3 March 2020

Wentworth Metal Works
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SUBJ: GLASS LOCK 3000 ENGINEERING REPORT
TEMPERED GLASS INFILL RAILING

The Glass Lock 3000 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 applications and is suitable for use in all natural environments. The Glass Lock 3000 system may be used for residential and commercial applications.

The Glass Lock 3000 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 = Varies by location

Refer to IBC Section 1607.8.1 for live loading criteria

The Glass Lock 3000 system will meet all applicable requirements of the 2012, 2015 and 2018 International Building Codes and International Residential Codes, 2010, 2013 and 2016 California Building and Residential Codes. 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 Glass Lock 3000 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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TYPICAL INSTALLATIONS

The following represent the conditions the Glass Lock 3000 is intended for use. A project specific analysis can be used to demonstrate the Glass Lock 3000 system may be used in applications outside of the limitations set below.

Max post spacing = 48” (Except for A111 top rail)
Railing height above walking surface = 42”
Allowable wind load = 25psf (Except for GRK fascia anchor where allowable wind load = 22.5psf)

A111 and wood cap rails have limited allowable post spacing when used in guard rails.

All standard anchorage details develop at least 8,400”# allowable moment strength.

LOAD CASES:

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

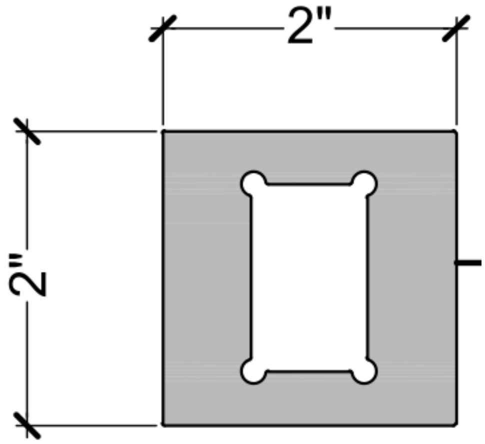
For top rail loads:
M_c = 200#*H
M_u = 50plf*S*H

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 1714.3.1 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 1714.3 and AC273.

2” Square Post
6061-T6 Aluminum



Full Section Properties:
-Area 2.939”
$I_x = 1.174in^4$
$I_y = 1.272in^4$
$S_x = 1.174in^4$
$S_y = 1.272in^3$
$Z_x = 1.644in^3$
$Z_y = 1.779in^4$

(Loading perpendicular to the glass will load the post about the y axis.

Based on 2015 ADM Chapter F

Yielding/Rupture/Local Buckling:
The aluminum extrusion has very thick walls and is not susceptible to local buckling. The post may be designed to its full plastic strength.
Yield strength, $F_y/\Omega = 35ksi/1.65 = 21.2ksi$
Ultimate strength, $F_u/\Omega = 38ksi/1.95 = 19.5ksi$ (Ultimate strength controls design)

Loading parallel to top rail, $M_{a,x} = 1.644in^3 * 19.5ksi = 31{,}000$”#
Loading perpendicular to top rail, $M_{a,y} = 1.779in^3 * 19.5ksi = 34{,}700$”#

At Welded End:
The heat affected zone caused by welding the baseplate reduces the strength of the aluminum. Only ultimate strength needs to be checked since this is a short length right at the end of post that is weakened.

Ultimate strength, $F_u/\Omega = 24ksi/1.95 = 12.3ksi$

Loading parallel to top rail, $M_{a,x} = 1.644in^3 * 12.3ksi = 20{,}200$”#
Loading perpendicular to top rail, $M_{a,y} = 1.779in^3 * 12.3ksi = 21{,}900$”#

At Drilled Holes:
Fascia mounts involve drilling fasteners through post. Max fastener size = 3/4” (Assume 1” hole maximum)
$Z_{reduced} = 1.779in^{4}-2*(0.605"*1"*0.6975") = 0.9350in^3$
$M_a = 19.5ksi*0.9350in^3 = 18{,}200$”#

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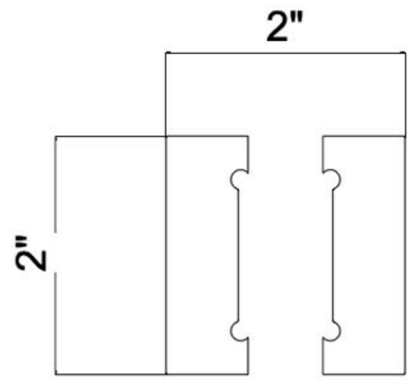
Reduced Section At Glass:
Top and bottom walls are removed to provide opening through post for glass to pass through. The opening is very long compared to the length of the post so it behaves differently than bending around a hole. The top of the post has a 1-5/8" portion that is solid so the halves also behave differently than two cantilevered flat bars.



Section Properties for each half:
$I_x = 0.4293in^4$
$I_y = 0.04075in^4$
$S_x = 0.4293in^3$
$S_y = 0.1082in^3$
$Z_x = 0.6310in^3$
$Z_y = 0.1926in^3$

For bending about the X-axis, each part of the post must be checked for lateral torsional buckling. For bending about the Y-axis, the aluminum may be designed to its plastic strength.
Yield strength, $F_y/\Omega = 35ksi/1.65 = 21.2ksi$
Ultimate strength, $F_u/\Omega = 38ksi/1.95 = 19.5ksi$ (Ultimate strength controls design)

Lateral torsional buckling is according to 2015 ADM F.4.2.4:
$\lambda = 2.3/0.605"*(2"*37"/1.3)^{1/2} = 28.68 < 66$
$M_{nmb} = 0.6310in^3*35ksi(1-28.68/66)+\pi^2*29000*28.68*0.4293in^3/66^3 = 24.75in\text{-}kips$

Total strength for loading parallel to the top rail, $M_{a,x} = 2*24{,}750"\#/1.65 = 30{,}000"\#$
Total rupture strength, $M_u/\Omega = 19.5ksi*2*0.6310in^3 = 24{,}600"\#$ (controls)

Strength for each bar for loading perpendicular to the top rail, $M_{a,y} = 19.5ksi*0.1926in^3 = 3{,}760"\#$

The post is modeled using SCIA Engineer 18.1 in order to analyze how the slot affects the bending of the post. At the glass slot the two halves are modeled as separate members. The model assumes a 42" total post height. Result diagrams of the model are shown following the calculations below.

A 200# load at the top of the post causes a moment of 2,015"# and a compression of 2,780#. The upper solid section provides significant moment resistance and causes a large portion of the moment to be resisted by a tension compression couple between the two halves.

Find allowable compression load:
$A = 1.243in^2$
$r_{min} = 0.182in$
$kl/r = 0.5*37"/0.182" = 102$
$F/\Omega = 51352/102^2 = 4.94ksi$

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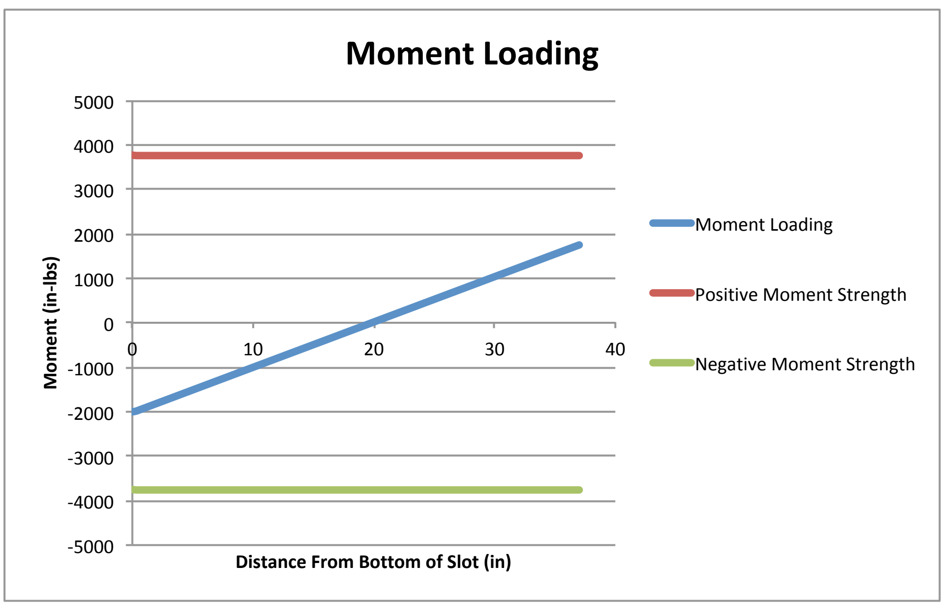
$P_a = 1.243in^2*4.94ksi = 6{,}140\#$

Check combined forces:
Moment varies along the length of post and axial strength varies along the length. Axial strength is a minimum at the middle of the slot and is 6,140#. Right at the end of the slots, the axial strength is controlled by yielding/rupture and is $19.5ksi*1.243in^2 = 24{,}200\#$.

The slenderness (λ) varies linearly along the post's length.
$\lambda = kL/r = 1*L/0.182”$ (where L is the distance from the bottom or top of the slot)
$\lambda = 66 = 1*L/0.182”$, L = 12.0” at transition between plastic and elastic buckling
$P_a = 51352/\lambda^2*1.243in^2$ (Between 12” and 18.5” from ends)
$\lambda = 17.8 = 1*L/0.182”$, L = 3.24” at transition between plastic buckling and yielding
$P_a = 1.243in^2(0.00047\lambda^2-0.232\lambda+25.2)$ (Between 3.24” and 12”)
$P_a = 24{,}200\#$ (Less than 3.24” for ends)

Moment at top of bar = 1,750”#
Moment at bottom of bar = - 2,020”#
Moment varies linearly between top and bottom.

Moment and axial loading and strength are plotted using excel and combined stress checks are performed at each location.



The chart above demonstrates moment loading is OK at points along the slot.

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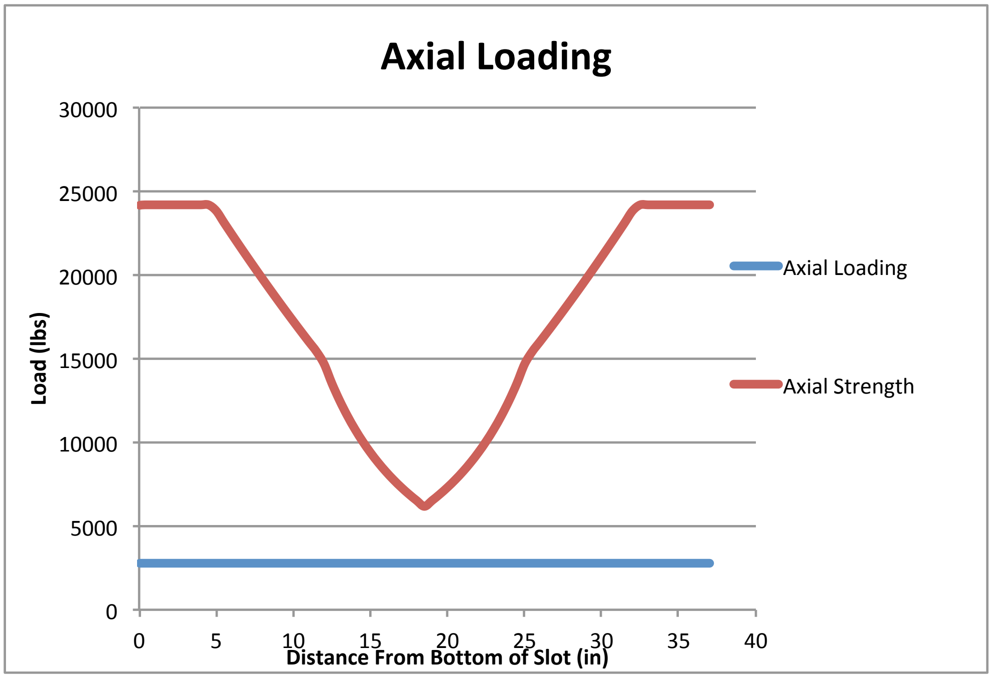
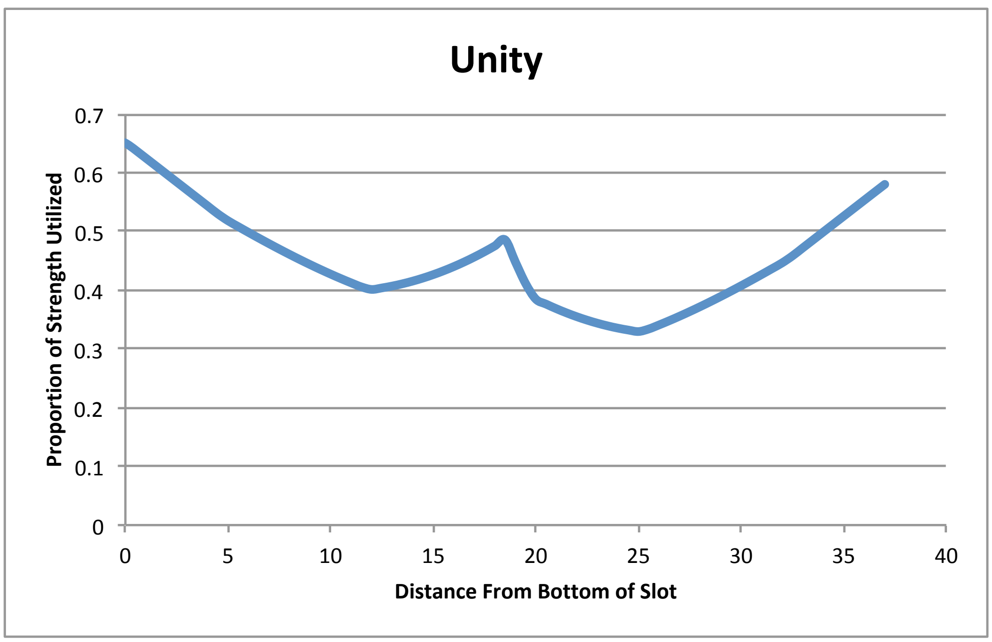


The chart above demonstrates the axial loading is OK at all points along the slot.



