06 April 2026

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
GLASS RAIL SYSTEM - GRS
9/16" THROUGH 1-1/16" LAMINATED TEMPERED GLASS

The GRS Glass Railing Dry Glaze Taper-Loc™ System utilizes laminated tempered glass 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 2018, 2021 and 2024 International Building Codes, 2022 and 2025 California Building Codes, 2017, 2020 and 2023 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 8-22 *Specification for the Design of Cold-Formed Stainless Steel Structural Members* or AISC *370 Specification for Structural Stainless Steel*.

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

Table of Contents

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.

Embedded base shoe:
Glass strength controls for all cases

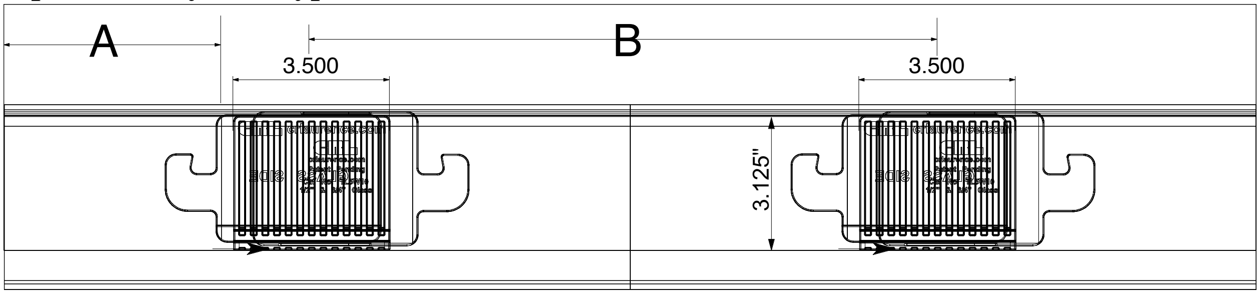
ALLOWABLE LOADS ON GLASS
The allowable load on the glass is dependent on the glass makeup and light width. Refer to table 2 for allowable moment for wind loading.
Calculate glass moment based on wind load-
$M_w = w*h^2*0.55*12$”: in-lb/ft
where:
w = wind load pressure in psf
h = effective cantilever 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® being 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 Tables 2-6 (pages 7 - 20) 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 Tables 7-10 (pages 21 - 25) for compliance with the International Building Code (all versions) and International Residential Code (all versions). Installations without a top rail must use an extra stiff ionoplast interlayer such as SentryGlas® demonstrated to retain the barrier in place in the event of the glass fracturing.

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

Taper-Loc® System Typical Installation



For two ply laminated glass with 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® sets:

6" to 14" (152 to 356mm)	1 Taper-Loc® Set
14" to 28" (356 to 711mm)	2 Taper-Loc® Sets
28" to 42" (711 to 1,067mm)	3 Taper-Loc® Sets
42" to 56" (1,067 to 1,422mm)	4 Taper-Loc® Sets
56" to 70" (1422 to 1778mm)	5 Taper-Loc® Sets
70" to 84" (1778 to 2134mm)	6 Taper-Loc® Sets

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.

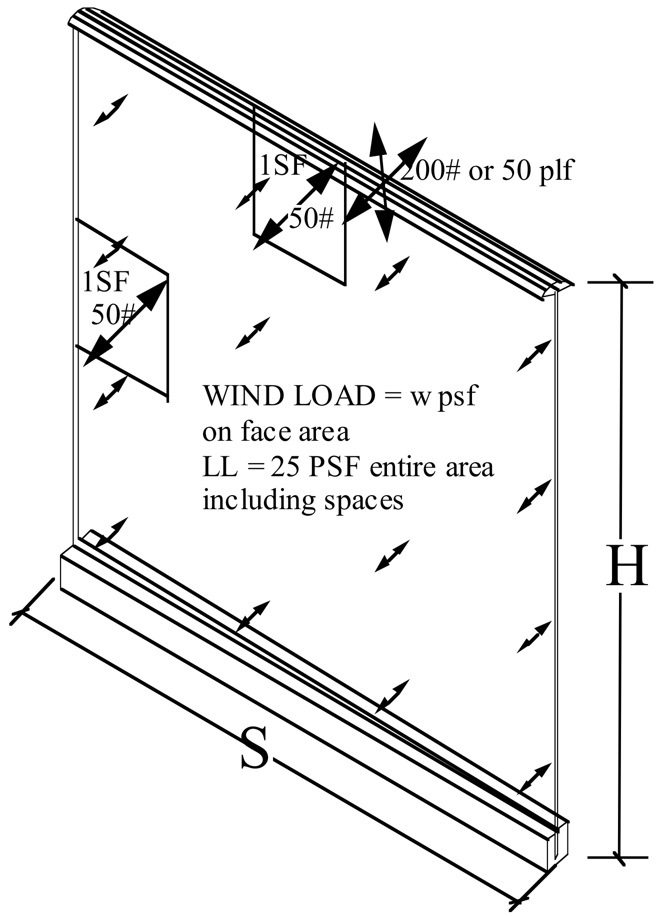
EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

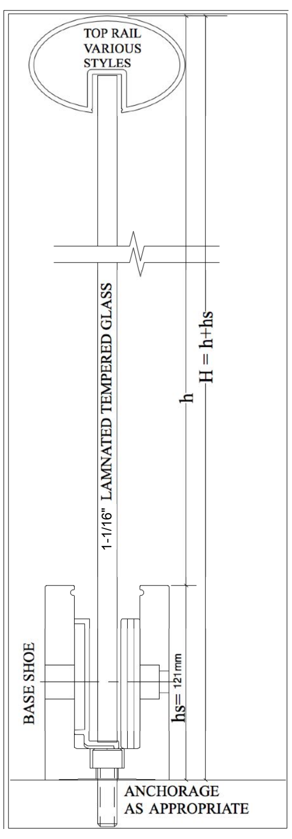
LOAD CASES:

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 7-22 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 1".

POOL FENCE

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

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

WIND LOADING ON FENCES OR GUARDS

Calculated in accordance with ASCE/SEI 7-16 (and 22) 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 $1.3 \le C_f \le 2.6$ (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.

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

9/16" 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 9/16" glass the average Modulus of Rupture F_r is 24,000 psi.

Allowable glass bending stress for live loads: 24,000/4 = 6,000 psi. – Tension stress calculated.
For wind loads the allowable stress in ASTM E1300-16 may be used - Maximum edge stress of 10,600 psi; however, recommend limiting to 9,600 psi because of support conditions.
Determine effective thickness of the laminated glass for stresses and deflections based on ASTM E1300-16 appendix X9.
For PVB interlayer G = 140 psi adjust for service temperature.
For SGP interlayer G = 15,600 psi adjust for service temperature (SentryGlas® product data published by Kuraray).
The values of G are selected as most appropriate for service conditions and load durations as G decreases with temperature and load duration.
$h_1 = h_2 = 0.219"$
$h_v = 0.06"$
a = least width - typically total glass height including portion in base shoe: 41" for 42" overall height including base shoe.
$h_s = 0.5(h_1+h_2)+h_v = 0.5(0.219*2)+0.06 = 0.279"$
$h_{s;1} = h_{s;2} = (h_s h_1)/(h_1+h_2) = (0.279*0.219)/(2*0.219) = 0.1395"$
$I_s = h_1 h^2_{s;2}+ h_2 h^2_{s;1}= 2*(0.219*0.1395"^2)= 0.00852$
$\Gamma = 1/[1+9.6(EI_s h_v)/(Gh^2_s a^2)]$
effective thickness for deflection:
$h_{ef;w} = (h_1{}^3+ h^3{}_2+ 12\Gamma I_s)^{1/3}$
effective thickness for glass stress:
$h_{1;ef;\sigma} = [h_{ef;w}{}^3/(h+2\Gamma h_s)]^{1/2}$
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

Exterior installations assumed.
For PVB interlayer shear modulus. G = 70 psi (T ≤ 122 F°)

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

Table 2	h_1, h_2	h_v		$h_{s;1}$ $h_{s;2}$		I_s	h_s	
6mm	0.219	0.06		0.1395		0.0085	0.279	
6mm	0.219	0.06		0.1395		0.0085	0.279	
Shortest Dimension	**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	0.0236	0.2646	0.2861	0.3636	0.3222	0.4052	2201	3480
24	0.0881	0.5900	0.3108	0.4333	0.3510	0.4605	2613	4496
36	0.1785	0.7640	0.3399	0.4628	0.3822	0.4790	3097	4864
41	0.2199	0.8077	0.3517	0.4697	0.3939	0.4829	3289	4944
48	0.2787	0.8520	0.3672	0.4764	0.4085	0.4866	3537	5020
60	0.3764	0.8999	0.3904	0.4835	0.4286	0.4904	3894	5099
72	0.4651	0.9283	0.4093	0.4876	0.4434	0.4925	4169	5143

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 2021 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 $S \leq h$

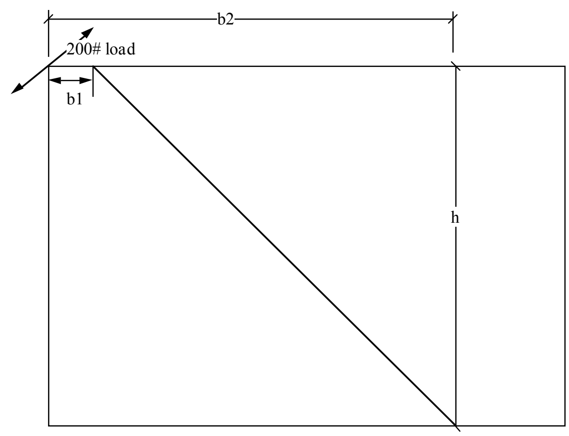
For wind load
$h = (M_{aw}/(0.55W))^{1/2}$
maximum wind load for given light height:
$W = M_{aw}/(0.55h^2)$

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

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 = 145.5/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 9/16" glass = 4.48"
The shear will transfer along the glass at a 45˚ angle to spread across the panel. - Deflection continuity of the glass requires that load be transferred across the full width with decreasing load as it gets farther from the corner.
$b_2 = b_1 + h$

$M_{ave} = 200*h/(b_2)$ 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:
$h \leq 3.75*b_2t^2$

LOAD TESTS ON 9/16" GLASS

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

The laminated glass fabricated with 0.06" SentryGlas+®

Glass test loads: - 38" effective glass cantilever height.
Effective thickness for deflection calculated from maximum deflection measured:
$t = [(P*38^3)/(\Delta*3*4*10.4x10^6)]^{1/3}$
P = load at deflection, Δ
TABLE 3

Test	Max Load	Defl in	Moment/ft	Defl at 800# load	Eff. thickness Defl - Inches	Glass stress at failure	Comment
1	851	4.5	8404	4.250	0.4358	22125	at failure
2	928	4.38	9164	3.750	0.4544	22195	at failure
3	886	4.5	8749	4.125	0.4401	22581	No failure
4	898	4.625	8868	4.375	0.4316	23802	at failure
5	886	4.875	8749	4.125	0.4401	22581	No failure
Ave	889.8	4.576	8787	4.125	0.4404	22657	

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

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

11/16" 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.

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 adjust for service temperature.

For SGP interlayer G = 15,600 psi adjust for service temperature (SentryGlas® product data published by Kuraray).

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

$h_1 = h_2 = 0.292"$

$h_v = 0.06"$

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 4	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:
$h = (M_{aL}/u) = M_{aL}/50plf$

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 $S \leq h$

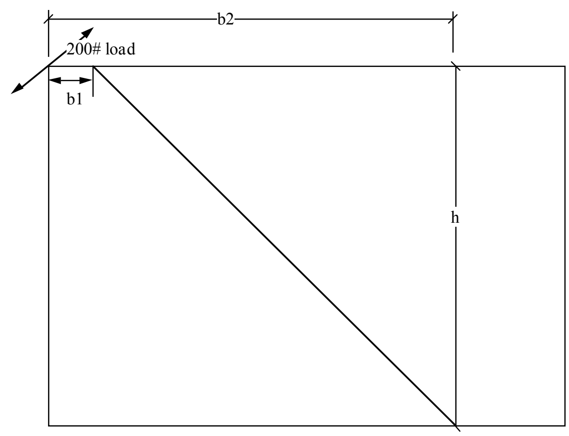
For wind load
$h = (M_{aw}/(0.55W))^{1/2}$
maximum wind load for given light height:
$W = M_{aw}/(0.55h^2)$

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

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:
$h \leq 3.75 * b_2 t^2$

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

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

13/16" 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.
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 adjust for service temperature.
For SGP interlayer G = 15,600 psi adjust for service temperature (SentryGlas® product data published by Kuraray).
The values of G are selected as most appropriate for service conditions and load durations as G decreases with temperature and load duration.

