Edward C. Robison, P.E.

03 April 2019

C.R. Laurence Co., Inc.
2503 East Vernon
Los Angeles, CA 90058

SUBJ: CRL P-SERIES POST RAILING SYSTEMS
STAINLESS STEEL POSTS FOR GUARDRAILS

I have reviewed the design drawings for the stainless steel post kits to verify that they will safely support the following loads when used in building guardrails, 42" total rail height:

200 pound point load on top rail, vertical or horizontal
50 plf load on top rail, vertical or horizontal or
25 psf uniform load on glass panel horizontal or
50 lb conc load on 1 sf
25 psf wind load (90 mph (3 sec gust) exposure C)

Allowable post spacing is as shown on page 3.

For these conditions the railing meets all requirements of the 2006, 2009, 2012 and 2015 International Building Codes. Stainless steel components are designed in accordance with SEI/ ASCE 8-02 *Specification for the Design of Cold-Formed Stainless Steel Structural Members* or AISC Design Guide 27 *Structural Stainless Steel* as applicable.

If you have any questions please call me at 253-858-0855.

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THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION

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

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POST SUMMARY

	Allowable[1]	To Concrete[2]	To Wood[3]	Commercial[4]	Residential[5]
Post	Post Strength Moment in-lb[7]	Post Moment in-lb[7]	Post Moment in-lb[8]	Max. Post Spacing ft.	Max. Post Spacing ft.
P1	13,881"#	11,053"#	8,400"#	5.263'	6.00'
P2[6]	13,056"#	11,053"#	8,400"#	5.263	6.00'
P3[6]	9,259"#	9,259"#	8,400"#	4.409'	5.00'
P4[6]	15,435"#	11,053"#	8,400"#	5.263'	6.00'
P5[6]	8,982"#	8,982"#	8,400"#	4.277'	5.00'
P6	10,084"#	8,400"#	8,400"#	4.800'	6.00'
P7	10,084"#	8,400"#	8,400"#	4.800'	6.00'
P8[6]	12,573"#	11,053"#	8,400"#	5.263	6.00'
P9[6]	see (9)	11,053"#[(9)]	8,400"#	see (9)	6.00'

(1) For anchorage to steel with 3/8" bolts the full allowable post strength is developed for all posts.
(2) Anchorage to concrete is as shown on page 34 for 5" baseplate and page 36 for round baseplate.
(3) Anchorage to wood is as shown on page 35 for 5" baseplate. For posts P6 and P7 use round baseplate with 4 hole option with 1/2" lag screws.
(4) Based on 50 plf live load. Limited to 4' on center for all post types mounted to wood and posts P6 and P7 mounted to concrete. Limited to 4.18' for installations using the swivel connection at top
(5) Based on 200# concentrated load.
(6) Posts must have lateral bracing, shear resistant infill panels (3/8" minimum tempered glass or other material with adequate shear strength) or top rail attached to solid support at ends.
(7) Wind load induced moments shall not exceed this value.
(8) Wind load induced moments may be increased to Moment*1.20 when mounted to wood accounting for increase in C_D from 1.33 for guard live load to 1.6 for wind load.
(9) Post strength dependent on the specified stanchion length see page 38.

FASCIA BRACKETS - Posts P1, P3, P6, P7, P8
For attachment to steel - 5' post spacing, allowable wind moment ≤ 12,000"# or post strength
For attachment to concrete - 4' post spacing, allowable wind moment ≤ 9,600"#
For attachment to wood - 4' post spacing, allowable wind moment ≤ 10,944"# or post strength
For interior and protected installation up to 5' post spacing
Allowable post moments at at top of fascia bracket.
P8 post is welded to 5x7x3/8" plate for fascia bracket.

Alternative anchorage may be designed for project specific conditions.

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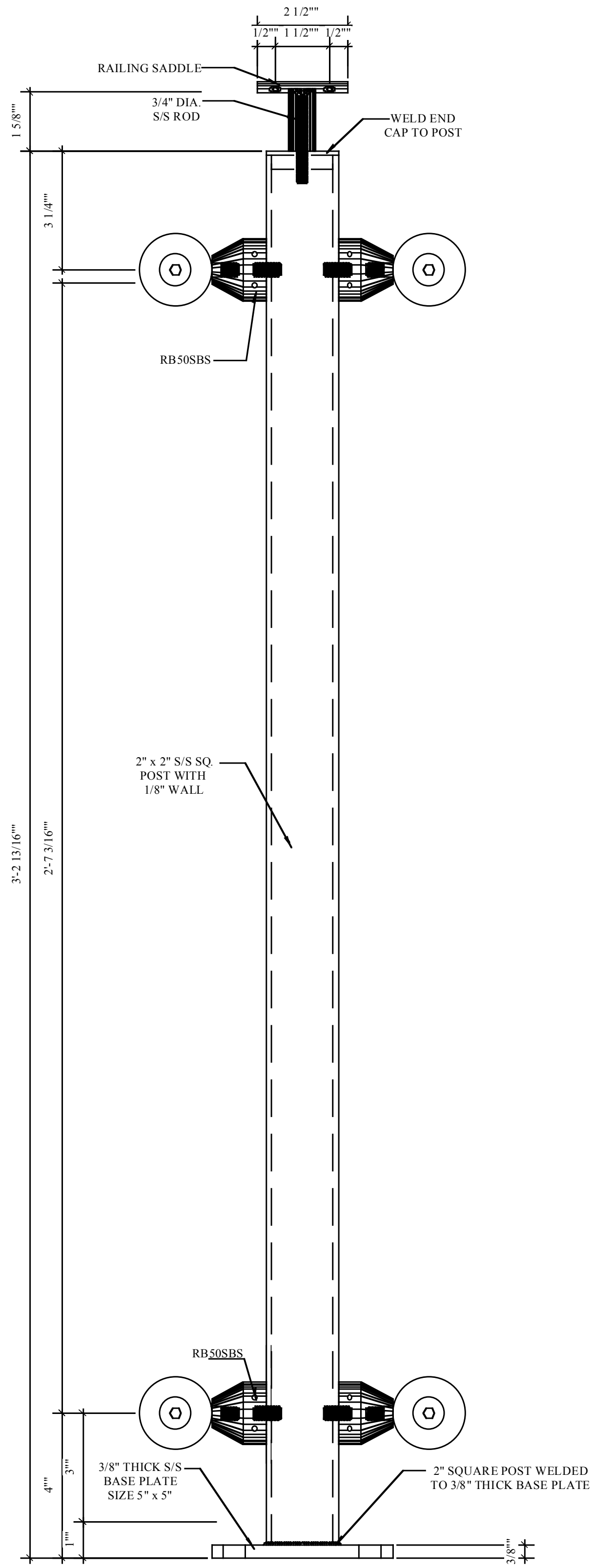
P1 POST

2”x2”x 3/16” 304 Stainless steel tube



Post Strength
S_{yy} = 0.618 in^3
F_y = 30 ksi SEI/ASCE 8-02 Table A1
Reserve strength method.
M_n = 1.25*0.618 in^3 *30 ksi = 23,181”#
Service moment
$M_s = ø\ M_n /1.67$
M_s = 23,181”# /1.67 = 13,881”#

Allowable top rail load for 42” total height:
P = 13,881#”/42” = 330#

Allowable load on lite based on 42” post height – 20” to center of lite
W = 13,881#”/20” = 694#
based on 5’ x 3’ lite
w = 694#/(5’*3’) = 46.3 psf

Weld to base plate : 3/16” fillet weld all around

Post Deflection:
Top of post deflection from 200# load:
Δ = 200#*H”^{3}/(3EI)
I = 0.618 in^4
For 42” post height:
Δ = 200#*42”^{3}/(3*27x10^6*0.618) = 0.296”

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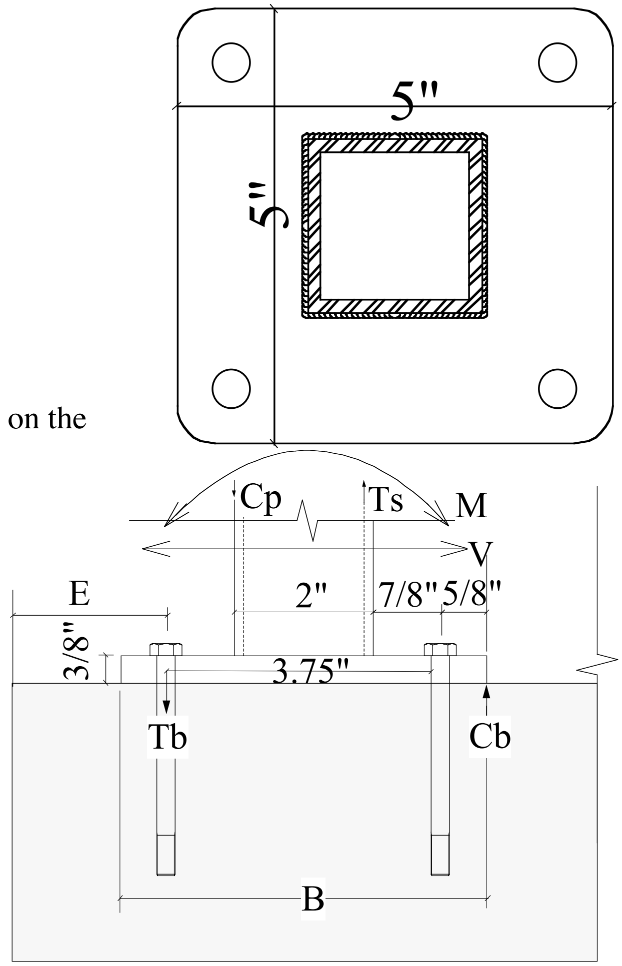
P1 POST continued:

Base Plate design:



for 3/8" plate $Z = \frac{5" \bullet 3/8^2}{4} = 0.176\ in^3$

$F_y = 45$ ksi

$M_n = Z\ F_y$

$M_n = 0.176*45ksi = 7{,}910\#"$

$M_s = M_n/1.67$

$M_s = 7{,}910\#"/1.67$

$M_s = 4{,}737\#"$

Calculate base plate reactions and moment based on the nominal strength of the posts.

$M = 19{,}805"\#$

$T_b = M/4.125"/2$ bolts

$T_b = 19{,}805/(4.125*2) = 2400\#$

Nominal anchor tension

Base plate moment

$M_u = 2*Tb*7/8"$

$M_u = 2*2400*(7/8") = 4201\#"$

$M_u < øM_n$ therefore okay

Service tension

$T_{bs} = 11{,}140\#"/(4.125*2) = 1{,}350\#$

Base plate anchor strength:

Service strength required for anchors

$T_s = 1{,}350\#$ (for allowable load on anchor)

Maximum allowable post moment based on base plate strength:

$M_{max} = 4{,}737\#"*2.5"/0.875" = 13{,}534"\#$

THIS BASE PLATE MAY BE USED WITH ANY OF THE POSTS IN THIS SERIES. THE STRENGTH AND ANCHORAGE WILL BE THE SAME FOR ALL POST TYPES.

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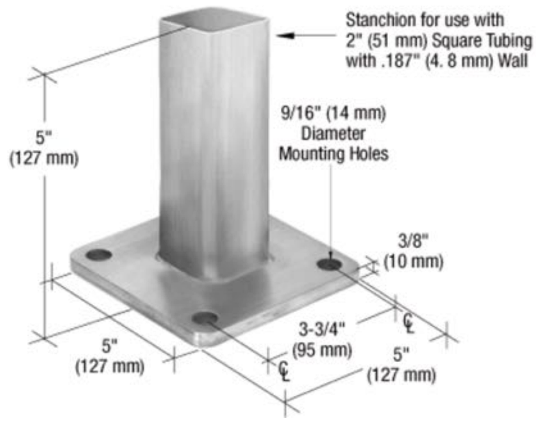
P1 POST continued:
Stanchion mounted posts
Surface Mount Stanchions
Cast stainless steel - stanchion cast integrally with the base plate
Fits 2x2x3/16" 316 stainless steel post



$I_{xx}=I_{yy}=0.6663in^4$
$S_{xx}=S_{yy}=0.6663in^3$
$Z_{xx}=Z_{yy}=0.8377in^3$
$r_x=r_y=0.7257in$
Determine flexural strength of stanchion according to AISC design guide 27.
b/t=1.625/.1875=8.667
$1.12(28,000/30)^{1/2}=34.2>8.67$ (Compact elements)
$L_b=5"$
$L_p=0.5*0.7257"*(28,000ksi/30ksi)^{1/2}=11.1"$
$L_b<.75L_p$ therefore continuous strength method applies

$\lambda_p=1.625/.1875(30*12*(1-0.27^2)/(28,000*\pi^2*.425))^{1/2}=0.4620<0.68$ CSM applies
$\varepsilon_{csm}/\varepsilon_y=.25/.462^{3.6}=4.029$
$\varepsilon_u=1-30/75=0.6$
$\varepsilon_y=30/28000=0.00107$
$E_{sh}=(75-30)/(0.16*0.6-0.00107)=474ksi$
$M_p=30ksi*0.8377in^3=25.131in\text{-}kips$
$M_n=25.131(1+(474*0.6636)/(28000*.8377)*(4.029-1)-(1-.6663/.8377)/(4.029)^2)=25.835k\text{-}in$
$M_n/\Omega = 25,835"\#/1.67 = 15,470"\#$
Stanchion exceeds post strength so will not lower strength of system.

Base plate mounting is the same as for the standard 5" square base plate.

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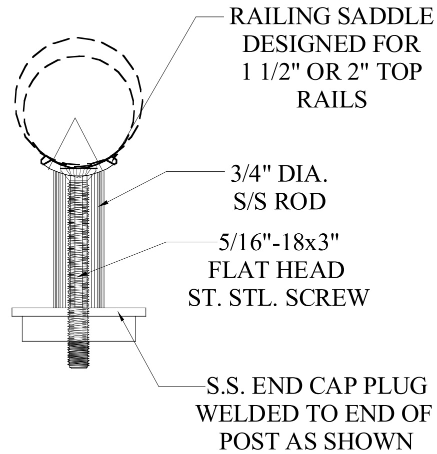
P1 POST continued:

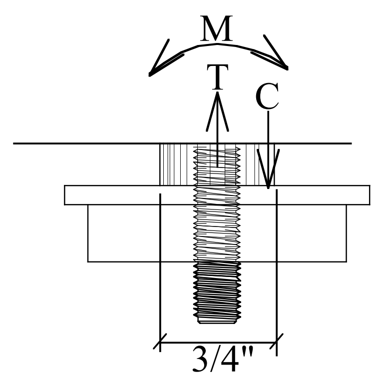
Top Rail Connection Bracket - Okay for all rails with wall thickness of 0.05" or greater.
Top rail to saddle: No significant bending
Check for shear transfer:
Two #8 self drilling screws into 0.05" wall tube.
V_a = 45ksi*0.164"*0.05"/2 = 185#/ screw
V_t = 185#*2 screws = 370#
Maximum applied load 295#



Saddle strength
1/8" sheet welded to 3/4" rod
Weld strength
V_w= 0.3*0.6*75 ksi*0.125*3/4"π = 4.0k
$M_n = S_w F_y$
$S_w = (0.875^3-0.625^3)/6 = 0.071$ in^3
M_n = 0.071 in^3*0.6*75 ksi = 3,195#"
$M_s = M_n$/1.67 = 3,195#"/1.67 = 1,913#"

Sheet strength:
Z_s = 2"*0.125^2/4 = 0.0078 in^3
F_y = 75 ksi for rolled SS sheet 1/4 hard
$M_n = Z_s$*F_y = 0.0078 in^3*75 ksi = 586 #"
$M_s = M_n$/1.67 = 586#"/1.67 = 351#"
Moment on saddle to rod connection
M = 1/2 rail diameter * P = 2"/2*300# = 300"# < M_s okay

Connection rod saddle to post:
Strength of screw 316 Condition CW ASTM F593-98
T_n = 71.2 ksi*0.0524 in^2 = 3,731#
Moment resistance of connection:
M_n = 3,731#*(0.75"/2) = 1,399#"
$M_s = M_n$/2 = 1,399/2 = 700#"



Maximum service load on top rail
$P_s = M_s/a$
a = 1.625"+ 1/2 Diameter top rail
a = 2.625" for 2" top rail and 2.375" for 1.5" top rail
P_s = 700#"/(2.625") = 266# for 2" rail
P_s = 700#/2.375" = 295# for 1.5" rail

THIS WILL CONTROL ALLOWABLE LIVE LOAD ON TOP RAIL – P_s = 266# MAXIMUM PER POST for 2" dia. rail, all wall thicknesses.
or for 1-½" top rail allowable load: P_s = 295#, all wall thicknesses.

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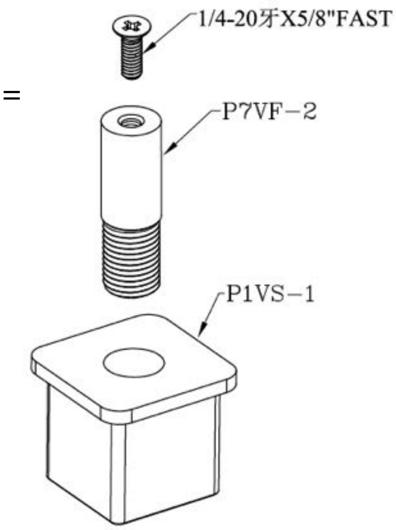
Adjustable Top Rail Connection Bracket



Find moment capacity of rod:
F_y = 30ksi
14MM diameter threaded portion with fine thread, pitch diameter = 13.026mm
Inside diameter = 8mm
Rod bears against cap above the thread portion so moment occurs above the thread portion.
Cylinder diameter = 18.75mm, inside diameter = 8mm
Z = $(18.75^3-8^3)/6 = 1{,}013mm^3$
Z = $1{,}013mm^3*(0.03937in/mm)^3 = 0.06182in^3$
$M_a = 30ksi/1.67*0.06182in^3 = 1{,}110$”#
Allowable load at full height = 1,110”#/(2.125”+1.5”/2) = 386#> 300# (Load at 6ft post spacing)

Check thread engagement of threaded rod into cap. Minimum embedment=
$A_s = 22.42mm^2/mm = 0.8827\ in^2/in$
T_a = 0.8827*75ksi*3/8”/2 = 12,413# > 300# OK

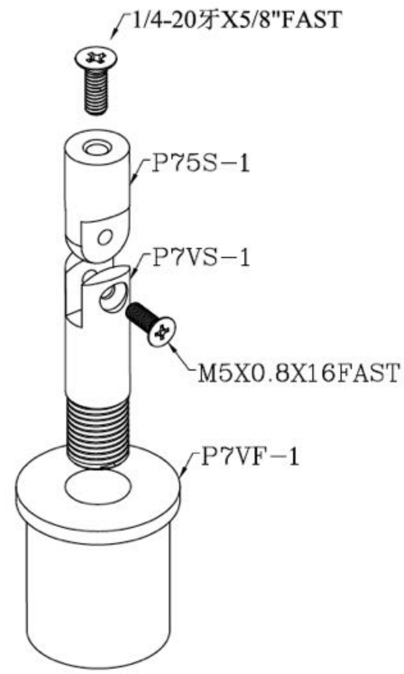
Check embedment of cap into post:
e = 1.8125”
Assume linear stress distribution, use 1# unit load.
M = 1#*2.125” = 2.125”#
P = 2.125”#/(2/3*1.8125”) = 1.759# (Magnitude of couple moment loads)
w = 1.759#/(1.8125”/2*1/2) = 3.881pli (peak distributed load)
Two failure modes, bending of face of post or tearing at corners of post.
Post wall bending strength
$Z=0.187^2*1/4 = 0.0350in^3/in$
$M_a = 30ksi/1.67*2*0.0350in^3 = 1{,}257$”#
$u_a = 1{,}257$”#*8/(2-2*.187)^2 = 3,804pli
w_a = 3,804pli*(2”-.187”*2)/1” = 6,185pli
Max grab rail load = 6,185pli/3.881pli*1# = 1,593#
Corner shear strength:
V_a = 1”*0.187”*0.6*30ksi/1.5 = 2,244pli (controls)
Corner tension strength
V_a = 1”*0.187”*30ksi/1.67 = 3,359pli

w_a = 2,244pli*2 = 4,488pli
Max grab rail load = 4,488pli/3.881pli*1# = 1,177#

Grab rail saddle is mounted using a single 1/4” screw
$As_s = 0.368in^2$
$As_n = 0.539in^2$

W = 0.368in^2*0.6*75ksi/2 = 8.28kli (controls)
W = 0.539in^2*0.6*75ksi/2 = 12.13kli
Tensile strength of screw = 0.0318in^2*75ksi/2 = 1,192#
Required thread engagement = 2*1,192#/8,230pli = 0.29 => 5/16”

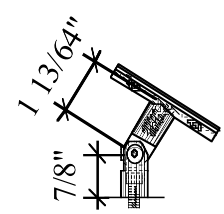
Swivel connection



Moment at swivel connection = 250#*1” = 250”#
Friction area = 14mm$^2\pi$/4=153.9mm^2=0.2386in^2
Required friction stress:
From direct load = 200#/0.2386in^2 = 838.2psi
From rotation = 250”#/(0.2386in^2*0.551”*2/3) = 2,852psi
Total = 838.2+2852psi = 3,690psi
Assume μ = 0.5
T= 3,690psi/(0.5*2/0.2386in^2) = 880#
Screw strength = 0.02444in^2*75ksi/2 = 916# > 880# ok
The screw should be torqued to approximately
(0.2*.1969*880) = 35”#

Friction will be adequate to hold swivel in place.

