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.

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DigiSigned by elrobison@narrows.com Date: 12/4/24 12:17:03 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 Professional Engineer — Edward C. Robison — 54027

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 Professional Engineer — No. 046121 — 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

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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
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Commonwealth of Pennsylvania — Registered Professional — EDWARD C. ROBISON — Engineer — No.73556
EXP 09/30/2025

Edward C. Robison — Registered Engineer — Agriculture — Commerce — No.124674 — State of Tennessee

State of Texas — EDWARD C. ROBISON — 100688 — Licensed Professional Engineer
FIRM #F-12044
EXP 12/31/2024

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

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Table 1: Standard Post Installations:

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.*

Table 2: 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

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Table 3: Standard Cable Infill Installations:

3-3/16” on center spacing:

Cable spreader isn’t required for post spacings 4’ or less when cable is installed as required herein.

One cable spreader picket mid-distance between posts shall be used for post spacings above 4’.

Stainless Steel Cable Installation Parameters:			
Max spacing (in)	**Max number of cables for commercial installation**	**Max number of cables for residential installation**	**Minimum cable pretension any installation (lbs)**
3-3/16”	12	10	200
Maximum cable pretension for commercial applications (lbs)	**Maximum cable pretension for residential applications (lbs)**	**Max cable length with minimum 125# pretension (ft)**	**Max post spacing with cable spreader (in)**
175	200	50	72
Single corner posts may only be used for residential applications with a max pretension of 150# per cable. Other applications use (2) posts each corner.			

LOAD CASES:
Picket rail Dead load = 5 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

Wind loading on the picket railing is insignificant compared to live loading.

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STAINLESS STEEL CABLE IN-FILL:

Stainless Steel Cable Installation Parameters:				
Max cable spacing (in)	**Max number of cables for commercial installation**	**Max number of cables for residential installation**	**Minimum cable pretension any installation (lbs)**	
3-3/16”	12	10	200	
Maximum cable pretension for commercial applications (lbs)	**Maximum cable pretension for residential applications (lbs)**	**Max cable length with minimum 130# pretension (ft)**	**Max post spacing with cable spreader (in)**	**Max post spacing without cable spreader (in)**
175	200	80	72	48
Single corner posts may only be used for residential applications with a max pretension of 150# per cable. Other applications use (2) posts each corner.				

Loading on cable infill is subject to interpretation with no clear guidance from the ICC. The only live load required to be applied to the cable infill is the 50# infill load on one square foot. However, the code also requires that a 4” sphere not be able to pass through the infill. There is no code defined force to apply to the 4” sphere.

A 45˚ cone (or pyramid) with a 5.6# load is a criteria that has been referenced by many manufacturers of cable railing. This assumes the cone covers one ninth of the one square foot [12”x12”/(4”x4”)] = 9.   50#/9 = 5.6#. The cone is chosen to have a 45˚ angle because that is approximately the contact angle between a 4” sphere and cables at 3-3/16” apart.

The maximum cable free span is when the posts are spaced 48” and no cable brace is used. For greater post spacings the cable spreader must be used so the span of the cable is lower. Max cable span = 48”-2.375” post wide = 45.625”.

1/8" cable application

Cable with minimum pretension of 130#. Allowable total cable length is 150'.				
Cable spacing, S (in)	**Cable area, A (in^2)**	**E (psi)**	**Cone diameter at measurement, D (in)**	**Cable free span, L_f (in)**
3.1875	0.0123	28000000	4	45.625
Cable diameter, D_c (in)	**Cable spread, SP = $D-S+D_c$ (in)**	**Cone angle, ø (°)**	**Cable vertical deflection (in), ∂_y = SP/2**	**Cable horizontal deflection (in), $\partial_x = \partial_y * \tan(ø)$**
0.125	0.9375	45	0.46875	0.46875
Minimum cable pretension, T_{pre} (lbs)	**Total cable length, L (in)**	**Total cable deflection, $\partial = (\partial_x^2+\partial_y^2)^{1/2}$ (in)**	**New cable length between supports, L_a (in) = $2((L_f/2)^2+\partial^2)^{1/2}$**	
200	1800	0.6629	45.6443	
Total new cable length, $L_{new} = (L-L_f+L_a)$ (in)	**Change in cable length (in), $L_\Delta = L_a-L_f$**	**Increase in cable tension, T_Δ (lbs) = L_Δ*E*A/L**	**Total cable tension, T = $T_{pre}+T_\Delta$ (lbs)**	
1800.0193	0.0193	3.7	203.7	
Cable slope, S = $\arctan(\partial/(L_f/2))$ (°)	**Force against cone per cable, P = sin(S)*T (lbs)**	**Total resisted load on cone = 2sinø*P (lbs)**		
1.6645	5.9	8.4	≥ 5.6# OK	