.

$h_1 = h_2 = 0.355"$
$h_v = 0.06"$
a = least width - typically total glass height including portion in base shoe: 41" for 42" overall height including base shoe.
$h_s = 0.5(h_1+h_2)+h_v = 0.5(0.355*2)+0.06 = 0.415"$
$h_{s;1} = h_{s;2} = (h_s h_1)/(h_1+h_2) = (0.415*0.355)/(2*0.355) = 0.208"$
$I_s = h_1 h^2_{s;2} + h_2 h^2_{s;1} = 2*(0.355*0.208"^2) = 0.0306$
$\Gamma = 1/[1+9.6(EI_s h_v)/(Gh^2_s a^2)]$
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

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

Table 5	h_1, h_2	h_v		$h_{s;1}$ $h_{s;2}$		I_s	h_s	
10mm	0.355	0.06		0.2075		0.0306	0.415	
10mm	0.355	0.06		0.2075		0.0306	0.415	
Shortest Dimension	Γ PVB	Γ SGP	$h_{ef;w}$ PVB	$h_{ef;w}$ SGP	$h_{1;ef;\sigma}$ PVB	$h_{1;ef;\sigma}$ SGP	All. wind mom. lb-in/ft PVB	All. wind mom. lb-in/ft SGP
12	0.0094	0.1816	0.4529	0.5384	0.5088	0.6023	5489	7690
24	0.0365	0.4703	0.4686	0.6399	0.5272	0.6901	5892	10096
36	0.0786	0.6664	0.4909	0.6938	0.5525	0.7272	6471	11210
41	0.0996	0.7215	0.5014	0.7075	0.5639	0.7356	6741	11473
48	0.1317	0.7803	0.5165	0.7216	0.5800	0.7440	7131	11734
60	0.1916	0.8473	0.5426	0.7370	0.6064	0.7527	7795	12010
72	0.2544	0.8888	0.5676	0.7462	0.6300	0.7577	8415	12170

Minimum glass thickness from ASTM C1036. If thicker glass is used in fabricating the laminated glass greater effective thicknesses may be calculated based on actual glass thickness.

GLASS PANELS LOADS:
From IBC 1607.9

At top – 200lb concentrated or 50 plf Any direction

Or On panel – 50 lbs on one square foot

Or Wind load on entire area; 10 psf minimum

DETERMINE MAXIMUM PANEL HEIGHT:
For 50 plf distributed load:

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

For 200# load, not top rail:

$h = M_{aL}*S/200\#$ 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 $S \le h$

For wind load

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

maximum wind load for given light height:

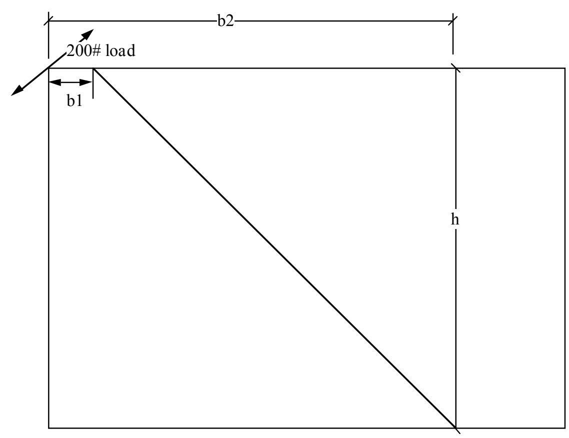
$W = M_{aw}/(0.55h^2)$

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 13/16" glass = 6.16"
The shear will transfer along the glass at a 45˚ angle to spread across the panel. - Deflection continuity of the glass requires that load be transferred across the full width with decreasing load as it gets farther from the corner.
$b_2 = b_1+h$

M_{ave} = 200*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.
$M_{min} = (1/2)M_{max}$
$M_{ave} = (M_{max} + M_{min})/2 = (M_{max} + (1/2)M_{max})/2 = (3/2)M_{max}/2= (3/4)M_{max}$
$M_{max} = 4/3M_{ave} = 1.3333*200*h/(b_2) \le 1000t^2$ (live load allowable stress)
Rearranging and simplifying:
$h \le 3.75*b_2t^2$

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

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

1-1/16" 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.
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 adjust for service temperature.
For SGP interlayer G = 15,600 psi adjust for service temperature (SentryGlas® product data published by Kuraray).
The values of G are selected as most appropriate for service conditions and load durations as G decreases with temperature and load duration.

$h_1 = h_2 = .469"$
$h_v = 0.06"$
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

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

Table 6	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:

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

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 $S \le h$

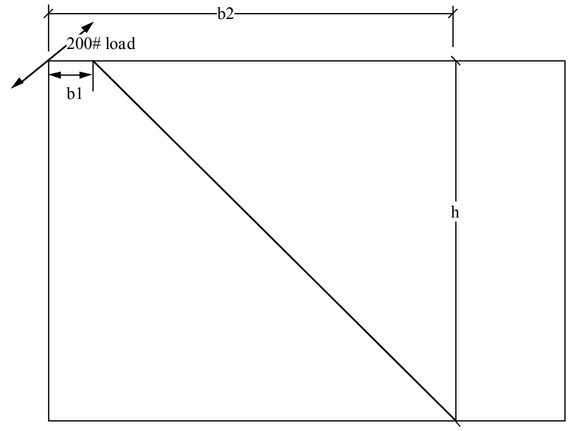
For wind load

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

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

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

FOR INSTALLATION WITHOUT A TOP RAIL

Maximum glass cantilever height based on light width for 200lb live load and no top rail:
Also verify for 50 plf live load- $h \leq 3000*2*t^2/50 = 120t^2$ (allowable stress reduced for residual condition)

<u>**TABLE 7 (9/16" GLASS)**</u>

Light width inches	Effective thickness PVB	Maximum height inches 200# PVB	50 PLF Max height inches PVB	Effective thickness SGP	Maximum height inches 200# load SGP	50 PLF Max height inches SGP
12	0.318	N/A	N/A	0.405	7.4	19.7
24	0.338	N/A	N/A	0.461	19.1	25.4
36	0.364	N/A	N/A	0.479	31.0	27.5
41	0.374	N/A	N/A	0.483	35.9	28.0
45	0.382	N/A	N/A	0.485	39.7	28.25
66	0.382	N/A	N/A	0.485	39.7	28.25
73	0.382	N/A	N/A	0.485	39.7	28.25

Limit effective thickness to value for 45" width.
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).
PVB interlayer should not be used without a 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 9/16" the lights must be connected together at the joints to limit differential deflection. Recommend using mall front clamps, H clip or similar within 12 inches of the top of the glass.
Example: Mall front clamp or structural silicone butt joint full height.

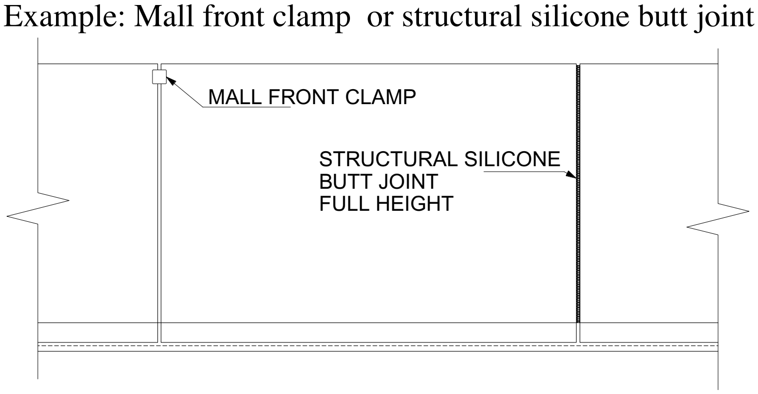
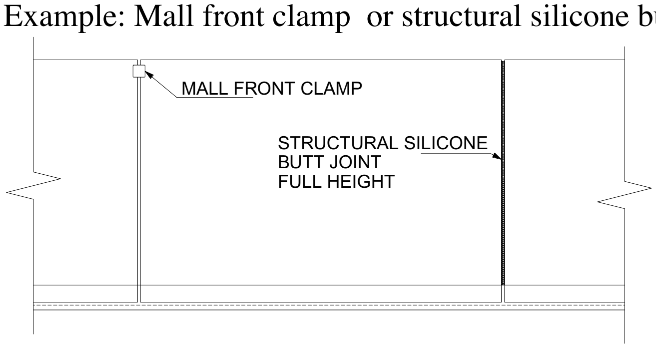


TABLE 8: (11/16" GLASS)

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.
Example: Mall front clamp or structural silicone butt joint full height.



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

TABLE 9 (13/16" GLASS)

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

Deflection limit of 1" applied.
For 42" guard height - required glass cantilever height:
For height inclusive of base shoe h_g = 38.5"
For height above base shoe h_g = 42.5" (42" clear glass height above top of base shoe).
*Maximum allowable height of 50 plf live load above base shoe for code compliance.
†Maximum allowable height based on light width for 200 lb live load and no top rail.

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

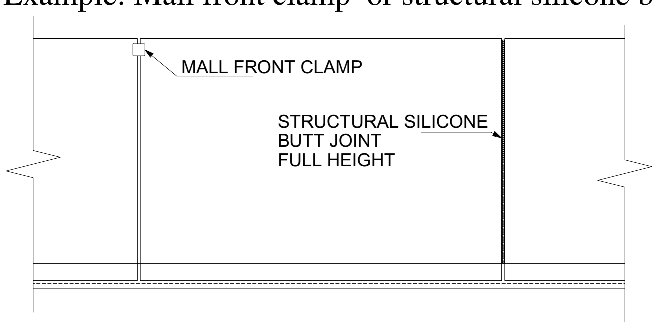


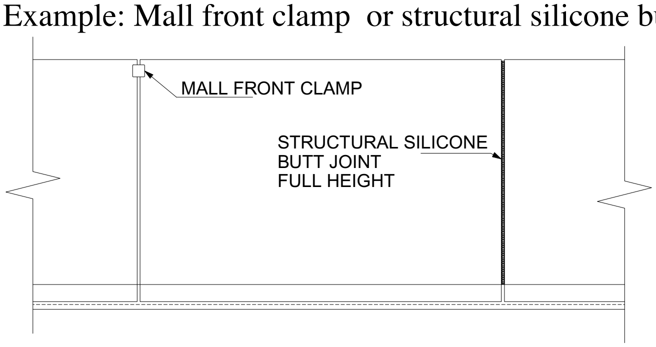
TABLE 10 (1-1/16" GLASS)

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 1" 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.

