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Architectural Railing Division
C.R.Laurence Co., Inc.
2503 E Vernon Ave.
Los Angeles, CA 90058
(T) 800.421.6144
(F) 800.587.7501
www.crlaurence.com

SUBJ: TAPER-LOC® SYSTEM DRY-GLAZE
LAMINATED TEMPERED GLASS RAIL SYSTEM
11/16" (17.52mm) LAMINATED GLASS - L68S AND 9BL68 BASE SHOES

The GRS Glass Railing Dry Glaze Taper-Loc™ System utilizes 11/16" (17.52mm) laminated tempered glass (5/16" 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 1607.9, 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.

Edward Robison, P.E.

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

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” socket head 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-
$M_w = w*h^2*0.55*12”$: 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.

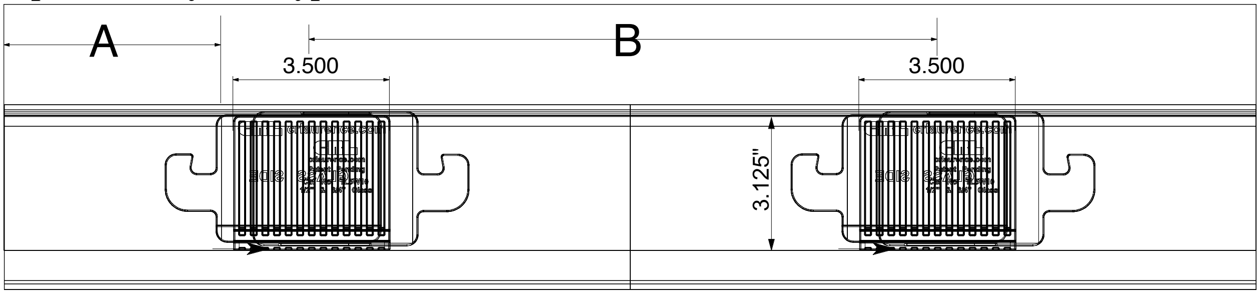
FOR INSTALLATION WITH A TOP RAIL: Maximum glass cantilever height for fall protection is limited to that height at which the glass bending moment does not exceed the allowable glass moments as shown in Table 2 (page 7 of 29) for 50 plf live load or 200 lb concentrated live load being applied at top of 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 9 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.)

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Taper-Loc® System Typical Installation



For two ply laminated glass with 5/16” Fully Tempered Glass and 1/16” interlayer maximum glass light height is 42”:
Edge Distance: 2” ≤ A ≤ 8 5/8”; 51mm ≤ A ≤ 219mm
Center to center spacing: 7” ≤ B ≤ 14”: 178mm ≤ B ≤ 356mm

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_{max} = 8 5/8*(42/h)
   B_{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.

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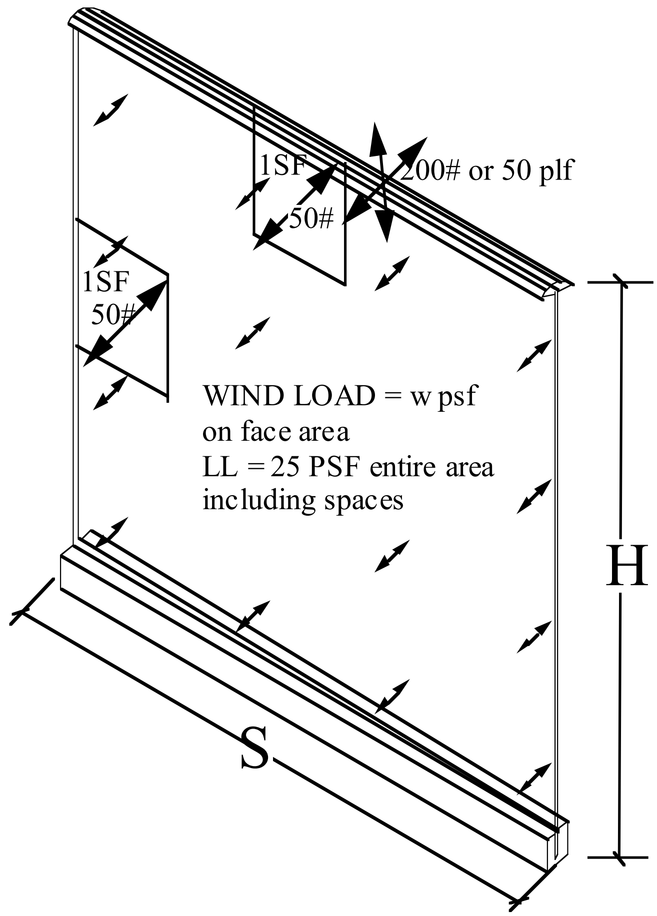
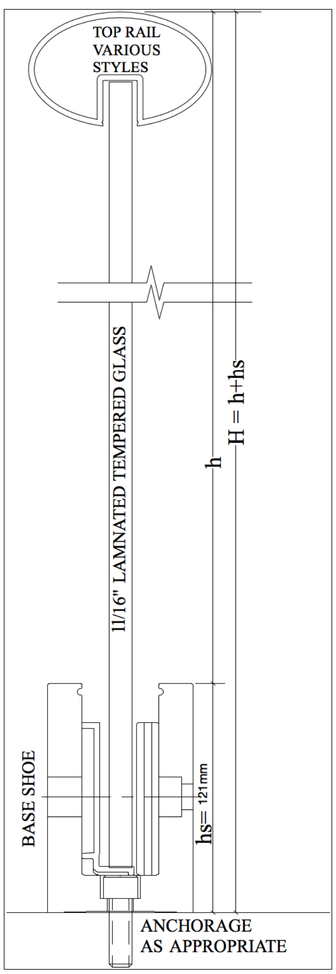
LOAD CASES:

Dead load = 8.3 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
$M_i = 25\ psf*H^2/2$ or
$M_w = w\ psf*\ H^2/2$

For top rail loads:
$M_c = 200\#*H$
$M_u = 50plf*H$





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 5/8”.