Option for increased span length. 54" max span. Cable with maximum pretension of 200#. Allowable total cable length is 80'.				
Cable spacing, S (in)	**Cable area, A (in^2)**	**E (psi)**	**Cone diameter at measurement, D (in)**	**Cable free span, L_f (in)**
3.1875	0.0123	28000000	4	54
Cable diameter, D_c (in)	**Cable spread, SP = $D-S+D_c$ (in)**	**Cone angle, ø (°)**	**Cable vertical deflection (in), ∂_y = SP/2**	**Cable horizontal deflection (in), $\partial_x = \partial_y * \tan(ø)$**
0.125	0.9375	45	0.46875	0.46875
Minimum cable pretension, T_{pre} (lbs)	**Total cable length, L (in)**	**Total cable deflection, $\partial = (\partial_x^2+\partial_y^2)^{1/2}$ (in)**	**New cable length between supports, L_a (in) = $2((L_f/2)^2+\partial^2)^{1/2}$**	
200	960	0.6629	54.0163	
Total new cable length, $L_{new} = (L-L_f+L_a)$ (in)	**Change in cable length (in), $L_\Delta = L_a-L_f$**	**Increase in cable tension, T_Δ (lbs) = L_Δ*E*A/L**	**Total cable tension, T = $T_{pre}+T_\Delta$ (lbs)**	
960.0163	0.0163	5.8	205.8	
Cable slope, S = atan($\partial/(L_f/2)$) (°)	**Force against cone per cable, P = sin(S)*T (lbs)**	**Total resisted load on cone = 2sinø*P (lbs)**		
1.4065	5.1	7.1	≥ 5.6# OK	

3/16" cable application

Cable with minimum pretension of 130#. Allowable total cable length is 80'.				
Cable spacing, S (in)	**Cable area, A (in^2)**	**E (psi)**	**Cone diameter at measurement, D (in)**	**Cable free span, L_f (in)**
3.1875	0.0196	28000000	4	45.625
Cable diameter, D_c (in)	**Cable spread, SP = $D-S+D_c$ (in)**	**Cone angle, ø (°)**	**Cable vertical deflection (in), ∂_y = SP/2**	**Cable horizontal deflection (in), ∂_x = ∂_y*tan(ø)**
0.1875	1	45	0.5	0.5
Minimum cable pretension, T_{pre} (lbs)	**Total cable length, L (in)**	**Total cable deflection, $\partial = (\partial_x^2+\partial_y^2)^{1/2}$ (in)**	**New cable length between supports, L_a (in) = $2((L_f/2)^2+\partial^2)^{1/2}$**	
200	1800	0.7071	45.6469	
Total new cable length, $L_{new} = (L-L_f+L_a)$ (in)	**Change in cable length (in), $L_\Delta = L_a-L_f$**	**Increase in cable tension, T_Δ (lbs) = L_Δ*E*A/L**	**Total cable tension, T = $T_{pre}+T_\Delta$ (lbs)**	
1800.0219	0.0219	6.7	206.7	
Cable slope, S = atan($\partial/(L_f/2)$) (°)	**Force against cone per cable, P = sin(S)*T (lbs)**	**Total resisted load on cone = 2sinø*P (lbs)**		
1.7754	6.4	9.1	≥ 5.6# OK	

Cable with minimum pretension of 150# which can be used with corner posts in commercial settings. Allowable total cable length is 150’.				
Cable spacing, S (in)	**Cable area, A (in^2)**	**E (psi)**	**Cone diameter at measurement, D (in)**	**Cable free span, L_f (in)**
3.1875	0.0123	28000000	4	45.625
Cable diameter, D_c (in)	**Cable spread, SP = $D-S+D_c$ (in)**	**Cone angle, ø (°)**	**Cable vertical deflection (in), ∂_y = SP/2**	**Cable horizontal deflection (in), ∂_x = ∂_y*tan(ø)**
0.125	0.9375	45	0.46875	0.46875
Minimum cable pretension, T_{pre} (lbs)	**Total cable length, L (in)**	**Total cable deflection, $\partial = (\partial_x^2+\partial_y^2)^{1/2}$ (in)**	**New cable length between supports, L_a (in) = $2((L_f/2)^2+\partial^2)^{1/2}$**	
200	1800	0.6629	45.6443	
Total new cable length, $L_{new} = (L-L_f+L_a)$ (in)	**Change in cable length (in), $L_\Delta = L_a-L_f$**	**Increase in cable tension, T_Δ (lbs) = L_Δ*E*A/L**	**Total cable tension, T = $T_{pre}+T_\Delta$ (lbs)**	
1800.0193	0.0193	3.7	203.7	
Cable slope, S = atan(∂/(L_f/2)) (°)	**Force against cone per cable, P = sin(S)*T (lbs)**	**Total resisted load on cone = 2sinø*P (lbs)**		
1.6645	5.9	8.4	≥ 5.6# OK	