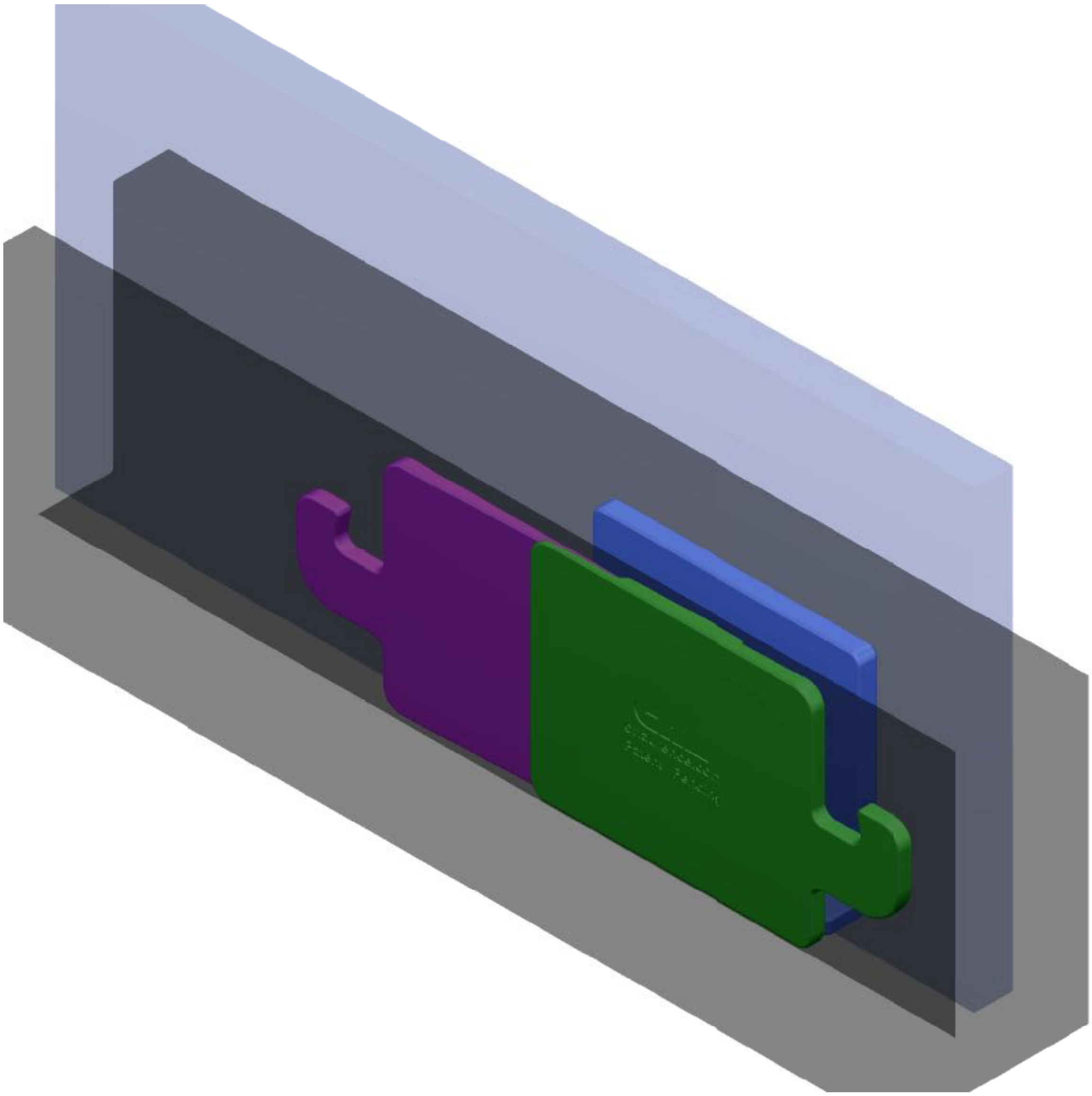
Example: Mall front clamp or structural silicone butt joint full height.



POOL FENCE

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

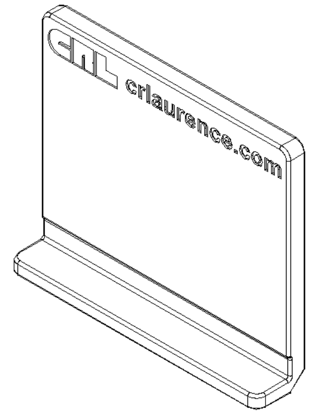
DRY-GLAZE TAPER-LOC SYSTEM



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.

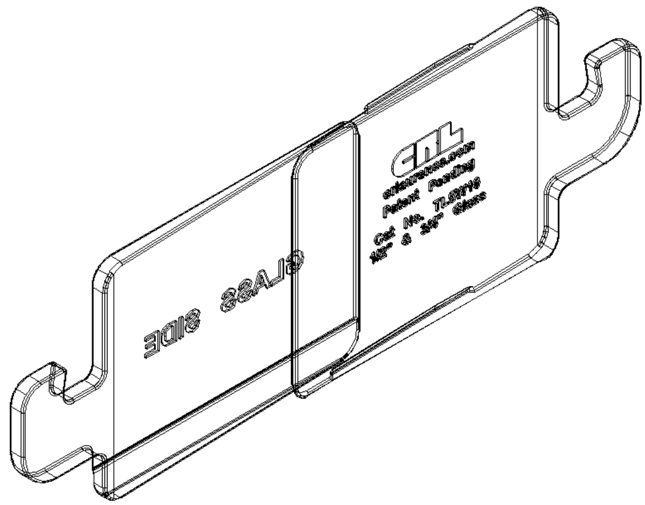
Glass may be extracted by reversing the installation tool to extract tapers.

The Taper-Loc setting plate is set against the glass during installation and then locked into place by the compression of the Taper-Loc shims.



Surface area of the setting plate in contact with the glass:
A = 2”*2.5” = 5 in^2
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} = 150#” design/minimum installation torque (preset on installation tool)
T_{max} = 200#” maximum installation torque
Compressive force generated by the installation torque:
C = (0.2*150#”/1.0”)/ sin(1.76˚)
C = 977#



Frictional force of shims and setting plate against aluminum base shoe:
coefficient of friction, μ= 0.65
f = 2*(977#0.65) = 1,270#

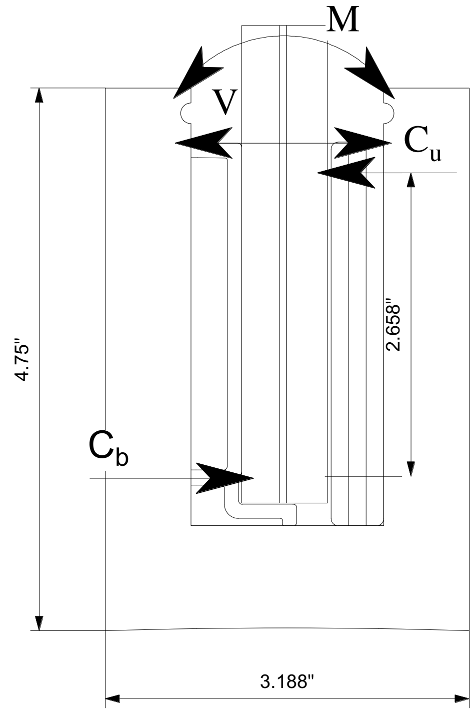
Frictional force of shims against glass:
μ = 0.20
f = 2*977*0.20 = 391#

Resistance to glass pull out:
U = 391# + weight of glass

Safety factor for 200# pullout resistance = 2*391/200 = 3.91
Based on two taper sets
Minimum recommended installation torque:

Extraction force required to remove tapers after installation at design torque:
T = 150*(0.7/0.2) = 525#”

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



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

R_{CU} @ 1/6*3.188 = 0.53"
e = 3.188-0.53 = 2.658"
From $\sum M$ about $R_{CU} = 0$
$0 = M+V*(0.53''/2) - R_{CB}*(2.658-0.53/2)$
Let M = V*42.5" (42" exposed glass height)
M_a = 233.3#' for 13/16" SGP laminated glass
V = 233.3/3.33' = 65.9#
substitute and simplify:
$0 = V*(42.5''+0.265'') - R_{CB}*2.393''$
Solving for - R_{CB}
R_{CB} = 65.9*42.765/2.393 = 1,178#
For C_B = 3,000 psi:
R_{CB} = 3.5"*(3.188"/2)*3,000psi/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 1-1/16" laminated glass based on minimum glass dimension and interlayer.

GLASS STRESS ADJUSTMENTS FOR THE TAPER-LOC SYSTEM

The Taper-Loc System provides a concentrated non-continuous support in the base shoe:
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 when installed per the instructions. 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 and M_r = 24,000 psi

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

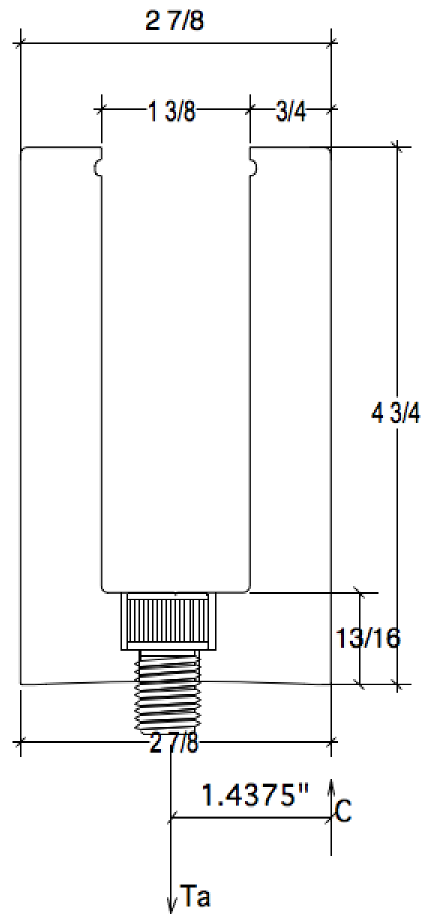
For 1-1/16” laminated glass, 12” width:
$M_a = 2*h_{ef;\sigma}^2*6{,}000$ for live load
V_a = 0.438*12*3,000/7.0 = 2,253# for live load

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)

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

9/16" LAMINATED GLASS BASE SHOE
L56S BASE SHOE
6063-T52 Aluminum extrusion
Fully tempered glass glazed in place, using the Taper-Loc dry-glazing system.

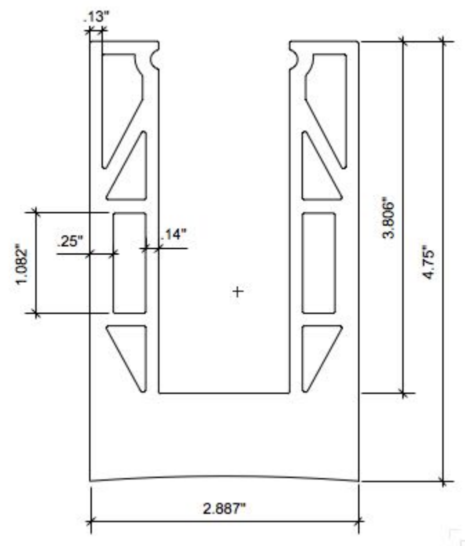


Shoe strength – Vertical legs:
Glass reaction by bearing on legs to form couple. Allowable moment on legs per 2020 ADM Chapter F.
$M_a = 1.5SF_y/\Omega_y$ or $\leq ZF_u/\Omega_r$
S_y = 12"*0.75"^{2}*/6 = 1.125 in^3/ft
Z_y = 12"*0.75"^{2}*/4 = 1.6875 in^3/ft
M_{ay} = 16ksi*1.5*1.125 in^3/ft/1.65 = 16,364#"/ft 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

9BL56 - 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



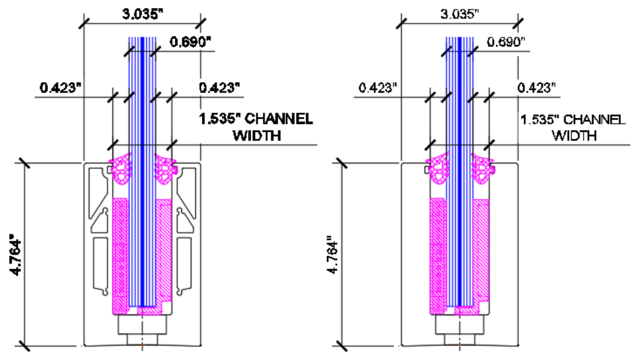
Allowable shear across cell - based on shear bending across cell legs allowing rotation at top
$V_a = [1.5(S_i+S_o)*P_{nt}/b]/\Omega$
S_i, S_o = section modulus of inside or outside leg
b = height of cell = 1.082"
$V_a = [1.5(0.14^2/6+0.25^2/6)*16ksi/1.082"]/1.65 = 1{,}400$ pli Won't control
Strength 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.

L68S10 Solid Base Shoe and
9BL68 Cored Base Shoe

CRL L68S Series Laminated Square Base Shoe and 9BL68 cored shoe.
6063-T52 Aluminum extrusion
Fully tempered glass glazed in place, using the Taper-Loc dry-glazing system.



