09 SEP 2010

Feeney Inc.
2603 Union Sreet
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SUBJ: FEENEY ARCHITECTURAL DESIGN-RAIL™
ALUMINUM RAILING
GLASS INFILL SYSTEMS
SERIES 100, 150, 200, 300, 350 AND 400 SERIES SYSTEMS

The Design-Rail System (DRS) utilizes aluminum extrusions and tempered glass infill 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 all natural environments. The DRS may be used for residential, commercial and industrial applications. The DRS 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 direction perpendicular to top 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 DRS system will meet or exceed all requirements of the 1997 Uniform Building Code, 2000, 2003, 2006 and 2009 International Building Codes, Florida Building Code (non-hurricane zones), 2007 California Building Code and 2005 Aluminum Design Manual. Wood components and anchorage to wood are designed in accordance with the National Design Specification for Wood Construction.

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EXP 03/21/2011

EXP 12/31/2011

EXP 02/28/2011

It is a violation of NY law for any person, unless he is acting under the direction of a licensed professional engineer to alter this item in any way. If an item bearing the seal of an engineer is altered, the altering engineer shall affix to the item his seal and the notation "altered by" followed by his signature and the date of such alteration, and a specific description of the alteration.

EXP 04/30/2012

EXP 12/31/2010

EXP 09/30/2011

FIRM #F-12044
EXP 12/31/2010

Signed 09/09/2010

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TYPICAL INSTALLATIONS:

<u>Surface mounted with base plates:</u>
3/8" mounting hardware depends on substrate refer to calculations for hardware specifics.

Residential Applications:
Rail Height 36" above finish floor.
Standard Post spacing 6' on center maximum.
All top rails

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

Pool Fence/Wind Fence
4' post spacing, 4' post height – 25 psf wind load
4' post spacing, 4' 5.75" post height – 20 psf wind load
4' post spacing, 5' post height – 16 psf wind load

<u>Core pocket /embedded posts or stainless steel stanchion mounted:</u>
Residential Applications:
Rail Height 36" above finish floor.
Standard Post spacing 6' on center maximum, series 100, 150 and 400.
8' on center Series 200, 300, and 350.

Commercial and Industrial Applications:
Rail Height 42" above finish floor.
Standard Post spacing 5' on center maximum, series 100, 150 and 400
6' on center Series 200, 300, and 350.

Pool Fence/Wind Fence
4' post spacing, 4' 9" post height – 25 psf wind load (3/8" glass)
4' post spacing, 5' post height – 23 psf wind load

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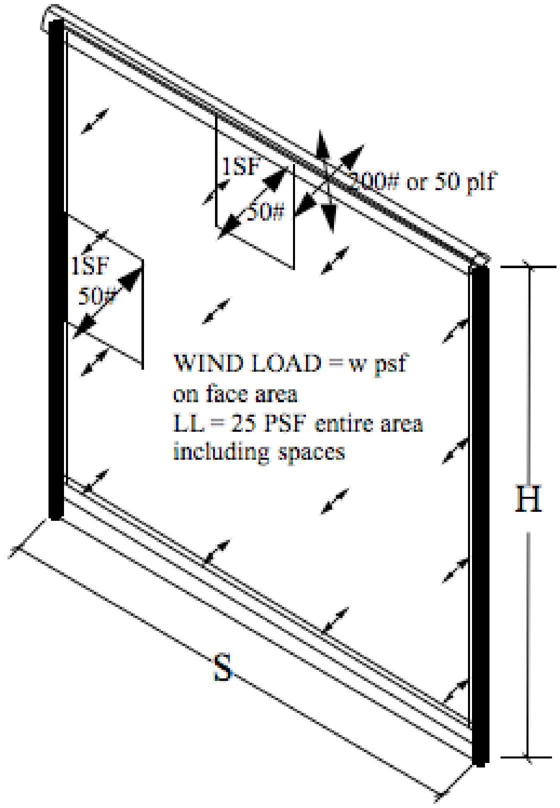
LOAD CASES:

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/2 =
= 0.5w*S*H^2

Solving for w :
w = 2M/(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 = 2*875/(5*3.5^2) = 28.57 psf

Allowable wind load adjustment for other post spacing:

w = 28.57*(5/S)

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WIND LOADING ON A BALCONY RAILS

Calculated in accordance with SEI/ASCE 7-05 Section 6.5.13 Design Wind Loads on Open Buildings and Other Structures. This section is applicable for free standing guardrails, wind walls and balcony railings because they are not part of the building structural frame, are open on all sides and do not receive loading from anything other than the railing surface. Section 6.5.12.4 Components and Cladding is not applicable because the rails are not part of the building envelope but are outside of the building envelope. Section 6.5.12.4.4 Parapets may be applied to any of the exposed rails but the results will be essentially the same as Section 6.5.13 because $GC_{pi} = 0$ for the railings because all sides are open with no internal pressure so the equation simplifies to:

$p = q_p(GC_p) = q_zGC_f$

For guardrails the coefficients have the following values:

$G = 0.85$ from section 6.5.8.2 for a relatively stiff structure.

$C_f = 1.2$ From Figure 6-20.

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

$I = 1.0$

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

$K_d = 0.85$ from Table 6-4.

K_{zt} From Figure 6-4 for the site topography, typically 1.0.

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

28.5 psf wind load is equivalent to the following wind speeds and exposures ($K_{zt} = 1.0$):

85 mph Exp B, 600’ above grade
85 mph Exp C, 350’ above grade
85 mph Exp D, 225’ above grade

95 mph Exp B, 275’ above grade
95 mph Exp C, 120’ above grade
95 mph Exp D, 60’ above grade

105 mph Exp B, 130’ above grade
105 mph Exp C, 45’ above grade
105 mph Exp D, 20’ above grade

115 mph Exp B, 70’ above grade
115 mph Exp C, 20’ above grade

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GLASS STRENGTH - 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 typical Modulus of Rupture for the glass, F_r is ≥ 24 ksi. 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). Other locations the glass stress may be increased by 33% (SF = 3.0) for glass when there is no fall hazard. Glass not used in guardrails may be designed for a safety factor of 2.5 in accordance with ASTM E1300-00.

Values for the modulus of rupture, F_r, modulus of Elasticity, E and shear modulus, G for glass are typically taken as:

F_R = 24,000 psi based on numerous published data from various glass manufacturers. This value is recognized in ASTM E 1300-00, ANSI Z97.1, ASTM C 1048-97b and CPSC 16 CFR 1201 (derivation of the value is required using the provided formulae and properties). This value is referenced in numerous publications, design manuals and manufacturers' literature.

E = 10,400 ksi is used as the standard value for common glass. 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. It can be found in ASTM E 1300 and numerous other sources.

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

$\nu = 0.22$ (Typical value of Poisson's ratio for common glasses).

$\mu = 5x10^{-6}$ in/(in˚F) (Typical thermal coefficient for common glass).

The safety factor of 4 is dictated by the building code (1997 UBC 2406.6, 2006 IBC 2407.1.1). It is applied to the modulus of rupture since glass as an inelastic material does not have a yield point. The safety factor of 4 is applicable to glass stresses. Non-glass elements are designed in accordance with the applicable code sections for the material.

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 1300-00 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.

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:

$I = 12"*(t)^3 /12= (t)^3 \text{ in}^3/\text{ft}$

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

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:

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

Simplifying:

$\Delta = [wL^4/t^3]/(9.58 \times 10^9)$for w in psf and L in inches

For concentrated load:

$\Delta = PL^3/(48EI)$

Simplifying:

$\Delta = PL^3/(4.992*10^8t^3)$

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

$w = [t^3*1.595*10^8]/L^3$

Solving for L

$L = [(t^3*1.595*10^8)/w]^{1/3}$

Solving for t

$t = [L^3w/(1.595*10^8)]^{1/3}$

For Concentrated load

Solving for P

$P = (8.32*10^6t^3)/L^2$

Solving for L

$L = [8.32*10^6*t^3/P]^{1/2}$

Solving for t

$t = [PL^2/(8.32*10^6)]^{1/3}$

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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 = 3.0 (no fall hazard, glass fence or wind screen)
M_{all} = 597”#*4/3 = 796”#

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
$\Delta = [25*36^4/0.25^3]/(9.58 \times 10^9) = 0.28$”
or $\Delta = 50*36^3/(4.992*10^8*0.25^3) = 0.30$”

Maximum width for 50# load at center of 2’ long light:
H = 2*597”#*4/50 = 95.5” (will not limit height for lights over 1’3” wide)

For Pool Fence or Wind Fence (SF = 4.0)
Allowable wind load:
For 48” wide light:
W = 796”#*8/(4’*48”)= 33.17 psf
For 60” wide light:
W = 796”#*8/(5’*60”)= 21.2 psf
Maximum width for 25 psf:
H = $\sqrt{}$(796”#/12*8/25) = 4.607’

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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 = 3.0 (no fall hazard, glass fence or wind screen)
M_{all} = 1,607"#*4/3 = 2,143#"

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
$\Delta = [25*36^4/0.366^3]/(9.58 \times 10^9) = 0.089$"
or $\Delta = 50*36^3/(4.992*10^8*0.366^3) = 0.095$"

Check maximum wind load based on deflection:

36" width w = $[0.366^3*1.595*10^8]/36^3$ = 167 psf (does not control)
42" width w = $[0.366^3*1.595*10^8]/42^3$ = 105 psf (does not control)

For Pool Fence or Wind Fence (SF = 4.0)
Allowable wind load:
For 48" wide light:
W = 1,607"#*8/(4'*48")= 67 psf
For 60" wide light:
W = 1,607"#*8/(5'*60")= 42.9 psf
Maximum width for 25 psf:
H = $\sqrt{}$(1,607"#/12*8/25) = 6.546'

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STANDARD POST – 2-3/8" Square

Post Strength

6005-T5 or 6061-T6

Post

-Area 0.995"

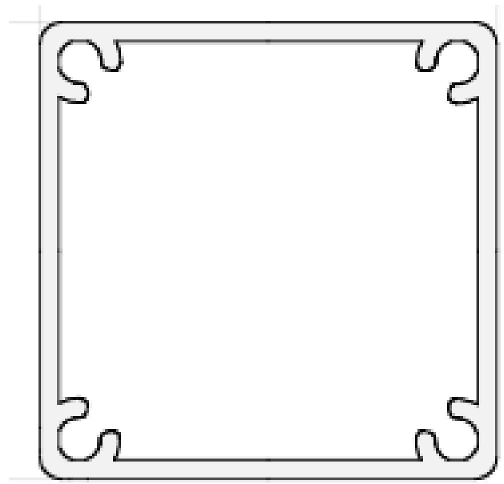
$I_{xx} = I_{yy} = 0.863 \text{ in}^4$

$S = 0.726 \text{ in}^3$

$r = 0.923 \text{ in}$

$J = 0.98 \text{ in}$

$k \leq 1$ for all applications



POST EXTRUSION ACTUAL SIZE

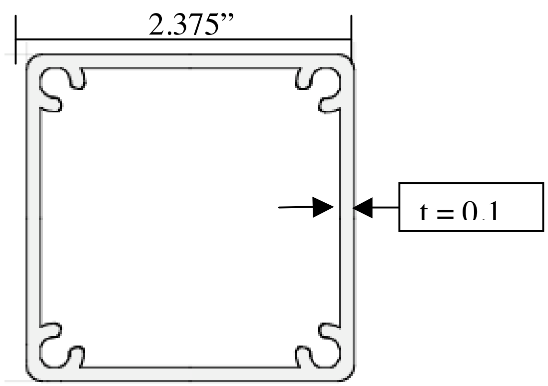
Allowable bending stress ADM Table 2-21

$$S_1 = \frac{L_B \; S_C}{0.5\sqrt{I_y J}} = \frac{L_B \bullet 0.726}{0.5\sqrt{0.863 \bullet 0.98}} = 1.58 \, L_B$$

for $L_B \leq \frac{146}{158} = 92" \rightarrow F_{CB} = 21$ ksi

for $L_B > 92"$ $F_{CB} = 2.39–0.24(1.58\, L_B)^{1/2}$

$M_{all} = 0.726 \bullet 19^{ksi} = 13{,}794^{\#"} = 1{,}149^{\#}ft$



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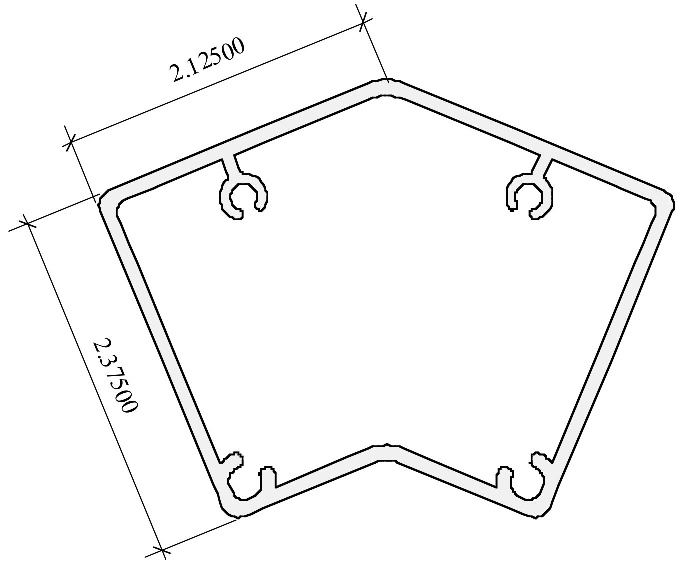
45° Corner Post



