The Hansen Aluminium Railing System (ARS) utilizes aluminium 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.


Edward Robison, P.E.
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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<td>74 - 84</td>
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SIGNED:
11 Apr 2018

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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
\[ M_i = 25 \text{ psf} \times H \times S \times \frac{H}{2} = 12.5 \times S \times H^2 \]

For top rail loads:
\[ M_c = 200\# \times H \]
\[ M_u = 50\text{plf} \times S \times H \]

For wind load surface area:
\[ M_w = w \text{ psf} \times H \times S \times H \times 0.55 = 0.55w \times S \times H^2 \]

Solving for w :
\[ w = M/(0.55 \times S \times H^2) \]

Wind load equivalent for 42” rail height, 5’ post spacing 50 plf top rail load:
\[ M_u = 50\text{plf} \times 5' \times 3.5' = 875\# = 10,500\#” \]
\[ w = 875/(0.55 \times 5 \times 3.5^2) = 26 \text{ psf} \]

Allowable wind load adjustment for other post spacing:
\[ w = 26 \times (5/S) \]
WIND LOADING
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.

\[ p = q_0(GC_p) = q_0GC_f \] (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.)

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

\[ Q_z = K_zK_nK_dV^2I \]

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).
\[ K_n \] From Figure 6-4 (Fig 26.8-1) for the site topography, typically 1.0.
\[ V = \text{Wind speed (mph)} \] 3 second gust, Figure 6-1 (Fig 26.5-1A) or per local authority.

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

\[ \text{For } C_f = 1.3: \quad F = q_h \times 0.85 \times 1.3 = 1.11 q_h \]
\[ \text{For } C_f = 2.6: \quad F = q_h \times 0.85 \times 2.6 = 2.21 q_h \]

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:

<table>
<thead>
<tr>
<th>Exposure</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.70</td>
<td>0.85</td>
<td>1.03</td>
</tr>
</tbody>
</table>

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 \( K_n = 1.0 \)

<table>
<thead>
<tr>
<th>Wind Speed</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>V</td>
<td>0.00169V^2</td>
<td>0.00205V^2</td>
<td>0.00249V^2</td>
<td>0.00337V^2</td>
<td>0.00409V^2</td>
<td>0.00495V^2</td>
</tr>
<tr>
<td>85</td>
<td>12.2</td>
<td>14.8</td>
<td>17.9</td>
<td>24.3</td>
<td>29.5</td>
<td>35.8</td>
</tr>
<tr>
<td>90</td>
<td>13.7</td>
<td>16.6</td>
<td>20.2</td>
<td>27.3</td>
<td>33.1</td>
<td>40.1</td>
</tr>
<tr>
<td>100</td>
<td>16.9</td>
<td>20.5</td>
<td>24.9</td>
<td>33.7</td>
<td>36.9</td>
<td>49.5</td>
</tr>
<tr>
<td>110</td>
<td>20.5</td>
<td>24.8</td>
<td>30.1</td>
<td>40.7</td>
<td>49.5</td>
<td>59.9</td>
</tr>
<tr>
<td>120</td>
<td>24.3</td>
<td>29.6</td>
<td>35.8</td>
<td>48.5</td>
<td>58.9</td>
<td>71.3</td>
</tr>
<tr>
<td>130</td>
<td>28.6</td>
<td>34.7</td>
<td>42.0</td>
<td>56.9</td>
<td>69.1</td>
<td>83.7</td>
</tr>
<tr>
<td>140</td>
<td>33.1</td>
<td>40.2</td>
<td>48.8</td>
<td>66.0</td>
<td>80.1</td>
<td>97.1</td>
</tr>
</tbody>
</table>

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 \( w = 0.6 \times 20.5 = 12.3 \) psf (ASD wind loads used herein)

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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):

\[
F_R = 24,000 \text{ psi}.
\]
\[
E = 10,400 \text{ ksi}. \quad \text{While the value of } E \text{ for glass varies with the stress and load duration this value is typically used as an average value for the stress range of interest.}
\]
\[
G = 3,800 \text{ ksi}. \quad \text{This is rarely used when checking the deflection in glass. The shear component of the deflection tends to be very small, under 1\% of the bending component and is therefore ignored.}
\]
\[
\mu = 0.22 \quad \text{Typical value of Poisson’s ratio for common glasses.}
\]

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:

\[
I = 12***(t^3)/12= (t^3) \text{ in}^3/\text{ft}
\]
\[
S = 12***(t^2)/6= 2*(t^2) \text{ in}^3/\text{ft}
\]

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

\[ M_w =WL^2/8 \] for uniform load \( W \) and span \( L \) or

\[ M_P =PL/4 \] for concentrated load \( P \) and span \( L \), highest moment \( P @ \) center

Maximum wind loads:

\[ W = Ma^*8/L^2 \] for uniform load \( W \) and span \( L \) (rail to rail distance)

Deflection can be calculated using basic beam theory:

\[ \Delta = (1-v^2)wL^4/(384EI) \] for uniform load

For concentrated load:

\[ \Delta = (1-v^2) PL^3/(48EI) \]

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 \( L \)

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

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 \( S = 2*(0.223)^2 = 0.0995 \text{ in}^3/\text{ft} \)
\( M_{allowable} = 6,000 \text{psi} * 0.0995 \text{ in}^3/\text{ft} = 597”/\text{ft} \)

For FS = 2.5 (no fall hazard, glass fence or wind screen)
\( M_{all} = 597”/4/2.5 = 955”/\text{ft} \)

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

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

for 36” wide lite (infill for 42” rail height)
\( W = 597”/8/(3”*36”) = 44 \text{ psf} \)

for 42” wide lite (infill for 48” rail height)
\( W = 597”/8/(3.5”*42”) = 32.5 \text{ psf} \)

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 \( \Delta = (1-0.22^2)*50*36^3/(4.992*10^8*0.25^3) = 0.285” \)
3/8” FULLY TEMPERED GLASS

Weight = 4.75 psi
t_{uvc} = 0.366”

For 3/8” glass S = 2*(0.366)^2 = 0.268 in^3/ft

M_{allowable} = 6,000 psi*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 = 25 psf*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 = 25 psf*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

Δ = [(1-0.22^2)* 25*36^4/0.366^3]/(9.58 x 10^9) = 0.085”

or

Δ = (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)
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.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1 &amp; H2</td>
<td>Glass pane thicknesses</td>
</tr>
<tr>
<td>Hv</td>
<td>Interlayer thickness</td>
</tr>
<tr>
<td>E</td>
<td>Young's Modulus</td>
</tr>
<tr>
<td>g</td>
<td>Shear Modulus</td>
</tr>
<tr>
<td>Hs</td>
<td>(0.5(h_1+h_2)+hv)</td>
</tr>
<tr>
<td>Hs;1</td>
<td>(h_{sh1}/(h_1+h_2))</td>
</tr>
<tr>
<td>Hs;2</td>
<td>(h_{sh2}/(h_1+h_2))</td>
</tr>
<tr>
<td>Is</td>
<td>(h_1(h_{sh1})^2+h_2(h_{sh1})^2)</td>
</tr>
<tr>
<td>a</td>
<td>Minimum Pane Width</td>
</tr>
<tr>
<td>Γ</td>
<td>(1/(1+9.6(Eishv/(G(\sigma)^2))))</td>
</tr>
<tr>
<td>h_{efw}</td>
<td>(\sqrt{(h_1)^2+(h_2)^2+2\Gamma Is})</td>
</tr>
<tr>
<td>h_{1;efσ}</td>
<td>(\sqrt{(h_{efw})^2/(h_1+2\Gamma h_{sh1})})</td>
</tr>
<tr>
<td>h_{2;efσ}</td>
<td>(\sqrt{(h_{efw})^2/(h_2+2\Gamma h_{sh1})})</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Laminated Glass Effective Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>h1</td>
</tr>
<tr>
<td>h2</td>
</tr>
<tr>
<td>hv</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>g</td>
</tr>
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</tr>
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<td>h_{sh1}</td>
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<tr>
<td>h_{sh1}</td>
</tr>
<tr>
<td>a</td>
</tr>
<tr>
<td>(he_{fw})</td>
</tr>
<tr>
<td>h_{1;efσ}</td>
</tr>
<tr>
<td>h_{2;efσ}</td>
</tr>
<tr>
<td>h_{1;efσ}</td>
</tr>
<tr>
<td>h_{2;efσ}</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Glass Size, ( t_{ave} ) (in)</th>
<th>( t_{ef,w} ) (in)</th>
<th>( t_{ef,\sigma} ) (in)</th>
<th>( I ) (in(^4)/ft)</th>
<th>( S ) (in(^3)/ft)</th>
<th>( W_a ) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>0.201</td>
<td>0.223</td>
<td>0.0081206</td>
<td>0.099458</td>
<td>29</td>
</tr>
<tr>
<td>5/16</td>
<td>0.218</td>
<td>0.243</td>
<td>0.0103602</td>
<td>0.118098</td>
<td>37</td>
</tr>
<tr>
<td>3/8</td>
<td>0.301</td>
<td>0.337</td>
<td>0.0272705</td>
<td>0.227138</td>
<td>98</td>
</tr>
</tbody>
</table>
**2-3/8” Square Post**
6061-T6 Aluminum

4 screw post
-Area 0.995”

\[ I_{xx} = I_{yy} = 0.863 \text{ in}^4 \]
\[ S = 0.726 \text{ in}^3 \]
\[ Z = 0.9748 \text{ in}^3 \]
\[ r = 0.923 \text{ in} \]
\[ J = 1.341 \text{ in}^4 \]
\[ k \leq 1 \text{ for all applications} \]

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.

\[ C_b=1.3 \text{ for cantilevered beam with concentrated load at free end (ADM 15 F.4.1)} \]
\[ \lambda = 2.3(L_bS_C/(C_b(I_yJ)^{1/2}))^{1/2} = 2.3(L_b*.726/(1.3*(.863*1.341)^{1/2}))^{1/2} = 1.657 L_b^{1/2} \]
Inelastic buckling controls when \( \lambda < C_c=65.7 \)

\[ 65.7 = 1.657 L_b^{1/2} \]
\[ L_b=1,572” \]

For \( L_b=42” \)
\[ \lambda = 1.657*42^{1/2} = 10.74 \]
\[ M_{amb}=M_p(1-\lambda/C_c)+\pi^2 E\lambda S_{sc}/C_c^3 \]
\[ M_p=35\text{ksi}*9748\text{in}^3=34,118”# \]
\[ M_{amb}=34,118(1-10.74/65.7)+\pi^2*10.1*10^6*10.74*.726/65.7^3=31,281”# \]
\[ M_{amb}/\Omega = 31,281”#/1.65 = 18,958”# \]

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, $F_c/\Omega=21.2$ksi (Local buckling does not apply)
$Z<1.5S$
$M_{nul}/\Omega = ZF_y/\Omega = 0.9748\text{in}^3\times21.2\text{ksi} = 20,666$” or
$M_{nul}/1.95 = ZF_u = 0.9748\text{in}^3\times38\text{ksi}/1.95 = 18,996$” (Controls)

Bending strength of post installed with top rail:
$M_s=19,000$”#

Strong axis deflections:
$\Delta = PL^3/(3EI) = PL^3/(3\times10,100,000\text{psi}\times0.863\text{in}^4) = PL^3/26,148,900$
$P_{1”} = 26,148,900/L^3$ 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:
$P = 10,100,000\text{psi}\times0.863\text{in}^4/(4\times42^2) = 1,235$# - 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

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

Addition of holes at base of post only affects rupture strength.
$M_{nul}/\Omega = ZF_y/\Omega = 0.7590\text{in}^3\times38\text{ksi}/1.95 = 14,791$”#

For loading perpendicular to bolt axis
$I_{red} = 0.8750\text{in}^4$
$S_{red} = 0.7365\text{in}^3$
$Z_{red} = 0.8666\text{in}^3$
$M_{nul}/\Omega = ZF_y/\Omega = 0.8666\text{in}^3\times38\text{ksi}/1.95 = 16,888$”#
2-3/8" Square Post
6 Screw Post

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

\[ k \leq 1 \quad \text{for all applications} \]

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.

