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Structural Calculations for

Residential/Commercial
Aluminum Cable Guardrail System

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Prepared for
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Scope of Work

Development and design for an aluminum cable railing system including:
Termination post, intermediate post, top rail, rail connecting blocks, cables, end cap, flat infill, base plate, stair facia, stair intermeadiate cap, and attachments.

General

The enclosed calculations were intended to be to designed and submitted in conformance with the following:
Professional Engineer Seals
State of Oregon

Building Codes (Meets or Exceeds Requirements)
2014 Oregon Structural Specialty Code and Oregon Residential Specialty Codes

Additional Design References
2010 Aluminum Design Manual
2011 Building Code Requirements for Structural Concrete (ACI318-11)
AISC Steel Construction Manual, 14th Edition
2012 National Design Specification for Wood Construction
ICC Report AC273: Acceptance Criteria for Handrails and Guards

Materials

6061-T6, T6510, T6511 Extrusions	Tensile Ultimate Strength,	F_{tu} =	38 ksi
	Tensile Yield Strength,	F_{ty} =	35 ksi
	Compressive Yield Strength,	F_{cy} =	35 ksi
	Shear Ultimate Strength,	F_{su} =	24 ksi
A554 Stainless Steel Grade 304/304L	Yield Stress,	F_y =	30 ksi
	Tensile Stress,	F_u =	90 ksi
Type 316 Stainless Steel Wire Rope	1x19 Strand Core		
	1/8" dia. with breaking strength = 1,869-lbs		
	7x7 Strand Core		
	1/8" dia. with breaking strength = 1,566-lbs		

Guardrail Loading Conditions

Uniform Load

Per 2012 IBC §1607.8.1, the uniform load shall be applied to the handrail in any direction. The railing system covered in this package covers all commercial and residential properties. — p = 50 plf

Concentrated Load

Per IBC §1607.8.1.1, the concentrated load shall be applied to the handrail in any direction — P = 200 lbs

Per IBC §1607.8.1.2, components including intermediate rails, balusters, and cables shall be designed for a concentrated load applied normal and horizontally over an area of 1ft². — P = 50 lbs

Per IBC §1013.2 and IRC §312.3 opening limitations shall not allow the passage of a sphere 4" in diameter through.

Part Numbers and Descriptions

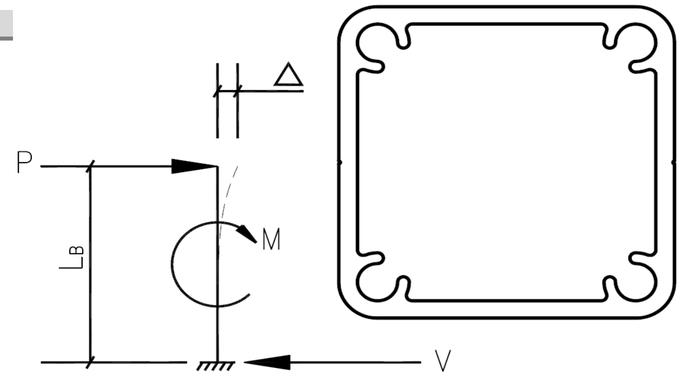
IP100 - SCRS Extruded Aluminum Intermediate Posts	Page 4, 5
TR100 - SCRS Extruded Aluminum Top Rail	Page 6
FI200 - SCRS Extruded Aluminum Flat Infill	
EC100 - SCRS Top Rail End Cap	
BP100 - SCRS Base Plate	Page 7, 8
SR200 - SCRS Extruded Aluminum Stair Rail	Page 9,10
RCB100 - SCRS Stair Grab Rail Connecting Block	
Stainless Steel Wire Rope	Page 11
TP100 - SCRS Extruded Aluminum Termination Posts	Page 12
ISPA200 - SCRS Stair Post Cap Assembly	Page 13
SCRS Extruded Aluminum Facia Mount Posts	Page 14, 15, 16

Aluminum Cable Guardrail System Summary

Total Post/Handrail Height Including Base Plate	42 in
Maximum Termination Post Spacing	5 ft
Maximum Stair Rail Post Spacing	5 ft
Cable Prestressing	255 lbs
Cable Spacing (On-Center)	3.125 in

Extruded Aluminum Post Input

Post Spacing, s =	5 ft
(See Page 3) Applied Load At Top, P =	250 lbs
Unbraced Length, L_B =	45.500 in
Post Area, A_p =	1.146 in^2
Compressive Modulus of Elasticity, E =	10100 ksi
Section Modulus, S =	0.744 in^3
Moment of Inertia, $I_x > I_y$ =	0.837 in^4
Torsion Constant, J =	0.073 in
Clear Height of Shear Area, h =	2.250 in
Thickness of Shear Area, t =	0.125 in



Flat Elements Supported on Both Ends in Uniform Compression, 6061-T6 (ADM 2010 Section B5.4.2, Table 2-19 Part VI)

Slenderness, S =	16.0	
Allowable Stress, $S \le S_1$ =	21.2 ksi	◄ Controls
Slenderness Limit, S_1 =	21	
Allowable Stress, $S_1 < S < S_2$ =	22.6 ksi	
Slenderness Limit, S_2 =	33	
Allowable Stress, $S \ge S_2$ =	36.3 ksi	
Allowable Bending Stress, F_b =	21.2 ksi	
Allowable Moment, S*Fc = M_{allow} =	**15.779 kip-in**	
Applied Moment, P*L_B = $M_{applied}$ =	**11.38 kip-in**	**OK**

Flexural Compression Closed Shapes Lateral Torsional Buckling 6061-T6 (ADM 2010 Section F3.1, Table 2-19 Part VI)

Slenderness, S =	274.0	
Allowable Stress, $S \le S_1$ =	-5.42 ksi	
Slenderness Limit, S_1 =	55	
Allowable Stress, $S_1 < S < S_2$ =	19.961 ksi	◄ Controls
Slenderness Limit, S_2 =	1685	
Allowable Stress, $S \ge S_2$ =	1.159 ksi	
Allowable Bending Stress, F_a =	**20.0 ksi**	
Applied Bending Stress, F_b =	**15.283 ksi**	**OK**

Deflection Check, Δ_{MAX} = L_B/12 (ICC Report AC273)