6005-T5 or 6061T6

Post Section Properties
Area 1.355”
$I_{xx} = 1.120\ in^4$
$I_{yy} = 1.742\ in^4$
$S_{xx} = 0.812\ in^3$
$S_{yy} = 0.900\ in^3$
$r_{xx} = 0.975$ in
$r_{yy} = 1.175$ in
J = 1.146 in
k = 1 for all applications

Allowable bending stress ADM Table 2-21

$$S_1 = \frac{L_B\ S_C}{0.5\sqrt{I_y J}} = \frac{L_B \bullet 0.900}{0.5\sqrt{1.120 \bullet 1.146}} =$$

$=1.58\ L_B$

for $L_B \leq \frac{146}{1.58} = 92” \rightarrow F_{CB} = 21$ ksi

for $L_B > 92”$ $F_{CB} = 2.39–0.24(1.58\ L_B)^{1/2}$

$M_{all} = 0.812 \bullet 19^{ksi} = 15{,}428\ ^{\#}” = 1{,}286^{\#}ft$

Connection to base plate
Post uses standard base plate

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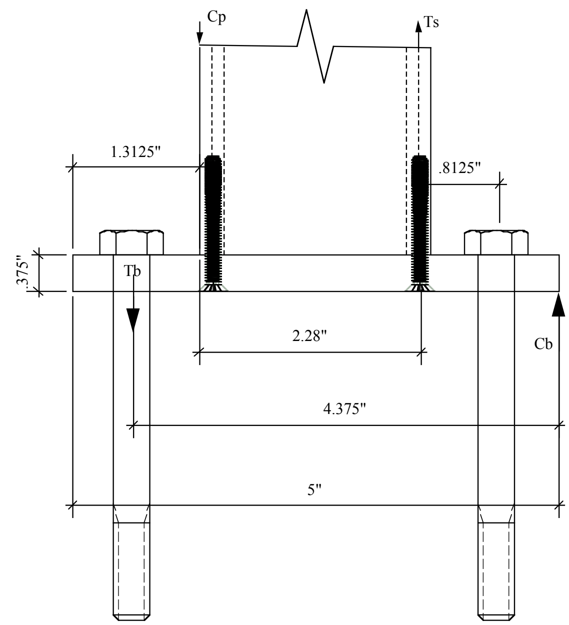
CONNECTION TO BASE PLATE



Failure modes → screw tension
→ screw shear
→ screw withdrawal

For screw withdrawal see ADM 5.4

$W = 2.3 \bullet e \bullet d \bullet \pi \bullet F_{sy}$

e = full thread engagement = 1"
d = max root diameter = 0.248"
minor = 0.185"

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.

F_{sy} = 20 ksi

$W = 2/3 \bullet 1" \bullet 0.248" \bullet \pi \bullet 20^{ksi}$
$W = 10.39^k$
$W' = \dfrac{10.39}{3.0} = 3.46^k$ Safety factor

Major root area

Screw tension → $T_y = 0.0483\ in^2 \bullet 110\ ksi = 5314^{\#}$ Vu = 0.0483* 45ksi =2,174#

average root area

$F_{tU} = 0.0376 \bullet 150\ ksi = 5640^{\#}$
Safety factors for screws calculated from SEI/ASCE 8-02 Section 5 LRFD factors
For yielding SF = 1.6/0.75 = 2.13 → $5,314^{\#}/2.13 = 2,495^{\#}$

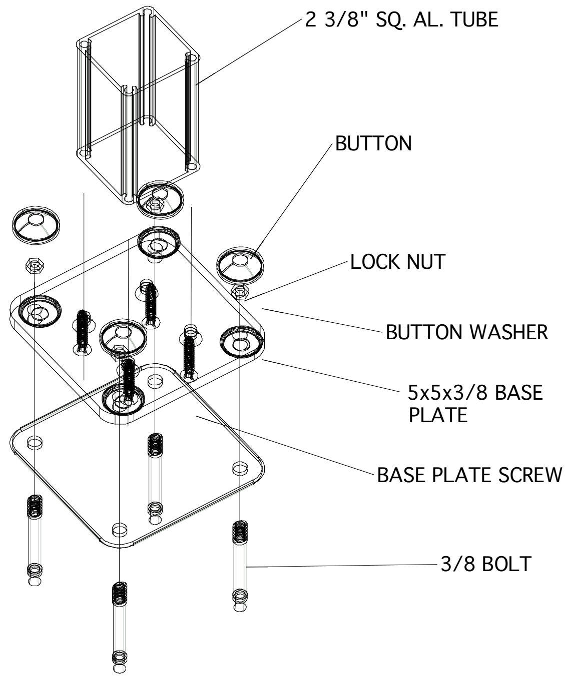
For fracture SF = 1.6/0.65 = 2.46 → $5640/2.46 = 2,293^{\#}$

Shear strength
For fracture SF = 1.6/(0.9*0.75) = 2.37 → $5,640/2.37 = 2,380^{\#}$

BASE PLATE DESIGN

Base plate bending stress

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



Base plate allowable moment

M_{all} = 24 ksi • 0.117 in^3 = 2,812 "#

→ Base plate bending stress

$T_B = C$

M = 0.8125" • T_B • 2

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

Maximum post moment for base plate strength

M_{all} = 2 • 1730 • 4.375" = 15,142#"

Limiting factor = screws to post

M_{ult} = 2 • 5,314# • 2.28" = 24,232#"

M_{all} = 2 • 2,293# • 2.28" = 10,456"#

For factors of safety refer to Aluminum Design Manual Section 5.3.2.1
and SEI/ASCE 8-02 section 5

BASE PLATE ANCHORAGE

3/8" mounting hardware depends on substrate, select appropriate fasteners for the substrate to provide the required strength.

$$T_{Des} = \frac{10{,}456}{2 \cdot 4.375"} = 1{,}195^{\#}$$

adjustment for concrete bearing pressure:

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

$$T'_{Des} = \frac{10{,}456}{2 \cdot (4.375"-0.087/2)} = 1{,}206^{\#}$$

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} = 10,456"#/42" = 250#

For 50 plf live load maximum post spacing is:

S_{max} = 250#/50 plf = 5.0' = 5'0"

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.

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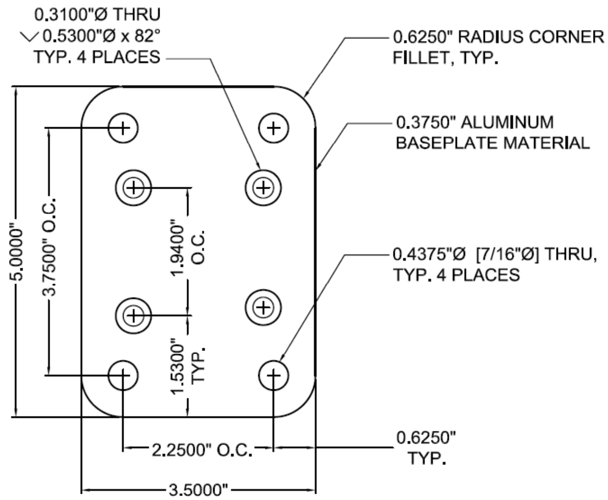
NARROW BASE PLATE

The narrow base plate attaches to the post with the same screws as the standard base plate.



NARROW POST BASEPLATE - TOP ELEVATION
SCALE: 1:2

For long dimension perpendicular to the guard the bolt loads may be assumed as the same as for the standard 5x5 base plate.

For base plate oriented with the long dimension parallel to the guard the design anchor load is:
T = 10,500/(2*2.8”) = 1,875#

When attached to steel with 3/8” bolts the narrow base plate may be oriented in either direction.

When attached to wood with the base plate oriented with the long dimension perpendicular to the guard there is no reduction in load with the lag screw sizes as calculated on page 10.

When attached to wood using lag screws with the base plate oriented with the long dimension parallel to the guard the allowable load per post is multiplied by 0.7.
For example if the base plate is attached with 6” lag screws on a weather exposed deck the maximum post height is reduced to:

H = 0.7*42” = 29.4”

When attached to wood using 3/8” hex bolts with the base plate oriented with the long dimension parallel to the guard the allowable load per post is the same as for the standard base plate provided that a base plate is used under the nuts with washers.

When installed to concrete the anchors shall be custom designed for the imposed loads based on the actual conditions of the proposed installation. The standard concrete anchor design shown herein for the 5x5 base plate may not be used because the anchor spacing is inadequate.

BASE PLATE MOUNTED TO WOOD – SINGLE FAMILY RESIDENCE

36” GUARDS

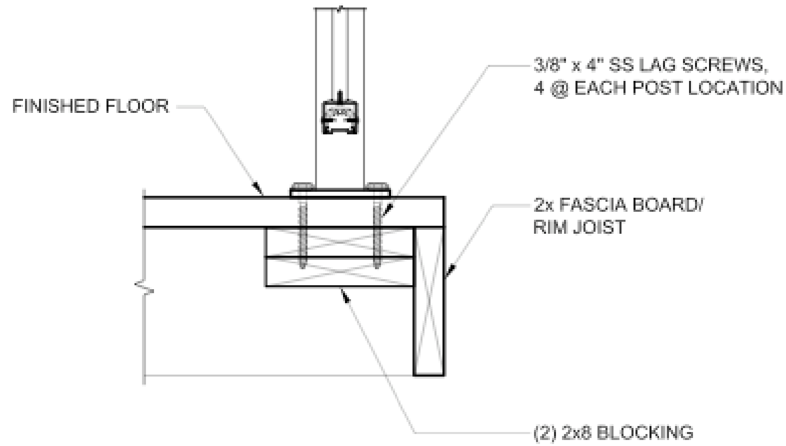
For 200# top load and 36” post height: M = 200#*36” = 7,200”#

$$T_{200} = \frac{7{,}200}{2*4.375''} = 823\#$$

Adjustment for wood bearing:

a = 2*823/(1.33*625psi*5”)= 0.4”

T = 7,200/[2*(4-0.4/2)]= 947#



Required embed depth:

For protected installations the minimum embedment is:

l_e = 947#/323#/in = 2.93” :

+7/32” for tip = 3.15”

For weather exposed installations the minimum embedment is:

l_e = 947#/243#/in = 3.90” :

+7/32” for tip = 4.12”

FOR 36” HIGH WEATHER EXPOSED INSTALLATIONS USE 5” LAG SCREWS AND INCREASE BLOCKING TO 4.5” MINIMUM THICKNESS.

42” HIGH GUARDS

For 200# top load and 42” post height: M = 200#*42” = 8,400”#

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

Adjustment for wood bearing:

a = 2*960/(1.33*625psi*5”)= 0.462”

T = 8,400/[2*(4-0.462/2)]= 1,114#

Required embed depth:

For protected installations the minimum embedment is:

l_e = 1,114#/323#/in = 3.45” : +7/32” for tip = 3.67” **4.5” minimum lag length**.

For weather exposed installations the minimum embedment is:

l_e = 1,114#/243#/in = 4.58” : +7/32” for tip = 4.80”

FOR 42” HIGH WEATHER EXPOSED INSTALLATIONS USE 6” LAG SCREWS AND INCREASE BLOCKING TO 5.5” MINIMUM THICKNESS.

3/8” Stainless steel bolts with heavy washers bearing on the wood may be used through the solid wood blocking with a minimum 3” nominal thickness.

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BASE PLATE MOUNTED TO CONCRETE - Expansion Bolt Alternative:

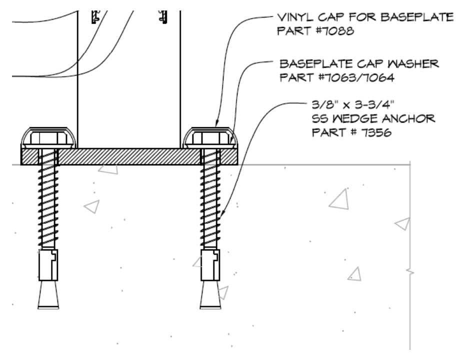
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8”x3.75” concrete anchors with 3” effective embedment. Anchor strength based on ESR-2251

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

$C_{a,min} = 1.5”$ (ESR-2251 Table 3)

$C_{ac} = 5.25”$ (ESR-2251 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 ($c_{a,min} \leq c_{ac}$)

$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$

$N_{cb} = 86.06/81*1.0*1.0*0.857*6{,}830 = 6{,}219 \leq 2*3{,}469$

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

$\varphi_{h,V}= \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} =1.225$

$V_b= [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(3.0/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{2500}(3.0)^{1.5} =1{,}688\#$

$V_{cb} = 43.03/40.5*1.0*1.4*1.225*1{,}688\# = 3{,}076\#$

Steel shear strength = 4,825#*2 = 9,650

Allowable shear strength

$ØV_N/1.6 = 0.70*3{,}076\#/1.6 = 1{,}346\#$

Shear load = 4.87*50/1,346 = 0.18 ≤ 0.2

Therefore interaction of shear and tension will not reduce allowable tension load:

$M_a = 2{,}526\#*4.375” = 11{,}053\# > 10{,}653\#$

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

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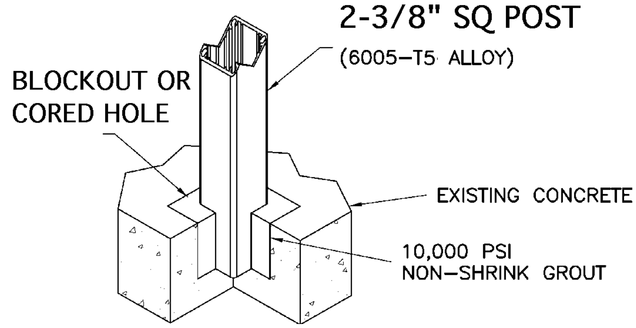
CORE MOUNTED POSTS

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



Max load – 6’•50 plf = 300#

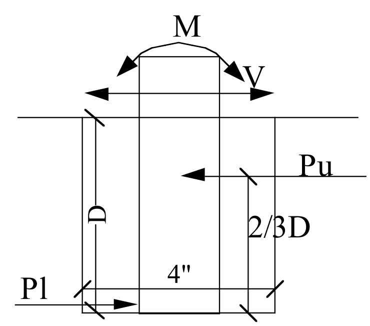
M = 300#•42” = 12,600”#

Check grout reactions

From $\Sigma M_{PL} = 0$

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

$$f_{Bmax} = \frac{5093\text{\#}\cdot 2}{2\text{”}\cdot 2.375\text{”}} \cdot 1/0.85 = 2{,}523 \text{ psi post to grout}$$



f_{Bconc} = 2523 • 2”/4” = 1,262 psi grout to concrete

Minimum required grout strength:
f'_c = 1.6*2,523/0.75 = 5,400 psi

Core mount okay for 6’ post spacing

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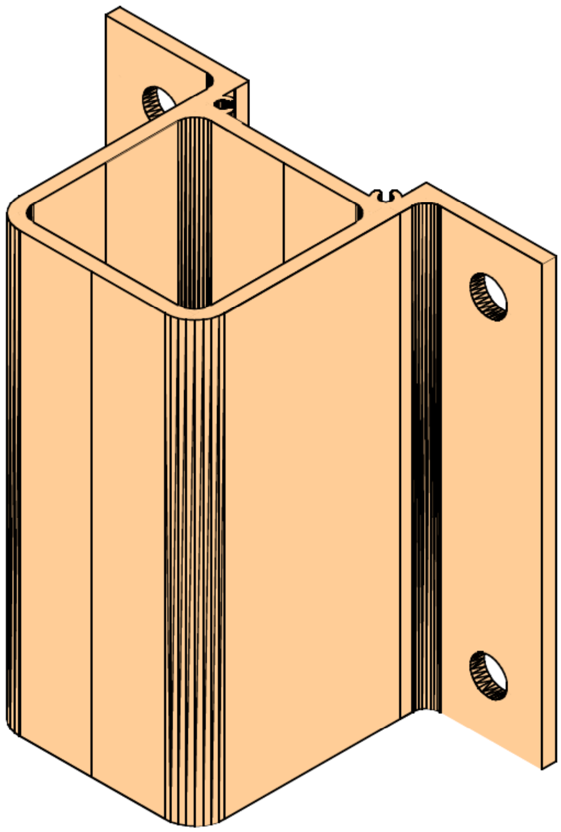
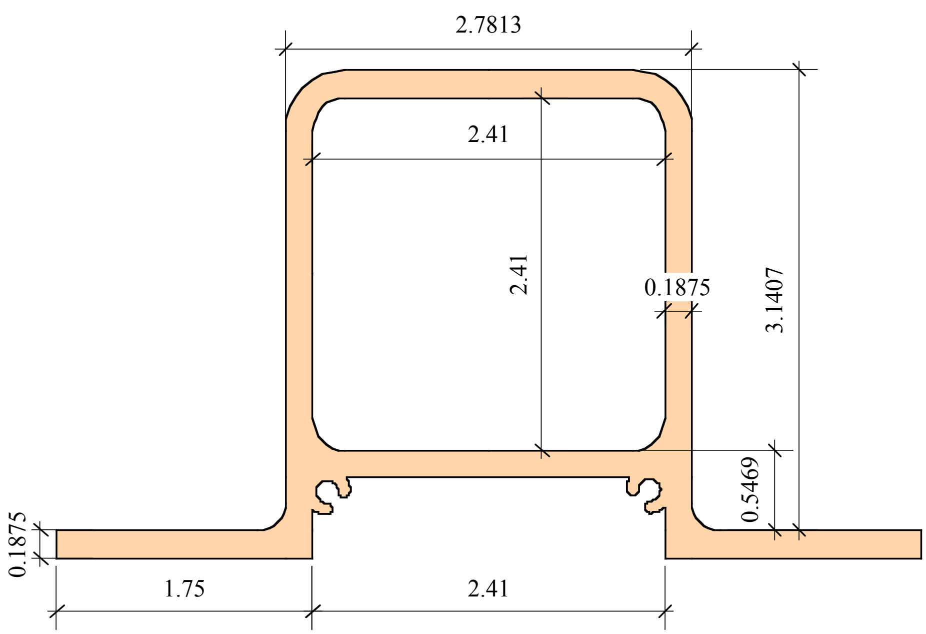
FASCIA BRACKET



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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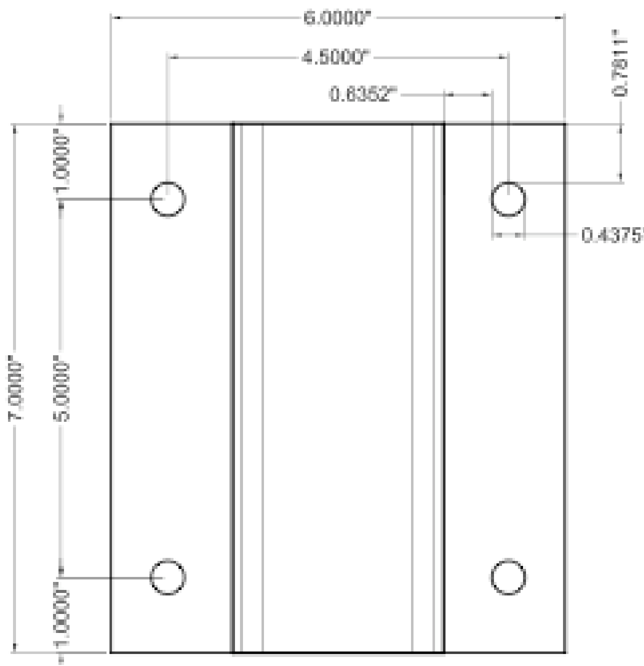
Allowable moment on bracket:

$Ma = F_t*S$

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

Ma_{yy} = 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
Ma_f = 0.0278 in^3*20 ksi = 557"#

Allowable bolt tension
T = Ma_f/0.375 = 1,485#
3/8" bolt standard washer

For Heavy washer
T=Ma_f/0.1875= 2,971#

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

T_{up} = [250#*(42"+ 9")/6"]/2 bolts = 1,062# tension

T_{bot} = [250#(42"+2")/6"]/2 bolts = 917# tension

For lag screws into beam face:
- 3/8" lag screw – withdrawal strength per NDS Table 11.2A

Wood species – G ≥ 0.43 – W = 243#/in

Adjustments – C_d = 1.33, C_m = 0.75 (where weather exposed)

No other adjustments required.

W' = 243#/in*1.33 = 323 #/in – where protected from weather

W' = 243#/in*1.33*0.75 = 243#/in – where weather exposed

For protected installations the minimum embedment is:

l_e = 1,062#/323#/in = 3.29" : +7/32" for tip = 3.50"

For weather exposed installations the minimum embedment is:

l_e = 1,062#/243#/in = 4.37" : +7/32" for tip = 4.59"

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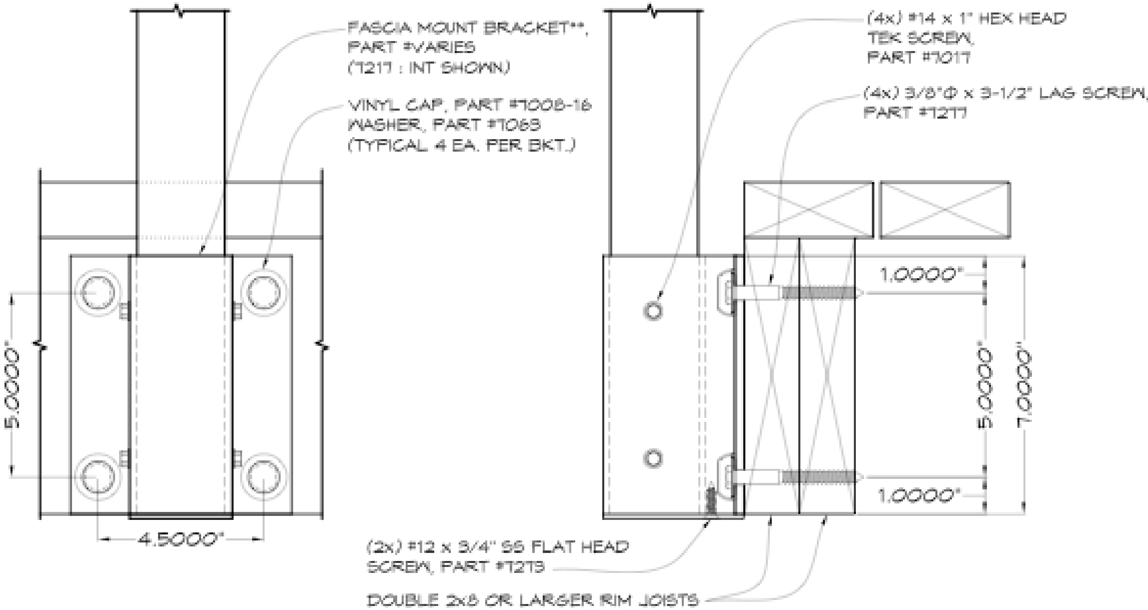
Fascia Brackets- Single Family Residence installations to wood deck:



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

T_{up} = [200#*(36"+ 9")/6"]/2 bolts = 750# tension

T_{bot} = [200#(36"+3")/6"]/2 bolts = 650# tension

For protected installations the minimum embedment is:

l_e = 750#/323#/in = 2.32" : +7/32" for tip = 2.54"

For weather exposed installations the minimum embedment is:

l_e = 750#/243#/in = 3.09" : +7/32" for tip = 3.31"

Requires 3-1/2" minimum wood thickness (4x)

4" lag screws are acceptable for installation on residential decks with 36" rail height. 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"+ 9")/6"]/2 bolts = 850# tension

T_{bot} = [200#(42"+3")/6"]/2 bolts = 750# tension

For protected installations the minimum embedment is:

l_e = 850#/323#/in = 2.63" : +7/32" for tip = 2.85" Requires 3.5" lag screw

For weather exposed installations the minimum embedment is:

l_e = 850#/243#/in = 3.50" : +7/32" for tip = 3.72" Requires 4" lag screw

Requires 4-1/2" minimum wood thickness (triple 2x)

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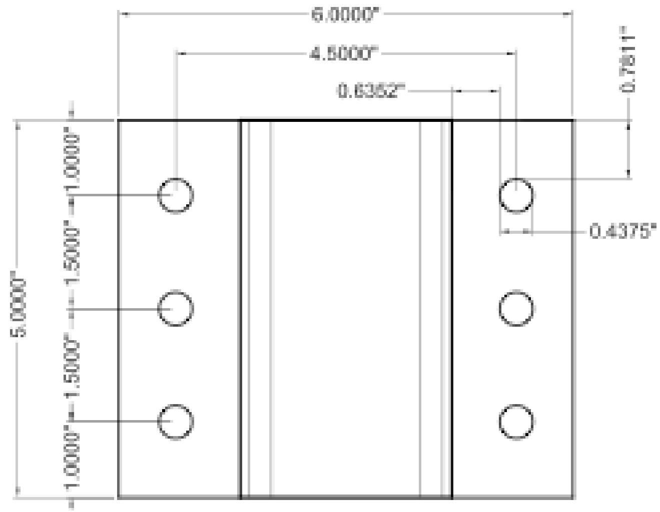
6 BOLT ALTERNATIVE:
5" bracket length



Anchor tension may be calculated from $\sum M$ about the end of the bracket with anchor load proportional to distance from the edge of bracket.