\[ C_b = 1.3 \quad \text{for cantilevered beam with concentrated load at free end (ADM 15 F.4.1)} \]
\[ \lambda = 2.3( L_B S_c/( C_b I_y J)^{1/2} )^{1/2} = 2.3( L_b^{*}.8388/(1.3^*(.889*.1.341)^{1/2} )^{1/2} = 1.768 L_B^{1/2} \]

Inelastic buckling controls when \( \lambda < C_c = 65.7 \)
65.7 = 1.768 L_B^{1/2}
\[ L_b = 1.381" > 48" \quad (\text{Much higher than practical post heights}) \]

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

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.
Yielding/Rupture/Local Buckling:

\[ \frac{b}{t} = \frac{1.95}{0.1} = 19.5 < 20.8 \]
\[ F_c/\bar{\Omega} = 21.2 \text{ ksi} \]
\[ Z < 1.5S \]
\[ M_{up}/\bar{\Omega} = ZF_u/\bar{\Omega} = 0.9996\text{in}^3/21.2\text{ksi} = 21,192'''' \text{ (Controls)} \]
\[ M_{up}/1.95 = ZF_u = 0.9996\text{in}^3/38\text{ksi}/1.95 = 19,479'''' \text{ (Controls)} \]

Bending strength of post installed with top rail:
\[ M_a = 19,500'''' \]

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

Deflection for 200# load for 42'' post height:
\[ \Delta = PL^3/(3EI) = 200*42^3/(3*10,100,000\text{psi}*0.9971\text{in}^4) = 0.49'''' \]

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

For bending parallel to bolts:
\[ S_{red} = 0.564 \text{ in}^3, \ A_r = 0.125*1.875^2 = 0.439 \text{ in}^2 \]
\[ F_{tb} = 21 \text{ ksi at reduced section} \]
\[ M_{red} = 21\text{ksi} * 0.564 \text{ in}^3 = 11,844'''' \]

To allow for shear stress from bolt bearing on post limit moment so that:
\[ M/11,844 + [(T_{bol}/0.439)/12000]^2 \leq 1.0 \]
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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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/Ω = 21.2 ksi
Z<1.5S
M_{np}/Ω = ZF_c/Ω = 1.127 in^3 * 21.2 ksi = 23,892#  or
M_{np}/1.95 = ZF_c = 1.127 in^3 * 38 ksi = 43,390# (Controls)

For bending about Y-axis
b/t = 1.812/0.09 = 20.1 < 20.8
F_c/Ω = 21.2 ksi
Z<1.5S
M_{np}/Ω = ZF_c/Ω = 1.340 in^3 * 21.2 ksi = 28,408#  or
M_{np}/1.95 = ZF_c = 1.340 in^3 * 38 ksi = 51,340# (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.

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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
\[ F_t = 24 \text{ ksi} \rightarrow S_{\text{min}} = 5'' \cdot 3/8^2 \div 6 = 0.117 \text{ in}^3 \]

Base plate allowable moment
\[ M_{\text{all}} = 24 \text{ ksi} \cdot 0.117 \text{ in}^3 = 2,812 \text{ in}'' \]
\[ \rightarrow \text{Base plate bending stress} \]
\[ T_B = C \]
\[ M = 0.8125'' \cdot T_B \cdot 2 \]
\[ T_{\text{all}} = \frac{2.812}{2 \cdot 0.8125} = 1,730'' \]

Maximum post moment for base plate strength
\[ M_{\text{all}} = 2 \cdot 1,730 \cdot 4.375'' = 15,142'' \]

Limiting factor = screws to post
\[ M_{\text{ult}} = 2 \cdot 5,314'' \cdot 2.28'' = 24,232'' \]
\[ M_{\text{all}} = 2 \cdot 2,993'' \cdot 2.28'' = 10,500'' \]

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**
\[ T_{\text{Des}} = \frac{10,500}{2 \cdot 4.375''} = 1,195'' \]

adjustment for concrete bearing pressure:
\[ a = 2 \cdot 1.195/(2 \cdot 3000 \text{ psi} \cdot 4.75'') = 0.087'' \]
\[ T'_{\text{Des}} = \frac{10,500}{2 \cdot (4.375''-0.087/2)} = 1,206'' \]

For 200'' top load and 42'' post ht
\[ T_{200} = \frac{8,400}{2 \cdot 4.375''} = 960'' \]

For 42'' post height the maximum live load at the top of the post is:
\[ P_{\text{max}} = 10,500''/42'' = 250'' \]

For 50 plf live load maximum post spacing is:
\[ S_{\text{max}} = 250''/50 \text{ plf} = 5'' = 5'0'' \]
LOAD TESTS:
Connection strength from load testing post/base plate assemblies:
42” from top of base plate to centerline of load.

\[ M_{\text{fail}} = (524.2\# \times 42") = 22,226"\# \]

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:
\[ SF = \frac{(1.05\alpha+1)}{M_MF_M (\alpha +1)} - \frac{\beta_0\sqrt{V_M^2+V_F^2+C_PV_F^2+V_Q^2}}{} \]

Where: \( M_M = 1.10, \quad F_M = 1.00, \quad V_M = 0.06, \quad V_Q = 0.21, \quad \beta_0 = 3.5, \quad V_F = 0.05, \quad V_P = 0.0192 \)
\( M_M = 1.10 \) selected because strength is controlled by steel screw not aluminum failure.
\( C_P = (n^2-1)/(n^2-3n) = (7^2-1)/(7^2-3\times 7) = 1.71; \quad \alpha = 0.2 \)
\[ SF = \frac{(1.05\alpha+1)}{[1\times1.1\times(0.2+1)]\times e\{3.5\sqrt{0.06^2+0.05^2+1.71\times0.0146^2+0.21^2}\}} = 2.07 \]

From test strengths
\[ M_{\text{allowable}} = 22,226"\#/2.07 = 10,895"\# \]

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_e = 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

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Allowable = 19 ksi

Bearing on nut:
Area = \((0.8^2-0.5625^2)\pi\) = 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\(_{200}\) = \frac{7,200}{2*4.375} = 823#

Adjustment for wood bearing:
Bearing Area Factor:
C\(_b\) = (5”+0.375)/5” = 1.075
a = \frac{2*823/(1.075*625\text{psi}*5")}{0.49”} = 0.49”
T = \frac{7,200/[2*(4.375-0.49/2)]}{323#/in} = 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.**

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BASE PLATE MOUNTED TO UNCRAKCED 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 ≥ 3,000 psi; edge distance =2.25”  spacing = 3.75”
h = 3.0”: embed depth
For concrete breakout strength:
N_{cb} = \left(\frac{A_{Ncg}}{A_{Nco}}\right)\phi_{ed,N}\phi_{c,N}\phi_{h,N}\phi_{cb,N}
A_{Ncg} = (1.5*3.2+3.75)*(1.5*3+2.25) = 86.06 in²  2 anchors
A_{Nco} = 9*3² = 81 in²
C_{a,min} = 1.5”  (ESR-2427 Table 3)
C_{ac} = 5.25”  (ESR-2427 Table 3)
\phi_{ed,N} = 1.0
\phi_{c,N} = \text{use 1.0 in calculations with } k = 24
\phi_{cp,N} = \text{max (1.5/5.25 or 1.5*3”/5.25) = 0.857 (c_{a,min} ≤ c_{ac})}
N_{cb} = 86.06/81*1.0*1.0*0.857*6,830 = 6,219 ≤ 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}(\phi_{ed,v}\phi_{c,v}\phi_{h,v}\phi_{cb,v})
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
\phi_{ed,v} = 1.0  \text{ (affected by only one edge)}
\phi_{c,v} = 1.4 uncracked concrete
\phi_{h,v} = \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} =1.225
V_{cb} = [7(1/d_a)²/\alpha\phi_{c,v}\phi_{h,v}f’_{c}(c_{a1})]^{1.5} = [7(1.625/0.375)²/\alpha\phi_{c,v}\phi_{h,v}f’_{c}(c_{a1})]^{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
\phi V_{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_n = 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.

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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 ΣM_{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 psi post to grout

f_{Bconcrete} = 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_0 = 2.375”+3*C_{a1}+2*(2”+1.5C_{a1})

\beta = (2.375”+3*C_{a1})/(2”+1.5C_{a1})

V_{n,min} = V*LF/\varphi = 5093#*1.6/0.75 = 10,865#
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
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

$k \leq 1$ 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_bI_{yy})^{1/2}) = 2.3(L_b*.8388/(1.3*($8.89*1.341)^{1/2}) = 1.768 L_b^{1/2}$

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 $L_b=42”$

$\lambda = 1.768*42”^{1/2} = 11.46$

$M_{amb}=M_{p}(1-\lambda/C_c)+\pi^2EI_{xx}/C_c^3$

$M_p=35ksi*.9996in^3=34,986”#$

$M_{amb}=34,986(1-11.46/65.7)+\pi^2*10*10^6*11.46*.8388/65.7^3=32,229”#$

$M_{amb}=32,229”#/1.65=19,533”#$

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:

\[ \frac{b}{t} = 1.95/0.1 = 19.5 < 20.8 \]
\[ F_c/\Omega = 21.2 \text{ ksi} \]
\[ Z < 1.5S \]
\[ \frac{M_{up}}{\Omega} = ZF_u/\Omega = 0.9996 \text{in}^3 \times 21.2 \text{ksi} = 21.192" \text{" or } \]
\[ \frac{M_{up}}{1.95} = ZF_u = 0.9996 \text{in}^3 \times 38 \text{ksi}/1.95 = 19.479" \text{# (Controls)} \]

Weak axis bending = 0.9011 \text{in}^3 \times 38 \text{ksi}/1.95 = 17.560" \text{# (Controls for weak axis bending)}

Bending strength of post installed with top rail:
\[ M_a = 19,500" \text{#} \]

Strong axis deflections:
\[ \Delta = \frac{PL^3}{(3EI)} = \frac{P}{3} \times 10,100,000 \text{psi} \times 0.9971 \text{in}^4 = \frac{PL^3}{30,212,130} \]
\[ P_{1"'} = 30,212,130/L^3 \text{ for 42" post height} = 408" \]
\[ L_{1"'} = \left(\frac{30,212,130}{P}\right)^{1/3} \text{ for 250" L = 49.5/16"} \]

For L/12 (maximum allowable post deflection from ASTM E-985 test loads)
\[ P = EI/(4L^2) \text{: for 42" height:} \]
\[ P = 10,100,000 \text{psi} \times 0.9971 \text{in}^4/(4 \times 42^2) = 1.427" \text{ - Deflection will not control post loads} \]

Deflection for 200# load for 42" post height:
\[ \Delta = \frac{PL^3}{(3EI)} = \frac{200 \times 42^3}{(3 \times 10,100,000 \text{psi} \times 0.9971 \text{in}^4)} = 0.49" \]

For posts directly fascia mounted with 3/8" (7/16" dia holes) bolts through post:

Reduced strength at bolt hole:
Bending perpendicular to bolts
\[ S_{red} = 0.6026 \text{ in}^3 \]
\[ F_{tb} = 21 \text{ ksi at reduced section} \]
\[ M_{red} = 21 \text{ksi} \times 0.6026 \text{ in}^3 = 12,655" \text{#} \]

For bending parallel to bolts:
\[ S_{red} = 0.564 \text{ in}^3, \quad A_f = 0.125 \times 1.875^2 = 0.439 \text{ in}^2 \]
\[ F_{tb} = 21 \text{ ksi at reduced section} \]
\[ M_{red} = 21 \text{ksi} \times 0.564 \text{ in}^3 = 11,844" \text{#} \]

To allow for shear stress from bolt bearing on post limit moment so that:
\[ \frac{M}{11,844} \times (\frac{0.439}{12000})^2 \leq 1.0 \]

For example if bolt tension = 2,000# the maximum allowable moment is:
\[ M_a = \{1.0 - (\frac{2000}{0.439}/12000)^2\} \times 11,844 = 10,137" \text{#} \]
Heavy Post
6061-T6 Aluminum

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

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

$k \leq 1$ 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
$F_y/\Omega = 35\text{ksi}/1.65 = 21.2\text{ksi}$
$F_u/\Omega = 38\text{ksi}/1.95 = 19.5\text{ksi}$ (Controls)

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

$\Delta = PL^3/(3EI) = PL^3/(3*10,100,000\text{psi}*1.0757\text{in}^4) = PL^3/32,593,710$
$P_{1''} = 32,593,710/L^3$ 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:
$P = 10,100,000\text{psi}*1.0757\text{in}^4/(4*42^2) = 1,540#$ - Deflection will not control post loads

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

\[ T_a = 2,293 \text{# per screw} \]
\[ V_a = 917 \text{# per screw} \]

\[ V_{des} = 6 \times 917 = 5,502 \text{#} \]
Limiting shear load on post so that screw shear stress doesn’t reduce the allowable tension:

\[ V_{0.2} = 0.2 \times 5,502 = 1,100 \text{#} \]

Base plate thickness and strength same as for standard post.

