23 August 2019

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

SUBJ: GLASS WINDSCREEN SYSTEM
FRAMELESS WINDSCREEN CLAMP SYSTEM
AFWC1/AFWC6, AFWC2/AFWC7, AFWC3/AFWC8, AFWC4, AFWC1S. AFWC4
DFWC3 - ALL VARIANTS
FRICTION FIT CLAMPS FWCR10/12, FWCS10/12, FWCS20/22, FWCR20/22

The frameless windscreen system uses stainless steel clamps to point support fully tempered glass lights. The windscreen system is designed for the following loading conditions:

200 lb concentrated load at top rail or top of glass horizontal (where applicable) or
50 lbs on one square foot at any location on glass (always applicable) or
wind load as shown in attached calculations for specified panel size.

The frameless windscreen clamps may be used for fall protection in one- and two-family residential occupancies and locations included in IBC 1607.8.1 Exceptions 1 and 2 when installed with 3 panels minimum of ½" tempered glass and installed with a top rail capable of spanning 10 feet. Allowable light sizes for fall protection are as indicated in tables 8 and 14. For these conditions the system will support the 200 lb load vertically or horizontally with a design factor of 4.0 against glass failure. Allowable wind loading for given panel height shall be in accordance with tables 5, 6, 10 to 13 or as calculated using the equations herein. This report is valid for glass light lengths from 36" to 72" long and up to 60" tall.

The clamps should be installed at 1/4 the panel length from each end. Clamp locations outside of 1/4 to 1/3 the light length from the ends are outside the scope of this report. Loads on the clamps shall not exceed the values given herein and as summarized in Table 15.

For these conditions the railing meets all applicable requirements of the 2006, 2009, 2012, 2015 and 2018 International Building Codes and 2010, 2013 and 2016 California Building Codes, 2010 and 2013 California Residential Codes and other state and local building codes adopting the International Building Codes. Stainless steel components are evaluated in accordance with SEI ASCE 8-02, *Specification for the Design of Cold-Formed Stainless Steel Structural Members* or AISC Design Guide 27 *Structural Stainless Steel*, as appropriate. The glass components are evaluated in accordance with GANA *Glazing Manual*, *Tempered Glass Engineering Standards Manual* and *Laminated Glazing Reference Manual*. The system may be designed and constructed using the methodology detailed herein to be compliant with ASTM E 2358 *Standard*

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

Specification for the Performance of Glass in Permanent Glass Railing Systems, Guards, and Balustrades and ICC Acceptance Criteria AC 439 *Acceptance Criteria for Glass Railing and Balustrade Systems* (July 2015).

Other glass thicknesses- This report provides the glass thicknesses the clamps are designed to support using the provided contact gaskets. Other glass thicknesses may be used but or outside of the scope of this report.

All glass is assumed to be a regular rectangle with the only holes being those used for installation of the clamps. Other glass shapes, such as trapezoidal are outside of the scope of this report. Notches in the glass are outside of the scope of this report.

For fall protection this report is limited to one- and two- family dwellings residential or locations listed in IBC 1607.8.1 exception 2 where the uniform live load is reduced to 20 plf. Any other application requiring fall protection is outside the scope of this report.

The specifier shall verify the suitability of the system for any specific installation to include but not limited to the wind load conditions, fall protection requirements, substrate support and any local codes or other requirements. This report may be used by a qualified professional as a guide in preparing a project specific design.

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

Sealed 23 AUG 2019

Registered Professional Engineer/Structural
CERTIFICATE NO. 42123
EDWARD C. ROBISON
Date Signed
ARIZONA U. S. A.
EXP 03/31/2020

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

COLORADO LICENSED
EDWARD C. ROBISON
54027
PROFESSIONAL ENGINEER

EDWARD C. ROBISON
LICENSE NO. 63683
STATE OF FLORIDA
PROFESSIONAL ENGINEER
EXP 02/28/2021

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

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION

04/30/2020

Signature Expiration Date of the License

LICENSED STRUCTURAL ENGINEER
EDWARD C. ROBISON
081.007077
STATE OF ILLINOIS
EXP 11/30/2020

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: 04/09/2020

COMMONWEALTH OF MASSACHUSETTS
EDWARD C. ROBISON
STRUCTURAL
No. 49757
REGISTERED PROFESSIONAL ENGINEER
EXP 06/30/2020

EDWARD C. ROBISON
LICENSED PROFESSIONAL ENGINEER
29000
STATE OF MISSISSIPPI

PROFESSIONAL ENGINEER-STATE OF NEVADA
EDWARD C. ROBISON
Exp: 12/31/2019
CIVIL
No. 17546

STATE OF NEW YORK
Edward C. Robison
088030
LICENSED PROFESSIONAL ENGINEER
EXP 03/31/2021

REGISTERED PROFESSIONAL ENGINEER
18195PE
OREGON
DEC. 6, 1995
EDWARD C. ROBISON
EXP 12/31/2020

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
EDWARD C. ROBISON
ENGINEER
NO. 73556
EXP 09/30/2019

STATE OF TEXAS
EDWARD C. ROBISON
100688
LICENSED PROFESSIONAL ENGINEER
FIRM #F-12044
EXP 12/31/2019

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

Typical loads:

Load cases illustrated on Figure 1.
Live Loads:
50 plf load along glass at 42” above finish floor horizontal:

LOADS ON CLAMPS
V_c = shear load on clamp
M_c = Moment on clamp
V_c = 50plf*S/2
M_c = 50plf*42”*S/2

or
200 lb concentrated load at 42” above finish floor horizontal
V_c = 200/Cc where
C_c from Figure 2
M_c = V_c*42”

or
Wind load uniform load over entire glass light area. **Wind load must be determined by a competent professional for a specific installation.**

V_c = w*H*S/2
M_c = V_c*H*0.55

All loads must be considered as acting inwards or outwards.

The clamp loads must be determined to verify that the allowable loads as shown in table 4 aren’t exceeded.

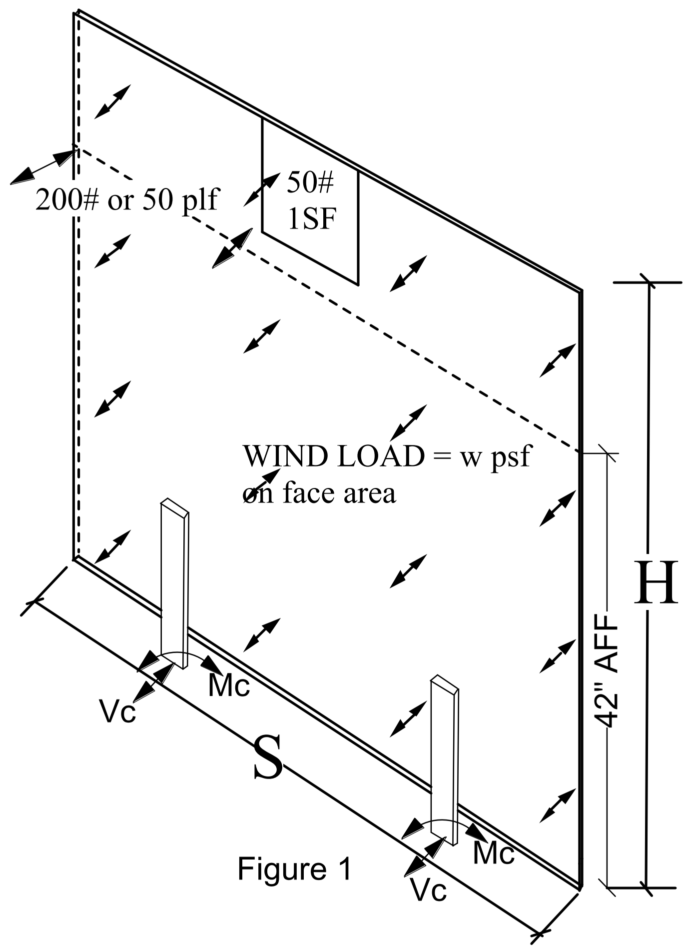
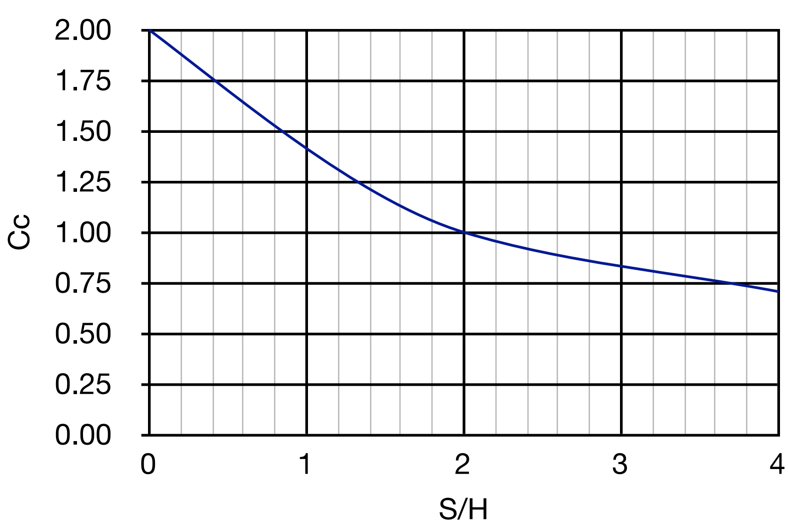


Figure 1

Figure 2



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

GLASS STRENGTH

All glass is fully tempered glass conforming to the specifications of ANSI Z97.1, ASTM C 1048-97b and CPSC 16 CFR 1201. The average Modulus of Rupture for the glass F_r is 24,000 psi. . In accordance with IBC 2407.1.1 glass used as structural balustrade panels shall be designed for a safety factor of 4.0. This is applicable only to structural panels (glass provides support to railing). Other locations the glass stress may be increased to as high as 10,600 psi edge stress in accordance with ASTM E1300-00.

Allowable glass bending stress: 24,000/4 = 6,000 psi. – Tension stress calculated.
Allowable compression stress = 24,000psi/4 = 6,000 psi.
Allowable bearing stress = 24,000 psi/4 = 6,000 psi.
For wind loads only: allowable stress from ASTM E-1300 = 10,600 psi

Bending strength of glass for the given thickness:

$$S = \frac{12''*(t_{min})^2}{6} = 2*(t_{min})^2 \text{ in}^3/\text{ft}$$

For 1/2" glass $t_{min} = 0.469"$; $t_{ave} = 0.50"$

$S = 2*(0.469)^2 = 0.44$ in^3/ft
$M_{allowable}$ = 6,000psi*0.44 in^3/ft = 2,640"#/ft = 220'#/ft
For wind load case:
$M_{allowable}$ = 10,600psi*0.44 in^3/ft = 4,664"#/ft = 352'#/ft

For 3/8" glass $t_{min} = 0.355"$; $t_{ave} = 0.380"$

$S = 2*(0.355)^2 = 0.252$ in^3/ft
$M_{allowable}$ = 6,000psi*0.252 in^3/ft = 1,512.3"#/ft = 126.0'#/ft
For wind load case:
$M_{allowable}$ = 10,600psi*0.252 in^3/ft = 2,671"#/ft = 222.6'#/ft

Laminated glass sizes are covered further 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

Determine Glass Stresses:
The glass is subject to stress concentrations which are a function of the glass height and width. The stress concentrations were modeled using finite element analysis models-
Models were prepared using SCIA Engineer 18. The stress concentration effects were normalized by dividing the peak model stress by the calculated average stress to develop the proposed design curves for determining the peak glass stress. Moment amplification factors vary greatly with glass width and vary slightly with glass height.

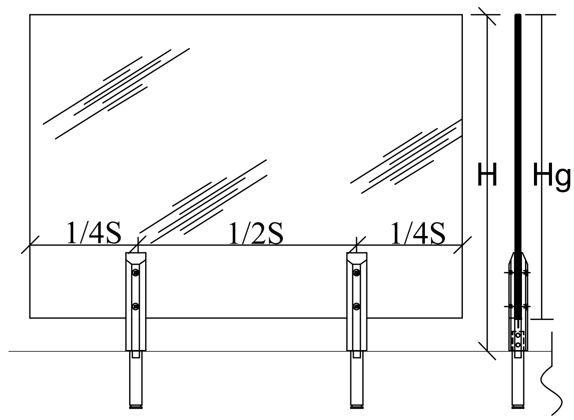


Figure 4

Linear interpolation for widths and heights between those shown may be used to determine the amplification factor (ß).

