Edward C. Robison, P.E.

21 January 2025

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 2015, 2018, 2021 and 2024 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.

Edward Robison, P.E.

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2:14:29PM PST

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THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION
04/30/2026
Signature Expiration Date of the License

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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,
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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: Typed or printed name:
Edward C. Robison
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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	26,000”#	11,053”#	8,400”#	5.263’	6.00’
P2[6]	15,800”#	11,053”#	8,400”#	5.263	6.00’
P3[6]	13,500”#	9,259”#	8,400”#	4.409’	5.00’
P4[6]	14,400”#	11,053”#	8,400”#	5.263’	6.00’
P5[6]	8,900”#	8,982”#	8,400”#	4.277’	5.00’
P6	11,900”#	8,400”#	8,400”#	4.800’	6.00’
P7	11,900”#	8,400”#	8,400”#	4.800’	6.00’
P8[6]	18,000”#	11,053”#	8,400”#	5.263	6.00’
P9[6]	12,500”#	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.

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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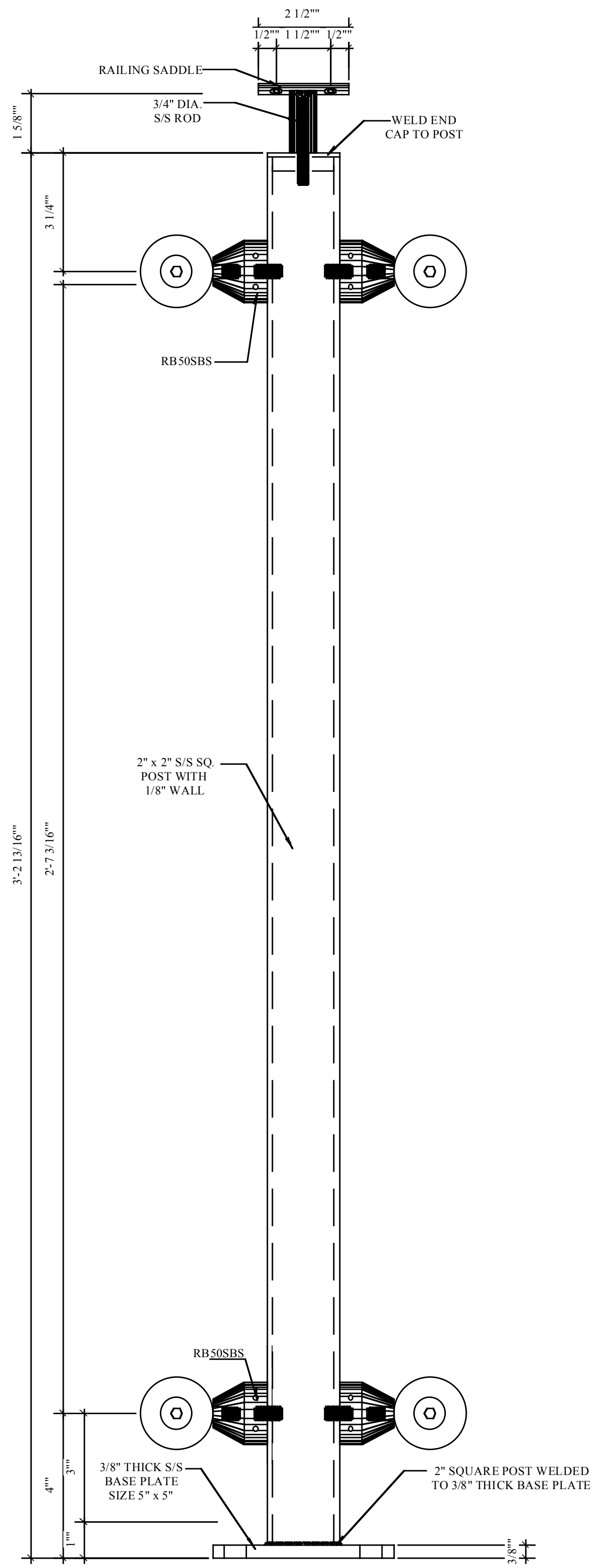
P1 POST

2”x2”x 3/16” 304 Stainless steel tube

Stainless steel post strength calculations are done according to AISC 370 Appendix 2 continuous strength method. For tubes, the cold working increase is also calculated.

Strength calculations are shown on the following page, allowable moment based on post strength = 26,000”#. The strength is limited by the full perimeter fillet weld.

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



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B (in)	H (in)	A_g (in^2)	I (in^4)	Z (in^3)	E (ksi)	F_y (ksi)	F_u (ksi)
2	2	1.27	0.672	0.843	28000	30	75
Coldworking stress increease per AISC 370 B4-1a							
r (in)	t (in)	ε_{corner}	ε_{wall}	ε_y	ε_u	n	A_{corner} (in^2)
0.1875	0.1875	0.167	0.0865	0.00307	0.6	0.17371	0.894
$F_{y,corner}$ (ksi)	$F_{y,wall}$ (ksi)	$F_{y,avg}$ (ksi) (See AISC 370 B4-1a)					
51.194	45.818	49.602	Use in place of F_y in appendix 2 calculations.				

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.68$	$\varepsilon_{csm}/\varepsilon_y = 0.25/\lambda_l^{3.6} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y)$)	
Case b)	$\lambda_l > 0.68$	$\varepsilon_{csm}/\varepsilon_y = (1-0.222/\lambda_l^{1.05})1/\lambda_l^{1.05}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
49.602	15	28000	0.00177
F_u (ksi)	E_{sh} (ksi)	υ	α
75	484.59	0.3	2
Section Properties:			
t_p (in)	b_p (in)	S (in^3)	Z (in^3)
0.1875	2	0.672	0.843
Find elastic buckling stress per AISC 370 C-A-1-2.			
Isolated flange	k	$F_{el,f}^{SS} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	4	889.69	1

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Isolate web	k	$F_{el,w}^{SS} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	23.9	5315.89	1
$F_{el,p}^{SS} = \min(\beta_f F_{el}^{SS}{}_f\ ,\ \beta_w F_{el}^{SS}{}_{,w})$			
889.69			
Isolated flange	k	$F_{el,f}^{F} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	6.97	1550.28	1
Isolate web	k	$F_{el,w}^{F} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	39.6	8807.91	1
$F_{el,p}^{F} = \min(\beta_f F_{el}^{F}{}_f\ ,\ \beta_w F_{el}^{F}{}_{,w})$			
1550.28			
$ø = \beta_f F_{el}^{SS}{}_f / (\beta_w F_{el}^{SS}{}_{,w})$	If $ø < 1$	$a_f = 0.24-[0.1(t_f/t_w)^2(H/B-1)]^{1/0.6} \leq 0.24$	**af**
0.17	If $ø \geq 1$	$a_w = 0.63-0.1H/B \leq 0.53$	0.24
	If $ø < 1$	$\zeta = t_w/t_f*(0.24-a_f*ø)^{0.6}$	ζ
	If $ø \geq 1$	$\zeta = t_f/t_w*(0.53-a_w/ø)$	0.38
$F_{el} = F_{el,p}^{SS}+\zeta(F_{el,p}^{F}-F_{el,p}^{SS})$ ksi	$\lambda_l = (F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.68$, $\varepsilon_{csm}/\varepsilon_y = 0.25/(\lambda_l)^{3.6} \leq (\Lambda$ and $0.1(1-F_y/F_u)/\varepsilon_y)$	$\varepsilon_{csm}/\varepsilon_y$
1141.07	0.2085	For $0.68 < \lambda_l < 1.00$, $\varepsilon_{csm}/\varepsilon_y = (1-0.222/(\lambda_l)^{1.05})*(1/(\lambda_l)^{1.05})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.02657	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
33.332	41.814	49.853	**29852**

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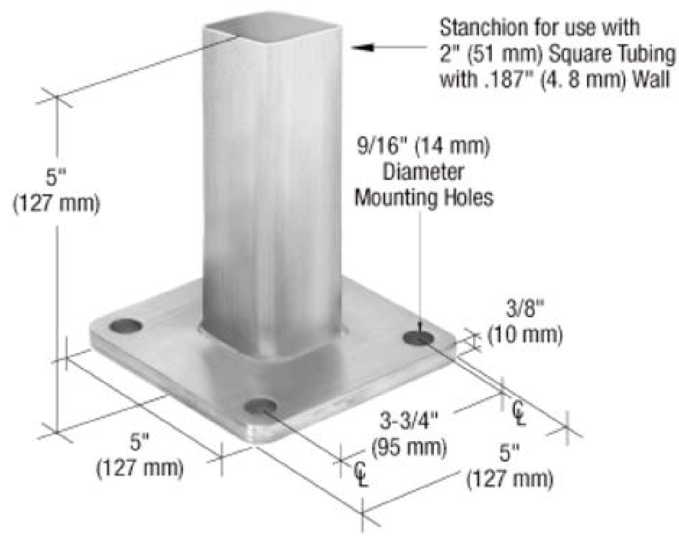
Rectangular weld group				
B (in) (Weld group width)	D (in) (Weld group depth)	I_x (in^4/in) = $(2D^3/12+2B(D/2)^2$	S_x(in^3/in) = $I_x/(D/2)$	L (in) = 2D+2B
2	2	5.3333	5.3333	8.0000
Weld size (in)	Weld Filler Strength (ksi)	V (lbs)	M (in-lbs)	T (lbs)
0.1875	82	619	26000	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
77	4875	4876		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.555	73.795			
$R_n/\Omega = F_{nw}*S_w*\sin(45^\circ)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
4892	Pass			

The post strength is limited by the weld to the post. The shear is assumed to be M/42”. The maximum moment carried by the 3/16” full perimeter fillet weld is 26000”#.

P1 POST continued:
Stanchion mounted posts
Surface Mount Stanchions
Cast stainless steel - stanchion cast integrally with the base plate
M_a = 20,300"#



B (in)	H (in)	A_g (in^2)	I (in^4)	Z (in^3)	E (ksi)	F_y (ksi)	F_u (ksi)
1.625	1.625	1.05	0.361	0.563	28000	30	75
Coldworking stress increease per AISC 370 B4-1a							
r (in)	t (in)	ε_{corner}	ε_{wall}	ε_y	ε_u	n	A_{corner} (in^2)
0.1875	0.1875	0.1667	0.1077	0.00307	0.6	0.17371	0.8938
$F_{y,corner}$ (ksi)	$F_{y,wall}$ (ksi)	$F_{y,avg}$ (ksi) (See AISC 370 B4-1a)					
51.194	47.539	50.651	Use in place of F_y in appendix 2 calculations.				