**Allowable moment on the posts based on screw tension strength:**

Strong axis bending -

\[ M_{base} = 3 \text{ screws} \times 2,293 \text{#} \times 2.28" = 15,684" \text{#} < 19,479" \text{#} \]

Doesn’t develop full post strength.

Weak axis bending -

\[ M_{base} = 2 \text{ screws} \times 2,293 \text{#} \times 2.28" + 2 \text{ screws} \times 0.5 \times 2,293 \text{#} \times 2.28" / 2 = 13,070" \text{#} \leq 17,560" \text{#} \]

6 screw connection won’t develop the full post strength for weak axis bending.

**LIMITING POST MOMENTS FOR SIX SCREW CONNECTION:**

**STRONG AXIS BENDING** \( M_A = 15,684" \text{#} = 1,307" \text{#} \)

**WEAK AXIS BENDING** \( M_A = 13,070" \text{#} = 1,089" \text{#} \)
FASCIA BRACKET
Allowable stresses
ADM Table 2-24  6063-T6 Aluminum

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

Section Properties
Area: 2.78 sq in
Perim: 28.99 in
Ix: 3.913 in^4
Iy: 5.453 in^4
Cxx: 1.975 in/1.353 in
Cyy: 2.954 in
Sxx: 1.981 in^3 front
Sxx: 2.892 in^3
Syy: 1.846 in^3

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Allowable moment on bracket:

\[ M_a = F_t \cdot S \]
\[ M_{axx} = 15 \text{ ksi} \cdot 1.981 \text{ in}^3 = 29,175 \text{#} - \text{Outward moment} \]
\[ M_{axy} = 15 \text{ ksi} \cdot 1.846 \text{ in}^3 = 27,690 \text{#} - \text{Sidewise moment} \]

Flange bending strength

Determine maximum allowable bolt load:

Tributary flange

\[ b_t = 8t = 8 \cdot 0.1875 = 1.5" \text{ each side of hole} \]
\[ b_t = 1.5" + 1" + 0.5" + 1.75" = 4.75" \]
\[ S = 4.75" \cdot 0.1875 \cdot \frac{1}{6} = 0.0278 \text{ in}^3 \]
\[ M_{af} = 0.0278 \text{ in}^3 \cdot 20 \text{ ksi} = 557 \text{#} \]

Allowable bolt tension

\[ T = \frac{M_{af}}{0.375} = 1,485 \text{#} \]
\[ 3/8" \text{ bolt standard washer} \]

For Heavy washer

\[ T = \frac{M_{af}}{0.1875} = 2,971 \text{#} \]

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

\[ T_{top} = \frac{[250\# \cdot (42" + 7")/5"]}{2 \text{ bolts}} = 1,225\# \text{ tension} \]
\[ T_{bot} = \frac{[250\# \cdot (42" + 3")/5"]}{2 \text{ bolts}} = 1,125\# \text{ tension} \]

For centerline holes:

\[ T = \frac{[250\# \cdot (42" + 5")/3"]}{2 \text{ bolts}} = 1,958\# \text{ tension} \]

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:

\[ l_e = 1.225#/389#/\text{in} = 3.15" : +7/32" \text{ for tip is 3.37"} \]

For weather exposed installations the minimum embedment is:

\[ l_e = 1.225#/272#/\text{in} = 4.50" : +7/32" \text{ for tip is 4.72" requires 5-1/2” screw} \]

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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

\[ T_{up} = \frac{200\#(36”+7”)/5”}{2 \text{ bolts}} = 860\# \text{ tension} \]
\[ T_{bot} = \frac{200\#(36”+3”)/5”}{2 \text{ bolts}} = 780\# \text{ tension} \]

For protected installations the minimum embedment is:
\[ l_e = \frac{860\#/323#/\text{in}}{2.66”} : +7/32” \text{ for tip = 2.88”} \]

For weather exposed installations the minimum embedment is:
\[ l_e = \frac{860\#/243#/\text{in}}{3.54”} : +7/32” \text{ for tip = 3.76”} \]

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

\[ T_{up} = \frac{200\#(42”+7”)/5”}{2 \text{ bolts}} = 980\# \text{ tension} \]
\[ T_{bot} = \frac{200\#(42”+3”)/5”}{2 \text{ bolts}} = 900\# \text{ tension} \]

For protected installations the minimum embedment is:
\[ l_e = \frac{980\#/323#/\text{in}}{3.03”} : +7/32” \text{ for tip = 3.25”} \]

For weather exposed installations the minimum embedment is:
\[ l_e = \frac{980\#/243#/\text{in}}{4.03”} : +7/32” \text{ for tip = 4.25”} \]

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.

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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:

\[ P = 200\# \text{ or } 50\text{plf} \times 4 = 200\# \]

\[ M_{\text{deck}} = 42'' \times 200\text{plf} = 8,400''\# \]

Load from glass infill lites:

- Wind = 25 psf
  \[ M_{\text{deck}} = 3.5'' \times 25\text{psf} \times 42''/2 \times 4' \text{o.c.} = 7,350''\# \]

- 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

- \( W = 305 \# /\text{in of thread penetration} \)
- \( C_D = 1.6 \) for guardrail live loads or wind loads.
- \( C_m = 1.0 \) for weather protected supports (lags into wood not subjected to wetting).
- \( T_b = W C_D C_m l_m = \text{total withdrawal load in lbs per lag} \)
- \( W' = W C_D C_m = 305''/'' \times 1.6 \times 1.0 = 488''/\text{in} \)

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

\[ T_b = 488\# \times 2.4'' = 1,171\# \]

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

\( Z'_{\|} = 220\# \times 1.6 \times 1.0 = 352\# \)

\( Z_T = 140\# \text{ per lag, (vertical load)} \)

\( Z_T = 140\# \times 1.6 \times 1.0 = 224\# \)

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

From $\Sigma M$ about end

\[ M = (8.5''T + 1.5'' \times 1.5/8.5''T) = 8.76''T \]

Allowable post moment

\[ M_a = 972'' \times 8.76'' = 8,515''# \]

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

For 3/8” carriage bolts:

Allowable load per bolt = 0.11 in$^2$$ \times 20$ ksi = 2,200#

For bearing on 2” square bearing plate – area = 3.8 in$^2$

\[ P_b = 3.8 \text{ in}^2 \times 1.19 \times 405 \times 1.33 = 2,436# \]

\[ M_a = 2,200'' \times 8.76'' = 19,272''# \text{ (exceeds post strength)} \]

For vertical load lag capacity is:

- 2 lags*187# = 374#/post for live load
- 2 lags*140# = 280#

\[ D + L = 200/374 + 60/280 = 0.75 < 1.0 \text{ okay} \]

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.

\[ S_h = 0.726 - 2 \times (7/16 \times 0.125) \times (2.255/2)^2 = 0.588 \text{ in}^3 \]

\[ M_{ared} = 19,000 \times 0.588 = 11,172''# \]

Maximum moment calculated at the centerline of the top hole must not exceed 11,172”## = 931’##
**STANCHION MOUNT**

2”x1-1/2”x 1/8” A500 steel tube

Stanchion Strength

\[ F_{yc} = 45 \text{ ksi} \]

\[ Z_{yy} = 0.543 \text{ in}^3 \]

\[ M_u = 0.543 \text{ in}^3 \times 45 \text{ ksi} = 24,435" \]

\[ M_s = \phi M_u / 1.6 = 0.9 \times 24,435 / 1.6 = 13,745" \]

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:

\[ A_{surface} \sqrt{f''c} = 7" \times 4" \times \sqrt{8,000 \text{ psi}} = 2,500" \]

(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 \[ \Sigma M \] about base of stanchion = 0

\[ P_u = M + V \times D = \frac{2/3 D}{2/3 \times 4} \]

For: \[ M = 10,500" \]

\[ P_u = 10,500 + 250 \times 4 = 4,312" \]

\[ f_{B_{max}} = \frac{P_u \times 2}{D \times 1.5" \times 0.85} = \frac{4,312 \times 2}{4" \times 1.5" \times 0.85} = 1.691 \text{ psi} \]

For: \[ M = 12,600" \]

\[ P_u = 12,600 + 300 \times 4 = 5,175" \]

\[ f_{B_{max}} = \frac{P_u \times 2}{D \times 1.5" \times 0.85} = 2.029 \text{ psi} \]

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”x1.5”x 1/8” powder coated A500 steel tube stanchion:
Stanchion Strength
Fy = 46 ksi
Zyy = 0.475 in³
Mn = 0.475 in³ * 46 ksi = 21,850#”
Ms = φMn/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
Fc = 21 ksi  From ADM Table 2-22
Syy = 0.719 in³
Ma = 0.719 in³ * 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:
Fc, = 9 ksi
Syy = 0.719 in³
Ma = 0.719 in³ * 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 \( \frac{d_c}{t} = \frac{(2*2/3)}{0.125} = 10.67 < \lambda_1 \)
\( \lambda_1 = 1.1/\sqrt{(F_{yc}/E_o)} = 1.1/\sqrt{(50/28*10^3)} = 26 \)
\( M_n = 0.543 in^3 * 50 ksi = 27,148\#'\)
\( M_s = \phi M_n/1.6 = 0.9*27,148/1.6 = 15,270\#'\)

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:
\( \phi P_n = \phi t L F_{ua}, \text{ Use } Z \text{ for } tL \)
\( P_n = 0.55*0.362*80 \text{ ksi} \)
\( P_n = 15,928 \)
\( P_s = 15,928/1.2 = 13,273\#'\)

Grout bond strength to stanchion:
\( A_{\text{surface}} \sqrt{f'c} = 7''*6''*\sqrt{10,000} \text{ psi} = 4,200\#' \) (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.
**Series 100 Top Rail**

Butts into post

Alloy  6063 – T6 Aluminum

Allowable Stress
ADM Table 2-21

\[ F_c / \Omega = 15.2 \text{ ksi} \]

Check lateral torsional buckling about strong axis:

\[
J = 0.2359 \text{ in}^4
\]

\[ \lambda = \frac{2.3(L_B S_c / (C_h I_y)^{1/2})^{1/2}}{1*(.2951*.2359)^{1/2}} = 2.219 L_B^{1/2} \]

Inelastic buckling controls when \( \lambda < C_c = 78 \)

\[ L_B = 78 \]

\[ L_B = 1,236" \]

For \( L_B = 60" \), \( \lambda = 17.19 \)

\[ Z_x = 0.3880 \text{ in}^3 \]

\[ M_{nmb} = M_p(1 - \lambda / C_c) + \pi^2 E I S_{xc} / C_c^3 \]

\[ M_p = 30 \text{ ksi} * 0.3880 \text{ in}^3 = 11,640"# \]

\[ M_{nmb} = 11,640(1 - 17.19/78) + \pi^2 * 10^6 * 17.19 * 0.2455/78^3 = 9,952"# \]

\[ M_{nmb}/\Omega = 9,952"#/1.65 = 6,032"# \]

Check local buckling about strong axis:

\[ R_b/t = 2.5"/0.065" = 38.46 > 31.2 \]

\[ F_c / \Omega = 18.5 - .593 * 38.46^{1/2} = 14.82 \text{ ksi} \]

\[ M_a = 14.82 \text{ ksi} * 0.2455 \text{ in}^3 = 3,638"# \] (Controls)