Table 1: Moment Amplification Factor For Uniform Loading At Top of Glass based on glass light width and height

Glass Height (in)			Glass Width (in)		
	36	48	60	68	72
36	2.13	2.71	3.35	3.77	3.98
42	2.15	2.76	3.38	3.80	4.02
48	2.19	2.81	3.43	3.85	4.07
60	2.22	2.83	3.45	3.88	4.10

Table 2: Moment Amplification Factor For Uniform Wind Loading based on glass light width and height

Glass Height (in)			Glass Width (in)		
	36	48	60	68	72
36	1.98	2.53	3.15	3.55	3.75
42	2.01	2.60	3.20	3.61	3.81
48	2.07	2.67	3.27	3.68	3.89
60	2.12	2.72	3.32	3.74	3.95

To calculate the peak glass moment (in-lb per foot) the average bending moment is calculated based on a simple cantilever and then multiplied by the amplification factor ß:
For 50 plf live load at 42" above finish floor or top of glass whichever is less:
M_{gL} = ß*50plf*h or = ß*50plf*H_g
where h = 42" minus height from walking surface to bottom of glass

50 lbs on one square foot concentrated load doesn't need to be checked if the 50 plf or 200 lb loads are checked. 50plf and 200 lb load cases only need to be checked if installation is required to provide fall protection or pool/spa fence. 50 lbs on one square foot concentrated load is the only live load check required when used as a wind screen only. Generally, the glass will be unable to hold the 200# concentrated live load except for shorter glass panels that are connected together. If the Frameless Windscreen System is to be used as a guard rail, then a qualified engineer must analyze live loading on the specific system.

For wind loads:
$M_{gw} = ßwH_g^2*0.55/12$ (in-lb per foot) for w in psf and H_g in inches

Peak Glass Stress:
$f_{bg} = M/(2*t_{min}^2)$ in psi

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

GLASS DEFLECTIONS

There is no building code defined limit on the glass deflections. ASTM E 2358 *Standard Specification for the Performance of Glass in Permanent Glass Railing Systems, Guards, and Balustrades* limits deflection to H/12 (ASTM E 6.2.1.1). ICC Acceptance Criteria AC 439 *Acceptance Criteria for Glass Railing and Balustrade Systems* (July 2015) Paragraph 3.3 applies a 1 inch deflection limit to the glass for the live load cases.

The finite element models were used to determine the deflection amplification caused by the reduced stiffness provided by the point supports using the clamps.
The deflection of glass may be estimated using the following equations:
For 50plf top rail loading:
$\Delta = \lambda 50H_g^3/(3*10.4x10^6*t_{ave}^3)$
For wind loading:
$\Delta = \lambda PH_g^4/(8*10.4x10^6*t_{ave}^3)$
λ = deflection amplification factor which is a function of the height and width and may be taken from the appropriate width curve in Figures 4 and 5.

Figure 4: Deflection Amplification Factor For 50plf Load

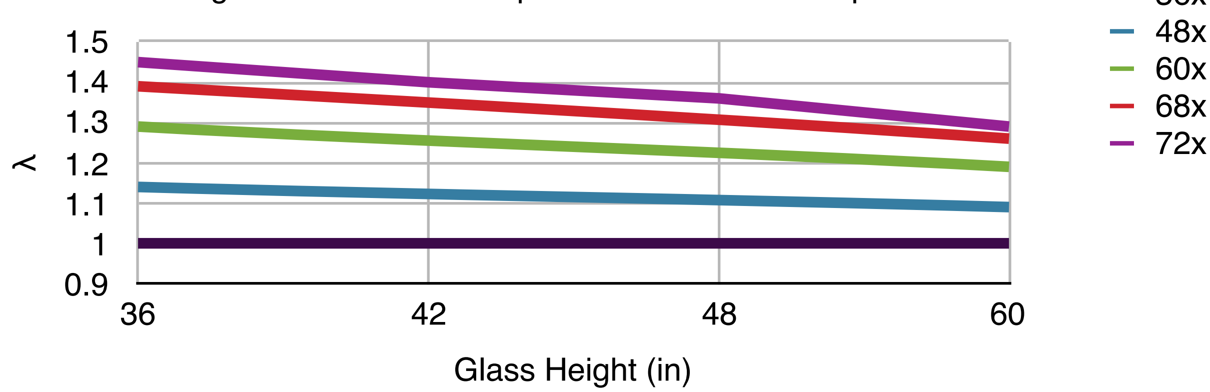


Figure 4: Deflection Amplification Factor For Wind Load

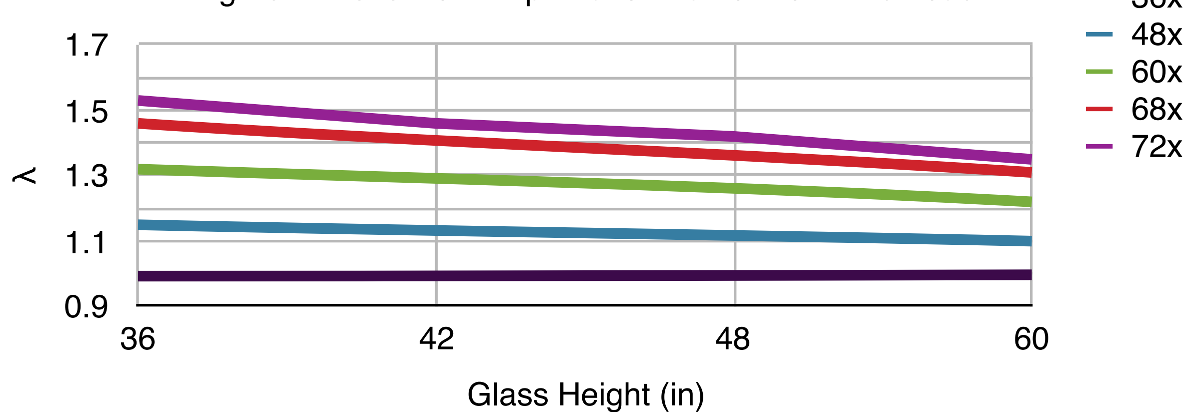


Table 3: Deflection Amplification Factor For Live Loading based on glass light width and height

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	1.00	1.14	1.29	1.39	1.45
42	1.00	1.12	1.25	1.35	1.40
48	1.00	1.11	1.23	1.31	1.36
60	1.00	1.09	1.19	1.26	1.29

Table 4: Moment Amplification Factor For Uniform Wind Loading based on glass light width and height

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	0.993	1.15	1.32	1.46	1.53
42	0.993	1.13	1.29	1.40	1.46
48	0.995	1.12	1.27	1.37	1.42
60	0.997	1.10	1.22	1.31	1.35

Allowable wind loads on the wind screen are a function of the glass light height and width. The allowable wind load may be calculated from:
$w_{all} = M_{all}*12/(ßH_g^2*0.55) = 462545t_{min}^2/(ßH_g^2)$
(derived from the equations on page 5)
Or taken from Tables 5 or 6.

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

Table 5 & 6: Allowable Uniform Wind Loading based on glass light width and height

1/2" Monolithic Tempered Glass - t_{min} = 0.469"

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	39.6	31.0	24.9	22.1	20.9
42	28.7	22.2	18.0	16.0	15.1
48	21.3	16.5	13.5	12.0	11.4
60	13.3	10.4	8.5	7.6	7.2

3/8" Monolithic Tempered Glass - t_{min} = 0.355"

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	22.7	17.8	14.3	12.7	12.0
42	16.4	12.7	10.3	9.2	8.7
48	12.2	9.5	7.7	6.9	6.5
60	7.6	6.0	4.9	4.3	4.1

Linear interpolation between the dimensions shown may be used.

The wind loads for a specific installation shall be determined by a qualified professional considering wind speed, exposure, terrain, location on structure, use and other applicable factors.

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

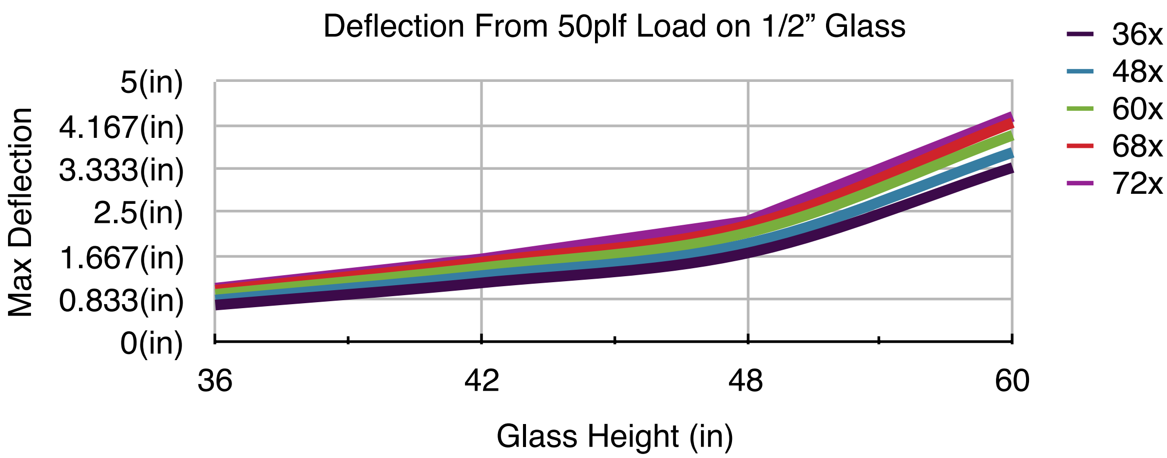


Table 7

Deflection From 50plf Load on 1/2" Monolithic Tempered Glass - t_{min} = 0.469"

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	0.7	0.8	0.9	1.0	1.1
42	1.2	1.3	1.4	1.6	1.6
48	1.7	1.9	2.1	2.3	2.3
60	3.4	3.7	4.0	4.2	4.3

FOR FALL PROTECTION:

Table 8

1/2" Monolithic Tempered Glass - t_{min} = 0.469"

Glass must be installed with a continuous structural top rail, $I_y \geq 0.1$ in^4 for stainless steel or 0.29 in^4 for aluminum.

Glass Height (in)	Glass Width (in)			
	36	42	48	54
33	OK	OK	OK	OK
36	OK	OK	OK	NO
39	OK	OK	NO	NO
42	OK	NO	NO	NO

The top rail must have alternative supports at the end attached to a wall or post.

LAMINATED GLASS:

Laminated tempered glass may be used with the glass clamps. The effective thickness of the laminated glass may be determined per ASTM E1300-12a appendix X9. For determining the effective glass thickness use the lesser of the glass width or glass height. Typically the laminated glass is fabricated with 0.06" interlayer of either PVB or Kuraray SentryGlas+® ionoplast interlayer. Table 9 may be used to select the effective glass thickness for 7/16" and 9/16" laminated glass made with either PVB or SGP evaluated at 120˚F.

Table 9 - Effective Thickness of Laminated Glass:

	PVB	SGP (psi)
G	70	1638.9

	$h_1 = h_2$	h_v		$h_{s;1} = h_{s;2}$		I_s	h_s
3/16"	0.18	0.06		0.12		0.005184	0.24
1/4"	0.219	0.06		0.1395		0.0085235	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**
3/16+0.06+ 3/16 = 7/16" Laminated Glass	36	0.144	0.798	0.274	0.394	0.310	0.406
	42	0.186	0.843	0.285	0.400	0.322	0.409
	48	0.230	0.875	0.296	0.404	0.332	0.412
	54	0.275	0.899	0.306	0.407	0.342	0.413
	60	0.319	0.916	0.316	0.409	0.350	0.414
1/4"x0.06"x 1/4" = 9/16" Laminated Glass	36	0.121	0.764	0.322	0.463	0.364	0.479
	42	0.158	0.815	0.334	0.471	0.376	0.484
	48	0.197	0.852	0.345	0.476	0.388	0.487
	54	0.237	0.879	0.356	0.481	0.398	0.489
	60	0.278	0.900	0.367	0.484	0.408	0.490

$h_{ef;w}$ = effective glass thickness for deflections

$h_{1;ef;\sigma}$ = effective glass thickness for bending stress

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

Table 10 - 13: Allowable Uniform Wind Loading based on glass light width and height

Table 10
9/16" PVB Laminated Tempered Glass - $t_{min} = 0.364"$

Glass Height (in)			Glass Width (in)		
	36	48	60	68	72
36	23.9	18.7	15.0	13.3	12.6
42	17.3	13.4	10.9	9.6	9.1
48	12.9	10.0	8.1	7.2	6.8
60	8.0	6.3	5.1	4.6	4.3

Table 11
7/16" PVB Laminated Tempered Glass - $t_{min} = 0.310"$

Glass Height (in)			Glass Width (in)		
	36	48	60	68	72
36	17.3	13.6	10.9	9.7	9.1
42	12.5	9.7	7.9	7.0	6.6
48	9.3	7.2	5.9	5.2	5.0
60	5.8	4.5	3.7	3.3	3.1

Table 12
9/16" SGP Laminated Tempered Glass - $t_{min} = 0.479"$

Glass Height (in)			Glass Width (in)		
	36	48	60	68	72
36	41.4	32.4	26.0	23.1	21.8
42	29.9	23.1	18.8	16.7	15.8
48	22.3	17.3	14.1	12.5	11.8
60	13.9	10.8	8.9	7.9	7.5

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

Table 13
7/16" SGP Laminated Tempered Glass - t_{min} = 0.406"

Glass Height (in)	Glass Width (in)				
	36	48	60	68	72
36	29.7	23.3	18.7	16.6	15.7
42	21.5	16.6	13.5	12.0	11.3
48	16.0	12.4	10.1	9.0	8.5
60	10.0	7.8	6.4	5.7	5.4

FOR FALL PROTECTION:
Table 14
9/16" SGP Laminated Tempered Glass - t_{min} = 0.479"
Glass must be installed with a continuous top rail or corners clamped together and the joint fully butt glazed with structural silicone.

