

28 April 2021

Wentworth Metal Works
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SUBJ: WL-3000 RAILING SYSTEM
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.

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Edward Robison, P.E.



Sealed 28 April 2021

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LOAD CASES:

Dead Load

Cap Rail = 1.3plf

Infill = 0.243 plf for pickets

Post = 1.3plf

Typical For System = $1.3\text{plf} + 11 * 0.243\text{plf} + 3.5' * 1.3\text{plf} / 4' = 5.1\text{plf}$

For top rail loads:

 $M_c = 200\# * H = 200\# * 42'' = 8,400''\#$ $M_u = 50\text{plf} * S * H = 50\text{plf} * 42'' * 4.00' = 8,400''\#$

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

 $M_u = 20\text{plf} * S * H = 20\text{plf} * 42'' * 4.00' = 3,360''\#$ **WIND LOAD**

Loading on open railing with 33% solid-

25 psf over full area including openings exceeds 120 mph Exp D (ASD)

 $M_w = 21.4\text{psf} * 3.5'^2 * 0.55 * 12'' * 4' = 6,917''\#$ **SEISMIC LOADS:** $F_P = [(0.4 * a_P S_{DS} I_P) / R_P] [1 + 2Z/h] W_P = [0.4 * 1.0 * 2.0 * 1.0 / 2.5] (1 + 2) = 0.96 W_P$ $S_{DS} = 2g$

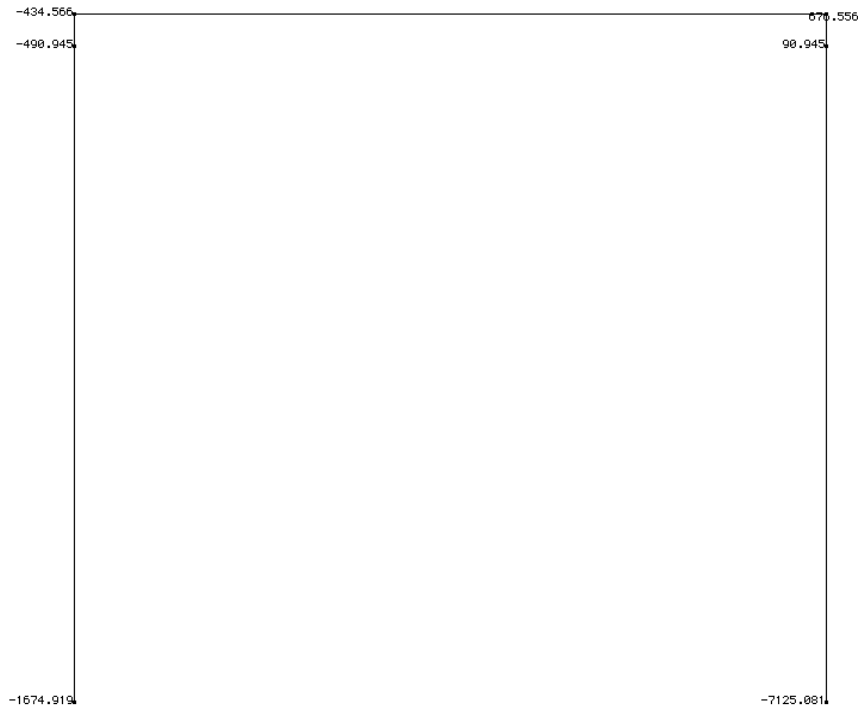
For guard weight:

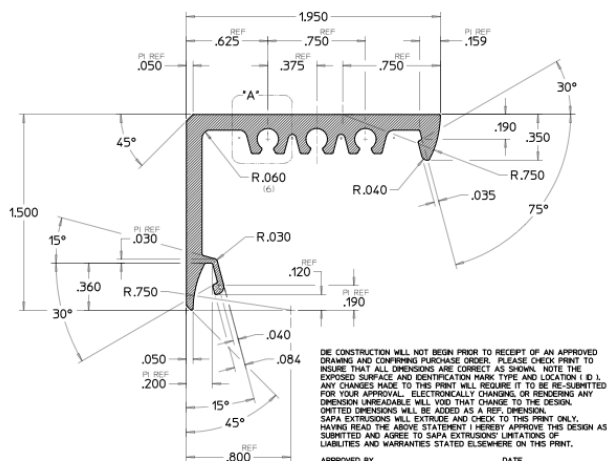
5.1 plf,

 $E = 0.96 * 5.1 = 4.9 \text{ plf}$

Seismic loads are much smaller than live loads and will not control any aspect of design.

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





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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:

$$I_{xx} = 0.2311 \text{ in}^4, \quad I_{yy} = 0.1962 \text{ in}^4$$

$$S_{xx} = 0.1716 \text{ in}^3, \quad S_{yy} = 0.1344 \text{ in}^3$$

$$r_x = 0.6400 \text{ in}, \quad r_y = 0.5897 \text{ in}$$

Total for assembled post section

$$I_{xx} = 0.6266 \text{ in}^4, \quad I_{yy} = 0.6392 \text{ in}^4$$

$$S_{xx} = 0.6266 \text{ in}^3, \quad S_{yy} = 0.6392 \text{ in}^3$$

Allowable stress:

$$F/\Omega = 15.2 \text{ ksi for 6063-T6}$$

$$F/\Omega = 19.5 \text{ ksi for 6005-T61 or 6061-T6}$$

Allowable moment:

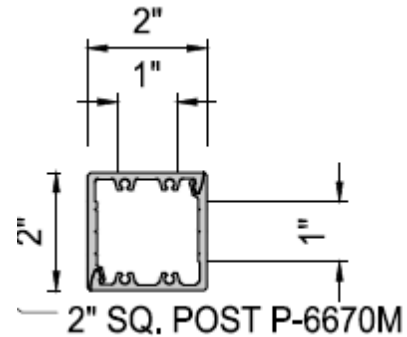
$$M_a = 15.2 \text{ ksi} * 0.6266 \text{ in}^3 = 9,520 \text{"}\# \text{ for 6063-T6}$$

$$M_a = 19.5 \text{ ksi} * 0.6392 \text{ in}^3 = 12,500 \text{"}\# \text{ for 6005-T61 or 6061-T6}$$

Check post deflection:

$$\Delta = Ph^3/(3EI) = 50 * 4.02 * 43^3 / (3 * 10.1 \times 10^6 * 0.6392) = .83" \leq h/12$$

Deflections are okay.



1-1/2"x2" 3 PIECE POST

Infill slide in elements are not considered in post strength.

Section properties for main extrusion:

$$A = 1.02\text{in}^2$$

$$I = 0.563\text{in}^4$$

$$S = 0.565\text{in}^3$$

$$Z = 0.703\text{in}^3$$

By inspection, local buckling is not a concern.

Allowable stress:

$$F/\Omega = 15.2\text{ksi for 6063-T6}$$

$$F/\Omega = 19.5\text{ksi for 6005-T61 or 6061-T6}$$

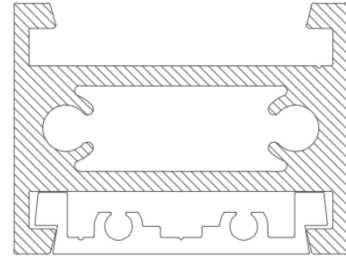
Allowable moment:

$$M_a = 15.2\text{ksi} \cdot 0.703\text{in}^3 = 10,700\text{"}\#$$

$$M_a = 19.5\text{ksi} \cdot 0.703\text{in}^3 = 13,700\text{"}\#$$

For welded 6061-T6 posts:

$$M_a = 24\text{ksi} / 1.95 \cdot 0.703\text{in}^3 = 8,650\text{"}\#$$



2"x2" 3 PIECE POST

Infill slide in elements are not considered in post strength.

