Edward C. Robison, P.E.

07 March 2023

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

SUBJ: CRL P-SERIES POST RAILING SYSTEMS
STAINLESS STEEL POSTS FOR GUARDRAILS
MINI POSTS - 16 INCH TALL

I have evaluated the stainless steel post kits to verify that they will safely support the following loads when used in building guardrails:

200 pound point load on top rail, vertical or horizontal
50 plf load on top rail, vertical or horizontal or
50 psf uniform load on glass panel horizontal or
50 lb conc load on 1 sf

Allowable post spacing is as shown on page 2.

For the installations using laminated tempered glass a top rail is optional. For installations using monolithic glass the top rail/cap rail is required when used as a guard. The top rail/cap rail may be as approved in ESR-3269 for the glass thickness, span and use.

Stainless steel members are analyzed according to the provision of AISC 370.

If you have any questions please e-mail me at elrobison@narrows.com or call at 253-858-0855.

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Sealed 07 March 2023

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Post Type	Glass Thickness and type	Glass height (inches)	Post Spacing (inches)	Allowable wind pressure (psf)
PWC1 OR PWC2	3/8" Laminated	12"	46	80
PWC1 OR PWC2	9/16" laminated	12"	64	80
PWC1 OR PWC2	3/8" monolithic	12"	52	80
PWC1 OR PWC2	1/2" monolithic	12"	68	80
PWC1 OR PWC2	9/16" lam/ 1/2" mon	16"	72	60

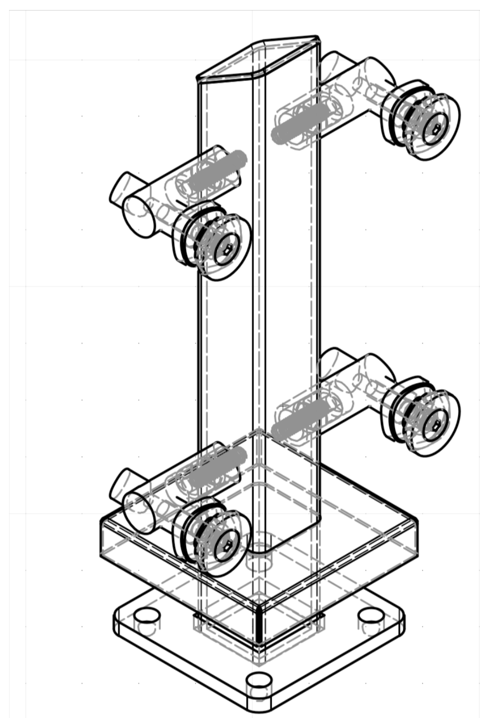
Typical Installation: 16" tall posts spacing up to 6' as allowed for glass height and type. Minimum glass height is 12". Maximum post spacing is achieved with a glass height of 16". All anchorage options are equivalent for post spacing.

PWC1

HSS2X2X3/16" 304/316 Stainless steel

The PWC1 mini post is a HSS2x2x1/4 welded to a stainless steel baseplate. The post passes through a hole in the baseplate and is welded from top and bottom producing a weld that develops the full strength of the post.

The post forms a plastic hinge at the baseplate. The shape is compact, is a closed tube and is relatively short. Therefore, a plastic hinge will form and the post may be designed according to AISC 370 Appendix 2, continuous strength method.



Moment strength calculations for the HSS2x2x1/4 are shown on the following two pages.

The PWC1 can also be used in an eccentric condition that creates torsion on the post.

The torsion strength of the post is calculated following the flexural strength calculations.

Post strength summary:
M_a = 17,000"#

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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)	$\varepsilon_y = F_y/E$
30	15	28000	0.00107
F_u (ksi)	E_{sh} (ksi)	υ	α
75	474.04	0.3	2
Section Properties:			
t_p (in)	b_p (in)	S (in^3)	Z (in^3)
0.174	2	0.641	0.797
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	766.18	1
Isolate web	k	$F_{el,w}^{SS} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_w
	23.9	4577.96	1
$F_{el,p}^{SS} = \min(\beta_f F_{el}^{SS}{}_f\ ,\ \beta_w F_{el}^{SS}{}_{,w})$			
766.18			
Isolated flange	k	$F_{el,f}^{F} = k\pi^2E/(12(1-\upsilon^2))(t_p/b_p)^2$ (ksi)	β_f
	6.97	1335.08	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	7585.23	1