Verify screw shear strength in the event of swivel slip into screw bearing:
a = 0.0145 in^2
F_v = 42.8 ksi
$\o Z_n$ = 0.65*0.0145*42.8 = 403#
$\o M_n$ = 2*403#*0.5” = 403#”
P_s = 403#”/(1.6*1.203”)= 209#
Shear strength controls the allowable load on the swivel.



Glass fittings:
May be used with the RB50, RB51F and ZP-Series clamps.

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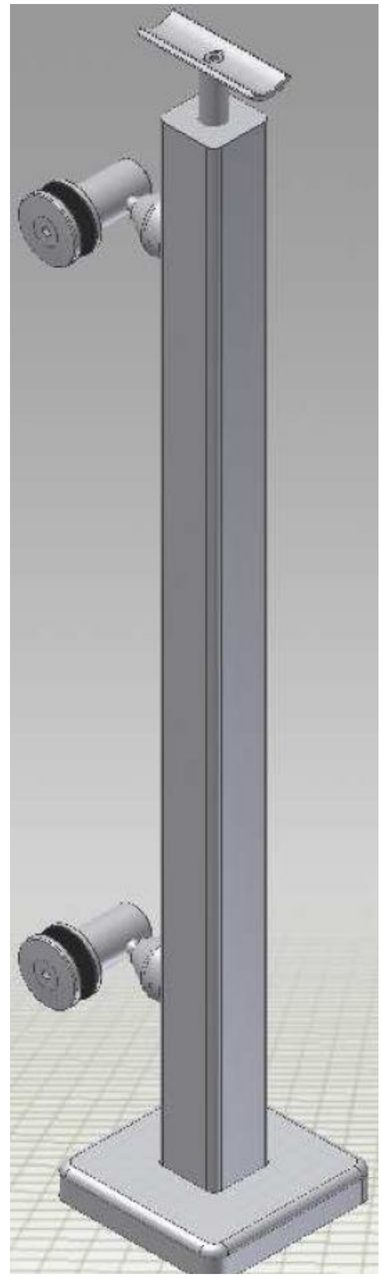
P1 POST continued:



For end posts the post strength is the same.

Infill panel light may extend past post but is supported on only one side of post.

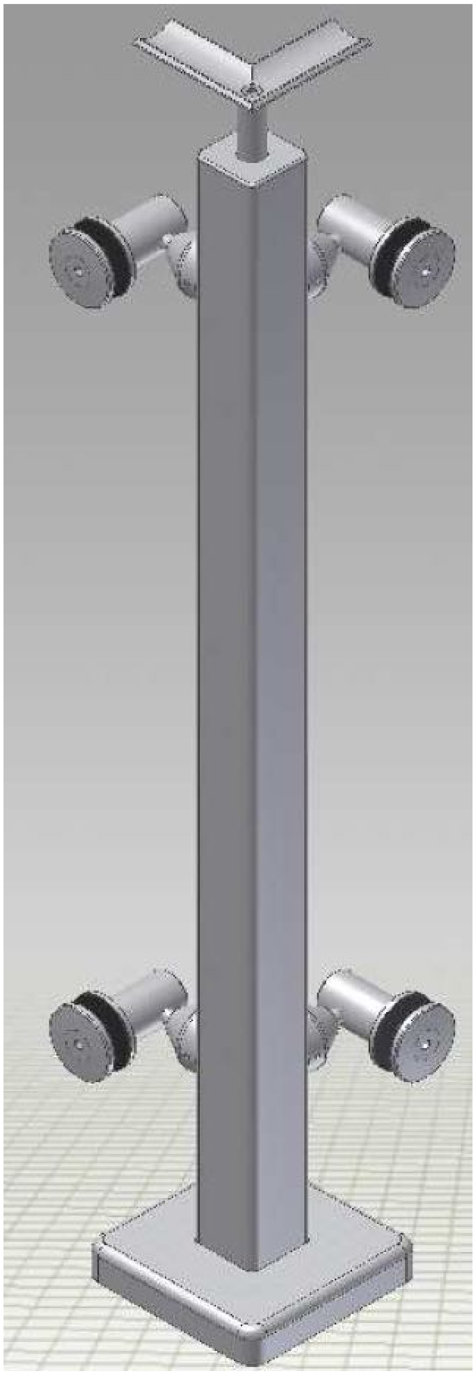
Maximum panel cantilever length past post is 2'-0" or less depending on infill panel strength and wind load conditions.
For 25 psf wind load or 50# concentrated load:
1/4" glass- 12" maximum
3/8" glass - 18" maximum
1/2" or thicker - 24" maximum

Top rail may extend past post with a cantilevered rail section. The length of the cantilever shall be in accordance with the grab rail strength but shall not exceed 2' - 6".

Corner Posts:
Strength of the post and base plate are the same as for the standard running post.

Infill panel may extend to corner from both directions or past corner on one side.

Maximum panel cantilever length past post is 2'-0" or less depending on infill panel strength and wind load conditions.
For 25 psf wind load or 50# concentrated load:
1/4" glass- 12" maximum
3/8" glass - 18" maximum
1/2" or thicker - 24" maximum



Post may be used for an inside corner condition by reversing the stand off direction.

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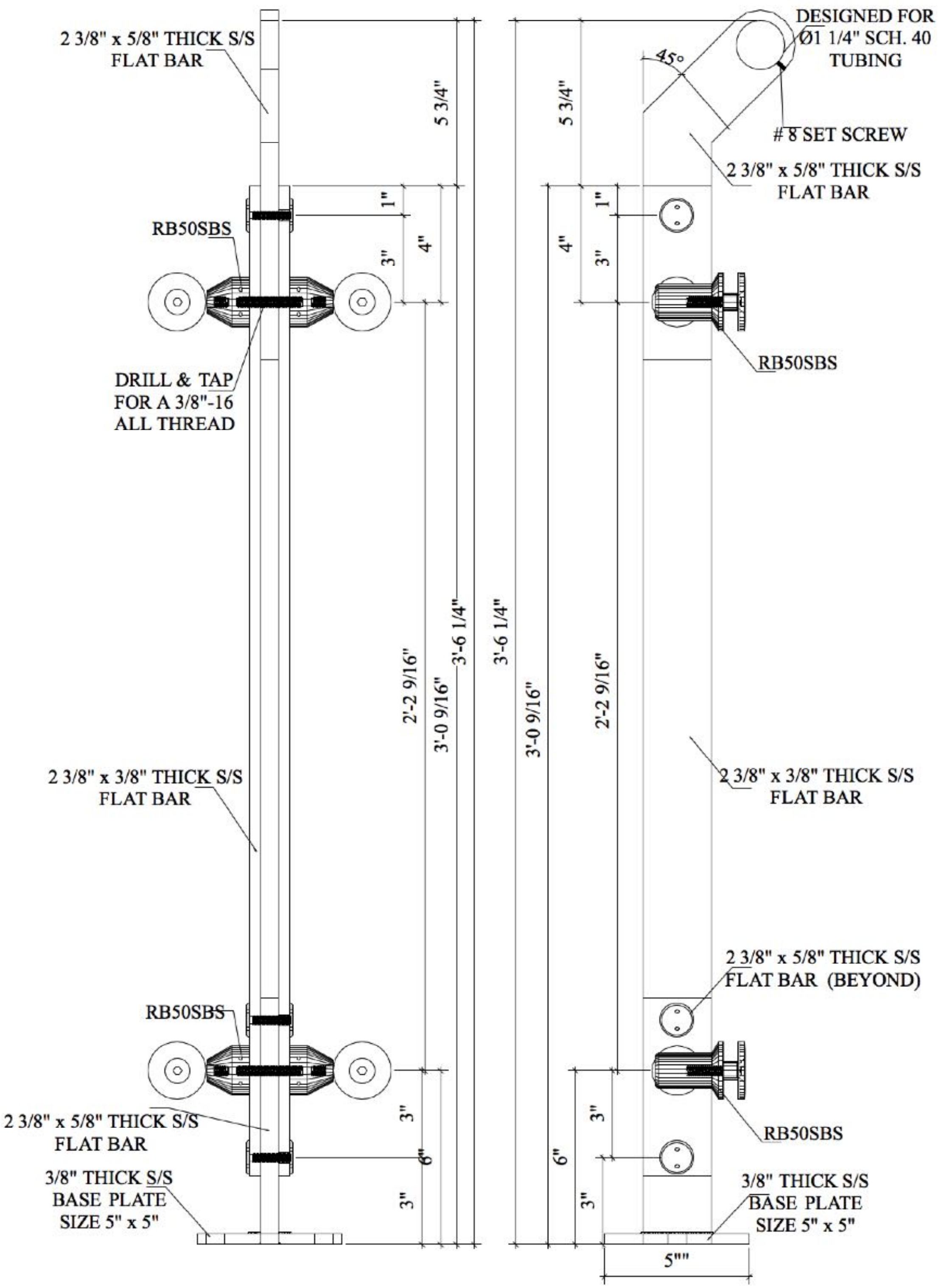
POST P2

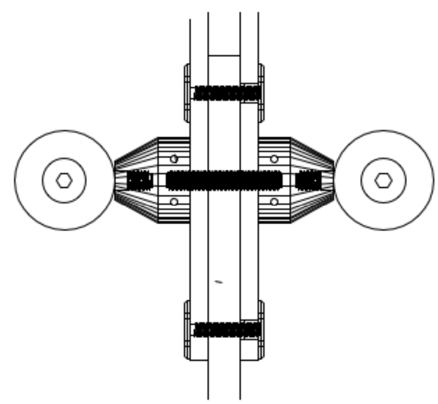


Stainless steel (304) bar post mounted on stainless steel bar stanchion attached to stainless steel base plate.

POST P2 continued:

Post Strength
(2) 2-3/8" x 3/8" bars
$Z_{yy} = 0.375*2.375^2/4 = 0.529\ in^3$
$F_y = 30$ ksi
$M_n = 2$ bars*Z*F_y (Assume non-composite action between the bars.)
$M_n = 2*0.529\ in^3 *30$ ksi = 31,740"# Strength per AISC DG 27
Service moment
$M_s = M_n /1.67$
M_s = 31,740"# /1.67 = 19,006"#

Strength of stanchion:
2-3/8" x 5/8" bars
$Z_{yy} = 0.625*2.375^2/4 = 0.881\ in^3$
$F_y = 30$ ksi SEI/ASCE 8-02 Table A1
$M_n = Z*F_y$ AISC DG 27
$M_n = 0.881\ in^3 *30$ ksi = 26,430"#
Service moment
$M_s = M_n /1.67$
M_s = 26,430"# /1.67 = 15,826"#



Splice between side bars and stanchion bar:
Bars are spliced using 5/16" stainless steel bolts
Strength of screw 316 Condition CW ASTM F593-98
$T_n = 71.2$ ksi*0.0524 in² = 3,731#
$V_n = 42.8$ksi*0.0988 in² = 4,229#
Moment resistance of connection from couple formed by shear in connection bolts acting in double shear:
M_s = 2*(4,229#)*(2"*2 bolts)/2 = 16,916#" at splice

Weld to base plate : 1/4" bevel weld each side. Weld filler to match post and base plate.

Weld strength:
$S_{weld} = 2*0.25"*2.375^2/6 = 0.47\ in^3$
$M_n = SF_y = 0.47\ in^3*75$ ksi = 35,250#"
$M_s = M_n /2.7$ = 35,250#"/2.7 = 13,056#"
Allowable top rail load for 42" total height:
Post moment will be controlled by the welds:
P = 13,056#"/42" = 310#

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POST P2 continued:
Allowable load on lite based on 42" post height – 20" to center of lite (17" above bottom bolt)
W = 11,808#"/21" = 562#
based on 5' x 3' lite
w = 562#/(5'*3') = 37.5 psf

Check lateral load on posts since post is not symmetric.
Since bottom 10" of post is braced by a bar between the side bars post stiffness will be greatly increased by shear transfer between side bars.

Weld strength:
M_{nb} = (0.625"^{2}*2.375"/4)*45 ksi = 10,437#" based on bar yielding at base plate weld.
M_{nw} = (0.25"*2.375"*0.675")*75 ksi = 30,059#" based on base plate weld.
M_s = 10,437/1.67 = 6,250"# or
M_s = 30,059/2.7 = 11,133"#

Top rail lateral force = 6,250/42 = 149#/ post
above 10" Z = 2*2.375"*.375" ^{2}/4 = 0.167 in^3
øM_n = 0.167*45 ksi = 7,515#"
M_s = 7,515/1.67 = 4,500#"
Top rail lateral force = 4,500/32 = 141#/ post (controls for lateral loads)
Lateral force will always be resisted by multiple posts.

Base Plate design:
Same as previously checked.

Glass Fittings:
Use either RB50 or RB51 fittings.
May use ZP series clamps.

Post Deflection:
Top of post deflection from 200# load:
Δ = 200#*H"^{3}/(3EI)
I_{ave} = 0.6875*2.375^3/12 = 0.7675 in^4
For 42" post height:
Δ = 200#*42"^{3}/(3*27x10^6*0.7675) = 0.238"

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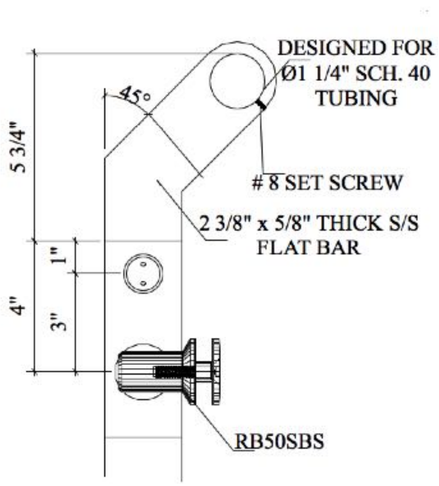
POST P2 continued:



Top bracket for grab rail attachment:

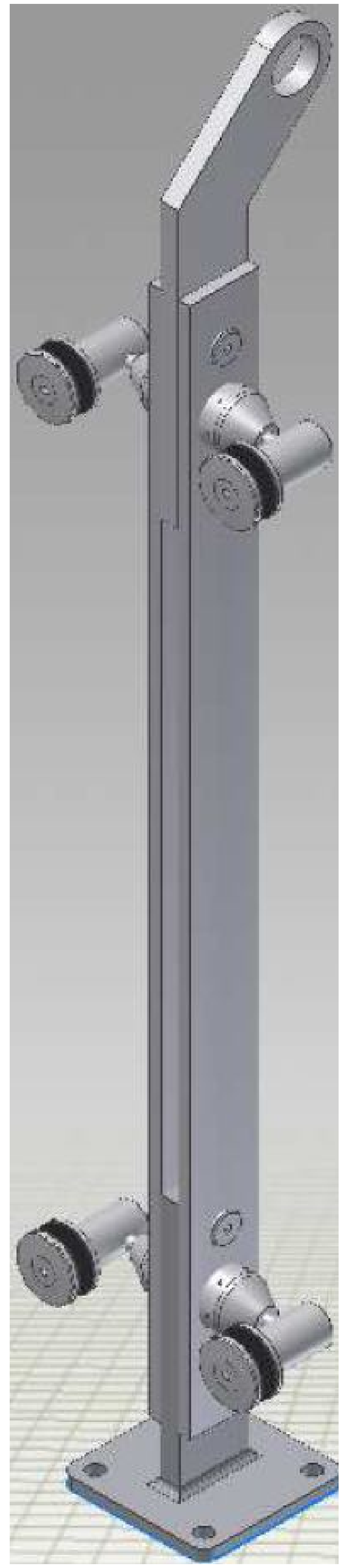
Stainless steel bar, hole diameter = 1 5/8"
Shear through bracket at set screw:

$$A = \frac{(5/8"-0.164")*(2.375-1.675)}{2}$$

$= 0.16\ in^2$

$V_n = 0.16in^2*25ksi = 4.0\ k$

Bending of bar okay based on strength of stanchion bar.



