

08 July 2016

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SUBJ: TAPER-LOC® SYSTEM DRY-GLAZE  
LAMINATED TEMPERED GLASS RAIL SYSTEM WITH SGP INTERLAYER  
9/16" LAMINATED GLASS - L56S AND 9BL56 BASE SHOES

The GRS Glass Railing Dry Glaze Taper-Loc™ System utilizing 9/16" laminated tempered glass with Sentry Glas+™ interlayer (1/4" glass plies with 0.06" interlayer) balustrade lights in a properly anchored, aluminum extruded base shoe and appropriate cap rail to construct guards for fall protection. The system is intended for interior and exterior weather exposed applications and is suitable for use in most natural environments. The system may be used for residential, commercial and industrial applications where not subject to vehicle impacts. This is an engineered system designed for the following criteria:

The design loading conditions are:

Conc. load = 200 lbs any direction, any location along top or 42" above walking surface\*

Uniform load = 50 plf perpendicular to glass at top or 42" above walking surface\*

Load of 50 lbs on one square foot at any location on glass.

Wind load = As stated for the application and components, 10 psf minimum - ASD level.

\*Refer to IBC Section 1607.7.1, applicable when fall protection is required.

Installations without a top rail shall comply with the recommendations herein and IBC 2407.1.2.

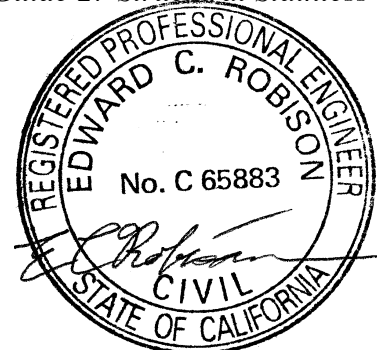
Glass stresses are designed for a safety factor of 4.0 (IBC 2407.1.1) for live loads.

The system will meet the applicable requirements of the 2009, 2012 and 2015 International Building Codes, 2010, 2013 and 2016 California Building Codes, and 2010 Florida Building Code (as wind loading permits) and other state codes adopting the IBC. Aluminum components are designed in accordance with the 2005 and 2010 Aluminum Design Manuals. 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 appropriate.

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Sealed 07/08/2016

**Typical Installations:**

Surface or fascia mounted to:

1/2" cap screw to steel @ 12" o.c.:

1/2" Wedge-Bolt to concrete @ 12" o.c. or @ 6" O.C.

1/2" x 6" lag screws to wood (moisture content maintained  $\leq 19\%$ ) @ 12" o.c. or @ 6" O.C.

Refer to Table 5 on page 18 for surface mounted anchor strength and allowable wind loads or Table 6 on page 20 for fascia mounted anchor strength and allowable wind loads.

**Embedded base shoe:**

Glass strength controls for all cases

**ALLOWABLE LOADS ON GLASS**

The allowable load on the glass is dependent on the glass makeup and light width. Refer to table 3 on page 7 of 24 for allowable moment for wind loading.

Calculate glass moment based on wind load-

$M_w = w * h^2 * 0.55 * 12$ : in-lb/ft

where:

w = wind load pressure in psf

h = effective cantilever height:

h = from top of base shoe to top edge of cap rail or glass if no cap rail installed when wet glazed.

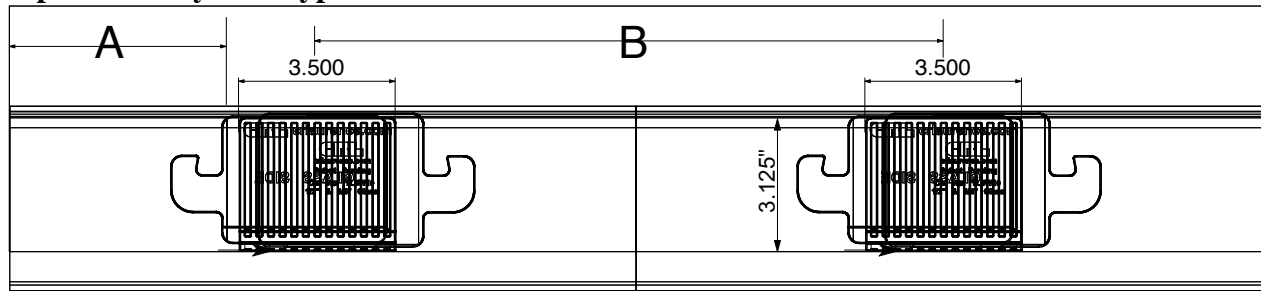
When installed with Taper-Locs® add 0.042 feet (1/2 in) to allow for that Taper-Locs® are set below top of base shoe.

**FOR INSTALLATION WITH A TOP RAIL:** Maximum glass cantilever height for fall protection is limited to that height at which the glass bending moment does not exceed the allowable glass moments as shown in Table 2 (page 7 of 24) for 50 plf live load or 200 lb concentrated live load being applied at top of glass or at 42 inches above the finish floor, whichever is less, for compliance with the International Building Code (all versions) and International Residential Code (all versions).

**FOR INSTALLATION WITHOUT A TOP RAIL:** Maximum glass cantilever height for fall protection is limited to the glass height as shown in Table 4 (page 10 of 24) for compliance with the International Building Code (all versions) and International Residential Code (all versions).

**REFER TO GRS TOP RAILS AND HANDRAILS ENGINEERING REPORT FOR CAP RAILS (REQUIRED FOR FALL PROTECTION) AND HANDRAILS (REQUIRED ALONG STAIRS AND RAMPS.)**

**Taper-Loc® System Typical Installation**



DETAIL 1

For two ply laminated glass with 1/4" Fully Tempered Glass and 1/16" ionoplast interlayer maximum glass light height is 42":

Edge Distance:  $2'' \leq A \leq 4''$ ;  $51\text{mm} \leq A \leq 102\text{mm}$

Center to center spacing:  $5'' \leq B \leq 8''$ ;  $127\text{ mm} \leq B \leq 203\text{ mm}$

Panel Width/Required quantity of Taper-Loc Plates:

6" to <10" (127 to 254 mm)	1 TL Plate
10" to <16" (254 to 406 mm)	2 TL Plates
16" to <24" (406 to 610 mm)	3 TL Plates
24" to <32" (610 to 813 mm)	4 TL Plates
32" to <40" (813 to 1,016 mm)	5 TL Plates
40" to <48" (1,016 to 1,219 mm)	6 TL Plates
48" to <56" (1,219 to 1,422 mm)	7 TL Plates
56" to <64" (1,422 to 1,626 mm)	8 TL Plates
64" to <72" (1,626 to 1,829 mm)	9 TL Plates
72" to <84" (1,067 to 1,422 mm)	10 TL Plates
80" to $\leq 84''$ (2,032 - 2,134 mm)	11 TL Plates

**Minimum Glass Lite Width = 6" when top rail/guardrail is continuous, welded corners or attached to additional supports at rail ends.**

NOTES:

1. For glass light heights over 42"  $A_{max}$  and  $B_{max}$  shall be reduced proportionally.

$$A_{max} = 4 * (42/h)$$

$$B_{max} = 8 * (42/h)$$

2. For glass light heights under 42"  $A_{max}$  and  $B_{max}$  shall not be increased.

3.  $A_{min}$  and  $B_{min}$  are for ease of installation and can be further reduced as long as proper installation is achieved.