Section properties for main extrusion:

$$A = 1.17\text{in}^2$$

$$I = 0.698\text{in}^4$$

$$S = 0.698\text{in}^3$$

$$Z = 0.846\text{in}^3$$

$$b/t = 1.01"/0.19" = 5.3 \text{ local buckling does not control}$$

Allowable stress:

$$F/\Omega = 15.2\text{ksi for 6063-T6}$$

$$F/\Omega = 19.5\text{ksi for 6005-T61 or 6061-T6}$$

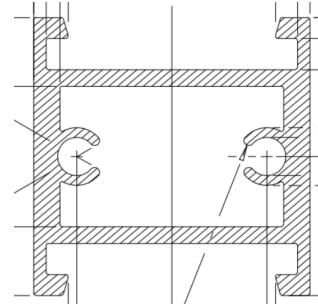
Allowable moment:

$$M_a = 15.2\text{ksi} * 0.846\text{in}^3 = 12,900\text{'\#}$$

$$M_a = 19.5\text{ksi} * 0.846\text{in}^3 = 16,500\text{'\#}$$

For welded 6061-T6 posts:

$$M_a = 24\text{ksi}/1.95 * 0.846\text{in}^3 = 10,400\text{'\#}$$

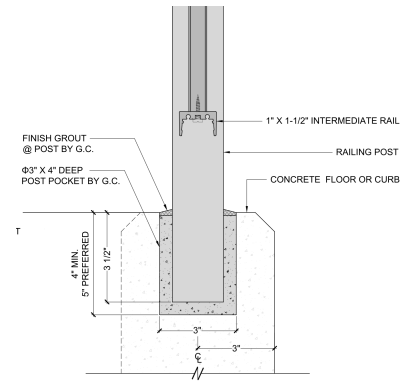


POST ANCHORAGE DETAILS**CORE MOUNT**

Assumed concrete strength $f'_c \geq 3,000$ psi

Check for concrete shear breakout or crushing failure modes.

Assume rectangular bearing reactions.

**Inputs**

Core Width (in)		Embed Width (in)		Concrete Strength		Edge Distance Embedment		Edge breakout calcs		
b_c (in)		b_s (in)		f'_c (psi)	λ	C_1 (in)	d (in)	w (in)	h (in)	β
3		1.5		3000	1	3	4	6	3.5	1.71428571
								Bearing Calcs		Strength
								$0.65 \cdot 0.85 f'_c / 1.6$	$(b_c - b_s) / 2 + b_s$	$(b_c - b_s) / 4 + d / 2$
α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2 + 4/\beta)\lambda\sqrt{f'_c}$ (psi)	$(2 + \alpha_s C_1 / b_s)\lambda\sqrt{f'_c}$ (psi)	$V_c C_1 b_s$	P_a (lbs)	f_s (psi)	b_b (in)	h_b (in)	P_a (lbs)	$\text{Min}(P_a) \cdot d / 2$
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,

$W = 151\text{pli}$

$C_d = 1.6$ for short duration live load

Spacing between rows of screws = 5.75"

$W' = 1.6 * 151\text{pli} = 241\text{pli}$

From above model for 4' post spacing max
load to single post = 162#

Load to upper screw pair =

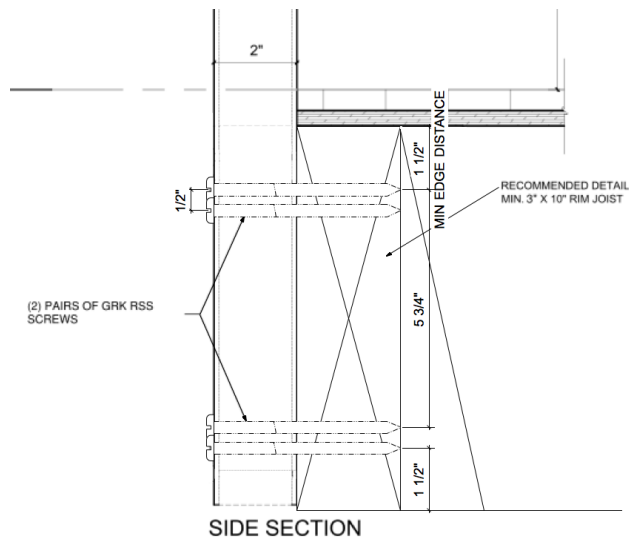
$162\# * (42'' + 2.75'' + 5.75'' + 1.5'') / (5.75'' + 1.5'')$
= 1,162#

Steel strength = 1,112# each OK

Required penetration = $(1,162\# / 2) / 241\text{pli} = 2.41''$

Required screw length = $2.41'' + 2'' \Rightarrow 4\text{--}1/2''$

3x10 minimum rim size.



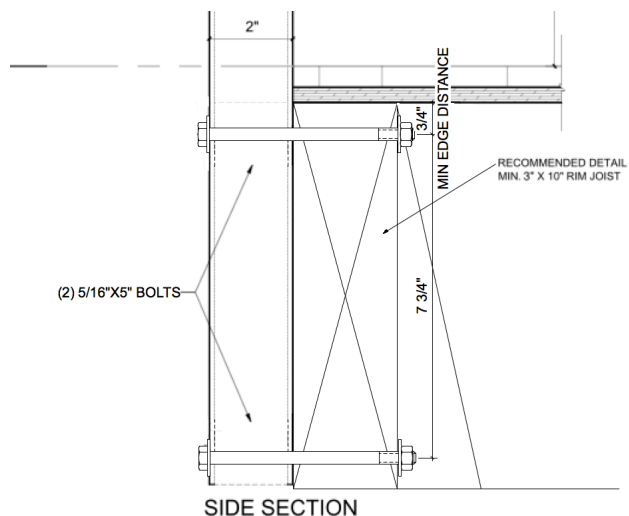
For posts made from the alternative higher strength allows increase embedment by 40%:

$L = 2.41 * 1.4 + 2 = 5\text{--}1/2''$

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 Simpson's instructions.

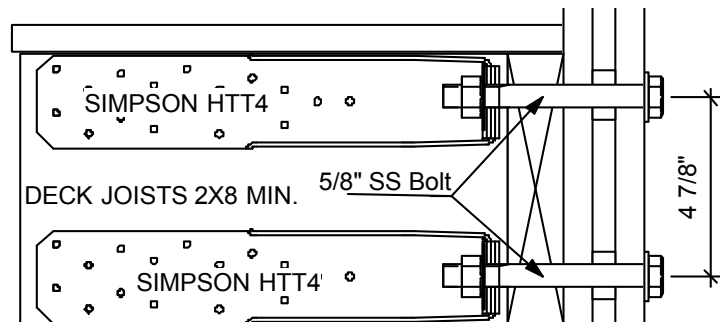
From Simpson load tables, $T_a = 3,000\#$

5/8" SS bolt strength, $T_a = 0.226\text{in}^2 \times 75\text{ksi} / 2 = 8,480\# > 3,000\#$ (Does not control)

Assuming 4-7/8" hold down spacing:

$a = 3000\# / (2" \times 744\text{psi}) = 2.02"$

Allowable load at top of post = $3,000\# \times (6.06" - 2.02" / 2) / (42" + 2" + 6.06" - 2.02" / 2) = 309\#$



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.