The stanchion is cast with the baseplate so there is no weld to check. M_a = 20,300"#

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.68$	$\varepsilon_{csm}/\varepsilon_y = 0.25/\lambda_l^{3.6} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y$))	

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Case b)	$\lambda_l > 0.68$	$\varepsilon_{csm}/\varepsilon_y = (1\text{-}0.222/\lambda_l^{1.05})1/\lambda_l^{1.05}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
50.651	15	28000	0.00181
F_u (ksi)	E_{sh} (ksi)	υ	α
75	485.66	0.3	2
Section Properties:			
t_p (in)	b_p (in)	S (in^3)	Z (in^3)
0.1875	1.625	0.444	0.563
Find elastic buckling stress per AISC 370 C-A-1-2.			
Isolated flange	k	$F_{el,f}{}^{SS} = k\pi^2E/(12(1\text{-}\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	4	1347.69	1
Isolate web	k	$F_{el,w}{}^{SS} = k\pi^2E/(12(1\text{-}\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	23.9	8052.47	1
$F_{el,p}{}^{SS} = \min(\beta_f F_{el}{}^{SS}{}_f\ ,\ \beta_w F_{el}{}^{SS}{}_{,w})$			
1347.69			
Isolated flange	k	$F_{el,f}{}^{F} = k\pi^2E/(12(1\text{-}\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	6.97	2348.35	1
Isolate web	k	$F_{el,w}{}^{F} = k\pi^2E/(12(1\text{-}\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	39.6	13342.16	1
$F_{el,p}{}^{F} = \min(\beta_f F_{el}{}^{F}{}_f\ ,\ \beta_w F_{el}{}^{F}{}_{,w})$			
2348.35			
$\o = \beta_f F_{el}{}^{SS}{}_f/\ (\beta_w F_{el}{}^{SS}{}_{,w})$	If $\o < 1$	$a_f = 0.24\text{-}[0.1(t_f/t_w)^2(H/B\text{-}1)]^{1/0.6} \leq 0.24$	**af**
0.17	If $\o \geq 1$	$a_w = 0.63\text{-}0.1H/B \leq 0.53$	0.24
	If $\o < 1$	$\zeta = t_w/t_f{}^*(0.24\text{-}a_f{}^*\o)^{0.6}$	ζ
	If $\o \geq 1$	$\zeta = t_f/t_w{}^*(0.53\text{-}a_w/\o)$	0.38

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$F_{el} = F_{el,p}^{SS}+\zeta(F_{el,p}^{F}-F_{el,p}^{SS})$ ksi	$\lambda_l = (F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.68$, $\varepsilon_{csm}/\varepsilon_y = 0.25/(\lambda_l)^{3.6} \leq (\Lambda$ and $0.1(1-F_y/F_u)/\varepsilon_y)$	$\varepsilon_{csm}/\varepsilon_y$
1728.48	0.1712	For $0.68 < \lambda_l < 1.00$, $\varepsilon_{csm}/\varepsilon_y = (1-0.222/(\lambda_l)^{1.05})*(1/(\lambda_l)^{1.05})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.02713	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
22.489	28.516	33.951	**20330**

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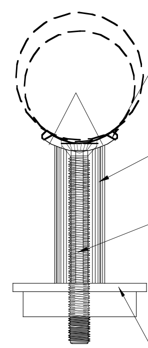
P1 POST continued:

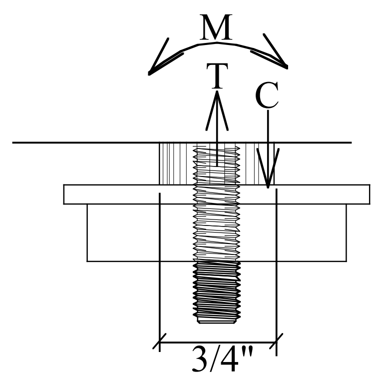
Top Rail Connection Bracket - Okay for all rails with wall thickness of 0.05" or greater.
Top rail to saddle: No significant bending
Check for shear transfer:
Two #8 self drilling screws into 0.05" wall tube.
V_a = 45ksi*0.164"*0.05"/2 = 185#/ screw
V_t = 185#*2 screws = 370#
Maximum applied load 295#



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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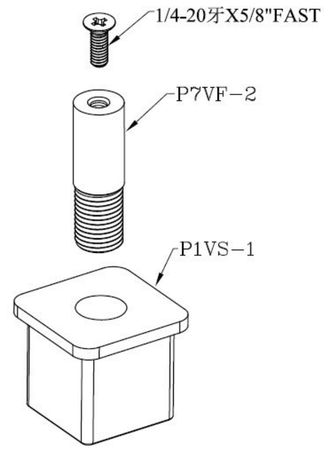
Adjustable Top Rail Connection Bracket



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
M_a =1,272"# (See plastic moment strength calculation on following page)
Allowable load at full height = 1,272"#/(2.125"+1.5"/2) = 442#> 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

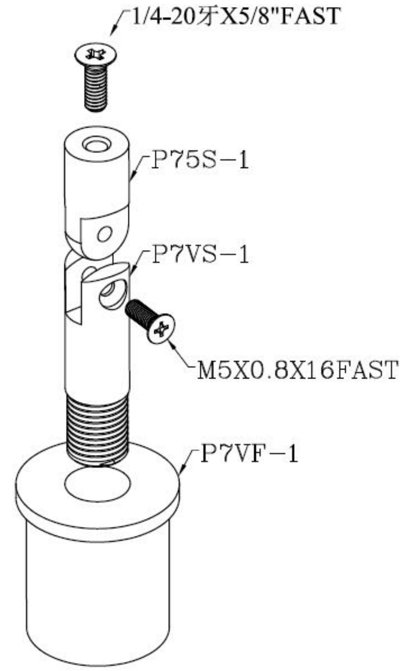
Check embedment of cap into post:
e = 1.8125"
Assume linear stress distribution, use 1# unit load.
M = 1#*2.125" = 2.125"#
P = 2.125"#/(2/3*1.8125") = 1.759# (Magnitude of couple moment loads)
w = 1.759#/(1.8125"/2*1/2) = 3.881pli (peak distributed load)
Two failure modes, bending of face of post or 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”

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.30$	$\varepsilon_{csm}/\varepsilon_y = 0.00444/\lambda_l^{4.5} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y$))	
Case b)	$\lambda_l > 0.3$	$\varepsilon_{csm}/\varepsilon_y = (1-0.224/\lambda_l^{0.342})1/\lambda_l^{0.342}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
30	15	28000	0.00107
F_u (ksi)	E_{sh} (ksi)	υ	α
75	474.04063	0.3	2
Section Properties:			
t (in)	D (in)	S (in^3)	Z (in^3)
0.2116	0.7382	0.038	0.062
Find elastic buckling stress per AISC 370 A 2-2 User note			
$F_{el}=E2t/((3*(1-\upsilon^2))^{1/2}D)$	$\lambda_l = (F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.30$, $\varepsilon_{csm}/\varepsilon_y = 0.00444/(\lambda_l)^{4.5} \leq$ Λ and $0.10(1-F_y/F_u)/\varepsilon_y$	$\varepsilon_{csm}/\varepsilon_y$
9715.9	0.05557	For $0.3 < \lambda_l$, $\varepsilon_{csm}/\varepsilon_y = (1-0.224/(\lambda_l)^{0.342})*(1/(\lambda_l)^{0.342})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.01607	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
1.145	1.855	2.1234	**1272**

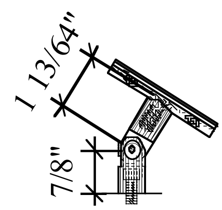
Swivel connection



Moment at swivel connection = 250#*1” = 250”#
Friction area = $14\text{mm}^2\pi/4=153.9\text{mm}^2=0.2386\text{in}^2$
Required friction stress:
From direct load = $200\#/0.2386\text{in}^2 = 838.2\text{psi}$
From rotation = $250''\#/(0.2386\text{in}^2*0.551''*2/3) = 2{,}852\text{psi}$
Total = 838.2+2852psi = 3,690psi
Assume $\mu = 0.5$
T= $3{,}690\text{psi}/(0.5*2/0.2386\text{in}^2) = 880\#$
Screw strength = $0.02444\text{in}^2*75\text{ksi}/2 = 916\# > 880\#$ 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 = $0.0145\ \text{in}^2$
$F_v = 42.8$ ksi
$øZ_n = 0.65*0.0145*42.8 = 403\#$
$øM_n = 2*403\#*0.5'' = 403\#''$
$P_s = 403\#''/(1.6*1.203'')= 209\#$
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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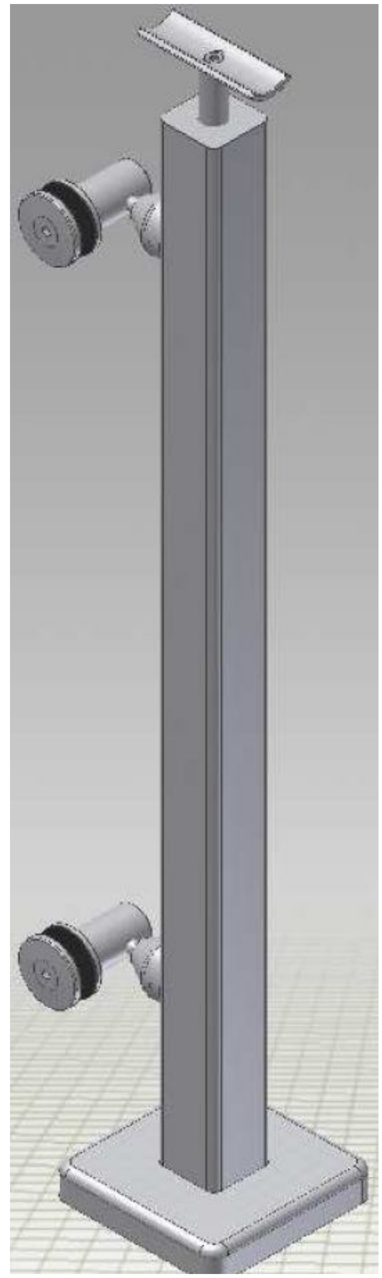
P1 POST continued:



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

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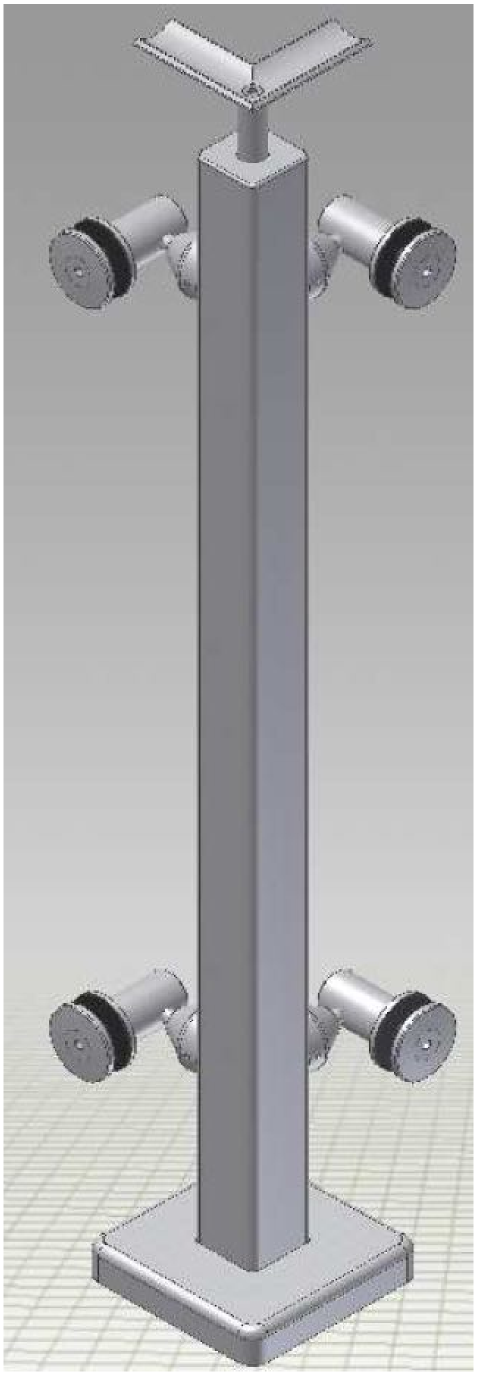

Post may be used for an inside corner condition by reversing the stand off direction.

