15 Frebruary 2021

Architectural Railing Division
C.R.Laurence Co., Inc.
2503 E Vernon Ave.
Los Angeles, CA 90058

SUBJ: ARS – ALUMINUM RAILING, PICKET, INFILL PANEL AND CABLE SYSTEMS
SERIES 100, 200, 300, 350 AND 400 SERIES SYSTEMS

The ARS Aluminum Railing System utilizes aluminum extrusions 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 all natural environments. The ARS may be used for residential, commercial and industrial applications. The ARS 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, perpendicular to rail

On In-fill Panels:

Concentrated load = 50# on one sf.
Distributed load = 25 psf on area of in-fill, including spaces
Wind load = To be determined based on infill

Refer to IBC Section 1607.8.1

The ARS system will meet all applicable requirements of the 2006, 2009, 2012, 2015 and 2018 International Building Codes and International Residential Codes, 2010, 2013 and 2016 California Building and Residential Codes, Florida Building Code and other state codes adopting these versions of the IBC and IRC. Aluminum components are designed per 2015 Aluminum Design Manual unless noted otherwise herein. Stainless steel components are designed in accordance with SEI/ASCE 8-02 *Specification for the Design of Cold-Formed Stainless Steel Structural Member*s or AISC Design Guide 27 *Structural Stainless Steel* as applicable. Wood components and anchorage to wood are designed in accordance with the *National Design Specification for Wood Construction*. The ARS system meets the requirements of ASTM E 985-00 *Standard Specification for Permanent Metal Railing Systems and Rails for Buildings* and ICC AC273 *Acceptance Criteria for Handrails and Guards*. The Specifier/project proponent is responsible for verifying that an installation complies with these recommendations and applicable codes for the specific project conditions and installation parameters. This report may be used in support of project specific evaluations but must not be used in place of a project specific review.

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SIGNATURE PAGE: Sealed 22 June 2021

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

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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,
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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.
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Date Lic. No. 58604

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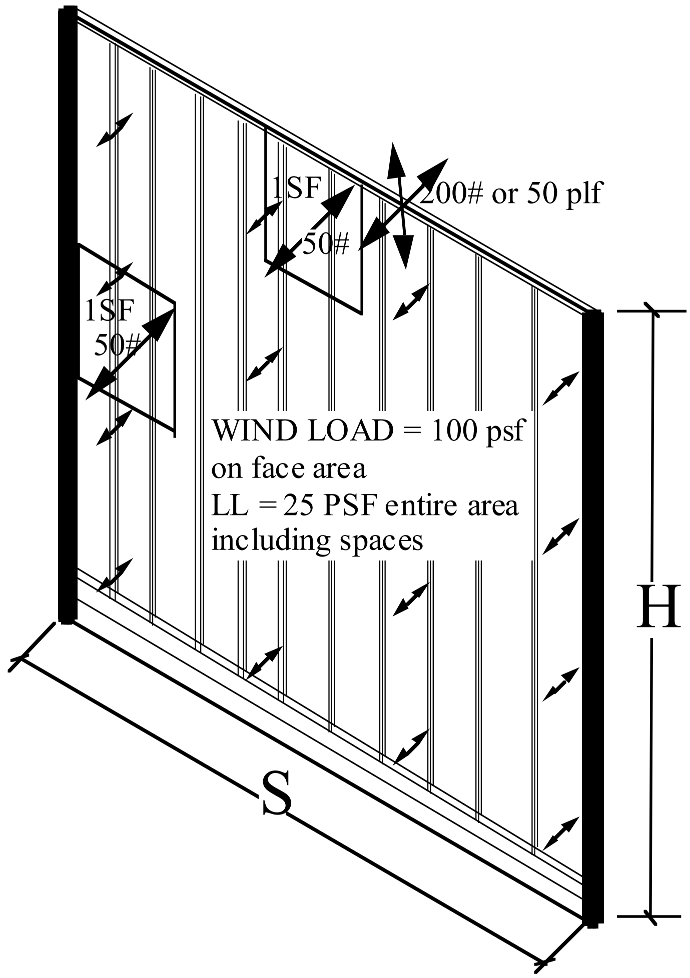
LOAD CASES:
Picket rail Dead load = 5 plf for 42” rail height or less.



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

For wind load surface area:
Pickets 3/4” wide by 4” on center
Top rail = 3” maximum
Post = 2.375”
Area for typical 4’ section by 42” high:
42”*2.375”+3”*48”+1.7”*45.625”
+0.75*36*11 = 618.3 in^2
% surface/area = 618.3/(48”*42”) =
30.67%
Wind load for 25 psf equivalent load =
25/0.3067 = 81.5 psf

NOTES ON ASTM E 985-00:
The loads given in ASTM E985-00 section 7 are test loads not allowable or service loads. The greatest test load of 365# concentrated load is less than the 500# ultimate load to which the 200# concentrated design live load in these calculations equates.

Compliance with ASTM E 985-00 while not directly demonstrated by testing is inferred from these calculations since all component strengths and applicable deflections are demonstrated as adequate to meet all testing criteria.

The test loads listed in ASTM E 985-00 do not meet the test load requirements of IBC 1709 or ICC AC273 *Acceptance Criteria For Handrails and Guards*. The engineering herein demonstrates the system has adequate strength to meet the test loads in IBC 1709 and AC273.

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POSTS

All ARS posts are extruded 6061-T6 aluminum or 6005A-T61. First, basic performance criteria for each post are listed below. Then, more specific performance criteria is shown with the supporting calculations on the following pages. Lastly, detailed sectional properties and moment strength calculations are shown for each post in accordance with the 2015 or 2020 ADM.

Basic Performance Criteria:

4 screw 2-3/8" square post
Allowable post moment, $M_{a,x}$ = 17,100"#
Second moment of area, I_x = 0.871in^4
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

6 screw 2-3/8" square post
Allowable post moment, $M_{a,x}$ = 19,500"#
Second moment of area, I_x = 0.997in^4
Handles top rail live loading at 60" tall with 72" post spacing.

Heavy 2-3/8" square post
Allowable post moment, $M_{a,x}$ = 26,200"#
Second moment of area, I_x = 1.264in^4
Handles top rail live loading at 60" tall with 72" post spacing.

135° corner post
Does not limit performance criteria below any of the 2-3/8" square posts. Note that the 135° post connection to the baseplate will control the allowable spacing when the six screw posts are used as the intermediate posts.

4" square post
Allowable post moment, $M_{a,x}$ = 49,500"#
Second moment of area, I_x = 5.48in^4
Handles top rail live loading at 60" tall with 72" post spacing.

Trim Line post
Allowable post moment, $M_{a,x}$ = 10,400"#
Second moment of area, I_x = 0.524in^4
Handles top rail live loading at 42" tall with 59" post spacing.

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Guard Rail Post Design

System: ARS
Post: 4 screw post

Post Properties:

E (psi)	10100000
I (in^4)	0.871
Ma (in-lbs)	17100
Δa	H/12

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	85.5
Δ at H=42"	0.56145775

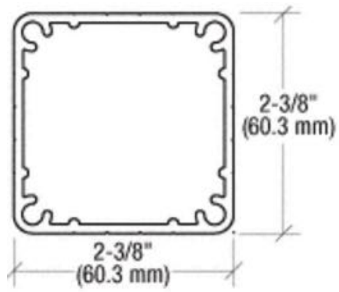
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

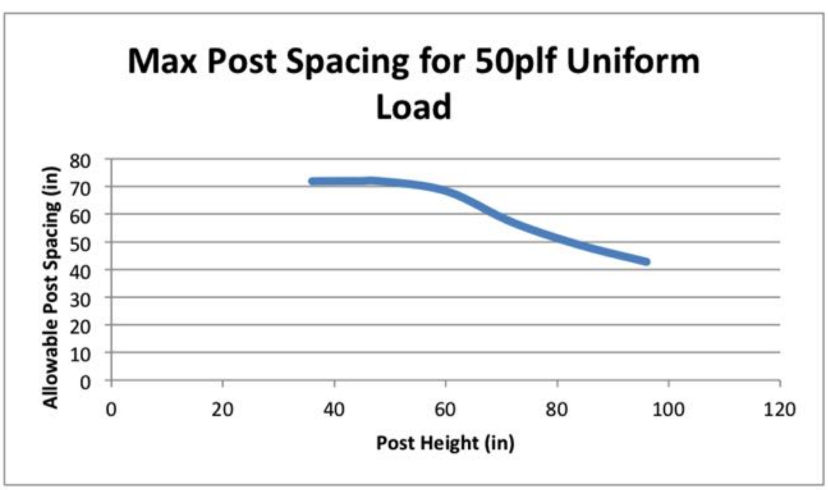
Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	68.4
72	57
84	48.85714286
96	42.75



Detailed calculations for this post are on pages 17 - 19

Note on shear in posts: As shear loads carried on the web elements and the flange elements provide the primary bending resistance the interaction of shear and bending need not be checked on any of the posts in accordance with ADM H.3.1



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Wind Load

*M=P/144*TW*H^2/2*

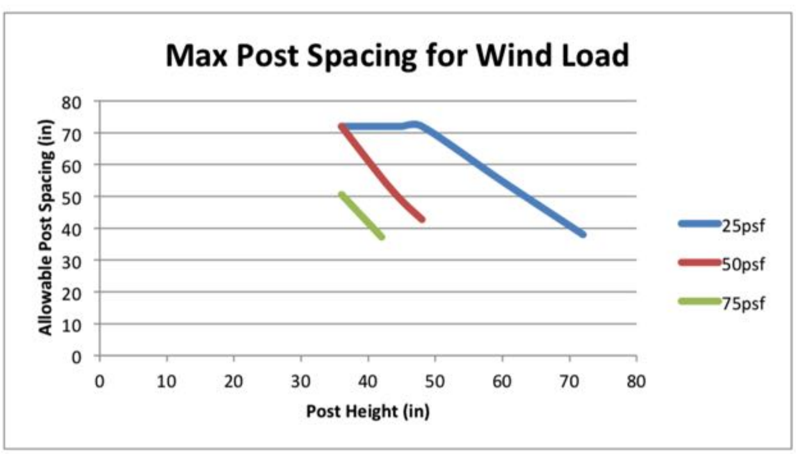
*TWmax = 2*Ma/(H²*P/144) < Δa*3EI/(H³*P/144*H/2))*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	54.72
72	38
84	<36"
96	<36"

P= 50psf	
Post Height (in)	Max Spacing (in)
36	72
42	55.83673469
45	48.64
48	42.75
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Max Spacing (in)
36	50.66666667
42	37.2244898
45	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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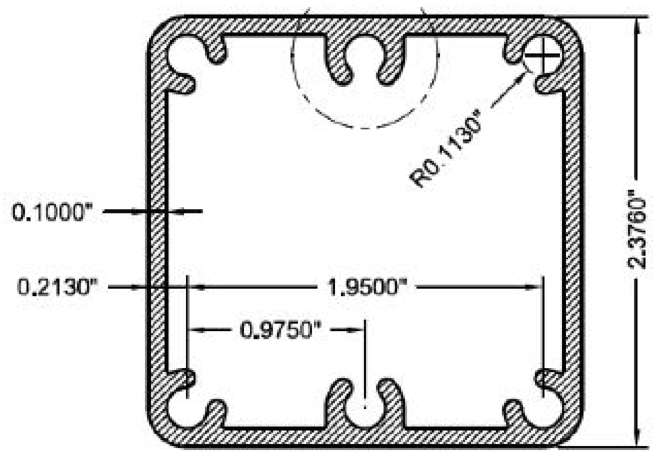
Guard Rail Post Design

System: ARS
Post: 6 screw post

Post Properties:

E (psi)	10100000
I (in^4)	0.997
Ma (in-lbs)	19500
Δa	H/12



Detailed calculations for this post are on pages 20 - 25

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	>96"
Δ at H=42"	0.49050121

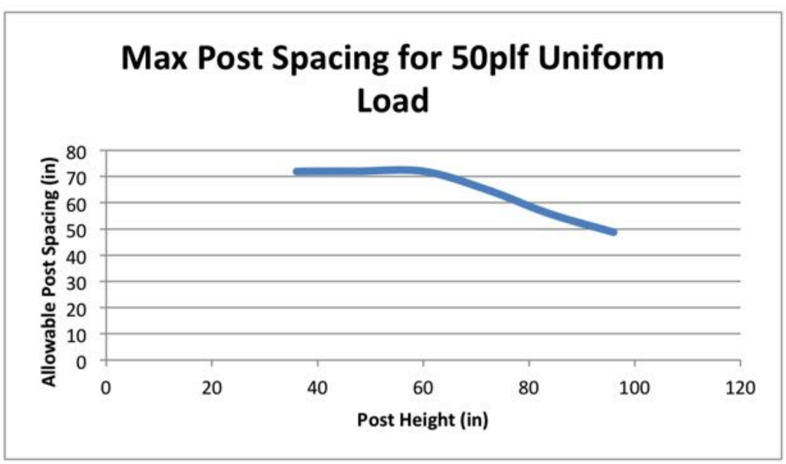
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H³*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	72
72	65
84	55.71428571
96	48.75



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Wind Load

*M=P/144*TW*H^2/2*

*TWmax = 2*Ma/(H²*P/144) < Δa*3EI/(H³*P/144*H/2))*

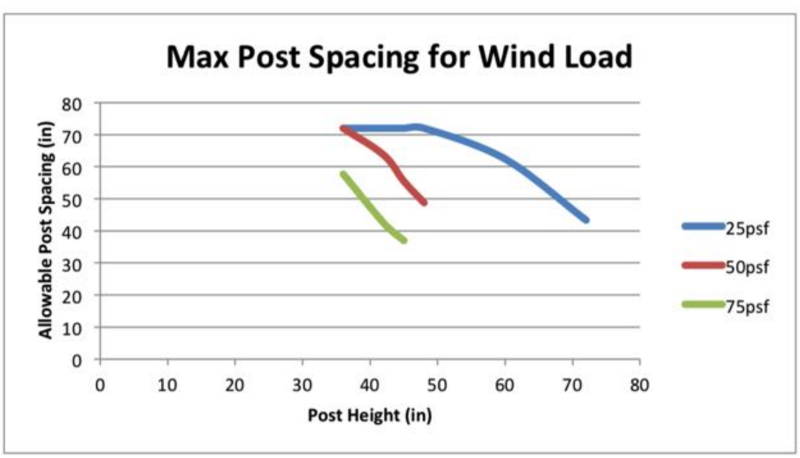
$TW_{max} = 2*Ma/(H^2*P/144) < \Delta a*3EI/(H^3*P/144*H/2))$

Allowable post height with respect to post spacing for different wind pressures:

P=	25psf
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	62.4
72	43.33333333
84	<36"
96	<36"

P=	50psf
Post Height (in)	Max Spacing (in)
36	72
42	63.67346939
45	55.46666667
48	48.75
60	<36"
72	<36"
84	<36"
96	<36"

P=	75psf
Post Height (in)	Max Spacing (in)
36	57.77777778
42	42.44897959
45	36.97777778
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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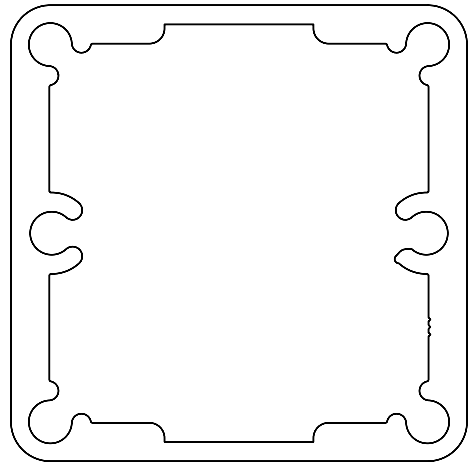
Guard Rail Post Design

System: ARS
Post: 6 screw strong post



Detailed calculations for this post are on pages 26 - 29

Post Properties:

E (psi)	10100000
I (in^4)	1.26
Ma (in-lbs)	26200
Δa	H/12

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	120
Δ at H=42"	0.38811881

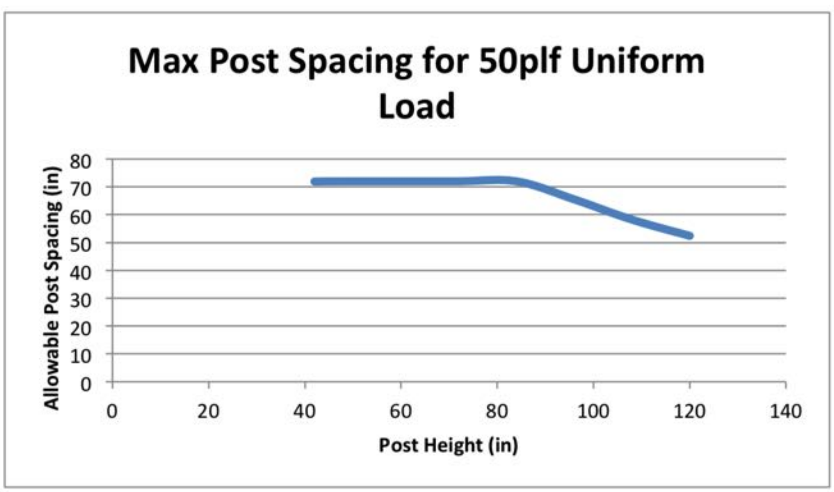
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
42	72
48	72
60	72
72	72
84	72
96	65.5
108	58.22222222
120	52.4



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Wind Load

*M=P/144*TW*H^2/2*

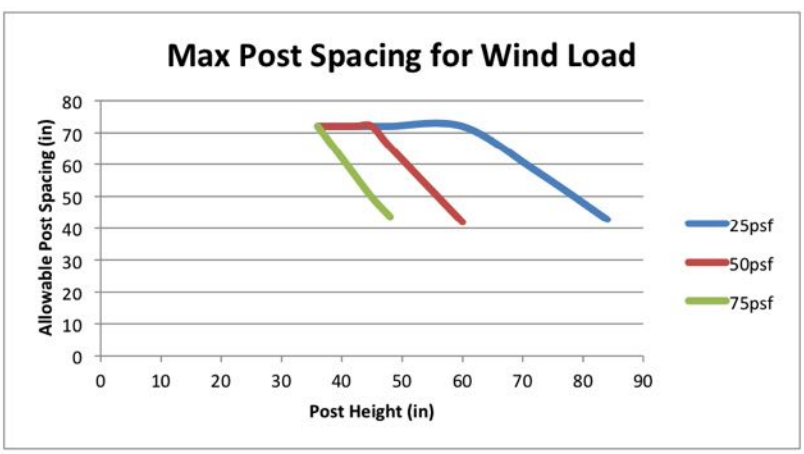
*TWmax = 2*Ma/(H²*P/144) < Δa*3EI/(H³*P/144*H/2))*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	72
72	58.22222222
84	42.7755102
96	<36"

P= 50psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	65.5
60	41.92
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Max Spacing (in)
36	72
42	57.03401361
45	49.68296296
48	43.66666667
60	<36"
72	<36"
84	<36"
96	<36"



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Guard Rail Post Design

System:	ARS
Post:	4" post

Post Properties:

E (psi)	10100000
I (in^4)	5.48
Ma (in-lbs)	49500
Δa	H/48

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	131.513998
Δ at H=42"	0.089239

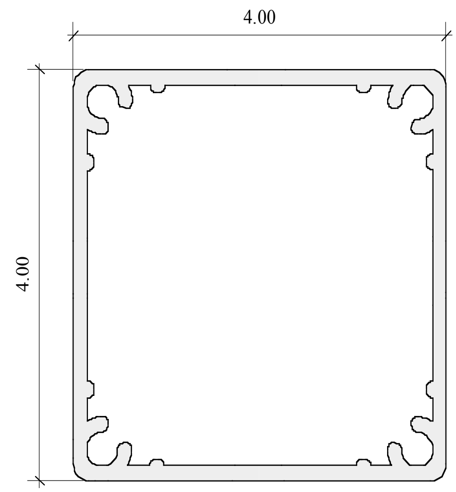
50plf uniform load along top rail

*M=50plf/12*TW*H*

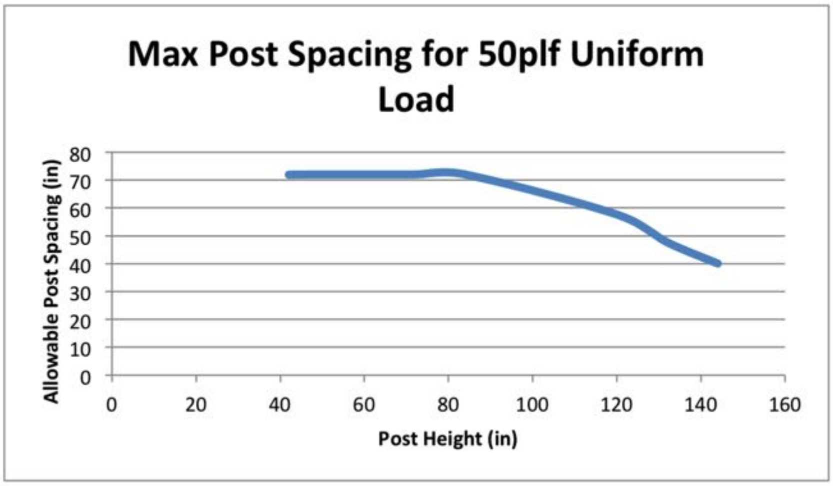
*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
42	72
48	72
60	72
72	72
84	72
120	57.65416667
132	47.64807163
144	40.03761574



Detailed calculations for this post are on pages 34 - 35



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Wind Load

*M=P/144*TW*H^2/2*

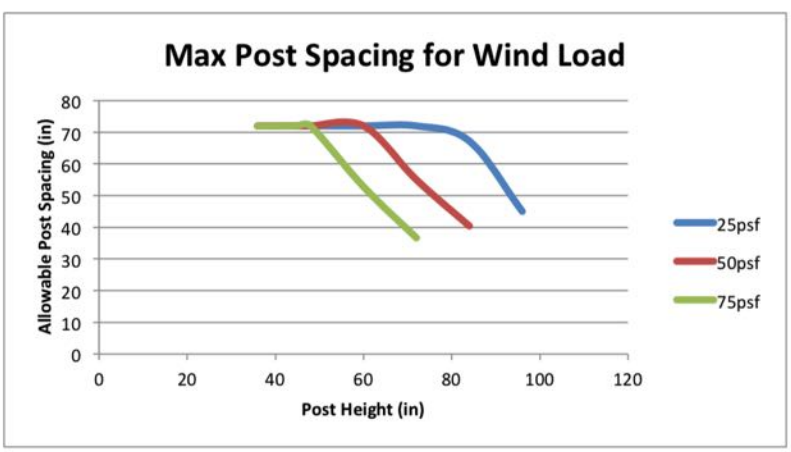
*TWmax = 2*Ma/(H²*P/144) < Δa*3EI/(H³*P/144*H/2))*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	72
72	72
84	67.23517979
96	45.04231771

P= 50psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	72
72	55
84	40.40816327
96	<36"

P= 75psf	
Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	52.8
72	36.66666667
84	<36"
96	<36"



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Guard Rail Post Design

System: ARS
Post: Trimline Post

Post Properties:

E (psi)	10100000
I (in^4)	0.524
Ma (in-lbs)	10400
Δa	H/12

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	52
Δ at H=42"	0.93326279

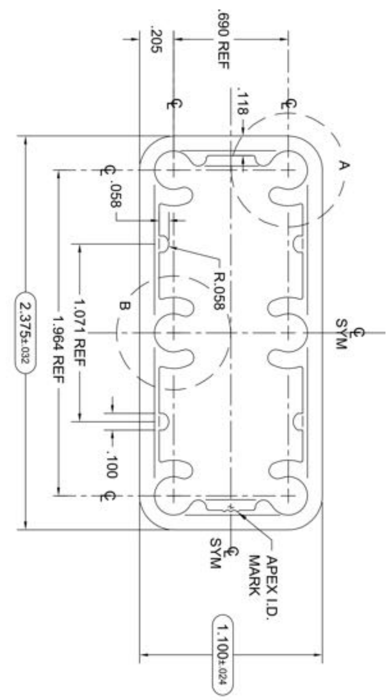
50plf uniform load along top rail

*M=50plf/12*TW*H*

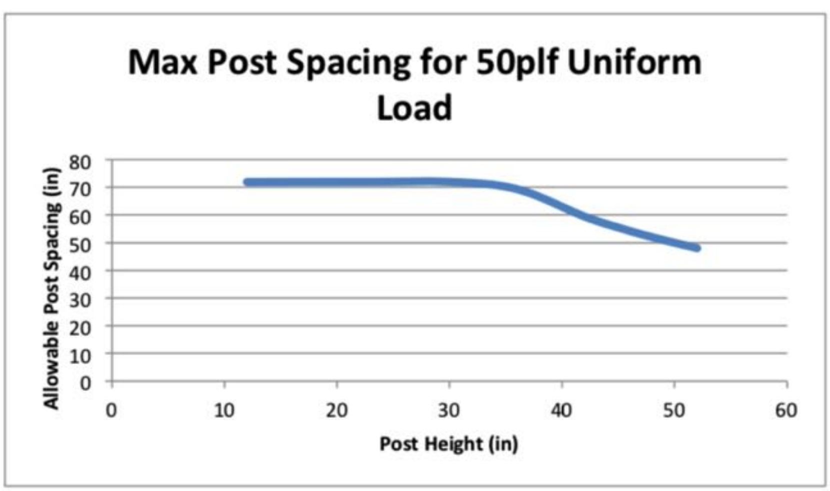
*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
12	72
24	72
30	72
36	69.33333333
42	59.42857143
45	55.46666667
48	52
52	48



Detailed calculations for this post are on pages 36 - 40



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Wind Load

*M=P/144*TW*H^2/2*

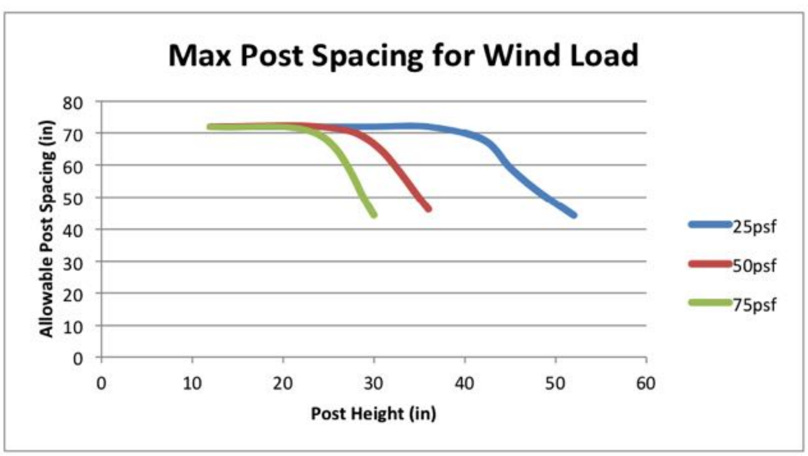
TWmax = $2*Ma/(H^2*P/144) < \Delta a*3EI/(H^3*P/144*H/2)$

Allowable post height with respect to post spacing for different wind pressures:

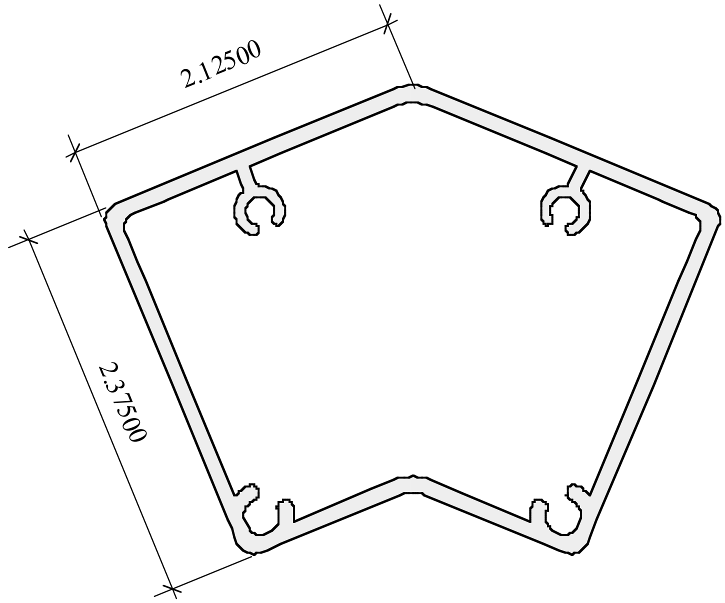
P=	25psf
Post Height (in)	Max Spacing (in)
12	72
24	72
30	72
36	72
42	67.91836735
45	59.16444444
48	52
52	44.30769231

P=	50psf
Post Height (in)	Max Spacing (in)
12	72
24	72
30	66.56
36	46.22222222
42	<36"
45	<36"
48	<36"
52	<36"

P=	75psf
Post Height (in)	Max Spacing (in)
12	72
24	69.33333333
30	44.37333333
36	<36"
42	<36"
45	<36"
48	<36"
52	<36"



135˚ Post
Detailed calculations for this post are on pages 30 - 33



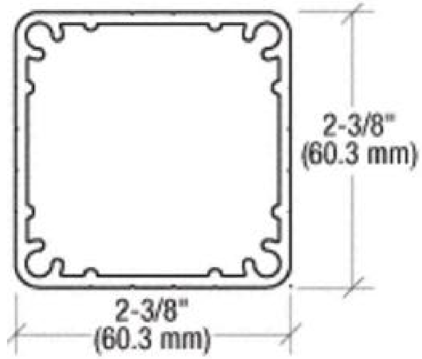
Weak axis moment strength,
$M_{a,y}$ = 22,200"#
Strong axis moment strength,
$M_{a,x}$ = 27,000"#

A SCIA model of one corner post and a six screw post on each side was created to determine how loading is shared with the adjacent posts. The posts are 42" tall and spaced 72" on center. The 50plf live load was checked normal to the top rail or normal to the weak axis of the corner post. The loading to the corner post is highest when it is normal to its weak axis. The baseplate moment was found to be 10,500"#. Note that for a long run of intermediate posts the baseplate moment would be assumed to be 50plf*6'*42" = 12,600"#. This is a reduction of 16.7%. Therefore, the 135˚ will only control allowable spacing if the intermediate post strength is greater than 22,200"#/ 0.833 = 26,700"#. Note that none of the 2-3/8" post develop this strength. Therefore, the 135˚ post does not limit post spacing for any of the 2-3/8" square posts. However, the anchorage detail may limit post spacing if the anchorage method for the 135˚ post is different than for the other posts.

2-3/8" Square 4 Screw Post

6061-T6 Aluminum extrusion

The bending strength about each axis of the 4 screw post is the same. Note the strength of the post will be limited by the connection to the baseplate.



Post shear strength:

All shear is assumed as carried by the web elements. In accordance with ADM G.2 the web shear strength is:

$V_a = V_n/\Omega = F_{su}\ A_n/k_t/\Omega$

F_{su} = 15 ksi and k_t = 1.0 for 6061-T6 or 6005A-T61

V_a = [15ksi*2*(2.12"*0.1")/1]/1.95 = 3,262# As this greatly exceeds any potential shear loading on any post for any proposed use in this report further checking of post shear is needed.

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion 4 Screw post

Section Properties

Ix (in4)	0.871
Sx (in3)	0.733
Zx (in3)	0.877
Iy (in4)	0.871
J (in4)	1.178
b	1.562
t	0.1

Cw (in6)	0
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	15.62 =	*b/t*	
λ1	20.8		
λ2	33		
F/Ω (ksi)	21.2 =	21.2	*for λ<λ1*
		27.3-.291λ	*for λ1<λ<λ2*
		580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	18592 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	0.877	
Mn/Ω (in-lbs)	17090.2564 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	609.240411 =	*See 2015 ADM F.4-9*	
λ	10.9513635 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	28.3850957 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	17203.0883 =	*Mnmb/1.65*1000*

Strength is controlled by rupture

Ma (in-lbs) **17090**

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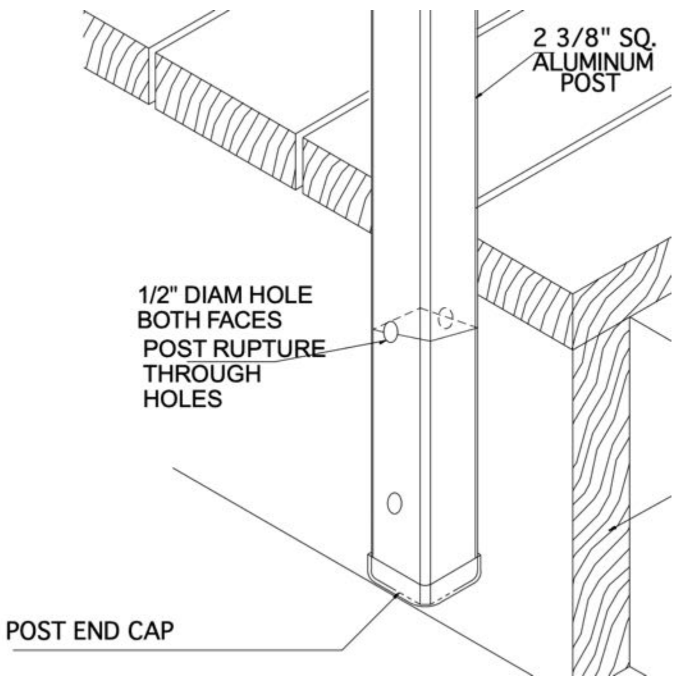
For posts directly fascia mounted with 3/8" bolts through post:
Reduced strength at bolt hole:
For loading parallel to bolt axis:
Assume 3/8" + 1/8" over size = 1/2" holes both sides of post
$Z_{red} = 0.7590\ in^3$
Reduced section properties for the holes

Addition of holes at base of post only affects rupture strength as the controlling failure mode at the holes.

$M_{nu}/\Omega = ZF_u/\Omega = 0.7590in^3*38ksi/1.95 = 14{,}800$"# (Reduced post strength for bolt holes front and back of post)

For loading perpendicular to bolt axis
$Z_{red}=0.8666in^3$

$M_{nu}/\Omega = ZF_u/\Omega = 0.8666in^3*38ksi/1.95 = 16{,}900$"# (Reduced post strength for bolt holes in sides of post)



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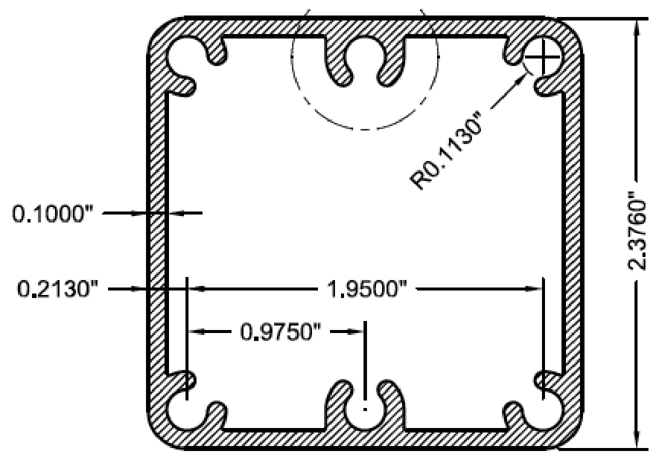
2-3/8" Square Post



6061-T6 Aluminum extrusion
Note the strength of the post will be limited by the connection to the baseplate.

Typical installation is where the top rail runs parallel to the strong axis of the post.

First calculate moment strength about the strong axis:

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	6 Screw post strong axis

Section Properties

Ix (in4)	0.997
Sx (in3)	0.838
Zx (in3)	1
Iy (in4)	0.889
J (in4)	1.209
b	1.597
t	0.1

Cw (in6)	0
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	15.97 =	*b/t*	
λ1	20.8		
λ2	33		
F/Ω (ksi)	21.2 =	21.2	*for λ<λ1*
		27.3-.291λ	*for λ1<λ<λ2*
		580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	21200 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	1	
Mn/Ω (in-lbs)	19487.1795 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	623.549608 =	*See 2015 ADM F.4-9*	
λ	11.5743621 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	32.225108 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	19530.3685 =	*Mnmb/1.65*1000*	

Strength is controlled by rupture

Ma (in-lbs) **19487**

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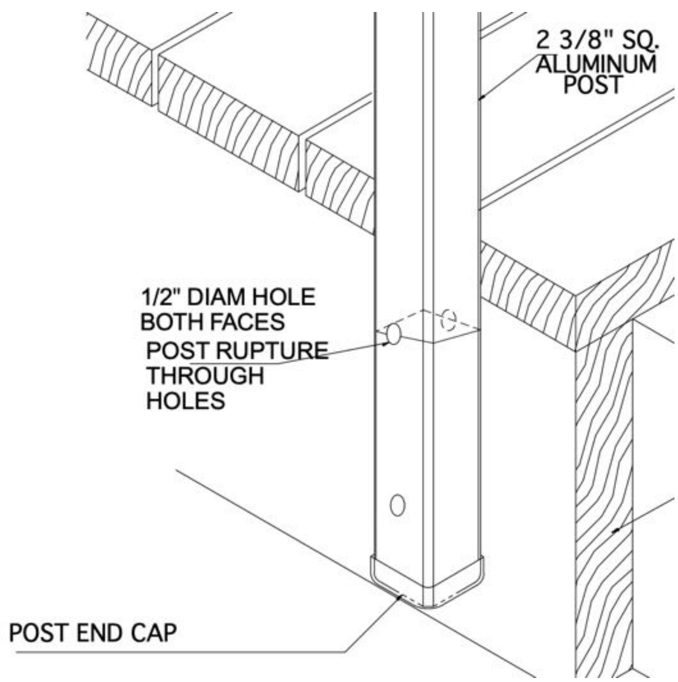
For posts directly fascia mounted with 3/8" bolts through post:
Reduced strength at bolt hole:
For loading parallel to bolt axis:
Assume 3/8" + 1/8" over size =1/2" holes both sides of post
$Z_{red} = 0.886\ in^3$
Reduced section properties for the holes.

Addition of holes at base of post only affects rupture strength as the controlling failure mode at the holes.

$M_{nu}/\Omega = ZF_u/\Omega = 0.886in^3*38ksi/1.95 = 17{,}300"\#$ (Reduced post strength for bolt holes front and back of post)

For loading perpendicular to bolt axis
$Z_{red}=0.987in^3$

$M_{nu}/\Omega = ZF_u/\Omega = 0.987in^3*38ksi/1.95 = 19{,}200"\#$ (Reduced post strength for bolt holes in sides of post)



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Next calculate moment strength about the weak axis:

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion 6 Screw post weak axis

Section Properties

Ix (in4)	0.889
Sx (in3)	0.838
Zx (in3)	0.902
Iy (in4)	0.997
J (in4)	1.209
b	1.597
t	0.1

Cw (in6)	0
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	15.97 =	*b/t*		
λ1	20.8			
λ2	33			
F/Ω (ksi)	21.2 =	21.2		*for λ<λ1*
		27.3-.291λ		*for λ1<λ<λ2*
		580/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	19122 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	0.902	
Mn/Ω (in-lbs)	17577.4359 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	660.340157 =	*See 2015 ADM F.4-9*	
λ	11.2473113 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	29.4580361 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	17853.3552 =	*Mnmb/1.65*1000*	

Strength is controlled by rupture

Ma (in-lbs)	**17577**

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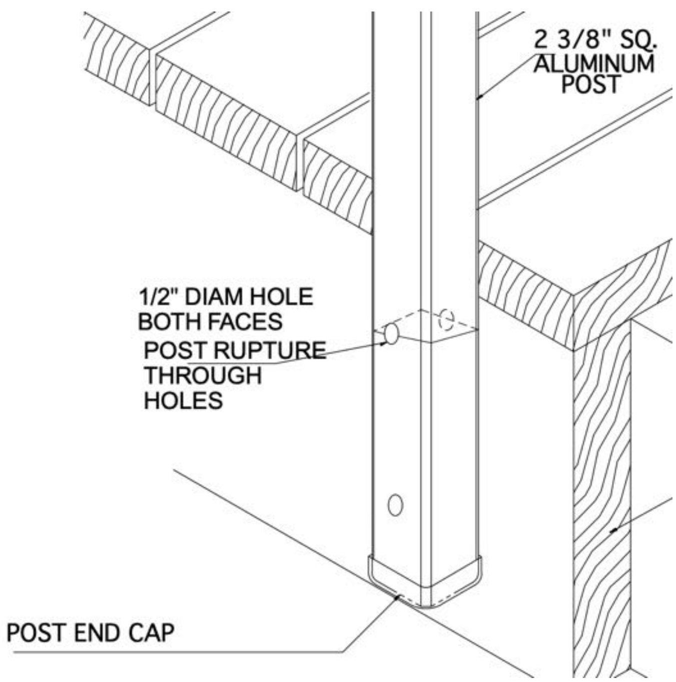
For posts directly fascia mounted with 3/8" bolts through post:
Reduced strength at bolt hole:
For loading parallel to bolt axis:
Assume 3/8" + 1/8" over size =1/2" holes both sides of post
$Z_{red} = 0.788 \text{ in}^3$
Reduced section properties for the holes

Addition of holes at base of post only affects rupture strength as the controlling failure mode at the holes.

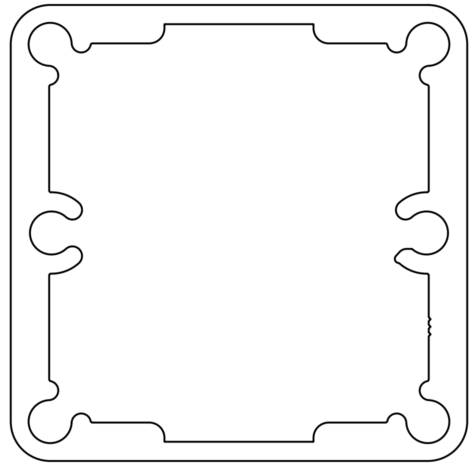
$M_{nu}/\Omega = ZF_u/\Omega = 0.788\text{in}^3 * 38\text{ksi}/1.95 = 15{,}400$"# (Reduced post strength for bolt holes front and back of post)

For loading perpendicular to bolt axis
$Z_{red}=0.889\text{in}^3$

$M_{nu}/\Omega = ZF_u/\Omega = 0.889\text{in}^3 * 38\text{ksi}/1.95 = 17{,}300$"# (Reduced post strength for bolt holes in sides of post)



Heavy Post
6061-T6 Aluminum



Heavy posts are typically used for cable rail corner and end posts that receive high cable loading. Typical installation is so that the strong axis is parallel with the top rail and cables run through the thinner portion of the wall.

First calculate moment strength about the strong axis:

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Heavy Post Strong Axis

Section Properties

Ix (in4)	1.264
Sx (in3)	1.006
Zx (in3)	1.347
Iy (in4)	1.076
J (in4)	2.34
b	1.597
t	0.1

Cw (in6)	0
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	15.97 =	*b/t*	
λ1	20.8		
λ2	33		
F/Ω (ksi)	21.2 =	21.2	*for λ<λ1*
		27.3-.291λ	*for λ1<λ<λ2*
		580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	28556 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	1.347	
Mn/Ω (in-lbs)	26249.2308 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	954.379372 =	*See 2015 ADM F.4-9*	
λ	10.2505939 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	43.3983106 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	26302.0064 =	*Mnmb/1.65*1000*

Strength is controlled by rupture

Ma (in-lbs) **26249**

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Next calculate moment strength about the weak axis:

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Heavy Post Weak Axis

Section Properties

Ix (in4)	1.076
Sx (in3)	0.8889
Zx (in3)	1.131
Iy (in4)	1.264
J (in4)	2.34
b	1.597
t	0.1

Cw (in6)	0
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	15.97 =	*b/t*	
λ1	20.8		
λ2	33		
F/Ω (ksi)	21.2 =	21.2	*for λ<λ1*
		27.3-.291λ	*for λ1<λ<λ2*
		580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	23977 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	1.131	
Mn/Ω (in-lbs)	22040 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	1034.39984 =	*See 2015 ADM F.4-9*	
λ	9.2553488 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

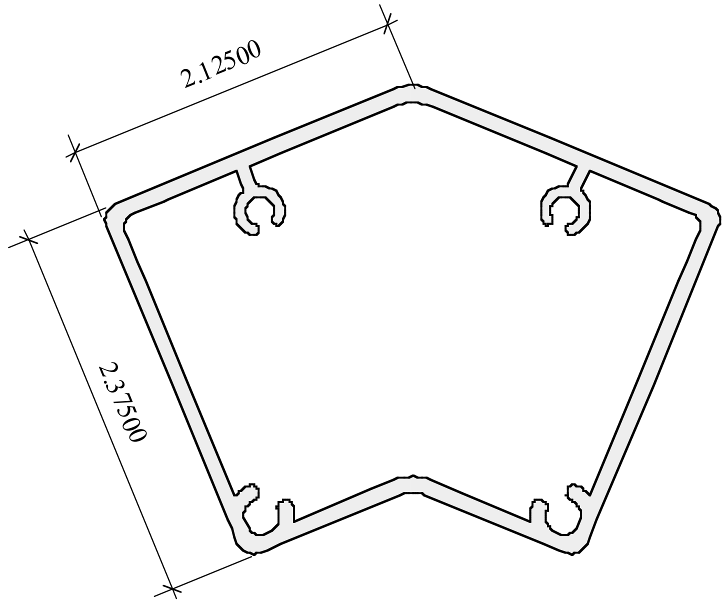
Mnmb (in-kip	36.8864557 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	22355.4277 =	*Mnmb/1.65*1000*

Strength is controlled by rupture

Ma (in-lbs) **22040**

CRL 135˚ Post – Corner post

First calculate moment strength about the strong axis which would be a vertical line in the image to the right.



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	135˚ Corner Post Strong Axis

Section Properties

Ix (in4)	1.819
Sx (in3)	0.9352
Zx (in3)	1.392
Iy (in4)	1.218
J (in4)	1.951
b	1.883
t	0.1

Cw (in6)	0.0342
βx (in)	-0.0157
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	18.83	=	*b/t*	
λ1	20.8			
λ2	33			
F/Ω (ksi)	21.2	=	21.2	*for λ<λ1*
			27.3-.291λ	*for λ1<λ<λ2*
			580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	29527	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795		
Znet	1.392		
Mn/Ω (in-lbs)	27126.1538	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5		**C2**	0.5
U (in)	0.003925	=	*C1*g0-C2*βx/2*	
Me	927.642248	=	*See 2015 ADM F.4-9*	
λ	10.0247269	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	44.5705501	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	27012.4546	=	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **27012**

Next calculate moment strength about the weak axis:

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion 135˚ Corner Post Weak Axis

Section Properties

Ix (in4)	1.218
Sx (in3)	0.8123
Zx (in3)	1.141
Iy (in4)	1.819
J (in4)	1.951
b	1.883
t	0.1

Cw (in6)	0.0342
βx (in)	-0.2282
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	18.83 =	*b/t*	
λ1	20.8		
λ2	33		
F/Ω (ksi)	21.2 =	21.2	*for λ<λ1*
		27.3-.291λ	*for λ1<λ<λ2*
		580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	24203 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795	
Znet	1.141	
Mn/Ω (in-lbs)	22234.8718 =	*Znet*Fu/Ω*1000kips/lbs*

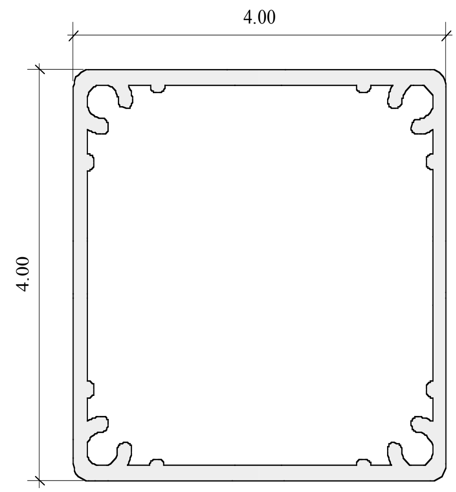
Lateral Torsional Buckling:

Lb (in)	42		
Cb	1.3		
C1	0.5	**C2**	0.5
U (in)	0.05705 =	*C1*g0-C2*βx/2*	
Me	1140.85482 =	*See 2015 ADM F.4-9*	
λ	8.42468353 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	37.2102185 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	22551.6476 =	*Mnmb/1.65*1000*	

Strength is controlled by rupture

Ma (in-lbs) **22235**

CRL Standard 4”x4” Square Post Strength 6005A-T61 or 6061-T6



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion 4" Square Post

Section Properties

Ix (in4)	5.48
Sx (in3)	2.52
Zx (in3)	3.2
Iy (in4)	5.48
J (in4)	7.2
b	3.15
t	0.12

Cw (in6)	0.0296
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	26.25 =	*b/t*		
λ1	20.8			
λ2	33			
F/Ω (ksi)	19.66125 =	21.2		*for λ<λ1*
		27.3-.291λ		*for λ1<λ<λ2*
		580/λ		*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	49546 =	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	19.4871795	
Znet	3.2	
Mn/Ω (in-lbs)	62358.9744 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5	**C2**	0.5	
U (in)	0 =	*C1*g0-C2*βx/2*		
Me	3778.12936 =	*See 2015 ADM F.4-9*		
λ	8.15403212 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*		

λ<Cc, inelastic buckling applies

Mnmb (in-kip	105.287484 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	63810.5963 =	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) **49546**

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Aluminum Extrusion Design

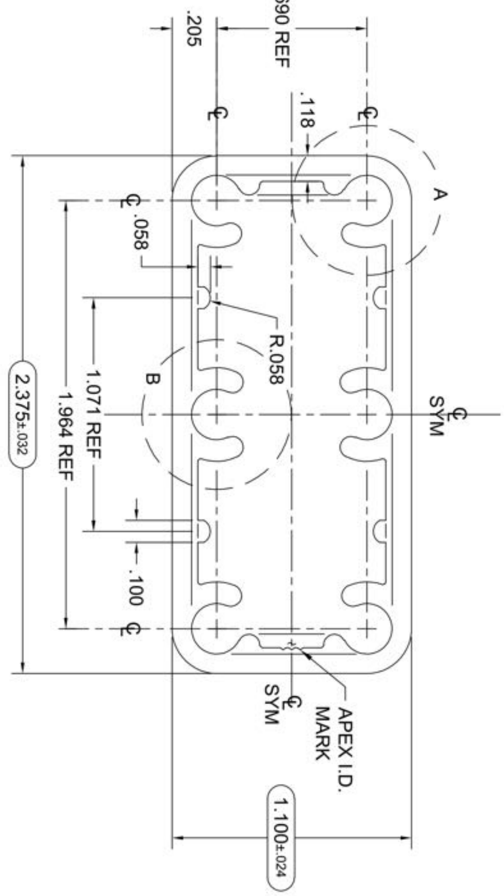
Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Trim Line Strong Axis

Section Properties

Ix (in4)	0.524
Sx (in3)	0.44
Zx (in3)	0.567
Iy (in4)	0.144
J (in4)	0.316
b	0.696
t	0.118

Cw (in6)	0.012
βx (in)	0
g0 (in)	0



Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	5.89830508	=	*b/t*	
λ1	20.8			
λ2	33			
F/Ω (ksi)	21.2	=	21.2	*for λ<λ1*
			27.3-.291λ	*for λ1<λ<λ2*
			580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	12027	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795		
Znet	0.567		
Mn/Ω (in-lbs)	11049.2308	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5		C2	0.5
U (in)	0	=	*C1*g0-C2*βx/2*	
Me	128.337847	=	*See 2015 ADM F.4-9*	
λ	18.4867055	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies				
Mnmb (in-kip	17.1067241	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	10367.7116	=	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **10368**

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Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Trim Line Weak Axis

Section Properties

Ix (in4)	0.144
Sx (in3)	0.44
Zx (in3)	0.31
Iy (in4)	0.524
J (in4)	0.316
b	1.55
t	0.118

Cw (in6)	0.012
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6061-T6
Fu (ksi)	38
Fy (ksi)	35
E (ksi)	10100
Cc	66

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	13.1355932	=	*b/t*	
λ1	20.8			
λ2	33			
F/Ω (ksi)	21.2	=	21.2	*for λ<λ1*
			27.3-.291λ	*for λ1<λ<λ2*
			580/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	6576	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	19.4871795		
Znet	0.31		
Mn/Ω (in-lbs)	6041.02564	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5		**C2**	0.5
U (in)	0	=	*C1*g0-C2*βx/2*	
Me	244.815633	=	*See 2015 ADM F.4-9*	
λ	13.3849676	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	10.6916093	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	6479.76319	=	*Mnmb/1.65*1000*

Strength is controlled by rupture

Ma (in-lbs) **6041**

TRIM LINE STANCHION

The trim line posts fit a 7/16"x2" flat bar stanchion.
$S = 0.292in^3$
$Z = 0.44in^3$

When used to reinforce post:
Yield limit state:
$M_a = 0.44*35ksi/1.65 = 9{,}330"\#$ (For 6061-T6 or 6005-T61 aluminum)
$M_a = 0.44*30ksi/1.67 = 7{,}900"\#$ (For 304 or 316 stainless)

When used as a stanchion where distance between the bottom of the post and the bottom of the stanchion is less than 1".
Rupture limit state:
$M_a = 0.44in^3*38ksi/1.95 = 8{,}570"\#$ (For 6061-T6 or 6005-T61 aluminum)
$M_a = 0.292in^3*70ksi/2 = 10{,}200"\#$ (For 304 or 316 stainless)

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POST ANCHORAGE

Posts can be attached to wood, concrete or steel using several standard connection details. Note that in most cases the post anchorage strength will be less than the post strength and will be the limiting strength. First, basic performance criteria for each anchorage detail are listed below. Then, more specific performance criteria is shown with the supporting calculations on the following pages. Lastly, detailed moment strength calculations are shown for each detail.

Basic Performance Criteria:
Note that many anchorage methods involve two connections. For instance a surface mount to concrete anchorage detail involves screwing the post to the baseplate and then anchoring the baseplate to the concrete. The performance criteria for both connections must be checked and the overall performance criteria is the more restrictive of the two.

Post to baseplate connections:
4 screw 2-3/8" square post screwed to baseplate (includes 135° post)
Allowable moment, $M_{a,x}$ = 10,500"#
Handles top rail live loading at 60" tall with 42" post spacing or 42" tall with 60" post spacing.

6 screw 2-3/8" square post screwed to baseplate (includes heavy post)
Allowable post moment, $M_{a,x}$ = 15,700"#
Handles top rail live loading at 60" tall with 60" post spacing or at 48" tall with 72" post spacing.

135° post at corner mixed with 6 screw square posts at intermediates
Allowable post moment, $M_{a,x}$ = 12,600"#
Handles top rail live loading at 60" tall with 60" post spacing or at 48" tall with 72" post spacing.

4" square post screwed to baseplate
Allowable post moment, $M_{a,x}$ = 17,300"#
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

Aluminum stanchion screwed to baseplate
Allowable post moment, $M_{a,x}$ = 12,400"#
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

Aluminum stanchion welded to baseplate
Allowable post moment, $M_{a,x}$ = 10,500"#
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

Steel stanchion welded to baseplate
Allowable post moment, $M_{a,x}$ = 13,600"#
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

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<u>Baseplate Anchorage Details:</u>

3/8”x4” KH-EZ w/ 4-3/16” anchor edge distance to uncracked concrete
Allowable moment, $M_{a,x}$ = 13,500”#
Handles top rail live loading at 48” tall with 66” post spacing or 60” tall with 54” post spacing.

3/8”x4” KH-EZ w/ 4-3/16” anchor edge distance to cracked concrete
Allowable moment, $M_{a,x}$ = 9,600”#
Handles top rail live loading at 42” tall with 54” post spacing or 48” tall with 48” post spacing.

3/8”x3-3/4” KB-TZ w/ 2-5/8” anchor edge distance to uncracked concrete
Allowable moment, $M_{a,x}$ = 14,200”#
Handles top rail live loading at 42” tall with 72” post spacing or 60” tall with 54” post spacing.

3/8”x3-3/4” KB-TZ w/ 2-5/8” anchor edge distance to cracked concrete
Allowable moment, $M_{a,x}$ = 11,000”#
Handles top rail live loading at 42” tall with 60” post spacing or 60” tall with 42” post spacing.