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**LOAD CASES:**

Dead load = 6.9 psf for glass  
 1.8 plf top rail  
 3.0 plf for base shoe

**Loading:**

Horizontal load to base shoe

$25 \text{ psf} \cdot H$  or  $W \cdot H$

Balustrade moments

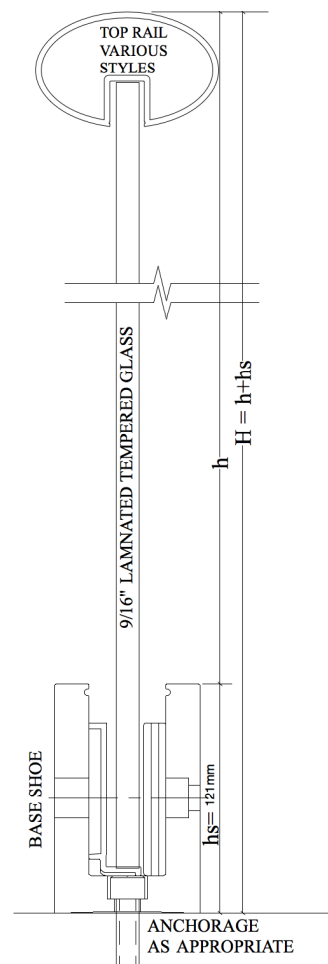
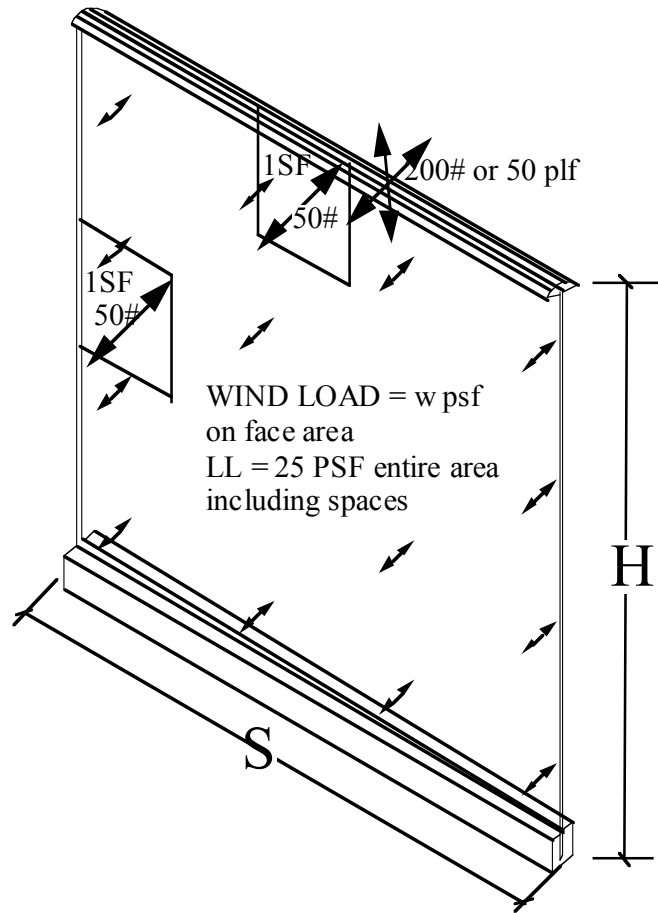
$M_i = 25 \text{ psf} \cdot H^2 / 2$  or

$M_w = w \text{ psf} \cdot H^2 / 2$

For top rail loads:

$M_c = 200\# \cdot H$

$M_u = 50\text{plf} \cdot H$



**FOR WIND**

SCREEN OR DIVIDER APPLICATIONS WHERE FALL PROTECTION IS NOT REQUIRED THE CAP RAIL MAY BE OMITTED.

THE 200# LOAD, 50 PLF LOAD AND 25 PSF LOAD CASES ARE APPLICABLE TO GUARD APPLICATIONS.

MINIMUM WIND LOAD IS 10 PSF.

WIND LOADS ARE ALLOWABLE STRESS DESIGN LOADS.

WIND LOADS CALCULATED AT STRENGTH LEVEL PER ASCE/SEI 7-10 SHALL BE ADJUSTED TO ASD LEVEL BY MULTIPLYING THE STRENGTH LEVEL LOADS BY 0.6.

WHEN INSTALLED WITHOUT A CAP RAIL DIFFERENTIAL DEFLECTION OF THE GLASS LIGHTS MUST BE CHECKED AND LIMITED TO UNDER 9/16" OR LIGHTS MUST BE CONNECTED.

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**WIND LOADING ON FENCES OR GUARDS**

Calculated in accordance with ASCE/SEI 7-10 Section 29.4 *Design Wind Loads on Solid Freestanding Walls and Solid Signs* (or ASCE/SEI 7-05 Section 6.5.14). This section is applicable for free standing building guardrails, wind walls and balcony railings that return to building walls. Section 29.6 *Parapets* may be applicable when the rail is along a roof perimeter.

**Wind loads must be determined by a qualified individual for a specific installation.**

$p = q_h(GC_p) = q_zGC_f$  (ASCE 7-10 eq. 29.4-1)

$G = 0.85$  from (section 26.9.4.)

$C_f = 2.5 * 0.8 * 0.6 = 1.2$  (Figure 29.4-1) with reduction for solid and end returns, will vary.

$q_h = 0.00256K_zK_{zt}K_dV^2$  Where:

$K_z$  from (Table 29.3-1) at the height  $z$  of the railing centroid and exposure.

$K_d = 0.85$  from (Table 26-6).

$K_{zt}$  From (Figure 26.8-1) for the site topography, typically 1.0.

$V =$  Wind speed (mph) 3 second gust, (Figure 26.5-1A) or per local authority.

Simplifying - Assuming  $1.3 \leq C_f \leq 2.6$  (Typical limits for fence or guard with returns.)

Adjustment for full height solid:  $f = 1.8 - 1 = 0.8$

Adjustment to Allowable Stress Design:  $w_{asd} = 0.6w_{strength}$

For  $C_f = 1.3$ :  $F = q_h * 0.85 * 1.3 * 0.8 * 0.6 = 0.53 q_h$

For  $C_f = 2.6$ :  $F = q_h * 0.85 * 2.6 * 0.8 * 0.6 = 1.06 q_h$

Wind Load will vary along length of fence in accordance with ASCE 7-10 Figure 29.4-1.

Typical exposure factors for  $K_z$  with height 0 to 15' above grade:

Exposure	B	C	D
$K_z =$	0.70	0.85	1.03

Centroid of wind load acts at 0.55h on the fence.

$w_{asd} = 0.53 * 0.00256 * K_z * V^2$  or  $w_{asd} = 1.06 * 0.00256 * K_z * V^2$

Table 1	W <sub>ASD</sub> in psf for C <sub>f</sub> = 1.3			W <sub>ASD</sub> in psf for C <sub>f</sub> = 2.6		
	Exp B K <sub>z</sub> =0.7	Exp C K <sub>z</sub> =0.85	Exp D K <sub>z</sub> =1.03	Exp B K <sub>z</sub> =0.7	Exp C K <sub>z</sub> =0.85	Exp D K <sub>z</sub> =1.03
100	9.5	11.5	14.0	19.0	23.1	28.0
110	11.5	14.0	16.9	23.0	27.9	33.8
120	13.7	16.6	20.1	27.4	33.2	40.2
130	16.1	19.5	23.6	32.1	39.0	47.2
140	18.6	22.6	27.4	37.2	45.2	54.8
150	21.4	25.9	31.4	42.7	51.9	62.9
160	24.3	29.5	35.8	48.6	59.0	71.6

For other values of  $C_f$  multiply wind load for  $C_f = 1.3$  value by  $C_f/1.3$

Where guard ends without a return the wind forces may be as much as 1.667 times  $C_f=2.6$  value.

MINIMUM WIND LOAD TO BE USED IS 10 PSF.

**GLASS STRENGTH**

All glass is fully tempered laminated glass conforming to the specifications of ANSI Z97.1, ASTM C 1048-97b and CPSC 16 CFR 1201. For the two ply 9/16" glass the minimum Modulus of Rupture  $F_r$  is 24,000 psi.

Allowable glass bending stress for live loads:  $24,000/4 = 6,000$  psi. – Tension stress calculated. For wind loads the allowable stress in ASTM E1300-12a may be used - Maximum edge stress of 10,600 psi; however, recommend limiting to 9,600 psi because of support conditions.

Determine effective thickness of the laminated glass for stresses and deflections based on ASTM E1300-12a appendix X9.

For SGP interlayer G from SentryGlas Plus product data published by DuPont/Kuraray.