$D_r = 0.680"$

$Z = 0.05241\text{in}^3$

$M_a = 0.05241\text{in}^3 * 75\text{ksi} / 2 = 1,970\#\text{'}$

Allowable load, $1,970\#\text{'}/(16\text{'}/8) = 2,960\#$

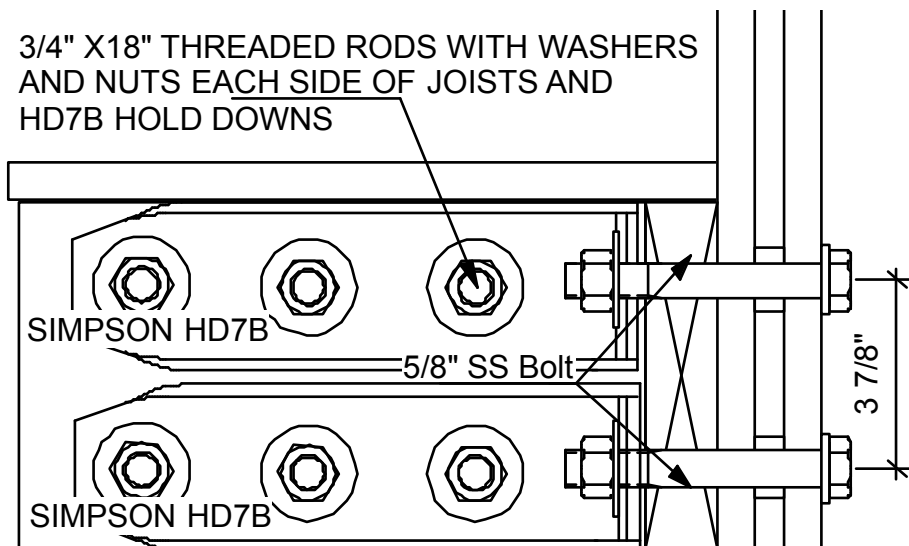
Assuming 3-7/8" hold down spacing:

$a = 2,960\# / (2\text{'}/2 * 744\text{psi}) = 1.99\text{'}$

Allowable load at top of post = $2,960\#(6.06\text{'}/2 - 1.99\text{'}/2) / (42\text{'}/2 + 2\text{'}/2 + 6.06\text{'}/2 - 1.99\text{'}/2) = 306\#$

Wind load that causes equivalent loading on anchorage = $8,400\#\text{'}/(4\text{'}/2 * 3\text{'}/2 * 31.1\text{'}) = 22.5\text{psf}$

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.

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 $f'_c = 5000\text{psi}$ minimum for grout

$$P_a = 0.65 * 0.85 * 1.5'' * 2'' * 5000\text{psi} / 1.6 = 5,180\#$$

$$M_a = 5,180\# * 2'' = 10,400''\#$$

Check welding to structural steel:

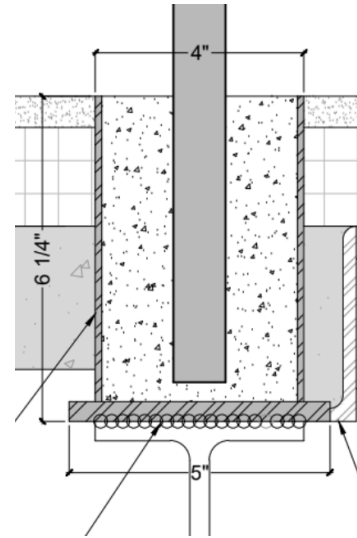
Assume 3-1/2" long 3/16" welds on two sides.

$$I_w = 2 * 3.5''^3 / 12 = 7.15\text{in}^4/\text{in}$$

Weld loading is primarily normal to the axis so the strength may be multiplied by 1.5.

$$R_n / \Omega = 1.5 * 928 * 3 * 1'' = 4,180\text{pli}$$

$$M_a = 4,180\text{pli} * 7.15\text{in}^4/\text{in} / 1.75'' = 17,100''\#$$



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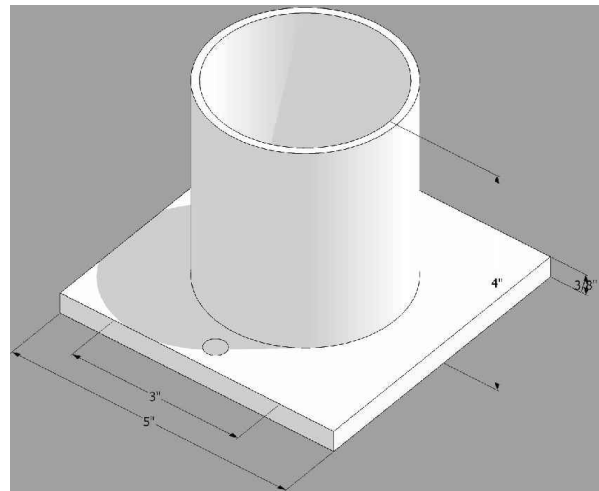
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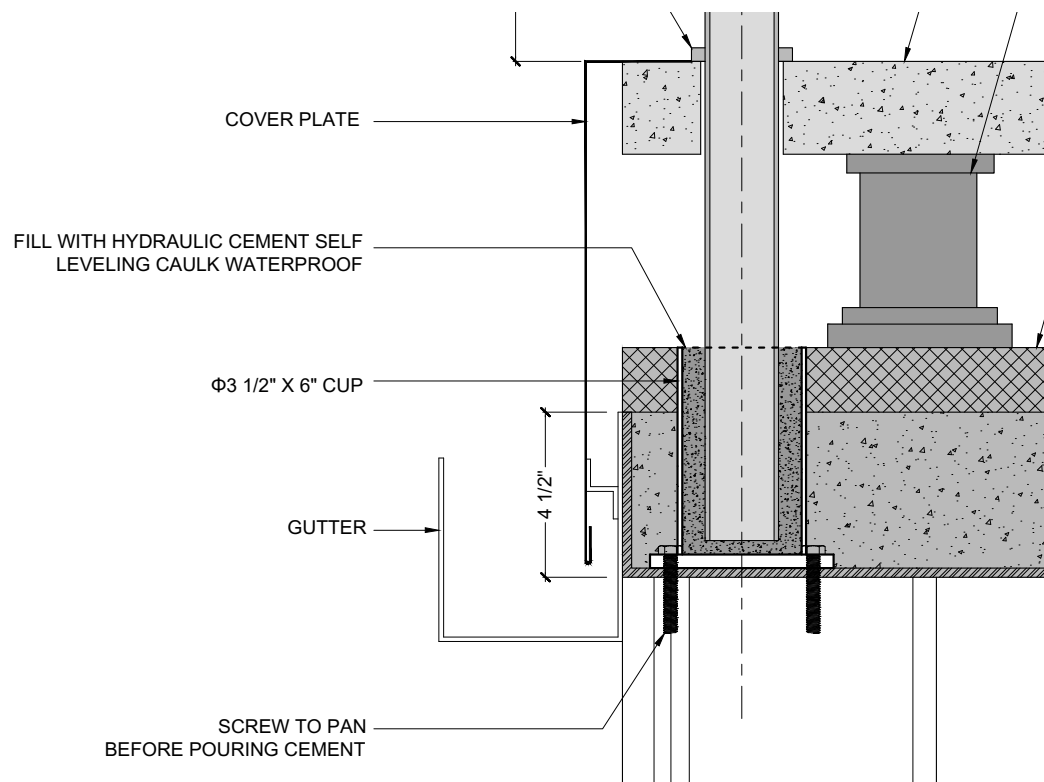
CUP MOUNTING - RECEIVER MOUNTED POST

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.



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-

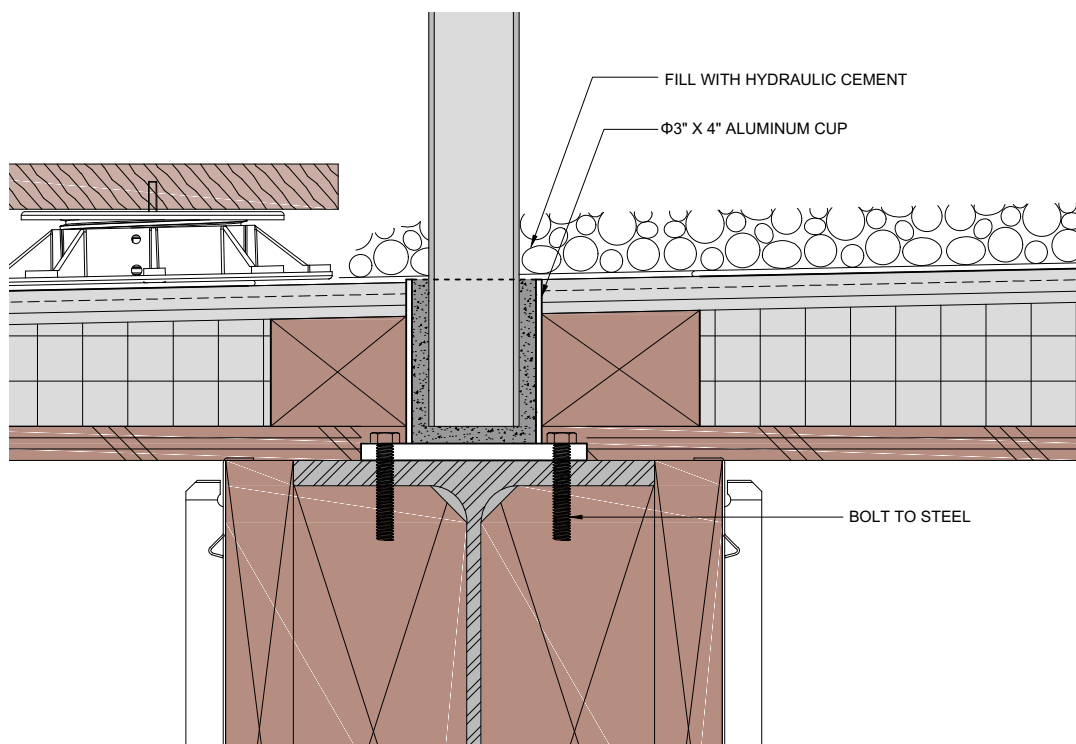
$$Z = (3.25^3 - 3^3)/6 = 1.221 \text{ in}^3$$