1/16" (17.52mm) LAMINATED GLASS
9BL68 SERIES BASE SHOE

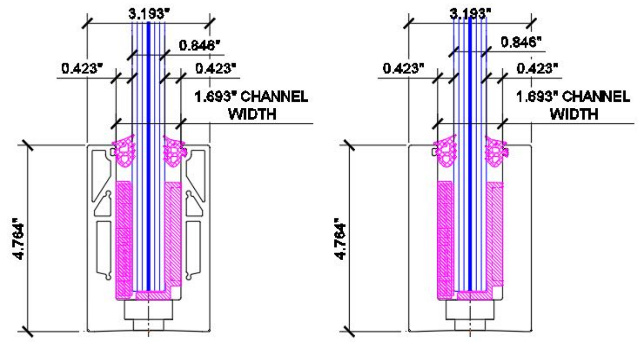
11/16" (17.52mm) LAMINATED GLASS
L68S SERIES BASE SHOE

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 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
Strength of 9BL68 shoe is essentially the same with controlling strength being determined by base shoe anchorage.

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

L21S Solid Base Shoe
9BL21 Cored Base Shoe



FOR 13/16mm LAMINATED GLASS
6063-T52 Aluminum extrusion
Fully tempered glass glazed in place with Taper-Loc-Laminated™ system.
See last page for 9BL21 shoe.
Shoe strength – Vertical legs:
Glass reaction by bearing on legs to form couple. Allowable moment on legs per 2020 ADM Chapter F.
$M_a = 1.5SF_y/\Omega_y$ or $\le ZF_u/\Omega_r$
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
Strength of 9BL21 hoe is essentially the same with controlling strength being determined by base shoe anchorage.

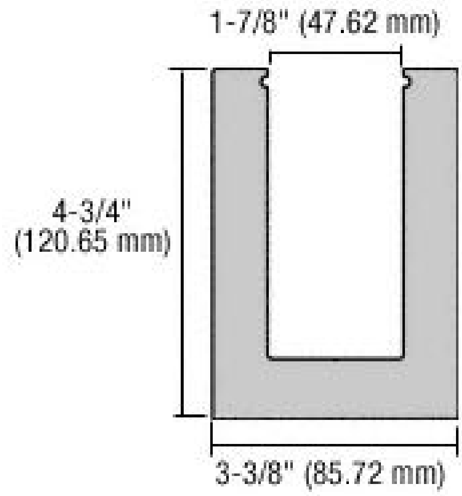
Maximum moment based on glass strength = 12,181”#/ft ≤ 16,364”#/ft
Maximum base shoe leg deflection:

$\Delta = Mh^2/(3Et^3) = 12{,}181*3.75^2/(3*10{,}400{,}000*0.75^3) = 0.013”$

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

$\Delta_{top} = 0.013*45.75/3.75 = 0.16”$

L25S Solid 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 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

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:
$\Delta_{top} = 0.013*45.75/3.75 = 0.16$"

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

L56S AND 9BL56 TYPICAL ANCHORAGE DETAILS

For anchorage to steel assume 1/2" stainless steel cap screw into a tapped hole. Assume at least A36 steel strength. Check pullout, fastener rupture and check bearing/shear failure of the shoe under the head. Pullover failure modes do not apply due to the thickness of the parts and size of the fastener so check for bearing directly under the head or shear through the aluminum shoe directly under the head.

1/2" Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5	0.779	1.12	0.142
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
3	6	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4" or 2.5 for d>1/4"	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	3036	3750	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4" or 2.5 for d>1/4"	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	3834		
Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			

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

Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	4327	5494	
Controlling allowable tension, T_a (lbs)			
3036			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 6" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 12" O.C. , M_a (in-lbs/ft)
0.060	4463	8926	4463

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

L56S and 9BL56 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 = 80 lbs (50 plf live load x 1.6 load factor)
M_u = 80lbs*42" = 3,360"#

Hilti HUS-EZ 3/8" x 4" screw in anchor into 4" deep holes. Installation per ESR-3027.
Nominal embed depth = 3.25"; Effective embed depth = 2.5":
For anchors at 12" on center:
For 4,000 psi cracked concrete:

3 Tension load

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

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

4 Shear load

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

For shear loads less than 20% of strength there is no reduction in the tension load strength:
V ≤ 0.2*3111 = 622# - As this greatly exceeds wind loads can check capacity based only on

For 2,500 psi uncracked concrete

Moment resistance of each anchor:
ϕM_n = 2,546#*[1.4375-0.5*2,546/(2*0.85*2.5ksi*12)] = 3,607#" = 300.58#' per anchor
$M_a = \phi M_n/\lambda$ = 300.58'#/1.6 = 187.86#'

For 6" spacing:
ϕM_n = 2*2,546#*[1.4375-0.5*2,546/(2*0.85*2.5ksi*6)] = 7,108#" = 592.3#' per anchor
$M_a = \phi M_n/\lambda$ = 7,108"#/1.6 = 4,442#'

L56S and 9BL56 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” socket head 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; $F'_{cT}*C_d*C_b$ = 560psi*1.33 = 745psi

For C = T =2,000#

A = 2,000#/745psi = 2.685in^2

b = A/(12”) = 2.685/(12) = 0.224”

M_a = 2,000#*(1.4375-0.224/2) = 2,651#” = 220.9#’ For 12” o.c. spacing

M_a = 2*2,000#*(1.4375-2*0.224/2) = 4,854#” = 220.9#’

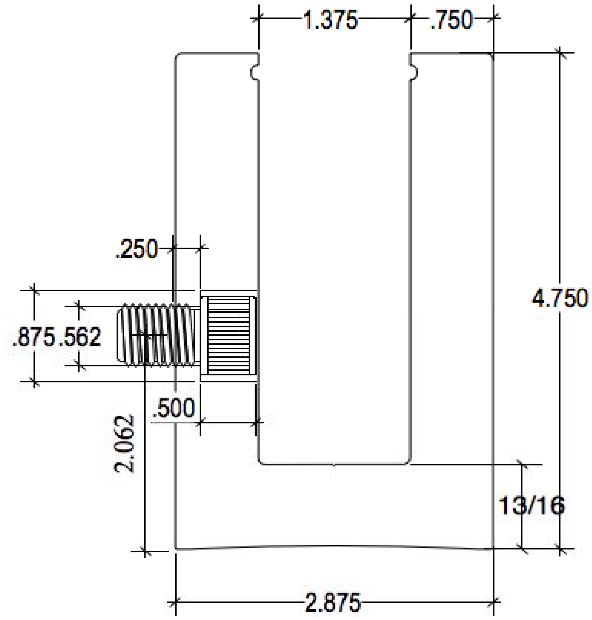
NOTE: DO NOT DIRECTLY LAG BASE SHOE TO WOOD WHERE EXPOSED TO WEATHER OR DIRECT SUNLIGHT BECAUSE BASE SHOE WILL LOOSEN WITH TIME AND WILL NOT BE ADEQUATELY ANCHORED.

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 L56S and 9BL56 base shoe strength - Must verify glass strength too.

Table 11		Allowable wind load in psf						
Surface Mounted	Allowable. Moment in-lbs/ft	Overall Guard height from bottom of base shoe top of top rail, ft.						
Mounting Substrate		3.00	3.25	3.5	3.75	4.0	4.5	5.0
Steel 12” o.c	4463.2	75.1	64.0	55.2	48.1	42.3	33.4	27.0
Steel 6” o.c	8926.5	150.3	128.0	110.4	96.2	84.5	66.8	54.1
Concrete 12” o.c.	2254.0	37.9	32.3	27.9	24.3	21.3	16.9	13.7
Concrete 6” o.c.	4442.0	74.8	63.7	54.9	47.9	42.1	33.2	26.9
Wood 12” o.c.	2651.0	44.6	38.0	32.8	28.6	25.1	19.8	16.1
Wood 6” o.c.	4854.0	81.7	69.6	60.0	52.3	46.0	36.3	29.4

L56S and 9BL56 Fascia Mounted Base Shoe:



M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
4.12	6	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	

Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4" or 2.5 for d>1/4"	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		
Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			
Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 6" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 12" O.C. , M_a (in-lbs/ft)
0.058	5983	11966	5983

For anchor into concrete:

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

Strength same as previously calculated,

M_a=øM_n/1.6= 2,546#*[2.25-0.5*2,546/(2*0.85*2.5ksi*12)]/1.6 = 3,547#" = 295.6#' per anchor

M_a=øM_n/1.6= 2*2,546#*[2.25-0.5*2,546/(2*0.85*2.5ksi*6)]/1.6 = 7,002#"/ft anchors 6" o.c

Lag screw strength same as previously calculated.

T_a = 2,005#

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

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

Allowable wind load on balustrade must be reduced for the dead load moment effect
$V_d = h_g$*6.8psf + 15psf
$M_d = [h_g$*6.8psf + 15psf]*1.52"   10.5 plf for base shoe and glazing + 4 plf for cap rail
h_g = actual height of glass (Typical approx 3.833' for 42" guard height above finish floor)
Assume h_g = guard height in feet + 0.333'
$M_d = h_g$*10.3"#/ft + 22.8"#/ft = 10.3h + 26.2"#
V_d = (h+0.333)*6.8psf + 15psf = (6.8h + 17.3)plf

Since the total shear load will typically by less than 20% of the shear strength for steel and concrete installations there is nor reduction required for combined shear and tension load on anchors.

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:

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

Assume T = 2000#

$\alpha = \tan^{-1}2000/41 = 88.83°$

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

Allowable tension component for 47# shear:

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

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

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

$M_a = 2*2{,}000\#*(2.25"-2*0.224/2) - 12.6h - 27"\# = 8{,}104"\# - 12.6h$ 6" 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 12		Allowable wind load in psf						
Fascia Mounted	Allowable. Moment in-lbs/ft	Overall Guard height from bottom of base shoe top of top rail, ft.						
Mounting Substrate		3.00	3.25	3.5	3.75	4.0	4.5	5.0
Steel 12" o.c	5983.2	99.8	85.0	73.2	63.8	56.0	44.2	35.8
Steel 6" o.c	11966.4	200.5	170.8	147.2	128.2	112.7	89.0	72.1
Concrete 12" o.c.	3547.0	58.8	50.0	43.1	37.5	33.0	26.0	21.0
Concrete 6" o.c.	7002.0	116.9	99.6	85.8	74.7	65.7	51.8	42.0
Wood 12" o.c.	4250.0	70.6	60.1	51.8	45.1	39.6	31.3	25.3
Wood 6" o.c.	8104.0	135.5	115.4	99.5	86.6	76.1	60.1	48.6

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

L68S AND 9BL68 TYPICAL ANCHORAGE DETAILS

For anchorage to steel assume M14 stainless steel cap screw into a tapped hole. Assume at least A36 steel strength. Check pullout, fastener rupture and check bearing/shear failure of the shoe under the head. Pullover failure modes do not apply due to the thickness of the parts and size of the fastener so check for bearing directly under the head or shear through the aluminum shoe directly under the head.

Holes are typically drilled at 30cm apart which is equivalent to 11.81".

