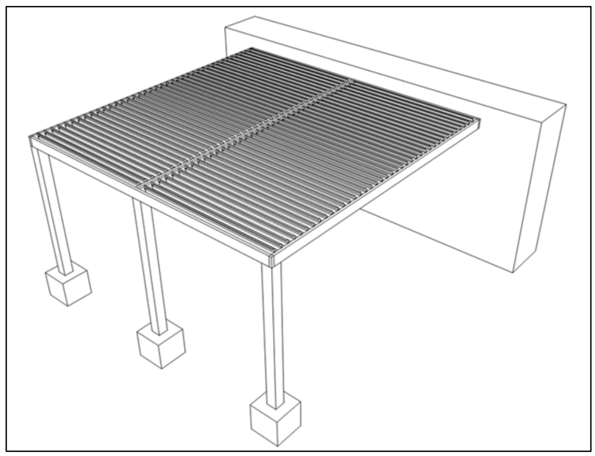
Calculation Booklet

Engineering Express Project 22-52551, Shane McArthur

Scope of Work:	Structural Design & Installation Of 1 Residential, Host Attached Pergola. Includes Calculaiton Of Loading, Members, Connections, Foundations, And Connection To Existing Host Structures As Required.
Project Information	22-52551
Project Address:	Shane McArthur 8609 SE 78th St Mercer Island, WA 98040
Design of:	At Grade, Residential, Host Attached Pergola With Mechanically Operated Louvered Roof
Prepared For:	StruXure Outdoor of Washington 9116 E Sprague Ave #547 Spokane, WA 99206 509-928-0880

General Notes:

This calculation package is to be submitted for permit alongside a set of certified drawings and details which bears the same project name, number, address, and certifying Professional Engineer as shown in the certification below. Any project notes, details, or design information in that drawing set shall also apply to this report (in the case of any uncertainty, the more stringent information shall apply). This structure shall be built in conformance with any building codes referenced on that drawing set, as well as any local building codes required for the project address. This document shall not be used or reproduced without the original signature & raised seal of the certifying P.E. Alterations, additions or other markings to this document are not permitted and invalidate our certification. Photocopies and unsealed documents are not to be accepted. Except as expressly provided herein, no additional cetifications or affirmations are intedned.



Project Designer: MD
Project Reviewer: RS
Sealing Engineer: Frank Bennardo PE

For Additional Information, Scan the QR Code here:

Engineer's Seal Below Valid For Pages 1 Through 46

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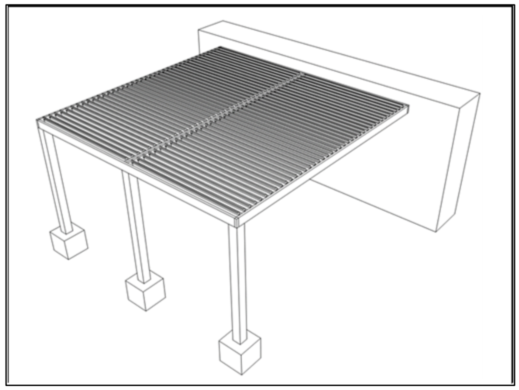
Frank Bennardo PE
PE# 56089
CA# 4018

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Design Overview Of: **Project Overview**



Structure Layout

Total Width	20.00 ft
Total Length	20.00 ft
Mean Roof Height	11.00 ft
Structure Support	Host Attached
Roof Style	Louvers
Roof Slope	0.0 / 12

Design Criteria (Detailed Calculations On Following Pages)

Loading Inputs

Dead Load	5.0 psf
Design Live Load	16.0 psf
Risk Category	II
Ultimate Wind Speed	110 mph
Exposure Category	D
HVHZ	NON-HVHZ
Wind Flow	Clear
Ground Snow Load	30.0 psf
Unredicible Snow Load?	FALSE
Design Snow Load	33.9 psf
Nominal Ice Thickness	1.00 in
Seismic Site Class	D (DEFAULT)
Response Acceleration, Ss	1.5 s
Response Acceleration, S_1	0.5 s
Seismic Site Category	D
TL	6 s
Total Effective Seismic Design Force, Fp	1871.4 lbs