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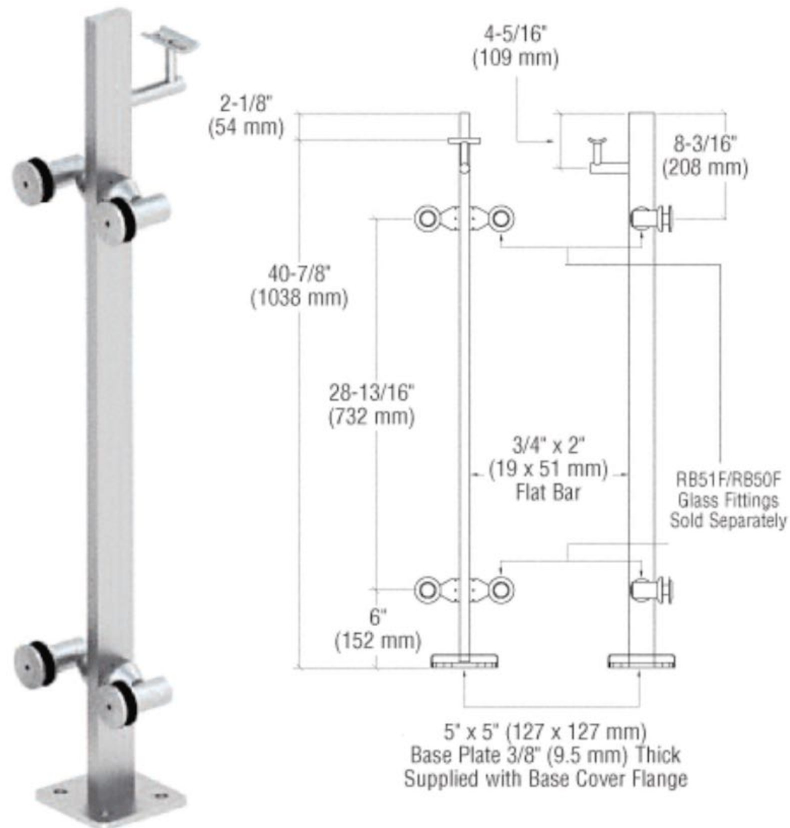
POST P3

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



M_a = 13,500”#

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Check 2-3/8”x3/8” flat bar strength. Strength is calculated per AISC 370 F9.					
d (in)	t (in)	S (in^3)	Z (in^3)		
2	0.75	0.5	0.75		
L (in)	Ld/t^2	F_y (ksi)	E (ksi)		
42	149.3333	30	28000		
$0.306E/F_y$	$2.0E/F_y$	M_a (in-lbs) See F9-1,2 or 3 as appropriate			
295.80	1866.67	13473			

Rectangular weld group				
B (in) (Weld group width)	D (in) (Weld group depth)	I_x (in^4/in) = $(2D^3/12+2B(D/2)^2$	$S_x(in^3/in) = I_x/(D/2)$	L (in) = 2D+2B
0.75	2	2.8333	2.8333	5.5000
Weld size (in)	Weld Filler Strength (ksi)	V (lbs)	M (in-lbs)	T (lbs)
0.25	70	321	13473	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
58	4755	4756		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.559	62.998			
$R_n/\Omega = F_{nw}*S_w*\sin(45°)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
5568	Pass			

1/4” full perimeter fillet weld develops post strength. M_a = 13,500”#.

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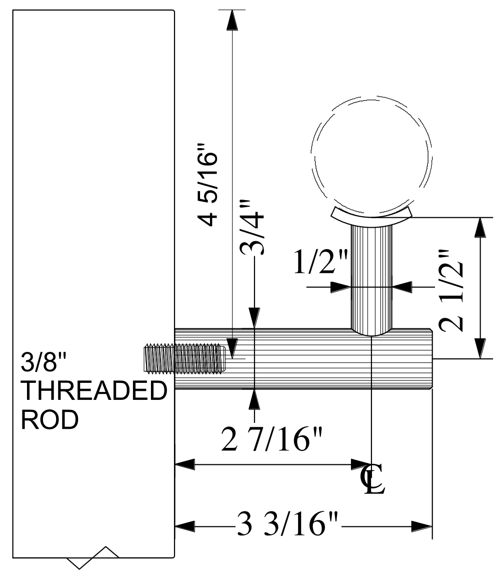
POST P3 continued:
Base Plate:
5” square base plate same as previously checked.

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:
M_a= 1437”# (See bar strength calculations on the following pages)
$V_S = [M_n/\Omega]/e = 1437/2.4375 = 590\# > 250\#$ OK for 5’ O.C. spacing

Bending in 1/2” vertical bar, hardened SS:
$Z = 0.5^3/6 = 0.02083\ in^3$
$øM_n = 0.02083*45ksi = 937”\#$
Vertical service load:
$V_S = [(M_n)/\Omega]/e = 426/(2.5”-0.375”) = 200\#$ OK for 4’ O.C. spacing

For 3/8” 316 SS rod ASTM F593-98 CW or stronger;
F_{ut} = 90ksi
$T_n = A*90\ ksi = 0.0775in^2*90ksi = 6,975\#$
$T_s = T_n/\Omega = 6,975\#/2 = 3,487\#$
$V_s = 3,487\#*0.375”/2.5” = 523\#$ OK for 5’ O.C. spacing

Allowable loading on the bracket is controlled by the strength of the vertical 1/2” round bar.
Allowable bracket spacing is 4’ O.C.

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Plastic strength of horizontal 3/4" round bar:

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method - Solid Round			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.30$	$\varepsilon_{csm}/\varepsilon_y = 0.00444/\lambda_l^{4.5} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y$))	
Case b)	$\lambda_l > 0.3$	$\varepsilon_{csm}/\varepsilon_y = (1-0.224/\lambda_l^{0.342})1/\lambda_l^{0.342}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
30	15	28000	0.00107
F_u (ksi)	E_{sh} (ksi)	υ	α
75	474.041	0.3	2
Section Properties:			
	D (in)	S (in^3)	Z (in^3)
	0.75	0.041	0.070
Solid round so no local buckling $\varepsilon_{csm}/\varepsilon_y = \Lambda$			$\varepsilon_{csm}/\varepsilon_y$
			15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.01607	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
1.243	2.109	2.400	**1437**

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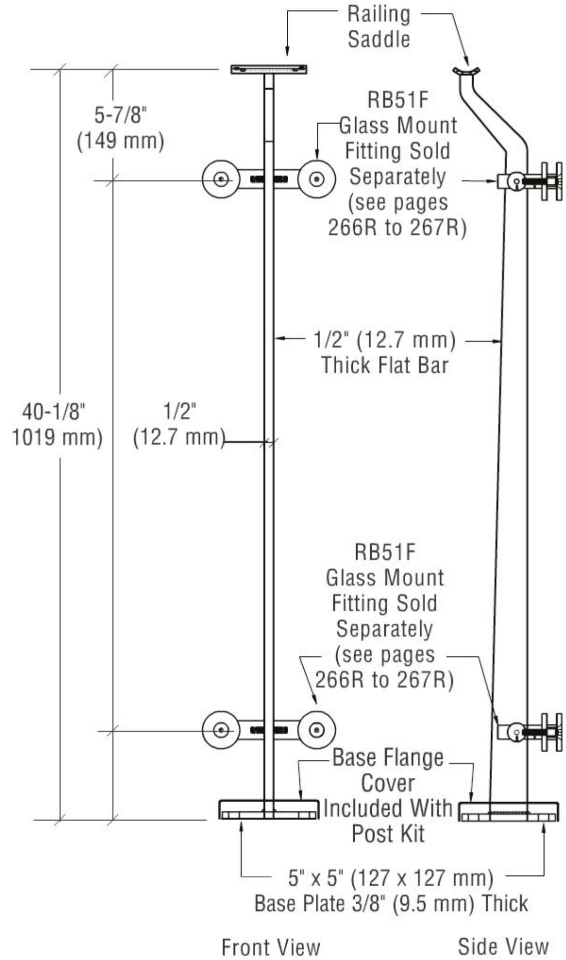
POST P5

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

$M_a = 8{,}900"\#$

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Check 2-3/8”x3/8” flat bar strength. Strength is calculated per AISC 370 F9.					
d (in)	t (in)	S (in^3)	Z (in^3)		
2	0.5	0.3333	0.5000		
L (in)	Ld/t^2	F_y (ksi)	E (ksi)		
40.1	320.8000	30	28000		
$0.306E/F_y$	$2.0E/F_y$	M_a (in-lbs) See F9-1,2 or 3 as appropriate each bar	Allowable spacing = $12M_a$/(50plf*42”) (in)		
295.80	1866.67	8900	50.86		

Rectangular weld group				
B (in) (Weld group width)	D (in) (Weld group depth)	I_x (in^4/in) = $(2D^3/12+2B(D/2)^2$	S_x(in^3/in) = $I_x/(D/2)$	L (in) = 2D+2B
0.5	2	2.3333	2.3333	5.0000
Weld size (in)	Weld Filler Strength (ksi)	V (lbs) = M/42”	M (in-lbs)	T (lbs)
0.25	70	212	8900	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
42	3814	3814		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.560	62.998			
$R_n/\Omega = F_{nw}*S_w*\sin(45^\circ)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
5568	Pass			

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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

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

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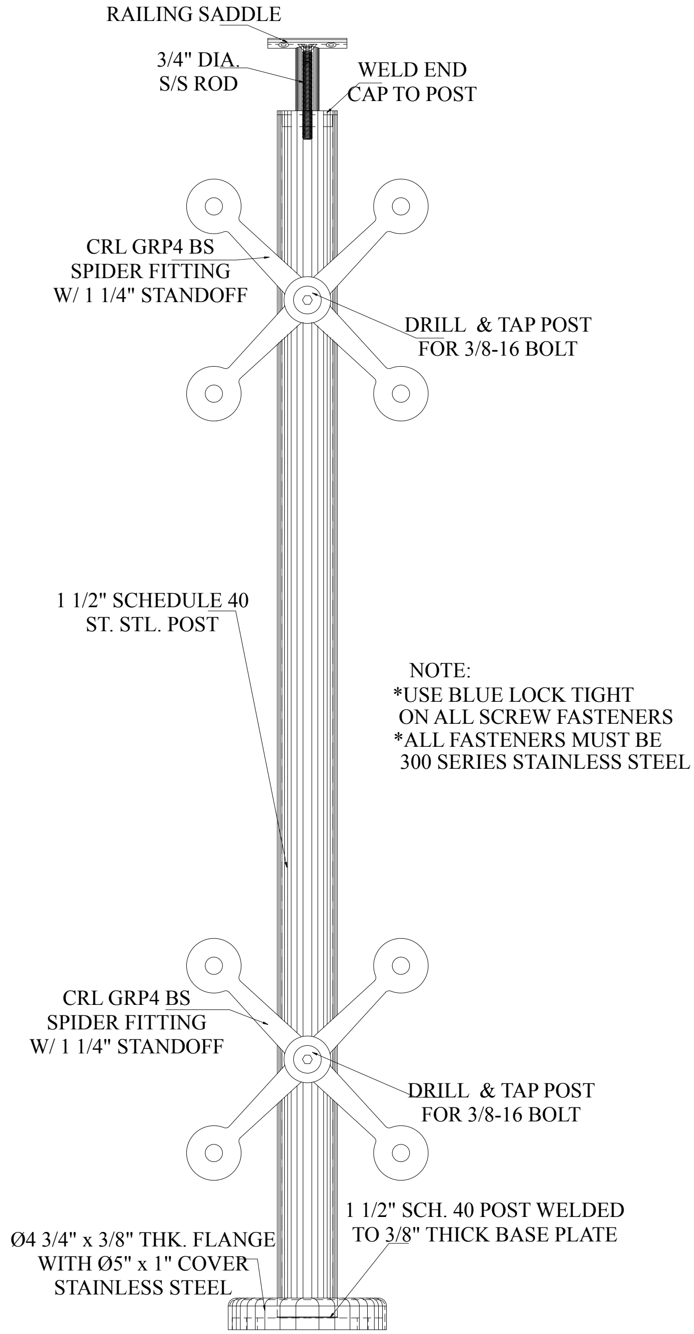
P6 POST

1-1/2” schedule 40 1/16 hard
304 Stainless steel pipe
Post Strength
$Z_{yy} = 0.421$ in^3

M_a = 11,900”# and is controlled by the plastic bending strength of the post.



