04 December 2024

Report Holder:
Frameless Hardware Company
2323 Firestone Blvd.
South Gate, CA 90280

SUBJ: FRAMELESS HARDWARE COMPANY
AR- ALUMINUM RAILING
STAINLESS STEEL CABLE INFILL

The AR utilizes aluminum extrusions and various infills to construct building guards and rails for decks, balconies, stairs, fences and similar locations. The system is intended for interior and exterior weather exposed applications and is suitable for use in most natural environments. The system may be used for residential, commercial and industrial applications as detailed herein. The railing system can be used in level and sloped applications such as stairs and ramps. The system is an engineered system designed for the following criteria:

The design loading conditions are:

On Top Rail:

Concentrated load = 200 lbs any direction, any location
Uniform load = 50 plf, any perpendicular to rail

For installations compliant with the IRC only the 200# top rail load is applicable.

On In-fill Panels:

Concentrated load = 50# on one sf.
Distributed load = 25 psf on area of in-fill, including spaces

Wind load is insignificant compared to live loading for most picket infill guard applications. A design professional should determine if wind loading is significant for a specific application.

Refer to IBC Section 1607.9.1 for loading.

The Aluminum Guard Rail System is engineered to the following codes and standards:
2024 California Building and Residential Codes
2021 Washington Building and Residential Codes
2021 and 2024 International Building and Residential Codes
ASCE 7-16
2020 Aluminum Design Manual
Anchorage calculations are also engineered to the following codes:
2018 American Wood Council NDS
ACI 318-19
The information herein is intended to assist a qualified individual in designing a code compliant installation and may be used as a guide in performing a site specific design, It remains the responsibility of the Specifier to verify compliance with local building codes for the project specific conditions.

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

TABLE OF CONTENTS

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

DigiSigned by
elrobison@narrows.com
Date: 12/4/24
12:18:23 PM PST

Engineering Seals

Registered Professional Engineer/Structural
CERTIFICATE NO.
42123
EDWARD C.
ROBISON
Date Signed
ARIZONA U.S.A.
EXP 03/31/2026

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

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

COLORADO LICENSED
EDWARD C. ROBISON
54027
PROFESSIONAL ENGINEER

EDWARD C. ROBISON
LICENSE
NO. 63683
STATE OF
FLORIDA
PROFESSIONAL ENGINEER
EXP 02/28/2025

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

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
Signature
04/30/2026
Expiration Date of the License

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

STATE OF MARYLAND
EDWARD C. ROBISON
52500
PROFESSIONAL ENGINEER

Professional Certification. I hereby certify that these documents were prepared or approved by me, and that I am a duly licensed professional engineer under the laws of the State of Maryland,
License No. 52500, Expiration Date: 04/09/2022

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

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

EDWARD C. ROBISON
LICENSED PROFESSIONAL
ENGINEER
29000
STATE OF MISSISSIPPI

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

STATE OF MISSOURI
EDWARD
ROBISON
NUMBER
PE-2024026643
PROFESSIONAL ENGINEER

STATE OF MINNESOTA
I hereby certify that this plan, specification. or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.
Signature: Typed or printed name: Edward C. Robison
Date Lic. No. 58604

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

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

STATE OF OHIO
Edward C.
Robison
E90702
REGISTERED
PROFESSIONAL ENGINEER

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

LICENSED PROFESSIONAL ENGINEER
E. C.
ROBISON
31646
OKLAHOMA

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

COMMONWEALTH OF
REGISTERED
PROFESSIONAL
EDWARD C. ROBISON
ENGINEER
NO.73556
PENNSYLVANIA
EXP 09/30/2025

EDWARD C. ROBISON
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No.124614
STATE OF TENNESSEE

STATE OF TEXAS
EDWARD C. ROBISON
100688
LICENSED
PROFESSIONAL ENGINEER
FIRM #F-12044
EXP 12/31/2024

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

SPECIFYING FOR WIND LOADING

The tables in this section are meant to be used by specifier's and bidders to determine preliminary maximum post spacings for their project. The tables are based on low, medium and high wind environments. Low wind environments are applications where the post reactions from the live load are greater than the reactions from the wind load. Medium wind applications covers most applications in the western part of the United States, non-special wind zones, at heights of 30 feet or less above the surrounding grade. The high wind applications cover most locations in the non-hurricane areas of the United States at heights of 60 feet or less above the surrounding grade. The tables assume the wind is calculated according to ASCE 7-16 Chapter 29 as wind acting on a freestanding fence. It is also assumed that the guard terminates at a return or wall and has a maximum length of 10 times the height of the rail.

Use Table 1 to determine if your application is a low, medium or high wind environment. Table 2 shows the allowable strength of the standard post mounting options and post spacing for the live load or low wind case (wind load where live load and wind load are equivalent). Tables 3 and 4 provide allowable post spacing for medium and high wind applications (wind controls). Tables 5A, 5B and 5C gives allowable wind pressure for various combinations of glass thickness, height and post spacing. Linear interpolation is permitted on these tables.

For installations more than 60 feet above adjacent grade, or on steep bluffs, cliffs or mountains, or in special wind zones a competent professional must determine the applicable wind loads on the guards. Tables 9 - 11 provide the strength and allowable wind loads for the glass options.

The wind speeds shown in the following table are basic wind speeds (ultimate) 3 second gust speed at 33 feet above the ground in Exposure C as specified in ASCE 7.

The exposure categories are as defined in ASCE 7:
Exposure B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of singe family dwellings or larger.

Exposure C : Open terrain with scattered obstructions including surface undulations or other irregularities having height generally less than 30 feet extending more than 1500 feet from the building site in any quadrant. Exposure C extends into adjacent Exposure B type terrain in the downwind direction for the distance of 1500 feet or 10 times the height of the building or structure, whichever is greater. This category includes open country and grasslands, and open water exposure for less than 1 mile.

Exposure D : Flat unobstructed areas exposed to wind flowing over open water for a distance of a least 1 mile. This exposure shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 1500 feet or 10 times the height of the building or structure, whichever is greater."

Table 1

<table>
<tr><th rowspan="2">Wind Environment For Specifier's Tables</th><th colspan="3">Project Wind Criteria</th></tr>
<tr><th>Exposure B</th><th>Exposure C</th><th>Exposure D</th></tr>
<tr><td rowspan="4">Low (live load controls, also use when picket or cable infill is specified. See additional cable information on the following page.)</td><td colspan="3">Guard is 30ft or less above grade AND</td></tr>
<tr><td>For 36” tall guard: The design wind speed is 120mph or less and the Exposure is B

For 42” tall guard: The design wind speed is 115mph or less and the Exposure is B</td><td>For 36” tall guard:
The design wind speed is 105mph or less and the Exposure is C

For 42” tall guard: The design wind speed is 95mph or less and the Exposure is C</td><td>For 36” tall guard:
The design wind speed is 95mph or less and the Exposure is D

For 42” tall guard: The design wind speed is 90mph or less and the Exposure is D</td></tr>
<tr><td colspan="3">Guard is 60ft or less above grade AND</td></tr>
<tr><td>For 36” tall guard: The design wind speed is 110mph or less and the Exposure is B

For 42” tall guard: The design wind speed is 105mph or less and the Exposure is B</td><td>For 36” tall guard:
The design wind speed is 95mph or less and the Exposure is C

For 42” tall guard: The design wind speed is 90mph or less and the Exposure is C</td><td>For 36” tall guard:
The design wind speed is 90mph or less and the Exposure is D</td></tr>
<tr><td rowspan="4">Medium</td><td colspan="3">Guard is 30ft or less above grade AND</td></tr>
<tr><td>The design wind speed is 140mph or less and the Exposure is B</td><td>The design wind speed is 115mph or less and the Exposure is C</td><td>The design wind speed is 110mph or less and the Exposure is D</td></tr>
<tr><td colspan="3">Guard is 60ft or less above grade AND</td></tr>
<tr><td>The design wind speed is 120mph or less and the Exposure is B</td><td>The design wind speed is 110mph or less and the Exposure is C</td><td>The design wind speed is 100mph or less and the Exposure is D</td></tr>
<tr><td rowspan="4">High</td><td colspan="3">Guard is 30ft or less above grade AND</td></tr>
<tr><td>The design wind speed is 150mph or less and the Exposure is B</td><td>The design wind speed is 120mph or less and the Exposure is C</td><td>The design wind speed is 115mph or less and the Exposure is D</td></tr>
<tr><td colspan="3">Guard is 60ft or less above grade AND</td></tr>
<tr><td>The design wind speed is 130mph or less and the Exposure is B</td><td>The design wind speed is 115mph or less and the Exposure is C</td><td>The design wind speed is 110mph or less and the Exposure is D</td></tr>
</table>