Bracket connection to side plate bars uses (2) 5/16" bolts at 3" apart:
Bolt strength from bottom connection:
$øM_n = 2*(3{,}082\#)*3.0" = 18{,}492\#"$
$M_s = M_n/2 = 18{,}492\#"/2 = 9{,}246\#"$

Check for vertical load of 250# on top rail:
$M = (250+15)*2.75" = 729\#"$
$V = 729\#"/3"+250\#*\sin 45^{\circ} = 420\# < (3{,}082/2)=1{,}541\#$

Horizontal load
$M = 250*5.88"+15*2.75" = 1{,}511\#"$
$V = 1{,}511\#"/3"+250\# = 754\# < 1{,}541\#$

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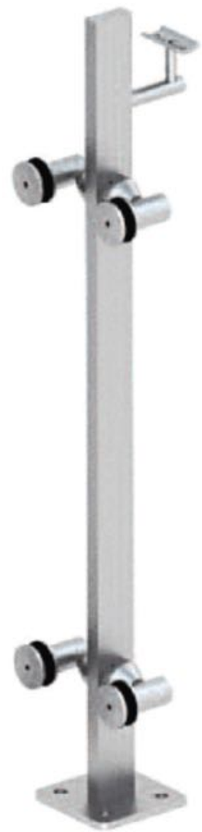
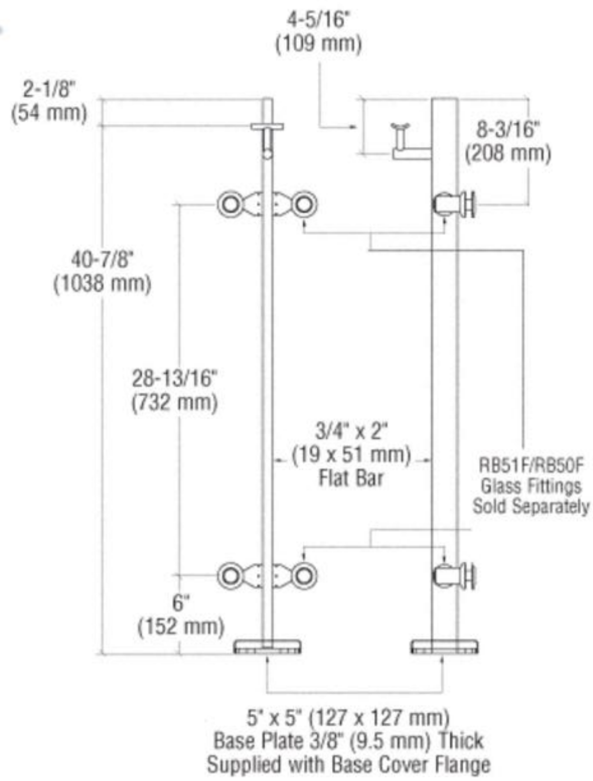
POST P3

Stainless steel (304) bar post mounted on stainless steel bar stanchion attached to stainless steel base plate.





Post Strength - 2” x 3/4” bars
$Z_{yy} = 0.75*2^2/4 = 0.75$ in^3
$F_y = 30$ ksi

$M_n = 1$ bar$*Z*F_y = 0.75$ in^3 $*30$ ksi $= 22{,}500$”# Strength per AISC DG 27
Service moment
$M_s = M_n/1.67 = 22{,}500$”# $/1.67 = 13{,}473$”#
Weld to base plate : 1/4” bevel weld down sides– Weld filler to match post and base plate.

Weld strength:
$S_{weld} = 2*0.25$”$*2^2/6 = 0.333$ in^3
$M_n = SF_y = 0.333$ in^{3*75} ksi $= 25{,}000$#”
$M_s = M_n/2.7 = 25{,}000$#”$/2.7 = 9{,}259$#”
Allowable top rail load for 42” total height:
Post moment will be controlled by the bar strength:
P = 9,259#”/42” = 220#

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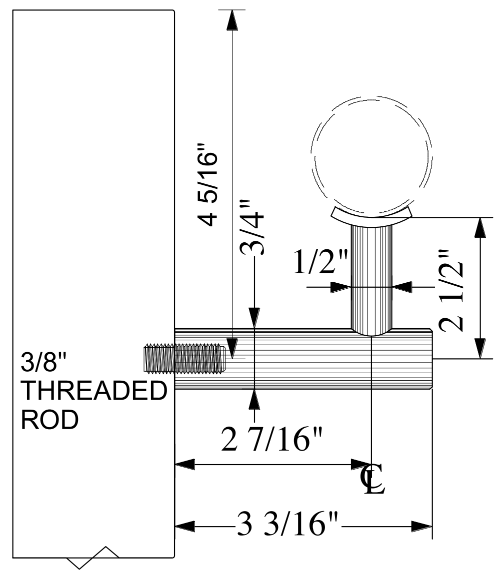
POST P3 continued:
Base Plate:
5” square base plate same as previously checked.

Glass fittings - RB51F/RB50F refer to fitting calcs.

Hand Rail Bracket:
3/4” Dia. 316 Stainless steel bar attached to post with 1/2” vertical bar.



Bending in ¾” horizontal bar:
$Z = 0.75^3/6 = 0.0703$ in^3
$øM_n = 0.0703*30$ksi $= 2,109$”#
Vertical service load:
$V_S = [(øM_n)/Ω]/e = [2,109/1.67]/2.4375 = 518$#

Bending in 1/2” vertical bar, hardened SS:
$Z = 0.5^3/6 = 0.02083$ in^3
$øM_n = 0.02083*45$ksi $= 937$”#
Vertical service load:
$V_S = [(M_n)/Ω]/e = [937/1.67]/2.625 = 214$#

For 3/8” 316 SS rod ASTM F593-98 CW or stronger;
$F_{ut} = 90$ksi
$T_n = A*90$ ksi $= 0.0775$in^{2*90}ksi $= 6,975$#
$T_s = T_n/Ω = 6,975\#/2 = 3,487$#

Couple moment strength:
$M_s = 3,487\#*2/3*3/4” = 1,743$”#
Allowable load per bracket
For maximum H = 2.4375”
$P = M_s/H$
$P = 1,743$”#/(2.4375”) = 715#

Post Deflection:
Top of post deflection from 200# load:
$Δ = 200\#*H”^3/(3EI)$
$I = 0.75*2^3/12 = 0.50$ in^4
For 42” post height:
$Δ = 200\#*42”^3/(3*27x10^6*0.50) = 0.366$”

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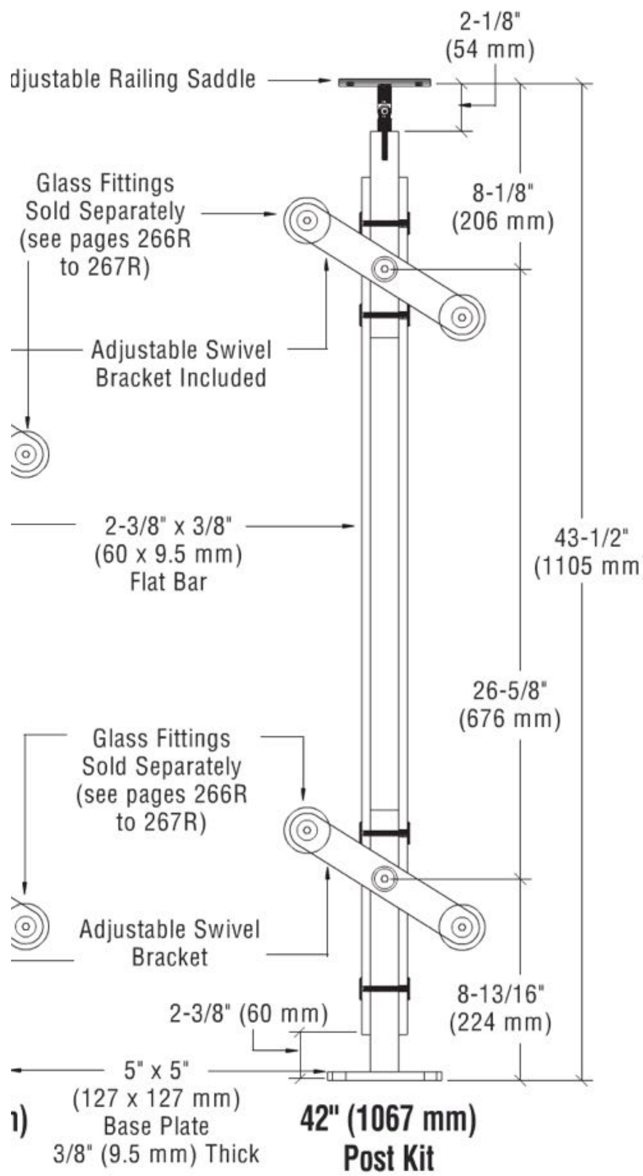
P4 POST

2-3/8"x3/8" Bars 304 Stainless Steel
F_y = 30 ksi
Post Strength
$Z_{yy} = 0.375*2.375^2/4 = 0.529$ in^3
F_y = 30 ksi SEI/ASCE 8-02 Table A1
M_n = 2 bars*Z*Fy
M_n = 2*0.529 in^3 * 30 ksi = 31,740"#
Service moment
$M_s = M_n / \Omega$
M_s = 31,740"# /1.67 = 19,006"#



Weld to base plate : 1/4" fillet weld down side, across ends– Weld filler to be same as post and base plate.

Weld strength:
$S_{weld} = 2*0.707*0.25"*2.375^2/6+$
$1.0*0.25*2.375 = 0.926$ in^3
$M_n = SF_y$ = 0.926 in^3*0.675 ksi = 41,674#"
$M_s = M_n$ /2.7 = 41,674#"/2.7 = 15,435#"

Splice between side bars and stanchion bar:
Bars are spliced using 5/16" stainless steel bolts.
Strength of screw 316 Condition CW
ASTM F593-98
T_n = 71.2 ksi*0.0524 in^2 = 3,731#
V_n = 42.8ksi*0.0988 in^2 = 4,229#
Moment resistance of connection from couple formed by shear in connection bolts acting in double shear:
M_s = 2*(4,229#)*(2"*2 bolts)/2 = 16,916#" at splice

Allowable top rail load for 42" total height:
P = 15,435#"/42" = 367#

Allowable load on lite based on 42" post height – 20" to center of lite
W = 15,435#"/20" = 772#
based on 5' x 3' lite
w = 772#/(5'*3'/2) = 102.9 psf

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P4 POST continued:

Check lateral load on posts since post is not symmetric.
Since bottom 10” of post is braced by a bar between the side bars post stiffness will be greatly increased by shear transfer between side bars.

Lateral load strength of post:
M_{nb} = (1”^{2}*2.375”/4)*30 ksi = 17,812#” based on bar yielding at base plate weld.
M_{nw} = (0.25”*2.375”*1”)*75 ksi = 44,531#” based on base plate weld.
M_s = 17,812/1.67 = 10,666”# or
M_s = 44,531/2.7 = 16,493”#

Top rail lateral force = 10,666/42 = 254#/ post

above 10” Z = 2*2.375”*.375” ^{2}/4 = 0.167 in^3
øM_n = 0.167*45 ksi = 7,515#“
M_s = 7,515/1.67 = 4,500#”
Top rail lateral force = 4,500/(32/2) = 282#/ post (controls for lateral loads)
Lateral force will always be resisted by multiple posts.

Base Plate:
5” square base plate same as previously checked.

Post Deflection:
Top of post deflection from 200# load:
Δ = 200#*H_1”^{3}/(3EI_1) + 200#* H*H_2”^{2}/(3EI_2)
I_1 = 2*0.375*2.375^3/12 = 0.353 in^4
I_2 = 1*2.375^3/12 = 1.116 in^4
For 42” post height:
Δ = 200#*34”^{3}/(3*27x10^6*0.353) + 200#*42*8”^{2}/(3*27x10^6*1.116) = 0.281”

Glass Fitting
May use either the RRF 10 surface mount or flush mount option.

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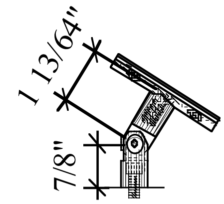
P4 POST continued:

Top Rail Connection Bracket



Top rail to saddle: No significant bending
Check for shear transfer:
Two #8 self drilling screws
V_a = 45ksi*0.164”*0.065” = 480#/ screw
V_t = 480#*2 screws = 960#
Maximum applied load 300

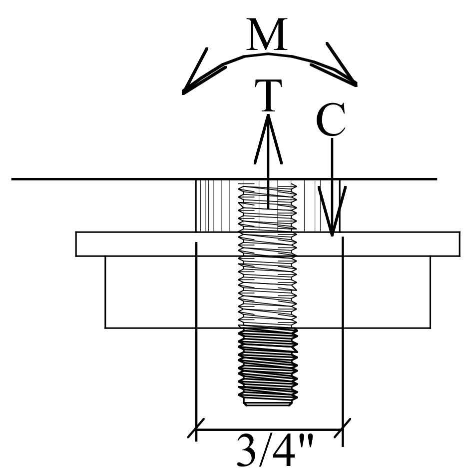
Saddle strength
1/8” sheet welded to 3/4” rod
Weld strength
V= 0.3*60 ksi*0.125*3/4”π = 5.3k
$M_n = S_w F_y$
$S_w = (0.875^3-0.625^3)/6 = 0.071$ in^3
$M_n = 0.071$ in^3*45 ksi = 3,195#”
M_s = øM$_n$/1.6 = 0.9*3,195#”/1.6 = 1,797#”

Swivel point strength
Shear in screw:
a = 0.0145 in^2
F_v = 42.8 ksi
øZ_n = 0.65*0.0145*42.8 = 403#
øM_n = 2*403#*0.5” = 403#”
P_s = 403#”/(1.6*1.203”)= 209#

Sheet strength:
Z_s = 2”*0.125^2/4 = 0.0078 in^3
F_y = 75 ksi for rolled SS sheet 1/4 hard
$M_n = Z_s*F_y$ = 0.0078 in^3*75 ksi = 586 #”
M_s = øM$_n$/1.6 = 0.9*586#”/1.6 = 330#”

Moment on saddle to rod connection
M = 1/2 rail diameter * P = 2”/2*300# = 300”# < M_s okay

Connection rod saddle to post:
Strength of screw 316 Condition CW ASTM F593-98
T_n = 71.2 ksi*0.0524 in^2 = 3,731#
Moment resistance of connection:
M_n = 3,731#*(0.75”/2) = 1,399#”
M_s = øM$_n$/1.6 = 0.75*1,399/1.6 = 659#”



Maximum service load on top rail
$P_s = M_s/a$
a = 2.078”+ 1/2 Diameter top rail
a = 3.078” for 2” top rail and 2.828” for 1.5” top rail
P_s = 659#”/(3.078”) = 214# for 2” rail
P_s = 659#/2.828” = 233# for 1.5” rail
ALLOWABLE LIVE LOAD ON TOP RAIL WILL BE LIMITED TO 209# BASED ON STRENGTH AT SWIVEL.

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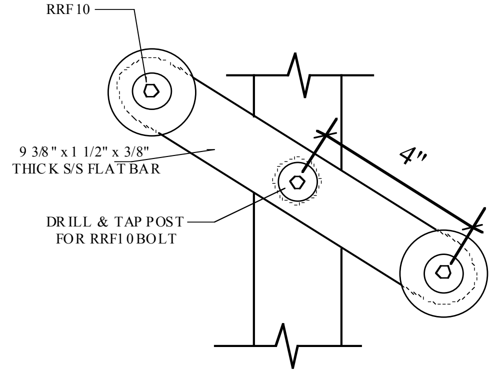
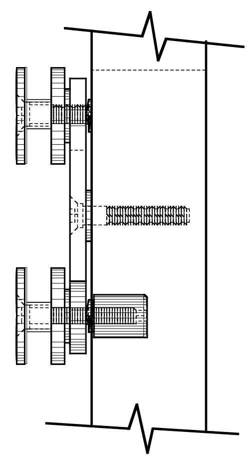
P4 POST continued:
GLASS STANDOFFS



Determine standoff strength:
$M = P*4"$ where P = Vertical or Horizontal
Shear on screw = Z
Standoff horizontal bending strength
$Z_{xx} = 1.5*0.375^2/4 = .0527\ in^3$
$M_s = øM_n/1.6 = 0.9*0.527*45/1.6 = 1,335\#"$
$H_s = 1,335\#"/4" = 334\#$

Strength of Screw into post 3/8”-16 screw 316 Condition
AF ASTM F879-98 3/8” countersunk cap screw rod 316:
$A_t = 0.0775\ in^2$; $A_v = 0.196\ in^2$
$V_n = 0.196\ in^2*42\ ksi = 8.2\ k$ or $0.0775*51\ ksi = 3,953\#$ (controls)
$T_n = 6,588\#$ from ASTM F 837 Table 4

Moment resistance of connection horizontal load:
$M_n = 3,953\#*(2.375"/2) = 4,694\#"$
$M_s = M_n/2 = 4,694/2 = 2,347\#"$
For vertical load:
Clamp force $= 0.33*T_n = 0.33*3,953 = 1,316\#$
$S_c = 1.5"^3/6 = 0.5625\ in^3$
$M_n = 0.5625\ in^3*1,316\# = 740\#"$
$M_s = 740\#"/2 = 370\#"$
$V = 370\#"/4" = 93\#$
$Z_n = F_{nv}*A_v = 42.8\ ksi*0.0454\ in^2 = 1,943\#$
$Z_s = Z_n/2 = 1,943\#/2 = 972\#$



Determine service load of standoff from interaction equation where:
$(M/M_s)^2+(Z/Z_s)^2 \leq 1.0$
$P = \sqrt{(H^2+V^2)}$
Z = P
M = 4”*P
substituting using P:
$(4P/2,347)^2 + (P/972)^2 = 1$ then solving for P
$P = \{1/[(4/2,347)^2 +1/972^2]\}^{1/2}$
P = 502# = V
For typical glass lite = 5’ x 3’
V = dead load weight of glass = 4.75 psf *5’*3’/(4 standoffs per lite) = 17.8#
$H = [P^2-V^2]^{1/2} = [249^2 - 17.8^2]^{1/2} = 433\# > 334\#$ standoff bending will control
Allowable horizontal load based on standoff strength
h = 334#/(5’*3’/4) = 89 psf

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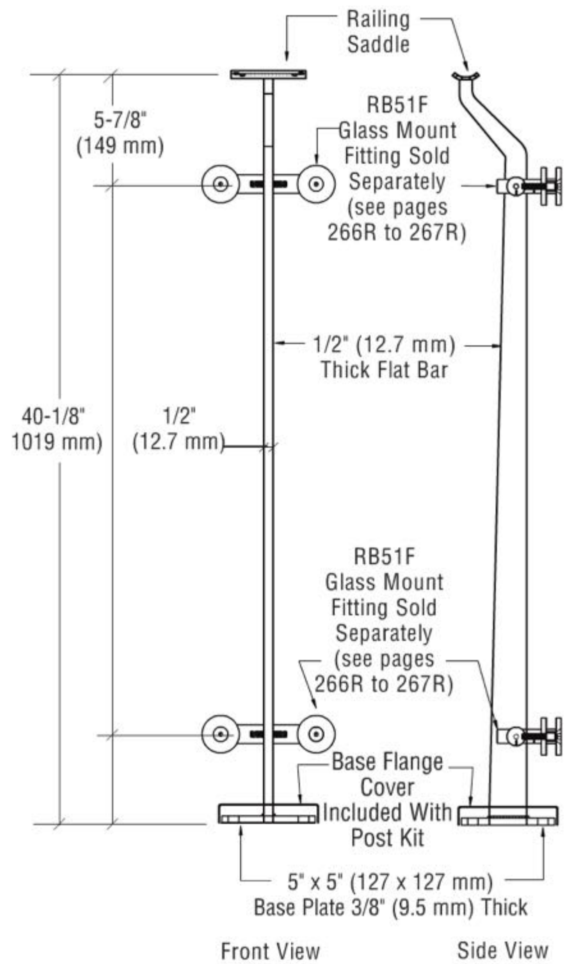
POST P5

1/2" x 2" Stainless steel (304) bar post mounted on stainless steel bar stanchion attached to stainless steel base plate.



Post Strength at bottom- 2" x 1/2" bars

$Z_{yy} = 0.5*2^2/4 = 0.5$ in^3

$F_y = 30$ ksi

$M_n = 1$ bar$*Z*F_y = 0.5$ in^3 $*30$ ksi = 15,000"#

Strength per AISC DG 27

Service moment

$M_s = M_n/1.67 = 15{,}000$"# /1.67 = 8,982"#

Weld to base plate : 1/4" bevel weld down sides–
Weld filler to match post and base plate.

