
Rail fasteners -Bottom rail connection block to post

#10x1.5" 55 PHP SMS Screw
Check shear @ post (6005-T5)
2x F_{upost}x dia screw x Post thickness x SF
Eq 5.4.3-2
$V= 38 \text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \dfrac{1}{3 \text{ (FS)}} =$
V = 240#/screw
Since minimum of 2 screws used for each
Allowable load = $2 \cdot 240\# = 480\#$
Rail Connection to RCB
2 screws each end
#8 Tek screw to 6063-T6
ADM Eq. 5.4.3-1
$2*30\text{ksi} \cdot 0.1248" \cdot 0.07" \cdot 1/3 = 175\#/\text{screw}$



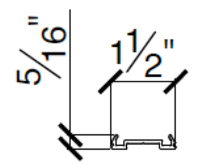
Allowable shear = 2*175 = 350#
OK

MID RAIL



$I_{xx} = 0.123\ in^4$
$I_{yy} = 0.177\ in^4$
$S_{xx} = 0.115\ in^3$
$S_{yy} = 0.209\ in^3$
$r_{xx} = 0.579\ in$
$r_{yy} = 0.695\ in$
$Z_{xx} = 0.1916\ in^3$
$Z_{yy} = 0.2397\ in^3$

Allowable stresses ADM Table 2-21 6063-T6 Aluminum
For vertical loads:
$F_{Cb} \rightarrow R_b/t = 1.75''/0.080'' = 21.6$
$F_c/\Omega = 15.2ksi$
$M_a = 0.1916in^3 * 15.2ksi = 2{,}912''\#$



For horizontal loads:
$b/t = 0.8667''/.0625'' = 13.9$
$F_c/\Omega = 15.2ksi$
$M_a = 0.2397in^3 * 15.2ksi = 3{,}643''\#$

Allowable vertical loading:
Distributed load = $2{,}912''\# * 8/72^2 = 4.493pli = 53.93plf$
Point load = $2{,}912''\# * 4/72 = 162\#$

Allowable horizontal loading:
Distributed load = $3{,}643''\# * 8/72''^2 = 5.622pli = 67.46plf$
Point load = $3{,}643''\# * 4/72'' = 202\#$

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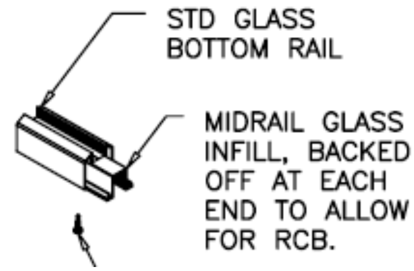
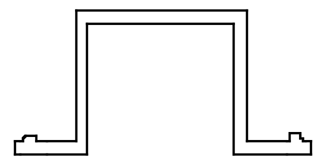


WIND SCREEN MID RAIL
Standard bottom rail with infill
Refer to bottom rail calculations for rail properties.

Check bottom rail strength for span used in privacy screen.

Midrail glass infill when installed in rail will stiffen the flanges (legs) continuously so that the flanges are equivalent to flat elements supported on both edges:
From ADM Table 2-21 section 16.
b/t = 1.1”/0.07 = 15.7 < 22
Therefore F_{ca} = 15.2 ksi

Strength of infill piece:
I_{xx}: 0.0162in^4
I_{yy}: 0.0378 in^4
S_{xx}: 0.0422 in^3
S_{yy}: 0.0490 in^3
F_{ca} = 15.2 ksi



When inserted into bottom rail determine the effective strength:
proportion of load carried by infill:

I_{yy} infill/ I_{yy} net = 0.0378/(.0378+0.172) = 0.18

0.046/0.18=0.256 or 0.204/(1-.18)=0.249<0.256 so standard bottom rail controls

Allowable Moments ➔ Horiz.=1,585”#/(1-.18) = 1,933”#
Maximum allowable load for 70” screen width L = 70”-1”*2-2.375*2 = 63.25”

W = 1,933”#*8/(63.25”^{2}) = 3.87 pli = 46.39 plf

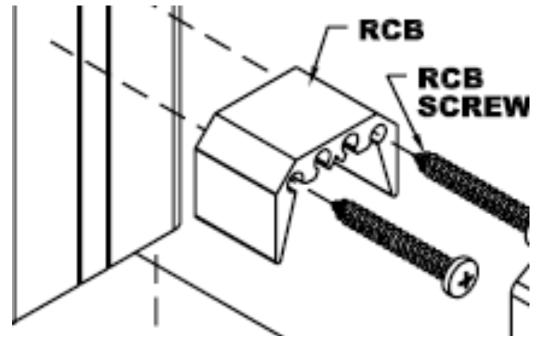
P = 1,933”#*4/63.25” = 122#

Maximum allowable load for 60” screen width L = 60”-1”*2-2.375*2 = 53.25”

W = 1,933”#*8/(53.25”^{2}) = 5.45 pli = 65.4 plf

P = 1,933”#*4/53.25” = 145#

STANDARD POST RAIL CONNECTION BLOCK



Can be used to connect top, mid and bottom rails to standard or 4”x4” post face, walls or other end butt connection conditions.

Rail snaps over block and is secured with either silicone adhesive or #8 tek screws.

Connection strength to post or wall: (2) #10x1.5” 55 PHP SMS Screw

Check shear @ post (6005-T5)
F_{upost}x dia screw x Post thickness / SF
Eq 5.4.3-2
$V = 38\text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\text{ (SF)}} =$ 240#/screw for standard post
Since minimum of 2 screws used for each, Allowable load = 2· 240# = 480#

For 4”x4” post:
$V = 38\text{ ksi} \cdot 0.19" \cdot 0.15" \cdot \frac{1}{3\text{ (SF)}} =$ 360#/screw for standard post
Since minimum of 2 screws used for each, Allowable load = 2· 360# = 720#

Connections to walls and other surfaces is dependant on supporting material. Alternative fasteners may be used for connections to steel, concrete or wood.

For connection to wood post:
(2) #10 x2-1/2” wood screws strength from NDS Table 11M, G ≥ 0.43
$Z' = n*C_D*Z = 2\text{ screws}*1.6*140\# = 448\#$

For connection to cold formed steel stud - 22 ga min based on CCFSS T.B. V2#1
Z = 2*175# = 350#

For connection to concrete or CMU - (2) 3/16” x 2” Tapcon screws
Z = 2*290 = 580#

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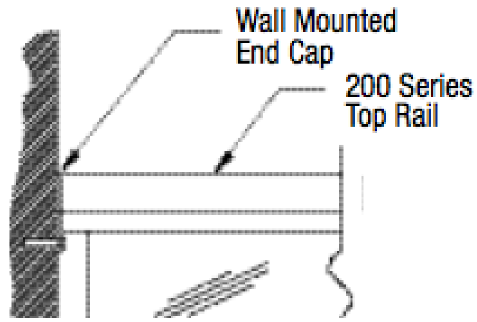
WALL MOUNT END CAPS
End cap is fastened to the top rail with
2) #10x1" 55 PHP SMS Screws

2x F_{upost}x dia screw x Cap thickness x SF
Eq 5.4.3-2
$V = 2*38 \text{ ksi} \cdot 0.19" \cdot 0.15" \cdot \frac{1}{3\,(FS)} =$
722#/screw , 1,444# per connection

Connection to wall shall use either:



#14x1-1/2" wood screw to wood, minimum 1" penetration into solid wood.

Allowable load = 2*175# = 350#
Wood shall have a G ≥ 0.43
From NDS Table 12M

For connection to steel studs or sheet metal blocking
Use #12 self drilling screws.
Minimum metal thickness is 18 gauge, 43 mil (0.0451")
Allowable load = 280#/screw

Table 3: Suggested Capacity for Screws Connecting Steel to Steel (lbs.)