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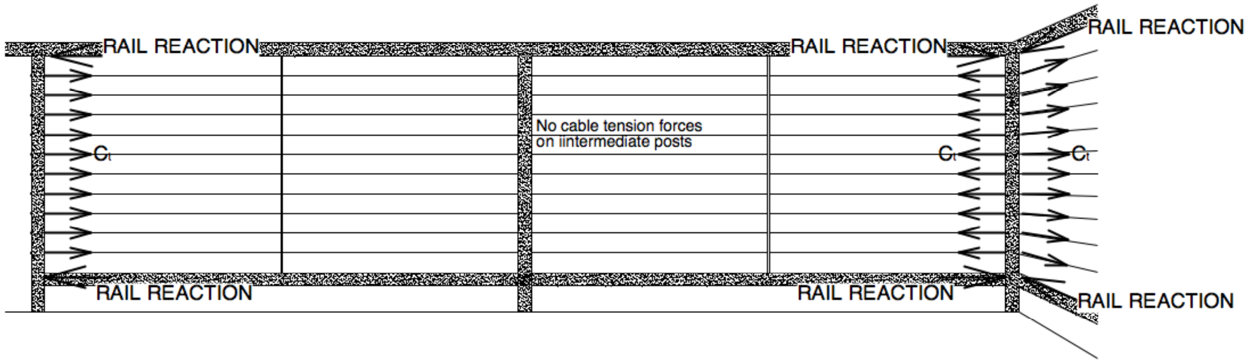
Cable induced forces on posts:



Cable tension forces occur where cables either change direction at the post or are terminated at a post. Top rail acts as a compression element to resist cable tension forces. The top rail infill piece inserts tight between the posts so that the post reaction occurs by direct bearing.

Bottom rail when present will be in direct bearing to act as a compression element.

When no bottom rail is present the post anchorage shall be designed to accommodate a shear load in line with the cables based on one half the total cable tension load.

Typical residential application:					
Span, L = 36"-1" (Assumes no bottom rail)	Cable pretension, P (lbs)	Number of cables, N	End post loading, w = P*N/L (pli)	End post moment = $wL^2/8$ (in-lbs)	Allowable moment on post, M_a (in-lbs)
35	200	10	57.1	8750	19,600"#
					> 8,750"#
Typical commercial application:					
Span, L = 42"-1" (Assumes no bottom rail)	Max able pretension, P (lbs)	Number of cables, N	End post loading, w = P*N/L (pli)	End post moment = $wL^2/8$ (in-lbs)	Allowable moment on post, M_a (in-lbs)
41	200	11	53.7	11275	19,600
					> 11,300"#
The following pages include calculations checking the interaction between cable loading on the end post and live loading on the end post.					

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Check infill loading on posts, commercial end post condition:			
Post span, L (in)	Load per cable, T (lbs)	Number of cables, N	Uniform load W(pli)
40.5	200	11	53.66
Post moment (in-lbs), $wL^2/8$	Check in combination with top rail loads.	$M_{a,y}$ (in-lbs)	$M_{a,x}$ (in-lbs)
11002		21800	19600
x (measured from top)	M_{infill} = wx(l-x)/2 (in-lbs)	$M_{toprail}$ = 200#*(x+1)	$M_{infill}/M_{a,y}+M_{toprail}/M_{a,x}$ (in-lbs) < 1.0 OK
20	11000.00	4200	0.72
21	10986.59	4400	0.73
22	10919.51	4600	0.74
23	10798.78	4800	0.74
24	10624.39	5000	0.74
25	10396.34	5200	0.74
26	10114.63	5400	0.74
27	9779.27	5600	0.73
28	9390.24	5800	0.73
29	8947.56	6000	0.72
30	8451.22	6200	0.70
31	7901.22	6400	0.69
32	7297.56	6600	0.67
33	6640.24	6800	0.65
34	5929.27	7000	0.63
35	5164.63	7200	0.60
36	4346.34	7400	0.58
37	3474.39	7600	0.55
38	2548.78	7800	0.51
39	1569.51	8000	0.48
40	536.59	8200	0.44
40.5	0.00	8300	0.42

For corner posts the corner will hold at most, half the 200# live load because the top rail will act as a tie and share the load with at least one other post. For 50plf live loads the corner post may be loaded in both directions but the loading will still be shared with one other post in each direction that is loaded. For worst case 6' spacing, P_{total}=2*6'/2*50plf/2 = 150# which controls over the 200# concentrated load's 100# net load to the post.