Weld strength:

$S_{\text{weld}} = 2*0.25\text{"}*2^2/6$

$S = 0.333$ in^3

$M_n = SF_y = 0.333$ in^3$*75$ ksi = 25,000#"

$M_s = M_n/2.7 = 25{,}000$#"/2.7 = 9,259#"

Allowable top rail load for 42" total height:
Post moment will be controlled by the bar strength:

P = 8,982#"/42" = 214#

Lateral load strength of post:

$M_{nb} = (0.5\text{"}^2*2\text{"}/4)*30$ ksi = 3,750#" based on bar yielding at base plate weld.

$M_{nw} = (0.25\text{"}*2\text{"}*0.5\text{"})*75$ ksi = 18,750#" based on base plate weld.

$M_s = 3{,}750/1.67 = 2{,}246$"# or

$M_s = 18{,}750/2.7 = 6{,}944$"#

Top rail lateral force = 2,246/42 = 53#/ post

This post may not be installed without lateral support (glass lights in place or top rail otherwise braced against longitudinal loads.

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POST P5 continued:
Base Plate:
5” square base plate same as previously checked.

Glass fittings - RB51F/RB50F refer to fitting calcs.

Hand Rail Bracket:
SS saddle attached to top of post.
1/8” saddle pressed onto top of post.

Top rail to saddle: No significant bending
Check for shear transfer:
Two #8 self drilling screws
V_a = 45ksi*0.164”*0.065” = 480#/ screw
V_t = 480#*2 screws = 960#
Maximum applied load 300

Sheet strength:
Z_s = 2”*0.125^2/4 = 0.0078 in^3
F_y = 75 ksi for rolled SS sheet 1/4 hard
M_n = Z_s*F_y = 0.0078 in^3*75 ksi = 586 #”
M_s = M_n/1.67 = 586#”/1.67 = 350#”

Post Deflection:
Top of post deflection from 200# load:
Δ = 200#*$H”^3$/(3EI)
Average I for tapered post bar-
I_{ave} = 0.5*$1.5”^3$/12 = 0.14 in^4
For 42” post height:
Δ = 200#*$42”^3$/(3*27x10^6*0.14) = 1.30”

Glass Fittings:
Use either RB50 or RB51 fittings.
May use ZP series clamps.

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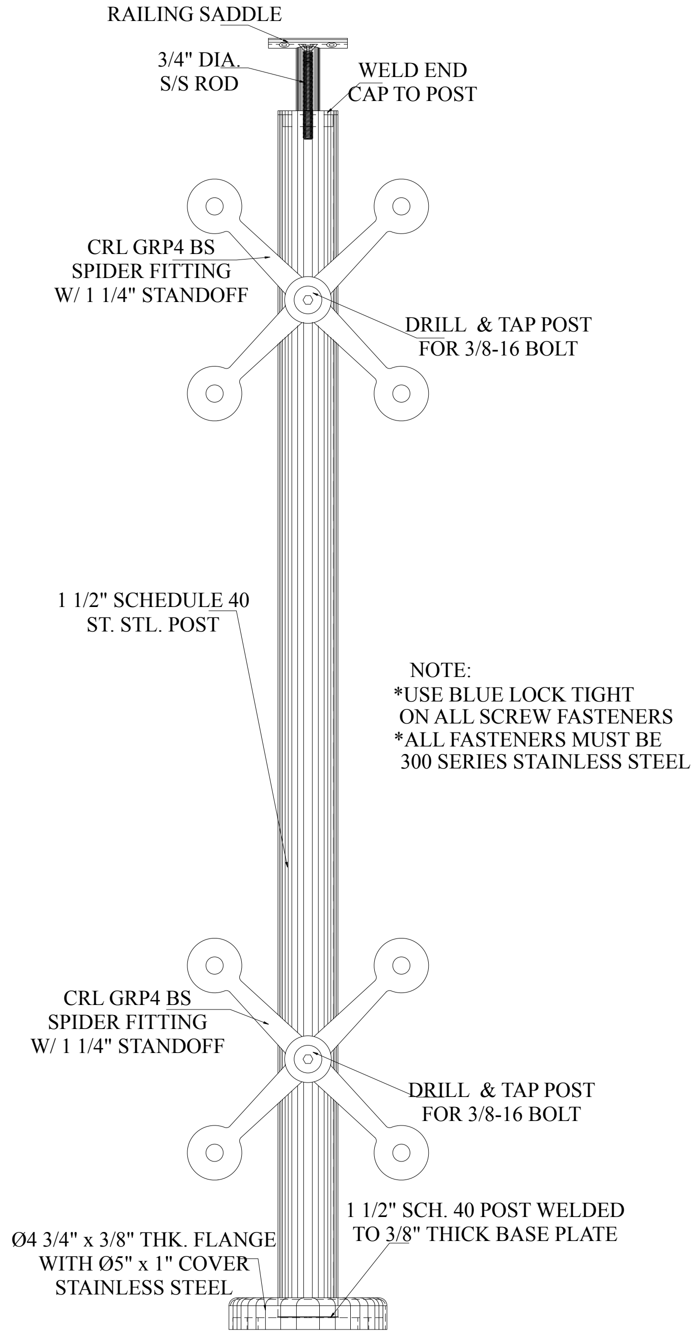
P6 POST



1-1/2" schedule 40 1/16 hard
304 Stainless steel pipe
Post Strength
$Z_{yy} = 0.421\ in^3$
$F_y = 40$ ksi SEI/ASCE 8-02 Table A1
$M_n = 0.421\ in^3 * 40\ ksi = 16{,}840\#"$

Service moment
$M_s = M_n / \Omega$
$M_s = 16{,}840\#"/1.67 = 10{,}084\#"$

Allowable top rail load for 42" total height:
$P = 10{,}084\#"/42" = 240\#$

Allowable load on lite based on 42" post height – 20" to center of lite
$W = 10{,}084\#"/20" = 504\#$
based on 5' x 3' lite
$w = 504\#/(5'*3'/2) = 67.2$ psf

Weld to base plate : 1/4" fillet weld all around – Weld filler to be same as post and base plate.
$S_w = 0.764$
$M_{nw} = 0.764*0.6*75\ ksi = 34{,}386\#"$
$M_s = M_{nw}/2.7 =$
$M_s = 34{,}386/2.7 = 12{,}736\#"$

Base plate weld does not control.

Post Deflection:
Top of post deflection from 200# load:
$\Delta = 200\#*H"^3/(3EI)$
$I = 0.293\ in^4$
For 42" post height:
$\Delta = 200\#*42"^3/(3*27x10^6*0.293) = 0.624"$

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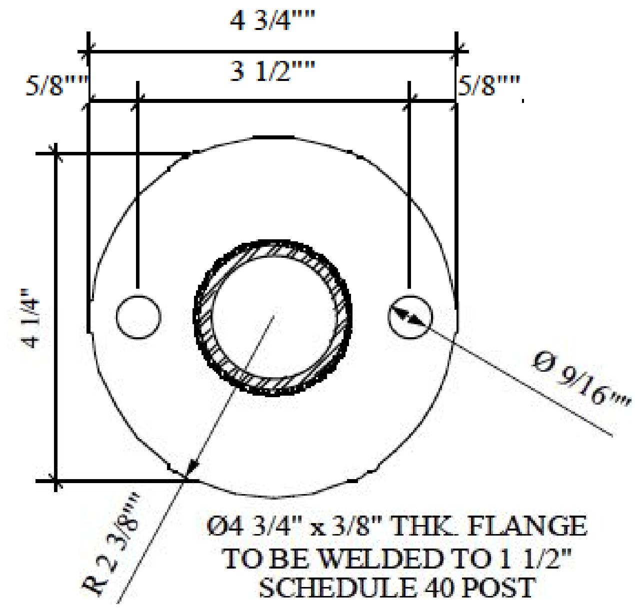
P6 POST continued:



Base Plate design:

for 3/8" plate $Z = \frac{4.25" \bullet 3/8^2}{4} = 0.149\ in^3$

$F_y = 45$ ksi

$M_n = Z\ F_y$

$M_n = 0.149*45ksi = 6,705\#"$

$M_s = M_n/\Omega$

$M_s = 6,705\#"/1.67$

$M_s = 4,015\#"$

Calculate base plate reactions and moment based on the design load of 250# at top rail.

M = 250#*42" = 10,500

$T_b = M/4.5"$

$T_b = 10,500/(4.5) = 2,333\#$

Design anchor tension (service load)

Base plate moment

$M = T_b*(4.75"-2")/2 = 1.375T_b$

M = 1.375*2,333 = 3,208#"

$M < M_s$ therefore okay

Check torsional strength:

$\tau = abc^2 = 0.33*0.375"*4.5"^2 = 2.5\ in^3$

$\tau_n = 25\ ksi*2.5\ in^3 = 62,648\#"$

$\tau_s = ø*\tau_n/1.6 = 0.85*62,648\#"/1.6 = 33,282\#"$

Torsion in base plate will not limit post moment

Required anchors for connections to steel

Tension on bolts

$T_n \geq 2,333\#*2 = 4,666\#$

Strength of bolt 316 Condition CW ASTM F593-98

$F_y = 71.2$ ksi

area required = 4,666/71,200 = 0.066 in^2

3/8" bolt, a = 0.0775 in^2

For concrete mounted base plates - anchors designed for 2,333# allowable tension per anchor, see round baseplate to concrete section in this report.

For mounting to wood use 4 hole baseplate option shown in this report.

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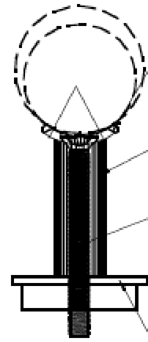

P6 POST continued:
Top Rail Connection Bracket
Top rail to saddle: No significant bending
Check for shear transfer:
Two #8 self drilling screws, 0.05" min wall thickness
V_a = 45ksi*0.164"*0.05"/2 = 185#/ screw
V_t = 185#*2 screws = 370#
Maximum applied load 295#

Saddle strength
1/8" sheet welded to 3/4" rod
Weld strength
V= 0.6*75 ksi*0.125*3/4" = 4.2 k
$M_n = S_w F_y$
$S_w = (0.875^3-0.625^3)/6 = 0.071$ in^3
M_n = 0.071 in^3*0.6*75 ksi = 3,195#"
$M_s = M_n/\Omega$ = 3,195#"/2.7 = 1,183#"

Sheet strength:
Z_s = 2"*0.125^2/4 = 0.0078 in^3
F_y = 75 ksi for rolled SS sheet 1/4 hard
$M_n = Z_s*F_y$ = 0.0078 in^3*75 ksi = 586 #"
$M_s = M_n/\Omega$ = 586#"/1.67 = 351#"

Moment on saddle to rod connection
M = 1/2 rail diameter * P = 2"/2*300# = 300"# < M_s okay

Connection rod saddle to post:
Strength of screw 316 Stainless steel
T_n = 71.2 ksi*0.0524 in^2 = 3,731#
Moment resistance of connection:
M_n = 3,731#*(0.75"/2) = 1,399#"
$M_s = M_n/\Omega$ = 1,399/2 = 700#"

Maximum service load on top rail
$P_s = M_s/a$
a = 1.625"+ 1/2 Diameter top rail
a = 2.625" for 2" top rail and 2.375" for 1.5" top rail
P_s = 700#"/(2.625") = 267# for 2" rail
P_s = 700#/2.375" = 295# for 1.5" rail
THIS WILL CONTROL ALLOWABLE LIVE LOAD ON TOP RAIL 267# MAXIMUM PER POST for 2" dia. rail - all wall thicknesses.
or for 1.5" top rail: P_s = 295# - all wall thicknesses.

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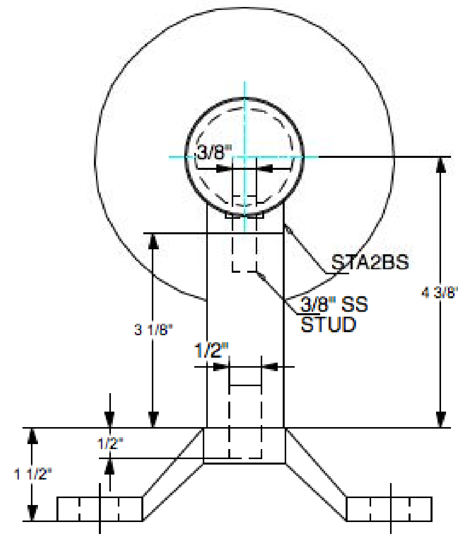
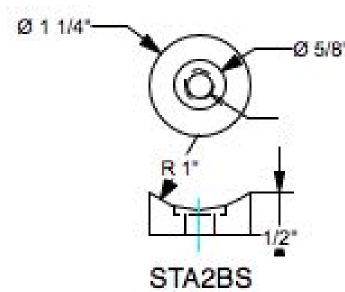
P6 POST continued:



Spider fitting connection to posts:
1-1/4” diameter standoff bar is attached to the post using a coped adapter and secured in place by tightening on a 3/8” diameter stainless steel stud.

Check strength of stud into pipe wall:
Screw tension strength:
$P_{nt} = 0.75*0.0775in^2*67.5ksi = 3,923\#$
Thread strength:
$P_{not} = 0.58tF_{tu}A_{sn} = 0.58*75ksi*0.375''*0.828'' =$
$P_{not} = 13.5k$

Shear strength:
$P_{nt} = 0.65*0.0878in^2*40.5ksi = 2,312\#$



Typical maximum panel load:
$D = 6.5psf*5'*3'/2 = 49\#$
$L = 50\#$
$p = 49+50 = 99\#$
$M_u = 99\#(3.125''+0.5''+1.5''+0.5''/2) = 532\#''$

$V_u/V_n = 99\#/2,312\# = 0.043$ will not reduce nominal tension strength.

Determine M_n:
$M_n = 3,923\#*(1.25''/2) = 2,452\#'' > 532\#''$

Check bending of the stud through the pipe adapter:
$M = 99\#*0.375'' = 37.1\#''$
$Z_{stud} = 0.375^3/6 = 0.00879in^3$
$M_n = 0.00879\ in^3*75ksi = 659\#'' > 37.1\#''$ okay.

The adapter bar will have adequate strength for the applied load.

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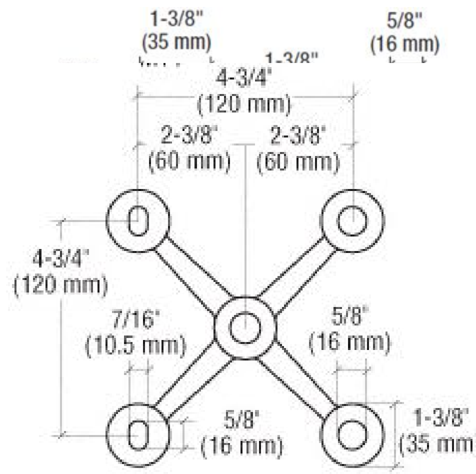
P6 POST continued:
GRP SPIDER FITTINGS



Check strength of spider fitting arm
horizontal bending strength at face of connection hub
$Z_x=Z_y=Z_z = 5/8^3/4 = 0.061$ in^3
$M_n = ZF_y$
$M_s = øM_n/1.6 = 0.9*0.061*45/1.6 = 1{,}545”\#$
$H_{sx} = H_{sx} = 1{,}545”\#/1.6875” = 916\#$
$H_{sz} = 1{,}545”\#/2.386” = 647\#$
$V_s = H_s = 460\#$ vertical or horizontal load acting alone
For interaction between vertical and horizontal:
$\sqrt{[H_s^2+V_s^2]} = 647\#$

Check strength of eyelet attachment to arm for loads in the glass plane with a maximum offset of 3”. Offset from glass fitting causes torsion at the eyelet to arm
b = 0.482” ; = c = 0.375”; $\alpha = 0.221$
$\tau_{max} = F_y\alpha bc^2 = 45ksi*0.221*0.482*0.375^2 = 674”\#$
$P_{ax} = P_{ay} = (674/1.67)/3” = 135\#$

For maximum dead load case $V_s = 111\#$ (next page)
$H_s = [647\#^2 - 111^2]^{½} = 637\#$
Determine connection strength to support post:
Loads on fasteners
$M = P*3.359”$ where P = V or H
Shear on fasteners = Z = 1/2*(H or V)
$C = T = M/(1.375”/2) = P*(3.359”/0.6875”) = 4.886P$
Assumes unbalanced horizontal loads (all horizontal load concentrated on a single arm.

Strength of bolt into support
screw 316 Condition AF ASTM F879-98 3/8” countersunk cap screw rod 316 Stainless steel
Shear strength:
$A_t = 0.0775$ in^2 ; $A_v = 0.196$ in^2
$V_n = 0.196$ in^2*42 ksi = 8.2 k or 0.0775*51 ksi = 3,953# (controls)
$T_n = 6{,}588\#$ from ASTM F 837 Table 4
Assumes attachment to support develops full cap screw strength:
$øV_n = 0.75*3{,}953 = 2{,}964\#$
$øT_n = 0.85*6{,}588 = 5{,}600\#$
Moment resistance of connection:
For horizontal loads:
$øM_n = 5{,}600\#*(1.375”/2) = 4{,}073\#”$
$M_s = øM_n/1.6 = 4{,}073/1.6 = 2{,}546\#”$
$V_s = øV_n/1.6 = 2{,}964/1.6 = 1{,}853\#$

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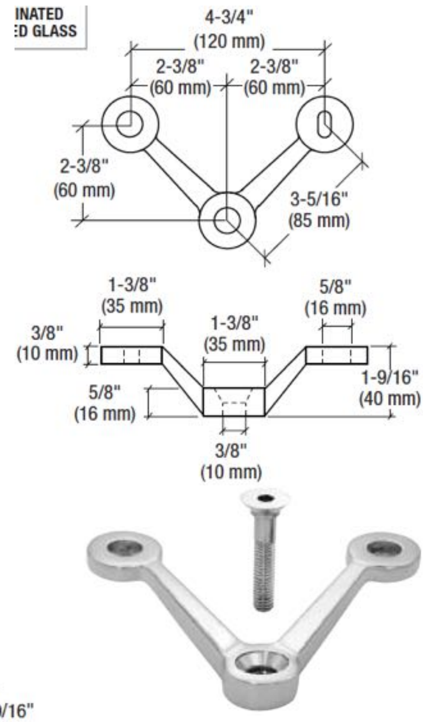
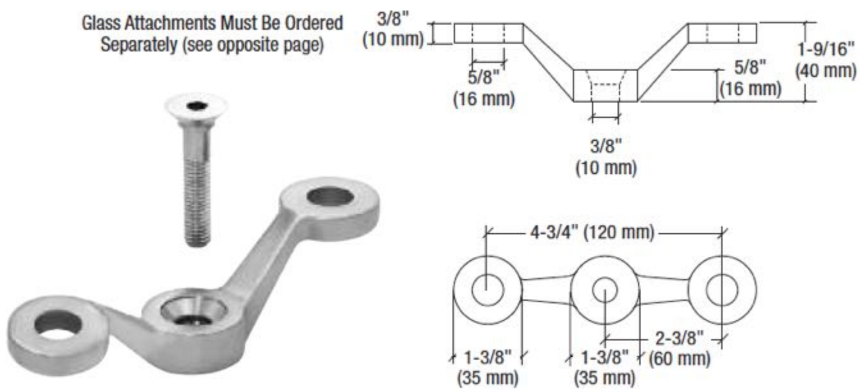
P6 POST continued: GRP (continued)

Check bending strength of stud for pure bending:
Applicable to loads in glass plane only:
$Z = 0.375''^3/6 = 0.0088in^3$
$\phi M_n = 0.9*85ksi*0.0088in^3 = 673\#''$
for maximum eccentricity = 1/2”
$P_n = 673\#''/0.5'' = 1{,}346\#$
Determine allowable load:
$P_{sv} = 1{,}346\#/1.6 = 841\#$
X or Y (in glass plane):
$V_x = V_y = [2{,}546''\#/(1.5625+3'')] = 558\#$ total

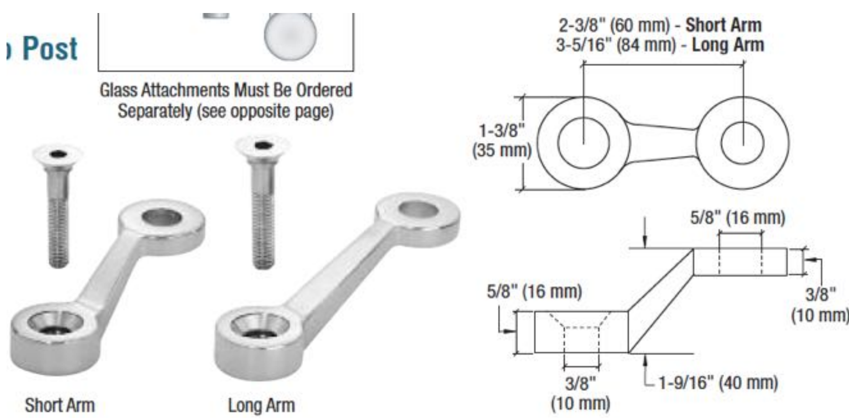
Out of plane loads (typically live and wind)
For dead load ≤ 0.2*558# = 111#
$R_a = (2{,}546''\#-111*4.5625)/4.886'' = 417\#$ single arm
$R_a = (2{,}546''\#-111*4.5625)/2.375'' = 858\#$ total on fitting
These strength parameters are applicable to all configurations:



GRP2V



GRP2



GRP1 (single arm)

FOR GRP FITTING LIMIT TOTAL LOAD ON A SINGLE ARM:
X or Y ≤ 111# and Z ≤ 417#
AND Z ≤ 858# TOTAL ON THE FULL FITTING.

