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SUBJ: TAPER-LOC® SYSTEM DRY-GLAZE
LAMINATED TEMPERED GLASS RAIL SYSTEM
13/16" (21.52mm) LAMINATED GLASS - L21S and 9BL21 BASE SHOES

The GRS Glass Railing Dry Glaze Taper-Loc™ System utilizes 13/16" (21.52mm) laminated tempered glass (3/8" 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 2021 IBC Section 1607.9, 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 2015, 2018 and 2021 International Building Codes, 2016 and 2020 California Building Codes, 2017 and 2020 Florida Building Code (as wind loading permits) and other state codes adopting the IBC when properly designed by a qualified professional and correctly installed. This report is intended to provide design guidance to said design professional and isn't intended to demonstrate code compliance of any specific installation. Aluminum components are designed in accordance with the 2015 and 2020 Aluminum Design Manuals (ADM). 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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Typical Installations:

Surface or fascia mounted to:

M14 Hex Screw to steel @ 11-13/16" o.c.:

M12 Expansion Anchor to concrete @ 11-13/16" o.c.

1/2" Hilti HUS-EZ screw-in anchor to concrete @ 11-13/16" o.c or @5-7/8" O.C.

1/2" x 6" lag screws to wood (moisture content ≤ 19%) @ 11-13/16" o.c. or @5-7/8" O.C.

Refer to Table 4 on page 22 for surface mounted anchor strength and allowable wind loads or Table 5 on page 27 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 2 for allowable moment for wind loading.

Calculate glass moment based on wind load-

$$M_w = w * h^2 * 0.55 * 12": \text{ in-lb/ft}$$

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.

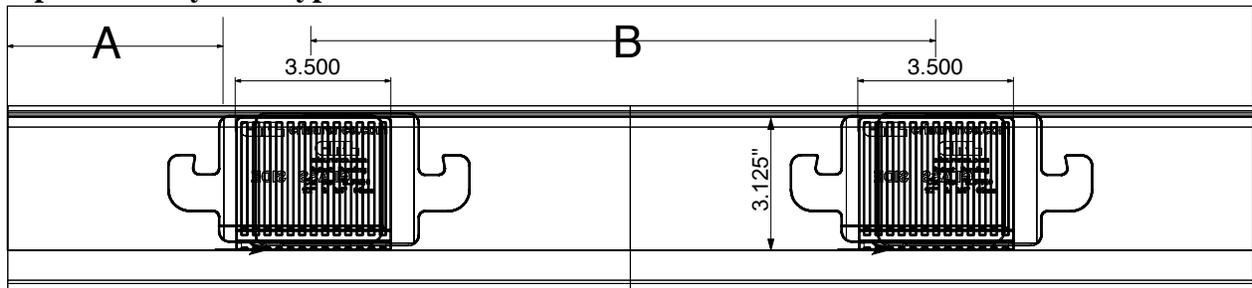
When installed with Taper-Locs® add 0.042 feet (1/2 in) to allow for Taper-Locs® are set below top of base shoe.

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 29) 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 3 (page 10 of 29) 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



For two ply laminated glass with 3/8" Fully Tempered Glass and 1/16" interlayer maximum glass light height is 42":

Edge Distance: $2'' \leq A \leq 8 \frac{5}{8}''$; $51\text{mm} \leq A \leq 219\text{mm}$

Center to center spacing: $7'' \leq B \leq 14''$; $178\text{mm} \leq B \leq 356\text{mm}$

Panel Width/Required quantity of Taper-Loc Plates:

6" to 14" (152 to 356mm)	1 TL Plate
14" to 28" (356 to 711 mm)	2 TL Plates
28" to 42" (711 to 1,067 mm)	3 TL Plates
42" to 56" (1,067 to 1,422 mm)	4 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.

$$A_{\text{max}} = 8 \frac{5}{8} * (42/h)$$

$$B_{\text{max}} = 14 * (42/h)$$

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.

LOAD CASES:

Dead load = 10 psf for glass
 1.8 plf top rail
 3.0 plf for base shoe

Loading:

Horizontal load to base shoe

25 psf*H or W*H

Balustrade moments

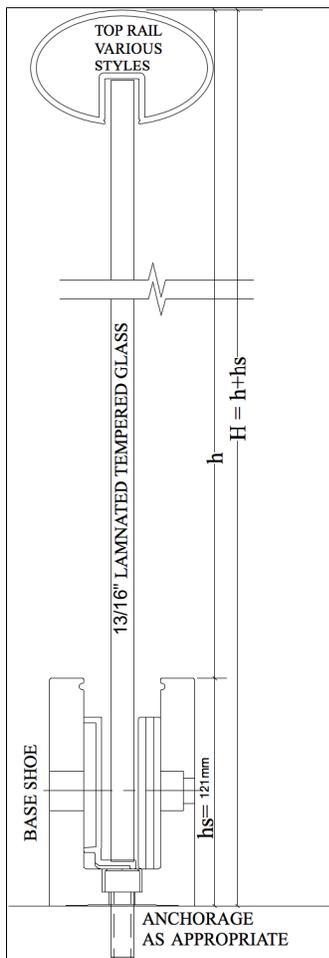
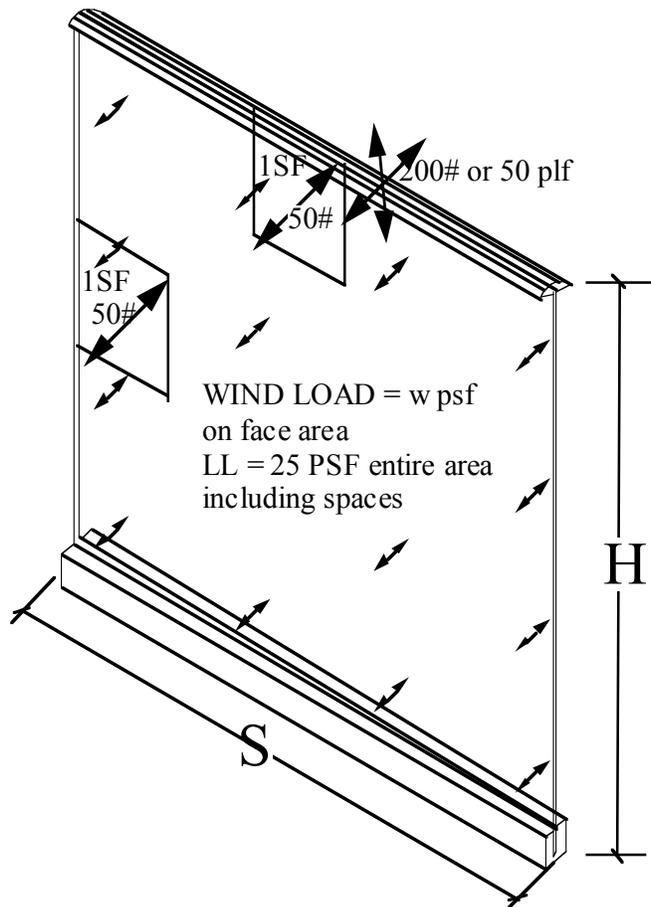
$M_i = 25 \text{ psf} * H^2 / 2$ or

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

For top rail loads:

$M_c = 200\# * H$

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



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 ONLY.
 MINIMUM WIND LOAD IS 10 PSF
 WIND LOADS ARE ALLOWABLE STRESS DESIGN LOADS.
 WIND LOADS CALCULATED AT STRENGTH LEVEL PER ASCE/SEI 7-16 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 3/4"

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WIND LOADING ON FENCES OR GUARDS

Calculated in accordance with ASCE/SEI 7-16 Section 29.3.1. *Design Wind Loads on Solid Freestanding Walls and Solid Signs*. This section is applicable for free standing building guardrails, wind walls and balcony railings that return to building walls. Section 30.8 *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 7-16 eq. 29.3-1)

G = 0.85 from (section 26.11.)

