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SUBJ: TAPER-LOC® SYSTEM DRY-GLAZE
LAMINATED TEMPERED GLASS RAIL SYSTEM
1-1/16" (25.52mm) LAMINATED GLASS - L25S BASE SHOE

The GRS Glass Railing Dry Glaze Taper-Loc™ System utilizes 1-1/16" (25.52mm) laminated tempered glass (1/2" 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 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" 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 19 for surface mounted anchor strength and allowable wind loads or Table 5 on page 24 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 25) 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 25) 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.)

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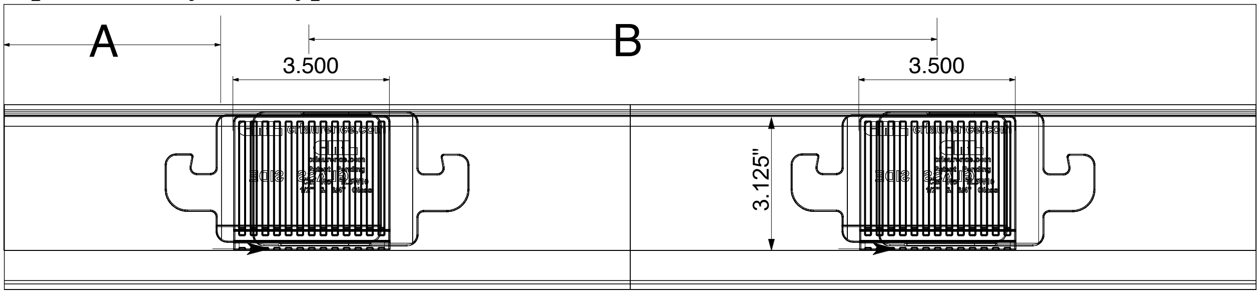
Taper-Loc® System Typical Installation



For two ply laminated glass with 1/2” 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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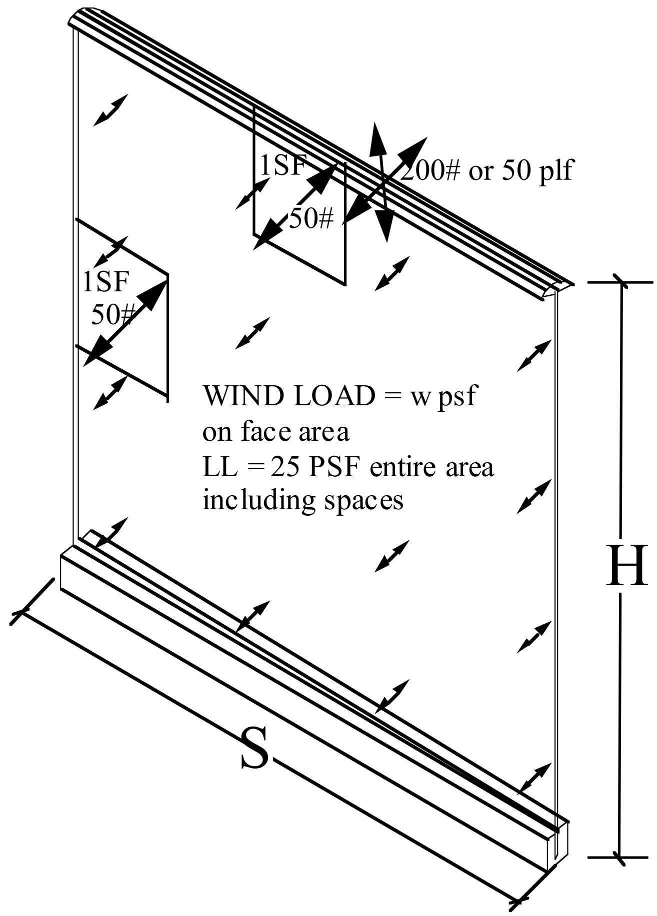
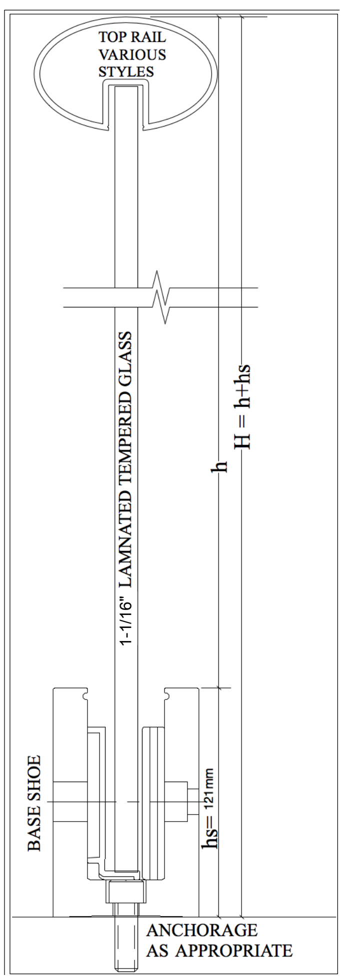
LOAD CASES:

Dead load = 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 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 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 29.10-1) 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 26.5-1B) 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.

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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 1-1/16” 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 E1300-12a may be used - Maximum edge stress of 10,600 psi; however, recommend limiting to 9,600 psi because of support conditions.
Determine effective thickness of the laminated glass for stresses and deflections based on ASTM E1300-16 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 byKuraray)
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.
$h_s = 0.5(h_1+h_2)+h_v = 0.5(0.469*2)+0.06 = 0.529”$
$h_{s;1} = h_{s;2} = (h_s h_1)/(h_1+h_2) = (0.529*0.469)/(2*0.469) = 0.2645”$
$I_s = h_1 h^2_{s;2} + h_2 h^2_{s;1} = 2*(0.469*0.2645”^2) = 0.0656$
$\Gamma = 1/[1+9.6(EI_s h_v)/(Gh^2_s a^2)]$
effective thickness for deflection:
$h_{ef;w} = (h_1^3 + h^3_2 + 12\Gamma I_s)^{1/3}$
effective thickness for glass stress:
$h_{1;ef;\sigma} = [h_{ef;w}{}^3/(h+2\Gamma h_{s;2})]^{1/2}$
$M_{aL} = 6{,}000psi*2*h_{1;ef;\sigma}{}^2 = 12{,}000\ h_{1;ef;\sigma}{}^2$ “#/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

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Table 2	h_1, h_2	h_v		$h_{s;1}$ $h_{s;2}$		I_s	h_s	
12mm	0.469	0.06		0.2645		0.0656	0.529	
12mm	0.469	0.06		0.2645		0.0656	0.529	
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	0.0071	0.1438	0.5962	0.6837	0.6695	0.7657	9504	12430
24	0.0279	0.4019	0.6112	0.8056	0.6870	0.8758	10005	16261
36	0.0607	0.6019	0.6334	0.8795	0.7121	0.9295	10750	18317
41	0.0773	0.6623	0.6441	0.8995	0.7239	0.9425	11109	18833
48	0.1030	0.7289	0.6599	0.9206	0.7410	0.9555	11640	19357
60	0.1521	0.8077	0.6883	0.9444	0.7704	0.9695	12582	19925
72	0.2053	0.8581	0.7166	0.9590	0.7982	0.9776	13506	20261

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:

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

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 $S \leq h$

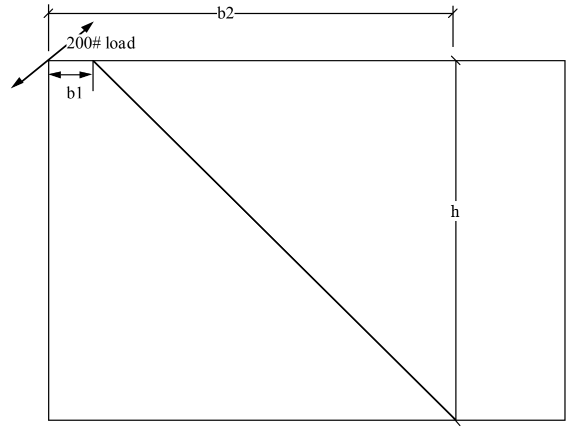
For wind load

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

maximum wind load for given light height:

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

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 1-1/16" 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.
$b_2 = b_1+h$

$M_{ave} = 200*h/(b_2)$ 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-
$H = [3Ebt^3/(1.333*200)]^{1/3} = [2.25Ebt^3/(88.89)]^{1/3}$