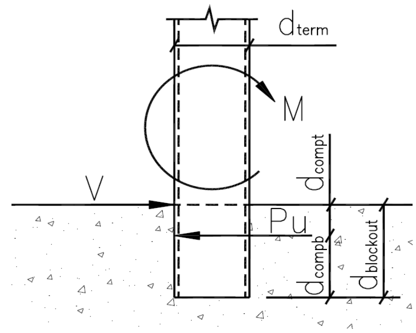
Allowable Deflection, Δ_{allow} =	3.792 in	
Applied Deflection, $PL_B^3/3EI$ = $\Delta_{applied}$ =	0.928 in	**OK**

Shear in Elements, Gross Section 6061-T6 (ADM 2010 Table Section G.2, Table 2-19 Part VI)

Allowable Stress, $S \le S_1$ =	12.7 ksi	◄ Controls
Slenderness Limit, S_1 =	35.3	
Allowable Stress, $S_1 < S < S_2$ =	14.8 ksi	
Slenderness Limit, S_2 =	63.0	
Allowable Stress, $S \ge S_2$ =	151.0 ksi	
Allowable Shear Stress, F_s =	13 ksi	
Allowable Shear, Ap*Fs = V_{allow} =	**14.557 kips**	
Applied Shear, P = $V_{applied}$ =	**0.250 kips**	**OK**

Core Mounted Posts Bearing Check

Existing Concrete Strength, f'_c =	2500 psi	
$V_{applied}$ =	0.250 kips	*(See Page 4)*
$M_{applied\ from\ post}$ =	11.375 kip-in	*(See Page 4)*
$M_{applied\ from\ shear}$ =	0.750 kip-in	
M_{total} =	12.125 kip-in	
Depth of Concrete Blockout, $d_{blockout}$ =	3.000 in	
Distance Bottom of Blockout to Applied P_u, d_{compb} =	2.000 in	
Distance from Applied P_u to Top of Concrete, d_{compt} =	1.000 in	
Width of Post, d_{term} =	2.250 in	
Loaded Area, A_1 =	2.250 in²	
Area of the Lower Base of Largest Fulcrum, A_2 =	6.500 in²	
Compression Load at Blockout, P_u =	6.063 kips	
Strength Reduction Factor, Ø =	0.65	*(Per ACI 318-11 §9.3.2.4)*
Concrete Bearing Strength, f_b =	5282 psi	*(Per ACI 318-11 §10.14.1)*
Maximum Applied Compression Load, f_{bmax} =	**2694 psi**	**OK < 5282 psi**



Core Mounted Posts Edge Distance Check

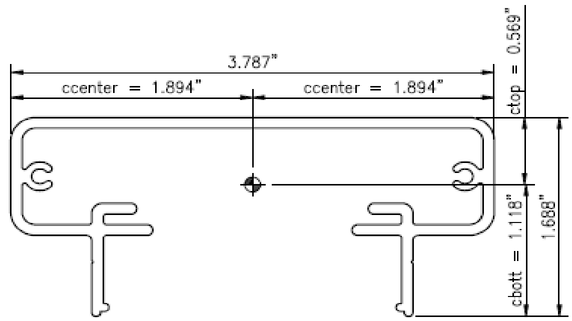
stance from Center of Post to Edge of Concrete, c_{a1} =	6.750 in	
Distance from Post Face to Edge of Concrete, c_{post} =	4.500 in	
Thickness of Concrete, h_{a1} =	4.000 in	
Projected Concrete Failure Area, A_{Vco} =	205.031 in²	*(Per ACI 318-11 §D.6.2.1 D-32)*
Projected Concrete Failure Area, A_{Vc} =	90.000 in²	*(Per ACI 318-11 §D.6.2.1 D-32)*
Shear Strength Modification Factor, $\psi_{ed,V}$ =	1.00	*(Per ACI 318-11 §D.6.2.6)*
Cracked Concrete Modification Factor, $\psi_{c,V}$ =	1.00	*(Per ACI 318-11 §D.6.2.7)*
Cracked Concrete Modification Factor, $\psi_{h,V}$ =	1.59	*(Per ACI 318-11 §D.6.2.8)*
Lightweight Concrete Factor, λ =	1.00	*(Per ACI 318-11 §8.6.1)*
Basic Concrete Breakout Strength, V_b =	9.752 kips	*(Per ACI 318-11 §D.6.2.2)*
Nominal Concrete Breakout Strength, V_{cb} =	6.811 kips	*(Per ACI 318-11 §D.6.2.1 D-31)*
Max Nominal Concrete Breakout Strength, V_{max} =	**6.811 kips**	**OK < 6.0625 kips**

Use 4,000psi Non-Shrink Grout in Min 3"SQx4"Deep Blockout or 3" Diax4"Deep Hole with 4 ½" Min Edge Distance (No Rebar) or 1 ¼" Min Edge Distance when #3 or Larger Slab Edge Rebar Present

Check Top Connection

Note: Lateral loads on top rail bears directly on post side. Only uplift loads affecting attachment are considered.

Diameter of Screw, d_{screw} =	0.194 in	
Thickness of Post, t_{post} =	0.125 in	
Area of Engaged Post in Shear, A_{Vpost} =	0.024 in²	
Number of Screws in Shear =	2	
Factor of Safety on Screw Connections, n_s =	3.00	
Tensile Ultimate Strength of Member Not in Contact with Screw Head, F_{tu2} =	38 ksi	
Shear Strength of Screw, V_{screw} =	**0.614 kips**	**OK > 0.250 kips**

