Edward C. Robison, PE

16 OCT 2020

C.R. Laurence Co., Inc.
2503 East Vernon
Los Angeles, CA 90058

SUBJ: CR LAURENCE ALUMINUM WINDSCREEN SYSTEM - AWS
ALUMINUM FRAMED GLASS WIND WALLS AND FENCES

The AWS is an engineered system designed for the following criteria:
The design loading conditions are:
Concentrated load = 200 lbs (1 sf area) @ 42” above grade or,
Distributed load = 50 plf @ 42” above finish grade or,
Concentrated load = 50 lbs on 1 sf area any location or,
Uniform load = 10 psf or,
Seismic loads will not affect design because of the small dead loads.
Wind load as calculated based on ASCE/SEI 7-05 or 7-10 and as limited for the specific configuration as shown in tables 2 to 11 as applicable.

For these conditions the system will meet all applicable requirements of the 2006, 2009, 2012, 2015 and 2018 International Building Codes and International Residential Codes along with state codes adopting the IBC and IRC and 2015 Aluminum Design Manual. The system will meet all requirements for a swimming pool enclosure when installed as recommended and in compliance with IBC Section 3109. When fall protection is required a top rail or a grab rail must be installed at 42” (36” for IRC compliance) above the walking surface. Refer to the appropriate tables herein to determine allowable post spacing, heights and allowable stress design wind loads. The supporting structure shall be designed by others and be adequate to support the AWS with all imposed loads. It is the specifier’s responsibility to verify suitability for any specific applications or installations based on specific conditions and requirements.

10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

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

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

COLORADO LICENSED
EDWARD C. ROBISON
54027
PROFESSIONAL ENGINEER

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

GEORGIA
REGISTERED
No. 046121
PROFESSIONAL
ENGINEER
EDWARD C. ROBISON

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

THIS WORK WAS PREPARED BY ME OR UNDER MY SUPERVISION
04/30/2022
Signature Expiration Date of the License

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

STATE OF MARYLAND
EDWARD C. ROBISON
52500
PROFESSIONAL ENGINEER

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland, License No. 52500, Expiration Date: 04/09/2020

COMMONWEALTH OF MASSACHUSETTS
EDWARD C. ROBISON
STRUCTURAL
No. 49757
REGISTERED
PROFESSIONAL ENGINEER
EXP 06/30/2022

EDWARD C. ROBISON
LICENSED PROFESSIONAL
ENGINEER
29000
STATE OF MISSISSIPPI

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

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

LICENSED PROFESSIONAL ENGINEER
E. C. ROBISON
31646
OKLAHOMA

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 09/30/2021

STATE OF TEXAS
EDWARD C. ROBISON
100688
LICENSED
PROFESSIONAL ENGINEER
FIRM #F-12044
EXP 12/31/2020

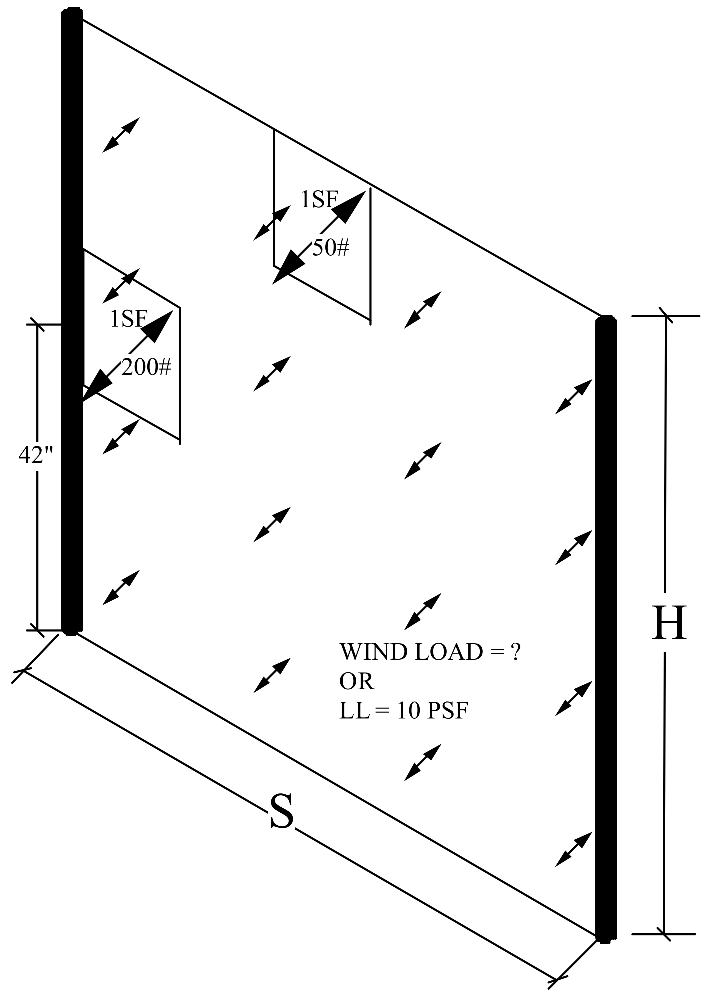
LICENSED STRUCTURAL ENGINEER
No. 10738527-2203
EDWARD C. ROBISON
STATE OF UTAH

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



Loading to Posts:
Live load = 200# @ 42" height
Any location along wall
(42" above finish floor)
Or:
50 lb on one square foot at any location on glass.
Or:
Wind load on solid area
Or:
10 psf live load on entire area including voids.

M_{200} = 200#x42" = 8,400"#

M_{50plf} = 50*42"*S
Maximum spacing when fall protection is required:
S = M_a/2,100"#/ft

M_{50}= 50lb x H*12"/ft
(will not govern post design)

M_{LL} = 10psf*(S)*(H^2/2)*12"/ft

M_{WL}= W*(S)*(H^2*0.55)('#)
Wind loading typically controls post design.
Wind loads are based on allowable stress design, strength level wind loads as specified in 2012 IBC - ASCE 7-10 must be adjusted to ASD levels by using 0.6W.
Determine the maximum post heights
M_a = Allowable post moment
for Wind load:
M_{WL}= W*(S)*(H^2*0.55) = M_a
Solving for S
S = M_a/(0.55*W*H^2)

Solving for H
H = $[M_a/(0.55*W*S)]^{1/2}$

Allowable wind load:
W = M_a/(0.55*S*H^2)

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

WIND LOADING ON FENCES OR GUARDS

For wind load surface area is full area of fence or guard:
Calculated in accordance with ASCE/SEI 7-05 Section 6.5.14 *Design Wind Loads on Solid Freestanding Walls and Solid Signs* **(or ASCE/SEI 7-10 Chapter 29.4)**. This section is applicable for free standing building guardrails, wind walls and balcony railings that return to building walls. Section 6.5.12.4.4 (29.6) *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.**

$p = q_p(GC_p) = q_zGC_f$ (ASCE 7-05 eq. 6-26 or 7-10 eq. 29.4-1)
G = 0.85 from section 6.5.8.2 (sec 26.9.4.)
C_f = 2.5*0.8*0.6 = 1.2 Figure 6-20 (29.4-1) with reduction for solid and end returns, will vary.
$q_z = K_zK_{zt}K_dV^2I$ Where:

I = 1.0
K_z from Table 6-3 (29.3-1) at the height z of the railing centroid and exposure.
K_d = 0.85 from Table 6-4 (Table 26-6).
K_{zt} From Figure 6-4 (Fig 26.8-1) for the site topography, typically 1.0.
V = Wind speed (mph) 3 second gust, Figure 6-1 (Fig 26.5-1A) or per local authority.

Simplifying - Assuming $1.3 \le C_f \le 2.6$ (Typical limits for fence or guard with returns.)