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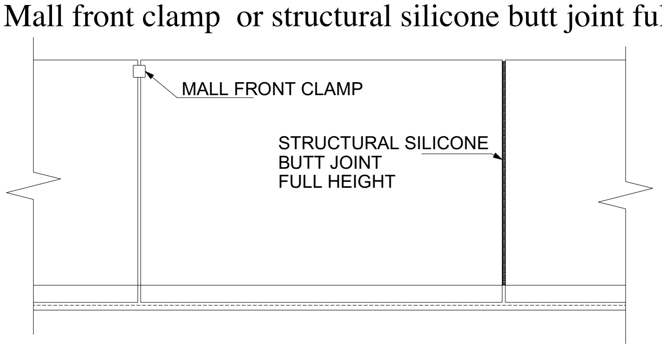
FOR INSTALLATION WITHOUT A TOP RAIL
TABLE 3

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	0.670	20.2	53.8	0.766	26.4	70.4
24	0.687	42.3	56.6	0.876	54.0	74.8
36	0.712	50.2	60.9	0.930	65.6	79.4
41	0.724	53.3	61.9	0.943	69.4	80.5
48	0.741	56.3	63.3	0.956	72.6	81.6
60	0.770	66.5	65.8	0.970	83.7	82.8
72	0.798	71.3	68.2	0.978	87.3	83.5

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.

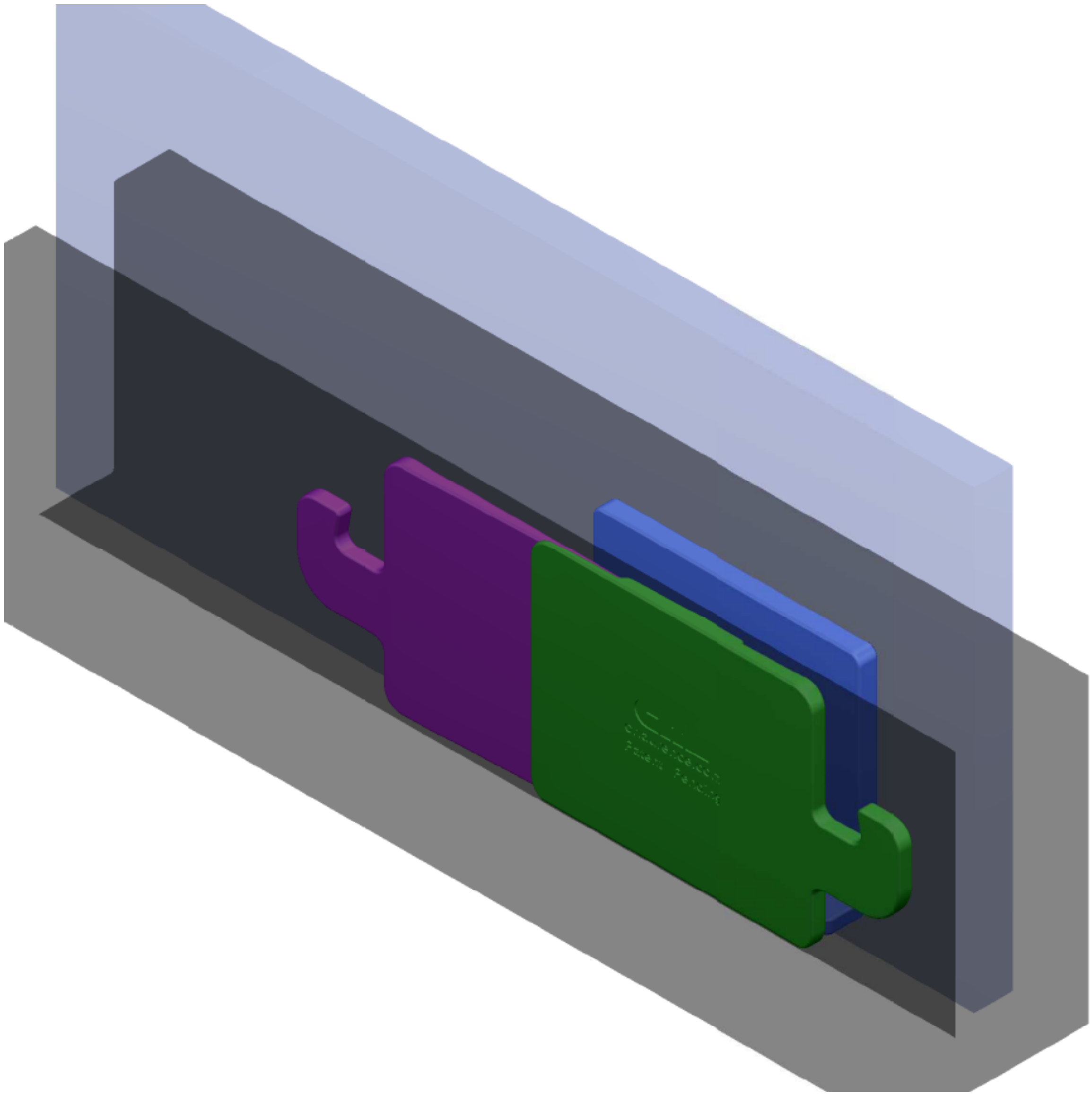
For installations without a top rail the differential deflection of glass lights must be checked based on 200 lb concentrated load on one light. Where deflection exceeds 3/4” the lights must be connected together at the joints to limit differential deflection. Recommend using mall front clamps, H clip or similar within 12 inches of the top of the glass.