$\sum M = M_g - 4*T*2+2.5^2/4.5*T*2$
$+ 1^2/4.5*T*2$
$M_g = 11.22T$
$T = M_g/11.22$

Typical Installation – Post load = 250# at 42" AFF – Top hole is 3" below finish floor
T_{up} = [250#*(42"+ 7")]/11.22 = 1,092# tension
T_{bot} = [250#(42"+2")]/11.22 = 980# tension

For lag screws into beam face:
- 3/8" lag screw – withdrawal strength per NDS Table 11.2A
Wood species – $G \geq 0.43$ – W = 243#/in
Adjustments – $C_d = 1.33$, $C_m = 0.75$ (where weather exposed)
No other adjustments required.
W' = 243#/in*1.33 = 323 #/in – where protected from weather
W' = 243#/in*1.33*0.75 = 243#/in – where weather exposed
For protected installations the minimum embedment is:
l_e = 1,092#/323#/in = 3.38" : +7/32" for tip = 3.60"
For weather exposed installations the minimum embedment is:
l_e = 1,092#/243#/in = 4.49" : +7/32" for tip = 4.71"

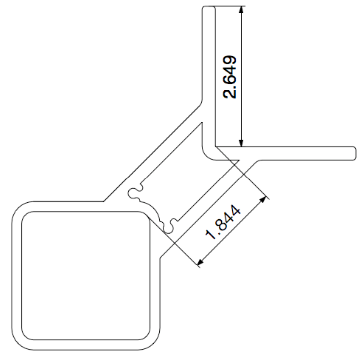
For residential installations:
36" ht: T_{bot} = [200#(36"+7")]/11.22 = 766# tension
For weather exposed installations the minimum embedment is:
l_e = 766#/243#/in = 3.15" : +7/32" for tip = 3.37"

42" ht: T_{bot} = [200#(42"+7")]/11.22 = 873# tension
For weather exposed installations the minimum embedment is:
l_e = 873#/243#/in = 3.59" : +7/32" for tip = 3.81"

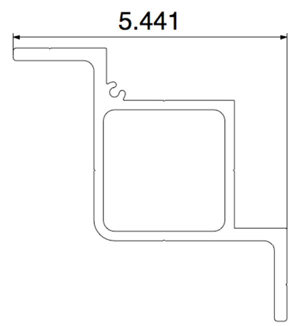
For centerline holes only (edge of concrete slab):
T = [250#*(42"+ 7")/2.5"]/2 bolts = 2,450# tension
Design anchors for 2,450# allowable tension load (Halfen anchor inbeds or similar)

Corner Conditions Fascia Brackets:

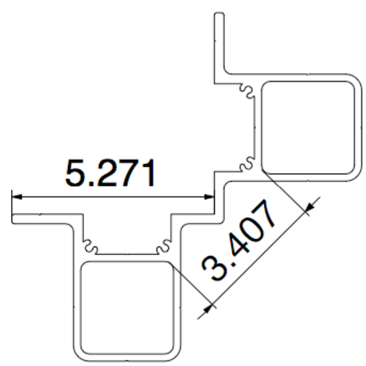
Single Outside Corner
Used at an outside corner for a single post, uses 4 anchors with 2 anchors in shear and 2 in tension based on direction of loading. Bracket strength will be similar to the standard fascia bracket for the same attachment method. May have top rail mitered corner with top rail extending two perpendicular directions or single top rail in one direction.



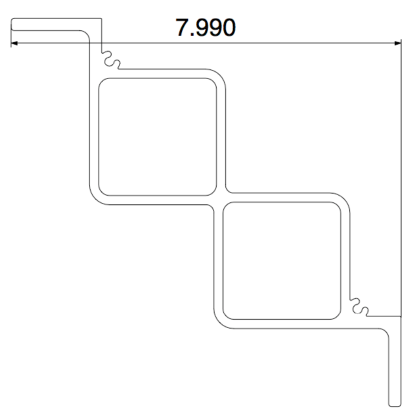
Single Inside Corner
Used at an inside corner for a single post, uses 4 anchors with 2 anchors in shear and 2 in tension based on direction of loading. Bracket strength will be similar to the standard fascia bracket for the same attachment method. May have top rail mitered corner with top rail extending two perpendicular directions or single top rail in one direction.



Double Outside Corner
Used at an Outside corner for two posts – top rail may intersect at corner or terminate at post or before the corner intersection. Uses 4 anchors with 2 anchors in shear and 2 in tension based on direction of loading. Bracket strength will be similar to the standard fascia bracket for the same attachment method.



Double Inside Corner
Used at an inside corner for two posts – top rail may intersect at corner or terminate at post or before the corner intersection. Uses 4 anchors with 2 anchors in shear and 2 in tension based on direction of loading. Bracket strength will be similar to the standard fascia bracket for the same attachment method.



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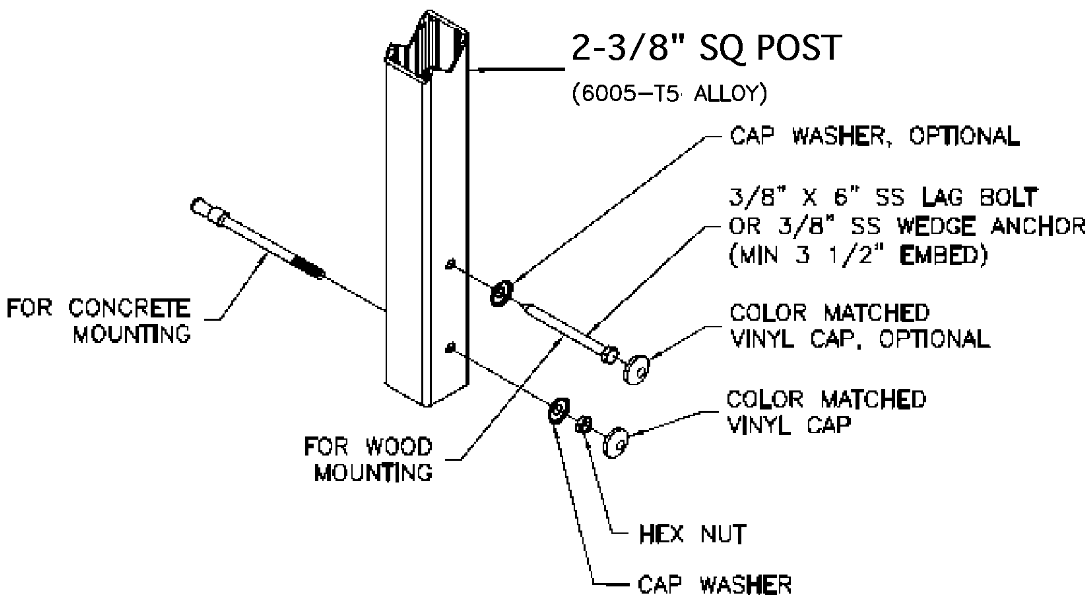
FASCIA MOUNTED POST
Commercial application – Load = 200# or 50 plf any direction on top rail



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 = 1.33 for guardrail live loads, = 1.6 for 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#/"*1.33*1.0 = 405#/in

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

T_b = 405*2.4" = 972#

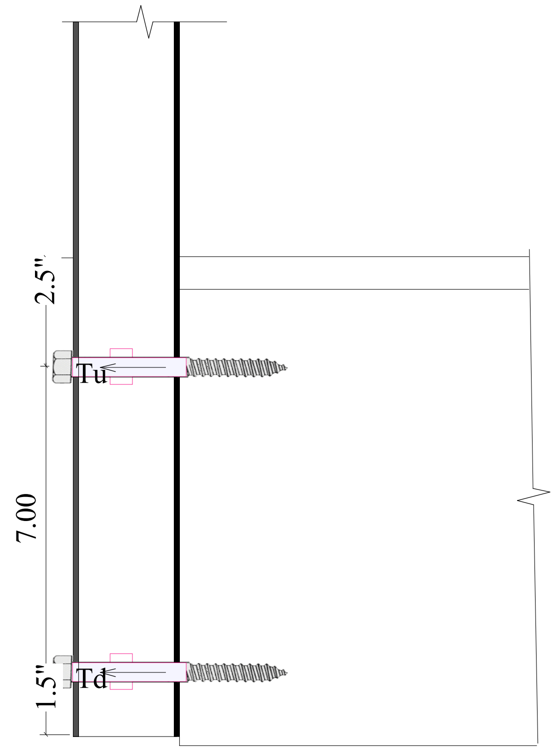
Z_{ll} = 220# per lag, (horizontal load) NDS Table 11K

Z'_{ll} = 220#*1.33*1.0 = 295#

Z_T = 140# per lag, (vertical load)

Z_T = 140#*1.33*1.0 = 187#

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



From $\sum$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 are four lags, two each way. Two lags will act in withdrawal and two will be in shear: Okay be inference from running posts.

For attachment to concrete – ITW Red Head Trubolt wedge anchor 3/8"x3.75" concrete anchors with 3" effective embedment, T_a = 1,263# (see page 10 for calculation). M_a = 1,263#*8.76" = 11,064"#

For attachment to steel – 3/8" bolts will develop full post strength.

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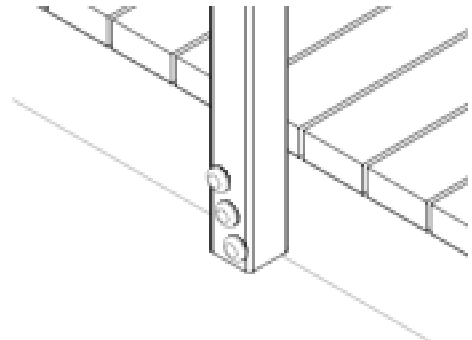
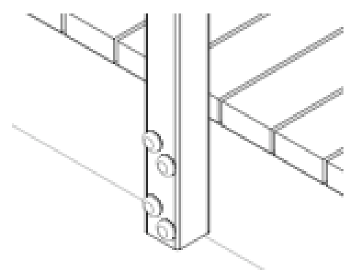
ALTERNATIVE FASCIA ATTACHMENT CONFIGURATIONS:



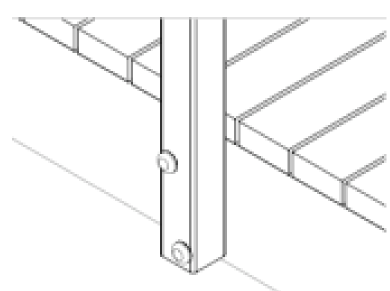
To 6x wood fascia:
3 Bolt pattern – 1" from top and bottom and at center:
$\sum M = M_g - 4.5*T + 2.75^2/4.5*T + 1^2/4.5*T$
$M_g = 6.4T$
$T = M_g/6.4$
For 36" residential guard:
T = (36"+7")*200#/6.4 = 1,344#
Exceeds 3/8" lag screw capacity Requires use of thru-bolts/carriage bolts.
For 42" residential guard:
T = (42"+7")*200#/6.4 = 1,531#
Exceeds 3/8" lag screw capacity Requires use of thru-bolts/carriage bolts.

Moment capacity of carriage bolts: T_a = 2,200#
M_a = 2,200#*6.4" = 14,080"# - develops full post strength.

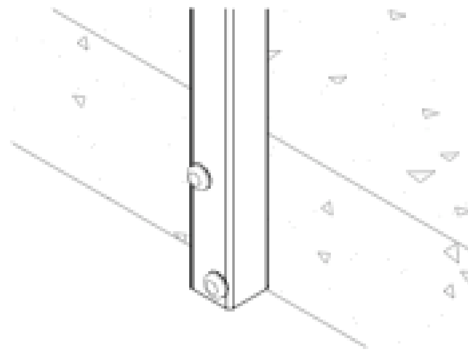


To 8x wood fascia
For (4) 3/8" lag screw pattern
Lag screws at 1" and 1.75" from top and bottom:
$\sum M = M_g - 6.5*T + 5.75^2/6.5*T$
$M_g = 11.59T$
$T = M_g/11.59$
For 36" residential guard:
T = (36"+9")*200#/11.59 = 777#
For weather exposed installations the minimum embedment is:
l_e = 777#/243#/in = 3.20" : +7/32" for tip = 3.42"

For 42" residential guard:
T = (42"+9")*200#/11.5 = 887#
For weather exposed installations the minimum embedment is:
l_e = 887#/243#/in = 3.65" : +7/32" for tip = 3.87"



For (2) 3/8" carriage bolt alternative:
Moment capacity of carriage bolts: T_a = 2,200#
M_a = 2,200#*6" = 13,200"# - develops full post strength.



To 8” nominal slab edge (7.5”).
ITW Red Head Trubolt wedge anchor 3/8”x3.75” concrete anchors with 3” effective embedment.
Anchor strength based on ESR-2251
Minimum conditions used for the calculations:
$f'_c \geq 3,000$ psi
edge distance =2.5” spacing = 2.5”
h = 3.0”: embed depth
For concrete breakout strength:
$A_{Ncg}= (1.5*3*2)*7.5 = 67.5\ in^2$ 2 anchors
$A_{Nco}= 9*3^2 = 81\ in^2$
$C_{a,min} = 1.5$” (ESR-2251 Table 3)
$C_{ac} = 5.25$” (ESR-2251 Table 3)
$\varphi_{ed,N} = 1.0$
$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}= 0.7+0.3*[2.5/(1.5*3)] = 0.87$
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6,830\#$
$N_{cb} = 69.5/81*1.0*1.0*0.87*6,830 = 5,098 \leq 2*3,469$
based on concrete breakout strength.
Determine allowable moment load on anchor group
$T_s = 0.65*5,098\#/1.6*5” = 11,391”\#$
Develops the full post strength.