$C_f = 2.5 * 0.8 * 0.6 = 1.2$ (Figure 29.3-1) with reduction for solid and end returns, will vary.

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

K_z from (Table 26.10-1) at the height z of the railing centroid and exposure.

$K_d = 0.85$ from (Table 26.6-1).

K_{zt} From ASCE 26.8 for the site topography, typically 1.0.

V = Wind speed (mph) 3 second gust, (Figure 26.5-1B) 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 7-16 Figure 29.3-1.

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 _{ASD} in psf for C _f = 1.3			W _{ASD} in psf for C _f = 2.6		
Wind speed	Exp B K _z = 0.7	Exp C K _z = 0.85	Exp D K _z = 1.03	Exp B K _z = 0.7	Exp C K _z = 0.85	Exp D K _z = 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 1048-18 and CPSC 16 CFR 1201. For the two ply 13/16" glass the average 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 E1300-16 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 E1300-16 appendix X9.

For interior installations with temperature $\leq 90^\circ\text{F}$

For PVB interlayer $G = 140$ psi

For SGP interlayer $G = 15,600$ psi (SentryGlas Plus product data published by Kuraray)

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

$$h_1 = h_2 = 0.355''$$

$$h_v = 0.06''$$

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

$$h_s = 0.5(h_1+h_2)+h_v = 0.5(0.355*2)+0.06 = 0.415''$$

$$h_{s;1} = h_{s;2} = (h_s h_1)/(h_1+h_2) = (0.415*0.355)/(2*0.355) = 0.208''$$

$$I_s = h_1 h_{s;2}^2 + h_2 h_{s;1}^2 = 2*(0.355*0.208^2) = 0.0306$$

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

effective thickness for deflection:

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

effective thickness for glass stress:

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

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

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

For Exterior installations, assumed for balance of calculations.

For heat and size PVB interlayer shear modulus. $G = 70$ psi ($T \leq 122^\circ\text{F}$)

PVB is not recommended for exterior applications due to exposed glass edges.

For SentryGlas interlayer use $G = 1,640$ psi (11.3 MPa)

(from Kuraray SentryGlas *Effective Laminate Thickness for the Design of Laminated Glass* based on 122°F , (50°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	h₁, h₂	h_v		h_{s;1} h_{s;2}		l_s	h_s	
10mm	0.355	0.06		0.2075		0.0306	0.415	
10mm	0.355	0.06		0.2075		0.0306	0.415	
Shortest Dimension	Γ PVB	Γ SGP	h_{ef;w} PVB	h_{ef;w} SGP	h_{1;ef;σ} PVB	h_{1;ef;σ} SGP	All. wind mom. lb-in/ft PVB	All. wind mom. lb-in/ft SGP
12	0.0094	0.1816	0.4529	0.5384	0.5088	0.6023	5489	7690
24	0.0365	0.4703	0.4686	0.6399	0.5272	0.6901	5892	10096
36	0.0786	0.6664	0.4909	0.6938	0.5525	0.7272	6471	11210
41	0.0996	0.7215	0.5014	0.7075	0.5639	0.7356	6741	11473
48	0.1317	0.7803	0.5165	0.7216	0.5800	0.7440	7131	11734
60	0.1916	0.8473	0.5426	0.7370	0.6064	0.7527	7795	12010
72	0.2544	0.8888	0.5676	0.7462	0.6300	0.7577	8415	12170

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.

GLASS PANELS LOADS:

From IBC 1607.9

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, not top rail:

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

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:

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

Solve for h:

$$h = 145.45/W$$

or solve for W:

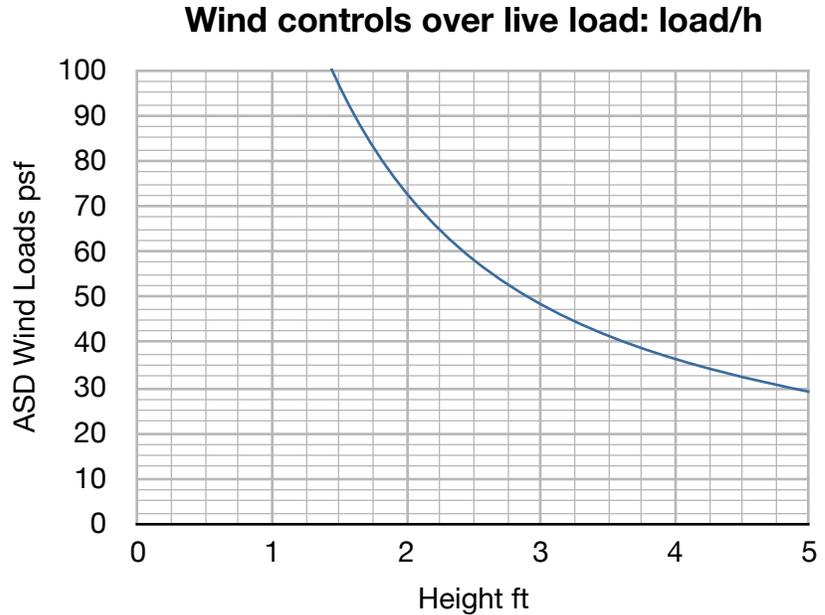
$$W = 145.45/h$$

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

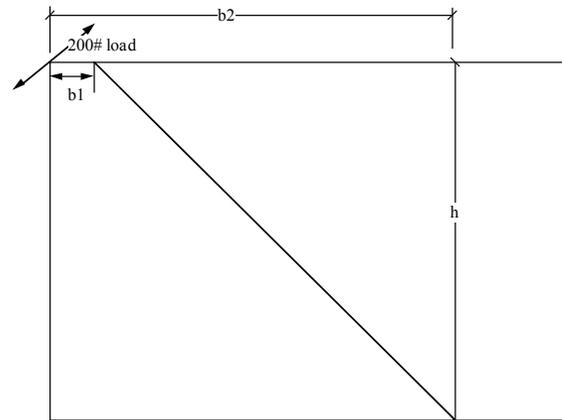
Worst case is load at end of light top corner with no top rail:

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

For 13/16" glass = 6.16"

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.

$$b_2 = b_1 + h$$



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

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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For deflection of glass limited to H/12 (ASTM 2358 limit)

$$H/12 = PH^3/(3EI)$$

$$H = [EI/(4P)]^{1/2}$$

$$I = b_2t^3$$

For 50 plf uniform load:

$$H = [Et^3/(4*50)]^{1/2} = [Et^3/(200)]^{1/2}$$

For 200# load at corner and 1.333 deflection amplification at loaded corner-

$$H = [Ebt^3/(4*1.333*200)]^{1/2} = [Ebt^3/(1066.4)]^{1/2}$$

The ASTM 2358 limit will not control

For 1" deflection limit:

Deflection limit of 1" applied to installations without a top rail for safety reasons.

$$H = [3Et^3/(50)]^{1/3}$$

For 200# load at corner and 1.333 deflection amplification at loaded corner-

$$H = [3Ebt^3/(1.333*200)]^{1/3} = [2.25Ebt^3/(88.89)]^{1/3}$$

FOR INSTALLATION WITHOUT A TOP RAIL

TABLE 3:

Light width inches	Effective thickness PVB	200# LL Maximum height inches PVB†	50 PLF Max height inches PVB*	Effective thickness SGP	200# LL Maximum height inches SGP†	50 PLF Max height inches SGP*
12	0.509	11.7	31.1	0.602	16.3	43.5
24	0.527	25.0	33.4	0.690	42.5	57.1
36	0.552	38.9	36.6	0.727	51.3	62.1
41	0.564	41.5	38.2	0.736	54.2	62.9
48	0.580	45.0	40.4	0.744	57.8	63.6
60	0.606	50.7	44.1	0.753	63.0	64.3
72	0.630	56.0	47.6	0.758	67.4	64.8

Deflection limit of 1” applied.

For 42” guard height - required glass cantilever height:

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

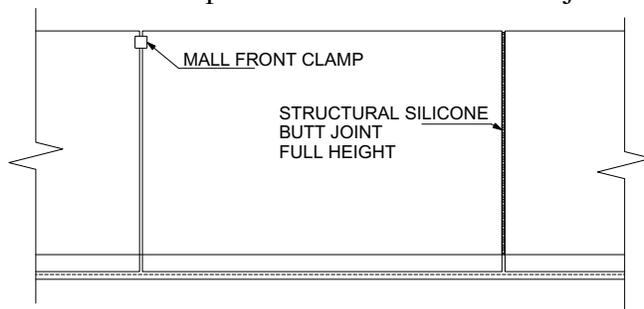
For height above base shoe $h_g = 42.5”$ (42” clear glass height above top of base shoe).

*Maximum allowable height of 50 plf live load above base shoe for code compliance.

†Maximum allowable height based on light width for 200 lb live load and no top rail.

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 3/4” 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 PVB interlayer

$$B_{\min} = h / (60 * t^2) = h / 19.7$$

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 light length:

For SGP interlayer

$$B_{\min} = h / (60 * 0.741^2) = h / 32.9$$

The equations don't include effect of variable effective thickness with respect to length and become more conservative as the length increases.

FOR 13/16” LAM. GLASS:

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

For PVB interlayer

$$h_{ef;\sigma} = 0.573'' \text{ typical}$$

$$M_{wa} = 2 * 0.573^2 * 9,600 = 6,304''\# = 525.3'\#$$

$$h = (525.3'\# / ft / (0.55 * W))^{1/2}$$

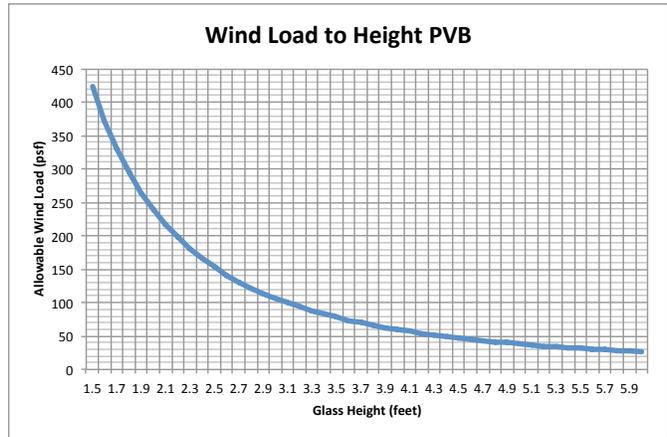
$$W = 955.1 / H^2$$

H = glass height in feet

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

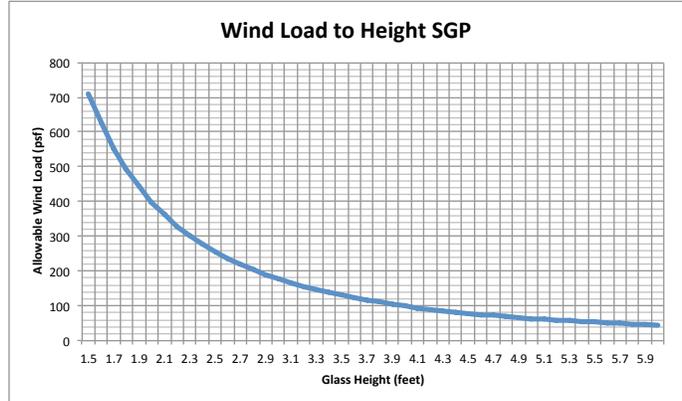
$$h_{ef;\sigma} = 0.741'' \text{ typical}$$