Mall front clamp or structural silicone butt joint full height.



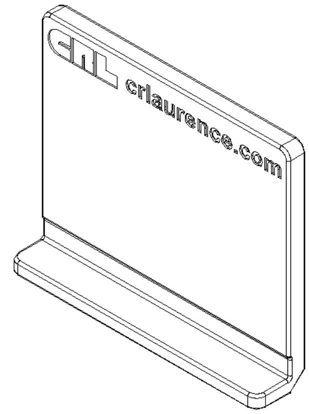
POOL FENCE
When installed as a pool fence the live loads are assumed as acting at 42” above finish floor.

DRY-GLAZE TAPER-LOC SYSTEM



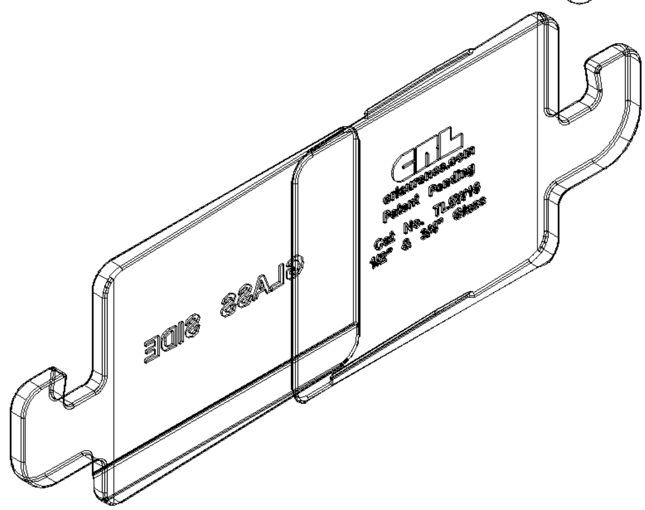
Glass is clamped inside the aluminum base shoe by the Taper-Loc Shoe Setting Plate (L shaped piece on the back side) and two Taper-Loc Shim Plates (front side). The glass is locked in place by the compressive forces created by the Taper-Loc shim plates being compressed together by the installation tool. 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
$Z = (2/3)*5 \text{ in}^2*80 = 267\#$ minimum
setting plate locks into place in the base shoe by friction created by the compression generated when the shim plates
are locked into place.



Installation force:
$T_{des} = 250\#"$ design installation torque
$T_{max} = 300\#"$ maximum installation torque
Compressive force generated by the installation torque:
$C = (0.2*250\#"/1.0")/ \sin(1.76^\circ)$
$C = 1{,}628\#$

Frictional force of shims and setting plate against aluminum base shoe:
coefficient of friction, $\mu = 0.65$
$f = 2*(1{,}628\#0.65) = 2{,}117\#$

Frictional force of shims against glass:
$\mu = 0.20$
$f = 1{,}628*0.20 = 326\#$

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

Safety factor for 200# pullout resistance = 2*593/200 = 5.93
Based on two taper sets
Minimum recommended installation torque:
$4/5.93*250 = 169\#"$