For $C_f = 1.3$: $F = q_z*0.85*1.3 = 1.11\ q_z$
For $C_f = 2.6$: $F = q_z*0.85*2.6 = 2.21q_z$

Wind Load will vary along length of fence in accordance with ASCE 7-05 Figure 6-20 (29.4-1).
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

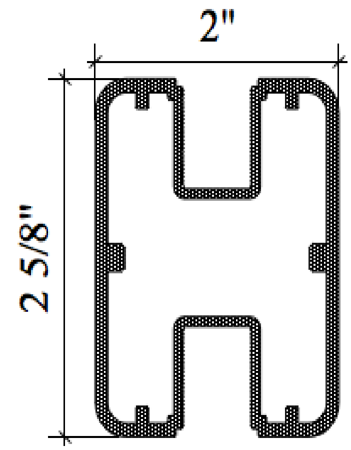
MINIMUM ASD WIND LOAD TO BE USED IS 10 PSF.
Centroid of wind load acts at 0.55h on the fence.
Typical wind load range for I = 1.0 and K_{zt} = 1.0

Table 1:	**Wind load in psf $C_f = 1.3$**			**Wind load in psf $C_f = 2.60$**		
Wind Speed	B	C	D	B	C	D
V	$0.00169V^2$	$0.00205V^2$	$0.00249V^2$	$0.00337V^2$	$0.00409V^2$	$0.00495V^2$
85	12.2	14.8	17.9	24.3	29.5	35.8
90	13.7	16.6	20.2	27.3	33.1	40.1
100	16.9	20.5	24.9	33.7	36.9	49.5
110	20.5	24.8	30.1	40.7	49.5	59.9
120	24.3	29.6	35.8	48.5	58.9	71.3
130	28.6	34.7	42.0	56.9	69.1	83.7
140	33.1	40.2	48.8	66.0	80.1	97.1

Where fence ends without a return the wind forces will be 1.667 times greater than above.
When I = 0.87 is applicable (occupancy category I) multiply above loads by 0.87.
For wind loads based on ASCE 7-10 wind speeds, figures 26.5-1A, B and C, multiply the wind loads by 0.6 to convert to Allowable Stress Design loads.
For example - Exp B with $C_f = 1.3$; 7-05 wind speed = 85 mph w= 12.2 psf:
7-10 wind speed= 110mph w = 0.6*20.5 = 12.3 psf (ASD wind loads used herein)

POST OPTIONS:

2" X 2–5/8" Rectangular Post (SP--)

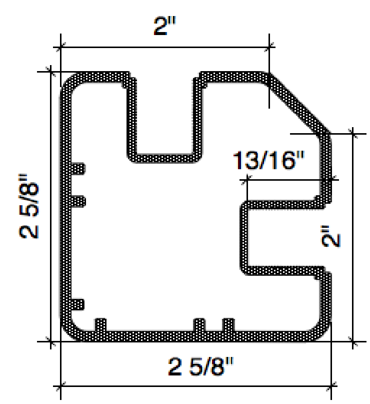


Area: 1.135 sq in
I_{xx}: 0.855 in^4 I_{yy}: 0.611 in^4
r_{xx}: 0.868 in r_{yy}: 0.734 in and J = 1.292 in^4
C_{xx}: 1.3125 in C_{yy}: 1.00 in
S_{xx}: 0.657 in^3 S_{yy}: 0.611 in^3
Z_{xx}: 0.874 in^3 Z_{yy}: 0.762 in^3
Allowable stress in aluminum post in accordance with ADM Design Aid Table 2-19:
6061-T6 Extruded aluminum
Check local buckling of flat edge with intermediate stiffener:
b/t = 2.098"/0.1" = 20.98
F_c/Ω = 27.3-0.291*20.98 = 21.2ksi = F_y

Check to 4 decimal places:
Note that F_y = 35ksi is only two significant figures
F_y/Ω = 35ksi/1.65 = 21.21ksi
F_c/Ω = 27.3-0.291*20.98 = 21.19ksi
% Difference = (21.21ksi-21.19ksi)/21.21ksi*100% = 0.094%
Since the estimated failure compressive stress is well within engineering precision of the yield stress, it can be assumed the shape will form a plastic hinge.
$M_a = F_y/\Omega*Z_{yy}$ = 21.2ksi*0.762in^3 = 16,100"#

Post Variations:

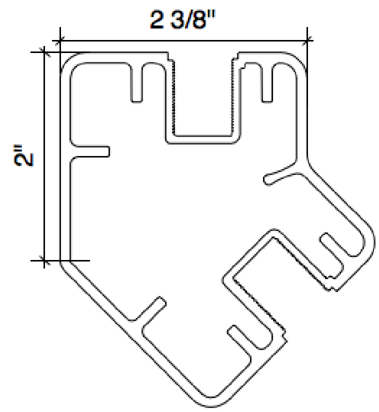
90˚ Corner Post (SP9--)



Area: 1.200 sq in
Primary bending axes are 45˚ to the faces of the post:
I_x: 1.067in^4 I_y: 0.935in^4
S_x: 0.614in^3 S_y: 0.570in^3
Z_x: 0.987in^3 Z_y: 0.881in^3

Allowable stress in aluminum post in accordance with the 2015 ADM Design Aid Table 2-19:
Check long sides for local buckling:
b/t = 2.098"/0.1" = 20.98
F_c/Ω = 27.3-0.291*20.98 = 21.2ksi = F_y
Note 1.5S<Z so design for 1.5S
$M_{a,x}$=1.5*0.614in^3*21.2ksi = 19,500"#
$M_{a,y}$=1.5*0.570in^3*21.2ksi = 18,100"#

Worst case is loading on panels both being inwards or both being outwards since this causes their loading to be additive in the weak axis. If loading from both sides is balanced, then loading in the strong axis will be zero and loading in the weak axis will be sin(45˚) times the loading from both sides. Note that the weak axis bending strength is greater than the bending strength of the intermediate post and the loading is sin(45˚) times the tributary width. Since the strength is greater and at the same spacing the loading is smaller, it can be assumed in most cases that the corner post does not control allowable post spacing.



135˚ Corner Post (SP5--)
I_x: 1.659in^4 I_y: 1.146in^4
S_x: 0.906in^3 S_y: 0.753in^3
Z_x: 1.415in^3 Z_y: 1.131in^3

Allowable stress in aluminum post in accordance with the 2015 ADM Design Aid Table 2-19:
Check long sides for local buckling:
b/t = 1.756"/0.1" = 17.56 < 20.8 Local buckling does not control

F_c/Ω = 21.2ksi
Note 1.5S<Z so design for 1.5S
$M_{a,x}$=1.5*0.906in^3*21.2ksi = 28,800"#
$M_{a,y}$=1.5*0.753in^3*21.2ksi = 23,900"#

Loading is balanced then loading in the X axis is zero and loading in the weak axis is cos(22.5˚) times the loading from each side. It can be seen that like the 90˚ post, the strength is much higher than the intermediate post and the loading is lower for the same tributary width. Therefore, it can be assumed in most cases that the corner posts do not control the allowable post spacing.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

The standard straight post will typically govern the wind screen design. Using the equations derived on page 2 determine the allowable wind loads based on the post strength (post directly core mounted in grout or other method that will develop the fill post strength.)

Solving for S
M = 16,100”# = 1,340’#
S = 1,340’#/(0.55*W*H^2) = 2,440’#/(W*H^2)
Example determine required post spacing for 20 psf wind load and 4’-0” screen height:
S = 2,440’#/(20*4^2) = 7’-7”

Solving for H
H = $[2{,}440’\#/(W*S)]^{1/2}$
Example determine maximum screen height for 20 psf wind load and 6’-0” post spacing:
H = $[2{,}440’\#/(20*6)]^{1/2}$ = 4’ - 6”

Allowable wind load:
W = 2,440’#/(S*H^2)
Example determine maximum wind load for 4’ screen height and 6’-0” post spacing:
W = 2,440’#/(6*4^2) = 25.4 psf

Table 2: 2" Post (SP)	Post strength (ft-#)=		1340			
Wind load ASD	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	90.2	67.7	60.2	54.1	49.2	45.1
3.5	66.3	49.7	44.2	39.8	36.2	33.1
4	50.8	38.1	33.8	30.5	27.7	25.4
4.5	40.1	30.1	26.7	24.1	21.9	20.1
5	32.5	24.4	21.7	19.5	17.7	16.2
5.5	26.8	20.1	17.9	16.1	14.6	13.4
6	22.6	16.9	15.0	13.5	12.3	11.3

Based on post strength, assumes anchorage method will develop the full post strength.

Maximum spacing when fall protection is required:
S = M_a/2,100”#/ft
S = 16,100”#/2,100”#/ft = 7.67’ = 7’ 8”

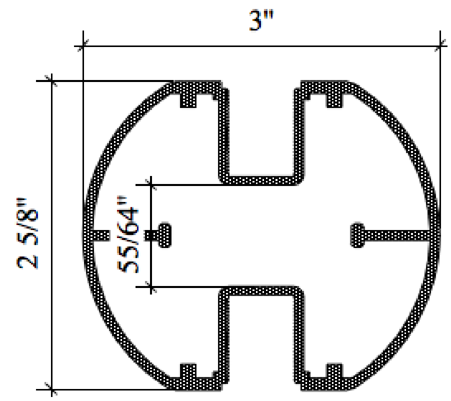
Post Deflections at maximum allowable wind load (ASD):
Δ = $UH^4/(8EI)$ = $M*H^2/(0.55*8EI)$ = $(1{,}340*12)*H^2/(0.55*8*10{,}100{,}000*0.611)$
Δ = $0.0005922*H^2$

Ht (in)	36	42	48	54	60	66	72
Defl’n	0.7675	1.0446	1.3644	1.7269	2.1319	2.5796	3.0700

3" Round Post (RP--)

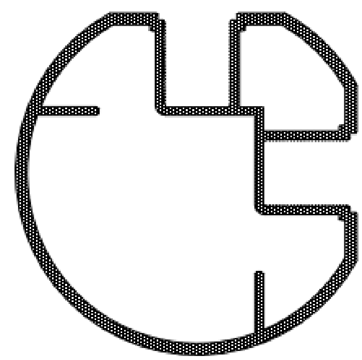
Area: 1.145 sq in

I_{xx}: 0.829 in^4	I_{yy}: 0.964 in^4
r_{xx}: 0.851 in	r_{yy}: 0.9178 in and J = 1.567 in^4
C_{xx}: 1.3125 in	C_{yy}: 1.5 in
S_{xx}: 0.6316 in^3	S_{yy}: 0.6427 in^3
Z_{xx} = 0.846 in^3	Z_{yy}: 0.921 in^3