M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
3	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4" or 2.5 for d>1/4"	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4" or 2.5 for d>1/4"	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		
Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			

Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2 - (D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	4331	8801	4400

L68S and 9BL68 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 318-19 Chapter 17 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-4 interior or HSL-3-R exterior

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

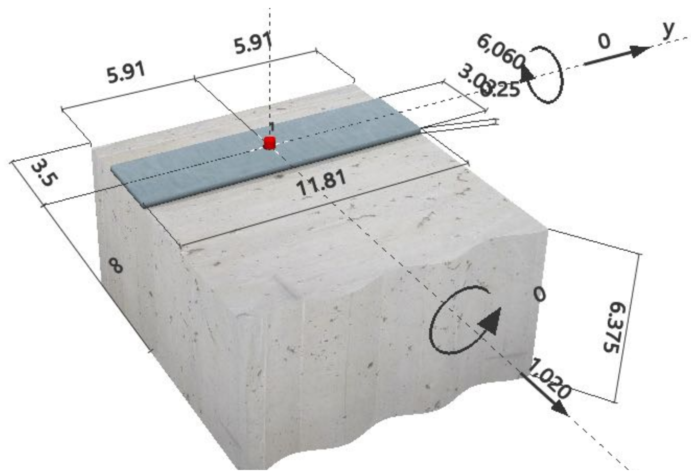
For anchors at 11 13/16" on center: For 4,000 psi cracked concrete:

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

V ≤ 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

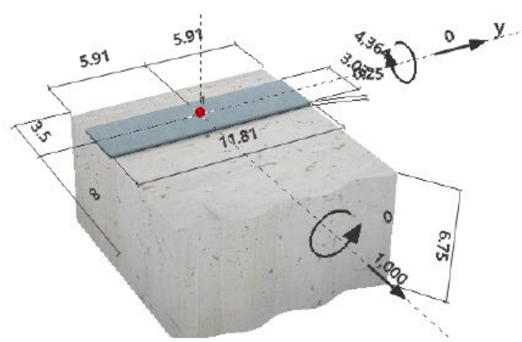
Maximum shear V_u = 0.2*5,098 = 1,020#

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)

L68S and 9BL68 ALTERNATIVE ANCHORAGE TO CONCRETE Hilti HUS-EZ CRC (KH-EZ CRC) 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 CRC (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

L68S and 9B68 FOR 1/2" HUS CRC 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
$M_a = 1{,}658/0.4925 = 3{,}365''\#/\text{ft}$

3 Tension load

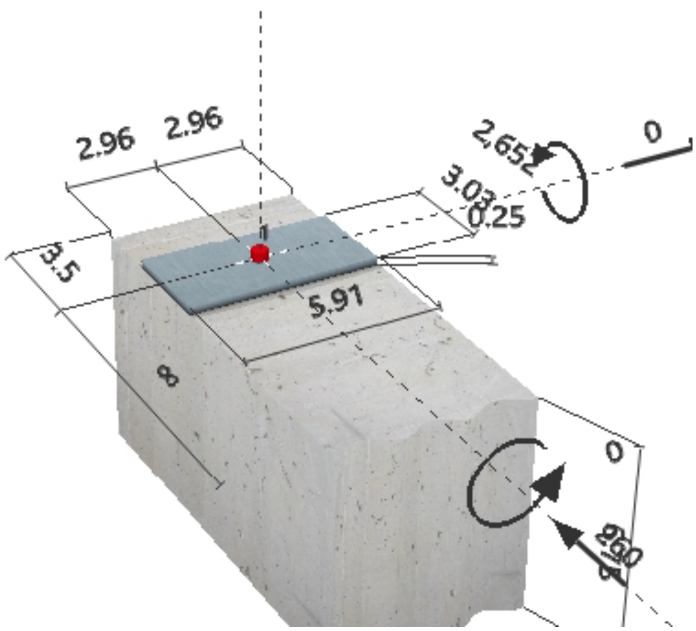
	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	2008	11778	18	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	2008	2009	100	OK

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



L68S and 9BL68 ALTERNATIVE ANCHORAGE TO CONCRETE Hilti HUS-4 M14x135mm OR HUS-EZ Stainless Steel 1/2" x 5"

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 318-25 Chapter 17. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 11 3/8" segment of base shoe and supporting concrete.

The analysis of this anchorage method utilizes the narrow base plate factor:
$\Psi_{cm,N} = 2\text{-}d/(1.5h_{ef}) > 1.0$
This accounts for the increased breakout strength caused by the overlap of the base shoe compression reaction on the concrete breakout wedge.

Unit loads used in the reports:
V_u = 1.6 load factor; M_u = 1.6 load factor

Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22":
Minimum concrete thickness = 6.75"
Edge distance CL shoe to edge of concrete = 2.0"
For anchors at 12" on center: For 4,000 psi cracked concrete:
For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:
$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$

Shear and tension strength as calculated on the next pages:

2" minimum edge distance
V_a = 472#
T_a = 2491#
$M_a = T_a*b/2 = 2491*3.036/2 = 3{,}781$

Based this evaluation the same strengths may be conservatively assumed as the same for the three variants of the HUS anchor - HUS-4 SS M14x135mm, HUS-EZ Stainless Steel or CRC 1/2" x 5", for anchoring all the base shoes in this report

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

MINIMUM EDGE DISTANCE: 2.0”

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-25 Chapter 17. Post installed anchors with narrow baseplate factor per ACI318-25 eq 17.6.2.7.1							
f’c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)	Anchor spacing perpendicular to edge (in)	Concrete thickness (in)	D (in)	Lever arm to anchor, d (in)
4000	3.26	2	12	0	7	0.556	1.5175
Area calculations, assumes one anchor in tension							
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)	Cac			
9	67.3842	18	95.6484	7.5			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)	
	1	0.823	1	1.0000	1330	547	
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	$\Psi_{cm,N}$ = 2-d/(1.5hef) > 1.0	N_b	N_{cbg} (lbs)
	1	0.823	1	0.652	1.690	6329	4041
Shear pryout	k_{cp}	V_{cbg} (lbs)					
	2	8082					
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)						
	N/A does not control						
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail		
0.65	0.7	1.6	239	80	Pass		
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail					
1642	0	Pass					
$øM_n/ALF=øT_n/ALF*d$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a < 1.2$					
2491	2100	1.177					
	Pass	<1.2 Pass					

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

L68S and 9BL68 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.

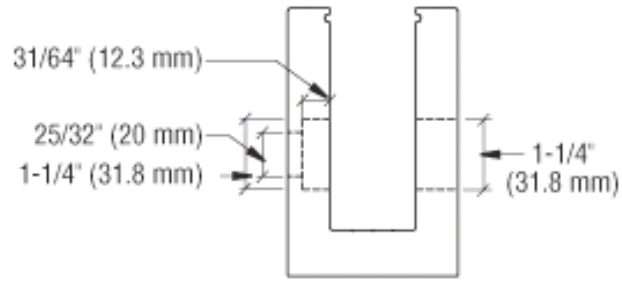
EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Summary of surface mounted L68S base shoe strength - Must verify glass strength too.

Table 13		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	4400.4	74.1	63.1	54.4	47.4	41.7	32.9	26.7
Steel 5-7/8” o.c	8800.8	148.2	126.2	108.9	94.8	83.3	65.8	53.3
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 HUS-EZ* 11-13/16” o.c.	2762.0	46.5	39.6	34.2	29.8	26.2	20.7	16.7
Concrete 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

*** HUS-EZ CRC or SS 1/2” x 5” or HUS-4 SS M14x135mm**

L68S and 9BL68 Fascia Mounted Base Shoe:



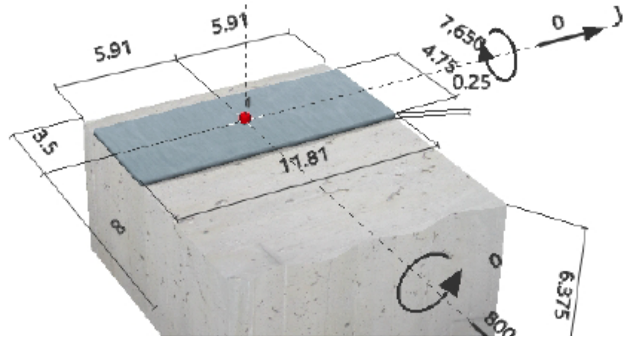
Hole Pattern "F"

M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
4.12	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		

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

Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			
Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	5980	12153	6077

For anchor into concrete - fascia mounted:
Hilti M12 HSL-4 interior or HSL-3-R exterior
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)

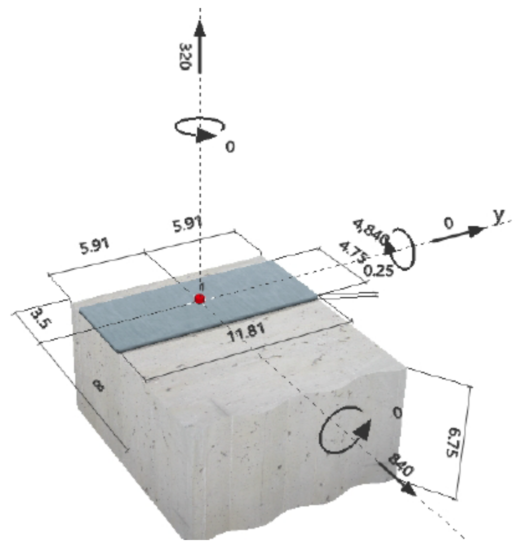
Allowable wind load on balustrade must be reduced for the dead load moment effect
$V_d = h_g$*8.3psf + 15psf (10.5 plf for base shoe and glazing + 4.5 plf for cap rail)
$M_d = [h_g$*8.3psf + 15psf]*1.52”
h_g = actual height of glass (Typical approx 3.833’ for 42” guard height above finish floor)
Assume 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.

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

L68S and 9BL68 ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED
Hilti HUS-EZ CRC (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 CRC (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

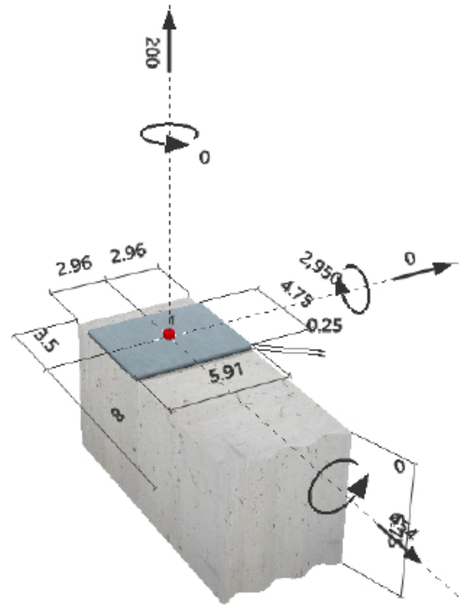
β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.191	1.000	100	OK

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

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

L68S and 9BL68 ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED
5.91” o.c. Hilti HUS-EZ CRC (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 CRC (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

L68S and 9BL68 ALTERNATIVE ANCHORAGE TO CONCRETE FASCIA MOUNTED Hilti HUS-4 SS M14x135mm OR HUS-EZ Stainless Steel 1/2" x 5"

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 318-25 Chapter 17. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 11 3/8" segment of base shoe and supporting concrete.

The analysis of this anchorage method utilizes the narrow base plate factor:
$\Psi_{cm,N} = 2\text{-}d/(1.5h_{ef}) > 1.0$
This accounts for the increased breakout strength caused by the overlap of the base shoe compression reaction on the concrete breakout wedge.

Unit loads used in the reports:
$V_u = 1.6$ load factor; $M_u = 1.6$ load factor

Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22":
Minimum concrete thickness = 6.75"
Edge distance CL shoe to edge of concrete = 2.0"
For anchors at 12" on center: For 4,000 psi cracked concrete:
For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:
$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$

Shear and tension strength as calculated on the next pages:

2" minimum edge distance
$V_a = 472\#$
$T_a = 3787\#$
$M_a = T_a*b/2 = 3787*4.75/2 = 8{,}994"\#$

Based this evaluation the same strengths may be conservatively assumed as the same for the three variants of the HUS anchor - HUS-4 SS M14x135mm, HUS-EZ Stainless Steel or CRC 1/2" x 5", for anchoring all the base shoes in this report.