The values of G are selected as most appropriate for service conditions and load durations.

$$h_1 = h_2 = 0.219''$$

$$h_v = 0.06''$$

a = least width - typically total glass height including portion in base shoe: 41" for 42" overall height including base shoe.

$$h_s = 0.5(h_1 + h_2) + h_v = 0.5(0.219 * 2) + 0.06 = 0.279''$$

$$h_{s;1} = h_{s;2} = (h_s h_1) / (h_1 + h_2) = (0.279 * 0.219) / (2 * 0.219) = 0.1395''$$

$$I_s = h_1 h_{s;2}^2 + h_2 h_{s;1}^2 = 2 * (0.219 * 0.1395^2) = 0.00852$$

$$\Gamma = 1 / [1 + 9.6(EI_s h_v) / (G h_s^2 a^2)]$$

effective thickness for deflection:

$$h_{ef;w} = (h_1^3 + h_2^3 + 12\Gamma I_s)^{1/3}$$

effective thickness for glass stress:

$$h_{1;ef;\sigma} = [h_{ef;w}^3 / (h + 2\Gamma h_s)]^{1/2}$$

$$M_{aL} = 6,000 \text{ psi} * 2 * h_{1;ef;\sigma}^2 = 12,000 h_{1;ef;\sigma}^2 \quad \text{“#/ft} = 1,000 h_{1;ef;\sigma}^2 \quad \text{“#/ft} \quad \text{For Live Loads}$$

$$M_{aW} = 9,600 \text{ psi} * 2 * h_{1;ef;\sigma}^2 \quad \text{For Wind Loads}$$

Exterior installations assumed.

For SentryGlas interlayer use  $G = 460$  psi (3.6 MPa)

(from DuPont SentryGlas *Effective Laminate Thickness for the Design of Laminated Glass* based on 140°F, (60°C) and short term load duration)

For cantilevered elements basic beam theory for cantilevered beams is used.

$$M_w = W * L^2 / 2 \quad \text{for uniform load } W \text{ and span } L \text{ or}$$

$$M_p = P * L \quad \text{for concentrated load } P \text{ and span } L,$$

$$\Delta = (1 - 0.222) * w / 12 * h^4 / (10,400,000 * h_{ef;w}^3) \quad \text{for wind load}$$

$$\Delta = (1 - 0.222) * 50 * h^3 / (3 * 10,400,000 * h_{ef;w}^3) \quad \text{for 50 plf live load load}$$

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Table 2	$h_1, h_2$	$h_v$	$h_{s;1} h_{s;2}$	$l_s$	$h_s$		
6mm	0.219	0.06	0.1395	0.0085	0.279		
6mm	0.219	0.06	0.1395	0.0085	0.279		
Shortest Dimension		$\Gamma$ SGP	$h_{ef;w}$ SGP	$h_{1;ef;\sigma}$ SGP		All. live load mom. lb-in/ft	All. wind mom. lb-in/ft SGP
12		0.0917	0.3121	0.3525	1410		2634
24		0.2877	0.3695	0.4105	1913		3573
36		0.4761	0.4116	0.4451	2249		4200
41		0.5410	0.4242	0.4543	2343		4375
48		0.6177	0.4383	0.4638	2442		4561
60		0.7163	0.4551	0.4744	2555		4772
72		0.7843	0.4660	0.4808	2625		4902

Minimum glass thickness from ASTM C1036. If thicker glass is used in fabricating the laminated glass greater effective thicknesses may be calculated based on actual glass thickness. Allowable glass stress reduced to 5,676 psi based on physical tests (see page 9 of 24).

#### GLASS PANELS LOADS:

From IBC 1607.7.1

At top – 200lb concentrated or 50 plf Any direction

Or On panel – 50 lbs on one square foot

Or Wind load on entire area; 10 psf minimum

#### DETERMINE MAXIMUM PANEL HEIGHT:

For 50 plf distributed load:

$$h = (M_{aL}/u) = M_{aL}/50\text{plf}$$

For 200# load, on top rail/ top of glass:

$$h = M_{aL} * S / 200\# \text{ where } S = \text{light length in feet when installed with cap rail}$$

For installation without a cap rail and load at corner of glass:

$$h = M_{aL} * (2/3 * S) / 200\# \text{ where } S \leq h$$

For wind load

$$h = (M_{aw} / (0.55W))^{1/2}$$

maximum wind load for given light height:

$$W = M_{aw} / (0.55h^2)$$

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Determine height at which wind load will control over 50 plf top load:

$$M_{aL} = 50plf \cdot h = (W \cdot 0.55h^2) / 1.6$$

1.6

Solve for h:

$$h = 145.5/W$$

or solve for W:

$$W = 145.45/h$$

or

$$W \cdot h = 145.45$$

Relationship of wind to height where wind load controls over 50 plf top load (See graph)

Below line 50 plf top load will control design.



Glass thickness and light width must be adequate to support the imposed load.

For 200 lb concentrated load

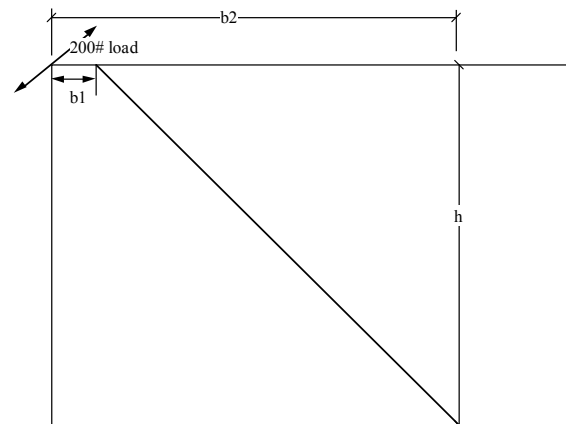
Worst case is load at end of light top corner with no top rail:

The load will be initially resisted by a strip = 8t

For 9/16" glass = 4.48"

The shear will transfer along the glass at a 45° angle to spread across the panel. - Deflection continuity of the glass requires that load be transferred across the full width with decreasing load as it gets farther from the corner.

$$b_2 = b_1 + h$$



$M_{ave} = 200 \cdot h / (b_2)$  average moment.

Peak moment at free edge will be greater based on triangular loading along strip considered and glass beyond assumed width carries no loading.

$$M_{min} = (1/2)M_{max}$$

$$M_{ave} = (M_{max} + M_{min}) / 2 = (M_{max} + (1/2)M_{max}) / 2 = (3/2)M_{max} / 2 = (3/4)M_{max}$$

$$M_{max} = 4/3 M_{ave} = 1.3333 \cdot 200 \cdot h / (b_2) \leq 1000t^2 \text{ (live load allowable stress)}$$

Rearranging and simplifying:

$$h \leq 3.75 \cdot b_2 t^2$$

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**LOAD TESTS**

CR Laurence performed in-house tests to verify the glass strength. Tests were performed on 5 glass lights 48" long x 41" tall set into the L56S base shoe with 6 sets of the LTL96X Taper-Loc® 'X' Tapers.

The laminated glass fabricated with 0.06" Sentry Glas+

Glass test loads: - 38" effective glass cantilever height.

Effective thickness for deflection calculated from maximum deflection measured:

$$t = [(P \cdot 38^3) / (\Delta \cdot 3 \cdot 4 \cdot 10.4 \times 10^6)]^{1/3}$$

P = load at deflection,  $\Delta$

**TABLE 3**

Test	Max Load	Defl in	Moment/ft	Defl at 800# load	Eff. thickness Defl - Inches	Glass stress at failure	Comment
1	851	4.5	8403.625	4.250	0.4358	22125	at failure
2	928	4.38	9164	3.750	0.4544	22195	at failure
3	886	4.5	8749.25	4.125	0.4401	22581	No failure
4	898	4.625	8867.75	4.375	0.4316	23802	at failure
5	886	4.875	8749.25	4.125	0.4401	22581	No failure
<b>Ave</b>	889.8	4.576	8786.775	4.125	0.4404	22657	

The tests confirm that the glass will meet the safety factor of 4 for live loads based on the 200 lb concentrated load and 50 plf uniform load (equivalent loads for the 4' long lights tested.)

Note on strength - The average modulus of rupture of the lights that failed (3) is 22,707 psi. The other two lights didn't fail so modulus of rupture can't be determined but would exceed this. Average modulus of rupture versus 24,000 psi based on direct testing for tempered soda glass: %MR = 22,707/24,000 = 0.946: 5.4% under. This is within the expected range of  $\geq 20,000$  psi.