ASD Design Load Combinations

Per ASCE 7-16, Ch 2.4

Components & Cladding

Gravity	38.9 psf	D + S
Uplift	-10.0 psf	Min Requirement
Lateral	15.9 psf	D + 0.6 W

Main Wind Force

Gravity	38.9 psf	D + S
Uplift	-13.5 psf	0.6 D + 0.6 W
Lateral	15.9 psf	D + 0.6 W

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Design Overview Of:	**Roof And Beam Design Overview**

<u>Roof Design -</u> **Louvers**

Max Louver Span	10.00 ft
Aluminum Alloy:	6063-T6
Louver Width	5.087 in
Louver Height	5.006 in
Louver Spacing	8 in

Strength Capacity % =	32%
Deflection Capacity =	44%

Louvers To Be Rotated To Open Position
During Named Wind Event (75 MPH+)

<u>Structural Beam Designs</u> - (Critical Members Shown)

Main Beam #1 Design
(⊥ Roof Member Span)

Beam #1 Material	6063-T6
Beam #1 Max Span	19.50 ft
Beam #1 Overhang L	0.00 ft
Beam #1 Overhang R	0.00 ft
Beam Width	2.0 in
Beam Height	8.0 in
Beam Thickness	0.250 in
# | | Beams in Section	1
Beam #1 Sx	8.150 in³
Beam Location	Edge
Beam #1 - # Spans	1

Strength Capacity % =	100%
Deflection Capacity =	73%

Main Beam #2 Design
(| | Roof Member Span)

Beam #2 Material	6063-T6
Beam #2 Max Span	10.00 ft
Beam #2 Overhang L	0.00 ft
Beam #2 Overhang R	0.00 ft
Beam Width	2.0 in
Beam Height	8.0 in
Beam Thickness	0.250 in
# | | Beams in Section	1
Beam #2 Sx	8.150 in³
1st Intermediate Beam #1 Offset "a"	0.00 ft
2nd Intermediate Beam #1 Offset "b"	0.00 ft
Beam Location	Edge
Beam #2 - # Spans	2

Strength Capacity % =	6%
Deflection Capacity =	0%

Work Prepared For: StruXure Outdoor of Washington
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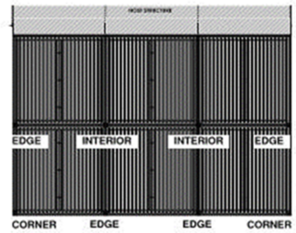
Design Overview Of: **Post & Connection Design**

Post Design (Critical Post Shown)

Post Material	6063-T6
Post Location	Edge
Post Height	11.00 ft
Post Width	8.0 in
Post Depth	8.0 in
Post Thickness	0.188 in
Post #1 Sx	14.910 in³
Fascia Height	8.0 in



Tributary Width	9.75 ft
Tributary Length	10.00 ft
Strength Capacity % =	32%
Deflection Capacity =	12%

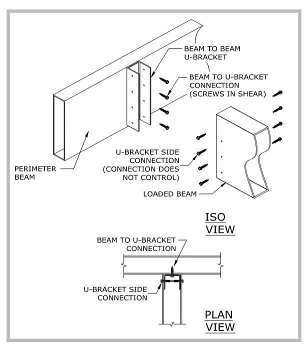
Reactions On Foundation

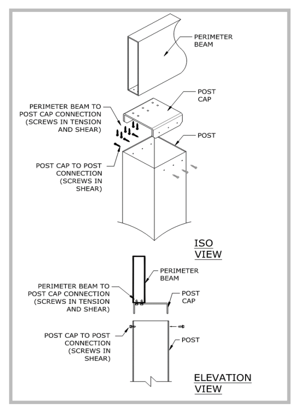
Gravity / Compression =	3.79 Kip
Uplift / Tension =	-1.32 Kip
Lateral / Shear =	0.54 Kip
Bending / Moment =	4.1 Kip-ft

Connection Design

Loaded Beam To Perimeter Beam

Total # Screws	6
Screw Type	-14 SMS, 316 SS
Tensile Strength	2985 lb
Shear Strength	2235 lb
Connection Interaction =	94%





Perimeter Beam to Post

Connection Orientation	Beam On Top Of Post
# Screws - Beam To Clip	6
# Screws - Clip To Post	6
Screw Type	#12-14 SMS, 316 SS