$$M_{wa} = 2 * 0.741^2 * 9,600 = 10,542''\# = 878.5'\#$$

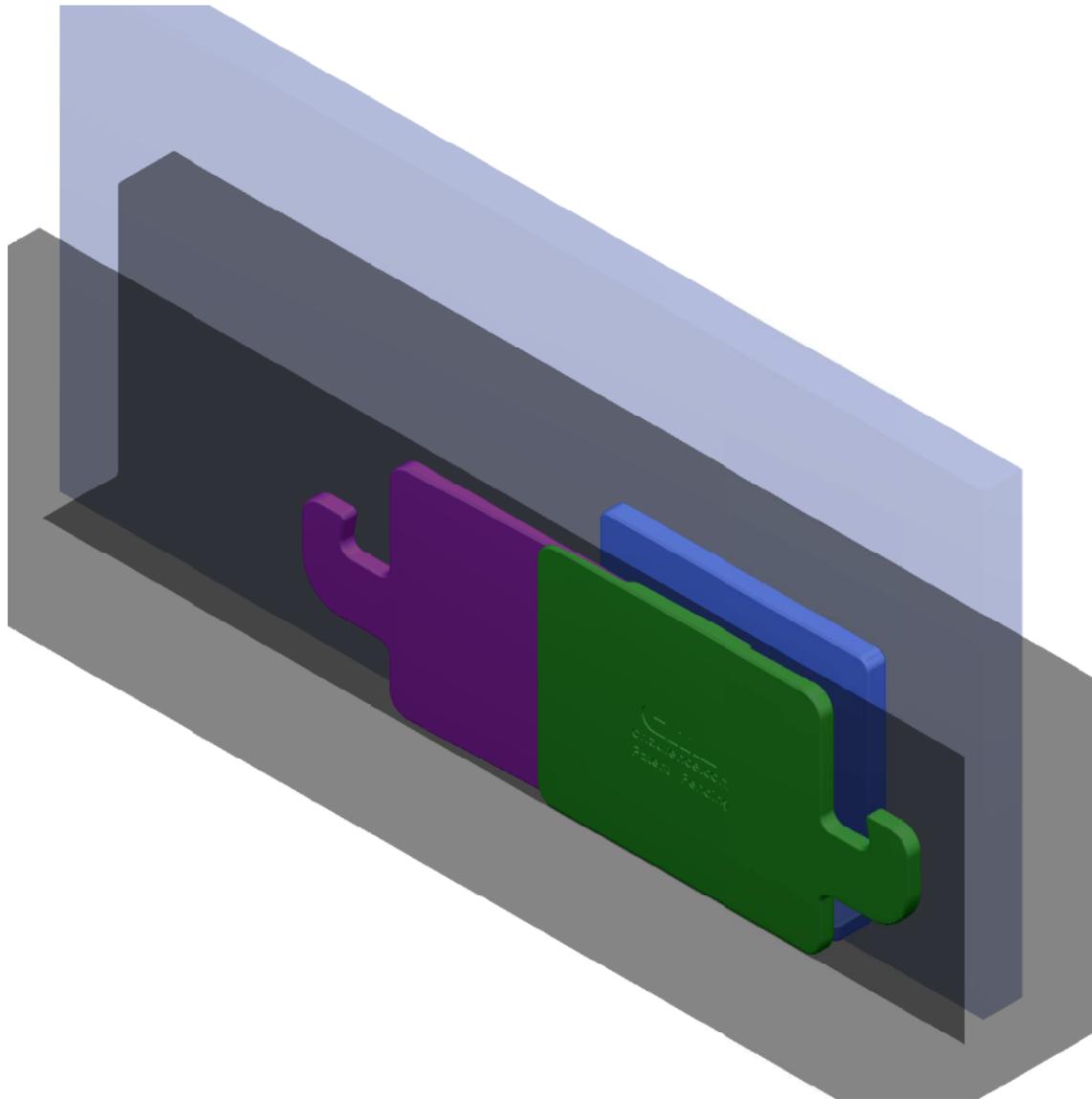
$$h = (878.5'\# / ft / (0.55 * W))^{1/2}$$

$$W = 1,597 / H^2$$

H = glass height in feet



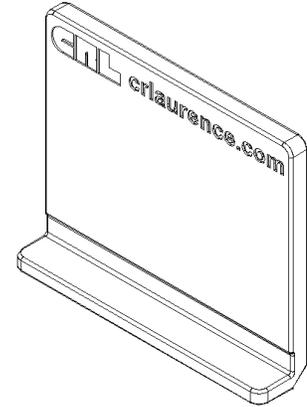
DRY-GLAZE TAPER-LOC SYSTEM



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:

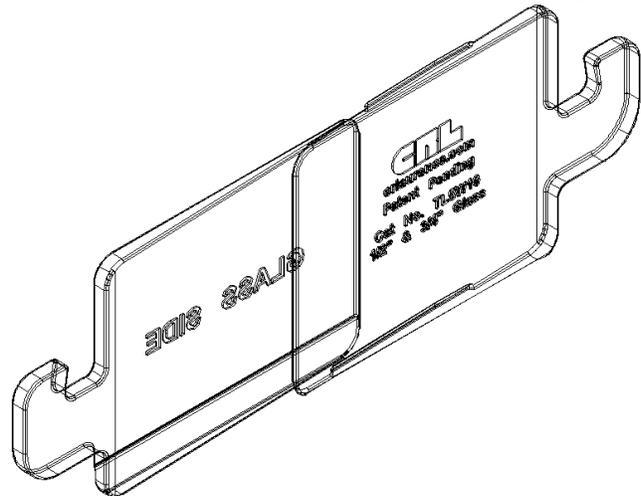
$$A = 2'' * 2.5'' = 5 \text{ in}^2$$

adhesive shear strength ≥ 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:

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

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

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:

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

$$\text{Safety factor for } 200\# \text{ pullout resistance} = 2 * 593 / 200 = 5.93$$

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 13/16" SGP laminated glass

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

substitute and simplify:

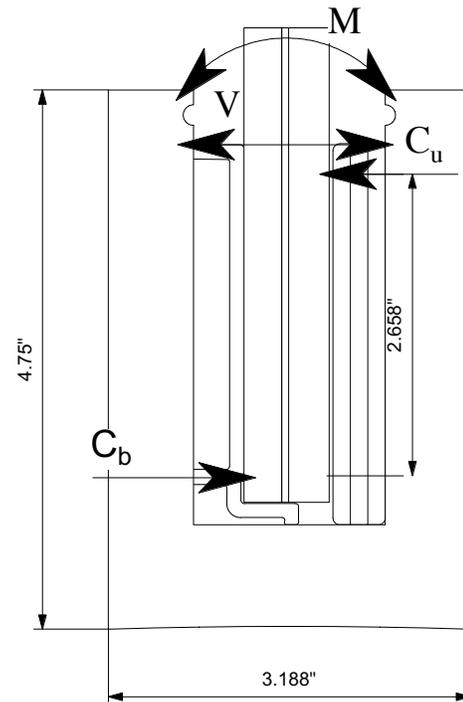
$$0 = V * (42.5'' + 0.265'') - R_{Cb} * 2.393''$$

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 13/16" 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 14" 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

$$C_V = 14''/3.5''*(2-3.5/14) = 7.0$$

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

Allowable Glass Loads:

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

$$V_a = t*b/7.0$$

For 13/16" laminated glass, 12" width:

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

$$V_a = 0.438*12*3,000/7.0 = 2,253\# \text{ for live load}$$

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:

$$e_{des} = 14/2 = 7'' \text{ for design conditions (no reduction in allowable loads)}$$

$$e_{max} = e + e_{des}/2 \text{ and}$$

$$(25*e*3.5') + 25*1.17*3.5^2/2 = 229.6 : \text{ solve for } e$$

$$e_{max} = 3.5'' + [229.6 - 25*1.17*3.5^2/2]/(25*3.5) = 10.4'' \text{ (to CL of Taper-Loc plates)}$$

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**L21S 4-3/4" x 3-3/16" (121mm x 81mm) GLASS
BALUSTRADE BASE SHOE**

FOR 13/16mm LAMINATED GLASS

6063-T52 Aluminum extrusion

Fully tempered glass glazed in place with Taper-Loc-Laminated™ system.

See last page for 9BL21 shoe.

Shoe strength – Vertical legs:

Glass reaction by bearing on legs to form couple.