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$F_{el,p}^F = \min(\beta_f F_{el}^F{}_f , \beta_w F_{el}^F{}_{,w})$			
1335.08			
$ø = \beta_f F_{el}^{SS}{}_f / (\beta_w F_{el}^{SS}{}_{,w})$	If $ø < 1$	$a_f = 0.24-[0.1(t_f/t_w)^2(H/B-1)]^{1/0.6} \le 0.24$	**af**
0.17	If $ø \ge 1$	$a_w=0.63-0.1H/B \le 0.53$	0.24
	If $ø < 1$	$\zeta = t_w/t_f*(0.24-a_f*ø)^{0.6}$	ζ
	If $ø \ge 1$	$\zeta = t_f/t_w*(0.53-a_w/ø)$	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 $\lambda_l < 0.68$, $\varepsilon_{csm}/\varepsilon_y = 0.25/(\lambda_l)^{3.6}+0.002/\varepsilon_y \le \Lambda$	$\varepsilon_{csm}/\varepsilon_y$
982.67	0.1747	For $0.68 < \lambda_l < 1.00$, $\varepsilon_{csm}/\varepsilon_y = (1-0.222/(\lambda_l)^{1.05})*(1/(\lambda_l)^{1.05})+0.002(f/F_y)^n/\varepsilon_y$	15
ε_{csm}	Case 1) $\varepsilon_{csm}/\varepsilon_y < 1.0$	$M_n=\varepsilon_{csm}/\varepsilon_y M_y$	
0.01607	Case 2) $\varepsilon_{csm}/\varepsilon_y \ge 1.0$	$M_n=M_P(1+E_{sh}S/(EZ)*(\varepsilon_{csm}/\varepsilon_y-1)-(1-S/Z)/(\varepsilon_{csm}/\varepsilon_y)^{\alpha})$	
M_y (in-kips)	M_p (in-kips)	M_n (in-kips)	**$M_a = M_n/1.67*1000$ (in-lbs)**
19.23	23.91	28.45	**17034**
Percent increase over Chapter F strength			
19.0			

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SS HSS Torsion Strength, Per AISC 370 G8-1 and G8.2.			
h(in)	t(in)	λ=h/t	E(ksi)
2	0.174	11.494	28000
F_y(ksi)	$0.74(E/F_y)^{1/2}$	$2.17(E/F_y)^{1/2}$	$5.99(E/F_y)^{1/2}$
30	22.607	66.295	182.998
See G8.2 a-d for calculation of C_v with respect to λ.	G8-10 controls	C_V	C (in^3) section constant for HSS2x2x1/4
		1.2	1.41
$T_n = 600CC_vF_y$ (in-lbs)	**$T_n/\Omega = T_n/1.67$ (in-lbs)**		
30456	**18237**		
Also check direct shear:			
Strength per AISC 370 G3	A_w = 2ht (in^2)	λ=h/t	k_v
	0.696	11.494	5
$0.33(k_vE/F_y)^{1/2}$	$0.97(k_vE/F_y)^{1/2}$	$2.68(k_vE/F_y)^{1/2}$	See G2.2-8 - 11 for calculation of C_{v2}.
22.543	66.264	183.079	
G2-8 controls	C_{v2}	$V_n=600F_yA_WC_{v2}$ (lbs)	**$V_n/\Omega = V_n/1.67$ (lbs)**
	1.2	15034	**9002**

For combined forces, AISC 370 H2-1 controls.	Combined forces will be the worst for short guards with high loading. Check for 1000# test load at 24" total height. V_{max} = 500#*(24"-5")/8" (Shear is highest between standoffs) T = 500#*3.25", M = 500#*24"		
M_r (in-lbs)	V_r (lbs)	T_r (in-lbs)	Torsion and shear are not very significant compared to moment. Therefore, the intermediate posts that have twice the moment and no torsion will control over the end posts with torsion.
12000	1188	1625	
H2-1 states $(P_r/P_c+M_r/M_c)+(V_r/V_c+T_r/T_c)^2 \le 1.0$		Utilization checking moment only,	
0.75		0.70	

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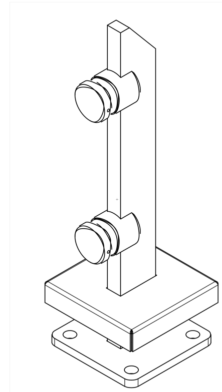
PWC2



Check 2”x3/4” 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)
12	42.6667	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	(Plastic moment strength controls for 2x3/4” flat bar, check strength based on Appendix 2. No flange elements and the flat bar has been shown to be compact so $\varepsilon_{csm}/\varepsilon_y = 15$.
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)**
15	22.5	26.02	**15582**
Percent increase over Chapter F strength			
15.7		34.7	