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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 (Figure 26.8) for the site topography, typically 1.0.
- V = Wind speed (mph) 3 second gust, (Figure 26.5-1A) or per local authority.

Simplifying - Assuming $1.3 \le C_f \le 2.6$ (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 11/16” 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 E1300-16 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 ≤ 90˚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.
$h_1 = h_2 = 0.292$”
$h_v = 0.06$”
a = least width - typically total glass height including portion in base shoe: 41” for 42” overall height including base shoe.
$h_s = 0.5(h_1+h_2)+h_v = 0.5(0.292*2)+0.06 = 0.352$”
$h_{s;1} = h_{s;2} = (h_s h_1)/(h_1+h_2) = (0.352*0.292)/(2*0.292) = 0.176$”
$I_s = h_1 h^2_{s;2} + h_2 h^2_{s;1} = 2*(0.292*0.176”^2) = 0.0181$
$\Gamma = 1/[1+9.6(EI_s h_v)/(Gh^2_s a^2)]$
effective thickness for deflection:
$h_{ef;w} = (h_1^3 + h^3_2 + 12\Gamma I_s)^{1/3}$
effective thickness for glass stress:
$h_{1;ef;\sigma} = [h_{ef;w}^3/(h+2\Gamma h_{s;1})]^{1/2}$
M_{aL} = 6,000psi*2* $h_{1;ef;\sigma}^2$ = 12,000 $h_{1;ef;\sigma}^2$ “#/ft = 1,000 $h_{1;ef;\sigma}^2$ ‘#/ft For Live Loads
M_{aW} = 9,600psi*2* $h_{1;ef;\sigma}^2$ For Wind Loads

For Exterior installations, assumed for balance of calculations.
For heat and size PVB interlayer shear modulus. G = 70 psi (T ≤ 122 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.
$M_w = W*L^2/2$ for uniform load W and span L or
$M_p = P*L$ for concentrated load P and span L,
$\Delta = (1-0.22^2)*w/12*h^4/(10,400,000*h_{ef;w}^3)$ for wind load
$\Delta = (1-0.22^2)*50*h^3/(3*10,400,000*h_{ef;w}^3)$ for 50 plf live load load

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Table 2	h_1, h_2	h_v		$h_{s;1}$ $h_{s;2}$		I_s	h_s	
8mm	0.292	0.06		0.1760		0.0181	0.352	
8mm	0.292	0.06		0.1760		0.0181	0.352	
Shortest Dimension	**Γ PVB**	**Γ SGP**	**$h_{ef;w}$ PVB**	**$h_{ef;w}$ SGP**	**$h_{1;ef;\sigma}$ PVB**	**$h_{1;ef;\sigma}$ SGP**	All. wind mom. lb-in/ft PVB	All. wind mom. lb-in/ft SGP
12	0.0114	0.2125	0.3739	0.4578	0.4202	0.5114	3743	5544
24	0.0441	0.5191	0.3901	0.5457	0.4394	0.5850	4092	7256
36	0.0940	0.7083	0.4125	0.5883	0.4647	0.6132	4578	7972
41	0.1186	0.7590	0.4227	0.5987	0.4757	0.6194	4798	8135
48	0.1557	0.8119	0.4372	0.6092	0.4910	0.6255	5110	8294
60	0.2237	0.8709	0.4616	0.6205	0.5151	0.6317	5624	8460
72	0.2932	0.9067	0.4841	0.6271	0.5358	0.6352	6086	8555

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

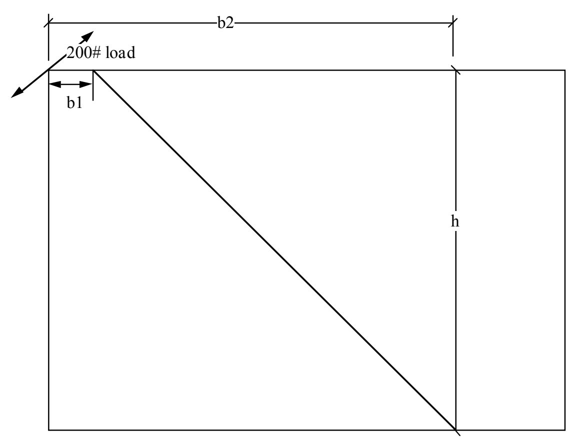
For 200# load, not top rail:
$h = M_{aL}*S/200\#$ where S = light length in feet when installed with cap rail
For installation without a cap rail and load at corner of glass:
$h = M_{aL}*(2/3*S)/200\#$ where $S \leq h$

For wind load
$h = (M_{aw}/(0.55W))^{1/2}$
maximum wind load for given light height:
$W = M_{aw}/(0.55h^2)$

Determine height at which wind load will control over 50 plf top load:
M_{aL}= 50plf*h = (W*0.55h^2)/1.6
Solve for h:
h = 145.45/W
or solve for W:
W = 145.45/h
or
W*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 11/16" glass = 5.152"
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*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.
$M_{min} = (1/2)M_{max}$
$M_{ave} = (M_{max} + M_{min})/2 = (M_{max} + (1/2)M_{max})/2 = (3/2)M_{max}/2= (3/4)M_{max}$
$M_{max} = 4/3M_{ave} = 1.3333*200*h/(b_2) \leq 1000t^2$ (live load allowable stress)
Rearranging and simplifying:
$h \leq 3.75*b_2t^2$

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

EDWARD C. ROBISON, PE
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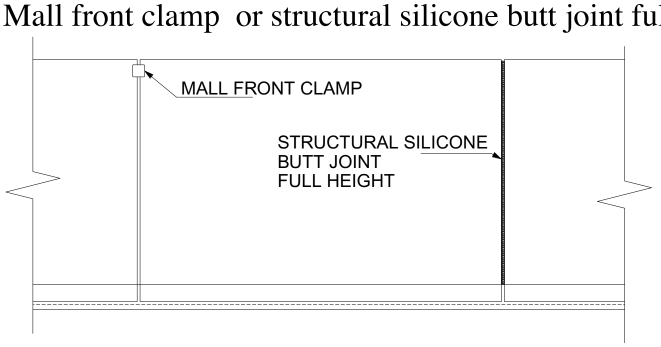
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

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	0.420	7.9	21.2	0.511	11.8	31.4
24	0.439	17.4	23.2	0.585	30.8	41.1
36	0.465	29.2	25.9	0.613	43.3	45.1
41	0.476	34.8	27.2	0.619	45.6	46.0
48	0.491	38.1	28.9	0.625	48.5	46.9
60	0.515	43.1	31.8	0.632	52.9	47.9
72	0.536	47.6	34.5	0.635	56.4	48.4

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 11/16” 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.