Check local buckling about weak axis:

\[ b/t = 1.186"/0.065" = 18.25 < 22.8 \]

\[ F_c / \Omega = 15.2 \text{ ksi} \] (local buckling does not control)

\[ M_a = (F_c / \Omega) \times Z_y = 15.2 \text{ ksi} * 0.3915 \text{ in}^3 = 5,951"# \] (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, \quad S_{xx} = 0.101 in^3
I_{yy} = 0.164 in^4, \quad S_{yy} = 0.193 in^3
r_{xx} = 0.476", \quad r_{yy} = 0.603"

b/t = .807"/.07" = 11.5>7.3
F_{c/\Omega} = 19-0.53*11.5 = 12.9ksi
Allowable Moments → Horiz. = 0.193in^3 • 12.9 ksi = 2,490"
Maximum allowable load for 72” o.c. post spacing
W = 2,490"#/(67.625")^2 = 4.36 pli = 52.27 plf
P = 2,490"#/(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 #10×1.5” 55 PHP SMS Screw
Check shear @ post (6005-T5 or 6061-T6)

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

V = 2×38 ksi × 0.1697" × 0.10” × 1
\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 = 2×30 ksi × 0.1409” × 0.07” × 1
\frac{1}{3} (FS)

V_{All} = 2×183 = 366#
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:

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

\[V_{All} = 2 \cdot 183 = 366\#
\]

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

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

\[V_{tot} = 2 \cdot 183# = 366# \geq 200# \text{ okay}
\]
**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 in²
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 ≥ 49 ppi
- Peel strength ≥ 40 psi
- Ult. tension adhesion ≥ 170 psi
- Tensile strength ≥ 48 psi @ 25% elongation
- Tensile strength ≥ 75 psi @ 50% elongation

Moment capacity:
\[ I_x=2.175”^2*2.175”^2+2*2.175^3/12+2*2.175^*(2.175”/2)^2 \]
\[ I_x=17.15in^4/in \]
\[ M_x=49ppi*17.15in^4/in/2.175”=386”# \]

\[ SF = 386”#/127.5”” = 3 > 2.0 \text{ okay} \]

Option #8 Tek screws:
Shear strength = \[ V = 2.38 \text{ ksi} \cdot 0.1309” \cdot 0.07“ \cdot \frac{1}{3} \frac{1}{(FS)} = 232# \]

Added moment capacity = 232#*2.375” = 551”"
Series 200 Top rail

Area: 0.887 sq in

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

6063-T6 Aluminum alloy
For 72” on center posts; \(L = 72”-2.375”-1” \times 2 = 67.625”\)

Calculate lateral torsional buckling strength per ADM F.4.2.1
\[r_{ye} = (0.254^{0.5}/0.874)(0.038*0.001661*67.625^2)^{1/2}/0.557 = 0.557 \text{ in}\]
\[\lambda = 67.625/0.557 = 121.4\]
\[C_c = 78.4 \text{ for 6063-T6}\]
\[F_c/\Omega = 60414/121.4^2 = 4.10 \text{ ksi} \text{ (limiting strength for horizontal loading)}\]

Check for local buckling of top element under vertical loading:
\[b/t = 3.125”/0.094” = 33.24\]
\[F_c/\Omega = 19.17*33.24 = 13.3 \text{ ksi} \text{ (limiting strength for vertical loading)}\]

Allowable Moments
\[\text{Horiz.} = 0.874 \text{ in}^3 \times 4.10 \text{ ksi} = 3.583”^2 = 299”\]
\[\text{Vertical load} = 0.457 \text{ in}^3 \times 13.3 \text{ ksi} = 6,078”^2 \text{ top compression}\]
\[\text{or} \quad 0.421 \text{ in}^3 \times 15.2 \text{ ksi} = 6,399”^2 \text{ controls vertical- bottom tension}\]

Maximum allowable load for 72” o.c. post spacing - vertical
\[W = 3.583”^2/8/(67.625”^2) = 6.268 \text{ pli} = 75.2 \text{ plf}\]
\[P = 3.583”^2/4/67.625” = 212”\]

For horizontal loading:
\[\Delta_{max} = 200*72^3/(48*10^6*1.529 \text{ in}^4) = 0.102”\]
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
$C_{yt}$: 0.545 in
$C_{yb}$: 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; $L = 72”-2.375”-1”\times2 = 67.625”$

Calculate lateral torsional buckling strength per ADM F.4.2.1
$r_{ye} = (0.1235^2/2.43)\times(0.038\times0.0008104\times67.625^2)^{1/2} = 0.737$ in

$\lambda = 67.625”/(.737) = 91.76$
$C_c = 78.4$ for 6063-T6
$F_c/\Omega = 60414/91.76^2 = 7.18$ ksi (limiting strength for horizontal loading)

Check for local buckling of top element under vertical loading:
$b/t = 2.571”/0.074” = 34.74$

$F_c/\Omega = 19.17\times34.74 = 13.1$ ksi (limiting strength for vertical loading)

Allowable Moments → Horiz. = 0.566 in$^3\times7.18$ ksi = 4,064#” = 339#’

Vertical load = 0.243 in$^3\times13.1$ ksi = 3,183#” top compression

or = 0.246 in$^3\times15.2$ ksi = 3,739#” controls vertical- bottom tension

Maximum allowable load for 72” o.c. post spacing - vertical
$W = 3,183"^#/(67.625"^2) = 5.568$ 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
$W = 4,064"^#/(67.625"^2) = 7.11$ pli = 85.3 plf
$P = 4,064"^#/4/67.625" = 240$#

For horizontal loading:
$\Delta_{max} = 200\times72^3/(48\times10^6\times0.8512\text{in}^4) = 0.182”$

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**Series 300 Top Rail**

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

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

$\lambda = (1.073/404)(0+0.038*0.005419*67.625^{1/2})^{1/2} = 0.886$ in  
$\lambda = 67.625”/(0.886”) = 76.33$  
$C_c = 78.4$ for 6063-T6  
$M_p = 0.864\text{in}^3*15.2\text{ksi} = 13,133’’#$  
$M_{mb} = 13,133 *(1-76.33/78.4)+\pi^2*10.1*10^6*76.33*.662/78.4^3 = 10,799’’#$

Check for local buckling of top curved element under vertical loading:  
$R_b/t = 1.5”/0.086” = 17.44 < 31.2$ Local buckling does not control

**Allowable Moments**  
$\rightarrow$ Horiz. $= 10,799’’#/1.65 = 6,545’’#$  
$Vertical = 0.575\text{in}^3*15.2 \text{ksi} = 8,740’’#$ controls vertical- bottom tension

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

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

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

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Top rail 300X

Wall thickness $t = 0.09375''$ min.
Allowable stresses ADM Table 2-24

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

Based on 72” max post spacing

$F_{Cb} = 16.7 - 0.073(59.4) = 12.36 \text{ ksi}$

$M_{\text{all horiz}} = 12.36 \text{ksi} \times (0.656) = 8.111''#$

Vertical loads shared with bottom rail
For vertical load $\rightarrow$ bottom in tension top comp.

$F_b = 18 \text{ ksi}$ line 3
$F_c = 18 \text{ ksi}$ line 16.1

$M_{\text{all vert}} = (0.309\text{in}^4) \times 18 \text{ ksi} = 5.562''#$

Allowable loads

Horizontal $\rightarrow$ uniform $\rightarrow W = \frac{8.111 \times 8}{72^2} = 12.5 \#$/in $= W = 150 \text{ plf}$

$P_H = 4 \times 8.111 = 451 \#$

Vertical $\rightarrow W = \frac{5.562 \times 8}{72^2} = 5.6 \#$/in $= 103 \text{ plf}$ (Top rail alone)

$P = 5.562 \times 4 = 309 \#$

For horizontal loading:
$\Delta_{\text{max}} = 200\times72^3/(48\times10^6\times0.984\text{in}^4) = 0.158''$

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Insert channel for glass – 6063-T6

\[ I_{yy} = 0.156 \text{ in}^4 \quad I_{xx} = 0.023 \text{ in}^4 \]

\[ S_{yy} = 0.125 \text{ in}^3 \quad S_{xx} = 0.049 \text{ in}^4 \]

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:
\[ R/t = 3”/0.09375 = 32 > 5 \quad \text{therefore can assume plane} \]
bending and error will be minimal

\[ M = 2.08”^*W \]

\[ M_{all} = S*F_b \]

\[ F_b = 20 \text{ ksi for flat element bending in own plane,} \]
ADM Table 2-21

\[ S = 12”/\text{ft}^*(0.094)^3/6 = 0.0177 \text{ in}^3 \]

\[ W_{all} = M_{all}/2.08” = (S* F_b)/2.08” = (0.0177 \text{ in}^3*20 \text{ ksi})/2.08” = 170 \text{ plf} \]

For 36” panel height – 1/2 will be tributary to top rail:
Maximum live load = 170 plf/(3’/2) = 113 psf.

Check deflection:

\[ \Delta = WL^3/(3EI) \]

\[ I = 12”^*0.09375^3/12 = .000824 \text{ in}^4 \]

\[ \Delta = 170\text{plf}^*2.08”^3/(3*10.1x10^6*.000824) = 0.06” \]

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
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)

\[ 2 \times F_{\text{rail}} \times \text{dia screw} \times \text{Rail thickness} \times \text{SF} \]
\[ V = 2 \times 30 \, \text{ksi} \times 0.1379'' \times 0.09'' \times \frac{1}{3} = 325\#/\text{screw} \]

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

Post bearing strength
\[ V_{\text{all}} = A_{\text{bearing}} \times F_B \]
\[ A_{\text{bearing}} = 0.09'' \times 2.25'' = 0.2025 \, \text{in}^2 \]
\[ F_B = 21 \, \text{ksi} \]
\[ V_{\text{all}} = 0.2025 \, \text{in}^2 \times 21 \, \text{ksi} = 4.25 \, \text{k} \]

Bracket tab bending strength
Vertical uplift force
For 5052-H32 aluminum stamping 1/8” thick
\[ F_B = 18 \, \text{ksi} – \text{ADM Table 2-09} \]
\[ S = 0.438'' \times (0.125)^3 / 12 = 0.00007 \, \text{in}^3 \]
\[ M_a = 18 \, \text{ksi} \times 0.00007 = 126''\# \]
\[ P_a = M_a / l = 126''\# / 1.158'' = 109# \]
Uplift limited by bracket strength:
\[ U_{\text{p,all}} = 2 \times 109 = 218\# \text{ per bracket} \]
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)

\[ V = 2 \cdot 30 \text{ ksi} \cdot 0.1379'' \cdot 0.09'' \cdot \frac{1}{3} = 325#/\text{screw}; \text{ for two screws = 650#} \]

or \( F_{\text{uplate}} \cdot \text{dia screw} \cdot \text{plate thickness} \cdot \text{SF} \)

\[ V = 38 \text{ ksi} \cdot 0.1379'' \cdot 0.125'' \cdot \frac{1}{3} = 218#/\text{screw}; \text{ for two screws = 436#} \]

Post to splice plate:
Screws into post screw chase so screw to post connection will not control.

splice plate screw shear strength

\[ V = 2 \cdot 38 \text{ ksi} \cdot 0.1379'' \cdot 0.125'' \cdot \frac{1}{3} = 416#/\text{screw}; \text{ for two screws = 832#} \]

Check moment from horizontal load:

\[ M = P \cdot 0.75''. \text{ For 200# maximum load from a single rail on to splice plates} \]

\[ M = 0.75 \cdot 200 = 150##'' \]

\[ S = 0.125\cdot(0.625)^2/6 = 0.008 \text{ in}^3 \]

\[ f_b = 150##''/(0.008\cdot2) = 9,216 \text{ psi} \]

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

Single full width bar may be used instead of the two ½” 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

\[
\begin{align*}
I_{yy} &= 0.156 \text{ in}^4 & I_{xx} &= 0.023 \text{ in}^4 \\
S_{yy} &= 0.125 \text{ in}^3 & S_{xx} &= 0.049 \text{ in}^4
\end{align*}
\]