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

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-25 Chapter 17. Post installed anchors with narrow baseplate factor per ACI318-25 eq 17.6.2.7.1							
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)	Anchor spacing perpendicular to edge (in)	Concrete thickness (in)	D (in)	Lever arm to anchor, d (in)
4000	3.26	2.375	12	0	7	0.556	2.375
Area calculations, assumes one anchor in tension							
A_{Vc} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)	Cac			
12.691	71.0517	25.383	95.6484	7.5			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)	
	1	0.846	1	1.0000	1721	728	
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	$\Psi_{cm,N} = 2-d/(1.5hef) > 1.0$	N_b	N_{cbg} (lbs)
	1	0.846	1	0.652	1.514	6329	3925
Shear pryout	k_{cp}	V_{cbg} (lbs)					
	2	7851					
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)						
	N/A does not control						
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail		
0.65	0.7	1.6	318	80	Pass		
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail					
1595	0	Pass					
$øM_n/ALF=øT_n/ALF*d$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a < 1.2$					
3787	2320	0.864					
	Pass	<1.2 Pass					

L68S and 9BL68 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 ≥ 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 14		**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	6077	101.2	86.2	74.3	64.7	56.8	44.8	36.3
Steel 5-7/8" o.c	12153.2	203.5	173.4	149.4	130.1	114.4	90.3	73.1
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 HUS-EZ* 11-13/16" o.c.	3074.0	50.7	43.1	37.1	32.3	28.4	22.4	18.1
Concrete 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

*** HUS-EZ CRC or SS 1/2" x 5" or HUS-4 SS M14x135mm**

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.

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

L21S10D AND L21SDC TYPICAL ANCHORAGE DETAILS

For anchorage to steel assume M14 stainless steel cap screw into a tapped hole. Assume at least A36 steel strength. Check pullout, fastener rupture and check bearing/shear failure of the shoe under the head. Pullover failure modes do not apply due to the thickness of the parts and size of the fastener so check for bearing directly under the head or shear through the aluminum shoe directly under the head.

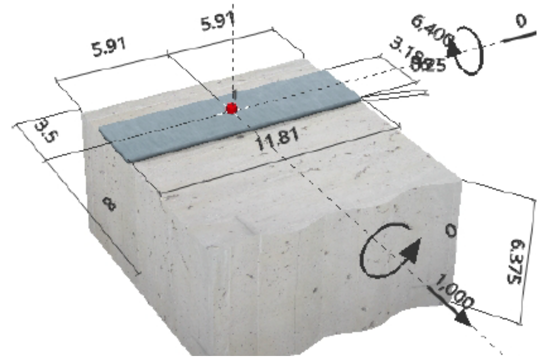
M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
3.19	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		
Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			

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

Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	4611	9370	4685

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

L21S and 9BL21 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 = (wind or live load x 1.6 load factor)
M_u
Hilti M12 HSL-4 interior or HSL-3-R exterior
Nominal embed depth = 3.25"; Effective embed depth = 2.5":
For anchors at 11.81" on center:

3 Tension load

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

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

4 Shear load

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

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

For 4,000 psi cracked concrete:
For shear loads less than 20% of strength there is no reduction in the tension load strength:
$V \leq 0.2*3111 = 622\#$ - As this greatly exceeds wind loads can check capacity based only on tension strength and tension load

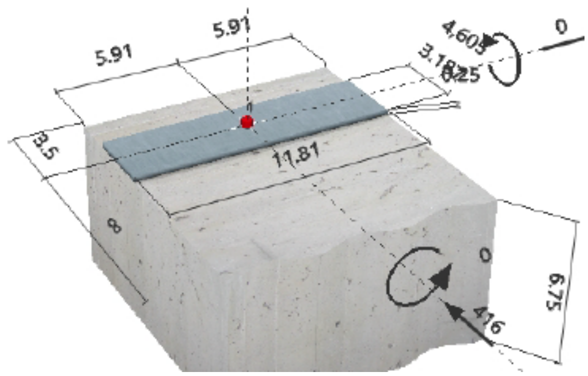
For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:
$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$

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

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

L21S and 9BL21 ALTERNATIVE ANCHORAGE TO CONCRETE
Hilti HUS-EZ CRC (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 CRC (KH-EZ) 1/2" Diameter
Nominal embed depth = 4.25" (hole depth); Effective embed depth = 3.22":
Minimum concrete thickness = 6.75"
For anchors at 11 13/16" on center: For 4,000 psi cracked concrete:
For 2,500 psi uncracked concrete strength is slightly more than for 4,000 psi cracked:
$\sqrt{4000}/1.4 = 45.2 \leq \sqrt{2500} = 50$
Maximum moment found by iteration (outward load controls)
Shear load: V_u = 416#; V_a = 416/1.6 = 260# per anchor
V_a = 260/0.984 = 264 plf
Moment load: M_u = 4,605"# ; M_a = 4,605/1.6 = 2,878"# per anchor
M_a = 2,878/0.984 = 2,925"#/ft

3 Tension load

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

4 Shear load

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

5 Combined tension and shear loads

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

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

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

L21S and 9BL21 FOR HUS ANCHORS AT 5.9” ON CENTER

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

Shear load: V_u = 201#; V_a = 201/1.6 = 126# per anchor
V_a = 126/0.4925 = 256 plf
Moment load: M_u = 2,795”# ; M_a = 2,795/1.6 = 1,747”# per anchor
M_a = 1,747/0.4925 = 3,547”#/ft

3 Tension load

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

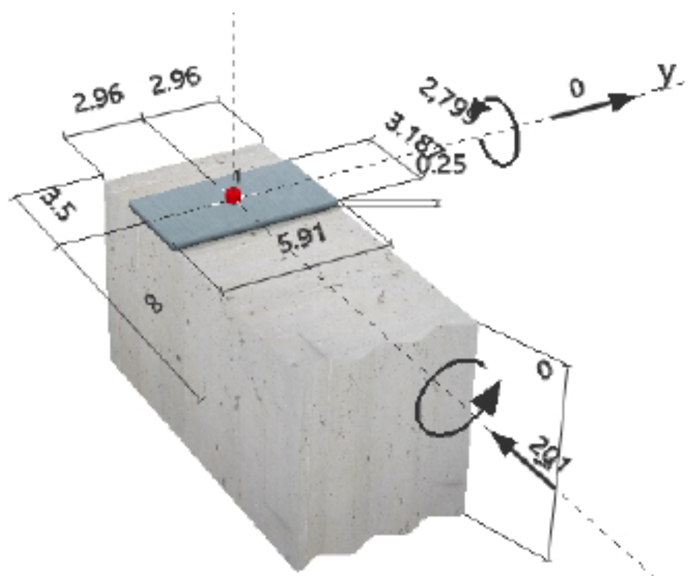
4 Shear load

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

5 Combined tension and shear loads

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

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



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

L21S and 9BL21 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 =2,005#

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

b = A/(12”) = 2.691/(12) = 0.224”

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

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

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

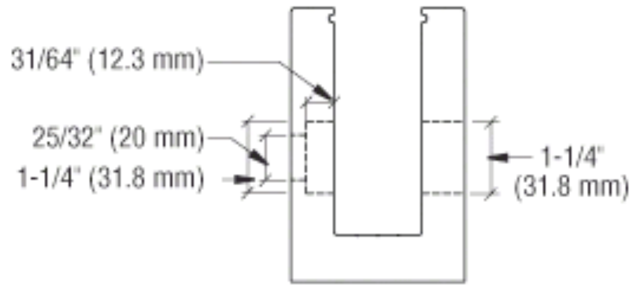
EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

Summary of surface mounted L21S and 9BL21 base shoe strength - Must verify glass strength too.

Table 15		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	4684.8	78.9	67.2	57.9	50.5	44.4	35.1	28.4
Steel 5-7/8” o.c	9369.5	157.7	134.4	115.9	101.0	88.7	70.1	56.8
Concrete 12M HSL 11-13/16” o.c.	4000.0	67.3	57.4	49.5	43.1	37.9	29.9	24.2
Concrete HUS-EZ* 11-13/16” o.c.	2925.0	49.2	42.0	36.2	31.5	27.7	21.9	17.7
Concrete HUS-EZ* 5-7/8” o.c.	3547.0	59.7	50.9	43.9	38.2	33.6	26.5	21.5
Wood 11-13/16” o.c.	2971.0	50.0	42.6	36.7	32.0	28.1	22.2	18.0
Wood 5-7/8” o.c.	5610.0	94.4	80.5	69.4	60.4	53.1	42.0	34.0

*** HUS-EZ CRC or SS 1/2” x 5” or HUS-4 SS M14x135mm**

L21S and 9BL21 Fascia Mounted Base Shoe:



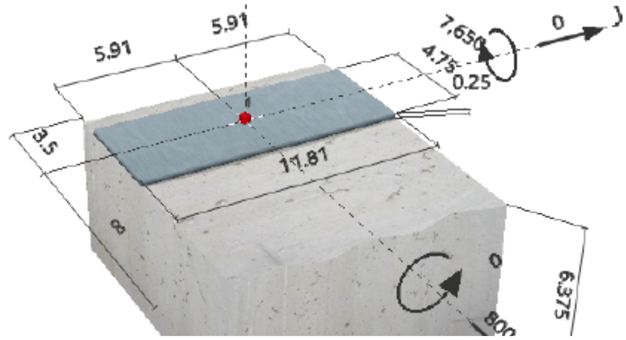
Hole Pattern "F"

M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
4.12	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		

Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			
Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	5980	12153	6077

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

For anchor into concrete - fascia mounted:
Hilti M12 HSL-4 interior or HSL-3-R exterior
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)

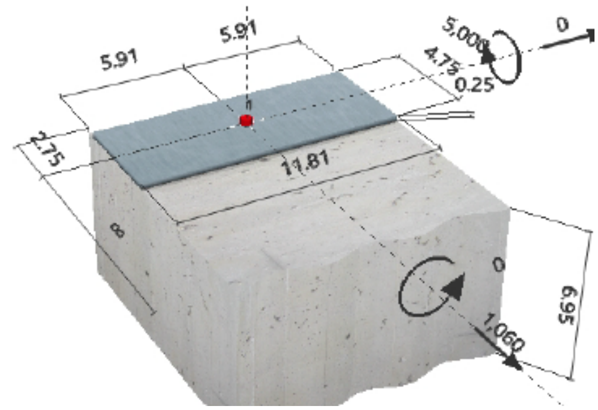
Allowable wind load on balustrade must be reduced for the dead load moment effect
$V_d = h_g$*8.3psf + 15psf (10.5 plf for base shoe and glazing + 4.5 plf for cap rail)
$M_d = [h_g$*8.3psf + 15psf]*1.52"
h_g = actual height of glass (Typical approx 3.833' for 42" guard height above finish floor)
Assume 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.

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

L21S and 9BL21 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

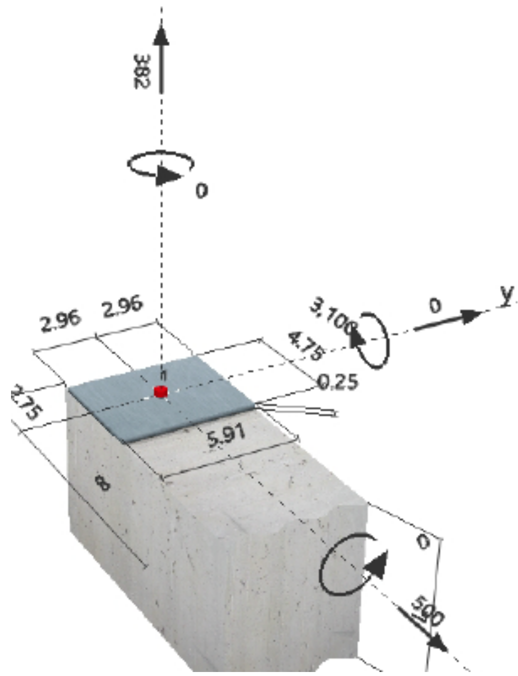
	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1060	5547	20	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1060	5943	18	OK
Concrete edge failure in direction y-**	1060	5301	20	OK

5 Combined tension and shear loads

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

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

L21S and 9BL21 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'$_c$ ≥ 4,000 psi for cracked condition or f'$_c$ ≥ 2,500 psi for uncracked condition. The post-installed concrete anchor strength was determined according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 5.91" segment of base shoe and supporting concrete.

Unit loads used in the reports:

V_u = 1.6 load factor; M_u

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

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

Minimum concrete thickness = 6.75"

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

Maximum moment found by iteration (outward load controls)

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

V_a = 284/0.4925 = 577 plf

Moment load: M_u = 3,100"# ; 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:

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

Assume T = 2000#

$\alpha = \tan^{-1} 2000/53 = 87.62°$

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

Allowable tension component for 47# shear:

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

Allowable wind loads:

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

M_a = 2*1,994#*(2.25"-2*0.224/2) - 14.9h - 28"# = 8,052"# - 14.9h 5-7/8" o.c.