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) :

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

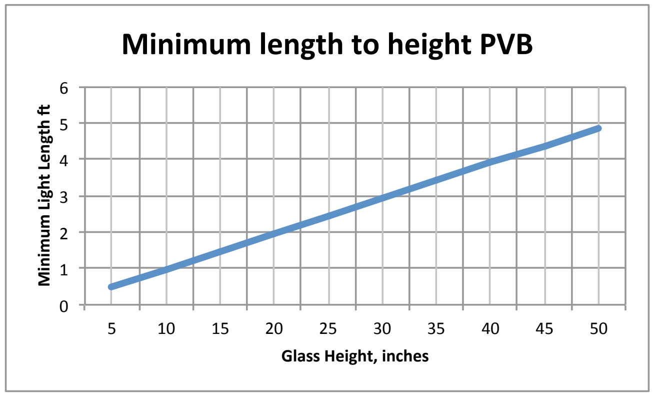
$B_{min} = 200h/(12{,}000*t^2) = h/(60t^2)$

B_{min} is minimum length in feet

h is cantilever height in inches

For PVB interlayer

$B_{min} = h/(60*t^2) = h/8.393$



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

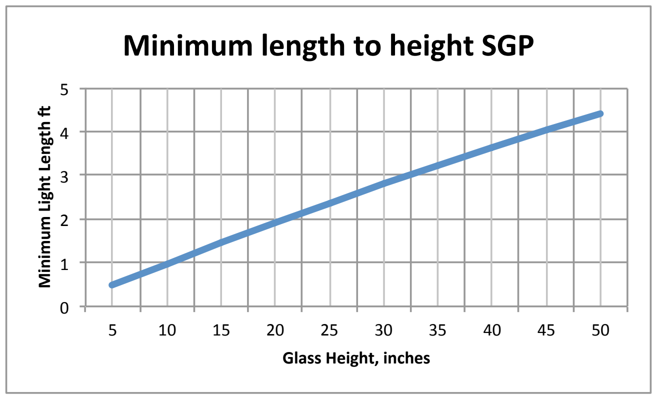
Maximum allowable height for SGP interlayer

h≤ 2,952"#/f/50plf = 59" (glass cantilever height in inches)

Minimum light length:

For SGP interlayer

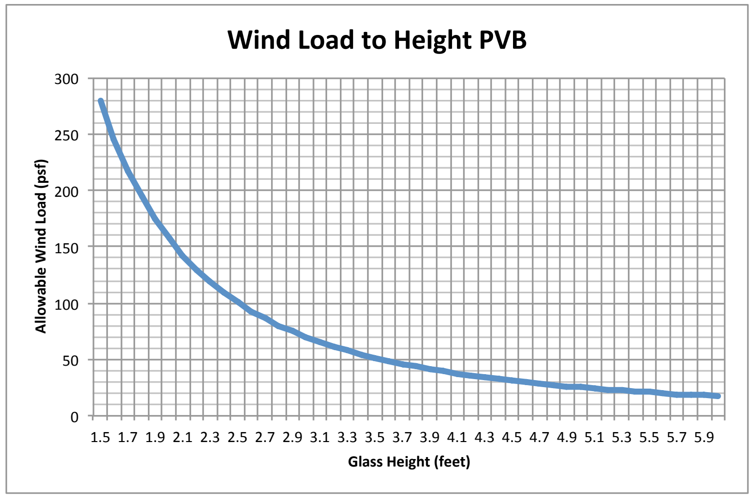
$B_{min} = h/(60*0.483^2) = h/14.0$



Graphs include effect of variable effective thickness with respect to length.

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

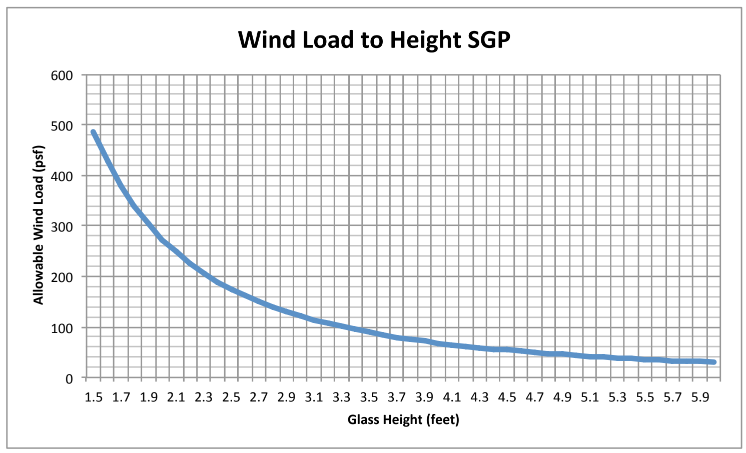
FOR 11/16” LAM. GLASS:
Determine relationship between allowable wind load ASD and wind screen height:



For PVB interlayer
$h_{ef;\sigma} = 0.374$” typical
$M_{wa} = 2*0.465^2*9{,}600 = 4{,}152$”# = 346.0’#
$h = (346.0’\#/ft/(0.55*W))^{1/2}$
$W = 629.0/H^2$
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
$h_{ef;\sigma} = 0.613$” typical
$M_{wa} = 2*0.613^2*9{,}600 = 7{,}215$”# = 601.2’#
$h = (601.2’\#/ft/(0.55*W))^{1/2}$
$W = 1{,}093/H^2$
H = glass height in feet