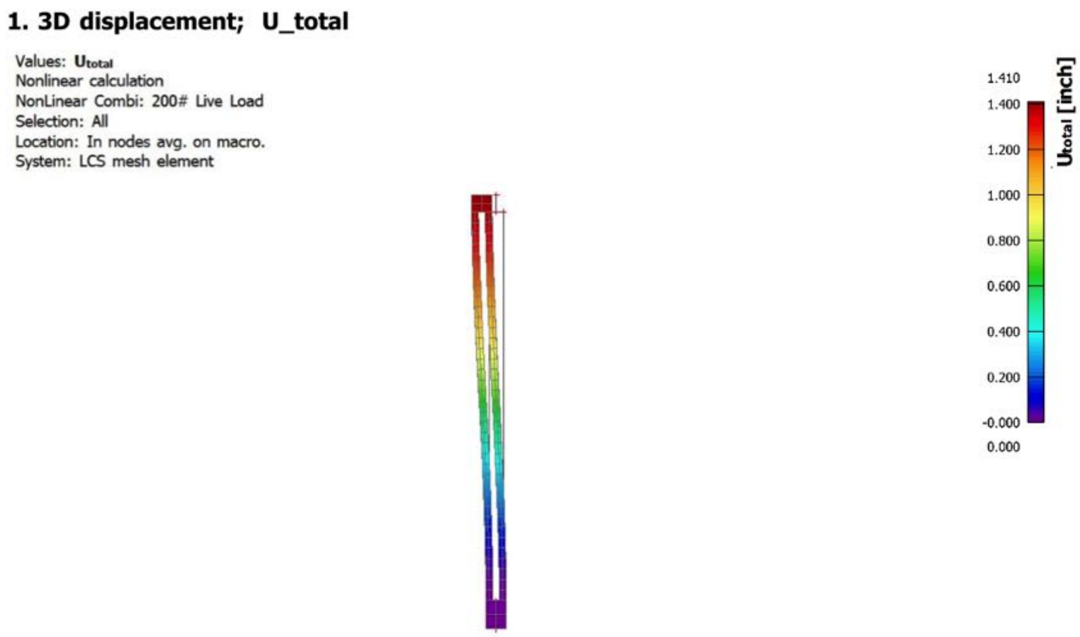
The above chart shows the combined stress is OK at all points along the slot.

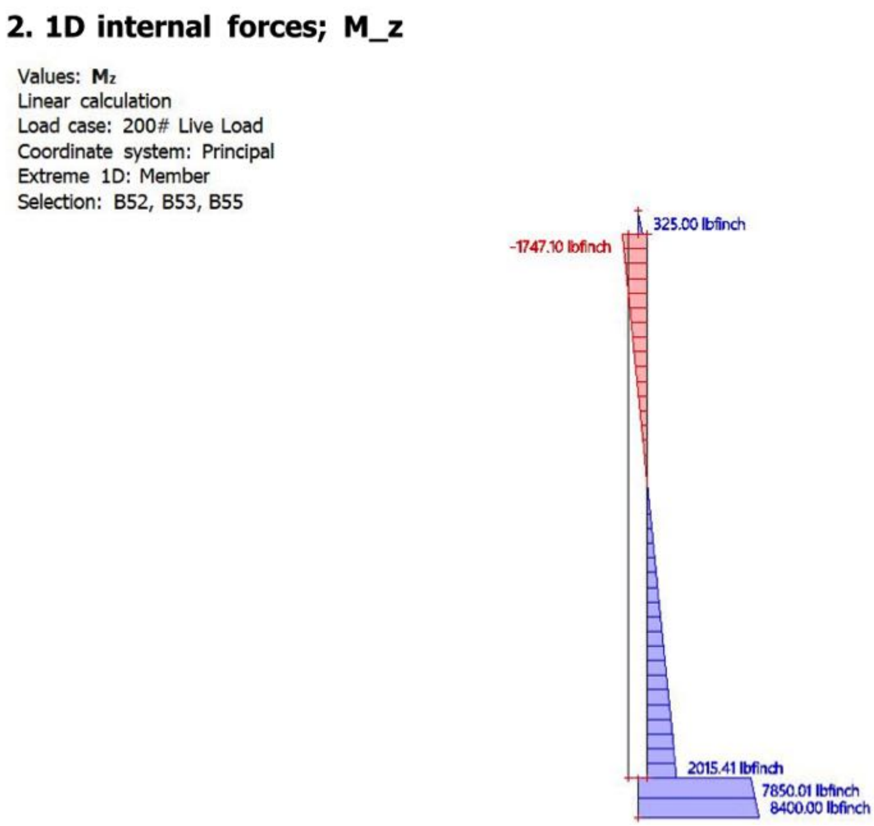
Max unity value = 0.652 and it occurs right at the bottom of the slot. Therefore, the expected failure mode is formation of a plastic hinge at the bottom of the slot.

Max allowable load at the top of the post = 200#/0.652 = 307#.

Allowable moment above base of slot = 307#*39.25” = 12,000”#

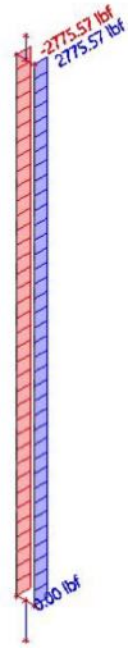
Model result diagrams are shown below. The deflection diagram shows the deflected shape to scale.



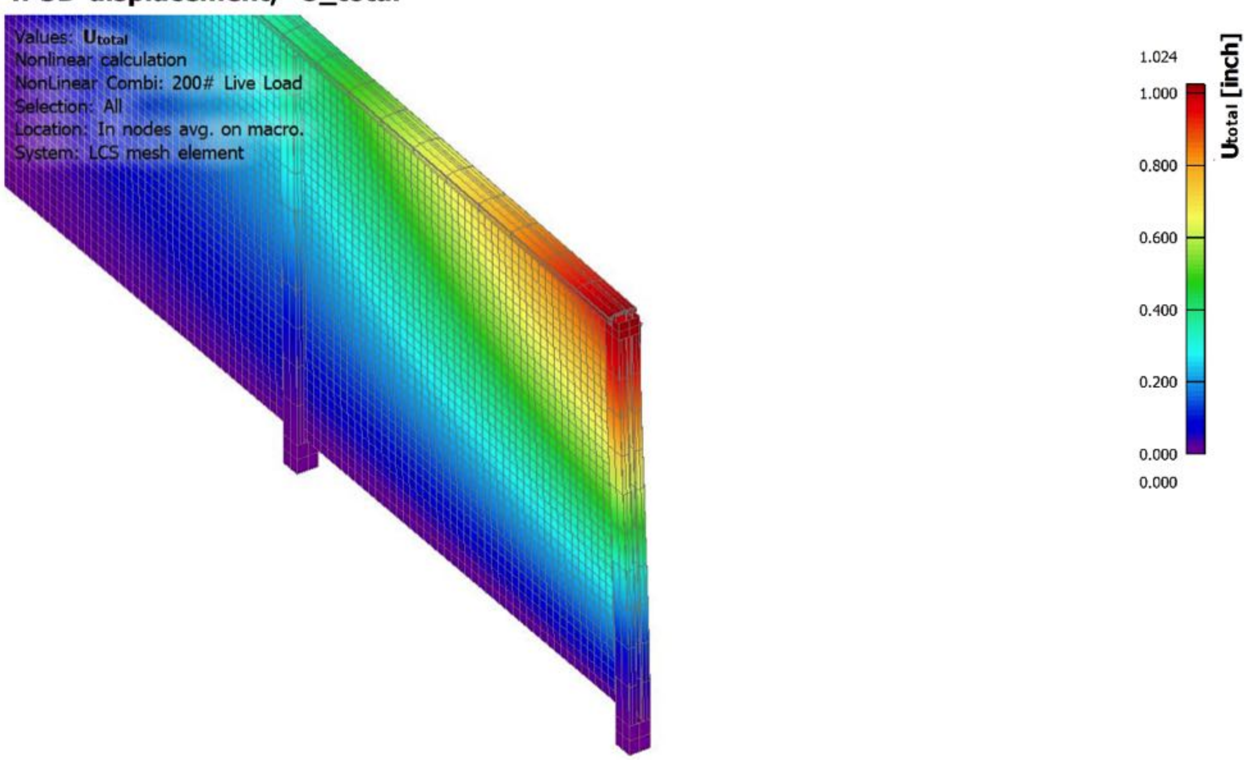


3. 1D internal forces; N

Values: **N**
Linear calculation
Load case: 200# Live Load
Coordinate system: Principal
Extreme 1D: Member
Selection: All



4. 3D displacement; U_total



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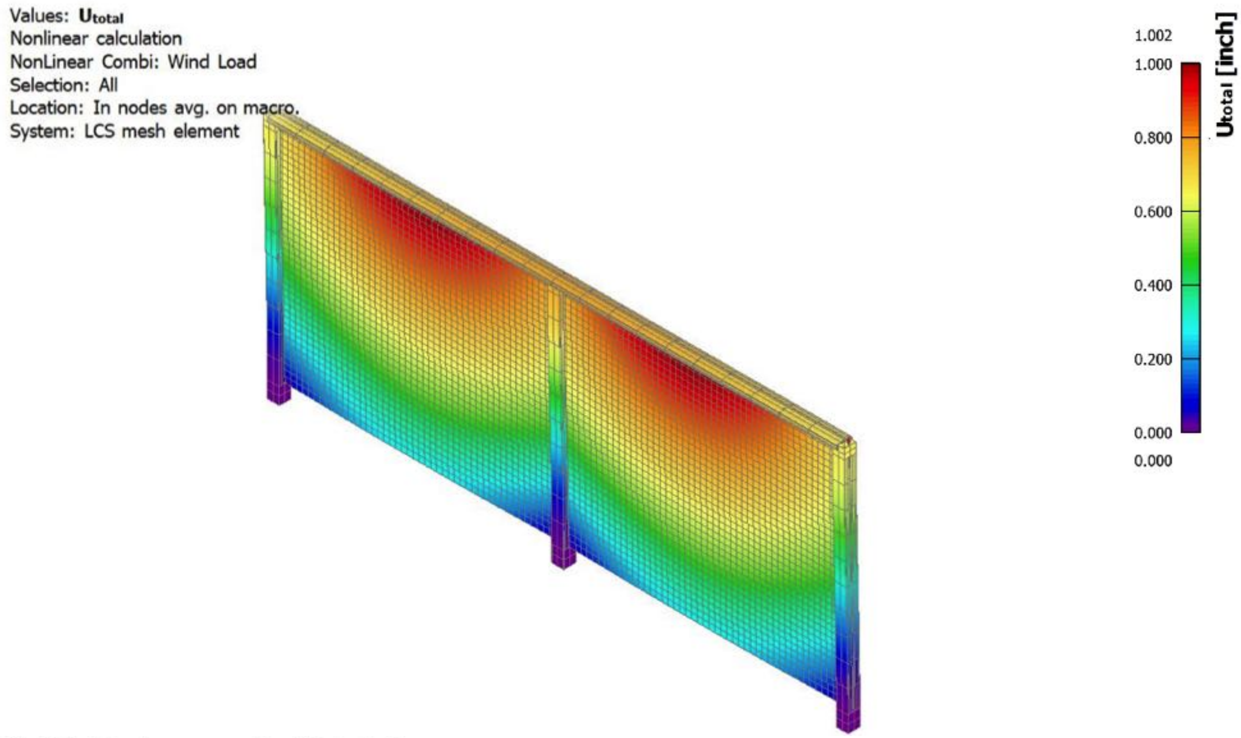
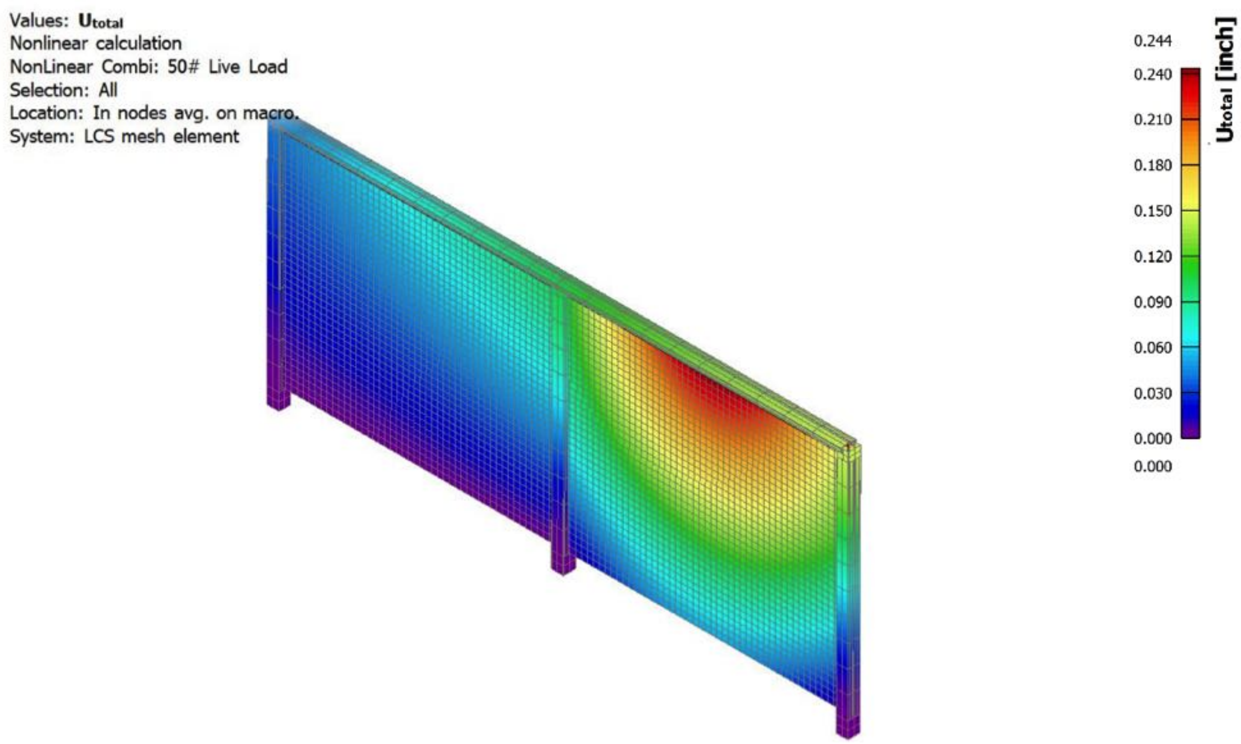
5. 3D displacement; U_total



6. 3D displacement; U_total



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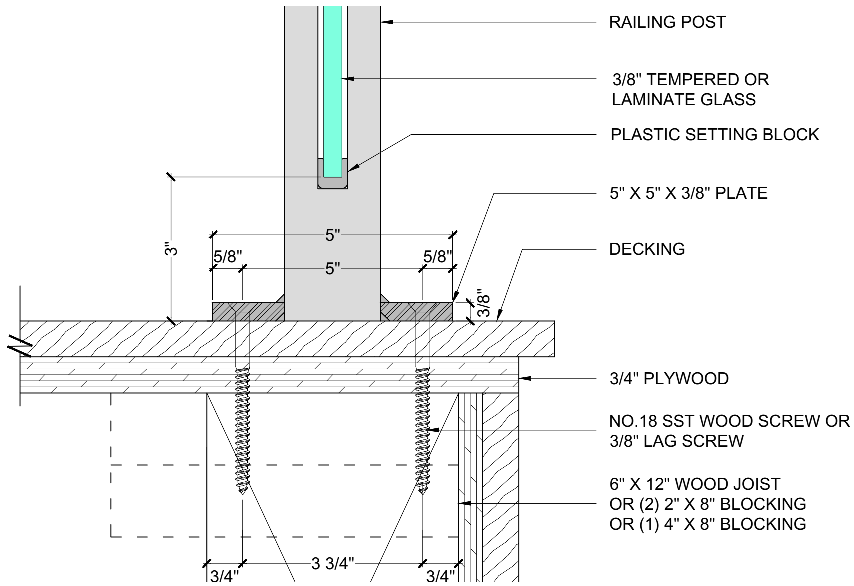
BASEPLATE MOUNT

The aluminum post is welded to an aluminum baseplate which is anchored to the structure with four fasteners.