Allowable stress in aluminum post in accordance with the 2015 ADM Design Table 2-19:
R_b/t = 1.42"/0.079" = 18.0 < 27.6 not subject to local buckling
F_c/Ω = 21.2ksi
Bending is primarily about the Y-axis.
M_a = 0.921in^3*21.2ksi = 19,500"#

90˚ Corner Post (R--CR)



Area: 1.218in^2
Primary bending axes are 45˚ to the faces of the post:

I_x: 1.106in^4	I_y: 1.033in^4
S_x: 0.714in^3	S_y: 0.663in^3
Z_x: 1.009in^3	Z_y: 0.953in^3

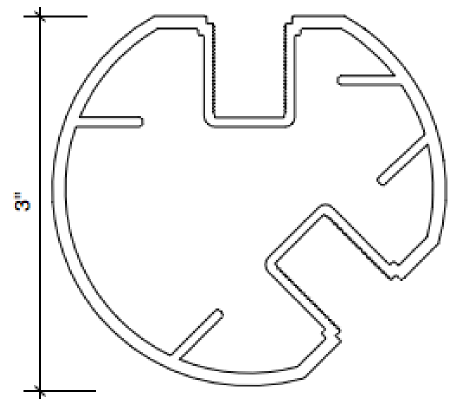
Allowable stress in aluminum post in accordance with the 2015 ADM Design Aid Table 2-19:
R_b/t = 1.42"/0.079" = 18.0 < 27.6 not subject to local buckling
F_c/Ω = 21.2ksi

Note for X-axis 1.5S<Z so design for 1.5S
$M_{a,x}$=1.5*0.714in^3*21.2ksi = 22,700"#
$M_{a,y}$=0.953in^3*21.2ksi = 20,200"#

For balanced loading, loading in the X-axis is zero and bending in the Y-axis is sin(45˚) times the loading from each direction. Since the loading to the corner post is less than the loading to an intermediate post with the same spacing and the strength of the corner post is higher, it can be assumed in most cases that the corner post does not limit post spacing.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

135˚ Corner Post (R--5)
Area: 1.313in^2
Primary bending axes are 45˚ to the faces of the post:
I_x: 1.309in^4 I_y: 1.109in^4
S_x: 0.813in^3 S_y: 0.706in^3
Z_x: 1.125in^3 Z_y: 1.081in^3



Allowable stress in aluminum post in accordance with the 2015 ADM Design Aid Table 2-19:
R_b/t = 1.42"/0.079" = 18.0 < 27.6 not subject to local buckling
F_c/Ω = 21.2ksi

Note for Y-axis 1.5S<Z so design for 1.5S
$M_{a,x}$=1.125in^3*21.2ksi = 23,800"#
$M_{a,y}$=1.5*0.706in^3*21.2ksi = 22,400"#

For balanced loading, loading in the Y-axis is zero and bending in the Z-axis is cos(22.5˚) times the loading from each direction. Since the loading to the corner post is less than the loading to an intermediate post with the same spacing and the strength of the corner post is higher, it can be assumed in most cases that the corner post does not limit post spacing.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

The standard straight post will typically govern the wind screen design. Using the equations derived on page 2 determine the allowable wind loads based on the post strength (post directly core mounted in grout or other method that will develop the fill post strength.)

Solving for S
M_a = 19,500"# = 1,625'#
S = 1,625'#/(0.55*W*H^2) = 2,955'#/(W*H^2)
Example determine required post spacing for 20 psf wind load and 4'-0" screen height:
S = 2,955'#/(20*4^2) = 9'-2"

Solving for H
H = [1,850.2#/(W*S)]$^{1/2}$
Example determine maximum screen height for 20 psf wind load and 6'-0" post spacing:
H = [2,955'#/(20*6)]$^{1/2}$ = 4'-11"

Allowable wind load (ASD):
W = 2,955'#/(S*H^2)
Example determine maximum wind load for 4' screen height and 6'-0" post spacing:
W = 2,955'#/(6*4^2) = 30.8 psf

Table 3: Allowable wind loads (psf) on 3" Round Post

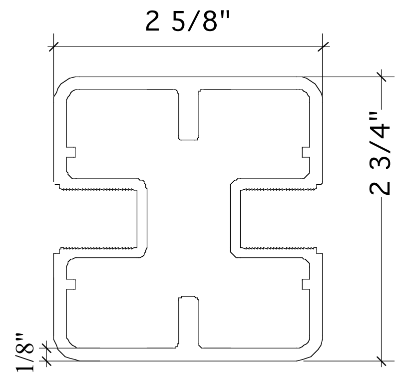
3" Post (RP)	Post strength (ft-#)=		1625			
Wind load ASD	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	109.4	82.1	73.0	65.7	59.7	54.7
3.5	80.4	60.3	53.6	48.2	43.9	40.2
4	61.6	46.2	41.0	36.9	33.6	30.8
4.5	48.6	36.5	32.4	29.2	26.5	24.3
5	39.4	29.5	26.3	23.6	21.5	19.7
5.5	32.6	24.4	21.7	19.5	17.8	16.3
6	27.4	20.5	18.2	16.4	14.9	13.7

Based on post strength, assumes anchorage method will develop the full post strength.

Maximum spacing when fall protection is required:
S = M_a/2,100"#/ft
S = 19,500"#/2,100"#/ft = 9.286' = 9' - 3"
Post Deflections at maximum allowable wind load (ASD):
Δ = UH^4/(8EI) = M*H^2/(0.55*8EI) = (1,625*12)*H^2/(0.55*8*10,100,000*0.964)
Δ = 0.0004552*H^2

Ht (in)	**36**	**42**	**48**	**54**	**60**	**66**	**72**
Defl'n	0.5899	0.8030	1.0488	1.3274	1.6387	1.9829	2.3598

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*



2-3/4” X 2–5/8” Post (BP)
Barrier System Post
Post size is 2-5/8”x2-3/4” with nonstructural cladding
Area: 1.74 sq in
I_{xx}: 1.634 in^4 I_{yy}: 1.336 in^4
r_{xx}: 0.969 in r_{yy}: 0.877 in
J: 1.048 in^4 C_w: 0.655in^6
C_{xx}: 1.375 in C_{yy}: 1.3125 in
S_{xx}: 1.188 in^3 S_{yy}: 1.018 in^3
Z_{xx}: 1.506in^3 Z_{yy}: 1.313in^3

Allowable stress in aluminum post in accordance with ADM Design Aid Table 2-19:
6061-T6 Extruded aluminum
Check local buckling of flat edge with intermediate stiffener:
b/t = 2.1”/0.125” = 16.8 < 20.8 (Local buckling does not control)
F_c/Ω = 21.2ksi
M_a = 1.506in^3*21.2ksi = 31,900”# = 2,660’#

Using the equations derived on page 2 determine the allowable wind loads based on the post strength (post directly core mounted in grout or other method that will develop the fill post strength.)

Solving for S
S = 2,660’#/(0.55*W*H^2) = 4,830’#/(W*H^2)
Example determine required post spacing for 30 psf wind load and 5’-0” screen height:
S = 4,830’#/(30*5^2) = 6’ 5”

Solving for H
H = [4,830’#/(W*S)]$^{1/2}$
Example determine maximum screen height for 30 psf wind load and 5’-0” post spacing:
H = [4,830’#/(30*5)]$^{1/2}$ = 5’-8”

Allowable wind load (ASD):
W = 4,830’#/(S*H^2)
Example determine maximum wind load for 4’ screen height and 6’-0” post spacing:
W = 4,830’#/(5*5^2) = 38.6 psf

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

Table 4: Allowable wind loads (psf) on 2-5/8" Barrier Post

2-5/8" post (BP)	Post strength (ft-#)=		2,660			
Wind load ASD	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	179.1	134.3	119.4	107.5	97.7	89.6
3.5	131.6	98.7	87.7	79.0	71.8	65.8
4	100.8	75.6	67.2	60.5	55.0	50.4
4.5	79.6	59.7	53.1	47.8	43.4	39.8
5	64.5	48.4	43.0	38.7	35.2	32.2
5.5	53.3	40.0	35.5	32.0	29.1	26.6
6	44.8	33.6	29.9	26.9	24.4	22.4

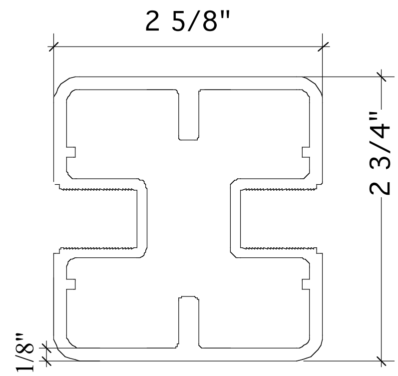
Based on post strength, assumes anchorage method will develop the full post strength.