Allowable moment on legs per 2020 ADM Chapter F.

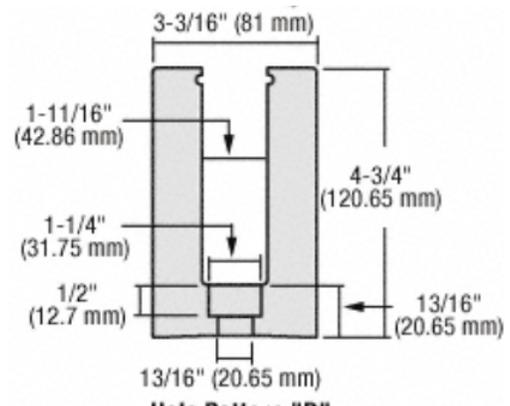
$$M_a = 1.5SF_y/\Omega_y \text{ or } \leq ZF_u/\Omega_r$$

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

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

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

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



Leg shear strength @ bottom 2020 ADM G.1

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

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

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

Maximum moment based on glass strength = 12,181''#/ft ≤ 16,364''#/ft

Maximum base shoe leg deflection:

$$\Delta = Mh^2/(3Et^3) = 12,181 \cdot 3.75^2 / (3 \cdot 10,400,000 \cdot 0.75^3) = 0.013''$$

Glass deflection from leg deflection for 42" glass height above shoe:

$$\Delta_{\text{top}} = 0.013 \cdot 45.75 / 3.75 = 0.16''$$

Base shoe anchorage:

For M14 hex head cap screw to tapped steel

$$T_n = A_{sn} \cdot t_c \cdot 0.6 \cdot F_{tu}$$

where $t_c = 0.25''$; $A_{sn} = 1.2218''$ and $F_{tu} = 58 \text{ ksi}$ (A36 steel plate)

$$T_n = 1.2218'' \cdot 0.25 \cdot 0.6 \cdot 58 \text{ ksi} = 10.63 \text{ k}$$

$$\text{Bolt tension strength} = 0.75 \cdot 67.5 \text{ ksi} \cdot 0.1789 \text{ in}^2 = 9.06 \text{ k}$$

Use 5/16" minimum for maximum load:

$$\text{Maximum service load: } 10.63\text{k} / 2 = 5,330\#$$

Maximum allowable moment for 11- 13/16" on center spacing and direct bearing of base shoe on steel:

$$M = 5,330\# \cdot [1.59375'' - 0.5 \cdot 5,330 / (30\text{ksi} \cdot 11.8125)] = 8,455\#'' \text{ per anchor}$$

For 5.875" o.c.

$$M = 2 \cdot 5,330\# \cdot [1.59375'' - 0.5 \cdot 5,330 / (30\text{ksi} \cdot 5.875)] = 16,828\#'' \text{ per 2 anchors (0.9844')}$$

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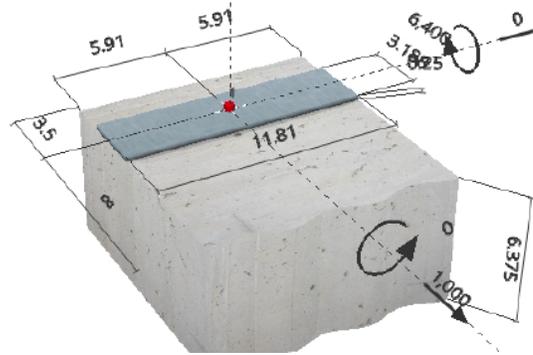
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ANCHORAGE TO CONCRETE

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 according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 11 13/16" segment of base shoe and supporting concrete.



Unit loads used in the reports:

$V_u = (\text{wind or live load} \times 1.6 \text{ load factor})$

M_u

Hilti M12 HSL-3

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

For anchors at 11.81" on center:

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	4398	11397	39	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	4398	4427	100	OK

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

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1000	9571	11	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1000	9534	11	OK
Concrete edge failure in direction y+**	1000	5098	20	OK

* anchor having the highest loading **anchor group (relevant anchors)

For 4,000 psi cracked concrete:

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 tension strength and tension load

For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:

$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$

Maximum moment $M_u = 6,400\#\text{'}$ maximized using the Hilti Profis software

Maximum shear $V_u = 0.2 \times 5,098 = 1,020\#$

$V_a = 1,020/1.6 = 637\#$ (total wind shear load per anchor - approx. 1 foot)

$M_a = 6,400/1.6 = 4,000\#\text{'}$ (total wind load moment per anchor - approx. 1 foot)

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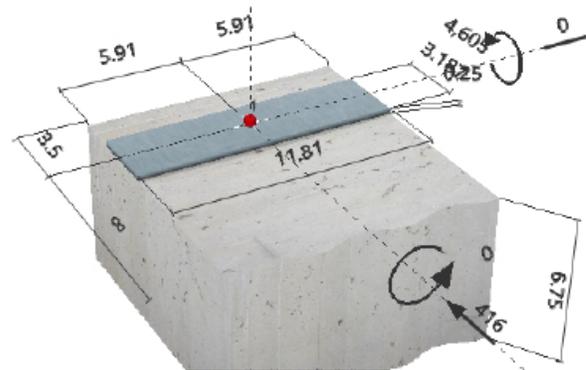
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ALTERNATIVE ANCHORAGE TO CONCRETE

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

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 according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 11 13/16" segment of base shoe and supporting concrete.



Unit loads used in the reports:

$V_u = 1.6$ load factor; M_u

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22"

Minimum concrete thickness = 6.75"

For anchors at 11 13/16" on center: For 4,000 psi cracked concrete:

For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:

$$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$$

Maximum moment found by iteration (outward load controls)

Shear load: $V_u = 416\#$; $V_a = 416/1.6 = 260\#$ per anchor

$$V_a = 260/0.984 = 264 \text{ plf}$$

Moment load: $M_u = 4,605\#\text{'}$; $M_a = 4,605/1.6 = 2,878\#\text{'}$ per anchor

$$M_a = 2,878/0.984 = 2,925\#\text{'}/\text{ft}$$

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	3194	11778	28	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	3194	3194	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	416	5547	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	416	6880	7	OK
Concrete edge failure in direction x-**	416	2083	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.200	1.000	100	OK

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

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FOR HUS ANCHORS AT 5.9" ON CENTER

Same concrete and anchor conditions as 11.81" spacing but with spacing reduced to 5.9".
 Maximum moment found by iteration (outward load controls)

Shear load: $V_u = 201\#$; $V_a = 201/1.6 = 126\#$ per anchor

$V_a = 126/0.4925 = 256$ plf

Moment load: $M_u = 2,795\#\text{'}$; $M_a = 2,795/1.6 = 1,747\#\text{'}$ per anchor

$M_a = 1,747/0.4925 = 3,547\#\text{'}/\text{ft}$

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2009	11778	18	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2009	2009	100	OK