The post was previously checked for the effects of the heat affected zone on the strength of the post.

The 3/8” thick baseplate is welded to the post using a 3/16” groove weld on the under side and a 3/16” fillet weld on the top side.



Assume F_{EXX} = 35ksi

Base metal shear strength, R_n/Ω = 24ksi/1.95*0.6*3/16” = 1.385kips/in
Fillet rupture, R_n/Ω = 35ksi*0.6/1.95*sin45°*3/16” = 1.428kips/in
Groove rupture = R_n/Ω = 35ksi*0.6/1.95*3/16” = 2.019kips/in

Both welds are controlled by the base metal shear strength.

Combined strength of welds = 2*1.385kips/in = 2.77kips/in
I_w = 2*2”*1”^{2}+2*2”^{3}/12 = 5.333in^4/in
M_a = 2.77kips/in* 5.333in^4/in/1” = 14,800”#

Check baseplate bending:
Z = 0.375”^{2}*5”/4 = 0.1758in^3
M_a = 0.1758in^3*24ksi/1.95 = 2,160”#
Max load on pair of fasteners = 2,160”#/0.875” = 2,470#

Baseplate is anchored to a wood substrate using stainless steel 3/8” lag screw fasteners.

Lag screws are designed according to NDS Chapter 12
Allowable steel strength per fastener, T_a = 1,415# (will not control design)
Pullout strength, W = 243pli (NDS Table 12.2A)
C_D = 1.6 for short term loading
W’ = 1.6*243pli = 389pli
Required penetration to maximize allowable loading = (2,470#/2)/389pli = 3.175” = 3-3/16”

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At 3-3/16” penetration in wood, the screw develops the allowable tension based on baseplate bending strength.
Assume allowable bearing stress = 625psi
C_b = (5”+0.375”)/5” = 1.075
F_c’ = 625psi*1.075 = 672psi
a = 2,470#/(5”*672psi) = 0.735”
M_a = 2,470#*(4.375”-0.735”/2) = 9,900”#

Minimum recommended penetration:
T = 8,400”#/(2*(4.375”-0.614”/2) = 1,032#
a = 2*1,032#/(5”*672psi) = 0.614”
e = 1,032/389 = 2.65”
Check allowable moment for 3” embedment:
T = 3”*389pli = 1167#
a = 2*1,167#/(5”*672psi) = 0.695”
M_a = (2*1167*(4.375”-0.695”/2) = 9,400”#
For maximum loading (3-3/16” embedment)
T = 2,470# (1,235#/lag)
a = 2,470#/(5”*672psi) = 0.735”
M_a = (2,470*(4.375”-0.735”/2) = 9,900”#
Penetration must be into wood that is securely fastened to the structure. Do not count wood members that the screws pass through but do not engage threads into, such as plywood sheathing directly beneath the baseplate.
Wind load that causes equivalent loading on anchorage:
For minimum embedment of 2.65”
w = 8,400”#/(4’*3’*24”) = 29psf
For 3” lag screw penetration:
w = 9,400”#/(4’*3’*24”) = 32.6psf
For 3-3/16” lag screw penetration:
w = 9,900”#/(4’*3’*24”) = 34.4psf

GRK RSS wood screw 5/16” x 6” alternative: Strength based on ESR-2442
Allowable tension strength = 1,415# (ESR-2442 Table 1)
Withdrawal strength = 196 pli (ESR-2442 Table 2 interpolated) for G = 0.50
Embedment from 6” screw: E = 5.75-.375-.75 = 4.625”
For weather exposed installation:
T = 4.625*196*(0.7*1.6) = 1,015#,
a = 2*1,015#/(5”*672psi) = 0.604”
M_a = (2*1,015*(4.375”-0.604”/2) = 8,268”#
w = 8,268”#/(4*3*24) = 28.7 psf

For protected installation: $M_c \leq 19\%$
T = 4.625*196*1.6 = 1,450#,
Develops full strength of base plate:
M_a = 9,900”# and w = 34.4 psf

BASE PLATE MOUNTED TO UNCRACKED CONCRETE - Expansion Bolt Alternative:
Base plate mounted to concrete with ITW Red Head Trubolt wedge anchor 3/8”x3.75” concrete anchors with 3” effective embedment. Anchor strength based on ESR-2427
Minimum conditions used for the calculations:
$f'_c \geq 3{,}000$ psi; edge distance =2.25” spacing = 3.75”
h = 3.0”: embed depth
For concrete breakout strength:
$N_{cb} = [A_{Ncg}/A_{Nco}]\varphi_{ed,N}\varphi_{c,N}\varphi_{cp,N}N_b$
$A_{Ncg} = (1.5*3*2+3.75)*(1.5*3+2.25) = 86.06\ in^2$ 2 anchors
$A_{Nco} = 9*3^2 = 81\ in^2$
$C_{a,cmin} = 1.5$” (ESR-2427 Table 3)
$C_{ac} = 5.25$” (ESR-2427 Table 3)
$\varphi_{ed,N} = 1.0$
$\varphi_{c,N} =$ (use 1.0 in calculations with k = 24)
$\varphi_{cp,N} =$ max (1.5/5.25 or 1.5*3”/5.25) = 0.857 ($c_{a,min} \leq c_{ac}$)
$N_b = 24*1.0*\sqrt{3000}*3.0^{1.5} = 6{,}830\#$
$N_{cb} = 86.06/81*1.0*1.0*0.857*6{,}830 = 6{,}219 \leq 2*4{,}200$
based on concrete breakout strength.
Determine allowable tension load on anchor pair
$T_s = 0.65*6{,}219\#/1.6 = 2{,}526\# > 2{,}470\#$ (Baseplate will control moment strength)
Check shear strength - Concrete breakout strength in shear:
$V_{cb} = A_{vc}/A_{vco}(\varphi_{ed,V}\varphi_{c,V}\varphi_{h,V}V_b$
$A_{vc} = (1.5*3*2+3.75)*(2.25*1.5) = 43.03$
$A_{vco} = 4.5(c_{a1})^2 = 4.5(3)^2 = 40.5$
$\varphi_{ed,V} = 1.0$ (affected by only one edge)
$\varphi_{c,V} = 1.4$ uncracked concrete
$\varphi_{h,V} = \sqrt{(1.5c_{a1}/h_a)} = \sqrt{(1.5*3/3)} = 1.225$
$V_b = [7(l_e/d_a)^{0.2}\sqrt{d_a}]\lambda\sqrt{f'_c}(c_{a1})^{1.5} = [7(1.625/0.375)^{0.2}\sqrt{0.375}]1.0\sqrt{3000}(3.0)^{1.5} = 1{,}636\#$
$V_{cb} = 43.03/40.5*1.0*1.4*1.225*1{,}636\# = 2{,}981\#$
Steel shear strength = 1,830#*2 = 3,660
Allowable shear strength
$ØV_N/1.6 = 0.70*2{,}981\#/1.6 = 1{,}304\#$
Shear load = 250/1,304 = 0.19 ≤ 0.2
Therefore interaction of shear and tension will not reduce allowable tension load:
$M_a = 2{,}470\#*(4.375\text{”}-2{,}470\#/(0.85*3000psi*5\text{”})) = 10{,}300\text{”}\#$

DEVELOPS FULL BASEPLATE MOUNTING STRENGTH.
ALLOWABLE SUBSTITUTIONS: Use same size anchor and embedment
Hilti Kwik Bolt TZ in accordance with ESR-1917
Powers Power Stud+ SD2 in accordance with ESR-2502
Powers Wedge-Bolt+ in accordance with ESR-2526
CONCRETE ANCHORS SHALL BE CHECKED FOR PROJECT CONDITIONS.

SIDE MOUNTING TO CONCRETE

Strength of the anchors-
Mounted with 1/2” threaded rod set with Hilti HIT-HY 200 adhesive.
The anchorage design was checked using Hilti Profis Anchor software.
The threaded rod is to be embedded:
Minimum of 3” into the concrete for minimum edge distance of 2 3/4”
Minimum of 3 7/16” into the concrete for minimum edge distance of 2”

3 Tension load

	Load N_{ua} [lb]	Capacity ϕN_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1,989	9,223	22	OK
Bond Strength**	1,989	1,991	100	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	1,989	2,228	90	OK

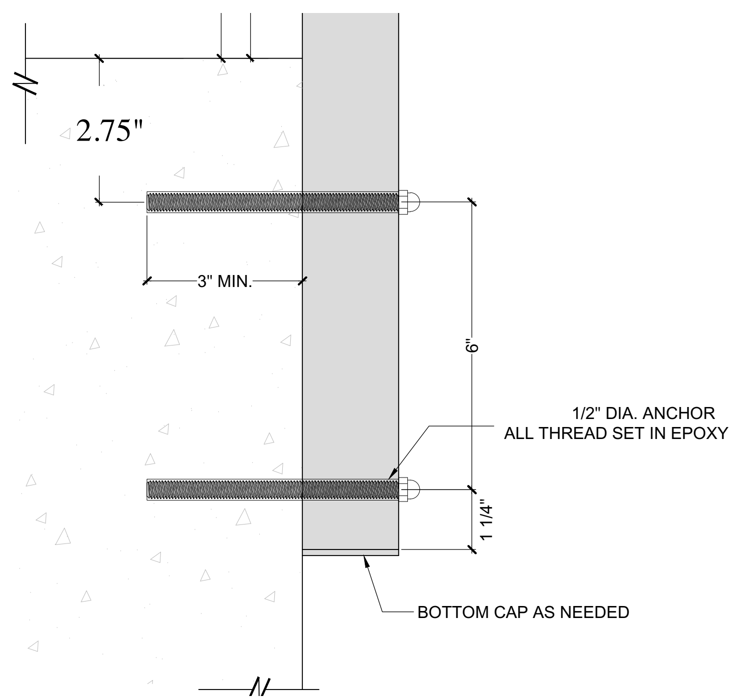
* anchor having the highest loading **anchor group (anchors in tension)

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	160	5,109	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)**	320	6,891	5	OK
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

FOR EDGE DISTANCE DOWN TO 2" THE EMBEDMENT MUST INCREASE TO 3 7/16"



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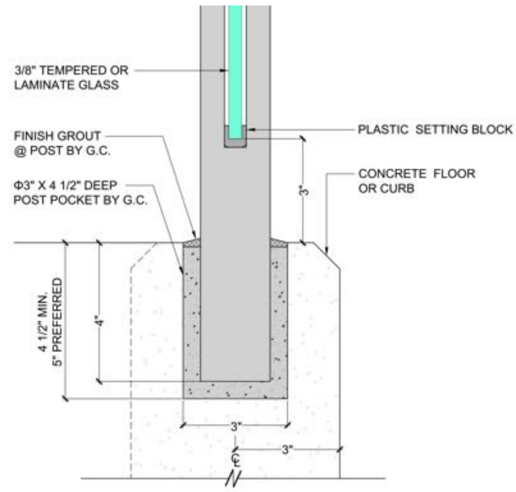
CORE MOUNTED POST

Core mounts resist moment through bearing between the face of the posts and the concrete/grout. The core mount can fail through concrete compressive failure or shear breakout. Shear breakout tends to control the strength when near the concrete edge.



The detail shown to the right should be used when concrete strength is 4,000psi or stronger. Weaker concretes will require greater embedment or edge distance.

Core mount option 1:
3" edge distance, 4" embedment, 4,000psi concrete

Inputs								
Post Width (in)	Concrete Strength		Edge Distance (in)	Embedment	Edge breakout calcs			
b (in)	f'_c (psi)	λ	C_1 (in)	d (in)	w (in)	h (in)	β	b_0 (in)
2	4000	1	3	4	5	3.5	1.428571429	12

						Bearing Calcs		Strength
						$0.65*0.85f'_c/1.6$	$f_a bd/2$	$Min(P_a)*d/2$
α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2+4/\beta)\lambda\sqrt{f'_c}$ (psi)	$(2+\alpha_s C_1/b_0)\lambda\sqrt{f'_c}$ (psi)	$V_c C_1 b_0$	P_a (lbs)	f_a (psi)	P_a (lbs)	M_a (in-lbs)
30	252.982213	303.5786554	600.8327554	9107.35966	4269.07484	1381.25	5525	8538.149682

M_a = 8,540"#

Core mount option 2:
3" edge distance, 4-1/2" embedment, 3,000psi concrete

Inputs								
Post Width (in)	Concrete Strength		Edge Distance (in)	Embedment	Edge breakout calcs			
b (in)	f'_c (psi)	λ	C_1 (in)	d (in)	w (in)	h (in)	β	b_0 (in)
2	3000	1	3	4.5	5	3.75	1.333333333	12.5

						Bearing Calcs		Strength
						$0.65*0.85f'_c/1.6$	$f_a bd/2$	$Min(P_a)*d/2$
α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2+4/\beta)\lambda\sqrt{f'_c}$ (psi)	$(2+\alpha_s C_1/b_0)\lambda\sqrt{f'_c}$ (psi)	$V_c C_1 b_0$	P_a (lbs)	f_a (psi)	P_a (lbs)	M_a (in-lbs)
30	219.089023	273.8612788	503.9047529	8215.83836	3851.17423	1035.9375	4661.71875	8665.142023

M_a = 8,670"#

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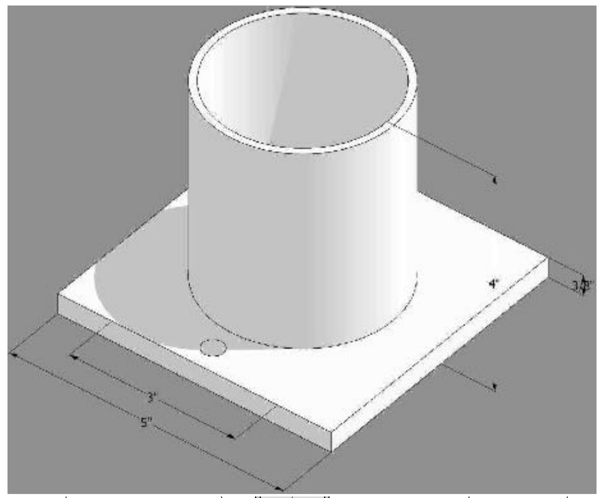
Core mount option 3:
4” edge distance, 4-1/2” embedment, 3,000psi concrete

Inputs

Post Width (in)	Concrete Strength		Edge Distance (in)	Embedment	Edge breakout calcs			
b (in)	f'_c (psi)	λ	C_1 (in)	d (in)	w (in)	h (in)	β	b_o (in)
2	3000	1	4	4.5	6	4.25	1.411764706	14.5