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P6 POST continued:

Glass Fittings:
Uses RRF10 fixed fitting or RSF10 combination swivel head fitting.
May use either surface mounted or flush mounted fittings.
Refer to pages 43 and 44.

Typical loads:
Lower lite size:
10’*6’
D = 10’*6’*6.5psf = 390#
W = 10’*6’*10.9psf = 654#
E = 1.76*390# = 686#

Upper lite size:
10’*6’
D = 7’*6’*6.5psf = 273#
W = 7’*6’*10.9psf = 458#
E = 1.76*273# = 480#

For lower light dead load is supported directly at bottom.
Spider fittings provided horizontal bracing to top.

Load to fitting: two fittings on top
D = 0
W = (654#/2)/2 = 164#
E = (0.5*686)/2 = 172#
E_{asd} = 0.7*172 = 120#
Load to fittings from upper lite
D = 273/4 = 68#
W = (#458/2)/2 = 115#
E = (0.5*480)/2 = 120#
E_{asd} = 0.7*120 = 84#

Check load combinations on support plate:
M_u = 1.2D+1.3W : 1.2*68*14= 1,142#” vertical; 1.3*(164+115)*3” =1,088#” horizontal
okay based on plate strength or
M_u = 1.2D+1.0E = 1.2*68*14= 1,142#” vertical ; 172#*(3”+11”) = 2,408#” horizontal
okay based on plate strength

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P7 POST

1-1/2” schedule 40 ASTM A312
Stainless steel pipe - 316 alloy
Post Strength
t= 0.145”
1-1/2” schedule 40 1/16 hard
304 Stainless steel pipe
Post Strength
$Z_{yy} = 0.421$ in^3
$F_y = 40$ ksi SEI/ASCE 8-02 Table A1
$M_n = 0.421$ in^3 *40 ksi = 16,840#”

Service moment
$M_s = M_n / \Omega$
M_s = 16,840#”/1.67 = 10,084#”

Allowable top rail load for 42” total height:
P = 10,084#”/42” = 240#

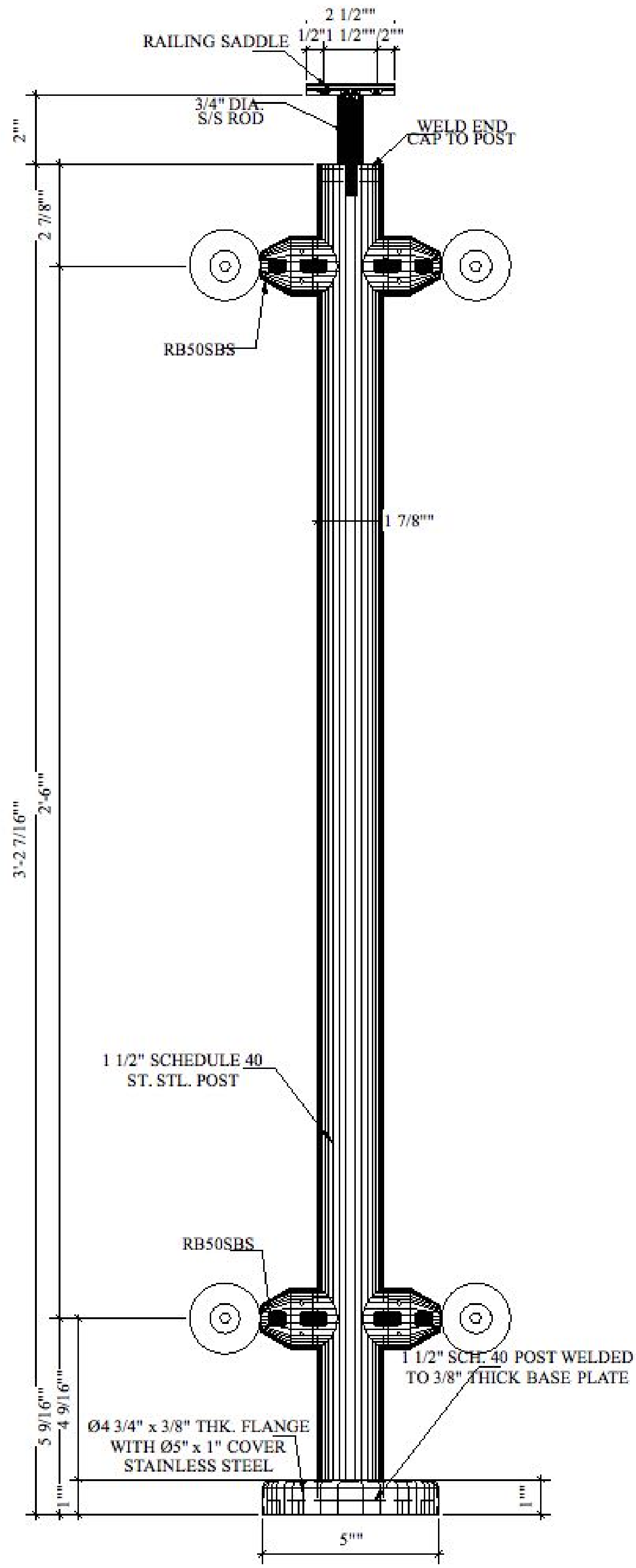
Allowable load on lite based on 42” post height – 20” to center of lite
W = 10,084#”/20” = 504#

based on 4.5’ x 3’ lite
w = 504#/(4.5’*3’/2) = 74.67 psf

Weld to base plate : 1/4” fillet weld all around – Weld filler to be same as post and base plate.
$S_w = 0.764$
M_{nw} = 0.764*0.6*75 ksi = 34,386#”
$M_s = M_{nw}/2.7$ =
M_s = 34,386/2.7 = 12,736#”

Base plate weld does not control.

Post Deflection:
Top of post deflection from 200# load:
$\Delta = 200\#*H"^3/(3EI)$
I = 0.293 in^4
For 42” post height:
$\Delta = 200\#*42"^3/(3*27\text{x}10^6*0.293) = 0.624"$



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P7 POST continued:

Base Plate design:
Same as for P6 Post.
See 4-3/4" Round Baseplate design and anchorage.

Glass fittings - RB51F/RB50F refer to fitting calcs.

Top Rail Connection Bracket -
Same as for P6 post. Adjustable/swivel bracket strength is same as for P1.

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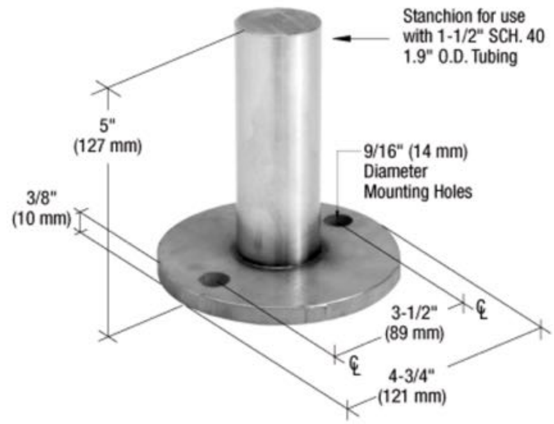
P7 POST continued:
Stanchion Mounted Posts
Surface Mount Stanchions
Cast stainless steel - stanchion cast integrally with the base plate
1.61" diameter solid 316 stainless steel.
$I_{xx}=I_{yy}=0.3298in^4$
$S_{xx}=S_{yy}=0.4097in^3$
$Z_{xx}=Z_{yy}=0.6955in^3$
Determine flexural strength of stanchion according to AISC design guide 27 and AISC Specification F11.



M_n=0.6955*30ksi=20,865"#<1.6*0.4097*30ksi=19,666"# (controls)
M_n/Ω=19,666"#/1.67=11,776"# (exceeds post strength will not limit system strength)

Base plate mounting per the 4-3/4" round base plate calculations.

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P8 POST

2"x1/2" Bars A304 hard Stainless Steel

Fy = 45 ksi

Post Strength

$Z_{yy} = 0.5*2^2/4 = 0.5\ in^3$

$F_y = 30$ ksi SEI/ASCE 8-02 Table A1

$M_n = 2\ bars*Z*F_y$

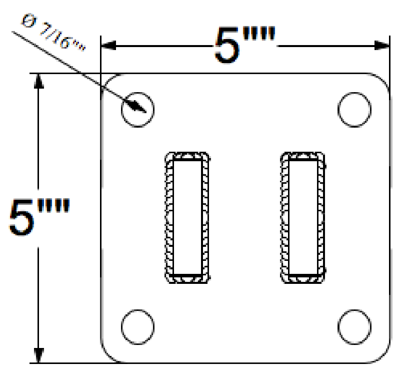
$M_n = 2*0.5\ in^3 *30\ ksi = 30{,}000"\#$

Service moment

$M_s = M_n /\Omega$

$M_s = 30{,}000"\# /1.67 = 17{,}964"\#$

Weld to base plate : 3/16" fillet weld all around –
Weld filler to be same as post and base plate.



Weld strength:

$S_{weld} = 2*[(.6875*2.1875^2)-(.3125*1.8125^2)]/6$

$S_{weld} = 0.754\ in^3$

$M_n = SF_y = 0.754\ in^3*0.6*75\ ksi = 33{,}948\#"$

$M_s = M_n /\Omega = 33{,}948\#"/2.7 = 12{,}573\#"$

Welds will control.

Allowable top rail load for 42" total height:

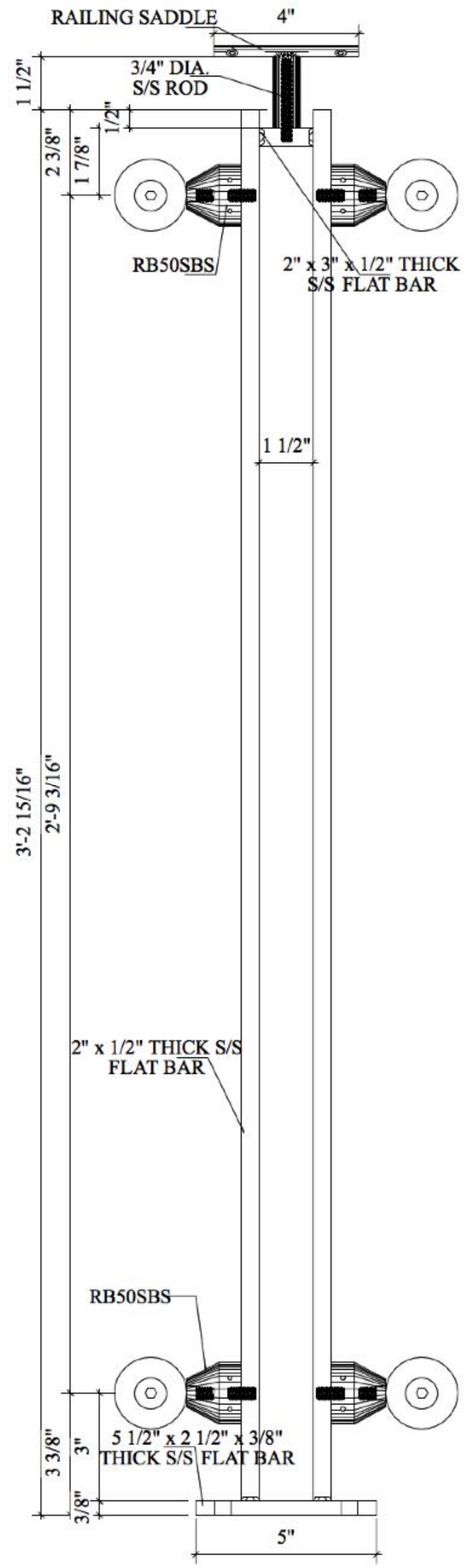
P = 12,573#"/42" = 299#

Allowable load on lite based on 42" post height – 20" to center of lite

W = 12,573#"/20" = 629#

based on 5' x 3' lite

w = 629#/(5'*3'/2) = 83.8 psf



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P8 POST continued:

Check lateral load on posts since post is not symmetric.
Z_{xx} = 2*0.5^2/4 x 2 bars = 0.25 in^3
M_n = 0.25*30 ksi = 7,500#"
$M_s = M_n / \Omega$
M_s = 7,500"# /1.67 = 4,491"#

For weld
S_{xw} = 0.1326"*2"*0.5"*2 = 0.265 in^3
M_{nw} = 0.265*0.6*70 ksi = 11,931#"
$M_s = øM_n$/1.6 = 7425/1.6 = 4,640

Top rail lateral force = 4,491/42 = 107#/ post

This post requires lateral support to resist lateral loads.

Post Deflection:
Top of post deflection from 200# load:
Δ = 200#*H"^{3}/(3EI)
I = 2*0.5*2^2/12 = 0.333 in^4
For 42" post height:
Δ = 200#*42"^{3}/(3*27x10^6*0.333) = 0.549"

Top Rail Connection Bracket
Same as for P6 post except that top cap is 1/2" flat bar.
Flat bar is welded between the post bars:
Weld strength:
A_{weld} = 0.707*3/16*2" = 0.265 in^2
$V_n = A_w$*0.6*F_e = 2*0.265 in^2*0.6*75 ksi = 23.85k

S_w = 2*0.707*3/16*2^2/6 = 0.1768 in^3
M_n =0.1768*0.6*75ksi = 7,954"#
$M_s = M_n / \Omega$ = 7,954#"/2.7 = 2,946#"

Welds won't limit top rail loads.

135° and 90° corner post variants have similar strength.

Glass Fittings:
Use either RB50 or RB51 fittings.
May use ZP series clamps.

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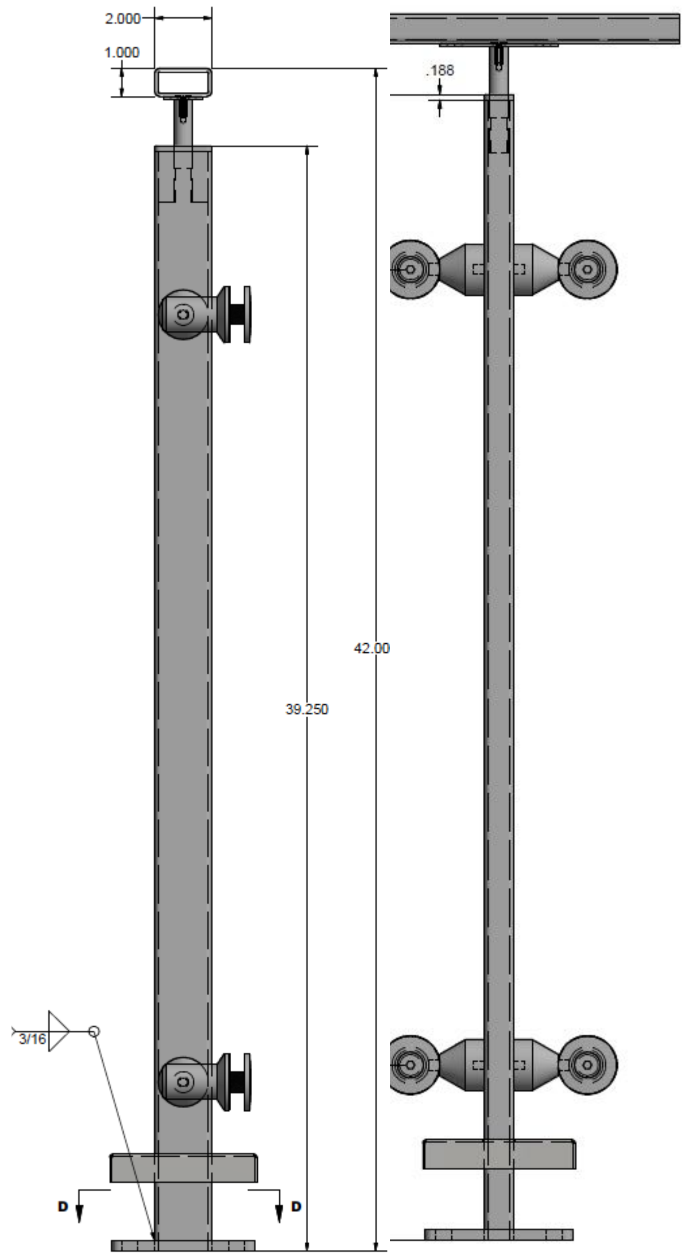
POST P9



Stainless steel (304) tube post mounted on stainless steel bar stanchion attached to stainless steel base plate.