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

Table 2

Live Load and Low Wind Application		
Post Height, H (in)	**Equivalent Wind Load in Pressure (psf)**	
36	*30.9*	
Post Option	**M_a (in-lbs)**	**Allowable Post Spacing, S_a (in)**
Custom Anchorage Detail	19500	72
Baseplate Mounted With Commercial Length Fasteners	10500	70
Core Mounted	10900	72
Baseplate Mounted With Residential Length Fasteners	7200	48

Live Load and Low Wind Application		
Post Height, H (in)	**Equivalent Wind Load in Pressure (psf)**	
42	*26.4*	
Post Option	**M_a (in-lbs)**	**Allowable Post Spacing, S_a (in)**
Custom Anchorage Detail	19500	72
Baseplate Mounted With Commercial Length Fasteners	10500	60
Core Mounted	10900	62
Baseplate Mounted With Residential Length Fasteners	7200	41

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

Table 3

Medium Wind Application				
Equivalent Wind Load in Pressure (psf)		**Post Height, H (in)**		
	38.2	*36*	*42*	*48*
Anchor Detail Option	**M_a (in-lbs)**	**Allowable Post Spacing, S_a (in)**		
Custom Anchorage Detail	19500	72	72	59
Baseplate Mounted With Commercial Length Fasteners	10500	57	42	32
Core Mounted	10900	59	43	33
Baseplate Mounted With Residential Length Fasteners	7200	39	28	22

Table 4

High Wind Application				
Equivalent Wind Load in Pressure (psf)		**Post Height, H (in)**		
	43.1	*36*	*42*	*48*
Post Option	**M_a (in-lbs)**	**Allowable Post Spacing, S_a (in)**		
Custom Anchorage Detail	19500	72	68	52
Baseplate Mounted With Commercial Length Fasteners	10500	50	37	28
Core Mounted	10900	52	38	29
Baseplate Mounted With Residential Length Fasteners	7200	34	25	19

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

Use the tables below if the wind pressure is known for your site. Check both the infill and the anchorage table appropriate for the application.

Tables 5A, 5B and 5C

Allowable Wind Load Accounting for 1/4”monolithic or 5/16” SG Laminated Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	47	28	18
48	47	28	18
60	32	27	18
72	21	18	16

Allowable Wind Load Accounting for 3/8” Monolithic Or 7/16” SG Laminated Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	89	76	66
48	50	43	37
60	32	27	24
72	21	18	16

Allowable Wind Load Accounting for 1/2” Monolithic or 9/16” SG Laminated Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	89	76	66
48	50	43	37
60	32	27	24
72	21	18	16

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

Table 6: Standard Post Installations:

This table does not account for wind loading. Use tables 1-5 on the previous pages to check wind loading.

Allowable Post Spacings:			
	Anchorage Type	**Anchorage Fasteners**	**Allowable Post Spacing**
Residential Applications	Surface mount to wood	(4) 3/8”x4” Lag Screws	72”
	Surface mount to concrete	(4) 3/8”x3-3/4” Hilti KH-EZ (Min edge distance = 3-1/2” to nearest anchor)	72”
	Fascia bracket to wood	(4) 3/8”x4” Lag Screws	72”
	Concrete core mount	Set 4” deep in 4” square or circle core or blockout	72”
	Direct Fascia to wood	(2) 3/8” lag screws[1]	72”
	Direct Fascia to concrete	(2) 3/8”x3-3/4” Hilti KH-EZ	72”
Commercial Applications	Surface mount to wood	(4) 3/8”x4” Lag Screws	48”
	Surface mount to concrete	(4) 3/8”x3-3/4” Hilti KH-EZ (Min edge distance = 3-1/2” to nearest anchor)	48”
	Surface mount to concrete	(4) 3/8”x5” Hilti KH-EZ(Min edge distance = 3-1/2” to nearest anchor)	60”
	Fascia bracket to wood	(4) 3/8”x4” Lag Screws	48”
	Concrete core mount	Set 4” deep in 4” square or circle core or blockout	60”
	Concrete core mount	Set 4-1/2” deep in 4” square or circle core or blockout	72”
	Direct Fascia to wood	(2) 3/8” lag screws[2]	72”
	Direct Fascia to concrete	(2) 3/8”x5” Hilti KH-EZ	72”

1) *Required lengths of screws are: 4” for 4x10 min beam size that is weather protected, 5” for 6x10 min beam size that is not protected from weather or 6x8 min beam size that is protected from water. 6” for 6x8 min beam size that is not protected from weather.*
2) *Required lengths of screws are: 5” for 6x10 min beam size that is weather protected, 7” for 8x10 min beam size that is not protected from weather.*

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

Table 7: Standard Top Rail Installations

Top Rail Engineering Properties The X-axis is taken as the horizontal axis and the Y-axis is the vertical axis.					
Top Rail	**I_x (in^4)**	**I_y (in^4)**	**$M_{a,x}$ (in-lbs) (Assumes max allowable free span)**	**$M_{a,y}$ (in-lbs) (Assumes max allowable free span)**	**Allowable post spacing/ Allowable span (in)**
X1	0.383	0.355	5130	4940	72
X2	0.147	0.976	2480	3530	60
X3	0.268	0.92	3090	3280	60
X35	0.238	1.36	3250	6830	65
X4	0.062	0.9	2480	11000	48

LOAD CASES:
Dead load = 20 plf for 42” rail height or less.

Loading:
Horizontal load to top rail from in-fill:
25 psf*H/2
Post moments
M_i = 25 psf*H/2*S*H=
= (25/2)*S*H^2

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

GLASS INFILL

The infill glass is recommended to be fully tempered glass with a mean modulus of rupture of 24,000psi or above. The glass must comply with Category II of the CPSC 16 CFR Part1201 or Class A of ANSI Z97.1. The FHC ARS guard system is compatible with monolithic or laminated panels. The compatible monolithic thicknesses are 1/4", 3/8" and 1/2" . Additionally, the compatible laminated thicknesses are 5/16", 7/16" and 9/16". All laminated panels are assumed to be laminated with a SentryGlass interlayer.

Table 8 - 1/4" (5/16") laminated glass

h_1 (in) (thickness of each ply)	h_2 (in)	h_v (in) thickness of interlayer	E (psi) (Young's modulus of glass)	g (psi) (Assumed shear modulus of interlayer assumes high temperature)
0.102	0.102	0.06	10400000	1600
h_s (in)	$h_{s;1}$ (in)	$h_{s;2}$ (in)	I_s (in^4/in)	
0.162	0.0810	0.0810	0.0013	
a (in)	Γ	$h_{ef;w}$ (in)	$h_{1;ef;\sigma}$ (in)	$h_{2;ef;\sigma}$ (in)
36	0.8716	0.2526	0.2575	0.2575

The laminated glass has higher effective thickness than the monolithic glass. The monolithic minimum glass thickness of 0.219" is assumed in design.

To meet the requirements of the IBC 2407.12 the glass used in the guardrail shall be designed using a safety factor of four for the live loads. For fully tempered glass, this results in an allowable design stress of 24,000psi/4 = 6,000psi. When designing for wind loads, the allowable glass stress is according to ASTM E1300 Table X7.1. Table X7.1 gives the allowable edge stress for fully tempered glass as 10,600psi. Even when glass is used in a guard, the allowable glass stress of 10,600psi is used when designing for wind loads. This is a generally accepted interpretation of the code, including by ICC-ES glass guard acceptance criteria.

All tables and recommendations in this report are based on fully tempered glass. If heat strengthened glass is used the allowable loads are to be multiplied by 0.43. Annealed glass is strictly prohibited and must not be used with this system for any application.

Table 9A - 1/4" Monolithic Glass

Glass minimum thickness, t_{min}	Design stress for live loads, F_{live} (psi)	Design stress for wind loads, F_{wind} (psi)	Allowable moment for live loads, $M_{a,live} = F_{live}*2*t_{min}^2$ (in-lbs/ft)
0.219	6000	10600	576
Max live load moment = 50#*42"/4 (in-lbs/ft)	Allowable moment for wind loads, $M_{a,wind} = F_{wind}*2*t_{min}^2$ (in-lbs/ft)		
525	1017		
< 576"#/ft OK			

Allowable wind load = $\min(M_a*8*12/(H^2), 384*10.4*10^6*0.219^3*12/(5*60*H^3))$.