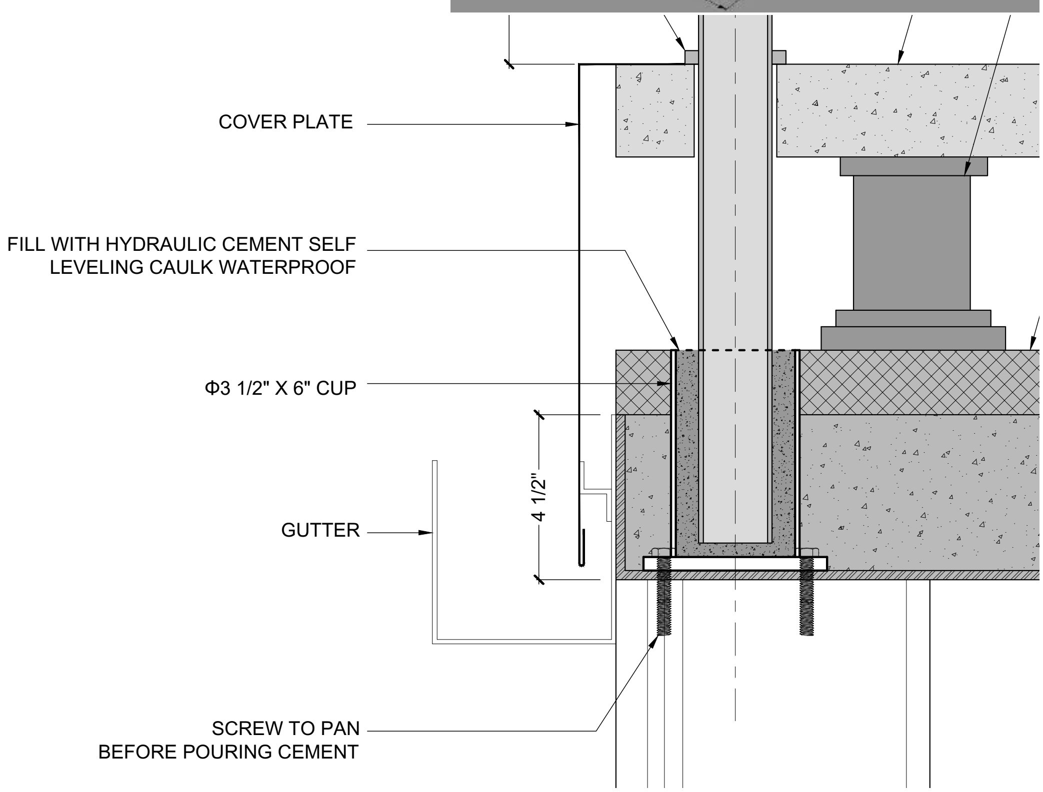
						Bearing Calcs		Strength
						$0.65*0.85f'_c/1.6$	$f_a bd/2$	$Min(P_a)*d/2$
α_s	$4\lambda\sqrt{f'_c}$ (psi)	$(2+4/\beta)\lambda\sqrt{f'_c}$ (psi)	$(2+\alpha_s C_1/b_o)\lambda\sqrt{f'_c}$ (psi)	$V_c C_1 b_o$	P_a (lbs)	f_a (psi)	P_a (lbs)	M_a (in-lbs)
30	219.089023	264.7325695	562.8321453	12707.1633	5956.48281	1035.9375	4661.71875	10488.86719

M_a = 10,500”#



CUP MOUNTING - RECEIVER MOUNTED POST
The post is grouted into a metal receiver set into a concrete or mounted to a steel or wood deck.

CUP MOUNTING IN CONCRETE SLAB
When set in concrete slab strength is same as for the core mounted post.



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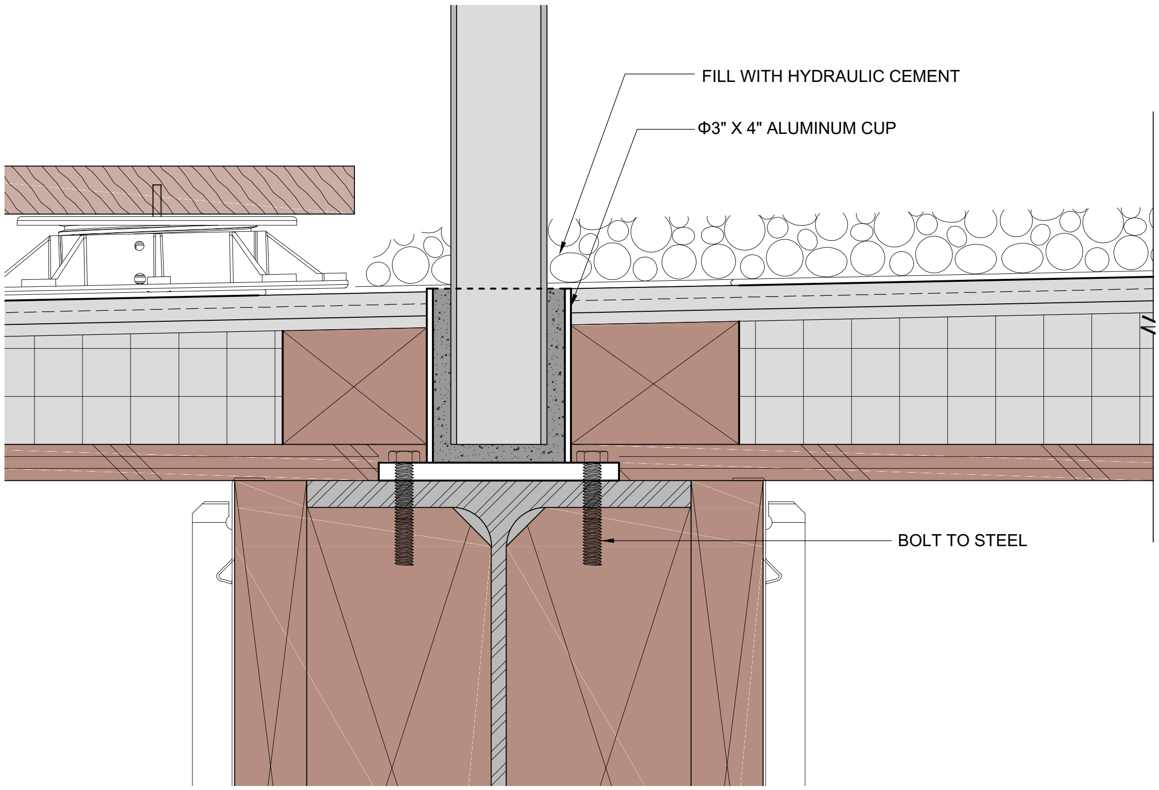
Strength of Cup:
3” i.d. x 1/8” thick welded to baseplate (6061-T6 aluminum):
Strength controlled by the HAZ at the weld to the base plate-
$Z = (3.25^3-3^3)/6 = 1.221$ in^3
F_{twa} = 24ksi*1.221/1.95 = 15,032”# allowable moment on the cup when not encased.
Strength is adequate for post strength

CUP MOUNTING TO STEEL BEAM OR DECK



Use (4) 3/8” SS bolts to anchor to the steel beam or steel deck of adequate strength and thickness.

Bolt strength: 75 ksi tension strength
T_n = 0.0775in^2*75ksi = 5,812#
M_n = 5,812#*3.75”*2 = 43,590”#

Thus will support the full post strength.

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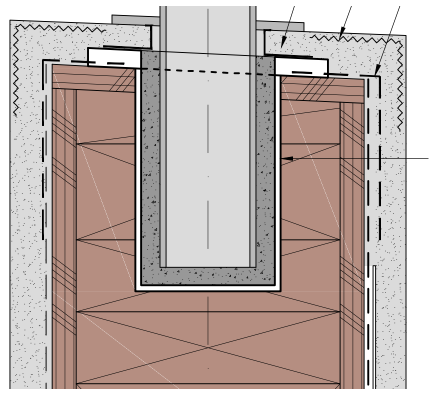
CUP MOUNTING TO WOOD:

Baseplate mounted receiver (same as steel but attached to wood)
T_a = 1.6*263pli*4.25” = 1,788”#
a = 2*1788/(605psi*5”) = 1.182”
M_a = 1,788*2*(4.25-1.182/2) = 12,190”#



Inset in wood:
The receiver is inset into wood by inlaying int a bored hole or by building up blocking around.

For built up blocking required shear connection between plies assuming 1-1/2” ply thickness:
V = (200#*47”)/4.5” = 2,089#

Connection - 16d sinker nails
Z’ = 1.6*99 = 158# each (NDS Table 12N G = 0.43)
n = 2089/158 = 14 nails to connect plies, each ply

#12 x 6” screws:
Z’ = 1.6*128 = 205# each (NDS Table 12L G = 0.43)
n = 2089/205 = 11 Screws to connect plies

#14 x 6” screws
Z’ = 1.6*141 = 226# each (NDS Table 12L G = 0.43)
n = 2089/226 = 10 Screws to connect plies

5/16” x 6” GRK RSS (ESR-2442) or Fastenmaster Timberlok (ESR-1078) screw
Z’ = 1.6*210 = 336# each (G = 0.42)
n = 2089/336 = 7 Screws to connect plies

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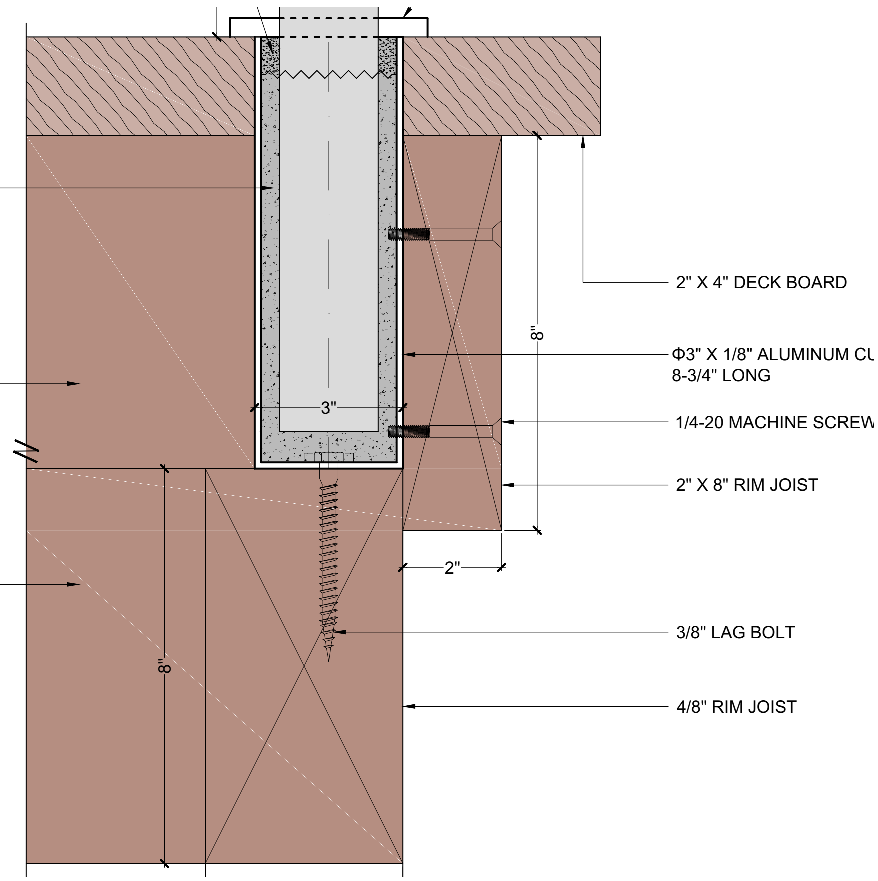
Cup Mounting 1:



To wood with receiver clamped by a block on side.
Required pullout strength of blocking:
R =(200#*50”)/7.25” = 1,380#

#12 screws:
W’ = 1.6*114pli*1.5” = 274# each (NDS Table 12.2B, G = 0.43)
n = 1380/274 = 6 screws

5/16” x 6” GRK RSS (ESR-2442) or Fastenmaster Headlok (ESR-1078) screw
W’ = 410# each
n = 1380/410 = 4 screws

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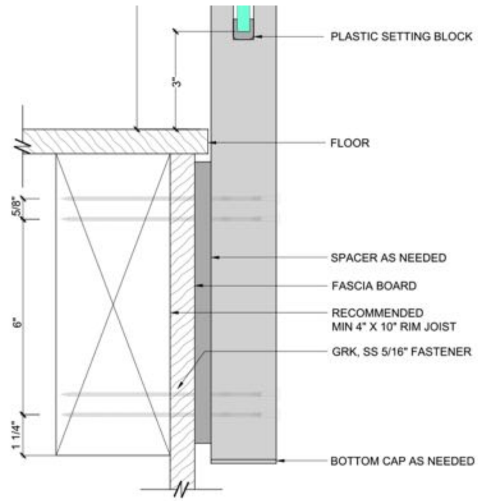
FASCIA MOUNTED POSTS



Uses (4) 5/16"X6" GRK RSS fasteners.
GRK fasteners are designed according to ESR 2442.
Allowable steel strength per fastener, T_a = 1,415# (will not control design)
Pullout strength, W = 165pli
C_D = 1.6 for short term loading
W' = 1.6*165pli = 264pli
p = 6"-2"-1/2"-3/4" = 2.75"
W'p = 264pli*2.75" = 726#

Assume allowable bearing stress = 625psi
C_b = (2"+0.375")/2" = 1.19
F_c' = 625psi* 1.19 = 744psi

a = 726#*2/(2"*744psi) = 0.976"
Allowable load at top of post = (2*726#)*(7.56"-0.976"/2)/(42"+2.375"+7.56"-0.976"/2) = 200#

LAG SCREW ALTERNATIVE
Lag screw tension same as calculated above:
T = 1,162#
Try 3/8" SS lag screw
Tension strength of lag screw = 0.055*70ksi/2 = 1,930#

Determine required anchor embedment:
Tension load on anchors = 200*(42"+2.375"+7.56"-0.976"/2)/(7.56"-0.976"/2) = 1,455#
From 2015 NDS Table 12.2A for G ≥ 0.43
W = 243 #/in
For weather protected (MC ≤ 19% at all times)
C_D = 1.6 and C_M = 1.0
W' = 243*1.6*1.0 = 389 #/in
Required embed depth = 1,455/389 = 3.74" (3-3/4") minimum beam thickness
Minimum lag length = 2" + 3.75" + 7/32" = 5.97" (6" lag)

For weather exposed (MC > 19% at any time in service)
C_D = 1.6 and C_M = 0.7
W' = 243*1.6*0.7 = 272 #/in
Required embed depth = 1,455/272 = 5.35" requires 5-1/2" minimum beam thickness
Minimum lag length = 2" + 5.35" + 7/32" = 7.57" (8" lag)

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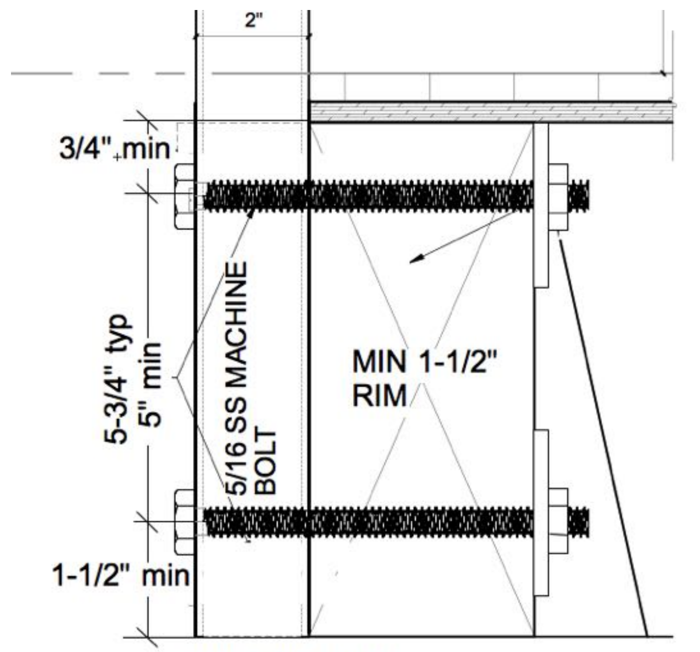
For **1/2” lag screw** the withdrawal strength is:
W = 302plf
length adjustment required for 1/2” lag screws:
a_l = 243/302 = 0.8
L_{dry} = 3.74*0.8 = 2.99” minimum length = 2” + 2.99” + 5/16” = 5.3” requires 6” lag screw
L_{wet} = 5.35*0.8 = 4.28” minimum length = 2” + 4.28” + 5/16” = 6.59” requires 7” lag screw

BOLTD CONNECTION:
Use 5/16” bolts through rim joist with bearing washer on back.
Strength will be same as above thus okay alternative.
Typical anchor spacing is 5 3/4”.