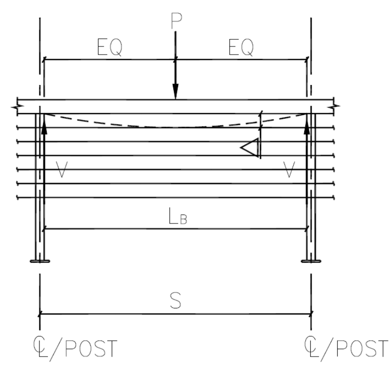


Extruded Aluminum Rail Input

Post Spacing, s =	5 ft	Left/Right to Centroid, C_{center} =	1.895 in
(See Page 3) Applied Load At Top, P =	250 lbs	Section Modulus Top, S_{xx} =	0.373 in³
Unbraced Length, L_B = 5'-0" x 12" - 2.25" =	57.750 in	Section Modulus Bottom, S_{xx} =	0.190 in³
Compressive Modulus of Elasticity, E =	10100 ksi	Section Modulus , S_{yy} =	0.879 in³
Rail Area, A_r =	0.928 in²	Torsion Constant, J =	0.005 in
Moment of Inertia x, I_{xx} =	0.212 in⁴	Clear Height of Shear Area, h =	1.643 in
Moment of Inertia y, I_{yy} =	1.664 in⁴	Thickness of Shear Area, t =	0.090 in
Top of Member to Centroid, C_{top} =	0.569 in		
Bottom of Member to Centroid, C_{bott} =	1.118 in		

Flexural Compression Closed Shapes Lateral Torsional Buckling 6061-T6 (ADM 2010 Section F3.1, Table 2-19 Part VI)

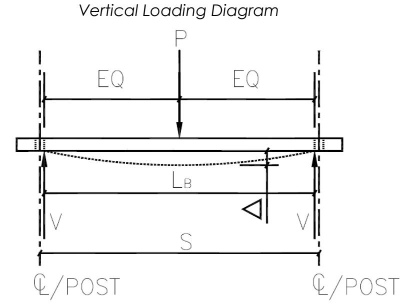
Slenderness, S =	469.7	
Allowable Stress, S≤S_1 =	-19.26 ksi	
Slenderness Limit, S_1 =	55	
Allowable Stress, S_1<S<S_2 =	18.742 ksi	◄ Controls
Slenderness Limit, S_2 =	1685	
Allowable Stress, S≥S_2 =	0.394 ksi	
Applied Moment, P*L_B/4 = $M_{APPLIED}$ =	3.609 kip-in	
Allowable Bending Stress, F_b =	**18.7 ksi**	
Vertical Compressive Stress Applied, C_{Vert} =	**9.672 ksi**	**OK**
Horizontal Compressive Stress Applied, C_{Horiz} =	**4.109 ksi**	**OK**



Vertical Loading Diagram

Flat Elements Supported on Both Ends in Uniform Tension 6061-T6 (ADM 2010 Section F8.1, Table 2-19 Part VI)

Tensile Yield Strength, F_{ty} =	35 ksi	
Factor of Safety on Yield Strength, n_y =	1.65	
Allowable Tensile Stress, F =	21 ksi	
Tensile Ultimate Strength, F_{tu} =	38 ksi	
Factor of Safety on Ultimate Strength, n_u =	1.95	
Coefficient for Tension Members, k_t =	1.00	
Allowable Tensile Stress, F =	**19.5 ksi**	**◄ Controls**
Vertical Tensile Stress Applied, T_{Vert} =	**18.999 ksi**	**OK**
Horizontal Tensile Stress Applied, T_{Horiz} =	**4.109 ksi**	**OK**



Horizontal Loading Diagram

Deflection Check, Δ_{MAX} = L_B/12 (ICC Report AC273)

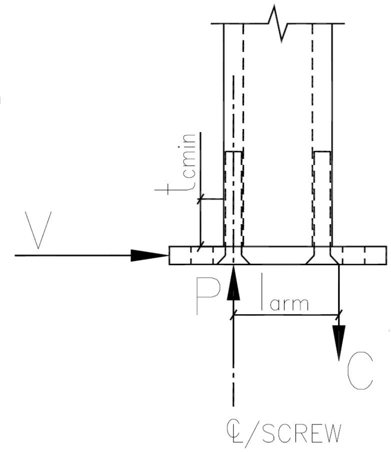
Allowable Deflection, Δ_{ALLOW} =	4.813 in	
Applied Deflection, $PL_B^3/48EI$ = $\Delta_{APPLIED}$ =	0.468 in	OK

Shear in Elements, Gross Section 6061-T6 (ADM 2010 Table Section G.2, Table 2-19 Part VI)

Slenderness, S =	14.3	
Allowable Stress, S≤S_1 =	12.7 ksi	◄ Controls
Slenderness Limit, S_1 =	35.3 in³	
Allowable Stress, S_1<S<S_2 =	14.970 ksi	
Slenderness Limit, S_2 =	63 in³	
Allowable Stress, S≥S_2 =	189.080 ksi	
Allowable Shear Stress, F_s =	189.1 ksi	
Allowable Shear, Ap*F_s = V_{ALLOW} =	**11.780 kips**	
Applied Shear, P = $V_{APPLIED}$ =	**0.250 kips**	**OK**

Tension Capacity of Screw (ADM 2010 Section J5.5)

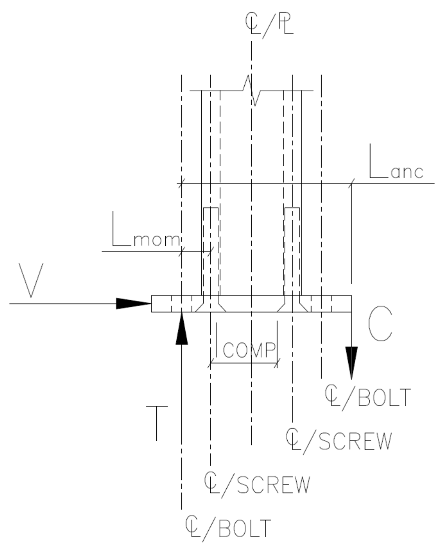
Screw Properties: 5/16"-18 x 2" 6-Lobe Flat Head Floorboard Thread Cutting Screw, Type F, Black Phosphate and Oil
Fu = 120 ksi, Fy = 48 ksi, Ft = 90 ksi, Min Dia = 0.3026 in, Area = 0.0702 in^2