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
\[
M = 2.08''*W
\]
\[
M_{all} = S*F_b
\]
\[
F_b = 20 \text{ ksi for flat element bending in own plane, ADM Table 2-24}
\]
\[
S = 12''/ft*(0.094)^2/6 = 0.0177 \text{ in}^3
\]
\[
W_{all} = M_{all}/2.08" = (S* F_b)/2.08" = (0.0177 \text{ in}^3*20 \text{ ksi})/2.08" = 170 \text{ plf}
\]

For 36” panel height – 1/2 will be tributary to top rail:
\[
\text{Maximum wind load} = 170 \text{ plf}/(3'./2) = 113 \text{ psf.}
\]

Insert channel for picket infill – 6063-T6

\[
\begin{align*}
I_{yy} &= 0.144 \text{ in}^4 & I_{xx} &= 0.0013 \text{ in}^4 \\
S_{yy} &= 0.115 \text{ in}^3 & S_{xx} &= 0.0057 \text{ in}^4
\end{align*}
\]

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
\[
M = 2.08''*W
\]
\[
M_{all} = S*F_b
\]
\[
F_b = 20 \text{ ksi for flat element bending in own plane, ADM Table 2-24}
\]
\[
S = 12''/ft*(0.094)^2/6 = 0.0177 \text{ in}^3
\]
\[
W_{all} = M_{all}/2.08" = (S* F_b)/2.08" = (0.0177 \text{ in}^3*20 \text{ ksi})/2.08" = 170 \text{ plf}
\]

For 36” panel height – 1/2 will be tributary to top rail:
\[
\text{Maximum live load} = 170 \text{ plf}/(3'./2) = 113 \text{ psf.}
\]
**Top Rail Series 320**

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

Allowable stresses ADM Table 2-21
6063-T6 Aluminum

For 72" on center posts; \( L = 72" - 2.375" - 1" \times 2 = 67.625" \)

Calculate lateral torsional buckling strength per ADM F.4.2.1
\[
r_{ye} = \left( \frac{0.118 - 0.129}{0.129} \right) \times (0 + 0.038 \times 0.00173 \times 67.625^2)^{1/2} = 0.907 \text{ in} \\
\lambda = \frac{67.625^\prime}{(0.907\prime)} = 74.6 \\
C_c = 78.4 \text{ for 6063-T6} \\
M_p = 0.669 \text{ in}^3 \times 25 \text{ksi} = 16,725\# \\
M_{mb} = 16,725\# \times (1 - 74.6/78.4) + \pi^2 \times 10.1 \times 10^6 \times 74.6 \times 0.531/78.4^3 \times 9,005\# \\
\]

Check for local buckling of top curved element under vertical loading:
\[ R_b/t = 3.687\prime / 0.1\prime = 36.87 > 31.2 \text{ Local buckling controls} \]
\[ F_c/\Omega = 18.5 \times 0.593 \times 36.87^{1/2} = 14.90 \text{ ksi} \]

Allowable Moments \( \Rightarrow \) Horiz. = 9,005\# (Inelastic lateral torsional buckling)
Vertical = 0.244\text{in}^3\times 15.2 \text{ ksi} = 3,709\# (Yielding)
Vertical = 0.201\text{in}^3\times 14.9 \text{ ksi} = 2,995\# (Local Buckling)

Maximum allowable load for 72” o.c. post spacing
\[ W = 2,995\# \times 8/(67.625\prime)^2 = 5.24 \text{ pli} = 62.9 \text{ plf} \]
\[ P = 2,995\# \times 4/67.625\prime = 177\# \text{ (Load share with bottom rail needed for 6’ spans)} \]

For horizontal loading:
\[ \Delta_{max} = 200 \times 72^3/(48 \times 10 \times 10^6 \times 0.796 \text{ in}^4) = 0.195" \]

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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
rxy: 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_w=(.263/.5*737)*(0+.038*.0008041*67.625^2)^1/2=0.907 in
λ=67.625”/(.907”) = 74.6
C_w=78.4 for 6063-T6
F_o/Ω = 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_o/Ω = 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:
Δ_max = 200*72^3/(48*10x10^6*1.398in^4) = 0.111”

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**Series 400 Top rail**

$I_{xx}: 0.611 \text{ in}^4$

$I_{yy}: 3.736 \text{ in}^4$

$r_{xx}: 0.717 \text{ in}$

$r_{yy}: 1.774 \text{ in}$

$C_{xx}: 1.358 \text{ in}$

$C_{yy}: 2.50 \text{ in}$

$S_{xx}: 0.450 \text{ in}^3$ bottom

$S_{xx}: 0.399 \text{ in}^3$ top

$S_{yy}: 1.494 \text{ in}^3$

6063-T6 Aluminum alloy

For 72” on center posts;

$L = 72” - 2.375” - 1” \times 2 = 67.625”$

Calculate lateral torsional buckling strength per ADM F.4.2.1

$r_{ye} = ((.611^{1/2}/1.494)*(.038*0.00219*67.625^{1/2})^{1/2}) = 0.568 \text{ in}$

$\lambda = 67.625” / (.568”) = 119$

$C_c = 78.4$ for 6063-T6

$F_c / \Omega = 60414/119^2 = 4.266 \text{ ksi (limiting strength for horizontal loading)}$

Check for local buckling of top curved element under vertical loading:

$R_{b/t} = 12”/0.087” = 138 > 31.2$ Local buckling controls

$F_c / \Omega = 18.5 - 0.593*119^{1/2} = 12.03 \text{ ksi}$

Allowable Moments ➔ Horiz. = 1.494 in$^3*4.266 \text{ ksi} = 6.373”#$

Vertical = 0.399 in$^3*12.03 \text{ ksi} = 4.800”#$

Vertical = 0.772 in$^3*15.2 \text{ ksi} = 11.734”#$

Maximum allowable load for 72” o.c. post spacing

$W = 4,800” #/8/(67.625”^2) = 8.40 \text{ plf} = 101 \text{ plf}$

$P = 4,800” #/4/67.625” = 284#$

For horizontal loading:

$\Delta_{max} = 200*72^3/(48*10^6*3.736 \text{ in}^4) = 0.042”$

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

I\textsubscript{xx}: 0.0138 in\textsuperscript{4};  I\textsubscript{yy}: 0.265 in\textsuperscript{4}
C\textsubscript{xx}: 0.573 in;  C\textsubscript{yy}: 1.344 in
S\textsubscript{xx}: 0.024 in\textsuperscript{3};  S\textsubscript{yy}: 0.197 in\textsuperscript{3}

Wood
2”x4” nominal
I\textsubscript{xx}: 0.984 in\textsuperscript{4};  I\textsubscript{yy}: 5.359 in\textsuperscript{4}
C\textsubscript{xx}: 0.75 in;  C\textsubscript{yy}: 1.75 in
S\textsubscript{xx}: 1.313 in\textsuperscript{3};  S\textsubscript{yy}: 3.063 in\textsuperscript{3}

For wood use allowable stress from NDS Table 4A for lowest strength wood that may be used:
F\textsubscript{b} = 725 psi (mixed maple #1), C\textsubscript{D} = 1.6, C\textsubscript{F} = 1.5
F’\textsubscript{b} = 725*1.6*1.5 = 1,740 psi
F’\textsubscript{b} = 725*1.6*1.5*1.1 = 1,914 psi for flat use (vertical loading)

Composite action between aluminum and wood:
n = E\textsubscript{a}/E\textsubscript{w} = 10.1/1.1 = 9.18

Composite Shape Section Properties
Effective properties adjusted for E=10.1*10\textsuperscript{3}ksi
I\textsubscript{xx}=0.2763in\textsuperscript{4}  I\textsubscript{yy}=0.8484in\textsuperscript{4}

Allowable Stress for aluminum: ADM Table 2-21
F\textsubscript{T} = 15.2 ksi
F\textsubscript{C} → 6’ span
Rail is braced by wood At 16” o.c. and legs have stiffeners therefore
F\textsubscript{C} = 15.2 ksi

Vertical loading: M\textsubscript{a,x}=1.914psi*0.2763in\textsuperscript{4}/1.0427”*9.18=4,656”# (Wood failure)
M\textsubscript{a,x}=15.2ksi*0.2763in\textsuperscript{4}/1.2073”=3,479”# (Aluminum failure controls)

Horizontal loading: M\textsubscript{a,y}=1.740psi*0.8484in\textsuperscript{4}/1.75”*9.18=7,744”# (Wood failure controls)
M\textsubscript{a,y}=15.2ksi*0.8484in\textsuperscript{4}/1.3434”=9,599”# (Aluminum failure)

Maximum allowable load for 72” o.c. post spacing
W = 3,479”#*8/(67.625”\textsuperscript{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.

\[ V = \frac{200\#}{2} = 100\# \text{ (Midspan 200# concentrated load)} \]

\[ v = \frac{VQ}{I} \]

\[ Q = YA' = 0.6338'' \times 0.26406in^2 = 0.1674in^3 \]

\[ v = \frac{100\# \times 0.1674in^3}{0.2763in^4} = 60.59\text{pli} = 727.0\text{plf} \]

Use #6 Wood Screws (Larger screws do not appreciably increase shear strength due to limited penetration and will increase probability of splitting)

\[ Z' = 1.6 \times 76\# = 122\# \text{ each} \]

Aluminum bearing = \[ 2 \times 0.138'' \times 0.062'' \times 30\text{ksi}/3 = 171\# \]

Screw spacing to create composite bending at service loading = \[ \frac{122\#/60.59\text{pli}}{2} \Rightarrow 2'' \text{ O.C staggered} \]

Adhesive strength to create composite bending in lieu of screws = \[ \frac{60.59\text{pli}}{2.6875''} = 22.5\text{psi} \]

**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, $R_b/t = 12.5''/0.086'' = 145$  
$F_c/\Omega = 18.5 - 0.593 \times 145^{1/2} = 11.4$ksi

Calculate lateral torsional buckling strength per ADM F.4.2.1  
$r_{ye} = ((.262/1.283)^2(0+.038\times.001801\times67.625)^{1/2})^{1/2}=0.472 \text{ in}$  
$\lambda = 67.625''/(.472'') = 143$  
$C_c=78.4 \text{ for 6063-T6}$  
$F_c/\Omega = 60414/143^2 = 2.95$ksi (limiting strength for horizontal loading)

Allowable Moments ➔ Horiz. = 1.283in$^3 \times 2.95$ ksi = 3,785"#  
Vertical = 0.221in$^3 \times 11.4$ ksi = 2,519"#

Maximum allowable load for 72" o.c. post spacing  
$W = 2.519''/8/(67.625''^2) = 4.41 \text{ pli = 52.9 plf}$  
$P = 2.519''/4/67.625'' = 150$# (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

b/t = 1.397”/0.07” = 19.96
Fc/Ω = 155/19.96 = 7.77 ksi

Allowable Moments ➔ Horiz. = 0.204in³*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 Fupost x dia screw x Post thickness x SF
V = 2×38 ksi × 0.1697” × 0.10” × 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

2×30ksi×0.1309”×0.07” × 1 = 232#/screw
3

Allowable tension = 2×232 = 464#
OK
**Picket bottom rail**

Bottom rail strength
6063-T6 Aluminum alloy
For 72” on center posts; \( L = 72" - 2.375" - 1" \times 2 = 67.625" \)

Calculate lateral torsional buckling strength per ADM F.4.2.1
\[
J = 0.001752 \text{in}^4
\]
\[
r_y = (0.125/1.522) * (0+.038*.
\]
\[
0.01752 \times 67.625^{3/2} = 0.927 \text{ in}
\]
\[
\lambda = 67.625"/(.927") = 73.0
\]

\( C_c = 78.4 \) for 6063-T6
\[
F_c/\Omega = 15.2 \text{ksi}
\]

Check local buckling of vertical legs:
\[
b/t = 1.5"/0.07" = 21.4 > 12.6
\]
\[
F_c/\Omega = 155/21.4 = 7.24 \text{ksi}
\]

Allowable Moments → Horiz. = 0.227 in^3 \times 7.24 ksi = 1,643”#
Vertical = 0.108 in^3 \times 15.2 ksi = 1,642”#