Glass Height (in)	Glass Width (in)			
	36	48	60	68
33	OK	OK	OK	OK
36	OK	OK	OK	NO
39	OK	OK	NO	NO
42	OK	NO	NO	NO

The top rail or end lights must have alternative supports at the end attached to a wall or post.

The glass height is the measured from bottom to top of glass and not height above the walking surface.

GLASS CLAMPS
The glass clamp is composed of an adjustable clamping system which is made up of stainless steel clamp bodies (either cast or wrought depending on style), 5/16" diameter screws through the glass, and gaskets with varying thicknesses to accommodate selected glass thickness. The clamp varies in height by style but all styles have a similar glass bite. Various clamps are available for installation on substrates of concrete, metal and wood and attachment to the surface or fascia.

The CRL clamps require either one or two 3/4" holes through the glass at each clamp depending on the specific style. The holes are used to tie the clamp bodies together through the glass to provide the clamping force and develop the full clamp strength. Where indicated herein some styles of clamps may be used without the glass hole at a reduced clamp strength.

The maximum allowable moment on the clamps shall not exceed the moments indicated herein.

Available clamp styles covered in this report


AFWC1/6


AFWC3/8


AFWC2/7


DFWC3


AFWC4

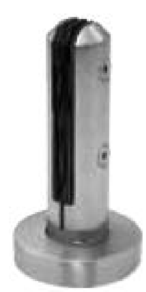

AFWC4S


AFWC1S

The Friction Fit Clamps are intended for installations without holes through the glass. These clamps don't have a through glass bolting option and are intended for locations with lesser wind loads or small light sizes and where fall protection typically isn't required.


FWCR10/ FWCR12


FWCS10/ FWCS12

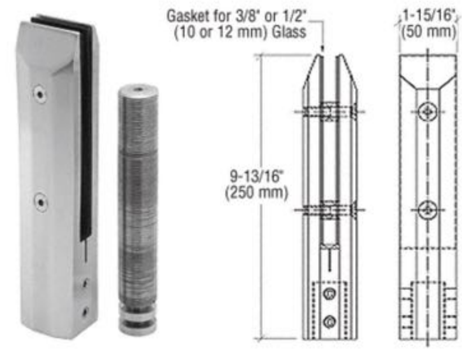

FWCR20/ FWCR22


FWCS20/ FWCS22

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

AFWC1/AFWC6

One Piece Post Option: 316 Stainless Steel casting with glass pocket mounted on 1-3/16” (30mm) stainless steel rod core mounted into concrete.



Check bending strength of bar:
Based on AISC Design Guide 27:

$Z = d^3/6 = 1.181^3/6 = 0.2746$ in^3

Wrought 316 stainless steel bars

$F_y \geq 50$ ksi and $F_u \geq 75$ ksi

$M_n = 0.2746*50$ksi = 13,730#”

$M_s = M_n/1.67$

$M_s = 13{,}730”\#/1.67$

$M_s = 8{,}222”\#$

Determine strength of anchorage into concrete:

Moment is resisted by vertical compression against the concrete by the embedded rod. The grooves on the end of the rod prevent withdrawal.

From $\sum$ M about centroid of C_l:

$M_n = C_l*D(2/3)$

$C_u = øF_B$

F_B = shear block failure from ACI 318 App D.6.2

$F_B = 1.4*\sqrt{1.5}*7(6/1.181)^{0.2}(1.181*2500)^{1/2}*6^{1.5} = 13{,}267\#$

$C_u = 0.85*13{,}267\# = 11{,}277\#$

$M_n = 11{,}277\#*6*(2/3) = 45{,}108\#”$

$M_s = 0.70*45{,}108/1.6 = 19{,}735\#”$

Minimum embed depth of 4.5”

$F_B = 1.4*\sqrt{1.5}*7(4.5/1.181)^{0.2}(1.181*2500)^{1/2}*4.5^{1.5} = 8{,}135\#$

$C_u = 0.85*8{,}135\# = 6{,}915\#$

$M_n = 6{,}915\#*4.5*(2/3) = 20{,}745\#”$

$M_s = 0.70*20{,}745/1.6 = 9{,}076\#”$

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

Check strength of clamp glass pocket.
Glass to clamp moment resisted by tension in cap screws:
Screw tension is proportional to distance from bottom screw to screw being considered:

5/16" diameter cap screws upper two.

Screw tension strength - ASTM F 879

$T = 3{,}980\#$ (ASTM F 879 Table 3)

$T_a = 3{,}980/2 = 1{,}990\#$

Moment resistance M_a

$M_a = 1{,}990\#*(5.625"+3.6875"^2/5.625") = 16{,}004"\# \geq 8{,}222"\#$

ALLOWABLE DESIGN MOMENT = 8,222"# WITH BOLTS THROUGH GLASS

For installations without bolts through glass:
Check for bending of side plates:

$S = 0.160\ in^3$

$F_y = 30$ ksi for cast 316 Stainless steel

$M_n = ZF_y = 0.160*30\ ksi = 4{,}800\ "\#$

$M_a = 4{,}800/1.67 = 2{,}874"\#$

ALLOWABLE DESIGN MOMENT = 2,874"# WITHOUT BOLTS THROUGH GLASS

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

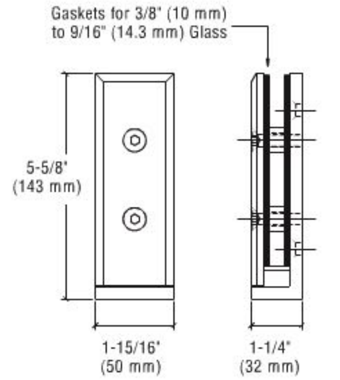
AFWC2/7

CRL Two-Piece Side Mount Frameless Windscreen Clamp

new

- 316 Grade Stainless Steel in Two Architectural Finishes
- Great for Windscreens or Glass Partitions (Not Designed for Guard Rail Applications)
- For 3/8" or 1/2" (10 or 12 mm) Tempered Monolithic Glass and 9/16" (13.52 mm) Tempered Laminated Glass

CRL Two-Piece Side Mount Frameless Windscreen Clamps are designed for attachment to the front side of walls or planter boxes. Due to the number of varying mounting conditions, anchors and mounting screws are not supplied. The back plate is fabricated to accept 8 mm (5/16") flat head screws. Glass fabrication requires two 3/4" (19 mm) holes per Clamp. An instruction sheet and glass fabrication template are supplied.



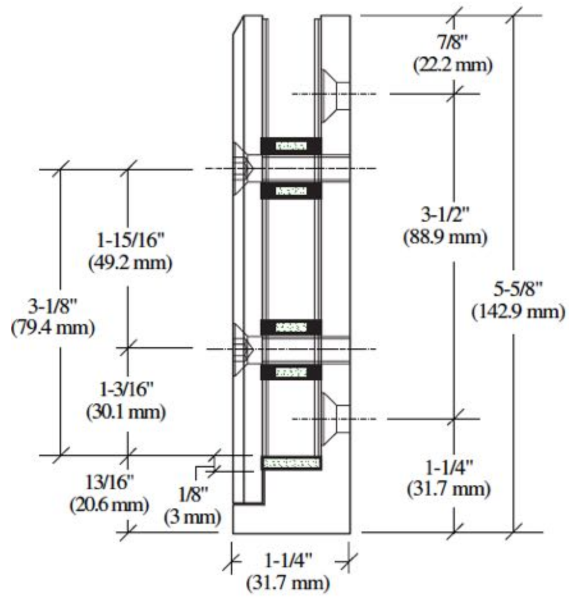
CAT. NO. CAP35S

Replacement Bolt

CAT. NO.	FINISH
AFWC2	Brushed Stainless
AFWC7	Polished Stainless

Minimum order: 1 each.

Clamp is mounted to a deck edge, beam web or wall face to support the glass light. Clamp may be installed to concrete, steel or other metal or wood. Clamp body is attached to the supporting structure using two 5/16" dia countersunk screws.



For anchorage to steel use two countersunk stainless steel screws:
Screw tension strength - ASTM F 879

$T = 3{,}980\#$ (ASTM F 879 Table 3)

$T_a = 3{,}980/2 = 1{,}990\#$

$V_a = 0.6*1990 = 1194\#$

Moment strength of anchorage to steel:

$M_s = 1{,}990\#*4.75" = 9453"\#$

$V_s = 0.2*2*1194 = 478\#$

For anchorage to concrete - Assumed conditions:
Minimum concrete strength f'c = 3,000 psi
Minimum concrete edge distance = 1.75"
Alternative anchorage may be designed for specific project conditions.

For anchorage to concrete using two 1/4" x 2-1/2" Tapcon screws with 1.75" effective embedment:

$T_a = 550\#$ (ITW Red Head technical information)

$V_a = 420\#$

$M_s = 550*4.75" = 2{,}613"\#$

$V_s = 0.2*2*420 = 168\#$

For 1/4” countersunk screw into Red Head Multi-Set II Drop-In Anchor
T_a = 590# (ITW Red Head technical information)
V_a = 270#
M_s = 590*4.75” = 2,803”#
V_s = 0.2*2*270 = 108#

For wood:
5/16” x 4” wood screw into wood with G ≥ 0.46
Assumes installation is in location where wood moisture content remains below 19%
Withdrawal strength of screw from National Design Specification for Wood Construction 11.2.2:
$W = 2850G^2D = 2850*0.46^2*0.3125$ = 180 pli
$W' = C_DWe$
C_D = 1.6 for wind loads
e = 3 1/8”
W’ = 1.6*180pli*3.125” = 900#

Shear strength based on NDS 11.3.1
$Z_\perp$ = 170#
$Z'_\perp = C_DZ_\perp$ = 1.6*170# = 272#
Combined lateral and withdrawal loads (NDS Table 11.4)
$Z'_\perp = (W'Z')/[W'Cos^2\alpha + Z'sin^2\alpha]$
For assumed dead load of 78# on bracket (39# on screw) and W = 700#
$\alpha = tan^{-1}(700/39) = 86.8^\circ$ resultant = $\sqrt{(700^2+39^2)}$ = 702#
$Z'_\perp = (900*272)/[900Cos^2 86.8^\circ + 272sin^2 86.8^\circ]$ = 894#
Screw withdrawal strength may be increased to 890# for maximum light size:
M_s = 890*4.75” = 4,228”#
V_s = 2*39 = 78# dead load

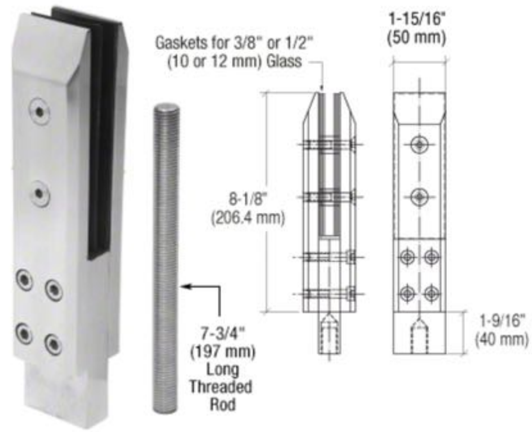
When attached to wood subject to wetting where moisture content will exceed 19% at any time after installation then the allowable loads are to be multiplied by C_M = 0.7.

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

AFWC3/8

Three Piece Core Mount Frameless Windscreen Clamp

Post is attached to 5/8” (16mm) rod set into concrete and grouted to base of side bars. For full clamp strength grout must be to base of side plates



Determine strength of anchorage into concrete:

Moment is resisted by a combination of vertical compression against concrete with tension in the anchor rod and by bending of center plate which is anchored by horizontal compression against concrete.