$M_{APPLIED}$ = Load x Length =	10.500 kip-in	
	(Page 4)	
Number of Screws in Tension =	2	
Resisting Moment Arm, Center of Screw to Compression Face, l_{arm} =	2.000 in	
Tension Applied, $P_{applied}$ =	**2.625 kips**	
Thread Stripping Area of Internal Thread Per Inch, A_{sn} =	0.663 in²	
Depth of Full Thread Engagement into t_2, t_{cmin} =	1.000 in	
Tensile Ultimate Strength of Member Not in Contact with Screw Head, F_{tu2} =	38 ksi	
Nominal Pull-Out Strength, P_{not} =	**14.613 kips**	
	(Eq. J.5-3)	
Thickness of Member in Contact with Screw Head, t_1 =	0.375 in	
Tensile Yield Strength of Member in Contact with Screw Head, F_{ty1} =	35 ksi	
Nominal Screw Head Diameter Abs Min, D =	0.568 in	
t_1/D =	0.66	<1.1
Nominal Pull-Over Strength, P_{nov} =	**9.150 kips**	
	(Eq. J5.10)	
Tensile Strength of Screw, F_t =	90 ksi	
Tensile Stress Area of Screw, A_t =	0.072 in²	
Nominal Tensile Strength of a Screw, P_{nt} =	**6.480 kips**	
Factor of Safety on Screw Connections, n_s =	3.00	
Ω =	2.00	
Pull-Out Strength, P_{not}/n_s =	**4.871 kips**	**OK > 2.625 kips**
Pull-Over Strength, P_{nov}/n_s =	**3.050 kips**	**OK > 2.625 kips**
Tensile Strength, P_{nt}/Ω =	**3.240 kips**	**OK > 2.625 kips**

Shear Capacity of Screw (ADM 2010 J5.6)

$V_{APPLIED}$ =	1.658 kips	*(TP-1)*	
Number of Screws in Shear =	4		
Shear Applied, $V_{applied}$ =	**0.414 kips**	**Per Screw**	
Tensile Ultimate Strength of Member in Contact with Screw Head, F_{tu1} =	38 ksi	Note:	1/2 of depth subtracted from t_1
Factor of Safety on Ultimate Strength, n_u =	1.95		as screw is countersunk
Check 1) Screw Shear and Bearing Strength, Pv =	**6.099 kips**	*(Eq. J.5-12)*	
Thickness of Member Not in Contact with Screw Head, t_2 =	1.000 in		
Check 2) Screw Shear and Bearing Strength, Pv =	**11.069 kips**	*(Eq. J.5-12)*	
Check 3) Screw Shear and Bearing Strength, Pv =	**N/A**	*(Eq. J.5-13)*	
Nominal Shear Strength of a Screw, P_{ss} =	5.522 kips		
Check 4) Screw Shear and Bearing Strength, Pv =	**1.473 kips**	*(Eq. J.5-14)*	
Minimum Screw Shear and Bearing Strength, Pv_{min} =	**1.473 kips**	**OK > 0.414 kips**	



Base Plate Anchorage (Lag Screws) Per 2012 National Design Specification for Wood Construction

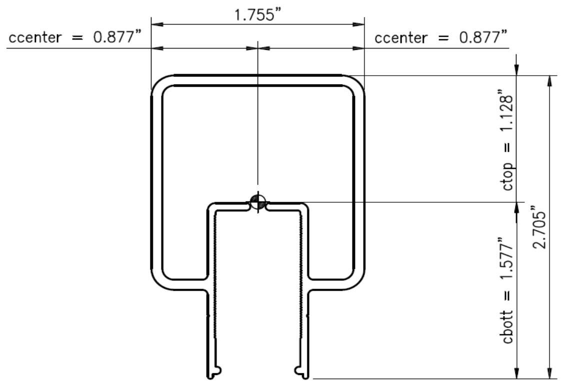
Applied Moment at TP100, Mapplied =	11.375 kip-in	*(Page 4)*
Edge of Baseplate to Centerline of Tension Anchorage, lanc =	4.360 in	
Number of Screws in Tension =	2	
Applied Tension at Anchor Bolt/Screw, Tapplied =	**1.304 kips**	
Vapplied =	0.250 kips	*(Page 4)*
Number of Screws in Shear =	4	
Shear Applied, Vapplied =	**0.063 kips**	**Per Screw**
Lag Screw Reference Withdrawl Design Value (G=0.46, D=3/8"), W =	269 lbs	*(Per Table 11.2A)*
Penetration Depth, d =	4.500 in	
Allowable Lag Screw Tension, Tallowable =	**1.937 kips**	**OK > 1.304 kips**
Lag Screw Reference Lateral Design Value (G=0.46, D=3/8"), Z =	170 lbs	*(Per Table 11K)*
Allowable Lag Screw Shear, Vallowable =	**0.170 kips**	**OK > 0.063 kips**

Use (4) ⅜" Dia SS304 Lag Screws with 6" Min Penetration into Min (1) 6x6 or (2) 3x6 Hem-Fir #2 (1.5" Min Edge Distance)

Base Plate Anchorage (Thru-Bolts) Per 2012 National Design Specification for Wood Construction

Bolt diameter =	0.375 in	
Diameter of washer =	2.500 in	
Area of Bearing under washer =	4.758 in²	
Washer bearing, $F_{c\ perp}$ =	521 psi	*(Per Table 4A)*
Allowable Thru-Bolt Tension, Tallowable =	**2.209 kips**	**OK > 1.304 kips**
Lag Screw Reference Lateral Design Value (G=0.46, D=3/8"), Z =	170 lbs	(No Thru-Bolt Values < 1/2" In NDS - Use Table 11K)
Allowable Thru-Bolt Shear, Vallowable =	**0.170 kips**	**OK > 0.063 kips**

Use (4) ⅜" Dia SS304 Thru-Bolts with Min 2" Dia Heavy Washer into Min (1) 6x or (2) 3x Hem-Fir #2

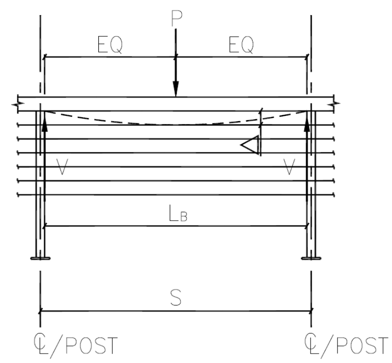


Extruded Aluminum Rail Input

Post Spacing, s =	5 ft	Left/Right to Centroid, Ccenter =	0.878 in
(See Page 2) Applied Load At Top, P =	250 lbs	Section Modulus Top, S_{xx} =	0.414 in³
Unbraced Length, L_B = 6'x12"-2.25" =	57.750 in	Section Modulus Bottom, S_{xx} =	0.296 in³
Compressive Modulus of Elasticity, E =	10100 ksi	Section Modulus , S_{yy} =	0.350 in³
Rail Area, A_r =	0.761 in²	Torsion Constant, J =	0.005 in
Moment of Inertia x, I_{xx} =	0.467 in⁴	Clear Height of Shear Area, h =	2.660 in
Moment of Inertia y, I_{yy} =	0.307 in⁴	Thickness of Shear Area, t =	0.090 in
Top of Member to Centroid, Ctop =	1.128 in		
Bottom of Member to Centroid, Cbott =	1.577 in		