Check infill loading on posts, commercial end post condition:			
Post span, L (in)	Load per cable, T (lbs)	Number of cables, N	Uniform load W(pli)
40.5	150	11	40.74
Post moment (in-lbs), $wL^2/8$	Check in combination with top rail loads.	$M_{a,y}$ (in-lbs)	$M_{a,x}$ (in-lbs)
8353		21800	19600
x (measured from top)	M_{infill} = wx(l-x)/2 (in-lbs)	$M_{toprail}$ = 75#*(x+1)	$(M_{infill}+M_{toprail})/M_{a,y}+(M_{infill}+M_{toprail})/M_{a,x}$ (in-lbs)
20	8351.85	1575	0.96
21	8341.67	1650	0.97
22	8290.74	1725	0.97
23	8199.07	1800	0.97
24	8066.67	1875	0.96
25	7893.52	1950	0.95
26	7679.63	2025	0.94
27	7425.00	2100	0.92
28	7129.63	2175	0.90
29	6793.52	2250	0.88
30	6416.67	2325	0.85
31	5999.07	2400	0.81
32	5540.74	2475	0.78
33	5041.67	2550	0.74
34	4501.85	2625	0.69
35	3921.30	2700	0.64
36	3300.00	2775	0.59
37	2637.96	2850	0.53

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Check infill loading on posts, commercial end post condition:			
Post span, L (in)	Load per cable, T (lbs)	Number of cables, N	Uniform load W(pli)
34.5	200	9	52.17
Post moment (in-lbs), $wL^2/8$	Check in combination with top rail loads.	$M_{a,y}$ (in-lbs)	$M_{a,x}$ (in-lbs)
7763		21800	19600
x (measured from top)	$M_{infill} = wx(l-x)/2$ (in-lbs)	$M_{toprail} = 75\#*(x+1)$	$(M_{infill}+M_{toprail})/M_{a,y}+(M_{infill}+M_{toprail})/M_{a,x}$ (in-lbs)
12	7043.48	975	0.78
13	7291.30	1050	0.81
14	7486.96	1125	0.83
15	7630.43	1200	0.86
16	7721.74	1275	0.87
17	7760.87	1350	0.88
18	7747.83	1425	0.89
19	7682.61	1500	0.89
20	7565.22	1575	0.89
21	7395.65	1650	0.88
22	7173.91	1725	0.86
23	6900.00	1800	0.84
24	6573.91	1875	0.82
25	6195.65	1950	0.79
26	5765.22	2025	0.75
27	5282.61	2100	0.72
28	4747.83	2175	0.67
29	4160.87	2250	0.62

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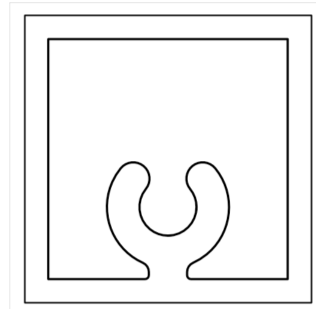

PICKETS 3/4" SQUARE

Used as infill at 4" O.C. max
$Z_x=0.050in^3$

b/t = 0.626"/0.06" = 10.4

Allowable moment, $M_a=0.050in^3*15.2ksi=760"\#$
Loading is spread over a 12"x12" square and the pickets are at 4" O.C. max. Assume infill load is carried by a minimum of three pickets.
Max picket span = 760"#*4/(50#/3)= 182" >> 60" (Pickets may be used at all guard heights considered ing this report.)