Rail fasteners - Bottom rail connection block to post

#10 \times 1.5”  55 PHP SMS Screw
Check shear @ post (6005-T5)
2x \( F_{u_{\text{pos}}} \times \text{dia screw} \times \text{Post thickness} \times \text{SF} \)
Eq 5.4.3-2
\[
V = 38 \text{ ksi} \times 0.19" \times 0.1" \times \frac{1}{3} = 240\#/{\text{screw}}
\]
\[
V = 240\#/\text{screw}
\]

Since minimum of 2 screws used for each
Allowable load = 2 \times 240# = 480#
Rail Connection to RCB
2 screws each end
#8 Tek screw to 6063-T6
ADM Eq. 5.4.3-1
\[
2*30ksi*0.1248"*0.07"* 1/3 = 175#/\text{screw}
\]

Allowable shear = 2 \times 175 = 350#
OK

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MID RAIL

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

Allowable stresses ADM Table 2-21 6063-T6 Aluminum
For vertical loads:
\[ F_{Cb} \rightarrow R_0/t = 1.75''/0.080'' = 21.6 \]
\[ F_c/\Omega = 15.2 \text{ksi} \]
\[ M_a = 0.1916 \text{in}^3 \times 15.2 \text{ksi} = 2,912''# \]

For horizontal loads:
\[ b/t = 0.8667''/0.0625'' = 13.9 \]
\[ F_c/\Omega = 15.2 \text{ksi} \]
\[ M_a = 0.2397 \text{in}^3 \times 15.2 \text{ksi} = 3,643''# \]

Allowable vertical loading:
Distributed load = 2,912''#*8/72 = 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''2 = 202#
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:
\[ \frac{I_{yy \text{ infill}}}{I_{yy \text{ net}}} = \frac{0.0378}{0.0378+0.172} = 0.18 \]
\[ 0.046/0.18=0.256 \text{ or } 0.204/(1-.18)=0.249<0.256 \text{ so standard bottom rail controls} \]

Allowable Moments \( \text{- Horiz.} = \frac{1.585”#/(1-.18)}{1.933”#} \)
Maximum allowable load for 70” screen width \( L = 70”-1”\times2-2.375\times2 = 63.25” \)
\[ W = 1.933”#\times8/(63.25”^2) = 3.87 \text{ pli} = 46.39 \text{ plf} \]
\[ P = 1.933”#\times4/63.25” = 122” \]

Maximum allowable load for 60” screen width \( L = 60”-1”\times2-2.375\times2 = 53.25” \)
\[ W = 1.933”#\times8/(53.25”^2) = 5.45 \text{ pli} = 65.4 \text{ plf} \]
\[ P = 1.933”#\times4/53.25” = 145” \]
**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)

\[ F_{\text{upost}} = \text{dia screw} \times \text{Post thickness} / \text{SF} \]

Eq 5.4.3-2

\[ V = 38 \text{ ksi} \times 0.19” \times 0.1” \times \frac{1}{3} \text{ (SF)} \]

Since minimum of 2 screws used for each, Allowable load = 2 \times 240# = 480#

For 4”x4” post:

\[ V = 38 \text{ ksi} \times 0.19” \times 0.15” \times \frac{1}{3} \text{ (SF)} \]

Since minimum of 2 screws used for each, Allowable load = 2 \times 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

\[ Z’ = n \times C_d \times Z = 2 \text{ screws} \times 1.6 \times 140# = 448# \]

For connection to cold formed steel stud - 22 ga min based on CCFSS T.B. V2#1

\[ Z = 2 \times 175# = 350# \]

For connection to concrete or CMU - (2) 3/16” x 2” Tapcon screws

\[ Z = 2 \times 290 = 580# \]

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

2x $F_{upost} \times \text{dia screw x Cap thickness x SF}$
Eq 5.4.3-2
$V = 2 \times 38 \, \text{ksi} \times 0.19” \times 0.15” \times \frac{1}{3} = \frac{722}{(FS)}$
$722#/$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 $G \geq 0.43$
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>
<thead>
<tr>
<th>Steel Thickness - Thinnest Component</th>
<th>1/4-14 Screw</th>
<th>#12-14 Screw</th>
<th>#10-18 Screw</th>
<th>#8-18 Screw *</th>
<th>#6 Screw *</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear</td>
<td>Pullout</td>
<td>Shear</td>
<td>Pullout</td>
<td>Shear</td>
</tr>
<tr>
<td>0.1017”</td>
<td>1000</td>
<td>320</td>
<td>890</td>
<td>280</td>
<td>780</td>
</tr>
<tr>
<td>0.0713”</td>
<td>600</td>
<td>225</td>
<td>555</td>
<td>195</td>
<td>520</td>
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<td>0.0451”</td>
<td>300</td>
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<td>280</td>
<td>120</td>
<td>280</td>
</tr>
<tr>
<td>0.0347”</td>
<td>200</td>
<td>110</td>
<td>185</td>
<td>95</td>
<td>175</td>
</tr>
</tbody>
</table>

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 $F_y = 33ksi, F_u = 45ksi$ minimum. Adjust values for other steel strengths.
3. * = Refer to Table 1 for limits on recommended total steel thickness of connected parts.
Wall Mounted End Caps – Cont.

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

<table>
<thead>
<tr>
<th>NOMINAL ANCHOR DIAMETER (inch)</th>
<th>EFFECTIVE EMBEDMENT DEPTH (Inches)</th>
<th>ALLOWABLE LOADS (pounds)</th>
<th>SHEAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/16</td>
<td>1.5</td>
<td>260 265 330 370 290</td>
<td></td>
</tr>
<tr>
<td>1/4</td>
<td>1.5</td>
<td>350 385 445 495 525</td>
<td></td>
</tr>
</tbody>
</table>

For Sf: 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 $\alpha = 0.3*1.2 + 0.7*1.6 = 1.49$.
7. $C_{af} = C_{ef} > C_{cf}$.
8. $h > h_{min}$.
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.
Excerpts from National Design Specifications For Wood Construction

**Table 11.2A  Lag Screw Reference Withdrawal Design Values, W**

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).

<table>
<thead>
<tr>
<th>Specific Gravity, G</th>
<th>1/4&quot;</th>
<th>5/16&quot;</th>
<th>3/8&quot;</th>
<th>7/16&quot;</th>
<th>1/2&quot;</th>
<th>5/8&quot;</th>
<th>3/4&quot;</th>
<th>1&quot;</th>
<th>1-1/8&quot;</th>
<th>1-1/4&quot;</th>
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</thead>
<tbody>
<tr>
<td>0.73</td>
<td>397</td>
<td>469</td>
<td>538</td>
<td>604</td>
<td>668</td>
<td>789</td>
<td>905</td>
<td>1016</td>
<td>1123</td>
<td>1226</td>
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<tr>
<td>0.71</td>
<td>381</td>
<td>450</td>
<td>516</td>
<td>579</td>
<td>640</td>
<td>757</td>
<td>868</td>
<td>974</td>
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<td>1176</td>
</tr>
<tr>
<td>0.68</td>
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<td>422</td>
<td>484</td>
<td>543</td>
<td>600</td>
<td>709</td>
<td>813</td>
<td>913</td>
<td>1009</td>
<td>1103</td>
</tr>
<tr>
<td>0.67</td>
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<td>413</td>
<td>473</td>
<td>531</td>
<td>597</td>
<td>694</td>
<td>796</td>
<td>893</td>
<td>987</td>
<td>1078</td>
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<td>0.58</td>
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<td>332</td>
<td>381</td>
<td>428</td>
<td>473</td>
<td>559</td>
<td>641</td>
<td>719</td>
<td>795</td>
<td>869</td>
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<td>0.55</td>
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<td>395</td>
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<td>734</td>
<td>802</td>
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<td>314</td>
<td>353</td>
<td>390</td>
<td>461</td>
<td>528</td>
<td>593</td>
<td>656</td>
<td>716</td>
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<td>305</td>
<td>342</td>
<td>378</td>
<td>447</td>
<td>513</td>
<td>576</td>
<td>636</td>
<td>695</td>
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<td>296</td>
<td>332</td>
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<td>434</td>
<td>498</td>
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<td>312</td>
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<td>302</td>
<td>334</td>
<td>395</td>
<td>453</td>
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<td>395</td>
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<td>535</td>
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<td>308</td>
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<td>397</td>
<td>438</td>
<td>479</td>
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<tr>
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<td>176</td>
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<td>227</td>
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<td>326</td>
<td>367</td>
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<td>443</td>
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<td>200</td>
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<td>337</td>
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<td>167</td>
<td>185</td>
<td>218</td>
<td>250</td>
<td>281</td>
<td>311</td>
<td>339</td>
</tr>
</tbody>
</table>

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.
### Table 11K LAG SCREWS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections1,2,3,4

for sawn lumber or SCL with ASTM A653, Grade 33 steel side plate (for t < 1/4") or ASTM A36 steel side plate (for t = 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)

<table>
<thead>
<tr>
<th>t₀ in</th>
<th>1/4 D</th>
<th>1/2 D</th>
<th>3/4 D</th>
<th>1 D</th>
<th>1 1/2 D</th>
<th>2 D</th>
<th>3 D</th>
<th>4 D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.075</td>
<td>85.4</td>
<td>128.5</td>
<td>192.9</td>
<td>295.7</td>
<td>384.0</td>
<td>476.3</td>
<td>568.6</td>
<td>660.9</td>
</tr>
<tr>
<td>0.105</td>
<td>108.0</td>
<td>162.0</td>
<td>243.0</td>
<td>349.5</td>
<td>445.5</td>
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<td>207.5</td>
<td>306.8</td>
<td>411.7</td>
<td>516.2</td>
<td>620.4</td>
<td>724.3</td>
<td>828.1</td>
</tr>
</tbody>
</table>

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 solid grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 8D, (doubled bearing strengths, F₀ of 61,850 psi for ASTM A653, Grade 33 steel and 87,000 psi for ASTM A36 steel and screw bending yield strength, Fₚ₉₀ of 70,000 psi for D = 1/4", 60,000 psi for D = 5/16", and 45,000 psi for D = 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 screws into the main member shall not be less than 4D. See 11.1.4.6 for minimum length of penetration, pₘᵢ₇ኦ.

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<table>
<thead>
<tr>
<th>Screw Diameter</th>
<th>Wood Screw</th>
<th>Gr. 40-7 Red Oak</th>
<th>Gr. 50-6 Mixed Pine</th>
<th>Gr. 50-6 Mixed Fir</th>
<th>Gr. 50-6 Doug-Fir (L)</th>
<th>Gr. 40-6 Doug-Fir (S)</th>
<th>Gr. 40-6 Hpm-Fir (L)</th>
<th>Gr. 40-6 Hpm-Fir (S)</th>
<th>Gr. 30 Redwood (Grades)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.036 in. (20 gage)</td>
<td>0.138</td>
<td>69</td>
<td>76</td>
<td>70</td>
<td>69</td>
<td>68</td>
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<td>0.038 in. (10 gage)</td>
<td>0.164</td>
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<tr>
<td>0.040 in. (10 gage)</td>
<td>0.184</td>
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<td>110</td>
<td>100</td>
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<td>0.045 in. (12 gage)</td>
<td>0.177</td>
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<td>0.063 in. (11 gage)</td>
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<td>116</td>
<td>123</td>
<td>118</td>
<td>111</td>
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<td>113</td>
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<tr>
<td>0.076 in. (10 gage)</td>
<td>0.177</td>
<td>131</td>
<td>138</td>
<td>132</td>
<td>126</td>
<td>124</td>
<td>126</td>
<td>126</td>
<td>128</td>
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<tr>
<td>0.120 in. (9 gage)</td>
<td>0.138</td>
<td>116</td>
<td>123</td>
<td>118</td>
<td>111</td>
<td>109</td>
<td>111</td>
<td>111</td>
<td>113</td>
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<tr>
<td>0.083 in. (7 gage)</td>
<td>0.131</td>
<td>129</td>
<td>136</td>
<td>130</td>
<td>126</td>
<td>124</td>
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<tr>
<td>0.179 in. (6 gage)</td>
<td>0.131</td>
<td>129</td>
<td>136</td>
<td>130</td>
<td>126</td>
<td>124</td>
<td>126</td>
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<tr>
<td>0.229 in. (3 gage)</td>
<td>0.131</td>
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<td>130</td>
<td>126</td>
<td>124</td>
<td>126</td>
<td>126</td>
<td>128</td>
</tr>
</tbody>
</table>

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 field-threaded wood screws (see Appendix L) inserted in sound grain with screw axis perpendicular to wood fibers; screw penetration, p, into the main member equal to 1.0D; dowel bearing strength, F, of 61,830 psi for ASTM A653, Grade 33 steel and screw bending yield strength, F_y, of 0.070 psi for p ≤ 0.85" or 0.125" D ≤ 0.25" D ≤ 0.177" or 0.001 for D > 0.25" and 0.000 psi for F_y < 0.177 psi. Where p ≤ 0.85" or D ≤ 0.177", the reduced penetration, p, shall be calculated using the provisions of 11.3.1 for the reduced penetration.
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 11.3.1 for the reduced penetration.