Flexural Compression Closed Shapes Lateral Torsional Buckling 6061-T6 (ADM 2010 Section F3.1, Table 2-19 Part VI)

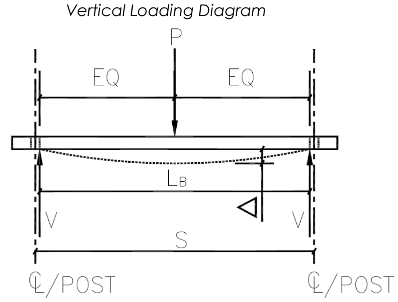
Slenderness, S =	1238.1	
Allowable Stress, S≤S1 =	-55.78 ksi	
Slenderness Limit, S1 =	55	
Allowable Stress, S1<S<S2 =	15.525 ksi	◄ Controls
Slenderness Limit, S2 =	1685	
Allowable Stress, S≥S2 =	0.057 ksi	
Applied Moment, P*L_B/4 = MAPPLIED =	3.609 kip-in	
Allowable Bending Stress, Fb =	**15.5 ksi**	
Vertical Compressive Stress Applied, CVert =	**8.719 ksi**	**OK**
Horizontal Compressive Stress Applied, CHoriz =	**10.321 ksi**	**OK**



Vertical Loading Diagram

Flat Elements Supported on Both Ends in Uniform Tension 6061-T6 (ADM 2010 Section F8.1, Table 2-19 Part VI)

Tensile Yield Strength, F_{ty} =	35 ksi	
Factor of Safety on Yield Strength, n_y =	1.65	
Allowable Tensile Stress, F =	21 ksi	
Tensile Ultimate Strength, F_{tu} =	38 ksi	
Factor of Safety on Ultimate Strength, n_u =	1.95	
Coefficient for Tension Members, k_t =	1.00	
Allowable Tensile Stress, F =	**19 ksi**	**◄ Controls**
Vertical Tensile Stress Applied, TVert =	**12.188 ksi**	**OK**
Horizontal Tensile Stress Applied, THoriz =	**10.321 ksi**	**OK**



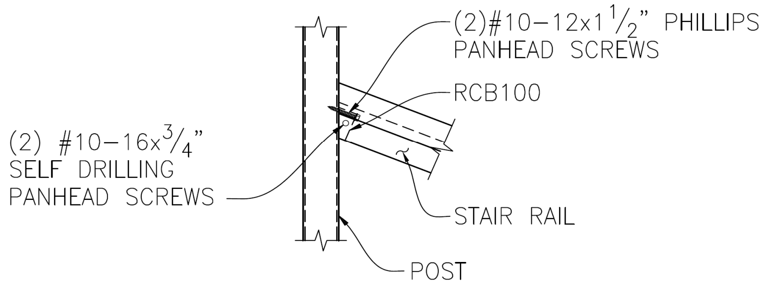
Horizontal Loading Diagram

Deflection Check, ΔMAX = L_B/12 (ICC Report AC273)

Allowable Deflection, ΔALLOW =	4.813 in	
Applied Deflection, $PL_B^3/48EI$ = ΔAPPLIED =	0.213 in	**OK**

Shear in Elements, Gross Section 6061-T6 (ADM 2010 Table Section G.2, Table 2-19 Part VI)

Slenderness, S =	25.6	
Allowable Stress, S≤S1 =	12.7 ksi	◄ Controls
Slenderness Limit, S1 =	35.3 in³	
Allowable Stress, S1<S<S2 =	13.761 ksi	
Slenderness Limit, S2 =	63 in³	
Allowable Stress, S≥S2 =	58.998 ksi	
Allowable Shear Stress, Fs =	12.7 ksi	
Allowable Shear, Ap*Fs = VALLOW =	**9.665 kips**	
Applied Shear, P = VAPPLIED =	**0.250 kips**	**OK**



Check Fascia Mount

Note: Uses (2) #10-12x1 1/2" Phillips Pan Head Sheet Metal Screws - Type A, 18-8 Stainless Steel

Diameter of Screw, d_{screw} =	0.189 in	
Thickness of Post, t_{post} =	0.125 in	
Area of Engaged Post in Shear, AV_{post} =	0.024 in²	
Number of Screws in Shear =	2	
Factor of Safety on Screw Connections, n_s =	3.00	
Tensile Ultimate Strength of Member Not in Contact with Screw Head, F_{tu2} =	38 ksi	
Shear Strength of Screw, V_{screw} =	**0.599 kips**	**OK > 0.250 kips**

Note: Uses (2) #10-16x3/4" Phillips Pan Head Self Drilling Screw Zinc #3 Point

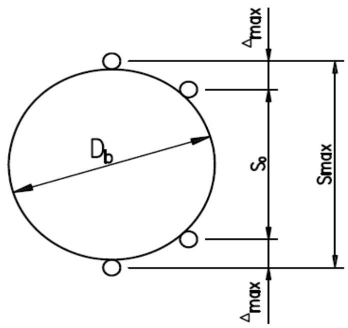
Diameter of Screw, d_{screw} =	0.194 in	
Thickness of Post, t_{post} =	0.125 in	
Area of Engaged Post in Shear, AV_{post} =	0.024 in²	
Number of Screws in Shear =	2	
Factor of Safety on Screw Connections, n_s =	3.00	
Tensile Ultimate Strength of Member Not in Contact with Screw Head, F_{tu2} =	38 ksi	
Shear Strength of Screw, V_{screw} =	**0.614 kips**	**OK > 0.250 kips**