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

$T = 250*(0.7/0.2) = 875\#"$

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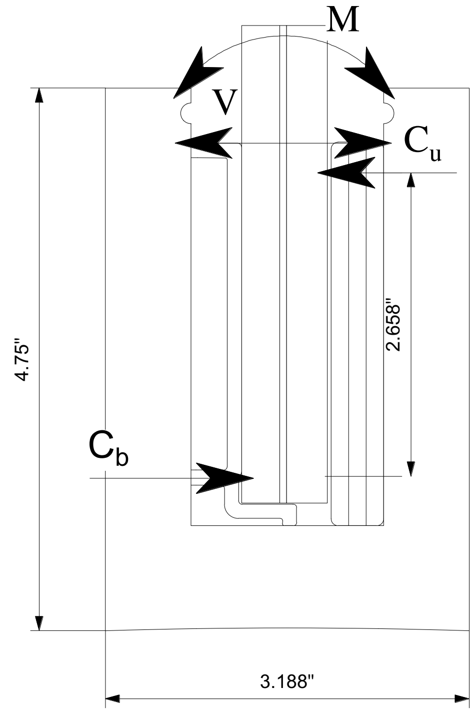
Glass anchorage against overturning:
Determine reactions of Taper-Loc plates on the glass:
Assuming elastic bearing on the wedges the reactions will have centroids at approximately 1/6*3.188" from the upper and lower edges of the bearing surfaces:



R_{CU} @ 1/6*3.188 = 0.53"
e = 3.188-0.53 = 2.658"
From ∑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)*3,000psi/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.

<u>Allowable moment on system is limited to allowable glass moment for 1-1/16" laminated glass based on minimum glass dimension and interlayer.</u>

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

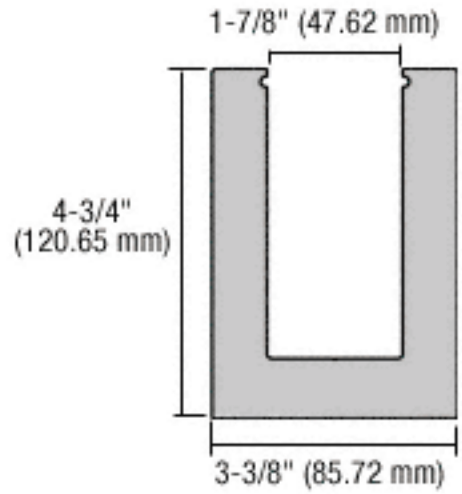
Allowable Glass Loads:
M_a = S*6,000 psi
V_a = t*b/7.0

For 1-1/16” laminated glass, 12” width:
$M_a = 2*h_{ef;\sigma}^2*6{,}000$ for live load
V_a = 0.438*12*3,000/7.0 = 2,253# 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)

L25S 4-3/4" x 3-3/8" (121mm x 85.7mm) GLASS BALUSTRADE BASE SHOE
FOR 1-1/16mm LAMINATED GLASS
6063-T52 Aluminum extrusion
Fully tempered glass glazed in place with Taper-Loc-Laminated™ system.



Shoe strength – Vertical legs:
Glass reaction by bearing on legs to form couple.
Allowable moment on legsper 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 in³/ft
Z_y = 12"*0.75"²*/4 = 1.6875 in³/ft
M_{ay} = 16ksi*1.5*1.125 in³/ft/1.65 = 16,364#"/ft or (controls)
M_{ar} = 22ksi*1.6875 in³/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

Maximum base shoe leg deflection:
$\Delta = Mh^2/(3Et^3) = 12{,}181*3.75^2/(3*10{,}400{,}000*0.75^3) = 0.013"$
Glass deflection from leg deflection for 42" glass height above shoe:
Δ_{top} = 0.013*45.75/3.75 = 0.16"

Base shoe anchorage:
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 in² = 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#*[1.6875"-0.5*5,330/(30ksi*11.8125)] = 8,954"# per anchor
For 5.875" o.c.
M = 2*5,330#*[1.6875"-0.5*5,330/(30ksi*5.875)] = 17,827"# per 2 anchors (0.9844')

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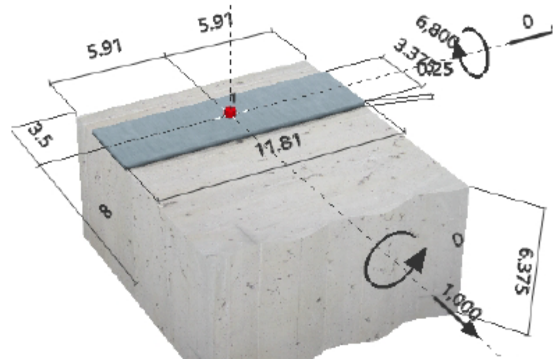
ANCHORAGE TO CONCRETE