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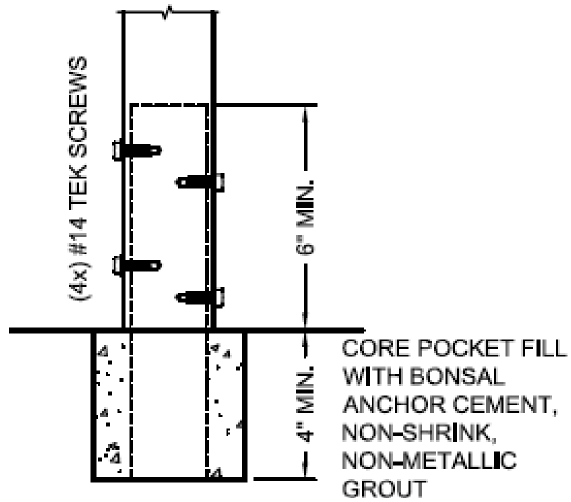
STANCHION MOUNT
2"x1-1/2"x 1/8" 304 1/4 Hard Stainless steel tube

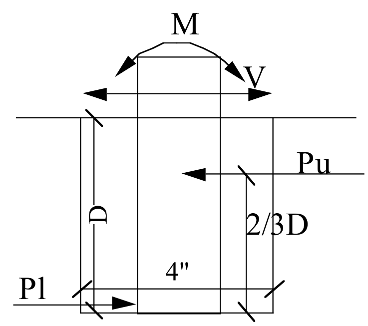


Stanchion Strength
F_{yc} = 50 ksi
Z_{yy} = 0.543 in^3
Reserve strength method from SEI ASCE 8-02 section 3.3.1.1 procedure II.
where $d_c/t = (2*2/3)/0.125 = 10.67 < \lambda_1$
$\lambda_1 = 1.1/\sqrt{(F_{yc}/E_o)} = 1.1/\sqrt{(50/28*10^3)} = 26$
$M_n = 0.543\ in^3 * 50\ ksi = 27{,}148\#"$
$M_s = øM_n/1.6 = 0.9*27{,}148/1.6 = 15{,}270\#"$
Equivalent post top load
42" post height
V = 15,270"#/42" = 363#
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 $\sum$M about base of stanchion = 0

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

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

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

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

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

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

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

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"

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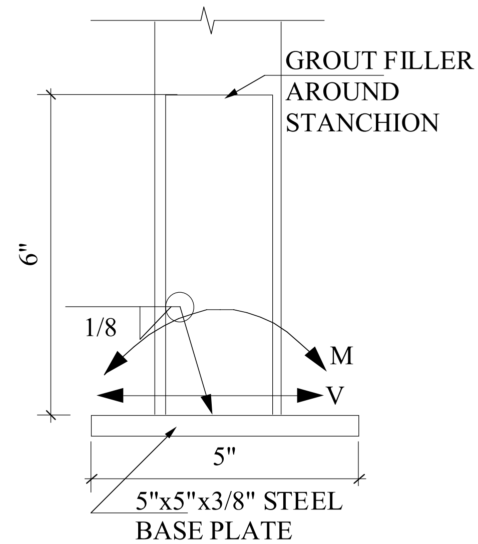
STANCHION WELDED TO BASE PLATE:
Stanchion is welded all around to base plate with 1/8" minimum throat fillet weld capable of developing the full stanchion bending strength.

2"x1-1/2"x 1/8" A304 1/4 hard Stainless steel tube

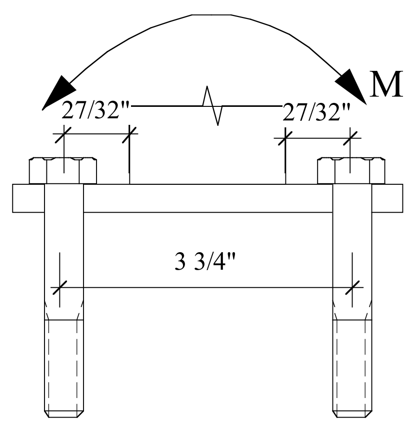


Stanchion Strength
$Z_{yy} = 0.362\ in^3$
$F_y = 75$ ksi SEI/ASCE 8-02 Table A1
$M_n = 0.362\ in^3 * 75\ ksi = 27{,}150\#"$
Service moment
$M_s = ø\ M_n /1.6$
$M_s = 0.9*27{,}150"\# /1.6 = 15{,}272\#"$
For Tension zone
Check compression zone $F_c = 50$ ksi:
$n = Es/Ea = 27/10.1 = 2.67$
$S_{eff} = 0.362+(.125"*2.375"*1.187"/2.67)$
$S_{eff} = 0.494$
$M_s = 0.9*0.494*50ksi/1.6 = 13{,}895"\#$

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
$P_n = 15{,}928$
$Ps = 15{,}928/1.2 = 13{,}273\#"$

Base plate bending stress
for 3/8" plate $S = \frac{5" \bullet 3/8^2}{6} = 0.117\ in^3$



Base plate allowable moment
$F_b = 0.75*50ksi = 37.5$ ksi
$M_{all} = 37.5\ ksi \bullet 0.117\ in^3 = 4{,}387$ "#
→ Base plate bending stress
$T_B = C$
$M = 0.84375" \bullet T_B \bullet 2$

$T_{all} = \frac{4.387}{2 \bullet 0.84375} = 2{,}600^{\#}$

Base plate anchorage is the same as previously calculated for the surface mounted post option for the specific substrate.

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POOL FENCE/WIND FENCE OPTION

The Design-Rail may be used to construct pool fences or wind walls.

Maximum allowable height for 48” on center post spacing:

For any of the detailed anchorage to wood or surface mounted to any substrate or direct fascia mounted (two bolts):
M_a = 9,600”#
Live load is 50 plf at 42” above finish floor or 200# at 42” above finish floor.

For 25 psf loading:
Maximum post height for 4’ o.c. post spacing:
$H_a = \sqrt{}(2*800'\#)/(25psf*4')) = 4'$
Required post spacing for 25 psf loading and 5’ post height:
$H_a = (2*800'\#)/(25psf*5^2)) = 2.56' = 2'\ 7''$

For 20 psf loading:
Maximum post height for 4’ o.c. post spacing:
$H_a = \sqrt{}(2*800'\#)/(20psf*4')) = 4.47'$
Required post spacing for 25 psf loading and 5’ post height:
$H_a = (2*800'\#)/(20psf*5^2)) = 3.2' = 2'\ 7''$

Maximum allowable wind load for 5’ post height and 4’ on center post spacing:
$W = 800'\#/(4'*5'^2/2) = 16$ psf
85 mph exposure C, $K_{zt} = 1.0$

For core mounted posts or steel stanchion mounted to concrete or steel or fascia mounted with fascia bracket:
25 psf wind loading:
Maximum post height for 4’ o.c. post spacing:
$H_a = \sqrt{}(2*1{,}150'\#)/(25psf*4')) = 4.78'\ \ 4'\ 9''$
Required post spacing for 25 psf loading and 5’ post height:
$H_a = (2*1{,}150'\#)/(25psf*5^2)) = 3.68' = 3'\ 8''$

Maximum allowable wind load for 5’ post height and 4’ on center post spacing:
$W = 1{,}150'\#/(4'*5'^2/2) = 23$ psf
105 mph exposure C, $K_{zt} = 1.0$

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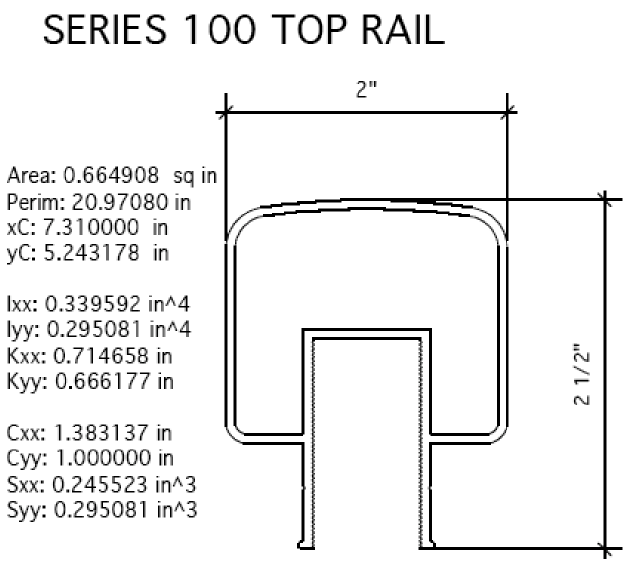
SERIES 100 TOP RAIL

Butts into post
Alloy 6063 – T6 Aluminum
Allowable Stress: ADM Table 2-24



F_T = 15 ksi

F_C → 6' span

$\frac{2 \text{ Lb SC}}{(\text{IyJ})} = \frac{2 \bullet 72'' \bullet 0.246}{(0.295 * 1.53)^{1/2}}$
= 52.7<130 therefore
Fc = 15 ksi

Allowable Moments ➜
Horiz.= $0.295 in^3 \cdot 15$ ksi = 4,425#" = 368.75 #'
Vertical load = $0.246 in^3 \cdot 15$ ksi = 3,690#" = 307.5 #'

Maximum allowable load for 72" o.c. post spacing - vertical
W = 3,690"#*8/(69.625"^{2}) = 6.09 pli = 73.1 plf
P = 3,690"#*4/69.625" = 212#

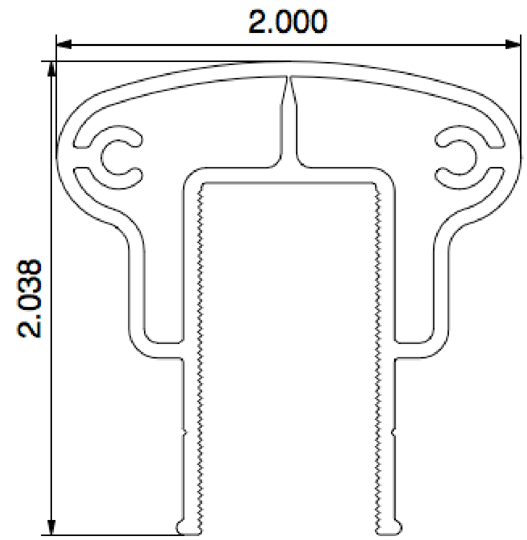
Maximum span without load sharing, P = 200# - vertical
S = 3,690"#*4/200# = 73.8" clear
Max post spacing =73.8"+2.375" = 76.175"

For horizontal loading rail strength is greater and therefore okay by inference.

Maximum allowable load for 72" length horizontal load
W = 4,425"#*8/72^2 = 6.8 pli = 81.9 plf
P = 4,425"#*4/72" = 245.8#

Maximum span for P = 200# and W = 50 plf horizontal load
W = √(368.75#'*8/50) = 7.68' = 7' 8.5"
P = 368.75#'*4/200 = 7.375' = 7'3.5" controls

SERIES 150 TOP RAIL



$A = 0.676\ in^2$
$I_{xx} = 0.1970\ in^4$
$I_{yy} = 0.2263\ in^4$
$S_{xx} = 0.1522\ in^3$
$S_{yy} = 0.2263\ in^3$
$r_{xx} = 0.540$ in
$r_{yy} = 00579$ in
Alloy 6063 – T6 Aluminum
Allowable Stress: ADM Table 2-24
$F_T = 18$ ksi

$F_C \rightarrow$ 6' span
$R_b/t = 0.3/0.065 = 4.6 < 35$
$F_c = 18$ ksi for horizontal loads
d/t = 0.75"/0.65 = 1.15 < 15
$F_c = 20$ ksi for vertical loads

Allowable Moments ➔

Horiz.= 0.2263in³ ▪18 ksi = 4,073"# = 339.45'#
Vertical load = 0.1522in³ ▪18 ksi = 2,739.6"# = 228.3#'

Maximum allowable load for 72" o.c. post spacing - vertical

W = 2,739.6"#*8/(69.625"²) = 4.52 pli = 54 plf
P = 2,793.6"#*4/69.625" = 160.5#

Maximum span without load sharing, P = 200# - vertical

S = 2,793.6"#*4/200# = 55.87" clear
Max post spacing =55.87"+2.375" = 58 1/4"

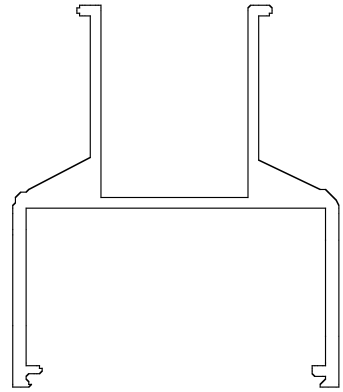
With loading sharing with bottom rail – load transferred by glass 200# concentrated load may be safely supported with 6' on center post spacing.

Maximum allowable load for 72" length horizontal load

W = 4,073"#*8/69.625² = 6.7 pli = 80.6 plf
P = 4,073"#*4/69.625" = 234#

Maximum post spacing is 6'.