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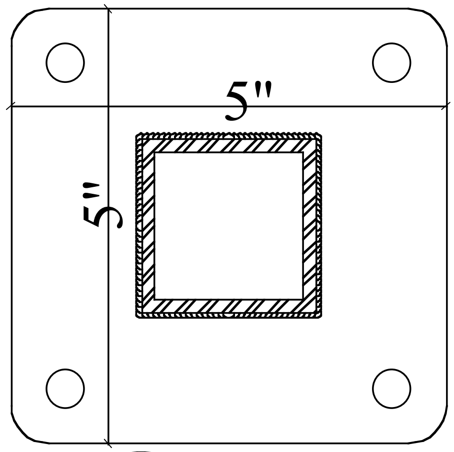
POST ANCHORAGE

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 maximum design load on posts.

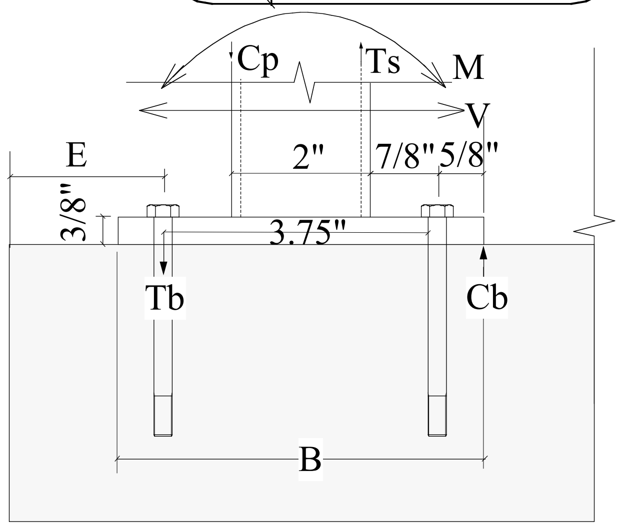
Live load

M = 300#x16" = 4,800"#

Maximum wind load:

W = 60psf*6'*1.333' = 480#

M = 480#x16"*0.55 = 4,224"#



Live load controls for tension

$T_b = M/4.125"/2$ bolts

$T_b = 4800/(4.125*2) = 582\#$

Nominal anchor tension

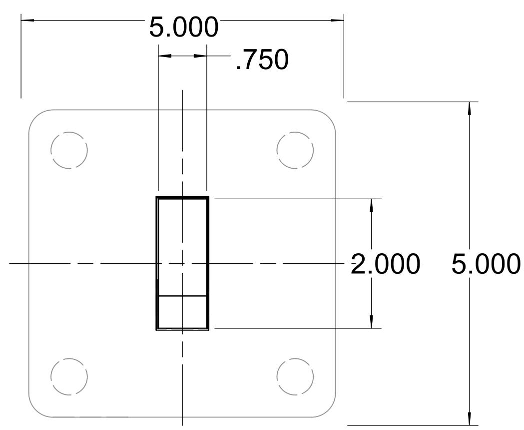
Base plate moment

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

$M_u = 1.6*2*582*(7/8") = 1{,}630\#"$

$M_u < \phi M_n$ therefore okay

Base plate anchor strength:



Service strength required for anchors

$T_s = 582\#$ (for allowable load on anchor)

This base plate is used with both of the posts in this series. the strength and anchorage will be the same for all post types.

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ANCHORAGE DESIGN
Design loads per screw:
T = 582# or Shear = 240#

MOUNTED TO STEEL
The baseplates may be attached to either structural steel sections or to cold formed steel members. Interaction between shear and tension may be assumed as not a consideration as the shear will be resisted by the compression side screws.

Self-tapping screws: (Strength per ESR-3064P for generic screws per AISI S100 Section E4)
For 1/4" screws minimum steel thickness = 118 mil, design thickness = 1/8" Grade 50
Screw length as needed to fully penetrate

Through bolts with nut and washer: 3/8" bolt with heavy washer
Minimum steel thickness = 54 mil

MOUNTED TO WOOD - Lag Screw Alternative:
Lag screw withdrawal strength:
W = 243#/in for 3/8" lag screw and G $\geq$ 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' = 582/227 = 2.564"
Required lag length:
L = 2.564"+3/8"+7/32" + T_d = 3.16" + decking thickness
With typical 3 1/2" lag screw with base plate on structural wood the wood embedment is over 3".
Shear strength of lag screw with 3" into wood per NDS Table 12K for G = 0.43
$Z_\perp$' = 1.33*1.60 = 213#
Shear load on the lag screws in tension:
Z = (480# - 2*213)/2 = 27#
Effective load angle on the screws in tension:
Tan^{-1}(582/27) = 87.3
Sin 87.3 = 0.9989 rounds to 1 thus can ignore the interaction as the strength will round to the full strength of W'.