Minimum edge distance is 1/2” for top bolt
and 1-1/4” for bottom bolt. Use 3/4” for top and 1-1/2” for bottom.

Tension strength of 5/16” bolt
T_n = 0.0524 in^2*70 ksi = 3,668”#
Ta = 3,668/2 = 1,834#



NOTES ON FRAMING:
The supporting beam must have adequate connection to the structure to resist all loading from the guards. This must be provided by proper detailing nd may require the addition of blocking, fasteners, ties or other hardware.

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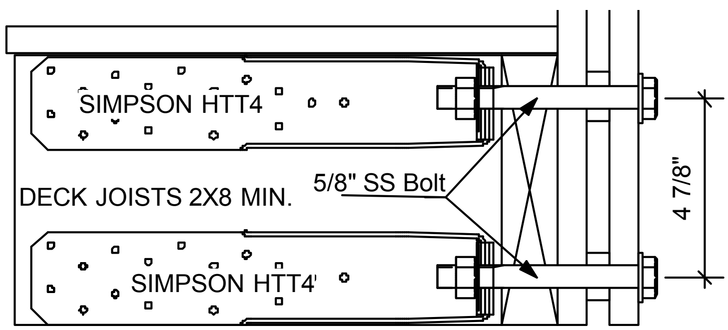
Hold Down Alternatives:
Bolts and hold downs can be used when the framing at the fascia is not adequate to attach directly. Bolts pass through the post and attach to hold downs which are anchored to joists that run perpendicular with the rail.

Bolts must be stainless steel because they are in contact with aluminum and weather.

Hold down option 1: Simpson HTT4
For use when hold downs are able to attach directly to joists. Hold down attaches with screws or nails according to Simpon's instructions.
From Simpson load tables, T_a = 3,000#
5/8" SS bolt strength, $T_a = 0.226in^2*75ksi/2$ = 8,480# > 3,000# (Does not control)
Assuming 4-7/8" hold down spacing:
a = 3000#/(2"*744psi) = 2.02"
Allowable load at top of post = 3,000#*(6.06"-2.02"/2)/(42"+2"+6.06"-2.02"/2) = 309#



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Hold down option 2: Simpson HD7B

For use when hold downs are not able to be attached directly to joists. Hold downs attach to threaded rods that attach to joists. Threaded rods have washers and nuts that hold them in place and keep the threaded rods from sliding in their holes.

From Simpson load tables, T_a = 6,645#

Assume 3/4" threaded rods span 16" max. Also assume that rods run multiple spans so that the center span behaves like a fixed span.

D_r = 0.680"

Z = 0.05241in^3

M_a = 0.05241in^3*75ksi/2 = 1,970"#

Allowable load, 1,970"#*3/(16"/8) = 2,960#

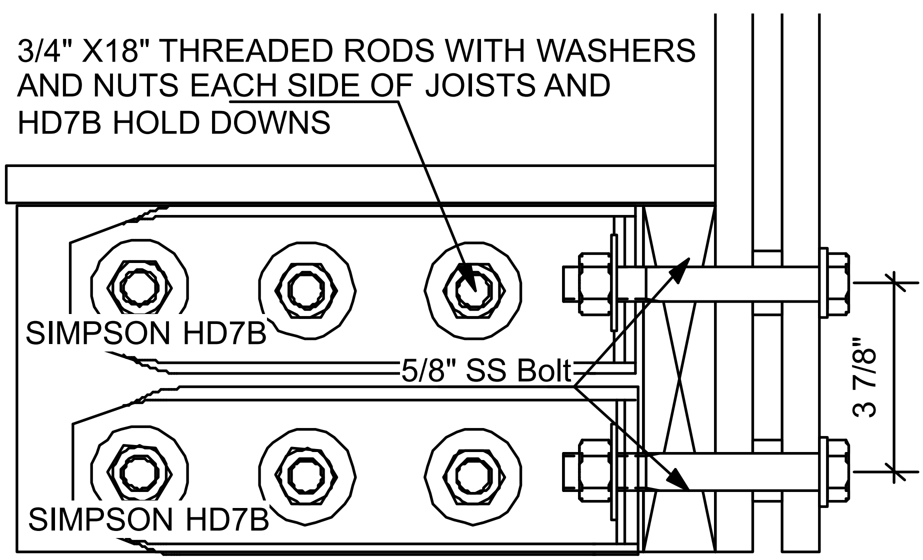
Assuming 3-7/8" hold down spacing:

a = 2,960#/(2"*744psi) = 1.99"

Allowable load at top of post = 2,960#(6.06"-1.99"/2)/(42"+2"+6.06"-1.99"/2) = 306#

Wind load that causes equivalent loading on anchorage = 8,400"#/(4'*3'*31.1") = 22.5psf

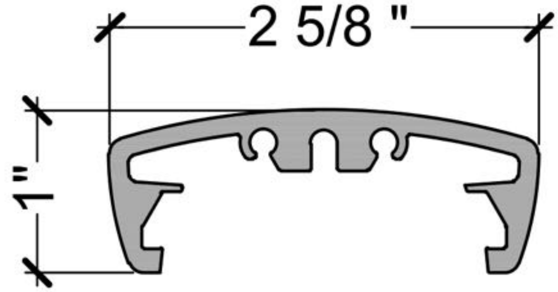
The GRK option limits wind load to 22.5psf. Other anchorages develop strength above 25psf.



Hold downs may be located at any location between joists on threaded rods.

For installation on joists deeper than 2x8 (7-1/4") set tension tie at top of joist and at 3-7/8" center to center for lower tension tie.

A11 TOP RAIL



Alloy 6063-T6 Aluminum

Allowable Stress
ADM Table 2-21
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b^2)^{1/2})^{1/2}$
$r_{ye} = ((0.0519)^{1/2}/0.0788*(0.0495+0.038*0.00568*48^2)^{1/2})^{1/2} = 1.462”$
$\lambda = 48”/(1.462”*1) = 32.83$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c^3$
$M_p = 30\text{ksi}*0.5583\text{in}^3 = 16.75\text{kip-in}$
$M_{nmb} = 16.75\text{kip-in}(1-32.83/78)+\pi^2 10{,}100*32.83*0.4264/78^3 = 12.64\text{kip-in}$
$M_a = 12.64\text{kip-in}/1.65 = 7{,}660”\#$

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1536\text{in}^3*15.2\text{ksi} = 2{,}330”\#$

Find maximum on center post spacing:
$L_{max}=2{,}330”\#*4/200\#+2” = 48.6”$ For single span condition
$L_{max}=2{,}330”\#*(64/13)/200\# = 57.4”$ For multiple span condition

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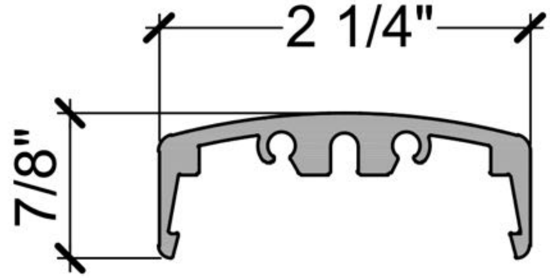
A111 TOP RAIL



Alloy 6063-T6 Aluminum

Allowable Stress
ADM Table 2-20
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b^2)^{1/2})^{1/2}$
$r_{ye} = ((0.02047)^{1/2}/0.2458*(0.01183+0.038*0.00400*48^2)^{1/2})^{1/2} = 0.5918”$
$\lambda = 48”/(0.5918”*1) = 81.11$
$M_{nmb}=\pi^2ES_x/\lambda^2$
$M_{nmb}=\pi^2*10,1000*0.2458/81.11^2 = 3,720”\#$
$M_a = 3,720”\#/1.65 = 2,260”\#$

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.07620in^3*15.2ksi = 1,160”\#$
Include contribution from sub rail:
Assume 520E sub rail, $I = 0.00589in^4$
Top rail load share = $0.02047in^4/(0.02047in^4+0.00589in^4) = 0.7766$
$M_a = 1,160”\#/0.7766 = 1,490”\#$

Find maximum top rail span:
$L_{max}=1,490”\#*4/200\# = 31.8”$ For single span condition
$L_{max}=1,490”\#*(64/13)/200\# = 36.7”$ For two span condition

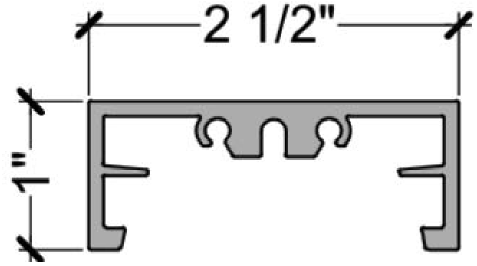
Use of top rail is extremely limited for 6063-T6, try 6061-T6:

$M_a = 0.07620in^3*19.5ksi/0.7766 = 1,910”\#$

Find maximum on center post spacing:
$L_{max}=1,910”\#*4/200\#+2” = 40.2”$ For single span condition
$L_{max}=1,910”\#*(64/13)/200\# = 47.0”$ For two or more spans condition

This top rail has limits below 48” O.C. post spacing when used for fall protection.

A222 TOP RAIL



Alloy 6063-T6 Aluminum

Allowable Stress
ADM Table 2-21
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b{}^2)^{1/2})^{1/2}$
$r_{ye} = ((0.05785)^{1/2}/0.3847(0.06424+0.038*0.00399*48^2)^{1/2})^{1/2} = 0.6341\text{in}$
$\lambda = 48\text{”}/(0.6341\text{”}*1) = 75.70$
$M_{nmb}=M_p(1-\lambda/C_c)+\pi^2E\lambda S_{xc}/C_c{}^3$
$M_p = 25\text{ksi}*0.4866\text{in}^3 = 12.17\text{kip-in}$
$M_{nmb} = 12.17\text{kip-in}(1-75.70/78)+\pi^2 10{,}100*75.70*0.3847/78^3 = 6.465\text{kip-in}$
$M_a = 6.465\text{kip-in}/1.65 = 3{,}920\text{”\#}$

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1512\text{in}^3*15.2\text{ksi} = 2{,}300\text{”\#}$

Find maximum on center post spacing:
L_{max}=2,300”#*4/200#+2” = 48.0” For single span condition
L_{max}=2,300”#*(64/13)/200# = 56.6” For two or more spans condition

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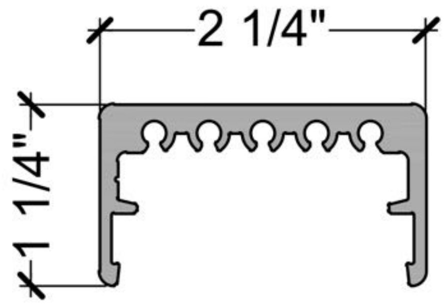
A225 TOP RAIL



Alloy 6063-T6 Aluminum

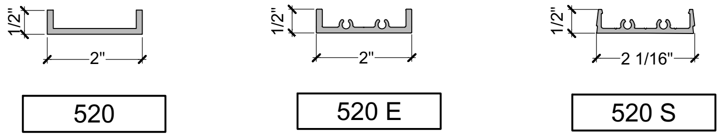
Allowable Stress
ADM Table 2-21
$F_c/\Omega = 15.2$ ksi

Check lateral torsional buckling about strong axis:
Lateral torsional buckling strength is calculated according to 2015 ADM F.4.2.1
$r_{ye} = ((I_y)^{1/2}/S_x(C_w+0.038JL_b{}^2)^{1/2})^{1/2}$
$r_{ye} = ((0.07776)^{1/2}/0.4498(0.06184+0.038*0.00660*48^2)^{1/2})^{1/2} = 0.4649$in
$\lambda = 48”/(0.4649”*1) = 103.2$
$M_{nmb}=\pi^2 ES_x/\lambda^2$
$M_{nmb}=\pi^2*10{,}1000*0.4498/103.2^2 = 4{,}210”\#$
$M_a = 4{,}210”\#/1.65 = 2{,}550”\#$

Bending about the weak axis:
Lateral torsional buckling and local buckling does not apply.
$M_a = 0.1715in^3*15.2ksi = 2{,}610”\#$

Find maximum on center post spacing:
$L_{max}=2{,}550”\#*4/200\#+2” = 53.0”$ For single span condition
$L_{max}=2{,}550”\#*(64/13)/200\# = 62.8”$ For two span condition

520 SUB RAILS
Sub rail options are:
520,
520S and
520E



The strength of the 3 are similar. Separate calculation of the sub rail strength is not provided herein as the sub rails are only used in an assembly with the top rail.

The top rail strengths take into account the coupled sub rail.

The 520S is a snap in sub rail which snaps into place on the bottom of the top rail and is retained in place by the interlocking tabs.

Force required to pull out the 520 S from the top rail is controlled by flexure in the locking tab.
Force to pullout the tab:
$R_{po} = [.1^3*3*10x10^6/(0.5^3)]*\sin 60 = 207$ k/ft
$R_t = 0.1*12"*15ksi = 18$ k/ft

Thus it requires substantially more force to pullout the sub rail than any realistic load condition while in service.