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I (in^4)	Z (in^3)	F_y (ksi)	F_u (ksi)			
0.293	0.421	30	75			
Coldworking stress increease per AISC 370 B4-1b						
D (in)	r (in)	t (in)	ε_{rnd}	ε_y	ε_u	n
1.9	0.95	0.135	0.0382	0.0031	0.6	0.1737
$F_{y,avg}$ (ksi)						
40.0516	Use in place of F_y in appendix 2 calculations.					

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.30$	$\varepsilon_{csm}/\varepsilon_y = 0.00444/\lambda_l^{4.5} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y$))	
Case b)	$\lambda_l > 0.3$	$\varepsilon_{csm}/\varepsilon_y = (1-0.224/\lambda_l^{0.342})1/\lambda_l^{0.342}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
40.0516	15	28000	0.0014
F_u (ksi)	E_{sh} (ksi)	υ	α
75	477.9192	0.3	2
Section Properties:			
t (in)	D (in)	S (in^3)	Z (in^3)
0.135	1.9	0.309	0.421

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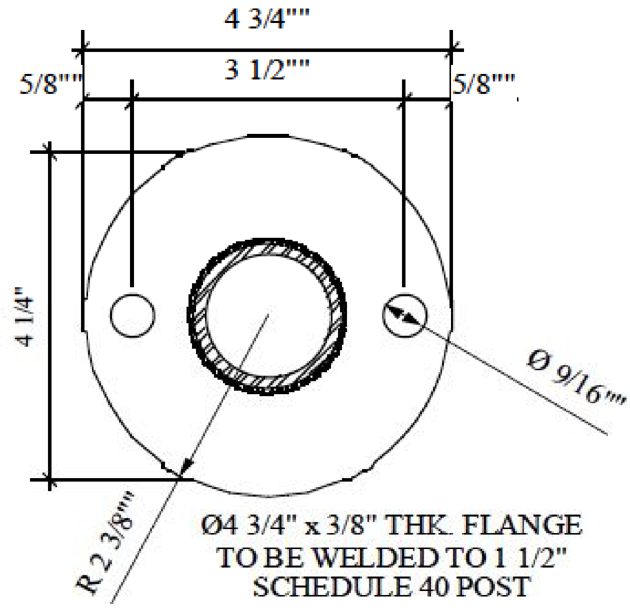
Find elastic buckling stress per AISC 370 A 2-2 User note			
F_{el}=E2t/((3*(1-υ^2))$^{1/2}$D)	λ_l = $(F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.30$, $\varepsilon_{csm}/\varepsilon_y = 0.00444/(\lambda_l)^{4.5} \leq \Lambda$ and $0.10(1-F_y/F_u)/\varepsilon_y$	$\varepsilon_{csm}/\varepsilon_y$
2408.2	0.129	For $0.3 < \lambda_l$, $\varepsilon_{csm}/\varepsilon_y = (1-0.224/(\lambda_l)^{0.342})*(1/(\lambda_l)^{0.342})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.0215	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**M_a = M_n/1.67*1000 (in-lbs)**
12.361	16.877	19.811	**11863**

Circular weld group				
D (in) (Weld group diameter)	R (in) (Weld group radius)	I_x (in^4/in) = πR^3	S_x(in^3/in) = $I_x/(R)$	L (in) = πD
1.9	0.95	2.6935	2.8353	5.9690
Weld size (in)	Weld Filler Strength (ksi)	V (lbs) = M/42”	M (in-lbs)	T (lbs)
0.25	70	282	11863	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
47	4184	4184		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.559	62.998			
R_n/Ω = $F_{nw}*S_w*\sin(45^\circ)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
5568	Pass	1/4” Weld OK		

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

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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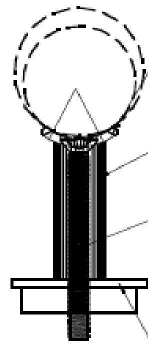
P6 POST continued:
Top Rail Connection Bracket



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#"

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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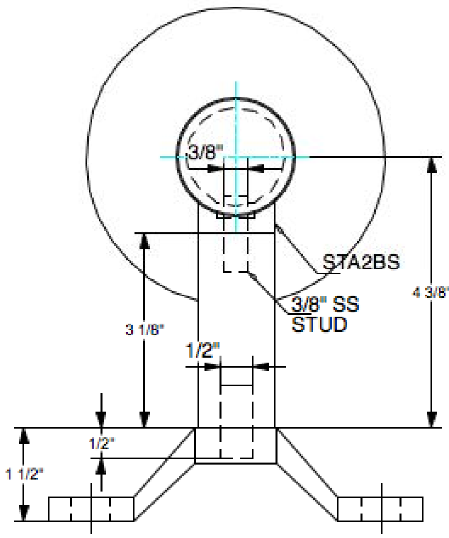
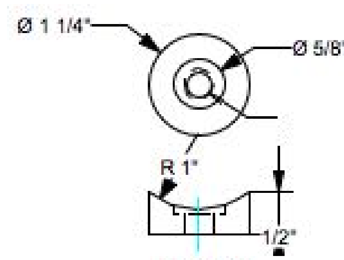
P6 POST continued:



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} = 0.58t$F_{tu}A_{sn}$ = 0.58*75ksi*0.375”*0.828” =
P_{not} = 13.5k

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



STA2BS

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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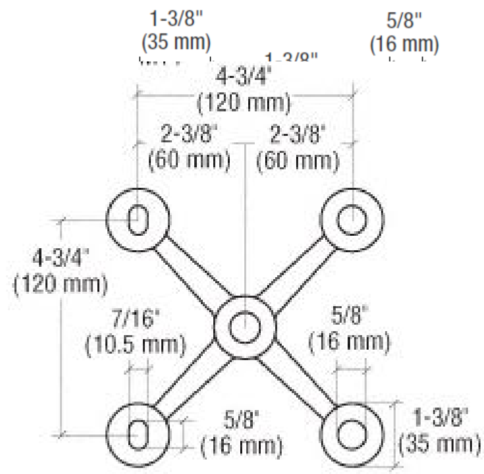
P6 POST continued:
GRP SPIDER FITTINGS

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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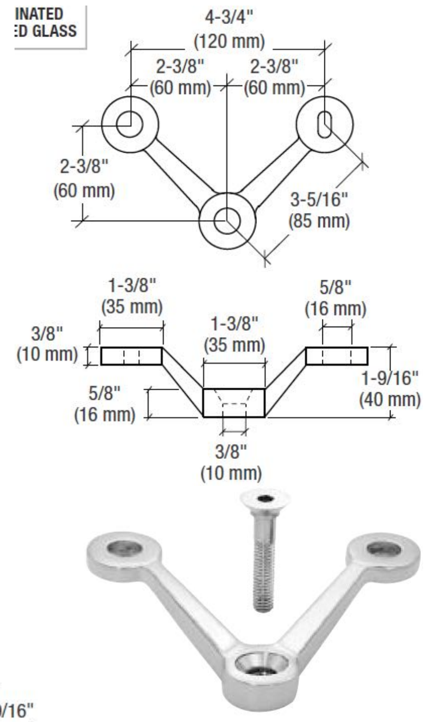
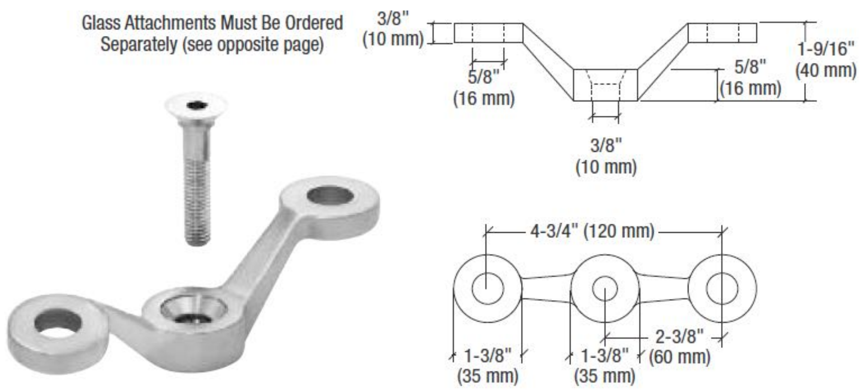
P6 POST continued: GRP (continued)

Check bending strength of stud for pure bending:
Applicable to loads in glass plane only:
$Z = 0.375"^3/6 = 0.0088in^3$
$øM_n = 0.9*85ksi*0.0088in^3 = 673\#"$
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

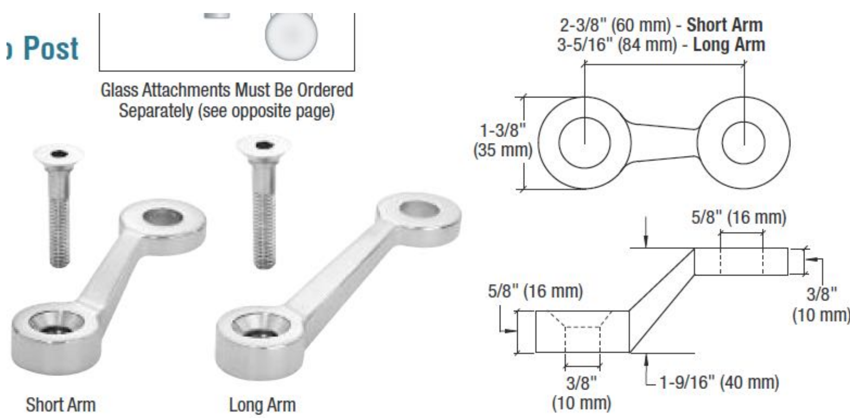
Out of plane loads (typically live and wind)
For dead load $\leq 0.2*558\# = 111\#$
$R_a = (2,546"\#-111*4.5625)/4.886" = 417\#$ single arm
$R_a = (2,546"\#-111*4.5625)/2.375" = 858\#$ total on fitting
These strength parameters are applicable to all configurations:



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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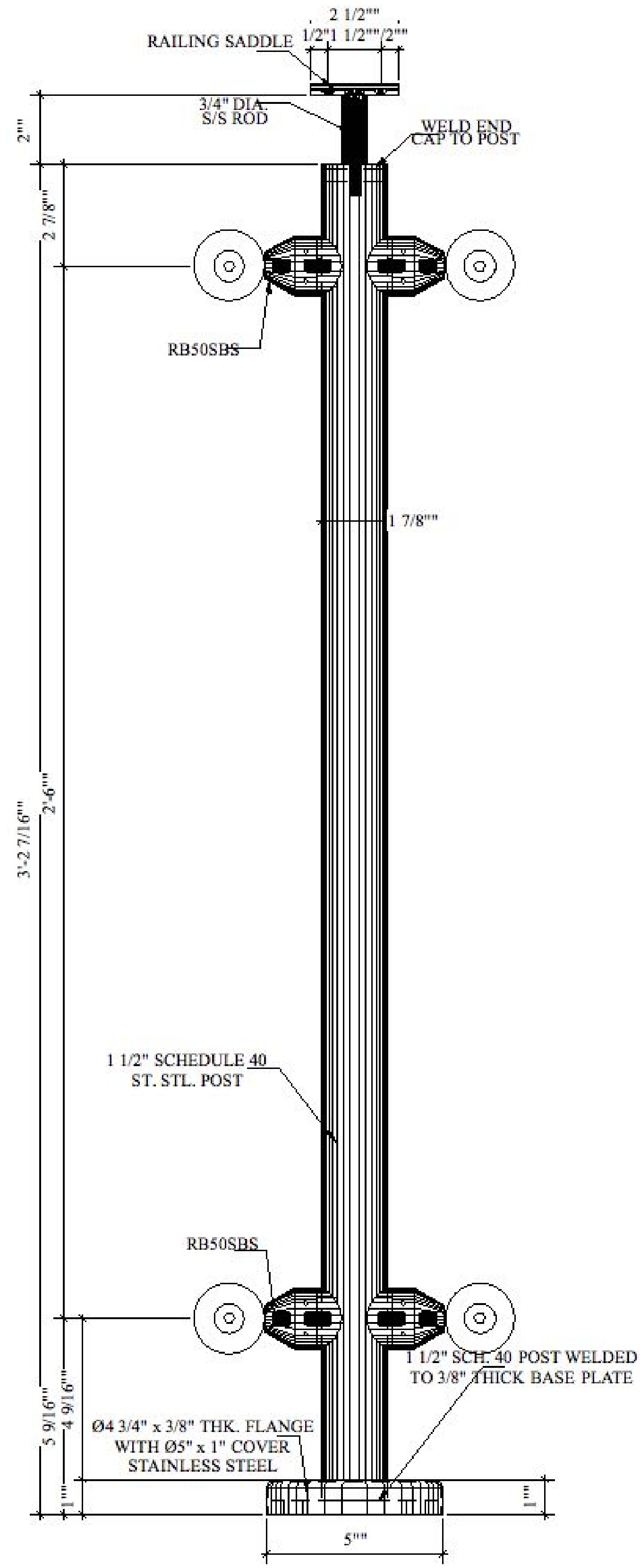
P7 POST

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

M_a = 11,900”# and is controlled by the plastic bending strength of the post.

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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P7 POST continued:

I (in^4)	Z (in^3)	F_y (ksi)	F_u (ksi)			
0.293	0.421	30	75			
Coldworking stress increease per AISC 370 B4-1b						
D (in)	r (in)	t (in)	ε_{rnd}	ε_y	ε_u	n
1.9	0.95	0.135	0.0382	0.00307	0.6	0.17371
$F_{y,avg}$ (ksi)						
40.0516175366	Use in place of F_y in appendix 2 calculations.					

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.30$	$\varepsilon_{csm}/\varepsilon_y = 0.00444/\lambda_l^{4.5} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y$))	
Case b)	$\lambda_l > 0.3$	$\varepsilon_{csm}/\varepsilon_y = (1-0.224/\lambda_l^{0.342})1/\lambda_l^{0.342}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
40.052	15	28000	0.001
F_u (ksi)	E_{sh} (ksi)	υ	α
75	477.919	0.3	2
Section Properties:			
t (in)	D (in)	S (in^3)	Z (in^3)
0.135	1.9	0.309	0.421
Find elastic buckling stress per AISC 370 A 2-2 User note			
$F_{el}=E2t/((3*(1-\upsilon^2))^{1/2}D)$	$\lambda_l = (F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.30$, $\varepsilon_{csm}/\varepsilon_y = 0.00444/(\lambda_l)^{4.5} \leq \Lambda$ and $0.10(1-F_y/F_u)/\varepsilon_y$	$\varepsilon_{csm}/\varepsilon_y$

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2408.2	0.1290	For $0.3 < \lambda_l$, $\varepsilon_{csm}/\varepsilon_y = (1-0.224/(\lambda_l)^{0.342})*(1/(\lambda_l)^{0.342})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.021	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
12.361	16.877	19.811	**11863**

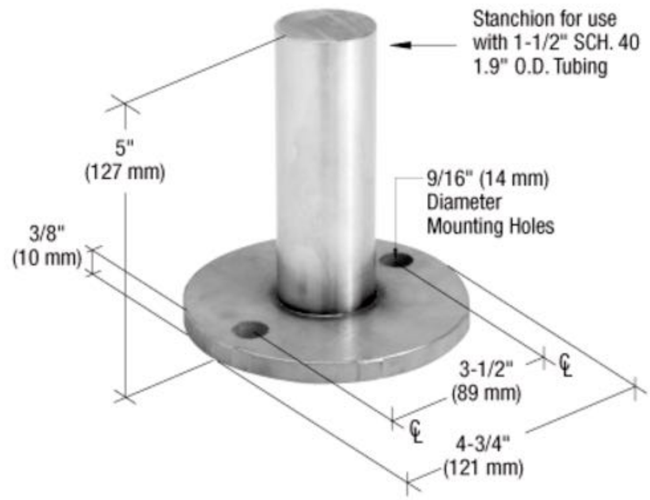
Circular weld group				
D (in) (Weld group diameter)	R (in) (Weld group radius)	I_x (in^4/in) = πR^3	S_x(in^3/in) = $I_x/(R)$	L (in) = πD
1.9	0.95	2.6935	2.8353	5.9690
Weld size (in)	Weld Filler Strength (ksi)	V (lbs) = M/42”	M (in-lbs)	T (lbs)
0.25	70	282	11863	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
47	4184	4184		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.559	62.998			
$R_n/\Omega = F_{nw}*S_w*\sin(45°)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
5568	Pass	1/4” Weld OK		

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.
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.
M_a = 14,200"#>11,800"# (does not limit post spacing)



Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method - Solid Round			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.30$	$\varepsilon_{csm}/\varepsilon_y = 0.00444/\lambda_l^{4.5} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y)$)	
Case b)	$\lambda_l > 0.3$	$\varepsilon_{csm}/\varepsilon_y = (1-0.224/\lambda_l^{0.342})1/\lambda_l^{0.342}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
30	15	28000	0.00107
F_u (ksi)	E_{sh} (ksi)	υ	α
75	474.04063	0.3	2
Section Properties:			
	D (in)	S (in^3)	Z (in^3)
	1.61	0.410	0.696
Solid round so no local buckling $\varepsilon_{csm}/\varepsilon_y = \Lambda$			$\varepsilon_{csm}/\varepsilon_y$
			15

ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.01607	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
12.291	20.866	23.742	**14217**

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

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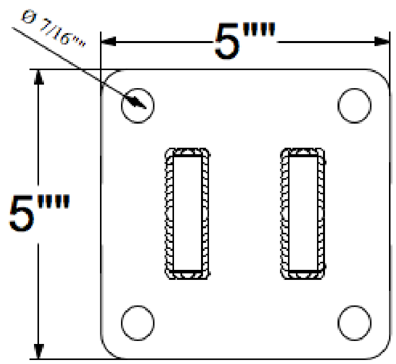
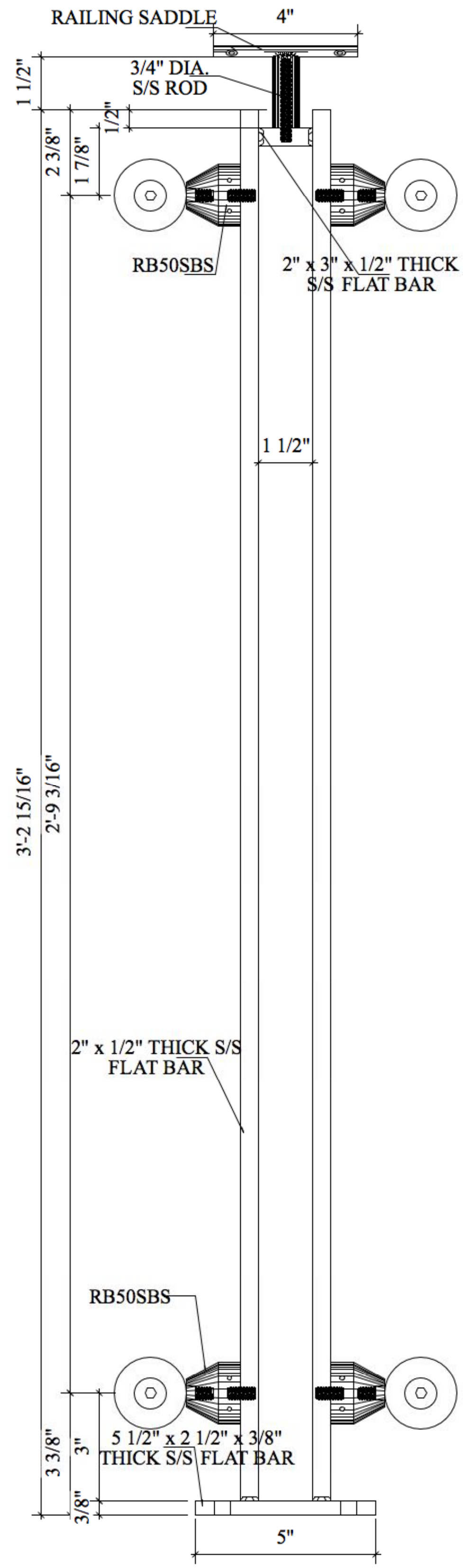
P8 POST
2"x1/2" Bars A304 Stainless Steel

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





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Check 2-3/8”x3/8” flat bar strength. Strength is calculated per AISC 370 F9.					
d (in)	t (in)	S (in^3)	Z (in^3)		
2	0.5	0.3333	0.5000		
L (in)	Ld/t^2	F_y (ksi)	E (ksi)		
36	288.0000	30	28000		
$0.306E/F_y$	$2.0E/F_y$	M_a (in-lbs) See F9-1,2 or 3 as appropriate each bar	For two bars, M_a (in-lbs)		
295.80	1866.67	8982	17964		

Rectangular weld group				
B (in) (Weld group width)	D (in) (Weld group depth)	I_x (in^4/in) = $(2D^3/12+2B(D/2)^2$	S_x(in^3/in) = $I_x/(D/2)$	L (in) = 2D+2B
0.5	2	2.3333	2.3333	5.0000
Weld size (in)	Weld Filler Strength (ksi)	V (lbs) = M/42”	M (in-lbs)	T (lbs)
0.25	70	214	8982	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
43	3849	3850		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.560	62.998			
$R_n/\Omega = F_{nw}*S_w*\sin(45^\circ)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
5568	Pass			

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P8 POST continued:

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

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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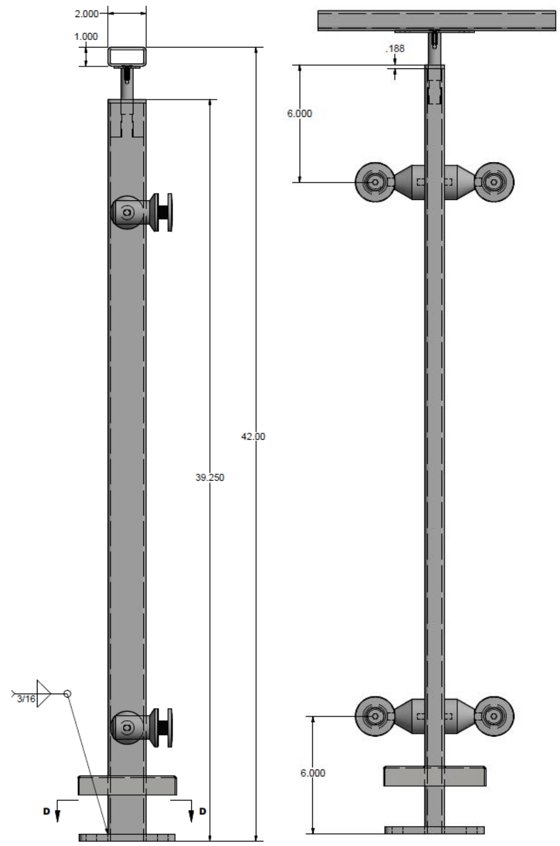
POST P9