EDWARD C. ROBISON, PE
10012 Creviston Dr NW
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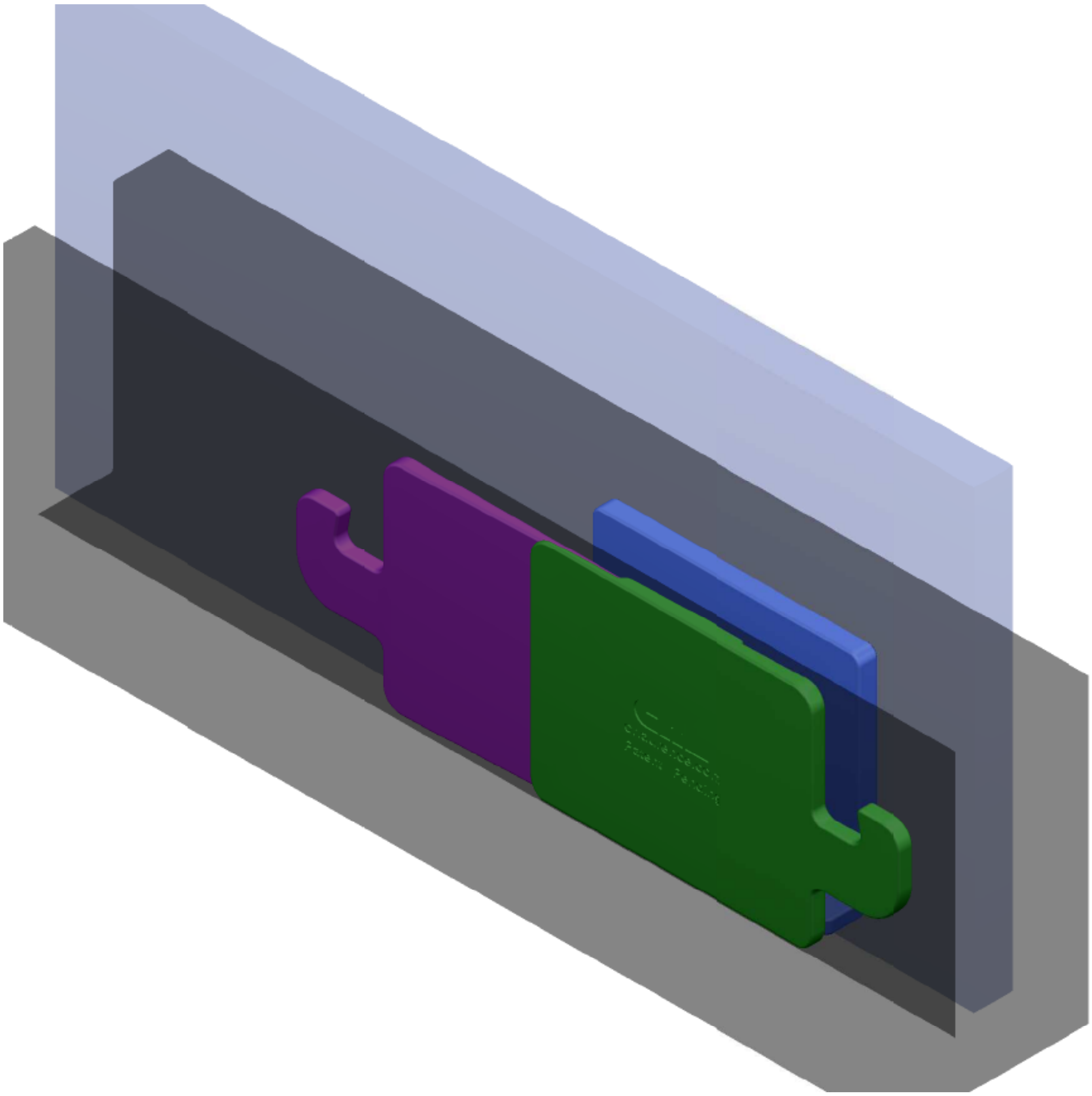
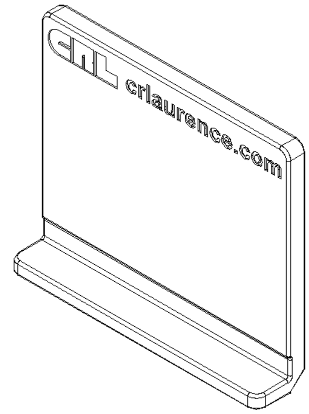
DRY-GLAZE TAPER-LOC SYSTEM



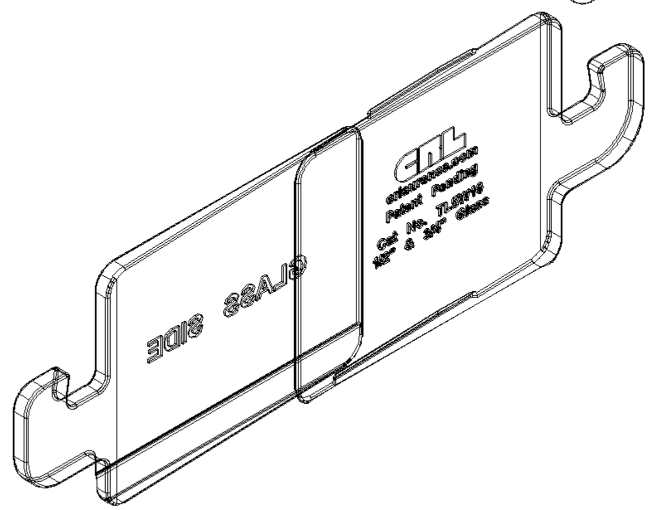
Glass is clamped inside the aluminum base shoe by the Taper-Loc Shoe Setting Plate (L shaped piece on the back side) and two Taper-Loc Shim Plates (front side). The glass is locked in place by the compressive forces created by the Taper-Loc shim plates being compressed together by the installation tool. 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:
$A = 2''*2.5'' = 5\ in^2$
adhesive shear strength ≥ 80 psi
3M™ VHB Tape
$Z = (2/3)*5\ in^2*80 = 267\#$ 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:
$\mu = 0.20$
$f = 1{,}628*0.20 = 326\#$