Post Deflections at maximum allowable wind load (ASD):
$\Delta = UH^4/(8EI) = M*H^2/(0.55*8EI) = (2,660*12)*H^2/(0.55*8*10,100,000*1.634)$
$\Delta = 0.00044*H^2$

Ht (in)	36	42	48	54	60	66	72
Defl'n	0.57	0.78	1.01	1.28	1.58	1.92	2.28

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

2-3/4” X 2–5/8” Post (HD)
Heavy Duty Post System
Post size is 2-5/8”x2-3/4”.
The post extrusion is the same as the barrier post system shown earlier in this report.



Area: 1.74 sq in
I_{xx}: 1.634 in^4 I_{yy}: 1.336 in^4
r_{xx}: 0.969 in r_{yy}: 0.877 in
J: 1.048 in^4 C_w: 0.655in^6
C_{xx}: 1.375 in C_{yy}: 1.3125 in
S_{xx}: 1.188 in^3 S_{yy}: 1.018 in^3
Z_{xx}: 1.506in^3 Z_{yy}: 1.313in^3

Allowable stress in aluminum post in accordance with ADM Design Aid Table 2-19:
6061-T6 Extruded aluminum
Check local buckling of flat edge with intermediate stiffener:
b/t = 2.1”/0.125” = 16.8 < 20.8 (Local buckling does not control)
F_c/Ω = 21.2ksi
M_a = 1.506in^3*21.2ksi = 31,900”# = 2,660’#

Using the equations derived on page 2 determine the allowable wind loads based on the post strength (post directly core mounted in grout or other method that will develop the fill post strength.)

Solving for S
S = 2,660’#/(0.55*W*H^2) = 4,830’#/(W*H^2)
Example determine required post spacing for 30 psf wind load and 5’-0” screen height:
S = 4,830’#/(30*5^2) = 6’ 5”

Solving for H
H = [4,830’#/(W*S)]$^{1/2}$
Example determine maximum screen height for 30 psf wind load and 5’-0” post spacing:
H = [4,830’#/(30*5)]$^{1/2}$ = 5’-8”

Allowable wind load (ASD):
W = 4,830’#/(S*H^2)
Example determine maximum wind load for 4’ screen height and 6’-0” post spacing:
W = 4,830’#/(5*5^2) = 38.6 psf

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

Table 5: Allowable wind loads (psf) on 2-5/8” HD Post

2-5/8" post (HD)	Post strength (ft-#)=		2,660			
Wind load ASD	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	179.1	134.3	119.4	107.5	97.7	89.6
3.5	131.6	98.7	87.7	79.0	71.8	65.8
4	100.8	75.6	67.2	60.5	55.0	50.4
4.5	79.6	59.7	53.1	47.8	43.4	39.8
5	64.5	48.4	43.0	38.7	35.2	32.2
5.5	53.3	40.0	35.5	32.0	29.1	26.6
6	44.8	33.6	29.9	26.9	24.4	22.4

Based on post strength, assumes anchorage method will develop the full post strength.

Post Deflections at maximum allowable wind load (ASD):

$\Delta = UH^4/(8EI) = M*H^2/(0.55*8EI) = (2{,}660*12)*H^2/(0.55*8*10{,}100{,}000*1.634)$

$\Delta = 0.00044*H^2$

Ht (in)	36	42	48	54	60	66	72
Defl’n	0.57	0.78	1.01	1.28	1.58	1.92	2.28

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

LIVE LOADS

When post is installed so that the base is lower than the finished floor height determine allowable post height - measured from base of post or bottom of cantilevered portion to point of live load application typically 42” above finish floor.

2" post (SP)	Post strength (ft-#)=		1340			
	Post Spacing					
	3	4	4.5	5	5.5	6
LL Post HT (in)	80.4	80.4	71.5	64.3	58.5	53.6

3" post (RP)	Post strength (ft-#)=		1620			
	Post Spacing					
	3	4	4.5	5	5.5	6
LL Post HT (in)	97.2	97.2	86.4	77.8	70.7	64.8

2-5/8" post (BP)	Post strength (ft-#)=		2660			
	Post Spacing					
	3	4	4.5	5	5.5	6
LL Post HT (in)	159.6	159.6	141.9	127.7	116.1	106.4

Linear interpolation is allowable between values shown.

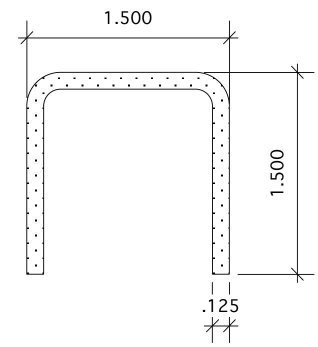
When post spacing is under 4’ the 200# concentrated load governs. For IRC compliant installations the post height may be determined based on the 4’ spacing column regardless of actual spacing. For IBC compliant installations the 50 plf distributed load governs for post spacing over 4’.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

1/8" (11 Gauge) Stanchion

Stanchions are break formed from HR Steel A1011 or A572 Grade 50 alloy steel ($F_y \geq 50$ ksi) powder coated, or 304 Stainless steel, ASTM A666 1/8 hard ($F_y \geq 50$ ksi).

t = 0.125"
$A = 0.511$ in^2
$I_{xx} = 0.117$ in^4
$I_{yy} = 0.187$ in^4
$C_{xx} = 0.932$"
$C_{yy} = 0.750$"
$Z_{xx} = 0.126$ in^3
$Z_{yy} = 0.250$ in^3



The stanchions are installed so that primary bending axis is YY with essentially no bending in the XX direction.
Stanchion strength: t/b = 1.25/0.125 = 10 < 20
Compression buckling of the flange is prevented because of confinement in grout and in the post above the grout therefore stanchion will develop the full plastic section:
$øM_{nyy} = øF_yZ_{yy}$
$ø = 0.9$
Determine the service moment on the stanchions based on a typical load factor of 1.6 (live or wind loads).
$M_s = øM_n/1.6$

$øM_{nyy}$ = 0.9*50 ksi*0.250 = 11,250"# = 937.5'#
M_s = 11,257"#/1.6 = 7,036"# = 586.3'#
S = 586.3'#/(0.55*W*H^2)

Table 6: Allowable wind loads (psf) on 11 gauge PST4 Stanchion (Any post)

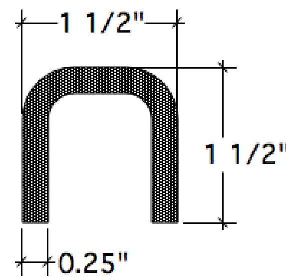
PST4 stanchion	Stanch. strength (ft-#)=		586.3			
Wind load	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	39.5	29.6	26.3	23.7	21.5	19.7
3.5	29.0	21.8	19.3	17.4	15.8	14.5
4	22.2	16.7	14.8	13.3	12.1	11.1
4.5	17.5	13.2	11.7	10.5	9.6	NA
5	14.2	10.7	9.5	NA	NA	NA
5.5	11.7	NA	NA	NA	NA	NA
6	9.9	NA	NA	NA	NA	NA

Based on stanchion strength, assumes anchorage method will develop the full stanchion strength.
NA = Not Allowed.
Maximum spacing when fall protection is required:
S = M_a/2,100"#/ft
S = 7,847"#/2,100"#/ft = 3.737' = 3' -8 7/8" Requires handrail able to span 7'-6" for fall protection.

1/4” Stanchion

Stanchion is made from A635 or A1011 Grade 50 (0.23” thick) steel sheet or 304 stainless steel (1/8 hard) with $F_y \geq 50$ ksi.

$I_{xx} = 0.189$ in^4 $I_{yy} = 0.278$ in^4
$S_{xx} = 0.221$ in^3 $S_{yy} = 0.371$ in^3
$Z_{xx} = 0.291$ in^3 $Z_{yy} = 0.489$ in^3
r = 0.550 in J = 0.070 in^4



t/b = 1.25/.25 = 5 < 20 therefore local buckling won’t control and full yield strength will be developed in stanchion.