Steel Thickness - Thinnest Component	¼ -14 Screw		#12-14 Screw		#10-16 Screw *		#8-18 Screw *		#6 Screw *	
	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout
0.1017"	1000	320	890	280	780	245	675	210	560	175
0.0713"	600	225	555	195	520	170	470	145	395	125
0.0566"	420	180	390	155	370	135	340	115	310	95
0.0451"	300	140	280	120	260	105	240	90	220	75
0.0347"	200	110	185	95	175	80	165	70	150	60

Notes:
1. Design values are based on CCFSS Technical Bulletin Vol. 2, No. 1 which outlines the proposed AISI Specification provisions for screw connections. For shear connections the cold-formed steel section should be evaluated for tension.
2. Based on Fy = 33ksi, Fu = 45ksi minimum. Adjust values for other steel strengths.
3. * = Refer to Table 1 for limits on recommended total steel thickness of connected parts.

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Wall Mounted End Caps – Cont.

For connection to masonry or concrete use 3/16 screw-in anchor-
Allowable shear load ≥ 290# per Tapcon

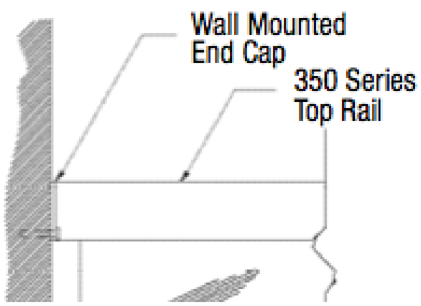
TABLE 2—EXAMPLE ALLOWABLE STRESS DESIGN VALUES FOR ILLUSTRATIVE PURPOSES FOR TAPCON WITH ADVANCED THREADFORM TECHNOLOGY ANCHOR[1,2,3,4,5,6,7,8]

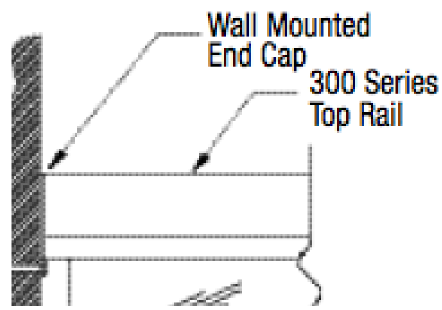
NOMINAL ANCHOR DIAMETER (inch)	EFFECTIVE EMBEDMENT DEPTH (inches)	ALLOWABLE LOADS (pounds)				
		Tension				Shear
		2,500 psi	3,000 psi	4,000 psi	5,000 psi	2,500 psi
$^3/_{16}$	1.5	260	285	330	370	290
$^1/_4$	1.5	350	385	445	495	525

For **SI:** 1 inch = 25.4 mm, 1 lbf = 4.45 N, 1 psi = 0.006895 MPa.

[1]Single anchor with static tension load only.
[2]Concrete determined to remain uncracked for the life of the anchorage.
[3]Load combination 9-2 from ACI 318 Section 9.2 (no seismic loading).
[4]Thirty percent dead load and 70 percent live load, controlling load combination 1.2D + 1.6L.
[5]Calculation of weighted average for $\alpha = 0.3^* 1.2 + 0.7^* 1.6 = 1.48$.
[6]Normal weight concrete
[7]$c_{a1} = c_{a2} > c_{ac}$.
[8]$h \geq h_{min}$.
[9]Condition B in accordance with ACI 318 Section D.4.4 applies.

300 and 350 Series end caps use same fasteners and have identical strengths





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Excerpts from National Design Specifications For Wood Construction

Table 11.2A Lag Screw Reference Withdrawal Design Values, W[1]

Tabulated withdrawal design values (W) are in pounds per inch of thread penetration into side grain of wood member. Length of thread penetration in main member shall not include the length of the tapered tip (see 11.2.1.1).

Specific Gravity, G[2]	Lag Screw Diameter, D										
	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1-1/8"	1-1/4"
0.73	397	469	538	604	668	789	905	1016	1123	1226	1327
0.71	381	450	516	579	640	757	868	974	1077	1176	1273
0.68	357	422	484	543	600	709	813	913	1009	1103	1193
0.67	349	413	473	531	587	694	796	893	987	1078	1167
0.58	281	332	381	428	473	559	641	719	795	869	940
0.55	260	307	352	395	437	516	592	664	734	802	868
0.51	232	274	314	353	390	461	528	593	656	716	775
0.50	225	266	305	342	378	447	513	576	636	695	752
0.49	218	258	296	332	367	434	498	559	617	674	730
0.47	205	242	278	312	345	408	467	525	580	634	686
0.46	199	235	269	302	334	395	453	508	562	613	664
0.44	186	220	252	283	312	369	423	475	525	574	621
0.43	179	212	243	273	302	357	409	459	508	554	600
0.42	173	205	235	264	291	344	395	443	490	535	579
0.41	167	198	226	254	281	332	381	428	473	516	559
0.40	161	190	218	245	271	320	367	412	455	497	538
0.39	155	183	210	236	261	308	353	397	438	479	518
0.38	149	176	202	227	251	296	340	381	422	461	498
0.37	143	169	194	218	241	285	326	367	405	443	479
0.36	137	163	186	209	231	273	313	352	389	425	460
0.35	132	156	179	200	222	262	300	337	373	407	441
0.31	110	130	149	167	185	218	250	281	311	339	367

1. Tabulated withdrawal design values, W, for lag screw connections shall be multiplied by all applicable adjustment factors (see Table 10.3.1).
2. Specific gravity,G, shall be determined in accordance with Table 11.3.3A.

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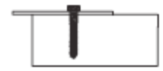
Table 11K LAG SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3,4]



for sawn lumber or SCL with ASTM A653, Grade 33 steel side plate (for t_s<1/4") or ASTM A36 steel side plate (for t_s=1/4")
(tabulated lateral design values are calculated based on an assumed length of lag screw penetration, p, into the main member equal to 8D)

Side Member Thickness	Lag Screw Diameter	G=0.67 Red Oak		G=0.55 Mixed Maple Southern Pine		G=0.5 Douglas Fir-Larch		G=0.49 Douglas Fir-Larch (N)		G=0.46 Douglas Fir(S) Hem-Fir(N)		G=0.43 Hem-Fir		G=0.42 Spruce-Pine-Fir		G=0.37 Redwood (open grain)		G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods		G=0.35 Northern Species	
t_s in.	D in.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.	$Z_{\parallel}$ lbs.	$Z_{\perp}$ lbs.
0.075	1/4	170	130	160	120	150	110	150	110	150	100	140	100	140	100	130	90	130	90	130	90
(14 gage)	5/16	220	160	200	140	190	130	190	130	190	130	180	120	180	120	170	110	170	110	160	100
	3/8	220	160	200	140	200	130	190	130	190	120	180	120	180	120	170	110	170	100	170	100
0.105	1/4	180	140	170	130	160	120	160	120	160	110	150	110	150	110	140	100	140	100	140	90
(12 gage)	5/16	230	170	210	150	200	140	200	140	190	130	190	130	190	120	180	110	170	110	170	110
	3/8	230	160	210	140	200	140	200	130	200	130	190	120	190	120	180	110	180	110	170	110
0.120	1/4	190	150	180	130	170	120	170	120	160	120	160	110	160	110	150	100	150	100	140	100
(11 gage)	5/16	230	170	210	150	210	140	200	140	200	140	190	130	190	130	180	120	180	120	180	110
	3/8	240	170	220	150	210	140	210	140	200	130	200	130	190	120	180	110	180	110	180	110
0.134	1/4	200	150	180	140	180	130	170	130	170	120	160	120	160	110	150	110	150	100	150	100
(10 gage)	5/16	240	180	220	160	210	150	210	140	200	140	200	130	200	130	190	120	180	120	180	120
	3/8	240	170	220	150	220	140	210	140	210	140	200	130	200	130	190	120	190	120	180	110
0.179	1/4	220	170	210	150	200	150	200	140	190	140	190	130	190	130	180	120	170	120	170	120
(7 gage)	5/16	260	190	240	170	230	160	230	160	230	150	220	150	220	150	210	130	200	130	200	130
	3/8	270	190	250	170	240	160	240	160	230	150	220	140	220	140	210	130	210	130	200	130
0.239	1/4	240	180	220	160	210	150	210	150	200	140	190	140	190	130	180	120	180	120	180	120
(3 gage)	5/16	300	220	280	190	270	180	260	180	260	170	250	160	250	160	230	150	230	150	230	140
	3/8	310	220	280	190	270	180	270	180	260	170	250	160	250	160	240	140	230	140	230	140
	7/16	420	290	390	260	380	240	370	240	360	230	350	220	350	220	330	200	330	200	320	190
	1/2	510	340	470	300	460	290	450	280	440	270	430	260	420	260	400	240	400	230	390	230
	5/8	770	490	710	430	680	400	680	400	660	380	640	370	630	360	600	330	590	330	580	320
	3/4	1110	670	1020	590	980	560	970	550	950	530	920	500	910	500	860	450	850	450	840	440
	7/8	1510	880	1390	780	1330	730	1320	710	1280	690	1250	650	1230	650	1170	590	1160	590	1140	570
	1	1940	1100	1780	960	1710	910	1700	890	1650	860	1600	820	1590	810	1500	740	1480	730	1460	710
1/4	1/4	240	180	220	160	210	150	210	150	200	140	200	140	190	130	180	120	180	120	180	120
	5/16	310	220	280	200	270	180	270	180	260	170	250	170	250	160	230	150	230	150	230	140
	3/8	320	220	290	190	280	180	270	180	270	170	260	160	250	160	240	150	240	140	230	140
	7/16	480	320	440	280	420	270	420	260	410	250	390	240	390	230	370	220	360	210	360	210
	1/2	580	390	540	340	520	320	510	320	500	310	480	290	480	290	460	270	450	260	440	260
	5/8	850	530	780	470	750	440	740	440	720	420	700	400	690	400	660	370	650	360	640	350
	3/4	1200	730	1100	640	1060	600	1050	590	1020	570	990	540	980	530	930	490	920	480	900	470
	7/8	1600	930	1470	820	1410	770	1400	750	1360	720	1320	690	1310	680	1240	630	1220	620	1200	600
	1	2040	1150	1870	1000	1800	950	1780	930	1730	900	1680	850	1660	840	1570	770	1550	760	1530	740