Connections
Pickets to top and bottom rails direct bearing for lateral loads –ok
#10 screw in to top and bottom infill pieces. Shear strength =
2x F_{upost}x dia screw x t_{rail} x SF ADM Eq 5.4.3-2

$$V= 30\ ksi \cdot 0.19'' \cdot 0.1'' \cdot \frac{1}{3\ (FS)} = 190\#$$

Connections
Pickets to top and bottom rails direct bearing for lateral loads –ok
#10 screw in to top and bottom infill pieces. Shear strength =
2x F_{upost}x dia screw x t_{rail} x SF ADM Eq 5.4.3-2

$$V= 38\ ksi \cdot 0.19'' \cdot 0.1'' \cdot \frac{1}{3\ (FS)} = 240\#$$

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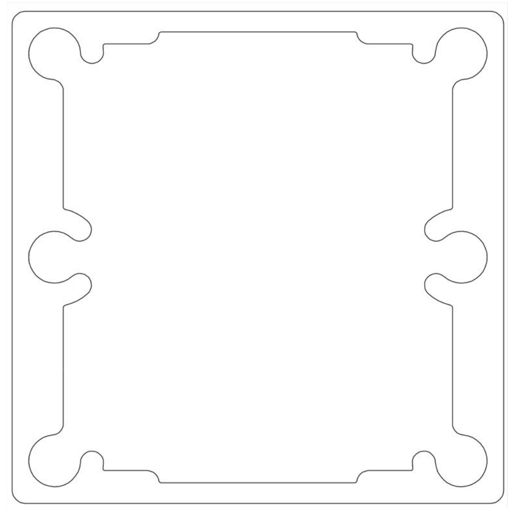
POST DESIGN - 2-3/8" Square

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 (in³)	**Z (in³)**	**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**		
9.16666666666667	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)**		
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.

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6005-T61 Aluminum. Lateral torsional buckling:				
S (in³)	**Z (in³)**	**E (psi)**	**J (in⁴)**	**C_w (in⁶)**
1.13	1.35	10100000	1.42	0.029
β_x (in)	**I_y (in⁴)**	**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				
$\lambda = \pi\sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2 EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E\lambda S_{xc}/C_c^3$ for $\lambda < C_c$ (in-lbs)				
$M_{nmb} = \pi^2 ES_{xc}/\lambda^2$ 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	9.382	32657	**19792**
36	656087	11.492	32411	**19643**
42	562329	12.413	32304	**19578**
48	492020	13.270	32204	**19518**
60	393600	14.837	32021	**19407**
72	327992	16.253	31856	**19307**

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

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		

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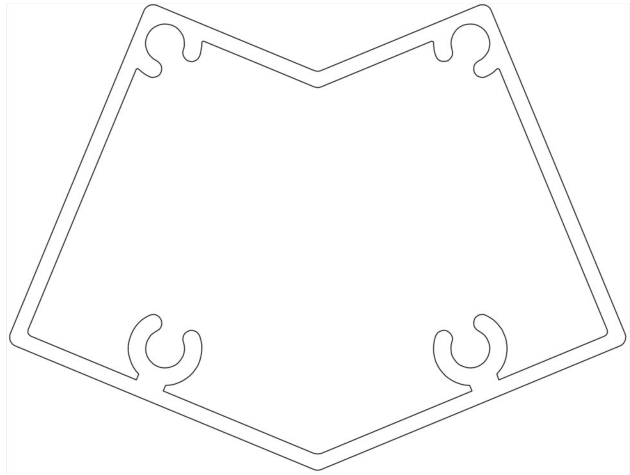
135° x 2.375" Post
First calculate moment strength about the strong axis.
I_x = 1.90 in^4
S_x = 0.956 in^3
Z_x = 1.45 in^3
I_y = 1.26 in^4
b = 1.97"
t = 0.1"
C_w = 0.0342 in^6
β = -0.0157 in
g_0 = 0 in

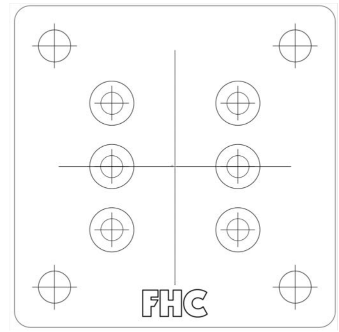


6005-T61 Aluminum. Local buckling of flange element supported on both sides.			
S (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
0.935	1.39	1.97	0.1
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
19.7	For **λ** ≤ 20.8, F_c/Ω = 21.2ksi For yielding, Strength is controlled by rupture, F/Ω = 38ksi/1.95 = 19.5ksi.		
F_c/Ω (ksi)	For 20.8 < **λ** < 33, F_c/Ω = 27.3-0.291**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)**		
27.105	27105		

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

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		

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BASEPLATE MOUNTED TO WOOD – SINGLE FAMILY RESIDENCE

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

$$T_{200} = \frac{7{,}200}{2*4.36”} = 826\#$$

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.