SERIES 100 – UNIVERSAL BOTTOM RAIL



Rail Properties:
6063-T6 Aluminum
I_{xx} = 0.102 in4, S_{xx} = 0.101 in3
I_{yy} = 0.164 in4, S_{yy} = 0.193 in3
r_{xx} = 0.476”, r_{yy} = 0.603”

For 72” on center posts; L = 72”-2.375”-1”x2 = 67.625” ; L_b = 1/2L = 33.81”
Lb/ry = 33.81”/0.603 = 56 From ADM Table 2-24
Fbc = 16.7-0.073· 56 = 12.6 ksi

Allowable Moments → Horiz.= 0.193in³ ·12.6 ksi =2,432”#
Maximum allowable load for 72” o.c. post spacing

W = 2,432”#*8/(67.625”²) = 4.25 pli = 51 plf

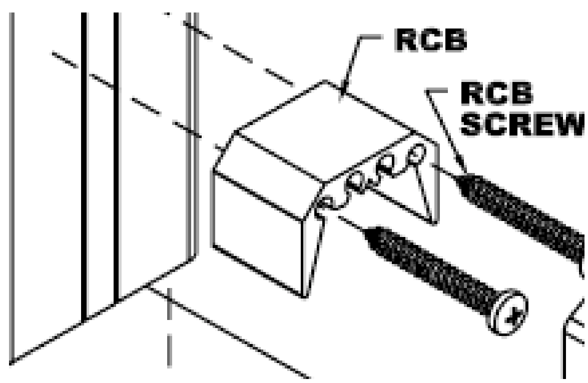
P = 2,432”#*4/67.625” = 144#

Max span for 50 plf load = (8*2,432/(50/12))1/2 = 68.33” clear span

Max span for 200# concentrated load:

L = 4*2,432”#/200# = 48.6”

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 end
#8 Tek screw to 6063-T6

$$V = 2 \cdot 38 \text{ ksi} \cdot 0.1309” \cdot 0.07” \cdot \frac{1}{3\ (FS)} = 232\#$$

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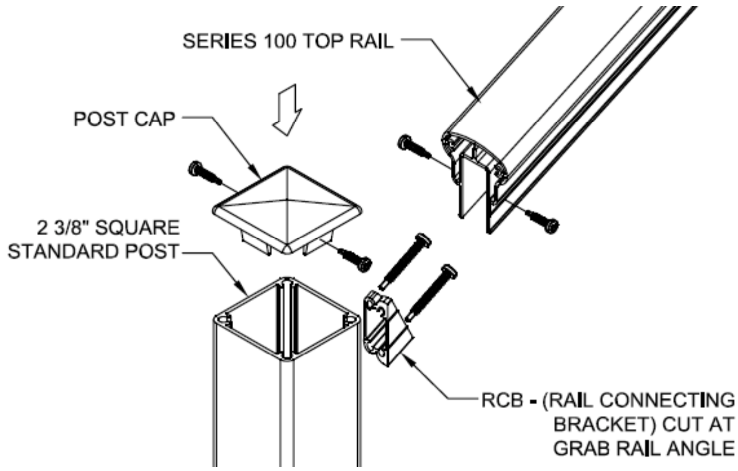
SERIES 100/150 TOP RAIL CONNECTION TO POST FACE:
Use RCB attached to post with 2 #10 screws same as bottom rail.

$$V = 2 \cdot 38\text{ ksi} \cdot 0.19" \cdot 0.10" \cdot \frac{1}{3\text{ (FS)}} = 481\text{\#/screw}$$

Since minimum of 2 screws used for each
Allowable load =
2 · 481# = 962#



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.

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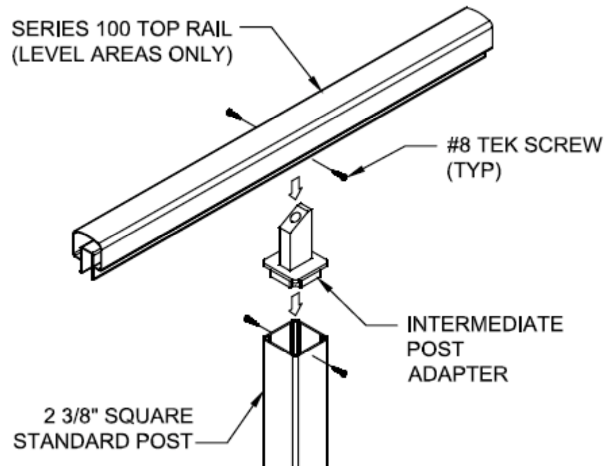
INTERMEDIATE POST FITTING – SERIES 100/150

Used for intermediate posts along stairways
Fitting locks into top of post with #8 Tek screws:



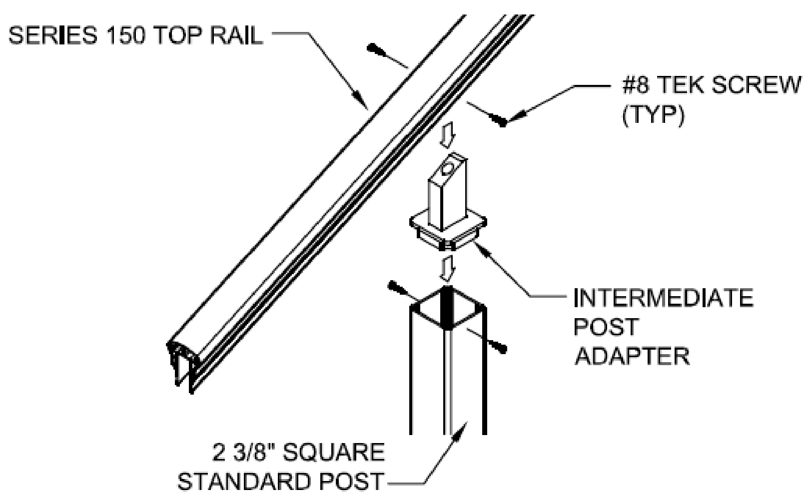
Maximum load on fitting is 300#
6' post spacing * 50 plf = 300#

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#"

#8 Tek screws:
Shear strength = V= $2 \cdot 38$ ksi $\cdot 0.1309" \cdot 0.07" \cdot \frac{1}{3\ (FS)}$ = 232#

Moment capacity
M= 232#*2.375" = 551#"



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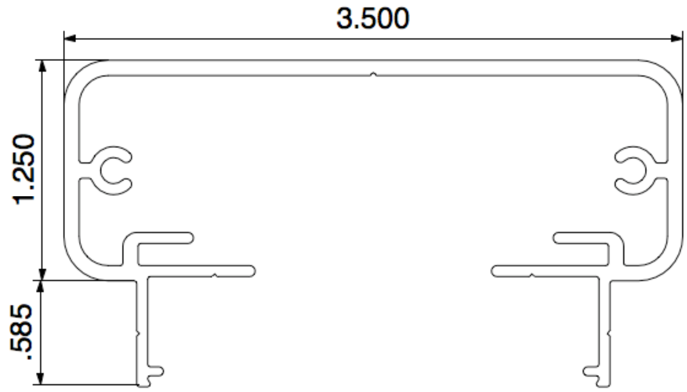
SERIES 200 TOP RAIL

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}: 0.457 in^3 top
S_{yy}: 0.874 in^3



6063-T6 Aluminum alloy from ADM Table 2-24
For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625" ; kL_b = 1/2L = 33.81"
$F_{bc} = 16.7 - 0.073 \cdot \dfrac{33.81}{1.313} = 14.82$ ksi
Ft = 15 ksi
Allowable Moments ➔ Horiz.= 0.874in^3 ·14.82 ksi = 12,953#" = 1,079#'
Vertical load = 0.457in^3 ·14.82 ksi = 6,773#" top compression
or = 0.213in^3 ·15 ksi = 3,195#" controls vertical- bottom tension

Maximum allowable load for 72" o.c. post spacing - vertical
W = 3,195"#*8/(67.625"^{2}) = 5.59 pli = 67 plf
P = 3,195"#*4/67.625" = 189# Load sharing with bottom rail required for 6 foot post spacing. Glass infill will transfer loads from top rail to bottom rail and provide required additional support.
With load sharing maximum span is 6'.

Maximum span without load sharing, P = 200#
S = 3,195#"*4/200# = 63.9" clear
Max post spacing =63.9"+2.375" = 66-1/4", 5' 6-1/4"

For horizontal load, maximum span for 50 plf load
L = (8Ma/50plf)$^{1/2}$ = (8*1,079/50plf)$^{1/2}$ = 13.14'
for 200# concentrated load
L = (4M/200#) = (4*1,079/200plf)= 21.58'
deflection limits will limit span to 6'.

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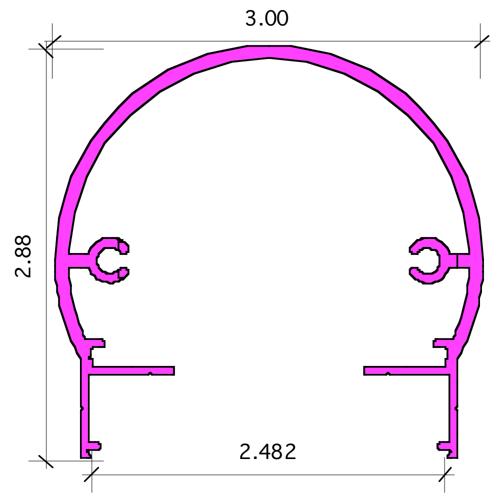
SERIES 300 TOP RAIL



Area: 0.881 sq in
Perim: 21.29 in
Ixx: 0.603 in^4
Iyy: 1.149 in^4
Kxx: 0.828 in
Kyy: 1.142 in
Cxx: 1.599 in
Cyy: 1.501 in
Sxx: 0.377 in^3
Syy: 0.766 in^3

Allowable stresses from ADM Table 2-21

$F_{Cb} \rightarrow L/r_y = \dfrac{(72 - 2\,3/8" - 2.1")}{1.142} = 59.1$

Based on 72" max post spacing
$F_{Cb} = 23.9 - 0.124(59.1) = 16.57$ ksi
$M_{all\ horiz} = 16.57^{ksi} \bullet (0.766) = 12{,}694"^{\#}$
Vertical loads shared with bottom rail
For vertical load → bottom in tension top comp.

$F_b = 19$ ksi

$M_{all\ vert} = (0.377in^4) \bullet 19\ ksi = 7.163"^{\#}$

Allowable loads

Horizontal → uniform → $W = \dfrac{12{,}694 \bullet 8}{72^2} = 19.6\ \#/in = W = 235$ plf

$P_H = \dfrac{4 \bullet 12{,}694}{72} = 705\ \#$

Vertical → $W = \dfrac{7{,}163 \bullet 8}{72^2} = 11.05\ \#/in = 132.6$ plf (Top rail alone)

$P = \dfrac{7{,}163 \bullet 4}{72} = 398\ \#$

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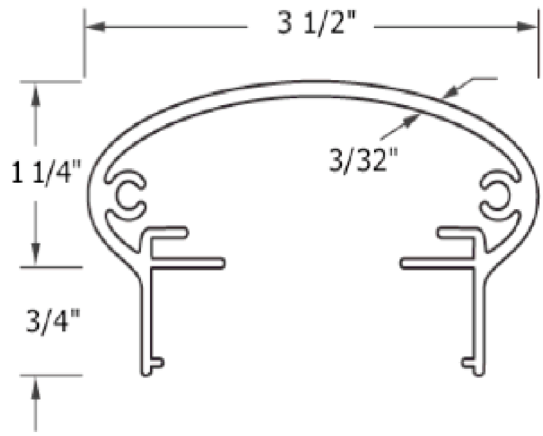
SERIES 350 TOP RAIL

Area: 0.887 in^2
I_{xx}: 0.243 in^4
I_{yy}: 1.463 in^4
r_{xx}: 0.522 in
r_{yy}: 1.281 in
C_{xx}: 1.157 in
C_{yy}: 1.750 in
S_{xx}: 0.210 in^3 bottom
S_{xx}: 0.288 in^3 top
S_{yy}: 0.836 in^3



Allowable stresses ADM Table 2-24 6063-T6 Aluminum

$F_{Cb} \rightarrow Rb/t = \frac{1.875"}{0.09375} = 10$ line 16.1

Based on 72" max post spacing
$F_{Cb} = 18.5 - 0.593(20)^{1/2} = 15.85$ ksi
$M_{all\ horiz} = 15.85^{ksi} \bullet (0.836) = 13{,}249"^{\#}$

Vertical loads shared with bottom rail
For vertical load → bottom in tension top comp.
$F_{bc} = 18$ ksi and $F_{bc} = 15.85$ ksi
For top rail acting alone
$M_{all\ vert} = (0.210in^3) \bullet 18\ ksi = 3{,}780"^{\#}$ Controls
$=(0.288in^4)* 15.85ksi = 4{,}565"^{\#}$

Allowable loads For 6' post spacing:
Horizontal → uniform → $W_H = \frac{13{,}249 \bullet 8}{72^2} = 20.44\ \#/in = W_H = 245$ plf
$P_H = \frac{4 \bullet 13{,}249}{72} = 736\#$

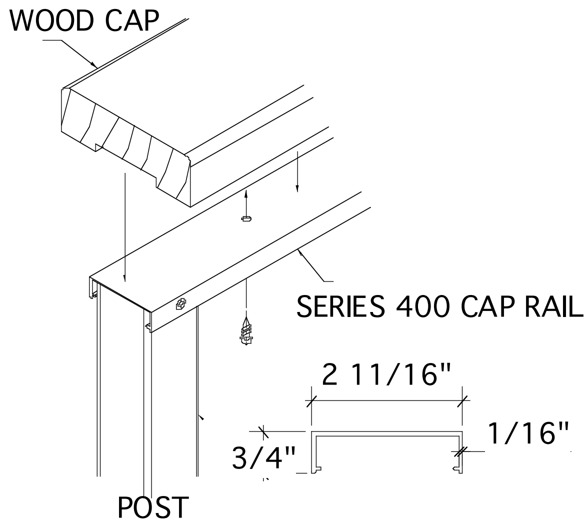
Vertical → $W = \frac{3{,}780 \bullet 8}{72^2} = 5.83\ \#/in = 70$ plf (Top rail alone)
$P = \frac{3{,}780 \bullet 4}{72} = 210\#$

SERIES 400 TOP RAIL
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 – varies G≥ 0.43
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

Allowable Stress for aluminum: ADM Table 2-24
F_T = 15 ksi
$F_C \rightarrow$ 6’ span



Rail is braced by wood At 16” o.c. and legs have stiffeners therefore
F_c = 15 ksi

For wood use allowable stress from NDS Table 4A for lowest strength wood that may be used: F_b = 725 psi (mixed maple #1), CD =1.33, CF = 1.5
F'_b = 725*1.33*1.5 = 1,445 psi
F'_b = 725*1.33*1.5*1.1 = 1,590 psi for flat use (vertical loading)
Composite action between aluminum and wood:
n = Ea/Ew = 10.1/1.1 = 9.18
The limiting stress on the aluminum = 9.18*1,445 psi = 13,267 psi < 15 ksi

Allowable Moments ➔
Horiz.= 0.197in^3 ▪13267 psi +3.063 in^3*1445psi = 7040”#
Vertical load = 0.024in^3 ▪13267 ksi +1.313*1,590= 2,405”#

Maximum allowable load for 72” o.c. post spacing - Horizontal load
W = 7,040”#*8/(69.625”^{2}) = 11.6 pli = 139 plf
P = 7,040”#*4/69.625” = 404#
Maximum span without load sharing, P = 200# or 50 lf - Vertical load
S = 2,405”#*4/200# = 48.1” clear
Max post spacing =48.1”+2.375” = 50.475”

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

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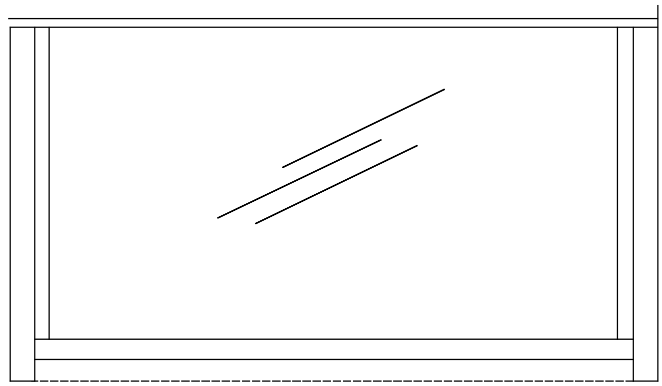
TOP RAIL VERTICAL LOAD – LOAD SHARING

For spans requiring load sharing with bottom rail or glass panel.