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 10.3.1).
2. Tabulated lateral design values, Z, are for "reduced body diameter" lag screws (see Appendix Table L2) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 8D; dowel bearing strengths, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and 87,000 psi for ASTM A36 steel and screw bending yield strengths, F_{yb}, of 70,000 psi for D = 1/4", 60,000 psi for D = 5/16", and 45,000 psi for D $\geq$3/8".
3. Where the lag screw penetration, p, is less than 8D but not less than 4D, tabulated lateral design values, Z, shall be multiplied by p/8D or lateral design values shall be calculated using the provisions of 11.3 for the reduced penetration.
4. The length of lag screw penetration, p, not including the length of the tapered tip, E (see Appendix Table L2), of the lag screw into the main member shall not be less than 4D. See 11.1.4.6 for minimum length of penetration, p_{min}.

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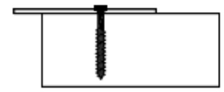
Table 11M WOOD SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections[1,2,3]

for sawn lumber or SCL with ASTM 653, Grade 33 steel side plate
(tabulated lateral design values are calculated based on an assumed length of wood screw penetration, p, into the main member equal to 10D)



Side Member Thickness t_s	Wood Screw Diameter D	Wood Screw Number	G=0.67 Red Oak	G=0.55 Mixed Maple Southern Pine	G=0.5 Douglas Fir-Larch	G=0.49 Douglas Fir-Larch(N)	G=0.46 Douglas Fir(S) Hem-Fir(N)	G=0.43 Hem-Fir	G=0.42 Spruce-Pine-Fir	G=0.37 Redwood (open grain)	G=0.36 Eastern Softwoods Spruce-Pine-Fir(S) Western Cedars Western Woods	G=0.35 Northern Species
in.	in.		lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
0.036	0.138	6	89	76	70	69	66	62	60	54	53	52
(20 gage)	0.151	7	99	84	78	76	72	68	67	60	59	57
	0.164	8	113	97	89	87	83	78	77	69	67	66
0.048	0.138	6	90	77	71	70	67	63	61	55	54	53
(18 gage)	0.151	7	100	85	79	77	74	69	68	61	60	58
	0.164	8	114	98	90	89	84	79	78	70	69	67
0.060	0.138	6	92	79	73	72	68	64	63	57	56	54
(16 gage)	0.151	7	101	87	81	79	75	71	70	63	61	60
	0.164	8	116	100	92	90	86	81	79	71	70	68
	0.177	9	136	116	107	105	100	94	93	83	82	79
	0.190	10	146	125	116	114	108	102	100	90	88	86
0.075	0.138	6	95	82	76	75	71	67	66	59	58	57
(14 gage)	0.151	7	105	90	84	82	78	74	72	65	64	62
	0.164	8	119	103	95	93	89	84	82	74	73	71
	0.177	9	139	119	110	108	103	97	95	86	84	82
	0.190	10	150	128	119	117	111	105	103	92	91	88
	0.216	12	186	159	147	145	138	130	127	114	112	109
	0.242	14	204	175	162	158	151	142	139	125	123	120
0.105	0.138	6	104	90	84	82	79	74	73	66	65	63
(12 gage)	0.151	7	114	99	92	90	86	81	80	72	71	69
	0.164	8	129	111	103	102	97	92	90	81	80	77
	0.177	9	148	128	119	116	111	105	103	93	91	89
	0.190	10	160	138	128	125	120	113	111	100	98	96
	0.216	12	196	168	156	153	146	138	135	122	120	116
	0.242	14	213	183	170	167	159	150	147	132	130	126
0.120	0.138	6	110	95	89	87	83	79	77	70	68	67
(11 gage)	0.151	7	120	104	97	95	91	86	84	76	75	73
	0.164	8	135	117	109	107	102	96	94	85	84	82
	0.177	9	154	133	124	121	116	110	107	97	95	93
	0.190	10	166	144	133	131	125	118	116	104	103	100
	0.216	12	202	174	162	159	152	143	140	126	124	121
	0.242	14	219	189	175	172	164	155	152	137	134	131
0.134	0.138	6	116	100	93	92	88	83	81	73	72	70
(10 gage)	0.151	7	126	110	102	100	96	91	89	80	79	77
	0.164	8	141	122	114	112	107	101	99	89	88	86
	0.177	9	160	139	129	127	121	114	112	101	100	97
	0.190	10	173	149	139	136	130	123	121	109	107	104
	0.216	12	209	180	167	164	157	148	145	131	129	126
	0.242	14	226	195	181	177	169	160	157	141	139	135
0.179	0.138	6	126	107	99	97	92	86	84	76	74	72
(7 gage)	0.151	7	139	118	109	107	102	95	93	84	82	80
	0.164	8	160	136	126	123	117	110	108	96	95	92
	0.177	9	184	160	148	145	138	129	127	113	111	108
	0.190	10	198	172	159	156	149	140	137	122	120	117
	0.216	12	234	203	189	186	178	168	165	149	146	143
	0.242	14	251	217	202	198	190	179	176	159	156	152
0.239	0.138	6	126	107	99	97	92	86	84	76	74	72
(3 gage)	0.151	7	139	118	109	107	102	95	93	84	82	80
	0.164	8	160	136	126	123	117	110	108	96	95	92
	0.177	9	188	160	148	145	138	129	127	113	111	108
	0.190	10	204	173	159	156	149	140	137	122	120	117
	0.216	12	256	218	201	197	187	176	172	154	151	147
	0.242	14	283	241	222	217	207	194	190	170	167	162

1. Tabulated lateral design values, Z, shall be multiplied by all applicable adjustment factors (see Table 10.3.1).
2. Tabulated lateral design values, Z, are for rolled thread wood screws (see Appendix L) inserted in side grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 10D; dowel bearing strength, F_e, of 61,850 psi for ASTM A653, Grade 33 steel and screw bending yield strengths, F_{yb}, of 100,000 psi for 0.099" $\leq$ D $\leq$ 0.142", 90,000 psi for 0.142" < D $\leq$ 0.177", 80,000 psi for 0.177"< D $\leq$ 0.236", 70,000 psi for 0.236" < D $\leq$ 0.273".
3. Where the wood screw penetration, p, is less than 10D but not less than 6D, tabulated lateral design values, Z, shall be multiplied by p/10D or lateral design values shall be calculated using the provisions of 11.3 for the reduced penetration.