Stanchion bending strength for bending about YY (typical post installation)
M_n = 0.489 x 50ksi = 24,450”#
Determine allowable load for wind or live loads.
$M_a = M_n/1.67$
M_a = 24,450”#/1.67 = 14,641”# = 1,220.1’#
S = 1,220.1’#/(0.55*W*H^2)

Table 8: Allowable wind loads (psf) on 1/4” Stanchion (with any post)

PST8 stanchion	Stanch. strength (ft-#)=		1220.1			
Wind load	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	82.2	61.6	54.8	49.3	44.8	41.1
3.5	60.4	45.3	40.2	36.2	32.9	30.2
4	46.2	34.7	30.8	27.7	25.2	23.1
4.5	36.5	27.4	24.3	21.9	19.9	18.3
5	29.6	22.2	19.7	17.7	16.1	14.8
5.5	24.4	18.3	16.3	14.7	13.3	12.2
6	20.5	15.4	13.7	12.3	11.2	10.3

Based on stanchion strength, assumes anchorage method will develop the full stanchion strength.
NA = Not allowed
Maximum spacing when fall protection is required:
S = M_a/2,100”#/ft
S = 14,641”#/2,100”#/ft = 6.972’ = 6’-11 2/3”

ALTERNATIVE STANCHIONS:

Custom stanchions may be produced using higher strength steels and alternative configurations to provide greater strength. For a stanchion made using steel with a different yield strength the allowable loads shall be adjusted by multiplying the tabulated value by

$F_{yhs}/50$ where: (note- Use of high strength steels will result in high deflections)
F_{yhs} = Yield strength of steel used based on mill certification or testing.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

PST10 Stanchion

Double 2-1/4”x-1/8” 304 Stainless Flat Bars
F_y = 30ksi minimum and F_u = 75ksi minimum
Z = 2*0.125”*2.25”²/4 = 0.316in³
Location where stanchion receives bending independently is only the small length that is between the top of the core mount and the bottom of the post. Therefore, the stanchion may be designed to its rupture strength.
M_a = 0.316in³*75ksi/2 = 11,800”# < 31,900”# (Does not develop post strength)

Table 9: Allowable wind loads (psf) on 1/4” Stanchion (with any post)

PST8 stanchion	Stanch. strength (ft-#)=		983			
Wind load	Post Spacing					
Screen Height	3	4	4.5	5	5.5	6
3	66.2	49.6	44.1	39.7	36.1	33.1
3.5	48.6	36.5	32.4	29.2	26.5	24.3
4	37.2	27.9	24.8	22.3	20.3	18.6
4.5	29.4	22.1	19.6	17.7	16.0	14.7
5	23.8	17.9	15.9	14.3	13.0	11.9
5.5	19.7	14.8	13.1	11.8	10.7	NA
6	16.5	12.4	11.0	NA	NA	NA

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

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 minimum Modulus of Rupture for the glass F_r is 24,000 psi. Safety Factor of 4.0 is applicable to the glass when subject to human impact. For wind loads ASTM E1300-12a allows edge stress of 10,600 psi for wind loads but recommend limiting to 9,600 psi because of unsupported edge and relatively high deflections.

Allowable glass bending stress: 24,000/4 = 6,000 psi. – Tension stress calculated.

Bending strength of glass for the given thickness:

$$I = \frac{12\text{”}*(t)^3}{12} = (t_{ave})^3 \text{ in}^3\text{/ft}$$

For deflection (I) use t_{ave}.

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

For bending (S) use t_{min}.

For 1/4” glass, t_{min} = 0.219”, t_{ave} = 0.2315”

$S = 2*(0.219)^2 = 0.0959$ in^3/ft

M_{aL} = 6,000psi*0.0959 in^3/ft = 575.5”#/ft = 47.96’#/ft Live load moment

M_{aW} = 9,600psi*0.0959 in^3/ft = 920.6”#/ft = 76.72’#/ft Wind load moment

$I = 0.2315^3 = 0.0124$ in^4

For 5/16” glass, t_{min} = 0.292”, t_{ave} = 0.312”

$S = 2*(0.292)^2 = 0.1705$ in^3/ft

M_{aL} = 6,000psi*0.1705 in^3/ft = 1,023.2”#/ft = 85.26’#/ft Live load moment

M_{aW} = 9,600psi*0.1705 in^3/ft = 1,636.8”#/ft = 136.4’#/ft Wind load moment

$I = 0.312^3 = 0.0304$ in^4

For 3/8” glass, t_{min} = 0.355”, t_{ave} = 0.3805” (or 9/16” laminated glass with PVB conservatively)

$S = 2*(0.355)^2 = 0.252$ in^3/ft

M_{aL} = 6,000psi*0.252 in^3/ft = 1,512.3”#/ft = 126.0’#/ft Live load moment

M_{aW} = 9,600psi*0.252 in^3/ft = 2,419.2”#/ft = 201.6’#/ft Wind load moment

$I = 0.3805^3 = 0.0551$ in^4

For 1/2” glass, t_{min} = 0.469”, t_{ave} = 0.50”

$S = 2*(0.469)^2 = 0.44$ in^3/ft

M_{aL} = 6,000psi*0.44 in^3/ft = 2,640”#/ft = 220’#/ft Live load moment

M_{aW} = 9,600psi*0.44 in^3/ft = 4,224”#/ft = 352’#/ft Wind load moment

$I = 0.5^3 = 0.125$ in^4

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

For 7/16" Laminated Glass
Find effective glass thickness according to ASTM E1300 appendix X-11
Assume 0.06" PVB interlayer with shear modulus of 140psi. This assumes elevated temperatures appropriate for most hot climates.

Laminated Glass Effective Thickness

h1	h2	hv	E	g
0.18	0.18	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.24	0.12	0.12	0.005184	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
39	0.283136279	0.308207981	0.343622342	0.343622342

Variable	Description
H1 & H2	Glass pane thicknesses
Hv	Interlayer thickness
E	Young's Modulus
g	Shear Modulus
Hs	$.5(h1+h2)+hv$
Hs;1	$hsh1/(h1+h2)$
Hs;1	$hsh2/(h1+h2)$
Is	$h1(hs;2)^2+h2(hs;1)^2$
a	Minimum Pane Width
Γ	$1/(1+9.6(Eishv/(G(ahs)^2))$
hef;w	$\sqrt[3]{((h1)^3+(h2)^3+12\Gamma Is)}$
h1;ef;σ	$\sqrt{((hef;w)^3/(h1+2\Gamma hs;2))}$
h2;ef;σ	$\sqrt{((hef;w)^3/(h2+2\Gamma hs;1))}$

Assume two equal plies of 3/16" tempered glass.
Effective glass thickness for deflection = 0.308"
Effective glass thickness for stress = 0.344"

$S = 2*(0.344)^2 = 0.237$ in^3/ft
$M_{aL} = 6{,}000$psi*0.237 in^3/ft = 1,420"#/ft = 119'#/ft Live load moment
$M_{aW} = 9{,}600$psi*0.237 in^3/ft = 2,270"#/ft = 190'#/ft Wind load moment
$I = 0.308^3 = 0.0292$ in^4/ft

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

GLASS IN SIMPLE SPANS

For panels simply supported on two opposite sides the moment and deflection are calculated from basic beam theory (applicable when glass is installed without structural top and bottom rails and is supported in posts only):

$M_w = W*L^2/8$ for uniform load W and span L or

$M_p = P*L/4$ for concentrated load P and span L, highest moment P @ center

Moment at edge of glass: $M_e = PL/(4*H^{0.8})$

$\Delta_{max}= (5/384)*wl^4/(EI) = (5/374)*(w/12)l^4/(10{,}400{,}000t^3) = (wl^4)/(9.34x10^9*t^3)$

l = glass span in inches

When glass is designed for a safety factor of 4.0 or greater the deflection will not govern the allowable loading

$W = M_{a}W*8/L^2 = (9{,}600*16*t^2/12)/L^2 = 12{,}800\ t^2/L^2$

Table 10: Allowable wind load (psf) for post spacing based on glass strength

	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	68.2	50.1	38.4	30.3	24.6	20.3	17.1
5/16"	121.3	89.1	68.2	53.9	43.7	36.1	30.3
3/8"	179.2	131.7	100.8	79.7	64.5	53.3	44.8
1/2"	312.8	229.8	176.0	139.0	112.6	93.1	78.2
7/16" Lam	168.3	123.6	94.7	74.8	60.6	50.1	42.1

Table 11: Check maximum glass span (post spacing, feet) for 200# concentrated load:

	Glass height (feet)						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	2.310	2.613	2.908	3.195	3.476	3.751	4.022
5/16"	4.107	4.645	5.169	5.680	6.179	6.669	7.150
3/8"	6.069	6.865	7.639	8.394	9.132	9.856	10.566
1/2"	10.693	12.096	13.460	14.789	16.090	17.365	18.617
7/16" Lam	5.732	6.484	7.215	7.928	8.625	9.308	9.979

Note: 50# concentrated load will not typically be the limiting factor for glass light size but may be taken as 4 times the span for the 200# concentrated load.