3/8” A307 or 304 Lag Screw w/ 4-1/4” penetration
Optimal lag screw penetration.
Allowable moment, $M_{a,x}$ = 11,400”#
Handles top rail live loading at 42” tall with 60” post spacing or 60” tall with 42” post spacing.
Higher strength connections will require higher strength material or larger diameter.

3/8” A307 or 304 Lag Screw w/ 3-1/2” penetration
Allowable moment, $M_{a,x}$ = 9,860”#
Handles top rail live loading at 42” tall with 54” post spacing or 48” tall with 48” post spacing.

3/8” A307 or 304 Lag Screw w/ 3” penetration
Allowable moment, $M_{a,x}$ = 8,700”#
Handles top rail live loading at 42” tall with 48” post spacing or 48” tall with 42” post spacing.

<u>Core Mount Details</u>

Post set in 4” Deep Core Mount, 3-5/8” edge distance measured from center of post
Allowable moment, $M_{a,x}$ = 12,600”#
Handles top rail live loading at 42” tall with 72” post spacing or 60” tall with 48” post spacing.

Stanchion set in 4” Deep Core Mount, 3-5/8” edge distance measured from center of post
Allowable moment, $M_{a,x}$ = 11,400”#
Handles top rail live loading at 42” tall with 60” post spacing or 60” tall with 42” post spacing.

<u>Fascia Mount Details</u>

Fascia Bracket To Wood, 3-3/8” lag Screw Penetration
Top lag screws located 2” below floor.
Allowable moment, $M_{a,x}$ = 10,600”# (measured at floor)

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Handles top rail live loading at 42” tall with 60” post spacing or 60” tall with 42” post spacing.

Fascia Bracket to Concrete, Uncracked Concrete
Allowable moment, $M_{a,x}$ = 11,300”# (measured at floor)
Handles top rail live loading at 42” tall with 60” post spacing or 60” tall with 42” post spacing.

Fascia Bracket to Concrete, Cracked Concrete
Allowable moment, $M_{a,x}$ = 8,000”# (measured at floor)
Handles top rail live loading at 42” tall with 48” post spacing.

Post Directly Fascia Mounted W/ 3/8” Lag Screws
Allowable moment, $M_{a,x}$ = 7,800”# (measured at floor)
Handles top rail live loading at 42” tall with 48” post spacing.

Post Directly Fascia Mounted W/ 3/8” Carriage Bolts
Allowable moment, $M_{a,x}$ = 17,400”# (measured at floor)
Handles top rail live loading at 60” tall with 66” post spacing or 54” tall with 72” post spacing.

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Post Anchorage Design

System: ARS
Detail Description:
4 screw 2-3/8" post screwed to baseplate

Ma (in-lbs) 10500

Load Cases:

200# concentrated load at top of post

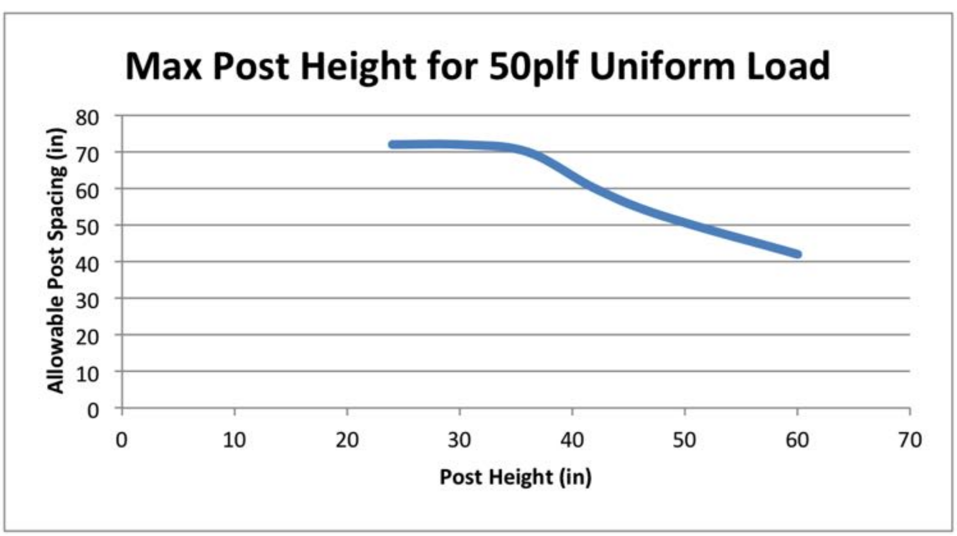
*M = 200#*H*
Hmax = Ma/200#
Hmax 52.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70
42	60
48	52.5
60	42
72	<36"
84	<36"
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

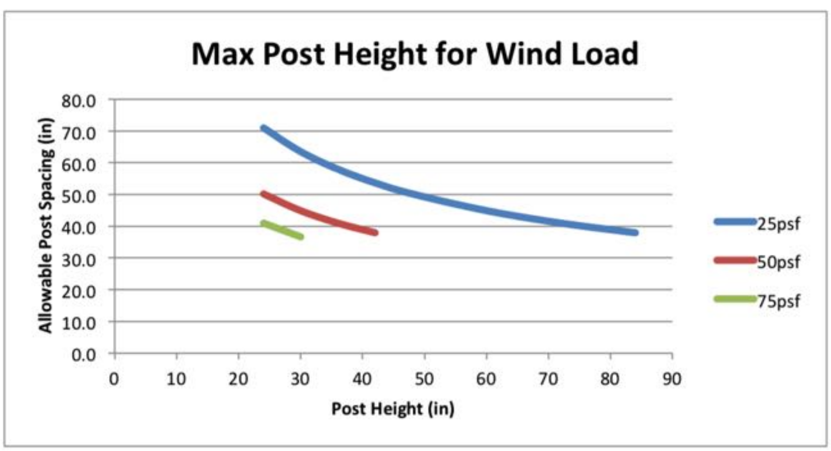
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	71.0
30	63.5
36	58.0
42	53.7
48	50.2
60	44.9
72	41.0
84	37.9
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	50.2
30	44.9
36	41.0
42	37.9
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	41.0
30	36.7
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
6 screw 2-3/8" post screwed to baseplate

Ma (in-lbs) 15700

Load Cases:

200# concentrated load at top of post

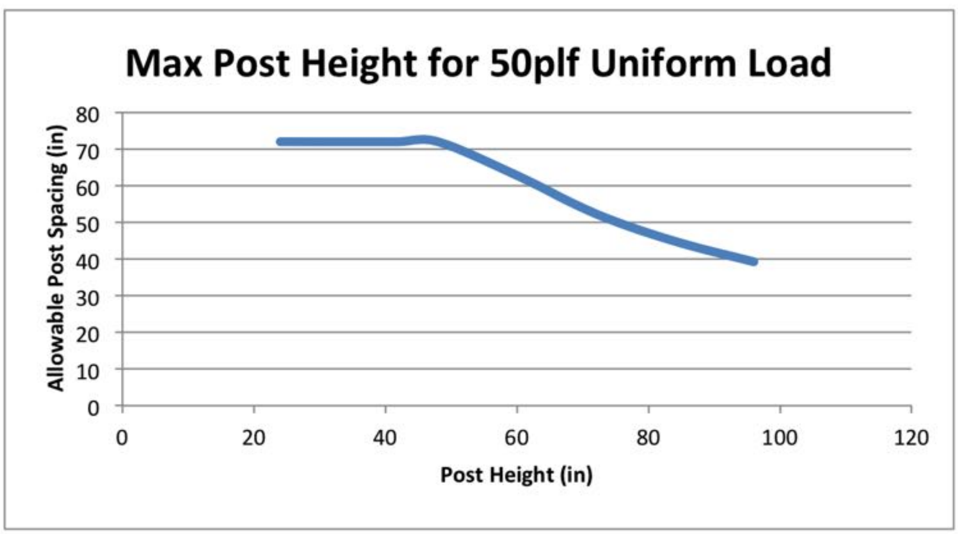
*M = 200#*H*
Hmax = Ma/200#
Hmax 78.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	62.8
72	52.33333333
84	44.85714286
96	39.25



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Wind Load

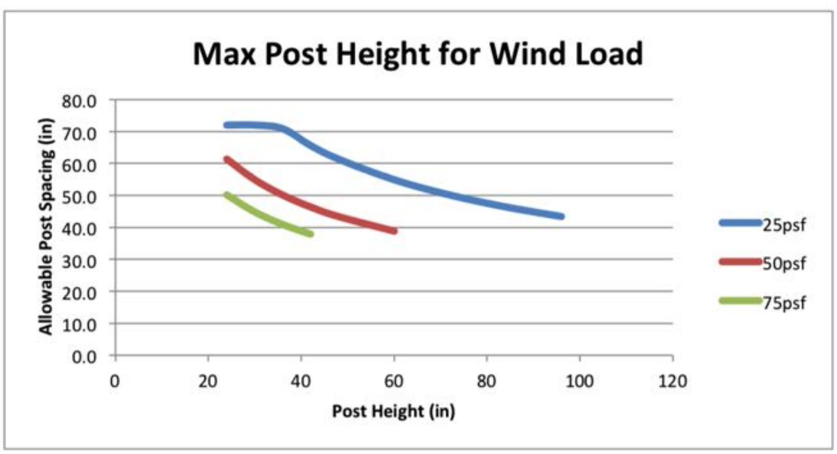
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	70.9
42	65.6
48	61.4
60	54.9
72	50.1
84	46.4
96	43.4

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	61.4
30	54.9
36	50.1
42	46.4
48	43.4
60	38.8
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	50.1
30	44.8
36	40.9
42	37.9
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
135˚ post at corners mixed with 6 screw intermediate posts

Ma (in-lbs) 12600

Load Cases:

200# concentrated load at top of post

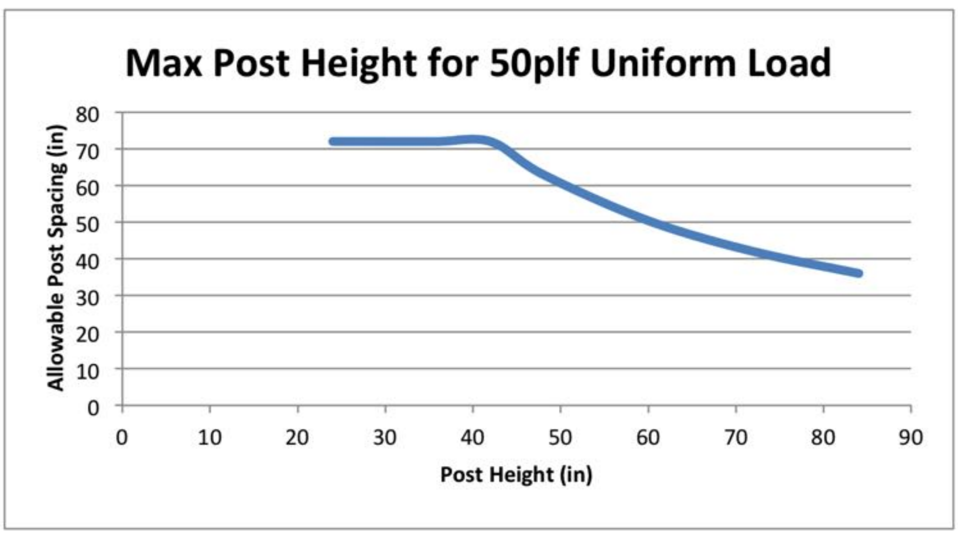
*M = 200#*H*
Hmax = Ma/200#
Hmax 63

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63
60	50.4
72	42
84	36
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

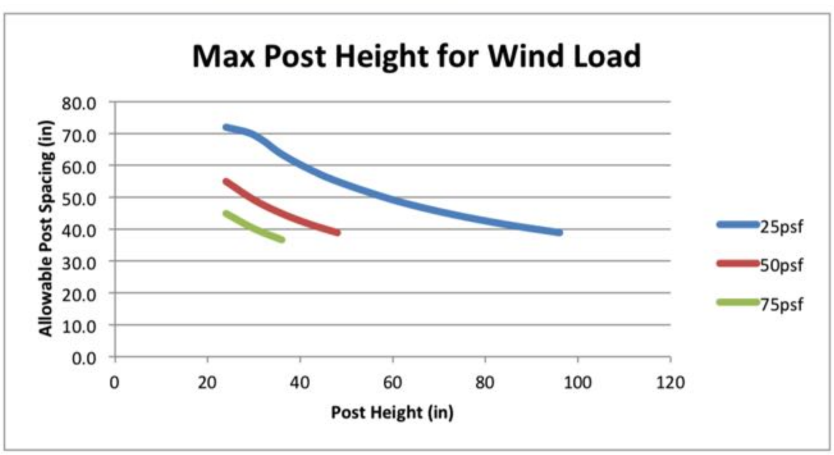
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P=	25psf
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	69.6
36	63.5
42	58.8
48	55.0
60	49.2
72	44.9
84	41.6
96	38.9

P=	50psf
Post Height (in)	Allowable Post Spacing (in)
24	55.0
30	49.2
36	44.9
42	41.6
48	38.9
60	<36"
72	<36"
84	<36"
96	<36"

P=	75psf
Post Height (in)	Allowable Post Spacing (in)
24	44.9
30	40.2
36	36.7
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
4" post screwed to post plate

Ma (in-lbs) 17300

Load Cases:

200# concentrated load at top of post

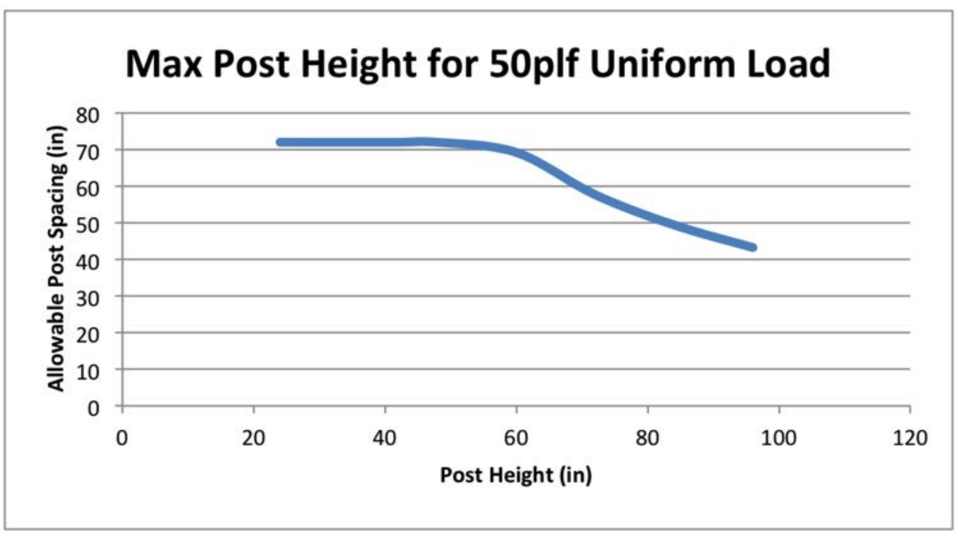
*M = 200#*H*
Hmax = Ma/200#
Hmax 86.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	69.2
72	57.66666667
84	49.42857143
96	43.25



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Wind Load

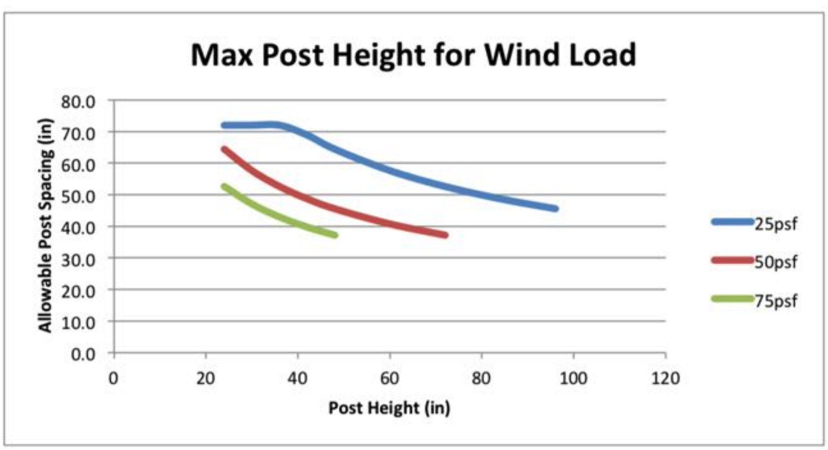
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	72.0
42	68.9
48	64.4
60	57.6
72	52.6
84	48.7
96	45.6

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	64.4
30	57.6
36	52.6
42	48.7
48	45.6
60	40.8
72	37.2
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.6
30	47.1
36	43.0
42	39.8
48	37.2
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Aluminum Stanchion Screwed to Baseplate

Ma (in-lbs) 12400

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 62

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	70.85714286
48	62
60	49.6
72	41.33333333
84	<36"
96	<36"

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Wind Load

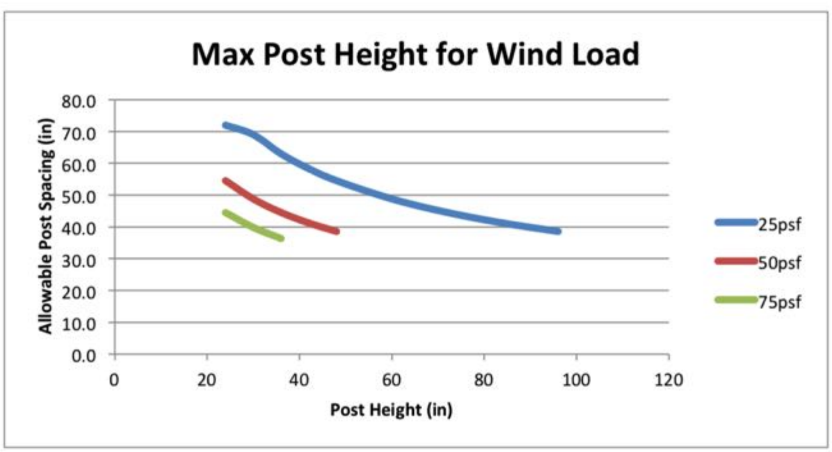
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	69.0
36	63.0
42	58.3
48	54.6
60	48.8
72	44.5
84	41.2
96	38.6

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	54.6
30	48.8
36	44.5
42	41.2
48	38.6
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	44.5
30	39.8
36	36.4
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS

Detail Description:

Aluminum Stanchion Welded to Baseplate

Ma (in-lbs) 10500

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

Hmax = Ma/200#

Hmax 52.5

50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70
42	60
48	52.5
60	42
72	<36"
84	<36"
96	<36"

Wind Load

*M=P/144*TW*H^2/2*

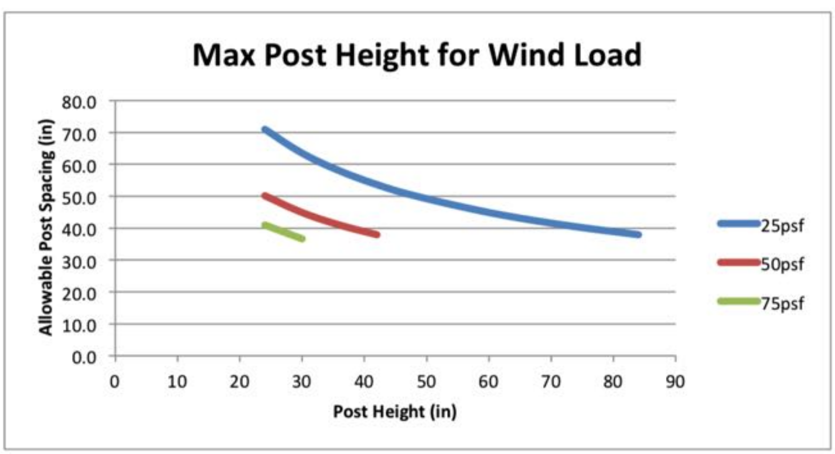
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	71.0
30	63.5
36	58.0
42	53.7
48	50.2
60	44.9
72	41.0
84	37.9
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	50.2
30	44.9
36	41.0
42	37.9
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	41.0
30	36.7
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Steel stanchion welded to baseplate

Ma (in-lbs) 13600

Load Cases:

200# concentrated load at top of post

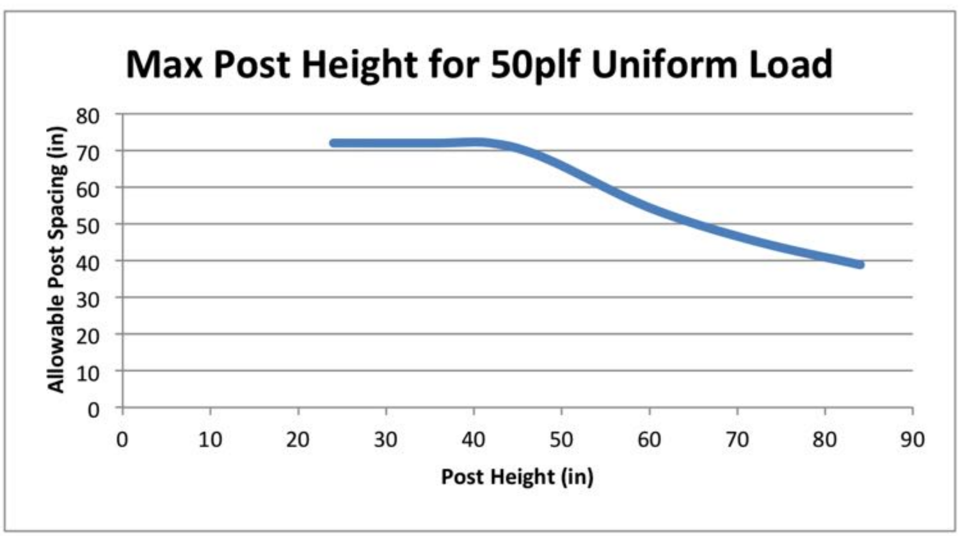
*M = 200#*H*
Hmax = Ma/200#
Hmax 68

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	68
60	54.4
72	45.33333333
84	38.85714286
96	<36"



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Wind Load

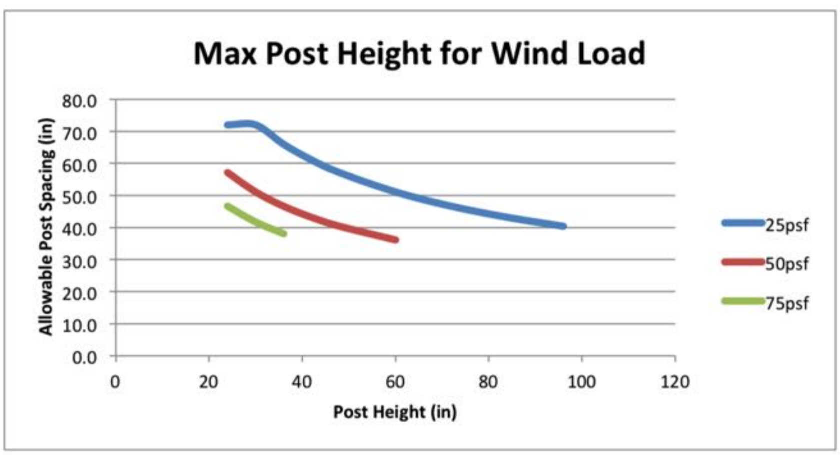
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	66.0
42	61.1
48	57.1
60	51.1
72	46.6
84	43.2
96	40.4

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	57.1
30	51.1
36	46.6
42	43.2
48	40.4
60	36.1
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	46.6
30	41.7
36	38.1
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in uncracked concrete and 5x5 baseplate

Ma (in-lbs) 13500

Load Cases:

200# concentrated load at top of post

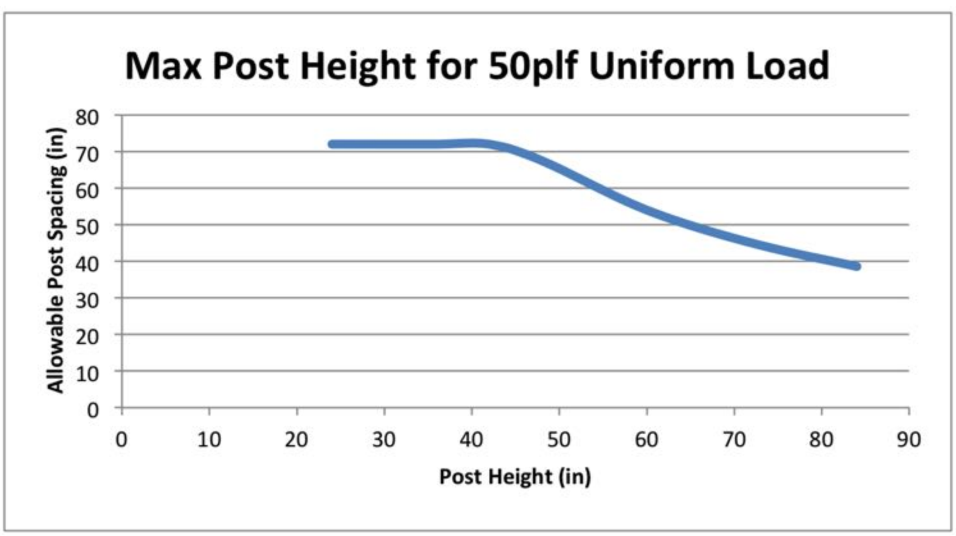
*M = 200#*H*
Hmax = Ma/200#
Hmax 67.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	67.5
60	54
72	45
84	38.57142857
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

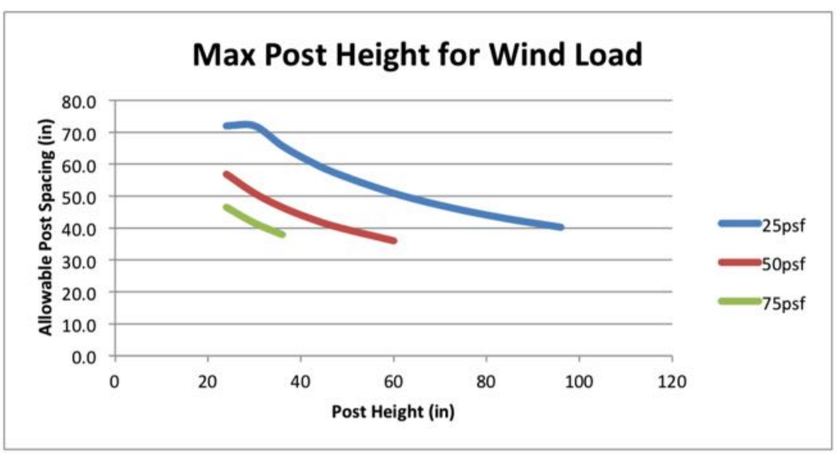
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P=	25psf
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	65.7
42	60.9
48	56.9
60	50.9
72	46.5
84	43.0
96	40.2

P=	50psf
Post Height (in)	Allowable Post Spacing (in)
24	56.9
30	50.9
36	46.5
42	43.0
48	40.2
60	36.0
72	<36"
84	<36"
96	<36"

P=	75psf
Post Height (in)	Allowable Post Spacing (in)
24	46.5
30	41.6
36	37.9
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in uncracked concrete and 3x5 baseplate

Ma (in-lbs) 7130

Load Cases:

200# concentrated load at top of post

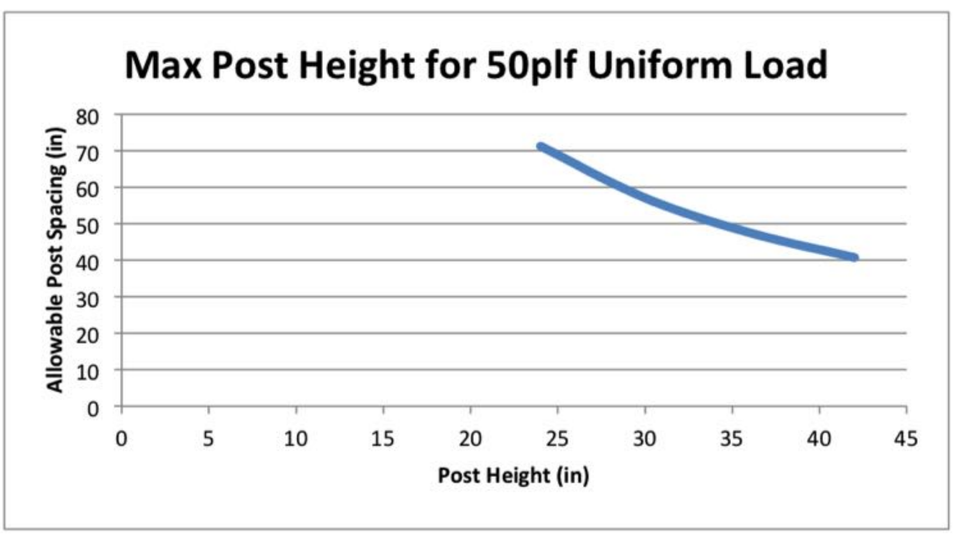
*M = 200#*H*
Hmax = Ma/200#
Hmax 35.65

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	71.3
30	57.04
36	47.53333333
42	40.74285714
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

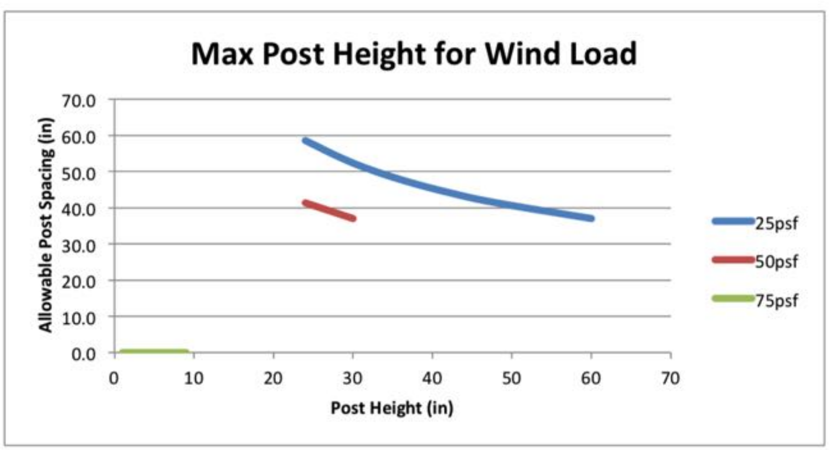
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	58.5
30	52.3
36	47.8
42	44.2
48	41.4
60	37.0
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	41.4
30	37.0
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in uncracked concrete and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 17800

Load Cases:

200# concentrated load at top of post

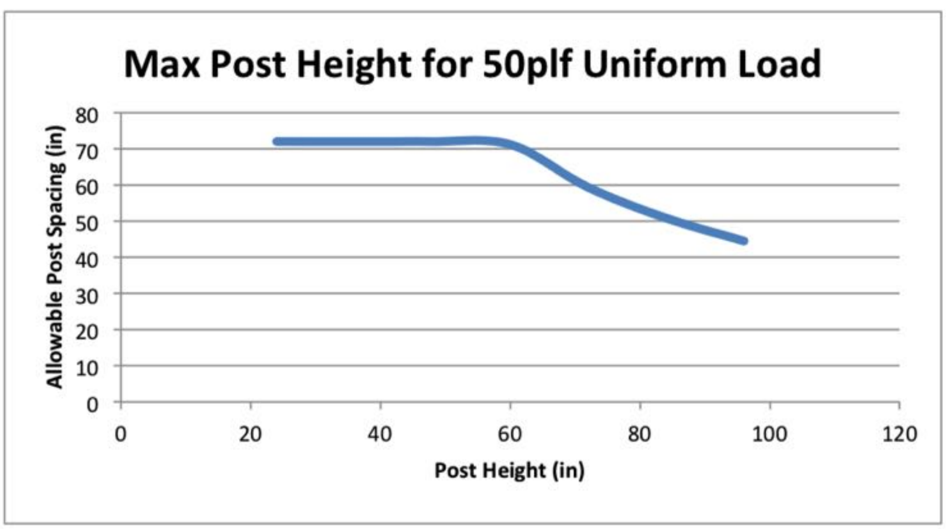
*M = 200#*H*
Hmax = Ma/200#
Hmax 89

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	71.2
72	59.33333333
84	50.85714286
96	44.5



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Wind Load

*M=P/144*TW*H^2/2*

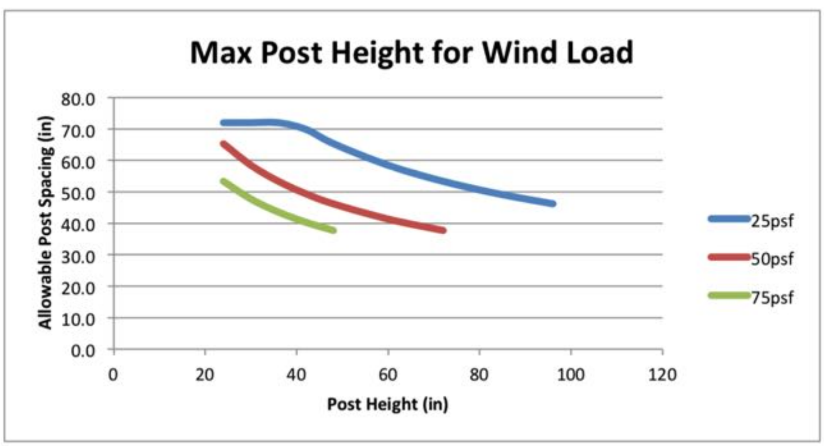
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	72.0
36	72.0
42	69.9
48	65.4
60	58.5
72	53.4
84	49.4
96	46.2

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	65.4
30	58.5
36	53.4
42	49.4
48	46.2
60	41.3
72	37.7
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	53.4
30	47.7
36	43.6
42	40.3
48	37.7
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in cracked concrete and 5x5" baseplate

Ma (in-lbs) 9600

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 48

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	64
42	54.85714286
48	48
60	38.4
72	<36"
84	<36"
96	<36"

Wind Load

*M=P/144*TW*H^2/2*

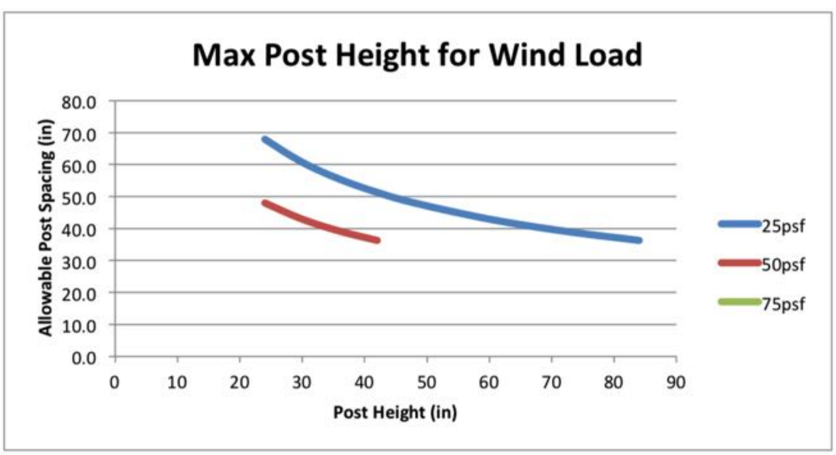
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	67.9
30	60.7
36	55.4
42	51.3
48	48.0
60	42.9
72	39.2
84	36.3
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	48.0
30	42.9
36	39.2
42	36.3
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	39.2
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in cracked concrete and 3x5" baseplate

Ma (in-lbs) 5120

Load Cases:

200# concentrated load at top of post

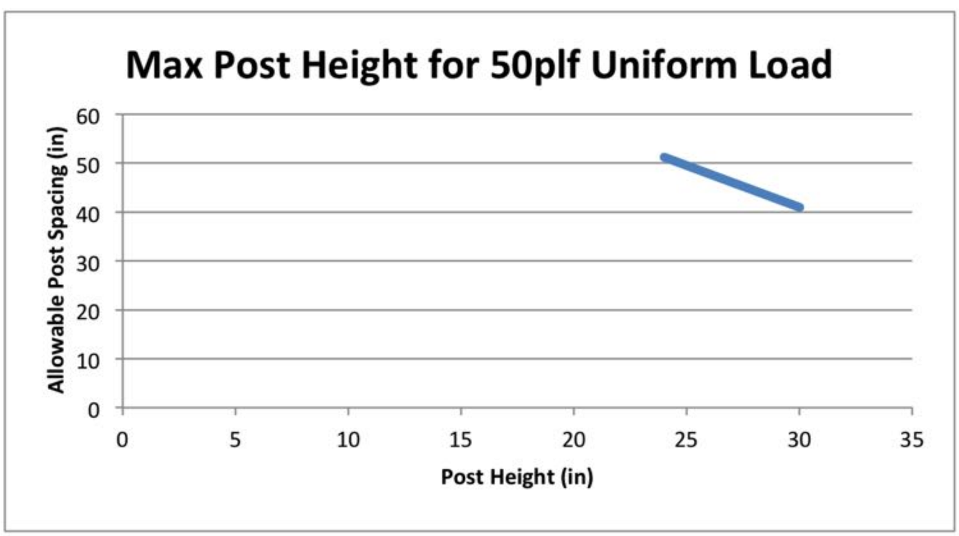
*M = 200#*H*
Hmax = Ma/200#
Hmax 25.6

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	51.2
30	40.96
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Wind Load

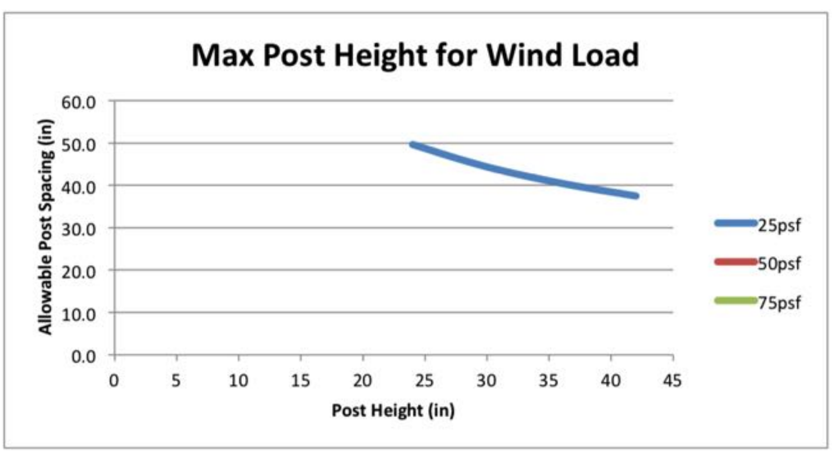
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	49.6
30	44.3
36	40.5
42	37.5
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8"x4" KH-EZ in cracked concrete and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 12700

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 63.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63.5
60	50.8
72	42.33333333
84	36.28571429
96	<36"

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Wind Load

M=P/144*TW*H^2/2

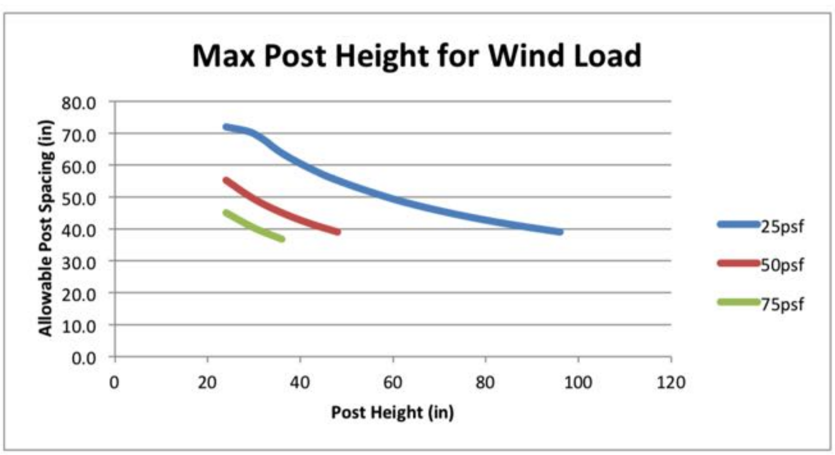
Hmax = (2*Ma/(TW*P/144))^(1/2)

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	69.8
36	63.7
42	59.0
48	55.2
60	49.4
72	45.1
84	41.7
96	39.0

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	55.2
30	49.4
36	45.1
42	41.7
48	39.0
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	45.1
30	40.3
36	36.8
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8"x3-3/4" KB-TZ in uncracked concrete and 5x5" baseplate

Ma (in-lbs) 14200

Load Cases:

200# concentrated load at top of post

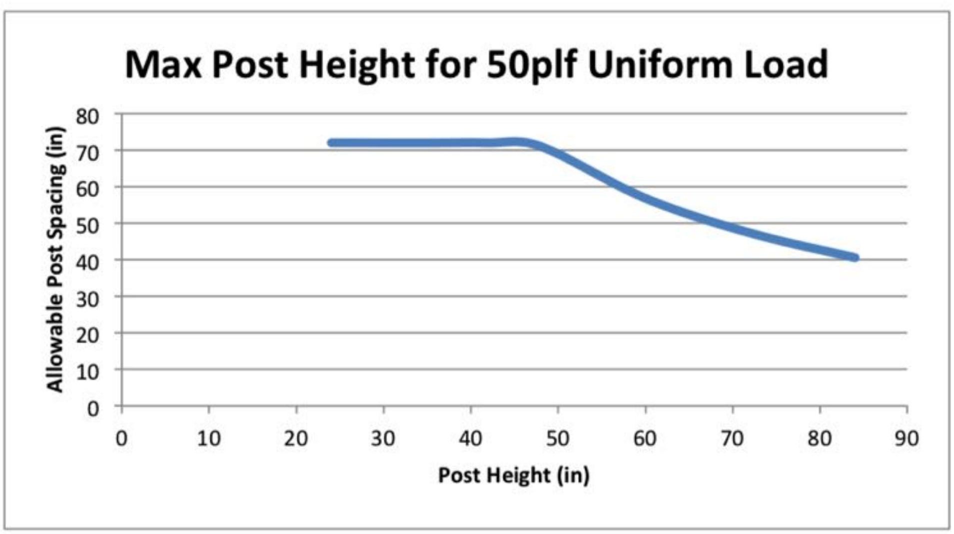
*M = 200#*H*
Hmax = Ma/200#
Hmax 71

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	71
60	56.8
72	47.33333333
84	40.57142857
96	<36"



Wind Load

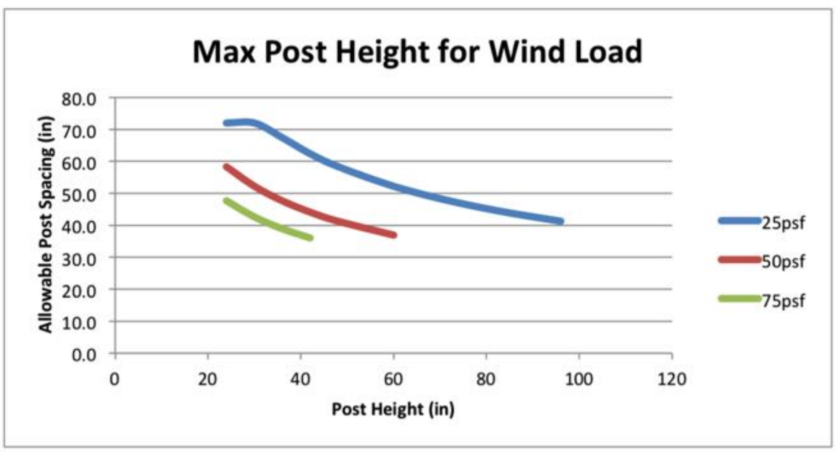
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	67.4
42	62.4
48	58.4
60	52.2
72	47.7
84	44.1
96	41.3

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	58.4
30	52.2
36	47.7
42	44.1
48	41.3
60	36.9
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	47.7
30	42.6
36	38.9
42	36.0
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8"x3-3/4" KB-TZ in uncracked concrete and 3x5" baseplate

Ma (in-lbs) 7490

Load Cases:

200# concentrated load at top of post

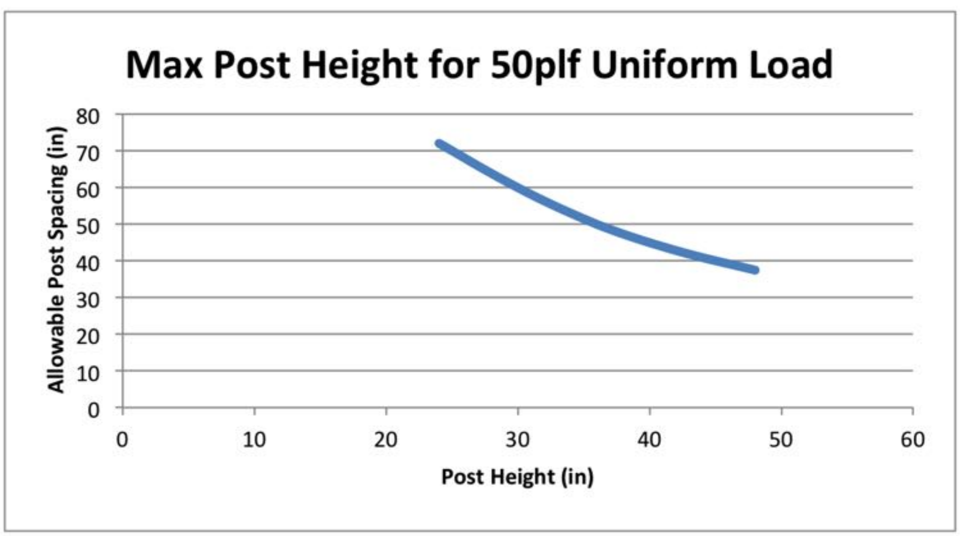
*M = 200#*H*
Hmax = Ma/200#
Hmax 37.45

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	59.92
36	49.93333333
42	42.8
48	37.45
60	<36"
72	<36"
84	<36"
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

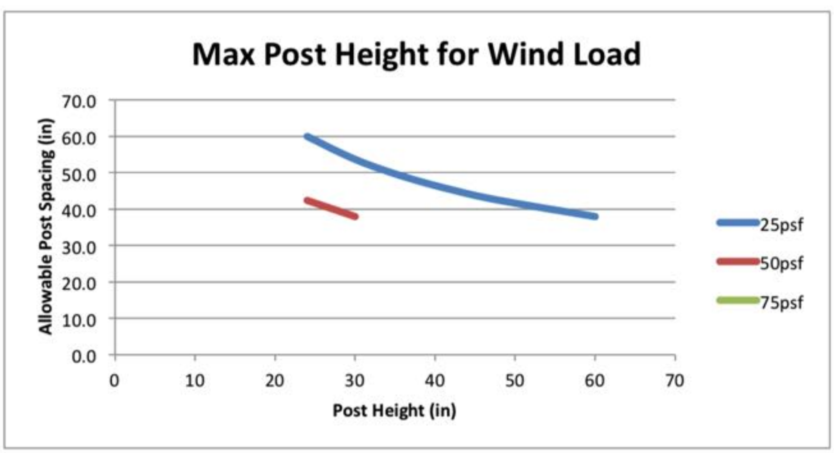
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	60.0
30	53.6
36	49.0
42	45.3
48	42.4
60	37.9
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	42.4
30	37.9
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS

Detail Description:

3/8"x3-3/4" KB-TZ in uncracked concrete and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 18800

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

Hmax = Ma/200#

Hmax 94

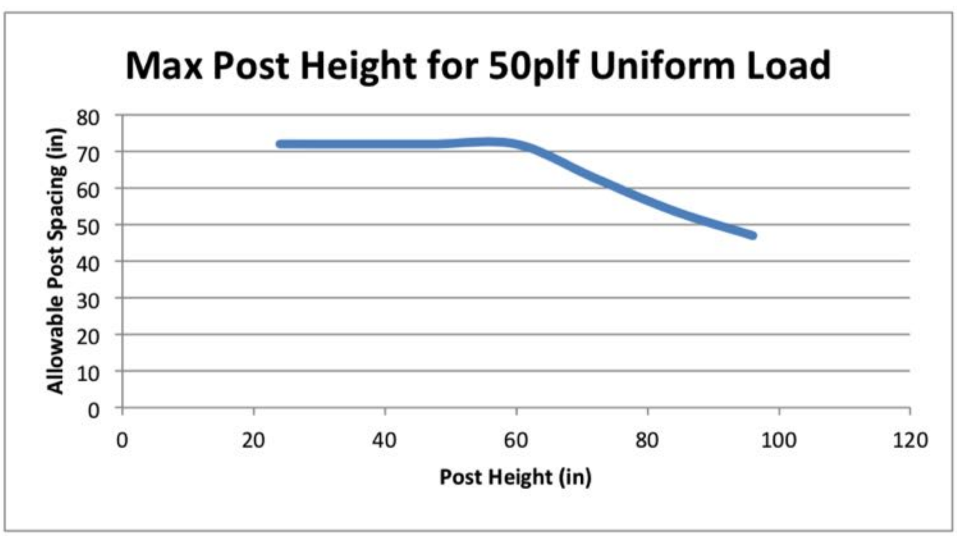
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	72
72	62.66666667
84	53.71428571
96	47



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Wind Load

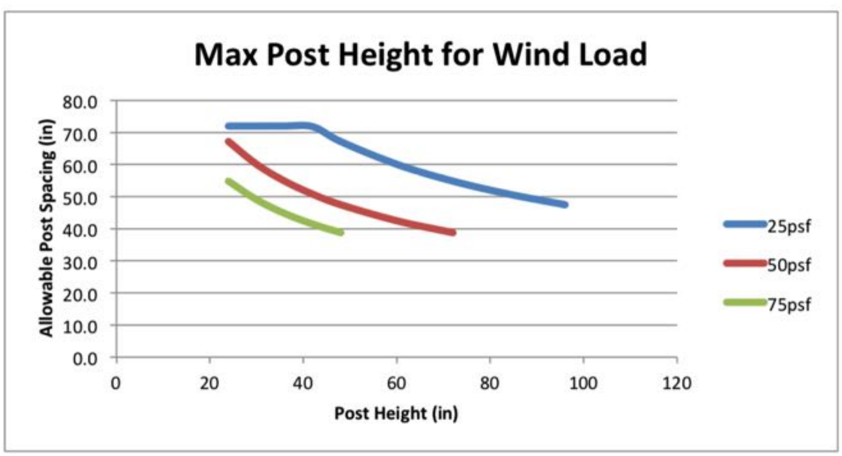
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	72.0
42	71.8
48	67.2
60	60.1
72	54.8
84	50.8
96	47.5

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	67.2
30	60.1
36	54.8
42	50.8
48	47.5
60	42.5
72	38.8
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	54.8
30	49.1
36	44.8
42	41.5
48	38.8
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8"x3-3/4" KB-TZ in cracked concrete and 5x5" baseplate

Ma (in-lbs) 11000

Load Cases:

200# concentrated load at top of post

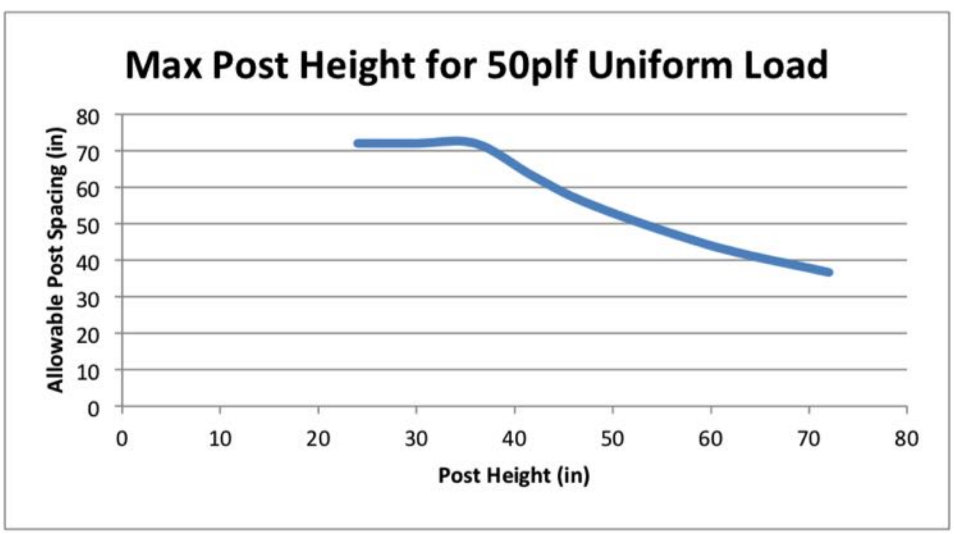
*M = 200#*H*
Hmax = Ma/200#
Hmax 55

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	62.85714286
48	55
60	44
72	36.66666667
84	<36"
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

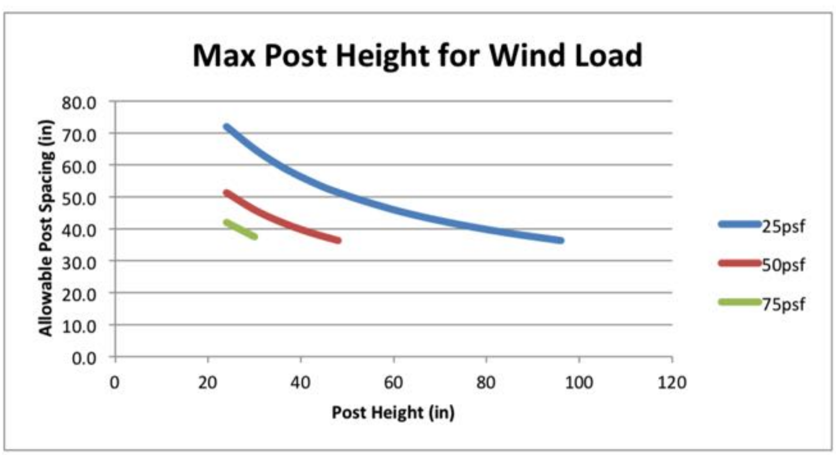
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	65.0
36	59.3
42	54.9
48	51.4
60	46.0
72	42.0
84	38.8
96	36.3

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	51.4
30	46.0
36	42.0
42	38.8
48	36.3
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	42.0
30	37.5
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS

Detail Description:

3/8"x3-3/4" KB-TZ in cracked concrete and 3x5" baseplate

Ma (in-lbs) 5840

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

Hmax = Ma/200#

Hmax 29.2

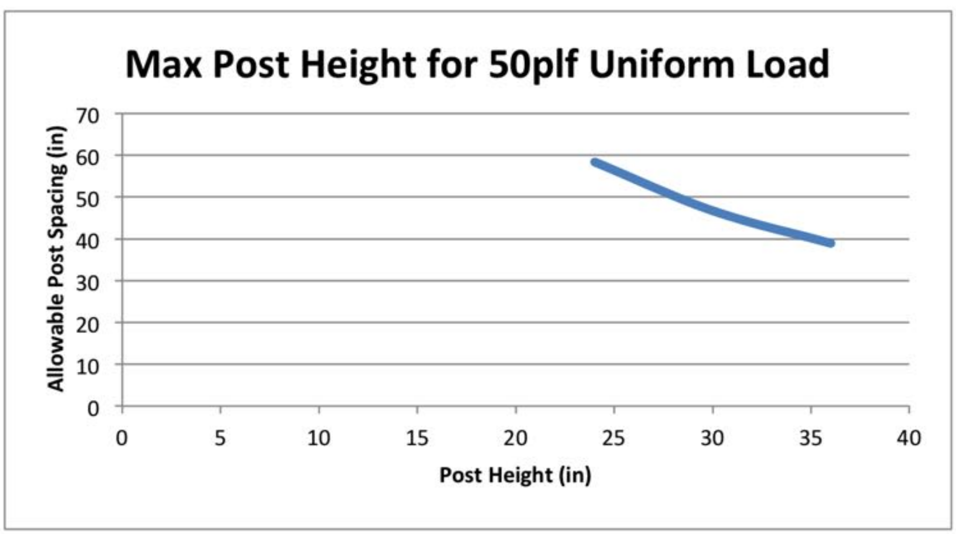
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	58.4
30	46.72
36	38.93333333
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Wind Load