VERTICAL PICKET INSTALLATIONS

LOAD CASES:

Picket rail Dead load = 5 plf for 42" rail height or less.

Loading:

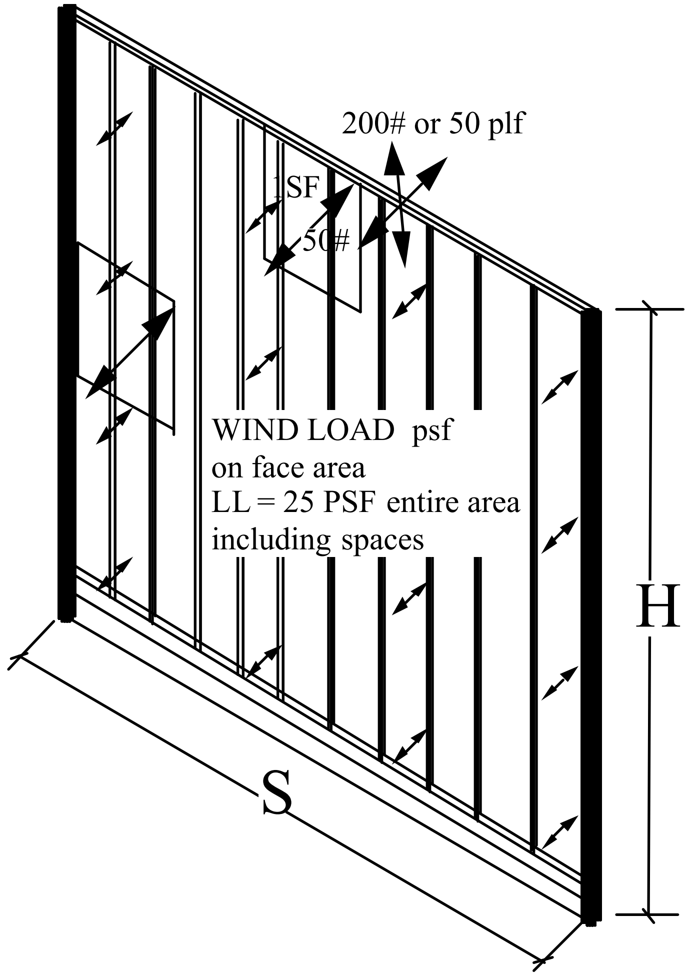
Horizontal load to top rail from in-fill:
25 psf*H/2

Post moments

M_i = 25 psf*H/2*S*H=
= (25/2)*S*H^2

For top rail loads:
M_c = 200#*H
M_u = 50plf*S*H

For wind load surface area:
Pickets 3/4" wide by 4" on center
Top rail = 3" maximum
Post = 2.375"
Area for typical 5' section by 42" high:
42"*2.375"+3"*60"+1.7"*57.625"
+0.75*36*18 = 863.7 in^2
% surface/area = 863.7/(60"*42") = 34.3%
Wind load for 25 psf equivalent load = 25/0.343 = 72.9 psf



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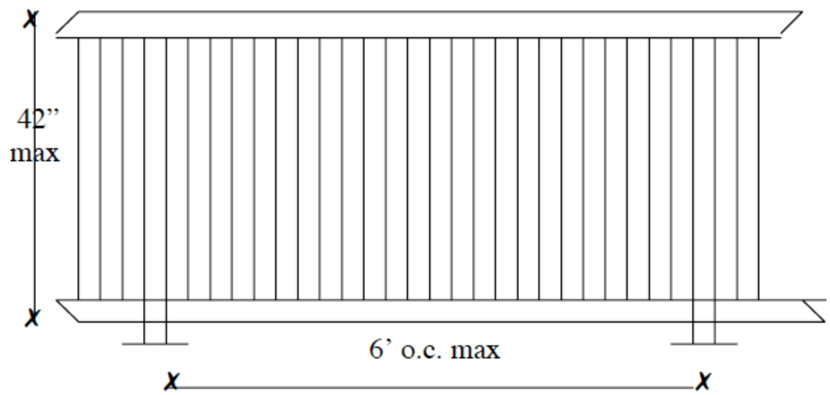
Picket Railing
Series 100
Top rail loading
50 plf or 200 lb conc.

Infill: 25 psf
Bottom rail loading
50 lb conc.



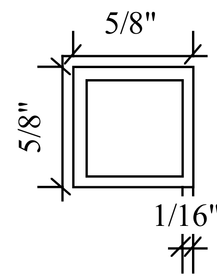
Picket infill panel is

Loading → 25 psf → 4
1/2” O.C → 25psf $\cdot$.375=9.4 plf

$$M = \frac{9.4/12\,(42\text{''}-6\text{''})2}{8} = 127 \text{ lb-in}$$

For 5/8" Square pickets t=0.062” → S= $0.625^3/6 - 0.5^3/6$ = 0.020 in^3

$$f_b = \frac{127 \text{ lb-in}}{0.02 in^3} = 6{,}350 \text{ psi}$$



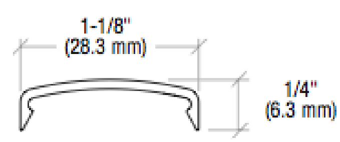
For 50 lb conc load → 1 SF - min 2 pickets

$$M = \frac{50/2 \cdot 36\text{''}}{4} = 225 \text{ lb-in}$$

$$f_b = \frac{225 \text{ lb-in}}{0.02 \, in^3} = 11{,}250 \text{ psi}$$

b/t = .5”/.0625” = 8 < 22.8
6063-T6 F_c/Ω= 15.2 ksi – compression ADM Table 2-21
Maximum allowable moment on picket = 15.2 ksi *0.02 in^3 = 304 in-lb
Maximum span = 304 in-lb*4/25 lb = 48.6” – concentrated load or
$(304\text{in-lb}*8/0.783 \text{ lb/in})^{1/2}$ = 55.73 in (based on 25 psf uniform load)
48.6” is the maximum allowable picket length.

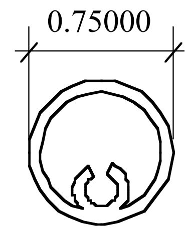
Connections
Pickets to top and bottom rails direct bearing –ok
Lap into top and bottom rail – 1” into bottom rail and 5/8” into top rail.



Allowable bearing pressure = 21 ksi (ADM Table 2-24 line 6)
Picket filler snaps between pickets to pressure lock pickets in place.
Bearing surface = 1.375”*.062” = 0.085 in^2
Allowable bearing = 0.085 in^2*21 ksi = 1,785#
Withdrawal prevented by depth into rails.