Check Stair Rail Adapter

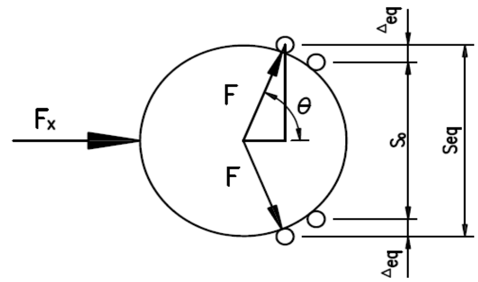
Note: Uses (2) #10-16x3/4" Phillips Pan Head Self Drilling Screw Zinc #3 Point See Check Above

Note: Uses (2) #10-16x3/4" Phillips Pan Head Self Drilling Screw Zinc #3 Point See Check Above

Applied Shear, V_{base} = V_{conn} = $V_{applied}$ =	1.658 kips	*(TP-1)*
Distance from Center of Bolt to Face of Base, l_{arm} =	0.655 in	
Applied Moment, $M_{applied}$ =	1.085 kip-in	
Distance from Edge of Base to Center of Screw, l_{base} =	1.450 in	
Applied Tension, $T_{applied}$ =	0.749 kips	
Diameter of Screw, d_{screw} =	0.194 in	
Thickness of Post, t_{post} =	0.125 in	
Area of Engaged Post in Shear, AV_{post} =	0.024 in²	
Number of Screws in Shear =	2	
Factor of Safety on Screw Connections, n_s =	3.00	
Tensile Ultimate Strength of Member Not in Contact with Screw Head, F_{tu2} =	38 ksi	
Shear Strength of Screw, V_{screw} =	**0.614 kips**	**OK > 0.250 kips**



Initial and Pass–Through Conditions



Conditions at Equilibrium

Check Cable Deflection

Note: A min load of 50psf shall be applied to a 4" sphere. Spacing and deflection of the cables shall not allow the sphere to pass through.

Diameter of Cable, D = 0.125 in

Intermediate Post Spacing, L = 5 ft

Prestress Force, F_{ps} = 255 lbs

Sphere Diameter, D_b = 4.000 in

Initial Cable Spacing, S_o = 3.125 in

Termination Post Spacing, L_T = 30 ft

Load Applied to Sphere, w_{sphere} = 50.0 psf

Projected Area of Sphere, A_{sphere} = 12.566 in²

Impact Factor, i_f = 2.00

Force Applied to Sphere, $F_{xsphere}$ = 8.727 lbs

Spread at Pass-Thru = D_b+D_{cable}, S_{max} = 4.125 in

Final Cable Spacing, S_{final} = S_{eq} = 4.124 in

Deflection at Pass-Thru = (S_{max}-S_o)/2, Δ'_{max} = 0.500 in

Deflection, Δ = Δ_{eq} = 0.500 in

Applied Angle = asin((S_o+2Δ)/(D_b+D)), θ = 88.7 °

Force Applied to Cable, T = F = 198.171 lbs

Maximum Cable Deflection = (D_b+D-S_o)/2, Δ_{max} = 0.500 in

Modulous of Elasticity, E = 29000 ksi

Moment of Inertia, I = 0.00001198 in⁴

Cross Sectional Area, A = 0.012 in²

Extensible, Flexible Cable, P_{ef} = 0.274 lbs

Flexural Bending, P_b = 0.039 lbs

Prestressing, P_{ps} = 8.492 lbs

Force in Cable Resisting Sphere, F_{xcable} = P = 8.804 lbs **OK**

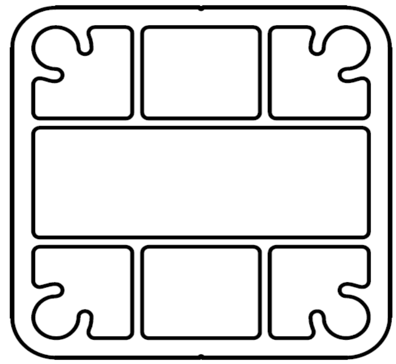
1/8" Diameter 1x19 Strand Core Breaking Strength = 1869 lbs

1/8" Diameter 7x7 Strand Core Breaking Strength = 1566 lbs **◄ Controls (OK < 310lbs)**

$$P_{ef} = \frac{4\Delta EA}{L} \times \frac{\sqrt{4\Delta^2 + L^2} - L}{\sqrt{4\Delta^2 + L^2} + L_T - L}$$

$$P_b = \frac{48EI\Delta}{L^3}$$

$$P_{ps} = \frac{4F_{ps}\Delta}{L}$$



Extruded Aluminum Post Input		
Post Spacing, s =	5 ft	
Prestress Force, F_{ps} =	255 lbs	
Initial Cable Spacing, S_o =	3.125 in	
Unbraced Length = 42" - 1" - 3/8", L_B =	40.625 in	
Distributed Load, w =	81.6 lb-in	
Post Area, A_p =	1.529 in²	
Compressive Modulus of Elasticity, E =	10100 ksi	
Compression Section Modulus, S_C =	0.836 in³	
Moment of Inertia, $I_x > I_y$ =	0.940 in⁴	
Torsion Constant, J =	0.260 in	
Clear Height of Shear Area, h =	2.250 in	
Thickness of Shear Area, t =	0.250 in	
Moment From Code Point Load, M_{pnt} =	11.375 kip-in	*(Page 4)*
Moment From Cable Prestress, M_{pstr} =	16.834 kip-in	◄ **Controls TP100 Design**

Flat Elements Supported on Both Ends in Uniform Compression, 6061-T6 (ADM 2010 Section B5.4.2, Table 2-19 Part VI)		
Slenderness, S =	16.0	
Allowable Stress, $S \le S_1$ =	21.2 ksi	◄ Controls
Slenderness Limit, S_1 =	21	
Allowable Stress, $S_1 < S < S_2$ =	22.6 ksi	
Slenderness Limit, S_2 =	33	
Allowable Stress, $S \ge S_2$ =	36.3 ksi	
Allowable Bending Stress, F_b =	21.2 ksi	
Allowable Moment, Sc*Fb = M_{allow} =	**17.715 kip-in**	
Applied Moment, $w{*}L_B^2/8$ = $M_{applied}$ =	**16.834 kip-in**	**OK**