Nominal strength of vertical compression of side plate against concrete:

$øF_B = øßf'c$ where:

$ß = \sqrt{(A2/A1)} = \sqrt{16/(1/2"*1\ 15/16)}$ but ≤ 2.0

and f'_c = 6 ksi for specified anchoring grout

$øF_B$ = 0.85*2*6,000 = 10,200 psi

Cp_n = 0.5”*1.9375”*10,200 psi = 9,881#

Bar tension strength, nominal

F_y = 75 ksi (1/4 hard A316)

T_b = 0.195 in^2*75 ksi = 14,625#

Cp_n controls

M_{Cpn} = 0.6875”*9,881# = 6,793#”

Nominal strength from bending of center flat bar

Bar bending:

Z = 1.9375”*0.9375^2/4 = 0.4257 in^3

F_y = 45 ksi

Mp_n = 45 ksi*0.425 in^3 = 19,157 #”

Check anchorage:

Center plate is anchored into concrete:

V_n = shear strength of bar

$V_n = A*F_{yv}$ = 0.195*42ksi = 8,190#

Allowable shear in bar must be reduced for tension:

$1 \geq (T/T_n)^2+(V/V_n)^2$

$V_b = V_n [1-(T/T_n)2]$ = 8190#*[1-(9881#/14625#)^2] = 4,452#

Check anchor strength of bar:

$C_u = V_b + C_l$ and $V_b = C_l$
C_u = 2*4,452# = 8,904#
Bearing length along bar for:
F_{BC} = 10,200 psi
$L = [C_u/(d_b F_{BC})]/0.85$
L = [8,904#/(1/2”*10,200psi)]/0.85 = 2.05” < 4” Bar length is adequate
Determine C_c
Maximum value for C_c if in full compression:
C = 1.56”*1.9375”*10,200 psi* 0.85 = 26,205# >V_b
C_c must be balanced with Vb and C_b
a = compression depth for C_c
$C_b + V_b = C_c$ and
C_b = (1.56”-a)*1.9375”*10,200 psi
C_c = a*1.9375”*10,200 psi
substituting into above and solving for a:
(1.56”-a)1.9375”*10,200psi+4,452# = 1.9375”*10,200psi*a
(1.56”-a)+0.2253” = a
1.79” = 2a
0.89” = a
Calculate C_c from a
C_c = 0.89”*1.9375”*10,200psi = 17,589#
C_b = 17,589-4,452 = 13,137#
Calculate the resisted moment by ∑M about Cc
Mh_n = (1.56”-0.89/2)*4,452+(1.56”/2-0.89”/2)*13,137 = 9,365#”

The total nominal moment strength of the anchorage:
$M_n = Mh_n + M_{Cpn}$ = 9,365#”+ 6,793#” = 16,158#”
$M_s = øM_n/1.6$ = 0.75*16,158#”/1.6 = 7,574#”
Allowable service moment based on clamp anchorage = 7,574#”

Minimum embed depth of rod into concrete = 3*1.9375 = 5.81 - Use 6” minimum
Minimum slab thickness at embed = 1.5*6” = 9”
Minimum slab thickness required for 1’ radius or 6” radius if slab thickness is 12” minimum.

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

Check moment strength of clamp
Glass to bracket moment resisted by tension in cap screws:
Screw tension is proportional to distance from bottom screw to screw being considered:

5/16" diameter cap screws upper two.
1/4" diameter cap screws for lower four.
$T_{5/16}$ = 3,980# (ASTM F 879 Table 3)
$T_{1/4}$ = 2,703# (ASTM F 879 Table 3)

Moment resistance M_n

M_n = 3,980#*(5.625"+3.6875"^{2}/5.625")+2,420#*(1.375"^{2}/5.625")*2
M_n = 33,635
M_s = 33,635/2.0 = 16,818#"

Check for bending of side bars:

Z = 1.75"*0.5625^2/4 = 0.1384 in^3 (effective dimensions)
F_y = 30 ksi
M_n = 0.1384*30 ksi = 4,153
M_s = 4,153/1.67 = 2,487#"

Moment Strength of fully installed clamp:

M_s = 2*2,487 = 4,974

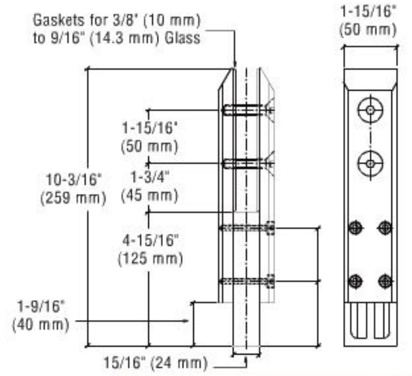
ALLOWABLE DESIGN MOMENT = 4,974"# WITH BOLTS THROUGH GLASS
ALLOWABLE DESIGN MOMENT = 2,487"# WITHOUT BOLTS THROUGH GLASS

DFWC3

CRL Three-Piece Core Mount Clamp

- **Made of Duplex 2205 Grade Brushed Stainless Steel**
- **Designed for 3/8" or 1/2" (10 or 12 mm) Tempered Monolithic Glass and 9/16" (13.52 mm) Tempered Laminated Glass**

Duplex 2205 is a more corrosive-resistant, higher grade of stainless steel than #316. It is ideally suited for areas within 1 kilometer of saltwater environments due to higher chromium content in the metal. Glass fabrication required two 19 mm (3/4") holes per clamp.



CAT. NO.	FINISH
DFWC3	Brushed Stainless

Minimum order: 1 each.

Clamp anchorage strength is same as calculated for the AFCW3 clamp.
Check moment strength of clamp
Glass to bracket moment resisted by tension in cap screws:
Screw tension is proportional to distance from bottom screw to screw being considered:

5/16" diameter cap screws upper two.
1/4" diameter cap screws for lower four.
$T_{5/16}$ = 3,980# (ASTM F 879 Table 3)
$T_{1/4}$ = 2,703# (ASTM F 879 Table 3)

Moment resistance M_n

$M_n = 3{,}980\#*(5.062"+3.125"^2/5.062")+2{,}420\#*(1.375"^2/5.062")*2$
M_n = 29,635
M_s = 29,635/2.0 = 14,817#"

Check for bending of side bars:

$Z = 1.75"*0.5625^2/4 = 0.1384\ in^3$ (effective dimensions)
F_y = 45 ksi
M_n = 0.1384*45 ksi = 6,228
M_s = 6,228/1.67 = 3,729#"

Moment Strength of fully installed clamp:

M_s = 2*3,729 = 7,458

ALLOWABLE DESIGN MOMENT = 7,458"# WITH BOLTS THROUGH GLASS
ALLOWABLE DESIGN MOMENT = 3,729"# WITHOUT BOLTS THROUGH GLASS

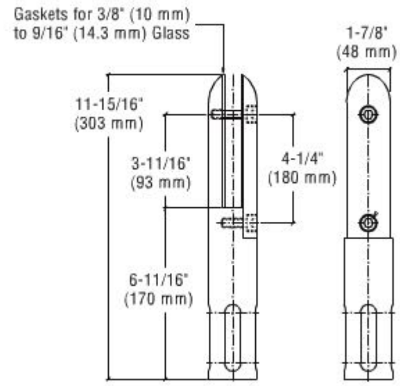
EDWARD C. ROBISON, PE
10012 Creviston Dr NW
Gig Harbor, WA 98329
253-858-0855/Fax 253-858-0856 elrobison@narrows.com

AFWC4

CRL Two-Piece Core Mount Round Clamp

- **Made of Duplex 2205 Grade Brushed Stainless Steel**
- **Designed for 3/8" or 1/2" (10 or 12 mm) Tempered Monolithic Glass and 9/16" (13.52 mm) Tempered Laminated Glass**

Duplex 2205 is a more corrosive-resistant, higher grade of stainless steel than #316. It is ideally suited for areas within 1 kilometer of saltwater environments due to higher chromium content in the metal. Glass fabrication requires one 19 mm (3/4") hole per clamp.



CAT. NO.	FINISH
AFWC4	Brushed Stainless

Minimum order: 1 each.

Clamp must be installed with bolt through the glass.

Strength of clamp anchorage into hole in concrete is similar to AFWC1 and won’t control clamp strength.

Check moment strength of clamp:
Glass to clamp moment resisted by tension in cap screws:
Screw tension is proportional to distance from bottom screw to screw being considered:

5/16” diameter cap screws connectig side plate to main clamp body

$T_{5/16}$ = 3,980# (ASTM F 879 Table 3)

Moment resistance M_n

M_n = 3,980#*4.25”

M_n = 16,915”#

M_s = 16,915/2.0 = 8,458#”

Check for bending of side bars:

Z = 0.083 in^3 (effective dimensions)

F_y = 60 ksi For cast Duplex 2205

M_n = 0.083*60 ksi = 4,980

M_s = 4,980/1.67 = 2,982#”

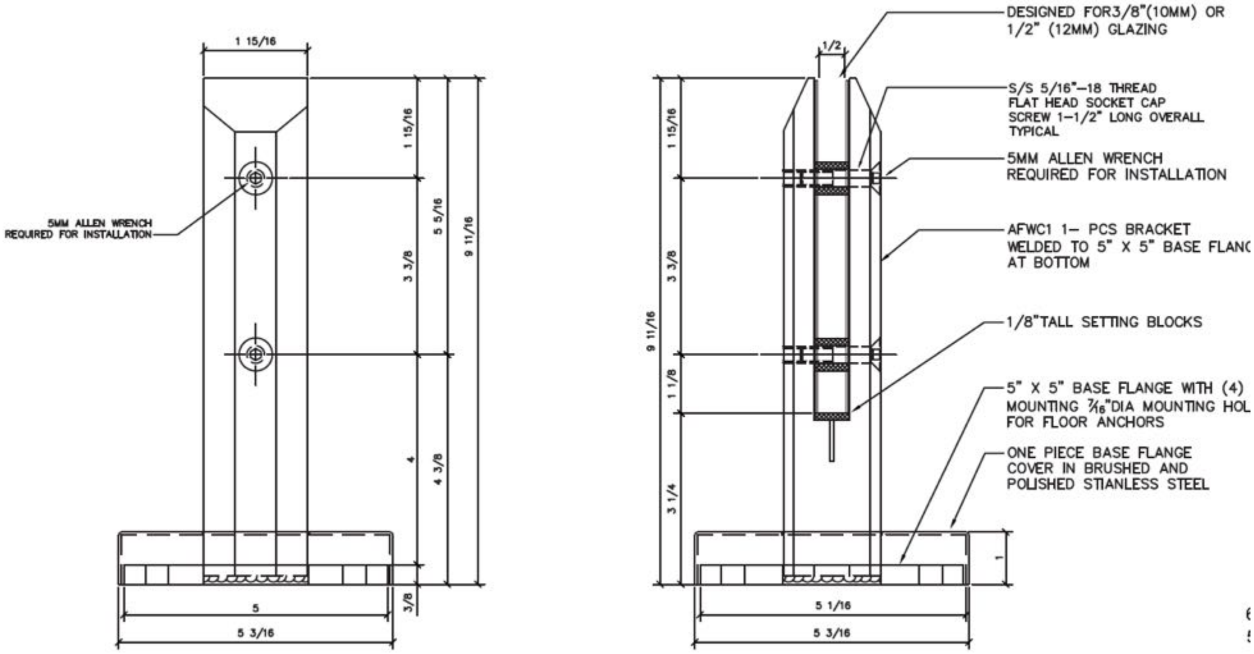
Moment Strength of fully installed clamp:

M_s = 2*2,982 = 5,964”#

ALLOWABLE DESIGN MOMENT = 5,964”# WITH BOLTS THROUGH GLASS

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

AFWC1S
One piece surface mounted clamp. Clamp body is similar to the AFWC1 clamp.



Check strength of clamp:
Glass to clamp moment resisted by tension in cap screws:
Screw tension is proportional to distance from bottom screw to screw being considered:

5/16" diameter cap screws upper two.

Screw tension strength - ASTM F 879

$T = 3{,}980\#$ (ASTM F 879 Table 3)

$T_a = 3{,}980/2 = 1{,}990\#$

Moment resistance M_a

$M_a = 1{,}990\#*(4.5"+1.125"^2/4.5") = 9{,}515"\#$

For installations without bolts through glass:
Check for bending of side plates:

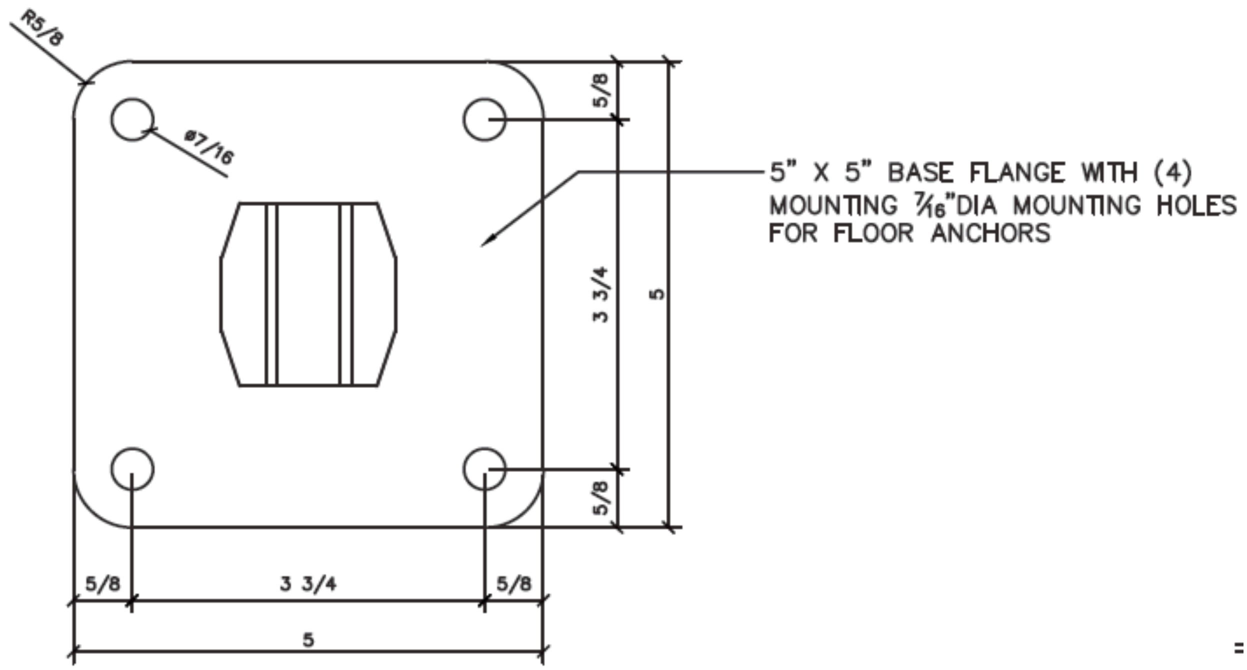
$S = 0.160\ in^3$

$F_y = 30$ ksi for cast 316 Stainless steel

$M_n = ZF_y = 0.160*30\ ksi = 4{,}800$ "#

$M_a = 4{,}800/1.67 = 2{,}874"\#$

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



CONNECTION TO STEEL:
3/8" SS Bolts to steel framing with sufficient strength to properly support the imposed loads-
Tension strength of 3/8" bolts (ASTM F 593)
$T_n = 0.0775in^2*70ksi = 5{,}425\#$
$T_s = 5{,}425\#/2 = 2{,}713\#$
Allowable moment based on anchor strength:
$M_s = 2*2{,}713\#*3.75" = 20{,}344"\#$