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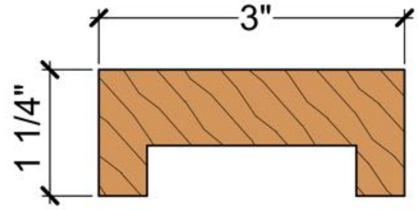
WDCR Wood cap rail

Minimum nominal bending strength for wood is 1,400psi when posts are spaced 48" O.C.
Strength factors:
C_D = 2 for impact live loading
C_f = 1.5 for 2x4 size
Wood must not receive damage from moisture- must be sealed.
F_b' = 2*1.5*1,400psi = 4,200psi
Flat wise bending is also allowed a 1.15 flat use factor,
F_b'=1.15*4,200psi = 4,830psi (vertical loads)



Check stability factor, $l_e = 1.37l_u+3d$ = 1.37*48"+3*3" = 74.8"

Beam Strength Calculation

Emin	Cm	Cf	Cd	t	d	le
470000	1	1.5	2	1.25	3	74.8

Rb	Braced?
11.9839893	No

Fb	Fb*	Fb'
1400	4200	3310.78848

CL	FcE
0.78828297	3927.13904

With the C_L factor, F_b' = 3,310psi.
C_L does not apply to flat wise bending.

Strong axis bending:
S = 1.653in^3
M_a = 1.653in^3*3,310psi = 5,470"#

Weak axis bending:
S = 0.3621in^3
M_a = 0.3621in^3*4,830psi = 1,750"#

Wood rails are always used with an aluminum sub rail.
Typical sub rail is the 520E
I_x = 0.00589in^4
Load share to wood = 0.2757/(10.1/1.4*0.00589+0.2757) = 0.866
M_a = 1,750"#/0.866 = 2,020"#

Find maximum on center post spacing:
L_{max}=2,020"#*4/200#+2" = 40.4" For single span condition
L_{max}=2,020"#*(64/13)/200# = 49.7" For two span condition

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.

μ = 0.22 Typical value of Poisson's ratio for common glasses.

$\nu = 5x10^{-6}$ 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.

Effective properties for laminated glass are calculated according to the appendix of ASTM E1300.

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

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	$\sqrt{(hef;w)^3/(h2+2\Gamma hs;1)}$

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The glass is analyzed using a 3rd order non-linear analysis that incorporates the aluminum posts in order to accurately model the support conditions.

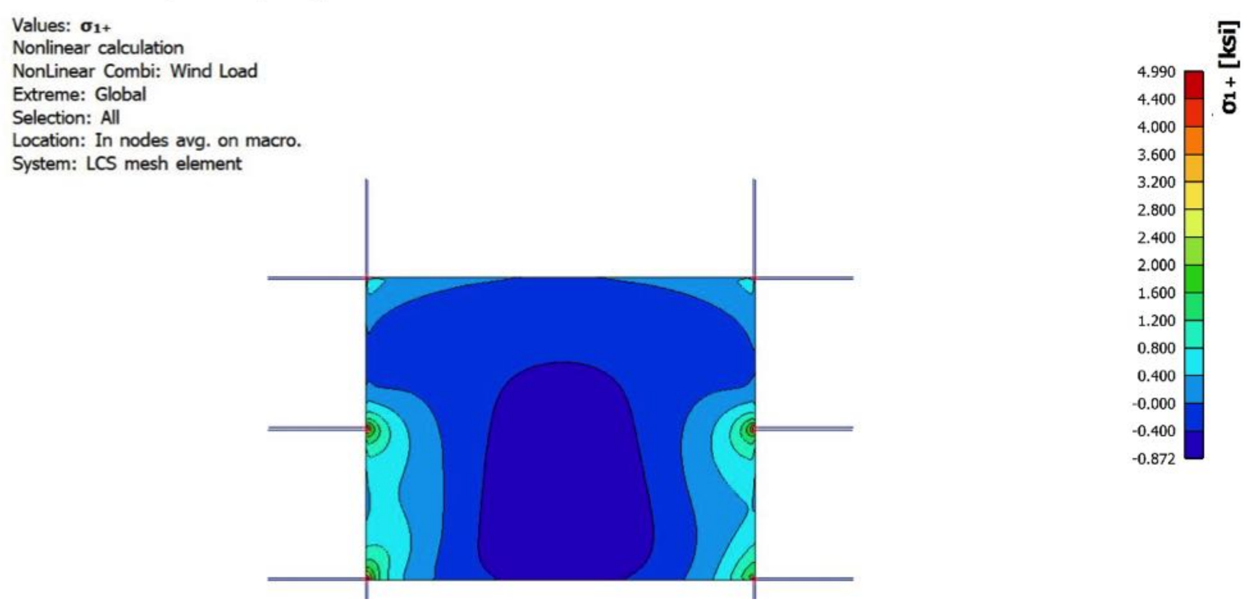
Model summary:
Posts at 48” O.C., 42” top rail height.
50# infill live load along the top edge of the glass results in a maximum glass stress of 1,030psi.
25psf wind load causes a maximum glass stress of 5,140psi. Allowable glass stress from wind loading is 10,600psi. Therefore, allowable wind loading = 10,600psi/5,140psi*25psf = 51.6psf.
The maximum stress is located at the cantilevered end panels.

Allowable wind load for laminated glass = $0.337^2/0.355^2 * 51.6$psf = 46.5psf

Glass is able to hold live loads per IBC 1607.8.

SCIA result diagrams are shown below.

7. 2D stress/strain; σ_1+



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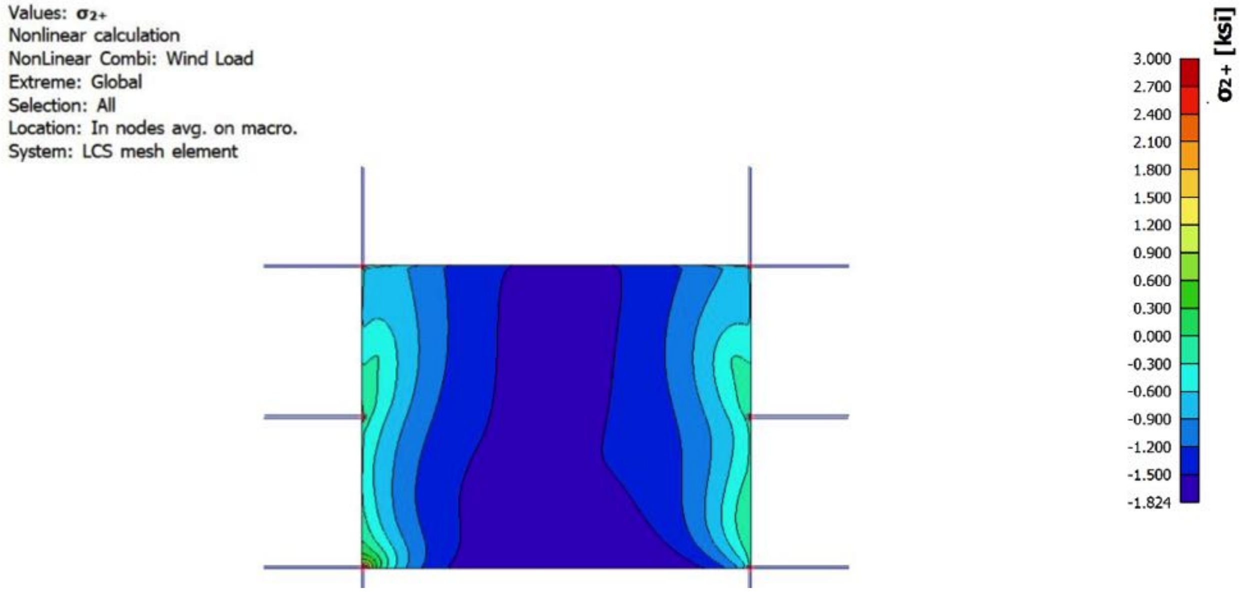
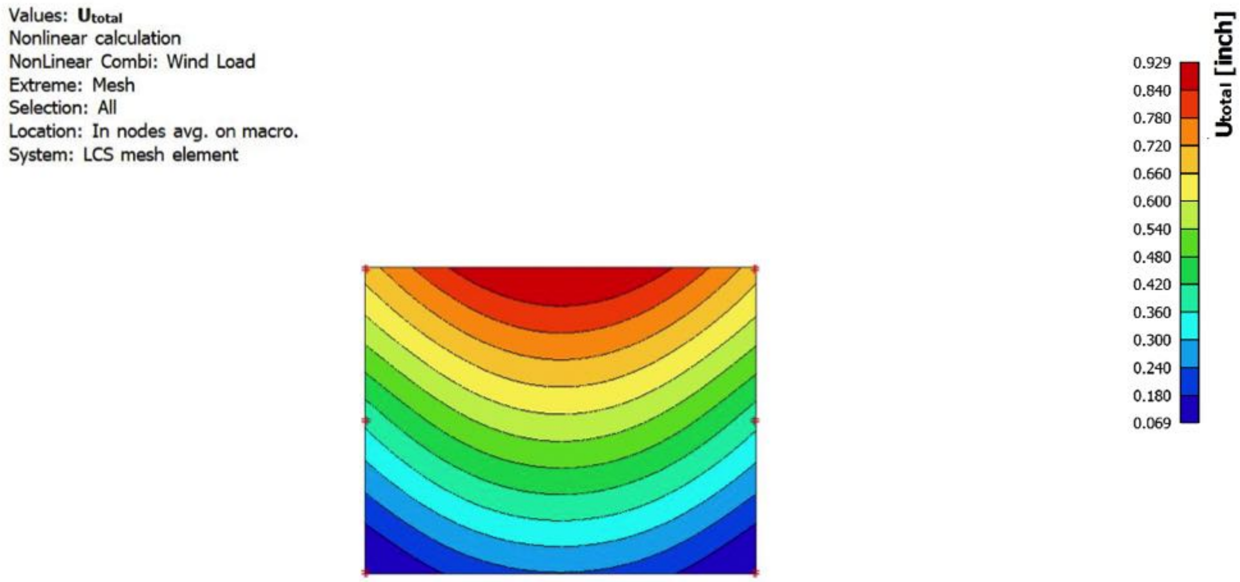
8. 2D stress/strain; σ_2+



9. 2D displacement; U_total



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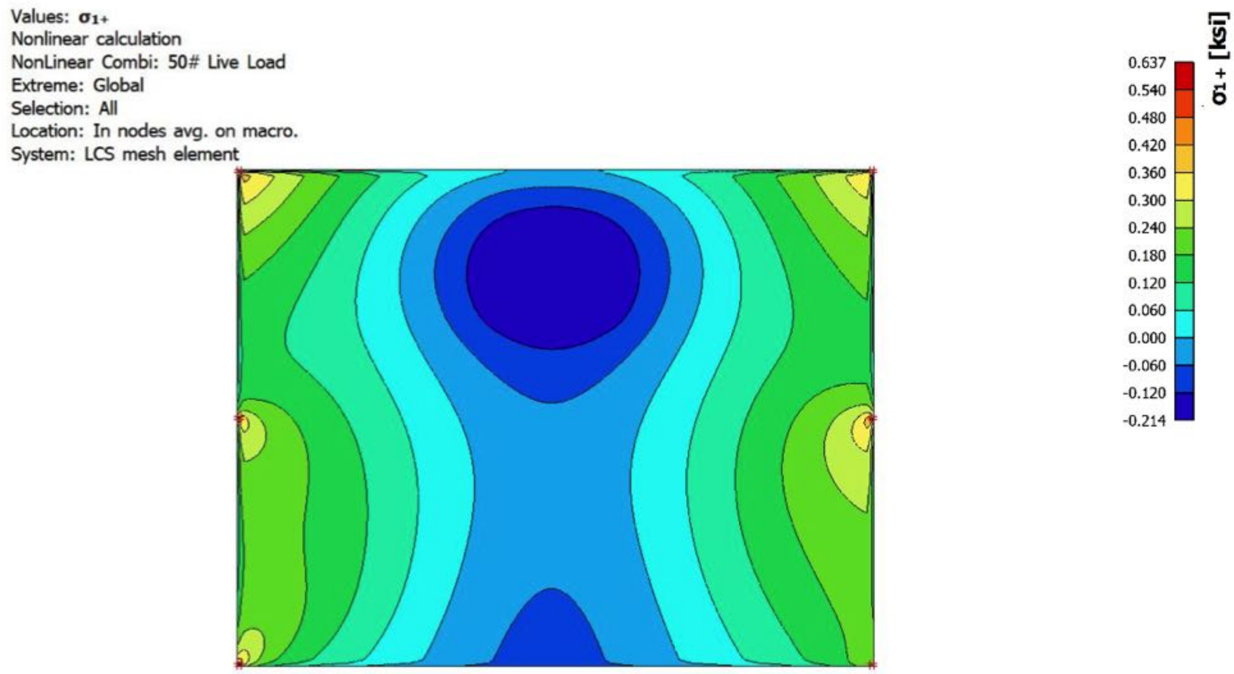
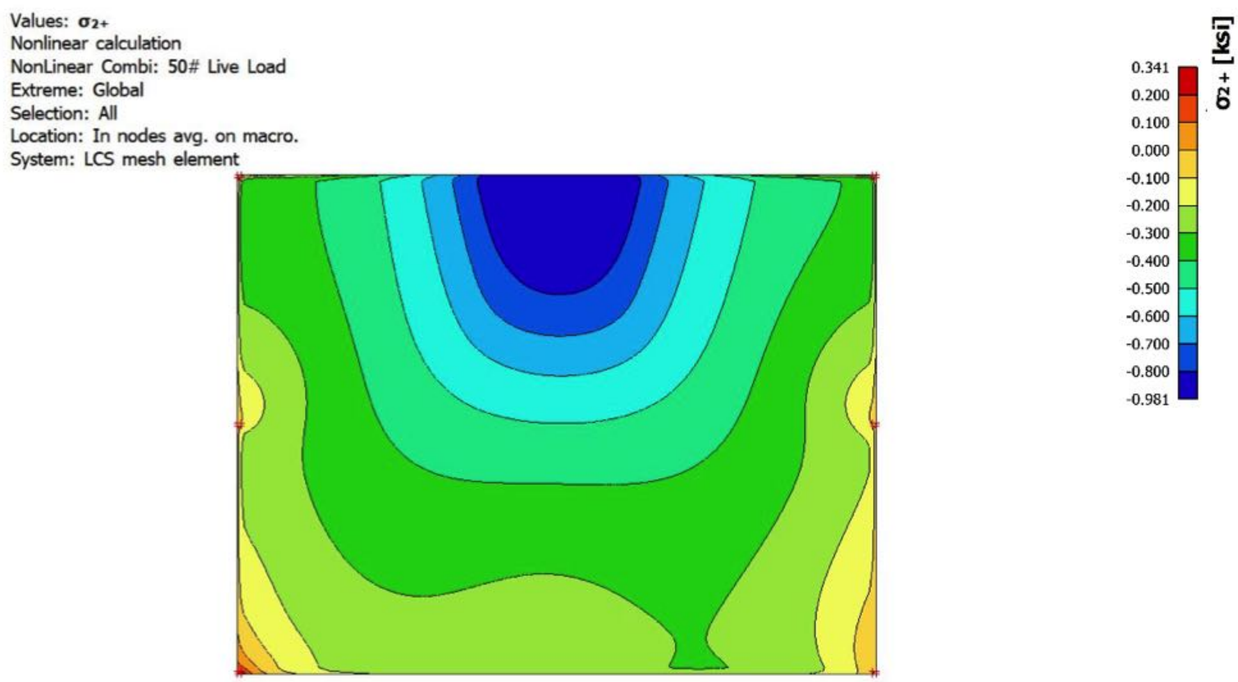
10. 2D stress/strain; σ_1+