*M=P/144*TW*H^2/2*

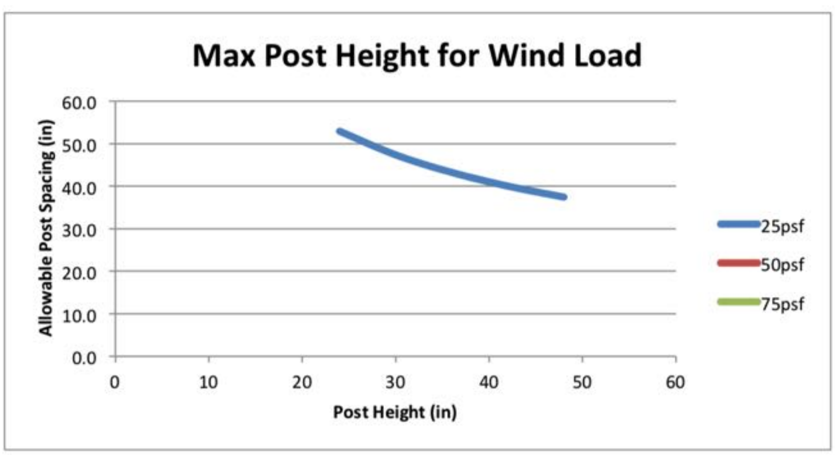
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.9
30	47.4
36	43.2
42	40.0
48	37.4
60	<36"
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	37.4
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8"x3-3/4" KB-TZ in cracked concrete and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 14500

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 72.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	58
72	48.33333333
84	41.42857143
96	36.25

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Wind Load

*M=P/144*TW*H^2/2*

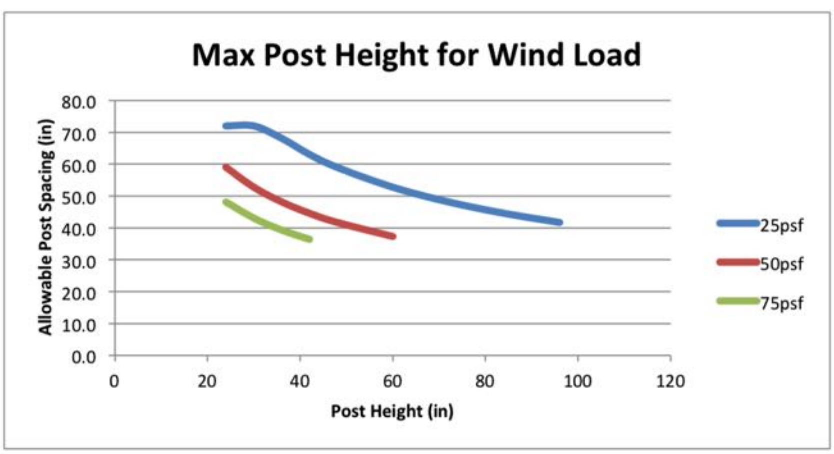
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	72.0
36	68.1
42	63.1
48	59.0
60	52.8
72	48.2
84	44.6
96	41.7

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	59.0
30	52.8
36	48.2
42	44.6
48	41.7
60	37.3
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	48.2
30	43.1
36	39.3
42	36.4
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 4-1/4" penetration and 5x5 baseplate

Ma (in-lbs) 11400

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 57

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	65.14285714
48	57
60	45.6
72	38
84	<36"
96	<36"

Wind Load

*M=P/144*TW*H^2/2*

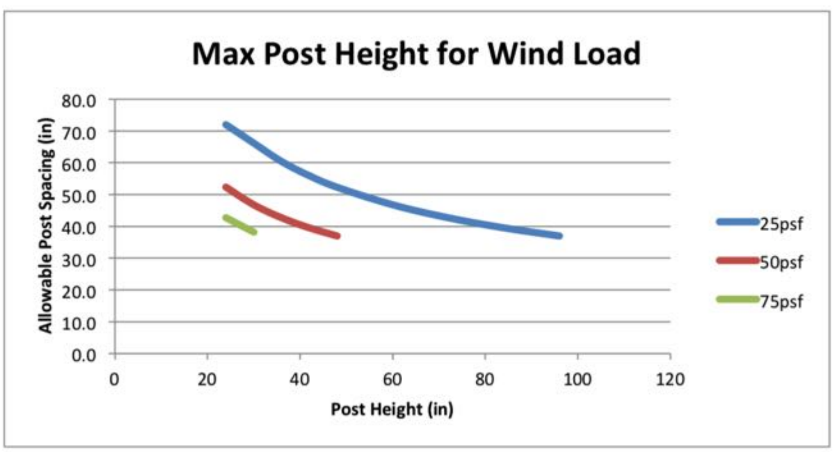
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	66.2
36	60.4
42	55.9
48	52.3
60	46.8
72	42.7
84	39.5
96	37.0

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.3
30	46.8
36	42.7
42	39.5
48	37.0
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	42.7
30	38.2
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 3-1/2" penetration and 5x5 baseplate

Ma (in-lbs) 9860

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 49.3

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	65.73333333
42	56.34285714
48	49.3
60	39.44
72	<36"
84	<36"
96	<36"

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Wind Load

*M=P/144*TW*H^2/2*

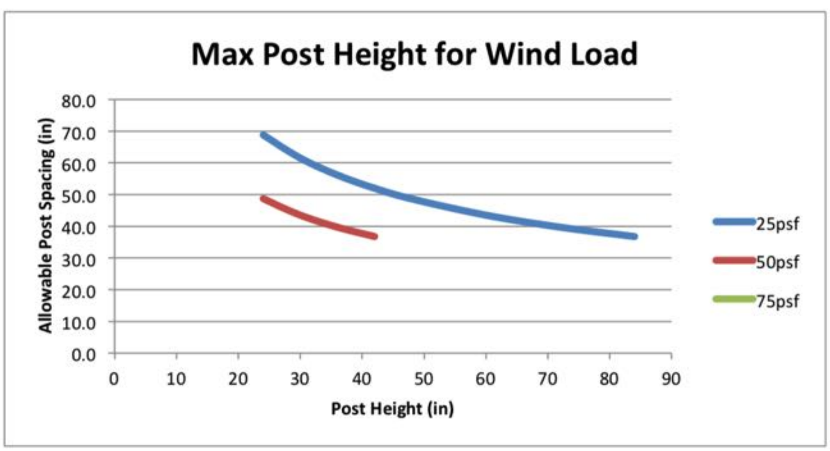
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	68.8
30	61.5
36	56.2
42	52.0
48	48.6
60	43.5
72	39.7
84	36.8
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	48.6
30	43.5
36	39.7
42	36.8
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	39.7
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 3" penetration and 5x5 baseplate

Ma (in-lbs) 8700

Load Cases:

200# concentrated load at top of post

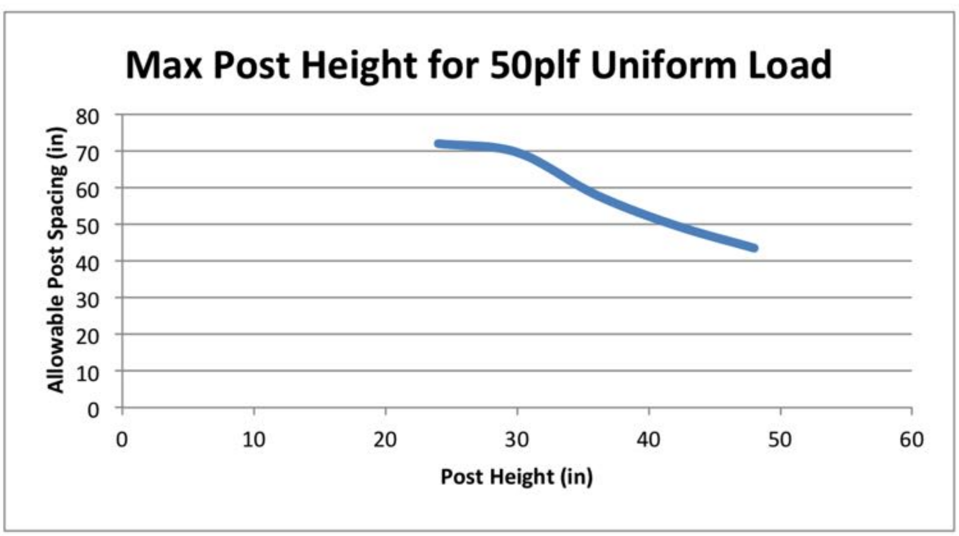
*M = 200#*H*
Hmax = Ma/200#
Hmax 43.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	69.6
36	58
42	49.71428571
48	43.5
60	<36"
72	<36"
84	<36"
96	<36"



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Wind Load

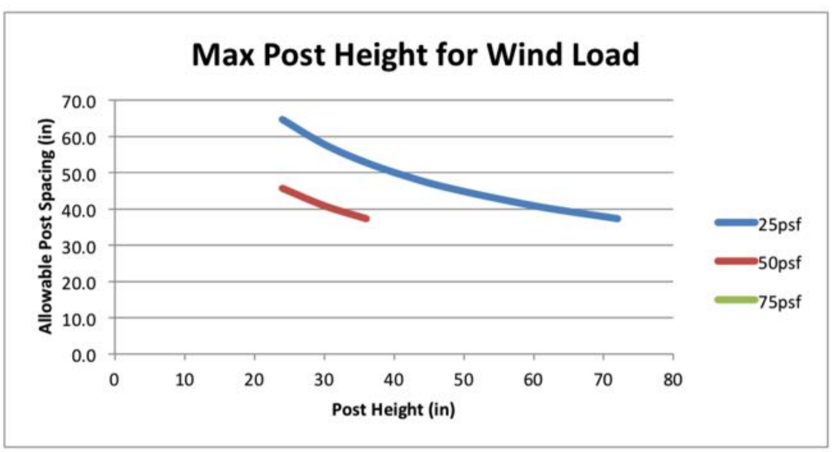
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	64.6
30	57.8
36	52.8
42	48.8
48	45.7
60	40.9
72	37.3
84	<36"
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	45.7
30	40.9
36	37.3
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	37.3
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 4-1/4" penetration and 3x5" baseplate

Ma (in-lbs) 4820

Load Cases:

200# concentrated load at top of post

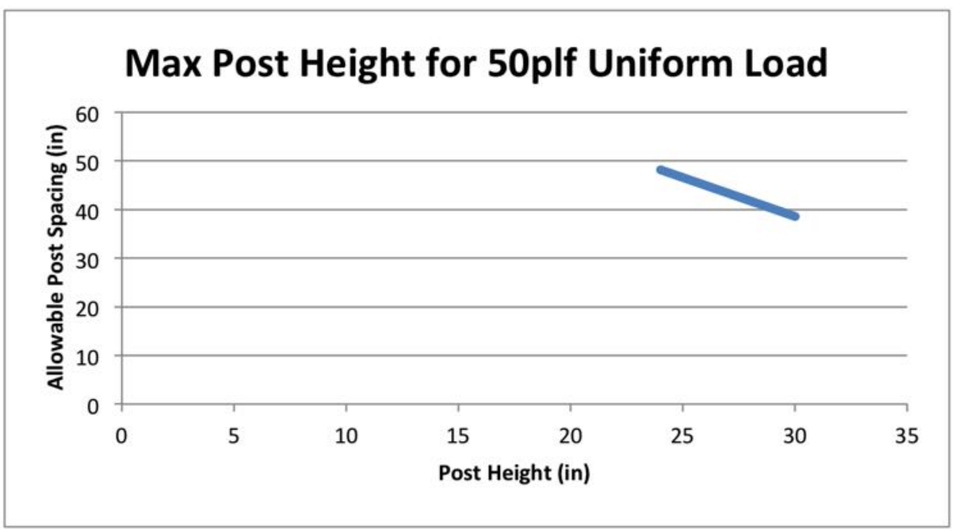
*M = 200#*H*
Hmax = Ma/200#
Hmax 24.1

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	48.2
30	38.56
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Wind Load

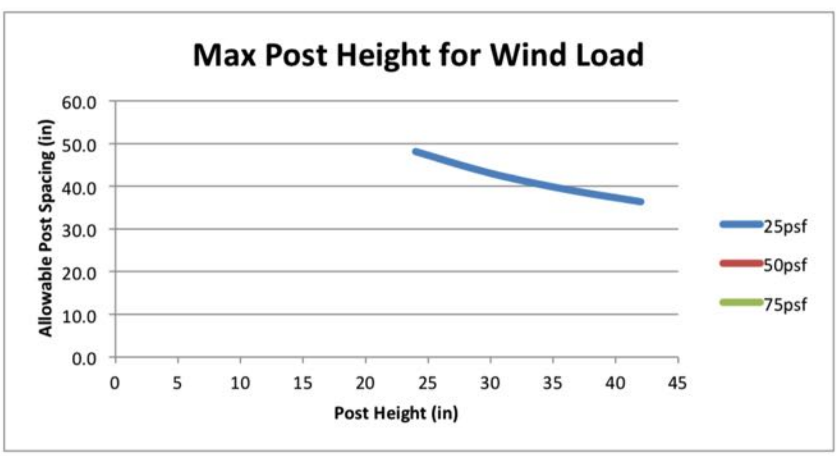
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	48.1
30	43.0
36	39.3
42	36.4
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 4-1/4" penetration and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 16000

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

Hmax = Ma/200#

Hmax 80

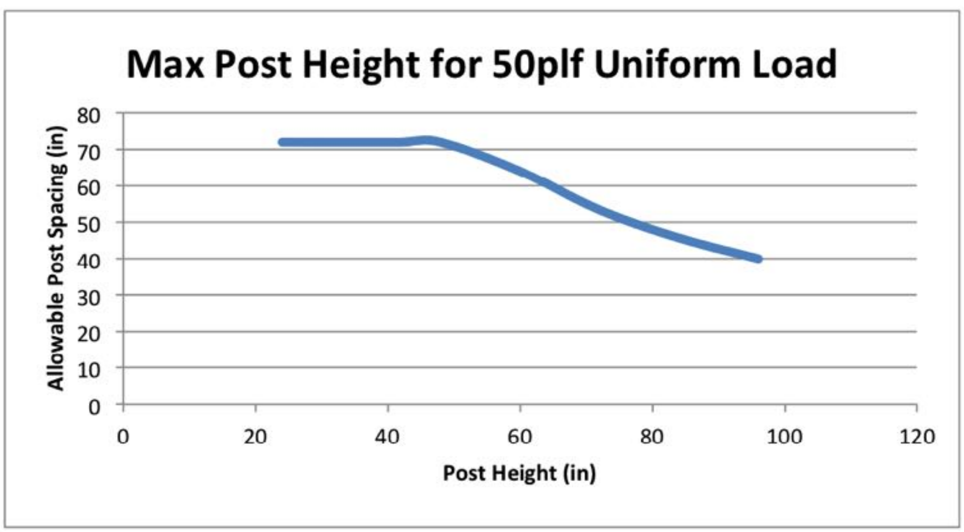
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	64
72	53.33333333
84	45.71428571
96	40



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Wind Load

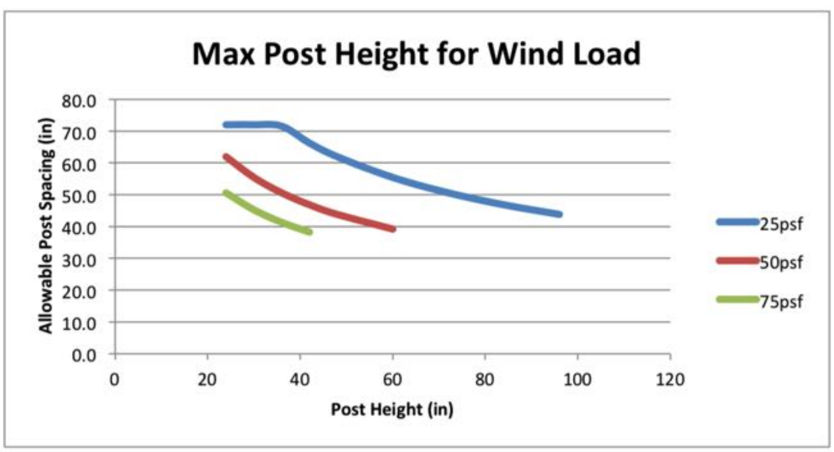
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	72.0
36	71.6
42	66.2
48	62.0
60	55.4
72	50.6
84	46.8
96	43.8

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	62.0
30	55.4
36	50.6
42	46.8
48	43.8
60	39.2
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	50.6
30	45.3
36	41.3
42	38.2
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 3-1/2" penetration and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 13600

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 68

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	68
60	54.4
72	45.33333333
84	38.85714286
96	<36"

Wind Load

*M=P/144*TW*H^2/2*

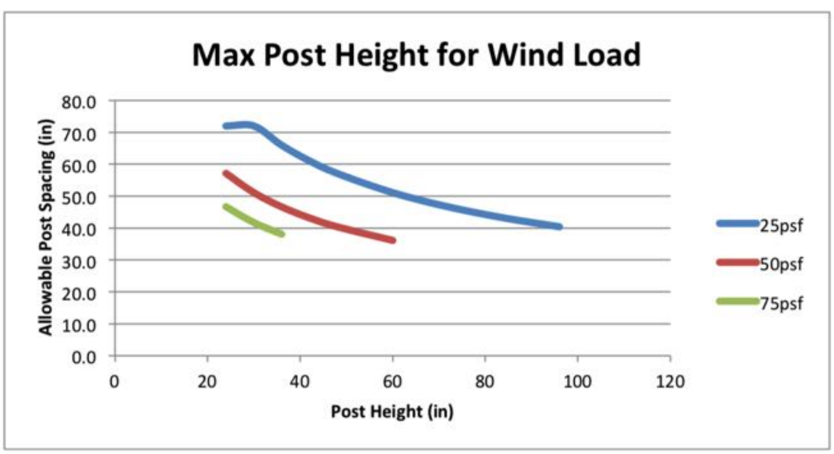
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	66.0
42	61.1
48	57.1
60	51.1
72	46.6
84	43.2
96	40.4

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	57.1
30	51.1
36	46.6
42	43.2
48	40.4
60	36.1
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	46.6
30	41.7
36	38.1
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



Post Anchorage Design

System: ARS
Detail Description:
3/8" Lag screw w/ 3" penetration and 6-1/2x6-1/2" baseplate

Ma (in-lbs) 11900

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 59.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	68
48	59.5
60	47.6
72	39.66666667
84	<36"
96	<36"

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Wind Load

*M=P/144*TW*H^2/2*

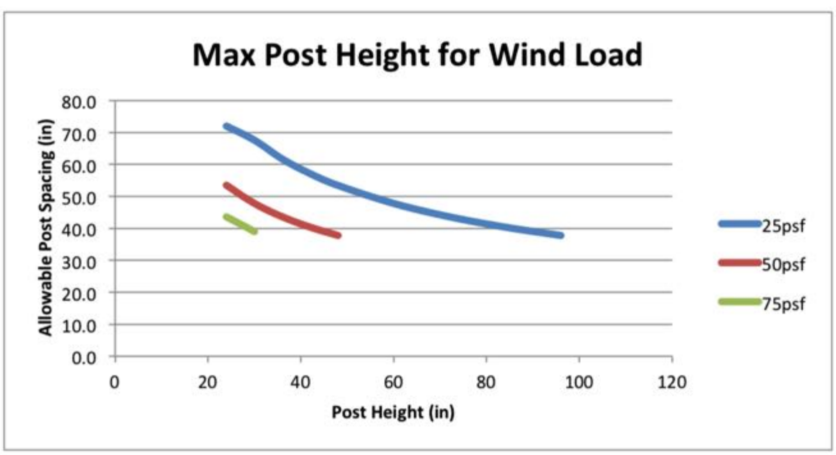
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	67.6
36	61.7
42	57.1
48	53.4
60	47.8
72	43.6
84	40.4
96	37.8

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	53.4
30	47.8
36	43.6
42	40.4
48	37.8
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	43.6
30	39.0
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Fascia bracket to wood beam. 3/8" lag screws with 3-3/8" penetration.

Ma (in-lbs) 10600

Load Cases:

200# concentrated load at top of post

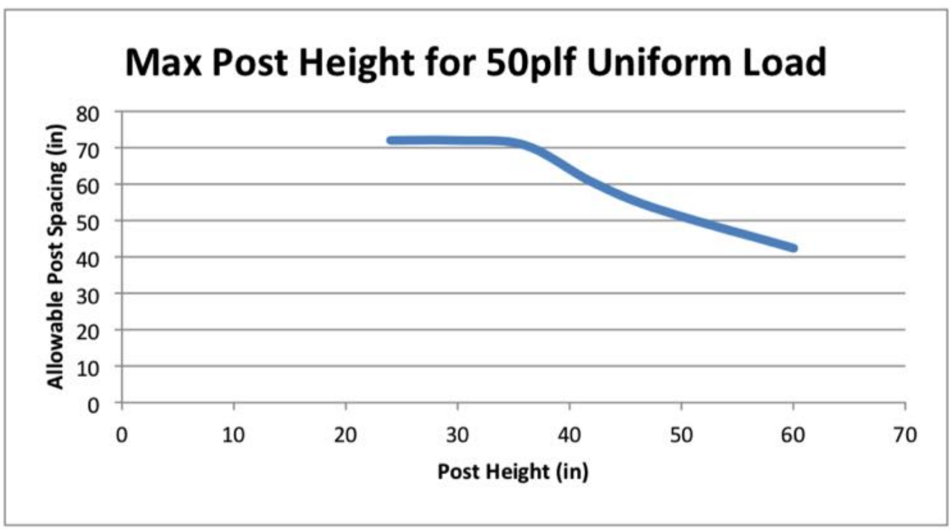
*M = 200#*H*
Hmax = Ma/200#
Hmax 53

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70.66666667
42	60.57142857
48	53
60	42.4
72	<36"
84	<36"
96	<36"



Wind Load

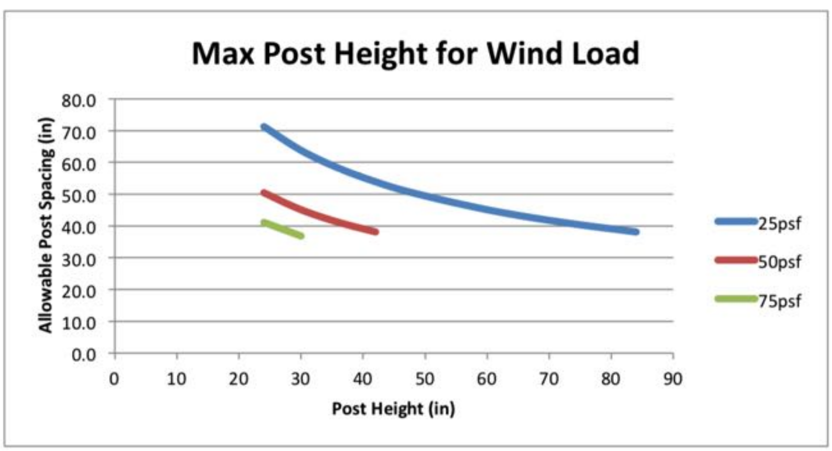
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	71.3
30	63.8
36	58.2
42	53.9
48	50.4
60	45.1
72	41.2
84	38.1
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	50.4
30	45.1
36	41.2
42	38.1
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	41.2
30	36.8
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Fascia bracket to uncracked concrete

Ma (in-lbs) 11300

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 56.5

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	64.57142857
48	56.5
60	45.2
72	37.66666667
84	<36"
96	<36"

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Wind Load

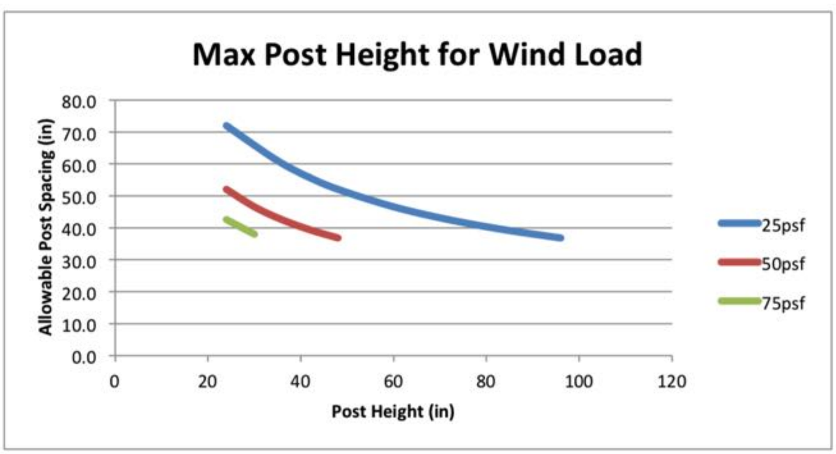
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	65.9
36	60.1
42	55.7
48	52.1
60	46.6
72	42.5
84	39.4
96	36.8

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.1
30	46.6
36	42.5
42	39.4
48	36.8
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	42.5
30	38.0
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Fascia bracket to cracked concrete

Ma (in-lbs) 8000

Load Cases:

200# concentrated load at top of post

*M = 200#*H*
Hmax = Ma/200#
Hmax 40

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	64
36	53.33333333
42	45.71428571
48	40
60	<36"
72	<36"
84	<36"
96	<36"

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Wind Load

*M=P/144*TW*H^2/2*

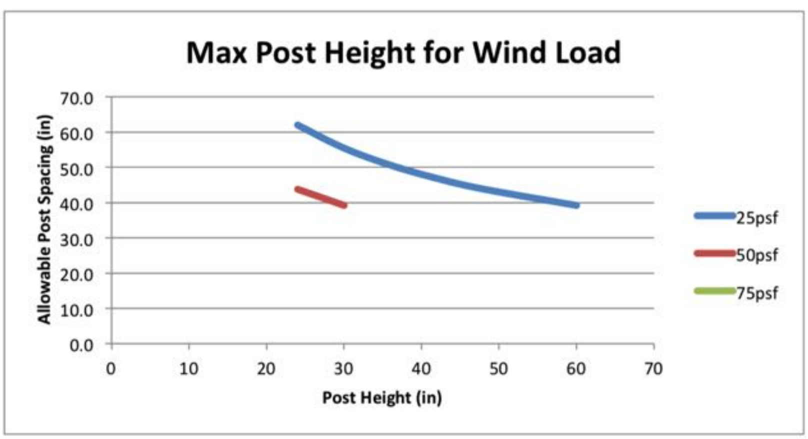
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	62.0
30	55.4
36	50.6
42	46.8
48	43.8
60	39.2
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	43.8
30	39.2
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS

Detail Description:

Direct post to wood fascia mount. 3/8"x5" lag screws.

Ma (in-lbs) 7800

Load Cases:

200# concentrated load at top of post

*M = 200#*H*

Hmax = Ma/200#

Hmax 39

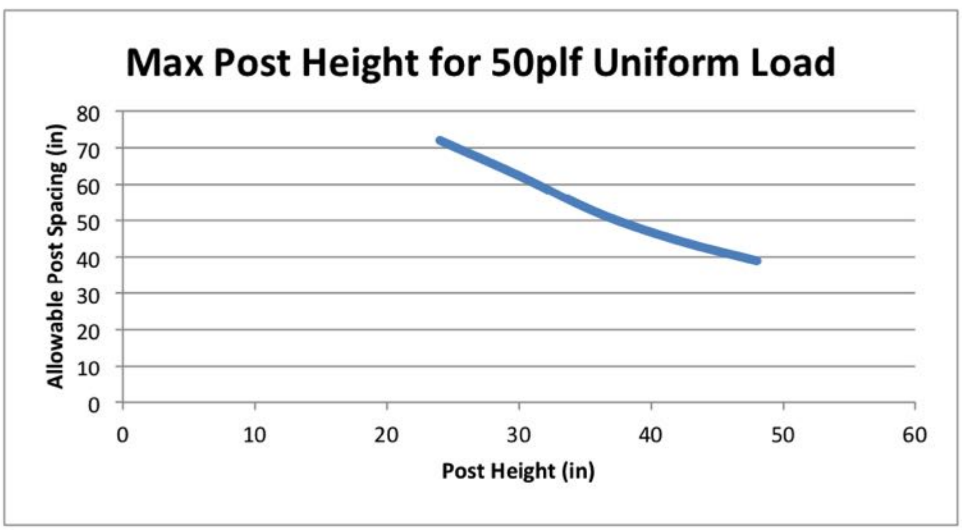
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	62.4
36	52
42	44.57142857
48	39
60	<36"
72	<36"
84	<36"
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

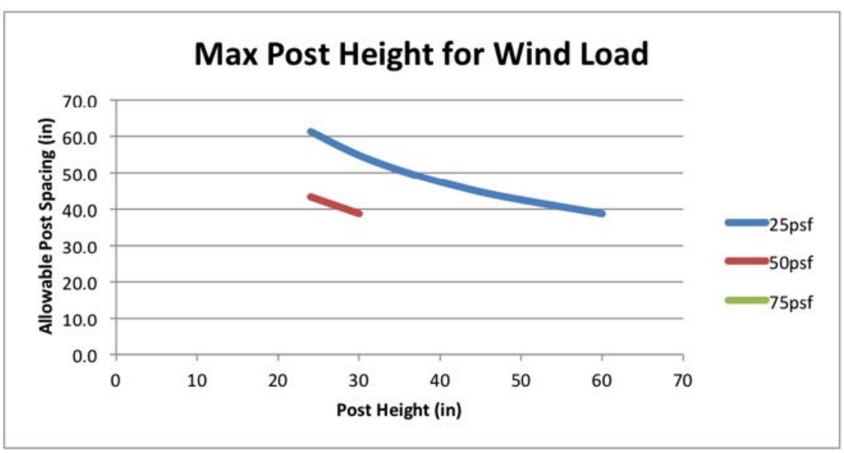
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	61.2
30	54.7
36	50.0
42	46.3
48	43.3
60	38.7
72	<36"
84	<36"
96	<36"

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	43.3
30	38.7
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	<36"
30	<36"
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Direct post to wood fascia mount. 3/8" carriage bolts.

Ma (in-lbs) 17400

Load Cases:

200# concentrated load at top of post

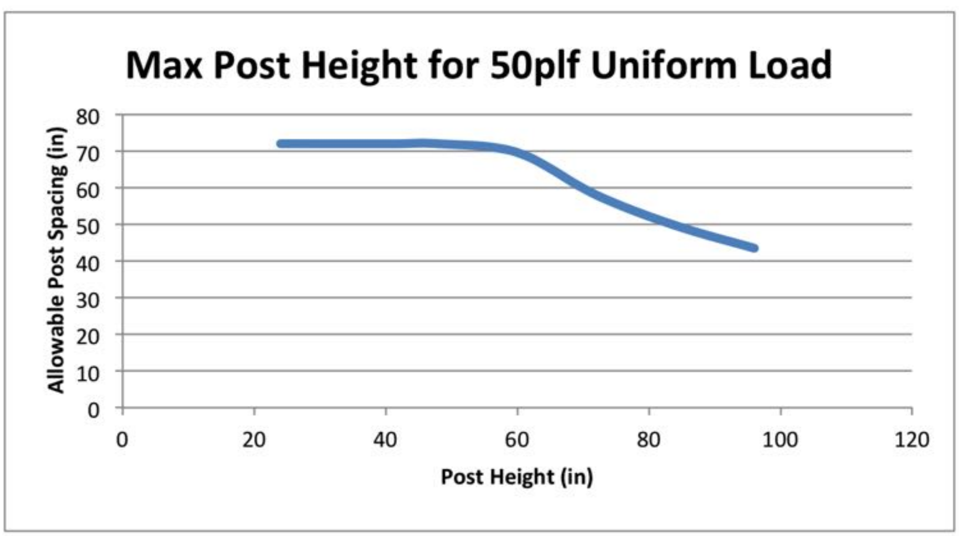
*M = 200#*H*
Hmax = Ma/200#
Hmax 87

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	69.6
72	58
84	49.71428571
96	43.5



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Wind Load

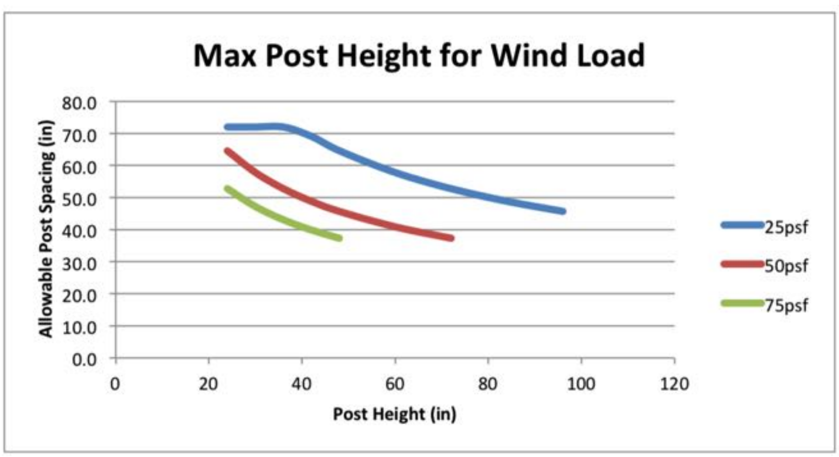
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	72.0
36	72.0
42	69.1
48	64.6
60	57.8
72	52.8
84	48.8
96	45.7

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	64.6
30	57.8
36	52.8
42	48.8
48	45.7
60	40.9
72	37.3
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.8
30	47.2
36	43.1
42	39.9
48	37.3
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Post in core mount

Ma (in-lbs) 12600

Load Cases:

200# concentrated load at top of post

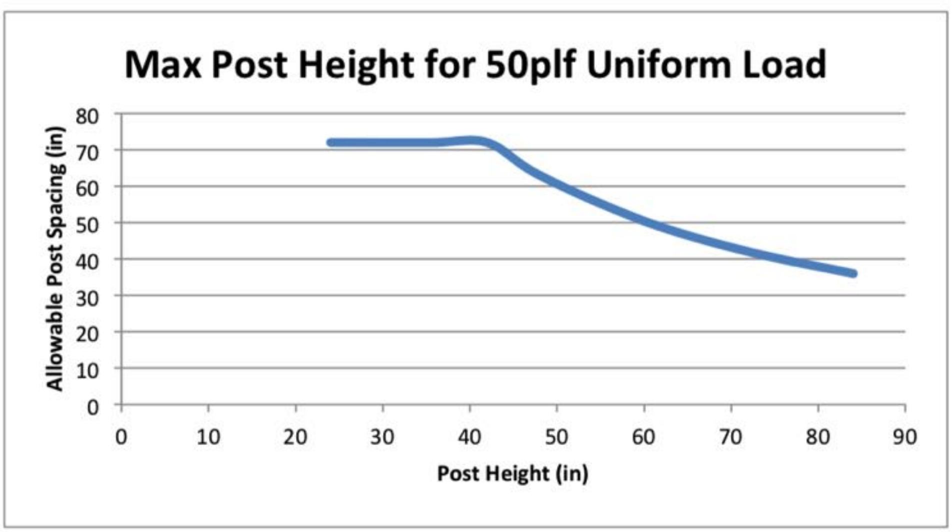
*M = 200#*H*
Hmax = Ma/200#
Hmax 63

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63
60	50.4
72	42
84	36
96	<36"



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Wind Load

*M=P/144*TW*H^2/2*

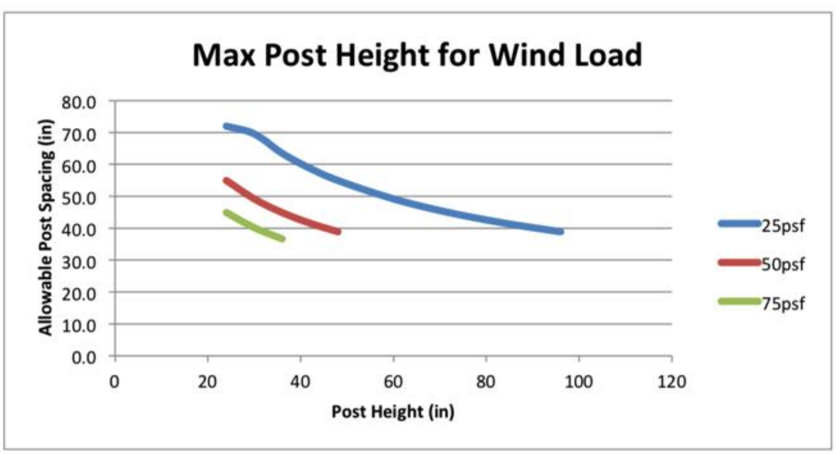
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	72.0
30	69.6
36	63.5
42	58.8
48	55.0
60	49.2
72	44.9
84	41.6
96	38.9

P= 50psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	55.0
30	49.2
36	44.9
42	41.6
48	38.9
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	**Allowable Post Spacing (in)**
24	44.9
30	40.2
36	36.7
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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Post Anchorage Design

System: ARS
Detail Description:
Stanchion in core mount

Ma (in-lbs) 11400

Load Cases:

200# concentrated load at top of post

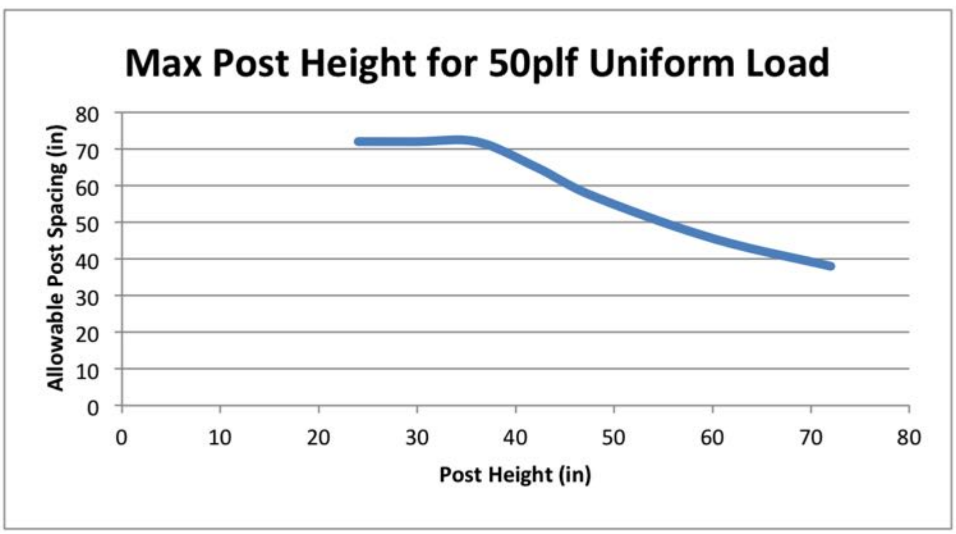
*M = 200#*H*
Hmax = Ma/200#
Hmax 57

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	65.14285714
48	57
60	45.6
72	38
84	<36"
96	<36"



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Wind Load

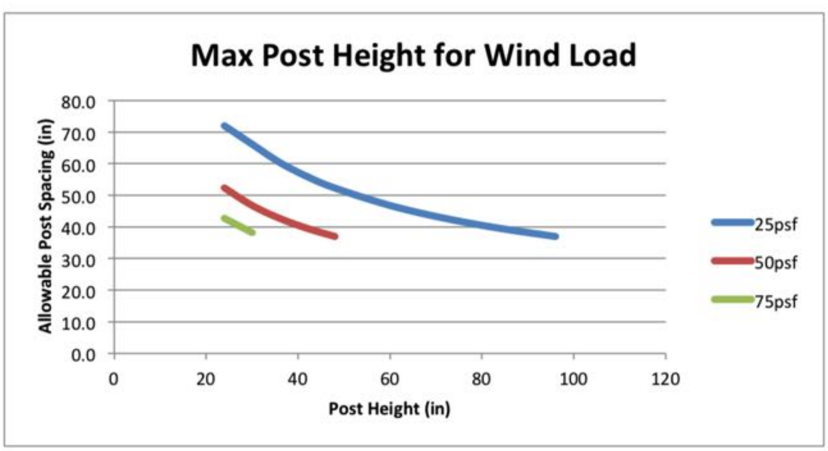
*M=P/144*TW*H^2/2*
*Hmax = (2*Ma/(TW*P/144))^(1/2)*

Allowable post height with respect to post spacing for different wind pressures:

P= 25psf	
Post Height (in)	Allowable Post Spacing (in)
24	72.0
30	66.2
36	60.4
42	55.9
48	52.3
60	46.8
72	42.7
84	39.5
96	37.0

P= 50psf	
Post Height (in)	Allowable Post Spacing (in)
24	52.3
30	46.8
36	42.7
42	39.5
48	37.0
60	<36"
72	<36"
84	<36"
96	<36"

P= 75psf	
Post Height (in)	Allowable Post Spacing (in)
24	42.7
30	38.2
36	<36"
42	<36"
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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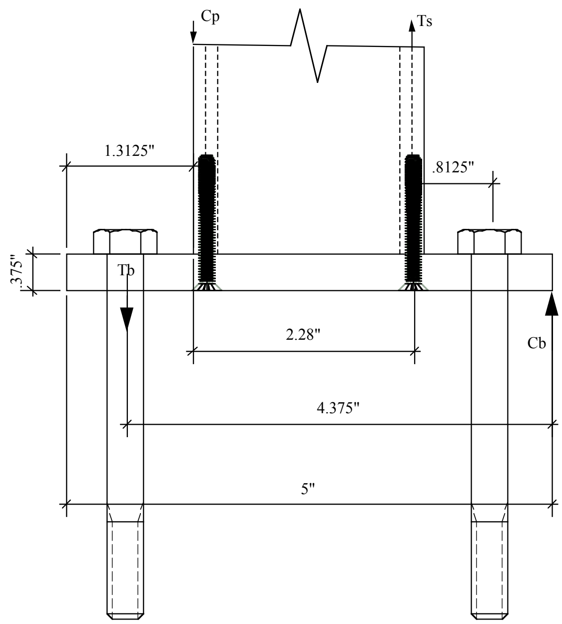
Connection to base plate



Failure modes → screw tension
→ screw shear
→ screw withdrawal

Testing has shown that screw withdrawal doesn't control the base plate to post connection. Thus ADM equation J.5-7 isn't applicable to this application.

Base plate to post screws are AISI 4037 steel alloy fabricated in accordance with SAE J429 Grade 8 and coated with Magni 550 corrosion protection. For 1/4" SAE J429 screw areas provided by manufacturer

Root area = 0.0483in^2

Tension rupture area = 0.0376 in^2

Screw tension → T_y = 0.0483 in^2 • 110 ksi = 5314 # V_u = 0.0483* 45ksi =2,174#

F_{tU} = 0.0376 • 150 ksi = 5,640#

Safety factors for screws calculated from SEI/ASCE 8-02 Section 5 LRFD factors - These safety factors were determine appropriate because AISC-360 and the ADM don't include this type of screw but SEI/ASCE 8-02 does.

For yielding SF = 1.6/0.75 = 2.13 → 5,314#/2.13 = 2,495#

For fracture SF = 1.6/0.65 = 2.46 → 5,640/2.46 = 2,293#

Shear strength

For fracture SF = 1.6/(0.9*0.75) = 2.37 → 2,174/2.37 = 917#

Base plate is 5083-H32 or 5052 H34 Aluminum plate, stamped

Allowable bending stress for flat plate = 24 ksi (1.5*Fy/1.65) ADM B.3.2.2

Base plate bending stress

$$F_t = 24 \text{ ksi} \rightarrow S_{min} = \frac{5" \bullet 3/8^2}{6} = 0.117 \text{ in}^3$$

Base plate allowable moment

$$M_{all} = 24 \text{ ksi} \bullet 0.117 \text{ in}^3 = 2{,}812 \text{ "\#}$$

→ Base plate bending stress

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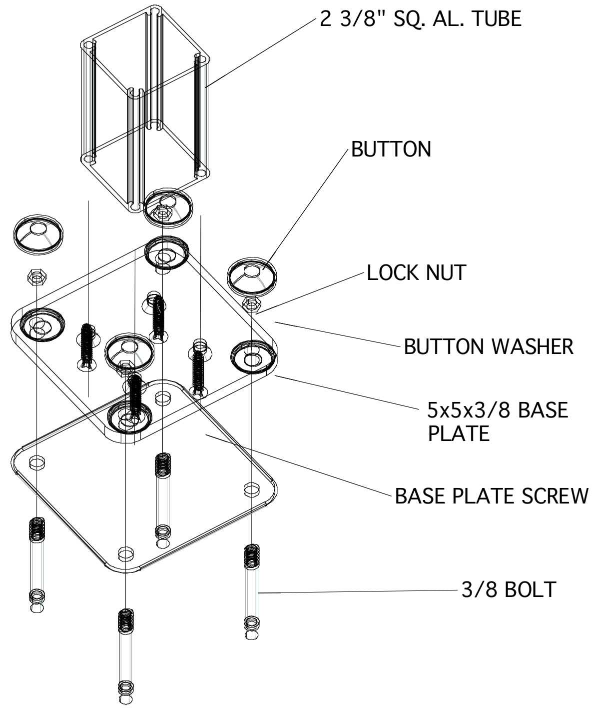

$T_B = C$

$M = 0.8125" \bullet T_B \bullet 2$

$$T_{all} = \frac{2{,}812}{2 \bullet 0.8125} = 1{,}730^{\#}$$

Maximum post moment for base plate strength
$M_{all} = 2 \bullet 1730 \bullet 4.375" = 15{,}142^{\#"}$

Limiting factor = screws to post
$M_{ult} = 2 \bullet 5{,}314^{\#} \bullet 2.28" = 24{,}232^{\#"}$
$M_{all} = 2 \bullet 2{,}293^{\#} \bullet 2.28" = 10{,}456^{"\#}$

Testing and experience has shown that screw tension rupture is the controlling failure mode.

Interaction of shear and tension on the screw isn't required as two screws on the compression face will take all shear so the tension screws don't carry shear loads.

Alternatively if the shear is distributed evenly to all screws the shear stress won't reduce the allowable tension load until it exceeds 20% of the strength:
V = 0.2*4*917# = 734#
As this will greatly exceed the allowable load on the post the interaction need not be considered.

As the screws are countersunk in the base plate the base plate flexure doesn't create significant prying action on the screw heads as the plate may pivot around the screw head like a socket joint. Therefore prying action on the baseplate screws may be ignored.

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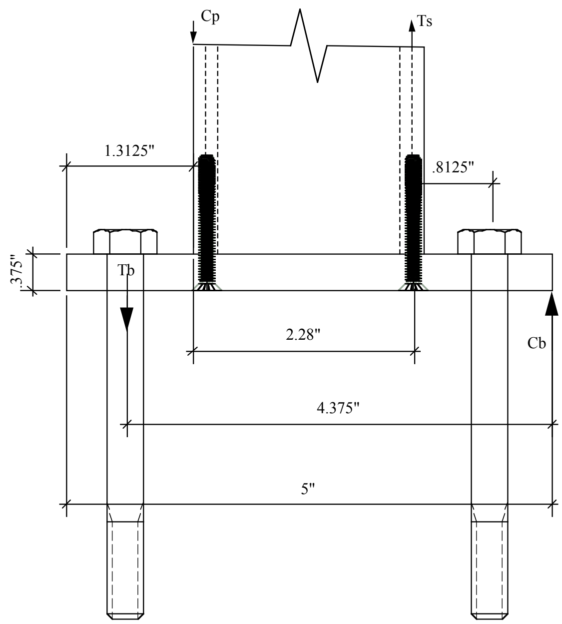
SIX SCREW CONNECTION TO BASE PLATE



Screws are the same as for the standard 4 screw connection.

Screw embedment length into the screw slots is adequate to develop the full screw tension strength.

Use same screw tension strength as used for the four screw connection:

T_a = 2,293# per screw
V_a = 917# per screw

V_{des} = 6*917 = 5,502#
limiting shear load on post so that screw shear stress doesn't reduce the allowable tension:
$V_{0.2}$ = 0.2*5,502# = 1,100#

Base plate thickness and strength same as for standard post.

Allowable moment on the posts based on screw tension strength:

Strong axis bending -
M_{base} = 3 screws*2,293#*2.28" = 15,684"# < 19,479"#
Doesn't develop full post strength.

Weak axis bending -
M_{base} = 2 screws*2,293#*2.28"+ 2 screws*0.5*2,293#*2.28"/2 = 13,070"# ≤ 17,560"#"#
6 screw connection won't develop the full post strength for weak axis bending.

LIMITING POST MOMENTS FOR SIX SCREW CONNECTION:
STRONG AXIS BENDING M_A = 15,684"# = 1,307'#
WEAK AXIS BENDING M_A = 13,070"# = 1,089'#

Note that the 135° post only has four screw slots. However, as noted in the 135° post strength calculations, the loading to the 135° corner post is reduced by approximately 17%. Required strength for 135° post to not limit spacing = 0.833*15,700"# = 13,100"# > 10,500"#. The presence of 135° posts screwed to baseplates will limit post spacing when other posts used the six screw pattern. Use M_a = 10,500"#/0.833 = 12,600"# for determining allowable post spacing.

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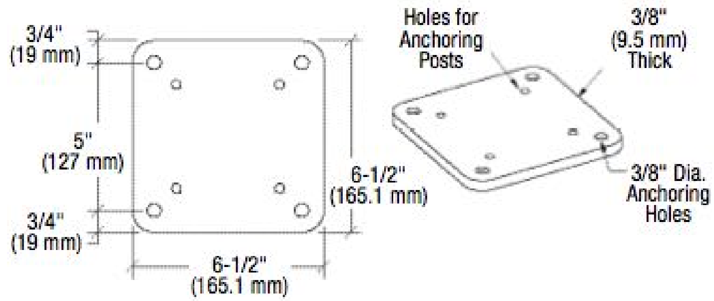
6-1/2” Square Base Plates for 4” Square Posts

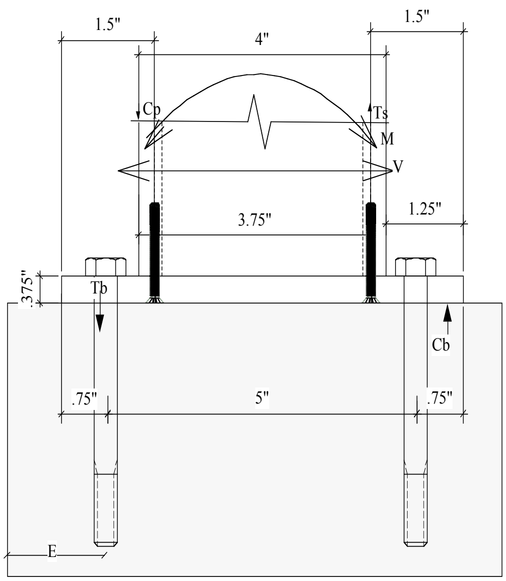
Connection to base plate



Failure modes → screw tension
→ screw shear
→ screw withdrawal

Testing has demonstrated that screw withdrawal doesn’t occur for the post alloy and screw embedment length

Base plate to post screws are AISI 4037 steel alloy fabricated in accordance with SAE J429 Grade 8 and coated with Magni 550 corrosion protection.



Screw tension → $T_y = 0.0483\text{in}^2 \bullet 110\text{ ksi} = 5314^{\#}$

$V_u = 0.0483 * 45\text{ksi} = 2{,}174\#$

$F_{tU} = 0.0376 \bullet 150\text{ ksi} = 5640^{\#}$

Safety factors for screws calculated from SEI/ASCE 8-02 Section 5 LRFD factors. These safety factors were determine appropriate because AISC-360 and the ADM don’t include this type of screw but SEI/ASCE 8-02 does.

For yielding SF = 1.6/0.75 = 2.13 → $5{,}314^{\#}/2.13 = 2{,}495^{\#}$

For fracture SF = 1.6/(0.9*0.75) = 2.37 → $5{,}640/2.37 = 2{,}380^{\#}$

Shear strength

SF = 1.6/0.65 = 2.46 $V_a = V_u/SF = 2{,}174^{\#}/2.46 = 884\#$

Base plate bending stress

$$F_t = 24\text{ ksi} \rightarrow S_{min} = \frac{6.5” \bullet 3/8^2}{6} = 0.152\text{ in}^3$$

6-1/2" Square Base Plates Cont.

Base plate allowable moment

M_{all} = 24 ksi • 0.152 in^3 = 3,648 "#

→ Base plate bending stress

$T_B = C$

M = 5/8" • T_B • 2 = 3,648#"

$$T_{all} = \frac{3{,}648\#"}{2 \cdot 0.625} = 2{,}918^{\#}$$

Maximum allowable post moment for base plate strength

M_{all} = 2 • 2,918 • 5.75" = 33,562#"

Limiting factor = screws to post

M_{ult} = 2 • 5,314# • 3.69" = 39,217#"

M_{all} = 2 • 2,380# • 3.69" = 17,264#"

As the screws are countersunk in the base plate the base plate flexure doesn't create significant prying action on the screw heads as the plate may pivot around the screw head like a socket joint. Therefore prying action on the baseplate screws may be ignored.

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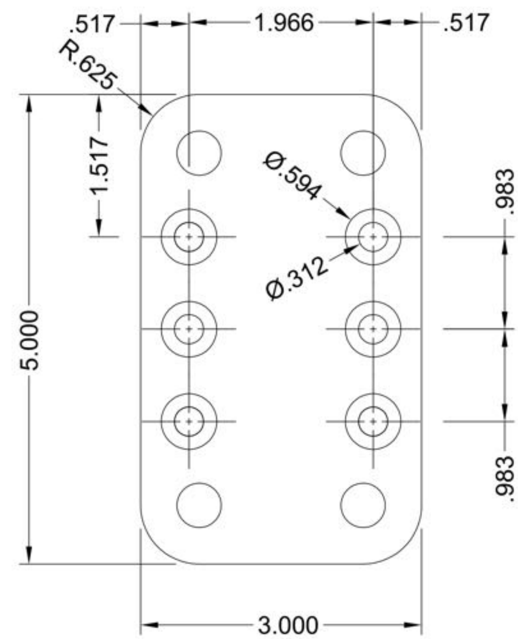
3X5" BASEPLATE

The 3x5" baseplate uses the same screw pattern as the 5x5 baseplate so the strength of the connection to the post is the same. Prying loading on the heel side anchors may be larger however. Allowable moments on the 3x5" baseplate will be lower because of the reduced effective distance between the tension and compression centroids.



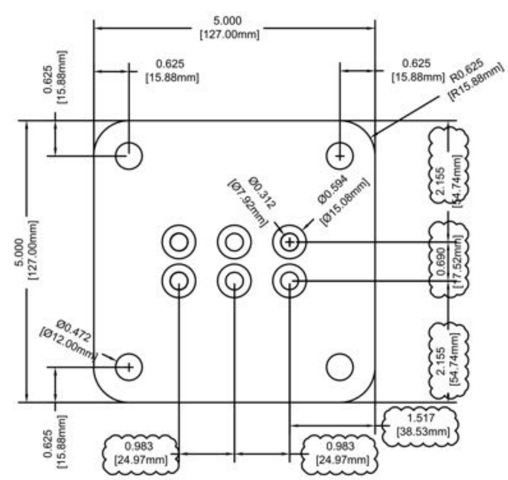
TRIM LINE POST BASEPLATE

Uses same screws as the 2-3/8" square post.

Strong axis bending -
M_{base} = 2 screws*2,293#*2.28"+ 2 screws*0.5*2,293#*2.28"/2 = 13,100"#

Weak axis bending -
M_{base} = 3 screws*2,293#*0.895" = 6,160"#

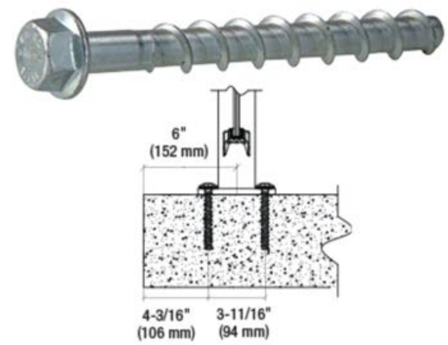
Anchorage to the structure is the same as for the 2-3/8" square posts.



Hilti 3/8"x4" Kwik HUS-EZ

Base plate mounted to concrete with Hilti 3/8"x4" KH-EZ concrete anchors. Anchor strength based on ESR-3027 AND ACI318.