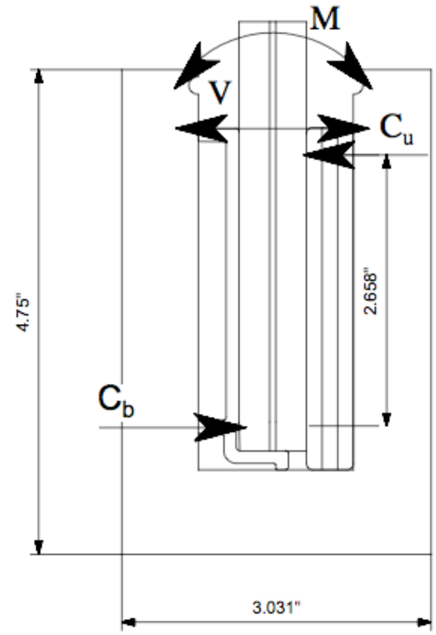
Resistance to glass pull out:
$U = 267\#+326\# = 593\#$

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:

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

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 11/16” 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)*3,000psi/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 11/16” laminated glass based on minimum glass dimension and interlayer.

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 psi
V_a = t*b/7.0

For 11/16” laminated glass, 12” width:
$M_a = 2*h_{ef;\sigma}{}^2*6{,}000$ for live load
V_a = 0.48*12*3,000/7.0 = 2,469# for live load

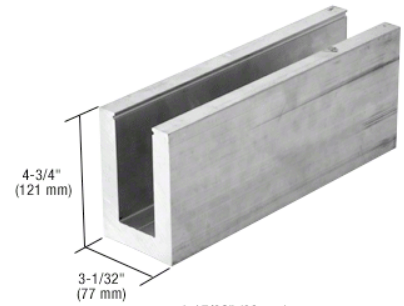
Since shear load in all scenarios is under 10% of allowable it can be ignored in determining allowable bending since it has less than 1% impact on allowable bending loads or rail heights.

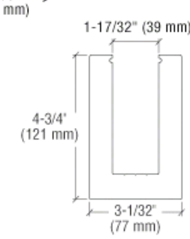
Maximum edge distance for edge of glass to centerline of Taper-Loc plates:
e_{des} = 14/2 = 7” for design conditions (no reduction in allowable loads)
$e_{max} = e + e_{des}/2$ and
$(25*e*3.5') + 25*1.17*3.5^2/2 = 229.6$: solve for e
$e_{max} = 3.5'' + [229.6 - 25*1.17*3.5^2/2]/(25*3.5) = 10.4''$ (to CL of Taper-Loc plates)

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

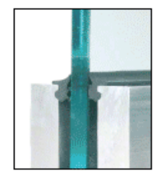
L68S10

CRL L68S Series 118-1/8” Laminated Square Base Shoe
6063-T52 Aluminum extrusion
Fully tempered glass glazed in place, using the Taper-Loc dry-glazing system.





Undrilled



Gasket Retention Groove Secures the Top Rubber Gasket in the Base Shoe

Shoe strength – Vertical legs:
Glass reaction by bearing on legs to form couple.
Allowable moment on legs per 2020 ADM Chapter F.

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

$S_y = 12”*0.75”^2*/6 = 1.125\ in^3/ft$

$Z_y = 12”*0.75”^2*/4 = 1.6875\ in^3/ft$

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

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

Leg shear strength @ bottom 2020 ADM G.1

$t_{min} = 0.75”$

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

V_{all} = 0.75”*12”/ft*9.6 ksi/1.65 = 52.36 k/ft

Base shoe anchorage:
Typical Guard design moment = 175#’ = 2,100#” or

For M14 hex head cap screw to tapped steel

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

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

T_n = 1.2218”*0.25*0.6*58 ksi = 10.63 k

Bolt tension strength = 0.75*67.5 ksi*0.1789 in^2 = 9.06 k

Use 5/16” minimum for maximum load:
Maximum service load: 10.63k/2 = 5,330#

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

M = 5,330#*[1.515625”-0.5*5,330/(30ksi*11.8125)] = 8,038”# per anchor

For 5.875” o.c.

M = 2*5,330#*[1.515625”-0.5*5,330/(30ksi*5.875)] = 15,995”# per 2 anchors (0.9844’)

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 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 M12 HSL-3

Nominal embed depth = 4.134”; Effective embed depth = 3.15”:

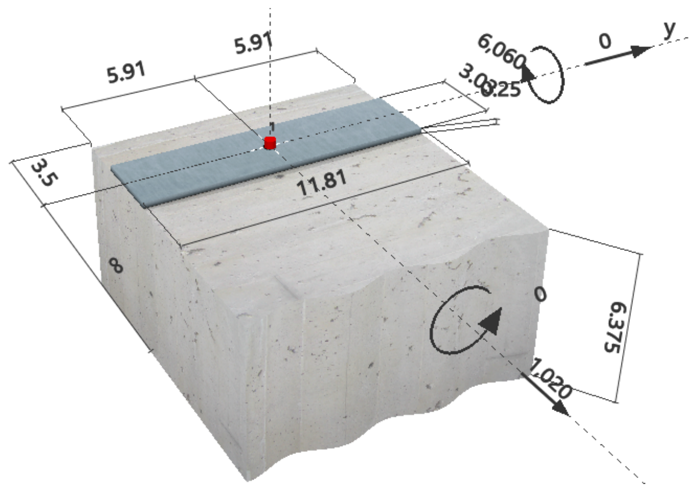
For anchors at 11 13/16” on center: For 4,000 psi cracked concrete:

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

$V \leq 0.2*3111 = 622\#$ - 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:

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



3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	4391	11397	39	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	4391	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*	1020	9571	11	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1020	9534	11	OK
Concrete edge failure in direction y+**	1020	5098	21	OK

