7 August 2020

Wentworth Metal Works
ATTN: Sean Wentworth
1483 67th ST
Emeryville, CA 94608

SUBJ: ALUMINUM FLAT BAR RAIL SYSTEM

The aluminum flat bar system uses aluminum extrusion posts, pickets and rails to form building guards. The guards are designed for the following loading criteria:

On Top Rail:
- Concentrated load = 200 lbs any direction, any location
- Uniform load = 50 plf, any direction perpendicular to rail

On In-fill Panels:
- Concentrated load = 50# on one sf.
  - Wind load = 21.4 psf over full area: 120 mph Exp D (ASD level) K_{zt} = 1.0
- Seismic loads – S_{DS} = 2, Site class D

Maximum post spacing is 4’ on center for posts made of 6061-T6 or 6005-T61 used in commercial applications. Unwelded 6063-T6 posts may be used at 4’ on center spacing in residential applications.

The railing system meets all applicable requirements of the 2018 International Building Code and 2019 California Building Code.

Edward Robison, P.E.

REGISTERED PROFESSIONAL ENGINEER
EDWARD C. ROBISON
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Sealed 28 August 2020

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LOAD CASES:
Dead Load
Cap Rail = 1.3plf
Infill = 0.243 plf for pickets
Post = 1.3plf

Typical For System = 1.3plf+11*0.243plf+3.5'*1.3plf/4' = 5.1plf
For top rail loads:
M_c = 200#*H = 200#*42" = 8,400"#
M_u = 50plf*S*H = 50plf*42"*4.00' = 8,400"#
Or check residential case with the 200# load over one post with at least 2 posts at 48" O.C. spacing. The post and top rail segment is modeled using VisualFEA and the maximum post moment was found to be 7,130"#.

For residential
M_u = 20plf*S*H = 20plf*42"*4.00' = 3,360"#

WIND LOAD
Loading on open railing with 33% solid-
25 psf over full area including openings exceeds 120 mph Exp D (ASD)
M_w = 21.4psf*3.5'^{2}*0.55*12"*4' = 6,917"#

SEISMIC LOADS:
$F_P = [(0.4 * a_P S_{DS} I_P)/R_P][1+2z/h)W_P = [0.4*1.0*2.0*1.0/2.5](1+2) = 0.96W_P$
S_{DS} = 2g
For guard weight:
5.1 plf,
E = 0.96*5.1 = 4.9 plf
Seismic loads are much smaller than live loads and will not control any aspect of design.

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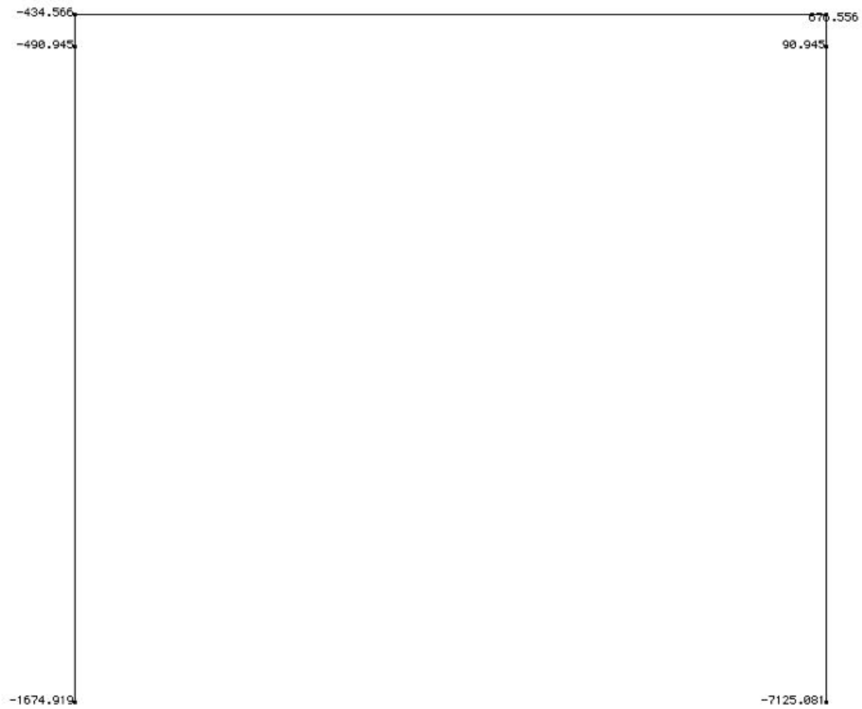
200# load at 44” post cantilever (Fascia Mount Condition) and 48” post spacing
M_{max}=7,125”#



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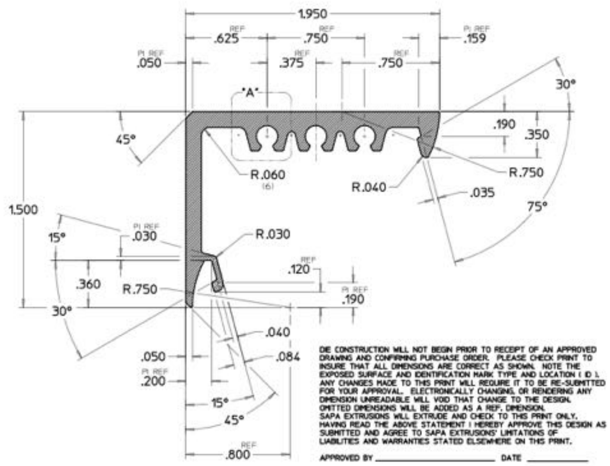
1-1/2"x2" TWO PART extruded aluminum post
6063-T6



Post sections snap together to form rectangular post. The interlocking nature of the channels allows the post to behave compositely.

Section properties of individual post extrusion:
$I_{xx} = 0.2096\ in^4$, $I_{yy} = 0.08057\ in^4$
$S_{xx} = 0.1718\ in^3$, $S_{yy} = 0.06859\ in^3$
$r_x = 0.6371$ in, $r_y = 0.3950$ in

Total for assembled post section
$I_{xx} = 0.4964\ in^4$, $I_{yy} = 0.3433\ in^4$
$S_{xx} = 0.4937\ in^3$, $S_{yy} = 0.4551\ in^3$

Allowable bending stress for 6063-T6 aluminum
$F_{ta} = 15.2$ ksi (ADM Table 2-24 line 2)
For compression:
$b/t = 0.911/0.121 = 7.53$
$F_{bc} = 15.2$ ksi (ADM Table 2-24 line 16)

$M_a = F_b S$
$M_{ax} = 0.4937*15.2$ ksi = 7,504"# For loading parallel with guard.
$M_{ay} = 0.4551*15.2$ ksi = 6,917"# For loading perpendicular with guard.

Check post deflection:
$\Delta = Ph^3/(3EI) = 50*4.02*42"^3/(3*10.1x10^6*0.4964) = .83" \leq h/12$

Deflections are okay.