PICKETS 3/4" ROUND

Area: 0.170 sq in
I_{xx}: 0.0093 in^4 S_{xx}: 0.022 in^3
I_{yy}: 0.0083 in^4 S_{yy}: 0.022 in^3
r_{xx}: 0.234267 in Z_{xx}: 0.03611in^3
r_{yy}: 0.221764 in Z_{yy}: 0.03133in^3



R_b/t = 0.75"/.0625" = 12 < 31.2
F_c/Ω = 15.2ksi

Allowable moment, M_a=0.03611in^3*15.2ksi=549"#

For 50 lb conc load → 1 SF - min 2 pickets

$$M= \frac{50/2 \cdot 36"}{4} = 225 \text{ lb-in} < 549"\#$$

Max picket span = 549"#*4/25# = 87"

Connections
#10 screw in to top and bottom infill pieces. Shear strength =
2x F_{upost}x dia screw x t_{rail} x SF ADM Eq 5.4.3-2

$$V= 38 \text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\ (FS)} = 240\#$$

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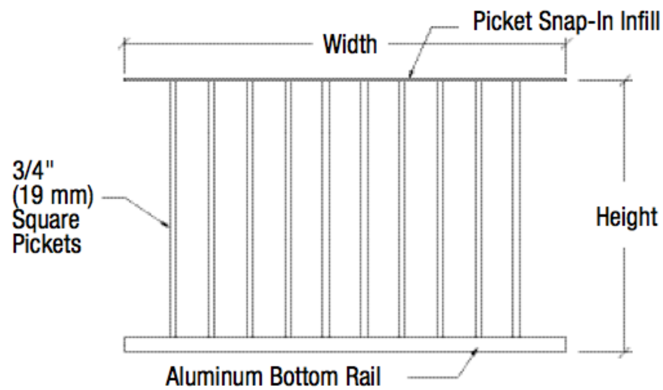
PICKETS 3/4" SQUARE



Z_x=0.0685in^3

b/t = 0.55"/0.1" = 5.5

Allowable moment,
M_a=0.0685in^3*15.2ksi=1,041"#

For 50 lb conc load → 1 SF - min 2 pickets

$$M = \frac{50/2 \cdot 36"}{4} = 225 \text{ lb-in} < 1{,}041"\#$$

Max picket span = 1,041"#*4/25# = 167"

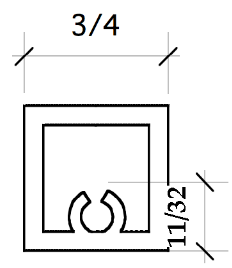
Connections
Pickets to top and bottom rails direct bearing for lateral loads –ok
#10 screw in to top and bottom infill pieces. Shear strength =
2x F_{upost}x dia screw x t_{rail} x SF ADM Eq 5.4.3-2

$$V = 30 \text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\ (FS)} = 190\#$$

Area: 0.288 sq in
Perim: 6.03 in

Ixx: 0.0196 in^4
Iyy: 0.0190 in^4
Kxx: 0.261 in
Kyy: 0.257 in

Cxx: 0.392 in
Cyy: 0.376 in
Sxx: 0.050 in^3
Syy: 0.051 in^3



PICKET

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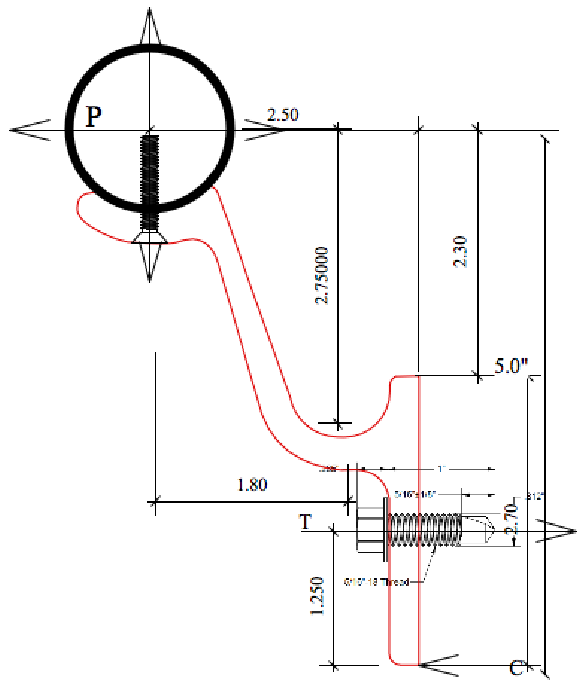
GRAB RAIL BRACKET
Loading 200 lb concentrated load or
50 plf distributed load



Grab rail bracket – 1-7/8" long
Aluminum extrusion 6063-T6
Allowable load on bracket:
Vertical load:
Critical point @ 1.8" from rail to root of double radius, t = 0.25"
M = P*1.8" or WS*1.8"
where P = 200#, W = 50 plf and
S = tributary rail length to bracket.
Determine allowable Moment:
F_T = 20 ksi, F_C = 20 ksi
From ADM Table 2-24
$S_V = 1.875"*0.25^2/6 = 0.0195$ in^3
$M_{Vall} = 0.0195$ in^3*20 ksi = 390"#

Determine allowable loads:
For vertical load:

P_{all} = 390"#/1.8" = 217#
S_{all} = 217#/50plf = 4'4"

Vertical loading will control bracket strength.

Allowable load may be increased proportionally by increasing the bracket length.

For 5' Post spacing: 5'/4.33'*1.875" = 2.165" – 2-11/64"

Grab rail connection to the bracket:

Two countersunk self drilling #8 screws into 1/8" wall tube
Shear – $2F_{tu}Dt/3$ = 2*30ksi*0.164"*0.125"/3*2 screws = 820# (ADM 5.4.3)
Tension – $1.2DtF_{ty}/3$ = 1.2*.164"*0.125"*25ksi*2 screws/3 = 410#

For residential installations only 200# concentrated load is applicable.

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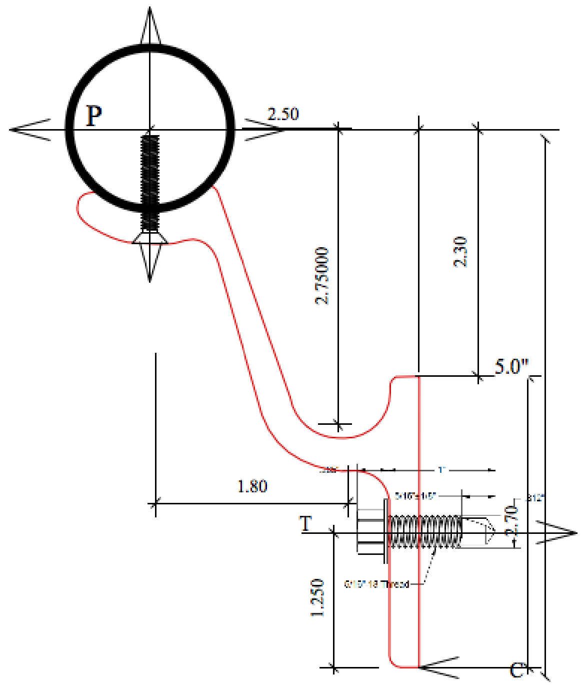

Connection to support:

Maximum tension occurs for outward Horizontal force = 200#:

Determine tension from ∑M about C

0= P*5" – T*1.25"

T = 200#*(5-1.25)"/1.25" = 600#

From ∑ forces – no shear force in anchor occurs from horizontal load

Vertical force = 200#+17# (DL):

Determine tension from ∑M about C

0= P*2.5" – T*1.25"

T = 217#*2.5"/1.25" = 434#

From ∑ forces – Z = P = 217#

CONNECTION TO STANDARD POST (0.1" WALL)

For 200# bracket load:

For handrails mounted to 0.1" wall thickness aluminum tube.

1/4" self drilling hex head screw at post screw slot - effective thickness = 0.125"

Shear – $2F_{tu}Dt/3$ (ADM 5.4.3)

2*38ksi*0.25"*0.125"/3= 792#

Tension – Pullout ADM 5.4.2.1

$P_t = 1.2DtF_{tu}/3$ = 1.2*.25*.125*38ksi/3 = 475#

Required attachment strength

T = 434# and V = 217# or

T = 600 # and V = 0

Two screws minimum , T_a=2*475#=950# > 600# OK

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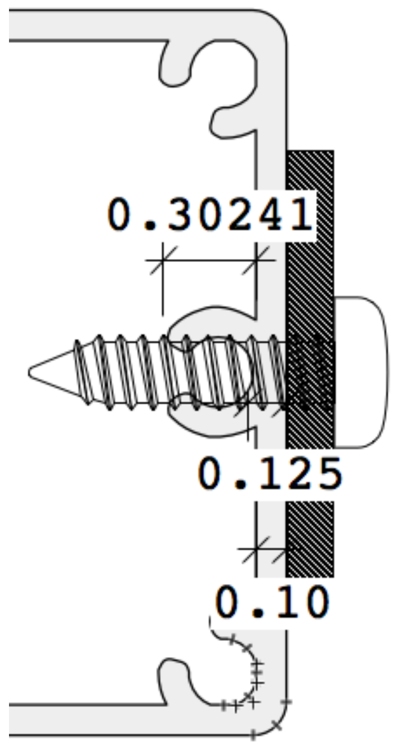
6 SCREW POST

For mounting to the 6 screw post with screw at the center screw slot:
For 200# bracket load:
For handrails mounted to 0.1" wall thickness aluminum tube.
1/4" self drilling hex head screw at post screw slot - thickness = 0.125"
This ignores contribution form the sides of the screw slot and considers only the bottom where there is full thread engagement.