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

Maximum moment M_u = 6,060”# maximized using the Hilti Profis software

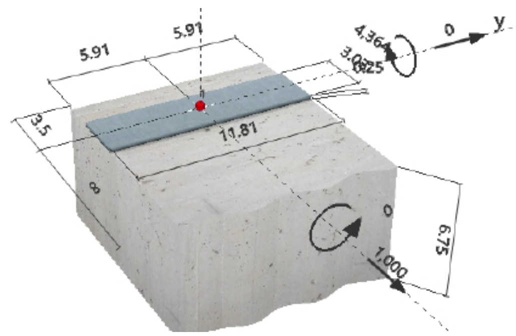
Maximum shear $V_u = 0.2*5{,}098 = 1{,}020\#$

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

$M_a = 6{,}060/1.6 = 3{,}788$”# (total wind load moment per anchor - approx. 1 foot)

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 = 400#; V_a = 400/1.6 = 250# per anchor

V_a = 250/0.984 = 254 plf

Moment load: M_u = 4,350"# ; M_a = 4,350/1.6 = 2,719"# per anchor

M_a = 2,719/0.984 = 2,762"#/ft

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	3180	11778	28	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	3180	3194	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*	400	5547	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	400	6880	6	OK
Concrete edge failure in direction x-**	400	2083	20	OK

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

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.996	0.192	1.000	99	OK

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

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

FOR 1/2" 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 = 260#; V_a = 260/1.6 = 163# per anchor
V_a = 163/0.4925 = 331 plf
Moment load: M_u = 2,652"# ; M_a = 2,652/1.6 = 1,658"# per anchor
M_a = 1,658/0.4925 = 3,365"#/ft

3 Tension load

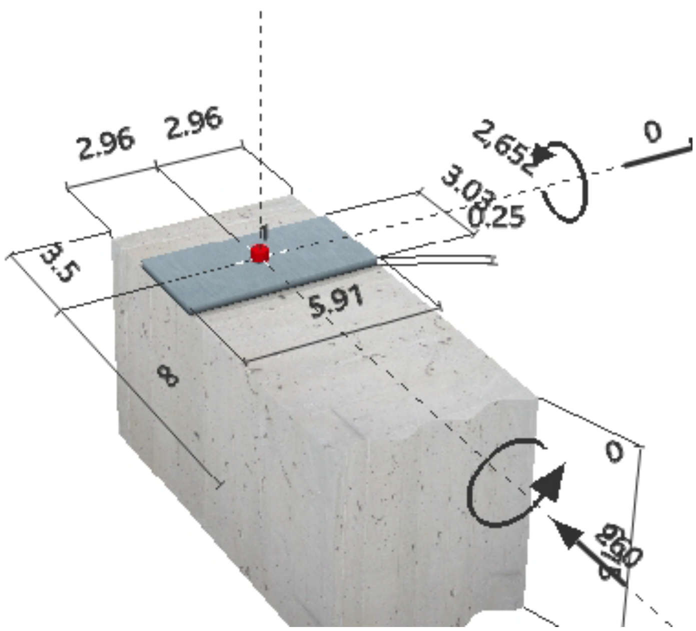
	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2008	11778	18	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2008	2009	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	260	5547	5	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	260	4327	7	OK
Concrete edge failure in direction x-**	260	1312	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.999	0.198	1.000	100	OK



EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

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).

$T_b = WC_DC_ml_m$ = total withdrawal load in lbs per lag

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

$Z'_{ll} = C_D*Z_{ll}$= 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 =2,000#

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

b = A/(12") = 2.685/(12) = 0.224"

M_a = 2,005#*(1.5156-0.224/2)(12/11.8125) = 2,859#" = 238.24#' For 11-13/16" o.c. spacing

For 5-7/8" o.c. spacing: M_a = (12/5.875)*2,005#*(1.5156-0.448/2) = 5,207 #" /ft

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.

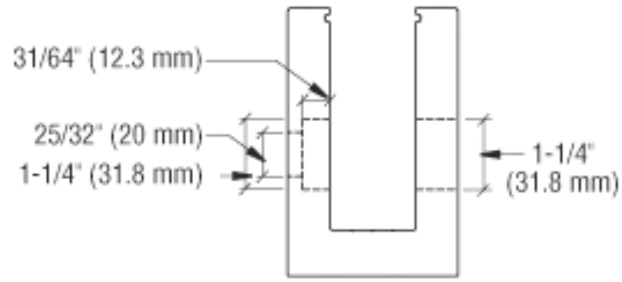
EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

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	8038.0	135.3	115.3	99.4	86.6	76.1	60.1	48.7
Steel 5-7/8” o.c	15995.0	269.3	229.4	197.8	172.3	151.5	119.7	96.9
Concrete 12M HSL 11-13/16” o.c.	3788.0	63.8	54.3	46.9	40.8	35.9	28.3	23.0
Concrete 12M HSL 5-7/8” o.c.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Concrete 1/2” HUS-EZ 11-13/16” o.c.	2762.0	46.5	39.6	34.2	29.8	26.2	20.7	16.7
Concrete 1/2” HUS-EZ 5-7/8” o.c.	3365.0	56.6	48.3	41.6	36.3	31.9	25.2	20.4
Wood 11-13/16” o.c.	2859.0	48.1	41.0	35.4	30.8	27.1	21.4	17.3
Wood 5-7/8” o.c.	5207.0	87.7	74.7	64.4	56.1	49.3	39.0	31.6

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Fascia Mounted Base Shoe:
Verify Anchor Pull through on base shoe:
For counter sunk screw
$P_{nov} = (0.27+1.45t/D)DtF_{ty}$
$=(0.27+1.45*.5*/.5).5*.5*16$ ksi = 6,880#

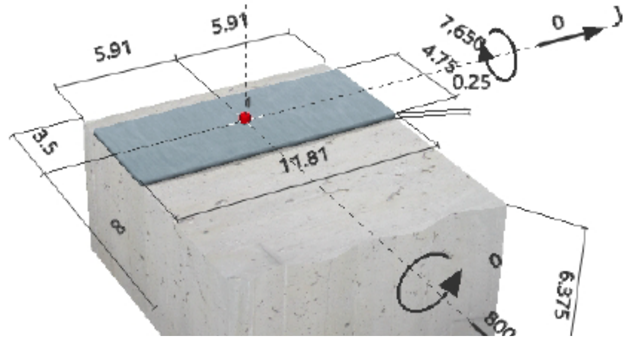


Hole Pattern "F"