**FOR INSTALLATION WITHOUT A TOP RAIL - NO FALL PROTECTION REQUIRED**

Maximum glass cantilever height based on light width for 200 lb live load and no top rail:

Also verify for 50 plf live load-  $h \leq 4500 * 2 * t^2 / 50 = 180t^2$  (allowable stress reduced for this use).

<b>TABLE 4</b>	<b>Effective thickness SGP</b>	<b>200# Live Load Max height inches SGP</b>	<b>50 PLF Live Load Max height inches SGP</b>
<b>Light width inches</b>			
12	0.353	5.6	22.4
24	0.411	15.2	30.3
36	0.445	26.7	35.7
41	0.454	31.7	37.1
45	0.464	36.3	38.7
66	0.464	39.7	38.8
73	0.464	39.7	38.8

Limit effective thickness to value for 45” width.

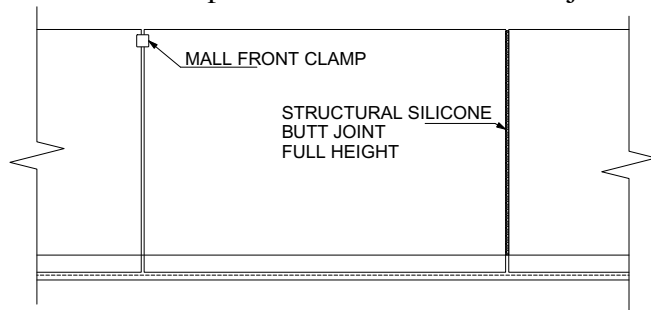
For 42” wind screen height - required glass cantilever height:

For height inclusive of base shoe  $h_g = 38.5”$

For height above base shoe  $h_g = 42.5”$  (42” clear glass height above top of base shoe).

**For installations without a top rail the differential deflection of glass lights must be checked based on 200 lb concentrated load on one light. Where deflection exceeds 9/16” the lights must be connected together at the joints to limit differential deflection. Recommend using mall front clamps, H clip or similar within 12 inches of the top of the glass.**

Mall front clamp or structural silicone butt joint full height.



**POOL FENCE**

When installed as a pool fence the live loads are assumed as acting at 42” above finish floor.

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**FOR INSTALLATIONS WITH A TOP RAIL:**

Top rail is assumed to have adequate stiffness to distribute load across length of light

Determine Minimum light length: S (ft) for height h (ft) :

$$M_{aL} = S_{yt} * 6,000\text{psi} = B * 2t^2 * 6,000\text{psi} \geq 200h$$

$$B_{min} = 200h / (12,000 * t^2) = h / (60t^2)$$

$B_{min}$  is minimum length in feet

h is cantilever height in inches

For lights smaller than the minimum required top rail must be continuous to additional supports such as wall, post or larger glass lights on each side.

For SGP Interlayer

Maximum allowable ht for SGP interlayer

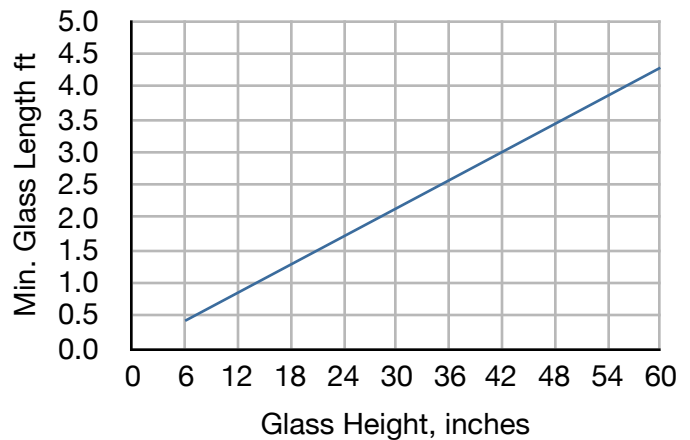
$$h \leq 2,952'' \# / f / 50\text{plf} = 59'' \text{ (glass cantilever height in inches)}$$

Minimum glass length:

For SGP interlayer

$$B_{min} = h / (60 * 0.483^2) = h / 14.0$$

**Minimum Length to height SGP**



FOR 9/16" LAM. GLASS:

Determine relationship between allowable wind load ASD and wind screen height:

NOTES:

Base Shoe anchorage may limit wind loads to less than that allowed by the glass strength.

Specifier shall be responsible to determine applicable load cases and wind load.

For SGP interlayer

$h_{ef;\sigma} = 0.483''$  typical

$M_{wa} = 2 * 0.483^2 * 9,600 = 4,479''\# =$

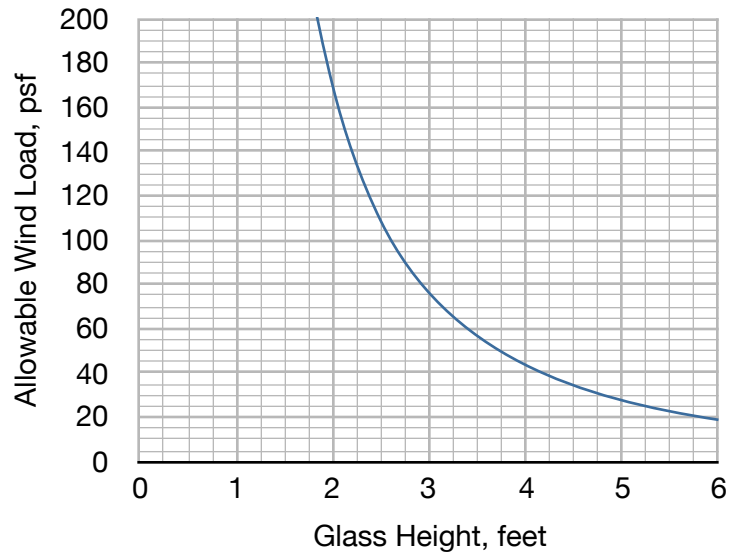
$373.26'\#$

$h = (373.26'\# / ft / (0.55 * W))^{1/2}$

$W = 678.66 / H^2$

H = glass height in feet

**Wind load to Height SGP**



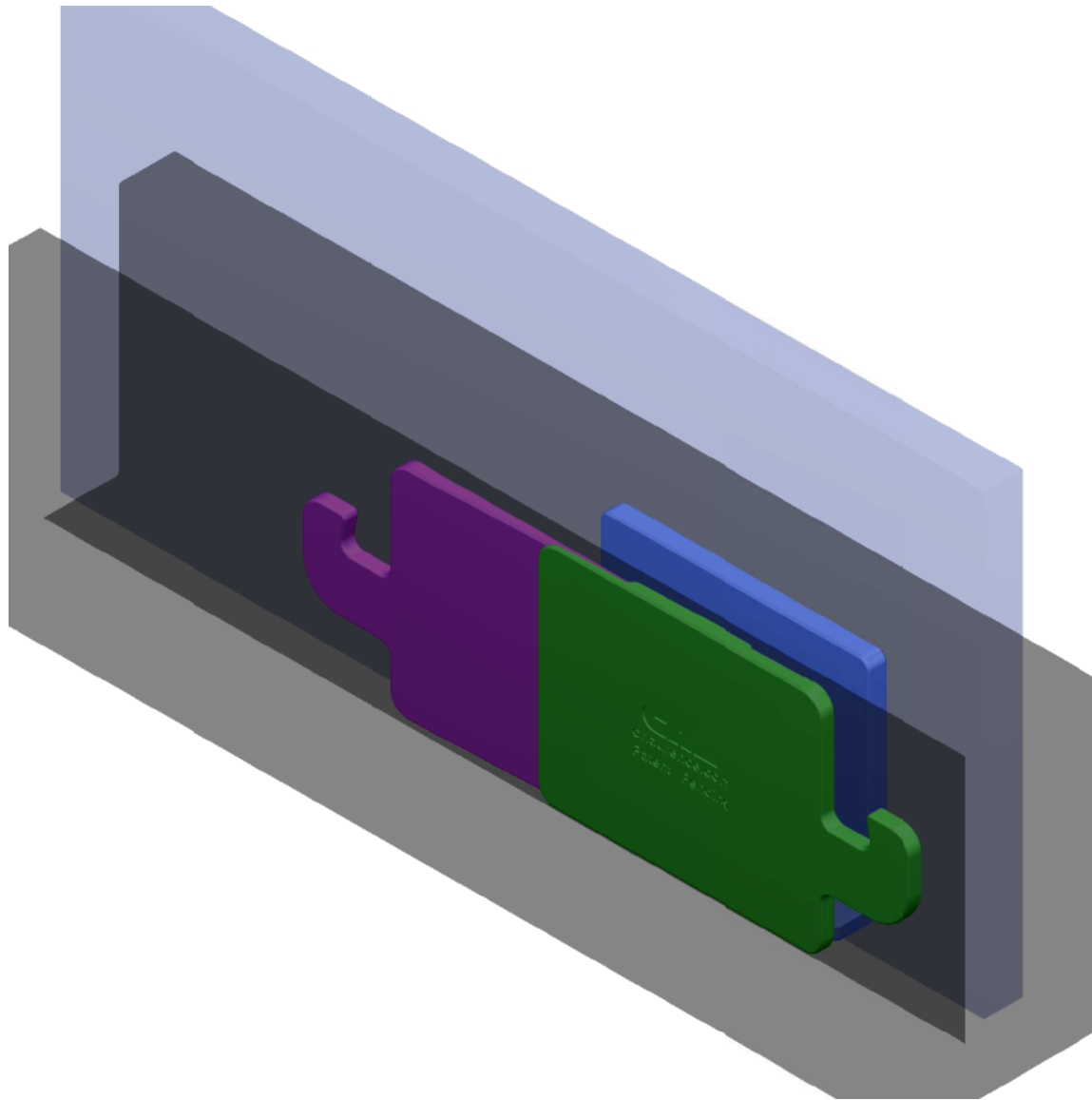
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**DRY-GLAZE TAPER-LOC SYSTEM**