Post Strength - 2” x 1” x 0.12” tube:
$I_x = 0.105$ in^4 $S_x = 0.210$ in^3, $Z_x = 0.25$in^3
$I_y = 0.332$ in^4 $S_y = 0.332$ in^3, $Z_x = 0.41$in^3
$F_y \geq 30$ ksi $F_u \geq 70$ ksi
For strong axis bending:
w/t = 0.675”/0.12” = 5.6
web:
w/t = 1.675”/0.12 = 13.96
$\lambda = (1.052/\sqrt{k})(w/t)(f/E_o)^{1/2}$
$\lambda = (1.052/\sqrt{k})(w/t)(f/E_o)^{1/2}$
$\lambda = (1.052/\sqrt{4})(5.6)^*$
$(1.25^*30\text{ksi}/27000)^{1/2}$
$\lambda = 0.11$
$\varrho = (1\text{-}0.22/\lambda)/\lambda = 8.87 > 5.6$
Use SEI/ASCE 8-023.3.2.1 Proc 2 with plastic modulus
M_n=1.25F_yZ =1.25*30^{ksi}*0.41=15,375”#
$M_a = M_n/1.65 =$
M_a = 15,375/1.65 = 9,318”#

For 42” post height-
Try 5” stanchion eight-
effective tube height
$h_e = h\text{-}h_s = 42\text{-}5 = 37$”
Allowable top post load:
R_a = 9318/37 = 252#
Okay for 5’ o.c. post spacing for 50 plf top rail loading and the standard stanchion

Check stanchion strength
3/4” x 1 3/4” X 5” bar for standard stanchion - cast with base plate
$Z = 0.75^*1.75^2/4 = 0.574$
For solid bar use the AISC DG 27 method:
M_{ny} = 1 bar*Z*F_y = 0.574 in^3 *30 ksi = 17,220”# at initial local distortion
M_{nu} = 1 bar*Z*F_u = 0.574 in^3 *70 ksi = 40,180”# at fracture

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POST P9 continued:

Service moment

based on fracture strength with safety factor of 2.5 since yield deflections will be very small:

$\partial = 0.002*1.7" = 0.0034$ about 1/300" deflection in the stanchion

$M_s = M_{nu} /2.5 = 40,180"\#/2.5 = 16,072"\#$

Allowable post top load:

$R_a = 16,072/42" = 383\#$ - post strength will control design

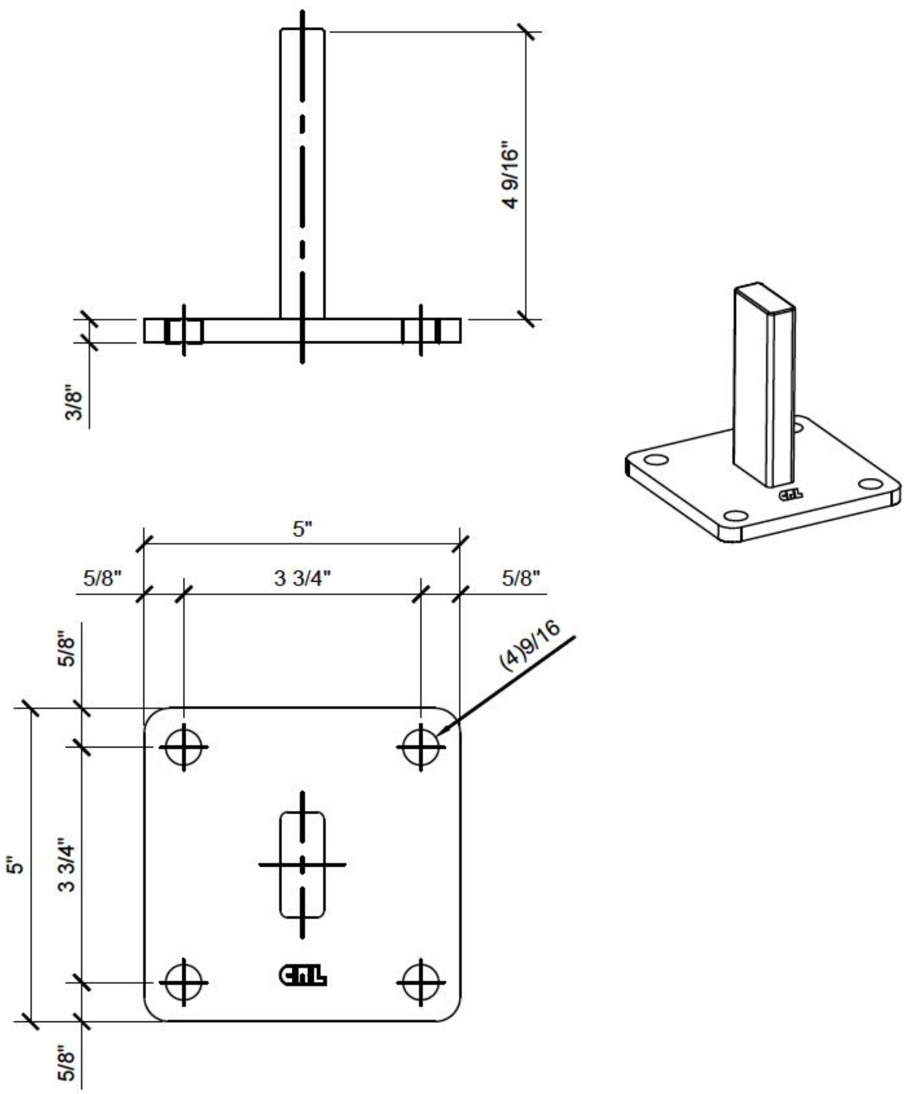
Based on physical testing performed by CR Laurence-

Fracture moment of the post and baseplate assembly:

$M_{fail} = 53,214"\# > 40,180"\#$ minimum strength calculated

Moment at full yield, plastic hinge, increasing deflection rate with decreasing rate of load increase:

$M_{plastic} = 24,300"\# > 17,220"\#$



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POST P9 continued:

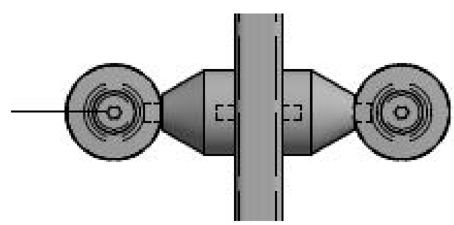
Base Plate:
5” square base plate same as previously checked, page 5.
Maximum allowable moment for baseplate strength = 13,534”#
$13{,}534 \geq 250*42” = 10{,}500”\#$

Post Deflection:
Top of post deflection from 200# load:
$\Delta = 200\#*H”^3/(3EI)$
$I = 0.75*2^3/12 = 0.50\ in^4$
Since there is a large transition in post stiffness (I) at the stanchion need to evaluate post deflection in two parts - deflection of tube alone + deflection of tube with stanchion:
$I_{tube} = 0.332\ in^4$
$I_{s+t} = 0.332\ in^4 + 0.334 = 0.666\ in^4$

For 42” post height:
5” stanchion and 200# load M = 8,400”#:
$\Delta = 200\#*(42”-5)^3/(3*20x10^6*0.332) + 200\#*5^2(3*42”-5)/(6*20x10^6*0.666) = 0.57”$

Based on physical testing which accounts for all deflection components including baseplate flexure and anchor deflection the typical post deflection at the 200# load will be approximately 3/4” for the 42” post height.

Glass fittings - RB51F/RB50F refer to fitting calcs.
Secure fittings to post with rod through post.
For end post with fitting on only one side use Hollo-bolt in post to anchor the fitting threaded rod for fitting located above the stanchion. Drill and tap stanchion for threaded rod if fitting is located within the stanchion height.



Z-clamps may be used with this post to support the glass.

Glass Lights - Uses typical glass lights as determined elsewhere herein.

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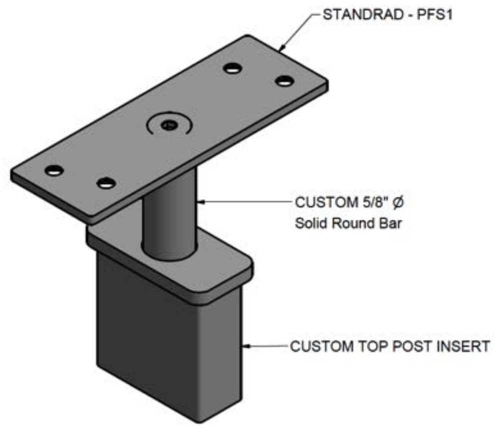
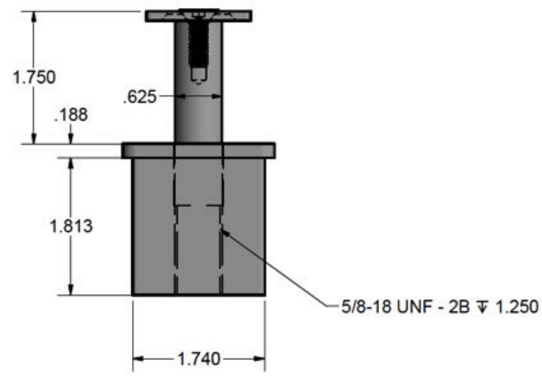
POST P9 continued:



Top Rail Mount:
Uses 5/8” solid round bar that is inserted into machined hole in solid bar inserted into the top of the post. End is threaded into tapped hole to provide positive restraint.

Bending strength of the bar-
per AISC DG 27 for connection element
$S = \pi d^3/32 = \pi * 0.625^3/32 = 0.024\ in^3$
$Z = d^3/6 = 0.625^3/6 = 0.0407\ in^3$

M_n = lesser of SF_u or ZF_y
$M_n = 0.024*70ksi = 1{,}680$”#
$M_n = 0.0407*30ksi = 1{,}221$”#
$Ma = 1{,}221/1.67 = 731$”#
Maximum top rail load based on maximum height of 2.25” to centerline of the top rail:
$R_a = 731/2.25 = 325\# > 322\#$ doesn’t control allowable top rail load.

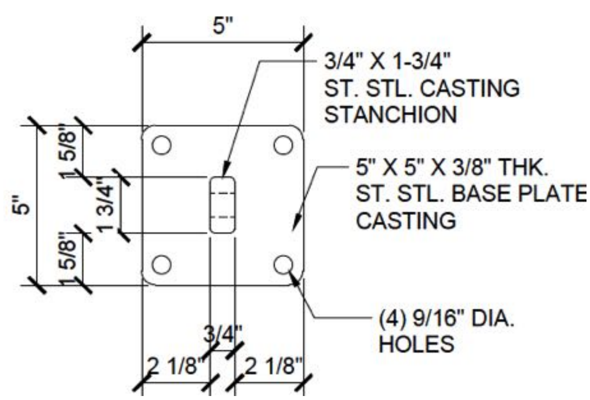


Connection of top rail to the mounting bar:
#1/4 SS screw ASTM F-879 or equal strength
Screw strength
$T_a = 0.02444in^2*75ksi/2 = 916\#$
$Va = 0.6*0.02444in^2*75ksi/2 = 550\#$

Connection to top rail:
Uses the PFS1 connection plate and (4) screws used by other posts in this report.

Baseplate
Anchorage of base plate is as calculated elsewhere for the substrate and anchors and 5” square base plate.

Baseplate covers - Covers are non-structural and are used to conceal the anchors. No design check is required.



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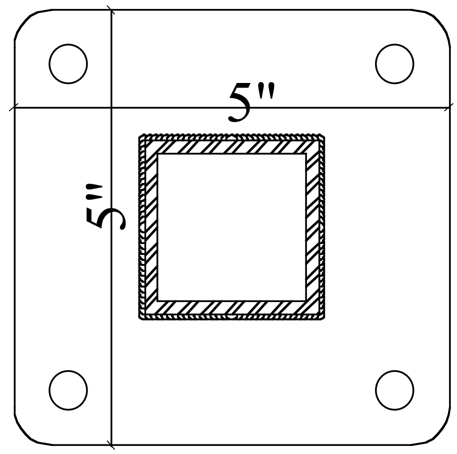
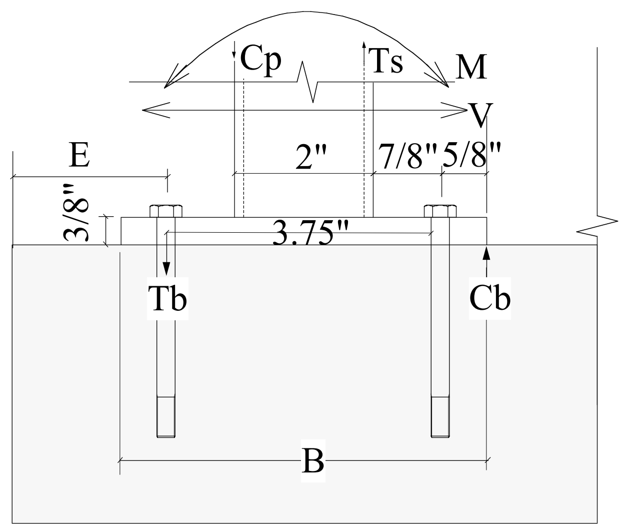
5”x5” Base Plate design:
May be used with any post.



for 3/8” plate $Z = \frac{5" \bullet 3/8^2}{4} = 0.176\ in^3$

$F_y = 45$ ksi
$M_n = Z\,F_y$
$M_n = 0.176*45ksi = 7{,}910\#”$
$M_s = M_n/1.67$
$M_s = 7{,}910\#”/1.67$
$M_s = 4{,}737\#”$

Calculate base plate reactions and moment based on the nominal strength of the posts.



$M = 19{,}805”\#$
$T_b = M/4.125”/2$ bolts
$T_b = 19{,}805/(4.125*2) = 2400\#$
Nominal anchor tension

Base plate moment
$M_u = 2*Tb*7/8”$
$M_u = 2*2400*(7/8”) = 4201\#”$
$M_u < øM_n$ therefore okay

Service tension
$T_{bs} = 11{,}140\#”/(4.125*2) = 1{,}350\#$

Base plate anchor strength:

Service strength required for anchors for anchorage to steel or concrete:
$T_s = 1{,}350\#$ (for allowable load on anchor).
For anchorage to steel use 3/8” bolts.

THIS BASE PLATE MAY BE USED WITH ANY OF THE POSTS IN THIS SERIES. THE STRENGTH AND ANCHORAGE WILL BE THE SAME FOR ALL POST TYPES.

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5"x5" BASE PLATE MOUNTED TO CONCRETE - Expansion Bolt Alternative:

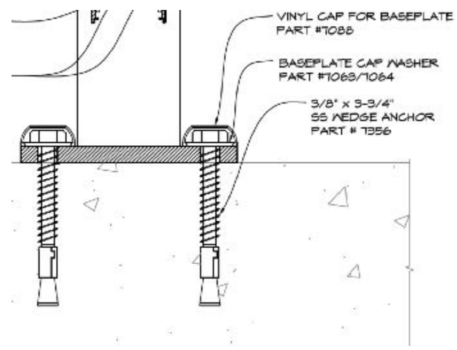
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8"x3.75" concrete anchors with 3" effective embedment. Anchor strength based on ESR-2427

Minimum conditions used for the calculations:

$f'_c \geq 3{,}000$ psi

edge distance =2.25" spacing = 3.75"

h = 3.0": embed depth



For concrete breakout strength:

$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$

$A_{Ncg} = (1.5*3*2+3.75)*(1.5*3+2.25) = 86.06$ in^2 2 anchors

$A_{Nco} = 9*3^2 = 81$ in^2

$C_{a,cmin} = 1.5$" (ESR-2427 Table 3)

$C_{ac} = 5.25$" (ESR-2427 Table 3)

$\varphi_{ed,N} = 1.0$

$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)

$\varphi_{cp,N} =$ max (1.5/5.25 or 1.5*3"/5.25) = 0.857 ($c_{a,min} \leq c_{ac}$)

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

$N_{cb} = 86.06/81*1.0*1.0*0.857*6{,}830 = 6{,}219 \leq 2*4{,}200$

based on concrete breakout strength.

Determine allowable tension load on anchor pair

$T_s = 0.65*6{,}219\#/1.6 = 2{,}526\#$

Check shear strength - Concrete breakout strength in shear:

$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$

$A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03$

$A_{vco} = 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$

$\varphi_{ed,V} = 1.0$ (affected by only one edge)

$\varphi_{c,V} = 1.4$ uncracked concrete

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

$V_b = [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5} = 1{,}636\#$

$V_{cb} = 43.03/40.5*1.0*1.4*1.225*1{,}636\# = 2{,}981\#$

Steel shear strength = 1,830#*2 = 3,660

Allowable shear strength

$ØV_N/1.6 = 0.70*2{,}981\#/1.6 = 1{,}304\#$

Shear load = 250/1,304 = 0.19 ≤ 0.2

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

$M_a = 2{,}526\#*4.375" = 11{,}053"\# > 10{,}500"\#$

DEVELOPS FULL BASEPLATE MOUNTING STRENGTH.

ALLOWABLE SUBSTITUTIONS: Use same size anchor and embedment

Hilti Kwik Bolt TZ in accordance with ESR-1917

Powers Power Stud+ SD2 in accordance with ESR-2502

Powers Wedge-Bolt+ in accordance with ESR-2526

Other anchors may be used if checked for the imposed loading conditions.

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5"x5" BASE PLATE MOUNTED TO WOOD - Lag Screw Alternative:
Moment at base plate attachment to deck:
M = 200#*42" = 8,400"#

Determine force on lag screws –
lever arm (centerline of lags to edge of base plate) = 4.375"
C = M/a
C = 8,400"#/4.375 = 1,920#
determine wood area required to support compression force based on F_{cT} = 405 psi, C_D = 1.33, and
C_b = 1.05
A = 1,920/(405*1.05) = 4.515 in^2
a = 4.515/(4.875) = 0.926

Lag withdrawal load:
T_l = 8400/[2*(4.375-0.926/2)]
T_l = 1,074#

Lag screw withdrawal strength:
W = 243#/in for 3/8" lag screw and G ≥ 0.43 (typ for Hem-Fir pressure treated wood) From NDS Table 11.24
C_D = 1.33 (IBC 16.7.1.3 and C_m = 0.7 (NDS table 10.3.3) for weather exposed wood.
W' = 243*1.33*0.7 = 227#/in
Required embedment length into the solid blocking:
e = T_l/W' = 1,074/227 = 4.73"
Required lag length:
L = 4.73"+3/8"+7/32" + T_d = 5.32"+ decking thickness

NOTE:
If lumber species is Southern Yellow Pine, Douglas Fir, Western Hemlock, or LVL/SCL:
W = 278#/in
e' = 4.73"*(243/278) = 4.134"
L = 4.134"+3/8"+7/32" + T_d = 4.73"+ decking thickness

NOTE 2:
For 36" rail height the embedment length is:
e_{36} = 4.73"*(36/42) = 4.054"
L_{36} = 4.054"+3/8"+7/32" + T_d = 4.65"+ decking thickness

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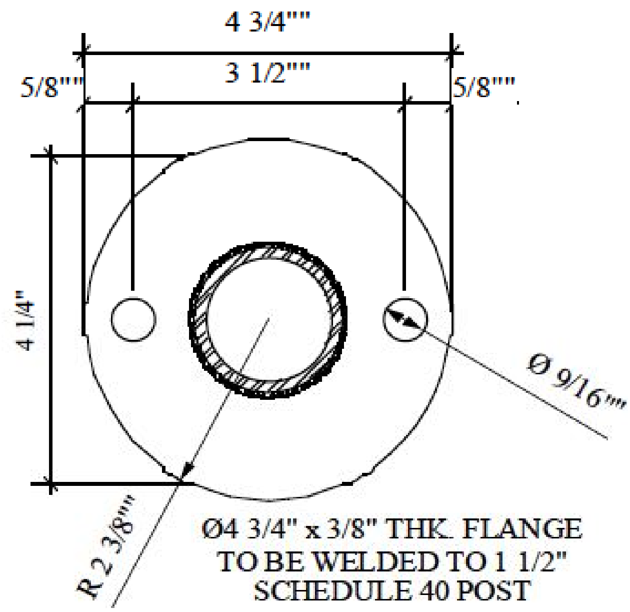
4-3/4" ROUND BASE PLATE
Mounted to Concrete
Designed per ACI 318-14 Chapter 17 and ICC ES-308
Maximum allowable post loads:
200# horizontal
8,400"# moment

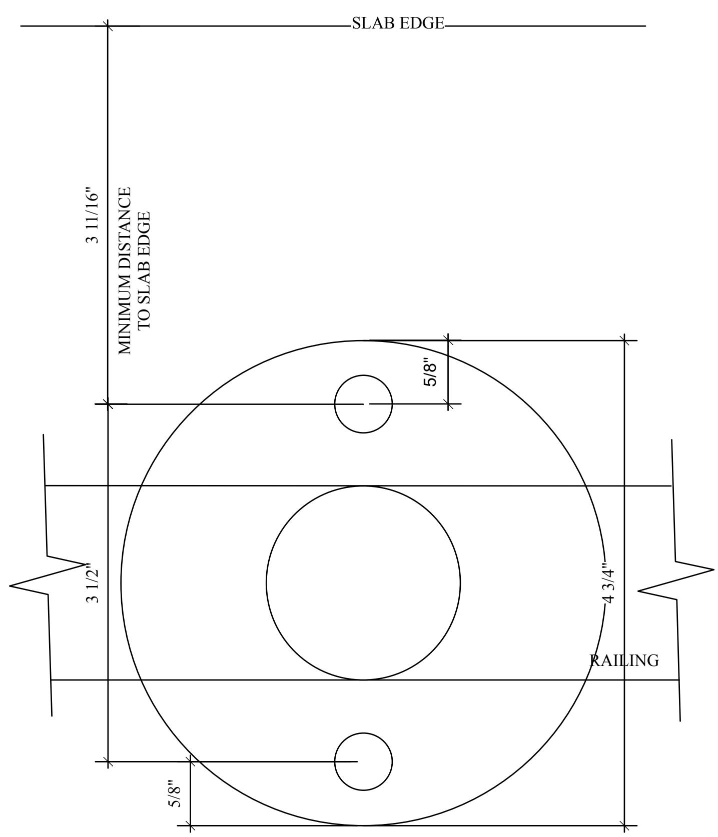


For anchors oriented along rail centerline-
For installation to 4,000 psi concrete:
Minimum slab thickness = 6.5"
Minimum edge distance = 5.25"
edge of slab to center of baseplate.