ALTERNATIVE ALLOYS TO INCREASE POST STRENGTH
Use 6061-T6 or 6005-T5 or 6005A-T61

$M_{ax} = 0.4937*19.5$ ksi = 9,630"# For loading parallel with guard.
$M_{ay} = 0.4551*19.5$ ksi = 8,870"# For loading perpendicular with guard.

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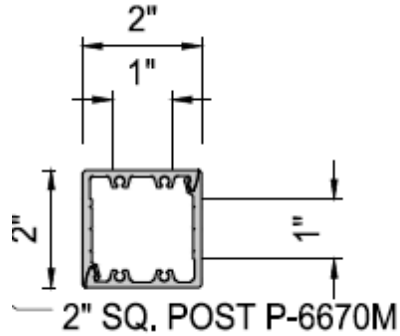
2”X2” 2 PIECE POST

Post sections snap together to form square post. The interlocking nature of the channels allows the post to behave compositely.



Section properties of individual post extrusion:
$I_{xx} = 0.2311\ in^4$, $I_{yy} = 0.1962\ in^4$
$S_{xx} = 0.1716\ in^3$, $S_{yy} = 0.1344\ in^3$
$r_x = 0.6400$ in, $r_y = 0.5897$ in

Total for assembled post section
$I_{xx} = 0.6266\ in^4$, $I_{yy} = 0.6392\ in^4$
$S_{xx} = 0.6266\ in^3$, $S_{yy} = 0.6392\ in^3$

Allowable stress:
$F/\Omega = 15.2$ksi for 6063-T6
$F/\Omega = 19.5$ksi for 6005-T61 or 6061-T6

Allowable moment:
$M_a = 15.2ksi*0.6266in^3 = 9{,}520”\#$ for 6063-T6
$M_a = 19.5ksi*0.6392in^3 = 12{,}500”\#$ for 6005-T61 or 6061-T6

Check post deflection:
$\Delta = Ph^3/(3EI) = 50*4.02*43”^3/(3*10.1x10^6*0.6392) = .83” \leq h/12$

Deflections are okay.

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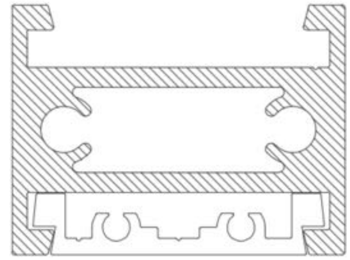
1-1/2”x2” 3 PIECE POST



Infill slide in elements are not considered in post strength.

Section properties for main extrusion:
$A = 1.02in^2$
$I = 0.563in^4$
$S = 0.565in^3$
$Z = 0.703in^3$
By inspection, local buckling is not a concern.

Allowable stress:
$F/\Omega = 15.2ksi$ for 6063-T6
$F/\Omega = 19.5ksi$ for 6005-T61 or 6061-T6

Allowable moment:
$M_a = 15.2ksi*0.703in^3 = 10,700”\#$
$M_a = 19.5ksi*0.703in^3 = 13,700”\#$

For welded 6061-T6 posts:
$M_a = 24ksi/1.95*0.703in^3 = 8,650”\#$

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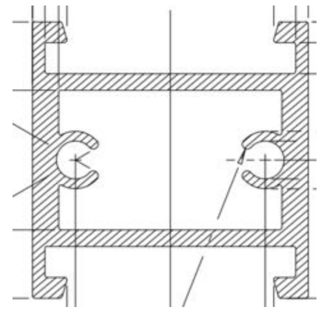
2”x2” 3 PIECE POST



Infill slide in elements are not considered in post strength.

Section properties for main extrusion:
$A = 1.17in^2$
$I = 0.698in^4$
$S = 0.698in^3$
$Z = 0.846in^3$
b/t= 1.01”/0.19” = 5.3 local buckling does not control

Allowable stress:
F/Ω = 15.2ksi for 6063-T6
F/Ω = 19.5ksi for 6005-T61 or 6061-T6

Allowable moment:
$M_a = 15.2ksi*0.846in^3 = 12{,}900”\#$
$M_a = 19.5ksi*0.846in^3 = 16{,}500”\#$

For welded 6061-T6 posts:
$M_a = 24ksi/1.95*0.846in^3 = 10{,}400”\#$

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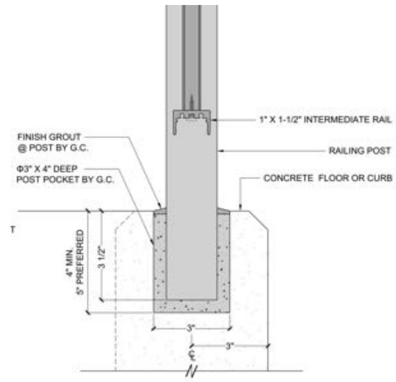
POST ANCHORAGE DETAILS



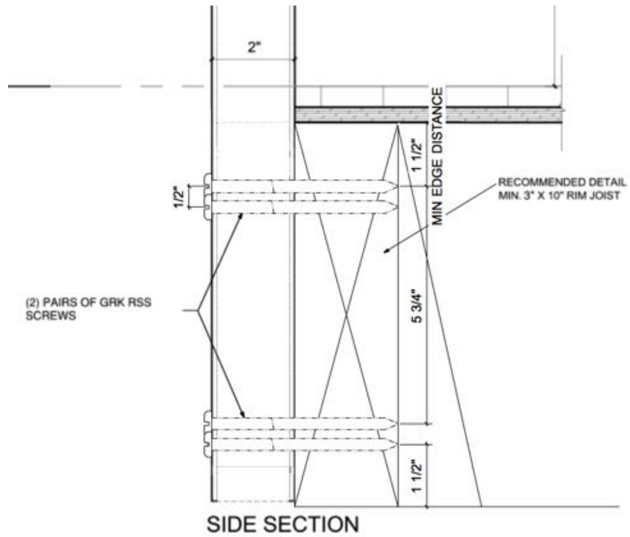
CORE MOUNT

Assumed concrete strength $f'_c \geq 3{,}000$ psi
Check for concrete shear breakout or crushing failure modes.
Assume rectangular bearing reactions.