Glass is clamped inside the aluminum base shoe by the Taper-Loc Shoe Setting Plate (L shaped piece on the back side) and two Taper-Loc Shim Plates (front side). The glass is locked in place by the compressive forces created by the Taper-Loc shim plates being compressed together by the installation tool. Use of the calibrated installation tool assures that the proper compressive forces are developed. Until the shim plates are fully installed the glass may be moved within the base shoe for adjustment.

Glass may be extracted by reversing the installation tool to extract tapers.

The Taper-Loc setting plate is bonded to the glass by adhesive tape to hold it in place during installation and to improve glass retention in the base shoe.

Surface area of the setting plate adhered to the glass:

$$A = 2'' * 2.5'' = 5 \text{ in}^2$$

adhesive shear strength  $\geq 80$  psi

3M™ VHB Tape

$$Z = (2/3) * 5 \text{ in}^2 * 80 = 267\# \text{ minimum}$$

setting plate locks into place in the base shoe by friction created by the compression generated when the shim plates are locked into place.

Installation force:

$T_{des} = 250\#''$  design installation torque

$T_{max} = 300\#''$  maximum installation torque

Compressive force generated by the installation torque:

$$C = (0.2 * 250\#'' / 1.0'') / \sin(1.76^\circ)$$

$$C = 1,628\#$$

Frictional force of shims and setting plate against aluminum base shoe:

coefficient of friction,  $\mu = 0.65$

$$f = 2 * (1,628\# * 0.65) = 2,117\#$$

Frictional force of shims against glass:

$\mu = 0.20$

$$f = 1,628 * 0.20 = 326\#$$

Resistance to glass pull out:

$$U = 267\# + 326\# = 593\#$$

Safety factor for 200# pullout resistance =  $2 * 593 / 200 = 5.93$

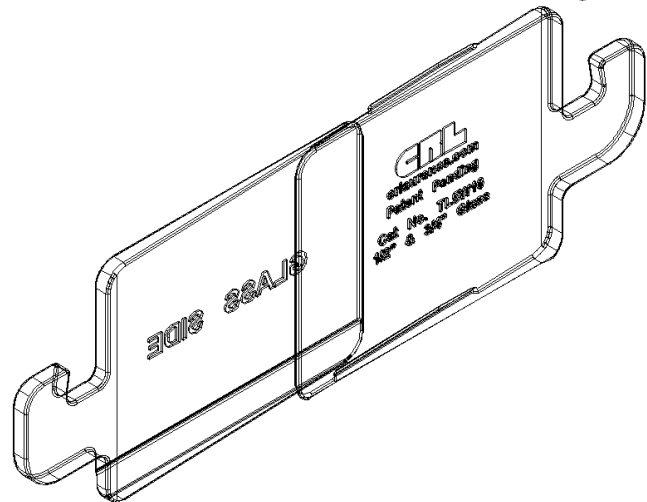
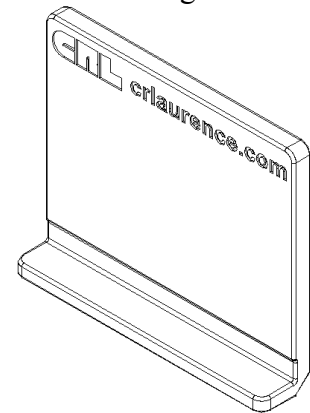
Based on two taper sets

Minimum recommended installation torque:

$$4 / 5.93 * 250 = 169\#''$$

Extraction force required to remove tapers after installation at design torque:

$$T = 250 * (0.7 / 0.2) = 875\#''$$



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Glass anchorage against overturning:

Determine reactions of Taper-Loc plates on the glass:

Assuming elastic bearing on the wedges the reactions will have centroids at approximately 1/6\*3.188" from the upper and lower edges of the bearing surfaces:

$$R_{Cu} @ 1/6 * 3.188 = 0.53''$$

$$e = 3.188 - 0.53 = 2.658''$$

From  $\sum M$  about  $R_{Cu} = 0$

$$0 = M + V * (0.53''/2) - R_{Cb} * (2.658 - 0.53/2)$$

Let  $M = V * 42.5''$  (42" exposed glass height)

$$M_a = 233.3\#'$$
 for 9/16" SGP laminated glass

$$V = 233.3 / 3.33' = 65.9\#$$

substitute and simplify:

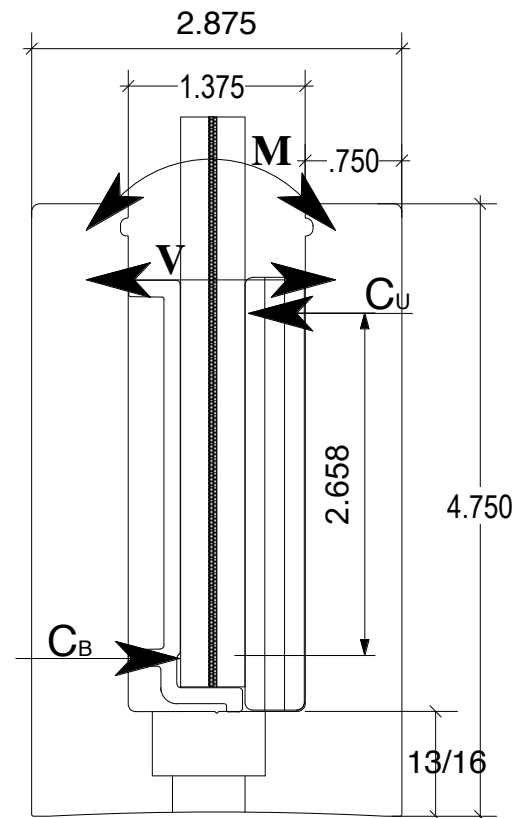
$$0 = V * (42.5'' + 0.265'') - R_{Cb} * 2.393''$$

Solving for  $- R_{Cb}$

$$R_{Cb} = 65.9 * 42.765 / 2.393 = 1,178\#$$

For  $C_B = 3,000$  psi:

$$R_{Cb} = 3.5'' * (3.188''/2) * 3,000 \text{psi} / 2 = 8,369\# > 1,178\#$$



Bearing strength is okay

$$M_a = 8,369 * (1/2 * 3.188'') = 13,340\#''$$

At maximum allowable moment determine bending in base shoe legs:

Bending at bottom of base shoe leg based on maximum allowable Taper-Loc reaction

$$M_i = R_c * [0.188 + (3.188 * 2/3)]$$

$$M_i = 8,369 * (2.313) = 19,360\#''$$

Strength of leg 12" length = 18,668#'' See base shoe calculations later in this report.