Minimum conditions used for the calculations:



$f'_c \geq 3{,}000$ psi; edge distance =4.1875" spacing = 3.75"

h_{nom} = 4"-3/8" = 3.625"

h_{ef} = 2.5"

For concrete breakout strength:

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

A_{Ncg}= (1.5*2.5"*2+3.75")*(1.5*2.5"*2) = 84.38 in^2 for 2 tension anchors

A_{Nco}= 9*2.5"^{2} = 56.25in^2

C_{ac} = 3.75"

$\varphi_{ed,N} = 1.0$

$\varphi_{c,N} = 1.0$

$k_{uncr} = 24$

$k_{cr} = 17$

$\varphi_{cp,N} = 1$

$N_b = 24*1.0*\sqrt{3000}*2.5^{1.5}$ = 5,200# for uncracked concrete

$N_b = 17*1.0*\sqrt{3000}*2.5^{1.5}$ = 3,680# for cracked concrete

N_{cb} = 84.38/56.25*1.0*1.0*1.0*5,200# = 7,800# for uncracked concrete

N_{cb} = 84.38/56.25*1.0*1.0*1.0*3,680# = 5,520# for cracked concrete

Determine allowable tension load on anchor pair assuming an average load factor of 1.6.

T_a = 0.65*7,800#/1.6 = 3,170#

T_a = 0.65*5,520#/1.6 = 2,240#

Anchor steel strength:

T_a = 2*0.65*10,335#/1.6 = 8,400# (Does not control)

Per ESR 3027, pullout strength does not control.

Find moment strength:

a = 3,170#/(0.85*3000psi*5") = 0.249" for uncracked concrete

a = 2,240#/(0.85*3000psi*5") = 0.176" for cracked concrete

5"x5" baseplate:

M_a = 3,170#*(4.375"-0.249"/2) = 13,500"# for uncracked concrete

M_a = 2,240#*(4.375"-0.176"/2) = 9,600"# for cracked concrete

6-1/2"x6-1/2" baseplate:

M_a = 3,170#*(5.75"-0.249"/2) = 17,800"# for uncracked concrete

M_a = 2,240#*(5.75"-0.176"/2) = 12,700"# for cracked concrete

3"x5" baseplate:

M_a = 3,170#*(2.375"-0.249"/2) = 7,130"# for uncracked concrete

M_a = 2,240#*(2.375"-0.176"/2) = 5,120"# for cracked concrete

Check shear strength - Concrete breakout strength in shear:

$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$

A_{vc} = (1.5*4.19"*2+3.75")*(4.19"*1.5) = 102.6in^2

A_{vco}= $4.5(c_{a1})^2 = 4.5(4.19)^2 = 79.00in^2$
$\varphi_{ed,V}$= 1.0 (affected by only one edge)
$\varphi_{c,V}$= 1.0 cracked concrete
V_b= $[7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(2.5/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(4.19)^{1.5}$ =2,940#
V_{cb} = 102.6/79.00*1.0*1.0*1.0*2,940# = 3,820#
V_a = 0.7*3,820#/1.6 = 1,670#
Steel shear strength = 5,185#*2 = 10,400#
Allowable shear strength
$ØV_N$/1.6 = 0.60*10,400#/1.6 = 3,900#
Controlling allowable shear strength = 1,670#
Note that the concrete breakout strength in shear is based off of the anchors closest to the edge which would not have tension loading if the force is towards the edge. It can be assumed the toe side anchors will resist all the shear and the heel side anchors will resist all the tension. Therefore interaction of shear and tension is not a concern.

Hilti 3/8"x3-3/4" Stainless Steel Kwikbolt - TZ

Base plate mounted to concrete with Hilti 3/4"x3-3/4" KB-TZ concrete anchors. Anchor strength based on ESR-1917 and ACI 318.
Minimum conditions used for the calculations:
$f'_c \geq 3{,}000$ psi; edge distance =2-5/8" spacing = 3.75"
h_{nom} = 3-1/16"
h_{ef} = 2.75"
For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
A_{Ncg}= (1.5*2.75"*2+3.75")*(1.5*2.75"+2.625") = 81.0 in^2 for 2 tension anchors
A_{Nco}= 9*2.75"^{2} = 68.1in^2
C_{ac} = 4.125"
$\varphi_{ed,N}$ = 0.7+0.3*2.625"/(1.5*2.75") = 0.89
$\varphi_{ec,N}$ = 1
k_{uncr} = 24
k_{cr} = 17
$\varphi_{cp,N}$ = 1 for cracked concrete
$\varphi_{cp,N} = C_{ac}/(1.5_{ef})$ max = 4.125/(1.5*2.75) = 1
N_b = 24*1.0*√3000*2.75$^{1.5}$ = 5,990# for uncracked concrete
N_b = 17*1.0*√3000*2.75$^{1.5}$ = 4,250# for cracked concrete
N_{cb} = 81.0/68.1*0.89*1.0*1.0*5,990# = 6,340# for uncracked concrete
N_{cb} = 81.0/68.1*0.89*1.0*1.0*4,250# = 4,500# for cracked concrete
Determine allowable tension load on anchor pair assuming an average load factor of 1.6.
T_a = 0.65*6,340#/1.6 = 2,580# for uncracked concrete
T_a = 0.65*4,500#/1.6 = 1,830# for cracked concrete
Anchor pullout strength:
$T_{a,}$ = 2*0.65*4,110#/1.6 = 3,340#for uncracked concrete (Does not control)
T_a = 2*0.65*3,160#/1.6 = 2,570# for cracked concrete (Does not control)
Find moment strength:
a = 3,340#/(0.85*3000psi*5") = 0.262" for uncracked concrete
a = 2,570#/(0.85*3000psi*5") = 0.202" for cracked concrete
5"x5" baseplate:
M_a = 3,340#*(4.375"-0.262"/2) = 14,200"# for uncracked concrete
M_a = 2,570#*(4.375"-0.202"/2) = 11,000"# for cracked concrete
6-1/2"x6-1/2" baseplate:
M_a = 3,340#*(5.75"-0.262"/2) = 18,800"# for uncracked concrete
M_a = 2,570#*(5.75"-0.202"/2) = 14,500"# for cracked concrete
3"x5" baseplate:
M_a = 3,340#*(2.375"-0.262"/2) = 7,490"# for uncracked concrete
M_a = 2,570#*(2.375"-0.202"/2) = 5,840"# for cracked concrete

Check shear strength - Concrete breakout strength in shear:

$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
$A_{vc} = (1.5*2.625"*2+3.75")*(2.625"*1.5) = 45.77in^2$
$A_{vco}= 4.5(c_{a1})^2 = 4.5(2.625")^2 = 31.01in^2$
$\varphi_{ed,V}= 1.0$ (affected by only one edge)
$\varphi_{c,V}= 1.0$ cracked concrete
$V_b= [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(2.75/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(2.625)^{1.5} =1,490\#$
$V_{cb} = 45.77/31.01*1.0*1.0*1.0*1,490\# = 2,200\#$
$V_a = 0.7*2,200\#/1.6 = 963\#$
Steel shear strength = 3,595#*2 = 7,190#
Allowable shear strength
$ØV_N/1.6 = 0.65*7,190\#/1.6 = 2,920\#$
Controlling allowable shear strength = 963#
Note that the concrete breakout strength in shear is based off of the anchors closest to the edge which would not have tension loading if the force is towards the edge. It can be assumed the toe side anchors will resist all the shear and the heel side anchors will resist all the tension. Therefore interaction of shear and tension is not a concern.

RAISED BASEPLATE DESIGN AND ANCHORAGE –

Baseplates are raised up and bear on nuts installed on embedded threaded rod.
Guard rail Height: 42”
loading: 200# concentrated load or
50 plf uniform load on top rail or
25 psf distributed load on area or
25 psf = 80 mph exp C wind load:

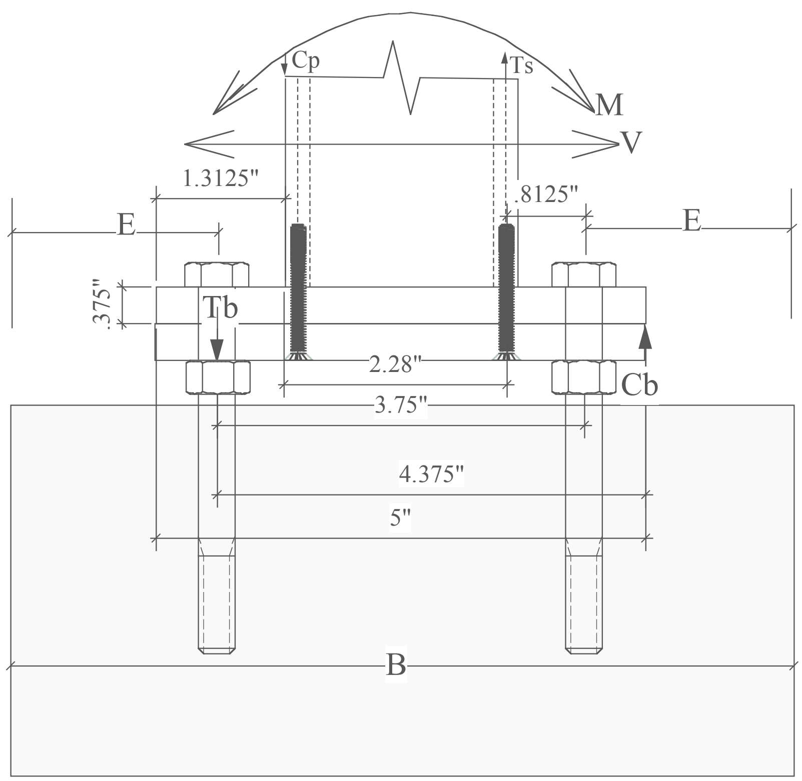
Design moment on posts:
M_l = 42”*200# = 8,400”#
M_l = 42”*50plf*5ft = 10,500”#
Mw = 3.5’*5’*25psf*42”/2 = 9,188”#

Design anchorage for 10,500”# moment.
Design shear = 438# (wind)

Bolt tension for typical design
T =10,500/(2*3.75)=1,400#

Anchor to concrete:
1/2” x 5” all-thread embedment depth = 3.5” and 4,000 psi concrete strength. Select anchor for:
Adjustment for anchor spacing = 3.75”
Adjustment for edge distance = 2-1/8”
T’ = 2,062#

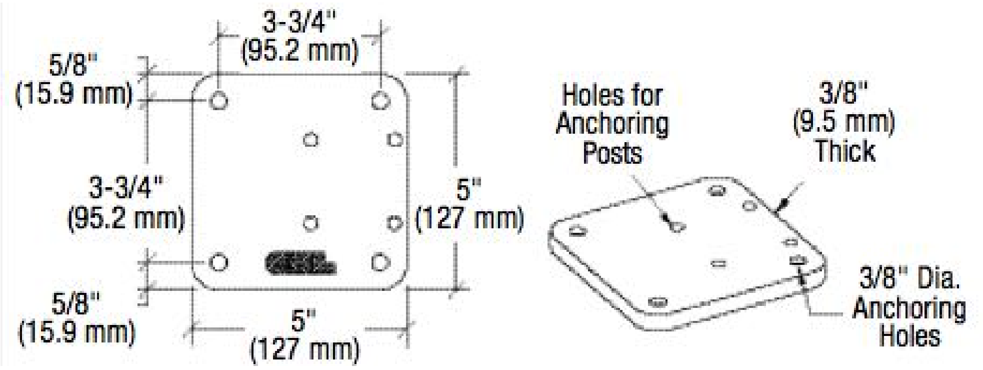
Check base plate strength:
Bending is biaxial because it sits on bearing nuts:
M = (3.75”-2.28”)/2*1,400#*2*√2 = 2,910”#



Bending stress in plate
The effective width at the post screws: 3.86”
$S = 2*3.86”*0.375^2/6 = 0.181\ in^3$
f_b = 2,910/0.181 = 16,080 psi
Base plate is 5083-H32 or 5052 H34 Aluminum plate, stamped
Allowable bending stress for flat plate = 24 ksi (1.5*Fy/1.65) ADM B.3.2.2

Bearing on nut:
Area = $(0.8^2 - 0.5625^2)\pi = 1.0$ in^2
f_B = 1,400#/1.0 = 1,400 psi - Okay
Screws to post – okay based on standard base plate design
Posts okay based on standard post design

OFFSET BASE PLATE



Offset base plate will have same allowable loads as the standard base plate.
Anchors to concrete are same as for standard base plate.

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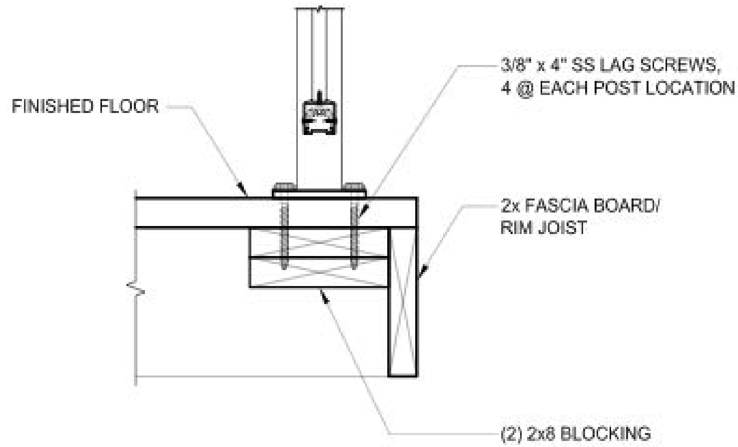
BASE PLATE MOUNTED TO WOOD



3/8" lag screw strength is according to AWC NDS. Assumed wood specific gravity is greater or equal than 0.43. This includes Hem-Fir, Douglas-Fir and Southern Pine.

From NDS Table 12.2A, W = 243pli.
C_D = 1.6 for short term guard rail live loads.
W' = 1.6*243pli = 389pli
Use either A307 steel or 304 stainless steel lag screws. F_u = 60ksi for A307 or 75ksi for 304.

$A_r = \pi/4*(0.265in)^2 = 0.0552in^2$
Design for a safety factor of 2 against screw rupture.
$P_a = 0.0552in^2*60ksi/2$ = 1,660#
Required penetration to develop strength of screw = 1,660#389pli = 4.27".

Check anchorage strength for 4-1/4", 2-1/2" and 1-1/2" screw penetration.
Assume wood bearing strength > 360psi This includes Hem-Fir, Douglas-Fir, Southern Pine and APA wood structural panels.
Bearing length, a = 2*389pli*p/(360psi*5") = 0.432p
5"x5" baseplate:
For 4-1/4" embedment, M_a = 2*389pli*4.25"*(4.375"-0.432*4.25"/2) = 11,400"#
For 3-1/2 embedment, M_a = 2*389pli*3.5"*(4.375"-0.432*3.5"/2) = 9,860"#
For 3" embedment, M_a = 2*389pli*3"*(4.375"-0.432*3"/2) = 8,700"#
6-1/2"x6-1/2" baseplate:
For 4-1/4" embedment, M_a = 2*389pli*4.25"*(5.75"-0.432*4.25"/2) = 16,000"#
For 3-1/2 embedment, M_a = 2*389pli*3.5"*(5.75"-0.432*3.5"/2) = 13,600"#
For 3" embedment, M_a = 2*389pli*3"*(5.75"-0.432*3"/2) = 11,900"#
3"x5" baseplate:
For 4-1/4" embedment, M_a = 2*389pli*4.25"*(2.375"-0.432*4.25"/2) = 4,820"#
For 3-1/2 embedment, M_a = 2*389pli*3.5"*(2.375"-0.432*3.5"/2) = 4,410"#
For 3" embedment, M_a = 2*389pli*3"*(2.375"-0.432*3"/2) = 4,030"#

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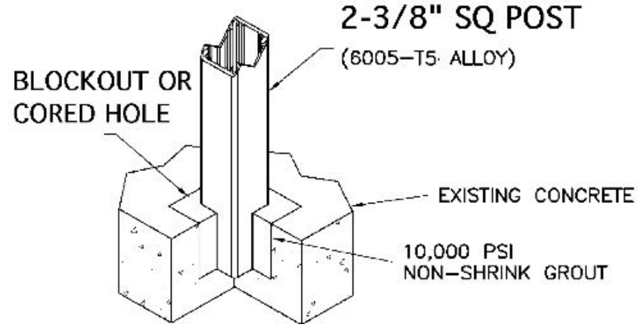
Core Mounted Posts

Mounted in either 4”x4”x4” blockout, or 2-3/8” to 6” dia by 4” deep cored hole.

Assumed concrete strength 2,500 psi for existing concrete



Max load – 6’•50 plf = 300#

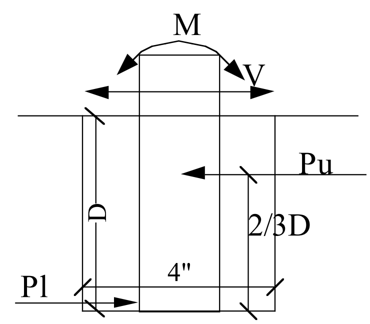
M = 300#•42” = 12,600”#

Check grout reactions

From $\Sigma M_{PL} = 0$

$$P_U = \frac{12{,}600''\# + 300\# \bullet 3.33''}{2.67''} = 5{,}093\#$$

$$f_{Bmax} = \frac{5{,}093\#\bullet 2\bullet 1/0.85}{2''\bullet 2.375''} = 2{,}523 \text{ psi post to grout}$$



$f_{Bconc} = 2{,}523\bullet 2''/4'' = 1{,}262$ psi grout to concrete

Core mount okay for 6’ post spacing

Design as 2-way shear:

Three sided breakout surface

Length of perpendicular break = $2.375''+3^*C_{a1}$

Length of parallel breaks = $2''+1.5C_{a1}$

$b_o = 2.375''+3^*C_{a1}+2^*(2''+1.5C_{a1})$

$\boldsymbol{\beta} = (2.375''+3^*C_{a1})/(2''+1.5C_{a1})$

$V_{n,min}=V^*LF/\phi=5093\#^*1.6/0.75=10{,}865\#$

λ	f'c	β	αs	d	bo
1	3000	1.70922661	30	2.3923629	20.7291774

ACI Table 22.6.5.2		
vc		
Least of:	4λ√f'c	219.089023
	(2+4/β)λ√f'c	237.72472
	$(2+\alpha_s d/b_o)\lambda\sqrt{f'c}$	299.183169
	$v_c d b_o$	10865.0004

$C_{a,min}$=2.39” measured from the face of the post

=2.39”+2.375”/2=3.58” measured from the center of the post

The core mount develops 12,600”# when the post is centered 3-5/8” from the edge of the concrete.

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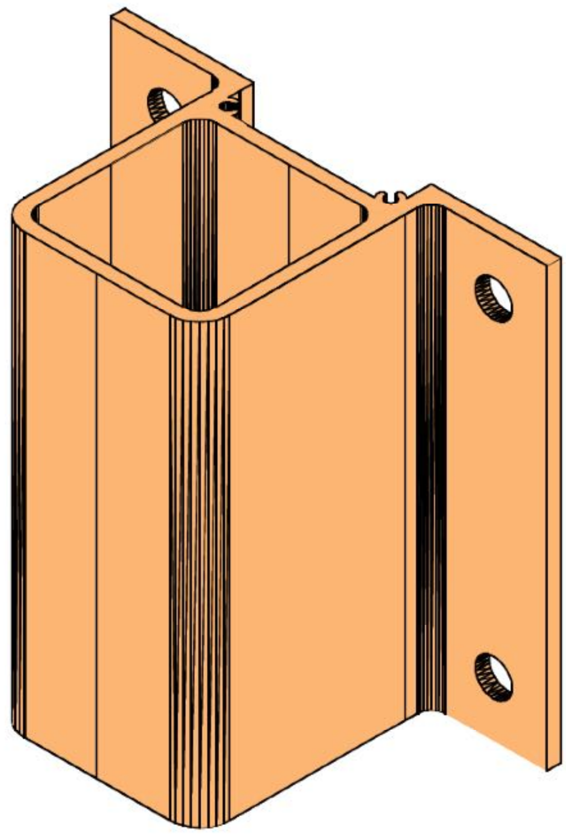
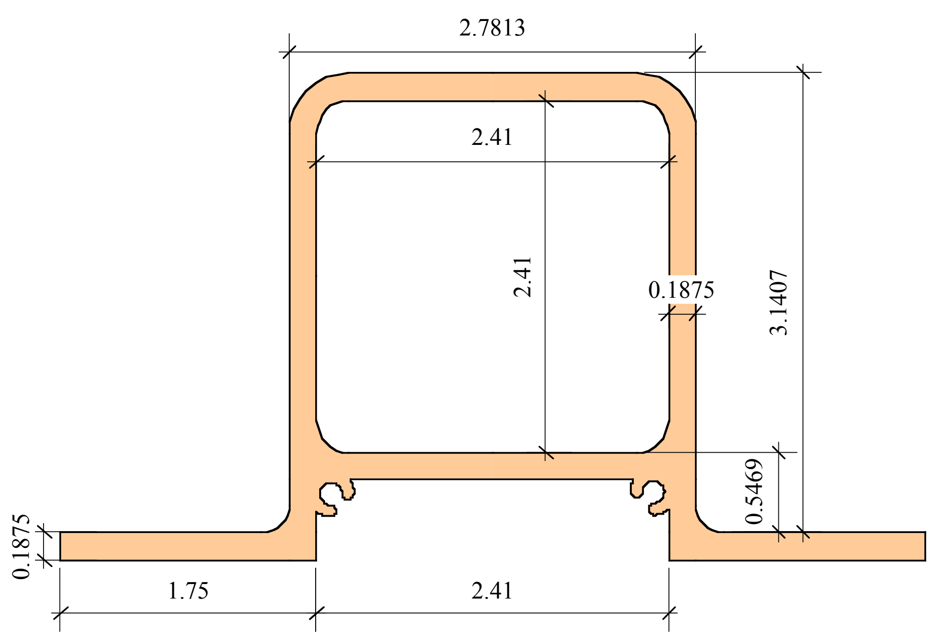
FASCIA BRACKET

Allowable stresses
ADM Table 2-21 6063-T6 Aluminum

F_t/Ω = 15.2 ksi, uniform tension
F/Ω = 22.7 ksi, flat element bending
F/Ω = 30.8 ksi, hole bearing



Section Properties
Area: 2.78 sq in
Perim: 28.99 in
I_{xx}: 3.913 in^4
I_{yy}: 5.453 in^4
C_{xx}: 1.975 in/1.353 in
C_{yy}: 2.954 in
S_{xx}: 1.981 in^3 front
S_{xx}: 2.892 in^3
S_{yy}: 1.846 in^3



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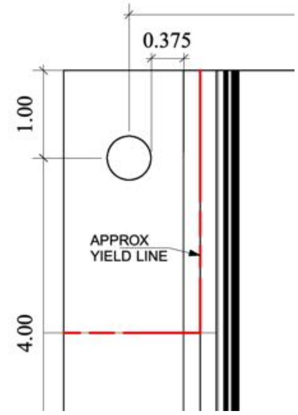
Allowable moment on bracket:

$Ma = F_t*S$

Ma_{xx} = 15.2 ksi*1.981 in^3 = 30,111"# - Outward moment

Ma_{yy} = 15.2 ksi*1.846 in^3 = 28,059"# - Sidewise moment



Flange bending strength

Determine maximum allowable bolt load:

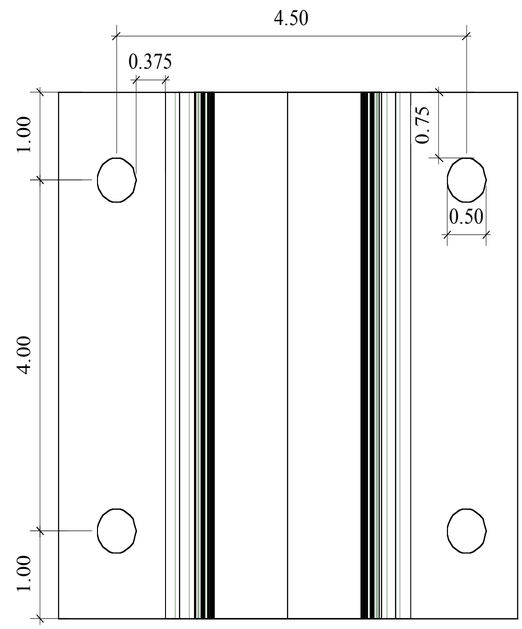
Tributary flange

b_f= 8t = 8*0.1875 = 1.5" each side of hole

b_t =1.5"+1"+0.5"+1.75" = 4.75"

S= 4.75"*0.1875^2/6=0.0278 in^3

Ma_f = 0.0278 in^3*22.7 ksi = 631"#



Allowable bolt tension

T = Ma_f/0.375 = 1,683#

3/8" bolt standard washer

For Heavy washer

T=Ma_f/0.1875= 3,366#

Typical Installation – Post load = 250# at 42" AFF – Top hole is 2" below finish floor

T_{up} = [250#*(42"+ 7")/5"]/2 bolts = 1,225# tension

T_{bot} = [250#*(42"+1")/5"]/2 bolts = 1,075# tension

For lag screws into beam face:

- 3/8" lag screw – withdrawal strength per NDS Table 12.2A

Wood species – G ≥ 0.43 – W = 243#/in

Adjustments – C_d = 1.6

No other adjustments required.

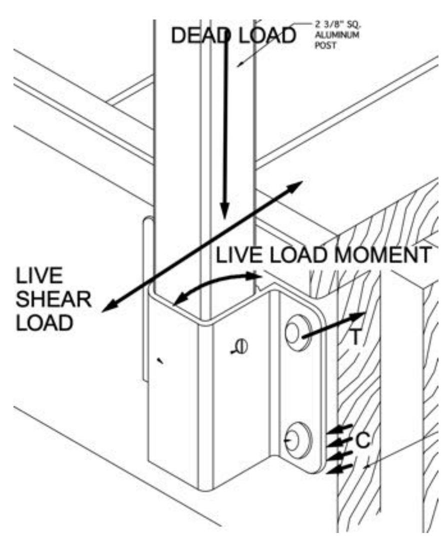
W' = 243#/in*1.6 = 389 #/in

The minimum embedment is:

l_e = 1,225#/389#/in = 3.15" : +7/32" for tip = 3.37"

When penetration = 3-3/8", M_a = 250#*42" = 10,600"# where moment is measured at the floor line.

Loads, except dead load and reactions are reversible



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Check for fascia bracket to concrete condition.
Use 3/8”x3-3/4” Hilti KB-TZ anchors.
h_{nom} = 3-1/16”, h_{ef} = 2.75”

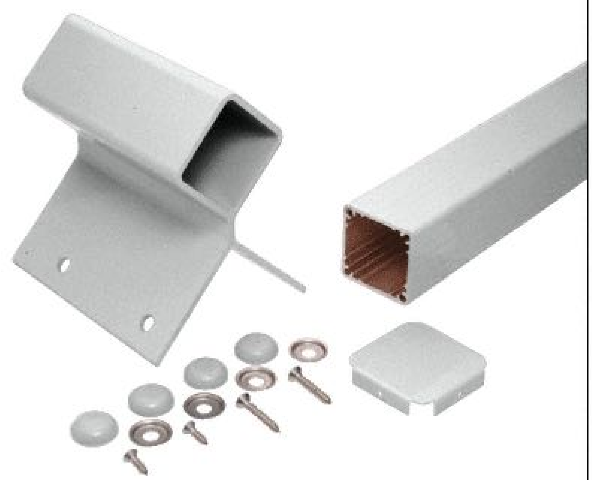
Recall, the Hilti KB-TZ with 2-3/4” effective embedment were analyzed as part of the baseplate anchorage detail. In this detail, the edge distance is reduced to 2-1/2” and the spacing is increased to 4-1/2”.

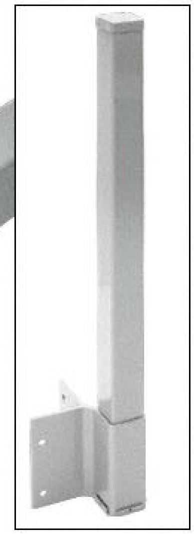
This changes A_{Ncg} to (1.5*2.75”*2+4.5”)*(1.5*2.75”+2.5”) = 84.47 in^2
and $\varphi_{ed,N}$ to 0.7+0.3*2.5”/(1.5*2.75”) = 0.88.
So,
N_{cb} = 84.47/68.1*0.88*1.0*1.0*5,990# = 6,540# for uncracked concrete
N_{cb} = 84.47/68.1*0.88*1.0*1.0*4,250# = 4,640# for cracked concrete
Determine allowable tension load on anchor pair assuming an average load factor of 1.6.
T_a = 0.65*6,540#/1.6 = 2,660# for uncracked concrete
T_a = 0.65*4,640#/1.6 = 1,890# for cracked concrete
Anchor pullout strength:
$T_{a,}$ = 2*0.65*4,110#/1.6 = 3,340#for uncracked concrete (Does not control)
T_a = 2*0.65*3,160#/1.6 = 2,570# for cracked concrete (Does not control)

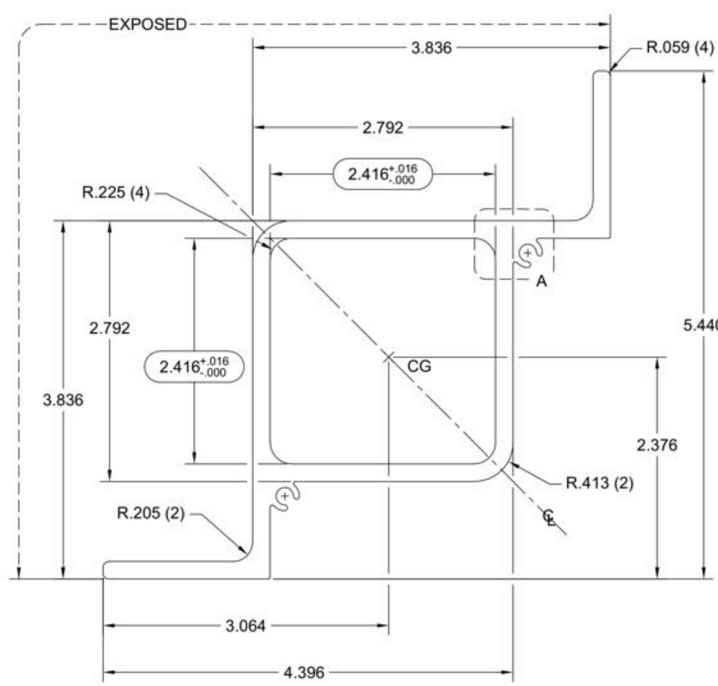
For 42” tall guard and 60” post spacing,
T_{max} = 250#*(42”+1.5”+6”)/5” = 2,480# < 2,660# but > 1,890#
OK for uncracked concrete but not for cracked concrete.
Max spacing for cracked concrete = 60”*1,890#/2,480# = 45.7”.
Find allowable moment measured at floor line:
M_a = 2,660#/2,480#*(250#*42”) = 11,300”# for uncracked concrete
M_a = 1,890#/2,480#*(250#*42”) = 8,000”# for cracked concrete
When used at 42” rail height and at spacings 48” or less. The maximum moment reaction from the 200# live load is 7,360”# < 8,000”# OK.

90˚ Corner Fascia Brackets

Loading from each direction is resisted by the adjacent anchors in tension. This bracket must be used as part of a corner rail with the welded top rail. This allows the corner post to braced and the bracket must only take half the load in each direction. Therefore, the anchors will receive the same or lower loading as the intermediate post brackets and are OK if the intermediate bracket anchors are. This is true for either the outside or inside corners.







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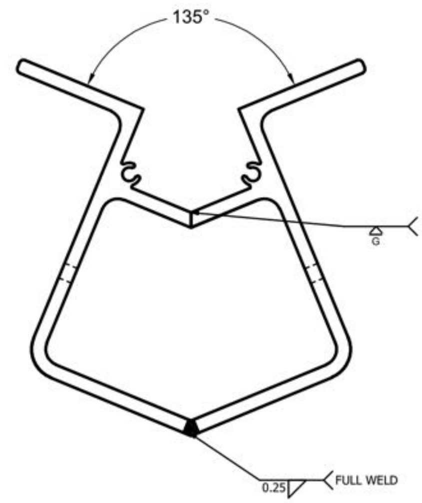
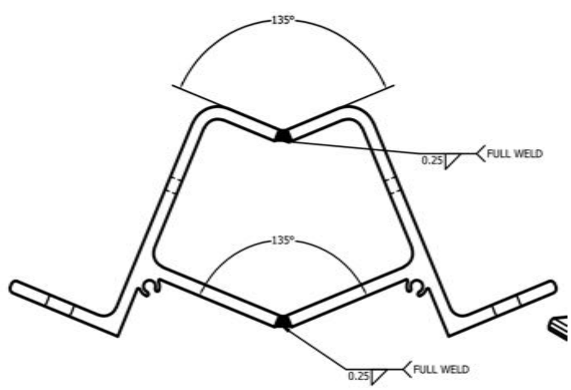
135˚ Fascia Brackets



Similar to the 90˚ corner bracket, each bracket leg can be assumed to be resisting the load tributary to the post from the same side. Therefore, the anchorage loading is the same as for the intermediate post fascia bracket without considering bracing from adjacent posts. Therefore, once bracing to adjacent posts is considered, the loading to the anchors is actually less than the loading to the intermediate fascia bracket anchors.

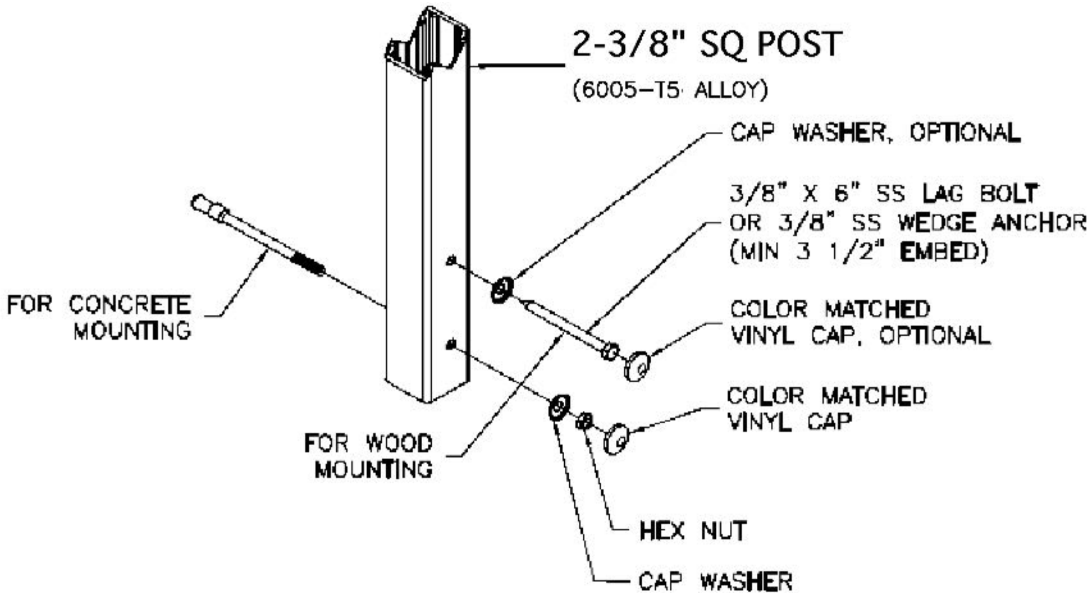
Check weld at front of fascia:
Weld rupture, T_a = 0.85*0.6*35ksi*0.707*0.25"/1.95 = 1,620pli
Base metal, 0.25"*11.6ksi/1.95 = 1,490pli (controls)
Post load resisted = 2*1,490pli*sin(45˚) = 2,110pli
M_a = 2,110pli*3"*6"/2 = 19,000"#
The fasteners will be the limiting failure mode.

Weld strength and loading are the same for either the inside or outside brackets.



FASCIA MOUNTED POST

Commercial application – Load = 200# or 50 plf any direction on top rail - 4' allowable spacing.



For 42" rail height and 4' on center post spacing:

$P = 200\#$ or $50plf*4 = 200\#$

$M_{deck} = 42''*200plf = 8{,}400''\#$

Load from glass infill lites:

Wind = 25 psf

$M_{deck} = 3.5'*25psf*42''/2*4'o.c. = 7{,}350''\#$

DL = 4'*(3 psf*3'+3.5plf)+10# = 60# each post (vertical load)

Typical anchor to wood: 3/8" lag screw. Withdrawal strength of the lags from *National Design Specification For Wood Construction* (NDS) 2015 Edition Table 12.2A.

For Doug-Fir Larch or equal, G = 0.50

W = 305 #/in of thread penetration.

$C_D = 1.6$ for guardrail live loads or wind loads.

$W' = WC_D = 305pli*1.6 = 488pli$

Lag screw design strength – 3/8" x 5" lag, $l_m = 5''-2.375''-7/32'' = 2.4''$

$T_b = 488*2.4'' = 1{,}171\#$

$Z_{ll} = 220\#$ per lag, (horizontal load) NDS Table 12K

$Z'_{ll} = 220\#*1.6 = 352\#$

Anchors to be minimum of 7" center to center and post shall extend 1-1/2" below bottom anchor.

From $\sum M$ about end

$M = (8.5''*T+1.5''*1.5/8.5*T) = 8.76''T$

Allowable post moment

$M_a=1{,}171\#*8.76'' = 10{,}300''\#$

Allowable moment measured at the floor line, M_a = 10,300"#-250#*10" = 7,800"#

When used at 42" rail height and at spacings 48" or less. The maximum moment reaction from the 200# live load is 7,360"# < 7,800"# OK.

For 3/8" carriage bolts:

Allowable load per bolt = 0.0775in^2*60ksi/2 = 2,330#

For bearing on 2" square bearing plate – area = 3.8 in^2

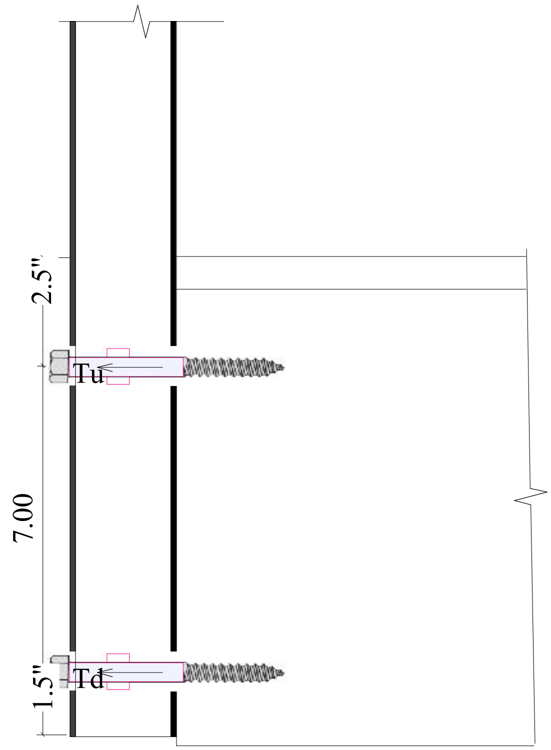
$F_{c,perp}$=405 psi for Hem-fir (Doug Fir and Southern Pine are stronger)

C_b=(2"+.375)/2"=1.19

P_b = 3.8 in^2*1.19*405*1.6 = 2,930#

M_a = 2,330#*8.76" = 20,400"# (exceeds post strength)

Allowable moment measured at the floor line, M_a = 20,400"#-300#*10" = 17,400"#



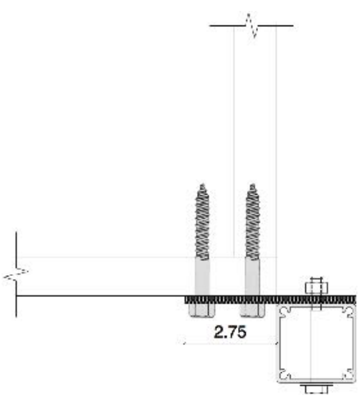
For vertical load lag capacity is:

2 lags*224# = 448#/post for D+L

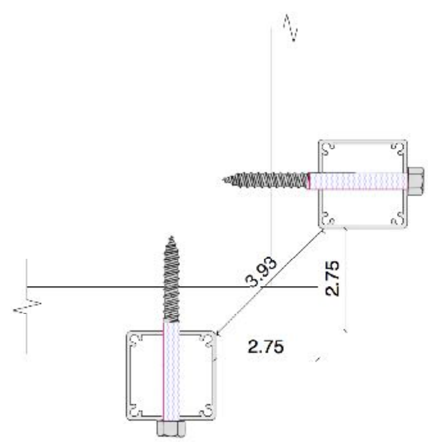
2 lags*0.9*140# = 252#/post for D

For corner posts:

For **interior corners** there are four lags, two each way. Two lags will act in withdrawal and two will be in shear: Okay be inference from running posts.



For **exterior corners** – requires either 2 posts (installed within 2" of corner each way) or 1/4"x 4"x8" mounting plate with four 3/8"x4" lag screws and two 3/8" bolts to post.



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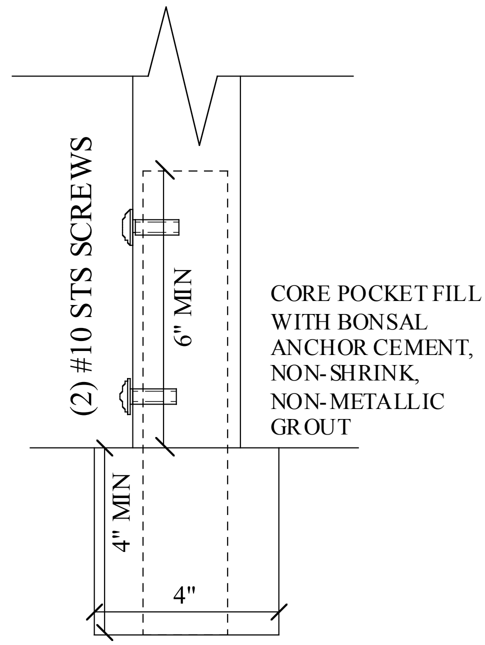
STANCHION MOUNT

Part PST5: 2”x1-1/2”x 1/8” 304 1/4 Hard Stainless steel tube
Edge distance same as for core mounted posts.
Stanchion Strength
F_{yc} = 50 ksi
Z_{yy} = 0.543 in^3
M_a = 50ksi*0.543in^3/1.67 = 16,300”#



Post may be attached to stanchion with screws or by grouting by pouring self-consolidating grout into top of post to cover the stanchion.
Grout bond strength to stanchion:

$A_{surface}\sqrt{f'c}$ = 7”*4”*√8,000 psi = 2,500#

(ignores mechanical bond) (similar to ACI318 chapter 12 development length provisions using the fundamental bond between the concrete and metal without adjustment for deformations)
for 200# maximum uplift the safety factor against pulling out:

SF = 2,500#/200# = 12.5 > 3.0 therefore okay.

The very high projected safety factor indicates that a more robust determination of the bond strength isn’t warranted.

Core mount strength is calculated by the same method shown for the post core mount:

Inputs

Core Width (in)	Stanchion Width (in)	Concrete Strength		Edge Distance	Embedment
b_c (in)	b_s (in)	f'_c (psi)	λ	C_1 (in)	d (in)
4	1.5	3000	1	3.625	4

Edge breakout calcs

w (in)	h (in)	β	b_0 (in)	α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2+4/\beta)\lambda\sqrt{f'_c}$ (psi)
7.625	3.8125	2	15.25	30	219.089023	219.089023

			Bearing Calcs				Strength
			$0.65*0.85f'_c/1.6$	$(b_c-b_s)/2+b_s$	$(b_c-b_s)/4+d/2$	$f_ab_bh_b$	$Min(P_a)*d/2$
$(2+\alpha_sC_1/b_0)\lambda\sqrt{f'_c}$ (psi)	$V_cC_1b_0$	P_a (lbs)	f_a (psi)	bb (in)	hb (in)	P_a (lbs)	M_a (in-lbs)
500.1335484	12111.5151	5677.272681	1035.9375	2.75	2.625	8287.5	11354.5454

M_a = 11,400”# when edge distance = 3-5/8” and embedment = 4”.

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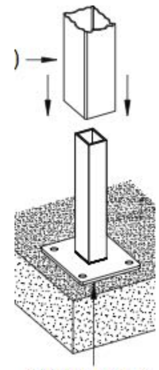
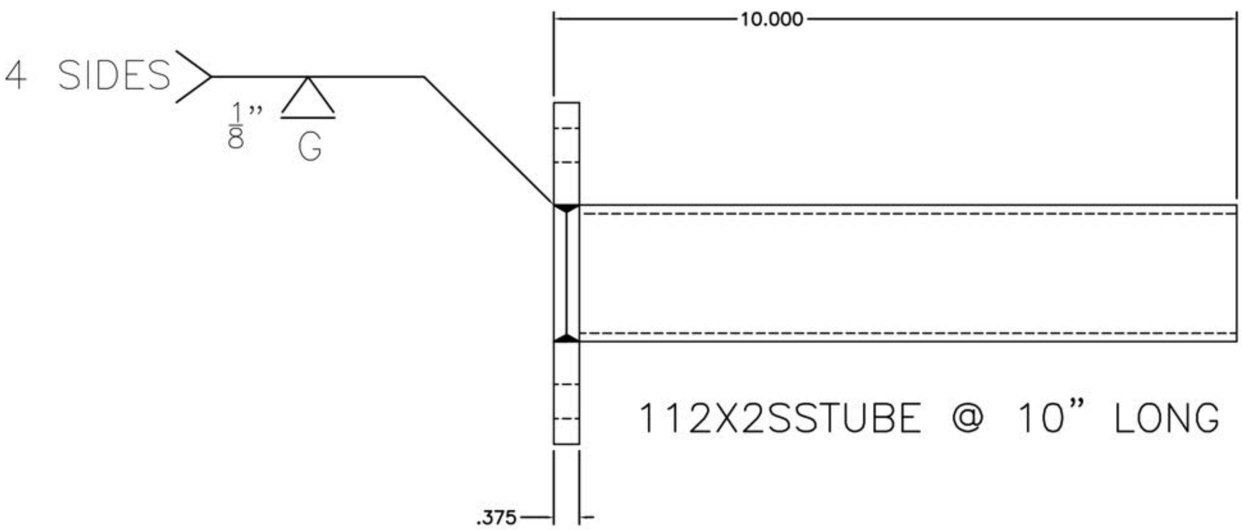
Base Plate Mounted Stanchions



2”x1-1/2”x 1/8” A304 SS Tube
Stanchion Strength
$Z_{yy} = (2^2*1.5 - 1.75^2*1.25)/4 = 0.362\ in^3$
$F_u = 75ksi$
$M_a = 0.362in^3*75ksi/2 = 13{,}600”\#$



Weld to base plate : 1/8” groove weld with reinforcing fillet weld built out to bottom face of base plate.
Minimum effective throat = $(0.125”^2+0.125”^2)^{1/2} = 0.177”$
Weld strength assumes 308L filler and loading normal to the weld’s axis.
Weld rupture, $R_n/\Omega = 0.177”*0.6*82ksi/1.88 = 4{,}630pli$ (controls)
Base metal, $R_n/\Omega = 0.125”*75ksi/2 = 4{,}690pli$
$I_{w,y} = 2*2”*0.75”^2+2*1.5”^3/12 = 2.81in^4/in$
$M_a = 4{,}630pli*2.81in^4/in/0.75” = 17{,}300”\#$ (Develops full strength of stanchion)
Since the 3/8” baseplate stanchion weld develops the full strength of the stanchion, the 1/2” baseplate weld also develops the full strength of the stanchion by inspection.

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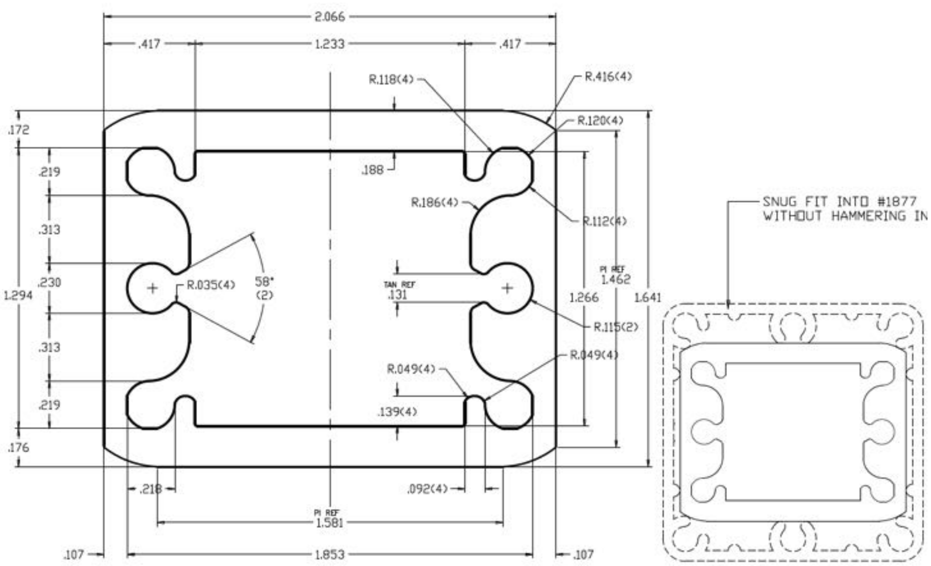
ALUMINUM STANCHION EXTRUSION



Extrusion screwed to an aluminum base plate using 6 screws and post installed over stanchion.

-Area 1.1428 in^2 I_{xx} = 0.4659 in^4 I_{yy} = 0.752 in^4

S_{xx} = 0.570 in^3 S_{yy} = 0.727 in^3 Z_{xx} = 0.707 in^3 Z_{yy} = 0.851 in^3

r_{xx} = 0.573 in r_{yy} = 0.728 in

Allowable moment = Mn/Ω per 2015 ADM

M_n = lesser of: 1.5SF_y or ZF_y and M_{nu} = ZF_u/k_t :

Ω = 1.65 for yield state, or 1.95 for rupture - M_{nu} will control as $F_u \leq 1.18\ F_y$

F_y = 35 ksi F_u = 38 ksi

Strong axis bending (typically perpendicular to rail)

M_{all} = 0.851 • 38^{ksi}/1.95 = 16,584 $^{\#”}$ = 1,382.0’# (Rupture controls)

Weak axis bending (typically parallel to rail)

M_{all} = 0.707 • 38^{ksi}/1.95 = 13,777$^{\#”}$ = 1,148.1’# (Rupture controls)

Strength of screws - same screws as used for base plates to posts-

Base plate pull over = [(0.27+1.45*0.375/0.25)*0.25*0.375*30ksi]/3 = 2,293#

Allowable moments on screwed base plate -

M_{ay} = 3screws*2,293”#*(1.797”) = 12,362”# For strong axis bending (90˚ to post’s strong axis)

M_{ax} = 2screws*2,293”#*(1.294+0.762) = 9,429”# For weak axis bending

Or when welded to baseplate:

M_a = 24ksi/1.95*0.851in^3 = 10,500”# (strong axis)

M_a = 24ksi/1.95*0.707in^3 = 8,700”# (weak axis)

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TOP RAILS

Top rails connect the tops of post together and provide a continuous barrier between posts. The 200# concentrated load and 50plf live load are applied directly to the top rail. Note for 72" max post spacing the maximum post moment is caused by the 200# concentrated load mid span:

Maximum top rail moment caused by live loading, M_{max} = 200#*72"/4 = 3,600"#
Moment caused by 50plf live load, M_{50plf} = 50plf/12*72"^{2}/8 = 2,700"# < 3,600"# (Does not control)

Therefore, allowable rail span = M_a4/200# = M_a/50#
Note that every rail is weakest under vertical loading. See the table below for allow post spacing based on top rail bending strength:

Top Rail Allowable Spans:

Top Rail		Ma (in-lbs)	Allowable Span (in)
	100	3750	72
	200	2640	52.8
200X		1790	35.8
	300	6430	72
	320	3000	60
	350	4300	72
	400	5130	72
	500	2600	52

Top rails may receive significant wind loading when glass infill is used. However, the bottom rail is much weaker and will receive the same loading. Therefore, none of the top rails shown will limit wind loading below that shown for the bottom rail. Note that the allowable span of the 200, 200X, 320 and 500 is limited by the vertical bending strength. When pickets attach to the top rail, load is shared with the bottom rail.

Check bending of 200 and 200X top rail when load can be shared with the picket bottom rail.
For series 200 top rail:
$I_{x,200}$ = 0.241in^4
$I_{x,bottom}$ = 0.119in^4
Load share to top rail = 0.241in^4/(0.241in^4+ 0.119in^4) = 0.669
M_a = 2,640"#/0.669 = 3,950"#
Allowable span > 72"

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For series 200X top rail:
$I_{x,200} = 0.132in^4$
$I_{x,bottom} = 0.119in^4$
Load share to top rail = $0.132in^4/(0.132in^4 + 0.119in^4) = 0.526$
M_a = 1,790”#/0.526 = 3,400”#
Allowable span = 3,400”#/50 = 68”.

For series 320 top rail:
$I_{x,200} = 0.118in^4$
$I_{x,bottom} = 0.119in^4$
Load share to top rail = $0.118in^4/(0.118in^4 + 0.119in^4) = 0.498$
M_a = 1,790”#/0.498 = 3,590”#
Allowable span = 3,590”#/50 = 72”.

For series 500 top rail:
$I_{x,200} = 0.257in^4$
$I_{x,bottom} = 0.119in^4$
Load share to top rail = $0.257in^4/(0.257in^4 + 0.119in^4) = 0.684$
M_a = 2,600”#/0.684 = 3,800”#
Allowable span = 3,800”#/50 > 72”.

For wood composite:
$I_{x,wood} = 0.984in^4$
$I_{x,bottom} = 0.119in^4$
Load share to top rail = $1.6/10.1*0.984in^4/(1.6/10.1*0.984in^4 + 0.119in^4) = 0.567$
$M_{a,bottom}$ = 1,770”#/0.433 = 4,090”#
$M_{a,wood}$ = 2,510”#/0.567 = 4,430”#
Allowable span = 4,090”#/50 > 72”.

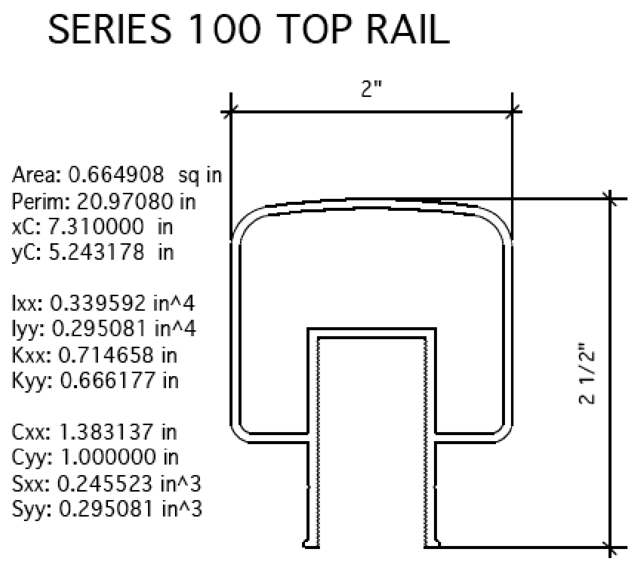
The preceding calculations show that when pickets and the picket bottom rail are used, only the 200X top rail has an allowable post spacing less than 72”.

NOTE ON SHEAR:
The maximum shear in the top rail spanning 72” is 150#. When used in a simple span the the maximum shear corresponds to the minimum moment. When used in multiple spans the maximum shear and maximum moment will coincide with the post. Since the allowable spans shown are based on simple spans the reduction in peak moment at the post will offset the added stress from the shear. Additionally all top rails will have web elements where the bending stress is minimal which will effectively resist the shear loads.
Web area required to resist 150 lb shear load:
$A_w = 150\#/(9.1 \text{ ksi}) = 0.0165 \text{ in}^2$ For 6063-T6 assumes $\lambda_1 < 16.1$ which applies to all top rails
For the minimum 0.08” thickness this requires under 1/4” of web. Thus further shear check of the top rails isn’t warranted as it clearly won’t impact the allowable spans on any of the top rails.