For inset bolt - M14
$t_{min} = 0.25”$
$P_{nov} = 0.6*F_{tu}*(A_v)$
$A_v = 0.25”*\pi*.75” = 0.589$ in^2
$P_{nov} = 0.6*22$ksi$*(0.649$ in$^2) = 8{,}571\#$
$P_a = 8{,}571\#/1.95 = 4{,}395\# \leq 5{,}330\#$
Tear through controls

For standard installation, 42” guard height and 25 psf max uniform load
Anchor Load T_a
$T_a = M_a/2.125”$
$M_a = T_a*(2.125” - T_a/(30\text{ksi}*s)$
For M14 anchors into steel support:
M = 4,395#*[2.25”-0.5*4,395/(30ksi*11.81)] = 9,861”# = 821.8’# per anchor
For 5.875” oc. spacing
M = 4,395#*[2.25”-0.5*4,395/(30ksi*5.875)] = 9,834”#/anchor: 19,668”#/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

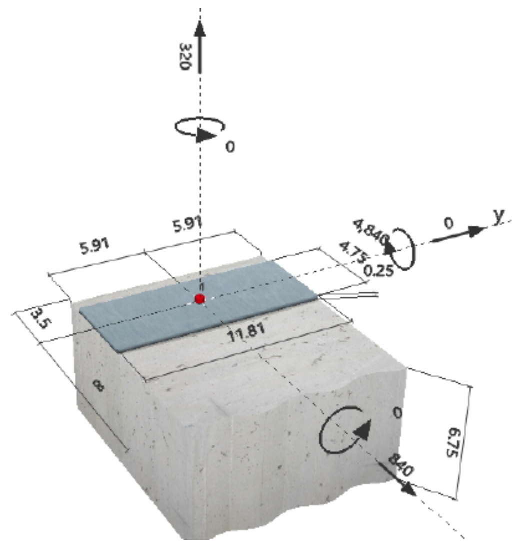
* anchor having the highest loading **anchor group (relevant anchors)

Allowable wind load on balustrade must be reduced for the dead load moment effect
$V_d = h_g$*8.3psf + 15psf (10.5 plf for base shoe and glazing + 4.5 plf for cap rail)
$M_d = [h_g$*8.3psf + 15psf]*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)*8.3psf + 15psf = (8.3h + 17.7)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 by 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.

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 = 840#; V_a = 840/1.6 = 525# per anchor
V_a = 525/0.984 = 534 plf
Moment load: M_u = 4,840”# ; M_a = 4,840/1.6 = 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*	2525	11778	22	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2525	2525	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	840	5547	16	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	840	5439	16	OK
Concrete edge failure in direction y-**	840	4397	20	OK

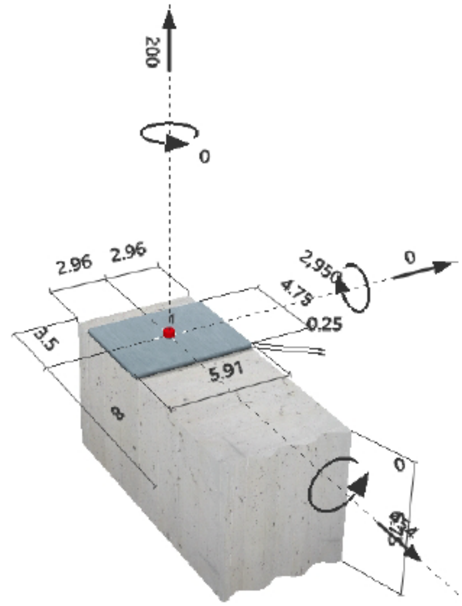
5 Combined tension and shear loads

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

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

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

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 11 13/16” 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 = 2,950”# ; M_a = 2,950/1.6 = 1,844”# per anchor
M_a = 1,844/0.4925 = 3,744”#/ft
With tension load of T_u = 200#; T_a = 200/1.6 = 125#
T = 125/0.4925 = 254 plf

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1583	11778	14	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	1583	1588	100	OK

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	454	5547	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	454	3421	14	OK
Concrete edge failure in direction y+**	454	2290	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.996	0.198	1.000	100	OK

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

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

FASCIA MOUNT ANCHORAGE TO WOOD

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

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

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

W'p = 2,005# from previous calculations

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

For typical installation with 42" height AFF:

$V_d = (8.3*3.5 + 17.7)$plf = 47#

Assume T = 2000#

$\alpha = \tan^{-1} 2000/47 = 88.65°$

$Z'_a = [(2005)512]/[(2005)\cos^2 88.65 + 512\sin^2 88.65] = 2002\#$

Allowable tension component for 47# shear:

$T = \sqrt{(2002^2 - 47^2)} = 2001 \geq 2000\#$ 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) - 12.6h - 27"# = 4,249"# - 12.6h for 11-13/16" o.c.

M_a = 2*2,000#*(2.25"-2*0.224/2) - 12.6h - 27"# = 8,104"# - 12.6h for 5-7/8" 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 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	164.9	140.5	121.1	105.4	92.6	73.2	59.2
Steel 5-7/8" o.c	19668.0	330.0	281.2	242.4	211.1	185.5	146.5	118.7
Concrete 12M HSL 11-13/16" o.c.	4781.0	79.4	67.6	58.3	50.7	44.5	35.1	28.4
Concrete 1/2" HUS-EZ 11-13/16" o.c.	3074.0	50.7	43.1	37.1	32.3	28.4	22.4	18.1
Concrete 1/2" HUS-EZ 5-7/8" o.c.	3744.0	61.9	52.7	45.4	39.5	34.7	27.4	22.1
Wood 11-13/16" o.c.	4249.0	70.4	60.0	51.7	45.0	39.5	31.2	25.2
Wood 5-7/8" o.c.	8104.0	135.3	115.3	99.4	86.5	76.0	60.0	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.