Stainless steel (304) tube post mounted on stainless steel bar stanchion attached to stainless steel base plate.
M_a = 12,500"#

Okay for 5' o.c. post spacing for 50 plf top rail loading and the standard stanchion



Check 2-3/8"x3/8" flat bar strength. Strength is calculated per AISC 370 F9.					
d (in)	t (in)	S (in^3)	Z (in^3)		
1.75	0.75	0.3828	0.5742		
L (in)	Ld/t^2	F_y (ksi)	E (ksi)		
3.5	10.8889	30	28000		
0.306E/F_y	2.0E/F_y	M_a (in-lbs) See F9-1,2 or 3 as appropriate each bar			
295.80	1866.67	10315			

For standard installation the post will sleeve the full height of the stanchion. In which case the full plastic strength may be used:

F_u (ksi)	$M_a = ZF_u/2*1000$ (in-lbs)		
75	21533		

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B (in)	H (in)	A_g (in^2)	I (in^4)	Z (in^3)	E (ksi)	F_y (ksi)	F_u (ksi)
1	2	0.608	0.350	0.366	28000	30	75
Coldworking stress increease per AISC 370 B4-1a							
r (in)	t (in)	ε_{corner}	ε_{wall}	ε_y	ε_u	n	A_{corner} (in^2)
0.24	0.12	0.1	0.0717	0.00307	0.6	0.17371	0.4566
$F_{y,corner}$ (ksi)	$F_{y,wall}$ (ksi)	$F_{y,avg}$ (ksi) (See AISC 370 B4-1a)					
46.945	44.397	46.310					

Design of Stainless Steel HSS Using AISC Appendix 2 Continuous Strength Method			
AISC 370 A.2.6			
Case 1)	$\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n = \varepsilon_{csm}/\varepsilon_y M_y$	
Case 2)	$\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n = M_p(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
Use AISC 370 A.2.3.1 to determine failure strain.			
Case a)	$\lambda_l \leq 0.68$	$\varepsilon_{csm}/\varepsilon_y = 0.25/\lambda_l^{3.6} \leq$ minimum(Λ, $0.10(1-F_y/F_u)/\varepsilon_y)$)	
Case b)	$\lambda_l > 0.68$	$\varepsilon_{csm}/\varepsilon_y = (1-0.222/\lambda_l^{1.05})1/\lambda_l^{1.05}$	
Material Properties:			
F_y (ksi)	Λ	E (ksi)	$\varepsilon_y = F_y/E$
46.310	15	28000	0.00165
F_u (ksi)	E_{sh} (ksi)	υ	α
75	481.77	0.3	2
Section Properties:			
t_p (in)	b_p (in)	S (in^3)	Z (in^3)
0.12	2	0.350	0.366
Find elastic buckling stress per AISC 370 C-A-1-2.			
Isolated flange	k	$F_{el,f}^{SS} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	4	364.42	1

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Isolate web	k	$F_{el,w}^{SS} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	23.9	2177.39	1
$F_{el,p}^{SS} = \min(\beta_f F_{el}^{SS}{}_f\ , \beta_w F_{el}^{SS}{}_{,w})$			
364.42			
Isolated flange	k	$F_{el,f}^{F} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	6.97	635.00	1
Isolate web	k	$F_{el,w}^{F} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	39.6	3607.72	1
$F_{el,p}^{F} = \min(\beta_f F_{el}^{F}{}_f\ , \beta_w F_{el}^{F}{}_{,w})$			
635.00			
$\text{ø} = \beta_f F_{el}^{SS}{}_f / (\beta_w F_{el}^{SS}{}_{,w})$	If ø < 1	$a_f = 0.24-[0.1(t_f/t_w)^2(H/B-1)]^{1/0.6} \leq 0.24$	**af**
0.17	If ø ≥ 1	$a_w=0.63-0.1H/B \leq 0.53$	0.24
	If ø < 1	$\zeta = t_w/t_f*(0.24-a_f*\text{ø})^{0.6}$	ζ
	If ø ≥ 1	$\zeta = t_f/t_w*(0.53-a_w/\text{ø})$	0.38
$F_{el} = F_{el,p}^{SS}+\zeta(F_{el,p}^{F}-F_{el,p}^{SS})$ ksi	$\lambda_l = (F_y/F_{el})^{1/2}$	For $\lambda_l \leq 0.68$, $\varepsilon_{csm}/\varepsilon_y = 0.25/(\lambda_l)^{3.6} \leq (\Lambda$ and $0.1(1-F_y/F_u)/\varepsilon_y)$	$\varepsilon_{csm}/\varepsilon_y$
467.38	0.3148	For $0.68 < \lambda_l < 1.00$, $\varepsilon_{csm}/\varepsilon_y = (1-0.222/(\lambda_l)^{1.05})*(1/(\lambda_l)^{1.05})$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.02481	Case 2) $\varepsilon_{csm}/\varepsilon_y \geq 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
16.209	16.950	20.8507591207799	**12485**

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Rectangular weld group				
B (in) (Weld group width)	D (in) (Weld group depth)	I_x (in^4/in) = $(2D^3/12+2B(D/2)^2$	S_x(in^3/in) = $I_x/(D/2)$	L (in) = 2D+2B
1	2	3.3333	3.3333	6.0000
Weld size (in)	Weld Filler Strength (ksi)	V (lbs) = M/42”	M (in-lbs)	T (lbs)
0.1875	70	298	12500	0.000
R_{shear} (pli)	R_{normal} (pli)	R_{net} (pli)		
50	3750	3750		
Angle between load and weld axis(rads)	$F_{nw}=0.6F_{EXX}(1.0+0.50\sin^{1.5}\theta)$ (ksi)			
1.558	62.997			
$R_n/\Omega = F_{nw}*S_w*\sin(45^\circ)/2*1000$ (pli)	Pass/Fail ($R_n/\Omega > R_{net}$)			
4176	Pass			

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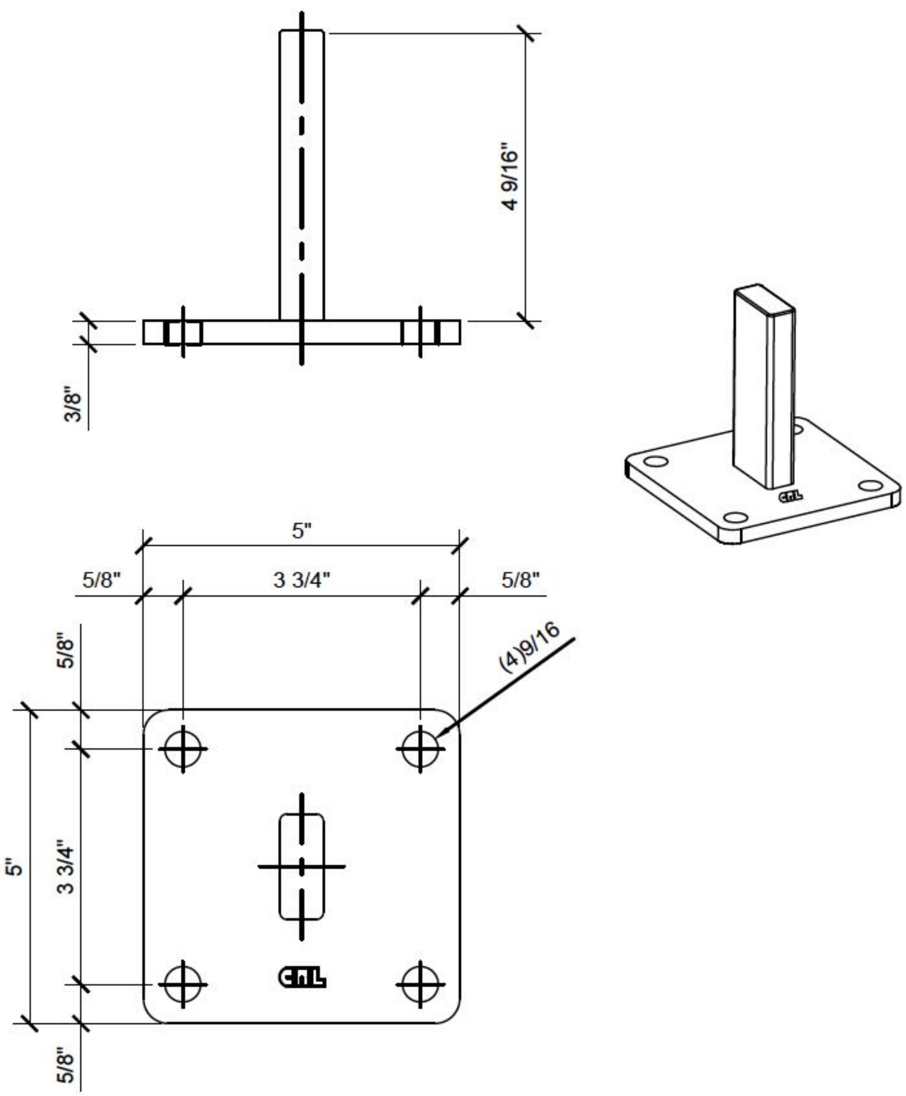
POST P9 continued:

Based on physical testing performed by CR Laurence-
Fracture moment of the post and baseplate assembly:
M_{fail} = 53,214”# > 43,000”# minimum strength calculated
Moment at full yield, plastic hinge, increasing deflection rate with decreasing rate of load increase:
$M_{plastic}$ = 24,300”#



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POST P9 continued:

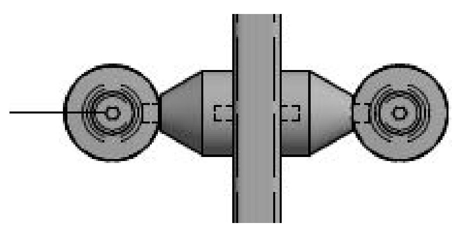
Base Plate:
5” square base plate same as previously checked, page 5.
Maximum allowable moment for baseplate strength = 13,534”#
13,534 ≥ 250*42” = 10,500”#

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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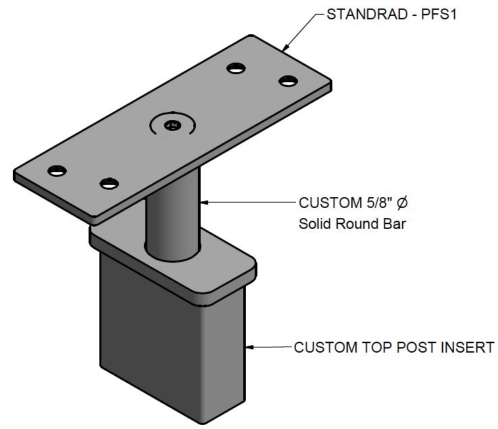
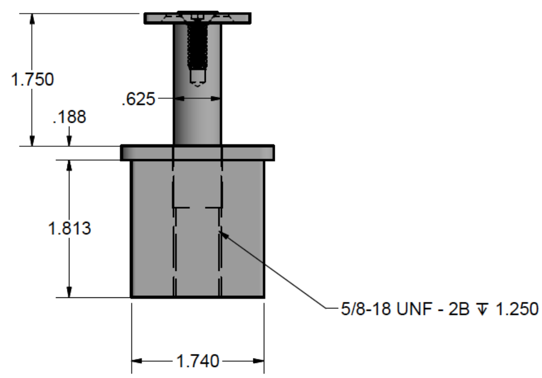
POST P9 continued:



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



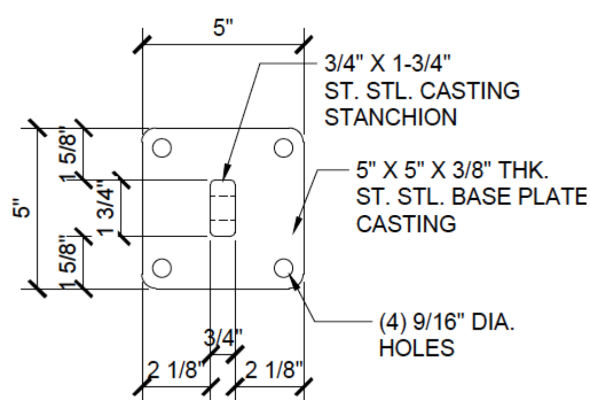
M_n = lesser of SF_u or ZF_y
$M_n = 0.024*70\text{ksi} = 1{,}680”\#$
$M_n = 0.0407*30\text{ksi} = 1{,}221”\#$
$Ma = 1{,}221/1.67 = 731”\#$
Maximum top rail load based on maximum height of 2.25” to centerline of the top rail:
$R_a = 731/2.25 = 325\# > 322\#$ doesn’t control allowable top rail load.