Series 100 Top Rail

Butts into post
Alloy 6063 – T6 Aluminum

First calculate bending strength under vertical loading:



Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 100 Top Rail Vertical Bending

Section Properties

Ix (in4)	0.339
Sx (in3)	0.251
Zx (in3)	0.403
Iy (in4)	0.291
J (in4)	0.19
b	1.55
t	0.065

Cw (in6)	0.118
βx (in)	-1.946
g0 (in)	-0.1

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	23.8461538 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	14.9461538 =	15.2		*for λ<λ1*
		19-0.17λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	3751 =	*F/Ω*1000(kips/lbs)*Sx*		

Rupture Strength

Fu/Ω	15.3846154			
Znet	0.403			
Mn/Ω (in-lbs)	6200 =	*Znet*Fu/Ω*1000kips/lbs*		

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5	**C2**	0.5	
U (in)	0.4365 =	*C1*g0-C2*βx/2*		
Me	151.717395 =	*See 2015 ADM F.4-9*		
λ	12.8419133 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*		
λ<Cc, inelastic buckling applies				
Mnmb (in-kip	9.09333409 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*		
Ma (in-lbs)	5511.11157 =	*Mnmb/1.65*1000*		

Strength is controlled by local buckling

Ma (in-lbs) **3751**

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First calculate bending strength under horizontal loading:

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 100 Top Rail Horizontal Bending

Section Properties

Ix (in4)	0.291
Sx (in3)	0.291
Zx (in3)	0.4
Iy (in4)	0.339
J (in4)	0.19
b	1.186
t	0.065

Cw (in6)	0.118
βx (in)	0
g0 (in)	-1

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

λ	18.2461538	=	*b/t*	
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2	=	15.2	*for λ<λ1*
			19-0.17λ	*for λ1<λ<λ2*
			484/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	6080	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154		
Znet	0.4		
Mn/Ω (in-lbs)	6153.84615	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	42			
Cb	1.3			
C1	0.5		C2	0.5
U (in)	-0.5	=	*C1*g0-C2*βx/2*	
Me	141.403678	=	*See 2015 ADM F.4-9*	
λ	14.3227596	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

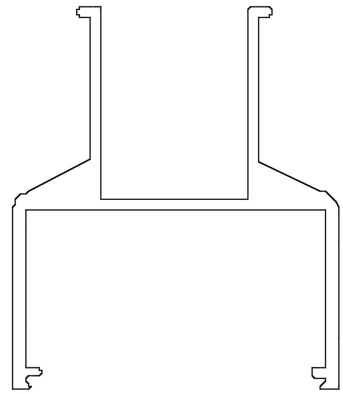
λ<Cc, inelastic buckling applies

Mnmb (in-kip	9.03925049	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	5478.33363	=	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 5478

SERIES 100 BOTTOM RAIL



First check strength under vertical loading. Generally the bottom rail will not receive significant vertical loading because the top rail is normally much stiffer.

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion Series 100 Bottom Rail Vertical Bending

Section Properties

Ix (in4)	0.091
Sx (in3)	0.091
Zx (in3)	0.146
Iy (in4)	0.166
J (in4)	0.002
b	0.862
t	0.076

Cw (in6)	0.034
βx (in)	-0.613
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element supported on one edge under flexural loading with compression edge free

λ	11.3421053 =	*b/t*	
λ1	6.5		
λ2	23		
F/Ω (ksi)	18.7128947 =	22.7	*for λ<λ1*
		27.9-0.810λ	*for λ1<λ<λ2*
		4932/λ^2	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S<Fy/Ω*Z*

Mn/Ω (in-lbs)	1703 =	*F/Ω*1000(kips/lbs)*Sx<Fy/Ω*Zx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.146	
Mn/Ω (in-lbs)	2246.15385 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0.15325 =	*C1*g0-C2*βx/2*	
Me	6.42711642 =	*See 2015 ADM F.4-9*	
λ	37.5684409 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	2.61011765 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	1581.88948 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **1582**

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Next check flexural strength under horizontal loading. The bottom rail may receive horizontal loading from wind loading on the infill or from the 50# concentrated infill load.

Aluminum Extrusion Flexural Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 100 Bottom Rail Horizontal Bending

Section Properties

Ix (in4)	0.166
Sx (in3)	0.195
Zx (in3)	0.258
Iy (in4)	0.091
J (in4)	0.002
b	0.862
t	0.076

Cw (in6)	0.034
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element supported on one edge under uniform loading

λ	11.3421053 =	*b/t*		
λ1	7.3			
λ2	12.6			
F/Ω (ksi)	12.9886842 =	15.2	*for λ<λ1*	
		19.0-0.530λ	*for λ1<λ<λ2*	
		155/λ	*for λ2<λ*	

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S<Fy/Ω*Z*

Mn/Ω (in-lbs)	2533 =	*F/Ω*1000(kips/lbs)*Sx<Fy/Ω*Zx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.258	
Mn/Ω (in-lbs)	3969.23077 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	4.32608891 =	*See 2015 ADM F.4-9*	
λ	67.0316863 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	3.65268875 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	2213.75076 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **2214**

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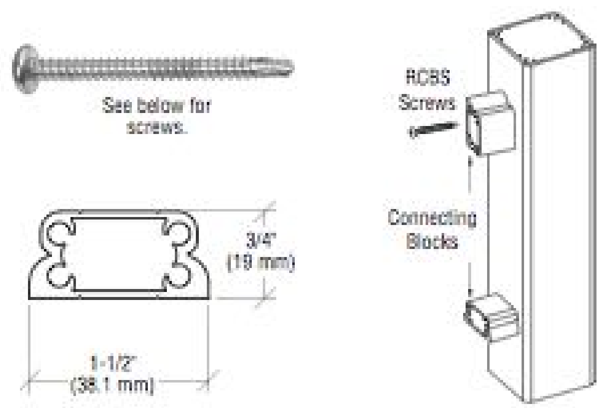
Rail attached to the posts using the Rail Connecting Blocks (part RCB1)
Rail fasteners -Bottom rail connection block to post #10x2" 55 PHP SMS Screw
CRL Part RCBS
Typical RCB length: $1 \leq L \leq 1.5$

Check shear @ post (6005-T5)



2x F_{upost}x dia screw x Post thickness x SF

$$V = 2 \cdot 38\text{ ksi} \cdot 0.1697" \cdot 0.10" \cdot \frac{1}{3\text{ (FS)}} =$$

V = 430#/screw

screw shear strength
shear area = 0.0226 in^2
$V_n = 0.0226\text{in}^2 * 0.5 * 80\text{ksi} = 904$
$V_a = 904/2 = 452\#$

Screw tilting:
$4.2(t_2{}^3D)^{1/2}F_{tu2} =$
$4.2(0.1^3 0.1697)^{1/2} 38\text{ ksi} = 2{,}079$
Screw tilting won't control for any of connections to the posts

Since minimum of 2 screws used for each
Allowable load = $2 \cdot 430\# = 860\#$

Rail Connection to RCB

2 screws each end
#8 Tek screw CRL Part TEK1 to 6063-T6

$$V = 2 \cdot 38\text{ ksi} \cdot 0.1309" \cdot 0.07" \cdot \frac{1}{3\text{ (FS)}} = 232\#$$

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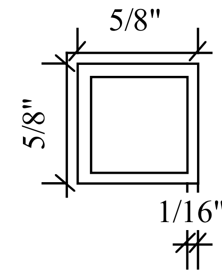
Picket Railing

5/8” Square pickets
Wall thickness = 0.062”

Bending strength is the same about both axis.



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	5/8" Picket

Section Properties

Ix (in4)	0.00662
Sx (in3)	0.0212
Zx (in3)	0.0268
Iy (in4)	0.00662
J (in4)	0.0113
b	0.5
t	0.06

Cw (in6)	0.000493
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

Note on shear: Shear will be carried by the web elements with bending resisted by the flange elements. As previously shown un 1/4” of web is required to resist the maximum design shear load thus further shear checks aren’t warranted.

Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	8.33333333 =	*b/t*	
λ1	22.8		
λ2	39		
F/Ω (ksi)	15.2 =	15.2	*for λ<λ1*
		19-0.170λ	*for λ1<λ<λ2*
		484/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	407 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.0268	
Mn/Ω (in-lbs)	412.307692 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	48		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	3.99257358 =	*See 2015 ADM F.4-9*	
λ	23.006578 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	0.57483252 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	348.383343 =	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **348**

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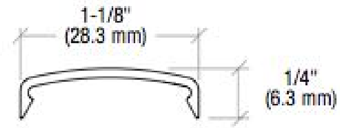
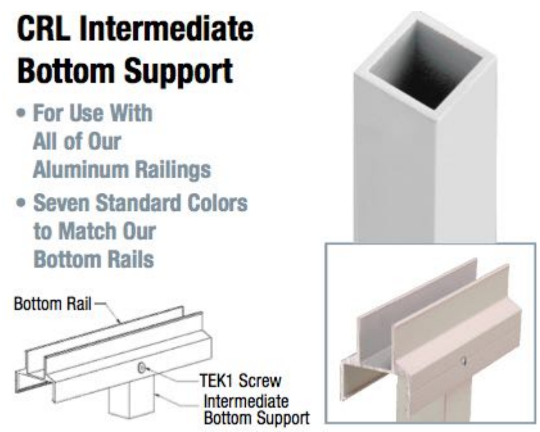
Connections



Pickets to top and bottom rails direct bearing –ok
Lap into top and bottom rail – 1" into bottom rail and 5/8" into top rail.

Allowable bearing pressure = 20.5 ksi (ADM Table 2-2)
Picket filler snaps between pickets to pressure lock pickets in place. Bearing surface = 1.375"*.062" = 0.085 in^2
Allowable bearing = 0.085 in^2*20.5 ksi = 1,743#
Withdrawal prevented by depth into rails.

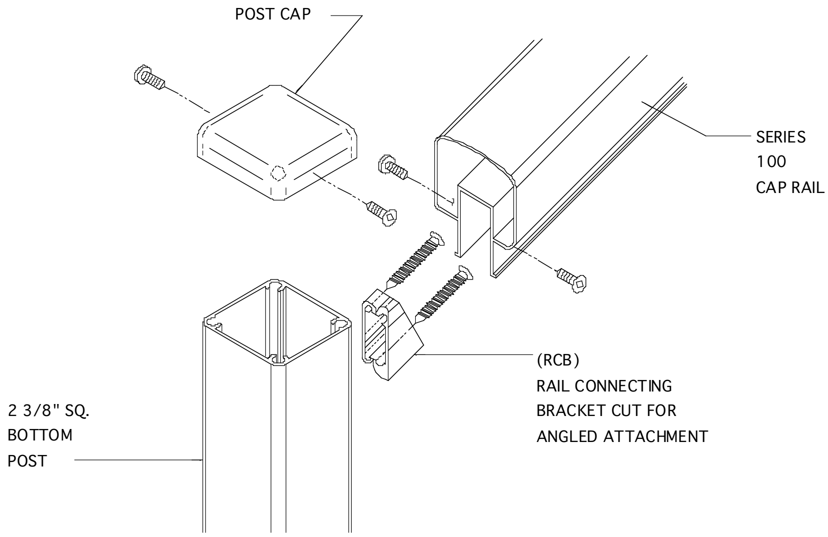
Intermediate post used to provide additional support to bottom rail.
1.4" square 0.1" wall thickness
Acts in compression only.
Secured to rail with two #8 tek screws
Shear strength of screws:



Top rail connection to post face:
Use RCB attached to post with two #10 screws same as bottom rail.

$V = 2 \cdot 38\text{ ksi} \cdot 0.19" \cdot 0.10" \cdot \frac{1}{3\text{ (FS)}} =$

V = 481#/screw



Since minimum of 2 screws
used for each
Allowable load =
$2 \cdot 481\# = 962\#$

screw shear strength
shear area = 0.0226 in^2
$V_n = 0.0226\text{in}^2 * 0.6 * 80\text{ksi} =$
1084
$V_a = 1084/2 = 542\#$

Screw tilting:
$4.2(t_2{}^3 D)^{1/2} F_{tu2} =$
$4.2(0.1^3 0.19)^{1/2} 38\text{ ksi} = 2{,}200$
Screw tilting won't control for any of connections to the posts.

The connection block can be cut square for use in horizontal rail applications or angled for use in sloped applications such as along stairs or ramps.

Connection of rail to RCB is with (2) #8 Tek screw to 6063-T6

$V = 2 \cdot 30\text{ ksi} \cdot 0.164" \cdot 0.07" \cdot \frac{1}{3\text{ (FS)}} = 239\#$

$V_{tot} = 2 * 230\# = 460\#$

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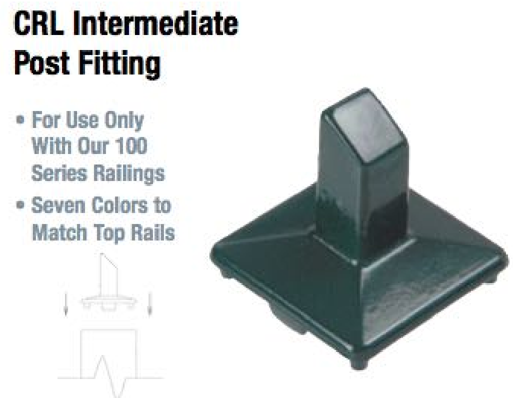
Intermediate post fitting

Used for intermediate posts along stairways

Fitting locks into top of post using structural silicone.



Maximum load on fitting is 300#

6' post spacing * 50 plf = 300#

Shear resisted by direct bearing between fitting and post

area = 2.175"*0.1875 = 0.408 in^2

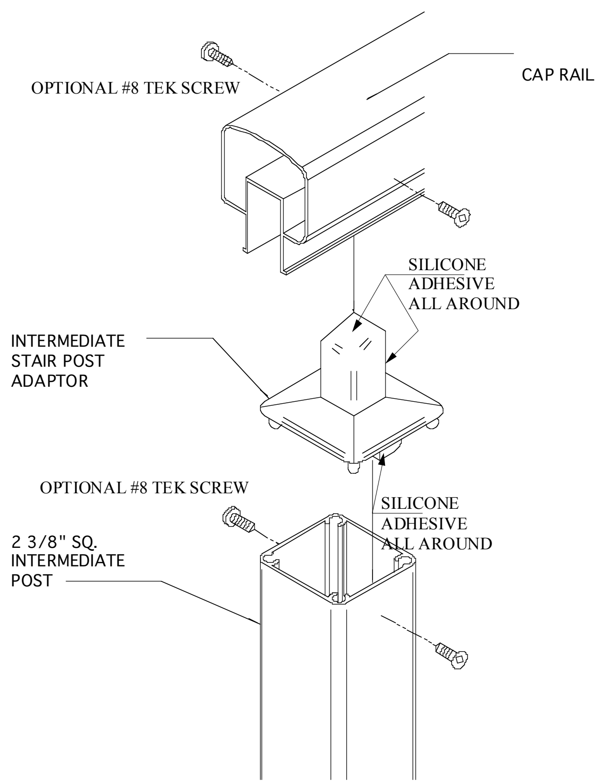
Bearing pressure = 300#/.408 = 736 psi

Moment of fitting to post:

This is an intermediate post with rotation of top rail restrained at rail ends.

Moment of fitting is created by eccentricity between bottom of top rail and top of post: e = 0.425"

M = 300# * (0.425") = 127.5#"



Use screws for positive connection

#8 Tek screws:

Shear strength =

$$V = 2 \cdot 30\text{ ksi} \cdot 0.164" \cdot 0.1" \cdot \frac{1}{3\text{ (FS)}} = 328\#$$

$$V = 0.246 \cdot 30\text{ ksi} \cdot 0.1" \cdot \frac{1}{3\text{ (FS)}} = 246\#$$

Vs = 0.0162in^2*0.5*80ksi/2 = 324#

Moment resistance = 246#*2.375" = 584#"

Screws are located 1.5d from top of post:

(ADM J.5.3)

s = 1.5*0.164 = 0.246 (1/4")

Screw will be at least 1/4" from bottom of the shear tab too as shear tab is 5/8" long

Strength of shear tab:

A = 0.75"*0.15" = 0.1125

Casting strength: A356.0 T61 F_{tu} = 28 ksi $F_{uv} = 0.6F_{tu}$

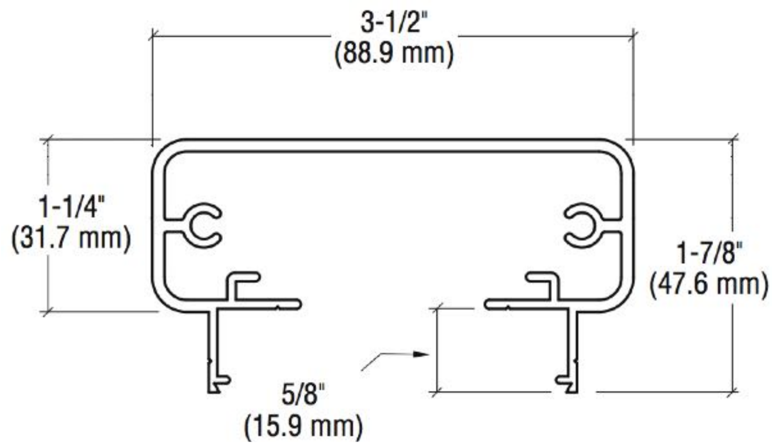
T_a = 0.1125*28 ksi/1.95 = 1,615#

V_a = 0.1125*0.6*28 ksi/1.95 = 969#

Shear load will be primarily resisted by tab on compression face.

Series 200 Top rail

First calculate bending strength under vertical loading:



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion Series 200 Top Rail Vertical Loading

Section Properties

Ix (in4)	0.241
Sx (in3)	0.201
Zx (in3)	0.398
Iy (in4)	1.43
J (in4)	0.00159
b	3
t	0.087

Cw (in6)	0.978
βx (in)	-3.61
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	34.4827586 =	*b/t*	
λ1	22.8		
λ2	39		
F/Ω (ksi)	13.137931 =	15.2	*for λ<λ1*
		19-0.170λ	*for λ1<λ<λ2*
		484/λ	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	2641 =	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.398	
Mn/Ω (in-lbs)	6123.07692 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0.9025 =	*C1*g0-C2*βx/2*	
Me	69.3716157 =	*See 2015 ADM F.4-9*	
λ	16.9948636 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	8.49961363 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	5151.28099 =	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) 2641

Next calculate bending strength under horizontal loading:

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 200 Top Rail Horizontal Loading

Section Properties

Ix (in4)	1.43
Sx (in3)	0.818
Zx (in3)	1.03
Iy (in4)	0.241
J (in4)	0.00159
b	0.75
t	0.087

Cw (in6)	0.978
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	8.62068966 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =		15.2	*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	15656 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	1.03	
Mn/Ω (in-lbs)	15846.1538 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	12.2283867 =	*See 2015 ADM F.4-9*	
λ	81.6587316 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ>Cc elastic buckling applies

Mnmb (in-kip	12.8232988 =	*π^2*E*Sx/λ^2*
Ma (in-lbs)	7771.69622 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) ***7772***

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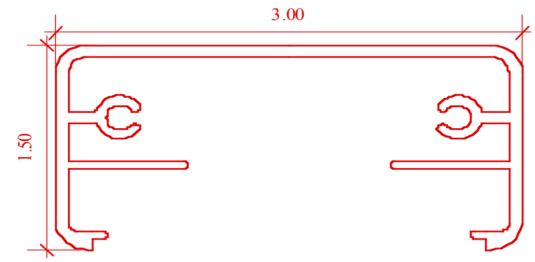
SERIES 200X TOP RAIL



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 200X Top Rail Vertical Loading

Section Properties

Ix (in4)	0.132
Sx (in3)	0.137
Zx (in3)	0.246
Iy (in4)	0.85
J (in4)	0.000927
b	2.57
t	0.074

Cw (in6)	0.271
βx (in)	-3.51
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	34.7297297 =	*b/t*	
λ1	22.8		
λ2	39		
F/Ω (ksi)	13.0959459 =	15.2	*for λ<λ1*
		19-0.170λ	*for λ1<λ<λ2*
		484/λ	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	1794 =	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.246	
Mn/Ω (in-lbs)	3784.61538 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0.8775 =	*C1*g0-C2*βx/2*	
Me	37.6250385 =	*See 2015 ADM F.4-9*	
λ	19.05164 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	5.19611611 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	3149.16128 =	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) **1794**

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Next calculate bending strength under horizontal loading:

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 200X Top Rail Horizontal Loading

Section Properties

Ix (in4)	0.85
Sx (in3)	0.567
Zx (in3)	0.681
Iy (in4)	0.132
J (in4)	0.000927
b	1.11
t	0.074

Cw (in6)	0.271
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	15 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =	15.2		*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	10351 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.681	
Mn/Ω (in-lbs)	10476.9231 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	5.36401535 =	*See 2015 ADM F.4-9*	
λ	102.649558 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ>Cc elastic buckling applies

Mnmb (in-kip	6.84556464 =	*π^2*E*Sx/λ^2*
Ma (in-lbs)	4148.82705 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **4149**

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Series 300 Top Rail

Aluminum Extrusion Design

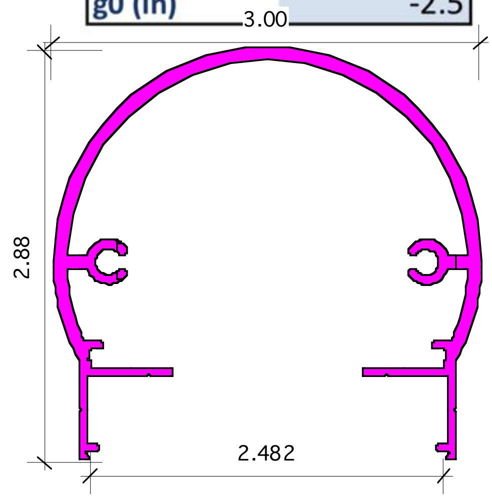
Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 300 Top Rail Vertical Loading

Section Properties

Ix (in4)	0.574
Sx (in3)	0.364
Zx (in3)	0.583
Iy (in4)	1.07
J (in4)	0.00168
b	1.41
t	0.086

Cw (in6)	0.906
βx (in)	-5.46
g0 (in)	-2.5



Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	16.3953488 =	*Rb/t*	
λ1	70		
λ2	189		
F/Ω (ksi)	20.816501 =	27.7-1.70λ^0.5	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	8273 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.583	
Mn/Ω (in-lbs)	8969.23077 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0.115 =	*C1*g0-C2*βx/2*	
Me	28.0602538 =	*See 2015 ADM F.4-9*	
λ	35.9596498 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	10.6051194 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	6427.34509 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 6427

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 300 Top Rail Horizontal Loading

Section Properties

Ix (in4)	1.07
Sx (in3)	0.712
Zx (in3)	0.583
Iy (in4)	0.574
J (in4)	0.00168
b	1.41
t	0.086

Cw (in6)	0.906
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	16.3953488 =	*Rb/t*		
λ1	70			
λ2	189			
F/Ω (ksi)	20.816501 =	27.7-1.70λ^0.5	*for λ<λ1*	
		18.5-0.593λ^0.5	*for λ1<λ<λ2*	
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*	

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	8833 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.583	
Mn/Ω (in-lbs)	8969.23077 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	18.4710844 =	*See 2015 ADM F.4-9*	
λ	61.9875341 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	12.262968 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	7432.10182 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **7432**

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Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 300X Top Rail Vertical Loading

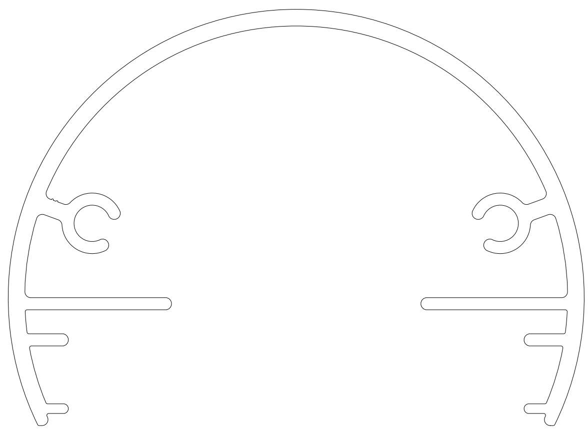
Section Properties

Ix (in4)	0.288
Sx (in3)	0.252
Zx (in3)	0.395
Iy (in4)	0.949
J (in4)	0.00166
b	1.5
t	0.087

Cw (in6)	0.312
βx (in)	-4.409
g0 (in)	1

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78



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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	17.2413793 =	*Rb/t*	
λ1	70		
λ2	189		
F/Ω (ksi)	20.6411342 =	27.7-1.70λ^0.5	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	5727 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.395	
Mn/Ω (in-lbs)	6076.92308 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	1.60225 =	*C1*g0-C2*βx/2*	
Me	70.7803983 =	*See 2015 ADM F.4-9*	
λ	18.8388474 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	8.48717637 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	5143.74325 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **5144**

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Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion Series 300X Top Rail Vertical Loading

Section Properties

Ix (in4)	0.949
Sx (in3)	0.632
Zx (in3)	0.792
Iy (in4)	0.288
J (in4)	0.00166
b	1.5
t	0.087

Cw (in6)	0.312
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	17.2413793 =		*Rb/t*	
λ1	70			
λ2	189			
F/Ω (ksi)	20.6411342 =		27.7-1.70λ^0.5	*for λ<λ1*
			18.5-0.593λ^0.5	*for λ1<λ<λ2*
			3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	12000 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*	

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.792	
Mn/Ω (in-lbs)	12184.6154 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	9.40394183 =	*See 2015 ADM F.4-9*	
λ	81.8491421 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ>Cc elastic buckling applies

Mnmb (in-kip	9.88888057 =	*π^2*E*Sx/λ^2*
Ma (in-lbs)	5993.26095 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **5993**

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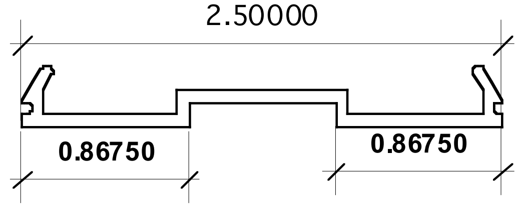
Top Rail Infill for Pickets

$I_{yy} = 0.144\ in^4$ $I_{xx} = 0.0013in^4$
$S_{yy} = 0.115\ in^3$ $S_{xx} = 0.0057\ in^4$
$Z_{yy} = 0.161in^3$

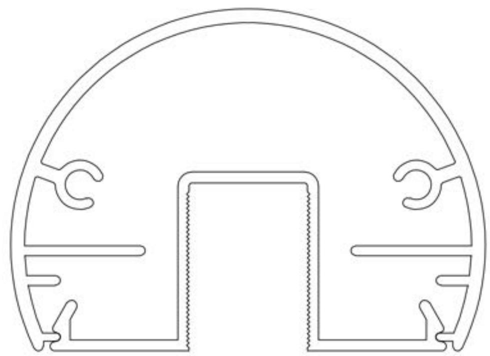


Infill for pickets provides negligible contribution to vertical load resistance but provides some horizontal load resistance.

b/t = 2.5”/0.063” = 39.7 > 34.7 (Flat element supported on both edges)
F/Ω = 27.9-0.150*39.7 = 21.9ksi
M_a = 21.9ksi*0.115in^3 = 2,520”# (For local buckling)
M_a = 15.2ksi*0.161in^3 = 2,450”# (For yielding) (controls)

Top Rail Infill for Glass

$I_{yy} = 0.1657\ in^4$ $I_{xx} = 0.06062in^4$
$S_{yy} = 0.1321\ in^3$ $S_{xx} = 0.08607\ in^4$
$Z_{yy} = 0.2087in^3$ $Z_{xx} = 0.1232in^3$



b/t = 1”/0.06” = 16.7 < 22.8 (Local buckling doe snot control for any of the elements)

Vertical bending:
$M_{a,x}$ = 15.2ksi*0.1232in^3 = 1,870”#

Horizontal bending:
Note $1.5S_{yy} < Z_{yy}$.
$M_{a,y}$ = 1.5*0.1321in^3*15.2ksi = 3,010”#

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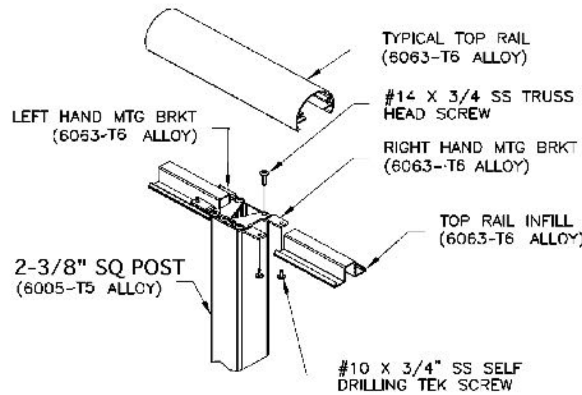
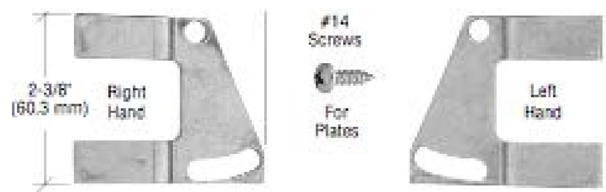
Adjustable Fastening Plates for Top Rails
Top rail connection to post:



For Vertical loads top rail is restrained by (2) #10 Tek screws each side.
Connection of bracket to post is with (2) #14 screws so is stronger.

For horizontal loads the top rail directly bears on side of post.

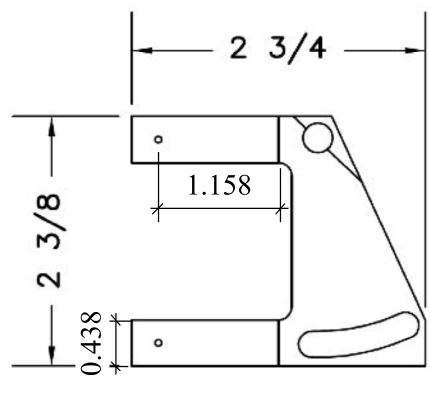
Tek screw strength: Check shear @ rail (6063-T6)
2x F_{urail}x dia screw x Rail thickness x SF
V= 2·30 ksi ·0.19" · 0.09"/3 = 342#/screw
Pullout strength:
$R_a = K_sDL_eF_{ty2}/\Omega$= 1.2*0.19*0.125*30ksi/3 = 285#



Since minimum of 2 screws used for each
Allowable load = 2· 342# = 684# Horiz.
Allowable uplift = 2*285# = 570#

Post bearing strength
$V_{all} = A_{bearing}*F_B$
$A_{bearing}$ = 0.09"*2.25" = 0.2025 in^2 F_B = 20.5 ksi
V_{all} = 0.2025 in^2 * 20.5 ksi = 4.15 k

Bracket tab bending strength
Vertical uplift force
For 6061-T6 aluminum stamping 1/8" thick
Check for shear rupture of tabs:
V_a=.438"*.125"*12.7ksi = 695# each



LHB

Tab uplift bending:
M_a= 0.438*0.125^2/4*30ksi/1.95 = 26"# each
Uplift resistance: at edge of screw head L =1.158-.2 = 0.958
U_a = 26/(0.958/2)*2 = 109# per bracket, 218# per post > 200#
Tab bending controls for uplift
Tension strength of #14 screws into post screw slot:
$R_n = 0.29DL_eF_{tu}$ = 0.29*0.25*(.75-.125-0.056)*38ksi = 1568
R_a = 1568/3 = 523#
Prying = 109*(1.158+0.625)/0.31 = 627# ≤ 2*523# OK

Screw strength controls connection strength:
P_a=2*295# = 590# > 300# OK

Strengths of Tek screws:
#10: T_a =885 , V_a = 573
#14: T_a =1605 , V_a = 990

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RAIL SPLICES:

Splice plate strength: 6061-T6 bars 5/8 x 1/8

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

$V = 2\cdot 30\text{ ksi}\cdot 0.19''\cdot 0.09''\cdot \frac{1}{3\text{ (FS)}} = 342\#/\text{screw}$

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

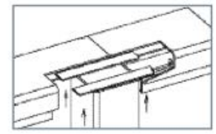
$V = 38\text{ ksi}\cdot 0.19''\cdot 0.125''\cdot \frac{1}{3\text{ (FS)}} = 300\#/\text{screw}$; for two screws = 600#

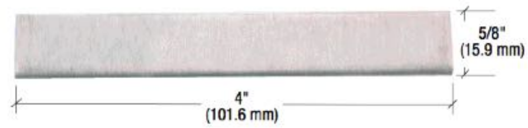
Pullout strength:

$R_a = K_s D L_e F_{ty2}/\Omega = 1.2*0.19*0.125*38\text{ksi}/3 = 361\#$

CRL 200 and 300 Series Splice Plates

- For Use with 200 and 300 Series Railings
- Aluminum Splice Plates Fit Under Top Rail to Splice Top Rails Together Over a Post





Post to splice plate:

1/4" Tek screws into post screw chase so screw to post connection will not control (see previous page).

splice plate screw shear strength

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

$V = 2\cdot 38\text{ ksi}\cdot 0.25''\cdot 0.125''\cdot \frac{1}{3\text{ (FS)}} = 792\#/\text{screw}$

Check moment from horizontal load:

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

$M = 0.75*200 = 150\#''$

$S = 0.125*(0.625)^2/6 = 0.008\text{ in}^3$

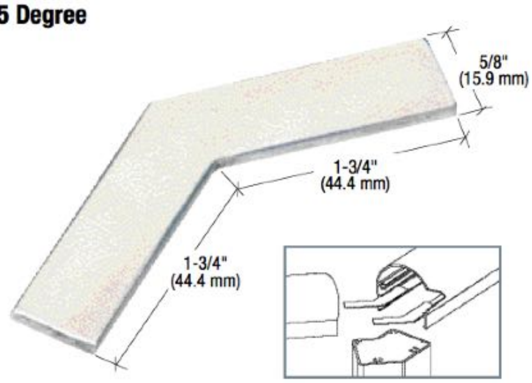
$f_b = 150\#''/(0.008*2) = 9{,}216\text{ psi} > 31.8\text{ksi OK}$

Vertical load:

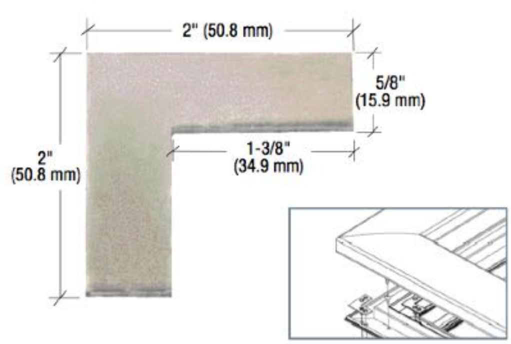
$M_a = 2*5/8''*0.125^2/4*38\text{ksi}/1.95 = 95''\#$

Allowable uplift = 95"#(1.125"/2) = 169# each bracket

45 Degree



90 Degree



For corner brackets screw strength and bending strength will be the same.

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Series 320 Top Rail

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

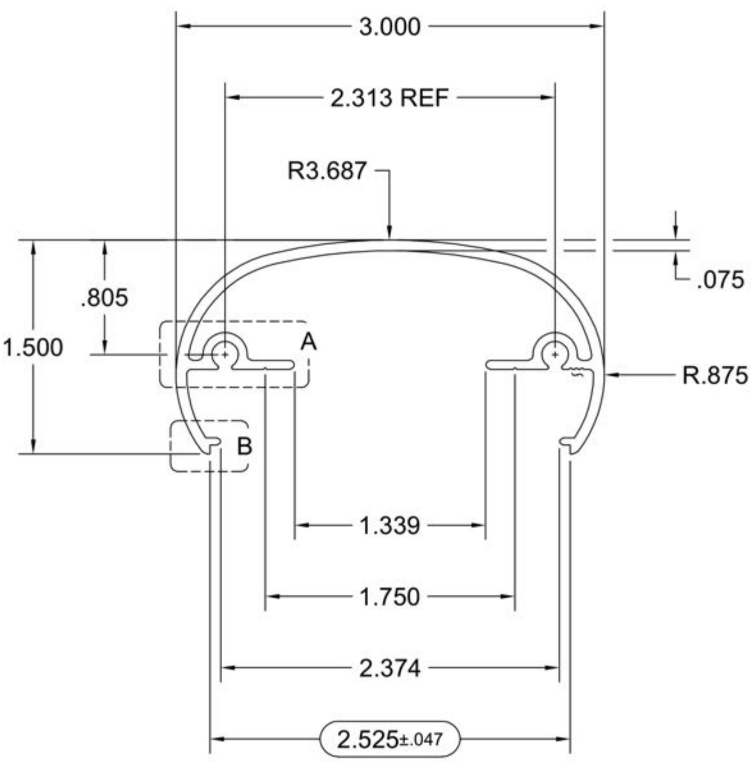
System ARS

Extrusion Series 320 Top Rail Vertical Loading

Section Properties

Ix (in4)	0.118
Sx (in3)	0.201
Zx (in3)	0.243
Iy (in4)	0.796
J (in4)	0.00157
b	3.59
t	0.1

Cw (in6)	0.0741
βx (in)	-3.66
g0 (in)	0



Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round hollow elements under uniform compression

λ	35.9	=	*Rb/t*	
λ1	31.2			
λ2	189			
F/Ω (ksi)	14.9469451	=	15.2	*for λ<λ1*
			18.5-0.593λ^0.5	*for λ1<λ<λ2*
			3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	3004	=	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	15.3846154		
Znet	0.243		
Mn/Ω (in-lbs)	3738.46154	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72			
Cb	1.14			
C1	0.5		C2	0.5
U (in)	0.915	=	*C1*g0-C2*βx/2*	
Me	36.0048031	=	*See 2015 ADM F.4-9*	
λ	23.5900232	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	5.23370562	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	3171.9428	=	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) **3004**

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Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion Series 320 Top Rail Horizontal Loading

Section Properties

Ix (in4)	0.796
Sx (in3)	0.531
Zx (in3)	0.669
Iy (in4)	0.118
J (in4)	0.00157
b	0.775
t	0.1

Cw (in6)	0.0741
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round hollow elements under uniform compression

λ	7.75 =	*Rb/t*	
λ1	31.2		
λ2	189		
F/Ω (ksi)	15.2 =	15.2	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	10136 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.669	
Mn/Ω (in-lbs)	10292.3077 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	4.66246149 =	*See 2015 ADM F.4-9*	
λ	106.549199 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ>Cc elastic buckling applies

Mnmb (in-kip	5.76292313 =	*π^2*E*Sx/λ^2*
Ma (in-lbs)	3492.68069 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 3493

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Series 350 Top Rail

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 350 Top Rail Vertical Loading

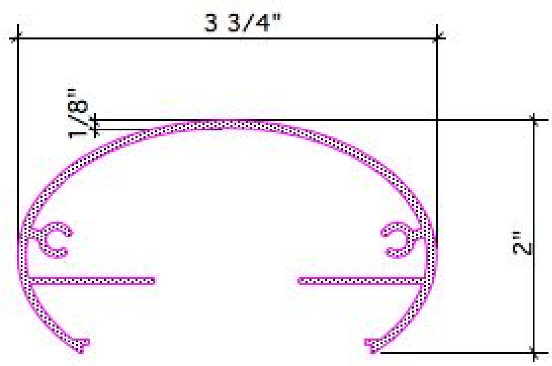
Section Properties

Ix (in4)	0.249
Sx (in3)	0.224
Zx (in3)	0.355
Iy (in4)	1.29
J (in4)	0.00101
b	2.82
t	0.07

Cw (in6)	0.719
βx (in)	-4.57
g0 (in)	-1.82

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78



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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	40.2857143 =	*Rb/t*	
λ1	70		
λ2	189		
F/Ω (ksi)	16.9099252 =	27.7-1.70λ^0.5	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	5091 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.355	
Mn/Ω (in-lbs)	5461.53846 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0.2325 =	*C1*g0-C2*βx/2*	
Me	31.318508 =	*See 2015 ADM F.4-9*	
λ	26.7014002 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	7.09323514 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	4298.93039 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **4299**

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Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 350 Top Rail Horizontal Loading

Section Properties

Ix (in4)	1.29
Sx (in3)	0.692
Zx (in3)	0.88
Iy (in4)	0.249
J (in4)	0.00101
b	0.747
t	0.07

Cw (in6)	0.719
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

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Local buckling/ Yielding:

Support Condition

Round profiles

λ	10.6714286	=	*Rb/t*	
λ1	70			
λ2	189			
F/Ω (ksi)	22.1465841	=	27.7-1.70λ^0.5	*for λ<λ1*
			18.5-0.593λ^0.5	*for λ1<λ<λ2*
			3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	13333	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154		
Znet	0.88		
Mn/Ω (in-lbs)	13538.4615	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72			
Cb	1.14			
C1	0.5		C2	0.5
U (in)	0	=	*C1*g0-C2*βx/2*	
Me	10.4802824	=	*See 2015 ADM F.4-9*	
λ	81.1291853	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

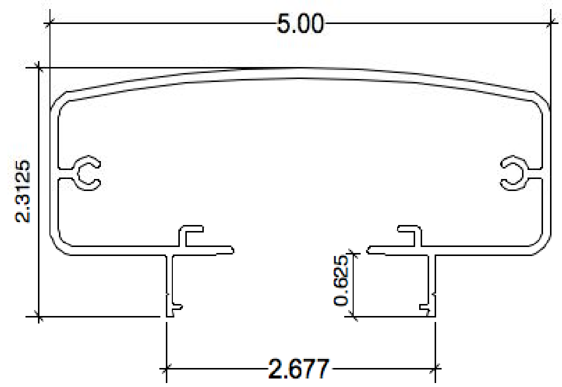
λ>Cc elastic buckling applies

Mnmb (in-kip	10.9103067	=	*π^2*E*Sx/λ^2*
Ma (in-lbs)	6612.30708	=	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 6612

Series 400 Top rail



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 400 Top Rail Vertical Loading

Section Properties

Ix (in4)	0.612
Sx (in3)	0.45
Zx (in3)	0.776
Iy (in4)	3.74
J (in4)	0.00254
b	12.5
t	0.087

Cw (in6)	7.85
βx (in)	-4.71
g0 (in)	2.18

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	143.678161 =	*Rb/t*	
λ1	70		
λ2	189		
F/Ω (ksi)	11.3919566 =	27.7-1.70λ^0.5	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	5126 =	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.776	
Mn/Ω (in-lbs)	11938.4615 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	2.2675 =	*C1*g0-C2*βx/2*	
Me	408.534561 =	*See 2015 ADM F.4-9*	
λ	10.4785796 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	17.7842841 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	10778.354 =	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) **5126**

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS

Extrusion Series 400 Top Rail Horizontal Loading

Section Properties

Ix (in4)	3.74
Sx (in3)	1.49
Zx (in3)	1.94
Iy (in4)	0.612
J (in4)	0.00254
b	1.04
t	0.087

Cw (in6)	7.85
βx (in)	0
g0 (in)	-2.5

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	11.954023 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =		15.2	*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	29394 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	1.94	
Mn/Ω (in-lbs)	29846.1538 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	-1.25 =	*C1*g0-C2*βx/2*	
Me	35.5461592 =	*See 2015 ADM F.4-9*	
λ	64.6408938 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	28.5382564 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	17295.913 =	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **17296**

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Series 500 Top Rail

Aluminum Extrusion Design

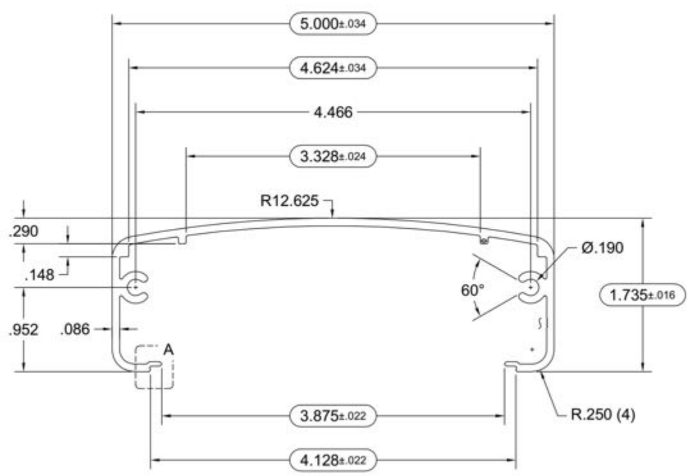
Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Series 500 Top Rail Vertical Bending

Section Properties

Ix (in4)	0.257
Sx (in3)	0.443
Zx (in3)	0.392
Iy (in4)	3.34
J (in4)	0.00248
b	25
t	0.086

Cw (in6)	1.05
βx (in)	-5.62
g0 (in)	0



Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round hollow elements under uniform compression

λ	290.697674 =	*Rb/t*	
λ1	31.2		
λ2	189		
F/Ω (ksi)	5.87337068 =	15.2	*for λ<λ1*
		18.5-0.593λ^0.5	*for λ1<λ<λ2*
		3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

*For λ>λ1, local buckling applies and the moment strength is calculated as F/Ω*S*

Mn/Ω (in-lbs)	2602 =	*F/Ω*1000(kips/lbs)*Sx*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.392	
Mn/Ω (in-lbs)	6030.76923 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	1.405 =	*C1*g0-C2*βx/2*	
Me	217.11117 =	*See 2015 ADM F.4-9*	
λ	14.2617014 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	9.33527188 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	5657.74053 =	*Mnmb/1.65*1000*

Strength is controlled by local buckling

Ma (in-lbs) **2602**

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Series 500 Top Rail Horizontal Bending

Section Properties

Ix (in4)	3.34
Sx (in3)	1.34
Zx (in3)	1.58
Iy (in4)	0.257
J (in4)	0.00248
b	1.032
t	0.086

Cw (in6)	1.05
βx (in)	0
g0 (in)	-2.5

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	12 =	*b/t*	
$\lambda 1$	22.8		
$\lambda 2$	39		
F/Ω (ksi)	15.2 =	15.2	*for $\lambda<\lambda 1$*
		19-0.170λ	*for $\lambda 1<\lambda<\lambda 2$*
		484/λ	*for $\lambda 2<\lambda$*

For $\lambda<\lambda 1$, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	23939 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	1.58	
Mn/Ω (in-lbs)	24307.6923 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	-1.25 =	*C1*g0-C2*βx/2*	
Me	8.43676165 =	*See 2015 ADM F.4-9*	
λ	125.827355 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ>Cc elastic buckling applies			
Mnmb (in-kip	11.1971776 =	*π^2*E*Sx/λ^2*	
Ma (in-lbs)	6786.16826 =	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 6786

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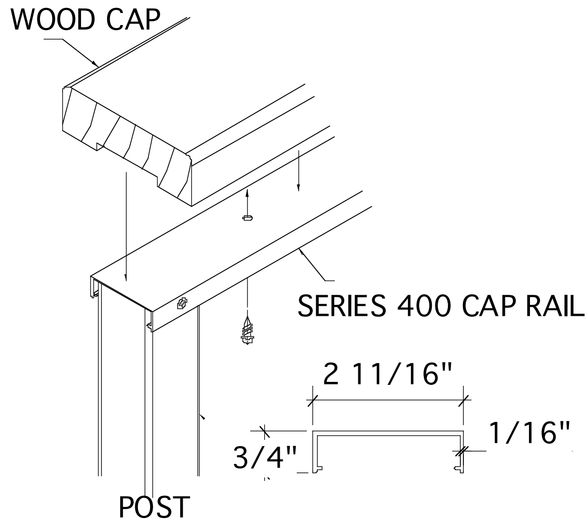
WOOD TOP RAIL
Alloy 6063 – T6 Aluminum



I_{xx}: 0.0138 in^4; I_{yy}: 0.265 in^4
C_{xx}: 0.573 in; C_{yy}: 1.344 in
S_{xx}: 0.024 in^3; S_{yy}: 0.197 in^3

Wood
2”x4” nominal
I_{xx}: 0.984 in^4; I_{yy}: 5.359 in^4
C_{xx}: 0.75 in; C_{yy}: 1.75 in
S_{xx}: 1.313 in^3; S_{yy}: 3.063 in^3

For wood use allowable stress from NDS Table 4A for lowest strength wood that may be used:
F_b = 725 psi (mixed maple #1), C_D =1.6, C_F = 1.5
F'_b = 725*1.6*1.5 = 1,740 psi
F'_b = 725*1.6*1.5*1.1 = 1,914 psi for flat use (vertical loading)

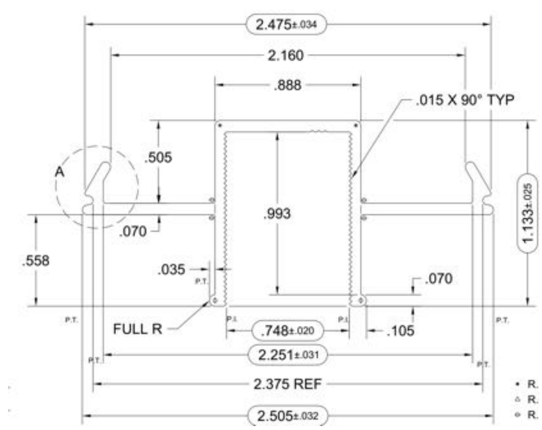
By inspection, vertical loading controls
Vertical loading: $M_{a,x}$=1,914psi*1.313in^3 = 2,510”#

Horizontal loading: $M_{a,y}$=1,740psi*3.063in^3 = 5,330”#

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

Glass Infill for wood cap rail:

Loading is primarily resisted by the top rail nut contribution from the glass infill piece may be taken into account.



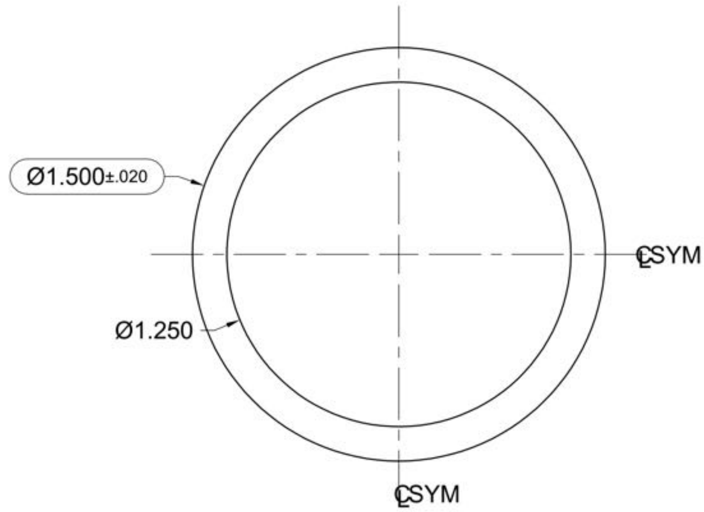
b/t = 1”/0.07” = 14.3 < 22.8 (Elements supported on two edges OK)
b/t = 0.558”/0.07” = 8.0 > 6.5
F/Ω = 27.8-0.81*8.0 = 21.3ksi for bottom elements supported one one side.

For vertical bending:
I_x = 0.0274in^4
S_x = 0.0406in^3
Z_x = 0.0721in^3
M_p is controlled by F_y1.5S_x in this case.

$M_{p,x}/\Omega = 1.5*0.0406in^3*15.2ksi = 948"\#$
Check local buckling over lower legs, $M_{a,x} = 21.3ksi*0.0406in^3 = 865"\#$ (controls)

For horizontal bending:
$I_y = 0.161in^4$
$S_y = 0.129in^3$
$Z_y = 0.203in^3$
M_p is controlled by $F_y 1.5S_y$ in this case.
$M_{p,x}/\Omega = 1.5*0.129in^3*15.2ksi = 2{,}940"\#$ (controls)

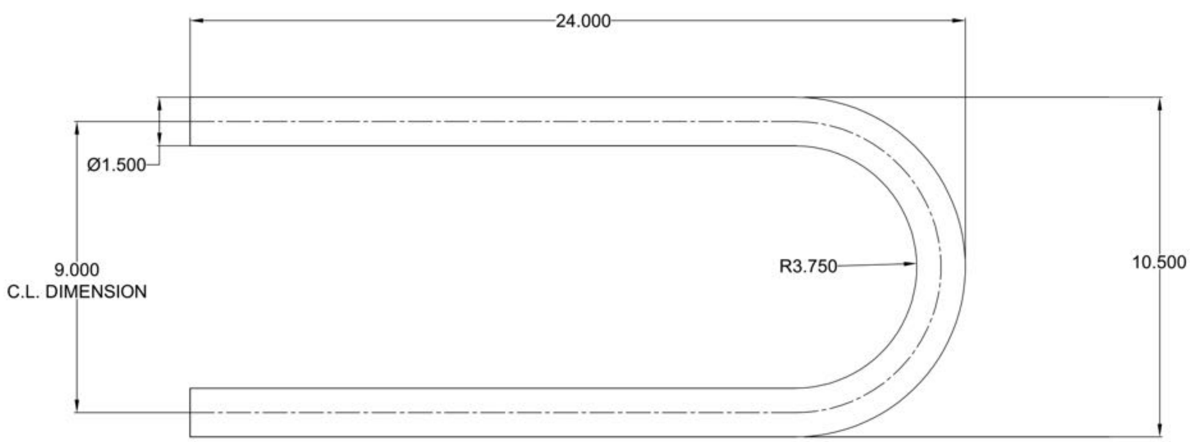
ARS HANDRAIL TUBING



1-1/2"X1/8" 6063-T6
$I = 0.129in^4$
$Z = 0.237in^3$
R_b/t = 0.75"/0.125" = 6 <70 (local buckling does not apply)

Lateral torsional buckling does not apply to a round tube.
F/Ω = 15.2ksi
M_a =15.2ksi*0.237in³ = 3,600"

Check bent handrail return:



Maximum moment occurs at the upper connection to the post when the 200# live load is concentrated at the end of the cantilever.

Recall allowable hand rail moment = 3,600"#
Allowable cantilever = 3,600#/200# = 18"

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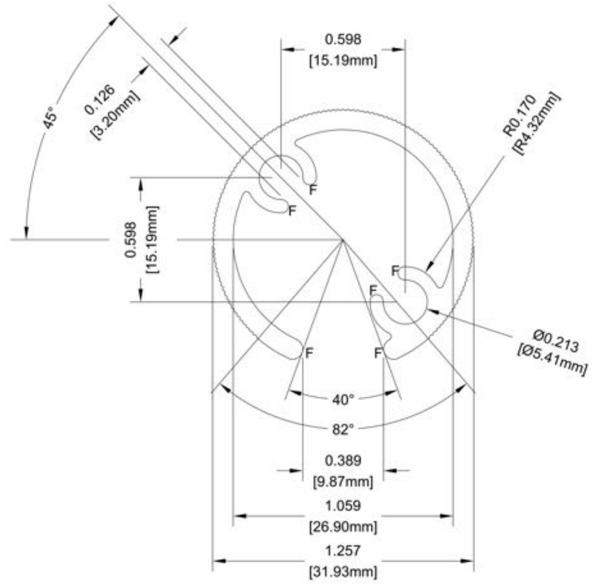
Hand Rail Splice/ Connection Block



Can be used to splice hand rails or attach to a face mount bracket.

$A = 0.321in^2$
$A_{v,x} = 0.161in^2$
$A_{v,y} = 0.226in^2$

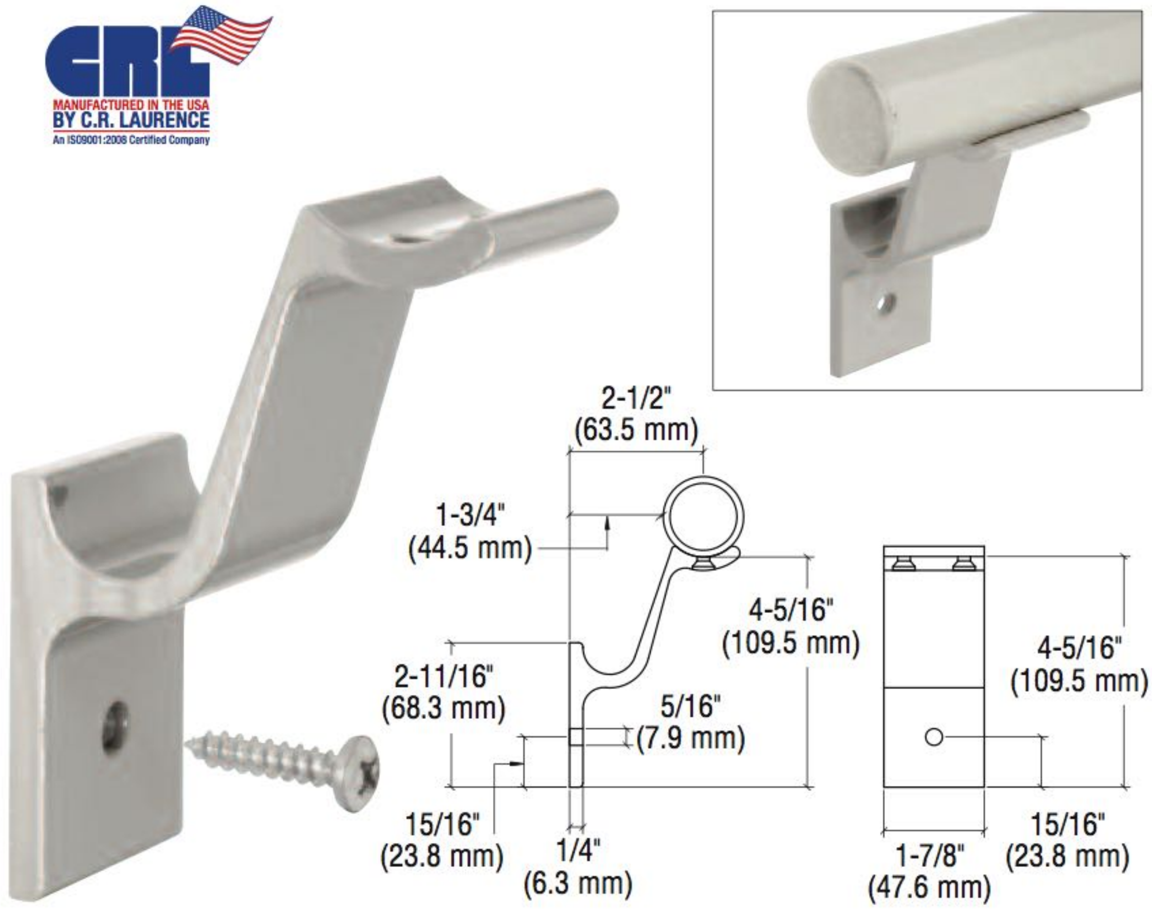
$F_v/\Omega = 9.1ksi$ (For 6063-T6)

$V_a = 0.161in^2*9.1ksi = 1,470\#$

In normal configurations the maximum shear carried by a hand rail is 200# < 1,470# OK.