Load applied to top rail will be transferred by glass lite to bottom rail and carried by the glass:

Glass stiffness:
$I = .25"*36"^3/12 = 972\ in^4$
If all load carried by the glass:
for 6' span:
$M_w = 50\ plf*6^2/8 = 225'\#$ or
$M_p = 200\#*6'/4 = 300'\#$

$f_b = 300'\#*12/(972/18") = 67\ psi$

Therefore glass will provide adequate vertical stiffness to prevent top rail from deflecting measurably from vertical loads.

Bottom rail will support glass and transfer loads to post through the bottom rail to post connection. The moment in the bottom rail:
$M = (200\#/2)*3" = 300"\#$
Moment in bottom rail will not be a consideration.
Shear strength of the connection = 464# (see bottom rail calculations).

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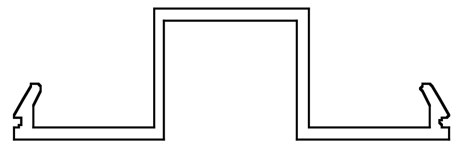
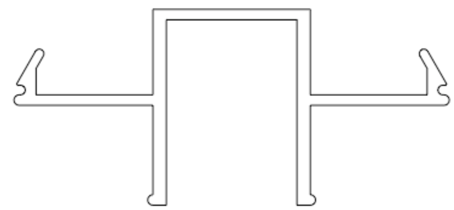
GLASS INFILL INSERT:



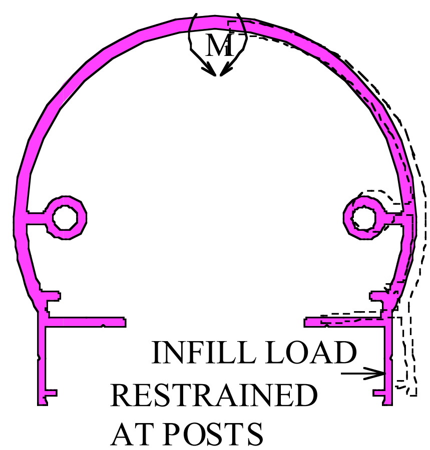
Series 200, 300, 350 and 400 top rails
Either infill option may be used as strength is equivalent for each style.



Insert channel for glass

$I_{yy} = 0.156$ in^4 $\quad$ $I_{xx} = 0.023$ in^4
$S_{yy} = 0.125$ in^3 $\quad$ $S_{xx} = 0.049$ 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
$M = 2.08\text{"}*W$
$M_{all} = S*F_b$
F_b = 20 ksi for flat element bending in own plane, ADM Table 2-24
$S = 12\text{"/ft}*(0.094)^2/6 = 0.0177$ in^3
$W_{all} = M_{all}/2.08\text{"} = (S*F_b)/2.08\text{"} = (0.0177$ 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.

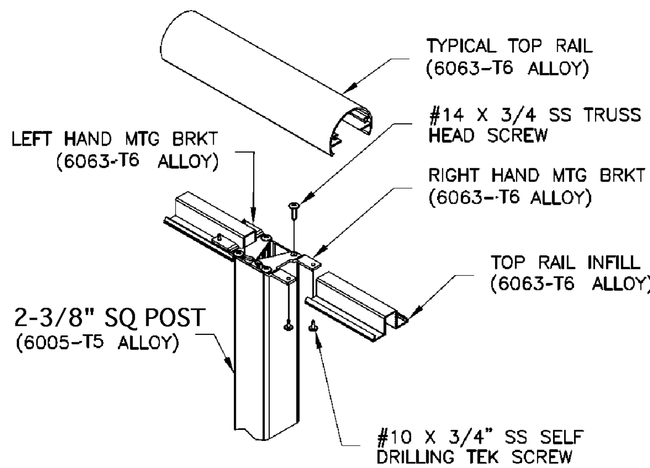
Check deflection:
$\Delta = WL^3/(3EI)$
$I = 12\text{"}*0.09375^3/12 = .000824$ in4
$\Delta = 170\text{plf}*2.08\text{"}^3/(3*10.1\text{x}10^6*.000824) = 0.06\text{"}$

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

Maximum horizontal load on glass infill is 94 plf

TOP RAIL TO POST CONNECTION:
Series 200, 300, 350 and 400 top rails.
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)

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



For Vertical upward 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.

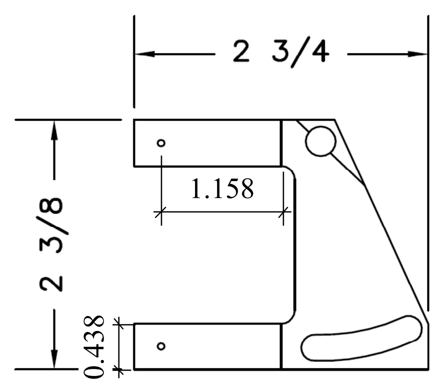
Tek screw strength: Check shear @ rail (6063-T6)
2x F_{urail}x dia screw x Rail thickness x SF

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

Since minimum of 2 screws used for each
Allowable load = 2 $\cdot$ 325# = 650#

Post bearing strength

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



LHB

Bracket tab bending strength
Vertical uplift force
For 6061T6 aluminum stamping 1/8" thick
$F_b = 28\text{ ksi}$ – ADM Table 2-21
$S = 0.438" * (.125)^3/12 = 0.00007\text{ in}^3$
$M_a = 28\text{ ksi} * 0.00007 = 196"\#$
$P_a = M_a/l = 196"\#/1.158" = 169\#$
Uplift limited by bracket strength:

$Up_{all} = 2 * 169 = 338\#$ 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:

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

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

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

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

$V = 38\text{ ksi} \cdot 0.1379" \cdot 0.098" \cdot \frac{1}{3\text{ (FS)}} = 171\#/\text{screw}$; for two screws = 342#

Post to splice plate:

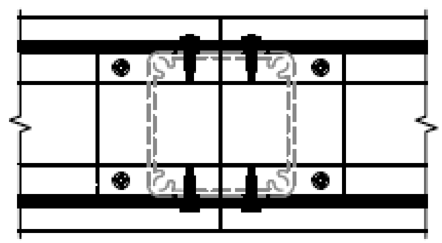
Screws into post screw chase so screw to post connection will not control.

splice plate screw shear strength

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

$V = 2 \cdot 38\text{ ksi} \cdot 0.1379" \cdot 0.098" \cdot \frac{1}{3\text{ (FS)}} = 326\#/\text{screw}$;

for two screws = 652#



BUTT SPLICE

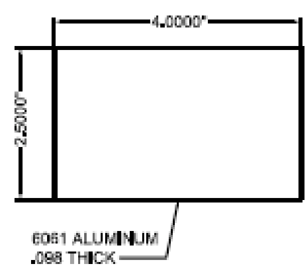
Check moment from horizontal load:

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

M = 0.75*200 = 150#"

$S = 0.098*(2.5)^2/6 = 0.6125\text{ in}^3$

$f_b = 150\#"/(0.6125) = 245\text{ psi}$



STANDARD SPLICE PLATE

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

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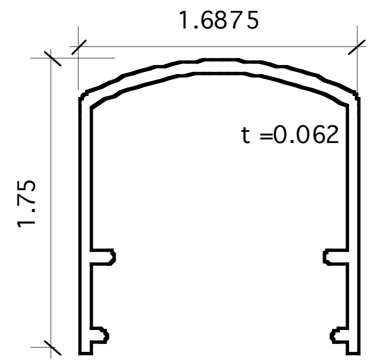
INTERMEDIATE RAIL



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

Allowable stresses: ADM Table 2-24 6063-T6 Aluminum
$F_t = 19$ ksi For vertical loads
$F_{Cb} \rightarrow Rb/t = \dfrac{1.25"}{3.75} = 0.33$ line 16.1 $F_{Cb} = 18$ ksi
$M_{all\ vert} = 18^{ksi} \bullet (0.115) = 2{,}070"^{\#}$

For horizontal loads:
$F_t = 15$ ksi For vertical loads
$F_{Cb} \rightarrow Lb/ry = \dfrac{35"}{0.695} = 50.4$ line 11
Based on 72" max post spacing
$F_{Cb} = (16.7\text{-}0.073*50.4)$ ksi = 13.0 ksi
$M_{all\ horiz} = 13^{ksi} \bullet (0.209) = 2{,}717"^{\#}$

For intermediate rail acting alone
Allowable loads

Horizontal → uniform → $W_H = \dfrac{2{,}717 \bullet 8}{70^2} = 4.44\ \#/in = W_H = 53$ plf

$P_H = \dfrac{4 \bullet 2{,}717}{70} = 155\ \#$ Not used for top rail 50# conc load appl.

Vertical → $W = \dfrac{2070 \bullet 8}{70^2} = 3.38\ \#/in = 40.6$ plf (Top rail alone)

$P = \dfrac{2070 \bullet 4}{70} = 118\#$ Not used for top rail 50# conc load appl.

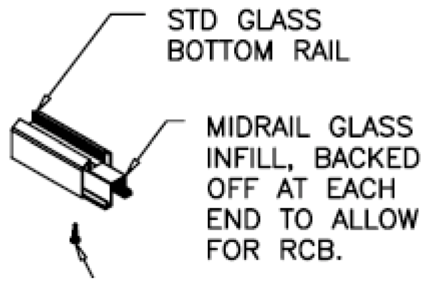
Maximum wind load for 3'6" lite height, 1'9" tributary width
$W_{max} = 53/1.75 = 30.3$ plf

Maximum span for 200# concentrated load:
L = 2,717*4/200# = 54"

May only be used as a top rail for single family residences with a maximum post spacing of 4' 6".

WIND SCREEN MID RAIL

Used with Intermediate Rail, Glass Bottom Rail or Standard Bottom Rail to install glass infill light below the rail.



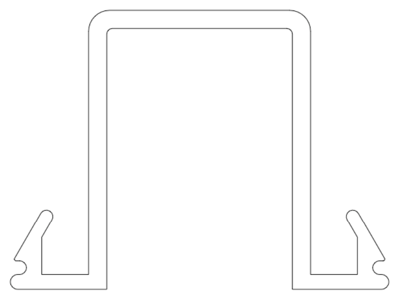
Refer to mid or bottom rail calculations for rail properties.