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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

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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} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)			
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	

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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} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)			
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)	$a=T_{n,min}/(0.85f'_c b_e)$ (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	0.49	10503	10500	1.198	
				Pass	<1.2 Pass	

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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)
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	

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} (in^2)	A_{nc} (in^2)	A_{vo} (in^2)	A_{No} (in^2)			
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	

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 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), $H = \min(d/2+b_c/4, d)$	$P_n = 0.85 b_b H f'_c$ (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		

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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		

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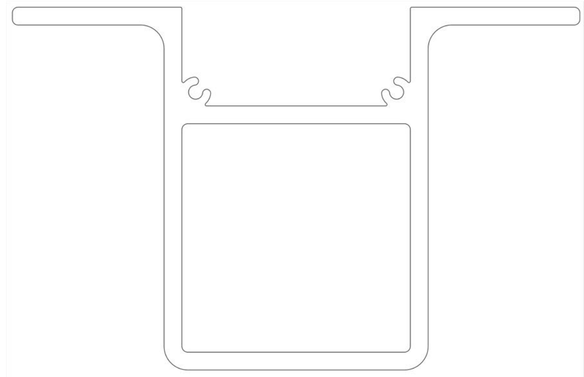
FASCIA MOUNTED POSTS WITH FASCIA BOOT



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

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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

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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

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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.

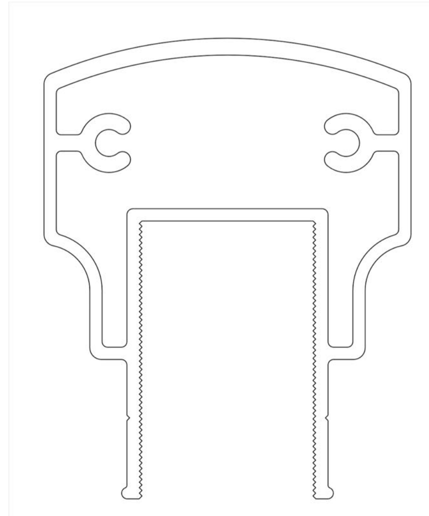
Top Rail Engineering Properties

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

X1

First check vertical bending:



X1 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in³)	**Z (in³)**	**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 $\lambda \leq 8.4$, 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 $\lambda \geq 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \leq 8.4$, 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

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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**

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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)**	**$U = C_1g_0+C_2\beta_x/2$**
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	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**

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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 $\lambda \le 22.8$, $F_c/\Omega = 15.2$ksi		
F_c/Ω (ksi)	For $22.9 < \lambda < 39$, $F_c/\Omega = 19.0-0.170\lambda$		
15.2	For $\lambda \ge 39$, $F_c/\Omega = 484/\lambda$		
If $\lambda \le 22.8$, local buckling does not control and the strength is controlled by ZF_y/Ω.			
If $\lambda > 22.8$, local buckling controls and the strength is calculated as F_c/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
4.94	4940		

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Top rail X2

X2 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in³)	**Z (in³)**	**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 $\lambda \leq 8.4$, $F_c/\Omega = 27.7\text{ksi}-0.17\lambda$		
F_c/Ω (ksi)	For $8.4 < \lambda < 13.7$, $F_c/\Omega = 18.5-0.593\lambda$		
22.7	For $\lambda \geq 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \leq 8.4$, 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

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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)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.14	0.5	0.5	0	-0.8735
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (in-lbs)				
$M_{nmb} = M_{np}(1-\lambda/C_c)+\pi^2 E\lambda S_{xc}/C_c^3$ 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	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**

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X2 Horizontal loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.651	0.777	10100000	0.001	0.279
β_x (in)	**I_y (in^4)**	**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**