When used to connect to a face mount end bracket, the hand rail extends to the end of the connection block so loading is transferred directly to one of the 1/4" screws.
$V_{max} = 200\#$
Hole bearing assuming 0.1" thick 6063-T6 endplate, $V_a = 2*0.25"*0.1"*30ksi/3 = 500\# > 200\#$ OK

CRL QUICK CONNECT ALUMINUM HAND RAIL BRACKET



Loading 200 lb concentrated load or
50 plf distributed load

Grab rail bracket – 1-7/8” long
Aluminum extrusion 6063-T6
Allowable load on bracket:
Vertical load:
Critical point @ 1.8” from rail to root of double radius, t = 0.25”
M = P*1.8” or WS*1.8”
where P = 200#, W = 50 plf and
S = tributary rail length to bracket.
Determine allowable Moment:
F_T = 20 ksi, F_C = 20 ksi
From ADM Table 2-21
$S_V = 1.875''*0.25^2/6 = 0.0195$ in^3
$M_{Vall} = 0.0195$ in^3*20 ksi = 390”#

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Determine allowable loads:
For vertical load:

P_{all} = 390"#/1.8" = 217#

S_{all} = 217#/50plf = 4'4"

Vertical loading will control bracket strength.

Allowable load may be increased proportionally by increasing the bracket length.

For 5' Post spacing: 5'/4.33'*1.875" = 2.165" – 2-11/64"

Grab rail connection to the bracket:

Two countersunk self drilling #8 screws into 1/8" wall tube

Shear – $2F_{tu}Dt/3$ = 2*30ksi*0.164"*0.125"/3*2 screws = 820# (ADM 5.4.3)

Tension – $1.2DtF_{ty}/3$ = 1.2*.164"*0.125"*25ksi*2 screws/3 = 410#

For residential installations only 200# concentrated load is applicable. Connection to support:

Maximum tension occurs for outward Horizontal force:

Determine tension from ∑M about C

0= P*5" – T*0.875"

T = 200#*4.12"/0.875" = 942#

From ∑ forces – no shear force in anchor occurs from horizontal load

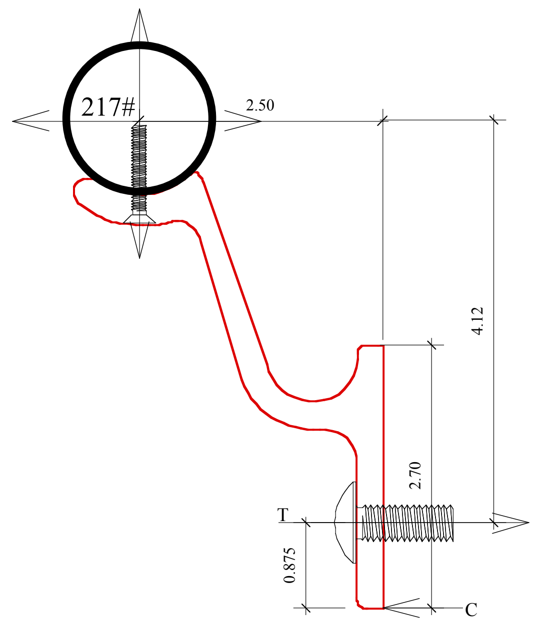
Vertical force:

Determine tension from ∑M about C

0= P*2.5" – T*0.875"

T = 200#*2.5"/0.875" = 571#

From ∑ forces – Z = P = 200#



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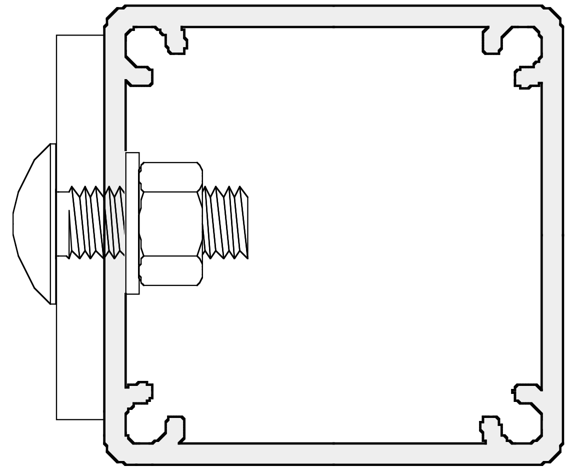
CONNECTION TO STANDARD POST (0.1" WALL)

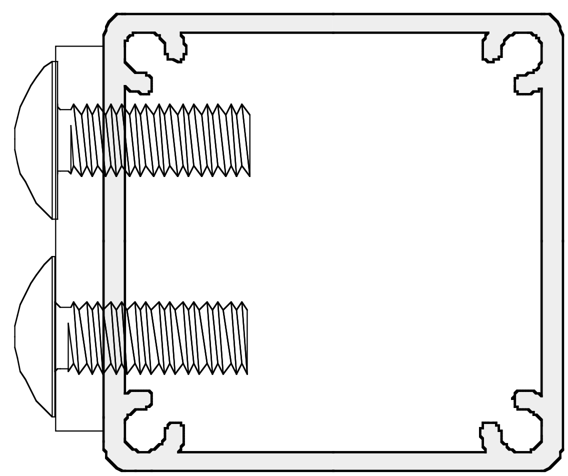
For 200# bracket load:
For handrails mounted to 0.1" wall thickness aluminum tube.
One Pan head self drilling #14
Shear – $2F_{tu}Dt/3$ (ADM 5.4.3)
2*38ksi*0.25"*0.1"/3= 633#
Tension – Pullout ADM 5.4.2.1
$P_t = 1.2DtF_{tu}/3 = 1.2*.25*.1*38ksi/3 = 380\#$



Screw tension strength, T_a=1,605# (ESR 1976)

Where a washer and nut is used only one screw/bolt is required.

For no washer and nut:
Required number of screws = 942#/380# => 3 screws minimum



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WELDED CORNERS

All top rails are 6063-T6 and welded with 5356 filler. The welds develop the full thickness of the part attached.

Check weld strength:
F_u = 35ksi
R_n/Ω = 0.6*35ksi*t/1.95 = (10.8t)kli

Check base metal affected by weld:
$F_{u,w}$ = 11.6ksi
$F_{y,w}$ = 8ksi
R_n/Ω = 11.6ksi*t/1.95 = (5.95t)kli < (10.8t)kli

The above calculations show that the base metal in the weld affected zone is by far the limiting failure mode. It can be conservatively assumed the failure section is perpendicular to the top rail. Therefore, the angle of the miter does not matter, since the failure section is not at the miter.

Therefore moment strength is calculated as $F_{y,w}Z/1.65$. Moment strengths at the corners are provided below for reference. However, in normal use the corners will not be subject to high moment loading. The splices used to connect the corner to the straight pieces of top rail will not transfer significant moment.

Using the welded corner bracket allows the corner post to braced in each direction. Therefore, the corner bracket may have to transfer significant force via shear and axial force. The shear strength is the limiting failure mode by inspection.

Assume shear is carried by the horizontal elements only. Shear strength is calculated as ,V_a=0.6$Fy_{,w}$*b_e*t/1.65. Where the effective width, b_e is taken as the width of the main horizontal element and t is taken as the thickness of that element.

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Top Rail	b_e (in)	t (in)	Z_x (in^3)	Z_y (in^3)	R_n/Ω (lbs)	M_x/Ω (in-lbs)	M_y/Ω (in-lbs)
200	3	0.087	0.398	1.03	759	1930	4994
300	2	0.086	0.583	0.583	500	2827	2827
320	2.3	0.075	0.243	0.669	502	1178	3244
350	2.5	0.07	0.355	0.88	509	1721	4267
400	4.5	0.086	0.776	1.94	1126	3762	9406
500	5	0.086	0.392	1.38	1251	1901	6691
1-1/2" HR	1.5	0.125	0.237	0.237	545	1149	1149

The minimum allowable load at the corner is 500#. Maximum loads expected to occur at the corner are between 200 and 300#. This would occur from a 200# load directly at the corner, or from a 50plf load where the corner is braced by a long stretch of posts. Cases where wind loading may be concern will be controlled by the bottom rail attachment. Therefore, the welded brackets are not a design concern in typical configurations.

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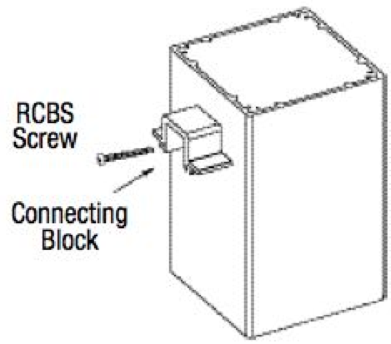
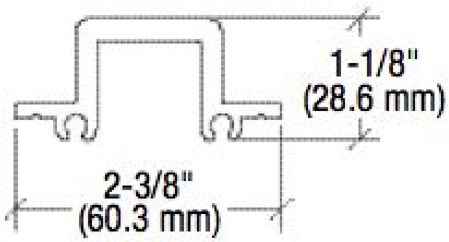
STANDARD POST RAIL CONNECTION BLOCK

Can be used to connect top rails to standard or 4"x4" post face, walls or other end butt connection.





Typical RCBS length is 1"

Top rail snaps over block and is secured with either silicone adhesive or #8 tek screws.

Connection strength to post or wall: (2) #10x1.5" 55 PHP SMS Screw

Check shear @ post (6005-T5)

F_{upost}x dia screw x Post thickness x SF

Eq 5.4.3-2

$V = 38\text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\text{ (FS)}} =$ 240#/screw for standard post

Since minimum of 2 screws used for each, Allowable load = 2· 240# = 480#

Screw tilting:

$4.2(t_2{}^3D)^{1/2}F_{tu2} = 4.2(0.1^3 0.19)^{1/2} 38\text{ ksi} = 2{,}200$

For 4"x4" post:

$V = 38\text{ ksi} \cdot 0.19" \cdot 0.125" \cdot \frac{1}{3\text{ (FS)}} =$ 300#/screw

Since minimum of 2 screws used for each, Allowable load = 2· 300# = 600#

Connections to walls and other surfaces is dependent on supporting material. Alternative fasteners may be used for connections to steel, concrete or wood.

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WALL MOUNT END CAPS

End cap is fastened to the top rail with
2) #10x1" 55 PHP SMS Screws

2x F_{upost}x dia screw x Cap thickness x SF
Eq J.5.5.1 Screws to top rail

$V = \frac{0.5*0.15*38\text{ksi} \le 2*38\text{ ksi} \cdot 0.19" \cdot 0.15"}{3\text{ (FS)}} =$

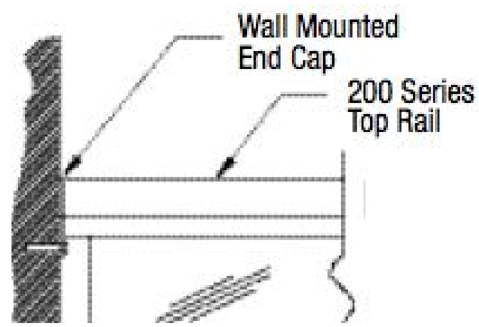
722#/screw , 1,444# per connection

Min screw edge distance ≥ 0.3"
Center of hole to edge

Screws to wall for shear on screw:

$V = \frac{0.325*0.15*38\text{ksi} \le 2*38\text{ ksi} \cdot 0.19" \cdot 0.15"}{3\text{ (FS)}} = 618\#$

Connection to wall shall use either:



#14x1-1/2" wood screw to wood, minimum 1" penetration into solid wood.

Allowable load = 2*175# = 350#
Wood shall have a G ≥ 0.43
From ADM Table 11M

For connection to steel studs or sheet metal blocking
Use #12 self drilling screws.
Minimum metal thickness is 18 gauge, 43 mil (0.0451")
Allowable load = 280#/screw

Table 3: Suggested Capacity for Screws Connecting Steel to Steel (lbs.)

Steel Thickness - Thinnest Component	¼ -14 Screw		#12-14 Screw		#10-16 Screw *		#8-18 Screw *		#6 Screw *	
	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout	Shear	Pullout
0.1017"	1000	320	890	280	780	245	675	210	560	175
0.0713"	600	225	555	195	520	170	470	145	395	125
0.0566"	420	180	390	155	370	135	340	115	310	95
0.0451"	300	140	280	120	260	105	240	90	220	75
0.0347"	200	110	185	95	175	80	165	70	150	60

Notes:
1. Design values are based on CCFSS Technical Bulletin Vol. 2, No. 1 which outlines the proposed AISI Specification provisions for screw connections. For shear connections the cold-formed steel section should be evaluated for tension.
2. Based on Fy = 33ksi, Fu = 45ksi minimum. Adjust values for other steel strengths.
3. * = Refer to Table 1 for limits on recommended total steel thickness of connected parts.

Wall Mounted End Caps

For connection to masonry or concrete use 3/16 screw-in anchor

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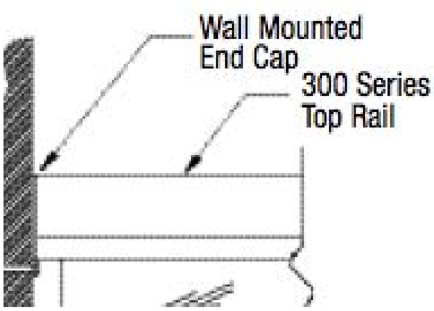
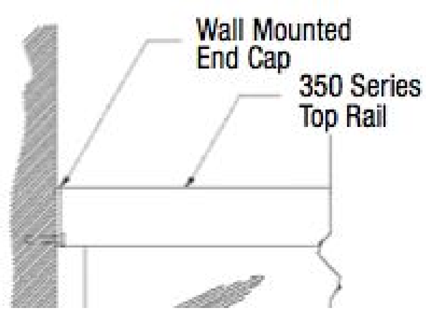
TABLE 3—ALLOWABLE TENSION AND SHEAR VALUES FOR TAPPER SCREW ANCHORS INSTALLED IN NORMAL-WEIGHT CONCRETE[1,2]

SCREW ANCHOR DIAMETER (inch)	SCREW ANCHOR MATERIAL AND COATING (AS APPLICABLE)	MINIMUM EMBEDMENT[3] (inches)	ALLOWABLE TENSION (pounds)						ALLOWABLE SHEAR[6] (pounds)
			With Special Inspection[4]			Without Special Inspection[5]			
			Concrete Strength, f'_c (psi)			Concrete Strength, f'_c (psi)			
			2000	3000	4000	2000	3000	4000	
$^3/_{16}$	Carbon steel, Perma-Seal coated	1	90	90	90	45	45	45	175
		$1^1/_2$	180	215	255	90	110	130	230
		$1^3/_4$	295	335	375	150	170	190	235

300 and 350 Series end caps use same fasteners and have identical strengths





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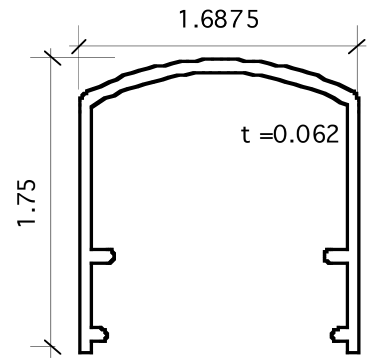
MID RAIL



Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Mid Rail Vertical Loading

Section Properties

Ix (in4)	0.111
Sx (in3)	0.107
Zx (in3)	0.177
Iy (in4)	0.561
J (in4)	0.000532
b	1.914
t	0.07

Cw (in6)	0.0486
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round hollow elements under uniform compression

λ	27.3428571	=	*Rb/t*	
λ1	31.2			
λ2	189			
F/Ω (ksi)	15.2	=	15.2	*for λ<λ1*
			18.5-0.593λ^0.5	*for λ1<λ<λ2*
			3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	2432	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154		
Znet	0.177		
Mn/Ω (in-lbs)	2723.07692	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72			
Cb	1.14			
C1	0.5		C2	0.5
U (in)	0	=	*C1*g0-C2*βx/2*	
Me	6.43064782	=	*See 2015 ADM F.4-9*	
λ	40.7263121	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	3.02993393	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	1836.32359	=	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 1836

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Mid Rail Horizontal Loading

Section Properties

Ix (in4)	0.181
Sx (in3)	0.212
Zx (in3)	0.239
Iy (in4)	0.111
J (in4)	0.000532
b	1.47
t	0.07

Cw (in6)	0.0486
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	21 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =		15.2	*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	3621 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.239	
Mn/Ω (in-lbs)	3676.92308 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	2.86045279 =	*See 2015 ADM F.4-9*	
λ	85.9530116 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ>Cc elastic buckling applies

Mnmb (in-kip	3.218447 =	*π^2*E*Sx/λ^2*
Ma (in-lbs)	1950.57394 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **1951**

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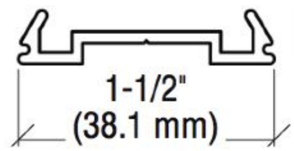

Filler for picket infill inserts into bottom of rail to attach 3/4" pickets.
May be used with either Mid Rail or standard bottom rail.
$I_y = 0.0386$ in^4; $S_y = 0.0515$ in^3
For infill: $M_a = 0.0515*15$ ksi = 773"#

Mid rail receives wind loading from the glass infill and the 50# concentrated live load.

Allowable moment under horizontal loading = 1,950"#
Allowable span as limited by 50# concentrated live load = 1,950"#/(50#/4) = 156" > 72" OK
Allowable span as limited by wind load = 1,950"#/(P/12*(H/2)/8)$^{1/2}$
Where P = wind pressure in PSF and H = infill height in feet.
The table below shows allowable spans with respect to different wind loads and infill heights.

Guard Rail Mid/Bottom Rail Design

System: ARS
Rail: Mid Rail

Extrusion Properties Properties:

E (psi) 10100000
Iy (in^4) 0.181
Ma (in-lbs) 1950
Δa (in) L/60

Load Cases:

50# concentrated load at mid span

M=50#L/4
*Δ=50#*L^3/(48EI)*
*Lmax=MIN(Ma*4/50 or (48EI/(50*60))^1/2*
Lmax (in) 156

Wind Load

M=P/12(H/2)*L^2/8*
*Δ=5*P/12*(H/2)*L^4/(384EI)*
Lmax=MIN((Ma/(P/12(H/2)/8))^(1/2) or (384EI*12*2/(60*5*PH))^1/3*

Allowable rail span with respect to infill height and wind load is shown in the table below.

		P (psf)		
		25	50	75
H (ft)	1.5	99.92	70.65	57.69
	2	86.53	61.19	49.96
	2.5	77.40	54.73	44.69
	3	70.65	49.96	40.79
	3.5	65.41	46.25	37.77
	4	61.19	43.27	35.33
	4.5	57.69	40.79	33.31
	5	54.73	38.70	31.60

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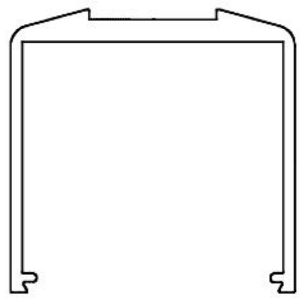
Picket Bottom Rail



Area: 0.446 sq in
Perim: 9.940 in

Ixx: 0.125 in^4
Iyy: 0.193 in^4
Kxx: 0.529 in
Kyy: 0.658 in

Cxx: 1.151 in
Cyy: 0.852 in
Sxx: 0.108 in^3
Syy: 0.227 in^3

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Picket Bottom Rail Vertical Loading

Section Properties

Ix (in4)	0.119
Sx (in3)	0.103
Zx (in3)	0.181
Iy (in4)	0.522
J (in4)	0.00154
b	1.56
t	0.132

Cw (in6)	0.0526
βx (in)	-2.88
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	11.8181818 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =	15.2		*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	2341 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.181	
Mn/Ω (in-lbs)	2784.61538 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0.72 =	*C1*g0-C2*βx/2*	
Me	20.7754615 =	*See 2015 ADM F.4-9*	
λ	22.2307355 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	3.71631382 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	2252.31141 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 2252

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Picket Bottom Rail Horizontal Loading

Section Properties

Ix (in4)	0.188
Sx (in3)	0.221
Zx (in3)	0.262
Iy (in4)	0.119
J (in4)	0.00154
b	1.44
t	0.07

Cw (in6)	0.0526
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	20.5714286 =	*b/t*	
λ1	22.8		
λ2	39		
F/Ω (ksi)	15.2 =	15.2	*for λ<λ1*
		19-0.170λ	*for λ1<λ<λ2*
		484/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	3970 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.262	
Mn/Ω (in-lbs)	4030.76923 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	C2	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	4.5116951 =	*See 2015 ADM F.4-9*	
λ	69.8774141 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	3.92598112 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	2379.38249 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **2379**

Guard Rail Mid/Bottom Rail Design

System:	ARS
Rail:	Picket Bottom Rail

Extrusion Properties Properties:

E (psi)	10100000
Iy (in^4)	0.188
Ma (in-lbs)	2380
Δa (in)	L/60

Load Cases:

50# concentrated load at mid span

M=50#L/4

*Δ=50#*L^3/(48EI)*

*Lmax=MIN(Ma*4/50 or (48EI/(50*60))^1/2*

Lmax (in) 174.300889

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Glass Bottom Rail

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System ARS
Extrusion Glass Bottom Rail Vertical Loading

Section Properties

Ix (in4)	0.0991
Sx (in3)	0.0968
Zx (in3)	0.165
Iy (in4)	0.192
J (in4)	0.000695
b	0.75
t	0.063

Cw (in6)	0.0623
βx (in)	-2.12
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	11.9047619 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =		15.2	*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	2200 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.165	
Mn/Ω (in-lbs)	2538.46154 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0.53 =	*C1*g0-C2*βx/2*	
Me	7.06342936 =	*See 2015 ADM F.4-9*	
λ	36.9607211 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	2.92188822 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	1770.84134 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **1771**

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	Glass Bottom Rail Horizontal Loading

Section Properties

Ix (in4)	0.192
Sx (in3)	0.228
Zx (in3)	0.269
Iy (in4)	0.0991
J (in4)	0.000695
b	1.41
t	0.063

Cw (in6)	0.0623
βx (in)	0
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	22.3809524 =	*b/t*		
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =	15.2		*for λ<λ1*
		19-0.170λ		*for λ1<λ<λ2*
		484/λ		*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	4076 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.269	
Mn/Ω (in-lbs)	4138.46154 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.14		
C1	0.5	**C2**	0.5
U (in)	0 =	*C1*g0-C2*βx/2*	
Me	3.08001451 =	*See 2015 ADM F.4-9*	
λ	85.901669 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ>Cc elastic buckling applies			
Mnmb (in-kip	3.43282404 =	*π^2*E*Sx/λ^2*	
Ma (in-lbs)	2080.49942 =	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) **2080**

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Guard Rail Mid/Bottom Rail Design

System: ARS
Rail: Glass Bottom Rail

Extrusion Properties Properties:

E (psi)	10100000
Iy (in^4)	0.192
Ma (in-lbs)	2080
Δa (in)	L/60

Load Cases:

50# concentrated load at mid span

M=50#L/4
*Δ=50#*L^3/(48EI)*
*Lmax=MIN(Ma*4/50 or (48EI/(50*60))^1/2*
Lmax (in) 166.4

Wind Load

M=P/12(H/2)*L^2/8*
*Δ=5*P/12*(H/2)*L^4/(384EI)*
Lmax=MIN((Ma/(P/12(H/2)/8))^(1/2) or (384EI*12*2/(60*5*PH))^1/3*

Allowable rail span with respect to infill height and wind load is shown in the table below.

		P (psf)		
		25	**50**	**75**
H (ft)	**1.5**	103.20	72.97	59.58
	2	89.37	63.19	51.60
	2.5	79.94	56.52	46.15
	3	72.97	51.60	42.13
	3.5	67.56	47.77	39.00
	4	63.19	44.69	36.49
	4.5	59.58	42.13	34.40
	5	56.52	39.97	32.63

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Glass bottom rail may be reinforced with the RCB extrusion or a 7/16" min thickness x 1-1/2" aluminum flat bar.
$I_y = 0.113in^4$ (For RCB, flat bar has greater stiffness)
The bottom rail is stiffer and has more slender elements so it can be assumed to control the overall strength.
$I_{net} = 0.113in^4+0.192in^4 = 0.305in^4$
$M_a = 2{,}080"\#*0.305in^4/0.192in^4 = 3{,}300"\#$

Guard Rail Mid/Bottom Rail Design

System: ARS
Rail: Glass Reinforced Bottom Rail

Extrusion Properties Properties:

E (psi)	10100000
Iy (in^4)	0.305
Ma (in-lbs)	3300
Δa (in)	L/60

Load Cases:

50# concentrated load at mid span

M=50#L/4
*Δ=50#*L^3/(48EI)*
*Lmax=MIN(Ma*4/50 or (48EI/(50*60))^1/2*
Lmax (in) 222.009009

Wind Load

M=P/12(H/2)*L^2/8*
*Δ=5*P/12*(H/2)*L^4/(384EI)*
Lmax=MIN((Ma/(P/12(H/2)/8))^(1/2) or (384EI*12*2/(60*5*PH))^1/3*

Allowable rail span with respect to infill height and wind load is shown in the table below.

		P (psf)		
		25	50	75
H (ft)	1.5	129.98	91.91	75.05
	2	112.57	79.60	64.99
	2.5	100.69	71.20	58.13
	3	91.91	64.99	53.07
	3.5	85.09	60.17	49.13
	4	79.60	56.28	45.96
	4.5	75.05	53.07	43.33
	5	71.20	50.34	41.10

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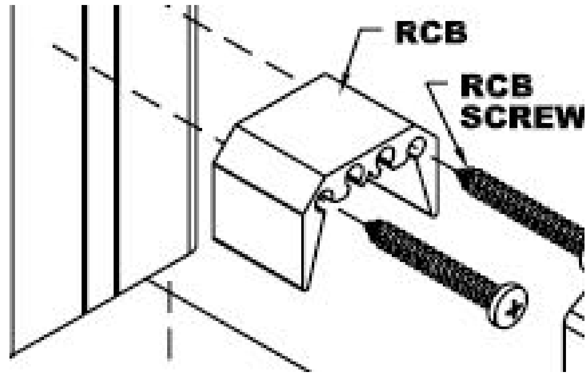
Rail fasteners -Bottom rail connection block to post
Typical RCB length is 1”
#10x1.5” 55 PHP SMS Screw
Check shear @ post (6005-T5)
2x F_{upost}x dia screw x Post thickness x SF
Eq 5.4.3-2
$V = 38 \text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\,(FS)} =$

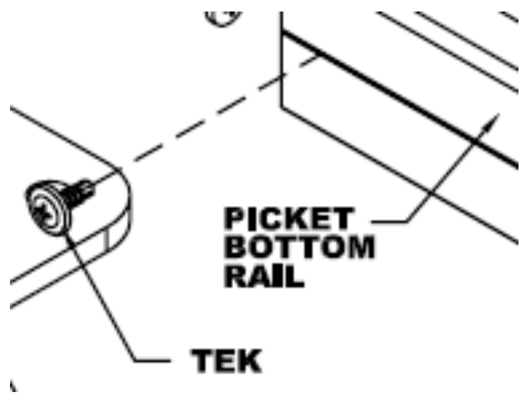
V = 240#/screw



Screw tilting:
$4.2(t_2{}^3D)^{1/2}F_{tu2} = 4.2(0.1^3 0.19)^{1/2}38 \text{ ksi} = 2{,}200$

Since minimum of 2 screws used for each
Allowable load = 2· 240# = 480#

Rail Connection to RCB
2 screws each end
#8 Tek screw to 6063-T6
ADM Eq. 5.4.3-1
$2*30_{ksi} \cdot 0.164" \cdot 0.07" \cdot 1/3 = 230$#/screw
Allowable shear = 2*230 = 460#
OK



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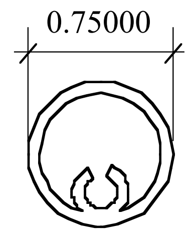
PICKETS 3/4" ROUND



The 50# concentrated infill load controls picket design. Pickets will be loaded about their strong axis.

For pickets at 4" O.C. max, the 50# live load over 1 square foot will be carried by 3 pickets minimum.
For maximum rail height of 60", M_{max} = 50#/3*60"/4 = 250"#
The calculations below show M_a = 423"# > 250"# OK
Δ = 50#/3*60"^{3}/(48*10.1*10^6*0.00906in^4) = 0.82"
L/Δ = 60"/0.82" = 73 > 60 OK
The pickets are OK up to the maximum 60" rail height considered for the ESR.

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	3/4" Round Picket Strong Axis

Section Properties

Ix (in4)	0.00906
Sx (in3)	0.0221
Zx (in3)	0.0336
Iy (in4)	0.00827
J (in4)	0.016
b	0.375
t	0.062

Cw (in6)	0
βx (in)	0.0774
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Round profiles

λ	6.0483871	=	*Rb/t*	
λ1	70			
λ2	189			
F/Ω (ksi)	23.5191103	=	27.7-1.70λ^0.5	*for λ<λ1*
			18.5-0.593λ^0.5	*for λ1<λ<λ2*
			3776/(λ(1+λ^0.5/35)^2)	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	502	=	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154		
Znet	0.0336		
Mn/Ω (in-lbs)	516.923077	=	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72			
Cb	1.32			
C1	0.5		C2	0.5
U (in)	-0.01935	=	*C1*g0-C2*βx/2*	
Me	4.09389441	=	*See 2015 ADM F.4-9*	
λ	23.1973501	=	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	0.69787054	=	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	422.951845	=	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 423

Connections
#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

$$V = 38\text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\text{ (FS)}} = 240\# > 200\#\text{ OK}$$

Note on shear loads:
Maximum shear assuming full 50# load acts on a single picket is under 50#
Picket cross-sectional area for the round pickets is 0.17 in^2
$f_v < 50/0.17 = 294$ psi <<< 9,200 psi
As this will occur at the ends of the picket where the bending moment is 0 and will be 0 at the peak moment further consideration of the shear on the pickets isn't warranted.

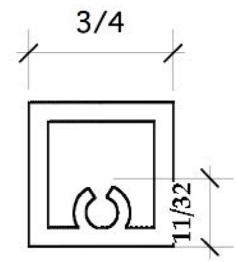
PICKETS 3/4" SQUARE

For pickets at 4" O.C. max, the 50# live load over 1 square foot will be carried by 3 pickets minimum.
For maximum rail height of 60", M_{max} = 50#/3*60"/4 = 250"#
The calculations below show M_a = 618"# > 250"# OK
Δ = 50#/3*60"^{3}/(48*10.1*10^6*0.0148in^4) = 0.50"
L/Δ = 60"/0.50" = 120 > 60 OK
The pickets are OK up to the maximum 60" rail height considered for the ESR.

Area: 0.288 sq in
Perim: 6.03 in

Ixx: 0.0196 in^4
Iyy: 0.0190 in^4
Kxx: 0.261 in
Kyy: 0.257 in

Cxx: 0.392 in
Cyy: 0.376 in
Sxx: 0.050 in^3
Syy: 0.051 in^3



PICKET

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	3/4" Square Picket Strong Axis

Section Properties

Ix (in4)	0.0148
Sx (in3)	0.0365
Zx (in3)	0.0486
Iy (in4)	0.0138
J (in4)	0.0206
b	0.63
t	0.062

Cw (in6)	0
βx (in)	0.0744
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	10.1612903 =		*b/t*	
λ1	22.8			
λ2	39			
F/Ω (ksi)	15.2 =		15.2	*for λ<λ1*
			19-0.170λ	*for λ1<λ<λ2*
			484/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	736 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.0486	
Mn/Ω (in-lbs)	747.692308 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.32		
C1	0.5	C2	0.5
U (in)	-0.0186 =	*C1*g0-C2*βx/2*	
Me	6.00007174 =	*See 2015 ADM F.4-9*	
λ	24.6251435 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	

λ<Cc, inelastic buckling applies

Mnmb (in-kip	1.02021906 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*
Ma (in-lbs)	618.31458 =	*Mnmb/1.65*1000*

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 618

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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

$$V = 38\text{ ksi} \cdot 0.19" \cdot 0.1" \cdot \frac{1}{3\ (FS)} = 240\# > 200\#\ \text{OK}$$

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PICKETS 5/8" SQUARE FOR SERIES 100

For pickets at 4" O.C. max, the 50# live load over 1 square foot will be carried by 3 pickets minimum.
For maximum rail height of 60", M_{max} = 50#/3*60"/4 = 250"#
The calculations below show M_a = 340"# > 250"# OK
Δ = 50#/3*56"^{3}/(48*10.1*10^6*0.006618in^4) = 0.91"
L/Δ = 56"/0.91" = 61 > 60 OK
The pickets are OK up to a 56" span which results in a 60" total rail height for normal configurations.

Aluminum Extrusion Design

Aluminum extrusion strength is according to ADM 2020.

System	ARS
Extrusion	5/8" Picket For Series 100

Section Properties

Ix (in4)	0.006619
Sx (in3)	0.02117
Zx (in3)	0.02677
Iy (in4)	0.006619
J (in4)	0.01125
b	0.502
t	0.062

Cw (in6)	0.000492
βx (in)	0.868
g0 (in)	0

Aluminum Properties

Alloy:	6063-T6
Fu (ksi)	30
Fy (ksi)	25
E (ksi)	10100
Cc	78

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Moment Strength

Moment strength is according to the 2020 ADM Design Table 2-21 and Chapter F4.

Local buckling/ Yielding:

Support Condition

Flat element under uniform compression supported on both sides

λ	8.09677419 =	*b/t*	
λ1	22.8		
λ2	39		
F/Ω (ksi)	15.2 =	15.2	*for λ<λ1*
		19-0.170λ	*for λ1<λ<λ2*
		484/λ	*for λ2<λ*

For λ<λ1, local buckling does not apply and the moment strength is calculated as the minimum of Zfy/Ω or 1.5SFy/Ω

Mn/Ω (in-lbs)	406 =	*F/Ω*1000(kips/lbs)*min(Zx or 1.5Sx)*

Rupture Strength

Fu/Ω	15.3846154	
Znet	0.02677	
Mn/Ω (in-lbs)	411.846154 =	*Znet*Fu/Ω*1000kips/lbs*

Lateral Torsional Buckling:

Lb (in)	72		
Cb	1.32		
C1	0.5	C2	0.5
U (in)	-0.217 =	*C1*g0-C2*βx/2*	
Me	3.03826264 =	*See 2015 ADM F.4-9*	
λ	26.3547155 =	*2.3(Lb*Sx/(Iy*J)^0.5)^0.5*	
λ<Cc, inelastic buckling applies			
Mnmb (in-kip	0.56032016 =	*Mp(1-λ/Cc)+π^2*E*λ*Sx/Cc^3*	
Ma (in-lbs)	339.587978 =	*Mnmb/1.65*1000*	

Strength is controlled by lateral torsional buckling

Ma (in-lbs) 340

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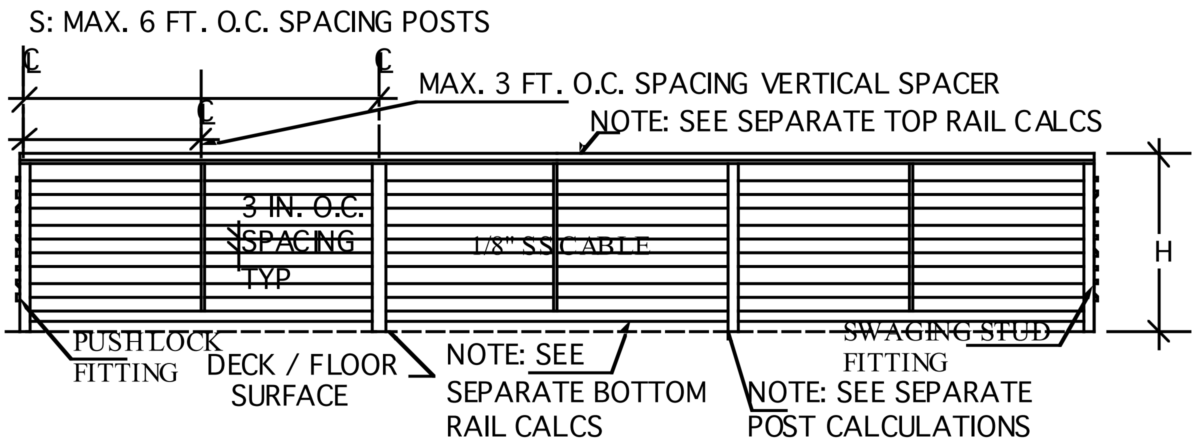
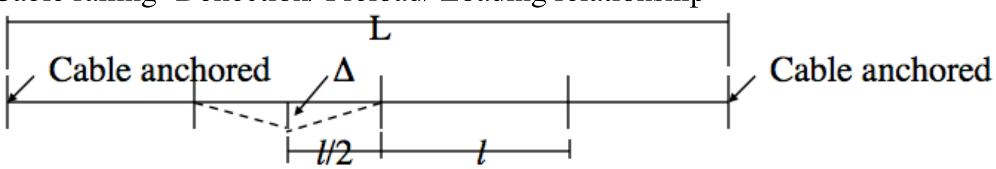
STAINLESS STEEL CABLE IN-FILL:



Cable railing- Deflection/ Preload/ Loading relationship



Cable Strain $= \in = \dfrac{C_{ta} \bullet L}{A \bullet E}$

$C_t = C_{tl} + C_{ta}$ $\qquad$ C_{ti} = installation tension

$C_{ta} = \dfrac{\in EA}{L}$ = Cable tension increase from loading

From cable theory

$C_t = \dfrac{l \bullet p}{4\Delta}$ $\qquad$ for concentrated load

To calculate allowable load for a given deflection:

Calculate $\quad \in = [[(l/2)^2 + \Delta^2]^{1/2} \bullet 2 - l]$

Then calculate $C_{ta} = \dfrac{\in AE}{L}$

Then calculate $C_t = C_{tl} + C_{ta}$

Then calculate load to give the assumed Δ for concentrated load

$$P = \frac{C_t 4\Delta}{l}$$

For uniform load – idealize deflection as triangular applying cable theory

$$C_t = \frac{Wl^2}{8\Delta}$$

Solving for $W = \frac{C_t\, 8\, \Delta}{l^2}$

See spreadsheet pages based on 36' maximum cable length and 3" clear cable spacing.

Cable rail loading requirements

Guardrail components 25 psf over entire area

IBC 1607.7.1.2 Components

50 lbs Conc. load over 1 sf

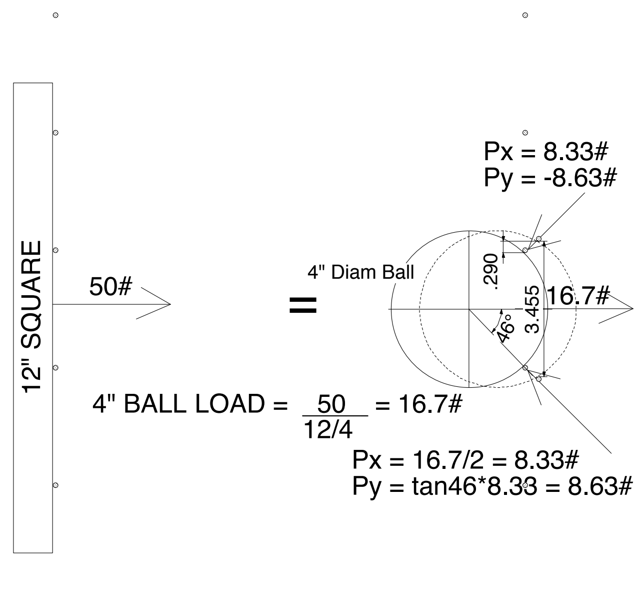
Application to cables

-Uniform load = $\frac{25 \text{ psf} \bullet 3"}{12"}$ = 6.25 plf

-Concentrated load 1 sf

3 cables minimum

50/3 = 16.7 lbs on 4" sphere



Produces 8.63 lb upward and downward on adjacent cables.

Deflection – since cables are 3" O.C. and maximum opening width = 4"

for 1/8" cable Δ_{all} = 4" – (3- 1/8) = 1 1/8"

for 3/16" cable Δ_{all} = 4" – (3- 3/16) = 1 3/16"

Cable Strain:

$\varepsilon = \sigma/E$ and $\Delta_L = L\,\varepsilon$

$\Delta_L = L(T/A)/E = L(T/0.0276 \text{ in}^2)/27 \times 10^6$ psi

Maximum cable free span length = 60.5"/2-2.375" = 27.875"

Additionally cable should be able to safely support 200 lb point load such as someone standing on a cable. This is not a code requirement but is recommended to assure a safe installation.

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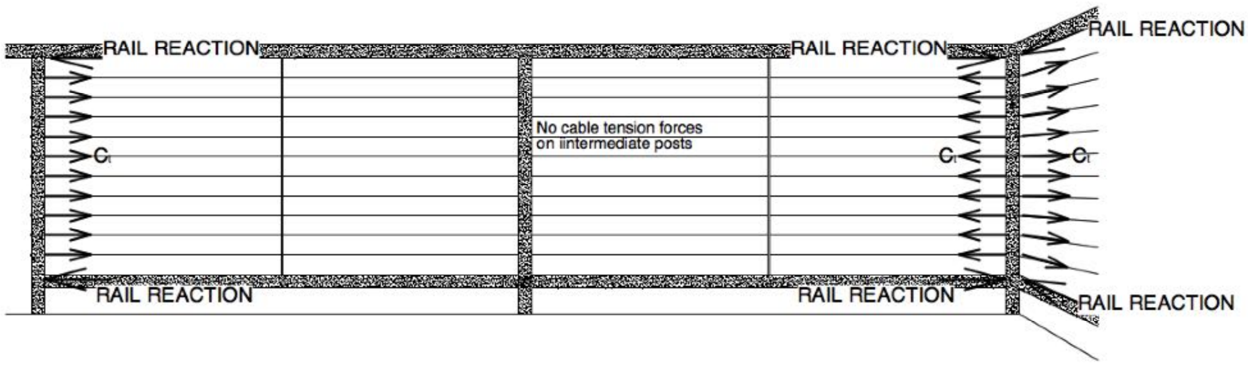
Cable railing					
Cable deflection calculations					
Cable = 3/16“ dia (area in^2) =		0.0278			
Modulus of elasticity (E, psi) =		27000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		200			
Total Cable length (ft) =		36			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	6.2	206.2	5.9	4.0
0.375	0.00803	13.9	213.9	9.2	6.3
0.55	0.01728	30.0	230.0	14.5	9.9
0.75	0.03213	55.7	255.7	21.9	15.0
1	0.05710	99.1	299.1	34.2	23.4
2	0.22783	395.3	595.3	136.1	93.3
2.5	0.35534	616.5	816.5	233.3	160.0
3	0.51056	885.8	1085.8	372.3	255.3

Cable railing					
Cable deflection calculations					
Cable = 1/8" dia (area in^2) =		0.0123			
Modulus of elasticity (E, psi) =		27000000			
Cable strain =Ct/(A*E) *L(in) = additional strain from imposed loading					
Cable installation load (lbs) =		200			
Total Cable length (ft) =		36			
Cable free span (inches) =		35			
Calculate strain for a given displacement (one span)				Imposed Cable load giving displ.	
delta (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.25	0.00357	2.7	202.7	5.8	4.0
0.375	0.00803	6.2	206.2	8.8	6.1
0.55	0.01728	13.3	213.3	13.4	9.2
0.75	0.03213	24.6	224.6	19.3	13.2
1	0.05710	43.8	243.8	27.9	19.1
2	0.22783	174.7	374.7	85.7	58.7
2.5	0.35534	272.5	472.5	135.0	92.6
3	0.51056	391.6	591.6	202.8	139.1

NOTE: Cable rail installations require special care to assure cables have adequate tension to provide the required resistance to infill loads. End posts and frame must have adequate strength to safely resist all cable loads.

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.

For 400 Series top rail no infill is used. Top rail extrusion is attached to post with (6) #8 screws in shear with total allowable shear load of 6*325# = 1,950#
Up to eight #8 screws may be used on a post if required to develop adequate shear transfer between the post and the 400 series top rail.

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 of 7*205#*1.25 = 1,784#

End post Cable loading
Cable tension - 200#/ Cable no in-fill load

$w = \frac{200\#}{3"} = 66.67\#/in$ $\qquad M_w = \frac{(39")^2 \bullet 66.67\#/in}{8} = 12{,}676\#"$

Typical post reactions for 200# installation tension :
11 cables*200#/2 = 1100# to top and bottom rails

For loaded Case

- 3 Cables @ center 220.7# ea based on 6' o.c. posts, 35" cable clear span

post deflection will reduce tension of other cables.

$\Delta = [Pa^2b^2/(3L)+2Pa(3L^2-4a^2)/24]/EI$ =

$\Delta = [220.7*15^2*24^2/(3*39)+220.7*15(3*39^2-4*15^2)/24]/(10{,}100{,}000*0.863) = 0.086"$

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Cable tension reduction for deflection will go from 200 at end cables to 271-220.7 at center, linear reduction = (200-50.3)/(39/2) = 7.7 pli

M_{conc} = 220.7# • 15”/2 +220.7#•18” +(3*(200-7.7*3)) + (6*(200-7.7*6)) + (9*(200-7.7*9)) +12*(200-7.7*12)+15*(200-7.7*15)/2
M_{conc} = 10,183#”

Typical post reactions for 200# installation tension with 50# infill load:
11 cables*200#/2+3*(221-200)/2 = 1132# to top and bottom rails.
Typical post reactions for 200# installation tension with 25 psf infill load:
11 cables*207.5#/2 = 1,141# to top and bottom rails.

For 200 # Conc load on middle cable tension
599.2# tension, post deflection will reduce tension of other cables
$\Delta = [Pa^2b^2/(3LEI) = [599.2*18^2 21^2/(3*39*10100000*0.863) = 0.084$
Cable tension reduction for deflection will go from 200 at end cables to 52 at center cables, linear reduction (200-52)/19.5” = 7.6 pli.
M_{200} = 599.2#/2 • 18” +(3)•(200-7.6*3) +(6) (200-7.6*6) +(9) (200-7.6*9) + (12) (200-7.6*12) +(15) (200-7.6*15) + (18) (200-7.6*18)/2 = 11,200#”

Post strength = 19,500”# (six screw post)
No reinforcement required. Note: post reinforcement may be required for other configurations.
Standard Cable anchorage okay.

Typical post reactions for 200# installation tension with 200# infill load on center cable:
11 cables*200#/2+(600#-200)/2 = 1,300# to top and bottom rails.

Typical post reactions for 200# tension with 200# infill load on top or bottom cable:
11 cables*200#/2+(600#-200)*33/36 = 1,467# to top and bottom rails.

Verify cable strength:
F_y = 110 ksi Minimum tension strength = 1,869# for ⅛” 1x19 cable
$øT_n$ = 0.85*110 ksi* 0.0123 = 1,150#
$T_s = øT_n / 1.6$ = 1,150# / 1.6 = 718#
Maximum cable pretension based on maximum service tension @ 200# cable load is 440#:

Δ (in)	strain (in)	Ct net (lb)	Ct tot (lbs)	Conc. Load (lb)	Uniform ld (plf)
0.19	0.00206	1.7	441.7	9.6	6.6
0.33	0.00622	5.1	445.1	16.8	11.5
2.437	0.33774	278.2	718.2	200.0	137.2

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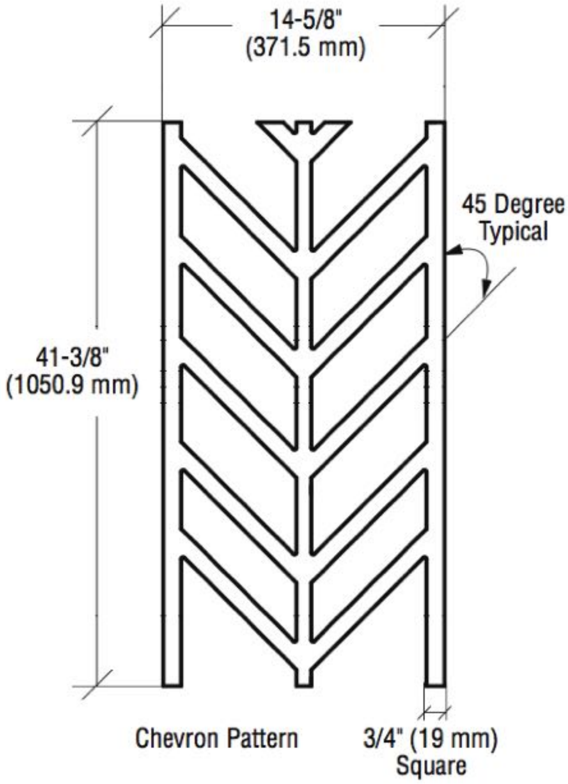
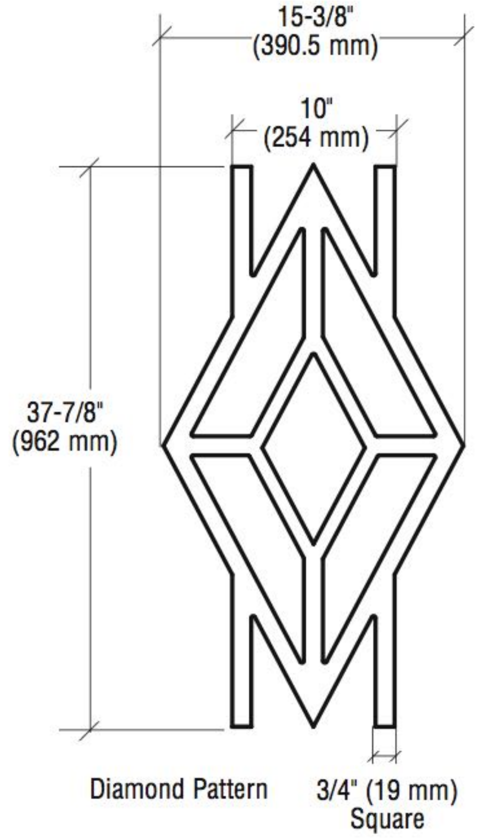
CRL Standard Cast Infills

Made from solid cast aluminum, 3/4” thick with patterns that conform to the code geometry requirements.
535.0-F aluminum casting in permanent molds



Chevron Pattern



Diamond Pattern

Infill strength based on 50# concentrated load at center of infill:
M = 50#*41.375”/4 = 517.2”#
At center bending is resisted by three 3/4” square solid aluminum bars:
$S = 3*0.75”^3/6 = 0.211$ in^3
f_b = 517.2/0.211 = 2,452 psi < 13.5ksi/2 = 6.75ksi

For diamond pattern casting: at center moment is resisted by four diagonal cast elements:
Two at 3/4” square and two at 1/2” x 3/4”
$S = 2*0.75”^3/6 + 2*0.5*0.75”^2/6 = 0.234$ in^3
M = 50*37.875”/4 = 473.44”#
f_b = 473.44”#/0.237 in^3 = 1,998 psi

May be mixed with 3/4” square pickets.

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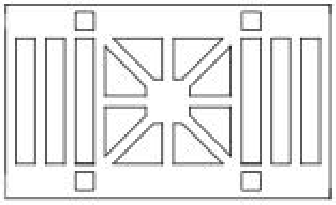
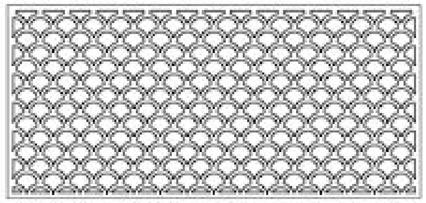
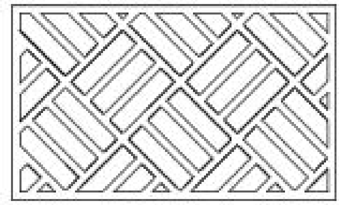
Custom Water Jet and Laser Cut Infill Panels

Custom patterns cut to specification with maximum opening sizes smaller than 4”.



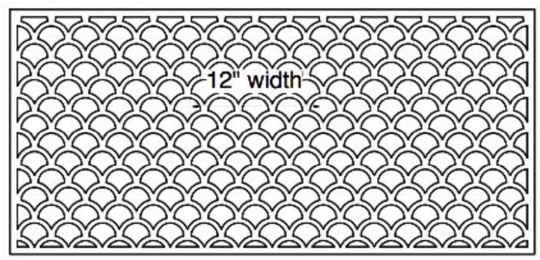




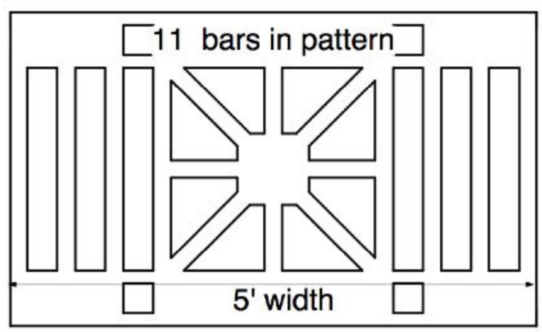
Fabricated from 5052-H32 or stronger aluminum sheet
Required strength based on a maximum panel width of 38”
Per ADM Table 2-9 F_b = 18 ksi
Design bending at centerline of plate:
M = 25psf*3.17’^{2}/8 = 31.4’# = 376.83”#/ft
Required minimum width of solid metal per foot of width:
S = b*t^2/6 ≥ 376.83”#/(18,000 psi) = 0.021 in^3

Determine required solid metal width based on thickness:
b = (0.021*6/t^2)

For t = 1/4”
b = (0.021*6/0.25^2) = 2.02”/ft
Design pattern so that there is a maximum of 9.98” of opening per foot.
In example there is 4 holes per foot maximum hole size = 9.98/4 = 2.495”



For second pattern the minimum bar width per foot must be 2.02” per foot average.
If panel is 5’ wide the total bar width is:
5’*2.02”/ft = 10.1”
There are 11 equivalent bars – 9 vertical and 2 diagonal:
Average width = 10.1/11 = 0.92”



For t = 3/8”
b = (0.021*6/0.375^2) = 0.896”/ft
Design pattern so that there is a maximum of 11.1” of opening per foot.
Calculate the required metal area similar to the two example shown for 1/4” sheet.

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GLASS STRENGTH FULLY TEMPERED INFILL PANELS

All glass is fully tempered glass conforming to the specifications of ANSI Z97.1, ASTM C 1048-97b and CPSC 16 CFR 1201. The average Modulus of Rupture for the glass F_r is 24,000 psi, F_r = 24 ksi typically used for design purposes. In accordance with IBC 2407.1.1 glass used as structural balustrade panels shall be designed for a safety factor of 4.0. Glass not used in guardrails may be designed for a lesser safety factor in accordance with ASTM E1300.