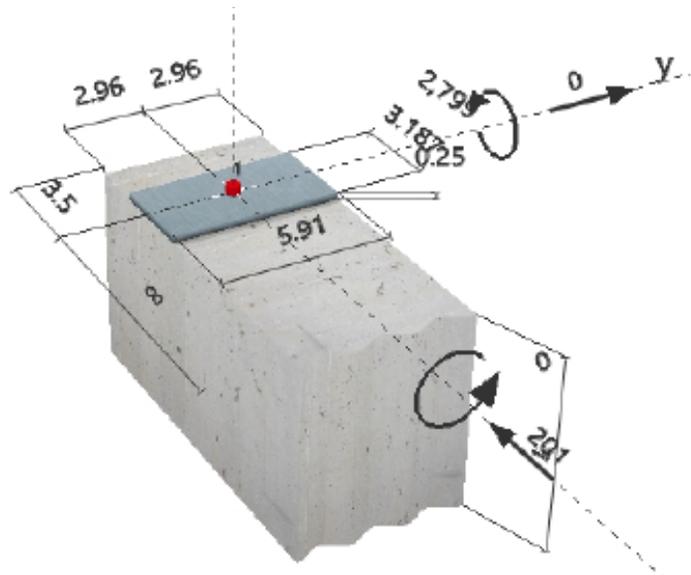
4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	201	5547	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	201	4327	5	OK
Concrete edge failure in direction x-**	201	1021	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.197	1.000	100	OK

$\beta_{N,V} = (\beta_N + \beta_V) / 1.2 \leq 1$



Installation to wood:

1/2" x 6" socket head 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 12.2A.

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).

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

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

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

$Z'_{||} = C_D * Z_{||} = 520\# * 1.6 = 832\#$ per lag, (horizontal load) NDS Table 12K

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

Determine moment strength of anchorage:

For pivoting about edge of base shoe:

Required compression area based on wood strength:

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

For $C = T = 2,005\#$

$A = 2,005\# / 745psi = 2.691in^2$

$b = A / (12") = 2.691 / (12) = 0.224"$

$M_a = 2,005\# * (1.59375 - 0.224/2) * (12/11.8125) = 2,971\#" = 247.55\#'$ For 11-13/16" o.c. spacing

$M_a = (12/5.875) * 2,005\# * (1.59375 - 2 * 0.224/2) = 5,609.56\#" / ft$ for 5-7/8" o.c.

NOTE: DO NOT DIRECTLY LAG BASE SHOE TO WOOD WHERE EXPOSED TO WEATHER 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 4		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 11-13/16” o.c	8455.0	142.3	121.3	104.6	91.1	80.1	63.3	51.2
Steel 5-7/8” o.c	16828.0	283.3	241.4	208.1	181.3	159.4	125.9	102.0
Concrete 12M HSL 11-13/16” o.c.	4000.0	67.3	57.4	49.5	43.1	37.9	29.9	24.2
Concrete 1/2” HUS-EZ 11-13/16” o.c.	2925.0	49.2	42.0	36.2	31.5	27.7	21.9	17.7
Concrete 1/2” HUS-EZ 5-7/8” o.c.	3547.0	59.7	50.9	43.9	38.2	33.6	26.5	21.5
Wood 11-13/16” o.c.	2971.0	50.0	42.6	36.7	32.0	28.1	22.2	18.0
Wood 5-7/8” o.c.	5610.0	94.4	80.5	69.4	60.4	53.1	42.0	34.0

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 - M14

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

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

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

$$P_{nov} = 0.6 * 22 \text{ ksi} * (0.649 \text{ in}^2) = 8,571\#$$

$$P_a = 8,571\# / 1.95 = 4,395\# \leq 5,330\#$$

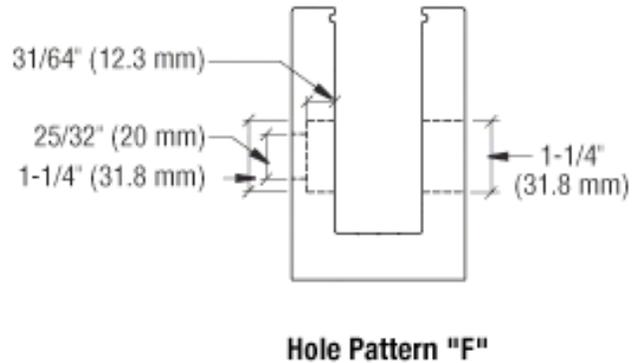
Tear through controls

For M14 anchors into steel support:

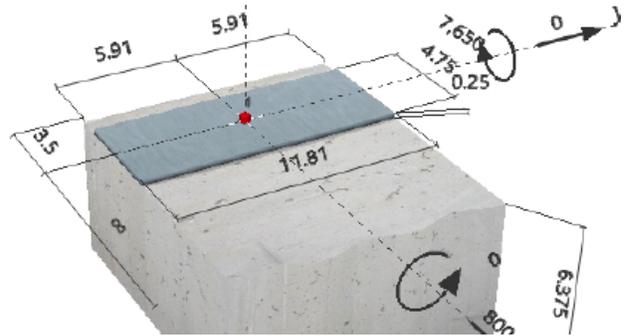
$$M = 4,395\# * [2.25'' - 0.5 * 4,395 / (30 \text{ ksi} * 11.81)] = 9,861''\# = 821.8' \# \text{ per anchor}$$

For 5.875" oc. spacing

$$M = 4,395\# * [2.25'' - 0.5 * 4,395 / (30 \text{ ksi} * 5.875)] = 9,834''\# / \text{anchor}: 19,668''\# / \text{ft}$$



For anchor into concrete - fascia mounted:
 Hilti M12 HSL-3
 Nominal embed depth = 4.134"; Effective embed depth = 3.15";
 Loading optimized using Profis software:



$M_u = 7,650''\#$
 $M_a = 7,650/1.6 = 4,781''\#$
 $V_u = 800\#$
 $V_a = 800/1.6 = 500\#$

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$
Steel Strength*	3476	11397	31
Pullout Strength*	N/A	N/A	N/A
Concrete Breakout Strength**	3476	3500	100

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

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	800	9571	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	800	7538	11	OK
Concrete edge failure in direction y+**	800	4030	20	OK

* anchor having the highest loading **anchor group (relevant anchors)

Allowable wind load on balustrade must be reduced for the dead load moment effect
 $V_d = h_g * 8.3\text{psf} + 15\text{psf}$ (10.5 plf for base shoe and glazing + 4.5 plf for cap rail)
 $M_d = [h_g * 8.3\text{psf} + 15\text{psf}] * 1.52''$
 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'
 $M_d = h_g * 12.6''\#/ft + 22.8''\#/ft = 12.6h + 27''\#$
 Height to reduce allowable wind load moment by 100''# (2% reduction):
 $h = (100 - 27)/12.6 = 5.794'$
 $V_d = (h + 0.333) * 8.3\text{psf} + 15\text{psf} = (8.3h + 17.7)\text{plf}$
 For most cases the dead load will have a minimum impact on the allowable wind load under 2%

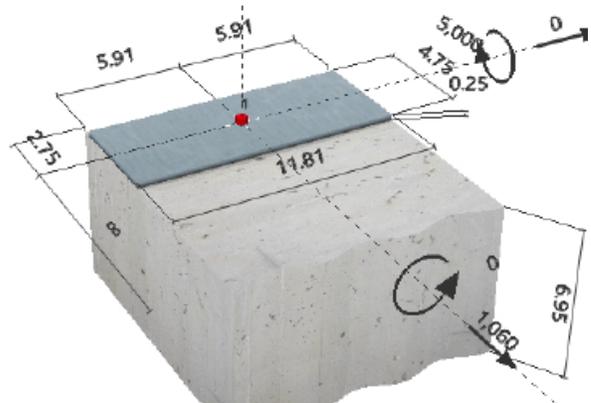
Since the total shear load will typically be less than 20% of the shear strength for steel and concrete installations there is no reduction required for combined shear and tension load on anchors.