BASE PLATE MOUNTED TO CONCRETE - Expansion Bolt Alternative:
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8"x3.75" concrete anchors with 3" effective embedment. Anchor strength based on ESR-2427
Minimum conditions used for the calculations:
$f'_c \geq 3{,}000$ psi; edge distance =2.25" spacing = 3.75"
h = 3.0": embed depth
For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
$A_{Ncg}= (1.5*3*2+3.75)*(1.5*3+2.25) = 86.06\ in^2$ 2 anchors
$A_{Nco}= 9*3^2 = 81\ in^2$
$C_{a,cmin} = 1.5"$ (ESR-2427 Table 3)
$C_{ac} = 5.25"$ (ESR-2427 Table 3)
$\varphi_{ed,N} = 1.0$
$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}=$ max (1.5/5.25 or 1.5*3"/5.25) = 0.857 ($c_{a,min} \leq c_{ac}$)
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$

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

$N_{cb} = 86.06/81*1.0*1.0*0.857*6{,}830 = 6{,}219 \leq 2*4{,}200$
based on concrete breakout strength.
Determine allowable tension load on anchor pair
$T_s = 0.65*6{,}219\#/1.6 = 2{,}526\#$
Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
$A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03$
$A_{vco}= 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$
$\varphi_{ed,V}= 1.0$ (affected by only one edge)
$\varphi_{c,V}= 1.4$ uncracked concrete
$\varphi_{h,V}= \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} =1.225$
$V_b= [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5} =1{,}636\#$
$V_{cb} = 43.03/40.5*1.0*1.4*1.225*1{,}636\# = 2{,}981\#$
Steel shear strength = 1,830#*2 = 3,660
Allowable shear strength
$ØV_N/1.6 = 0.70*2{,}981\#/1.6 = 1{,}304\#$
Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load:
$M_a = 2{,}526\#*4.375" = 11{,}053"\# > 9{,}515"\#$
DEVELOPS FULL CLAMP STRENGTH.
ALLOWABLE SUBSTITUTIONS: Alternative concrete anchors may be designed for project conditions.
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

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

TO WOOD:

For 3/8" SS bolts to wood beams with bearing plates between bolt head and beam and framing has adequate strength to resist the loads:

Tension strength of 3/8" bolts (ASTM F 593)

$T_n = 0.0775in^2*70ksi = 5,425\#$

$T_s = 5,425\#/2 = 2,713\#$

Allowable moment based on anchor strength:

$M_s = 2*2,713\#*3.75" = 20,344"\#$

For 3/8" Lag screws:

For 8,400"# design load

$$T_{200} = \frac{8,400}{2*4.375"} = 960\#$$

Adjustment for wood bearing (assumes Hem-fir or similar wood):

$a = 2*960/(1.075*625psi*5") = 0.572"$

$T = 8,400/[2*(4.375-0.572/2)] = 1,027\#$

Required embed depth will depend on wood density and moisture content:

Withdrawal strength of 3/8" lag screw to wood with $G \geq 0.46$

$C_D = 1.6$ for guard impact loads and wind loads

W = 269 pli from NDS Table 11.2A

$W' = 1.6*269 = 430\#/in$

$Z_\perp = 170\#$ (NDS Table 11K)

$Z'_\perp = 1.6*170\# = 272\#$ each

Shear load will equal wind or live load - assume 100# per lag:

Combined lateral and withdrawal loads (NDS Table 11.4)

$Z'_\perp = (W'Z')/[W'Cos^2\alpha + Z'sin^2\alpha]$

Resultant = $\sqrt{(1027^2+100^2)} = 1,032\#$

$\alpha = tan^{-1}(1027/100) = 84.4^\circ$

Try assuming 3" embedment: $W'e = 430*3 = 1,290\#$

$Z'_\perp = (272*1,290)/[1,290Cos^2 84.4^\circ + 272sin^2 84.4^\circ] = 1,246\# \geq 1,032\#$

3" embedment assumption is good.

For protected installations the minimum embedment is 3" with

Allowable shear load = 400#

Allowable moment = 8,400"#

For weather exposed installations the minimum embedment is:

$l_e = 3"/C_M = 3/0.7 = 4.286"$

Lesser embedment will reduce allowable load by:

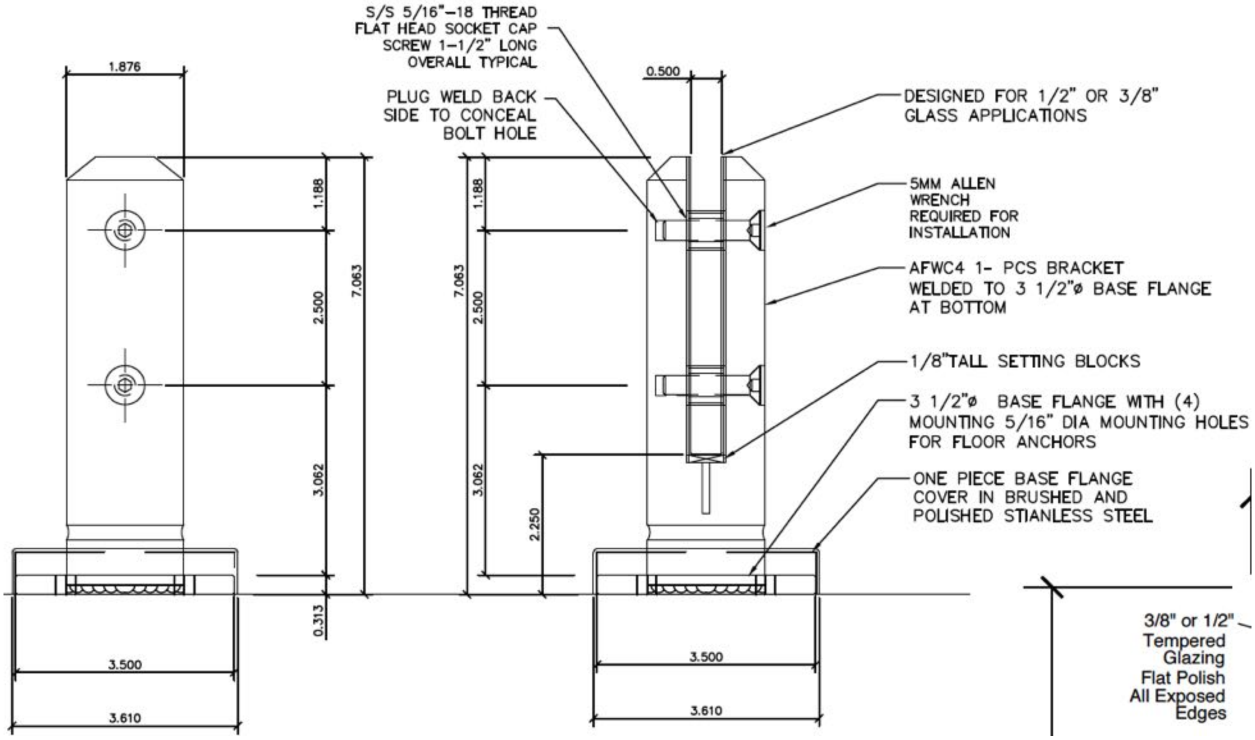
$M' = l_e/3"*8,400"\#$ for moisture content always below 19%

$M' = l_e/4.286*8,400"\#$ for moisture content that may go over 19%

Minimum embedment depth $l_e \geq 2.375"$ with reduced allowable moment load.

AFWC4S

One piece surface mounted clamp. Clamp body is similar to the AFWC1 clamp.



Clamp must be installed with bolt through the glass.

Check moment strength of clamp body

Glass to bracket moment resisted by tension in cap screws:

Screw tension is proportional to distance from bottom screw to screw being considered:

5/16" diameter cap screws connecting side plate to main clamp body

$T_{5/16}$ = 3,980# (ASTM F 879 Table 3)

Ta = 3,980#/2 = 1,990#

Moment resistance M_a

M_a = 1,990#*(3.312"+0.812"^{2}/3.312") = 6,987"#

Check for bending of side bars:

Z = 0.114 in^3

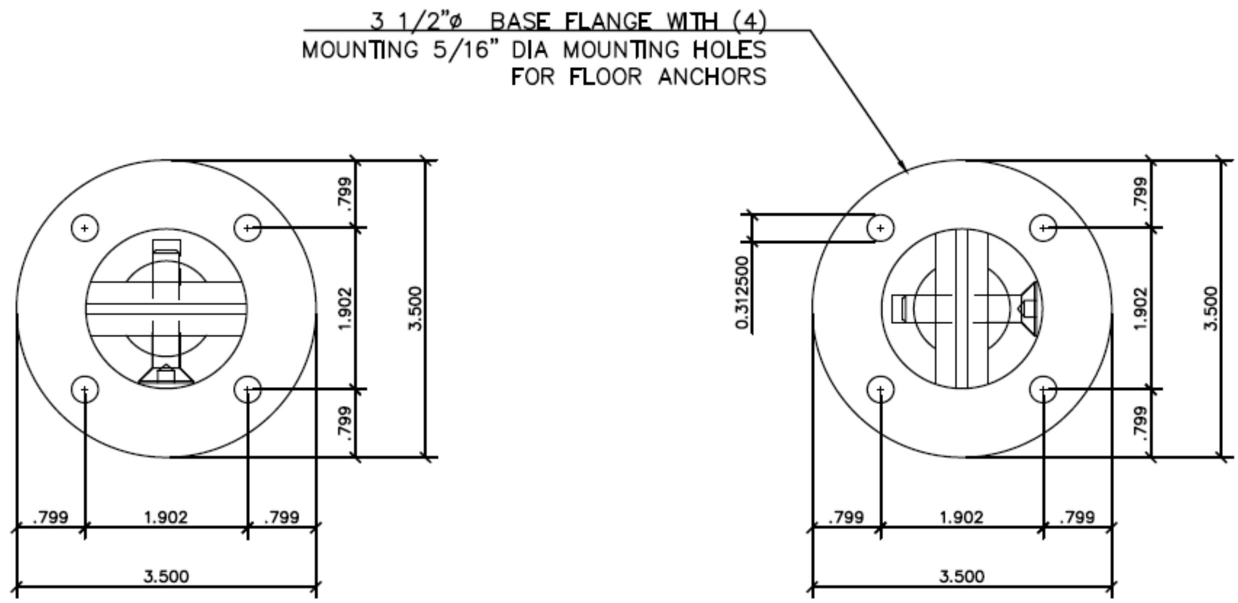
F_y = 30 ksi For cast 316 SS

M_n = 0.114*30 ksi = 3,420"#

M_s = 3,420/1.67 = 2,048#"

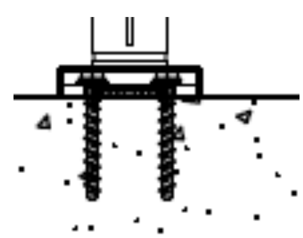
Moment Strength of fully installed clamp:

M_s = 2*2,048 = 4,096"#



CONNECTION TO STEEL:
3/8" SS Bolts to steel framing with sufficient strength to properly support the imposed loads-
Tension strength of 3/8" bolts (ASTM F 593)
$T_n = 0.0775in^2*70ksi = 5{,}425\#$
$T_s = 5{,}425\#/2 = 2{,}713\#$
Allowable moment based on anchor strength:
$M_s = 2*2{,}713\#*1.902" = 10{,}320"\#$

BASE PLATE MOUNTED TO CONCRETE -
Base plate mounted to concrete with Hilti HUS anchors with 3" embedment. Anchor strength based on ESR-3027 and ACI 318-08 Appendix D.



Minimum conditions used for the calculations:
$f'_c \geq 3{,}000$ psi; edge distance = 2.25" spacing = 1.902"
h = 3.0": embed depth
For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
$A_{Ncg}= (1.5*3*2+1.902)*(1.5*3+2.25) = 73.589$ in^2 2 anchors
$A_{Nco}= 9*3^2 = 81$ in^2
$C_{a,cmin} = 1.5"$ (ESR-3027 Table 2)
$C_{ac} = 3.2"$ (ESR-3027 Table 2)
$\varphi_{ed,N} = 1.0$
$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}=$ max (1.5/3.2 or 1.5*2.25"/3.2) = 1.05 but ≤ 1.0
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$
$N_{cb} = 73.589/81*1.0*1.0*0.857*6{,}830 = 5{,}318$
based on concrete breakout strength.