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

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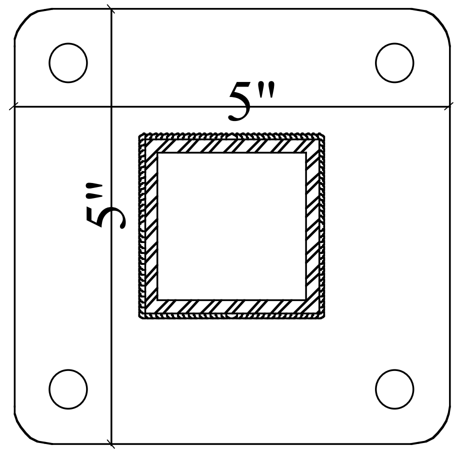
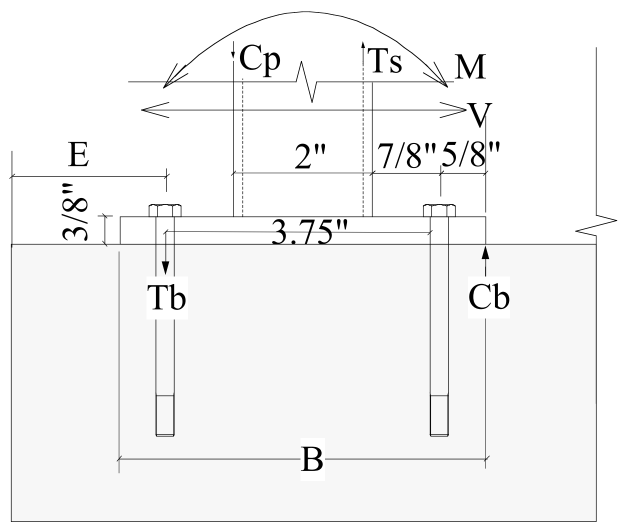
5”x5” Base Plate design:
May be used with any post.



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

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

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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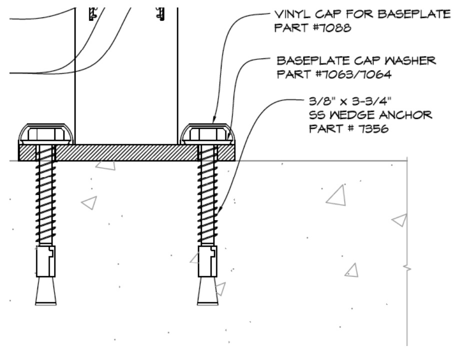
5"x5" BASE PLATE MOUNTED TO CONCRETE - Expansion Bolt Alternative:
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8"x3.75" concrete anchors with 3" effective embedment. Anchor strength based on ESR-2427
Minimum conditions used for the calculations:



$f'_c \geq 3,000$ 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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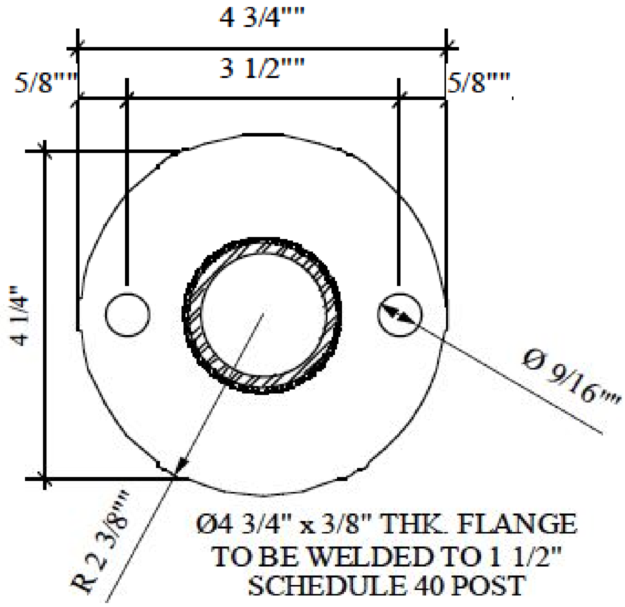
4-3/4" ROUND BASE PLATE
Mounted to Concrete
Designed per ACI 318-14 Chapter 17 and ICC ES-308
Maximum allowable post loads:
200# horizontal
8,400"# moment

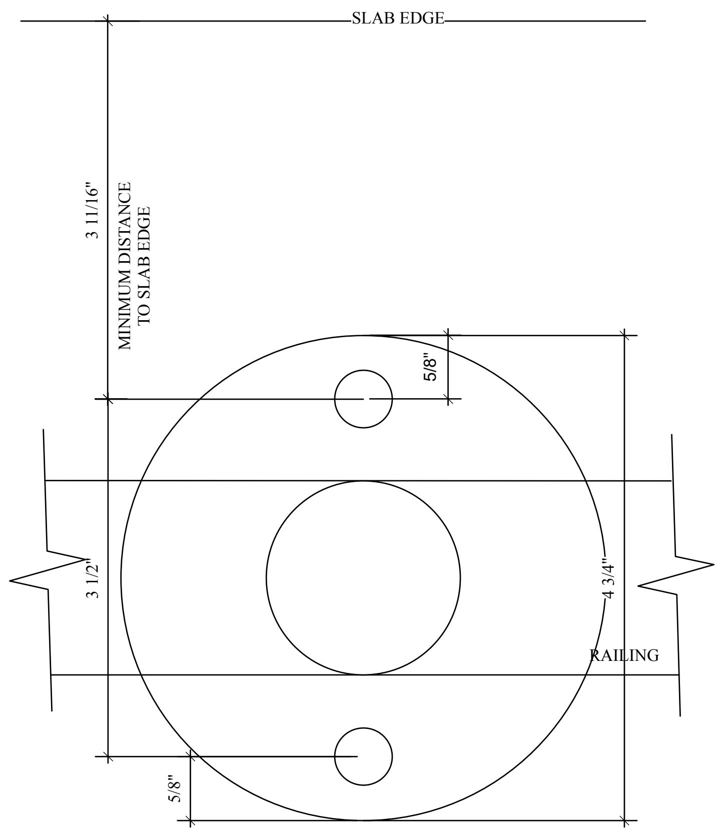


For anchors oriented along rail centerline-
For installation to 4,000 psi concrete:
Minimum slab thickness = 6.5"
Minimum edge distance = 5.25"
edge of slab to center of baseplate.

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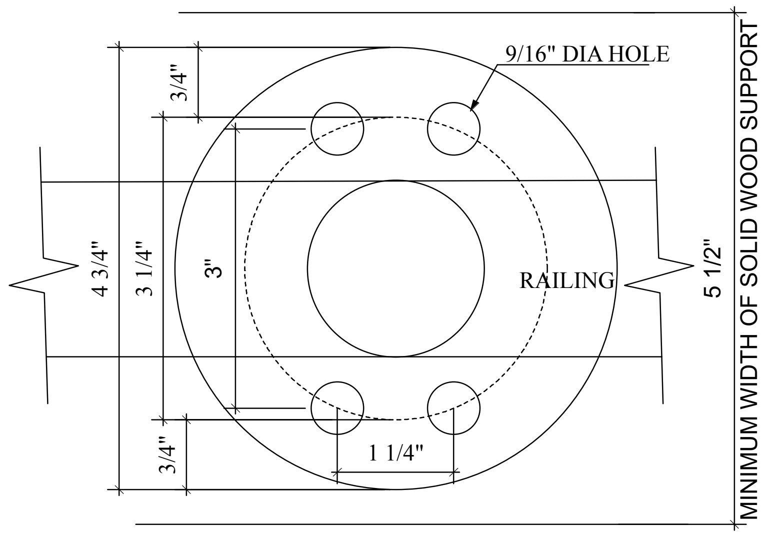
4-3/4" ROUND BASE PLATE
Mounted to Wood
4 Hole configuration



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 = 1.6 (NDS Table 2.3.2)
C_m = 0.7 (NDS table 10.3.3) for weather exposed wood.

W' = 302*1.6*0.7 = 338#/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*1.05) = 4.94 in^2
a = 4.94/(4.875) = 1.01"

Lag withdrawal load:
T_l = 8,400/[2*(4.0-1.01/2)]
T_l = 1,200#

Required embedment length into the solid blocking:
e = T_l/W' = 1,200#/338pli = 3.55"
Required lag length:
L = 3.55"+3/8"+5/16" + T_d = 4.24"+ decking thickness

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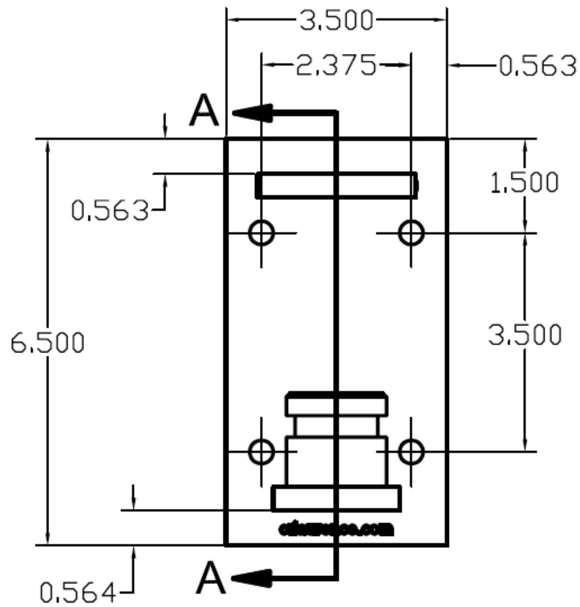
FASCIA BRACKETS


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).