Values for the modulus of rupture, F_r, modulus of Elasticity, E and shear modulus, G for glass based on AAMA CW-12-84 *Structural Properties of Glass* (values are consistent with those used in ASTM E1300) are typically taken as:

F_r = 24,000 psi

E = 10,400 ksi is used as the standard value for common glass. While the value of E for glass varies with the stress and load duration this value is typically used as an average value for the stress range of interest.

G = 3,800 ksi: The shear component of the deflection tends to be very small, under 1% of the bending component and is therefore ignored.

$\mu = 0.22$ Typical value of Poisson's ratio for common glasses.

$\nu = 5\text{x}10^{-6}$ in/(inF°) Typical coefficient of thermal expansion.

The safety factor of 4 is dictated by the building code (IBC 2407.1.1). It is applied to the modulus of rupture since glass as an inelastic material does not have a yield point.

There is no deflection limits for the glass in guards other than practical limits for the opening sizes, retention in the frames and occupant comfort. Refer to ASTM E 1300-12a for a standard method of calculating deflections but the deflection limits are concerned with glazing in windows and similar parts of the building envelope rather than a free standing guard. IBC 2403.3 applies a limit of L/175 or 3/4" on the supporting frame for glass to be considered as fully supported along the frame element. From IBC Table 1604.3 footnote h similar types of construction have a limit of L/60.

The shear strength of glass tracks closely to the modulus of rupture because failure under shear load will be a tensile failure with strength limited by the modulus of rupture. Thus shear loads are transformed using Mohr's circle to determine the critical tension stress to evaluate the failure load. The safety factor of 4 is applicable to this case same as the bending case. Thus the shear stress is limited based on principal stresses of 0 and 6,000 psi to 6,000/2 = 3,000 psi.
Bearing stress can be derived in a similar fashion with the principal stresses being –6,000 psi and 6,000 psi so the bearing stress = 6,000 psi.

Bending strength of glass for the given thickness:

$I = 12\text{"}*(t)^3/12= (t)^3$ in^3/ft

$S = 12\text{"}*(t)^2/6= 2*(t)^2$ in^3/ft

t_{ave} = Average glass thickness; t_{min} = minimum glass thickness allowed by standard
For lites simply supported on two opposite sides the moment and deflection are calculated from basic beam theory

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

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

Maximum wind loads:

$W = M_a*8/L^2$ for uniform load W and span L (rail to rail distance)

Deflection can be calculated using basic beam theory:

$\Delta = (1-\nu^2)5wL^4/(384EI)$ for uniform load

$(1-\nu^2) = 0.9516$

Simplifying:

$\Delta = [wL^4/t^3]/(10.07 \times 10^9)$ for w in psf and L in inches

For concentrated load:

$\Delta = (1-\nu^2) PL^3/(48EI)$

Simplifying:

$\Delta = PL^3/(5.246*10^8 t^3)$

Maximum allowable deflection: Use L/60 deflection limit for infill. This will prevent glass from deflecting enough to disengage from the frame.

For uniform load (wind load)

Solving for w

$w = [t^3*1.676*10^8]/L^3$

Solving for L

$L = [(t^3*1.676*10^8)/w]^{1/3}$

Solving for t

$t = [L^3 w/(1.676*10^8)]^{1/3}$

For Concentrated load

Solving for P

$P = (8.74*10^6 t^3)/L^2$

Solving for L

$L = [8.74*10^6*t^3/P]^{1/2}$

Solving for t

$t = [PL^2/(8.74*10^6)]^{1/3}$

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Guard Infill Design

System: ARS
Infill Description:
Monolithic Tempered Glass

Allowable Live Load Stress (psi)	6000
Allowable Wind Load Stress (psi)	10600
Young's modulus (psi)	10400000

Load Cases:

50# Concentrated Load At Mid Span

*M = 50#*Span/(1ft*4)*

*Δ = 50#*span3/(48EI)*

*Max span = min(S*6000psi*4/50 or(48EI/(50*60))$^{(1/2)}$)*

$I = t_{min}^{3}$

$S = 2t^{2}$

Allowable Spans For 50# Live Load

t_{nom}	t_{min}	I (in^4)	S (in^3)	Max span (in)
1/4	0.219	0.01050346	0.095922	41.81
5/16	0.292	0.02489709	0.170528	64.37
3/8	0.355	0.04473888	0.25205	86.28

Wind Load

$M = W/12Span^{2}/8$

$\Delta = 5*W/12*span^{4}/(384EI)$

*Max span = min((S*6000psi*8/W)$^{1/2}$ or(384EI/(5*W/12*60))$^{(1/3)}$)*

Allowable Spans With Respect To Wind Loading

	Wind Load (psf)		
t_{nom} (in)	25	50	75
1/4	40.64	32.25	28.18
5/16	54.18	43.01	37.57
3/8	65.88	52.29	45.68

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Laminated Glass Panels

The 2015 and 2018 IBC require laminated glass panels where a walking surface is directly below the guard.

Glass sizes checked in this report are 1/4”, 5/16” and 7/16”

Glass is assumed to use a PVB interlayer with a shear modulus (G) of 140psi. This low value for G accounts for high exterior temperatures that may be present in the southern part of the US and Hawaii.

Effective thickness calculated according to ASTM E1300 appendix X11.

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

1/4” Laminated Glass:
lami+0.06”+lami, (.102” glass + 0.06” interlayer + .102” glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.102	0.102	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.162	0.081	0.081	0.001338444	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.372604684	0.200887242	0.223453105	0.223453105

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5/16" Laminated Glass:
1/8"+0.06"+1/8", (.115" glass + 0.06" interlayer + .115" glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.115	0.115	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.175	0.0875	0.0875	0.001760938	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.345016429	0.21780446	0.242724016	0.242724016

7/16" Laminated Glass:
3/16"+0.06"+3/16", (.180" glass + 0.06" interlayer + .180" glass)

Laminated Glass Effective Thickness				
h1	h2	hv	E	g
0.18	0.18	0.06	10400000	140
hs	hs;1	hs;2	Is	
0.24	0.12	0.12	0.005184	
a	Γ	hef;w	h1;ef;σ	h2;ef;σ
36	0.251798561	0.301209506	0.337137597	0.337137597

Guard Infill Design

System: ARS

Infill Description:

Laminated Tempered Glass

Allowable Live Load Stress (psi)	6000
Allowable Wind Load Stress (psi)	10600
Young's modulus (psi)	10400000

Load Cases:

50# Concentrated Load At Mid Span

*M = 50#*Span/(1ft*4)*

*Δ = 50#*span3/(48EI)*

Max span = $\min(S*6000psi*4/50 \text{ or} (48EI/(50*60))^{(1/2)})$

$I = t_{e,w}{}^{3}$

$S = 2t_{e,\sigma}{}^{2}$

Allowable Spans For 50# Live Load

t_{nom}	$t_{e,w}$ (in)	$t_{e,\sigma}$ (in)	I (in^4)	S (in^3)	Max span (in)
1/4	0.201	0.223	0.0081206	0.099458	36.76
5/16	0.218	0.243	0.01036023	0.118098	41.52
3/8	0.301	0.337	0.0272709	0.227138	67.36

Wind Load

$M = W/12Span^2/8$

$\Delta = 5*W/12*span^4/(384EI)$

Max span = $\min((S*6000psi*8/W)^{1/2} \text{ or} (384EI/(5*W/12*60))^{(1/3)})$

Allowable Spans With Respect To Wind Loading

	Wind Load (psf)		
t_{nom} (in)	25	50	75
1/4	37.30	29.60	25.86
5/16	40.45	32.11	28.05
3/8	55.85	44.33	38.73

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DESIGN PROCEDURE AND FIGURES

For most projects, the required rail height, connection detail and wind loading are dictated for the job. The design figures below will assist the specifier determine the maximum post spacing that can be used for the given rail height, connection detail and wind loading. As the specifier goes through the procedure and selects components, the maximum post spacing the particular component will allow can be determined. Generally, this ESR considers a max fence height of 60" and a max post spacing of 72" although some conditions may go taller. For fence heights or spacings outside the scope of the design figures, a project specific analysis is required.

Step 1: Select infill

The infill will likely be selected by the building architect. All infills considered in this ESR meet guard rail requirements. Note that wind loading may be a significant factor for glass infill. Also, cable infill requires the heavy posts at corners and ends. For glass infill, consult the allowable span charts to determine if the glass thickness and span is OK for the project wind loading and to determine if the bottom and mid rail (if used) is OK for project wind loading. When a mid rail is used, the mid rail wind load tables are more restrictive than the bottom rail tables and will control allowable post spacing. Other infills are OK at the 60" max railing height considered in this ESR.

Step 2: Select anchorage method.

Anchorage method is normally dictated by architectural details for the building. Verify the project specific conditions are compatible with one of the standard anchorage details. Read the max post spacing for the 50plf live load table for the given rail height. Where significant wind loading is present, also consult wind load charts for max post spacing.

Step 3: Select posts.

Posts are selected similarly to the anchorage detail. Use heavy posts at cable corner posts and end posts to resist cable infill loading. Verify the post is compatible with the previously selected anchorage detail. For instance, if the six screw baseplate is used, then the six screw post must also be used. Consult the 50plf live load table for maximum post spacing for the given height and post. Where significant wind loading is present, also consult wind load charts for max post spacing.

Step 4: Select top rail.

This is also primarily an aesthetics consideration and will be dictated by the building architect. Top rails do not limit the post spacing except for the series 200X top rail which limits post spacing to 68" O.C. max. All other top rails are OK at 72" post spacing. Wind load charts are not provided for top rails because in every case they are stronger and stiffer than the bottom rail which receives the same loading.

STEP 1: INFILL

a) Picket infill

Does not restrict allowable post height or spacing

b) Cable infill - (Does not restrict allowable post height or spacing)

Limit post height to 42” above finished floor. Heavy posts required at corners and ends. Does not restrict post spacing.

c) Glass infill

Laminated glass required where a walking surface is below the guard rail. Select the glass thickness from the tables below that meets required allowable span and wind loading requirements. Additionally, check the allowable spans for the bottom rail and mid rail for the specific rail height and wind load. The mid rail will control over the bottom rail when used.

Monolithic glass design tables:

Allowable Spans For 50# Live Load

t_{nom}	t_{min}	I (in^4)	S (in^3)	Max span (in)
1/4	0.219	0.01050346	0.095922	41.81
5/16	0.292	0.02489709	0.170528	64.37
3/8	0.355	0.04473888	0.25205	86.28

Allowable Spans With Respect To Wind Loading

	Wind Load (psf)		
t_{nom} (in)	25	50	75
1/4	40.64	32.25	28.18
5/16	54.18	43.01	37.57
3/8	65.88	52.29	45.68

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Laminated glass design tables:

Allowable Spans For 50# Live Load

t_{nom}	$t_{e,w}$ (in)	$t_{e,\sigma}$ (in)	I (in^4)	S (in^3)	Max span (in)
1/4	0.201	0.223	0.0081206	0.099458	36.76
5/16	0.218	0.243	0.01036023	0.118098	41.52
3/8	0.301	0.337	0.0272709	0.227138	67.36

Allowable Spans With Respect To Wind Loading

	Wind Load (psf)		
t_{nom} (in)	25	50	75
1/4	37.30	29.60	25.86
5/16	40.45	32.11	28.05
3/8	55.85	44.33	38.73

Bottom rail design table:

Allowable rail span with respect to infill height and wind load is shown in the table below.

		P (psf)		
		25	50	75
H (ft)	1.5	103.20	72.97	59.58
	2	89.37	63.19	51.60
	2.5	79.94	56.52	46.15
	3	72.97	51.60	42.13
	3.5	67.56	47.77	39.00
	4	63.19	44.69	36.49
	4.5	59.58	42.13	34.40
	5	56.52	39.97	32.63

Mid rail design table:

Allowable rail span with respect to infill height and wind load is shown in the table below.

		P (psf)		
		25	50	75
H (ft)	1.5	99.92	70.65	57.69
	2	86.53	61.19	49.96
	2.5	77.40	54.73	44.69
	3	70.65	49.96	40.79
	3.5	65.41	46.25	37.77
	4	61.19	43.27	35.33
	4.5	57.69	40.79	33.31
	5	54.73	38.70	31.60

STEP 2: ANCHORAGE METHOD

Allowable post moment and post spacing information given for each anchorage type below. Note some anchorage details include two connections, such as post to baseplate and baseplate to slab. Check allowable post height when subject to 200# concentrated load to determine if the anchorage detail is valid for the rail height. If the detail is valid, then check allowable tributary width with respect to the 50plf live load and wind loads.

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<u>Post to baseplate connections:</u>

a) 4 screw 2-3/8" square post screwed to baseplate (includes 135° post)

Allowable moment, $M_{a,x}$ = 10,500"#

Max post height = 52.5" when subject to 200# concentrated load

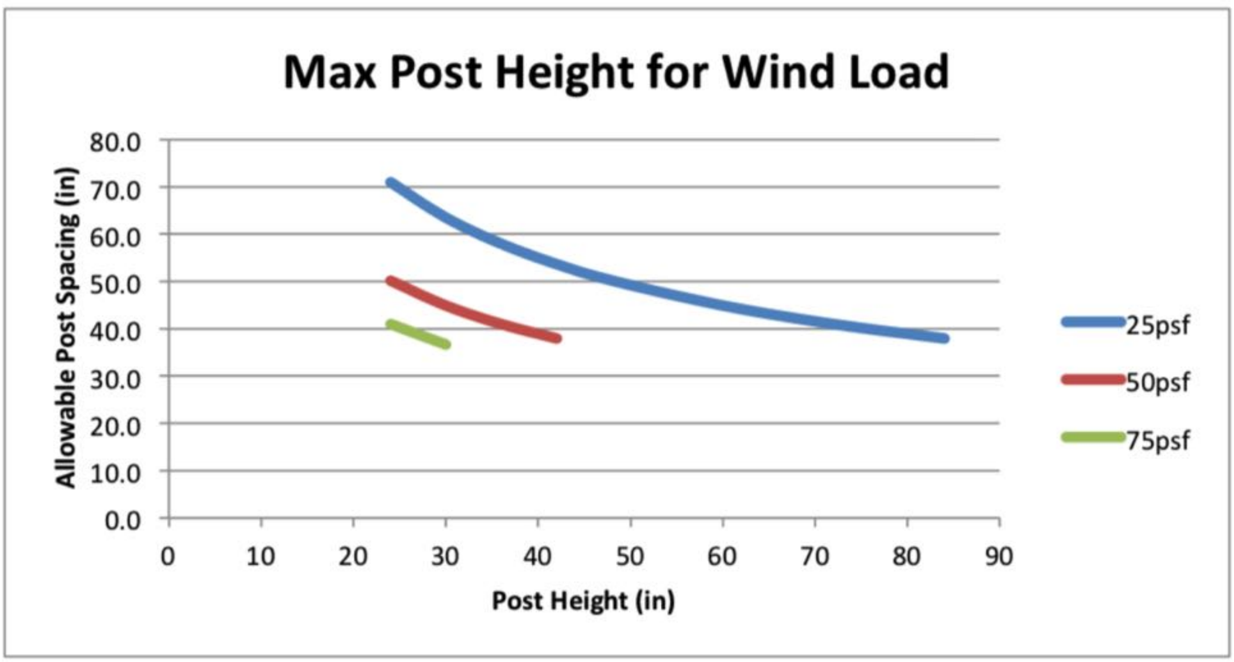
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70
42	60
48	52.5
60	42
72	<36"
84	<36"
96	<36"



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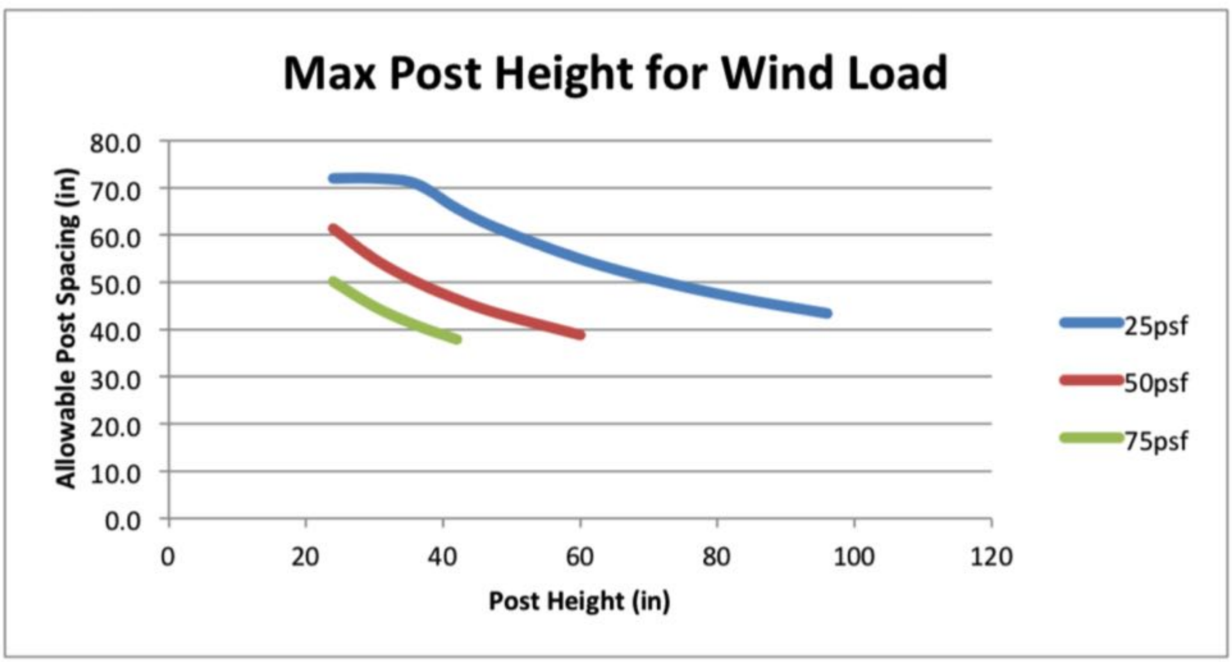
b) 6 screw 2-3/8" square post screwed to baseplate (includes heavy post, 135° post not used)
Allowable post moment, $M_{a,x}$ = 15,700"#
Max post height = 60" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	62.8
72	52.33333333
84	44.85714286
96	39.25



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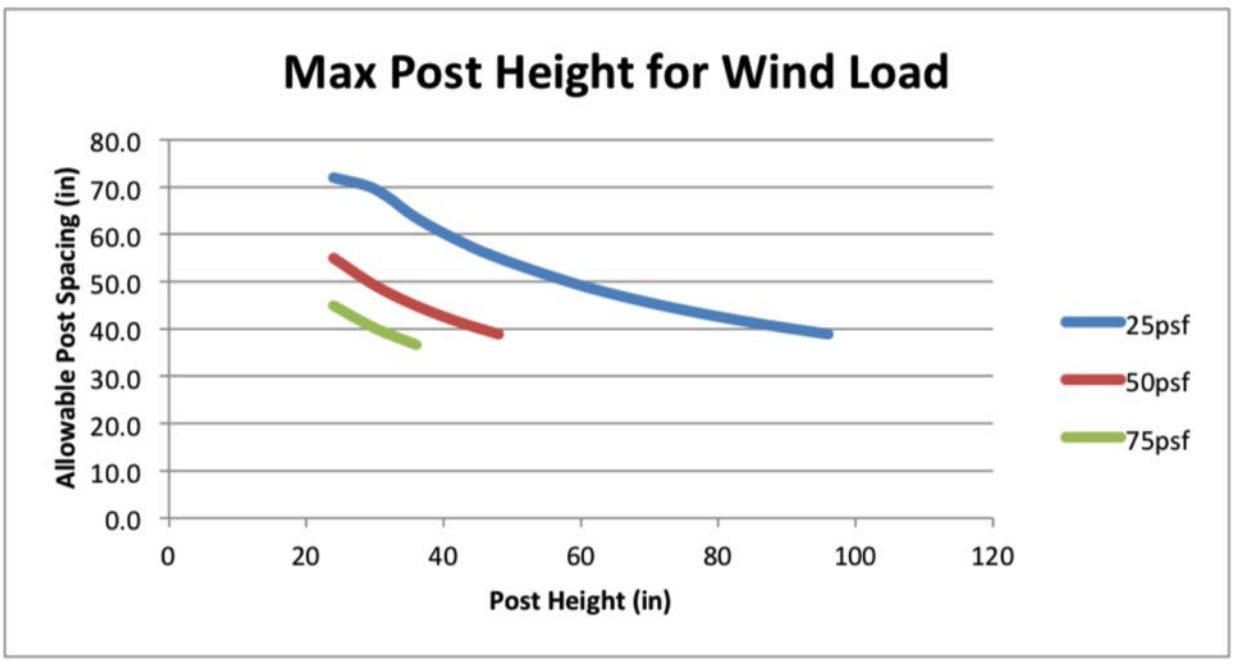
c) 135° post at corner mixed with 6 screw square posts at intermediates
Allowable post moment, $M_{a,x}$ = 12,600”#
Max post height = 60” when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63
60	50.4
72	42
84	36
96	<36"



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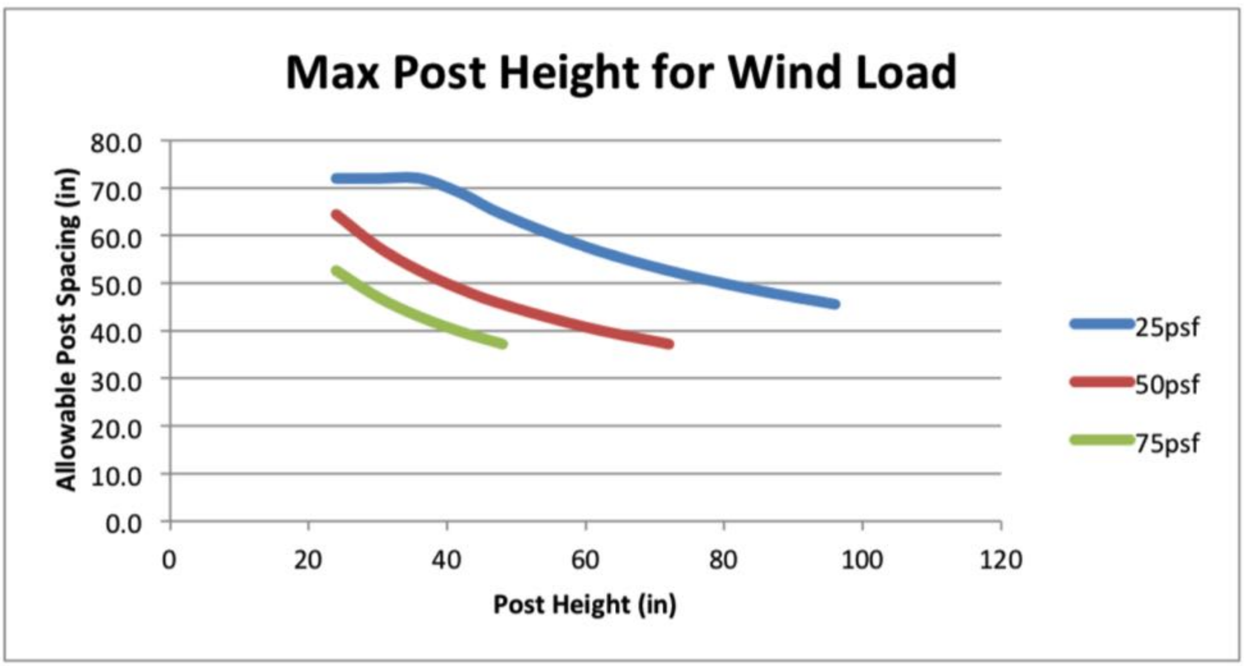
d) 4" square post screwed to baseplate
Allowable post moment, $M_{a,x}$ = 17,300"#
Max post height = 60" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	69.2
72	57.66666667
84	49.42857143
96	43.25



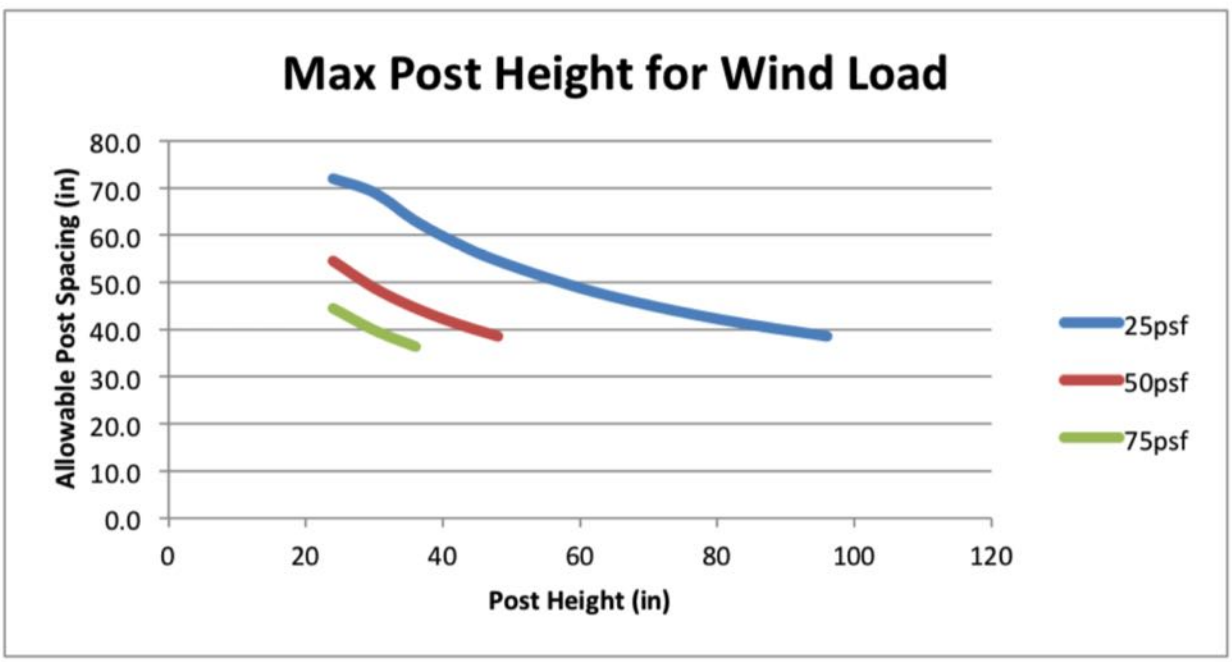
e) Aluminum stanchion screwed to baseplate
Allowable moment, $M_{a,x}$ = 12,400"#
Max post height = 60" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	70.85714286
48	62
60	49.6
72	41.33333333
84	<36"
96	<36"



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f) Aluminum stanchion welded to baseplate
Allowable moment, $M_{a,x}$ = 10,500"#
Max post height = 52" when subject to 200# concentrated load

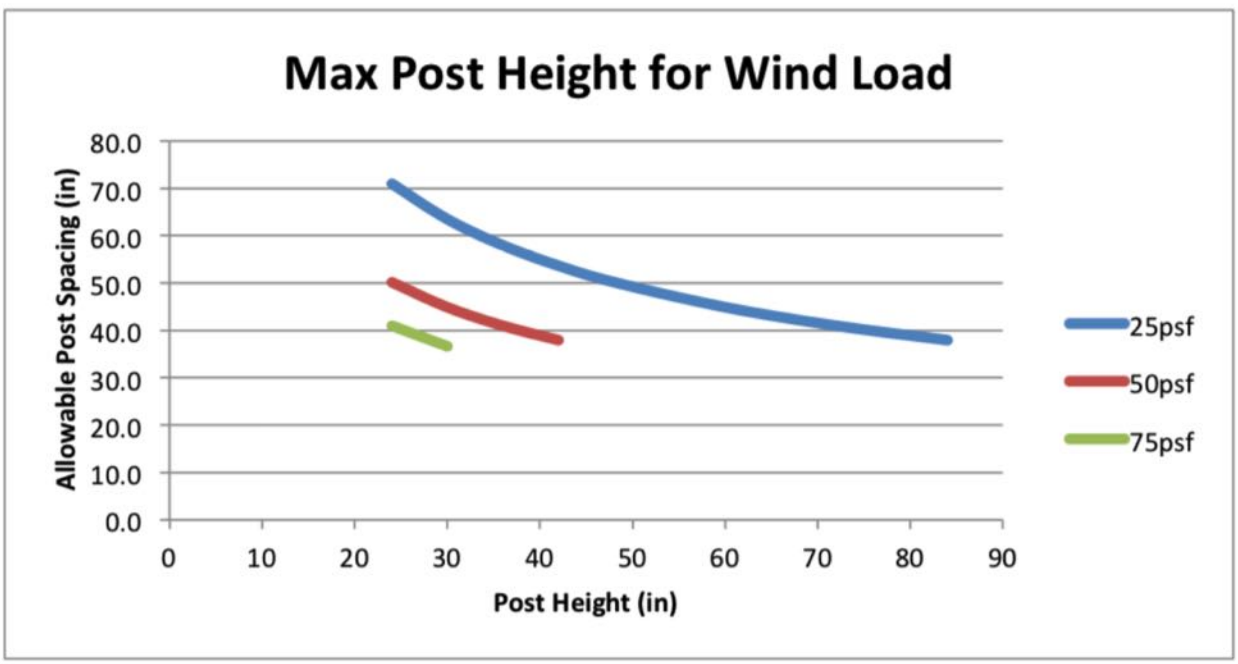
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70
42	60
48	52.5
60	42
72	<36"
84	<36"
96	<36"



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f) Steel stanchion welded to baseplate
Allowable moment, $M_{a,x}$ = 13,500"#

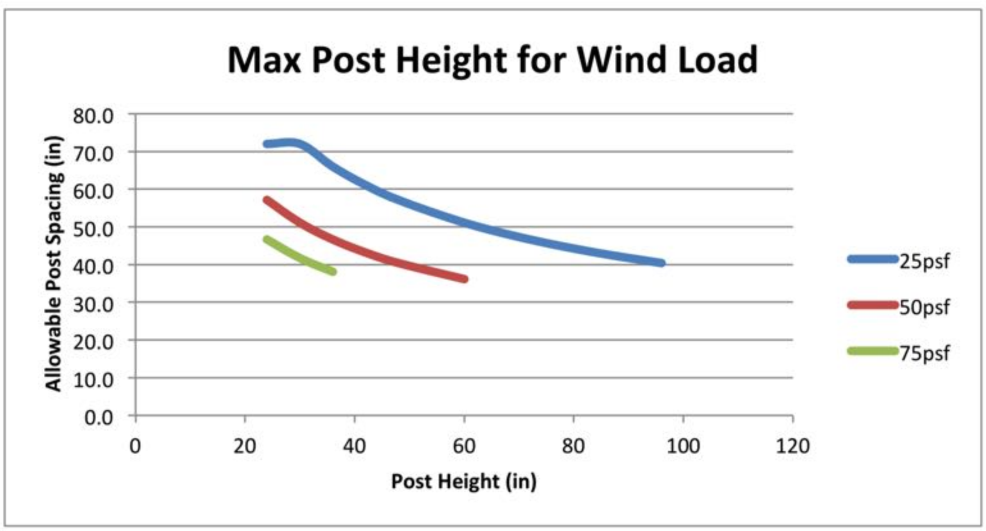
Max post height = 68" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	68
60	54.4
72	45.33333333
84	38.85714286
96	<36"



Baseplate Anchorage Details:

g) 3/8"x4" KH-EZ uncracked concrete and 5x5 baseplate

Allowable moment, $M_{a,x}$ = 13,500"#

Max post height = 60" when subject to 200# concentrated load

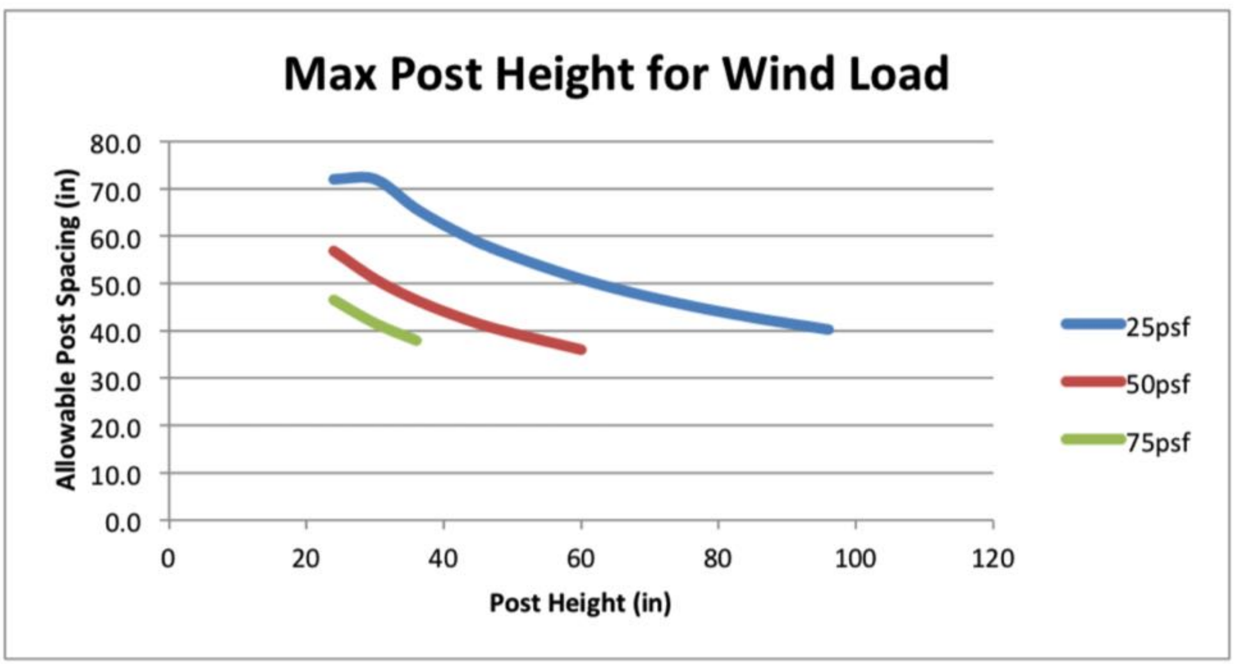
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	67.5
60	54
72	45
84	38.57142857
96	<36"



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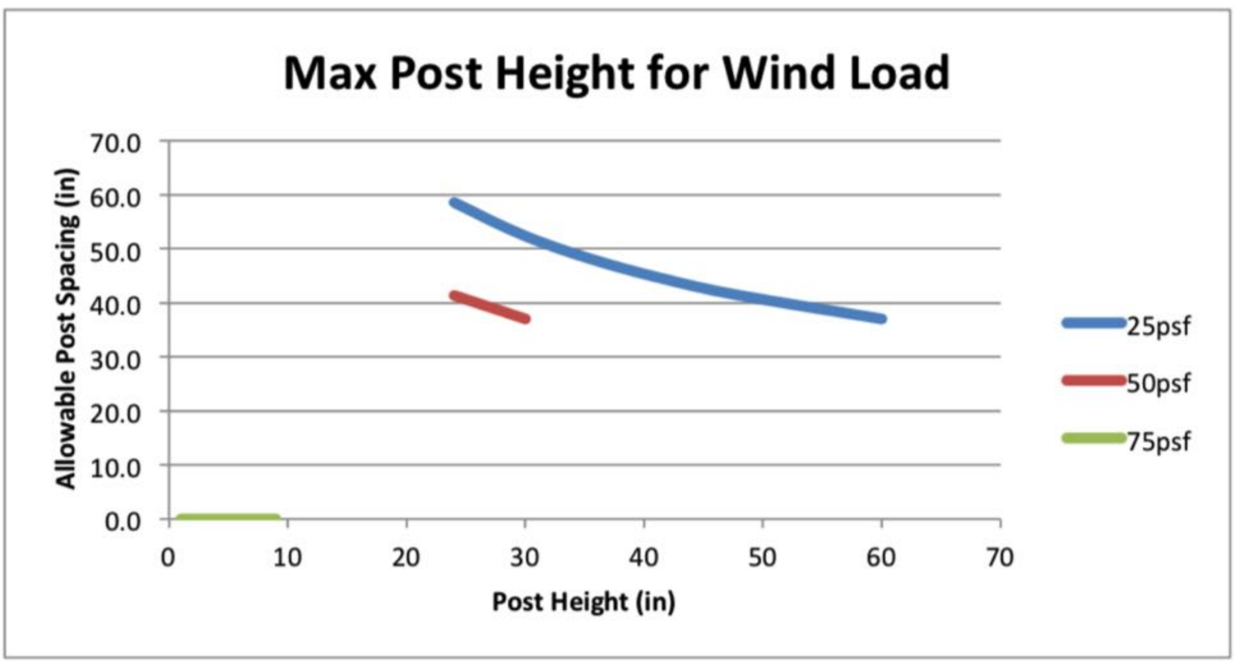
h) 3/8"x4" KH-EZ in uncracked concrete and 3x5 baseplate
Allowable moment, $M_{a,x}$ = 7,130"#
Max post height = 35" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	71.3
30	57.04
36	47.53333333
42	40.74285714
48	<36"
60	<36"
72	<36"
84	<36"
96	<36"



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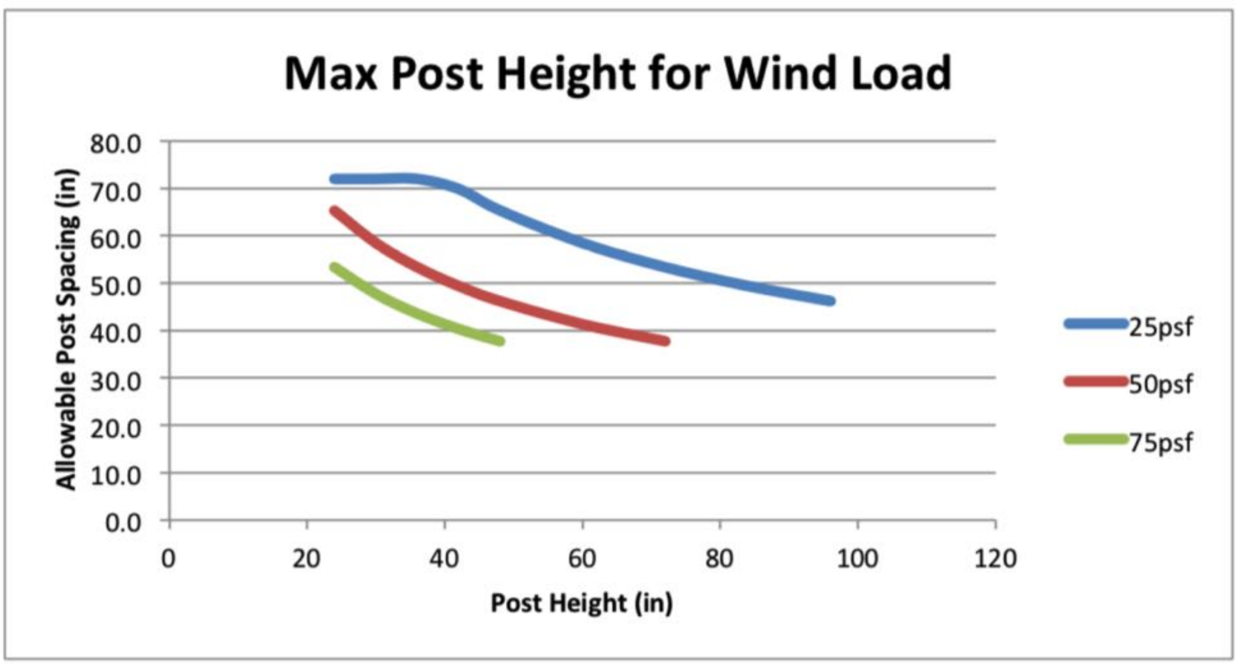
i) 3/8”x4” KH-EZ in uncracked concrete and 6-1/2x6-1/2” baseplate
Allowable moment, $M_{a,x}$ = 17,800”#
Max post height = 84” when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	71.2
72	59.33333333
84	50.85714286
96	44.5



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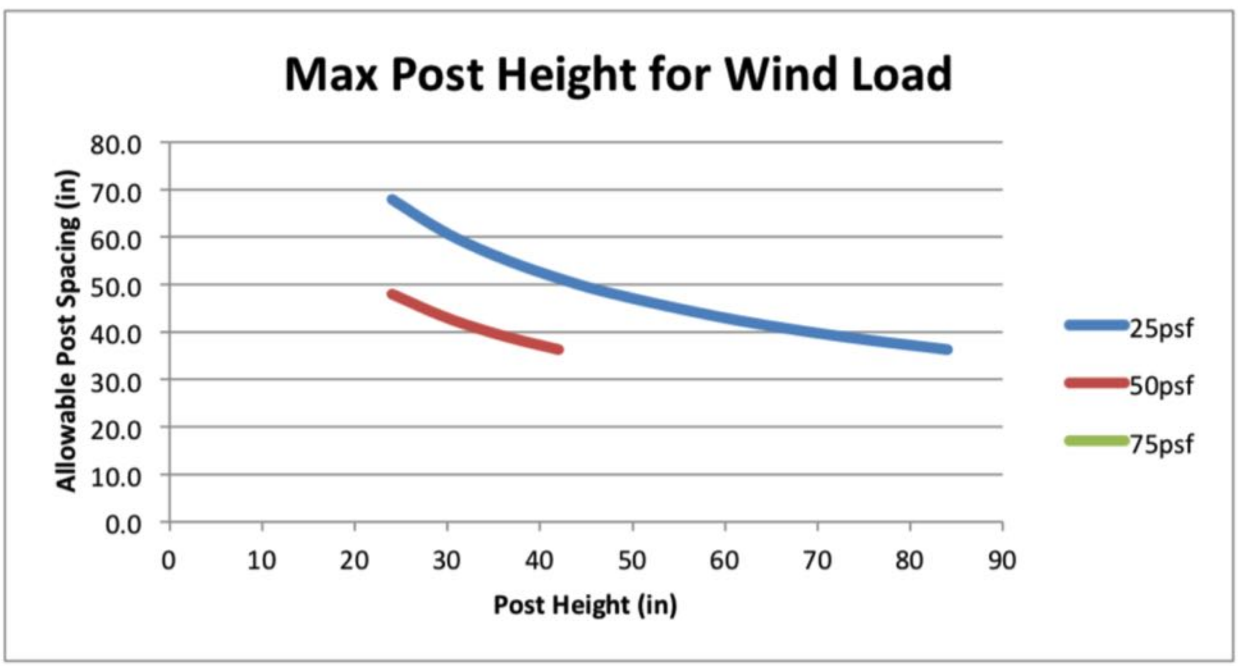
j) 3/8"x4" KH-EZ in cracked concrete and 5x5" baseplate
Allowable moment, $M_{a,x}$ = 9,600"#
Max post height = 48" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	64
42	54.85714286
48	48
60	38.4
72	<36"
84	<36"
96	<36"



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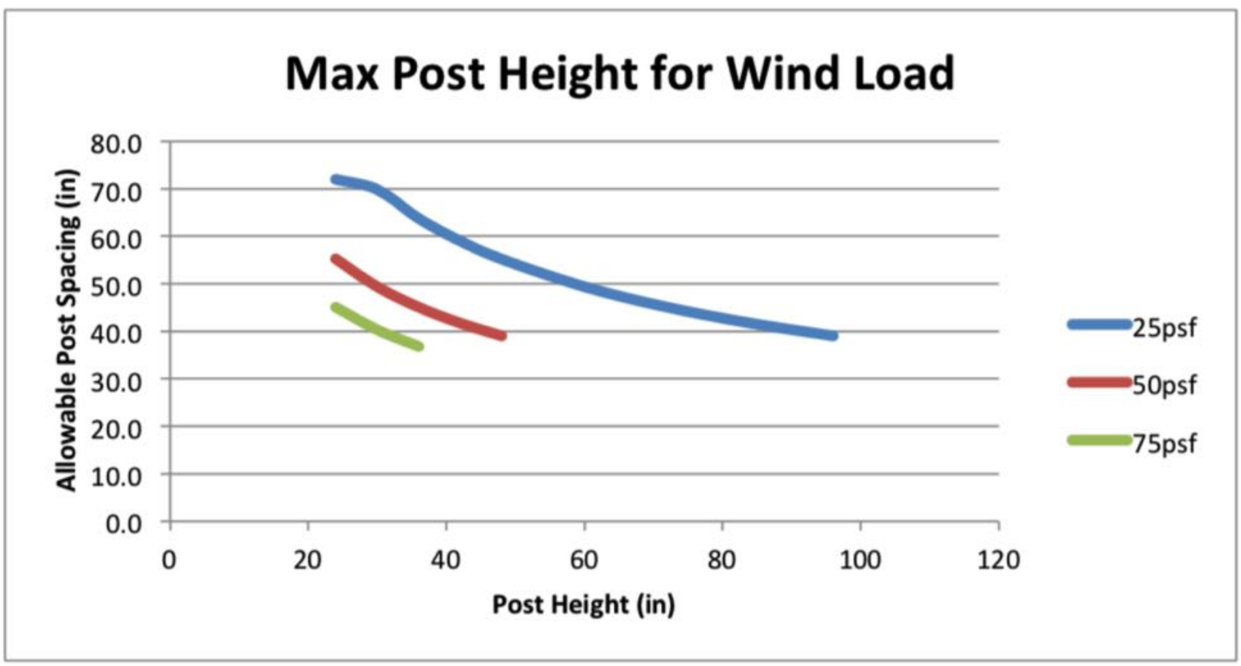
k) 3/8"x4" KH-EZ in cracked concrete and 6-1/2x6-1/2" baseplate
Allowable moment, $M_{a,x}$ = 12,700"#
Max post height = 60" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63.5
60	50.8
72	42.33333333
84	36.28571429
96	<36"



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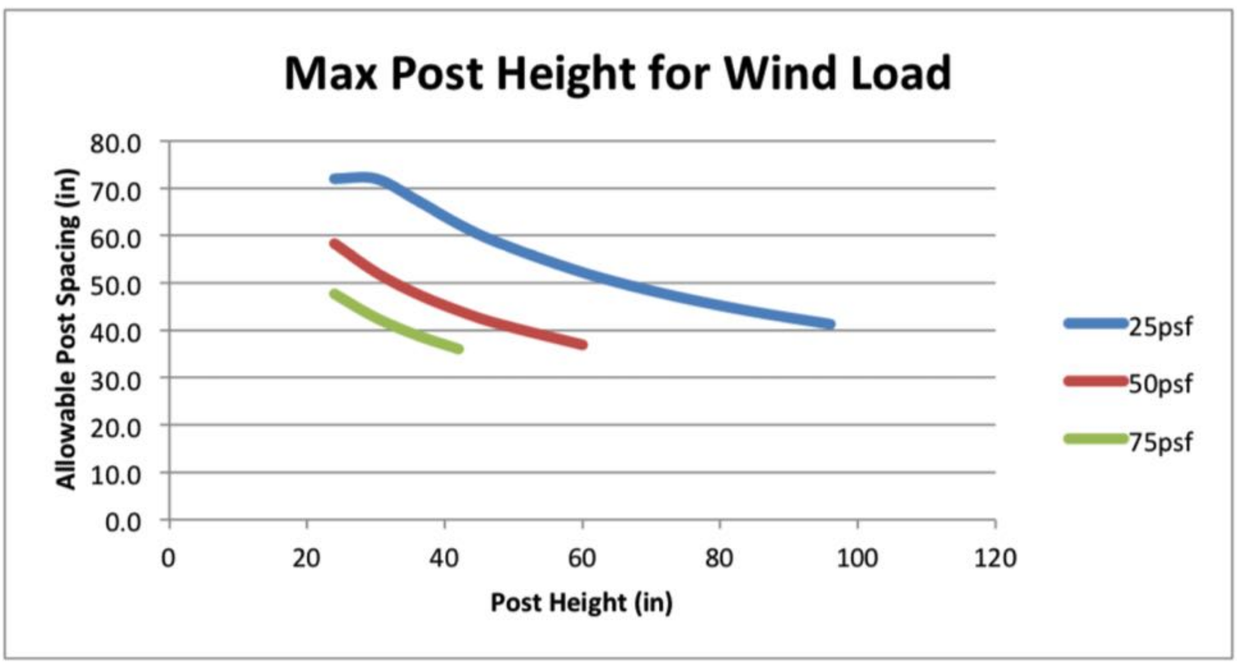
l) 3/8"x3-3/4" KB-TZ in uncracked concrete and 5x5" baseplate.
Allowable moment, $M_{a,x}$ = 14,200"#
Max post height = 60" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	71
60	56.8
72	47.33333333
84	40.57142857
96	<36"



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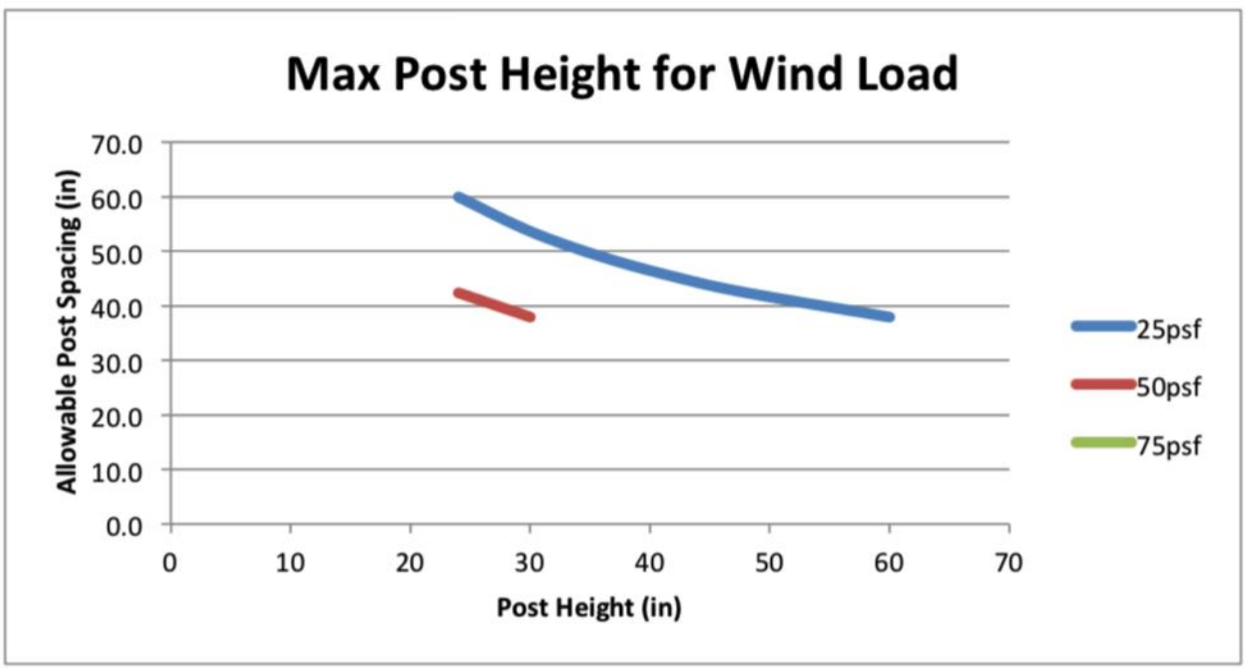
m) 3/8”x3-3/4” KB-TZ in uncracked concrete and 3x5” baseplate.
Allowable moment, $M_{a,x}$ = 7,490”#
Max post height = 36” when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	59.92
36	49.93333333
42	42.8
48	37.45
60	<36"
72	<36"
84	<36"
96	<36"



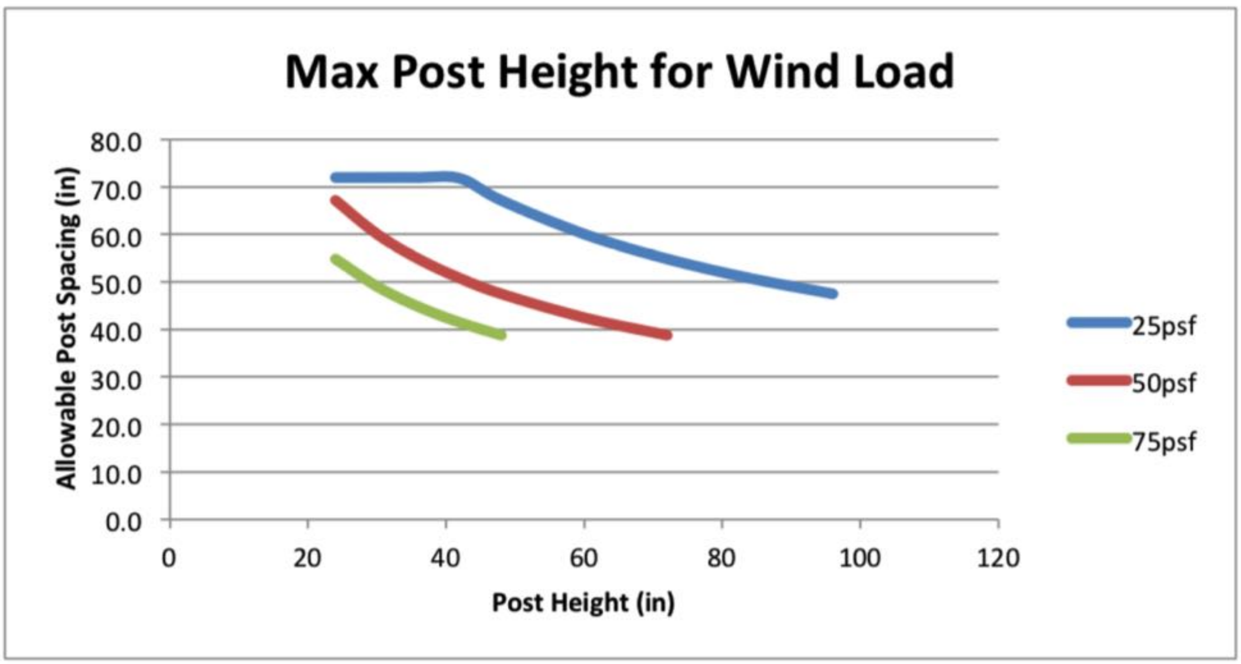
n) 3/8"x3-3/4" KB-TZ in uncracked concrete and 6-1/2x6-1/2" baseplate.
Allowable moment, $M_{a,x}$ = 18,800"#
Max post height = 94" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	72
72	62.66666667
84	53.71428571
96	47



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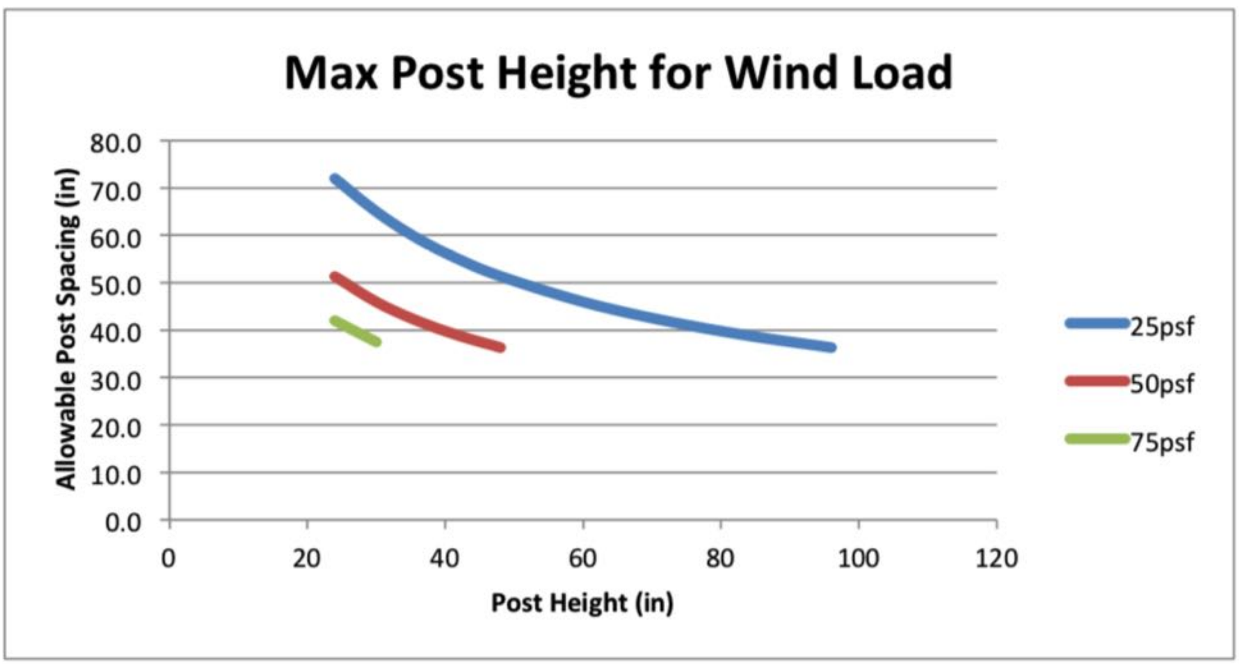
o) 3/8”x3-3/4” KB-TZ in cracked concrete and 5x5” baseplate.
Allowable moment, $M_{a,x}$ = 11,000”#
Max post height = 55” when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	62.85714286
48	55
60	44
72	36.66666667
84	<36"
96	<36"



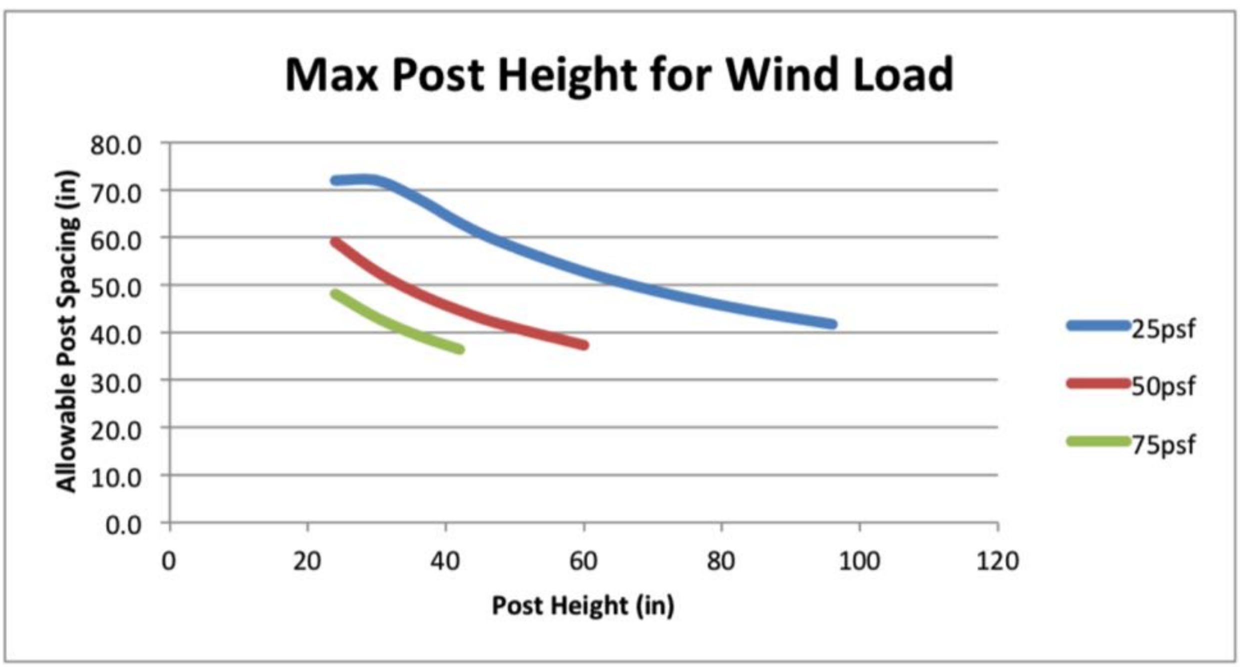
p) 3/8"x3-3/4" KB-TZ in cracked concrete and 6-1/2x6-1/2" baseplate.
Allowable moment, $M_{a,x}$ = 14,500"#
Max post height = 72" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	58
72	48.33333333
84	41.42857143
96	36.25



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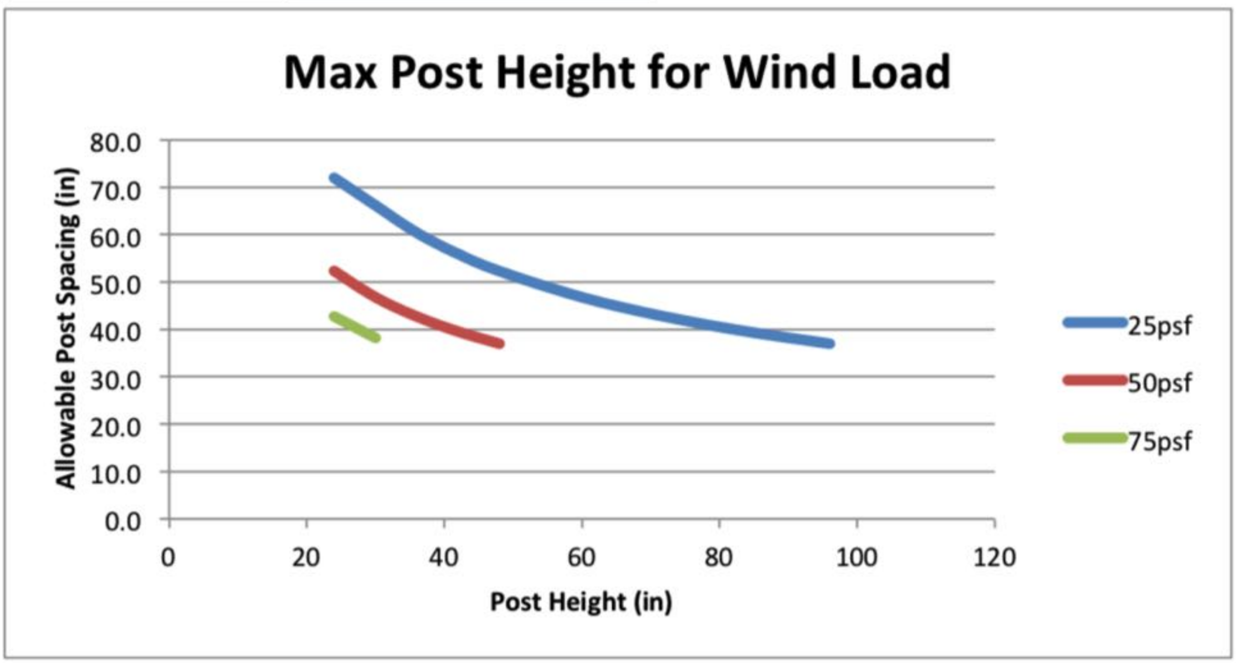
q) 3/8" A307 or 304 Lag Screw w/ 4-1/4" penetration and 5x5" baseplate
Optimal lag screw penetration.
Allowable moment, $M_{a,x}$ = 11,400"#
Max post height = 57" when subject to 200# concentrated load
Higher strength connections will require higher strength material or larger diameter.