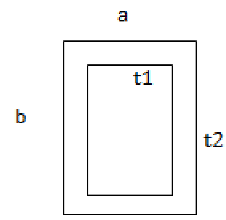
Flexural Compression Closed Shapes Lateral Torsional Buckling 6061-T6 (ADM 2010 Section F3.1, Table 2-19 Part VI)		
Slenderness, S =	137.4	
Allowable Stress, $S \le S_1$ =	7.63 ksi	
Slenderness Limit, S_1 =	55	
Allowable Stress, $S_1 < S < S_2$ =	21.110 ksi	◄ Controls
Slenderness Limit, S_2 =	1685	
Allowable Stress, $S \ge S_2$ =	4.609 ksi	
Allowable Bending Stress, F_a =	**21.1 ksi**	
Applied Bending Stress, F_b =	**20.145 ksi**	**OK**

Deflection Check, Δ_{MAX} = $L_B/12$ (ICC Report AC273)		
Allowable Deflection, Δ_{allow} =	3.385 in	
Applied Deflection, $PL_B^3/3EI$ = $\Delta_{applied}$ =	0.600 in	**OK**

Shear in Elements, Gross Section 6061-T6 (ADM 2010 Table Section G.2, Table 2-19 Part VI)		
Allowable Stress, $S \le S_1$ =	12.7 ksi	◄ Controls
Slenderness Limit, S_1 =	35.3	
Allowable Stress, $S_1 < S < S_2$ =	14.8 ksi	
Slenderness Limit, S_2 =	63.0	
Allowable Stress, $S \ge S_2$ =	151.0 ksi	
Allowable Shear Stress, F_s =	13 ksi	
Allowable Shear, Ap*Fs = V_{allow} =	**19.418 kips**	
Applied Shear, P = $V_{applied}$ =	**1.658 kips**	**OK**

Extruded Aluminum Stair Handrail Connection Input

Post Spacing, s =	5 ft	t1 =	0.125 in
Force, F =	250 lbs	t2 =	0.125 in
Moment Arm =	2.750 in	a =	0.625 in
Initial Cable Spacing, S_o =	0.000 in	b =	1.750 in
Unbraced Length, L_B =	1.600 in	J =	0.078 in^4
Post Area, A_p =	0.531 in^2	Sx =	0.178 in^3
Compressive Modulus of Elasticity, E =	10100 ksi	Sy =	0.079 in^3
Compression Section Modulus, Sy =	0.079 in^3	Ix =	0.100 in^4
Moment of Inertia, Iy =	0.019 in^4	Iy =	0.019 in^4
Torsion Constant, J =	0.078 in^4	Area =	0.531 in^2
Moment From Railing, M =	0.688 kip-in	wall/thick =	13.000



Flat Elements Supported on Both Ends in Uniform Compression, 6061-T6 (ADM 2010 Section B5.4.2, Table 2-19 Part VI)

Slenderness, S =	12.0	
Allowable Stress, S≤S_1 =	21.2 ksi	◄ Controls
Slenderness Limit, S_1 =	21	
Allowable Stress, S_1<S<S_2 =	23.8 ksi	
Slenderness Limit, S_2 =	33	
Allowable Stress, S≥S_2 =	48.3 ksi	
Allowable Bending Stress, F_b =	21.2 ksi	
Allowable Moment, Sc*F_b = M_{allow} =	**1.670 kip-in**	
Applied Moment, $M_{applied}$ =	**0.688 kip-in**	**OK**

Deflection Check, Δ_{MAX} = L_B/12 (ICC Report AC273)

Allowable Deflection, Δ_{allow} =	0.133 in	
Applied Deflection, $PL_B^3/3EI$ = $\Delta_{applied}$ =	0.002 in	**OK**

Shear in Elements, Gross Section 6061-T6 (ADM 2010 Table Section G.2, Table 2-19 Part VI)

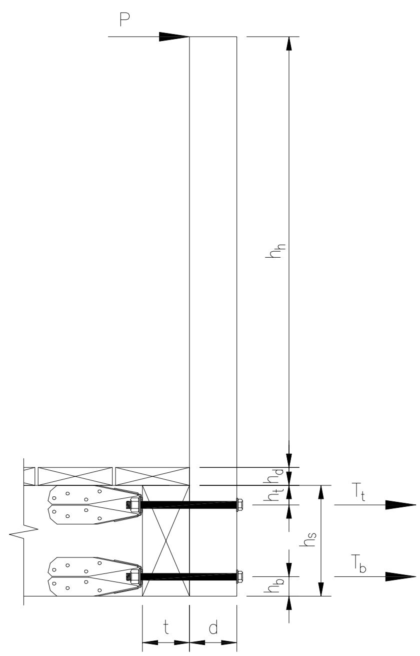
Allowable Stress, S≤S_1 =	12.7 ksi	◄ Controls
Slenderness Limit, S_1 =	35.3	
Allowable Stress, S_1<S<S_2 =	15.2 ksi	
Slenderness Limit, S_2 =	63.0	
Allowable Stress, S≥S_2 =	268.5 ksi	
Allowable Shear Stress, F_s =	13 ksi	
Allowable Shear, Ap*F_s = V_{allow} =	**6.747 kips**	
Applied Shear, P = $V_{applied}$ =	**0.250 kips**	**OK**

Fillet Weld Strength (ADM 2005 Section 7.3.2)

Filler Shear Ultimate Strength, Fsw =	17 ksi	**◄ Controls**
Base Metal Strength, Fswb =	21 ksi	
Weld Fillet Weld size, Se =	0.188 in	
Weld Section Modulus, S =	0.271 in^3	
Factor of Safety, nu =	1.95	
Allowable Weld Strength, Fsw/nu =	**6.164 ksi**	
Applied Weld Stress, M/S = fw =	**2.537 ksi**	**OK**

Shear Capacity of Screw (ADM 2010 J5.6)

Tensile Ultimate Strength of Member in Contact with screw head, Ftu1 =	38 ksi	
Thick. of Member in Contact with Screw, t1 =	0.065 in	
Tensile Ultimate Strength of Member Not in Contact with screw head, Ftu2 =	38 ksi	
Thick. of Member Not in Contact with Screw, t2 =	0.140 in	
Nominal Shear Strength of Screw, Pns =	2.363 kips	
Screw Diameter, D =	0.190 in	
No. of Screws, n =	2	
Safety Factor, ns =	3.00	
Safety Factor, nu =	1.95	
Allowable Shear Strength, Va =	**0.963 kips**	
Applied Shear, v =	**0.250 kips**	**OK**