$$F_{twa} = 24\text{ksi} * 1.221/1.95 = 15,032\text{"}\# \text{ allowable moment on the cup when not encased.}$$

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

$$T_n = 0.0775\text{in}^2 * 75\text{ksi} = 5,812\#$$

$$M_n = 5,812\# * 3.75\text{' } * 2 = 43,590\text{'"}\#$$

Thus will support the full post strength.

Check welds to structural steel:

Assume at least 3" long 3/16" welds on two sides.

$$I_w = 2 * 3\text{' }^3 / 12 = 4.5\text{in}^4/\text{in}$$

Welds will be loaded primarily normal to their axis, so the strength may be multiplied by 1.5.

$$R_n/\Omega = 1.5 * 928 * 3 * 1\text{' } = 4,180\text{pli}$$

$$M_a = 4,180\text{pli} * 4.5\text{in}^4/\text{in}/2\text{' } = 9,410\text{'"}\#$$

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CUP MOUNTING TO WOOD:

Baseplate mounted receiver (same as steel but attached to wood)

$$T_a = 1.6 * 263 \text{ pli} * 4.25'' = 1,788''\#$$

$$a = 2 * 1788 / (605 \text{ psi} * 5'') = 1.182''$$

$$M_a = 1,788 * 2 * (4.25 - 1.182 / 2) = 12,190''\#$$

Inset in wood:

The receiver is inset into wood by inlaying into 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

$$Z' = 1.6 * 99 = 158\# \text{ each (NDS Table 12N } G = 0.43)$$

$$n = 2089 / 158 = 14 \text{ nails to connect plies, each ply}$$

#12 x 6'' screws:

$$Z' = 1.6 * 128 = 205\# \text{ each (NDS Table 12L } G = 0.43)$$

$$n = 2089 / 205 = 11 \text{ Screws to connect plies}$$

#14 x 6'' screws

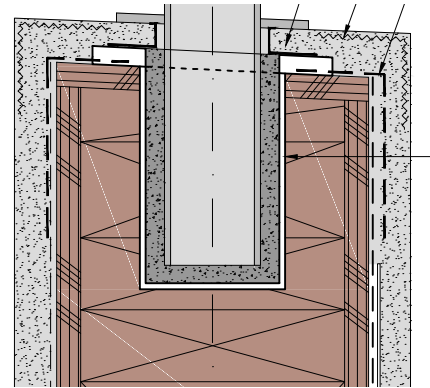
$$Z' = 1.6 * 141 = 226\# \text{ each (NDS Table 12L } G = 0.43)$$

$$n = 2089 / 226 = 10 \text{ Screws to connect plies}$$

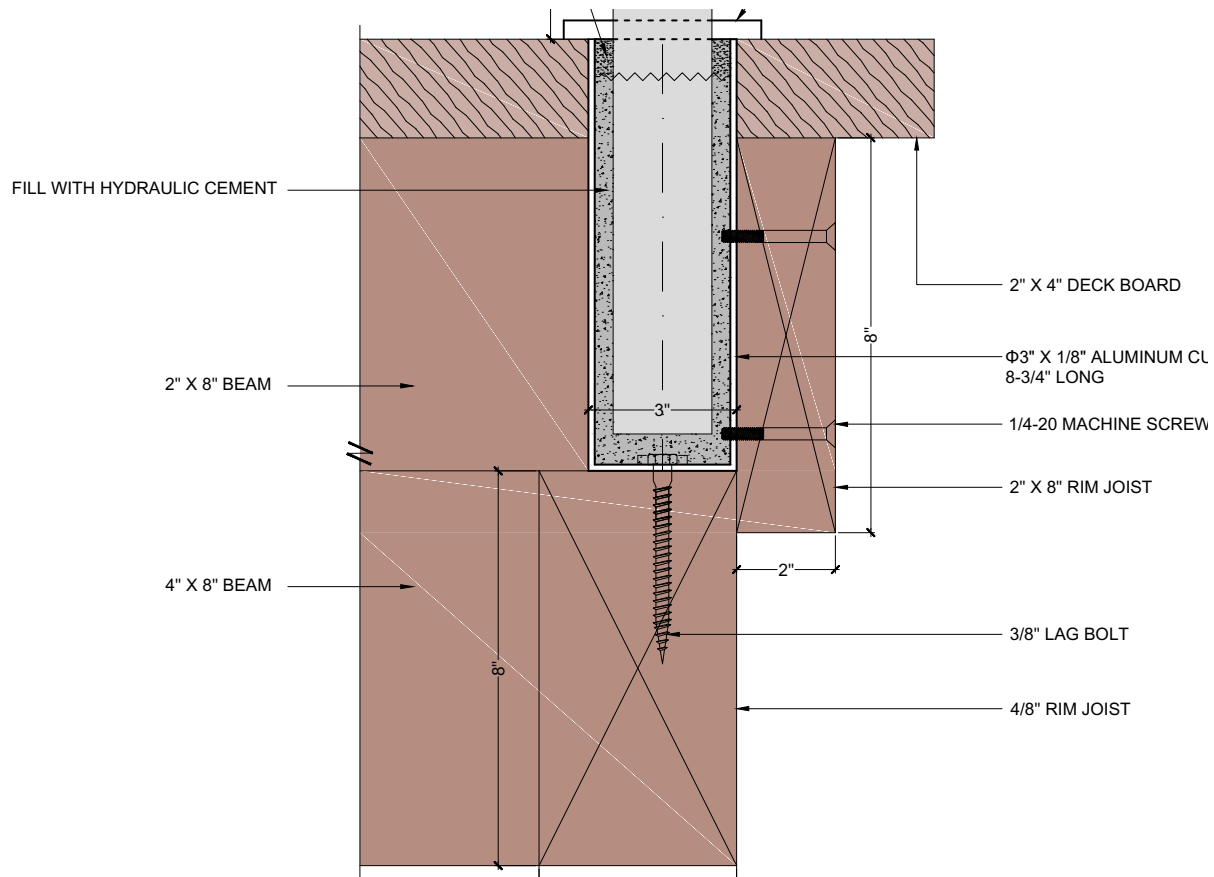
5/16'' x 6'' GRK RSS (ESR-2442) or Fastenmaster Timberlok (ESR-1078) screw

$$Z' = 1.6 * 210 = 336\# \text{ each (G = 0.42)}$$

$$n = 2089 / 336 = 7 \text{ Screws to connect plies}$$



Cup Mounting 1:



To wood with receiver clamped by a block on side.