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:
$V_u = 1.6$ 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*	4404	11397	39	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	4404	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

Maximum moment $M_u = 6{,}800"\#$ 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)
$M_a = 6{,}800/1.6 = 4{,}250"\#$ (total wind load moment per anchor - approx. 1 foot)

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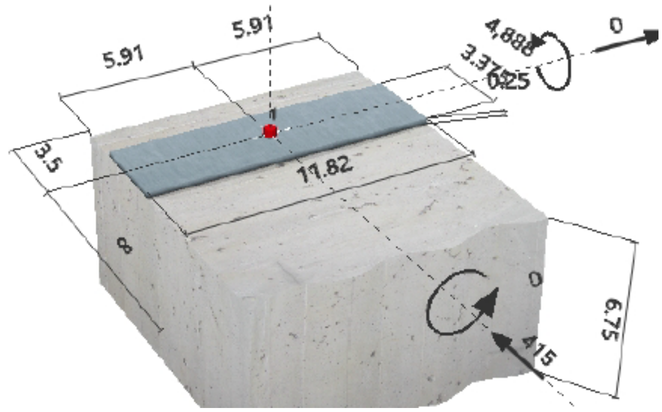
ALTERNATIVE ANCHORAGE TO CONCRETE
Hilti HUS-EZ (KH-EZ) 1/2" Diameter



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 = 415#; V_a = 415/1.6 = 259# per anchor
V_a = 259/0.984 = 264 plf
Moment load: M_u = 4,888"# ;
M_a = 4,888/1.6 = 3,055"# per anchor
M_a = 3,055/0.984 = 3,105"#/ft

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	3193	415	-415	0

max. concrete compressive strain: 0.26 [‰]
max. concrete compressive stress: 1149 [psi]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	3193	11778	28	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	3193	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*	415	5547	8	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	415	6880	7	OK
Concrete edge failure in direction x-**	415	2083	20	OK

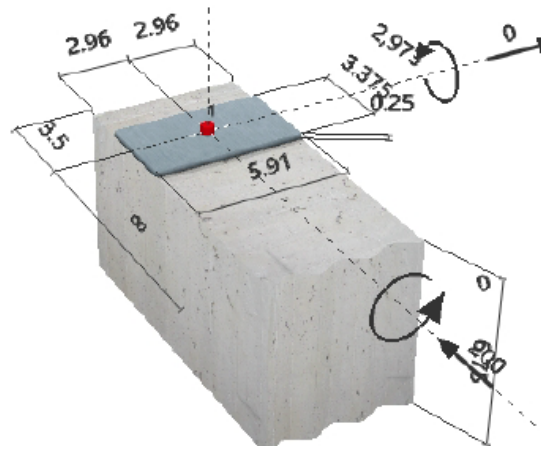
5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.199	1.000	100	OK

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

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: $V_u = 200\#$; $V_a = 200/1.6 = 125\#$ per anchor

$V_a = 125/0.4925 = 255$ plf

Moment load: $M_u =$ 2,973”# ; $M_a =$ 2,973/1.6 = 1,858”# per anchor

$M_a = 1{,}858/0.4925 = 3{,}773$”#/ft

2 Load case/Resulting anchor forces

Load case: Design loads

Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	2009	200	-200	0

max. concrete compressive strain: 0.25 [‰]
max. concrete compressive stress: 1093 [psi]
resulting tension force in (x/y)=(0.000/0.000): 2009 [lb]
resulting compression force in (x/y)=(-1.480/0.000): 2009 [lb]

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*	200	5547	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	200	4327	5	OK
Concrete edge failure in direction x-**	200	1021	20	OK

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.196	1.000	100	OK

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

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Installation to wood:

1/2" x 6" lag screws into solid wood, Douglas Fir or Southern Pine or equivalent density wood.
Typical anchor to wood: 1/2" lag screw. Withdrawal strength of the lags from *National Design Specification For Wood Construction* (NDS) Table 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 =2,005#
A = 2,005#/745psi = 2.691in^2
b = A/(12") = 2.691/(12) = 0.224"
M_a = 2,005#*(1.6875-0.224/2)(12/11.8125) = 3,209#" = 267.42#' For 11-13/16" o.c. spacing

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.