Safety Factor = 2.34 for guard rail application.
Shear – $F_{tu}Dt/2.34$ (2015ADM 5.5)
38ksi*0.2496"*0.125"/2.34= 507#
Tension – Pullout 2015 ADM 5.4.1
$P_t = 0.58A_{sn}F_{tu}(t_c)]/2.34 =$
0.58*0.682*38ksi(0.10)/2.34= 642#

Required attachment strength
T = 434# and V = 217# or
T = 600 # and V = 0
For combined shear and tension (Vertical load case)
$(T/P_t)^2 + (V/Z_a)^2 \leq 1$
$(434/642)^2 + (217/508)^2 = 0.639 \leq 1$
Or
$(434/642) + (217/508) = 1.10 \leq 1.2$
Or
$600 \leq 642\#$ therefore okay

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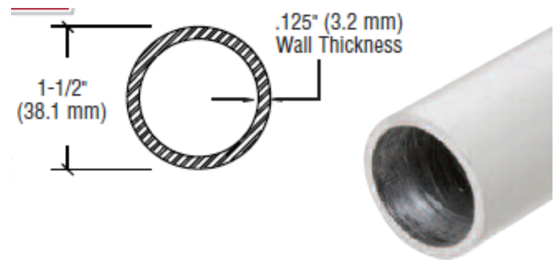
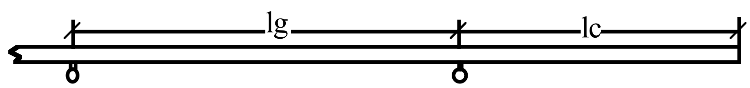
GRAB RAIL –1-1/2” x 1/8” WALL
6063-T6 Aluminum



Pipe properties:
O.D. = 1.50”
I.D. = 1.25”, t = 0.125”
A = 0.540 in^2
I = 0.129 in^4
S = 0.172 in^3
Z = 0.237 in^3

Allowable stresses from ADM Table 2-21
R_b/t = 0.625/0.125 = 5 < 70;
F_c/Ω = 27.7-1.70*$5^{1/2}$ = 23.90 ksi, Use 22.7ksi max
M_a= Z*F_y = 0.237*22.7 ksi = 5,380”# = 448.3’#

Allowable Span:
Check based on simple span and cantilevered section.



M = w$(lg)^2$/8 or = P(lg)/4 Solve for lg:
lg = $(8M/w)^{1/2}$ = $[8*(448.3’\#/50plf)]^{1/2}$ = 8.47’ or
lg = (4M/P) = 4*448.3’#/200# = 8.97’
Maximum allowable span for supports at both ends=8’-5 5/8”-Controlling span

For cantilevered section
M = w$(lc)^2$/2 or = P(lc) Solving for lc
lc = (2M/w)1/2 = $(2*448.3’\#/50plf)^{1/2}$ = 4.23’ or
lc = M/P = 448.3’#/200# = 2.24’ = 2’ -2 7/8” ----- Controlling span

Locate splice within lc of a support.

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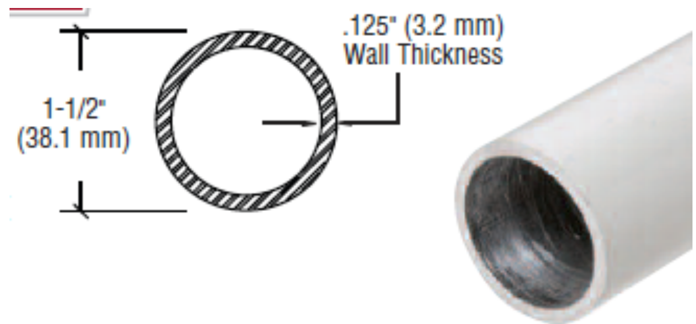
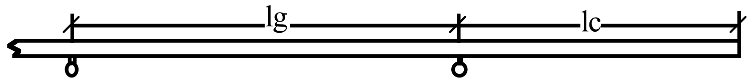
GRAB RAIL –1-1/2" x 1/8" WALL
Stainless Steel

Pipe properties:
O.D. = 1.50"
I.D. = 1.25", t = 0.125"
A = 0.540 in^2
I = 0.129 in^4
S = 0.172 in^3
Z = 0.236 in^3 minimum
r = 0.488 in, J = 0.255 in^4



Stainless steel tube in accordance with ASTM A554-10
Rail Service Loading:
Brushed stainless steel, $F_y \geq 45$ ksi, $F_u \geq 91$ ksi (Requires Mill Certification Tests)
$øM_n$= 0.9*1.25*S*F_y = 0.9*1.25*0.172*45 ksi
$øM_n$ = 8,707.5"#
M_l = $øM_n$/1.6 = 5,442.2"# = 453.52'#

Allowable Span:
Check based on simple span and cantilevered section.

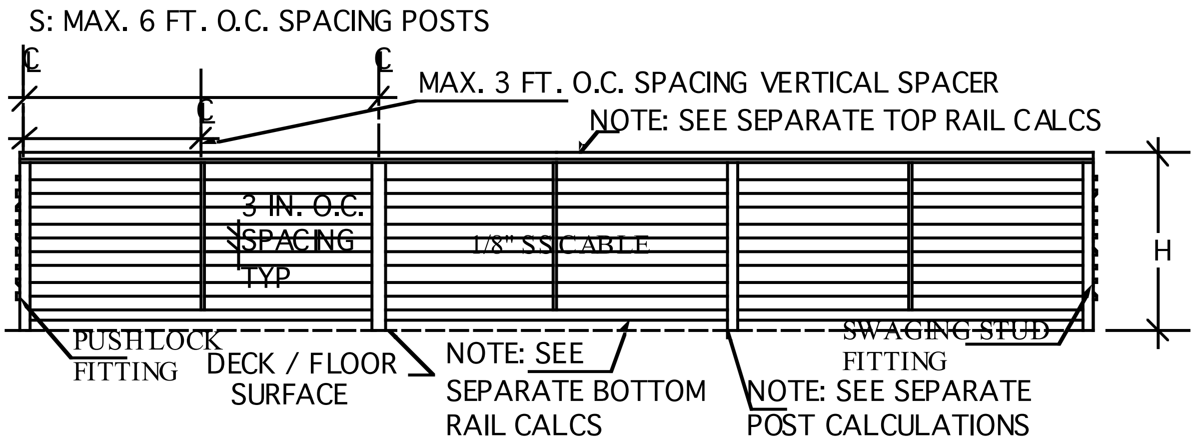


M = w$(lg)^2$/8 or = P(lg)/4 Solve for lg:
lg = $(8M/w)^{1/2}$ = $[8*(453.52'\#/50plf)]^{1/2}$ = 8.518' or
lg = (4M/P) = 4*453.52'#/200# = 9.07'
Maximum allowable span for supports at both ends = 8'-6 3/16"-Controlling span

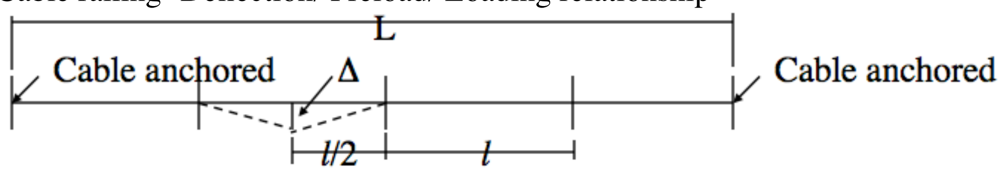
For cantilevered section
M = w$(lc)^2$/2 or = P(lc) Solving for lc
lc = $(2M/w)^{1/2}$ = $(2*453.52'\#/50plf)^{1/2}$ = 4.259' or
lc = M/P = 453.52'#/200# = 2.268' = 2' -3 3/16" ----- Controlling span

Locate splice within lc of a support.