Inputs						Edge breakout calcs			
Core Width (in)	Embed Width (in)	Concrete Strength		Edge Distance	Embedment				
b_c (in)	b_s (in)	f'_c (psi)	λ	C_1 (in)	d (in)	w (in)	h (in)	β	b_0 (in)
3	1.5	3000	1	3	4	6	3.5	1.71428571	13

						Bearing Calcs				Strength
						$0.65*0.85f'_c/1.6$	$(b_c-b_s)/2+b_s$	$(b_c-b_s)/4+d/2$	$f_a b_b h_b$	$\text{Min}(P_a)*d/2$
α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2+4/\beta)\lambda\sqrt{f'_c}$ (psi)	$(2+\alpha_s C_1/b_0)\lambda\sqrt{f'_c}$ (psi)	$V_c C_1 b_0$	P_a (lbs)	f_a (psi)	b_b (in)	h_b (in)	P_a (lbs)	M_a (in-lbs)
30	219.089023	237.3464416	488.7370513	8544.4719	4005.221202	1035.9375	2.25	2.375	6215.625	8010.4424

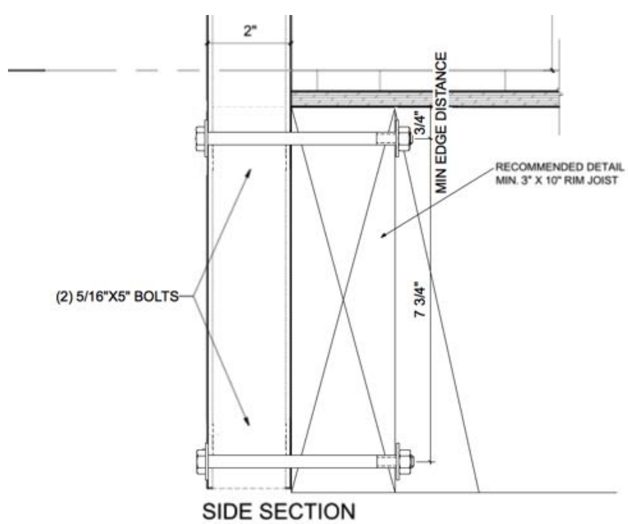
FASCIA MOUNT



Uses (4) 1/4" GRK RSS fasteners.
From ESR 2442,
W = 151pli
C_d=1.6 for short duration live load
Spacing between rows of screws = 5.75"
W'=1.6*151pli = 241pli
From above model for 4' post spacing max load to single post = 162#
Load to upper screw pair =
162#*(42"+2.75"+5.75"+1.5")/(5.75"+1.5")
= 1,162#
Steel strength = 1,112# each OK
Required penetration = (1,162#/2)/241pli = 2.41"
Required screw length = 2.41"+2" => 4-1/2"
3x10 minimum rim size.

For posts made from the alternative higher strength allows increase embedment by 40%:
L = 2.41*1.4 + 2 = 5-1/2"

ALTERNATIVE CONNECTION:
Use 5/16" bolts through rim joist with bearing washer on back.
Strength will be same as above thus okay alternative.



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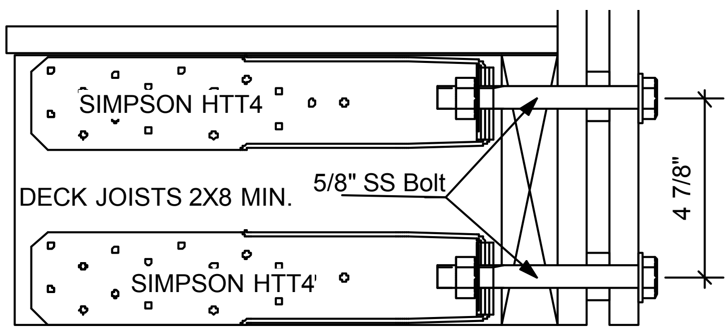
Hold Down Alternatives:
Bolts and hold downs can be used when the framing at the fascia is not adequate to attach directly. Bolts pass through the post and attach to hold downs which are anchored to joists that run perpendicular with the rail.

Bolts must be stainless steel because they are in contact with aluminum and weather.

Hold down option 1: Simpson HTT4
For use when hold downs are able to attach directly to joists. Hold down attaches with screws or nails according to joist per Simpon's instructions.
From Simpson load tables, T_a = 3,000#
5/8" SS bolt strength, T_a = 0.226in^2*75ksi/2 = 8,480# > 3,000# (Does not control)
Assuming 4-7/8" hold down spacing:
a = 3000#/(2"*744psi) = 2.02"
Allowable load at top of post = 3,000#*(6.06"-2.02"/2)/(42"+2"+6.06"-2.02"/2) = 309#



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Hold down option 2: Simpson HD7B

For use when hold downs are not able to be attached directly to joists. Hold downs attach to threaded rods that attach to joists. Threaded rods have washers and nuts that hold them in place and keep the threaded rods from sliding in their holes.

From Simpson load tables, T_a = 6,645#

Assume 3/4" threaded rods span 16" max. Also assume that rods run multiple spans so that the center span behaves like a fixed span.

D_r = 0.680"

Z = 0.05241in^3

M_a = 0.05241in^3*75ksi/2 = 1,970"#

Allowable load, 1,970"#*3/(16"/8) = 2,960#

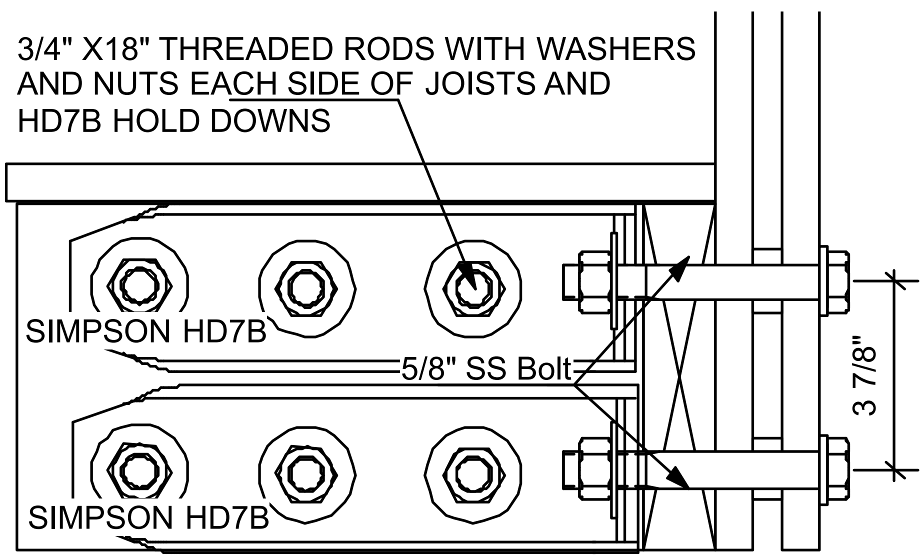
Assuming 3-7/8" hold down spacing:

a = 2,960#/(2"*744psi) = 1.99"

Allowable load at top of post = 2,960#(6.06"-1.99"/2)/(42"+2"+6.06"-1.99"/2) = 306#

Wind load that causes equivalent loading on anchorage = 8,400"#/(4'*3'*31.1") = 22.5psf

The GRK option limits wind load to 22.5psf. Other anchorages develop strength above 25psf.



Hold downs may be located at any location between joists on threaded rods.

For installation on joists deeper than 2x8 (7-1/4") set tension tie at top of joist and at 3-7/8" center to center for lower tension tie.