Hilti HIS-N B7 threaded rod insert with 4.75" embedment set with either Hilti HIT-HY 200 OR HIT-RE500-SD adhesive.

When installed to 5,000 psi concrete may use:
3/8" x 4" Hilti KWIK HUS-EZ (KH-EZ)

For base plate rotated with the anchors perpendicular to railing
3/8" x 4" Hilti KWIK HUS-EZ (KH-EZ)



3,000 psi concrete
Minimum slab edge distance of 3-11/16":

3,500 psi concrete
Minimum slab edge distance 3-3/8"

3,750 psi concrete
Minimum slab edge distance 3-3/16"

4,000 psi concrete
Minimum slab edge distance 3-1/16"

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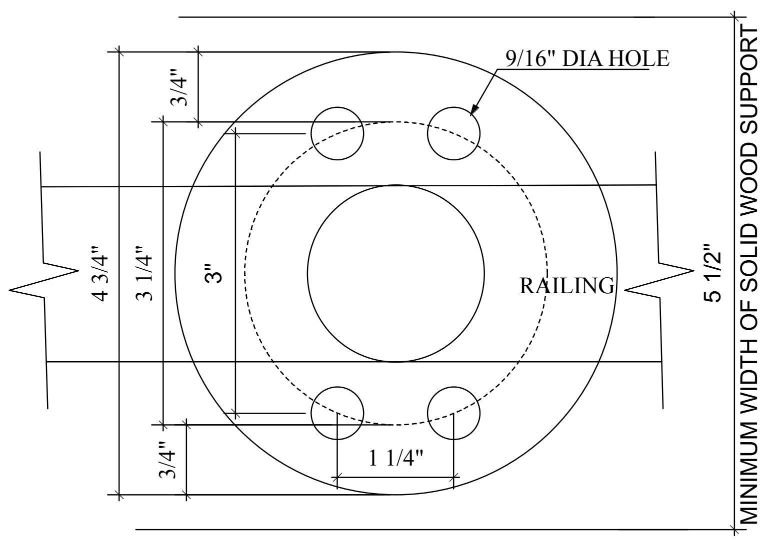
4-3/4" ROUND BASE PLATE
Mounted to Wood
4 Hole configuration



1/2" Lag screws into solid wood framing or blocking.

Lag screw withdrawal strength:
W = 302#/in for 1/2" lag screw and G ≥ 0.43 (typ for Hem-Fir pressure treated wood) From NDS Table 11.24

C_D = 1.33 (IBC 16.7.1.3
C_m = 0.7 (NDS table 10.3.3) for weather exposed wood.

W' = 302*1.33*0.7 = 281#/in

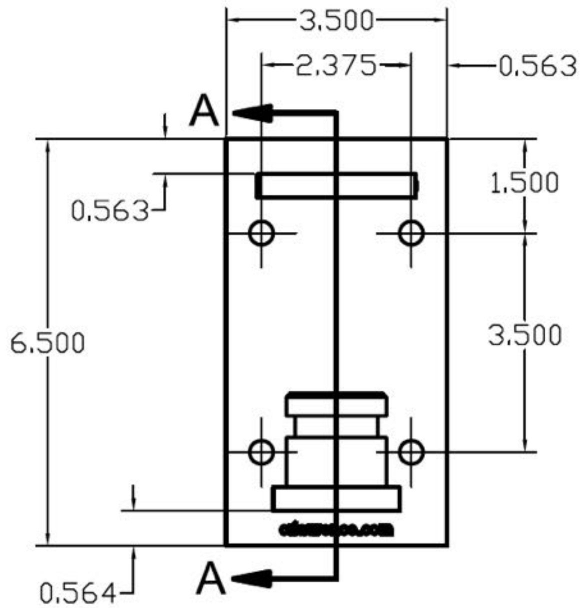
Determine force on lag screws –
lever arm (centerline of lags to edge of base plate) = 4"
C = M/a
C = 8,400"#/4 = 2,100#
determine wood area required to support compression force based on:
F_{cT} = 405 psi and C_b = 1.05
A = 2,100/(405*1.33*1.05) = 3.395 in^2
a = 3.395/(4.875) = 0.696

Lag withdrawal load:
T_l = 8,400/[2*(4.0-0.696/2)]
T_l = 1,150#

Required embedment length into the solid blocking:
e = T_l/W' = 1,150/281 = 4.09"
Required lag length:
L = 4.09"+3/8"+5/16" + T_d = 4.78"+ decking thickness

FASCIA BRACKETS

Bracket is used to support railing posts by mounting to the side of a wall, beam, fascia or similar vertical surface. The post is anchored at the bottom by fitting over a cylinder on a plate attached to the fascia plate. The upper ring locks the post from rotating and creates a couple with the bottom support to resist horizontal forces and induced moments. Maximum height from top of bracket to top of guardrail is 48". For 5' post spacing the design load on the post is 250#
(50plf load controls).

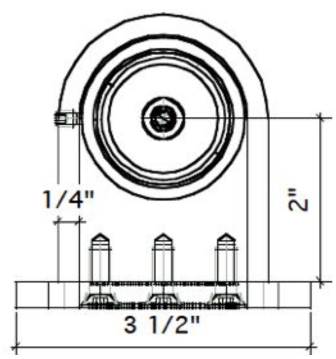


Determine forces on the brackets:

Vertical forces
D = 5.5psf*3.5*5/2 + 40#
D = 100# (rounded up)
L = 250#
For horizontal forces bottom plate must also support vertical dead load.
From $\sum$M about the base = 0 determine upper ring load:

H_t = 250#*(48"+4.75")/4.75" = 2,776#
Bottom bracket load: H_b = 2,776#-250# = 2,526#
with V = 100#

TOP RING:



Ring is secured to the back plate by three 5/16" screws installed through the back plate and into tapped holes in the ring. Screws A-2 or similar grade with minimum yield strength $F_y \geq 72$ ksi
Screw tension strength:
$A_T = 0.0524$ in^2
$T = \phi A_T F_y/1.6 = 0.75*0.0524$ in^2*72 ksi/1.6 = 1,770# each
T_{total} = 3*1,770# = 3,540# > 2,776# - okay
Shear strength, threads not in shear plane
$A_v = 0.0524$ in^2
$V_s = \phi A_v F_v/1.6 = 0.65*0.0524$ in^2*46 ksi/1.6 = 980#
V_{total} = 3*980 = 2,940#
For loads that are not either parallel or perpendicular to the rail the reaction will be a combination of shear and tension. The interaction case will be okay since they have pure shear/pure tension cases.

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FASCIA BRACKETS Cont.

Bottom fitting strength:

Post end fitting is inlaid into bottom plate so that shear forces are directly transferred.

Bending stresses from vertical forces:

V_m = 350# (D+L)

M_V = 350#"*2" = 700#"

V_D = 100#

M_D = 100# * 2" = 200#"

Tension on screw:

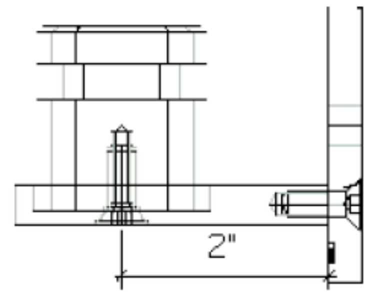
$H_L + V_D$ = 2,526# + 200#"/0.25" = 3,326#

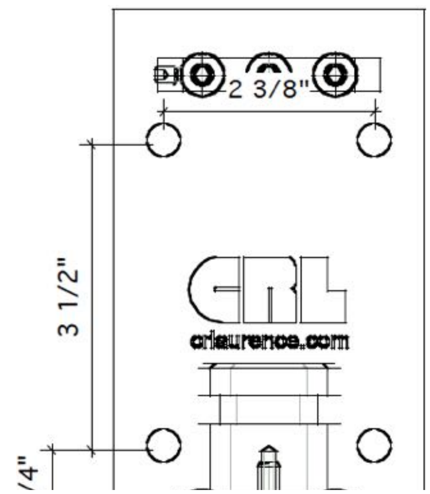
For vertical load

T_V = 700#"/0.25" = 2,800#"

Allowable tension for three screws = 3,540# > 3,326# okay

Plate bending is okay from inference.





CORNER BRACKETS

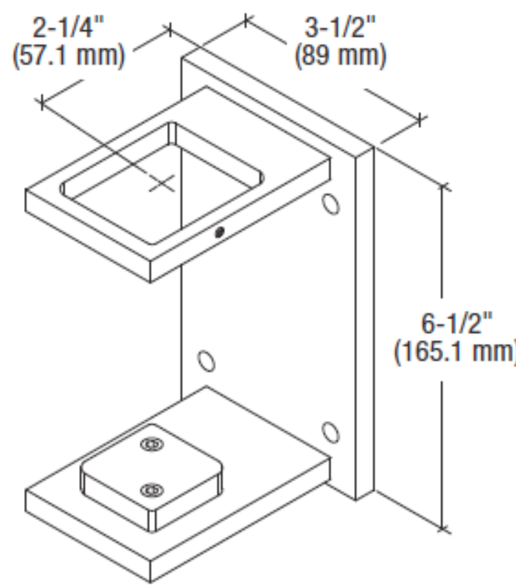
Inside and outside corner brackets have strength similar to the straight bracket.

P8 Post Fascia Bracket

Post bars are elongated and welded to 5" wide x 7" tall x 3/8" plate. Plate would be mounted same as for fascia brackets.

weld strength: Weld develops the full post strength

M_w = 0.26"*6"^{2}/6*0.6*75ksi/2.7 = 26,000"# each bar



FASCIA BRACKET ATTACHMENT

Bracket is fastened to the structural support using four bolts.

For horizontal loads:

M_H = 250#*(48"+4.75") = 13,187.5#"

Dead load will add shear and moment

M_D = 100#*2" = 200#"

M_T = 13,875.5#"+200#" = 14,075.5#"

V = 100#

Determine tension on anchors, will be greatest for outward force.

From $\sum$M about the bottom of the plate = 0

14,075.5#" – 2(1.45"*T_l) – 2(4.95*T_U) = 0

from similar triangles

$T_l = T_U$ *(1.45/4.95) = 0.29 T_U

Solving above for T_U

T_U = 14,075.5/(0.849+9.9) = 1,309#

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FASCIA BRACKETS Cont.

Anchor Alternatives

To steel
3/8" stud or bolt Grade A-2 or stronger:
$T_a = 0.0775in^2*20ksi = 1.55k \geq 1{,}309\#$
Okay for full post strength

Anchorage to Concrete:
Designed per ACI 318-14 Chapter 17 and ICC ES-308
Limited to 200# live load (4' post spacing for commercial installations)
3/8" x 4" Hilti KWIK HUS-EZ (KH-EZ)
f'c ≥ 3,000 psi, uncracked.
Set top of bracket at 2" down from edge of concrete.
$L_u = 1.6*200 = 320\#$
$D_u = 1.2*100\# = 120\#$
$M_u = 320*(42"+2"+6.5") + 120*2.25" = 16{,}430"\#$

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1902	6718	29	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	3895	4324	91	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	30	3111	1	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	120	13454	1	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.901	0.010	1.000	76	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

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FASCIA BRACKETS Cont.

Anchor Alternatives:
To wood:
3/8" x 6" Lag screw
Limited to 200# live load (4' post spacing for commercial installations)
Set top of bracket at top of beam
L_u = 200
D_u = 100#
M_u = 200*(42"+6.5") + 100*2.25" = 9,925"#

Lag screw withdrawal strength:
W = 243#/in for 3/8" lag screw and G ≥ 0.43 (typ for Hem-Fir pressure treated wood) From NDS Table 11.24
C_D = 1.33 (IBC 16.7.1.3 and C_m = 0.7 (NDS table 10.3.3) for weather exposed wood.
W' = 243*1.33*0.7 = 227#/in
Embedment length into the solid beam:
L = 6 - 5/16" - 7/32" = 5.469"
Total withdrawal strength
W" = 5.469"*227#/in = 1,241#
Allowable moment on bracket:
M_a = 2*1241#*[5" - 0.5*2*1,241/(405*3.5")] = 10,237

NOTE:
If lumber species is Southern Yellow Pine, Douglas Fir, Western Hemlock, or LVL/SCL:
W = 278#/in
Increased allowable loads:
278/243 = 1.14

For interior installations or locations where wood moisture content will remain below 19% may increase allowable loads by 1/0.7 = 1.43 up to 250# live load maximum.
Or reduce required lag screw length:
L_{Dry} = 5.469*0.7 + 5/16 + 7/32 = 4-3/8"

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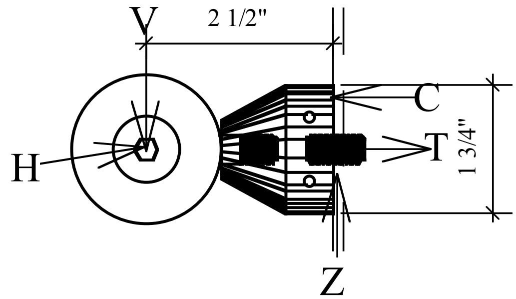
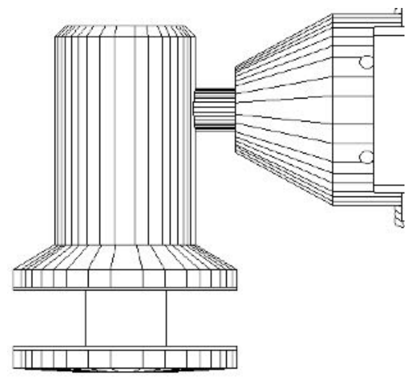
GLASS STANDOFFS
RB50S or RB50F



Determine standoff strength:
$M = P*2.5"$ where $P = V$ or H
Shear on screw $= Z = H$ or V
$C = T = M/(1.75"/2) = P*(2.5"/0.875") = 2.86P$

Strength of Screw into post
Strength of screw 316 Condition CW ASTM F593-98
$T_n = 71.2$ ksi*0.0524 in^2 = 3,731#
Check for pull out strength:
$P_{On} = 1.2*D\ t_c\ F_{y1} = 1.2*(5/16)*(1/8)*75$ ksi = 3,516#

Moment resistance of connection:
M_n = 3,516#*(0.75"/2) = 1,319#"



$M_s = M_n/2$ = 1,319/2 = 660#"

Shear Strength
$Z_n = F_{nv}*A_v$ = 42.8 ksi*0.0454 in^2 = 1,943#
$Z_s = Z_n/1.67$ = 1,943#/1.67 = 1,163#

Determine service load of standoff from interaction equation where:
$(M/M_s)^2+(Z/Z_s)^2 \leq 1.0$

$P = \sqrt{(H^2+V^2)}$
$Z = P$
$M = 2.5"*P$
substituting using P:
$(2.5P/660)^2 + (P/1{,}163)^2 = 1$ then solving for P
$P = \{1/[(2.5/660)^2 + 1/1{,}163^2]\}^{1/2}$
P = 257# = V
For typical glass lite = 5' x 3'
V = dead load weight of glass = 4.75 psf *5'*3'/(4 standoffs per lite) = 17.8#
$H = [P^2-V^2]^{1/2} = [257^2 - 17.8^2]^{1/2}$ = 256#
Allowable horizontal load based on standoff strength
h = 256#/(5'*3'/4) = 68.27 psf
Standoff strength may limit load on glass lite.