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

Summary of fascia mounted L21S base shoe strength - Must verify glass strength too.

Table 16		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	6076.6	101.1	86.1	74.2	64.6	56.7	44.8	36.2
Steel 5-7/8" o.c	12153.2	203.4	173.2	149.3	130.0	114.3	90.2	73.0
Concrete 12M HSL 11-13/16" o.c.	4781.0	79.3	67.5	58.1	50.6	44.4	35.1	28.4
Concrete HUS-EZ* 11-13/16" o.c.	3074.0	50.5	43.0	37.0	32.2	28.3	22.3	18.0
Concrete HUS-EZ* 5-7/8" o.c.	3934.0	65.0	55.3	47.7	41.5	36.4	28.7	23.2
Wood 11-13/16" o.c.	4235.0	70.1	59.7	51.4	44.7	39.3	31.0	25.0
Wood 5-7/8" o.c.	8052.0	134.3	114.4	98.6	85.9	75.4	59.5	48.2

*** HUS-EZ CRC or SS 1/2" x 5" or HUS-4 SS M14x135mm**

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.

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

L25S10D TYPICAL ANCHORAGE DETAILS

For anchorage to steel assume M14 stainless steel cap screw into a tapped hole. Assume at least A36 steel strength. Check pullout, fastener rupture and check bearing/shear failure of the shoe under the head. Pullover failure modes do not apply due to the thickness of the parts and size of the fastener so check for bearing directly under the head or shear through the aluminum shoe directly under the head.

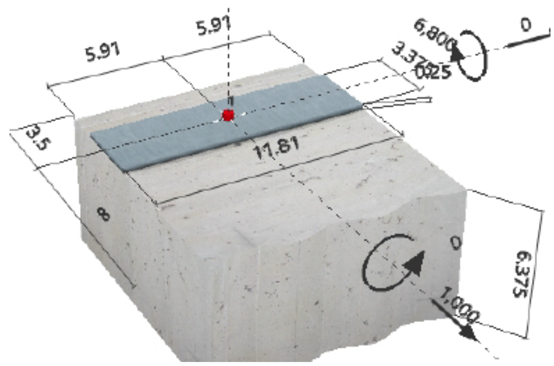
M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
3.375	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		
Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			

Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2 - (D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	4883	9923	4962

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

L25S 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-4 interior or HSL-3-R exterior
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 ≤ 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)

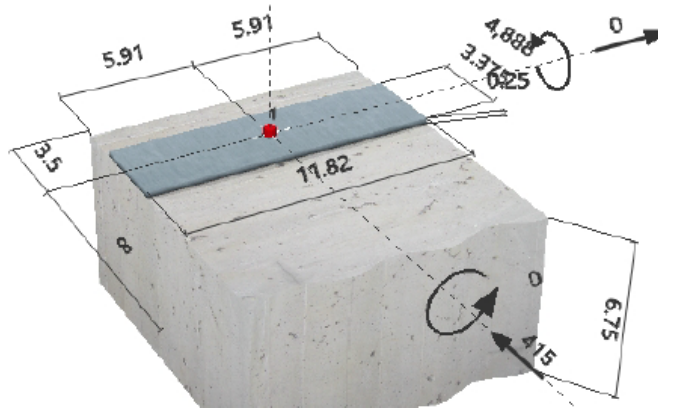
4 Shear load

	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)

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

L25S 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

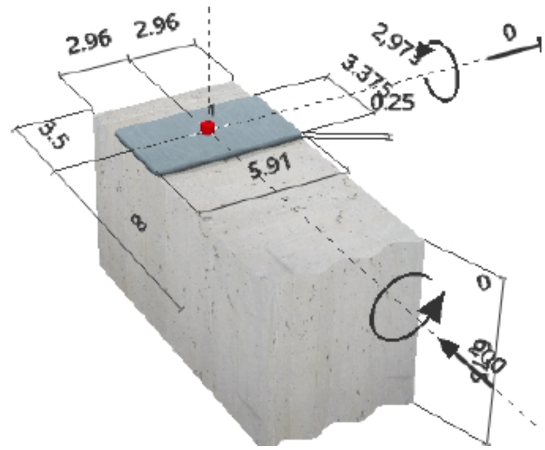
β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
1.000	0.199	1.000	100	OK

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

L25S HUS ANCHORS AT 5.9" ON CENTER

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

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]

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

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

L25S 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.

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

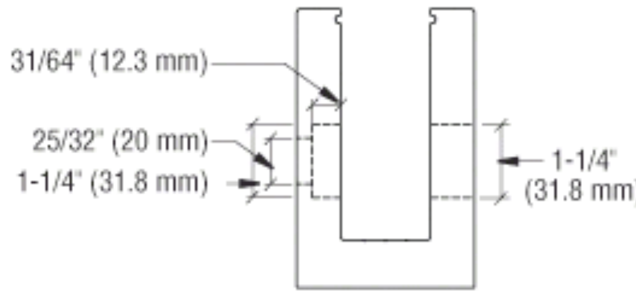
Summary of L25S surface mounted base shoe strength - Must verify glass strength too.

Table 17		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	4961.6	83.5	71.2	61.4	53.5	47.0	37.1	30.1
Steel 5-7/8” o.c	9923.3	167.1	142.3	122.7	106.9	94.0	74.2	60.1
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 HUS-EZ* 11-13/16” o.c.	3105.0	52.3	44.5	38.4	33.5	29.4	23.2	18.8
Concrete 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

*** HUS-EZ CRC or SS 1/2” x 5” or HUS-4 SS M14x135mm**

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.

L25S Fascia Mounted Base Shoe:

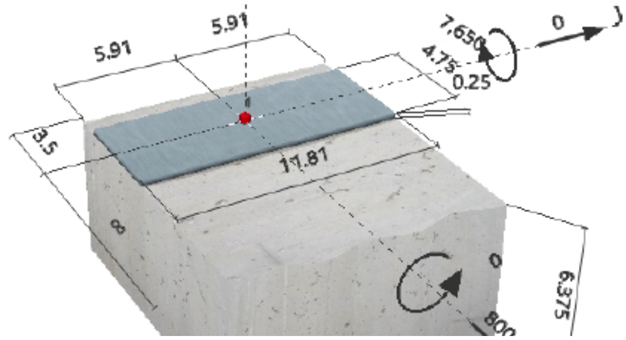


M14 Cap Screw:			
Screw Diameter, D (in)	Screw external thread stripping area, $A_{s,ext}$ (in^2/in)	Screw external thread stripping area, $A_{s,int}$ (in^2/in)	Screw net tensile area, A_{net} (in^2)
0.5512	0.7559	1.0551	0.1789
Screw ultimate strength, $F_{u,screw}$ (ksi)	Screw penetration, P (in)	Screw head diameter, D_h (in)	
67.5	0.25	0.8268	
Substrate ultimate strength, $F_{u,sub}$ (ksi)	Base shoe yield strength, $F_{y,base}$ (ksi)	Base shoe yield strength, $F_{u,base}$ (ksi)	
58	16	22	
Base shoe width, b (in)	Effective length of shoe, L (in)	Thickness of shoe below head, t (in)	
4.12	5.81	0.3125	
Pullout strength per AAMA TIR-A9-14 10.0.			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	External thread stripping strength, $T_a = A_{s,ext}*F_{u,screw}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	Internal thread stripping strength, $T_a = A_{s,int}*F_{u,base}*P/(\Omega*(3)^{1/2})*1000$ (lbs)	
2.5	2946	3533	
Fastener strength per AAMA TIR-A9-14 7.0			
Ω= 3.0 for d≤1/4” or 2.5 for d>1/4”	$T_a = A_{net}F_{u,screw}*1000/\Omega$ (lbs)		
2.5	4829		

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

Pullover/Bearing under head strength per ADM 2020 J.7 and J.8. Check bearing directly under screw head and shear through the thickness of shoe below the screw head.			
Ω per ADM 2020	Bearing under head, $T_a = 1.33\pi/4(D_h^2-(D+1/16")^2)*F_{u,base}/(\Omega)$ (lbs)	Shear through shoe below head, $T_a = 1000\pi D_h*0.6*F_{u,base}*t/\Omega$ (lbs)	
1.95	3617	5494	
Controlling allowable tension, T_a (lbs)			
2946			
Base shoe anchorage moment strength:			
Bearing blocking width, $a = T_a/(L*1000*F_{u,base}/(1.33*1.95))$	Allowable moment per screw, $M_a = T_a*(b/2-a/2)$ (in-lbs)	Allowable moment for screws at 5.9" O.C. , M_a (in-lbs/ft)	Allowable moment for screws at 11.81" O.C. , M_a (in-lbs/ft)
0.060	5980	12153	6077

For anchor into concrete - fascia mounted:
Hilti M12 HSL-4 interior or HSL-3-R exterior
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

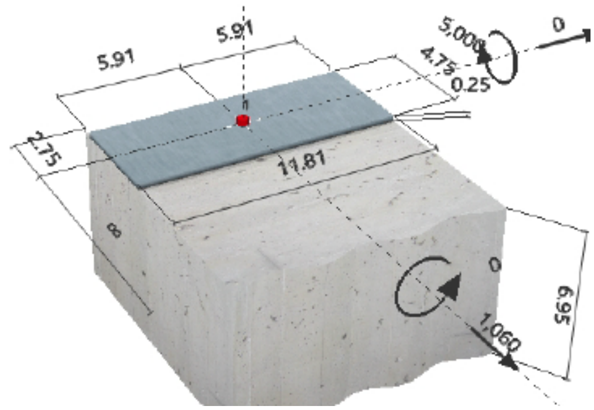
* anchor having the highest loading **anchor group (relevant anchors)

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.

L25S 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

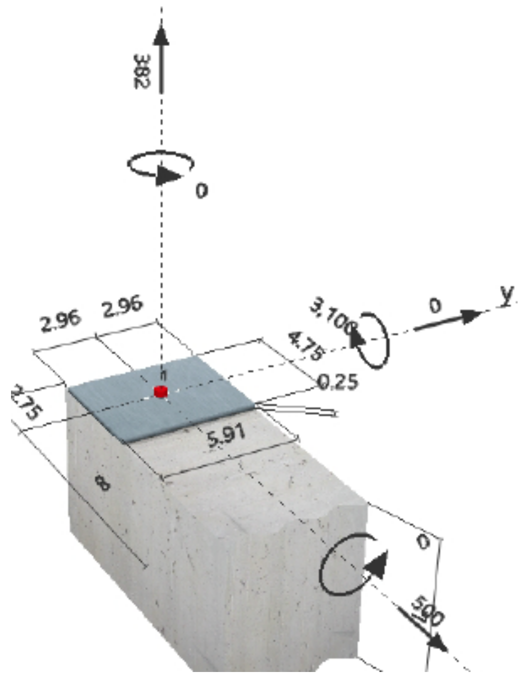
	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	1060	5547	20	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1060	5943	18	OK
Concrete edge failure in direction y-**	1060	5301	20	OK

5 Combined tension and shear loads

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

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

L25S ALTERNATIVE ANCHORAGE TO CONCRETE - FASCIA MOUNTED 5.91" o.c.
Hilti HUS-EZ (KH-EZ) 1/2" Diameter



Anchorage designed for concrete with strength $f'_c \geq 4{,}000$ psi for cracked condition or $f'_c \geq 2{,}500$ psi for uncracked condition. The post-installed concrete anchor strength was determined according to ACI 318-19 Chapter 17. Hilti Profis software was used to do the calculations. Tension and shear condition B assumed - no supplemental concrete reinforcement assumed. The anchorage was evaluated based on a 5.91" segment of base shoe and supporting concrete.

Unit loads used in the reports:

V_u = 1.6 load factor; M_u

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

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

Minimum concrete thickness = 6.75"

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

Maximum moment found by iteration (outward load controls)

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

$V_a = 284/0.4925 = 577$ plf

Moment load: $M_u = 3{,}100"\#$; $M_a = 3{,}100/1.6 = 1{,}938"\#$ 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

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

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

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:

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

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:

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

For 5-7/8" o.c.