Table 12: Check maximum glass span (post spacing, feet) for 50 plf live load:

	Glass height (feet)						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	4.299	4.572	4.823	5.056	5.273	5.478	5.672
5/16"	5.732	6.096	6.431	6.741	7.031	7.304	7.563
3/8"	6.968	7.411	7.818	8.195	8.547	8.880	9.194
1/2"	9.249	9.837	10.377	10.877	11.346	11.786	12.204
7/16" Lam	6.771	7.202	7.597	7.964	8.307	8.629	8.935

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

GLASS GLAZED IN STRUCTURAL BOTTOM SHOE AND POSTS (3 SIDES)
When glass is supported at posts and along the bottom the glass stresses are determined from flat plate theory where (**loads must also be checked for load share to posts**):

$M = 1/8*wH^2/[1+2(S/2H)^3]$ or solving for w:

$w = 8M_a[1+2(S/2H)^3]/b^2 = 8*2*t^2*9{,}600/12[1+2(S/2H)^3]/S^2 = 12{,}800*t^2[1+2(S/2H)^3]/S^2$

Table 13: Allowable wind load (psf) (ASD) for post spacing based on glass strength

3'6" Height	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	78.9	62.6	52.7	46.4	42.5	40.0	38.5
5/16"	140.4	111.4	93.7	82.5	75.5	71.1	68.5
3/8"	207.5	164.6	138.4	122.0	111.6	105.1	101.2
1/2"	362.1	287.3	241.6	212.9	194.7	183.4	176.7
7/16” Lam	194.8	154.6	130.0	114.5	104.7	98.6	95.1

4'0" Height	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	75.4	58.5	48.0	41.1	36.5	33.5	31.4
5/16"	134.1	104.0	85.3	73.1	65.0	59.5	55.9
3/8"	198.1	153.7	126.0	108.0	96.0	88.0	82.6
1/2"	345.8	268.3	220.0	188.5	167.6	153.6	144.2
7/16” Lam	186.1	144.4	118.3	101.4	90.2	82.6	77.6

4'6" Height	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	73.3	56.0	45.1	37.9	33.0	29.6	27.2
5/16"	130.2	99.6	80.2	67.4	58.6	52.5	48.3
3/8"	192.5	147.2	118.5	99.6	86.7	77.7	71.4
1/2"	336.0	256.9	206.9	173.8	151.2	135.6	124.6
7/16” Lam	180.8	138.2	111.3	93.5	81.4	72.9	67.0

5'0" Height	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	71.9	54.4	43.3	35.8	30.7	27.0	24.4
5/16"	127.8	96.7	76.9	63.7	54.6	48.1	43.4
3/8"	188.9	143.0	113.7	94.2	80.7	71.1	64.2
1/2"	329.7	249.5	198.5	164.4	140.8	124.0	112.0
7/16” Lam	177.4	134.3	106.8	88.4	75.7	66.7	60.3

6'0" Height	Post spacing, feet						
Glass thickness	3	3.5	4	4.5	5	5.5	6
1/4"	70.3	52.6	41.2	33.5	28.1	24.2	21.3
5/16"	125.1	93.5	73.3	59.6	50.0	43.0	37.9
3/8"	184.8	138.2	108.3	88.1	73.9	63.6	56.0
1/2"	322.6	241.2	189.0	153.7	128.9	111.0	97.8
7/16" Lam	173.6	129.8	101.7	82.7	69.4	59.7	52.6

Load to Posts with glass supported continuously along the bottom:
Bottom of glass is rigidly anchored by a continuous heavy base shoe such as the CRL B5S that is adequately anchored to develop the full imposed glass moment.
The load share to the posts is proportioned so that the glass deflection and the post deflection are the same at the ends of the glass lights. Thus the load share is proportional to the relative deflection of the post and glass.
For post: $k_p = EI_p$
E = modulus of elasticity for post material and I_p = moment of inertia for post; may have to be adjusted for custom stanchions or solid grouting of post.

For Glass: $k_g = EI_g$
E = modulus of elasticity = 10,400 ksi and $I_p = St_{ave}^3$ where S is glass length in feet
$R_p = (EI_p)/(EI_p+EI_g)$ Assumes post is loaded by glass on both sides
<u>**Table 14**</u>
Load share to post:
For the **standard AWS 2" post (SP)**:
$R_p = (10.1*0.611)/(10.1*0.611+10.4*S*t^3)$

Post load share	Post spacing, feet						
Ave glass t (in)	3.00	3.50	4.00	4.50	5.00	5.50	6.00
0.25	0.93	0.92	0.90	0.89	0.88	0.87	0.86
0.31	0.87	0.85	0.83	0.81	0.80	0.78	0.76
0.38	0.79	0.76	0.74	0.71	0.69	0.67	0.65
0.50	0.61	0.58	0.54	0.51	0.49	0.46	0.44
7/16" Lam	0.87	0.85	0.84	0.82	0.80	0.79	0.77

3" Round post (RP):
$R_p = (10.1*0.964)/(10.1*0.964+10.4*S*t^3)$

Post load share	Post spacing, feet						
Ave glass t (in)	3.00	3.50	4.00	4.50	5.00	5.50	6.00
0.25	0.95	0.94	0.94	0.93	0.92	0.92	0.91
0.31	0.91	0.90	0.88	0.87	0.86	0.85	0.84
0.38	0.86	0.84	0.82	0.80	0.78	0.76	0.75
0.50	0.71	0.68	0.65	0.62	0.60	0.58	0.56
7/16" Lam	0.91	0.90	0.89	0.88	0.87	0.85	0.84

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

2-5/8" Barrier post (BP) or HD Post:

$R_p = (10.1*1.188)/(10.1*1.188+10.4*S*t^3)$

Post load share	Post spacing, feet						
Ave glass t (in)	3.00	3.50	4.00	4.50	5.00	5.50	6.00
0.25	0.96	0.95	0.95	0.94	0.94	0.93	0.92
0.31	0.93	0.92	0.90	0.89	0.88	0.87	0.86
0.38	0.88	0.86	0.85	0.83	0.81	0.80	0.78
0.50	0.75	0.73	0.70	0.67	0.65	0.63	0.61
7/16" Lam	0.93	0.92	0.91	0.90	0.89	0.88	0.87

Post strength will typically control allowable load on wind screen even with continuous bas shoe.

Load to posts $= W*R_p$

This value shall be compared with the allowable wind loads in Tables 2 through 6 as appropriate.

Load to glass base shoe $= W*(1-R_p)$

This wind load shall be used to design the base shoe and anchorage (design not included in this report).

To determine moment in post:

$M = (W*R_p)*S*H^2*0.55$

W = design wind load

R_p = load share from table 11

S = glass length, post spacing

H = wind screen height

This must be checked against the allowable post moment and allowable moment for anchorage method.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

CONCRETE CORE MOUNTS

Core mount depth and edge distance requirements will vary by the loading and strength of concrete. The wind load tables below may be used to find an acceptable combination of depth and edge distance for a desired loading.

Check concrete failure modes:
Stanchions loaded with moment will resist moment by creating couple moment bearing reactions on the concrete. Assume stanchions experience plastic deformation and the bearing reactions are uniform.
Distance between centroids of bearing reactions = d/2 (d = embedment depth)
Bearing reaction depth = d/2
Allowable bearing stress = $0.65*0.85f'_c/1.6 = 0.3453f'_c$
Stanchion width = 1.5”
Allowable bearing load , $P_a = 1.5"*d/2*0.3453f'_c = 0.2590df'_c$
Allowable moment, $M_a = (0.2590df'_c)*d/2 = 0.1295*d^2*f'_c$
Bearing stress check limits moment strength with respect to embedment depth and concrete strength.

Check edge breakout:
Edge breakout is calculated as the concrete two way shear strength.
v_c is calculated according to ACI318-14 Table 22.6.5.2
Note that the perimeter of the breakout is offset $0.5c_1$ from the stanchion edges where c_1 is the edge distance to the face of the stanchion.
The width of the breakout section, $w = 1.5"+c_1$
The height of the breakout section, $h = d/2+0.5C_1$
The total perimeter of the breakout, $b_0= w+2h$

Once v_c is determined the allowable bearing load is calculated as,$P_a= 0.75*v_c*c_1*b_0/1.6$
And $M_a = P_a*d/2$

Sample calculations are shown below for several different embedment depth, edge distance and concrete strength combinations. The calculations first calculate the allowable load against shear breakout failure then the allowable load against compression failure. Lastly, the lesser of the two failure modes is multiplied by d/2 to find the allowable moment.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

Concrete Strength	Edge Distance	Embedment						
f'c (psi)	c1 (in)	d (in)	β	αs	w (in)	h (in)	bo (in)	4λ√f'c (psi)
3000	3	4.75	1.08064516	30	4.1875	3.875	16.125	219.089023
3000	3	5.5	0.98529412	30	4.1875	4.25	16.875	219.089023
3000	3	6.5	0.88157895	30	4.1875	4.75	17.875	219.089023
3000	5	4.75	1.26923077	30	6.1875	4.875	22.125	219.089023
3000	5	5.5	1.17857143	30	6.1875	5.25	22.875	219.089023
3000	5	6.5	1.07608696	30	6.1875	5.75	23.875	219.089023
5000	3	4.75	1.08064516	30	4.1875	3.875	16.125	282.842712
5000	3	5.5	0.98529412	30	4.1875	4.25	16.875	282.842712
5000	3	6.5	0.88157895	30	4.1875	4.75	17.875	282.842712
5000	5	4.75	1.26923077	30	6.1875	4.875	22.125	282.842712
5000	5	5.5	1.17857143	30	6.1875	5.25	22.875	282.842712
5000	5	6.5	1.07608696	30	6.1875	5.75	23.875	282.842712