11. 2D stress/strain; σ_2+



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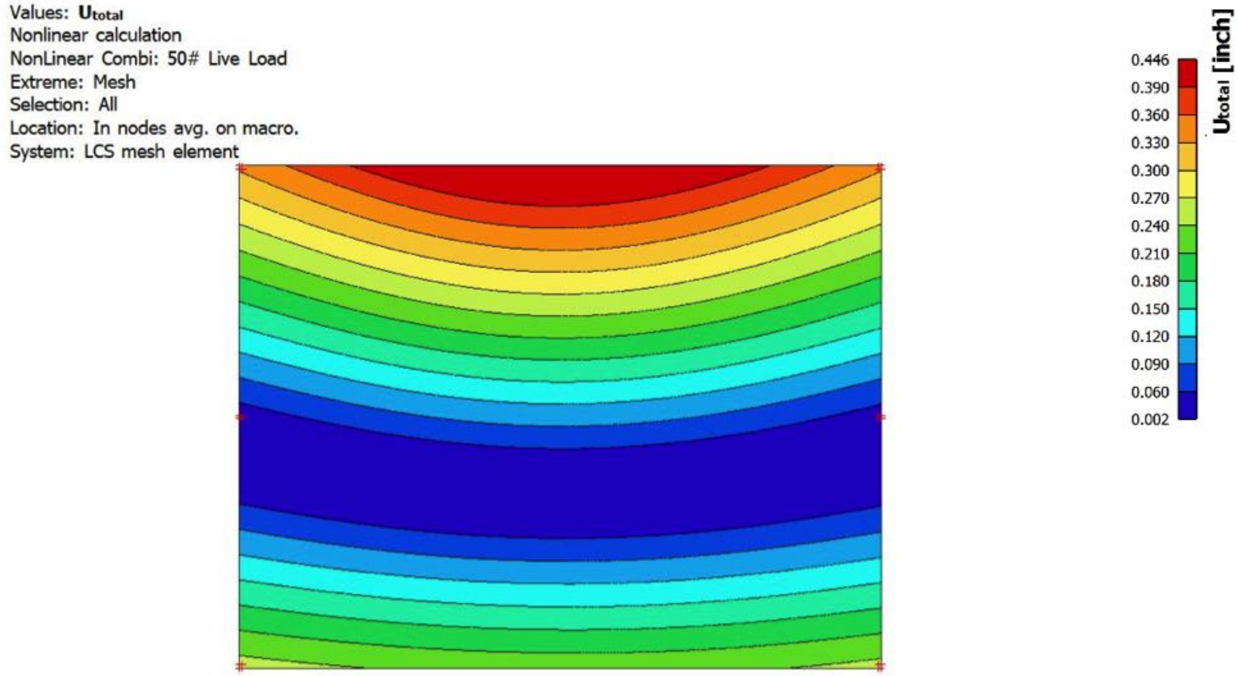
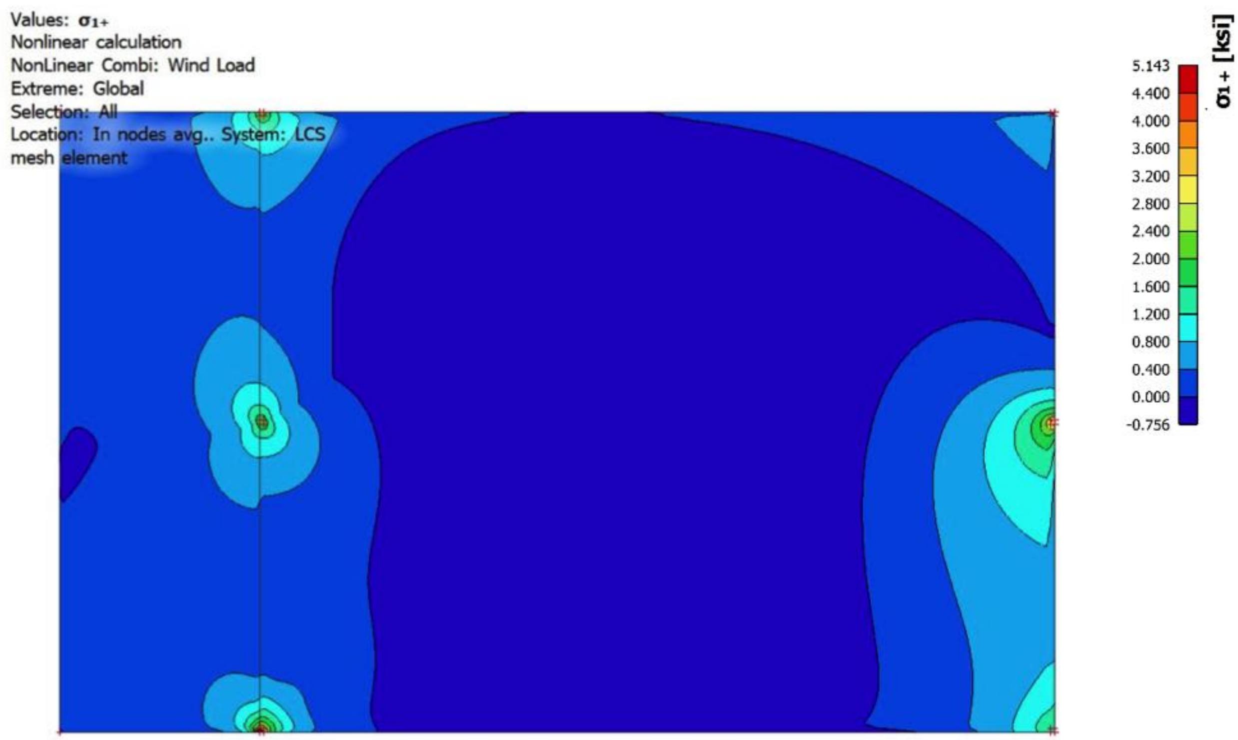
12. 2D displacement; U_total



13. 2D stress/strain; σ_1+



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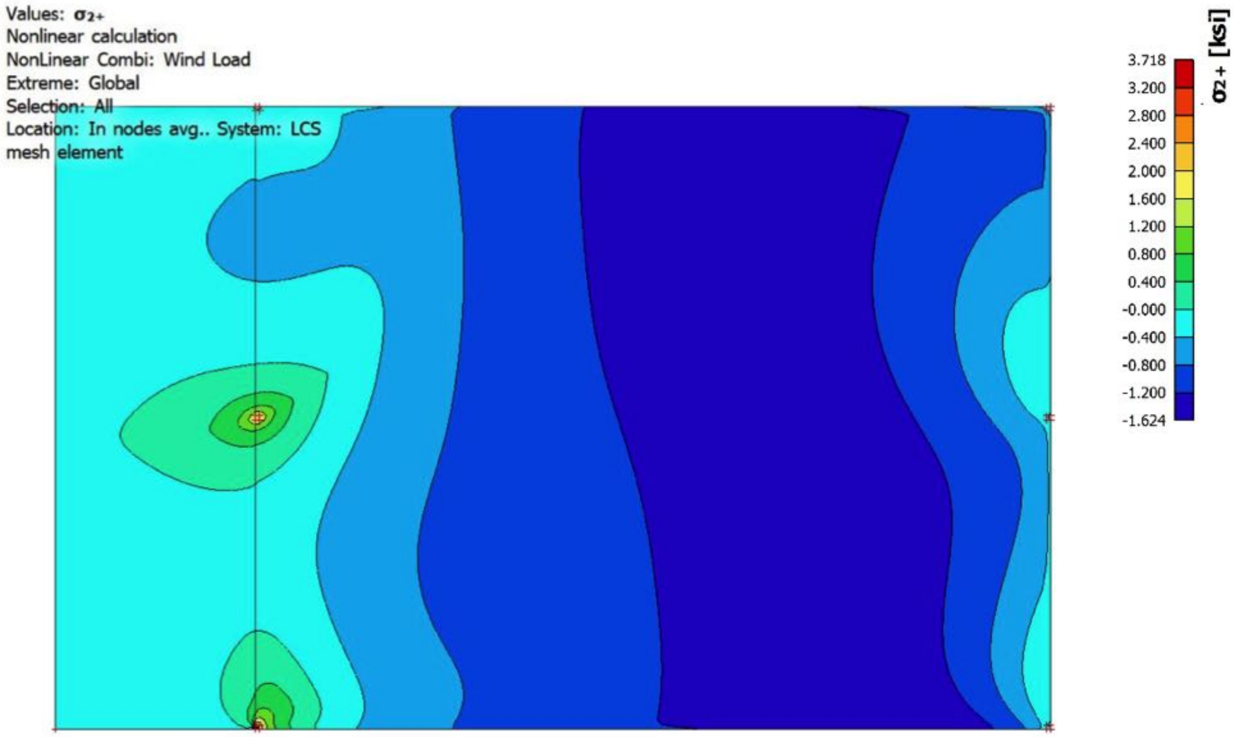
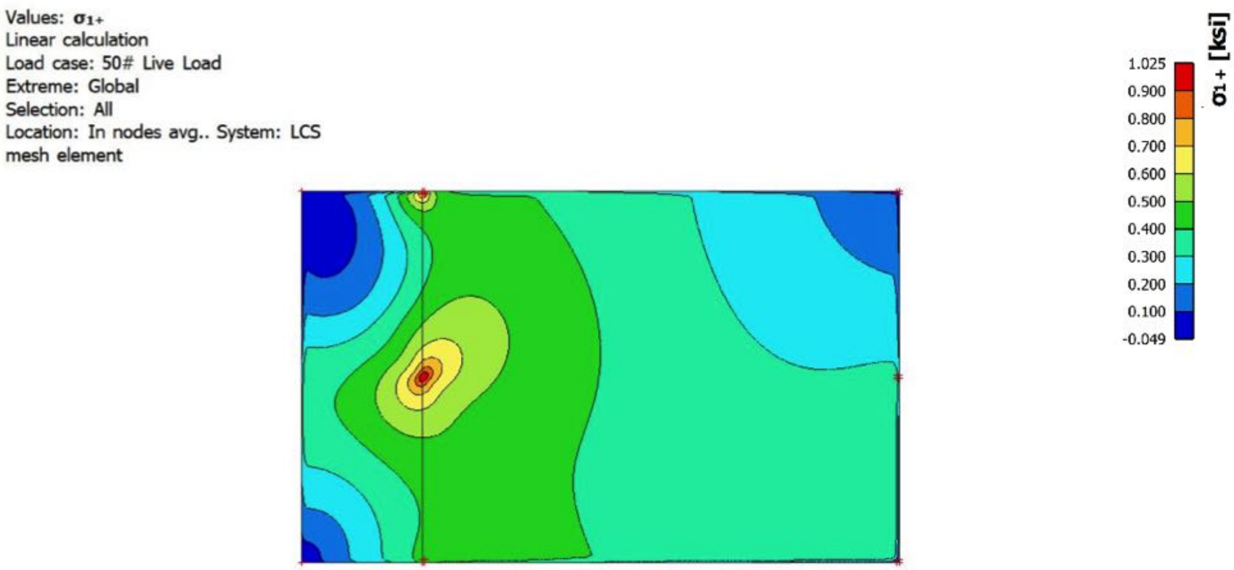
14. 2D stress/strain; σ_2+

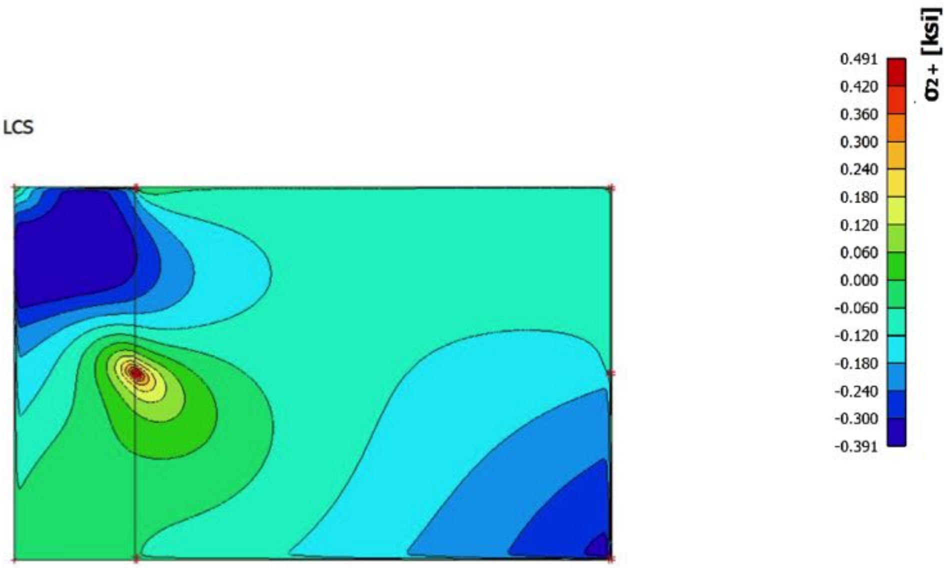


15. 2D stress/strain; σ_1+



16. 2D stress/strain; σ_2+

Values: σ_{2+}
Linear calculation
Load case: 50# Live Load
Extreme: Global
Selection: All
Location: In nodes avg.. System: LCS
mesh element



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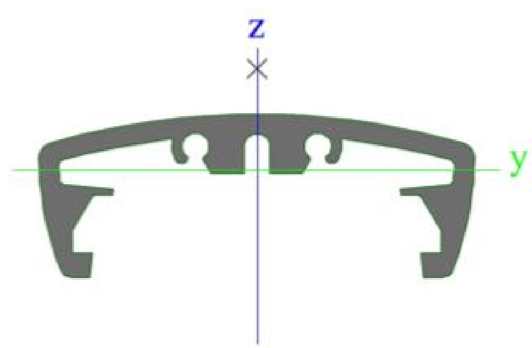
EXTRUSION SECTIONAL PROPERTIES

Each extrusion is modeled using SCIA Engineer 18.1. The Engineering properties for each extrusion are listed below.