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

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

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

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

Summary of fascia mounted L25 base shoe strength - Must verify glass strength too.

Table 18		**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	6077	100.8	85.9	74.0	64.4	56.5	44.6	36.1
Steel 5-7/8" o.c	12153	203.1	173.0	149.1	129.9	114.1	90.1	72.9
Concrete 12M HSL 11-13/16" o.c.	4781	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	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	64.8	55.1	47.5	41.3	36.3	28.6	23.1
Wood 11-13/16" o.c.	4246	70.0	59.6	51.3	44.7	39.2	30.9	25.0
Wood 5-7/8" o.c.	8090	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.

USE OF MONOLITHIC BASE SHOES WITH LAMINATED GLASS

Testing by CR Laurence has demonstrated that the monolithic glass series base shoes may be safely used with the corresponding laminated glass thickness:
CRL 8B series shoe for 1/2” glass may be used in place of the 9BL56 for 9/16” laminated,
CRL 8B58 shoe for 5/8” glass may be used in place of the 9BL68 for 11/16” laminated.
Special gaskets and Taper-Locs are required.

When installing laminated glass into the monolithic base shoe the laminated glass strength shall be taken as given in this report. The strength of the base shoe anchorage shall be as given in the monolithic base shoe report - *GRS Glass Rail System - Wet Glazed or Taper-Loc® System Dry-Glazed Base Shoes* dated 03 April 2025 or latest version, summarized below for surface mounted cases.

Surface mounted to steel with ½” cap screws @ 12” o.c.: [A]

1/2” cap screw to steel	36” Height	42” Height	anchorage strength
Base Shoe	Allowable wind load		
8B	75.3 psf	55.3 psf	4,470”# each
8B58	78.9 psf	58.0 psf	4,688”# each

For anchorage to concrete Surface Mounted:
3 3/8” diameter x 4” Hilti HUS-EZ (KH-EZ) in accordance with ESR-3027 or Hilti HSL-4 M8 x 3-3/4” anchor in accordance with ESR-1545. f’c = 3,000 psi[B]
embed depth = 2.5” effective depth
Concrete anchors ≥ 3.75” edge distance [ABC]

Anchor spacing to concrete	**12” O.C.**		
Total Guard Height AFF	36”	42”	anchorage strength
Base Shoe	Allowable wind load		
8B	42.7 psf	31.4 psf	2,539”#/lf
8B58	45.6 psf	33.5 psf	2,707”#/lf

Anchor spacing to concrete	**6” O.C.** [ABC]		
Total Guard Height AFF	36”	42”	
8B	68.6 psf	50.4 psf	4,073”#/lf
8B58	73.2 psf	53.8 psf	4,670”#/lf

[A] Linear interpolation between guard heights and anchor spacing is permitted.
[B] Adjustment for concrete strength other than $f'_c = 3{,}000$ psi

$$W' = \frac{W^*\sqrt{X}}{\sqrt{3{,}000}}$$

[C] Adjustment for sand light-weight concrete:

$$W' = 0.6^*W$$

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

ALLOWABLE WIND LOADS ON GLASS

122˚F ambient temperature and 10,600 psi peak edge stress

TABLE 19

9/16”	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	0.2861	0.3222	2201	37.1	27.2	20.8	13.3	9.3
24	0.3108	0.351	2613	44.0	32.3	24.7	15.8	11.0
36	0.3399	0.3822	3097	52.1	38.3	29.3	18.8	13.0
41	0.3517	0.3939	3289	*	40.7	31.1	19.9	13.8
48	0.3672	0.4085	3538	*	*	33.5	21.4	14.9
60	0.3904	0.4286	3894	*	*	*	23.6	16.4
72	0.4093	0.4434	4168	*	*	*	*	17.5

TABLE 20

9/16”	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	0.3636	0.4052	3481	58.6	43.1	33.0	21.1	14.6
24	0.4333	0.4605	4496	75.7	55.6	42.6	27.2	18.9
36	0.4628	0.4790	4864	81.9	60.2	46.1	29.5	20.5
41	0.4697	0.4829	4944	*	61.1	46.8	30.0	20.8
48	0.4764	0.4866	5020	*	*	47.5	30.4	21.1
60	0.4835	0.4904	5098	*	*	*	30.9	21.5
72	0.4876	0.4925	5142	*	*	*	*	21.6

* Allowable load is same as last value in column

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

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

TABLE 21

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	0.4372	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

TABLE 22

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

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

TABLE 23:

13/16”	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	0.4529	0.5088	5489	92.4	67.9	52.0	33.3	23.1
24	0.4686	0.5272	5892	99.2	72.9	55.8	35.7	24.8
36	0.4909	0.5525	6471	108.9	80.0	61.3	39.2	27.2
41	0.5014	0.5639	6741	*	83.4	63.8	40.9	28.4
48	0.5165	0.5800	7131	*	*	67.5	43.2	30.0
60	0.5426	0.6064	7795	*	*	*	47.2	32.8
72	0.5676	0.6300	8415	*	*	*	*	35.4

TABLE 24:

13/16”	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	0.5384	0.6023	7690	129.5	95.1	72.8	46.6	32.4
24	0.6399	0.6901	10096	170.0	124.9	95.6	61.2	42.5
36	0.6938	0.7272	11210	188.7	138.7	106.2	67.9	47.2
41	0.7075	0.7356	11473	*	141.9	108.6	69.5	48.3
48	0.7216	0.7440	11734	*	*	111.1	71.1	49.4
60	0.7370	0.7527	12010	*	*	*	72.8	50.5
72	0.7462	0.7577	12170	*	*	*	*	51.2

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

TABLE 25:

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	10005	168.4	123.7	94.7	60.6	42.1
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

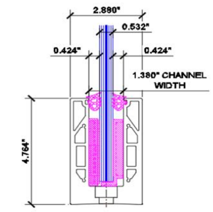
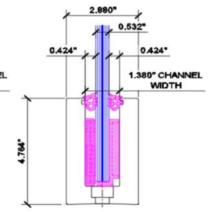
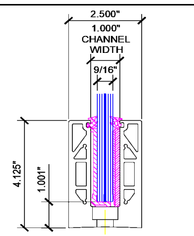
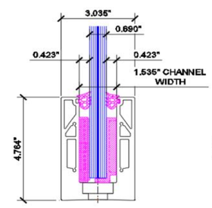
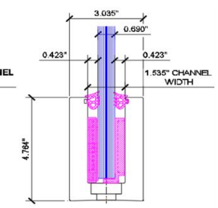
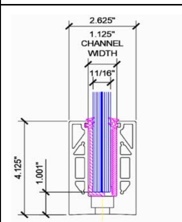
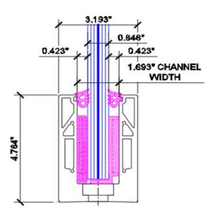
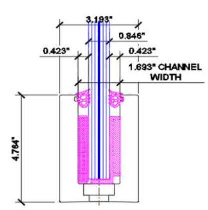
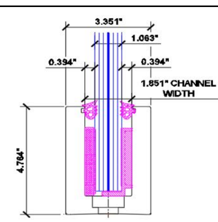
TABLE 26:

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

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

Base Shoe Options

GLASS	HOLLOW	SOLID	TAPER	GASKETS
9/16" LAMI	 9/16" (13.52mm) LAMINATED GLASS 9BL56 SERIES BASE SHOE	 9/16" (13.52mm) LAMINATED GLASS L56S SERIES BASE SHOE	LTL96X1	LBSG100 - PUSH IN SLBSG1 - DROP SIDE
9/16" LAMI	 9/16" (13.52mm) LAMINATED GLASS 8B SERIES BASE SHOE		TL5	MBSG1 - PUSH IN SG6X - DROP SIDE
17.52mm (11/16") LAMI	 1/16" (17.52mm) LAMINATED GLASS 9BL68 SERIES BASE SHOE	 11/16" (17.52mm) LAMINATED GLASS L68S SERIES BASE SHOE	LTL96X1	LBSG100 - PUSH IN SLBSG1 - DROP SIDE
17.52mm (11/16") LAMI	 11/16" (17.52mm) LAMINATED GLASS 8B58 SERIES BASE SHOE		TL6	MBSG1 - PUSH IN SG6X - DROP SIDE
21.52mm LAMI	 27/32" (21.52mm) LAMINATED GLASS 9BL21 SERIES BASE SHOE	 27/32" (21.52mm) LAMINATED GLASS L21S SERIES BASE SHOE	LTL10X1	LBSG100 - PUSH IN SLBSG1 - DROP SIDE
25.52mm LAMI		 1 1/16" (25.52mm) LAMINATED GLASS L25S SERIES BASE SHOE	LTL10X	LBSG100 - PUSH IN SLBSG1 - DROP SIDE

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

Seals

Edward C. Robison

Digitally signed by Edward C. Robison
Date: 2026.04.06 16:38:17 -07'00'

LICENSED PROFESSIONAL ENGINEER
EDWARD C. ROBISON
P30224
IOWA

PROFESSIONAL ENGINEER
LICENSED
1671692
STATE OF IDAHO
EDWARD C. ROBISON

Registered Professional Engineer/Structural
CERTIFICATE NO.
42123
EDWARD C. ROBISON
Date Signed
ARIZONA U. S. A.
EXP 03/31/2029

STATE OF ARKANSAS
LICENSED PROFESSIONAL ENGINEER
No. 21040
EDWARD C. ROBISON

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

COLORADO LICENSED
EDWARD C. ROBISON
54027
PROFESSIONAL ENGINEER

LICENSED STRUCTURAL ENGINEER
EDWARD C. ROBISON
081.007077
STATE OF ILLINOIS
EXP 11/30/2026

EDWARD C. ROBISON
LICENSE
NO. 63683
STATE OF FLORIDA
PROFESSIONAL ENGINEER
EXP 02/28/2027

GEORGIA
REGISTERED
No. 046121
PROFESSIONAL
ENGINEER
EDWARD C. ROBISON

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

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, License No. 52500, Expiration Date: 04/09/2022

EDWARD C. ROBISON
REGISTERED
NO. PE12300805
STATE OF INDIANA
PROFESSIONAL ENGINEER

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION
04/30/2028
Signature Expiration Date of the License

COMMONWEALTH OF MASSACHUSETTS
EDWARD C. ROBISON
STRUCTURAL
No. 49757
REGISTERED
PROFESSIONAL ENGINEER
EXP 06/30/2026

STATE OF MICHIGAN
Edward C. Robison
ENGINEER
NO
6201313877
REGISTERED PROFESSIONAL ENGINEER

EDWARD C. ROBISON
LICENSED PROFESSIONAL
ENGINEER
29000
STATE OF MISSISSIPPI

EDWARD C. ROBISON
LICENSE
NO. 89716
STATE OF MONTANA
PROFESSIONAL ENGINEER

STATE OF MISSOURI
EDWARD ROBISON
NUMBER
PE-2024026643
PROFESSIONAL ENGINEER

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

PROFESSIONAL ENGINEER-STATE OF NEVADA
EDWARD C. ROBISON
Exp: 12/31/2025
CIVIL
No. 17546

STATE OF NEW YORK
Edward C. Robison
088030
LICENSED PROFESSIONAL ENGINEER
EXP 03/31/2027

STATE OF OHIO
Edward C. Robison
E90702
REGISTERED
PROFESSIONAL ENGINEER

STATE OF NEW JERSEY
EDWARD C. ROBISON
24GE06161300
LICENSED
PROFESSIONAL ENGINEER

LICENSED PROFESSIONAL ENGINEER
E. C. ROBISON
31646
OKLAHOMA

REGISTERED PROFESSIONAL
ENGINEER
18195PE
OREGON
DEC. 6, 1995
EDWARD C. ROBISON
EXP 12/31/2026

COMMONWEALTH OF PENNSYLVANIA
REGISTERED
PROFESSIONAL
EDWARD C. ROBISON
ENGINEER
NO.73556
EXP 09/30/2027

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

STATE OF TEXAS
EDWARD C. ROBISON
100688
LICENSED
PROFESSIONAL ENGINEER
FIRM #F-12044
EXP 12/31/2026

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