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

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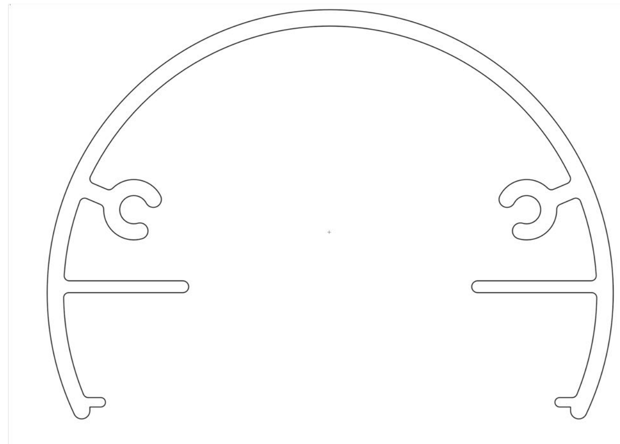
X2 Horizontal loading, Direct Strength Method			
S (in^3)	**Z (in^3)**	**F_e (ksi)**	**E (ksi)**
0.651	0.777	8.50	10100
$B_p = F_{cy}(1+(F_{cy}/(1500ksi))^{1/3})$	**$D_p = B_p/10*(B_p/E)^{1/2}$**	**k_1**	**F_{cy} (ksi)**
31.386	0.175	0.35	25
k_2	**$\lambda_{eq} = \pi(E/F_e)^{1/2}$**	**$\lambda_1 = (B_p-F_{cy})/D_p$**	**$\lambda_2 = (k_1B_p)/D_p$**
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 (in³) (Assumes downward loading)	**Z (in³)**	**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		

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)**	**$U = C_1g_0+C_2\beta_x/2$**
1.32	0.5	0.5	0	-1.09275
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (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	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**

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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

X3 Horizontal loading, Direct Strength Method			
S (in³)	**Z (in³)**	**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		

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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)**	**$U = C_1g_0+C_2\beta_x/2$**
1.32	0.5	0.5	-0.875	-0.4375
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_bM_e))}$				
$M_e = \pi^2EI_y/(L_b^2)(U+\sqrt{(U^2+0.038JL_b^2/I_y+C_w/I_y)})$ (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	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**

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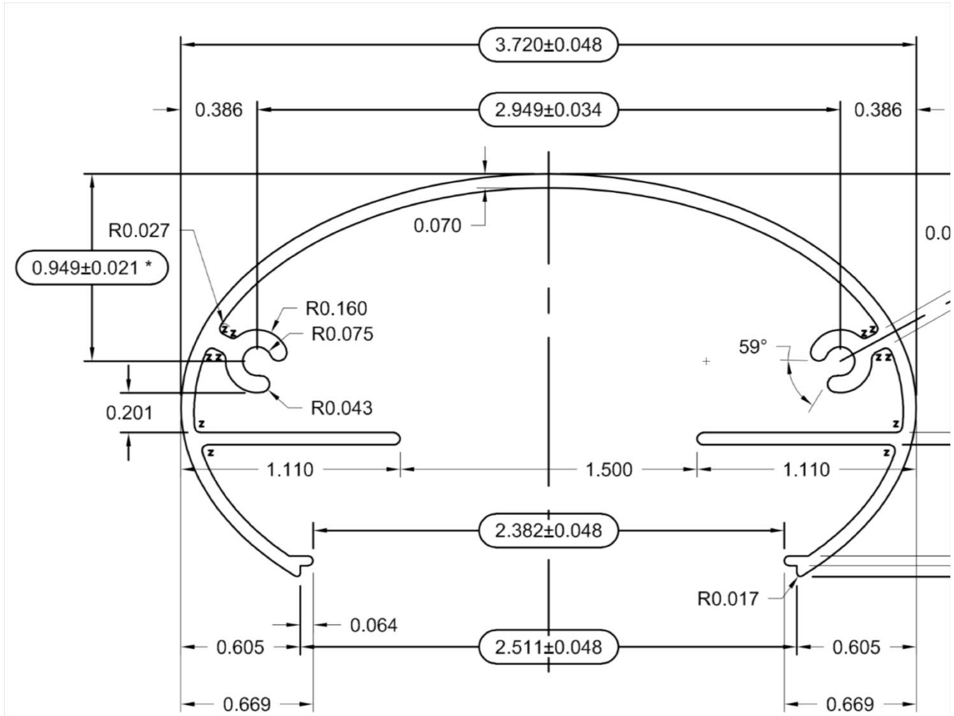
X35
Vertical bending:



X35 Vertical loading, local buckling of round hollow element under flexural compression			
S_c (in^3)	**Z (in^3)**	**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 $\lambda \leq 8.4$, F_c/Ω = 27.7ksi-0.17λ		
F_c/Ω (ksi)	For $8.4 < \lambda < 13.7$, F_c/Ω = 18.5-0.593λ		
22.7	For $\lambda \geq 13.7$, $F_c/\Omega = 3776/(\lambda^2(1+\lambda/35)^2)$		
If $\lambda \leq 8.4$, 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	5.3504	N/A	5350.4

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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)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.32	0.5	0.5	0	-1.1275
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y + C_w/I_y)})$ (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	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**

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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