A11		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [$inch^2$]	0.72267	
A_y [$inch^2$], A_z [$inch^2$]	0.41568	0.47416
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	1.10e+01	1.10e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	0.159
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.05194	0.56497
i_y [inch], i_z [inch]	0.268	0.884
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.07882	0.42638
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.15357	0.55826
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.35e+00	5.35e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.94e+01	1.94e+01
d_y [inch], d_z [inch]	0.000	0.613
I_t [$inch^4$], I_w [$inch^6$]	0.00568	0.04948
$β_y$ [inch], $β_z$ [inch]	-2.922	0.000
Picture		

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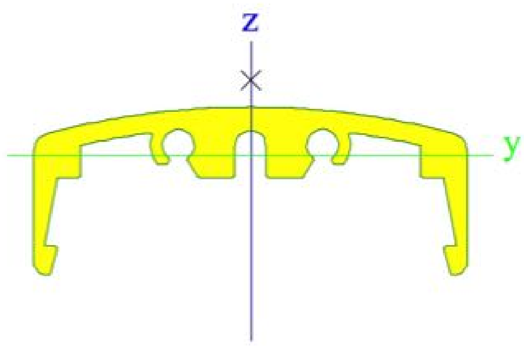
A111		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [$inch^2$]	0.53225	
A_y [$inch^2$], A_z [$inch^2$]	0.32466	0.26383
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	8.74e+00	8.74e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	0.189
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.02047	0.27528
i_y [inch], i_z [inch]	0.196	0.719
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.03266	0.24579
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.07620	0.32497
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.65e+00	2.65e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.13e+01	1.13e+01
d_y [inch], d_z [inch]	0.000	0.386
I_t [$inch^4$], I_w [$inch^6$]	0.00400	0.01183
$β_y$ [inch], $β_z$ [inch]	-2.549	0.000
Picture		

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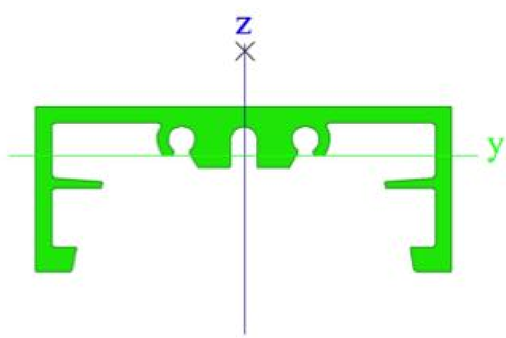
A222		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [inch^2]	0.66275	
A_y [inch^2], A_z [inch^2]	0.37998	0.32995
A_L [inch^2/inch], A_D [inch^2/inch]	1.20e+01	1.20e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.001	0.200
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.05785	0.48132
i_y [inch], i_z [inch]	0.295	0.852
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.08265	0.38466
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.15120	0.48655
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.26e+00	5.26e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.69e+01	1.69e+01
d_y [inch], d_z [inch]	0.006	0.630
I_t [inch^4], I_w [inch^6]	0.00399	0.06424
β_y [inch], β_z [inch]	-2.706	-0.014
Picture		

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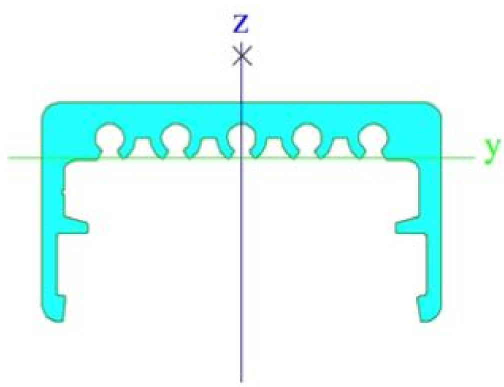
A225		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [inch^2]	0.75473	
A_y [inch^2], A_z [inch^2]	0.41813	0.35870
A_L [inch^2/inch], A_D [inch^2/inch]	1.18e+01	1.18e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	0.311
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.07776	0.50963
i_y [inch], i_z [inch]	0.321	0.822
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.08312	0.44982
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.17145	0.56130
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.97e+00	5.97e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.95e+01	1.95e+01
d_y [inch], d_z [inch]	-0.001	0.578
I_t [inch^4], I_w [inch^6]	0.00660	0.06184
β_y [inch], β_z [inch]	-2.439	0.001
Picture		

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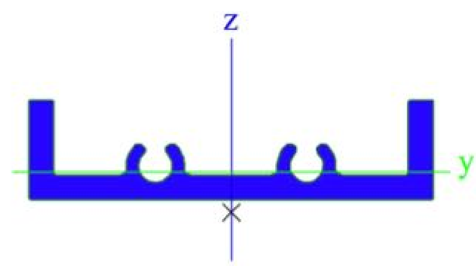
520E		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [$inch^2$]	0.37539	
A_y [$inch^2$], A_z [$inch^2$]	0.24782	0.20625
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	6.86e+00	6.86e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	-0.112
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.00589	0.17058
i_y [inch], i_z [inch]	0.125	0.674
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.01626	0.17058
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.03415	0.22469
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.19e+00	1.19e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	7.82e+00	7.82e+00
d_y [inch], d_z [inch]	0.000	-0.200
I_t [$inch^4$], I_w [$inch^6$]	0.00175	0.00356
β_y [inch], β_z [inch]	1.988	0.000
Picture		

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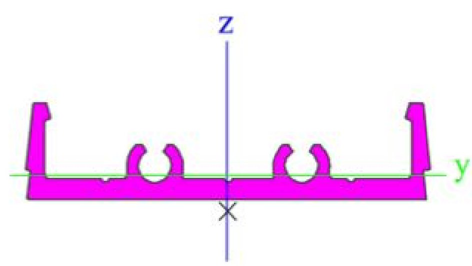
520S		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour	■	
A [inch^2]	0.32028	
A_y [inch^2], A_z [inch^2]	0.22334	0.18055
A_L [inch^2/inch], A_D [inch^2/inch]	7.17e+00	7.17e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.006	-0.126
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.00433	0.13977
i_y [inch], i_z [inch]	0.116	0.661
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.01158	0.13502
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.02613	0.18642
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	9.10e-01	9.10e-01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	6.49e+00	6.49e+00
d_y [inch], d_z [inch]	0.003	-0.179
I_t [inch^4], I_w [inch^6]	0.00098	0.00285
β_y [inch], β_z [inch]	1.934	-0.003
Picture		

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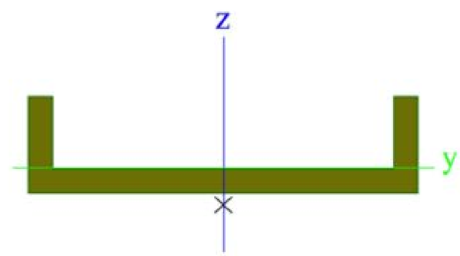
520		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		
A [$inch^2$]	0.34375	
A_y [$inch^2$], A_z [$inch^2$]	0.23294	0.13208
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	5.75e+00	5.75e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	-0.119
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.00569	0.16585
i_y [inch], i_z [inch]	0.129	0.695
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.01539	0.16585
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.03015	0.21289
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1.05e+00	1.05e+00
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	7.41e+00	7.41e+00
d_y [inch], d_z [inch]	0.000	-0.190
I_t [$inch^4$], I_w [$inch^6$]	0.00171	0.00359
β_y [inch], β_z [inch]	2.198	0.000
Picture		

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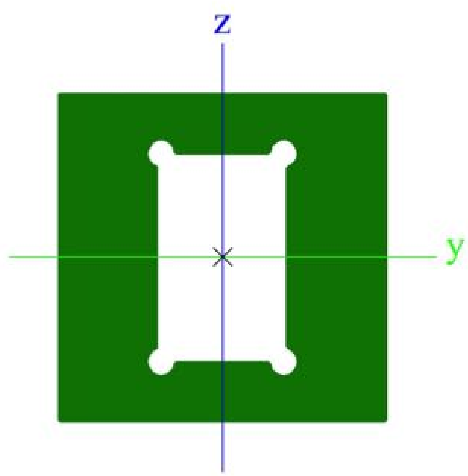
2020GL Solid		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6061-T6 Extrusion	
Fabrication	general	
Colour		
A [$inch^2$]	2.93894	
A_y [$inch^2$], A_z [$inch^2$]	1.68840	2.21451
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	7.97e+00	1.28e+01
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.000	0.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	1.17371	1.27225
i_y [inch], i_z [inch]	0.632	0.658
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	1.17371	1.27225
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	1.64370	1.77940
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	5.72e+01	5.72e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	6.19e+01	6.19e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [$inch^4$], I_w [$inch^6$]	1.88315	0.01735
$β_y$ [inch], $β_z$ [inch]	0.000	0.000
Picture		

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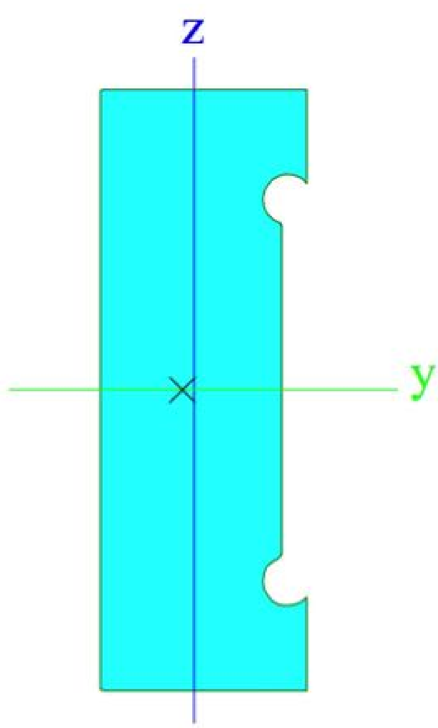
2020GL Half Post		
Type	General cross-section	
Shape type	Thin-walled	
Item material	6063-T6	
Fabrication	general	
Colour		

A [$inch^2$]	1.24287	
A_y [$inch^2$], A_z [$inch^2$]	1.07333	1.01921
A_L [$inch^2$/inch], A_D [$inch^2$/inch]	5.71e+00	5.71e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	-0.031	0.000
α [deg]	0.00	
I_y [$inch^4$], I_z [$inch^4$]	0.43252	0.04133
i_y [inch], i_z [inch]	0.590	0.182
$W_{el.y}$ [$inch^3$], $W_{el.z}$ [$inch^3$]	0.43252	0.11020
$W_{pl.y}$ [$inch^3$], $W_{pl.z}$ [$inch^3$]	0.63636	0.19520
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.22e+01	2.22e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	6.79e+00	6.79e+00
d_y [inch], d_z [inch]	-0.038	0.000
I_t [$inch^4$], I_w [$inch^6$]	0.11546	0.01098
$β_y$ [inch], $β_z$ [inch]	0.000	0.236
Picture		

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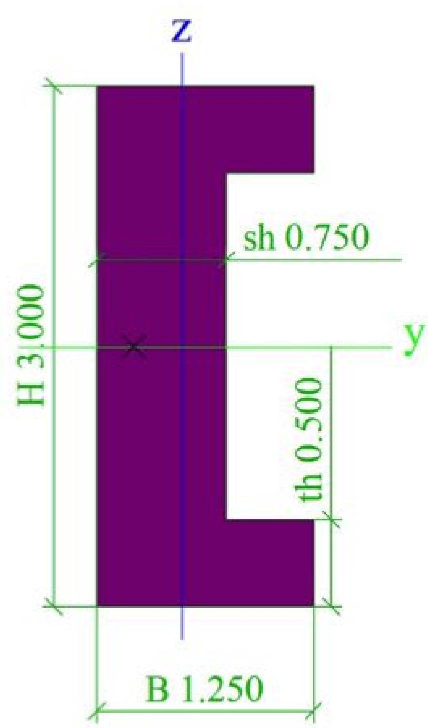
WDCR		
Type	U g	
Detailed	3.000; 1.250; 0.500; 0.750	
Shape type	Thick-walled	
Item material	DF-L SS	
Fabrication	general	
Colour	■	
A [inch^2]	2.75000	
A_y [inch^2], A_z [inch^2]	2.24043	2.01442
A_L [inch^2/inch], A_D [inch^2/inch]	9.50e+00	9.50e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.489	1.500
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	2.47917	0.27569
i_y [inch], i_z [inch]	0.949	0.317
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1.65278	0.36210
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2.31250	0.71354
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	8.05e+01	8.05e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	2.48e+01	2.48e+01
d_y [inch], d_z [inch]	-0.276	0.000
I_t [inch^4], I_w [inch^6]	0.41102	0.24798
$β_y$ [inch], $β_z$ [inch]	0.000	1.483

Picture



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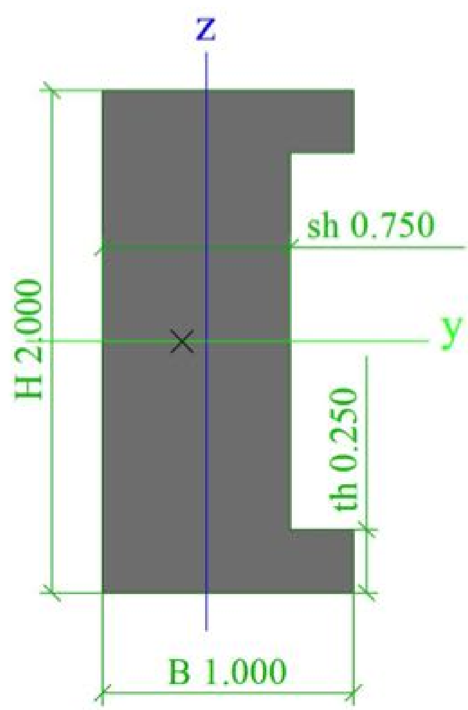
WDHR		
Type	U g	
Detailed	2.000; 1.000; 0.250; 0.750	
Shape type	Thick-walled	
Item material	DF-L SS	
Fabrication	general	
Colour	■	
A [inch^2]	1.62500	
A_y [inch^2], A_z [inch^2]	1.41163	1.29612
A_L [inch^2/inch], A_D [inch^2/inch]	6.50e+00	6.50e+00
$c_{Y.UCS}$ [inch], $c_{Z.UCS}$ [inch]	0.413	1.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.59635	0.09981
i_y [inch], i_z [inch]	0.606	0.248
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	0.59635	0.17017
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	0.85937	0.34180
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.99e+01	2.99e+01
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1.19e+01	1.19e+01
d_y [inch], d_z [inch]	-0.097	0.000
I_t [inch^4], I_w [inch^6]	0.21855	0.02559
β_y [inch], β_z [inch]	0.000	0.498
Picture		

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Explanations of symbols	
A	Area
A_y	Shear Area in principal y-direction - Calculated by 2D FEM analysis
A_z	Shear Area in principal z-direction - Calculated by 2D FEM analysis
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis
i_z	Radius of gyration about the principal z-axis

Explanations of symbols	
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid - Calculated by 2D FEM analysis
d_z	Shear center coordinate in principal z-direction measured from the centroid - Calculated by 2D FEM analysis
I_t	Torsional constant - Calculated by 2D FEM analysis
I_w	Warping constant - Calculated by 2D FEM analysis
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

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