Note: For concrete top of base shoe to be 1.625" below top of deck.

ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

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 according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 11 13/16" segment of base shoe and supporting concrete. Unit loads used in the reports:



$V_u = 1.6$ load factor; M_u

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22";

Minimum concrete thickness = 6.75"

For anchors at 11 13/16" on center: For 2,500 psi cracked concrete:

Maximum moment found by iteration (outward load controls)

Shear load: $V_u = 1,060\#$; $V_a = 840/1.6 = 525\#$ per anchor

$V_a = 525/0.984 = 534$ plf

Moment load: $M_u = 5,000''\#$; $M_a = 5,000/1.6 = 3,025''\#$ per anchor

$M_a = 3,025/0.984 = 3,074''\#/ft$

With tension load of $T_u = 320\#$; $T_a = 320/1.6 = 200$

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2759	11778	24	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2759	2759	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1060	5547	20	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1060	5943	18	OK
Concrete edge failure in direction y-**	1060	5301	20	OK

5 Combined tension and shear loads

β_N	β_V	C	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.200	1.000	100	OK

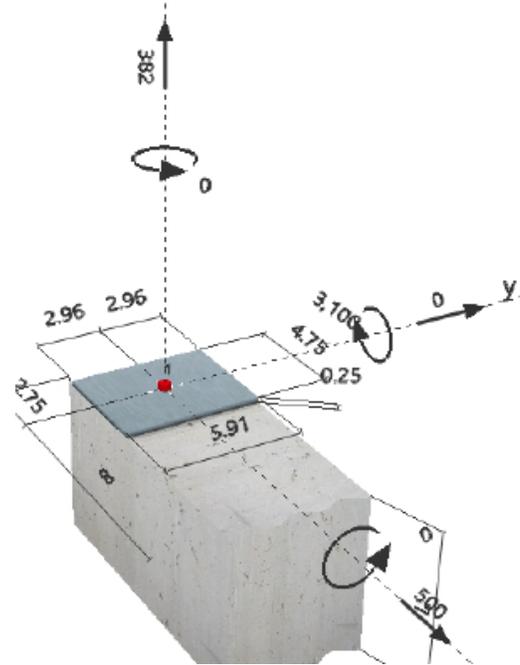
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ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED 5.91" o.c.

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

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 according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 5.91" segment of base shoe and supporting concrete.



Unit loads used in the reports:

$V_u = 1.6$ load factor; M_u

Hilti HUS-EZ (KH-EZ) 1/2" Diameter

Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22":

Minimum concrete thickness = 6.75"

For anchors at 5.91" on center: For 2,500 psi cracked concrete:

Maximum moment found by iteration (outward load controls)

Shear load: $V_u = 454\#$; $V_a = 454/1.6 = 284\#$ per anchor

$V_a = 284/0.4925 = 577$ plf

Moment load: $M_u = 3,100\#\text{ft}$; $M_a = 3,100/1.6 = 1,938\#\text{ft}$ per anchor

$M_a = 1,938/0.4925 = 3,934\#\text{ft}$

With tension load of $T_u = 382\#$; $T_a = 382/1.6 = 239\#$

$T = 239/0.4925 = 485$ plf

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1828	11778	16	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	1828	1829	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	500	5547	10	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	500	3939	13	OK
Concrete edge failure in direction y+**	500	2623	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.191	1.000	100	OK

$\beta_{N,V} = (\beta_N + \beta_V) / 1.2 \leq 1$

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

$$Z'_a = [(W'p)Z'] / [(W'p)\cos^2 \alpha + Z'\sin^2 \alpha] \quad (\text{NDS 12.4.1})$$

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

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

$$Z'_\perp = Z_\perp * C_D = 320\# * 1.6 = 512 \quad Z_\perp \text{ from NDS Table 12K for } 1/2'' \text{ lag and } \geq 1/4'' \text{ side plate.}$$

For typical installation with 42” height AFF:

$$V_d = (9.8 * 3.5 + 18.3)plf = 53\#$$

Assume $T = 2000\#$

$$\alpha = \tan^{-1}2000/53 = 87.62^\circ$$

$$Z'_a = [(2005)512] / [(2005)\cos^2 87.62 + 512\sin^2 87.62] = 1,995\#$$

Allowable tension component for 47# shear:

$$T = \sqrt{(1995^2 - 53^2)} = 1994 < 2000\# \text{ assumed adjust to } 1,994\#$$

Allowable wind loads:

$$M_a = 1,994\# * (2.25'' - 0.224/2) - 14.9h - 28''\# = 4,235''\# - 14.9h$$

$$M_a = 2 * 1,994\# * (2.25'' - 2 * 0.224/2) - 14.9h - 28''\# = 8,052''\# - 14.9h \quad 5-7/8'' \text{ o.c.}$$

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

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

Table 5		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 11-13/16” o.c	9861.0	164.8	140.4	121.0	105.3	92.6	73.1	59.1
Steel 5-7/8” o.c	19668.0	329.9	281.0	242.3	211.0	185.4	146.4	118.6
Concrete 12M HSL 11-13/16” o.c.	4781.0	79.3	67.5	58.1	50.6	44.4	35.1	28.4
Concrete 1/2” HUS-EZ 11-13/16” o.c.	3074.0	50.5	43.0	37.0	32.2	28.3	22.3	18.0
Concrete 1/2” HUS-EZ 5-7/8” o.c.	3934.0	65.0	55.3	47.7	41.5	36.4	28.7	23.2
Wood 11-13/16” o.c.	4235.0	70.1	59.7	51.4	44.7	39.3	31.0	25.0
Wood 5-7/8” o.c.	8052.0	134.3	114.4	98.6	85.9	75.4	59.5	48.2

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 6:

13/16”	EFFECTIVE THICKNESS		PVB Interlayer	Allowable wind Pressure, psf for glass height in inches				
	t _θ for defl.	t _e for stress		All. Moment “#/ft	36	42	48	60
12	0.4529	0.5088	5489	92.4	67.9	52.0	33.3	23.1
24	0.4686	0.5272	5892	99.2	72.9	55.8	35.7	24.8
36	0.4909	0.5525	6471	108.9	80.0	61.3	39.2	27.2
41	0.5014	0.5639	6741	*	83.4	63.8	40.9	28.4
48	0.5165	0.5800	7131	*	*	67.5	43.2	30.0
60	0.5426	0.6064	7795	*	*	*	47.2	32.8
72	0.5676	0.6300	8415	*	*	*	*	35.4

TABLE 7:

13/16”	EFFECTIVE THICKNESS		SentryGlas+ Interlayer	Allowable wind Pressure, psf for glass height in inches				
	t _θ for defl.	t _e for stress		All. Moment “#/ft	36	42	48	60
12	0.5384	0.6023	7690	129.5	95.1	72.8	46.6	32.4
24	0.6399	0.6901	10096	170.0	124.9	95.6	61.2	42.5
36	0.6938	0.7272	11210	188.7	138.7	106.2	67.9	47.2
41	0.7075	0.7356	11473	*	141.9	108.6	69.5	48.3
48	0.7216	0.7440	11734	*	*	111.1	71.1	49.4
60	0.7370	0.7527	12010	*	*	*	72.8	50.5
72	0.7462	0.7577	12170	*	*	*	*	51.2

* Allowable load is same as last value in column

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

9BL 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- 2020 ADM Chapter D

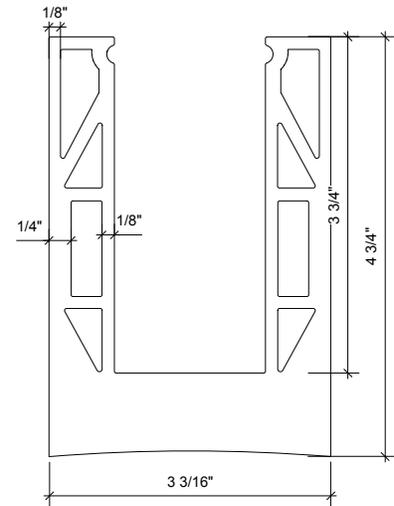
At 3rd cell - Rectangular cell used for fascia mounted option.

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_i = 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_i, S_o = 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_v = area of vertical leg, A_d = 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:

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

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

Strain in cell walls:

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

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

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

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

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

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

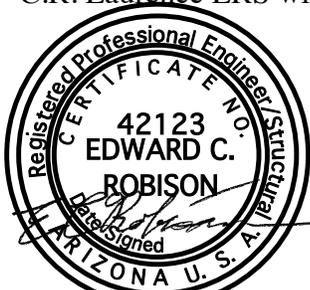
For mounting options, 9B series strength is same as for solid wall base shoes.

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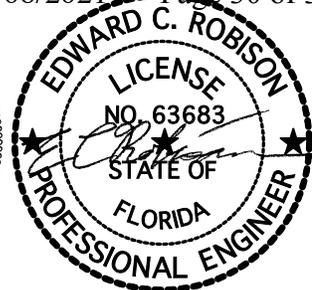
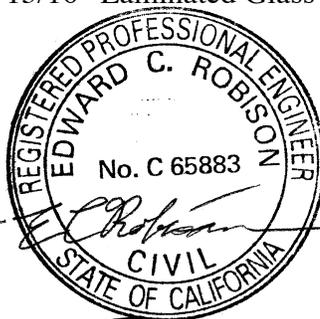
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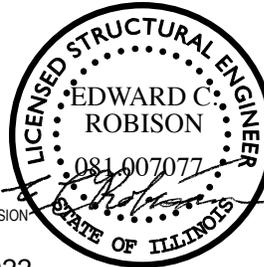
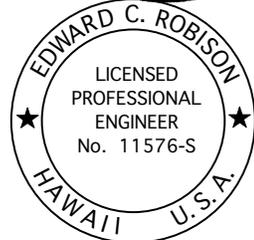
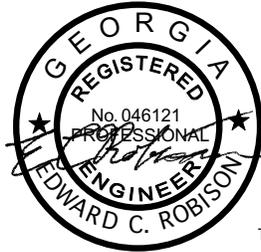
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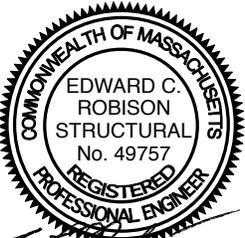
E. C. Robison 04/30/2022

Signature

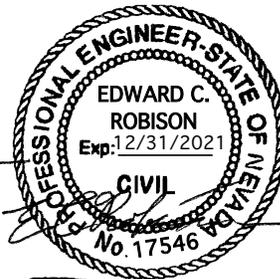
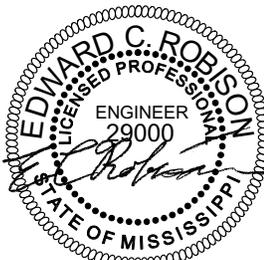
Expiration Date of the License

EXP 11/30/2022

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



EXP 06/30/2022



EXP 03/31/2024

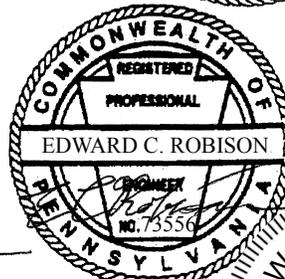
STATE OF MINNESOTA

I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

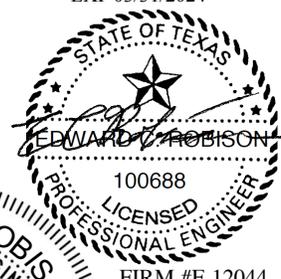
Signature: *E. C. Robison* Typed or printed name: Edward C. Robison
Date: _____ Lic. No. 58604



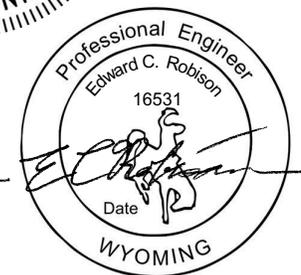
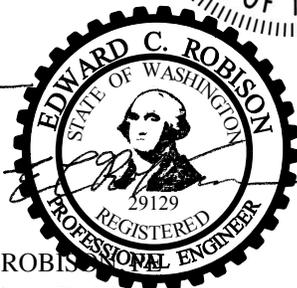
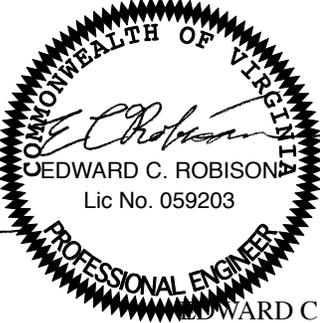
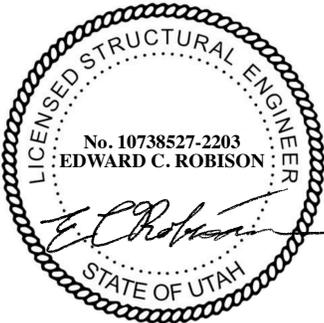
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