X35 Horizontal loading, Direct Strength Method			
S (in³)	**Z (in³)**	**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 $\lambda_{eq} \leq \lambda_1$	**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	30.927	487.493
F_c, controlling	**$F_c/\Omega = F_c/1.65$**		
25.0	15.2		
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/Ω*S.			
M_a(in-kips)	**M_a(in-lbs)**		
23.33	23325		

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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)**	**$U = C_1 g_0 + C_2 \beta_x / 2$**
1.32	0.5	0.5	0	0
F.4.2.5 Any Shape				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b^2)(U+\sqrt{(U^2+0.038 J L_b^2/I_y + C_w/I_y)})$ (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	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**

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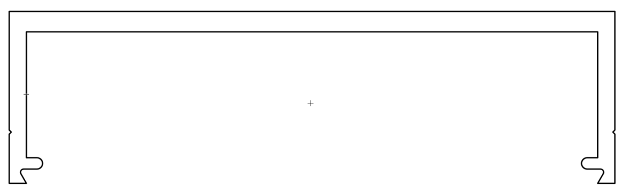
X4 TOP RAIL
WOOD OR COMPOSITE MATERIAL
Aluminum rail is Alloy 6063 – T6 Aluminum



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.

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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

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

342#/screw; for two screws = 684#

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

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

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

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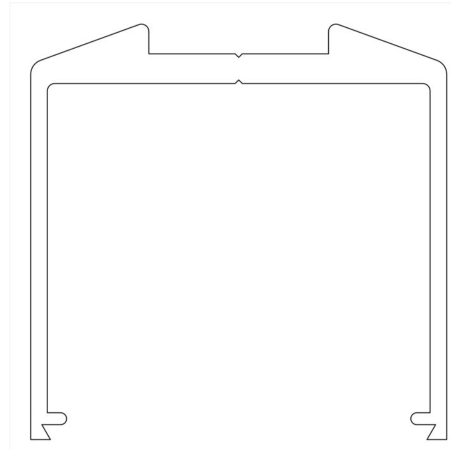
PICKET BOTTOM RAIL



Picket Bottom Rail Horizontal loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.249	0.299	10100000	0.002	0.065
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.13	25000	78	7475
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				
$\lambda = \pi \sqrt{(ES/(C_b M_e))}$				
$M_e = \pi^2 E I_y/(L_b{}^2)(U+\sqrt{(U^2+0.038JL_b{}^2/I_y+C_w/I_y)})$ (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)				
$M_{nmb} = \pi^2 ES_{xc}/\lambda^2$ 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)**

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24	20580	32.527	6059	**3672**
36	11214	44.064	5557	**3368**
42	9091	48.940	5345	**3239**
48	7644	53.371	5152	**3122**
60	5809	61.220	4810	**2915**
72	4697	68.084	4511	**2734**

Picket bottom rail horizontal loading, Local buckling of flange element supported on one side.			
S (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
0.249	0.299	1.5	0.07
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
21.4285714285714	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**		
7.23333333333334	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)**		
1.8011	1801		

Max considered span = 72"post spacing - 2.375" post width (in)	M_a(in-lbs)	Designed for picket or cable infill only. Max loading is from 50# point load. M_{max} = 50#*L/4 (in-lbs)	
69.625	1801	870	< 1,800"# OK

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

Picket bottom rail vertical loading, Local buckling of flange element supported on one side.			
S (in^3)	**Z (in^3)**	**b (in)**	**t (in)**
0.249	0.299	1.55	0.125
λ = b/t	**Allowable compression stress is calculated according to ADM 2020 Design Table 2-21**		
12.4	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**		
12.428	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)**		
3.094572	3095		

Picket bottom rail vertical loading, lateral torsional buckling:				
S (in^3)	**Z (in^3)**	**E (psi)**	**J (in^4)**	**C_w (in^6)**
0.108	0.189	10100000	0.002	0.065
β_x (in)	**I_y (in^4)**	**F_y (psi)**	**C_c**	**$M_p = ZF_y$ (in-lbs)**
0	0.218	25000	78	4725
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
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)				

$M_{nmb} = \pi^2 E S_{xc}/\lambda^2$ 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	26650	18.824	4012	**2431**
36	14521	25.502	3759	**2278**
42	11772	28.323	3652	**2213**
48	9898	30.888	3555	**2154**
60	7523	35.430	3383	**2050**
72	6083	39.403	3232	**1959**

M_a = 2,050"#
M_a for X2 = 2,480"#
Combined = 4,530"#
M_{max} = 200#*60"/4 = 3000"# < 4,530"# OK

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