Allowable load for Taper-Locs exceeds base shoe strength which exceeds glass strength.

**Allowable moment on system is limited to allowable glass moment for 9/16" laminated glass based on minimum glass dimension and interlayer.**

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### GLASS STRESS ADJUSTMENTS FOR THE TAPER-LOC SYSTEM

The Taper-Loc System provides is a concentrated support:

Stress concentration factor on glass based on maximum 8" glass width to each Taper-Loc set.

Moment concentration factor

Full scale tests and numerous FEA models indicate that there is no appreciable bending stress concentration associated with the concentrated point supports that the Taper-loc system employs. This is because of the purely elastic behavior of the glass for short duration loads up to failure combined with the ratio of the glass height to clear spacing between supports being greater than 2. The glass curvature must be nearly constant across the width of the glass so bending stress must be nearly constant. Thus bending stress will be accurately modeled as constant across the glass width.

$F_b = 6,000$  psi Allowable bending stress based on an SF = 4.0

Shear concentration factor:

Accounts for effect of point support

$$C_V = 8''/3.5''*(2-3.5/8) = 3.57$$

$F_{va} = 3,000$  psi maximum allowable shear stress

Allowable Glass Loads:

$$M_a = S*6,000 \text{ psi}$$

$$V_a = t*b/3.57$$

For 9/16" laminated glass, 12" width:

$$M_a = 2*h_{ef;\sigma}^2*6,000 \text{ for live load}$$

$$V_a = 0.438*12*3,000/3.57 = 4,415\# \text{ for live load}$$

Since shear load in all scenarios is under 10% of allowable it can be ignored in determining allowable bending since it has less than 1% impact on allowable bending loads or rail heights.

Maximum edge distance for edge of glass to centerline of Taper-Loc plates:

$$e_{des} = 8/2 = 4'' \text{ for design conditions (no reduction in allowable loads)}$$

For single < 10" wide light can increase e to 5".



**9/16" LAMINATED GLASS BASE SHOE**

**L56S BASE SHOE**

6063-T52 Aluminum extrusion

Fully tempered glass glazed in place, using the Taper-Loc dry-glazing system.

Shoe strength – Vertical legs:

Glass reaction by bearing on legs to form couple. Allowable moment on legs per 2015 ADM Chapter F.

$$M_a = 1.5SF_y/\Omega_y \text{ or } \leq ZF_u/\Omega_r$$

$$S_y = 12'' * 0.75''^2 * 6 = 1.125 \text{ in}^3/\text{ft}$$

$$Z_y = 12'' * 0.75''^2 * 4 = 1.6875 \text{ in}^3/\text{ft}$$

$$M_{ay} = 16\text{ksi} * 1.5 * 1.125 \text{ in}^3/\text{ft} / 1.65 = 16,364\#''/\text{ft} \text{ or (controls)}$$

$$M_{ar} = 22\text{ksi} * 1.6875 \text{ in}^3/\text{ft} / 1.95 = 19,038\#''/\text{ft}$$

Leg shear strength @ bottom 2015 ADM G.1

$$t_{\min} = 0.75''$$

$$F_{so} = 0.6 * F_{ty} = 0.6 * 16 \text{ ksi} = 9.6 \text{ ksi}$$

$$V_{\text{all}} = 0.75'' * 12''/\text{ft} * 9.6 \text{ ksi} / 1.65 = 52.36 \text{ k/ft}$$

Base shoe anchorage:

Typical Guard design moment = 175#' = 2,100#' or  
(maximum allowable moment) = 211.6' # = 2,539' # Based on glass strength

$$\text{Typical Anchor load} - 12'' \text{ o.c.} - T_a = 2,539\# / (1.4375'') = 1,766\#$$

For 1/2" cap screw to tapped steel, CRL Screw part SHCS12x34 or SHCS12x1

$$T_n = A_{sn} * t_c * 0.6 * F_{tu}$$

where  $t_c = 0.25''$ ;  $A_{sn} = 1.107''$  and  $F_{tu} = 58 \text{ ksi}$  (A36 steel plate)

$$T_n = 1.107'' * 0.25 * 0.6 * 58 \text{ ksi} = 9.63 \text{ k}$$

$$\text{Bolt tension strength} = 0.75 * 67.5 \text{ ksi} * 0.1419 \text{ in}^2 = 7.18 \text{ k}$$

Use 5/16" minimum for maximum load:

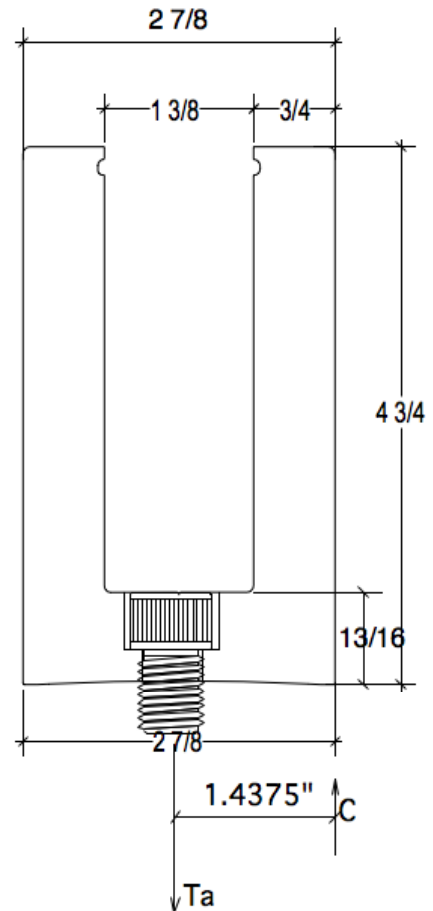
$$\text{Maximum service load: } 7.18\text{k} / 2 = 3,592\#$$

Maximum allowable moment for 12" on center spacing and direct bearing of base shoe on steel:

$$M = 3,592\# * [1.4375'' - 0.5 * 3,592 / (30\text{ksi} * 12)] = 5,146\#'' \text{ per anchor}$$

for 6" o.c.

$$M = 2 * 3,592\# * [1.4375'' - 0.5 * 3,592 / (30\text{ksi} * 6)] = 5,146\#'' \text{ per foot}$$



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**ANCHORAGE TO CONCRETE**

Anchorage designed for concrete with strength  $f'_c \geq 4,000$  psi for cracked condition or  $f'_c \geq 2,500$  psi for uncracked condition. The post-installed concrete anchor strength was determined using the Hilti Profis Anchor 2.4.9 software using the ACI 318-11 Appendix D method. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 1 foot segment of base shoe and supporting concrete. Profis reports are attached as supplements to this report.

Unit loads used in the reports:

$V_u = 80$  lbs (50 plf live load x 1.6 load factor)

$M_u = 80\text{lbs} \times 42'' = 3,360''\#$

Hilti HUS-EZ 3/8" x 4" screw in anchor into 4" deep holes. Installation per ESR-3027.

Nominal embed depth = 3.25"; Effective embed depth = 2.5":

For anchors at 12" on center:

For 4,000 psi cracked concrete:

**3 Tension load**

	Load $N_{ua}$ [lb]	Capacity $\phi N_n$ [lb]
Steel Strength*	2528	6718
Pullout Strength*	N/A	N/A
Concrete Breakout Strength**	2528	2546

\* anchor having the highest loading    \*\*anchor group (anchors in tension)

**4 Shear load**

	Load $V_{ua}$ [lb]	Capacity $\phi V_n$ [lb]
Steel Strength*	80	3111
Steel failure (with lever arm)*	N/A	N/A
Pryout Strength**	80	5483
Concrete edge failure in direction y+**	80	4943

For shear loads less than 20% of strength there is no reduction in the tension load strength:

$V \leq 0.2 \times 3111 = 622\#$  - As this greatly exceeds wind loads can check capacity based only on

For 2,500 psi uncracked concrete

Moment resistance of each anchor:

$\phi M_n = 2,546\# \times [1.4375 - 0.5 \times 2,546 / (2 \times 0.85 \times 2.5\text{ksi} \times 12)] = 3,607\#'' = 300.58\#'$  per anchor

$M_a = \phi M_n / \lambda = 300.58\#'' / 1.6 = 187.86\#'$

For 6" spacing:

$\phi M_n = 2 \times 2,546\# \times [1.4375 - 0.5 \times 2,546 / (2 \times 0.85 \times 2.5\text{ksi} \times 6)] = 7,108\#'' = 592.3\#'$  per anchor

$M_a = \phi M_n / \lambda = 7,108\#'' / 1.6 = 4,442\#'$

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**Installation to wood:**

1/2" x 6" lag screws into solid wood, Douglas Fir or Southern Pine or equivalent density wood.  
 Typical anchor to wood: 1/2" lag screw. Withdrawal strength of the lags from *National Design Specification For Wood Construction* (NDS) Table 11.2A.