Required pullout strength of blocking:

$$R = (200\# \times 50\text{'}) / 7.25\text{' } = 1,380\#$$

#12 screws:

$$W' = 1.6 \times 114\text{pli} \times 1.5\text{' } = 274\# \text{ each (NDS Table 12.2B, } G = 0.43)$$

$$n = 1380 / 274 = 6 \text{ screws}$$

5/16" x 6" GRK RSS (ESR-2442) or Fastenmaster Headlok (ESR-1078) screw

$$W' = 410\# \text{ each}$$

$$n = 1380 / 410 = 4 \text{ screws}$$

3/4" x 2-1/2" flat cap rail

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

$$\begin{aligned} I_{xx} &= 0.07226 \text{ in}^4 \\ S_{xx} &= 0.1842 \text{ in}^3 \\ I_{yy} &= 0.6394 \text{ in}^4 \\ S_{yy} &= 0.5123 \text{ in}^3 \end{aligned}$$
$$\begin{aligned} M_{a,x} &= 0.1842 \text{ in}^3 * 15.2 \text{ ksi} = 2,800''\# \\ M_{a,y} &= 0.5123 \text{ in}^3 * 15.2 \text{ ksi} = 7,787''\# \end{aligned}$$

For single span condition:
 $M = PL/4$
 $2,800'' \# = 200 \# L/4$
 $L = 56''$

$$\begin{aligned} M &= (13/64)PL \\ 2,800'' &= (13/64)200\#L \\ L &= 69'' \end{aligned}$$
$$\Delta = 200 * 56^3 / (48 * 10.1 \times 10^6 * .07226) = 1''$$

Deflection limits may control allowable top rail span.

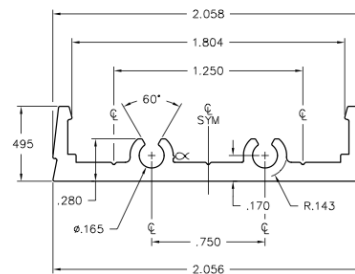
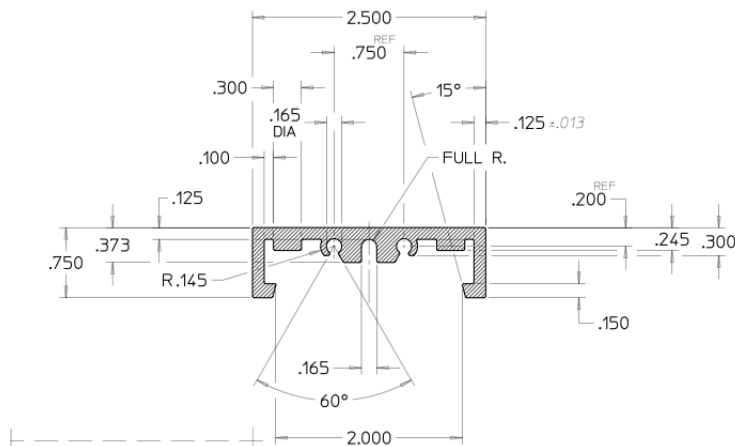
VQ/I

V=100# (200# load at mid point)

$$Q=0.0974$$

$$v = 100 \# \cdot .0974 / .07226 / 2 = 68 \text{ pli} = 810 \text{ plf}$$

Top rails are screwed to posts with #10 screws



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Strength of #10 SS screw (ASTM F 879 Table 3)

(6) screws into insert slots.

$$T_n = 1,330\#$$

$$V_n = 0.6T_n = 0.6 * 1,330\# = 798\# \text{ for shear through threads}$$

Strength into screw slots:

For withdrawal:

$$T_n > 0.29D_u F_{tu} = 0.29 * 0.19'' * 1'' * 30\text{ksi} = 1,653\#$$

For shear:

$$V_n = d_e F_{tu} = 0.1 * (0.29 * 1'') * 30\text{ksi} = 870\#$$

Bearing on post:

$$V_a = 2 * F_{ut1} D / 3 = 2 * 30\text{ksi} * 0.19 * 0.125 / 3 = 475\#$$

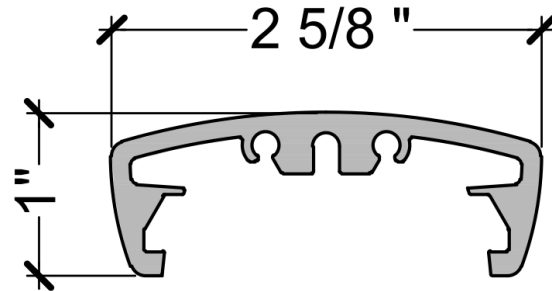
Connection strength is adequate for all design load conditions.

A11 TOP RAIL

Alloy 6063-T6 Aluminum

Allowable Stress

ADM Table 2-21

 $F_c/\Omega = 15.2 \text{ ksi}$ 

Check lateral torsional buckling about strong axis:

Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1

$$r_{ye} = ((I_y)^{1/2}/S_x(C_w + 0.038JL_b^2)^{1/2})^{1/2}$$

$$r_{ye} = ((0.0519)^{1/2}/0.0788*(0.0495 + 0.038*0.00568*48^2)^{1/2})^{1/2} = 1.462''$$

$$\lambda = 48''/(1.462''*1) = 32.83$$

$$M_{nmb} = M_p(1 - \lambda/C_c) + \pi^2 E I_{xc}/C_c^3$$

$$M_p = 30 \text{ ksi} * 0.5583 \text{ in}^3 = 16.75 \text{ kip-in}$$

$$M_{nmb} = 16.75 \text{ kip-in} (1 - 32.83/78) + \pi^2 10,100 * 32.83 * 0.4264/78^3 = 12.64 \text{ kip-in}$$

$$M_a = 12.64 \text{ kip-in} / 1.65 = 7,660''\#$$

Bending about the weak axis:

Lateral torsional buckling and local buckling does not apply.

$$M_a = 0.1536 \text{ in}^3 * 15.2 \text{ ksi} = 2,330''\#$$

Find maximum on center post spacing:

$$L_{\max} = 2,330''\# * 4/200\# + 2'' = 48.6'' \text{ For single span condition}$$

$$L_{\max} = 2,330''\# * (64/13)/200\# = 57.4'' \text{ For multiple span condition}$$

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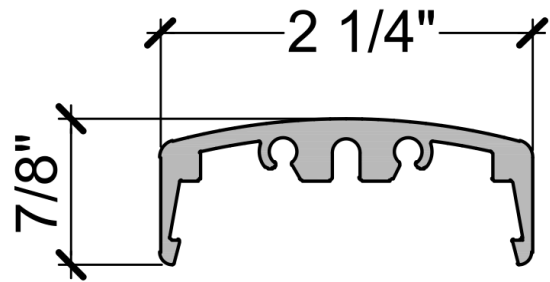
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

A111 TOP RAIL

Alloy 6063-T6 Aluminum

Allowable Stress

ADM Table 2-20

 $F_c/\Omega = 15.2 \text{ ksi}$ 

Check lateral torsional buckling about strong axis:

Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1

$$r_{ye} = ((I_y)^{1/2}/S_x(C_w + 0.038JL_b^2)^{1/2})^{1/2}$$

$$r_{ye} = ((0.02047)^{1/2}/0.2458*(0.01183 + 0.038*0.00400*48^2)^{1/2})^{1/2} = 0.5918''$$

$$\lambda = 48''/(0.5918''*1) = 81.11$$

$$M_{nmb} = \pi^2 E S_x / \lambda^2$$

$$M_{nmb} = \pi^2 * 10,1000 * 0.2458 / 81.11^2 = 3,720''\#$$

$$M_a = 3,720''\#/1.65 = 2,260''\#$$

Bending about the weak axis:

Lateral torsional buckling and local buckling does not apply.

$$M_a = 0.07620 \text{ in}^3 * 15.2 \text{ ksi} = 1,160''\#$$

Include contribution from sub rail:

Assume 520E sub rail, $I = 0.00589 \text{ in}^4$

$$\text{Top rail load share} = 0.02047 \text{ in}^4 / (0.02047 \text{ in}^4 + 0.00589 \text{ in}^4) = 0.7766$$

$$M_a = 1,160''\#/0.7766 = 1,490''\#$$

Find maximum top rail span:

$$L_{\max} = 1,490''\# * 4/200\# = 31.8'' \text{ For single span condition}$$

$$L_{\max} = 1,490''\# * (64/13)/200\# = 36.7'' \text{ For two span condition}$$

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

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

Find maximum on center post spacing:

$$L_{\max} = 1,910''\# * 4/200\# + 2'' = 40.2'' \text{ For single span condition}$$

$$L_{\max} = 1,910''\# * (64/13)/200\# = 47.0'' \text{ For two or more spans condition}$$

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

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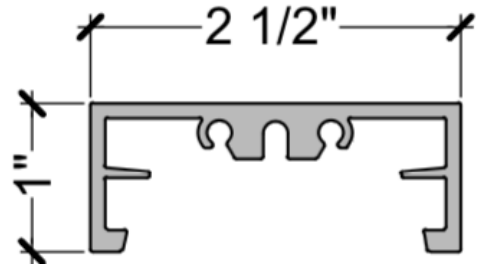
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A222 TOP RAIL

Alloy 6063-T6 Aluminum

Allowable Stress

ADM Table 2-21

 $F_c/\Omega = 15.2 \text{ ksi}$ 

Check lateral torsional buckling about strong axis:

Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1

$$r_{ye} = ((I_y)^{1/2}/S_x(C_w + 0.038JL_b^2))^{1/2}$$

$$r_{ye} = ((0.05785)^{1/2}/0.3847(0.06424 + 0.038 * 0.00399 * 48^2))^{1/2} = 0.6341 \text{ in}$$

$$\lambda = 48''/(0.6341'' * 1) = 75.70$$

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

$$M_p = 25 \text{ ksi} * 0.4866 \text{ in}^3 = 12.17 \text{ kip-in}$$

$$M_{nmb} = 12.17 \text{ kip-in}(1 - 75.70/78) + \pi^2 * 10,100 * 75.70 * 0.3847/78^3 = 6.465 \text{ kip-in}$$

$$M_a = 6.465 \text{ kip-in}/1.65 = 3,920''\#$$

Bending about the weak axis:

Lateral torsional buckling and local buckling does not apply.

$$M_a = 0.1512 \text{ in}^3 * 15.2 \text{ ksi} = 2,300''\#$$

Find maximum on center post spacing:

$$L_{\max} = 2,300''\# * 4/200\# + 2'' = 48.0'' \text{ For single span condition}$$

$$L_{\max} = 2,300''\# * (64/13)/200\# = 56.6'' \text{ For two or more spans condition}$$

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A225 TOP RAIL

Alloy 6063-T6 Aluminum

Allowable Stress

ADM Table 2-21

 $F_c/\Omega = 15.2 \text{ ksi}$

Check lateral torsional buckling about strong axis:

Lateral torsional buckling strength is calculated

according to 2015 ADM F.4.2.1

$$r_{ye} = ((I_y)^{1/2}/S_x(C_w + 0.038JL_b^2)^{1/2})^{1/2}$$

$$r_{ye} = ((0.07776)^{1/2}/0.4498(0.06184 + 0.038*0.00660*48^2)^{1/2})^{1/2} = 0.4649 \text{ in}$$

$$\lambda = 48''/(0.4649''*1) = 103.2$$

$$M_{nmb} = \pi^2 E S_x / \lambda^2$$

$$M_{nmb} = \pi^2 * 10,1000 * 0.4498 / 103.2^2 = 4,210''\#$$

$$M_a = 4,210''\# / 1.65 = 2,550''\#$$

Bending about the weak axis:

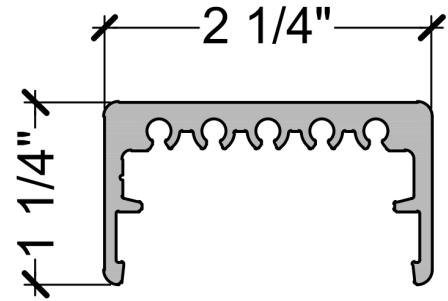
Lateral torsional buckling and local buckling does not apply.

$$M_a = 0.1715 \text{ in}^3 * 15.2 \text{ ksi} = 2,610''\#$$

Find maximum on center post spacing:

$$L_{\max} = 2,550''\# * 4 / 200\# + 2'' = 53.0'' \text{ For single span condition}$$

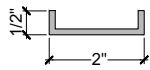
$$L_{\max} = 2,550''\# * (64/13) / 200\# = 62.8'' \text{ For two span condition}$$



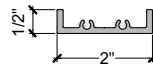
520 SUB RAILS

Sub rail options are:

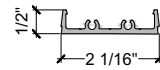
520,
520S and
520E



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:

$$R_{po} = [.1^3 * 3 * 10 \times 10^6 / (0.5^3)] * \sin 60 = 207 \text{ k/ft}$$

$$R_t = 0.1 * 12'' * 15 \text{ ksi} = 18 \text{ k/ft}$$

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

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

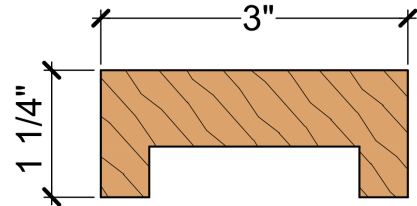
$C_f = 1.5$ for 2x4 size

Wood must not receive damage from moisture- must be sealed.

$F_b' = 2 * 1.5 * 1,400\text{psi} = 4,200\text{psi}$

Flat wise bending is also allowed a 1.15 flat use factor,

$F_b' = 1.15 * 4,200\text{psi} = 4,830\text{psi}$ (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, $F_b' = 3,310\text{psi}$.

C_L does not apply to flat wise bending.

Strong axis bending:

$S = 1.653\text{in}^3$

$M_a = 1.653\text{in}^3 * 3,310\text{psi} = 5,470''\#$

Weak axis bending:

$S = 0.3621\text{in}^3$

$M_a = 0.3621\text{in}^3 * 4,830\text{psi} = 1,750''\#$

Wood rails are always used with an aluminum sub rail.

Typical sub rail is the 520E

$I_x = 0.00589\text{in}^4$

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

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INFILL PICKETS**0.7" Diameter Round Picket**

$$I_x = 0.00809 \text{ in}^4; S_x = 0.02254 \text{ in}^3$$

$$I_y = 0.00897 \text{ in}^4; S_y = 0.02548 \text{ in}^3$$

Allowable stress for 6063-T6 aluminum:

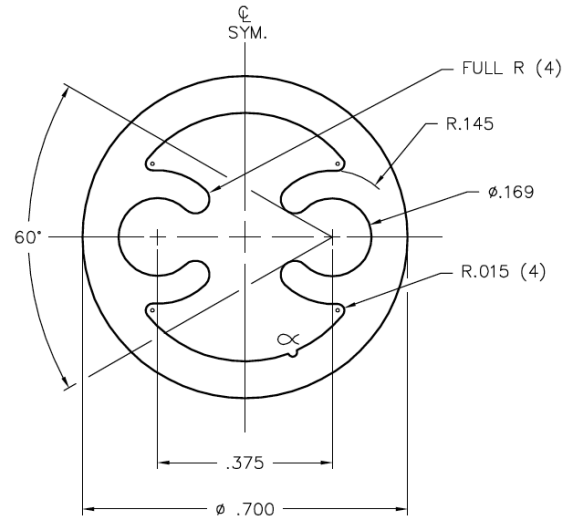
$$F_t = F_c = 17.7 \text{ ksi (ADM Table 2-24)}$$

$$M_a = 17.7 \text{ ksi} * 0.02254 = 3,989 \text{''\#}$$

Maximum moment on bottom rail from 50 lbs load at any location:

$$M = 50 \text{ \#} * L / 4 = 3,989 \text{''\#}$$

$$L = 319 \text{'' (Greatly exceeds practical spans)}$$

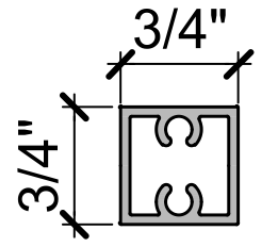
**3/4" Picket**

$$I_x = 0.0163 \text{ in}^4$$

$$S_x = 0.0435 \text{ in}^3$$

$$I_y = 0.0141 \text{ in}^4$$

$$S_y = 0.0376 \text{ in}^3$$



Allowable bending stress for 6005 aluminum

$$F_y / \Omega = 35 \text{ ksi} / 1.65 = 21.2 \text{ ksi}$$

$$\text{Or } F_u / \Omega = 38 \text{ ksi} / 1.95 = 19.5 \text{ ksi (controls)}$$