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	65.14285714
48	57
60	45.6
72	38
84	<36"
96	<36"



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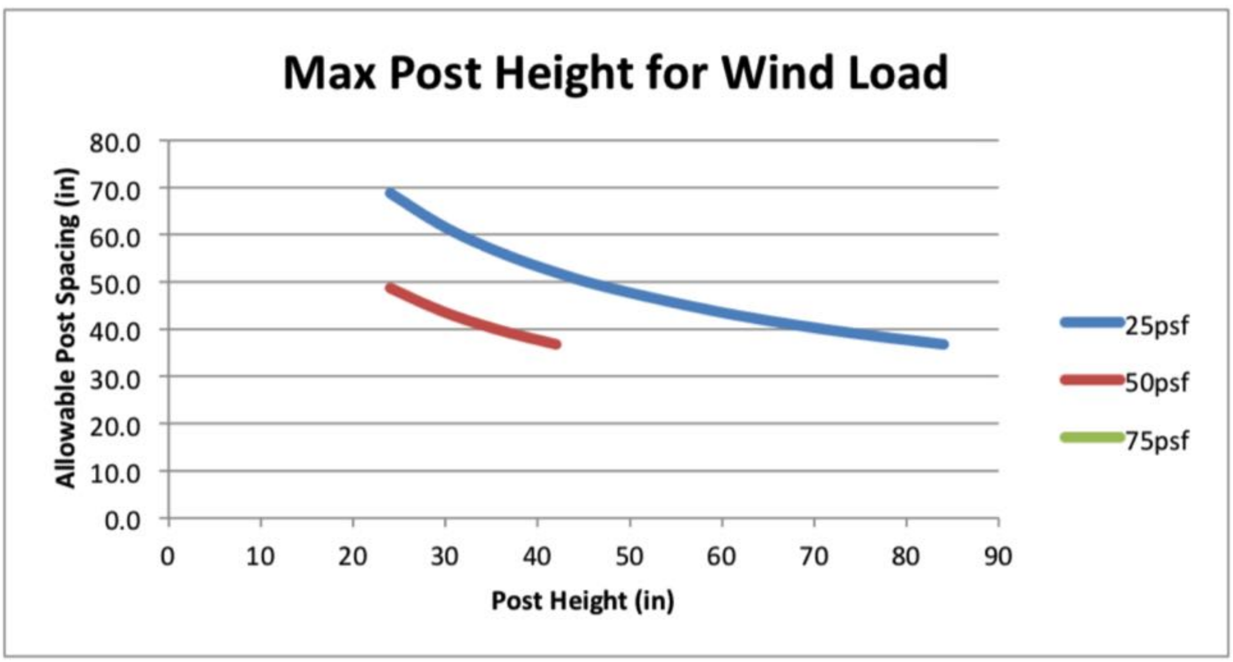
r) 3/8" A307 or 304 Lag Screw w/ 3-1/2" penetration and 5x5" baseplate
Allowable moment, $M_{a,x}$ = 9,860"#
Max post height = 49" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	65.73333333
42	56.34285714
48	49.3
60	39.44
72	<36"
84	<36"
96	<36"



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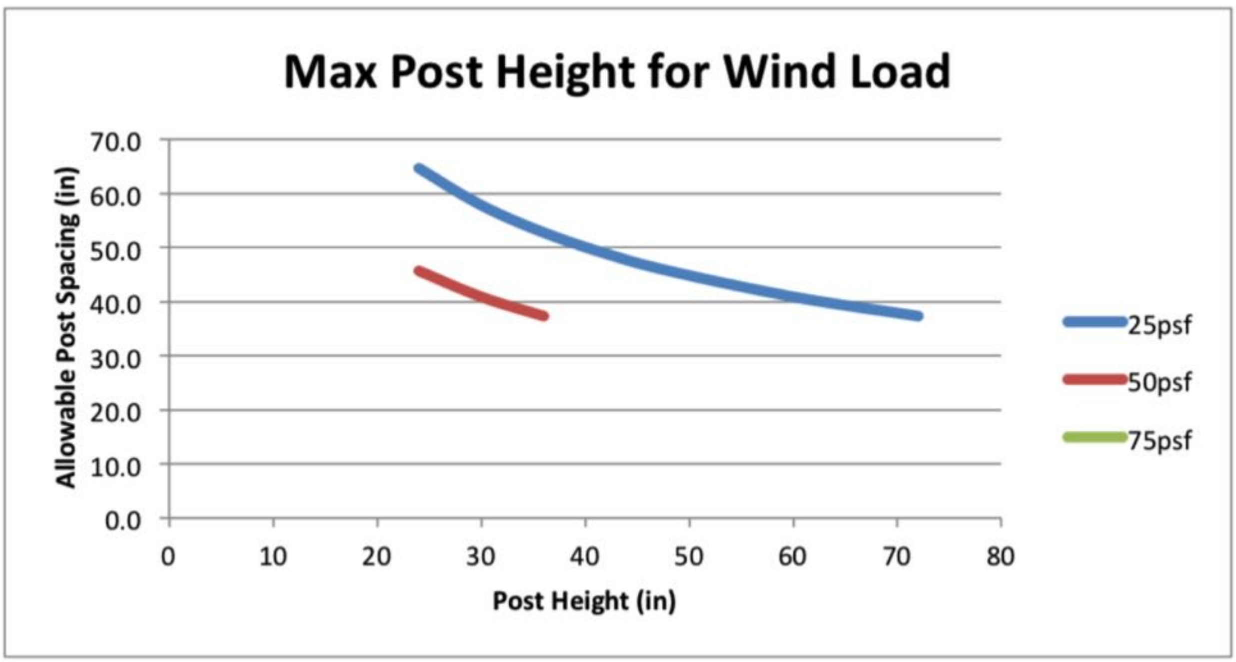
s) 3/8" A307 or 304 Lag Screw w/ 3" penetration and 5x5" baseplate
Allowable moment, $M_{a,x}$ = 8,700"#
Max post height = 43" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	69.6
36	58
42	49.71428571
48	43.5
60	<36"
72	<36"
84	<36"
96	<36"



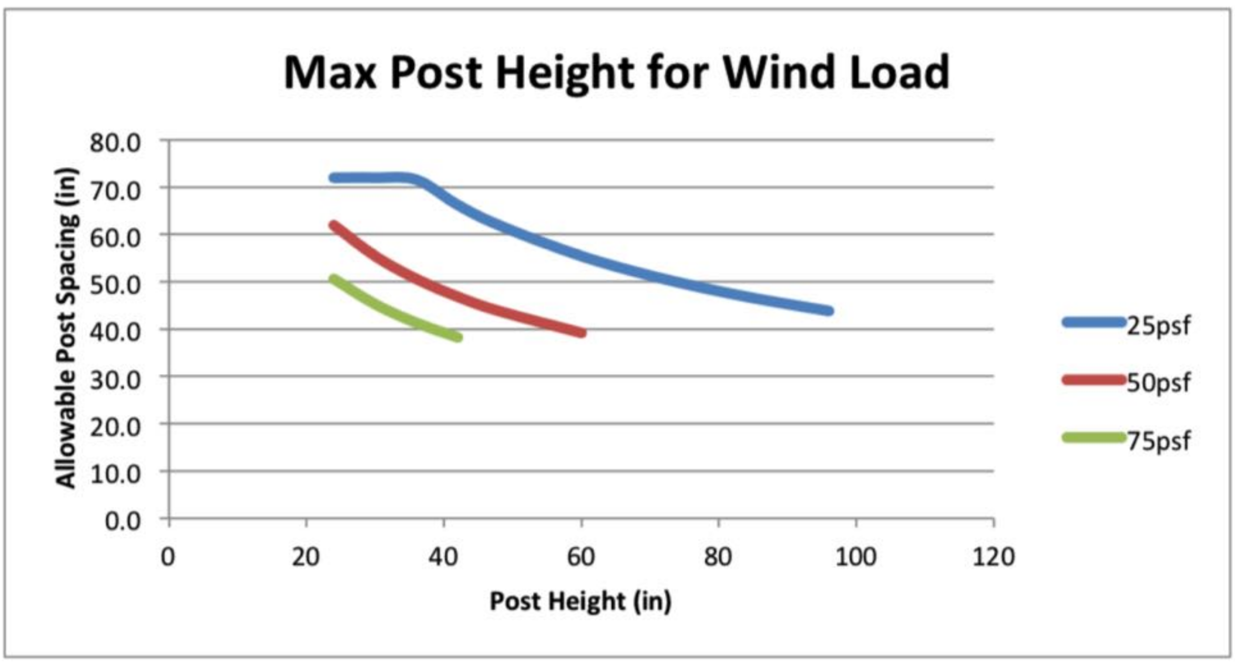
t) 3/8" A307 or 304 Lag Screw w/ 4-1/4" penetration and 6-1/2x6-1/2" baseplate
Optimal lag screw penetration.
Allowable moment, $M_{a,x}$ = 16,000"#
Max post height = 80" when subject to 200# concentrated load
Higher strength connections will require higher strength material or larger diameter.

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	72
60	64
72	53.33333333
84	45.71428571
96	40



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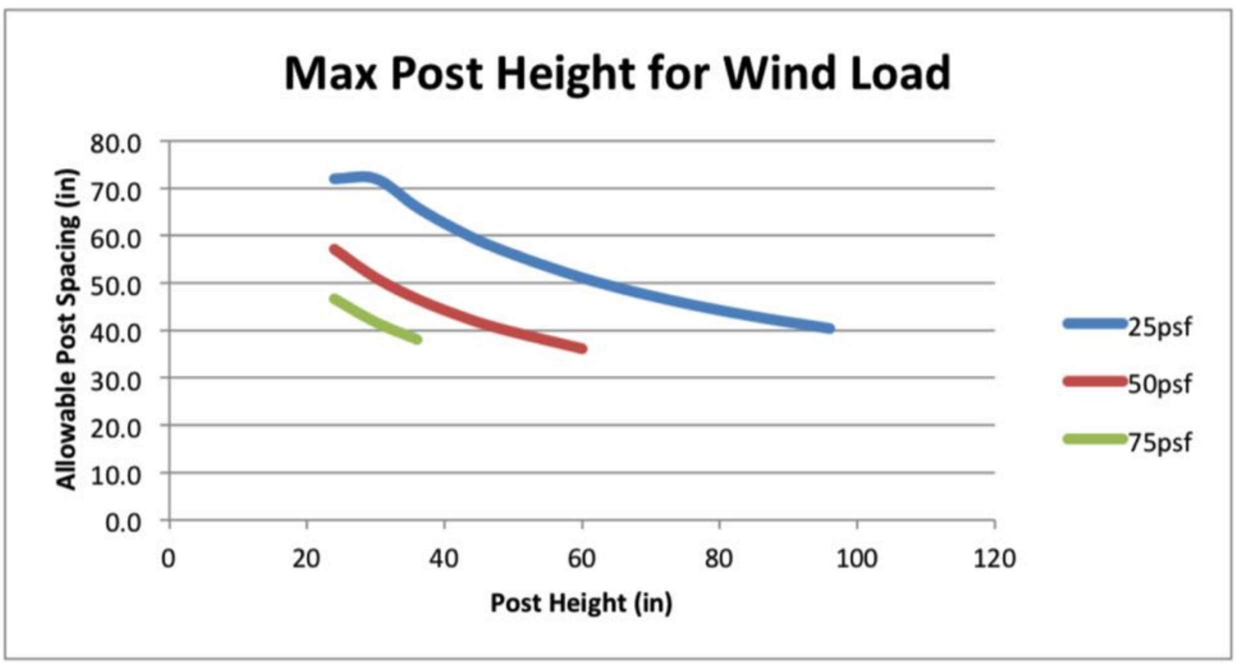
u) 3/8" A307 or 304 Lag Screw w/ 3-1/2" penetration and 6-1/2x6-1/2" baseplate
Allowable moment, $M_{a,x}$ = 13,600"#
Max post height = 68" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	68
60	54.4
72	45.33333333
84	38.85714286
96	<36"



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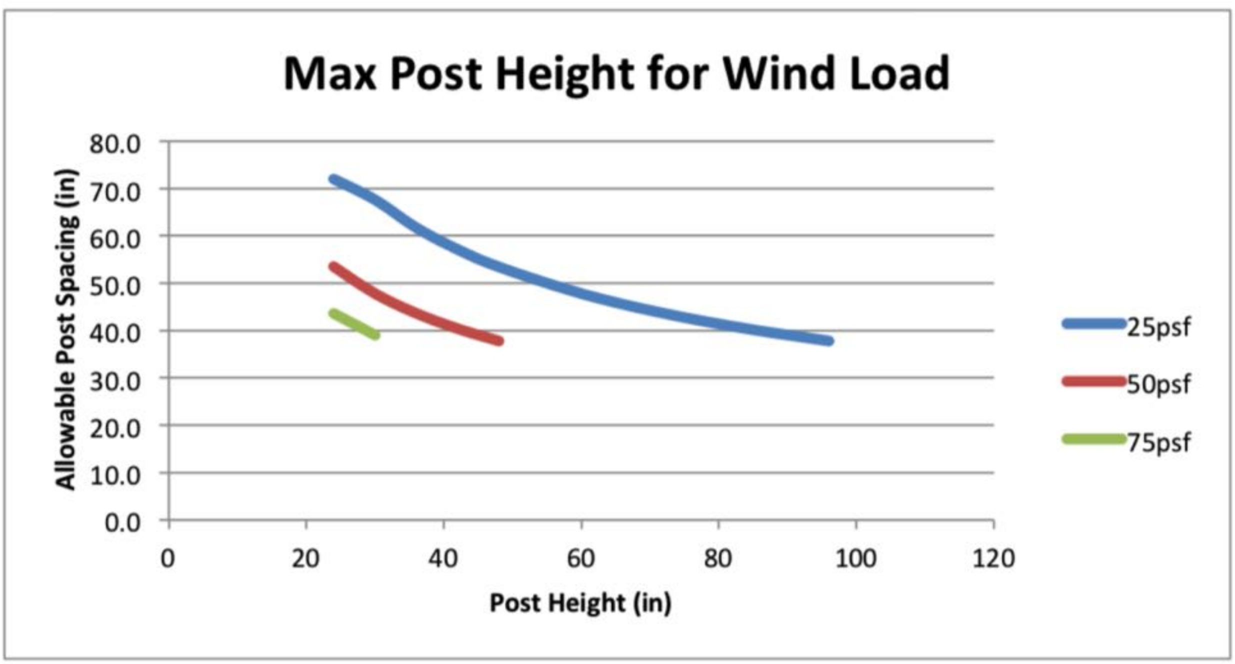
v) 3/8" A307 or 304 Lag Screw w/ 3" penetration and 6-1/2x6-1/2" baseplate
Allowable moment, $M_{a,x}$ = 11,900"#
Max post height = 59" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	68
48	59.5
60	47.6
72	39.66666667
84	<36"
96	<36"



Core Mount Details

w) Post set in 4” Deep Core Mount, 3-5/8” edge distance measured from center of post

Allowable moment, $M_{a,x}$ = 12,600”#

Max post height = 60” when subject to 200# concentrated load

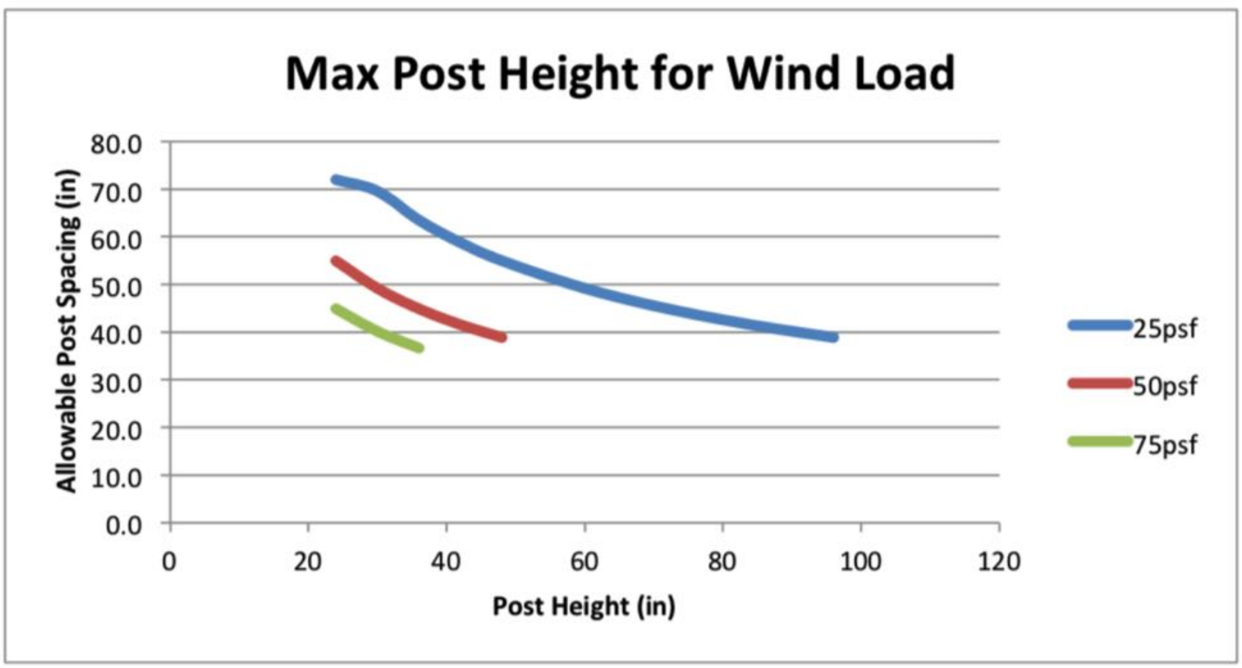
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	72
48	63
60	50.4
72	42
84	36
96	<36"



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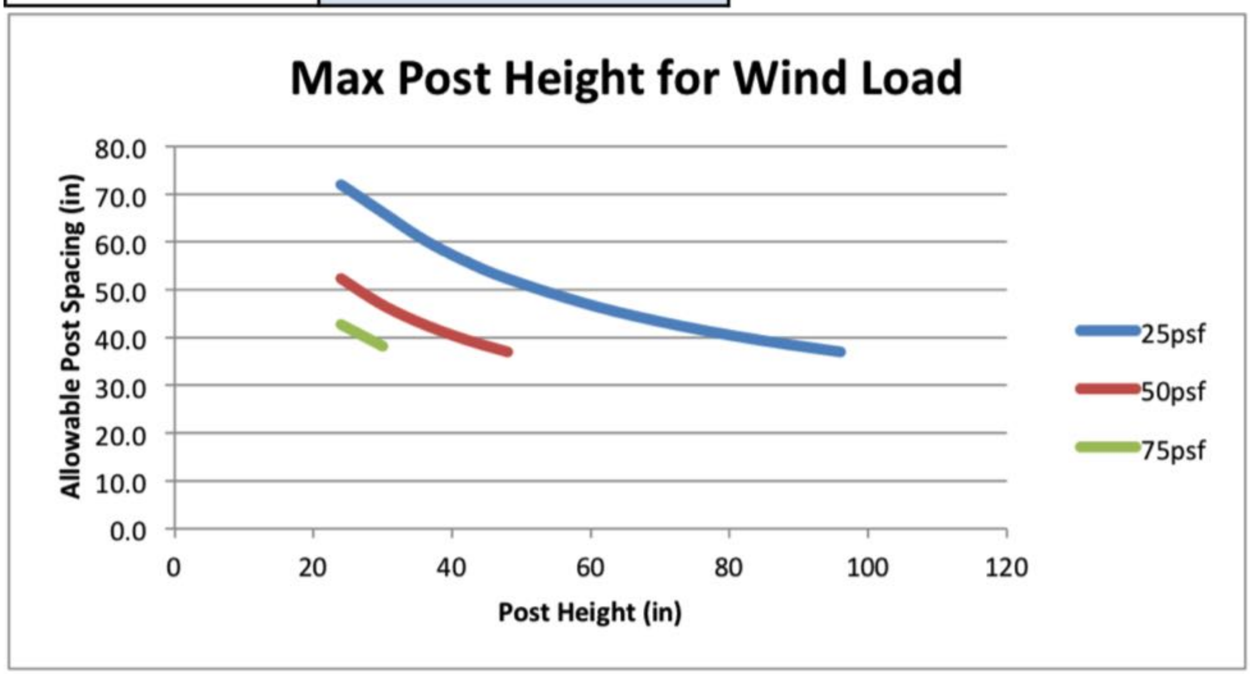
x) Stanchion set in 4" Deep Core Mount, 3-5/8" edge distance measured from center of post
Allowable moment, $M_{a,x}$ = 11,400"#
Max post height = 57" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	65.14285714
48	57
60	45.6
72	38
84	<36"
96	<36"



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Fascia Mount Details

y) Fascia Bracket To Wood, 3-3/8" lag Screw Penetration

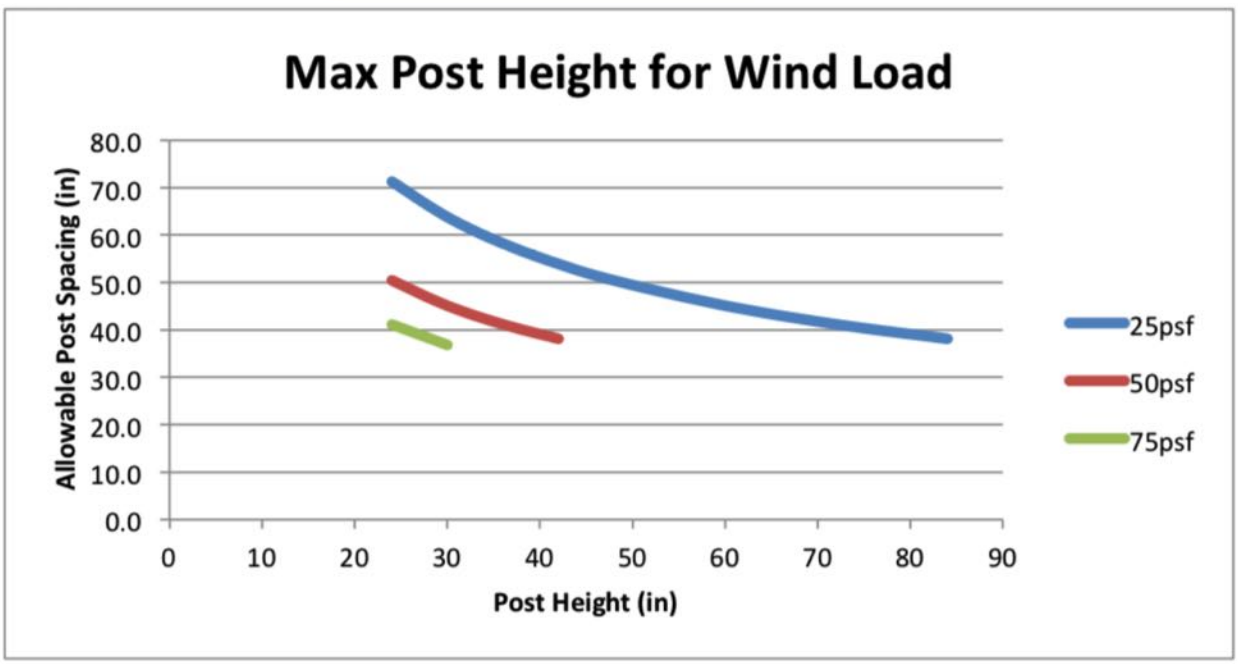
Top lag screws located 2" below floor.
Allowable moment, $M_{a,x}$ = 10,600"# (measured at floor)
Max post height = 53" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	70.66666667
42	60.57142857
48	53
60	42.4
72	<36"
84	<36"
96	<36"



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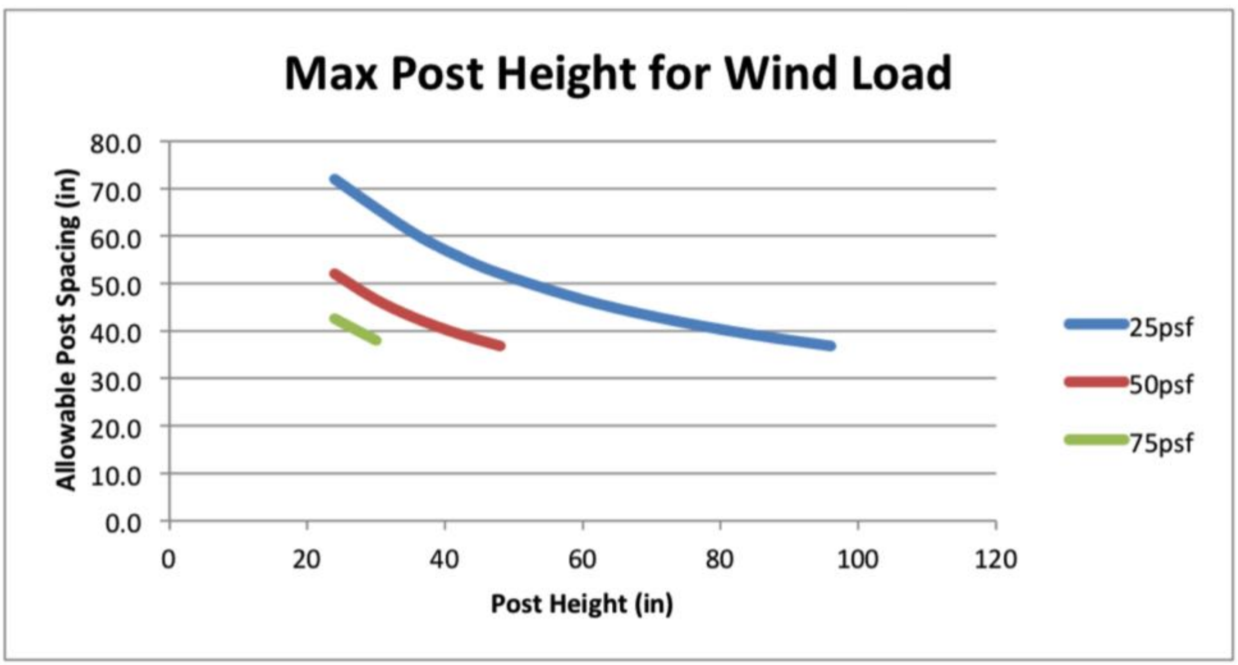
z) Fascia Bracket to Concrete, Uncracked Concrete
Allowable moment, $M_{a,x}$ = 11,300"# (measured at floor)
Max post height = 56.5" when subject to 200# concentrated load

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	72
36	72
42	64.57142857
48	56.5
60	45.2
72	37.66666667
84	<36"
96	<36"



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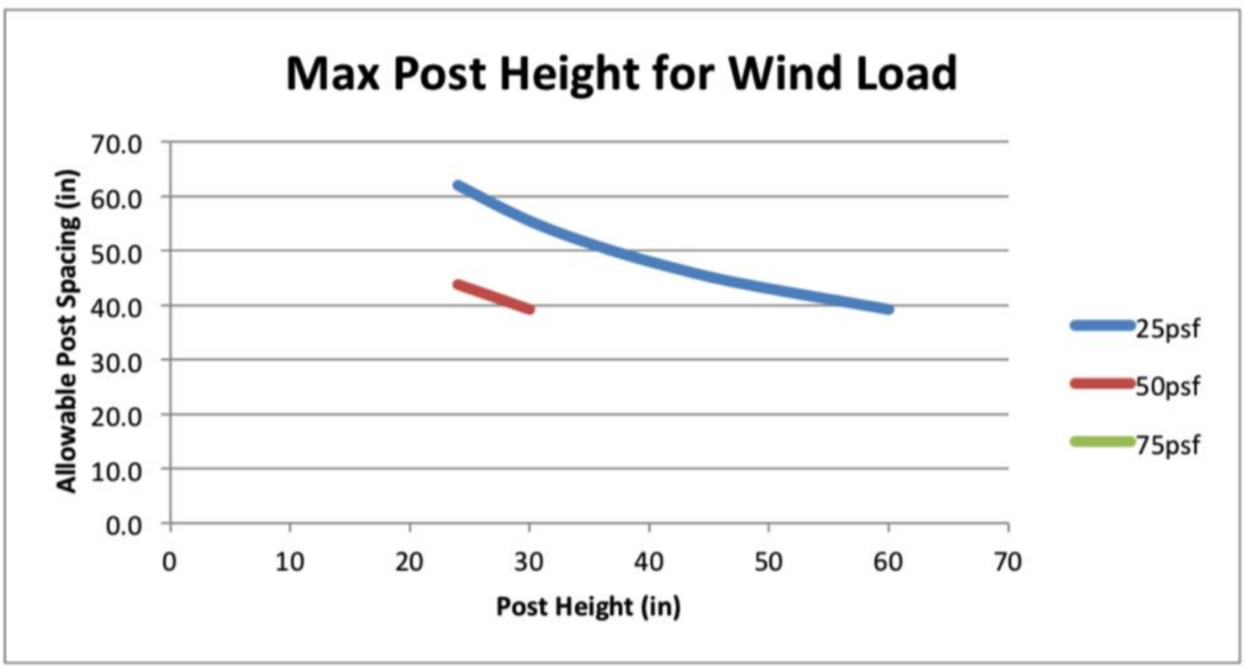
aa) Fascia Bracket to Concrete, Cracked Concrete
Allowable moment, $M_{a,x}$ = 8,000”# (measured at floor)
Max post height = 40” when subject to 200# concentrated load (42” allowed when there are at least 3 posts with a continuous top rail and spacing 48” on center max)

50plf uniform load along top rail

*M=50plf/12*TW*H*
*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	64
36	53.33333333
42	45.71428571
48	40
60	<36"
72	<36"
84	<36"
96	<36"



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ab) Post Directly Fascia Mounted W/ 3/8" Lag Screws
Allowable moment, $M_{a,x}$ = 7,800"# (measured at floor)
Max post height = 39" when subject to 200# concentrated load (42" allowed when there are at least 3 posts with a continuous top rail and spacing 48" on center max)

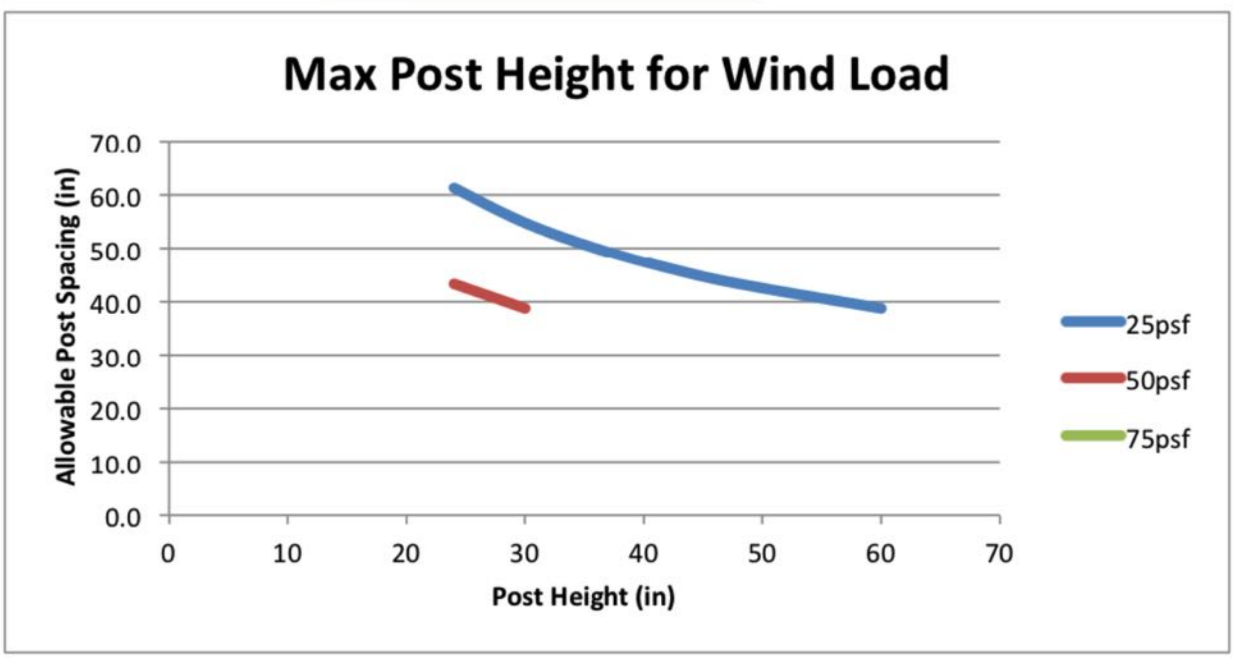
50plf uniform load along top rail

*M=50plf/12*TW*H*

*Hmax = Ma/(TW*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Allowable Post Spacing (in)
24	72
30	62.4
36	52
42	44.57142857
48	39
60	<36"
72	<36"
84	<36"
96	<36"



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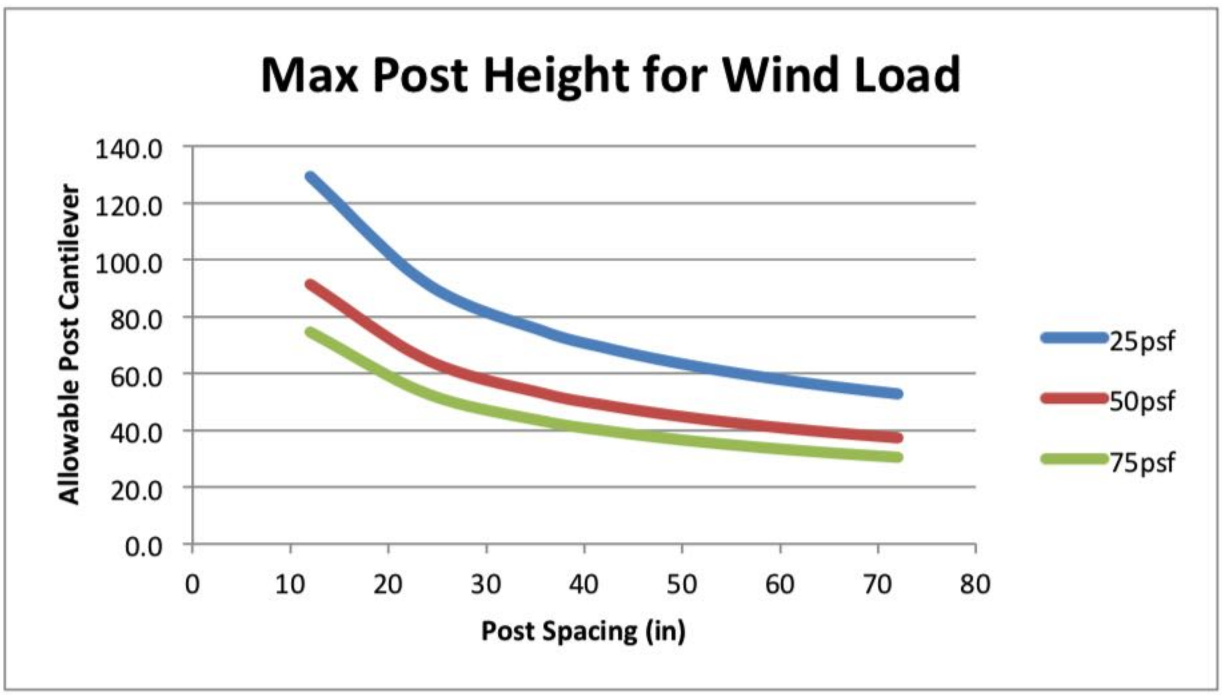
ac) Post Directly Fascia Mounted W/ 3/8" Carriage Bolts
Allowable moment, $M_{a,x}$ = 17,400"# (measured at floor)
Handles top rail live loading at 60" tall with 66" post spacing or 54" tall with 72" post spacing.

Allowable post height with respect to post spacing:

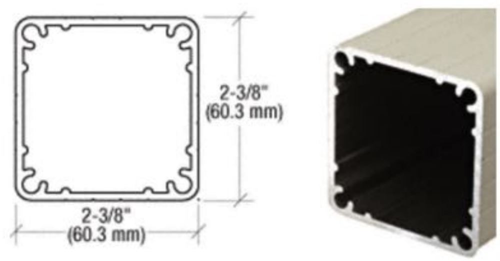
Tributary Width (in)	Max Height (in)
12	348
24	174
36	116
42	99.42857143
48	87
54	77.33333333
60	69.6
66	63.27272727
72	58



STEP 3: POSTS

Check allowable post spacing for selected posts with regard to the 50plf top rail live load and the applicable wind loading. All posts may be used at 60" tall.

4 screw 2-3/8" post
May be used at 60" tall when spacing is limited to 66".
Spacing may be increased to 72" for post height 57" or less.



200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	85.5
Δ at H=42"	0.56145775

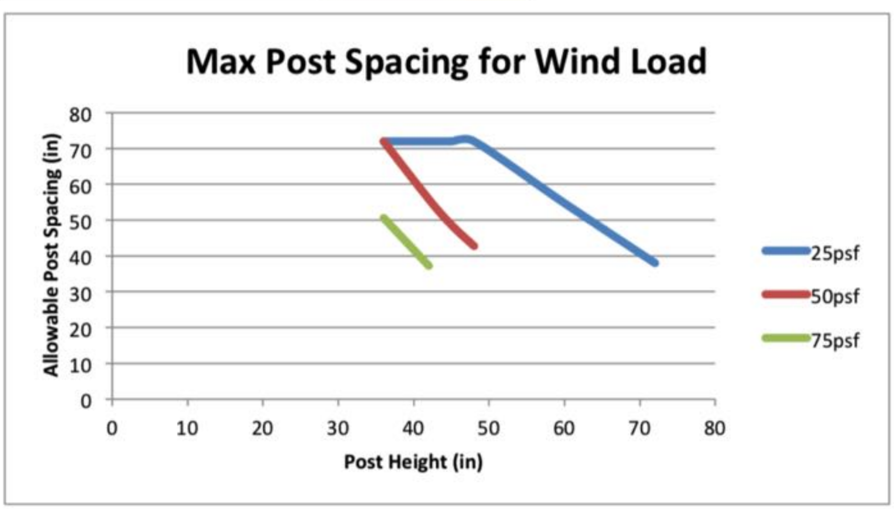
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H³*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	68.4
72	57
84	48.85714286
96	42.75



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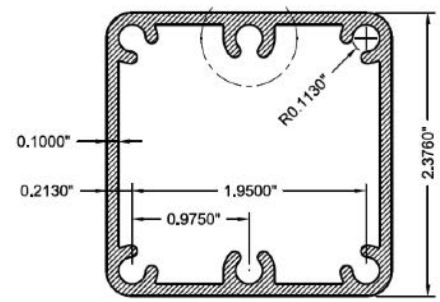
6 screw 2-3/8" post



200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	>96"
Δ at H=42"	0.49050121

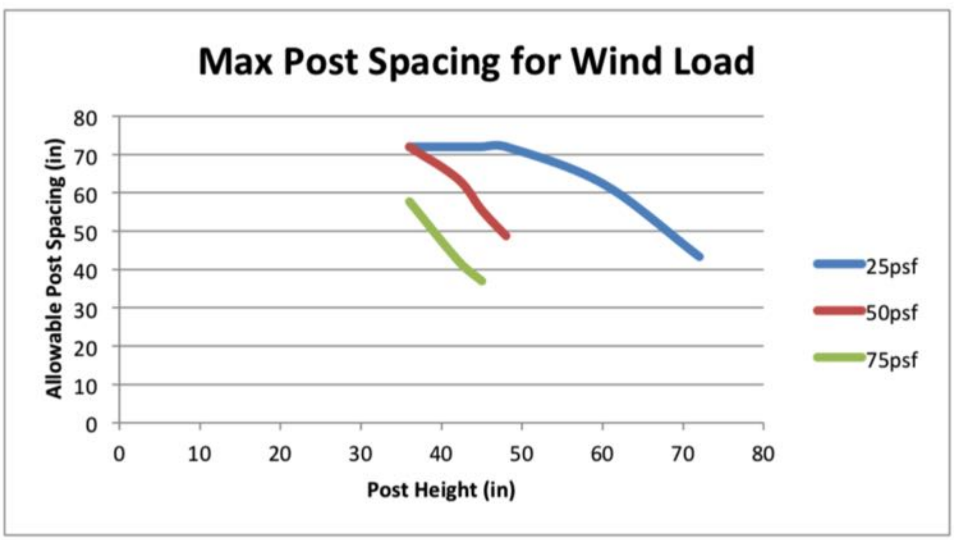
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
36	72
42	72
45	72
48	72
60	72
72	65
84	55.71428571
96	48.75



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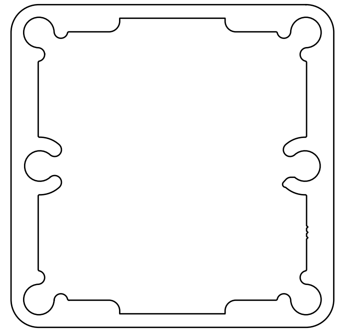
6 screw 2-3/8" heavy post



200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	120
Δ at H=42"	0.38811881

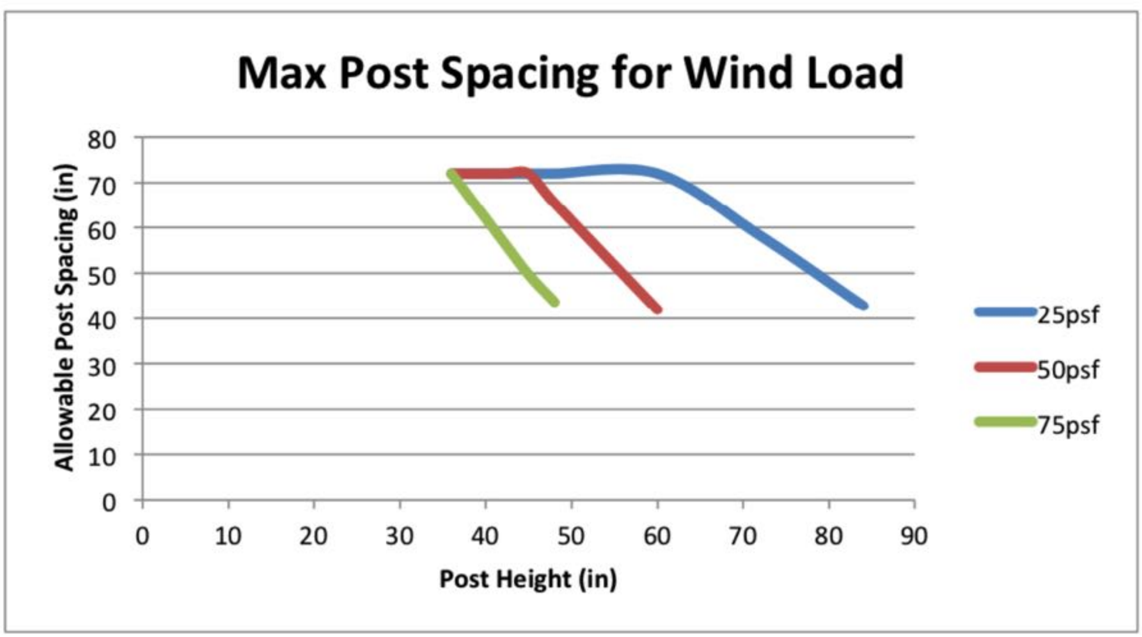
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
42	72
48	72
60	72
72	72
84	72
96	65.5
108	58.22222222
120	52.4



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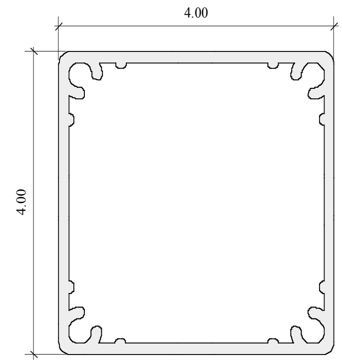
4" post
Designed for H/48 deflection criteria.



Load Cases:

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	131.513998
Δ at H=42"	0.089239

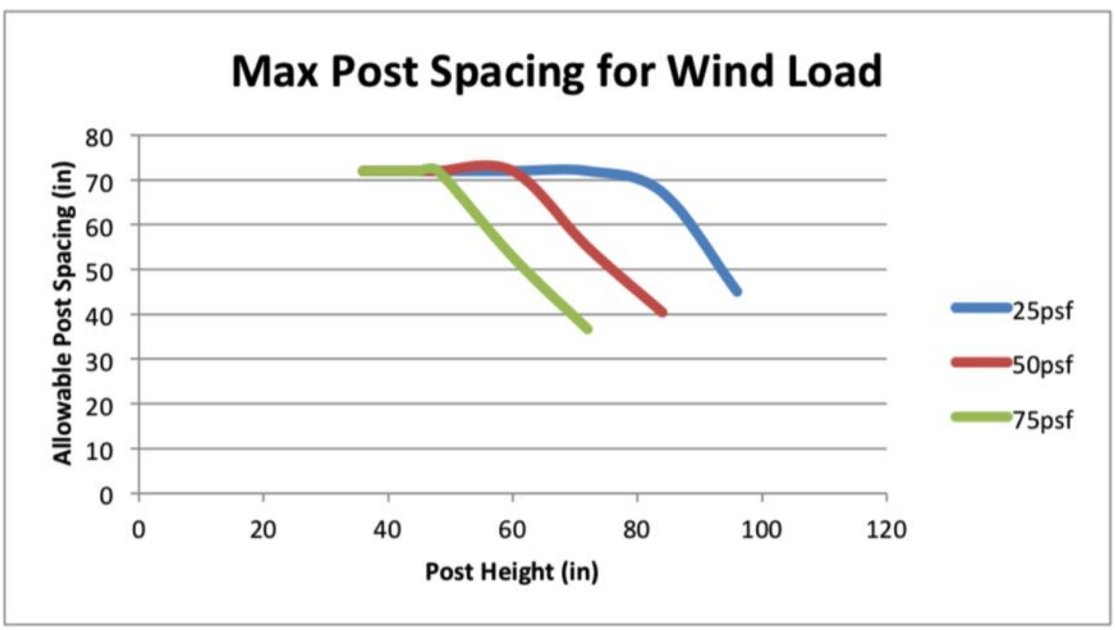
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H^3*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
42	72
48	72
60	72
72	72
84	72
120	57.65416667
132	47.64807163
144	40.03761574



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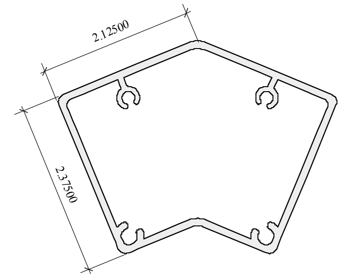
135˚ post

When 135˚ degree posts are used along with any of the other 2-3/8" posts, the intermediate posts limit the allowable post spacing.



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Trim-Line Post

200# concentrated load at top of post

*M = 200#*H*

*Hmax = Ma/200# <(Δa*3EI/200#)^(1/3)*

Hmax	52
Δ at H=42"	0.93326279

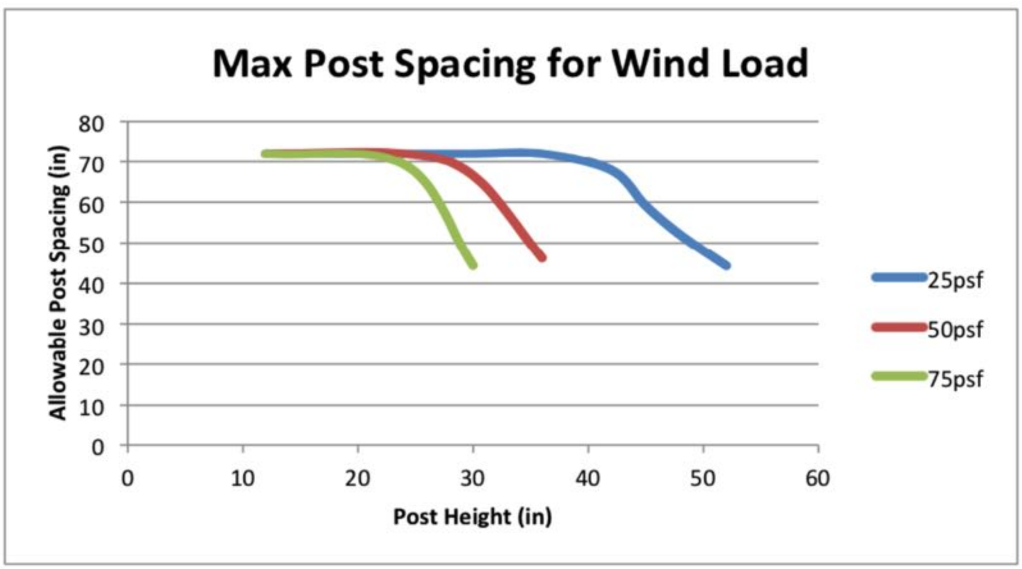
50plf uniform load along top rail

*M=50plf/12*TW*H*

*TWmax = Ma/(H*50plf/12) < Δa*3EI/(H³*50plf/12)*

Allowable post height with respect to post spacing:

Post Height (in)	Max Spacing (in)
12	72
24	72
30	72
36	69.33333333
42	59.42857143
45	55.46666667
48	52
52	48



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STEP 4: TOP RAIL

When pickets or glass infill attach to top rail:
Series 200X top rail max post spacing = 68”
All other top rails, max post spacing = 72”

When pickets or glass infill do <u>not</u> attach to top rail:
Examples, when a mid rail is used or cable infill without a picket spreader in the middle.
Allowable post spacing according to the table below:

Top Rail Allowable Spans:

Top Rail		Ma (in-lbs)	Allowable Span (in)
	100	3750	72
	200	2640	52.8
200X		1790	35.8
	300	6430	72
	320	3000	60
	350	4300	72
	400	5130	72
	500	2600	52

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