(2+4/β)λ√f'c (psi)	(2+αsc1/bo)λ√f'c (psi)	vcdbo (psi)	Pa (lbs)	fa (psi)	Pa (lbs)	Ma (in-lbs)
312.2836074	415.250125	10598.4315	4968.01476	1035.9375	3690.52734	8765
331.9035199	401.6632088	11091.3818	5199.08521	1035.9375	4273.24219	11751
358.0634033	385.3209041	11748.6489	5507.17915	1035.9375	5050.19531	16413
282.1601054	480.8818386	24236.7232	11360.964	1035.9375	3690.52734	8765
295.438228	468.7068443	25058.307	11746.0814	1035.9375	4273.24219	11751
313.1423915	453.6633958	26153.7521	12259.5713	1035.9375	5050.19531	16413
403.1564036	536.0856062	13682.5162	6413.67948	1726.5625	6150.87891	14608
428.4856017	518.5449729	14318.9123	6711.99015	1726.5625	7122.07031	18458
462.2578659	497.4471482	15167.4405	7109.73771	1726.5625	8416.99219	23107
364.2671297	620.8157842	31289.4751	14666.9414	1726.5625	6150.87891	14608
381.4091123	605.0979341	32350.1352	15164.1259	1726.5625	7122.07031	19586
404.265089	585.6769256	33764.3488	15827.0385	1726.5625	8416.99219	27355

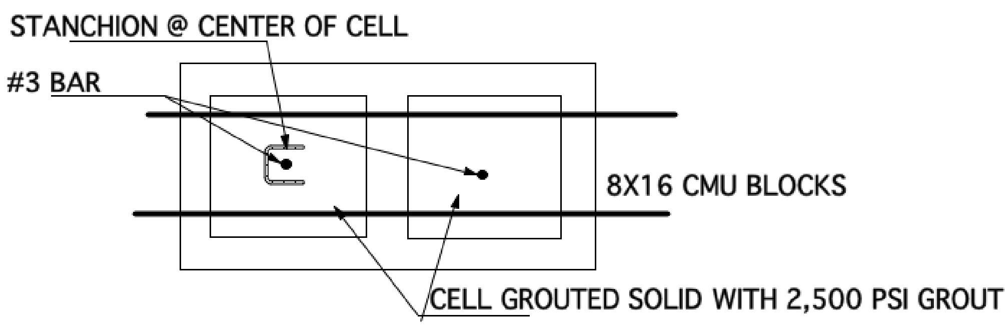
The results of the calculations above are summarized in the table below. A concrete strength, embedment depth and edge distance combination should be selected that develops the strength of the stanchion already selected for the required wind loading.

Table 15 Concrete Failure Allowable Moments

Concrete Strength (psi)	Edge Distance (in)	Embedment Depth (in)	Allowable Moment (in-lbs)	Max Stanchion Developed
3,000.00	3.00	4.75	8,760.00	10 ga
3,000.00	3.00	5.50	11,000.00	10 ga
3,000.00	3.00	6.50	14,000.00	10 ga
3,000.00	5.00	4.75	8,760.00	10 ga
3,000.00	5.00	5.50	11,700.00	10 ga
3,000.00	5.00	6.50	16,400.00	1/4”
5,000.00	3.00	4.75	11,500.00	10 ga
5,000.00	3.00	5.50	14,200.00	10 ga
5,000.00	3.00	6.50	18,100.00	1/4”
5,000.00	5.00	4.75	14,600.00	1/4”
5,000.00	5.00	5.50	19,600.00	1/4”
5,000.00	5.00	6.50	29,500.00	1/4”

CONCRETE MASONRY UNIT CONSTRUCTION (CMU)

When stanchions or posts are embedded into the grouted cells of CMU:
The CMU wall shall be designed for the imposed moments from the posts.
The stanchion shall be embedded a minimum of 15” into the CMU unless engineered for less.
The minimum wall thickness shall be 8” nominal.
A bond beam with (2) #3 bar or larger shall be constructed along the top course or as other engineering requires to accommodate the AWS loading. The reinforcement bar shall pass between the stanchion/post and each face of the wall.

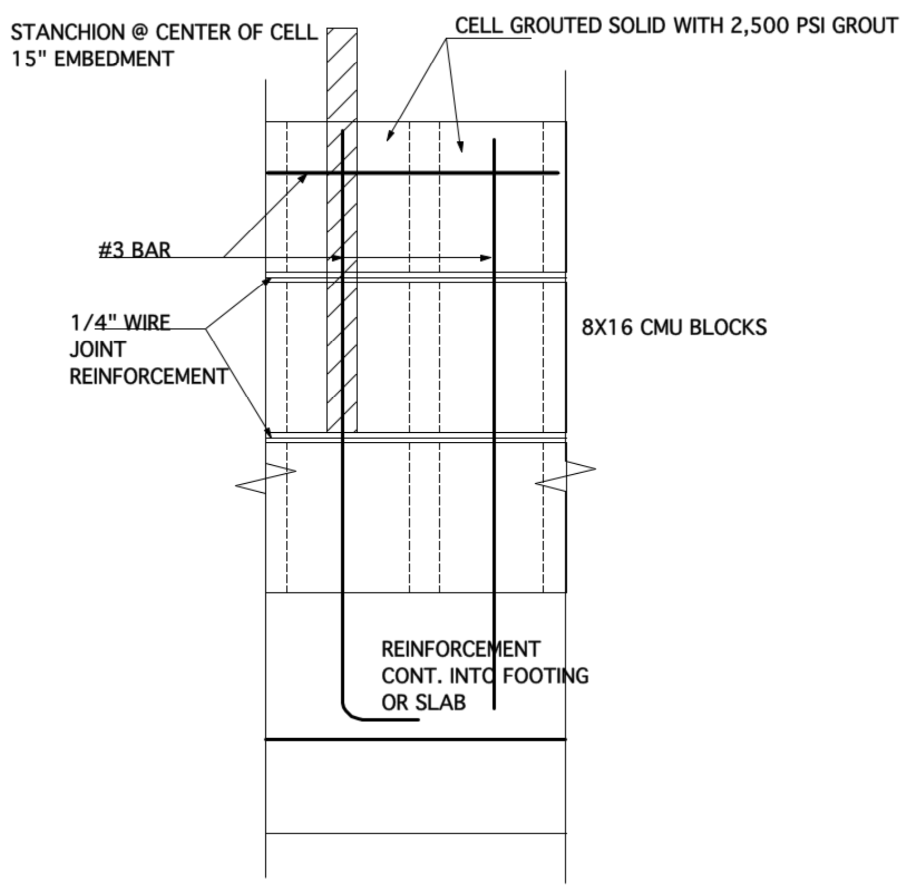


Additional reinforcement may be required depending on project requirements and specific AWS configuration. Maximum allowable moment for this detail is 9,600”# per post.

Other CMU wall configurations shall be engineered to support the imposed loads from the AWS posts.

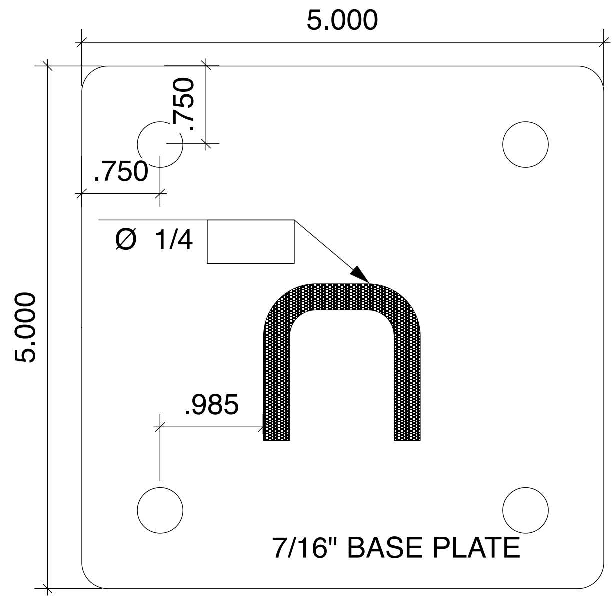
Surface mounted base plate installation shall be engineered for the specific application.

Face mounted post installations shall be engineered for the specific application.



Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

BASEPLATE MOUNTED STANCHION
Stanchion is welded to base plate be inserting stanchion through a water-jet cut hole in the base plate and then fully welded from the bottom and ground flat.



Check base plate strength:
5”x5”x7/16” A36 steel or stainless steel
$Z = 6\text{”}*0.4375\text{”}^2/4 = 0.287\text{in}^3$
Maximum allowable bolt tension load
$øM_n = 0.9*Z*F_y = 0.9*0.287*36 = 9{,}302\text{#”}$
$øT_n = øM_n/(2*a)$
a = 0.985”
$øT_n = 9{,}302/(2*0.985) = 4{,}722\text{#}$
$T_s = 4{,}722/1.6 = 2{,}951\text{#}$

Maximum allowable moment on base plate:
$M_a = 2T_s*(5\text{”}-0.75\text{”})$
$M_a = 2*2{,}951\text{#}*4.25\text{”} = 25{,}084\text{”#}$
Base plate strength is adequate to develop the full stanchion plastic moment.

Higher strength stanchions will require custom base plate size and anchorage which must be designed in conjunction with the custom stanchion.