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	8954.0	150.7	128.4	110.7	96.5	84.8	67.0	54.3
Steel 5-7/8” o.c	17827.0	300.1	255.7	220.5	192.1	168.8	133.4	108.0
Concrete 12M HSL 11-13/16” o.c.	4250.0	71.5	61.0	52.6	45.8	40.2	31.8	25.8
Concrete 1/2” HUS-EZ 11-13/16” o.c.	3105.0	52.3	44.5	38.4	33.5	29.4	23.2	18.8
Concrete 1/2” HUS-EZ 5-7/8” o.c.	3773.0	63.5	54.1	46.7	40.7	35.7	28.2	22.9
Wood 11-13/16” o.c.	3209.0	54.0	46.0	39.7	34.6	30.4	24.0	19.4
Wood 5-7/8” o.c.	6318.0	106.4	90.6	78.1	68.1	59.8	47.3	38.3

Note: Alternative anchorages may be designed using the principles shown in this report to provide greater strength and allow for higher wind loads. Must verify that glass.

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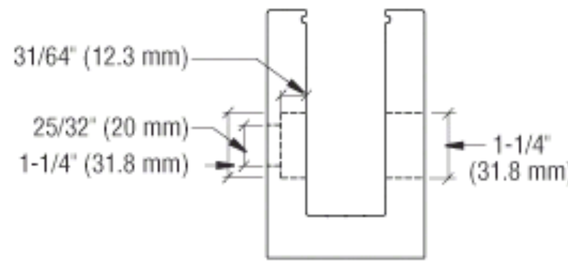
Fascia Mounted Base Shoe:



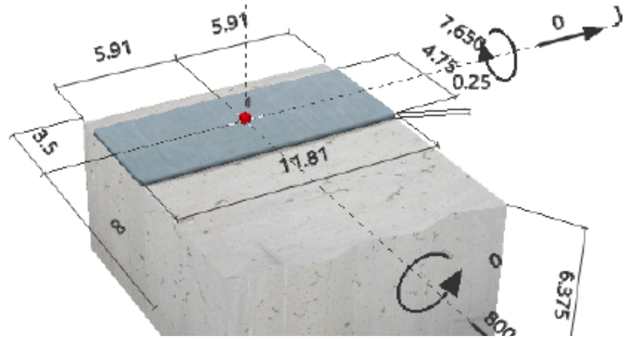
Hole Pattern "F"

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#

For inset bolt - M14
t_{min} = 0.25"
P_{nov} = 0.6*F_{tu}*(A_v)
A_v = 0.25"*π*.75" = 0.589 in^2
P_{nov} = 0.6*22ksi*(0.649 in^2) = 8,571#
P_a = 8,571#/1.95 = 4,395# ≤ 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/(30ksi*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

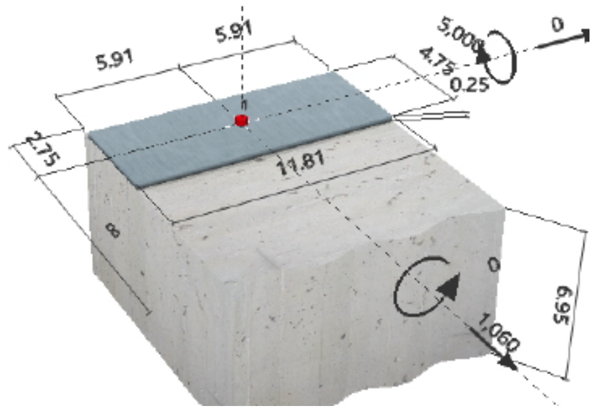
	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	800	9571	9	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	800	7538	11	OK
Concrete edge failure in direction y+**	800	4030	20	OK