Table 9B - Allowable Wind Load 1/4" monolithic or 5/16" laminated glass

	Allowable Wind Load on Glass Infill (psf)		
	Glass Infill Height, H (in)		
	33	39	45
Any Glass Width	47	28	18

Allowable Wind Load Accounting for 1/4" Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	39	45
36	47	28	18
48	47	28	18
60	32	27	18
72	21	18	16

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

Also check for 3/8” monolithic or 7/16” laminated glass.

t_{min} = 0.355” for 3/8” nominal. Check laminated:

Table 10A - 3/8” Monolithic or 7/16” laminated glass

h_1 (in) (thickness of each ply)	h_2 (in)	h_v (in) thickness of interlayer	E (psi) (Young’s modulus of glass)	g (psi) (Assumed shear modulus of interlayer assumes high temperature)
0.18	0.18	0.06	10400000	1600
h_s (in)	$h_{s;1}$ (in)	$h_{s;2}$ (in)	I_s (in^4/in)	
0.24	0.1200	0.1200	0.0052	
a (in)	Γ	$h_{ef;w}$ (in)	$h_{1;ef;\sigma}$ (in)	$h_{2;ef;\sigma}$ (in)
36	0.7937	0.3937	0.4059	0.4059

0.355” monolithic glass controls.

Glass minimum thickness, t_{min}	Design stress for live loads, F_{live} (psi)	Design stress for wind loads, F_{wind} (psi)	Allowable moment for live loads, $M_{a,live} = F_{live}*2*t_{min}^2$ (in-lbs/ft)
0.355	6000	10600	1512
Max live load moment = 50#*42”/4 (in-lbs/ft)	Allowable moment for wind loads, $M_{a,wind} = F_{wind}*2*t_{min}^2$ (in-lbs/ft)		
525	2672		
< 576”#/ft OK			

Table 10B - Allowable Wind Load 3/8” monolithic or 7/16” laminated glass

	Allowable Wind Load on Glass Infill (psf)		
	Glass Infill Height, H (in)		
	33	*39*	*45*
Any Glass Width	199	120	78

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

Table 10B - Continued

Allowable Wind Load Accounting for 3/8" Monolithic Or 7/16" SG Laminated Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	89	76	66
48	50	43	37
60	32	27	24
72	21	18	16

Also check for 1/2" monolithic or 9/16" laminated glass.
t_{min} = 0.469" for " nominal. Check laminated:

Table 11A - 1/2" Monolithic or 9/16" laminated glass

h_1 (in) (thickness of each ply)	h_2 (in)	h_v (in) thickness of interlayer	E (psi) (Young's modulus of glass)	g (psi) (Assumed shear modulus of interlayer
0.219	0.219	0.06	10400000	1600
h_s (in)	$h_{s;1}$ (in)	$h_{s;2}$ (in)	I_s (in^4/in)	
0.279	0.1395	0.1395	0.0085	
a (in)	Γ	$h_{ef;w}$ (in)	$h_{1;ef;\sigma}$ (in)	$h_{2;ef;\sigma}$ (in)
36	0.7597	0.4622	0.4786	0.4786

0.469" monolithic glass controls for stress

Glass minimum thickness, t_{min}	Design stress for live loads, F_{live} (psi)	Design stress for wind loads, F_{wind} (psi)	Allowable moment for live loads, $M_{a,live} = F_{live}*2*t_{min}^2$ (in-lbs/ft)
0.469	6000	10600	2640
Max live load moment = 50#*42"/4 (in-lbs/ft)	Allowable moment for wind loads, $M_{a,wind} = F_{wind}*2*t_{min}^2$ (in-lbs/ft)		
525	4663		
< 2,640"#/ft OK			

Table 11B - Allowable Wind Load 1/2" monolithic or 9/16" laminated glass

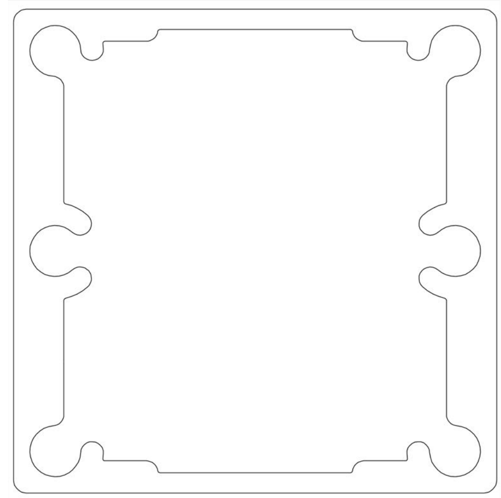
	Allowable Wind Load on Glass Infill (psf)		
	Glass Infill Height, H (in)		
	33	*39*	*45*
Any Glass Width	199	120	78

Allowable Wind Load Accounting for 1/2" Monolithic or 9/16" Laminated Glass, Top and Bottom Rails (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	89	76	66
48	50	43	37
60	32	27	24
72	21	18	16

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

POST DESIGN - 2-3/8" Square

Post flexural strength is calculated according to the 2020 Aluminum Design Manual Chapter F. Possible failure modes are local buckling, lateral torsional buckling and yielding. The aluminum alloy is 6005-T61A.



6005-T61 Aluminum. Local buckling of flange element supported on both sides.			
S (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
1.13	1.35	2.2	0.24
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
9.167	For $\lambda \le 20.8$, F_c/Ω = 21.2ksi For yielding, Strength is controlled by rupture, F/Ω = 38ksi/1.95 = 19.5ksi.		
F_c/Ω (ksi)	For $20.8 < \lambda < 33$, $F_c/\Omega = 27.3\text{-}0.291\lambda$		
19.5	For $\lambda \ge 33$, $F_c/\Omega = 580/\lambda$		
If $\lambda \le 20.8$, local buckling does not control and the strength is controlled by ZF/Ω. Note that for 6005-T61A $F_u/(k_t\Omega) < F_y/\Omega$.			
If $\lambda > 20.8$, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
26.325	26325		

The above calculations show that local buckling does not control. Lateral torsional buckling calculations are shown on the following page and will control design. Because the post is square it may be appropriate to design to the yield strength rather than the lateral torsional buckling strength. Engineering judgment should be used to determine if the specific installation is susceptible to a lateral torsional buckling failure. In this report, allowable spacing tables are based on the more conservative lateral torsional buckling.

6005-T61 Aluminum. Lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
1.13	1.35	10100000	1.42	0.029
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	1.04	25000	65.7	33750
Any loading distribution:				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.3	0	1	-1.1875	0
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	984385.1	9.4	32657.0	**19792**
36	656086.8	11.5	32411.1	**19643**
42	562329.2	12.4	32303.8	**19578**
48	492020.5	13.3	32203.9	**19518**
60	393599.8	14.8	32021.4	**19407**
72	327992.4	16.3	31856.4	**19307**

For typical 42” post height, M_a = 19,600”#

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

Post under weak axis bending:

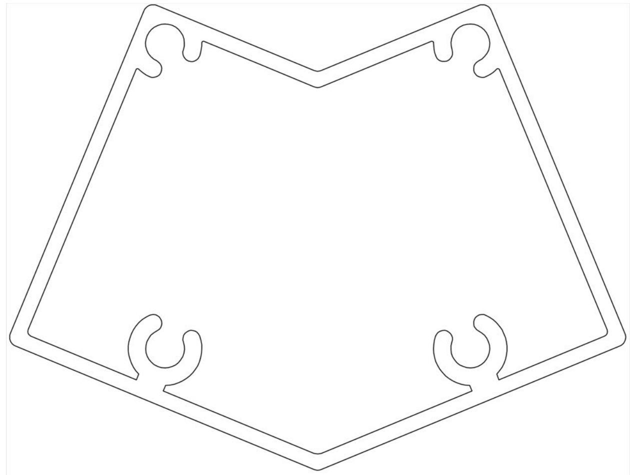
6005-T61 Aluminum. Local buckling of flange element supported on both sides.			
S (in³)	**Z (in³)**	**b (in)**	**t (in)**
0.871	1.12	0.937	0.1
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
9.37	For **λ** ≤ 20.8, F_c/Ω = 21.2ksi For yielding, Strength is controlled by rupture, F/Ω = 38ksi/1.95 = 19.5ksi.		
F_c/Ω (ksi)	For 20.8 < **λ** < 33, F_c/Ω = 27.3-0.291**λ**		
19.5	For **λ** ≥ 33, F_c/Ω = 580/**λ**		
If **λ** ≤ 20.8, local buckling does not control and the strength is controlled by ZF/Ω. Note that for 6005-T61A $F_u/(k_t\Omega) < F_y/\Omega$.			
If **λ** > 20.8, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
21.84	21840		

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

135° x 2.375" Post

First calculate moment strength about the strong axis.