Extruded Aluminum Post Facia Mount Input

Anchor Design Criteria

(See Page 3) Applied Load At Top, P =	250	lbs
Height of handrail point load, h_h =	42	in
Height of decking, h_d =	1.5	in
Height of beam, h_s =	9.25	in
Height of top anchor, h_t =	2	in
Height of bot anchor, h_b =	1.25	in
Anchor bolt spacing, s =	6	in
Unbraced Post Length, L_b =	45.5	in
Lumber species =	DF	
Thickness of joist =	3	in
Simpson Holdown =	DTT2Z	
Tension allowed, $T_{allowed}$ =	2145	lbs

Extruded Aluminum Post Facia Mount Output

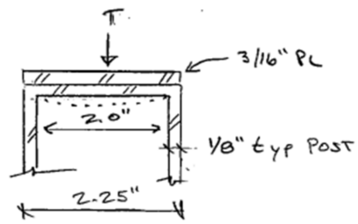
Anchor Calcs

Tension applied per top bolt, T_t =	1819	lbs
Tension applied per bottom bolt, T_b =	1569	lbs
Tension max =	**1819 lbs**	**OK**

Distribute Force 'T' over 3" Trib

$T/3 = 1.85^k/3 = 0.62^k$

Bending Load, $M_a = \frac{P\lambda}{4} = \frac{0.62(2)}{4} = 0.31^{k''}$



FLEXURAL YIELDING/RUPTURE, M_n: (ADM Sect F.2)

$M_n = Z F_{cy} = \left[\tfrac{1}{4}(3.0)(\tfrac{3}{16})^2\right] \times 35\,ksi = 0.923^{k''}$

$M_n = 1.5 S_t F_y = 1.5(\tfrac{1}{6} \times 3 \times \tfrac{3}{16}^2) \times 35\,ksi = 0.923^{k''}$

$M_n = 1.5 S_c F_{cy} = 1.5(\tfrac{1}{6} \times 3 \times \tfrac{3}{16}^2) \times 35\,ksi = 0.923^{k''}$

$M_n = Z F_{tu}/K_t = (\tfrac{1}{4} \times 3 \times \tfrac{3}{16}^2) \times 38\,ksi / 1.0 = 1.00^{k''}$

$M_n/\Omega_b = \begin{cases} 0.923/1.65 = 0.568^{k''} \\ 1.00/1.95 = 0.514^{k''} \end{cases}$

LTB, M_n: (ADM Sect F.4)

$\lambda = \frac{2.3}{t}\left(\frac{d\,L_b}{C_b}\right)^{1/2} = \frac{2.3}{3/16}\left(\frac{3(2)}{1.0}\right)^{1/2} = 30.5 < C_c = 65.7$

$M_n\left(1 - \frac{\lambda}{C_c}\right) + \frac{\pi^2 E \lambda S}{C_c^3} = 0.923\left(1 - \frac{30.5}{65.7}\right) + \frac{\pi^2 (10{,}100)(30.5)(\tfrac{1}{6} \times 3 \times \tfrac{3}{16}^2)}{(65.7)^3}$

$= 0.502 + 0.189$

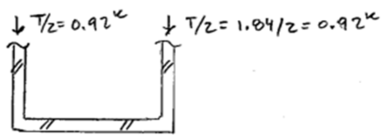
$= 0.691^{k''}$

$M_n/\Omega_b = 0.691/1.65 = 0.418^{k''}$ [govs]

$DCR = M_a/(M_n/\Omega_b) = 0.31^{k''}/0.418^{k''} = 0.74 < 1.0 \therefore$ ✓ ok

Compression on Post Elements

Distribute force 'T' equally to each side



Compression per Member Buckling (ADM Sect E.2)

$\lambda_1 = \frac{B_c - F_{cy}}{D_c} = \frac{39.4-35}{0.246} = 17.9$; $\lambda_2 = 65.7$

$\lambda = k\lambda/r = 0.5(2.25-2(0.125))/[1/8\sqrt{1/12}] = 27.7$ (E.2.1)

$F_c = (B_c - D_c\lambda)\left[0.85 - 0.15\left(\frac{C_c-\lambda}{C_c-\lambda_1}\right)\right] = (39.4-0.246(65.7))\left[0.85-0.15\left(\frac{65.7-27.7}{65.7-17.9}\right)\right]$

$= (23.24)[0.73]$

$= 16.96$ ksi

$P_{nc} = F_c A_g = 16.96(1/8 \times 3) = 6.36^k$

$P_{nc}/\Omega = 6.36/1.65 = 3.85^k > 0.92^k$ ✓ ok

Check Torsional and Flexural-Torsional Buckling (E2.2)

$\lambda = \pi\sqrt{E/F_e} = \pi\sqrt{10{,}100/204} = 22.1 < k\lambda/r$ ∴ use above results

$F_e = \left(\frac{\pi^2 E C_w}{(K_z L_z)^2} + GJ\right)\frac{1}{I_x+I_y} = 204$ ksi

$E = 10{,}100$ ksi

$C_w = \frac{(1/8)^3}{36}\left[(2.25-1/8)^3 + (1/8 \times 1/8)^3\right] = 0.000521$ in^6

$K_z = 0.5$

$L_z = 2''$

$G = 3800$ ksi

$J \cong \beta a b^3 = 1/3(2.25)(1/8)^3 = 0.00146$ in^4

$I_x = 1/12(1/8)^3(3) = 0.000366$ in^4

$I_y = 1/12(3.0)^3(1/8) = 0.281$ in^4

Check Local Buckling (E.3)

F_c: $\lambda_{eq} = 14.34 < \lambda_1$ (B.5.4.6)

$\lambda_1 = (B_p - F_{cy})/D_p = (45-35)/0.246 = 40.7$

$\lambda_2 = K_1 B_p / D_p = 0.35(45)/0.246 = 64.0$

∴ $F_c = F_{cy} = 35$ ksi ∴ F_c above govs

Check Buckling btn Member Buckling and Local Buckling (E.4)

No interaction b/c $F_e > F_c$