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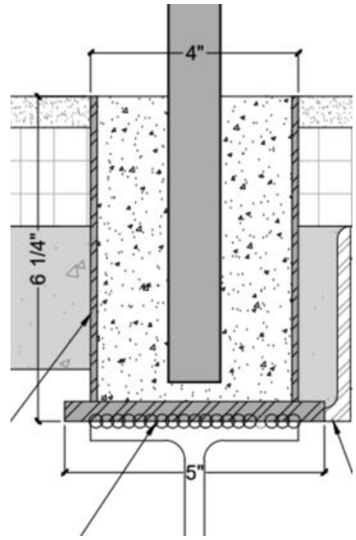
CUP MOUNT



Similar to core mounts but edge breakout is restrained by the steel tube. Assume at least 4" embedment in grout.

Check grout crushing:
Assume f'_c = 5000psi minimum for grout
P_a = 0.65*0.85*1.5"*2"*5000psi/1.6 = 5,180#
M_a = 5,180#*2" = 10,400"#

Check welding to structural steel:
Assume 3-1/2" long 3/16" welds on two sides.
I_w = 2*3.5"^{3}/12 = 7.15in^4/in
Weld loading is primarily normal to the axis so the strength may be multiplied by 1.5.
R_n/Ω = 1.5*928*3*1" = 4,180pli
M_a = 4,180pli* 7.15in^4/in/1.75" = 17,100"#

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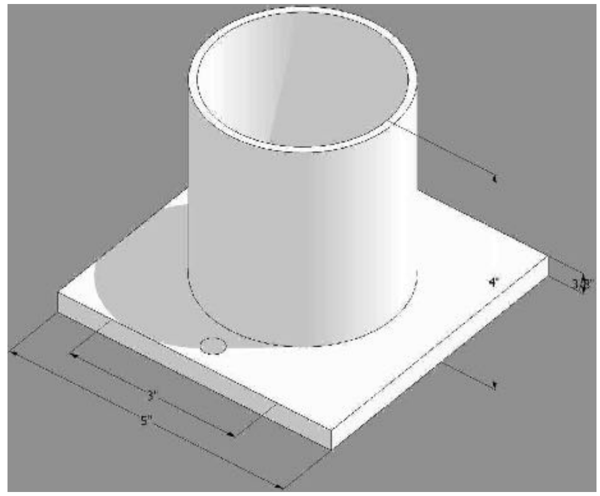
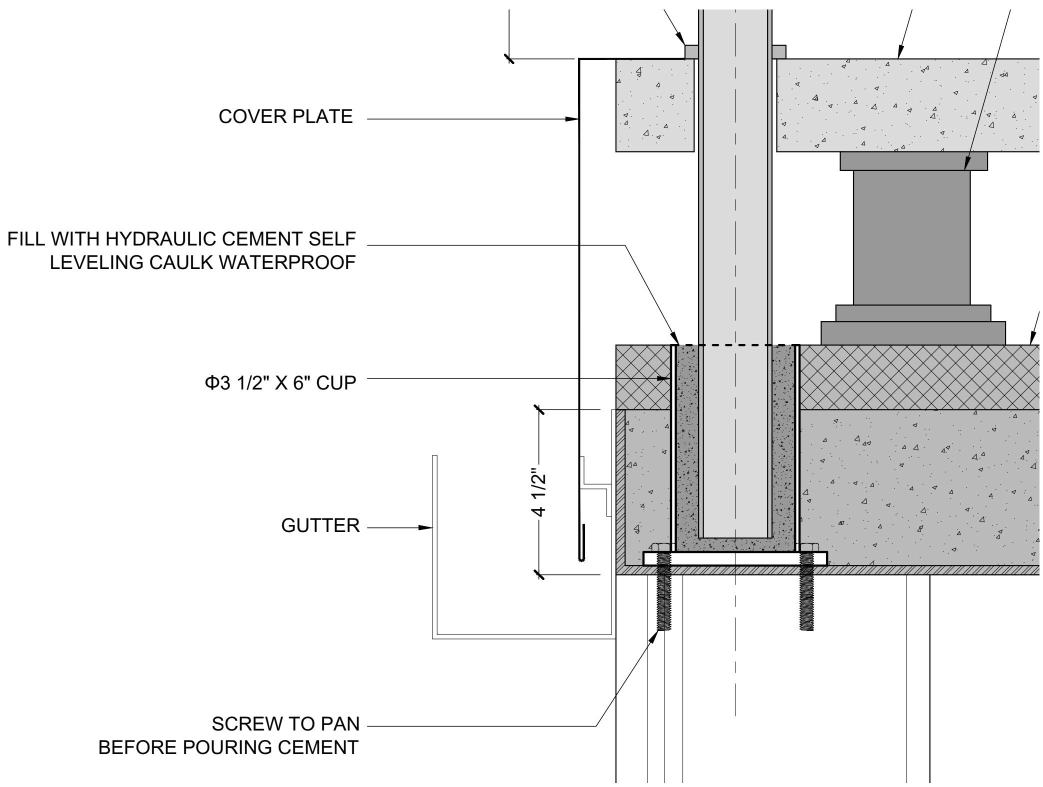
CUP MOUNTING - RECEIVER MOUNTED POST

The post is grouted into a metal receiver set into a concrete or mounted to a steel or wood deck.



CUP MOUNTING IN CONCRETE SLAB

When set in concrete slab strength is same as for the core mounted post.



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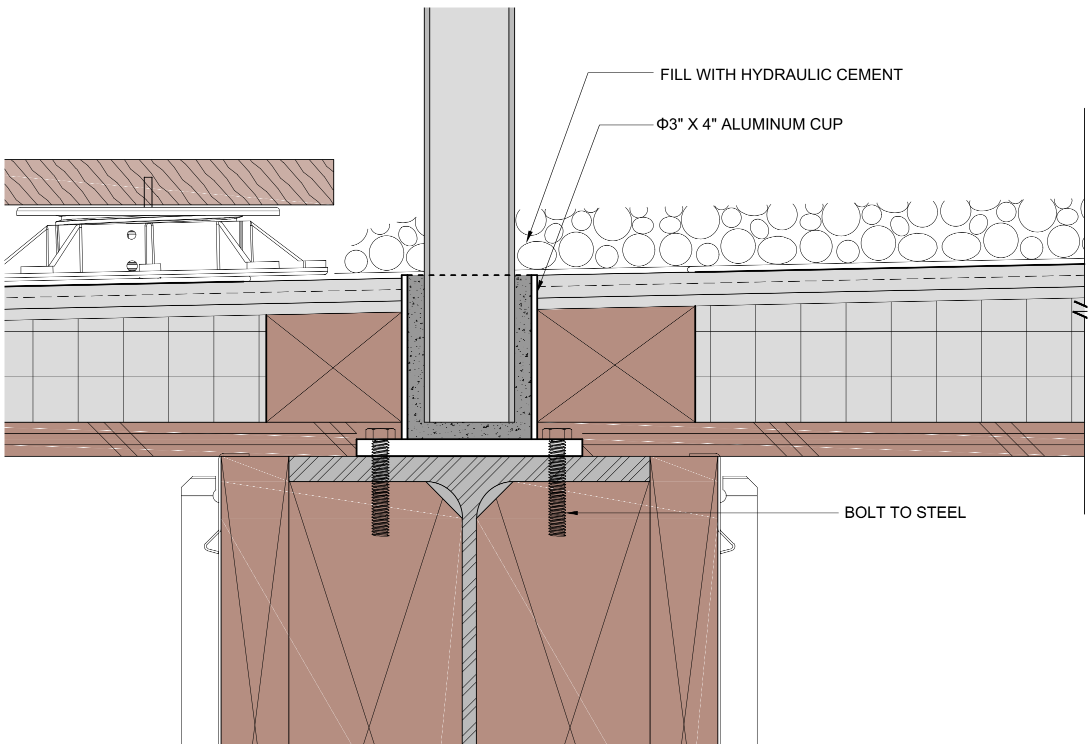
Strength of Cup:
3" i.d. x 1/8" thick welded to baseplate (6061-T6 aluminum):
Strength controlled by the HAZ at the weld to the base plate-
$Z = (3.25^3-3^3)/6 = 1.221\ in^3$
F_{twa} = 24ksi*1.221/1.95 = 15,032"# allowable moment on the cup when not encased.
Strength is adequate for post strength