$I_x = 1.90\ in^4$
$S_x = 0.956\ in^3$
$Z_x = 1.45\ in^3$
$I_y = 1.26\ in^4$
b = 1.97"
t = 0.1"
$C_w = 0.0342\ in^6$
β = -0.0157 in
$g_0 = 0$ in



6005-T61 Aluminum. Local buckling of flange element supported on both sides.			
S (in^3)	Z (in^3)	b (in)	t (in)
0.935	1.39	1.97	0.1
λ = b/t	Allowable compression stress is calculated according to ADM 2020 Design Table 2-21		
19.7	For λ ≤ 20.8, $F_c/Ω$ = 21.2ksi For yielding, Strength is controlled by rupture, F/Ω = 38ksi/1.95 = 19.5ksi.		
$F_c/Ω$ (ksi)	For 20.8 < λ < 33, $F_c/Ω$ = 27.3-0.291λ		
19.5	For λ ≥ 33, $F_c/Ω$ = 580/λ		
If λ ≤ 20.8, local buckling does not control and the strength is controlled by ZF/Ω. Note that for 6005-T61A $F_u/(k_tΩ) < F_y/Ω$.			
If λ > 20.8, local buckling controls and the strength is calculated as $F_c/Ω$*S.			
M_a(in-kips)	M_a(in-lbs)		
27.105	27105		

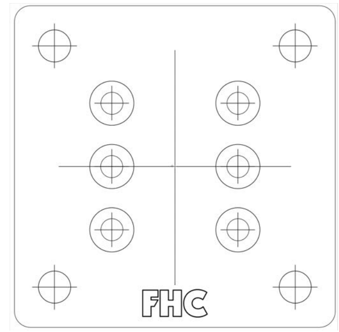
As this post has greater strength than the standard post it will not govern post spacing.

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

6005-T61 Aluminum. Lateral torsional buckling:				
S_x (in³)	**Z_x (in³)**	**E (psi)**	**J (in⁴)**	**C_w (in⁶)**
0.956	1.45	10100000	2.0	0.039
β_x (in)	**I_y (in⁴)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	1.26	25000	65.7	36250
Any loading distribution:				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.3	0	1	-1.5	0
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	1285865	7.550	34621	**20983**
36	857031	9.248	34255	**20761**
42	734560	9.990	34095	**20664**
48	642718	10.680	33946	**20573**
60	514154	11.940	33674	**20409**
72	428452	13.080	33428	**20260**

CONNECTION TO BASEPLATE

5/16"x2" Type F 410 SS fasteners with MagniCoat finish. The MagniCoat finish provides galvanic separation between the fastener and aluminum.



Tested Strength with four screws (lbs)	Tested strength with six screws (lbs)	Safety factor, Ω	Load height above baseplate (in)
731	861	2.5	40
Allowable moment load on connection with four screws (lbs)	Allowable moment load on connection with six screws (lbs)		
11696	13776		

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

BASEPLATE MOUNTED TO WOOD – SINGLE FAMILY RESIDENCE

For 200# top load and 36" post height: M = 200#*36" = 7,200"#

$T_{200} = \dfrac{7{,}200}{2*4.36"} = 826\#$

Assume Hem-fir, G = 0.43
Adjustment for wood bearing:
Bearing Area Factor:
C_b = (5"+0.375)/5" = 1.075
a = 2*826#/(1.075*625psi*5")= 0.492"
T = 7,200/[2*(4.36-0.49/2)]= 875#

Required embed depth:
Based on NDS Table 11.2A for 3/8" lag screws into Hem-fir, G = 0.43
W = 253pli
W' = WC_DC_m = 243*1.6*1.0 = 389 pli for dry conditions
W' = WC_DC_m = 243*1.6*0.7 = 272 pli for wet conditions

For protected installations the minimum embedment is:
l_e = 875#/389#/in = 2.25" : +7/32" for tip = 2.47"

For weather exposed installations the minimum embedment is:
l_e = 875#/272#/in = 3.22" : +7/32" for tip = 3.44"
FOR WEATHER EXPOSED INSTALLATIONS USE 5" LAG SCREWS AND INCREASE BLOCKING TO 4.5" MINIMUM THICKNESS.

For 42" guard height and 200# load increase lag screw embedment to:
e = 42/36*2.25"+7/32 = 2.85" for dry conditions
e = 2.85/0.7 = 4.06" for wet conditions
Lag screw length shall be as needed to achieve the required embedment into solid wood.

Alternative anchorage may be designed for the specific project conditions to include post loading, lumber species and exposure conditions.

For guards subject to the requirements of the International Residential code post spacing may be up to 6' on center.

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

BASE PLATE MOUNTED TO CONCRETE

Anchor options for baseplate surface mounts are summarized in the table below. Calculations for each option are shown on the following pages. The residential and commercial applications both assume a max top rail height of 42”. The commercial application must resist the 50 pounds per foot uniform load or the 200 pound concentrated load. The residential application only needs to resist the 200 pound concentrated load.

Anchor Option Summary

Anchor option	Allowable spacing for commercial applications (feet)	Allowable spacing for residential applications (feet)
(4) 3/8”x3-3/4” US Ultrawedge Anchors	4	6
(4) 3/8”x5” US Ultrawedge Anchors	5	6
(4) 3/8”x4” Hilti KH-EZ Anchors	4	6
(4) 3/8”x5” Hilti KH-EZ Anchors	5	6

Also verify spacing for wind loads.

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

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume Hilti KH-EZ per ESR 3027.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
3000	2.5	3.5	3.75		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
67.6875	81.5625	55.125	56.25			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0513	2247	2900
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.98	1	1	3681	5230
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	10460				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	1178	200	Pass	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail				
2125	0	Pass				
Baseplate effective width, b_e (in)	Lever arm to anchor, d (in)	$a=T_{n,min}/(0.85f'_c b_e)$ (in)	$øM_n/ALF=øT_n/ALF*(d-a/2)$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a < 1.2$	
5	4.375	0.41	8860	8400	1.118	
				Pass	<1.2 Pass	

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

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume Hilti KH-EZ per ESR 3027.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
3000	3.55	3.5	3.75		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
67.6875	127.08	55.125	113.4225			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0513	2410	3111
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.89718309859154	1	1	6228	6261
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	12521				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	1264	250	Pass	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail				
2543	0	Pass				
Baseplate effective width, b_e (in)	Lever arm to anchor, d (in)	$a=T_{n,min}/(0.85f'_cb_e)$ (in)	$øM_n/ALF=øT_n/ALF*(d-a/2)$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a < 1.2$	
5	4.375	0.49	10503	10500	1.198	
				Pass	<1.2 Pass	

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

Also check 3/8" US Ultrawedge Anchors:
Standard lengths are 3-3/4" or 5". For 3/8" and 3-3/4" long anchor baseplate, h_{nom} = 3.75"-0.5"-0.375" = 2.875". h_{ef} = 2.875"-0.375" = 2.5".

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume US Ultrawedge per ESR 3981.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
	2.5	3.5	3.75		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
67.6875	81.5625	55.125	56.25			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0513	0	0
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.98	1	1	0	0
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	0				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	0	200	Fail	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail				
0	0	Fail				
Baseplate effective width, b_e (in)	Lever arm to anchor, d (in)	$a=T_{n,min}/(0.85f'_cb_e)$ (in)	$øM_n/ALF=øT_n/ALF*(d-a/2)$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a < 1.2$	
5	4.375			8400		

For 5" long anchor, h_{ef} = 5"-0.5"-0.375"-0.375" = 3.75".

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume US Ultrawedge per ESR 3981.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
3000	3.75	3.5	3.75		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
67.6875	136.875	55.125	126.5625			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0513	2437	3145
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.88666666666666	1	1	6762	6484
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	12968				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	1278	250	Pass	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail				
2634	0	Pass				
Baseplate effective width, b_e (in)	Lever arm to anchor, d (in)	$a=T_{n,min}/(0.85f'_c b_e)$ (in)	$øM_n/ALF=øT_n/ALF*(d-a/2)$ (in-lbs)	M_{max} (in-lbs)	Combined, $M/M_a+T/T_a+V/V_a$ < 1.2	
5	4.375	0.51	10854	10500	1.163	
				Pass	<1.2 Pass	

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

Core Mounted Posts
Mounted in either 4”x4”x4” blockout, or 4” to 6” dia by 4” minimum deep cored hole.
Core mount okay for 6’ post spacing.
Assumed concrete strength 2,500 psi for existing concrete and 3,000 psi for grout
Max load – 6’•50 plf = 300#
M = 300#•42” = 12,600”#
Or M = 250#*42” = 10,500”# for 5’ max spacing
Maximum post spacing = 5’ commercial or 6’ residential (for 6’ commercial max post spacing see higher embedment option on the following page)