For Doug-Fir Larch or denser,  $G = 0.50$

$W = 378\#/in$  of thread penetration.

$C_D = 1.6$  for guardrail live loads (impact loads) and 1.6 for wind loads.

$C_m = 1.0$  for weather protected supports (lags into wood not subjected to wetting).

$T_b = WC_D C_m l_m =$  total withdrawal load in lbs per lag

$W' = WC_D C_m = 378\#/' * 1.6 * 1.0 = 605\#/in$

Determine lag screw thread embedment - assume 1-1/2" thick decking over structural beam/block

Lag screw design strength –  $l_m = 6'' - 13/16'' - 5/16'' - 1.5'' - 1/16'' = 3.31''$

$T_b = 605 * 3.31'' = 2,005\#$

Steel strength =  $60\text{ksi} * A_t / 1.67 = 35.93\text{ksi} * 0.110\text{in}^2 = 3,952\# > 2,005\#$

$Z'_{||} = C_D * Z_{||} = 520\# * 1.6 = 832\#$  per lag, (horizontal load) NDS Table 11K

$Z'_{\perp} = C_D * Z_{\perp} = 1.6 * 320\# = 512\#$  per lag, (horizontal load)

Determine moment strength of anchorage:

For pivoting about edge of base shoe:

Required compression area based on wood strength:

$F_{cT} = 560\text{psi}$ ;  $F'_{cT} * C_d * C_b = 560\text{psi} * 1.33 = 745\text{psi}$

For  $C = T = 2,000\#$

$A = 2,000\# / 745\text{psi} = 2.685\text{in}^2$

$b = A / (12'') = 2.685 / (12) = 0.224''$

$M_a = 2,000\# * (1.4375 - 0.224/2) = 2,651\#'' = 220.9\#'$  For 12" o.c. spacing

$M_a = 2 * 2,000\# * (1.4375 - 2 * 0.224/2) = 4,854\#'' = 220.9\#'$

**NOTE: DO NOT DIRECTLY LAG BASE SHOE TO WOOD WHERE EXPOSED TO WEATHER OR DIRECT SUNLIGHT BECAUSE BASE SHOE WILL LOOSEN WITH TIME AND WILL NOT BE ADEQUATELY ANCHORED.**

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**Summary of surface mounted base shoe strength - Must verify glass strength too.**

Table 5		Allowable wind load in psf						
Surface Mounted	Allowable. Moment in-lbs/ft	Overall Guard height from bottom of base shoe top of top rail, ft.						
Mounting Substrate		3.00	3.25	3.5	3.75	4.0	4.5	5.0
Steel 12" o.c	5146.0	86.6	73.8	63.6	55.4	48.7	38.5	31.2
Steel 6" o.c	10255.0	172.6	147.1	126.8	110.5	97.1	76.7	62.2
Concrete 12" o.c.	2254.0	37.9	32.3	27.9	24.3	21.3	16.9	13.7
Concrete 6" o.c.	4442.0	74.8	63.7	54.9	47.9	42.1	33.2	26.9
Wood 12" o.c.	2651.0	44.6	38.0	32.8	28.6	25.1	19.8	16.1
Wood 6" o.c.	4854.0	81.7	69.6	60.0	52.3	46.0	36.3	29.4

**Fascia Mounted Base Shoe:**

Verify Anchor Pull through on base shoe:

For counter sunk screw

$$P_{nov} = (0.27 + 1.45t/D)DtF_{ty}$$

$$= (0.27 + 1.45 * .5 / .5) * .5 * .5 * 16 \text{ ksi} = 6,880\#$$

For inset bolt

$$t_{min} = 0.25''$$

$$P_{nov} = 0.6 * F_{tu} * (A_v)$$

$$A_v = 0.25'' * \pi * .75'' = 0.589 \text{ in}^2$$

$$P_{nov} = 0.6 * 22 \text{ ksi} * (0.589 \text{ in}^2) = 7.775 \text{ k}$$

$$P_a = 7.775 \text{ k} / 1.95 = 3,987\#$$

For standard installation, 42" guard height and 25 psf max uniform load

Anchor Load  $T_a$

$$T_a = M_a / 2''$$

$$T_a = 2,100\# / 2.125'' = 988.2\#$$

For anchors into steel support:

$$M = 3,592\# * [2.25'' - 0.5 * 3,592 / (30 \text{ ksi} * 12)] = 8,064\# = 672.00 \text{ per anchor}$$

$$M = 2 * 3,592\# * [2.25'' - 0.5 * 3,592 / (30 \text{ ksi} * 6)] = 16,092\# / \text{ft anchors } 6'' \text{ o.c.}$$

For anchor into concrete:

3/8" diameter x 4" Screw-in anchor Powers Wedge-Bolt® (CRL #WBA38X4)

Strength same as previously calculated,

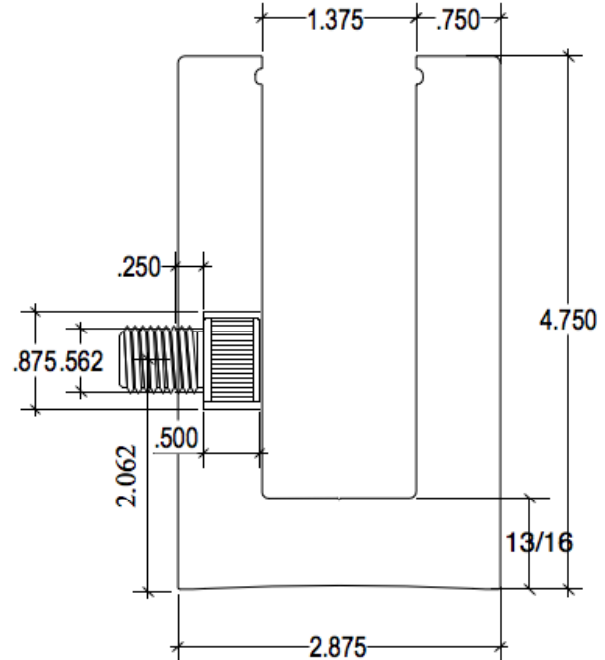
$$M_a = \phi M_n / 1.6 = 2,546\# * [2.25 - 0.5 * 2,546 / (2 * 0.85 * 2.5 \text{ ksi} * 12)] / 1.6 = 3,547\# = 295.6\# \text{ per anchor}$$

$$M_a = \phi M_n / 1.6 = 2 * 2,546\# * [2.25 - 0.5 * 2,546 / (2 * 0.85 * 2.5 \text{ ksi} * 6)] / 1.6 = 7,002\# / \text{ft anchors } 6'' \text{ o.c.}$$

Lag screw strength same as previously calculated.

$$T_a = 2,005\#$$

Note: Fascia mounted base shoe may be directly lagged to wood beam where weather exposed because of reduced wood stresses.



Allowable wind load on balustrade must be reduced for the dead load moment effect

$$V_d = h_g * 6.8 \text{ psf} + 15 \text{ psf}$$

$$M_d = [h_g * 6.8 \text{ psf} + 15 \text{ psf}] * 1.52'' = 10.5 \text{ plf for base shoe and glazing} + 4 \text{ plf for cap rail}$$

$h_g$  = actual height of glass (Typical approx 3.833' for 42" guard height above finish floor)

Assume  $h_g$  = guard height in feet + 0.333'

$$M_d = h_g * 10.3\# / \text{ft} + 22.8\# / \text{ft} = 10.3h + 26.2\#$$

$$V_d = (h + 0.333) * 6.8 \text{ psf} + 15 \text{ psf} = (6.8h + 17.3) \text{ plf}$$

Since the total shear load will typically be less than 20% of the shear strength for steel and concrete installations there is no reduction required for combined shear and tension load on anchors.