CUP MOUNTING TO STEEL BEAM OR DECK
Use (4) 3/8" SS bolts to anchor to the steel beam or steel deck of adequate strength and thickness.



Bolt strength: 75 ksi tension strength
$T_n = 0.0775in^2*75ksi = 5{,}812\#$
M_n = 5,812#*3.75"*2 = 43,590"#
Thus will support the full post strength.
Check welds to structural steel:
Assume at least 3" long 3/16" welds on two sides.
$I_w = 2*3"^3/12 = 4.5in^4/in$
Welds will be loaded primarily normal to their axis, so the strength may be multiplied by 1.5.
R_n/Ω = 1.5*928*3*1" = 4,180pli
M_a = 4,180pli*4.5in⁴/in/2" = 9,410"#

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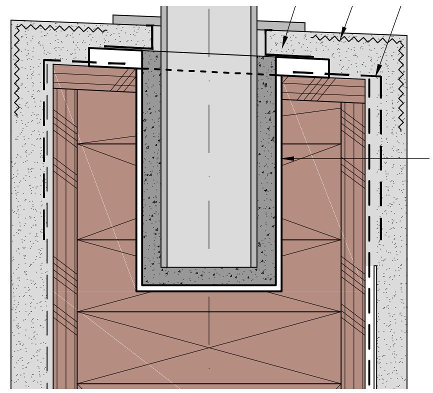
CUP MOUNTING TO WOOD:

Baseplate mounted receiver (same as steel but attached to wood)
T_a = 1.6*263pli*4.25” = 1,788”#
a = 2*1788/(605psi*5”) = 1.182”
M_a = 1,788*2*(4.25-1.182/2) = 12,190”#

Inset in wood:
The receiver is inset into wood by inlaying int a bored hole or by building up blocking around.



For built up blocking required shear connection between plies assuming 1-1/2” ply thickness:
V = (200#*47”)/4.5” = 2,089#

Connection - 16d sinker nails
Z’ = 1.6*99 = 158# each (NDS Table 12N G = 0.43)
n = 2089/158 = 14 nails to connect plies, each ply

#12 x 6” screws:
Z’ = 1.6*128 = 205# each (NDS Table 12L G = 0.43)
n = 2089/205 = 11 Screws to connect plies

#14 x 6” screws
Z’ = 1.6*141 = 226# each (NDS Table 12L G = 0.43)
n = 2089/226 = 10 Screws to connect plies

5/16” x 6” GRK RSS (ESR-2442) or Fastenmaster Timberlok (ESR-1078) screw
Z’ = 1.6*210 = 336# each (G = 0.42)
n = 2089/336 = 7 Screws to connect plies

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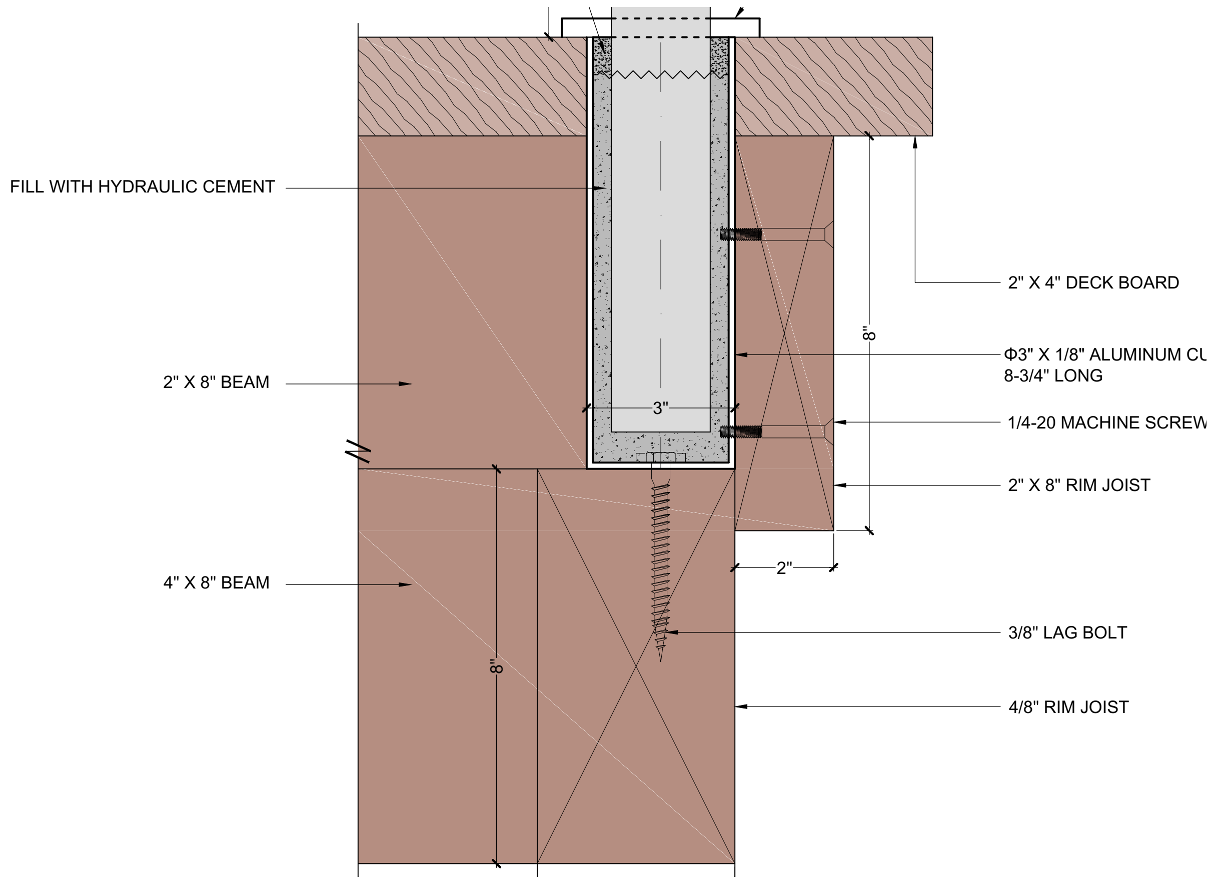
Cup Mounting 1:



To wood with receiver clamped by a block on side.
Required pullout strength of blocking:
R =(200#*50”)/7.25” = 1,380#

#12 screws:
W’ = 1.6*114pli*1.5” = 274# each (NDS Table 12.2B, G = 0.43)
n = 1380/274 = 6 screws

5/16” x 6” GRK RSS (ESR-2442) or Fastenmaster Headlok (ESR-1078) screw
W’ = 410# each
n = 1380/410 = 4 screws

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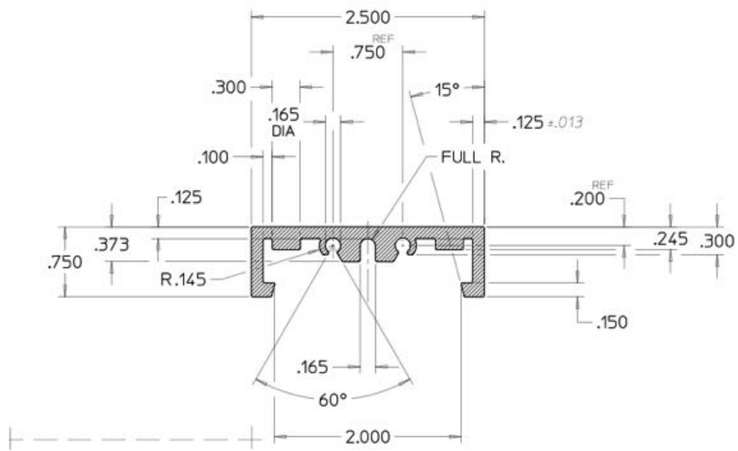
GUARD RAIL TOP RAIL:

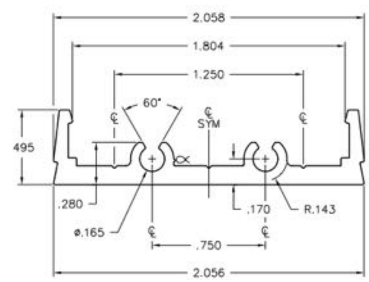


3/4” x 2-1/2” flat cap rail

Section properties:

Top	Insert
$I_{xx} = 0.02793 in^4$	$0.00431\ in^4$
$S_{xx} = 0.05266\ in^3$	$0.01147\ in^3$
$I_{yy} = 0.4835\ in^4$	$0.1560\ in^4$
$S_{yy} = 0.3874\ in^3$	$0.1514\ in^3$

Built Up Section
$I_{xx} = 0.07226\ in^4$
$S_{xx} = 0.1842\ in^3$
$I_{yy} = 0.6394\ in^4$
$S_{yy} = 0.5123\ in^3$



Allowable stress - 6063-T6 aluminum
$M_{a,x} = 0.1842in^3 * 15.2ksi = 2{,}800”\#$
$M_{a,y} = 0.5123in^3 * 15.2ksi = 7{,}787”\#$

Vertical loading controls allowable post spacing:
For single span condition:
M=PL/4
2,800”#=200#L/4
L=56”

For double span condition
M=(13/64)PL
2,800”#=(13/64)200#L
L=69”
When pickets are installed top rail span may increase due to load sharing with the bottom rail.

Toprail deflection at failure
$\Delta = 200*56^3/(48*10.1x10^6*.07226) = 1”$
Deflection limits may control allowable top rail span.

Check shear flow between top and insert.
VQ/I
V=100# (200# load at mid point)
Q=0.0974
v = 100#*.0974/.07226/2 = 68pli = 810plf

Top rails are screwed to posts with #10 screws

Strength of #10 SS screw (ASTM F 879 Table 3)
(6) screws into insert slots.
$T_n = 1{,}330\#$
$V_n = 0.6T_n = 0.6*1{,}330\# = 798\#$ for shear through threads

Strength into screw slots:
For withdrawal:
$T_n > 0.29DL_uF_{tu} = 0.29*0.19”*1”*30ksi = 1{,}653\#$

For shear:
$V_n = d_etF_{tu} = 0.1*(0.29*1”)*30ksi = 870\#$

Bearing on post:
$V_a = 2*F_{u1}t_1D/3 = 2*30ksi*0.19*0.125/3 = 475\#$

Connection strength is adequate for all design load conditions.

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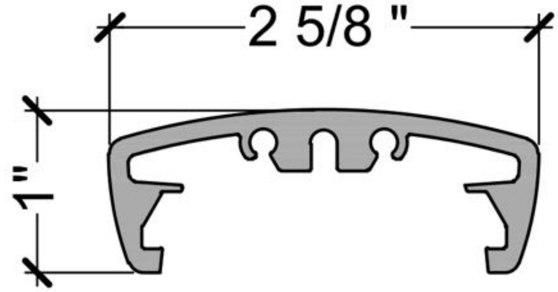
A11 TOP RAIL



Alloy 6063-T6 Aluminum

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

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b{}^2)^{1/2})^{1/2}$
$r_{ye} = ((0.0519)^{1/2}/0.0788*(0.0495+0.038*0.00568*48^2)^{1/2})^{1/2} = 1.462$”
$\lambda = 48$”/(1.462”*1) = 32.83
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c{}^3$
$M_p = 30$ksi*0.5583in^3 = 16.75kip-in
$M_{nmb} = 16.75$kip-in(1-32.83/78)+$\pi^2$10,100*32.83*0.4264/78^3 = 12.64kip-in
$M_a = 12.64$kip-in/1.65 = 7,660”#

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1536$in^3*15.2ksi = 2,330”#

Find maximum on center post spacing:
L_{max}=2,330”#*4/200#+2” = 48.6” For single span condition
L_{max}=2,330”#*(64/13)/200# = 57.4” For multiple span condition

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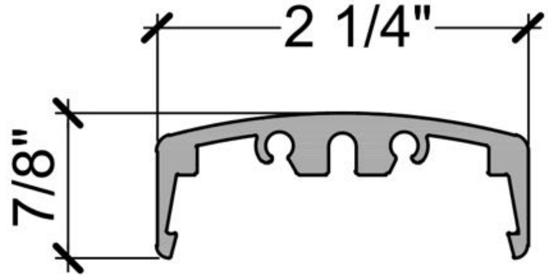
A111 TOP RAIL