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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
  \( M_i = \frac{25 \text{ psf} \times H}{2} \times S \times H = \frac{(25/2) \times S \times H^2}{2} \)

For top rail loads:
  \( M_c = 200\# \times H \)
  \( M_u = 50\text{ plf} \times S \times H \)

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:
  \( 42” \times 2.375” + 3” \times 60” + 1.7” \times 57.625” \)
  \( + 0.75 \times 36 \times 18 = 863.7 \text{ in}^2 \)
  % surface/area = \( \frac{863.7}{(60” \times 42”)} = 34.3\% \)
  Wind load for 25 psf equivalent load = \( \frac{25}{0.343} = 72.9 \text{ psf} \)

EDWARD C. ROBISON, PE
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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

\[
\text{Loading } \rightarrow 25 \text{ psf } \rightarrow 4 \\
\frac{1}{2}'' \text{ O.C } \rightarrow 25 \text{ psf } \cdot 0.375 = 9.4 \text{ plf}
\]

\[
M = \frac{9.4}{12} \left(\frac{42''-6''}{2}\right) = \frac{127}{8} \text{ lb-in}
\]

**For 5/8'' Square pickets**

\[
t = 0.062'' \rightarrow S = 0.625 \cdot \frac{3}{6} - 0.5 \cdot \frac{3}{6} = 0.020 \text{ in}^3
\]

\[
f_s = \frac{127}{0.02} \text{ lb-in} = 6,350 \text{ psi}
\]

For 50 lb conc load \( \Rightarrow 1 \text{ SF} \) - min 2 pickets

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

\[
f_s = \frac{225}{0.02} \text{ lb-in} = 11,250 \text{ psi}
\]

\[
b/t = \frac{0.5''}{0.0625''} = 8 < 22.8
\]

6063-T6 \( F_c/\Omega = 15.2 \text{ ksi} \) – compression ADM Table 2-21

Maximum allowable moment on picket = 15.2 ksi \( \cdot 0.02 \text{ in}^3 = 304 \text{ in-lb} \)

Maximum span = 304 in-lb \( \cdot 4/25 \text{ lb} = 48.6'' \) – concentrated load or

\( (304 \text{ in-lb} \cdot 8/0.783 \text{ lb/in})^{1/2} = 55.73 \text{ in} \) (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'' \( \cdot 0.062'' = 0.085 \text{ in}^2 \)

Allowable bearing = 0.085 in\(^2\) \( \cdot 21 \text{ ksi} = 1,785# \)

Withdrawal prevented by depth into rails.

EDWARD C. ROBISON, PE
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PICKETS 3/4” ROUND

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

R_b/t = 0.75”/.0625” = 12 < 31.2
F_c/\Omega = 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 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  \quad ADM Eq 5.4.3-2
V = 38 ksi \cdot 0.19” \cdot 0.1” \cdot \frac{1}{3 (FS)} = 240#
PICKETS 3/4” SQUARE

\[ Z_x = 0.0685 \text{in}^3 \]
\[ b/t = 0.55”/0.1” = 5.5 \]

Allowable moment,
\[ M_a = 0.0685 \text{in}^3 \times 15.2 \text{ksi} = 1,041”# \]

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”# \times 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 =
\[ 2 \times F_{\text{upost}} \times \text{dia screw} \times t_{\text{rail}} \times \text{SF} \quad \text{ADM Eq 5.4.3-2} \]
\[ V = 30 \text{ ksi} \times 0.19” \times 0.1” \times \frac{1}{3} \quad \text{(FS)} = 190# \]

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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:
\[ P_{all} = \frac{390”#}{1.8”} = 217# \]
\[ S_{all} = \frac{217#}{50 plf} = 4'4” \]
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_Dt/3 = 2*30ksi*0.164”*0.125”/3*2 screws = 820# (ADM 5.4.3)
Tension – 1.2DtF_y/3 = 1.2*.164”*0.125”*25ksi*2 screws/3 = 410#

For residential installations only 200# concentrated load is applicable.
Connection to support:

Maximum tension occurs for outward

Horizontal force = 200#:
Determine tension from $\sum M$ about C
0 = P*5” – T*1.25”
T = 200#*(5-1.25)”/1.25” = 600#
From $\sum$ forces – no shear force in anchor
occurs from horizontal load

Vertical force = 200#+17# (DL):
Determine tension from $\sum M$ about C
0 = P*2.5” – T*1.25”
T = 217#*2.5”/1.25” = 434#
From $\sum$ 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
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
\( P_t = 0.58 A_{sn} F_{tu}(t_c)/2.34 = \)
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
**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 in²
- I = 0.129 in⁴
- S = 0.172 in³
- Z = 0.237 in³

Allowable stresses from ADM Table 2-21

- \( R_b/t = 0.625/0.125 = 5 < 70; \)
- \( F_c/\Omega = 27.7 - 1.70 \times 5^{1/2} = 23.90 \text{ ksi} \), Use 22.7 ksi max
- \( M_a = Z \times F_y = 0.237 \times 22.7 \text{ ksi} = 5380'\# = 448.3'\# \)

**Allowable Span:**

Check based on simple span and cantilevered section.

\[
M = w(lg)^2/8 \quad \text{or} \quad P(lg)/4 \quad \text{Solve for lg:}
\]

- \( lg = (8M/w)^{1/2} = [8 \times (448.3'\#/50 \text{plf})]^{1/2} = 8.47' \) or
- \( lg = (4M/P) = 4 \times 448.3'\#/200'\# = 8.97' \)

Maximum allowable span for supports at both ends=8’-5 5/8”-Controlling span

For cantilevered section

\[
M = w(lc)^2/2 \quad \text{or} \quad P(lc) \quad \text{Solving for lc}
\]

- \( lc = (2M/w)^{1/2} = (2 \times 448.3'\#/50 \text{plf})^{1/2} = 4.23' \) or
- \( lc = M/P = 448.3'\#/200'\# = 2.24' = 2' -2 7/8” ----- Controlling span

Locate splice within \( lc \) of a support.
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 in²
I = 0.129 in⁴
S = 0.172 in³
Z = 0.236 in³ minimum
r = 0.488 in, J = 0.255 in⁴

Stainless steel tube in accordance with ASTM A554-10
Rail Service Loading:
Brushed stainless steel, F_y ≥ 45 ksi, F_u ≥ 91 ksi (Requires Mill Certification Tests)
\( \phi M_n = 0.9 \times 1.25 \times S \times F_y = 0.9 \times 1.25 \times 0.172 \times 45 \) ksi
\( \phi M_n = 8,707.5” \#
\( M_l = \phi M_n / 1.6 = 5,442.2” \# = 453.52’ \#

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

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

For cantilevered section
\[ M = w(lc)^2/2 \text{ or } P(lc) \]
Solving for \( lc \)
\[ lc = (2M/w)^{1/2} = (2 \times 453.52’/50 \text{ plf})^{1/2} = 4.259’ \text{ or } \]
\[ lc = M/P = 453.52’/200 = 2.268’ = 2’ - 3 3/16” \text{ ----- Controlling span } \]

Locate splice within \( lc \) of a support.
STAINLESS STEEL CABLE IN-FILL:

S: MAX. 6 FT. O.C. SPACING POSTS

MAX. 3 FT. O.C. SPACING VERTICAL SPACER

NOTE: SEE SEPARATE TOP RAIL CALCS

PUSH LOCK FITTING

DECK / FLOOR SURFACE

NOTE: SEE SEPARATE BOTTOM RAIL CALCS

NOTE: SEE SEPARATE POST CALCULATIONS

Cable railing- Deflection/ Preload/ Loading relationship

Cable Strain = \[\varepsilon = \frac{C_{\text{ta}} \cdot L}{A \cdot E}\]

\[C_t = C_{\text{ti}} + C_{\text{ta}}\]

\[C_{\text{ta}} = \varepsilon EA = \text{Cable tension increase from loading}\]

\[L\]

From cable theory

\[C_t = \frac{l \cdot P}{4\Delta}\]

for concentrated load

To calculate allowable load for a given deflection:

Calculate \[\varepsilon = \left[\left(\frac{l}{2}\right)^2 + \Delta^2\right]^{1/2} \cdot 2 - l\]

Then calculate \[C_{\text{ta}} = \frac{\varepsilon AE}{L}\]

Then calculate \[C_t = C_{\text{ti}} + C_{\text{ta}}\]

Then calculate load to give the assumed \(\Delta\) for concentrated load

\[P = \frac{C_{\text{ta}} 4\Delta}{l}\]

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For uniform load – idealize deflection as triangular applying cable theory

\[ C_t = \frac{WF}{8A} \]

Solving for \( W = \frac{C_t 8A}{l^2} \)

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 = \( \frac{25 \text{ psf} \times 3”}{12”} = 6.25 \text{ plf} \)
-Concentrated load 1 sf
  3 cables minimum
  \( 50/3 = 16.7 \text{ lbs on 4” sphere} \)

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 \( \Delta_{all} = 4” - (3 - 1/8) = 1 1/8” \)
for 3/16” cable \( \Delta_{all} = 4” - (3 - 3/16) = 1 3/16” \)

Cable Strain:

\[ \varepsilon = \frac{\sigma}{E} \text{ and } \Delta_L = L \varepsilon \]

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

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.
Cable railing
Cable deflection calculations

<table>
<thead>
<tr>
<th>Cable = 1/8&quot; dia (area in^2)</th>
<th>0.0123</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity (E, psi)</td>
<td>26000000</td>
</tr>
</tbody>
</table>

Cable strain = Ct/(A*E) *L(in) = additional strain from imposed loading

<table>
<thead>
<tr>
<th>Cable installation load (lbs)</th>
<th>150</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Cable length (ft)</td>
<td>36</td>
</tr>
<tr>
<td>Cable free span (inches)</td>
<td>35</td>
</tr>
</tbody>
</table>

Calculate strain for a given displacement (one span)

<table>
<thead>
<tr>
<th>delta (in)</th>
<th>strain (in)</th>
<th>Ct net (lb)</th>
<th>Ct tot (lbs)</th>
<th>Conc. Load (lb)</th>
<th>Uniform ld (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.00357</td>
<td>2.6</td>
<td>152.6</td>
<td>4.4</td>
<td>3.0</td>
</tr>
<tr>
<td>0.375</td>
<td>0.00803</td>
<td>5.9</td>
<td>155.9</td>
<td>6.7</td>
<td>4.6</td>
</tr>
<tr>
<td>0.55</td>
<td>0.01728</td>
<td>12.8</td>
<td>162.8</td>
<td>10.2</td>
<td>7.0</td>
</tr>
<tr>
<td>0.75</td>
<td>0.03213</td>
<td>23.7</td>
<td>173.7</td>
<td>14.9</td>
<td>10.2</td>
</tr>
<tr>
<td>1</td>
<td>0.05710</td>
<td>42.2</td>
<td>192.2</td>
<td>22.0</td>
<td>15.1</td>
</tr>
<tr>
<td>2</td>
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<td>168.3</td>
<td>318.3</td>
<td>72.7</td>
<td>49.9</td>
</tr>
<tr>
<td>2.5</td>
<td>0.35534</td>
<td>262.4</td>
<td>412.4</td>
<td>117.8</td>
<td>80.8</td>
</tr>
<tr>
<td>3.13</td>
<td>0.55542</td>
<td>410.2</td>
<td>560.2</td>
<td>200.4</td>
<td>137.4</td>
</tr>
</tbody>
</table>

EDWARD C. ROBISON, PE
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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 $6 \times 325 = 1,950$#

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 $7 \times 205 \times 1.25 = 1,784$#

End post Cable loading

Cable tension - 200#/ Cable no in-fill load

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

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.