Steel strength:
T_{ns} = 9,200# each

Pullout strength - Per ESR-3027 Table 3 will not limit tension strength.

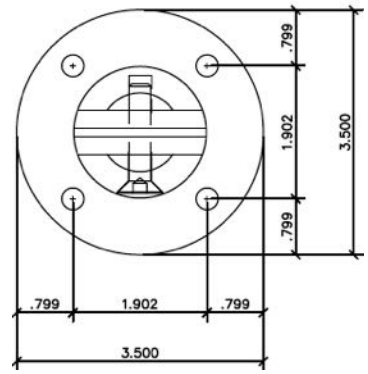
Determine allowable tension load on anchor pair
T_s = 0.65*5,318#/1.6 = 2,160#

Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03
$A_{vco} = 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$
$\varphi_{ed,V}$= 1.0 (affected by only one edge)
$\varphi_{c,V}$= 1.4 uncracked concrete
$\varphi_{h,V} = \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)}$ =1.225
$V_b = [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5}$ =1,636#
V_{cb} = 43.03/40.5*1.0*1.4*1.225*1,636# = 2,981#
Steel shear strength = 5,185*2 = 10,370#
Allowable shear strength
$ØV_N/1.6$ = 0.70*2,981#/1.6 = 1,304#

Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load for shear loads under 250#

M_a = 2,160#*1.902" = 4,108"# > 4,096"#
DEVELOPS FULL CLAMP STRENGTH.
ALLOWABLE SUBSTITUTIONS: Alternative concrete anchors may be designed for project conditions.
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

TO WOOD:
For 3/8" SS bolts to wood beams with bearing plates between bolt head and beam and framing has adequate strength to resist the loads:



Tension strength of 3/8" bolts (ASTM F 593)
T_n = 0.0775in^2*70ksi = 5,425#
T_s = 5,425#/2 = 2,713#
Allowable moment based on anchor strength:
M_s = 2*2,713#*1.902" = 10,282"#

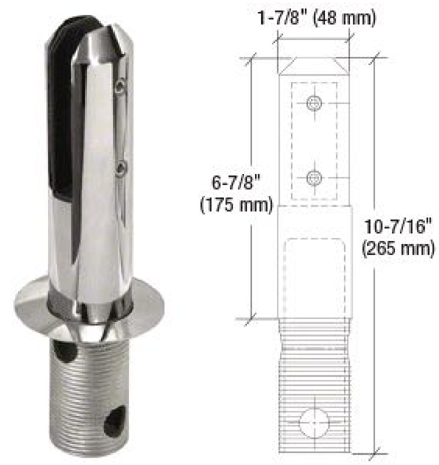
For 3/8" Lag screws:
For 4,096"# design load based on clamp strength
$T_{200} = \frac{4{,}096}{2*2.30"} = 890\#$
Adjustment for wood bearing (assumes Hem-fir or similar wood):
a = 2*890/(1.075*625psi*2.5")= 1.06"
T = 4,096/[2*(2.701-1.06/2)]= 943#
Required embed depth will depend on wood density and moisture content:
Withdrawal strength of 3/8" lag screw to wood with G ≥ 0.46
C_D = 1.6 for guard impact loads and wind loads
W = 269 pli from NDS Table 11.2A
W' = 1.6*269 = 430#/in
$Z_\perp$ = 170# (NDS Table 11K)
$Z'_\perp$ = 1.6*170# = 272# each
Shear load will equal wind or live load - assume 63# per lag:
Combined lateral and withdrawal loads (NDS Table 11.4)
$Z'_\perp = (W'Z')/[W'\cos^2\alpha + Z'\sin^2\alpha]$
Resultant = $\sqrt{(943^2+63^2)}$ = 945#
$\alpha = \tan^{-1}(943/63) = 86.2^\circ$
Try assuming 3" embedment: W'e = 430*3 = 1,290#
$Z'_\perp = (272*1{,}290)/[1290\cos^2 86.2^\circ + 272\sin^2 86.2^\circ] = 1{,}266\# \geq 945\#$
3" embedment assumption is good.

For protected installations the minimum embedment is 3"*945/1266 = 2.24 but not less then 2.375" with Allowable shear load = 250# and Allowable moment = 4,096"#

For weather exposed installations the minimum embedment is:
l_e = 3"/C_M = 2.24/0.7 = 3.2"

Lesser embedment will reduce allowable load by:
no reduction for moisture content always below 19%
M' = l_e/3.2*4,096"# for moisture content that may go over 19%
Minimum embedment depth l_e ≥ 2.375" with reduced allowable moment load.

FRICTION FIT CLAMPS
FWCR10



One piece stainless steel clamp for installation into cored hole. Glass is locked into clamp by tightening two set screws which press a bearing plate against the glass.

Check for bending of side bars:

$Z = 0.140\ in^3$

$F_y = 45$ ksi For 316 SS based on tested strength

$M_n = 0.140*45$ ksi = 6,300"#

$M_s = 6{,}300/1.67 = 3{,}772$#"

Allowable shear load:

$V_a = 0.6*45ksi*0.309in^2/2 = 4{,}171$#

Anchorage is achieved by embedding clamp base into hole in concrete and grouted in place.

Moment is resisted by pry out from concrete. The grooves on the end of the rod prevent withdrawal.

From $\sum$ M about centroid of C_l:

$M_n = C_l*D(2/3)$

$C_u = øF_B$

F_B = shear block failure from ACI 318 App D.6.2

$F_B = 2*1.4*7(3.5/1.875)^{0.2}(1.875*2500)^{1/2}*3.5^{1.5} = 9{,}956$#

$C_u = 0.85*9{,}956\# = 8{,}463$#

$M_n = 8{,}463\#*3.5*(2/3) = 19{,}746$#"

$M_s = 0.70*19{,}746/1.6 = 8{,}639$#"

NOTE ON CLAMP STRENGTH

The clamps were tested as part of a balustrade assembly in Australia by Azuma Design Pty Ltd, test reports revision date 18 April 2012. Based on the test reports the allowable moment for the clamp may be calculated as:

Maximum test load = 3.6 kN (809.3 lbs)

Maximum moment = 3.6kN*1035mm = 3.7778kN-m (809.3#*40.75" = 32,979"#

Load resisted by 3 glass lights with two clamps on each.

Based on load distribution from top rail to glass, glass to clamp the load to the center for clamps are assumed equal and the end clamps receive 1/2 the load of the middle clamps.

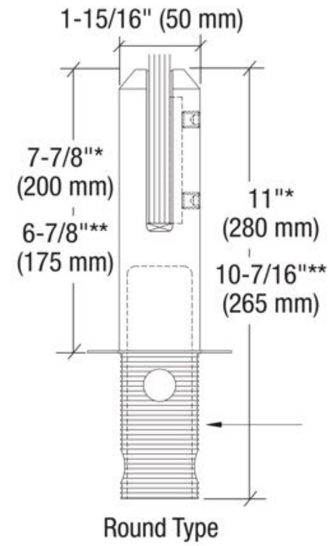
Maximum clamp moment = 32,979"#/4 = 8,245"# (test report indicated yielding of clamp occurred at the final test load but ultimate strength of the clamp wasn't reached)

SF = 8,245/3,772 = 2.19

Recommended design moment on clamp M_a = 3,772"#

Allowable shear load: V_a = 3,772/6.875" = 549#

FWCR12



Very similar to FWCR10 except slightly taller dimensions and fabricated from 2205 Duplex Stainless Steel. Dimensions on the image to the right with the single asterisk refer to the FWCR12 and double asterisks refer to FWCR10.

For 2205 Duplex Steel, F_y = 65ksi

M_n = 0.140*65ksi = 9,100”#

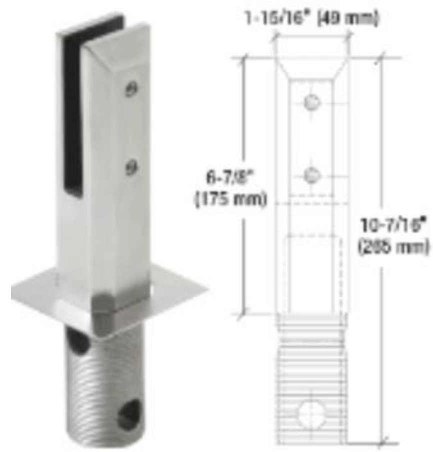
M_s = 9,100/1.67 = 5,450”#

Anchorage calculations are the same as for the FWCR10.

Recommended design moment on clamp M_a = 5,450”#
Allowable shear load: V_a = 5,450”#/7.875” = 692#

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

FWCS10



One piece stainless steel clamp for installation into cored hole. Glass is locked into clamp by tightening two set screws which press a bearing plate against the glass.

Check for bending of side bars:

$Z = 0.192$ in^3

$F_y = 45$ ksi For 316 SS based on tested strength

$M_n = 0.192*45$ ksi $= 8{,}640$"#

$M_s = 8{,}640/1.67 = 5{,}174$"#

Allowable shear load:

$V_a = 0.6*45$ksi$*1.117$in$^2/2 = 15{,}082$#

Anchorage is achieved by embedding clamp base into hole in concrete and grouted in place.

Moment is resisted by pry out from concrete. The grooves on the end of the rod prevent withdrawal.

From $\sum$ M about centroid of C_l:

$M_n = C_l*D(2/3)$

$C_u = øF_B$

F_B = shear block failure from ACI 318 App D.6.2

$F_B = 2*1.4*7(3.5/1.875)^{0.2}(1.875*2500)^{1/2}*3.5^{1.5} = 9{,}956$#

$C_u = 0.85*9{,}956\# = 8{,}463$#

$M_n = 8{,}463\#*3.5*(2/3) = 19{,}746$#"

$M_s = 0.70*19{,}746/1.6 = 8{,}639$#"

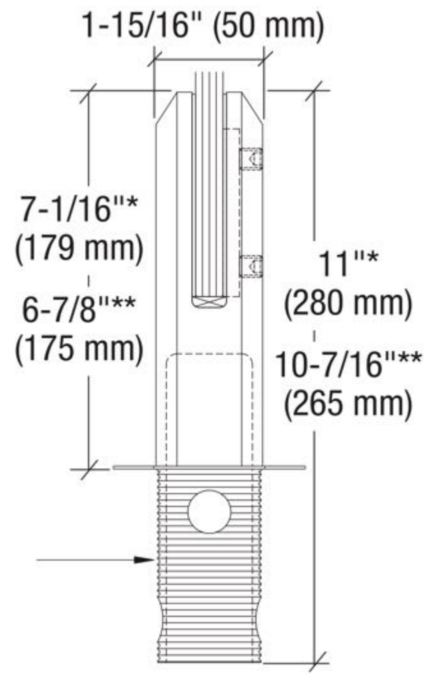
NOTE ON CLAMP STRENGTH

Based on the tests performed on the FWCR10 clamps and that the FWCR10 clamps are fabricated by the same methods from the same stainless steel grade and type the moment strength strength of the clamp may be calculated the same way. Thus as demonstrated for the FWCR10 clamp the yield strength of 45 ksi is appropriate.

Recommended design moment on clamp $M_a = 5{,}174$"#

Allowable shear load: $V_a = 5{,}174/6.875" = 753$#

FWCS12



Very similar to FWCS10 except slightly taller dimensions and fabricated from 2205 Duplex Stainless Steel. Dimensions on the image to the right with the single asterisk refer to the FWCS12 and double asterisks refer to FWCS10.

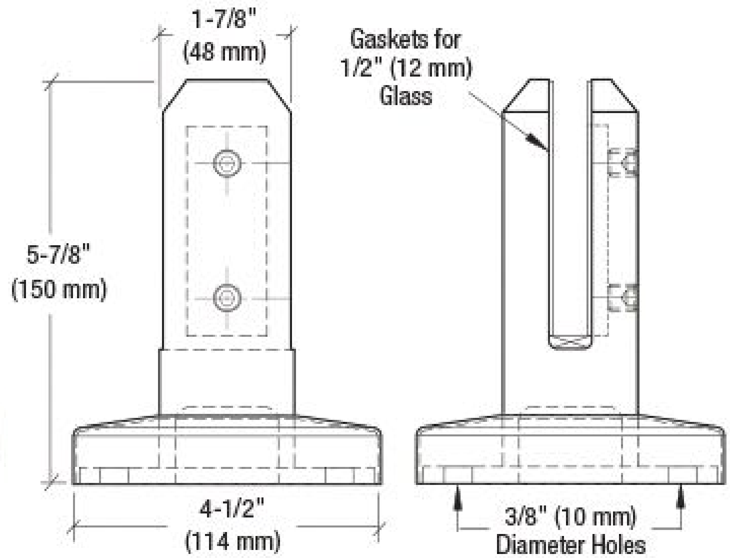
For 2205 Duplex Steel, F_y = 65ksi

M_n = 0.192*65ksi = 12,500”#
M_s = 12,500/1.67 = 7,480”#

Anchorage strength is slightly reduced for the FWCS12 because the baseplates are slightly smaller.
Reduction = (1.902”-(4.3125”-3.8125”)/2)/1.902” = 0.869
Anchorage to steel strength, M_a = 10,300”#*0.869 = 8,950”#
Anchorage to concrete strength, M_a = 4,108”#*0.869 = 3,570”# (Less than clamp strength)
Required lag screw embedment to achieve clamp strength, P = 2.05”/0.869 = 2.36” (Not to be less than 2-3/8”)

Recommended design moment on clamp M_a = 7,480”#
Allowable shear load: V_a = 7,480”#/7.0625” = 1,060#

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



FWCR20

One piece stainless steel clamp with base shoe for surface mounting. Glass is locked into clamp by tightening two set screws which press a bearing plate against the glass.