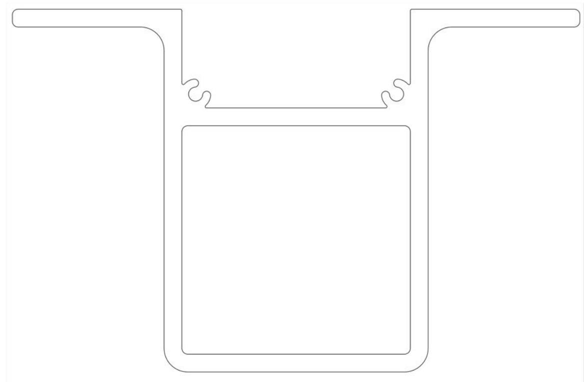
CONCRETE CORE MOUNT Failure modes are concrete crushing or shear breakout.				
Core width (in), b_c	Stanchion width (in), b_s	Grout strength (psi), f'_c	Edge distance (in), c	Embedment (in), d
4	2.375	3000	3.8	4
Edge breakout calculations				
Breakout width (in), $b_B = b_s+c$	Breakout height (in), $H = d/2+c/2$	$\beta = b_B/H$	Perimeter (in), $b_0 = b_B+2H$	α_s Three sided breakout
6.175	3.9	1.583	13.975	30
λ	$4\lambda(f'_c)^{0.5}$	$(2+4/\beta)\lambda(f'_c)^{0.5}$	$(2+\alpha_s C/b_0)\lambda(f'_c)^{0.5}$	v_c = minimum of previous three cells (psi)
1	219.09	247.917	556.35	219.09
$V_n = v_c b_0 c$ (lbs)	$V_a = øV_n/LF = 0.75V_n/1.6$ (lbs)			
11635	5454			
Concrete Crushing Calculations				
Bearing width (in), $b_b = \min(b_s+b_c/2,b_c)$	Breakout height (in), $H = \min(d/2+b_c/4,d)$	$P_n = 0.85b_bHf'_c$ (lbs)	$P_a = øP_n/LF = 0.65P_n/1.6$	
4	3	30600	12431.25	
Allowable Moment Calculation				
$M_a = \min(V_a,P_a)*d/2$ (in-lbs)	$M_{max} = 50plf*5'*42"$			
10908	10500	OK		

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

For 6' O.C. commercial option, use 4-1/2" minimum embedment

CONCRETE CORE MOUNT Failure modes are concrete crushing or shear breakout.				
Core width (in), b_c	Stanchion width (in), b_s	Grout strength (psi), f'_c	Edge distance (in), c	Embedment (in), d
4	2.375	3000	3.8	4.5
Edge breakout calculations				
Breakout width (in), $b_B = b_s+c$	Breakout height (in), $H = d/2+c/2$	$\beta = b_B/H$	Perimeter (in), $b_0 = b_B+2H$	α_s Three sided breakout
6.175	4.15	1.488	14.475	30
λ	$4\lambda(f'_c)^{0.5}$	$(2+4/\beta)\lambda(f'_c)^{0.5}$	$(2+\alpha_s C/b_0)\lambda(f'_c)^{0.5}$	v_c = minimum of previous three cells (psi)
1	219.089	256.787	540.911	219.089
$V_n = v_c b_0 c$ (lbs)	$V_a = øV_n/LF = 0.75V_n/1.6$ (lbs)			
12051	5649			
Concrete Crushing Calculations				
Bearing width (in), $b_b = \min(b_s+b_c/2, b_c)$	Breakout height (in), $H = \min(d/2+b_c/4, d)$	$P_n = 0.85 b_b H f'_c$ (lbs)	$P_a = øP_n/LF = 0.65P_n/1.6$	
4	3.25	33150	13467.1875	
Allowable Moment Calculation				
$M_a = \min(V_a,P_a)*d/2$ (in-lbs)	M_{max} = 50plf*6'*42"			
12710	12600	OK		

FASCIA MOUNTED POSTS WITH FASCIA BOOT



For Fascia boot, the post slides inside the boot and the boot is connected to the structure with (4) 3/8" lag screws or concrete anchors.			
Assume upper anchors are 4" below the walking surface max	Max post spacing for residential (ft)	Max post spacing for commercial (ft)	
	6	5	
Upper anchors are at least 5" above the bottom of the bracket	$T_{residential}$ = (36"+9")/5" *200#	$T_{commercial}$ = (42"+9")/5" *50plf*S	
	1800	2550	

For anchorage to wood: 3/8" lag screws	Recall W' for dry applications	W' for wet applications	
	389	272	
Required penetration for residential dry applications	Required penetration for residential wet applications	Required penetration for commercial dry applications	Required penetration for commercial wet applications
2.53	3.53	3.50	4.91

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

Also check 3/8" US Ultrawedge Anchors:
Standard lengths are 3-3/4" or 5". For 3/16" bracket and 3-3/4" long anchor baseplate, h_{nom} = 3.75"-0.5"-0.1875" = 3.06". h_{ef} = 3.06"-0.375" = 2.69".

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume US Ultrawedge per ESR 3981.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
3000	2.69	2.5	4.5		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
45	82.14495	28.125	65.1249			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0000	1376	2202
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.88587360594795	1	1	4108	4590
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	9181				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	895	200	Pass	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail	Combined, $M/M_a+T/T_a+V/V_a$ < 1.2			
1865	1800	Pass	1.189			
			<1.2 Pass			

OK for residential

For 5" long anchor, h_{ef} = 5"-0.5"-0.1875"-0.375" = 3.94".

Baseplate with moment anchorage. Concrete failure modes are according to ACI 318-19 Chapter 17. Post installed anchors. Assume US Ultrawedge per ESR 3981.						
f'c (psi)	hef (in)	Edge distance to nearest anchors (in)	Anchor spacing parallel with edge (in)		Concrete thickness (in)	D (in)
3500	3.94	2.5	4.5		4.75	0.375
Area calculations, assumes two anchors in tension						
A_{Vc} (in²)	A_{nc} (in²)	A_{vo} (in²)	A_{No} (in²)			
45	137.2512	28.125	139.7124			
Shear breakout	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_b	V_{cbg} (lbs)
	1	1	1	1.0000	1605	2567
Tension breakout	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b	N_{cbg} (lbs)
	1	0.82690355329949	1	1	7866	6389
Shear pryout	k_{cp}	V_{cbg} (lbs)				
	2	12779				
Also check pullout:	Pullout from cracked concrete, $N_{p,cr}$ (lbs)					
	N/A does not control					
Ø Tension	Ø Shear	Also divide by 1.6 to convert to ASD. ALF	$øV_n$/ALF (lbs)	V (lbs)	Pass/Fail	
0.65	0.65	1.6	1043	200	Pass	
$øT_n$/ALF (lbs)	T (lbs)	Pass/Fail	Combined, $M/M_a+T/T_a+V/V_a$ < 1.2			
2596	2550	Pass	1.174			
			<1.2 Pass			

OK for commercial

TOP RAILS

A top rail is required for all installations. The railing system may be used with the X1, X2, X3, X35 and X4 top rails. The maximum post spacing allowed for picket rail is 72" but the top rail may limit the allowable spacing further.

The top rail must hold the 200# or 50plf uniform live loads. For the 200# live load, maximum moment M = 200#L/4 where L is the post spacing. For the 50plf live load, maximum moment M = $(50plf/12)L^2/8$ where L is the post spacing in inches. By setting equations equal to each other, $(50plf/12)L^2/8=200L/4$, the span where 50plf live load controls over the 200# concentrated load can be solved. Solving for L gives L = 96" which is greater than the maximum post spacing of 72". Therefore, for all considered spans the 200# live load at center span will control.

Top Rail Engineering Properties

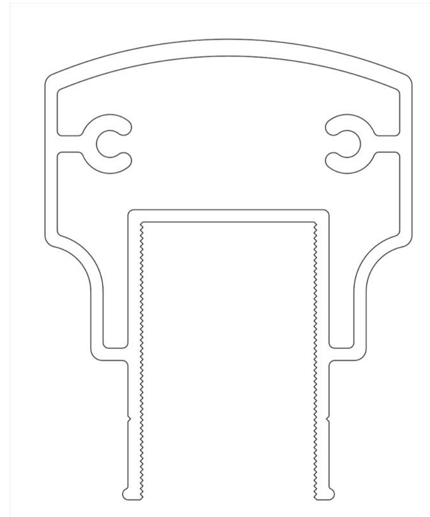
The X-axis is taken as the horizontal axis and the Y-axis is the vertical axis.