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For wood the allowable tension load must be adjusted for the shear loading effects:

$$Z'_a = [(W'p)Z'] / [(W'p)\cos^2 \alpha + Z'\sin^2 \alpha] \quad (\text{NDS 11.4.1})$$

$$\alpha = \tan^{-1}V/T$$

$W'p = 2,005\#$  from previous calculations

$$Z'_\perp = Z_\perp * C_D = 320\# * 1.6 = 512 \quad Z_\perp \text{ from NDS Table 11K for } 1/2'' \text{ lag and } \geq 1/4'' \text{ side plate.}$$

For typical installation with 42" height AFF:

$$V_d = (6.8 * 3.5 + 17.3)plf = 41\#$$

Assume  $T = 2000\#$

$$\alpha = \tan^{-1}2000/41 = 88.83^\circ$$

$$Z'_a = [(2005)512] / [(2005)\cos^2 88.83 + 512\sin^2 88.83] = 2002\#$$

Allowable tension component for 47# shear:

$$T = \sqrt{(2002^2 - 41^2)} = 2002 \geq 2000\# \text{ assumed}$$

Since it would require significant increase in guard height for shear load to be large enough to reduce allowable tension load under 2,000# can assume 2,000# tension load on anchor for determining allowable wind loads:

$$M_a = 2,000\# * (2.25'' - 0.224/2) - 10.3h - 26.2''\# = 4,250''\# - 10.3h$$

$$M_a = 2 * 2,000\# * (2.25'' - 2 * 0.224/2) - 12.6h - 27''\# = 8,104''\# - 12.6h \quad 6'' \text{ o.c.}$$

Allowable wind load for fascia mounted base shoes: Assumes top of base shoe is flush with finish floor:

**Summary of fascia mounted base shoe strength - Must verify glass strength too.**

Table 6		Allowable wind load in psf						
Fascia Mounted	Allowable. Moment in-lbs/ft	Overall Guard height from bottom of base shoe top of top rail, ft.						
Mounting Substrate		3.00	3.25	3.5	3.75	4.0	4.5	5.0
Steel 12" o.c	8064.0	134.8	114.8	99.0	86.2	75.7	59.8	48.4
Steel 6" o.c	16092.0	269.9	230.0	198.3	172.7	151.7	119.9	97.1
Concrete 12" o.c.	3547.0	58.8	50.0	43.1	37.5	33.0	26.0	21.0
Concrete 6" o.c.	7002.0	116.9	99.6	85.8	74.7	65.7	51.8	42.0
Wood 12" o.c.	4250.0	70.6	60.1	51.8	45.1	39.6	31.3	25.3
Wood 6" o.c.	8104.0	135.5	115.4	99.5	86.6	76.1	60.1	48.6

NOTE: The wind load must be checked for the glass based on the specific light size and interlayer. The allowable wind load is the lesser of the anchorage strength or glass strength.

**TABLE 7**

9/16"	EFFECTIVE THICKNESS		SentryGlas+ Interlayer	Allowable wind Pressure, psf for glass height in inches				
	width inches	$t_{\theta}$ for defl.		$t_e$ for stress	All. Moment "#/ft	36	42	48
12	0.3121	0.3525	2386	40.2	29.5	22.6	14.5	10.0
24	0.3695	0.4105	3235	54.5	40.0	30.6	19.6	13.6
36	0.4116	0.4451	3804	64.0	47.0	36.0	23.1	16.0
41	0.4242	0.4543	3963	*	49.0	37.5	24.0	16.7
48	0.4383	0.4638	4130	*	*	39.1	25.0	17.4
60	0.4551	0.4744	4321	*	*	*	26.2	18.2
72	0.4660	0.4808	4438	*	*	*	*	18.7

\* Allowable load is same as last value in column

Calculated from:  $w_{all} = M_{all} * 12 / (0.55 * h_g^2)$

Allowable bending stress of 9,600 psi used

Table 7 applies to wind loads only.

**9B Series - Square, Cored Base Shoe**

6063-T52 Aluminum extrusion

Shoe strength – Vertical legs:

Glass reaction by bearing on legs to form couple.

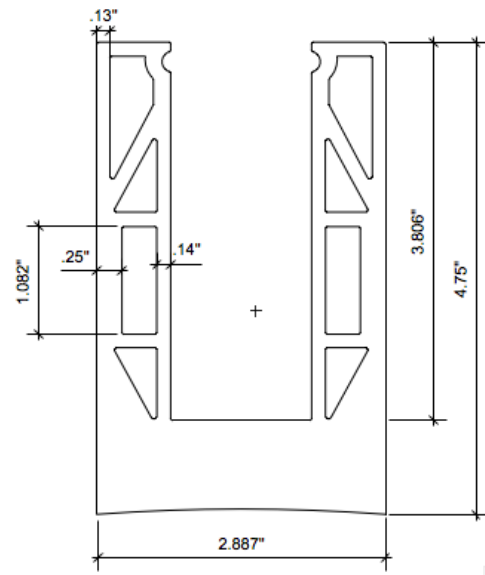
Allowable moment on legs: Same for all widths of 9B series base shoes.

Tension force on inside element will control moment strength of the base shoe legs- 2015 ADM Chapter D At 3<sup>rd</sup> cell - Rectangular cell used for fascia mounted option. Based on yielding as rupture will result in higher allowable load.

Moment resistance across cell

$$M_a = P_{nt} * e / \Omega = A_i * F_{ty} * c / 1.65 =$$

$$0.14'' * 16\text{ksi} * (0.75 - 0.14) / 1.65 = 828''\#\text{' = } 9,937''\#\text{/ft}$$

A<sub>i</sub> = area of inside leg

Allowable shear across cell - based on shear bending across cell legs allowing rotation at top

$$V_a = [1.5(S_i + S_o) * P_{nt} / b] / \Omega$$

S<sub>i</sub>, S<sub>o</sub> = section modulus of inside or outside leg

b = height of cell = 1.082"

$$V_a = [1.5(0.14^2/6 + 0.25^2/6) * 16\text{ksi} / 1.082''] / 1.65 = 1,400 \text{ pli Won't control}$$

Strength at bottom cell

Vertical leg allowable tension load:

$$M_a = P_{nt} * e / \Omega = A_v * F_{ty} * c / 1.65 = 0.14'' * 16\text{ksi} * (0.75 - 0.14) / 1.65 = 828''\#\text{' = } 9,937''\#\text{/ft}$$

A<sub>v</sub> = area of vertical leg, A<sub>d</sub> = Area of diagonal load

Allowable shear across cell:

$$V_a = A_d * F_{ty} / \Omega$$

$$V_a = (0.14 * 16\text{ksi}) / 1.65 = 1,358 \text{ pli} = 16,290 \text{ plf (shear won't control)}$$

Maximum allowable glass shear load reaction on top of base shoe, based on base shoe leg strength:

$$V_a = M_a / B = 9,937''\#\text{/ft} / 3.806'' = 2,611 \text{ plf}$$

Check leg deflection for 3,000''#/ft moment on rail:

Strain in cell walls:

$$\epsilon = (\sigma/E) * B = [(3,000 / (0.14'' * 12'' * 0.61'')) / 10,100,000] * 3.806'' = 0.00107''$$

$$\Delta_\epsilon = (2 * 0.00107'') / (0.75 / 2) = 0.0057''$$

$$\Delta_b = 3,000 * 3.806^2 / (3 * 10,100,000 * 0.75^3) = 0.00339''$$

$$\Delta_\Gamma = \Delta_\epsilon + \Delta_b = 0.0057 + 0.00339 = 0.00909''$$

Glass deflection at 42" above base shoe from base shoe leg deflection

$$\Delta_g = 0.00909 * (42 / 3.806) = 0.10'' \text{ based on } 3,000''\# \text{ glass moment; } 0.069'' \text{ for typical } 50 \text{ plf LL.}$$

**For mounting options, 9B series strength is same as for solid wall base shoes.**

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