Minimum strength is about the y axis.

$$M_{a,y} = 0.0376 \text{ in}^3 * 19.5 \text{ ksi} = 733 \text{''\#}$$

$$\text{Max span} = 38 \text{''}$$

$$\text{Max picket spacing} = 4.25 \text{''}$$

Loading is applied over a 12"x12" square so loading is applied to at least 3 pickets.

$$M = (50 \text{ \#} / 3) * 38 \text{''} / 4 = 158 \text{''\#} < 733 \text{''\# OK}$$

$$\Delta = 50 \text{ \#} / 3 * 38 \text{''}^3 / (48 * 10.1 * 10^6 * 0.0141) = 0.134 \text{''}$$

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1-3/4"x5/8" Pickets

Section properties:

$$I_x = 0.0341 \text{ in}^4$$

$$S_x = 0.1072 \text{ in}^3$$

$$I_y = 0.2328 \text{ in}^4$$

$$S_y = 0.2642 \text{ in}^3$$

$$M_{a,x} = 0.1072 \text{ in}^3 * 15.2 \text{ ksi} = 1,630''\#$$

$$M_{a,y} = 0.2642 \text{ in}^3 * 15.2 \text{ ksi} = 4,020''\#$$

Max span = 48"

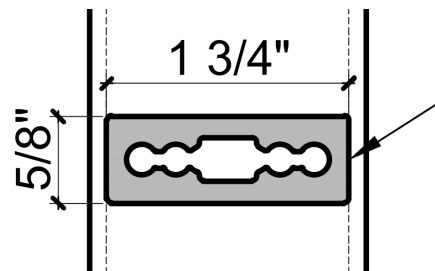
Check for 50# mid span:

$$M = 50\# * 48'' / 4 = 600''\# < 1,630''\# \text{ OK}$$

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:

$$I_x = 0.0204 \text{ in}^4$$

$$S_x = 0.0654 \text{ in}^3$$

$$I_y = 0.0816 \text{ in}^4$$

$$S_y = 0.130 \text{ in}^3$$

$$M_{a,x} = 0.0654 \text{ in}^3 * 15.2 \text{ ksi} = 994''\#$$

$$M_{a,y} = 0.130 \text{ in}^3 * 15.2 \text{ ksi} = 1,980''\#$$

Max span = 48"

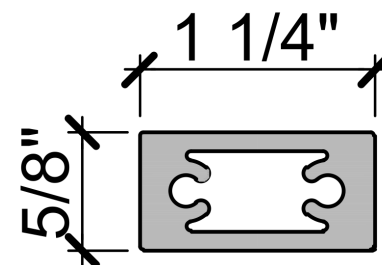
Check for 50# mid span:

$$M = 50\# * 48'' / 4 = 600''\# < 994''\# \text{ OK}$$

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

$$I_y = 0.1698 \text{ in}^4; S_y = 0.2189 \text{ in}^3$$

Allowable stress for 6063-T6 aluminum:

$$F_t = F_c = 15.2 \text{ ksi (ADM Table 2-24)}$$

$$M_a = 15.2 \text{ ksi} * 0.2189 = 3,330 \text{''\#}$$

Maximum moment on bottom rail from 50 lbs load at any location:

Maximum span (face to face of posts): 58''

$$M = 50 \text{ \#} * 58 / 4 = 725 \text{''\#} \leq 3,940 \text{''\#}$$

Live load deflection:

$$\Delta = 50 * 48^3 / [48 * 10 \times 10^6 * 0.1698] = 0.0678 \text{''}$$

Connection to post:

(2) #10 screws through post wall into rail:

Strength of #10 SS sheet metal screw (ASTM F 879 Table 3)

$$T_n = 1,330 \text{ \#}$$

$$V_n = 0.6 T_n = 0.6 * 1,330 \text{ \#} = 798 \text{ \# for shear through threads}$$

Strength into screw slots:

For withdrawal:

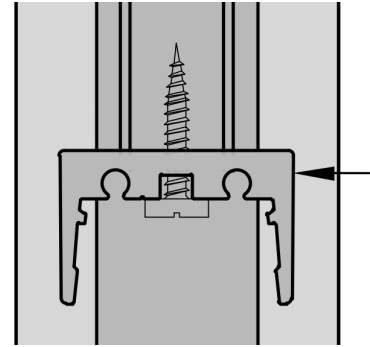
$$T_n > 0.29 D L_u F_{tu} = 0.29 * 0.19 \text{''} * 1 \text{''} * 30 \text{ ksi} = 1,653 \text{ \#}$$

For shear:

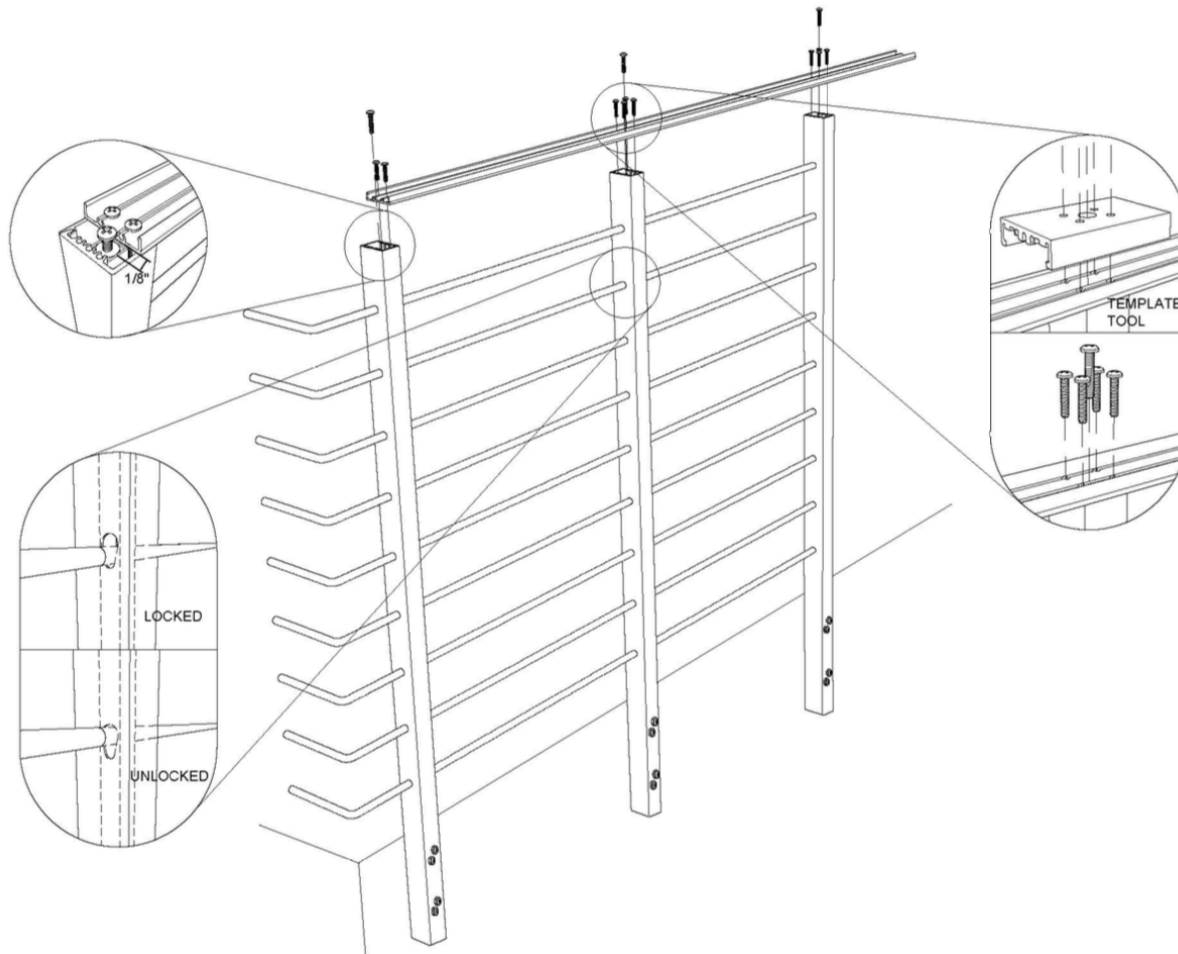
$$V_n = d_e t F_{tu} = 0.1 * (0.29 * 1 \text{''}) * 30 \text{ ksi} = 870 \text{ \#}$$