Top Rail	I_x (in^4)	I_y (in^4)	$M_{a,x}$ (in-lbs) (Assumes max allowable free span)	$M_{a,y}$ (in-lbs) (Assumes max allowable free span)	Allowable post spacing/ Allowable span (in)
X1	0.383	0.355	5130	4940	72
X2	0.147	0.976	2480	3530	60
X3	0.268	0.92	3090	3280	60
X35	0.238	1.36	3250	6830	65
X4	0.062	0.9	2480	11000	48

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

X1

First check vertical bending:



X1 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in^3)	**Z (in^3)**	**R_b (in)**	**t (in)**
0.225	0.614	2.41	0.125
$\lambda = (R_b/t)^{1/2}$	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
4.39089968002003	For $\lambda \le 8.4$, F_c/Ω = 27.7ksi-0.17λ		
F_c/Ω (ksi)	For 8.4 < λ < 13.7, F_c/Ω = 18.5-0.593λ		
22.7	For $\lambda \ge 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \le 8.4$, local buckling does not control and the strength is controlled by min(ZF_y/Ω,1.5SF_y).			
If $\lambda > 8.4$, local buckling may control both the local buckling strength (F_c/Ω*S) and the yielding strength must be assessed.			
F_y/Ω (ksi)	**ZF_y/Ω(in-kips)**	SF_c/Ω (in-kips) (Only applies if > 6.5)	**M_a(in-lbs) = min(1.5SF_y/Ω,ZF_y/Ω, SF_c/Ω)*1000**
15.2	9.3328	N/A	5130

X1 Vertical loading, Lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.247	0.468	10100000	0.157	0.108
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
-2.22	0.355	25000	78	11700
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.14	0.5	0.5	0	-0.555
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
.				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	163001	11.511	10571	**6406**
36	114055	13.761	10350	**6273**
42	99215	14.754	10252	**6214**
48	87801	15.684	10161	**6158**
60	71389	17.394	9993	**6057**
72	60150	18.949	9841	**5964**

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

X1 horizontal loading, Lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.355	0.503	10100000	0.157	0.108
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.383	25000	78	12575
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.14	0.5	0.5	-1	-0.5
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y + C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E \lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2 E S_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
.				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	171201	13.465	11408	**6914**
36	119364	16.126	11178	**6774**
42	103724	17.300	11076	**6713**
48	91718	18.397	10981	**6655**
60	74490	20.414	10806	**6549**
72	62716	22.248	10647	**6453**

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

X1 horizontal loading, local buckling of flange element supported on both sides.			
S_c (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
0.237	0.325	0.743	0.125
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
5.944	For **λ** ≤ 22.8, F_c/Ω = 15.2ksi		
F_c/Ω (ksi)	For 22.9 < **λ** < 39, F_c/Ω = 19.0-0.170**λ**		
15.2	For **λ** ≥ 39, F_c/Ω = 484/**λ**		
If **λ** ≤ 22.8, local buckling does not control and the strength is controlled by ZF_y/Ω.			
If **λ** > 22.8, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
4.94	4940		

Top rail X2

X2 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in^3)	**Z (in^3)**	**R_b (in)**	**t (in)**
0.15	0.264	1.375	0.125
$\lambda = (R_b/t)^{1/2}$	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
3.3166247903554	For $\lambda \le 8.4$, $F_c/\Omega = 27.7\text{ksi}-0.17\lambda$		
F_c/Ω (ksi)	For $8.4 < \lambda < 13.7$, $F_c/\Omega = 18.5-0.593\lambda$		
22.7	For $\lambda \ge 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \le 8.4$, local buckling does not control and the strength is controlled by $\min(ZF_y/\Omega, 1.5SF_y)$.			
If $\lambda > 8.4$, local buckling may control both the local buckling strength (F_c/Ω*S) and the yielding strength must be assessed.			
F_y/Ω (ksi)	**ZF_y/Ω(in-kips)**	SF_c/Ω (in-kips) (Only applies if > 6.5)	**M_a(in-lbs) = $\min(1.5SF_y/\Omega, ZF_y/\Omega, SF_c/\Omega)$*1000**
15.2	4.0128	N/A	3420

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

X2 Vertical loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.15	0.264	10100000	0.001	0.279
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
-3.494	0.976	25000	78	6600
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.14	0.5	0.5	0	-0.8735
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	27284	21.926	5436	**3294**
36	13136	31.599	4922	**2983**
42	10128	35.986	4689	**2842**
48	8172	40.062	4472	**2711**
60	5862	47.300	4088	**2478**
72	4595	53.426	3763	**2280**

X2 Horizontal loading, lateral torsional buckling:				
S (in³)	**Z (in³)**	**E (psi)**	**J (in⁴)**	**C_w (in⁶)**
0.651	0.777	10100000	0.001	0.279
β_x (in)	**I_y (in⁴)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.147	25000	78	19425
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.14	0.5	0.5	-1	-0.5
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	25835	46.940	14154	**8578**
36	12163	68.411	11743	**7117**
42	9251	78.442	10546	**6392**
48	7354	87.978	8384	**5081**
60	5107	105.580	5822	**3528**
72	3866	121.347	4407	**2671**

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

X2 Horizontal loading direct buckling analysis

For local buckling use the continuous strength method. SCIA Engineer is used to create a 3D model of the flange. The model uses a 3rd order non-linear analysis that accounts for large deflections.

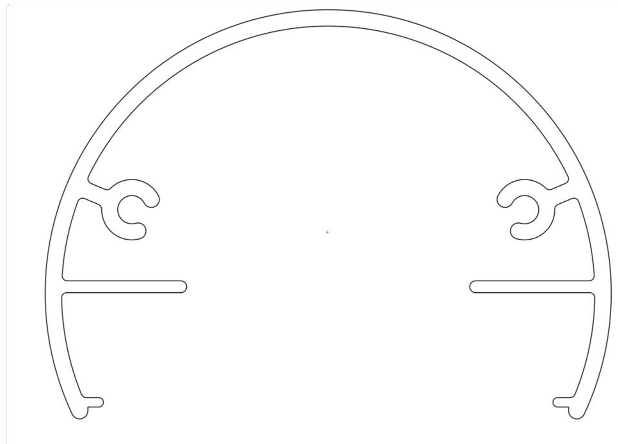
The supported edge of the leg is assumed to be able slide in the direction of the length of the top rail. The model uses a 3rd order non-linear analysis that accounts for large deflections and buckling modes. A 72" length of the element is created with a uniform load at one end that is factored until the element is either unable to bear the load or experiences high lateral displacement. The aluminum is assumed to be elastic so the buckling stress found is the elastic buckling stress, F_e, which is used with the provisions of ADM B.5.5.5 to determine the allowable compression stress.

Element area, A_e = Lt (in^2)	Total load applied to model, P_{cr} (lbs)	Elastic buckling stress, $F_e = P_{cr}/A_e$ (psi)
0.234	1990	8504

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

X2 Horizontal loading, Direct Strength Method			
S (in^3)	**Z (in^3)**	**F_e (ksi)**	**E (ksi)**
0.651	0.777	8.50	10100
$B_p = F_{cy}(1+(F_{cy}/(1500ksi))^{1/3})$	**$D_p = B_p/10*(B_p/E)^{1/2}$**	**k_1**	**F_{cy} (ksi)**
31.386	0.175	0.35	25
k_2	**$\lambda_{eq} = \pi(E/F_e)^{1/2}$**	**$\lambda_1 = (B_p-F_{cy})/D_p$**	**$\lambda_2 = (k_1B_p)/D_p$**
2.27	108.266	36.499	62.786
	For $\lambda_{eq} \leq \lambda_1$	**For $\lambda_1 < \lambda_{eq} \leq \lambda_2$**	**For $\lambda_{eq} \geq \lambda_2$**
F_c formula(ksi)	F_{cy}	$B_p-D_p\lambda_{eq}$	$k_2(B_pE)^{1/2}/\lambda_{eq}$
F_c (ksi) =	25	12.444	11.805
F_c, controlling	**$F_c/\Omega = F_c/1.65$**		
11.8	7.2		
If $\lambda_{eq} \leq \lambda_1$ local buckling does not control and the strength is controlled by ZF_y/Ω.			
If $\lambda_{eq} > \lambda_1$, local buckling controls and the strength is calculated as $F_c/\Omega*S$.			
M_a(in-kips)	**M_a(in-lbs)**		
7.68	7685		

X3
Vertical bending:



X3 Vertical loading, local buckling of flange element supported on both sides.			
S_c (in^3) (Assumes downward loading)	**Z (in^3)**	**b (in)**	**t (in)**
0.225	0.37	1.25	0.07
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
17.9	For **λ** ≤ 22.8, F_c/Ω = 15.2ksi		
F_c/Ω (ksi)	For 22.9 < **λ** < 39, F_c/Ω = 19.0-0.170**λ**		
15.2	For **λ** ≥ 39, F_c/Ω = 484/**λ**		
If **λ** ≤ 22.8, local buckling does not control and the strength is controlled by ZF_y/Ω.			
If **λ** > 22.8, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
5.624	5624		

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

X3 Vertical loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.225	0.370	10100000	0.002	0.251
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
-4.371	0.92	25000	78	9250
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.32	0.5	0.5	0	-1.09275
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E \lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2 E S_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	21957	27.818	7266	**4404**
36	11452	38.519	6503	**3941**
42	9210	42.952	6186	**3749**
48	7746	46.837	5909	**3581**
60	6000	53.216	5454	**3306**
72	5022	58.165	5101	**3092**

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

X3 Horizontal loading direct buckling analysis

For local buckling use the continuous strength method. SCIA Engineer is used to create a 3D model of the flange. The model uses a 3rd order non-linear analysis that accounts for large deflections.