11/16”	EFFECTIVE THICKNESS		PVB Interlayer	Allowable wind Pressure, psf for glass height in inches				
width inches	t_e for defl.	t_e for stress	All. Moment “#/ft	36	42	48	60	72
12	0.3739	0.4202	3743	63.0	46.3	35.4	22.7	15.8
24	0.3901	0.4394	4092	68.9	50.6	38.8	24.8	17.2
36	0.4125	0.4674	4578	77.1	56.6	43.4	27.7	19.3
41	0.4227	0.4757	4798	*	59.3	45.4	29.1	20.2
48	0.7372	0.4910	5110	*	*	48.4	31.0	21.5
60	0.4616	0.5151	5624	*	*	*	34.1	23.7
72	0.4841	0.5358	6086	*	*	*	*	25.6

11/16”	EFFECTIVE THICKNESS		SentryGlas+ Interlayer	Allowable wind Pressure, psf for glass height in inches				
width inches	t_e for defl.	t_e for stress	All. Moment “#/ft	36	42	48	60	72
12	0.4578	0.5114	5544	93.3	68.6	52.5	33.6	23.3
24	0.5457	0.5850	7256	122.2	89.7	68.7	44.0	30.5
36	0.5883	0.6132	7972	134.2	98.6	75.5	48.3	33.6
41	0.5987	0.6194	8135	*	100.6	77.0	49.3	34.2
48	0.6092	0.6255	8294	*	*	78.5	50.3	34.9
60	0.6205	0.6317	8460	*	*	*	51.3	35.6
72	0.6271	0.6352	8555	*	*	*	*	36.0

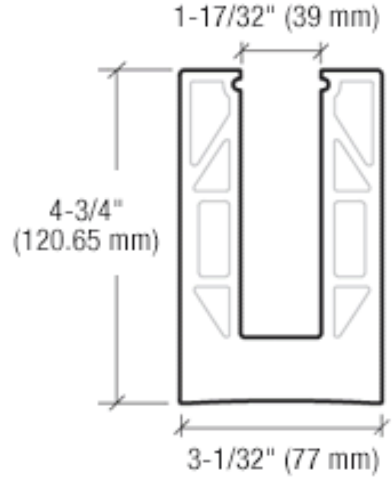
* Allowable load is same as last value in column
Calculated from: $w_{all} = M_{all}*12/(0.55*h_g^2)$

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- 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

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

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"

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

Strength at bottom cell

Vertical leg allowable tension load:

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

A_v = area of vertical leg, A_d = Area of diagonal load

Allowable shear across cell:

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

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

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

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

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

Strain in cell walls:

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

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

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

$\Delta_T = \Delta_\epsilon + \Delta_b = 0.0057+0.00339 = 0.00909"$

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

$\Delta_g = 0.00909*(42/3.806) = 0.10"$ 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.

EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Registered Professional Engineer/Structural
CERTIFICATE NO.
42123
EDWARD C.
ROBISON
Date Signed
ARIZONA U. S. A.
EXP 03/31/2023

REGISTERED PROFESSIONAL ENGINEER
EDWARD C. ROBISON
No. C 65883
CIVIL
STATE OF CALIFORNIA

COLORADO LICENSED
EDWARD C. ROBISON
54027
PROFESSIONAL ENGINEER

EDWARD C. ROBISON
LICENSE
NO. 63683
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
EXP 02/28/2023

GEORGIA
REGISTERED
No. 046121
PROFESSIONAL
ENGINEER
EDWARD C. ROBISON

EDWARD C. ROBISON
LICENSED
PROFESSIONAL
ENGINEER
No. 11576-S
HAWAII U.S.A.

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION

04/30/2022

Signature Expiration Date of the License

LICENSED STRUCTURAL ENGINEER
EDWARD C.
ROBISON
081.007077
STATE OF ILLINOIS
EXP 11/30/2022

STATE OF MARYLAND
EDWARD C. ROBISON
52500
PROFESSIONAL ENGINEER

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland,
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COMMONWEALTH OF MASSACHUSETTS
EDWARD C.
ROBISON
STRUCTURAL
No. 49757
REGISTERED
PROFESSIONAL ENGINEER
EXP 06/30/2022

EDWARD C. ROBISON
LICENSED PROFESSIONAL
ENGINEER
29000
STATE OF MISSISSIPPI

PROFESSIONAL ENGINEER-STATE OF NEVADA
EDWARD C.
ROBISON
Exp: 12/31/2021
CIVIL
No. 17546

STATE OF NEW YORK
Edward C. Robison
088030
LICENSED PROFESSIONAL ENGINEER
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: Typed or printed name: Edward C. Robison
Date Lic. No. 58604

COMMONWEALTH OF PENNSYLVANIA
REGISTERED
PROFESSIONAL
EDWARD C. ROBISON
ENGINEER
NO. 73556
EXP 09/30/2021

STATE OF TEXAS
EDWARD C. ROBISON
100688
LICENSED
PROFESSIONAL ENGINEER
FIRM #F-12044
EXP 12/31/2021

LICENSED PROFESSIONAL ENGINEER
E. C.
ROBISON
31646
OKLAHOMA

REGISTERED PROFESSIONAL
ENGINEER
18195PE
OREGON
DEC. 6, 1995
EDWARD C. ROBISON
EXP 12/31/2022

EDWARD C. ROBISON
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 124674
STATE OF TENNESSEE

LICENSED STRUCTURAL ENGINEER
No. 10738527-2203
EDWARD C. ROBISON
STATE OF UTAH

COMMONWEALTH OF VIRGINIA
EDWARD C. ROBISON
Lic No. 059203
PROFESSIONAL ENGINEER

EDWARD C. ROBISON
STATE OF WASHINGTON
29129
REGISTERED
PROFESSIONAL ENGINEER

Professional Engineer
Edward C. Robison
16531
Date
WYOMING

EDWARD C. ROBISON PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com