Alloy 6063-T6 Aluminum

Allowable Stress
ADM Table 2-20
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b^2)^{1/2})^{1/2}$
$r_{ye} = ((0.02047)^{1/2}/0.2458*(0.01183+0.038*0.00400*48^2)^{1/2})^{1/2} = 0.5918$”
$\lambda = 48$”/(0.5918”*1) = 81.11
$M_{nmb}=\pi^2ES_x/\lambda^2$
$M_{nmb}=\pi^2*10{,}1000*0.2458/81.11^2 = 3{,}720$”#
$M_a = 3{,}720$”#/1.65 = 2,260”#

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.07620in^3*15.2ksi = 1{,}160$”#
Include contribution from sub rail:
Assume 520E sub rail, $I = 0.00589in^4$
Top rail load share = $0.02047in^4/(0.02047in^4+0.00589in^4) = 0.7766$
$M_a = 1{,}160$”#/0.7766 = 1,490”#

Find maximum top rail span:
L_{max}=1,490”#*4/200# = 31.8” For single span condition
L_{max}=1,490”#*(64/13)/200# = 36.7” For two span condition

Use of top rail is extremely limited for 6063-T6, try 6061-T6:

$M_a = 0.07620in^3*19.5ksi/0.7766 = 1{,}910$”#

Find maximum on center post spacing:
L_{max}=1,910”#*4/200#+2” = 40.2” For single span condition
L_{max}=1,910”#*(64/13)/200# = 47.0” For two or more spans condition

This top rail has limits below 48” O.C. post spacing when used for fall protection.

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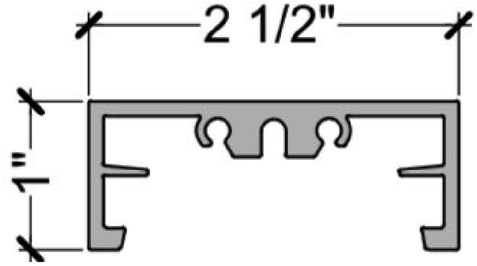
A222 TOP RAIL



Alloy 6063-T6 Aluminum

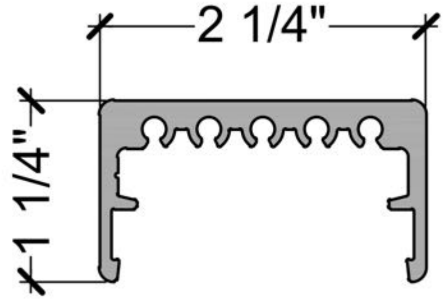
Allowable Stress
ADM Table 2-21
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b^2)^{1/2})^{1/2}$
$r_{ye} = ((0.05785)^{1/2}/0.3847(0.06424+0.038*0.00399*48^2)^{1/2})^{1/2} = 0.6341$in
$\lambda = 48"/(0.6341"*1) = 75.70$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$
$M_p = 25$ksi$*0.4866$in$^3 = 12.17$kip-in
$M_{nmb} = 12.17$kip-in$(1-75.70/78)+\pi^2 10,100*75.70*0.3847/78^3 = 6.465$kip-in
$M_a = 6.465$kip-in$/1.65 = 3,920$"#

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1512$in$^3*15.2$ksi $= 2,300$"#

Find maximum on center post spacing:
L_{max}=2,300"#*4/200#+2" = 48.0" For single span condition
L_{max}=2,300"#*(64/13)/200# = 56.6" For two or more spans condition

A225 TOP RAIL



Alloy 6063-T6 Aluminum

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

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b{}^2)^{1/2})^{1/2}$
$r_{ye} = ((0.07776)^{1/2}/0.4498(0.06184+0.038*0.00660*48^2)^{1/2})^{1/2} = 0.4649$in
$\lambda = 48"/(0.4649"*1) = 103.2$
$M_{nmb}=\pi^2 ES_x/\lambda^2$
$M_{nmb}=\pi^2*10{,}1000*0.4498/103.2^2 = 4{,}210"\#$
$M_a = 4{,}210"\#/1.65 = 2{,}550"\#$

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1715in^3*15.2ksi = 2{,}610"\#$

Find maximum on center post spacing:
$L_{max}=2{,}550"\#*4/200\#+2" = 53.0"$ For single span condition
$L_{max}=2{,}550"\#*(64/13)/200\# = 62.8"$ For two span condition

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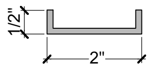
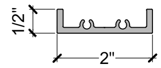
520 SUB RAILS

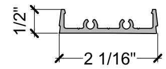
Sub rail options are:
520,
520S and
520E



520



520 E



520 S

The strength of the 3 are similar. Separate calculation of the sub rail strength is not provided herein as the sub rails are only used in an assembly with the top rail.

The top rail strengths take into account the coupled sub rail.

The 520S is a snap in sub rail which snaps into place on the bottom of the top rail and is retained in place by the interlocking tabs.

Force required to pull out the 520 S from the top rail is controlled by flexure in the locking tab.
Force to pullout the tab:
$R_{po} = [.1^3 * 3 * 10x10^6/(0.5^3)] * \sin 60 = 207$ k/ft
$R_t = 0.1 * 12" * 15ksi = 18$ k/ft

Thus it requires substantially more force to pullout the sub rail than any realistic load condition while in service.

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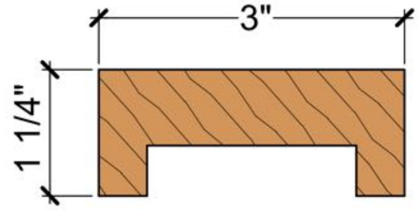
WDCR Wood cap rail

Minimum nominal bending strength for wood is 1,400psi when posts are spaced 48” O.C.
Strength factors:
C_D = 2 for impact live loading
C_f = 1.5 for 2x4 size
Wood must not receive damage from moisture- must be sealed.
F_b’ = 2*1.5*1,400psi = 4,200psi
Flat wise bending is also allowed a 1.15 flat use factor,
F_b’=1.15*4,200psi = 4,830psi (vertical loads)



Check stability factor, l_e = 1.37l_u+3d = 1.37*48”+3*3” = 74.8”

Beam Strength Calculation

Emin	Cm	Cf	Cd	t	d	le
470000	1	1.5	2	1.25	3	74.8

Rb	Braced?
11.9839893	No

Fb	Fb*	Fb'
1400	4200	3310.78848

CL	FcE
0.78828297	3927.13904

With the C_L factor, F_b’ = 3,310psi.
C_L does not apply to flat wise bending.