Check for bending of side bars:

$Z = 0.140 \text{ in}^3$

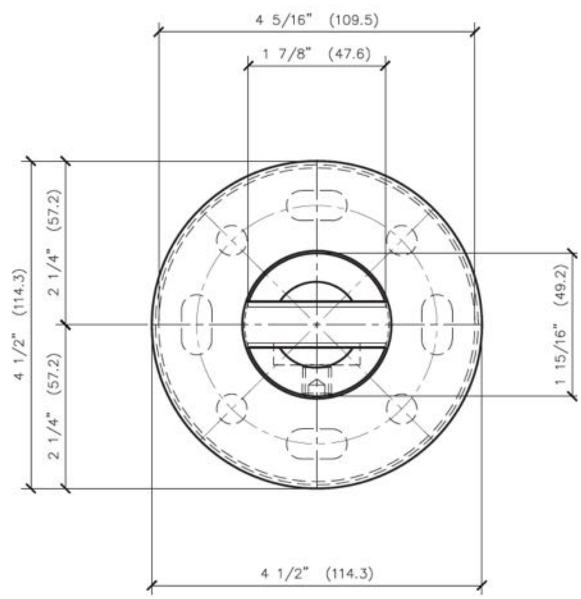
$F_y = 45$ ksi For 316 SS

$M_n = 0.140*45 \text{ ksi} = 6{,}300"\#$

$M_s = 6{,}300/1.67 = 3{,}772\#"$

Allowable shear load:

$V_a = 0.6*45\text{ksi}*0.309\text{in}^2/2 = 4{,}171\#$



Strength of base plate mounts:

CONNECTION TO STEEL:

3/8" SS Bolts to steel framing with sufficient strength to properly support the imposed loads-

Tension strength of 3/8" bolts (ASTM F 593)

$T_n = 0.0775\text{in}^2*70\text{ksi} = 5{,}425\#$

$T_s = 5{,}425\#/2 = 2{,}713\#$

Allowable moment based on anchor strength:

$M_s = 2*2{,}713\#*1.902" = 10{,}320"\#$

BASE PLATE MOUNTED TO CONCRETE -

Base plate mounted to concrete with Hilti HUS anchors with 3" embedment. Anchor strength based on ESR-3027 and ACI 318-08 Appendix D.

Minimum conditions used for the calculations:

$f'_c \geq 3{,}000$ psi; edge distance = 2.25" spacing = 1.902"

h = 3.0": embed depth

For concrete breakout strength:

$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$

A_{Ncg}= (1.5*3*2+1.902)*(1.5*3+2.25) = 73.589 in^2 2 anchors
A_{Nco}= 9*3^2 = 81 in^2
$C_{a,cmin}$ = 1.5" (ESR-3027 Table 2)
C_{ac} = 3.2" (ESR-3027 Table 2)
$\varphi_{ed,N}$ = 1.0
$\varphi_{c,N}$ = (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}$= max (1.5/3.2 or 1.5*2.25"/3.2) = 1.05 but ≤ 1.0
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$
N_{cb} = 73.589/81*1.0*1.0*0.857*6,830 = 5,318
based on concrete breakout strength.

Steel strength:
T_{ns} = 9,200# each

Pullout strength - Per ESR-3027 Table 3 will not limit tension strength.

Determine allowable tension load on anchor pair
T_s = 0.65*5,318#/1.6 = 2,160#

Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03
$A_{vco}= 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$
$\varphi_{ed,V}$= 1.0 (affected by only one edge)
$\varphi_{c,V}$= 1.4 uncracked concrete
$\varphi_{h,V}= \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} =1.225$
$V_b= [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5} =1{,}636\#$
V_{cb} = 43.03/40.5*1.0*1.4*1.225*1,636# = 2,981#
Steel shear strength = 5,185*2 = 10,370#
Allowable shear strength
$ØV_N$/1.6 = 0.70*2,981#/1.6 = 1,304#

Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load for shear loads under 250#

M_a = 2,160#*1.902" = 4,108"# > 3,772"#
DEVELOPS FULL CLAMP STRENGTH.
ALLOWABLE SUBSTITUTIONS: Alternative concrete anchors may be designed for project conditions.
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

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

TO WOOD:

For 3/8" SS bolts to wood beams with bearing plates between bolt head and beam and framing has adequate strength to resist the loads:

Tension strength of 3/8" bolts (ASTM F 593)

$T_n = 0.0775in^2*70ksi = 5,425\#$

$T_s = 5,425\#/2 = 2,713\#$

Allowable moment based on anchor strength:

$M_s = 2*2,713\#*1.902" = 10,282"\#$

For 3/8" Lag screws:

For 3,772#" design load based on clamp strength

$$T_{200} = \frac{3,772}{2*2.30"} = 820\#$$

Adjustment for wood bearing (assumes Hem-fir or similar wood):

$a = 2*820/(1.075*625psi*2.5")= 0.976"$

$T = 3,772/[2*(2.701-0.976/2)]= 852\#$

Required embed depth will depend on wood density and moisture content:

Withdrawal strength of 3/8" lag screw to wood with $G \geq 0.46$

$C_D = 1.6$ for guard impact loads and wind loads

W = 269pli from NDS Table 11.2A

$W' = 1.6*269 = 430\#/in$

$Z_\perp = 170\#$ (NDS Table 11K)

$Z'_\perp = 1.6*170\# = 272\#$ each

Shear load will equal wind or live load - assume 75# per lag:

Combined lateral and withdrawal loads (NDS Table 11.4)

$Z'_\perp = (W'Z')/[W'Cos^2\alpha + Z'sin^2\alpha]$

Resultant = $\sqrt{(852^2+75^2)} = 855\#$

$\alpha = tan^{-1}(855/75) = 85.0^\circ$

Try assuming 3" embedment: $W'e = 430*3 = 1,290\#$

$Z'_\perp = (272*1,290)/[1290Cos^285^\circ + 272sin^285^\circ] = 1,254\# \geq 855\#$

3" embedment assumption is good.

For protected installations the minimum embedment is 3"*855/1254 = 2.05" but not under 2.375" with Allowable shear load = 300# and Allowable moment = 3,772"#

For weather exposed installations the minimum embedment is:

$l_e = 3"/C_M = 2.05/0.7 = 2.93"$

Lesser embedment will reduce allowable load by:

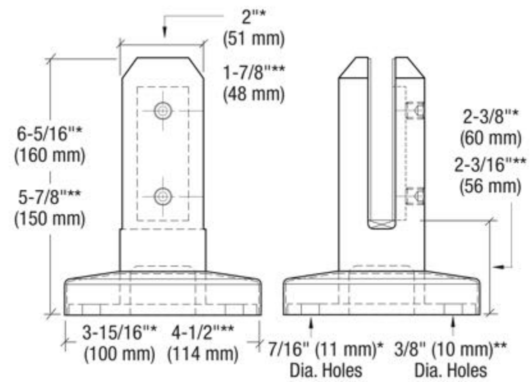
No reduction below 2.375" for moisture content always below 19%

$M' = l_e/4.286*2.93"\#$ for moisture content that may go over 19%

Minimum embedment depth $l_e \geq 2.375"$ with reduced allowable moment load.

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

FWCR22



Very similar to FWCS20 except slightly taller dimensions and fabricated from 2205 Duplex Stainless Steel. Dimensions on the image to the right with the single asterisk refer to the FWCS22 and double asterisks refer to FWCS20.

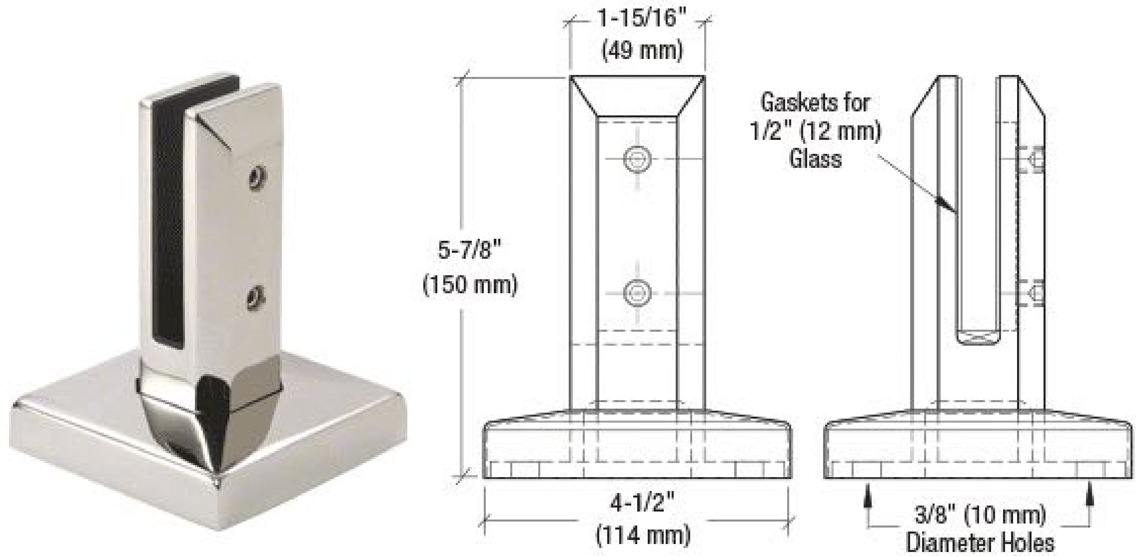
For 2205 Duplex Steel, F_y = 65ksi

M_n = 0.140*65ksi = 9,100"#
M_s = 9,100/1.67 = 5,450"#

Anchorage strength is slightly reduced for the FWCR22 because the baseplates are slightly smaller.
Reduction = (1.902"-(4.3125"-3.8125")/2)/1.902" = 0.869
Anchorage to steel strength, M_a = 10,300"#*0.869 = 8,950"#
Anchorage to concrete strength, M_a = 4,108"#*0.869 = 3,570"# (Less than clamp strength)
Required lag screw embedment to achieve clamp strength, P = 2.05"/0.869 = 2.36" (Not to be less than 2-3/8")

Recommended design moment on clamp M_a = 5,450"#
Allowable shear load: V_a = 5,450"#/6.3125" = 863#

FWCS20



One piece stainless steel clamp with base shoe for surface mounting. Glass is locked into clamp by tightening two set screws which press a bearing plate against the glass.

Check for bending of side bars:

$Z = 0.192\ in^3$

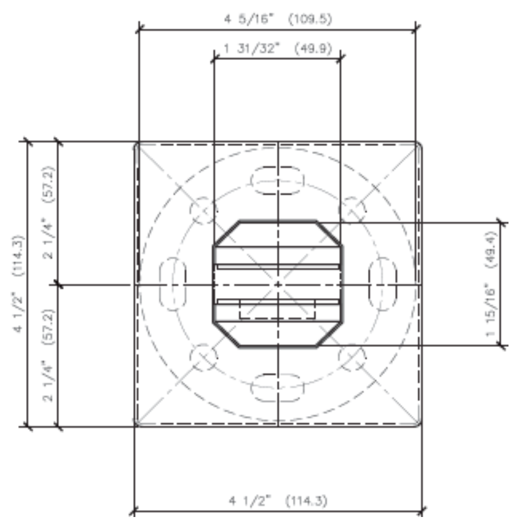
$F_y = 45$ ksi For 316 SS based on tested strength

$M_n = 0.192*45\ ksi = 8,640"\#$

$M_s = 8,640/1.67 = 5,174"\#$

Allowable shear load:

$V_a = 0.6*45ksi*1.117in^2/2= 15,082\#$



Strength of base plate mounts:

CONNECTION TO STEEL:

3/8" SS Bolts to steel framing with sufficient strength to properly support the imposed loads-

Tension strength of 3/8" bolts (ASTM F 593)

$T_n = 0.0775in^2*70ksi = 5,425\#$

$T_s = 5,425\#/2 = 2,713\#$

Allowable moment based on anchor strength:

$M_s = 2*2,713\#*1.902" = 10,320"\#$

BASE PLATE MOUNTED TO CONCRETE -

Base plate mounted to concrete with Hilti HUS anchors with 3" embedment. Anchor strength based on ESR-3027 and ACI 318-08 Appendix D.