STAINLESS STEEL CABLE IN-FILL:



Cable railing- Deflection/ Preload/ Loading relationship



Cable Strain = $\in = \dfrac{C_{ta} \bullet L}{A \bullet E}$

$C_t = C_{tl} + C_{ta}$ C_{ti} = installation tension

$C_{ta} = \dfrac{\in EA}{L}$ = Cable tension increase from loading

From cable theory

$C_t = \dfrac{l \bullet p}{4\Delta}$ for concentrated load

To calculate allowable load for a given deflection:

Calculate $\in = [[(l/2)^2 + \Delta^2]^{1/2} \bullet 2 - l]$

Then calculate $C_{ta} = \dfrac{\in AE}{L}$

Then calculate $C_t = C_{tl} + C_{ta}$

Then calculate load to give the assumed Δ for concentrated load

$$P = \frac{C_t 4\Delta}{l}$$

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For uniform load – idealize deflection as triangular applying cable theory

$$C_t = \frac{Wl^2}{8\Delta}$$

Solving for $W = \frac{C_t\, 8\, \Delta}{l^2}$

See spreadsheet pages based on 36' maximum cable length and 3" clear cable spacing.

Cable rail loading requirements

UBC table 16-B Line 9

Guardrail components 25 psf over entire area

IBC 1607.7.1.2 Components

50 lbs Conc. load over 1 sf

Application to cables

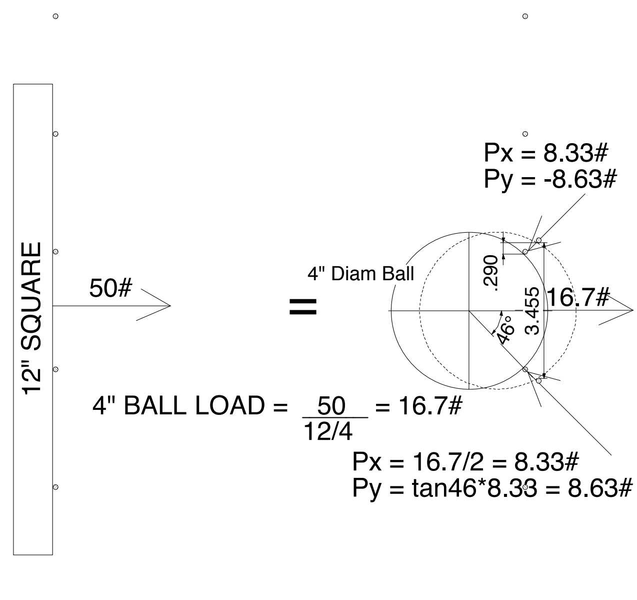
-Uniform load = $\frac{25 \text{ psf} \bullet 3"}{12"}$ = 6.25 plf

-Concentrated load 1 sf

3 cables minimum

50/3 = 16.7 lbs on 4" sphere

Produces 8.63 lb upward and downward on adjacent cables.



Deflection – since cables are 3" O.C. and maximum opening width = 4"

for 1/8" cable Δ_{all} = 4" – (3- 1/8) = 1 1/8"

for 3/16" cable Δ_{all} = 4" – (3- 3/16) = 1 3/16"

Cable Strain:

$\varepsilon = \sigma/E$ and $\Delta_L = L\,\varepsilon$

$\Delta_L = L(T/A)/E = L(T/0.0276 \text{ in}^2)/26 \times 10^6$ psi

Maximum cable free span length = 60.5"/2-2.375" = 27.875"

Additionally cable should be able to safely support 200 lb point load such as someone standing on a cable. This is not a code requirement but is recommended to assure a safe installation.

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Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		150			
Total Cable length (ft) =		36			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	2.6	152.6	4.4	3.0
0.375	0.00803	5.9	155.9	6.7	4.6
0.55	0.01728	12.8	162.8	10.2	7.0
0.75	0.03213	23.7	173.7	14.9	10.2
1	0.05710	42.2	192.2	22.0	15.1
2	0.22783	168.3	318.3	72.7	49.9
2.5	0.35534	262.4	412.4	117.8	80.8
3.13	0.55542	410.2	560.2	200.4	137.4

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		200			
Total Cable length (ft) =		36			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	2.6	202.6	5.8	4.0
0.375	0.00803	5.9	205.9	8.8	6.1
0.55	0.01728	12.8	212.8	13.4	9.2
0.75	0.03213	23.7	223.7	19.2	13.1
1	0.05710	42.2	242.2	27.7	19.0
2	0.22783	168.3	368.3	84.2	57.7
2.5	0.35534	262.4	462.4	132.1	90.6
3.02	0.51734	382.1	582.1	200.9	137.8

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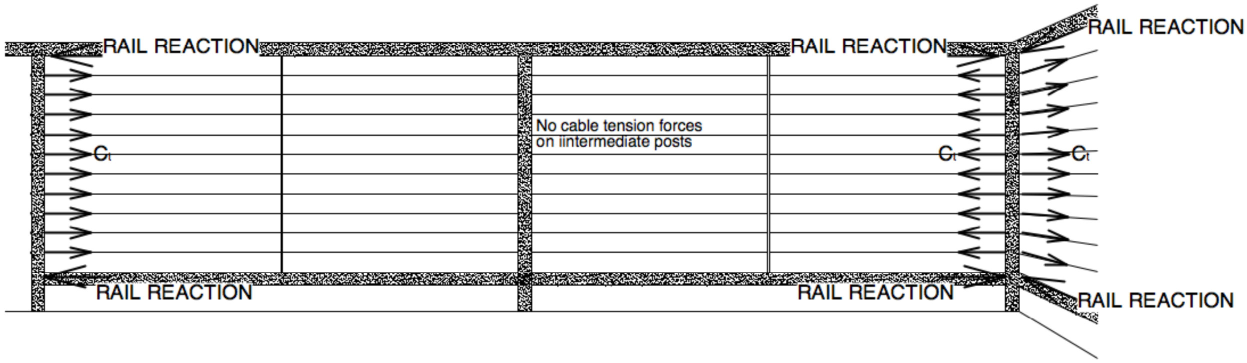
Cable induced forces on posts:



Cable tension forces occur where cables either change direction at the post or are terminated at a post.

Top rail acts as a compression element to resist cable tension forces. The top rail infill piece inserts tight between the posts so that the post reaction occurs by direct bearing.

For 400 Series top rail no infill is used. Top rail extrusion is attached to post with (6) #8 screws in shear with total allowable shear load of 6*325# = 1,950#
Up to eight #8 screws may be used on a post if required to develop adequate shear transfer between the post and the 400 series top rail.

Bottom rail when present will be in direct bearing to act as a compression element.

When no bottom rail is present the post anchorage shall be designed to accommodate a shear load in line with the cables of 7*205#*1.25 = 1,784#

End post Cable loading
Cable tension - 200#/ Cable no in-fill load

$w = \frac{200\#}{3''} = 66.67\#/in$ $\quad\quad M_w = \frac{(39'')^2 \bullet 66.67\#/in}{8} = 12{,}676\#''$

Typical post reactions for 200# installation tension :
11 cables*200#/2 = 1100# to top and bottom rails

For loaded Case

- 3 Cables @ center 220.7# ea based on 6' o.c. posts, 35" cable clear span

post deflection will reduce tension of other cables.