Strong axis bending:
S = 1.653in^3
M_a = 1.653in^3*3,310psi = 5,470”#

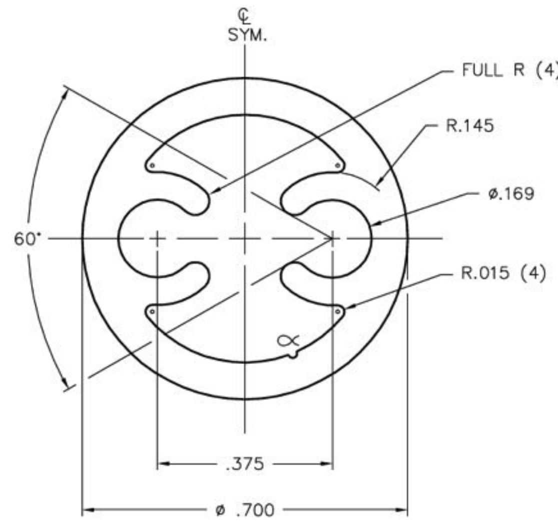
Weak axis bending:
S = 0.3621in^3
M_a = 0.3621in^3*4,830psi = 1,750”#

Wood rails are always used with an aluminum sub rail.
Typical sub rail is the 520E
I_x = 0.00589in^4
Load share to wood = 0.2757/(10.1/1.4*0.00589+0.2757) = 0.866
M_a = 1,750”#/0.866 = 2,020”#

Find maximum on center post spacing:
L_{max}=2,020”#*4/200#+2” = 40.4” For single span condition
L_{max}=2,020”#*(64/13)/200# = 49.7” For two span condition

INFILL PICKETS

0.7" Diameter Round Picket



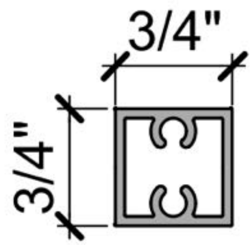
$I_x = 0.00809$ in^4; $S_x = 0.02254$ in^3
$I_y = 0.00897$ in^4; $S_y = 0.02548$ in^3

Allowable stress for 6063-T6 aluminum:
$F_t = F_c = 17.7$ ksi (ADM Table 2-24)

$M_a = 17.7$ksi*0.2254 = 3,989"#

Maximum moment on bottom rail from 50 lbs load at any location:
M=50#L/4=3,989"#
L = 319" (Greatly exceeds practical spans)

3/4" Picket



$I_x = 0.0163$in^4
$S_x = 0.0435$in^3
$I_y = 0.0141$in^4
$S_y = 0.0376$in^3

Allowable bending stress for 6005 aluminum
$F_y/\Omega = 35$ksi/1.65 = 21.2ksi
Or F_u/Ω=38ksi/1.95 = 19.5ksi (controls)
Minimum strength is about the y axis.
$M_{a,y}$=0.0376in^3*19.5ksi = 733"#

Max span = 38"
Max picket spacing = 4.25"
Loading is applied over a 12"x12" square so loading is applied to at least 3 pickets.
M = (50#/3)*38"/4 = 158"# < 733"# OK
Δ= 50#/3*38"^{3}/(48*10.1*10^6*.0141) = 0.134"

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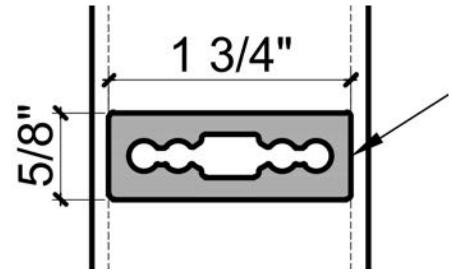
1-3/4”x5/8” Pickets



Section properties:
$I_x = 0.0341in^4$
$S_x = 0.1072in^3$
$I_y = 0.2328in^4$
$S_y = 0.2642in^3$

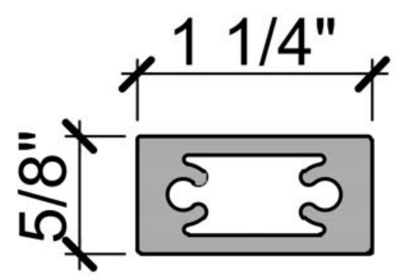
$M_{a,x} = 0.1072in^3*15.2ksi = 1{,}630”\#$
$M_{a,y} = 0.2642in^3*15.2ksi = 4{,}020”\#$

Max span = 48”

Check for 50# mid span:
M = 50#*48”/4 = 600”# < 1,630”# OK

Check extreme case of 200# mid span:
M = 200#*48”/4 = 2,400”# < 1.65* 1,630”# = 2,690”#
200# load on pickets is not a code requirement so the pickets are not required to carry the load with safety factors. However, due to the potential for climbing the horizontal pickets it should be considered. Yielding of the pickets is not expected if someone tries climbing them like a ladder.

1-1/4”x5/8” Pickets



Section properties:
$I_x = 0.0204in^4$
$S_x = 0.0654in^3$
$I_y = 0.0816in^4$
$S_y = 0.130in^3$

$M_{a,x} = 0.0654in^3*15.2ksi = 994”\#$
$M_{a,y} = 0.130in^3*15.2ksi = 1{,}980”\#$

Max span = 48”

Check for 50# mid span:
M = 50#*48”/4 = 600”# < 994”# OK

Check extreme case of 200# mid span:
M = 200#*48”/4 = 2,400”# > 1.65* 994”# = 1,640”#
Occupants climbing horizontal pickets like a ladder may cause failure of the pickets.

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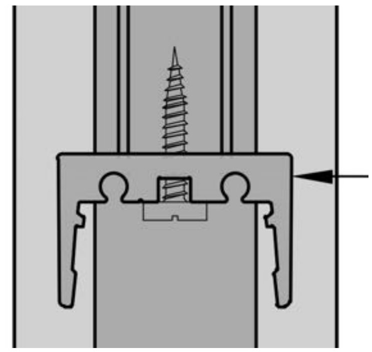
BOTTOM RAIL



$I_y = 0.1698$ in^4; $S_y = 0.2189$ in^3

Allowable stress for 6063-T6 aluminum:
$F_t = F_c = 15.2$ ksi (ADM Table 2-24)

$M_a = 15.2$ksi*0.2189 = 3,330"#

Maximum moment on bottom rail from 50 lbs load at any location:
Maximum span (face to face of posts): 58"
M = 50#*58/4 =725"# ≤ 3,940"#

Live load deflection:
$\Delta = 50*48^3/[48*10x10^6*0.1698] = 0.0678$"

Connection to post:
(2) #10 screws through post wall into rail:
Strength of #10 SS sheet metal screw (ASTM F 879 Table 3)
$T_n = 1,330$#
$V_n = 0.6T_n = 0.6*1,330$# = 798# for shear through threads

Strength into screw slots:
For withdrawal:
$T_n > 0.29DL_uF_{tu} = 0.29*0.19$"*1"*30ksi = 1,653#

For shear:
$V_n = d_etF_{tu} = 0.1*(0.29*1$")*30ksi = 870#

Bearing on post extrusion:
$V_a = 2*F_{u1}t_1D/3 = 2*30$ksi*0.19*0.125/3 = 475 per screw.

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