The supported edge of the leg is assumed to be able slide in the direction of the length of the top rail. The model uses a 3rd order non-linear analysis that accounts for large deflections and buckling modes. A 72” length of the element is created with a uniform load at one end that is factored until the element is either unable to bear the load or experiences high lateral displacement. The aluminum is assumed to be elastic so the buckling stress found is the elastic buckling stress, F_e, which is used with the provisions of ADM B.5.5.5 to determine the allowable compression stress.

Element area, A_e = Lt (in^2)	Total load applied to model, P_{cr} (lbs)	Elastic buckling stress, $F_e = P_{cr}/A_e$(psi)
0.366	3050	8333

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

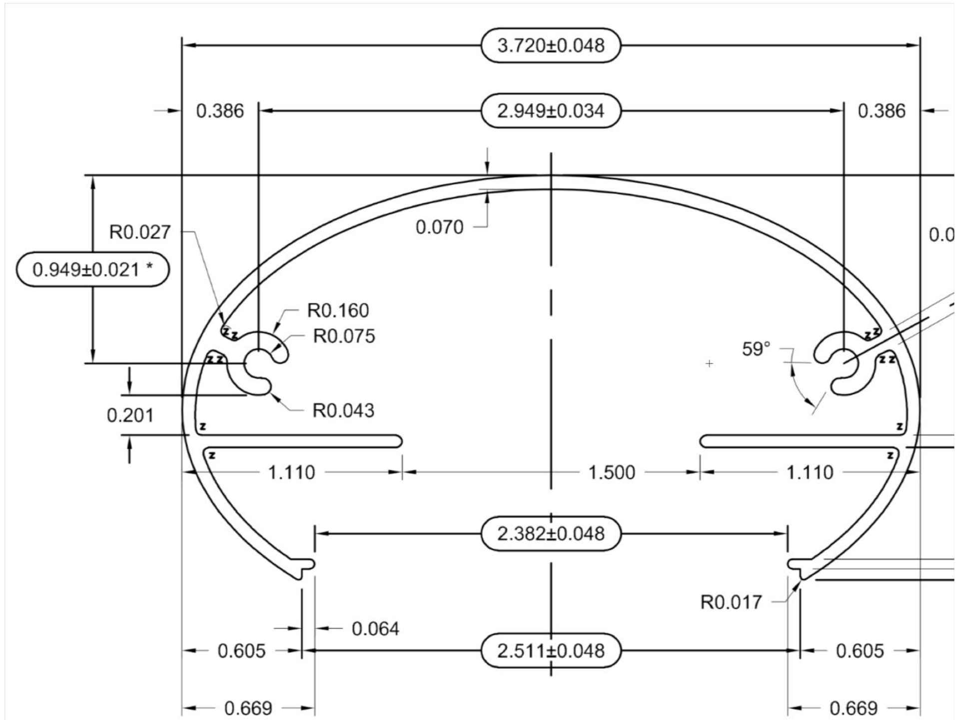
X3 Horizontal loading, Direct Strength Method			
S (in^3)	**Z (in^3)**	**F_e (ksi)**	**E (ksi)**
0.268	0.334	8333.33	10100
$B_p = F_{cy}(1+(F_{cy}/(1500ksi))^{1/3})$	**$D_p = B_p/10*(B_p/E)^{1/2}$**	**k_1**	**F_{cy} (ksi)**
31.386	0.175	0.35	25
k_2	**$\lambda_{eq} = \pi(E/F_e)^{1/2}$**	**$\lambda_1 = (B_p-F_{cy})/D_p$**	**$\lambda_2 = (k_1B_p)/D_p$**
2.27	3.459	36.499	62.786
	For $\lambda_{eq} \le \lambda_1$	**For $\lambda_1 < \lambda_{eq} \le \lambda_2$**	**For $\lambda_{eq} \ge \lambda_2$**
F_c formula(ksi)	F_{cy}	$B_p-D_p\lambda_{eq}$	$k_2(B_pE)^{1/2}/\lambda_{eq}$
F_c (ksi) =	25	30.781	369.533
F_c, controlling	**$F_c/\Omega = F_c/1.65$**		
25.0	15.2		
If $\lambda_{eq} \le \lambda_1$ local buckling does not control and the strength is controlled by ZF_y/Ω.			
If $\lambda_{eq} > \lambda_1$, local buckling controls and the strength is calculated as $F_c/\Omega*S$.			
M_a(in-kips)	**M_a(in-lbs)**		
8.35	8350		

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

X3 Horizontal loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.268	0.334	10100000	0.002	0.251
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.268	25000	78	8350
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.32	0.5	0.5	-0.875	-0.4375
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y + C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E \lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2 E S_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	32413	24.988	7082	**4292**
36	16190	35.356	6555	**3973**
42	12699	39.922	6324	**3833**
48	10403	44.108	6111	**3704**
60	7632	51.497	5736	**3476**
72	6052	57.829	5415	**3282**

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

X35
Vertical bending:



X35 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in³)	**Z (in³)**	**R_b (in)**	**t (in)**
0.245	0.352	2.64	0.07
$\lambda = (R_b/t)^{1/2}$	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
6.14119578862991	For $\lambda \le 8.4$, $F_c/\Omega = 27.7\text{ksi} - 0.17\lambda$		
F_c/Ω (ksi)	For $8.4 < \lambda < 13.7$, $F_c/\Omega = 18.5 - 0.593\lambda$		
22.7	For $\lambda \ge 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \le 8.4$, local buckling does not control and the strength is controlled by $\min(ZF_y/\Omega, 1.5SF_y)$.			
If $\lambda > 8.4$, local buckling may control both the local buckling strength (F_c/Ω*S) and the yielding strength must be assessed.			
F_y/Ω (ksi)	**ZF_y/Ω(in-kips)**	SF_c/Ω (in-kips) (Only applies if > 6.5)	**M_a(in-lbs) = $\min(1.5SF_y/\Omega, ZF_y/\Omega, SF_c/\Omega)$*1000**
15.2	5.3504	N/A	5350.4

X35 Vertical loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.209	0.352	10100000	0.001	0.63
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
-4.51	1.36	25000	78	8800
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.32	0.5	0.5	0	-1.1275
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	46036	18.516	7524	**4560**
36	21254	27.251	6922	**4195**
42	15992	31.416	6635	**4021**
48	12575	35.427	6358	**3854**
60	8554	42.955	5840	**3539**
72	6365	49.796	5368	**3253**

X35 Horizontal loading direct buckling analysis

For local buckling use the continuous strength method. SCIA Engineer is used to create a 3D model of the flange. The model uses a 3rd order non-linear analysis that accounts for large deflections.

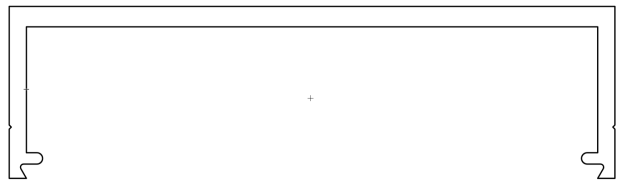
The supported edge of the leg is assumed to be able slide in the direction of the length of the top rail. The model uses a 3rd order non-linear analysis that accounts for large deflections and buckling modes. A 72" length of the element is created with a uniform load at one end that is factored until the element is either unable to bear the load or experiences high lateral displacement. The aluminum is assumed to be elastic so the buckling stress found is the elastic buckling stress, F_e, which is used with the provisions of ADM B.5.5.5 to determine the allowable compression stress.