$\Delta = [Pa^2b^2/(3L)+2Pa(3L^2-4a^2)/24]/EI =$

$\Delta = [220.7*15^2*24^2/(3*39)+220.7*15(3*39^2-4*15^2)/24]/(10{,}100{,}000*0.863) = 0.086''$

Cable tension reduction for deflection will go from 200 at end cables to 271-220.7 at center, linear reduction = (200-50.3)/(39/2) = 7.7 pli

M_{conc} = 220.7# • 15"/2 +220.7#•18" +(3*(200-7.7*3)) + (6*(200-7.7*6)) + (9*(200-7.7*9)) +12*(200-7.7*12)+15*(200-7.7*15)/2
M_{conc} = 10,183#"

Typical post reactions for 200# installation tension with 50# infill load:
11 cables*200#/2+3*(221-200)/2 = 1132# to top and bottom rails.
Typical post reactions for 200# installation tension with 25 psf infill load:
11 cables*207.5#/2 = 1,141# to top and bottom rails.

For 200 # Conc load on middle cable tension
599.2# tension, post deflection will reduce tension of other cables
$\Delta = [Pa^2b^2/(3LEI) = [599.2*18^2 21^2/(3*39*10100000*0.863) = 0.084$
Cable tension reduction for deflection will go from 200 at end cables to 52 at center cables, linear reduction (200-52)/19.5" = 7.6 pli.
M_{200} = 599.2#/2 • 18" +(3)•(200-7.6*3) +(6) (200-7.6*6) +(9) (200-7.6*9) + (12) (200-7.6*12) +(15) (200-7.6*15) + (18) (200-7.6*18)/2 = 11,200#"

Post strength = 17,560"# (Weak axis for standard six screw post)
No reinforcement required.
Standard Cable anchorage okay.

Typical post reactions for 200# installation tension with 200# infill load on center cable:
11 cables*200#/2+(600#-200)/2 = 1,300# to top and bottom rails.

Typical post reactions for 200# tension with 200# infill load on top or bottom cable:
11 cables*200#/2+(600#-200)*33/36 = 1,467# to top and bottom rails.

Verify cable strength:
F_y = 110 ksi Minimum tension strength = 1,869# for ⅛" 1x19 cable
$øT_n$ = 0.85*110 ksi* 0.0123 = 1,150#
$T_s = øT_n / 1.6$ = 1,150# / 1.6 = 718#
Maximum cable pretension based on maximum service tension @ 200# cable load is 440#:

Δ (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.19	0.00206	1.7	441.7	9.6	6.6
0.33	0.00622	5.1	445.1	16.8	11.5
2.437	0.33774	278.2	718.2	200.0	137.2

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CABLE LENGTH/SPAN OPTIONS:
For a **maximum cable free span of 42”** (Maximum post spacing of 44-3/8” on center)
The Maximum allowable cable length is 36’.
Required minimum cable installation tension is 373#

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		373			
Total Cable length (ft) =		36			
Cable free span (inches) =		42			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00298	2.2	375.2	8.9	5.1
0.375	0.00670	4.9	377.9	13.5	7.7
0.55	0.01440	10.6	383.6	20.1	11.5
0.75	0.02678	19.8	392.8	28.1	16.0
1	0.04759	35.2	408.2	38.9	22.2
2	0.19005	140.4	513.4	97.8	55.9
2.5	0.29657	219.0	592.0	141.0	80.6
3.03	0.43493	321.2	694.2	200.3	114.5

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For a **maximum cable length of 60’**.
Maximum cable free span is 35”
Required minimum cable installation tension is 349#.
Intermediate tensioning device is required (turnbuckle or similar device).

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		349			
Total Cable length (ft) =		60			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	1.6	350.6	10.0	6.9
0.375	0.00803	3.6	352.6	15.1	10.4
0.55	0.01728	7.7	356.7	22.4	15.4
0.75	0.03213	14.2	363.2	31.1	21.3
1	0.05710	25.3	374.3	42.8	29.3
2	0.22783	101.0	450.0	102.8	70.5
2.5	0.35534	157.5	506.5	144.7	99.2
3.03	0.52075	230.8	579.8	200.8	137.7

NOTE: WHEN CABLE LENGTH EXCEEDS 36’ AN ADDITIONAL TENSIONING DEVICE IS REQUIRED TO TAKE UP CABLE STRAIN AND ASSURE ADEQUATE CABLE PRETENSION, WHEN LENGTH EXCEEDS 72’ THREE DEVICES ARE REQUIRED.

For a **maximum cable pretension of 440#**.
Maximum allowable cable length is 98.4'.
Maximum cable free span is 35"
Two intermediate tensioning devices are required (turnbuckle or similar device).

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		440			
Total Cable length (ft) =		98.4			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	1.0	441.0	12.6	8.6
0.375	0.00803	2.2	442.2	19.0	13.0
0.55	0.01728	4.7	444.7	28.0	19.2
0.75	0.03213	8.7	448.7	38.5	26.4
1	0.05710	15.4	455.4	52.0	35.7
2	0.22783	61.6	501.6	114.6	78.6
2.5	0.35534	96.0	536.0	153.1	105.0
3.02	0.51734	139.8	579.8	200.1	137.2

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For a **maximum cable pretension of 440#**.
Maximum allowable cable length is 45.2'.
Maximum cable free span is 42"
Intermediate tensioning device is required (turnbuckle or similar device).

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		440			
Total Cable length (ft) =		45.2			
Cable free span (inches) =		42			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00298	1.8	441.8	10.5	6.0
0.375	0.00670	3.9	443.9	15.9	9.1
0.55	0.01440	8.5	448.5	23.5	13.4
0.75	0.02678	15.8	455.8	32.6	18.6
1	0.04759	28.0	468.0	44.6	25.5
2	0.19005	111.8	551.8	105.1	60.1
2.5	0.29657	174.5	614.5	146.3	83.6
3.03	0.43493	255.9	695.9	200.8	114.7

For a **maximum post spacing of 60” on center with intermediate cable spreader**.
Maximum allowable cable length is 144’. (1/8” cable may not exceed this length.)
Maximum cable free span is 27.625” (Posts @ 60” on center with center picket)
Required cable pretension is 354#
Three intermediate tensioning devices are required (turnbuckle or similar device).

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		26000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		354			
Total Cable length (ft) =		144			
Cable free span (inches) =		27.625			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00452	0.8	354.8	12.8	11.2
0.375	0.01018	1.9	355.9	19.3	16.8
0.55	0.02189	4.0	358.0	28.5	24.8
0.75	0.04069	7.5	361.5	39.3	34.1
1	0.07230	13.4	367.4	53.2	46.2
2	0.28809	53.2	407.2	117.9	102.4
2.5	0.44884	82.9	436.9	158.1	137.4
2.95	0.62302	115.0	469.0	200.3	174.1

For 1/8” diameter cable:
Cable pretension, free span and total length under no circumstance shall exceed the following limits.
MAXIMUM CABLE PRETENSION SHALL NOT EXCEED 440#.
MAXIMUM CABLE FREE SPAN MAY NOT EXCEED 42”.
MAXIMUM CABLE LENGTH SHALL NOT EXCEED 144’.

Cable installation parameters are dependent on each other and must be balanced for the specific installation as shown in the examples herein. When cable length increases the allowable free span decreases. When cable free span increases the allowable cable length decreases.

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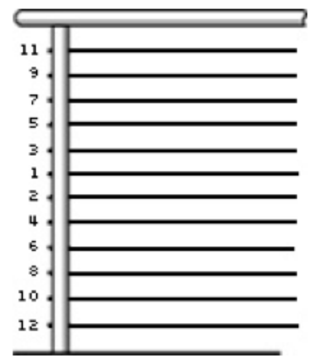
Cable installation instructions:

The desired cable installation tension is 200 lbs for all runs.
Cable tension is determined by the turn of the nut method:
Cables are pulled tight by hand when setting the quick connect bracket. The cable tension is increased to 200 lbs minimum by straining the cable by 0.153" (31' length). This requires 8.5 turns of the threaded terminal from the snug condition which is attained when the cable is pulled tight by hand. For every 5 feet of cable above 31' the nut shall be turned an additional 1/2 turn to achieve the required pretension. For every 5 feet of cable less than 31' the nut shall be turned 1/2 turn less to achieve the required pretension.

When installing the cables start with the lowest then go to the highest cable and alternate back and forth until all cables are installed, installing the center cable last, working from largest number down to 1 as shown in illustration.

Recommended Cable Tensioning Sequence



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