BASE PLATE ANCHORAGE TO STEEL
3/8” A307 or ASTM F593 Group 1 or 2 Condition CW stainless steel bolts into 1/4” tapped steel or with nuts.
Tensile area of 3/8” threaded rod (UNC) = 0.0775 in^2
Rod strength $øP_n$= (0.75*60ksi) * 0.0775 in^2 = 3,488#
Check thread strength into standoff – minimum thread embed = 1/4”
Internal thread stripping area = 0.828 in^2 for 3/8 – 16 threads
Strength of threads $øP_n = 0.65*0.58*A_{sn}*t*F_{tu}$ = 0.65*0.58*0.828*(1/4)*75ksi = 4,541#
Shear strength $øV_n$= 0.65*0.5*60 ksi*0.0775 in^2 = 1,511#

Develops full stanchion strength.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

BASE PLATE MOUNTED TO CONCRETE - Expansion Bolt Alternative:
Base plate mounted to concrete with Hilti Kwikbolt 3/8”x3.75” concrete anchors with 2.75” effective embedment. Anchor strength based on ESR-1917
Minimum conditions used for the calculations:
$f'_c \geq 3{,}000$ psi uncracked concrete
edge distance = 2.75” spacing = 3.5” (Assume 4-1/2” from center of post to edge of concrete)
h = 2.75”: effective embed depth (3.0625”” nominal)
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*2.75*2+3.5)*(1.5*2.75+2.25) = 74.91 in^2 2 anchors
A_{Nco}= 9*2.75”^{2} = 68.06 in^2
$C_{a,cmin}$ = 2.5” (ESR-1917 Table 3)
C_{ac} = 4.125” (ESR-1917 Table 3)
$\varphi_{ed,N}$ = 0.7+0.3*2.75”/(1.5*2.75”) = 0.9
$\varphi_{c,N}$ = (use 1.0 in calculations with k = 24)
$\varphi_{cp,N}$= max (2.75”/4.125 or 1.5*2.75”/4.125) = 1
N_b = 24*1.0*$\sqrt{3000}$*2.75$^{1.5}$ = 5,995#
N_{cb} = 74.91/68.06*0.9*1.0*1.0*5,995 = 5,940#

Concrete pullout strength per ESR 1917 Table 3, $N_{p,uncr}$=4,110# each
$N_{p,uncr}$=2*4,110# = 8,220# > 5,940# (Pullout strength does not control for tension strength of anchor pair)

Determine allowable tension load on anchor pair
T_s = 0.65*5,940#/1.6 = 2,410#

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*2.75”*2+3.5”)*(2.75”*1.5) = 48.47in^2
A_{vco}= 4.5(c_{a1})^2 = 4.5(2.75”)^2 = 34.03in^2
$\varphi_{ed,V}$= 1.0 (affected by only one edge)
$\varphi_{c,V}$= 1.4 uncracked concrete
$\varphi_{h,V}$ > 1 (Depends on slab thickness, assume minimum value of 1)
$V_b= [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(2.75/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(2.75)^{1.5}$ =1,590#
V_{cb} = 48.47/34.03*1.0*1.4*1.0*1,590# = 3,170#
øV_{cb}/LF = 0.7*3,170#/1.6 = 1,380#
Steel shear strength = 0.65*3,595#*2/1.6 = 2,920#
Concrete breakout controls shear strength, V_a = 1,380# (will not control anchor design)

M_a = 2,410#*(4.25”-0.5*2,410/(0.85*3000*4.75”)) = 10,000”# ≤ 14,641”# doesn’t develop full strength of stanchion.

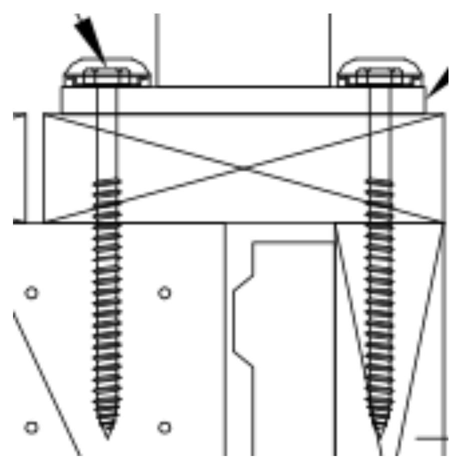
Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

ATTACHMENT TO WOOD:

Check required embedment for 3/8" lag screws:
From National Design Specification for Wood Construction Table 11.2A
For full PTS8 stanchion strength:
T = 14,641/3.75" = 3,905#
adjusted for wood bearing pressure:
a = 3,905/(1.25*625psi*5) = 1.00"
T' = 14,641/(4.25-1.0/2) = 3,905#
G ≥ 0.49 (pressure treated Doug-Fir, Southern Pine, LVL, or denser wood)
W = 296#/" (MC ≤ 19%)
C_D = 1.6 for wind loads (NDS Table 2.3.2)
C_M = 0.7 (NDS Table 10.3.3) where moisture content of wood may exceed 19%.
W' = 296#/"*1.6 = 477#/" (MC ≤ 19%)
W' = 296#/"*1.6*0.7 = 332#/" (MC ≥ 19%)
Allowable tension load on 3/8" lag screw stainless steel
$T_a = AF_{yt}$ = 0.0775in^2*75 ksi/3 = 1,938#

Required embedment depth e:
e = 1,938/332#/" = 5.837"
Req. lag screw length = 5.837" + 0.5 + 7/32" = 6.556" Use 7" lag screws for weather exposed installation.
e = 1,938/477#/" = 4.062"
Req. lag screw length = 4.062" + 0.5 + 7/32" = 4.78" Use 5" lag screws for protected installation.

When there is decking between the base plate and the solid limber backing the lag screw length shall be increased by the decking thickness.
Recommend using 3/8" x (7"+t_d) stainless steel lag screws, (4) per base plate.
Where t_d = material thickness between the base plate and the solid lumber backing.
Minimum blocking under base plate is 6x8 nominal.
Attachment of blocking to joists:
Required number of screws for 1/4" x 3" screws:
Z' = 159#*1.6*0.7 = 178#
From ∑M about center of block:
solving for N:
N = 2,810/(178) = 16 screws each end



Typical installation will use 3/8"x6" SS lag screws:
A typical installation is a 3/4" decking above solid blocking or beam that is protected from moisture,
p = 6"-3/4"-7/32" = 5.03"
W'p = 477pli*5.03" = 2,400# > 1,940# greater than assumed minimum steel strength

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

Allowable bearing stress on panel product is 360psi
M_a = 2*1,940#*(4.375”-2*1,940#/(5”*360psi)/2) = 12,800”# for typical installation

For through bolts:
Use 3/8” bolts to minimum 4x solid blocking with 2” square plate washers on the backside under the nut. Blocking shall be adequately secured.

Wood framing shall have adequate strength to carry the imposed loads from the posts.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*

FASCIA MOUNTED POSTS
Posts may be fascia mounted to steel, concrete, CMU or wood using a minimum of two 3/8" anchors designed for the imposed loads and moments as calculated.

ALTERNATIVES
Alternative anchors may be designed based on the post moments as calculated.

DESIGN STEPS:
1) Determine wind load using ASCE SEI 7-05 or 7-10 for project conditions as illustrated in table 1.
Wind loads are to be ASD level wind loads, adjust strength level by $W_{strength}$*0.6 =W_{ASD}
2) Select Post Spacing using Tables 2 – 4 for directly embedded posts or Tables 5 or 6 for stanchion mounted posts.
3) Select glass thickness from Tables 7 – 9 (No bottom rail) or Table 10 (with base shoe).
4) Select anchors based on substrate.

DESIGN EXAMPLE:
5' Tall AWS windscreen surface mounted to wood deck, 100 mph exposure D wind load.
1) From Table 1 wind load = 24.9 psf (ASD)

2) Since posts are surface mounted to wood use base plate mounted stanchion. Can use any post – select 2" post.
From Table 5 Cannot use 10 gauge stanchion because allowable wind load is too low.
From Table 6, 1/4" stanchion, post spacing is between 3' and 4' on center, interpolating :
S = 3'+(30.6-24.9)/(30.6-22.9) = 3.7' or from the equation for the post spacing:
S = 1,146.1'#/(W*0.55*H^2) = 1,146.1/(24.9*0.55*5^2) = 3.347' Use 3'4" as maximum post spacing.

3) Glass will not use a structural bottom rail therefore select glass thickness from Tables 7- 9 for a maximum height of 5' and spacing of 3'4":
From Table 7, 1/4" glass, S= 3.5', W = 50.1 psf and S = 3', W = 68.2 so interpolating at S = 3'4", W > 24.9 psf : 50.1+(2"/6")(68.2-50.1) = 56.1 psf
Select 1/4" glass.

4) Attached to wood framing use 3/8" x 6" lag screws (page 22).

LIMITATIONS
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. THIS REPORT IS NOT INTENDED TO CERTIFY THE AWS FOR A SPECIFIC INSTALLATION.

Edward C. Robison, PE
10012 Creviston DR NW *253-858-0855*
Gig Harbor, WA 98329 *FAX 253-858-0856*