When directly mounted on solid wood 3/8" x 3-1/2" lag screws.
When attached with non-structural wood materials between the baseplate and structural wood the lag screw length must be a minimum 3.16" + the thickness of the non-structural materials.

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ANCHORAGE TO CONCRETE

Typical anchorage is Hilti Kwik HUS-EZ 3/8” either 3” or 4”. Equivalent anchor may be used with the same embedment.

For 3” with 2-1/8” nominal embedment: May be used in uncracked concrete with f’c ≥ 2,500 psi and 2 5/16” edge distance.

For 4” with 3-3/8” nominal embedment: May be used in cracked or uncracked concrete with f’c ≥ 2,500 psi and 1 9/16” edge distance.

3/8” KH-EZ breakout per ACI 318 Chapter 17. Cracked concrete minimum concrete strength						
f’c (psi)	hef (in)	Edge distance anchors (in)	Spacing parallel to edge (in)	Concrete thickness (in)	D (in)	Lever arm to bolts (in)
2500	1.54	2.31	3.75	3.67	0.375	4.375
Area calculations						
A_{Vc} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)	C_{ac} (în)		
37.006	38.6694	24.012	21.3444	3.75		
Shear breakout						
$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg}	
1	1	1	1	998	1538	
Tension breakout						
$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg}	
1	1	1.0	1	1624	2943	
Design checks						
Nominal strengths are multiplied by the reduction factor of 0.65 and divided by the load factor of 1.6 to determined the allowable load.						
V_a	V	Pass/Fail				
625	240	Pass				
T_a (lbs)						
1196	on anchor group					
$M_a = T_a$*(4.375”) (in-lbs)		M	$V/V_a+M/M_a <$ 1.2			
5231		4224.00	1.19	< 1.2 OK	Pass	

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3” anchor at 1.5” edge distance in cracked concrete requires f’c ≥ 4,500 psi

3/8” KH-EZ breakout per ACI 318 Chapter 17. Cracked concrete minimum edge distance						
f’c (psi)	hef (in)	Edge distance anchors (in)	Spacing parallel to edge (in)	Concrete thickness (in)	D (in)	Lever arm to bolts (in)
4500	1.54	1.5	3.75	3.67	0.375	4.375
Area calculations						
A_{Vc} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)	C_{ac} (în)		
18.563	31.8897	10.125	21.3444	3.75		
Shear breakout						
$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg}	
1	1	1	1	701	1285	
Tension breakout						
$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg}	
1	1	1.0	1	2179	3256	
Design checks						
Nominal strengths are multiplied by the reduction factor of 0.65 and divided by the load factor of 1.6 to determined the allowable load.						
V_a	V	Pass/Fail				
522	240	Pass				
T_a (lbs)						
1323	on anchor group					
$M_a = T_a$*(4.375”) (in-lbs)		M	$V/V_a+M/M_a <$ 1.2			
5787		4224.00	1.19	< 1.2 OK	Pass	

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3" anchor at 1.5" edge distance in uncracked concrete requires f'c ≥ 3,000 psi

3/8" KH-EZ breakout per ACI 318 Chapter 17. Uncracked concrete minimum edge distance						
f'c (psi)	hef (in)	Edge distance anchors (in)	Spacing parallel to edge (in)	Concrete thickness (in)	D (in)	Lever arm to bolts (in)
3000	1.54	1.5	3.75	3.67	0.375	4.375
Area calculations						
A_{Vc} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)	C_{ac} (în)		
18.563	31.8897	10.125	21.3444	3.75		
Shear breakout						
$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg}	
1	1	1	1	572	1049	
Tension breakout						
$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg}	
1	1	1.4	1	1779	3722	
Design checks						
Nominal strengths are multiplied by the reduction factor of 0.65 and divided by the load factor of 1.6 to determined the allowable load.						
V_a	V	Pass/Fail				
426	240	Pass				
T_a (lbs)						
1512	on anchor group					
$M_a = T_a$*(4.375") (in-lbs)		M	$V/V_a+M/M_a <$ 1.2			
6615		4224.00	1.20	< 1.2 OK	Pass	