Bearing on post extrusion:

$$V_a = 2 * F_{ut1} D / 3 = 2 * 30 \text{ ksi} * 0.19 * 0.125 / 3 = 475 \text{ per screw.}$$



HORIZONTAL BAR INFILL CONTINUOUS BAR OPTION WITH WEDGE LOCK CLAMP



1/2" Diameter solid stainless steel rods installed horizontally through the posts which are locked in place by the wedge lock insert when it is tightened. The wedge lock uses a teardrop shaped hole to wedge the bar tight when the 4 screws at the top are tightened pulling the wedge lock upward inside the post, tight against the bars so it is locked together.

Check bar strength:

1/2 diam:

$$I = \pi D^4 / 64 = \pi * .5^4 / 64 = 0.00307 \text{ in}^4$$

For 4' post spacing the bars span 46"

For 50 lb load on bars:

Check deflections:

$$\Delta = PL^3 / (48EI) = 50 * 46^3 / (48 * 27 * 10^6 * 0.00307) = 1.223''$$

Because of high flexibility of the bar it will act more like a tension element also.

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Determine the deflection/tension required to resist the 50# load assuming 50% split between tension and bending resistance.

Try

$$\Delta = 0.611''$$

$$T = [0.5 * 50\# * 46/4] / 0.611 = 471\#$$

Tension resistance of wedge lock system to prevent rod from sliding:

(2) 1/4 screws minimum (end of run) tightened to 75''# torque each-

$$\text{Grip strength} = (2) * 75''\# * 20/'' = 3,000\#$$

$$\text{Resistance} = 3,000\# / 3 = 1,000\# \geq 471\#$$

The tension load on the post will be resisted by the compression in the adjacent bars.

Compression buckling resistance:

$$kL/r = 1 * 46 / (.5/4) = 368$$

$$F_e = \pi^2 E / (L_c/r)^2 = \pi^2 * 28 \times 10^6 / 368^2 = 2040 \text{ psi}$$

$$C_a = 0.877 * 2040 * 0.196 = 351\#$$

$$2 * 351 = 702\# \geq 471\#$$

SCREWED BAR OPTION (used for single span or where the wedge lock can't be used):

1/2'' Diameter solid stainless steel rods installed horizontally between the posts.

Rods will be installed to the posts by drilling a 3/16'' diameter hole x 1'' deep hole in the end and screwing to the post with a #10 x 3/4'' SS screw

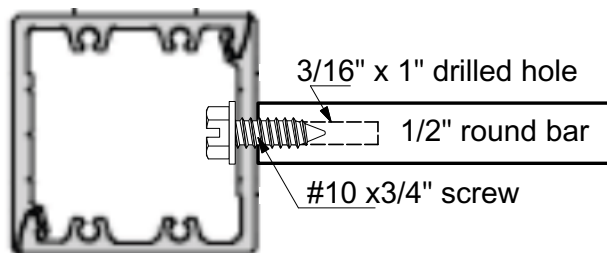
Screw tension strength:

#10 screw strength from ESR1976:

$T_a = 885\#$ (thread engagement is adequate to develop full screw strength)

$$V_a = 573\#$$

Bar strength and screw tension strength will be adequate to safely support the 50 lb infill load.

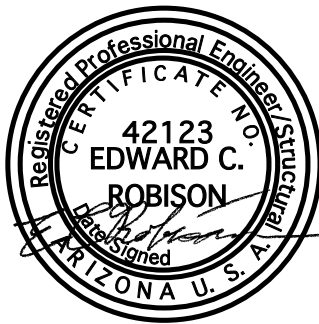


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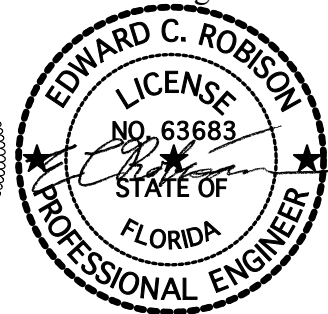
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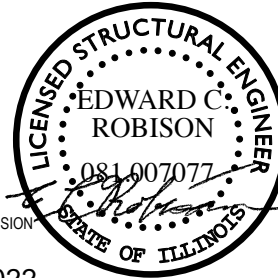
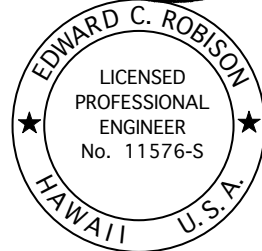
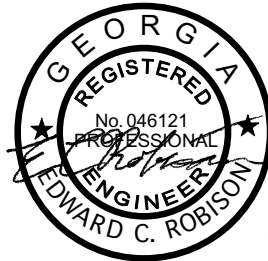
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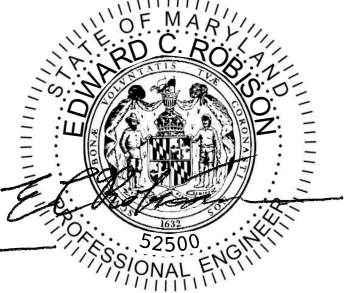
EXP 03/31/2023



EXP 02/28/2023



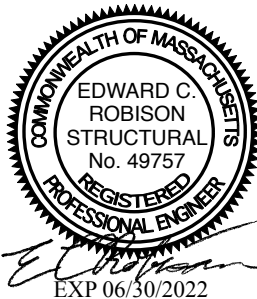
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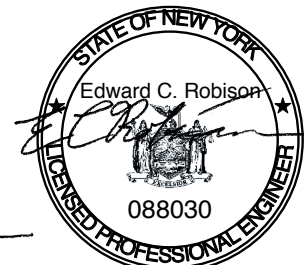
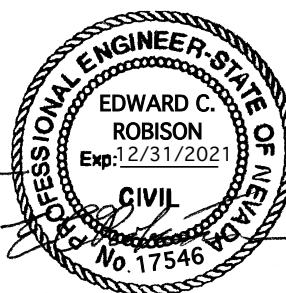
THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION

Signature Edward C. Robison Expiration Date of the License 04/30/2022

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland, License No. 52500, Expiration Date: 04/09/2022

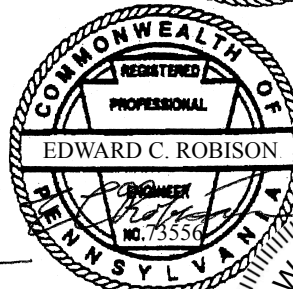


EXP 06/30/2022



EXP 03/31/2024

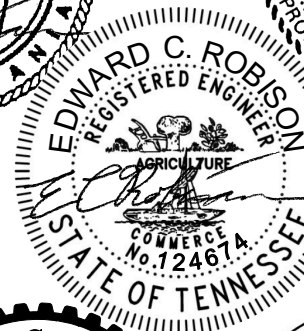
STATE OF MINNESOTA
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.
Signature Edward C. Robison Typed or printed name: Edward C. Robison
Date 04/30/2022 Lic. No. 58604



EXP 09/30/2021



EXP 12/31/2022



FIRM #F-12044
EXP 12/31/2021