Mid rail 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 2-24 section 16.

b/t = 1.1”/0.07 = 15.7 < 23

Therefore F_{ca} = 15 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 ksi

When inserted into intermediate rail or bottom rail determine the effective strength: ratio of load carried by infill:

I_{yy} infill/ I_{yy} rail = 0.0378/0.172 = 0.22

S_{yy} infill ≤ 0.22*0.204 = .045 < 0.049

Allowable Moments → Horiz.= (0.204in^3 +0.049) *15 ksi = 3,795”#

Maximum allowable load for 70” screen width L = 70”-1”*2-2.375*2 = 63.25”

W = 3,795”#*8/(63.25”^{2}) = 7.59 pli = 91 plf

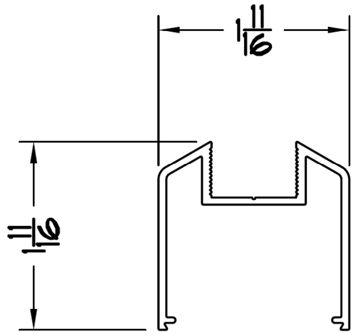
P = 3,795”#*4/63.25” = 240#

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

W = 3,795”#*8/(53.25”^{2}) = 10.7 pli = 128.5 plf

P = 3,795”#*4/53.25” = 285#

GLASS INFILL BOTTOM RAIL

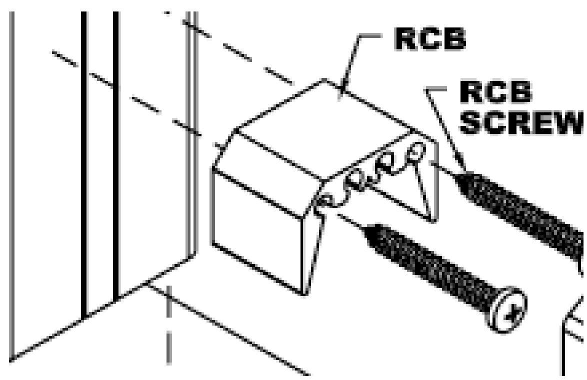


Area: 0.3923 sq in
Perim: 11.648 in

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

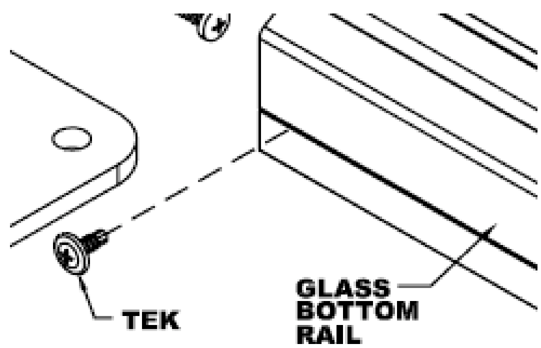
For 72" on center posts; L = 72"-2.375"-1"x2 = 67.625" ; L_b = 1/2L = 33.81"
L_b/r_y = 33.81"/0.662 = 51.07 From ADM Table 2-24:
F_{bc} = 16.7-0.073· 51.07 = 12.97 ksi

Allowable Moments → Horiz.= 0.204in^3 ·12.97 ksi =2,646"#
Maximum allowable load for 72" o.c. post spacing
W = 2,646"#*8/(67.625"^{2}) = 4.63 pli = 55.5 plf
P = 2,646"#*4/67.625" = 156.5#
Max span for 50 plf load =
(8*2646/(50/12))1/2 = 71.28" 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= 2·38 ksi ·0.1697" · 0.10" · $\frac{1}{3\text{ (FS)}}$ =
V = 430#/screw
Since minimum of 2 screws used for each
Allowable load = 2· 430# = 860#

Rail Connection to RCB



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

2*30ksi·0.1309"·0.07"· $\frac{1}{3}$ = 232#/screw
Allowable tension = 2*232 = 464#

OK

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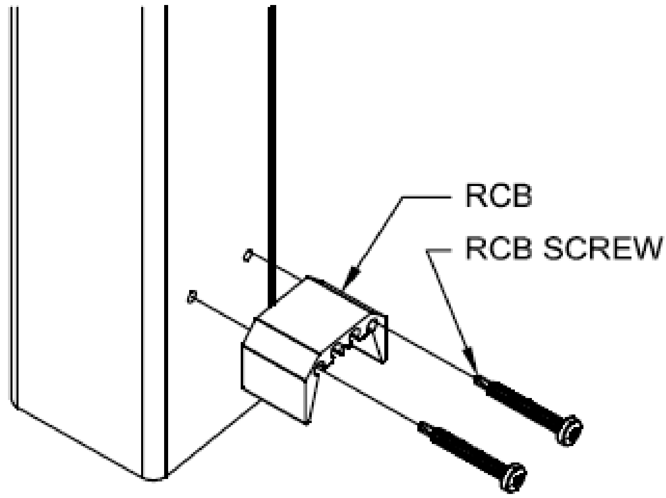
STANDARD POST RAIL CONNECTION BLOCK

Can be used to connect top rails to 2-3/8" standard post face, wood posts, walls or other end butt connection.

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

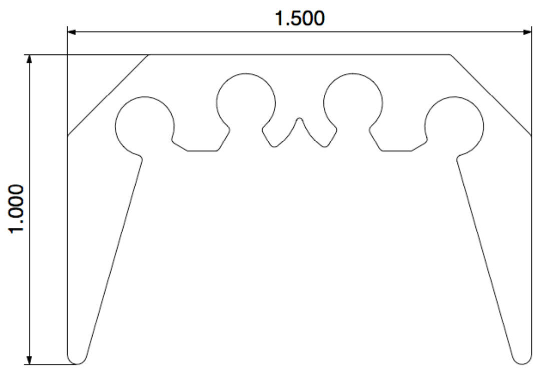


Connection strength to post or wall:
(2) #10× 1.5" 55 PHP SMS Screw

Check shear @ post (6005-T5)
F_{upost} x dia screw x Post thickness x SF
Eq 5.4.3-2
$V = 38 \text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\,(FS)} =$
V = 240#/screw for standard post

Since minimum of 2 screws used for each, Allowable load = 2· 240# = 480#
For attachment to wood posts: Use Four #10 x2.5" screws
Z_{II} = 139# per screw (NDS Table 11M, G ≥ 0.43)
V_a = 4*139# = 556#

Standard RCB



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WALL MOUNT END CAPS

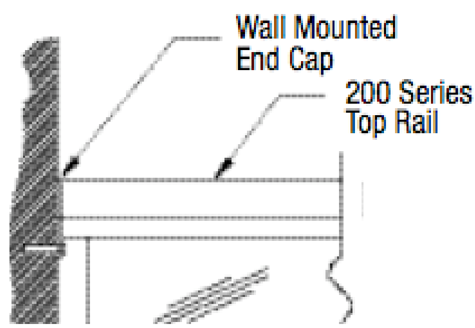
End cap is fastened to the top rail with
2) #10× 1" 55 PHP SMS Screws

2x F_{upost}x dia screw x Cap thickness x SF
Eq 5.4.3-2

$$V = 2*38\text{ ksi} \cdot 0.19" \cdot 0.15" \cdot \frac{1}{3\text{ (FS)}} =$$

722#/screw , 1,422# 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 ADM Table 11M

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 continued

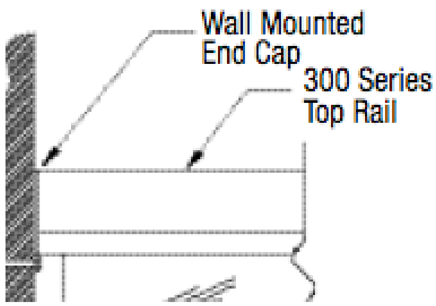
For connection to masonry or concrete use two 3/16 screw-in (Tapcon) anchor

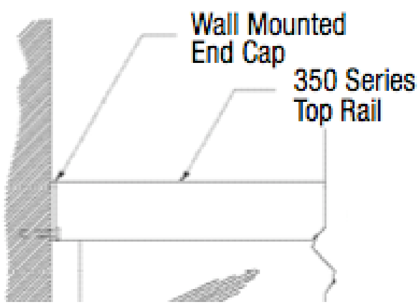
Page 3 of 3 ER-5878

TABLE 3—ALLOWABLE TENSION AND SHEAR VALUES FOR TAPPER SCREW ANCHORS INSTALLED IN NORMAL-WEIGHT CONCRETE[1,2]

SCREW ANCHOR DIAMETER (inch)	SCREW ANCHOR MATERIAL AND COATING (AS APPLICABLE)	MINIMUM EMBEDMENT[3] (inches)	ALLOWABLE TENSION (pounds)						ALLOWABLE SHEAR[6] (pounds)
			With Special Inspection[4]			Without Special Inspection[5]			
			Concrete Strength, f'_c (psi)			Concrete Strength, f'_c (psi)			
			2000	3000	4000	2000	3000	4000	
$^{3}/_{16}$	Carbon steel, Perma-Seal coated	1	90	90	90	45	45	45	175
		$1^{1}/_{2}$	180	215	255	90	110	130	230
		$1^{3}/_{4}$	295	335	375	150	170	190	235

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





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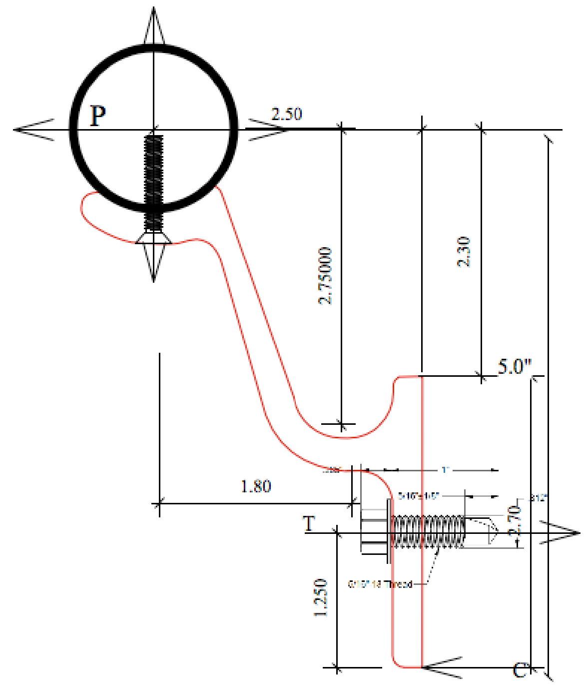
GRAB RAIL BRACKET
Loading 200 lb concentrated load or
50 plf distributed load



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 – F_{tu}Dt/3 = 30ksi*0.164"*0.125"/2.34*2 screws = 525# (ADM 5.4.3)
Tension – 1.2DtF_{ty}/3 = 1.2*.164"*0.125"*25ksi*2 screws/2.34 = 525#
Safety Factor = 2.34 for guard rail application.

For residential installations only 200# concentrated load is applicable.

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Connection to support:

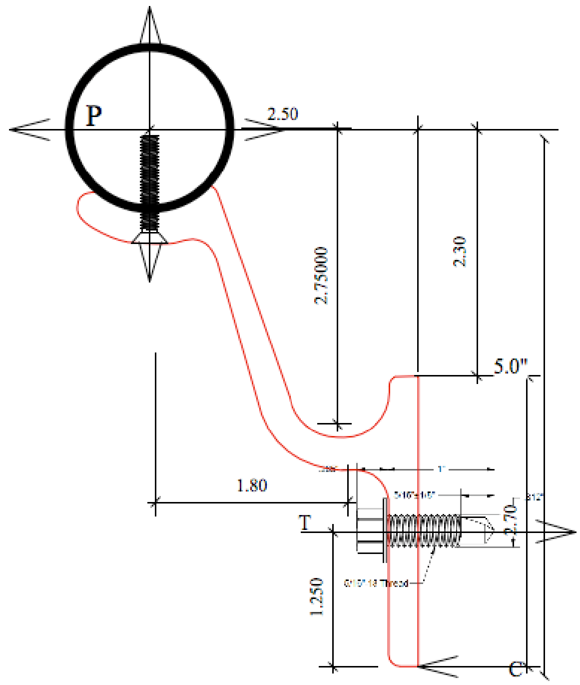
Maximum tension occurs for outward Horizontal force = 200#:

Determine tension from ∑M about C

0= P*5" – T*0.875"

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.

5/16" self drilling hex head screw

Safety Factor = 2.34 for guard rail application.

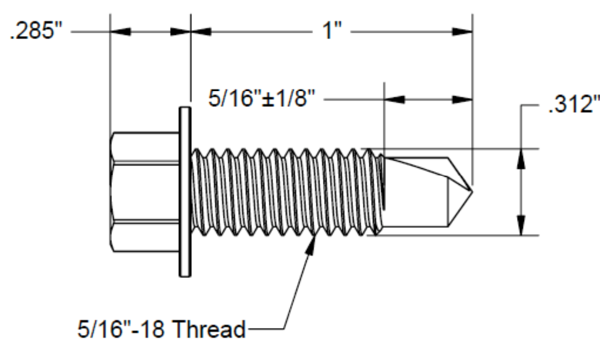
Shear – $F_{tu}Dt/2.34$ (ADM 5.4.3)

38ksi*0.3125"*0.1"/2.34= 507#

Tension – Pullout ADM 5.4.2.1

$P_t = 0.58A_{sn}F_{tu}(t_c)]/2.34 =$

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)

$(T/P_t)^2 + (V/Z_a)^2 \leq 1$

$(434/642)^2 + (217/508)^2 = 0639 \leq 1$

Or

$(434/642) + (217/508) = 1.10 \leq 1.2$

Or

600 ≤ 642# therefore okay

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Excerpts from National Design Specifications

Table 11.2A Lag Screw Withdrawal Design Values (W)[1]

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

Specific Gravity G	Lag Screw Unthreaded Shank 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).

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	
t_s	D	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$	$Z_{\parallel}$	$Z_{\perp}$
in.	in.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
0.075	1/4	170	130	160	120	150	110	150	110	150	100	140	100
(14 gage)	5/16	220	160	200	140	190	130	190	130	190	130	180	120
	3/8	220	160	200	140	200	130	190	130	190	120	180	120
0.105	1/4	180	140	170	130	160	120	160	120	160	110	150	110
(12 gage)	5/16	230	170	210	150	200	140	200	140	190	130	190	130
	3/8	230	160	210	140	200	140	200	130	200	130	190	120
0.120	1/4	190	150	180	130	170	120	170	120	160	120	160	110
(11 gage)	5/16	230	170	210	150	210	140	200	140	200	140	190	130
	3/8	240	170	220	150	210	140	210	140	200	130	200	130
0.134	1/4	200	150	180	140	180	130	170	130	170	120	160	120
(10 gage)	5/16	240	180	220	160	210	150	210	140	200	140	200	130
	3/8	240	170	220	150	220	140	210	140	210	140	200	130

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