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

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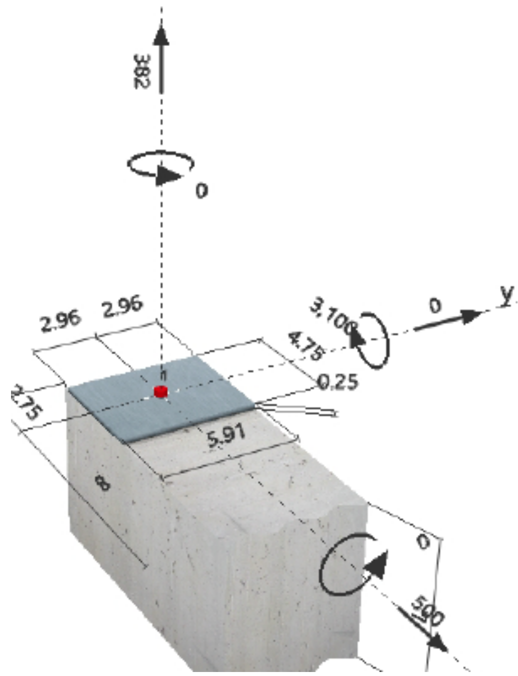
ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED 5.91" o.c.
Hilti HUS-EZ (KH-EZ) 1/2" Diameter



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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"# ; M_a = 3,100/1.6 = 1,938"# per anchor
M_a = 1,938/0.4925 = 3,934"#/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

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

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:

$V_d = (13.1*3.5 + 19.4)\text{plf} = 65.3\#$

Assume T = 2000#

$\alpha = \tan^{-1}2000/65.3 = 88.1°$

$Z'_a = [(2005)512]/[(2005)\cos^2 88.1 + 512\sin^2 88.1] = 1{,}999\#$

Allowable wind loads:

M_a = 1,999#*(2.25"-0.224/2) - 19.4h - 28"# = 4,246"# - 19.4h

For 5-7/8" o.c.

$\alpha = \tan^{-1}2*2000/65.3 = 89.06°$

$Z'_a = [2*2005*2*512]/[(2*2005)\cos^2 89.06 + 2*512\sin^2 89.06] = 4{,}007\#$

M_a = 4,007#*(2.25"-0.448/2) - 19.4h - 28"# = 8,090"# - 19.4h

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	164.6	140.1	120.8	105.2	92.4	72.9	59.0
Steel 5-7/8" o.c	19668.0	329.7	280.8	242.1	210.8	185.3	146.3	118.4
Concrete 12M HSL 11-13/16" o.c.	4781.0	79.0	67.3	57.9	50.4	44.3	34.9	28.2
Concrete 1/2" HUS-EZ 11-13/16" o.c.	3074.0	50.3	42.8	36.8	32.0	28.1	22.1	17.9
Concrete 1/2" HUS-EZ 5-7/8" o.c.	3934.0	64.8	55.1	47.5	41.3	36.3	28.6	23.1
Wood 11-13/16" o.c.	4246.0	70.0	59.6	51.3	44.7	39.2	30.9	25.0
Wood 5-7/8" o.c.	8090.0	134.7	114.7	98.9	86.1	75.6	59.7	48.3

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:

1-1/16”	EFFECTIVE THICKNESS		PVB Interlayer	Allowable wind Pressure, psf for glass height in inches				
width inches	t_∂ for defl.	t_e for stress	All. Moment “#/ft	36	42	48	60	72
12	0.5962	0.6695	9504	160.0	117.6	90.0	57.6	40.0
24	0.6112	0.6870	1005	16.9	12.4	9.5	6.1	4.2
36	0.6334	0.7121	10750	181.0	133.0	101.8	65.2	45.2
41	0.6441	0.7239	11109	*	137.4	105.2	67.3	46.8
48	0.6599	0.7410	11640	*	*	110.2	70.5	49.0
60	0.6883	0.7704	12582	*	*	*	76.3	53.0
72	0.7166	0.7982	13506	*	*	*	*	56.8

TABLE 7:

1-1/16”	EFFECTIVE THICKNESS		SentryGlas+ Interlayer	Allowable wind Pressure, psf for glass height in inches				
width inches	t_∂ for defl.	t_e for stress	All. Moment “#/ft	36	42	48	60	72
12	0.6837	0.7657	12430	209.3	153.7	117.7	75.3	52.3
24	0.8056	0.8758	16261	273.8	201.1	154.0	98.6	68.4
36	0.8795	0.9295	18317	308.4	226.6	173.5	111.0	77.1
41	0.8995	0.9425	18833	*	232.9	178.3	114.1	79.3
48	0.9206	0.9555	19357	*	*	183.3	117.3	81.5
60	0.9444	0.9695	19925	*	*	*	120.8	83.9
72	0.9590	0.9776	20261	*	*	*	*	85.3

* Allowable load is same as last value in column

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

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