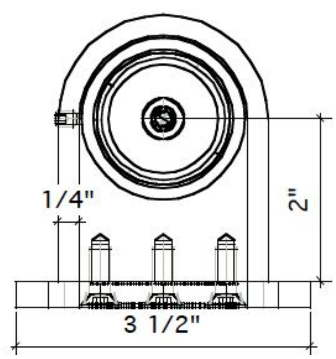


Determine forces on the brackets:

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 = $øA_TF_y/1.6 = 0.75*0.0524$ 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
$V_s = øA_vF_v/1.6 = 0.65*0.0524$ 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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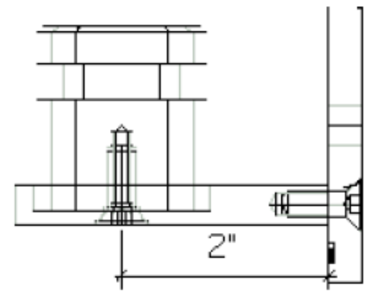
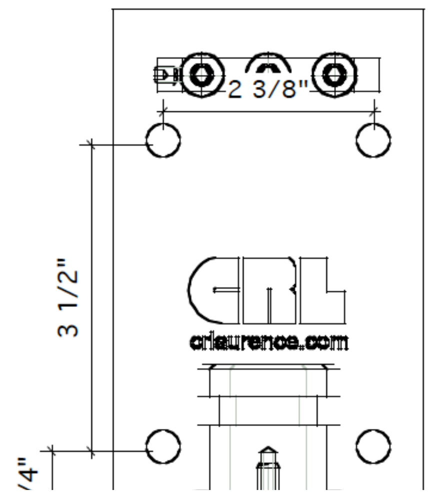
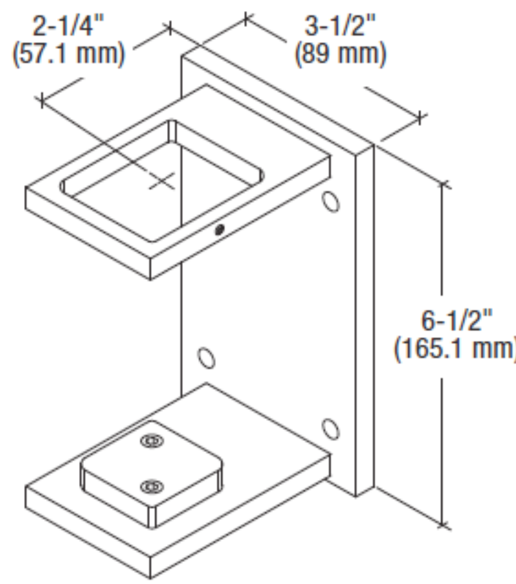
FASCIA BRACKETS Cont.
Bottom fitting strength:
Post end fitting is inlaid into bottom plate so that shear forces are directly transferred.
Bending stresses from vertical forces:
V_m = 350# (D+L)
M_V = 350#"*2" = 700#"
V_D = 100#
M_D = 100# * 2" = 200#"
Tension on screw:
$H_L + V_D$ = 2,526# + 200#"/0.25" = 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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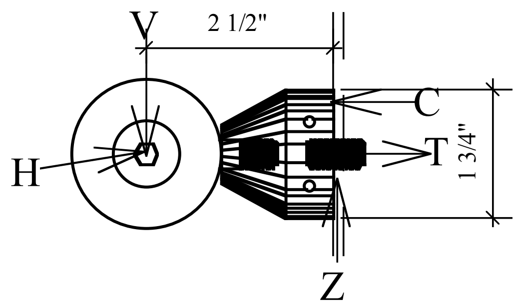
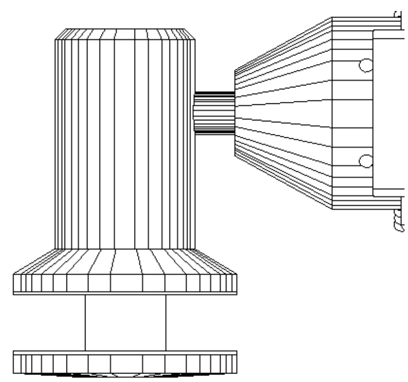
GLASS STANDOFFS
RB50S or RB50F



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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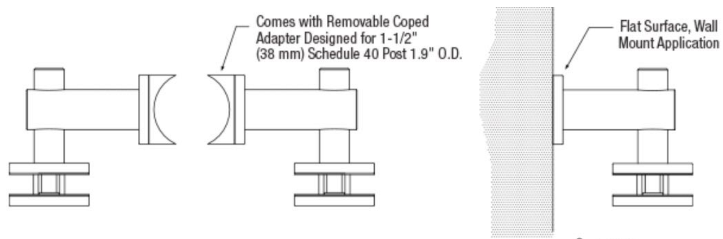
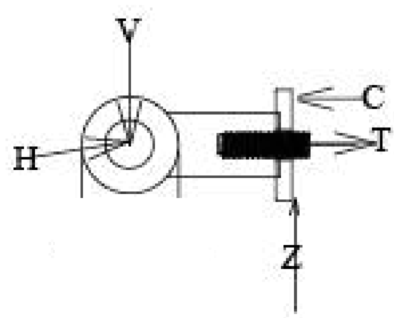
GLASS STANDOFFS
RB51F



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#



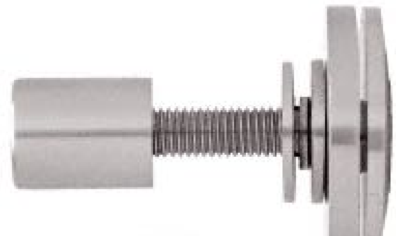
Moment resistance of connection:
M_n = 4,192#*(0.75”/2) = 1,572#”

$M_s = M_n/2$ = 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

RRF10BS/PS
Rigid fixed head
Mount to spider fitting

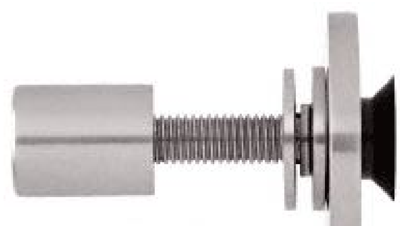


Cap Mount
(Standard hole through glass)

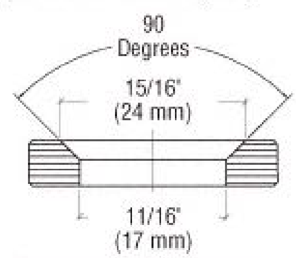
Cap mount glass fabrication

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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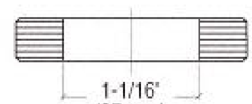
RSF10BS/PS
Combination Swivel head
Mount to spider fitting

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#



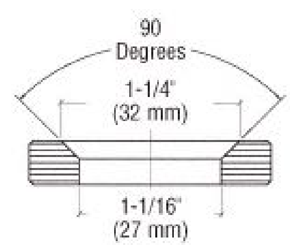
Cap mount glass fabrication

Strength of swivel ball joint: Shear failure around socket rim:
$øV_n = 0.85*42ksi*0.95*0.55"*\pi*0.065"= 3,809\#$
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

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



Flush mount glass fabrication

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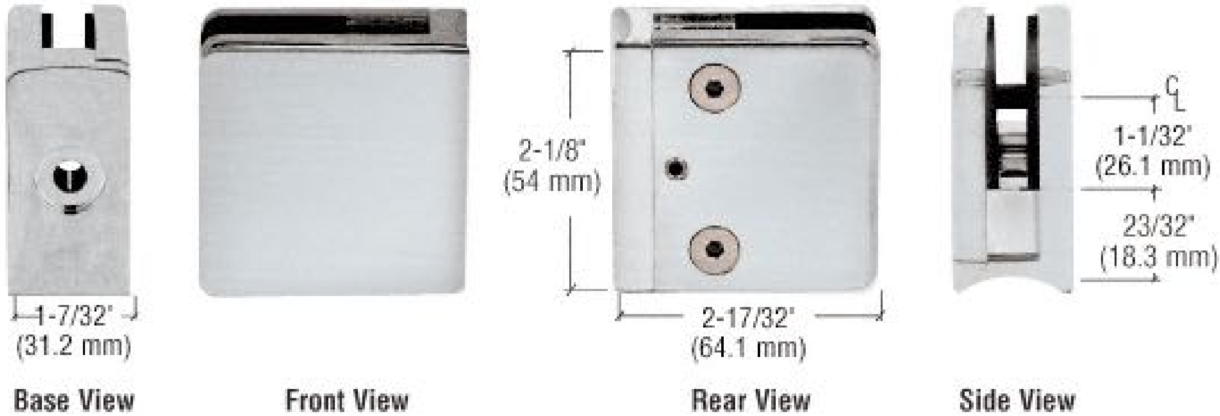
Z-CLAMPS - CRL Z712BS
Clamps to the glass on the side of the glass using a hole through the glass light at the clamp.



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)
$D = L' x B' x D_G$
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 $\sum M$ 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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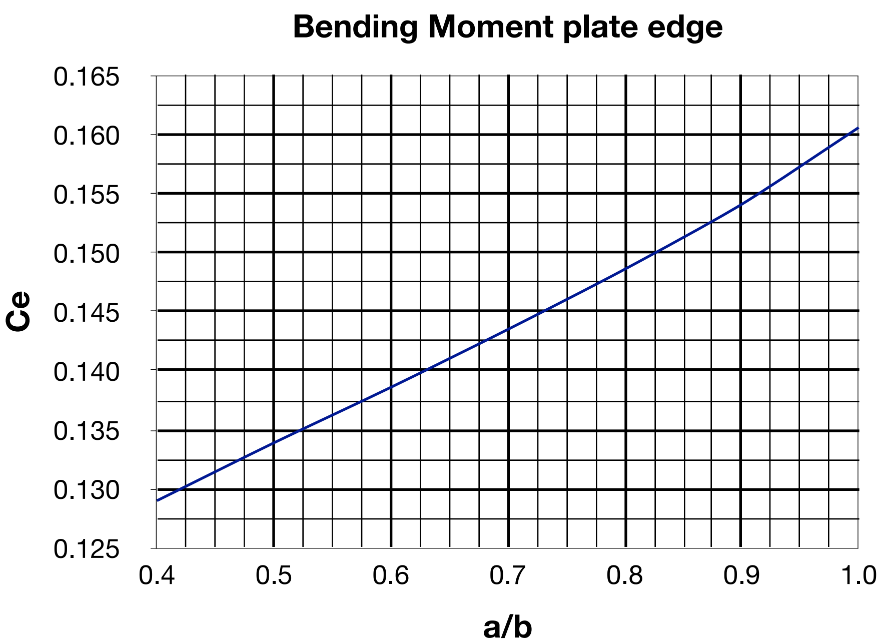
GLASS INFILL LIGHTS

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.

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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.

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 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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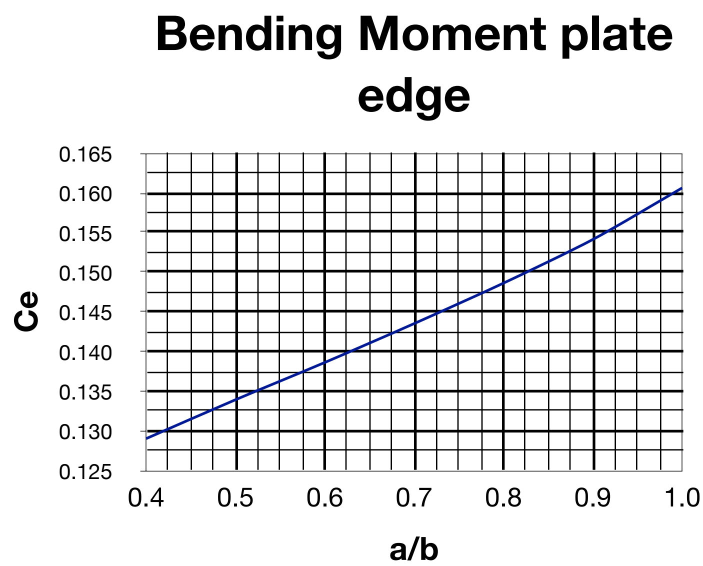
GLASS INFILL DESIGN



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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