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3/8” KH-EZ breakout per ACI 318 Chapter 17. Cracked concrete minimum edge distance KH-EZ SS316 3/8” x 4” (CRL part WBA38x4SS #2245627						
f’c (psi)	hef (in)	Edge distance anchors (in)	Spacing parallel to edge (in)	Concrete thickness (in)	D (in)	Lever arm to bolts (in)
2500	2.5	1.56	3.75	3.67	0.375	4.375
Area calculations						
A_{Vc} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)	C_{ac} (în)		
19.726	59.7375	10.951	56.25	3.75		
Shear breakout						
$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg}	
1	1	1	1	610	1099	
Tension breakout						
$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg}	
1	1	1.0	1	3360	3568	
Design checks						
Nominal strengths are multiplied by the reduction factor of 0.65 and divided by the load factor of 1.6 to determined the allowable load.						
V_a	V	Pass/Fail				
447	240	Pass				
T_a (lbs)						
1450	on anchor group					
$M_a = T_a*(4.375”)$ (in-lbs)		M	$V/V_a+M/M_a <$ 1.2			
6342		4224.00	1.20	< 1.2 OK	Pass	

GLASS STRENGTH

All glass is fully tempered glass conforming to the specifications of ANSI Z97.1, ASTM C 1048 and CPSC 16 CFR 1201. The median F_r for the tempered glass is 24 ksi. In accordance with IBC 2407.1.1 glass used as structural balustrade panels shall be designed for a safety factor of 4.0. For loads other than guard live loads glass may be designed for stresses in accordance with ASTM E1300.

Values for the modulus of rupture, F_r, modulus of Elasticity, E and shear modulus, G for glass are typically taken as:

F_r = 24,000 psi based on numerous published data from various glass manufacturers. This value is recognized in ASTM E 1300, ANSI Z97.1, ASTM C 1048 and CPSC 16 CFR 1201 (derivation of the value may be required). This value is referenced in numerous publications, design manuals and manufacturers' literature.

E = 10,400 ksi is used as the standard value for common glass. While the value of E for glass varies with the stress and load duration this value is typically used as an average value for the stress range of interest. It can be found in ASTM E 1300 and numerous other sources.

G = 3,800 ksi: This is available from various published sources but is rarely used when checking the deflection in glass. The shear component of the deflection tends to be very small, about 1% of the bending component and is therefore ignored.

$\mu = 0.22$ Typical value of Poisson's ratio for common glasses.

$\nu = 5\text{x}10^{-6}$ in/(inF°) Typical coefficient of thermal expansion.

Maximum allowable glass stress for tempered glass in guard rail application = 24,000psi/4 = 6,000psi

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The glass has point supports near each corner. Peak bending stress is at the center edge of the glass lights remote from the point supports. Since the length to width exceeds 2 the stress amplification the peak bending moment is negligible and may be safely assumed:
c_{fb} = 0.1339/0.125 = 1.07

3/8" laminated t_{min} (in)	I = $t_{,ef;w}^3$ (in^4/ft)	S = $2t_{ef;\sigma}^2$ (in^3/ft)	M_a = S*6,000psi (in-lbs) Live load	M_a = S*9,600psi (in-lbs) wind load
0.315	0.0313	0.1985	1191	1905
	width (ft)	Max span inches	width (ft)	Max span
50plf Live Load	1	46.2	1.333	53.4
Wind Load (psf)		80.0		60.0

9/16" laminated t_{min} (in)	I = $t_{,ef;w}^3$ (in^4/ft)	S = $2t_{ef;\sigma}^2$ (in^3/ft)	M_a = S*6,000psi (in-lbs) Live load	M_a = S*9,600psi (in-lbs) wind load
0.438	0.0840	0.3837	2302	3683
	width (ft)	Max span inches	width (ft)	Max span
50plf Live Load	1	64.3	1.333	74.2
Wind Load (psf)		80.0		60.0

3/8" monolithic t_{min} (in)	I = $t_{,ef;w}^3$ (in^4/ft)	S = $2t_{ef;\sigma}^2$ (in^3/ft)	M_a = S*6,000psi (in-lbs) Live load	M_a = S*9,600psi (in-lbs) wind load
0.355	0.0447	0.2521	1512	2420
	width (ft)	Max span inches	width (ft)	Max span
50plf Live Load	1	52.1	1.333	60.1
Wind Load (psf)		80.0		60.0

1/2" monolithic t_{min} (in)	I = $t_{,ef;w}^3$ (in^4/ft)	S = $2t_{ef;\sigma}^2$ (in^3/ft)	M_a = S*6,000psi (in-lbs) Live load	M_a = S*9,600psi (in-lbs) wind load
0.469	0.1032	0.4399	2640	4223
	width (ft)	Max span inches	width (ft)	Max span
50plf Live Load	1	68.8	1.333	79.5
Wind Load (psf)		80.0		60.0

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