Element area, A_e (in^2)	Total load applied to model, P_{cr} (lbs)	Elastic buckling stress, $F_e = P_{cr}/A_e$(psi)
0.362	5250	14503

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

X35 Horizontal loading, Direct Strength Method			
S (in^3)	**Z (in^3)**	**F_e (ksi)**	**E (ksi)**
0.734	0.933	14502.76	10100
$B_p = F_{cy}(1+(F_{cy}/(1500ksi))^{1/3})$	**$D_p = B_p/10*(B_p/E)^{1/2}$**	**k_1**	**F_{cy} (ksi)**
31.386	0.175	0.35	25
k_2	**$\lambda_{eq} = \pi(E/F_e)^{1/2}$**	**$\lambda_1 = (B_p-F_{cy})/D_p$**	**$\lambda_2 = (k_1B_p)/D_p$**
2.27	2.622	36.499	62.786
	For $\lambda_{eq} \le \lambda_1$	**For $\lambda_1 < \lambda_{eq} \le \lambda_2$**	**For $\lambda_{eq} \ge \lambda_2$**
F_c formula(ksi)	F_{cy}	$B_p-D_p\lambda_{eq}$	$k_2(B_pE)^{1/2}/\lambda_{eq}$
F_c (ksi) =	25	30.927	487.493
F_c, controlling	**$F_c/\Omega = F_c/1.65$**		
25.0	15.2		
If $\lambda_{eq} \le \lambda_1$ local buckling does not control and the strength is controlled by ZF_y/Ω.			
If $\lambda_{eq} > \lambda_1$, local buckling controls and the strength is calculated as $F_c/\Omega*S$.			
M_a(in-kips)	**M_a(in-lbs)**		
23.33	23325		

X35 Horizontal loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.734	0.933	10100000	0.001	0.63
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.238	25000	78	23325
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.32	0.5	0.5	0	0
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	68167	28.516	19194	**11633**
36	30926	42.336	17192	**10420**
42	23016	49.074	16216	**9828**
48	17879	55.679	15259	**9248**
60	11829	68.454	13409	**8127**
72	8531	80.607	11261	**6825**



X4 TOP RAIL
WOOD OR COMPOSITE MATERIAL
Aluminum rail is Alloy 6063 – T6 Aluminum

Aluminum Section
I_{xx}: 0.0153 in^4; I_{yy}: 0.322 in^4
S_{xx}: 0.0257 in^3; S_{yy}: 0.240 in^3

Wood – varies G≥ 0.43
1-1/4" x 4" nominal or 1"x3-1/2" true
I_{xx}: 0.292in^4; I_{yy}: 3.57in^4
C_{xx}: 0.5in; C_{yy}: 1.75in
S_{xx}: 0.583in^3; S_{yy}: 2.04in^3

Allowable Stress for aluminum: ADM Table 2-24
F_T = 15.2 ksi
F_C → 6' span
Rail is braced by wood At 16" o.c. and legs have stiffeners therefore
F_c = 15.2 ksi

C_F = 1.5 and C_d = 1.6
Minimum design strength F_b' = 3,590psi
Minimum nominal strength = 3,590psi/(1.6*1.5) = 1,500psi
Or use 2x4 with nominal strength = 0.583in^3/1.313in^3*1500psi = 666psi to use at the same max spacing

Allowable Moments ➔
Horiz.= 0.24in^3*15,200psi +2.04in^3*3,590psi = 11,000"#
Vertical load = 0.0257in^3 ·15,200psi +0.583in^3*3,590psi= 2,480"#

Maximum allowable load for 72" o.c. post spacing - Horizontal load (Assumes top rail is supported by picket)
W = 9,940"#*8/(69.625"^{2}) = 16.4 pli = 197 plf
P = 9,940"#*4/69.625" = 571#
Maximum span without load sharing, P = 200# or 50 lf - Vertical load
S = 2,410"#*4/200# = 48" clear
Max post spacing =48"+2.375" = 50.375" maximum post spacing

COMPOSITES: Composite materials, plastic lumber or similar may be used provided that the size and strength is comparable to the wood.

RAIL SPLICES:

Splice plate strength:

Vertical load will be direct bearing from rail/plate to post no bending or shear in plate.

Horizontal load will be transferred by shear in the fasteners.

Rail to splice plates:

#10 Tek screw strength: Check shear @ rail (6063-T6)

2x F_{urail}x dia screw x rail thickness x SF

$$V = 2 \cdot 30\ ksi \cdot 0.19" \cdot 0.09" \cdot \frac{1}{3\ (FS)} =$$

342#/screw; for two screws = 684#

or $F_{urplate}$x dia screw x plate thickness x SF

$$V = 38\ ksi \cdot 0.19" \cdot 0.125" \cdot \frac{1}{3\ (FS)} = 301\#/screw; \text{ for two screws} = 602\#$$

Top rail to splice piece:

Splice plate screw shear strength

2x F_{uplate}x dia screw x plate thickness x SF

$$V = 2 \cdot 38\ ksi \cdot 0.19" \cdot 0.125" \cdot \frac{1}{3\ (FS)} = 602\#/screw; \text{ for two screws} = 1{,}200\#$$

Check moment from horizontal load:

M = P*0.75". For 200# maximum load from a single rail on to splice plates

M = 0.75*200 = 150#"

S = 0.075 in^3

f_b = 150#"/(0.075) = 2,000 psi

May be used with (2) #10 tek screws per leg, four screws per splice minimum.

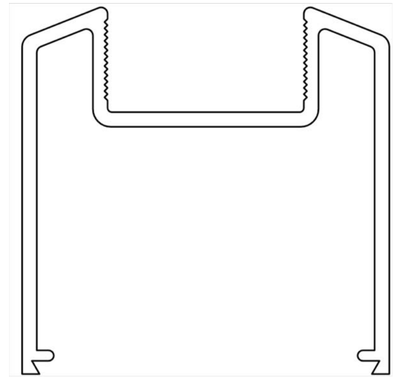
Other variations of splice

SPLST Stair splice

SPL=135 Deg splice

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

GLASS BOTTOM RAIL



Glass Bottom Rail Horizontal loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.234	0.273	10100000	0.001	0.07
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
-2.03	0.099	25000	78	6825
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1g_0+C_2\beta_x/2$**
1.14	0.5	0.5	0	-0.5075
F.4.2.5 Any Shape				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2ES_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	9961	45.322	5087	**3083**
36	5343	61.884	4452	**2698**

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

42	4329	68.750	4189	**2539**
48	3651	74.867	3954	**2396**
60	2808	85.364	3201	**1940**
72	2306	94.197	2629	**1593**

Glass bottom rail horizontal loading, Local buckling of flange element supported on one side.			
S (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
0.234	0.273	1.53	0.07
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
21.8571428571429	For **λ** ≤ 7.3, F_c/Ω = 15.2ksi		
F_c/Ω (ksi)	For 7.3 < **λ** < 12.6, F_c/Ω = 19.0-0.530**λ**		
7.09150326797384	For **λ** ≥ 12.6, F_c/Ω = 155/**λ**		
If **λ** ≤ 7.3, local buckling does not control and the strength is controlled by ZF_y/Ω.			
If **λ** > 7.3, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
1.65941176470588	1659		

Max considered span = 72”post spacing - 2.375” post width (in)	M_a(in-lbs)	Max loading is from 50# point load. M_{max} = 50#*L/4 (in-lbs)	
69.625	1659	870	< 1,660”# OK

Check max loading on bottom rail: P_a = min(1,660"#*2*12*12*8/(L^2H) or (384*10.1*10^6*.205*144*2)/(175*5HL^3)

Allowable Wind Load on Bottom Rail (psf)			
Bottom Rail Span, L (in)	**Glass Infill Height, H (in)**		
	33	*39*	*45*
36	89	76	66
48	50	43	37
60	32	27	24
72	21	18	16

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

X2 railing requires contribution from the bottom rail for vertical loads when spans are greater than 48”.

Glass bottom rail vertical loading, Local buckling of flange element supported on one side.			
S (in³)	**Z (in³)**	**b (in)**	**t (in)**
0.094	0.159	0.896	0.07
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
12.8	For $\lambda \leq 7.3$, $F_c/\Omega = 15.2$ksi		
F_c/Ω (ksi)	For $7.3 < \lambda < 12.6$, $F_c/\Omega = 19.0 - 0.530\lambda$		
12.109375	For $\lambda \geq 12.6$, $F_c/\Omega = 155/\lambda$		
If $\lambda \leq 7.3$, local buckling does not control and the strength is controlled by ZF_y/Ω.			
If $\lambda > 7.3$, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
1.13828125	1138		

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

Glass bottom rail vertical loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.094	0.159	10100000	0.001	0.07
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.205	25000	78	3975
Uniform Load on Simple Span (more conservative than point load at center span)				
C_b	**C_1**	**C_2**	**g_o(in)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.14	0.5	0.5	0	0
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E \lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2 E S_{xc}/\lambda^2$ for $\lambda \geq C_c$ (in-lbs)				
Lateral torsional buckling strength varies with unbraced length.				
L_b (in)	M_e (in-lbs)	λ	M_{nmb} (in-lbs)	**M_{nmb}/Ω (in-lbs)**
24	23752	18.602	3394	**2057**
36	12026	26.143	3159	**1914**
42	9471	29.459	3055	**1852**
48	7775	32.513	2960	**1794**
60	5701	37.970	2790	**1691**
72	4499	42.745	2641	**1600**

M_a = 1,600"#
M_a for X2 = 2,480"#
Combined = 4,530"#
M_{max} = 200#*60"/4 = 3000"# < 4,080"# OK