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

EDWARD C. ROBISON, PE
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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

\[ M_{\text{conc}} = 220.7 \times 15''/2 + 220.7 \times 18'' + (3 \times (200 - 7.7 \times 3)) + (6 \times (200 - 7.7 \times 6)) + (9 \times (200 - 7.7 \times 9)) + 12 \times (200 - 7.7 \times 12) + 15 \times (200 - 7.7 \times 15)/2 \]
\[ M_{\text{conc}} = 10,183# \]

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
\[ \Delta = \left[ \frac{P a^2 b^2}{3LEI} \right] = \left[ \frac{599.2 \times 18^2 \times 21^2}{3 \times 39 \times 10100000 \times 0.863} \right] = 0.084 \]

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.

\[ M_{200} = 599.2 \times 18'' + (3 \times (200 - 7.6 \times 3)) + (6 \times (200 - 7.6 \times 6)) + (9 \times (200 - 7.6 \times 9)) + (12 \times (200 - 7.6 \times 12)) + (15 \times (200 - 7.6 \times 15)) + (18 \times (200 - 7.6 \times 18))/2 = 11,200# \]

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:
\[ F_y = 110 \text{ ksi} \]
Minimum tension strength = 1,869# for ⅛” 1x19 cable
\[ \phi T_s = 0.85 \times 110 \text{ ksi} \times 0.0123 = 1,150# \]
\[ T_s = \phi T_n/1.6 = 1,150#/1.6 = 718# \]

Maximum cable pretension based on maximum service tension @ 200# cable load is 440#:

<table>
<thead>
<tr>
<th>( \Delta ) (in)</th>
<th>strain (in)</th>
<th>Ct net (lb)</th>
<th>Ct tot (lbs)</th>
<th>Conc. Load (lb)</th>
<th>Uniform ld (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.19</td>
<td>0.00206</td>
<td>1.7</td>
<td>441.7</td>
<td>9.6</td>
<td>6.6</td>
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<tr>
<td>0.33</td>
<td>0.00622</td>
<td>5.1</td>
<td>445.1</td>
<td>16.8</td>
<td>11.5</td>
</tr>
<tr>
<td>2.437</td>
<td>0.33774</td>
<td>278.2</td>
<td>718.2</td>
<td>200.0</td>
<td>137.2</td>
</tr>
</tbody>
</table>

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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#

<table>
<thead>
<tr>
<th>Cable railing</th>
<th>Cable deflection calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cable = 1/8” dia (area in^2) = 0.0123</td>
</tr>
<tr>
<td></td>
<td>Modulus of elasticity (E, psi) = 26000000</td>
</tr>
</tbody>
</table>

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)

<table>
<thead>
<tr>
<th>delta (in)</th>
<th>strain (in)</th>
<th>Ct net (lb)</th>
<th>Ct tot (lbs)</th>
<th>Conc. Load (lb)</th>
<th>Uniform Id (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.00298</td>
<td>2.2</td>
<td>375.2</td>
<td>8.9</td>
<td>5.1</td>
</tr>
<tr>
<td>0.375</td>
<td>0.00670</td>
<td>4.9</td>
<td>377.9</td>
<td>13.5</td>
<td>7.7</td>
</tr>
<tr>
<td>0.55</td>
<td>0.01440</td>
<td>10.6</td>
<td>383.6</td>
<td>20.1</td>
<td>11.5</td>
</tr>
<tr>
<td>0.75</td>
<td>0.02678</td>
<td>19.8</td>
<td>392.8</td>
<td>28.1</td>
<td>16.0</td>
</tr>
<tr>
<td>1</td>
<td>0.04759</td>
<td>35.2</td>
<td>408.2</td>
<td>38.9</td>
<td>22.2</td>
</tr>
<tr>
<td>2</td>
<td>0.19005</td>
<td>140.4</td>
<td>513.4</td>
<td>97.8</td>
<td>55.9</td>
</tr>
<tr>
<td>2.5</td>
<td>0.29657</td>
<td>219.0</td>
<td>592.0</td>
<td>141.0</td>
<td>80.6</td>
</tr>
<tr>
<td>3.03</td>
<td>0.43493</td>
<td>321.2</td>
<td>694.2</td>
<td>200.3</td>
<td>114.5</td>
</tr>
</tbody>
</table>

EDWARD C. ROBISON, PE
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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).

<table>
<thead>
<tr>
<th>Cable deflection calculations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable = 1/8” dia (area in^2) = 0.0123</td>
</tr>
<tr>
<td>Modulus of elasticity (E, psi) = 26000000</td>
</tr>
<tr>
<td>Cable strain = Ct/(A*E) *L(in) = additional strain from imposed loading</td>
</tr>
<tr>
<td>Cable installation load (lbs) = 349</td>
</tr>
<tr>
<td>Total Cable length (ft) = 60</td>
</tr>
<tr>
<td>Cable free span (inches) = 35</td>
</tr>
</tbody>
</table>

Calculate strain for a given displacement (one span)

<table>
<thead>
<tr>
<th>delta (in)</th>
<th>strain (in)</th>
<th>Ct net (lb)</th>
<th>Ct tot (lbs)</th>
<th>Conc. Load (lb)</th>
<th>Uniform Id (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.00357</td>
<td>1.6</td>
<td>350.6</td>
<td>10.0</td>
<td>6.9</td>
</tr>
<tr>
<td>0.375</td>
<td>0.00803</td>
<td>3.6</td>
<td>352.6</td>
<td>15.1</td>
<td>10.4</td>
</tr>
<tr>
<td>0.55</td>
<td>0.01728</td>
<td>7.7</td>
<td>356.7</td>
<td>22.4</td>
<td>15.4</td>
</tr>
<tr>
<td>0.75</td>
<td>0.03213</td>
<td>14.2</td>
<td>363.2</td>
<td>31.1</td>
<td>21.3</td>
</tr>
<tr>
<td>1</td>
<td>0.05710</td>
<td>25.3</td>
<td>374.3</td>
<td>42.8</td>
<td>29.3</td>
</tr>
<tr>
<td>2</td>
<td>0.22783</td>
<td>101.0</td>
<td>450.0</td>
<td>102.8</td>
<td>70.5</td>
</tr>
<tr>
<td>2.5</td>
<td>0.35534</td>
<td>157.5</td>
<td>506.5</td>
<td>144.7</td>
<td>99.2</td>
</tr>
<tr>
<td>3.03</td>
<td>0.52075</td>
<td>230.8</td>
<td>579.8</td>
<td>200.8</td>
<td>137.7</td>
</tr>
</tbody>
</table>

**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.

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10012 Creviston Dr NW  
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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).

<table>
<thead>
<tr>
<th>Cable railing</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable deflection calculations</td>
<td></td>
</tr>
<tr>
<td>Cable = 1/8” dia (area in^2) =</td>
<td>0.0123</td>
</tr>
<tr>
<td>Modulus of elasticity (E, psi) =</td>
<td>26000000</td>
</tr>
<tr>
<td>Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading</td>
<td></td>
</tr>
<tr>
<td>Cable installation load (lbs) =</td>
<td>440</td>
</tr>
<tr>
<td>Total Cable length (ft) =</td>
<td>98.4</td>
</tr>
<tr>
<td>Cable free span (inches) =</td>
<td>35</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Calculate strain for a given displacement (one span)</th>
<th>Imposed Cable load giving displ.</th>
</tr>
</thead>
<tbody>
<tr>
<td>delta (in)</td>
<td>strain (in)</td>
</tr>
<tr>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>0.25</td>
<td>0.00357</td>
</tr>
<tr>
<td>0.375</td>
<td>0.00803</td>
</tr>
<tr>
<td>0.55</td>
<td>0.01728</td>
</tr>
<tr>
<td>0.75</td>
<td>0.03213</td>
</tr>
<tr>
<td>1</td>
<td>0.05710</td>
</tr>
<tr>
<td>2</td>
<td>0.22783</td>
</tr>
<tr>
<td>2.5</td>
<td>0.35534</td>
</tr>
<tr>
<td>3.02</td>
<td>0.51734</td>
</tr>
</tbody>
</table>

EDWARD C. ROBISON, PE  
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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).

<table>
<thead>
<tr>
<th>Cable railing</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable deflection</td>
<td>calculations</td>
</tr>
<tr>
<td>Cable = 1/8” dia (area in^2) =</td>
<td>0.0123</td>
</tr>
<tr>
<td>Modulus of elasticity (E, psi) =</td>
<td>26000000</td>
</tr>
<tr>
<td>Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading</td>
<td></td>
</tr>
<tr>
<td>Cable installation load (lbs) =</td>
<td>440</td>
</tr>
<tr>
<td>Total Cable length (ft) =</td>
<td>45.2</td>
</tr>
<tr>
<td>Cable free span (inches) =</td>
<td>42</td>
</tr>
</tbody>
</table>

**Cable installation load (lbs) = 440**

**Total Cable length (ft) = 45.2**

**Cable free span (inches) = 42**

**Calculate strain for a given displacement (one span)**

<table>
<thead>
<tr>
<th>delta (in)</th>
<th>strain (in)</th>
<th>Ct net (lb)</th>
<th>Ct tot (lbs)</th>
<th>Conc. Load (lb)</th>
<th>Uniform Id (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.00298</td>
<td>1.8</td>
<td>441.8</td>
<td>10.5</td>
<td>6.0</td>
</tr>
<tr>
<td>0.375</td>
<td>0.00670</td>
<td>3.9</td>
<td>443.9</td>
<td>15.9</td>
<td>9.1</td>
</tr>
<tr>
<td>0.55</td>
<td>0.01440</td>
<td>8.5</td>
<td>448.5</td>
<td>23.5</td>
<td>13.4</td>
</tr>
<tr>
<td>0.75</td>
<td>0.02678</td>
<td>15.8</td>
<td>455.8</td>
<td>32.6</td>
<td>18.6</td>
</tr>
<tr>
<td>1</td>
<td>0.04759</td>
<td>28.0</td>
<td>468.0</td>
<td>44.6</td>
<td>25.5</td>
</tr>
<tr>
<td>2</td>
<td>0.19005</td>
<td>111.8</td>
<td>551.8</td>
<td>105.1</td>
<td>60.1</td>
</tr>
<tr>
<td>2.5</td>
<td>0.29657</td>
<td>174.5</td>
<td>614.5</td>
<td>146.3</td>
<td>83.6</td>
</tr>
<tr>
<td>3.03</td>
<td>0.43493</td>
<td>255.9</td>
<td>695.9</td>
<td>200.8</td>
<td>114.7</td>
</tr>
</tbody>
</table>

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10012 Creviston Dr NW
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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).

<table>
<thead>
<tr>
<th>Cable railing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable deflection calculations</td>
</tr>
<tr>
<td>Cable = 1/8” dia (area in^2) = 0.0123</td>
</tr>
<tr>
<td>Modulus of elasticity (E, psi) = 26000000</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Calculate strain for a given displacement (one span)</th>
<th>Imposed Cable load giving displ.</th>
</tr>
</thead>
<tbody>
<tr>
<td>delta (in)</td>
<td>strain (in)</td>
</tr>
<tr>
<td>0.25</td>
<td>0.00452</td>
</tr>
<tr>
<td>0.375</td>
<td>0.01018</td>
</tr>
<tr>
<td>0.55</td>
<td>0.02189</td>
</tr>
<tr>
<td>0.75</td>
<td>0.04069</td>
</tr>
<tr>
<td>1</td>
<td>0.07230</td>
</tr>
<tr>
<td>2</td>
<td>0.28809</td>
</tr>
<tr>
<td>2.5</td>
<td>0.44884</td>
</tr>
<tr>
<td>2.95</td>
<td>0.62302</td>
</tr>
</tbody>
</table>

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.
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.

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