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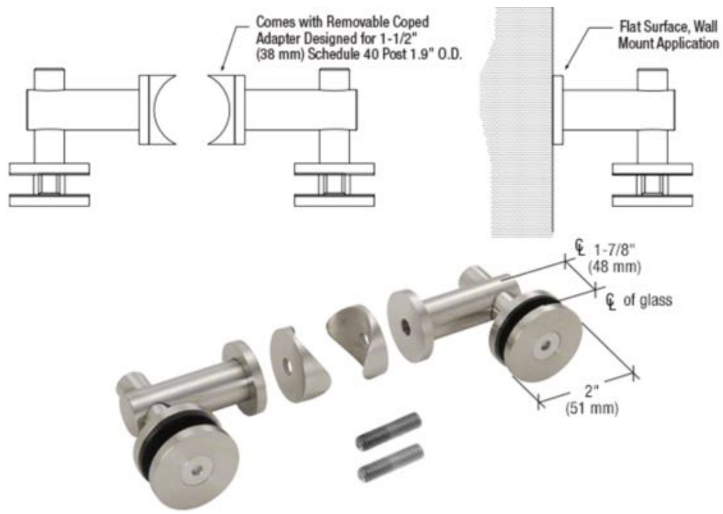
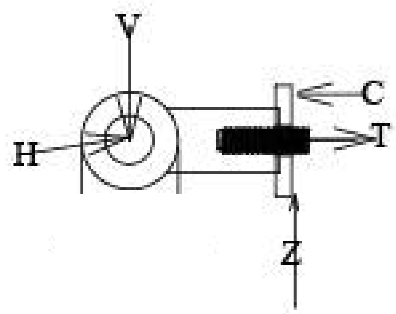
GLASS STANDOFFS
RB51F



Stainless steel strength based on ASTM A276-00a Cond A cold finished Table 2- 304 or 316 SS
Determine standoff strength:
M = P*2.5” where P = V or H
Shear on screw = Z = H or V
C = T = M/(1.75”/2) =
P*(2.5”/0.875”) = 2.86P

Strength of Screw into post
Strength of screw 316 Condition CW
ASTM F593-98
Strength from Table 2 Group 1
T_n = 80 ksi*0.0524 in^2 = 4,192#
Check for pull out strength: (0.145” min SS post wall)
P_{On} = K t_c 0.6F_{u1} = 0.682”*(0.145”)*0.675 ksi = 4,450#



Moment resistance of connection:
M_n = 4,192#*(0.75”/2) = 1,572#”

M_s = M_n/2 = 1,572/2 = 786#”

Z_n = F_{nv}*A_v = 0.58*80 ksi*0.0454 in^2 = 2,107#
Z_s = Z_n/2 = 2,107#/2 = 1,053#

Determine service load of standoff from interaction equation where:
$(M/M_s)^2+(Z/Z_s)^2 \leq 1.0$
$P = \sqrt{(H^2+V^2)}$
Z = P
M = 2.5”*P
substituting using P:
$(2.5P/786)^2 + (P/1{,}053)^2 = 1$ then solving for P
$P = \{1/[(2.5/786)^2 + 1/1{,}053^2]\}^{1/2}$
P = 301# = V
For typical glass lite = 5’ x 3’ - dead load carried by two standoffs
V = dead load weight of glass = 6.7 psf *5’*3’/(2 standoffs per lite) = 50#
$H = [P^2-V^2]^{1/2} = [301^2 - 50^2]^{1/2}$ = 297#
Allowable horizontal load based on standoff strength
h = 297#/(5’*3’/4) = 79.2 psf

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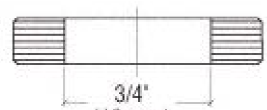
RRF10BS/PS
Rigid fixed head
Mount to spider fitting

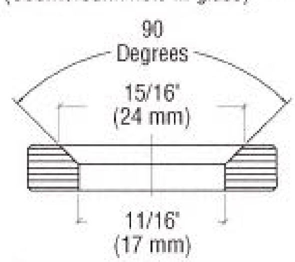

Cap Mount
(Standard hole through glass)



Cap mount glass fabrication

Fitting strength:
M10 threaded rod 316 Stainless steel
Shear strength:
$A_t = 57.99mm^2 = 0.0899in^2$
$A_v = 78.54mm^2 = 0.1217in^2$
$øV_n = 0.65*0.1217in^2*42.8$ ksi = 3,386#
$øT_n = 0.75*0.0899in^2*71.2$ ksi = 4,800#
For typical installation
$øM_n$ = 0.9*4,800#*0.39" = 1,691#" (based on rod in tension couple)
or for pure bending in rod:
$Z = 0.39"^3/6 = 0.00989in^3$
$øM_n = 0.9*71.2ksi*0.00989in^3$ = 634#"
for typical eccentricity = 1/4"+3/16" = 0.4375
P_n = 634#"/0.4375" = 1,449#
Determine allowable load
$(M/M_s)+(Z/Z_s) \leq 1.2$
Typical will be L = 200# or W = 350# and D = 100#
P_u = 1.6*200+1.2*100 = 440# or 1.6*350#+1.2*100 = 680#
M_u = 680#*0.4375" = 297.5#"
combined:
(680#/4,800#)+(297.5#"/634#") = 0.61 < 1.2 okay
Max allowable: F_R = 765# $F_x = F_y$ = 139#
FITTING SUPPORTS
Fittings are supported by steel with a minimum thickness of 1/4" designed for the concentrated load on the fitting.


Flush Mount
(Countersunk hole in glass)



Flush mount glass fabrication

STRENGTH OF COUNTERSUNK FITTING:
Check failure of bearing ring:
$øV_n = 0.65*(3/16"*0.9375"*\pi*25$ ksi = 8.97k
Will not control

Check for glass stress:
$\sigma = P_n/(0.5t*0.6875\pi) = P_n/(1.08t)$
Using maximum from above with 3/8" glass:
σ = 906#/(1.08*0.375) = 2236 psi

Bearing area:
$A = (1/4)*13/16\pi = 0.638$ in^2
FITTING REQUIRES TEMPERED GLASS

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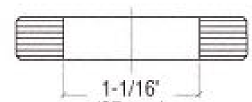
RSF10BS/PS
Combination Swivel head
Mount to spider fitting

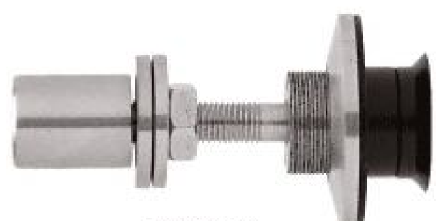
Cap Mount
(Standard hole through glass)

Fitting strength:
M10 threaded rod 316 Stainless steel
Shear strength:
$A_t = 57.99mm^2 = 0.0899in^2$
$A_v = 78.54mm^2 = 0.1217in^2$
$øV_n = 0.65*0.1217in^2*42.8$ ksi = 3,386#
$øT_n = 0.75*0.0899in^2*71.2$ ksi = 4,800#
Strength of swivel ball joint: Shear failure around socket rim:
$øV_n = 0.85*42ksi*0.95*0.55"*\pi*0.065"= 3,809\#$



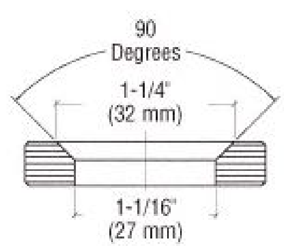
Cap mount glass fabrication

For typical installation
$øM_n = 0.9*3,809\#*0.39" = 1,337\#"$ (based on rod in tension couple)
or for pure bending in rod:
$Z = 0.39"^3/6 = 0.00989in^3$
$øM_n = 0.9*71.2ksi*0.00989in^3 = 634\#"$
for typical eccentricity = 1/4"+3/16" = 0.4375
$P_n = 634\#"/0.4375" = 1,449\#$
Determine allowable load
$(M/M_s)+(Z/Z_s) \leq 1.2$
Typical will be L = 200# or W = 350# and D = 100#
$P_u = 1.6*200+1.2*100 = 440\#$ or $1.6*350\#+1.2*100 = 680\#$
$M_u = 680\#*0.4375" = 297.5\#"$
combined:
(680#/3,809#)+(297.5#"/634#") = 0.65 < 1.2 okay
Max allowable: $F_R = 550*1.35=742$ # $F_x = F_y = 135\#$
STRENGTH OF COUNTERSUNK FITTING:
Check failure of bearing ring:
$øV_n = 0.65*(3/16"*0.9375"*\pi*25$ ksi = 8.97k



Flush Mount
(Countersunk hole in glass)

Check for glass stress:
$\sigma = P_n/(0.5t*0.6875\pi) = P_n/(1.08t)$
Using maximum from above with 3/8" glass:
$\sigma = 906\#/(1.08*0.375) = 2236$ psi



Flush mount glass fabrication

Bearing area:
$A = (1/4)*13/16\pi = 0.638$ in^2
FITTING REQUIRES TEMPERED GLASS

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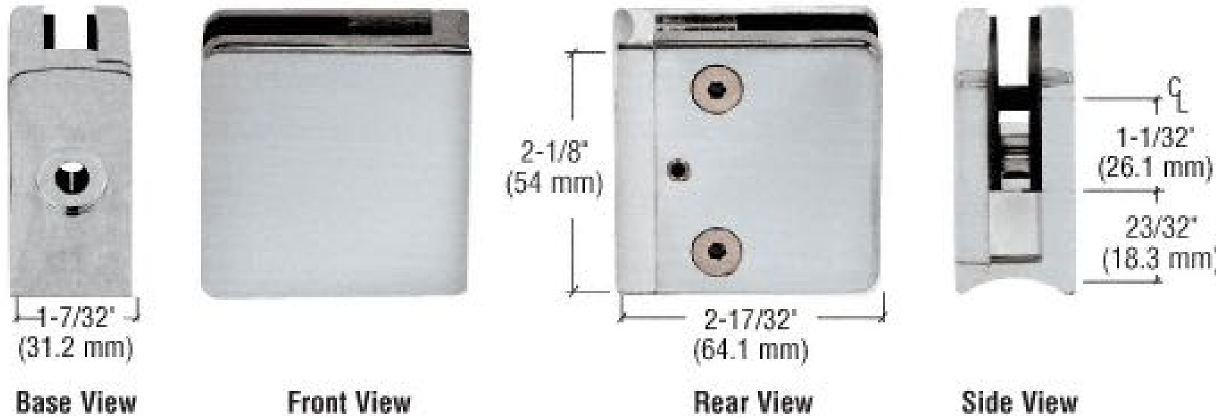
Z-CLAMPS - CRL Z712BS

Clamps to the glass on the side of the glass using a hole through the glass light at the clamp.



The Z-clamp provides a simple connection point (shear transferred only with no moment resistance.)

Load to the Z- clamp:
Vertical load V = D/4 (lite weight/ four clamps per lite)
$D = L'xB'x\ D_G$
L = light length, B = light height and D_G = light weight in psf
Glass dead load:
$D_{1/2"} = 6.5$ psf

Horizontal load (H), live or wind:
W or L in psf
H = L'xB'x W or L
Load per clamp = H/4

Glass bearing on clamp pin:
allowable = t*d*3,000 psi

Clamp Strength:
Screws holding clamp back to front: two #10 SS screws into tapped holes.
$øT_n = 0.75*67.5ksi*0.0175\ in^2 = 886\#$ each
$T_s = øT_n/1.6 = 886/1.6 = 554\#$
Total allowable service tension load on bracket = 2*554# = 1,108#

Check screw shear strength
Screw strength, $A_v = 0.0175\ in^2$, $F_{nt} = 33.7$ ksi
$V_s = ø\ A_v\ F_{nv}\ /1.6 = 0.65*.0175*33.7/1.6 = 240\#$
Total allowable service shear load on bracket = 2*240# = 480#

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Shear carried by through glass pin in clamp:

Pin strength, $A_v = 0.0767$ in^2, $F_{nt} = 33.7$ ksi

$T_s = ø\ A_v\ F_{nv}\ /1.6 = 0.65*0.0767*33.7/1.6 = 1{,}050\#$

Pin strength will not control clamp loading

FOR LARGE SERIES CLAMPS (2-1/8" TALL)

Moment on screw from vertical load

$M_V = V*1.469"$

Screw tension from ∑M and solving for T

$T_V = M_V/(2.125"/2) = 1.469V/1.0625" = 1.382V$

setting $T_V = T_s$ and solving for V

$V = 1{,}375\#/1.382 = 995\#$ (for all large clamps)

Vertical load will not be limiting for any of the clamp styles or attachments.

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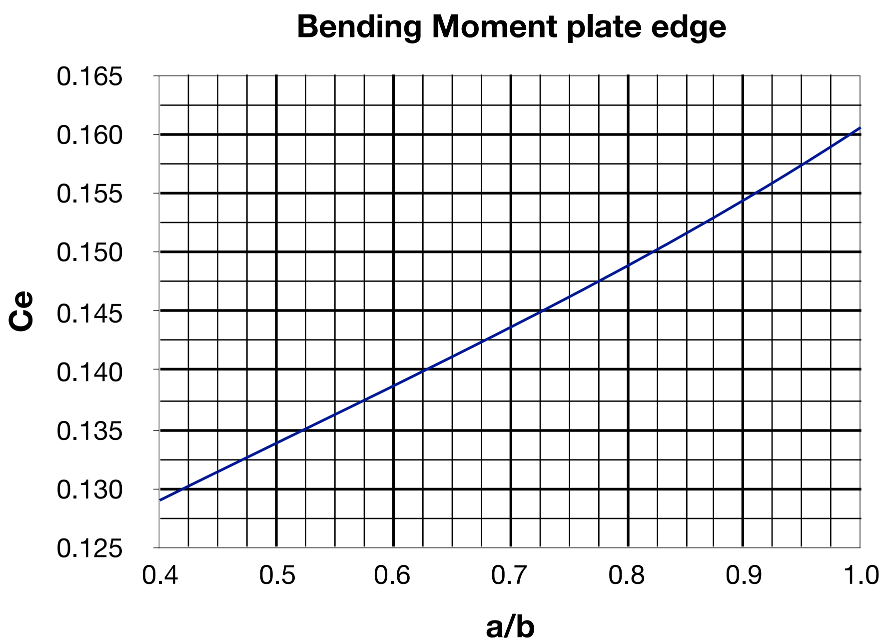
GLASS INFILL LIGHTS

Check Glass loads based on the 25psf or 50# concentrated load
L = On center distance between glass support points
H = height of glass light; H_1 = vertical distance between upper and lower glass supports
Check bending moment assuming bending along long axis of lite, simple support so that bending moment is 0 at the glass edge:
$M_{yy} = 0$ at top and bottom edge and maximum at center of sides
$M_{xx} = 0$ at side edges and maximum to center of top and bottom



a = short side and b = long side
$M_l = C_e*W*L^2$ or $2*C_e*50\#/H*L$
For bending along short axis
$M_t = C_e*W*H_1^2$
W = wind load ≥ 25 psf
for 5' long x 3' tall light:
a/b = ⅗ = 0.6; from graph: $C_e = 0.138$
$M_l = 0.138*25psf*5'^2 = 86.25'\#$
or for concentrated load
$M_l = 2*0.138*50\#*5' = 69.0'\#$

Stress concentration at the glass support points from shear stress in glass:

Fitting diameter = 2", hole size = 0.5"

$\sigma = (1-a^2/r^2)(1+3a^2/r^2) = (1-.25^2/1^2)(1+3*.25^2/1^2) = 1.11$

$f_v = 1.11*V/(2"*0.5")$

Glass shear stresses never govern the glass design.

GLASS STRENGTH

All glass is fully tempered glass conforming to the specifications of ANSI Z97.1, ASTM C 1048-12 and CPSC 16 CFR 1201. The median Modulus of Rupture for the glass F_r is 24,000 psi. In accordance with IBC 2407.1.1 glass used as structural balustrade panels shall be designed for a safety factor of 4.0 for the guard loads required by IBC 1607.7. For wind loads or for glass not used in guardrails design is in accordance with ASTM E1300-12.

For Guard Live Loads:

Allowable glass bending stress: 24,000/4 = 6,000 psi. – Tension stress calculated.
Allowable compression stress = 24,000psi/4 = 6.000 psi.
Allowable bearing stress = 24,000 psi/4 = 6,000 psi.
Allowable shear stress = 0.5*24,000psi/4 = 3,000 psi

For wind load on glass - recommended maximum edge stress is 9,600 psi. ASTM E1300-12 allows wind load stress of 10,600 psi but the lower stress is recommended because of the point supports and exposure.

Bending strength of glass for the given thickness:

$$S = \frac{12\text{''} * (t)^2}{6} = 2 * (t)^2 \text{ in}^3/\text{ft}$$

For 1/4" glass $S = 2*(0.25)^2 = 0.125$ in³/ft

$M_{alllive}$ = 6,000psi*0.125 in^3/ft = 750#"/ft = 62.5'# Live loads
$M_{allwind}$ = 9,600psi*0.125 in^3/ft = 1,200#"/ft = 100'# Wind loads

For 3/8" glass $S = 2*(0.360)^2 = 0.259$ in^3/ft

$M_{alllive}$ = 6,000psi*0.259 in^3/ft = 1,555#"/ft = 129.583'#
$M_{allwind}$ = 9,600psi*0.259 in^3/ft = 2,486#"/ft = 207.2'#

For 1/2" glass $S = 2*(0.469)^2 = 0.44$ in^3/ft

$M_{alllive}$ = 6,000psi*0.44 in^3/ft = 2,640#"/ft = 220.0'#
$M_{allwind}$ = 9,600psi*0.44 in^3/ft = 4,224#"/ft = 352.0'#

For 5/8" glass $S = 2*(0.595)^2 = 0.708$ in^3/ft

$M_{alllive}$ = 6,000psi*0.708 in^3/ft = 4,248#"/ft = 354.0'#
$M_{allwind}$ = 9,600psi*0.708 in^3/ft = 6,797#"/ft = 566.4'#

For 3/4" glass $S = 2*(0.719)^2 = 1.034$ in^3/ft

$M_{alllive}$ = 6,000psi*1.034 in^3/ft = 6,204#"/ft = 517.0'#
$M_{allwind}$ = 9,600psi*1.034 in^3/ft = 9,926#"/ft = 827.1'#

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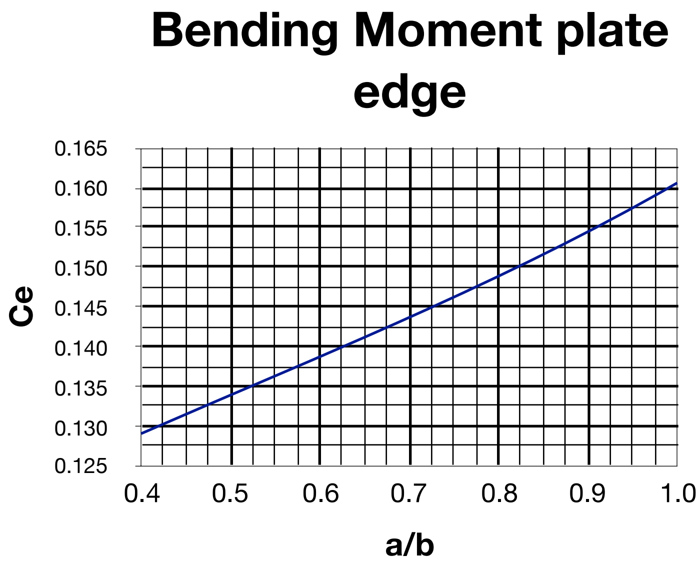
GLASS INFILL DESIGN

Determine the maximum allowable light size for live loads, either 50# concentrated or 25 psf:



For 1/4" Glass:
M_a = 750#"/ft = 62.5#'/ft

For square light C_e = 0.161
Maximum L:
L = √[(62.5#'/ft)/(25psf*0.161)]
L = 3.94' square
L = 62.5#'/ft/(0.161*50) = 7.764'
(will only control when H ≤ 0.894')

For 3/8" Glass:
M_a = 1,555#"/ft = 129.58#'/ft (SF = 4.0)

For square light C_e = 0.161
Maximum L:
L = √[(129.58#'/ft)/(25psf*0.161)]
L = 5.674' square
L = 129.58#'/ft/(0.161*50) = 16.097'
(will only control when H ≤ 0.894')

Check for 36" high glass light:
For ¼" glass estimate a/b = 0.7:
C_e = 0.144
L = √[(62.5#'/ft)/(25psf*0.144)] = 4.167' = 50"
a/b = 36/50 = 0.72 approximately 0.7 therefore use 36" x 50" maximum for ¼" glass

For 3/8" glass estimate a/b = 0.5:
C_e = 0.134
L = √[(129.58#'/ft)/(25psf*0.134)] = 6.219'
a/b = 3/6.219 = 0.48 approximately 0.5 therefore use 36" x 72" maximum for 3/8" glass

Other glass sizes may be checked using the edge moment coefficient C_e and the formula herein.

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