Minimum conditions used for the calculations:

$f'_c \geq 3,000$ psi; edge distance = 2.25" spacing = 1.902"

h = 3.0": embed depth

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

For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
A_{Ncg}= (1.5*3*2+1.902)*(1.5*3+2.25) = 73.589 in^2 2 anchors
A_{Nco}= 9*3^2 = 81 in^2
$C_{a,cmin}$ = 1.5" (ESR-3027 Table 2)
C_{ac} = 3.2" (ESR-3027 Table 2)
$\varphi_{ed,N}$ = 1.0
$\varphi_{c,N}$ = (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}$= max (1.5/3.2 or 1.5*2.25"/3.2) = 1.05 but ≤ 1.0
N_b = 24*1.0*√3000*$3.0^{1.5}$ = 6,830#
N_{cb} = 73.589/81*1.0*1.0*0.857*6,830 = 5,318
based on concrete breakout strength.

Steel strength:
T_{ns} = 9,200# each

Pullout strength - Per ESR-3027 Table 3 will not limit tension strength.

Determine allowable tension load on anchor pair
T_s = 0.65*5,318#/1.6 = 2,160#

Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03
A_{vco}= $4.5(c_{a1})^2 = 4.5(3)^2$ = 40.5
$\varphi_{ed,V}$= 1.0 (affected by only one edge)
$\varphi_{c,V}$= 1.4 uncracked concrete
$\varphi_{h,V}$= $\sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)}$ =1.225
V_b= $[7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5}$ =1,636#
V_{cb} = 43.03/40.5*1.0*1.4*1.225*1,636# = 2,981#
Steel shear strength = 5,185*2 = 10,370#
Allowable shear strength
$ØV_N$/1.6 = 0.70*2,981#/1.6 = 1,304#

Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load for shear loads under 250#

M_a = 2,160#*1.902" = 4,108"# < 5,174"#
LIMITS CLAMP STRENGTH.
ALLOWABLE SUBSTITUTIONS: Alternative concrete anchors may be designed for project conditions.
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

TO WOOD:
For 3/8" SS bolts to wood beams with bearing plates between bolt head and beam and framing has adequate strength to resist the loads:
Tension strength of 3/8" bolts (ASTM F 593)
$T_n = 0.0775in^2*70ksi = 5,425\#$
$T_s = 5,425\#/2 = 2,713\#$
Allowable moment based on anchor strength:
$M_s = 2*2,713\#*1.902" = 10,282"\#$

For 3/8" Lag screws:
For 5,174"# design load based on clamp strength
$T_{200} = \frac{5,174}{2*2.30"} = 1,125\#$
Adjustment for wood bearing (assumes Hem-fir or similar wood):
a = 2*1,125/(1.075*625psi*2.5")= 1.34"
T = 4,096/[2*(2.701-1.34/2)]= 1,008#
Required embed depth will depend on wood density and moisture content:
Withdrawal strength of 3/8" lag screw to wood with $G \geq 0.46$
$C_D = 1.6$ for guard impact loads and wind loads
W = 269 pli from NDS Table 11.2A
W' = 1.6*269 = 430#/in
$Z_\perp = 170\#$ (NDS Table 11K)
$Z'_\perp = 1.6*170\# = 272\#$ each
Shear load will equal wind or live load - assume 67# per lag:
Combined lateral and withdrawal loads (NDS Table 11.4)
$Z'_\perp = (W'Z')/[W'Cos^2\alpha + Z'sin^2\alpha]$
Resultant = $\sqrt{}(1008^2+67^2) = 1,010\#$
$\alpha = tan^{-1}(1,010/67) = 86.2^\circ$
Try assuming 3" embedment: W'e = 430*3 = 1,290#
$Z'_\perp = (272*1,290)/[1290Cos^2 86.2^\circ + 272sin^2 86.2^\circ] = 1,266\# \geq 1,010\#$
3" embedment assumption is good.

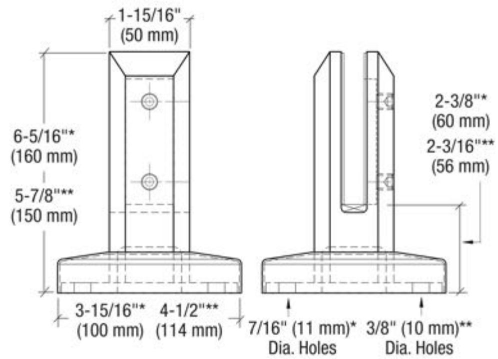
For protected installations the minimum embedment is 3"*1010/1266= 2.4" with
Allowable shear load = 268#
Allowable moment = 5,174"#

For weather exposed installations the minimum embedment is:
$l_e = 3"/C_M = 2.4/0.7 = 3.42"$

Lesser embedment will reduce allowable load by:
$M' = l_e/2.4"*5,425"\#$ for moisture content always below 19%
$M' = l_e/3.42*5,425"\#$ for moisture content that may go over 19%
Minimum embedment depth $l_e \geq 2.375"$ with reduced allowable moment load.

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

FWCS22



Very similar to FWCS20 except slightly taller dimensions and fabricated from 2205 Duplex Stainless Steel. Dimensions on the image to the right with the single asterisk refer to the FWCS22 and double asterisks refer to FWCS20.

For 2205 Duplex Steel, F_y = 65ksi

M_n = 0.192*65ksi = 12,500"#
M_s = 12,500/1.67 = 7,480"#

Anchorage strength is slightly reduced for the FWCS22 because the baseplates are slightly smaller.
Reduction = (1.902"-(4.3125"-3.8125")/2)/1.902" = 0.869
Anchorage to steel strength, M_a = 10,300"#*0.869 = 8,950"#
Anchorage to concrete strength, M_a = 4,108"#*0.869 = 3,570"# (Less than clamp strength)
Required lag screw embedment to achieve clamp strength, P = 2.4"/0.869 = 2.76"

Recommended design moment on clamp M_a = 7,480"#
Allowable shear load: V_a = 7,480"#/6.3125" = 1,180#

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

TABLE 15: CLAMP STRENGTH SUMMARY TABLE

	BOLTS THRU GLASS		NO BOLTS THRU GLASS	
CLAMP STYLE	SHEAR LBS	MOMENT IN-LBS	SHEAR LBS	MOMENT IN-LBS
AFWC1 AND AFWC6	1194	8,222	597	2,874
AFWC2 AND AFWC7	TO STEEL 478	TO STEEL 9,453	N/A	N/A
AFWC2 AND AFWC7	CONCRETE 108	CONCRETE 2,803	N/A	N/A
AFWC2 AND AFWC7	WOOD 78	WOOD 4,228	N/A	N/A
AFWC3 AND AFWC8	1194	4,974	597	2,487
DFWC3	1194	7,458	597	3,729
AFWC1S	400	8,400	400	2,874
AFWC4	597	5,964	N/A	N/A
AFWC4S	400	4,096	400	2,048
FWCR10	N/A	N/A	549	3,772
FWCS10	N/A	N/A	753	5,174
FWCR20 TO STEEL	N/A	N/A	1000	3,772
FWCR20 CONCRETE	N/A	N/A	600	3,772
FWCR20 WOOD	N/A	N/A	600	3,772
FWCS20 TO STEEL	N/A	N/A	1000	5,174
FWCS20 CONCRETE	N/A	N/A	250	4,108
FWCS20 WOOD	N/A	N/A	268	5,174

Appendix
Determination of moment and deflection amplification factors
36” Wide 50plf

Height	36	42	48	60
Mave	1800	2100	2400	3000
Calculated Stress	4090.90909090909	4772.72727272727	5454.54545454545	6818.18181818182
Stress by FEA	8729	10250	11920	15110
ß	2.13375555555556	2.14761904761905	2.18533333333334	2.21613333333333
Calculated Defl.	0.724776969032481	1.15091898323213	1.71798985252144	3.35544893070593
Defl. by FEA	0.726	1.153	1.720	3.358
λ	1.00168745837654	1.00180813488889	1.00117005782986	1.00076027659689

36” Wide 10psf

Height	36	42	48	60
Mave	540	735	960	1500
Calculated Stress	1227.27272727273	1670.45454545455	2181.81818181818	3409.09090909091
Stress by FEA	2431	3358	4512	7213
ß	1.98081481481481	2.010231292517	2.068	2.11581333333333
Calculated Defl.	0.163074818032308	0.302116233098435	0.515396955756431	1.25829334901472
Defl. by FEA	0.162	0.300	0.513	1.255
λ	0.99340904962963	0.992995301587302	0.995349301679687	0.997382685828151

36” Wide 200# Concentrated - Overstressed at all heights

Height	36	42	48	60
Stress by FEA	14100	16040	18340	22530
Defl. by FEA	1.202	1.816	2.619	4.897

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

48” Wide 50plf

Height	36	42	48	60
Mave	1800	2100	2400	3000
Calculated Stress	4090.90909090909	4772.72727272727	5454.54545454545	6818.18181818182
Stress by FEA	11080	13150	15300	19280
ß	2.70844444444445	2.7552380952381	2.805	2.82773333333333
Calculated Defl.	0.724776969032481	1.15091898323213	1.71798985252144	3.35544893070593
Defl. by FEA	0.823	1.288	1.901	3.646
λ	1.13552173311831	1.11910570488889	1.106525744148	1.08659081848489

48” Wide 10psf

Height	36	42	48	60
Mave	540	735	960	1500
Calculated Stress	1227.27272727273	1670.45454545455	2181.81818181818	3409.09090909091
Stress by FEA	3109	4344	5820	9257
ß	2.53325925925925	2.60048979591836	2.6675	2.71538666666667
Calculated Defl.	0.163074818032308	0.302116233098435	0.515396955756431	1.25829334901472
Defl. by FEA	0.187	0.342	0.578	1.388
λ	1.14671291531322	1.13201464380952	1.12146568493345	1.10308140870874

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

60” Wide 50plf

Height	36	42	48	60
Mave	1800	2100	2400	3000
Calculated Stress	4090.90909090909	4772.72727272727	5454.54545454545	6818.18181818182
Stress by FEA	13700	16140	18690	23550
ß	3.34888888888889	3.38171428571429	3.4265	3.454
Calculated Defl.	0.724776969032481	1.15091898323213	1.71798985252144	3.35544893070593
Defl. by FEA	0.932	1.442	2.106	3.978
λ	1.28591282535391	1.25291182177778	1.22585124522656	1.185534359828

60” Wide 10psf

Height	36	42	48	60
Mave	540	735	960	1500
Calculated Stress	1227.27272727273	1670.45454545455	2181.81818181818	3409.09090909091
Stress by FEA	3861	5350	7132	11330
ß	3.14599999999999	3.20272108843537	3.26883333333334	3.32346666666667
Calculated Defl.	0.163074818032308	0.302116233098435	0.515396955756431	1.25829334901472
Defl. by FEA	0.215	0.390	0.652	1.539
λ	1.31841324487883	1.29089389206349	1.26504433663773	1.223085221904

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

68” Wide 50plf

Height	36	42	48	60
Mave	1800	2100	2400	3000
Calculated Stress	4090.90909090909	4772.72727272727	5454.54545454545	6818.18181818182
Stress by FEA	15430	18150	21020	26470
ß	3.77177777777778	3.80285714285715	3.85366666666667	3.88226666666667
Calculated Defl.	0.724776969032481	1.15091898323213	1.71798985252144	3.35544893070593
Defl. by FEA	1.01	1.552	2.254	4.219
λ	1.39353213906379	1.34848761955556	1.31199843624913	1.25735783411622

68” Wide 10psf

Height	36	42	48	60
Mave	540	735	960	1500
Calculated Stress	1227.27272727273	1670.45454545455	2181.81818181818	12760
Stress by FEA	4355	6029	8031	11330
ß	3.54851851851851	3.60919727891155	3.680875	0.887931034482759
Calculated Defl.	0.163074818032308	0.302116233098435	0.515396955756431	1.25829334901472
Defl. by FEA	0.238	0.423	0.704	1.648
λ	1.45945280130773	1.4001233752381	1.36593744324074	1.30971049103171

72” Wide 50plf

Height	36	42	48	60
Mave	1800	2100	2400	3000
Calculated Stress	4090.90909090909	4772.72727272727	5454.54545454545	6818.18181818182
Stress by FEA	16300	19170	22190	27940
ß	3.98444444444445	4.01657142857143	4.06816666666667	4.09786666666667
Calculated Defl.	0.724776969032481	1.15091898323213	1.71798985252144	3.35544893070593
Defl. by FEA	1.05	1.609	2.331	4.344
λ	1.44872153070988	1.39801326022223	1.35681825860547	1.29461067347733

72” Wide 10psf

Height	36	42	48	60
Mave	540	735	960	1500
Calculated Stress	1227.27272727273	1670.45454545455	2181.81818181818	12760
Stress by FEA	4603	6370	8483	13470
ß	3.75059259259258	3.81333333333332	3.88804166666667	1.05564263322884
Calculated Defl.	0.163074818032308	0.302116233098435	0.515396955756431	1.25829334901472
Defl. by FEA	0.25	0.441	0.730	1.704
λ	1.53303865683585	1.45970309333333	1.41638399654224	1.35421521645511

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