<u>Beam To Post Clip</u>

Tensile Strength	2985 lb
Shear Strength	2235 lb
Connection Interaction =	68%

<u>Post Clip To Post</u>

Tensile Strength	2985 lb
Shear Strength	2235 lb
Connection Interaction =	59%

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Design Overview Of: **Foundation and Anchorage Design**

Concrete Foundation Design & Reactions

Footing Type	Isolated Footing
4" Slab Over Footing?	FALSE
Footing Lendth	36.0 in
Footing Width	36.0 in
Footing Depth	30.0 in
Footing Name	36" x 36" x 30" Isolated Footing
Required Reinforcement	(4) #5, Each Way, Top & Bottom

Footing Design Capacities

Uplift Capcity % =	39%
Sliding Capacity % =	45%
Overturning Capacity (X) % =	96%
Overturning Capacity (Y) % =	69%
Bearing Pressure Capacity =	50%

Baseplate Design

Post Attachment	Bolted Baseplate
Baseplate Length	12.0 in
Baseplate Width	12.0 in
Baseplate Thickness	0.250 in

Anchorage To Concrete -	3/8" Dia, Has Threaded Rods With Hy-200 Epoxy @ 4.5" Embed
Anchor Diameter	0.375 in
Anchor Embedment	4.50 in
Design Tension Strength	7,431 lbs
Design Shear Strength	22,954 lbs
Strength Capacity % =	84%

NOT USED

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Design Loading from Structure Classificaition & Wind**

Loading Design Criteria:

Design Standard:	ASCE 7-16	
Risk Category:	II	
Overall Width or Projection X, W =	20.00 ft	
Overall Length Y, L =	20.00 ft	
Total Area, A =	400.0 ft²	
Installaton Elevation =	0.00 ft	
Structure Height =	11.00 ft	
Mean Roof height, h =	11.00 ft	
Roof Slope, Θ =	0.00 º	(0" Per 12" of Slope)
Structure Type =	Host Attached	

Dead and Live Loading:

Design Dead Load:	**5.0 psf**
Design Roof Live Load:	20.00 psf

(Not-Occupiable Ordinary Flat, Pitched, and Curved Roofs)

Live Load Reduction For Ordinary Roofs, Awnings, And Canopies (Per IBC 1607.13.2.1)

$L_{reduced} = L_{design} * R_1 * R_2$

Reduction for Large Area, R_1 =	0.80
Reduction for Large Slope, R_2 =	1.00
Reduced Roof Live Load, L_R =	**16.00 psf**

Wind Design Conditions:

Ultimate Wind Velocity, Vult =	110 mph	(3-Second Gust)		
Exposure Category:	D			
Wind Flow Through Structure:	Clear			
Roof Wind Porosity:	50%	(0% = Solid)	Roof Type:	Louvers
Wall Wind Porosity:	100%		Wall Type:	Open Walls

Directionality Factor, Kd =	0.85
Gust Effect Factor, G =	0.85
Velocity Pressure Coefficient, Kz =	0.98
Topographic Factor, Kzt =	1
Velocity Pressure, q_z =	**25.70 psf**

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Design Loading from Structure Classificaition & Wind**

Gravity & Uplift Loads on Components & Cladding for Structure Support, Open Structures

(Per ASCE 7-16 Chapter 30.11)

Note: Loading Not Applicable For Components And Cladding On Enclosed Structures

Effective Component Length, L_1 = 10.00 ft

Roof Component Considered: Louver Blade

Effective Component Width, W_1 = 0.42 ft

Effective Wind Area, A_e = 4.24 ft^2

Least Horizontal Dimension, a = 3.00 ft

Host Structure Eave Height, he = 26.00 ft

A ≤ a^2

Positive Pressure Coefficient, CN_p = 0.6

Negative Pressure Coefficient, CN_n = -0.5

Velocity Pressure With Roof Porosity, q_z = 12.85 psf

C&C Gravity Wind Load, WL_p = 6.55 psf = qz * G * CNp

C&C Uplift Wind Load, WL_n = -5.24 psf = qz * G * CNn

Gravity & Uplift Loads On Monoslope, Free Roof Main Wind Force Resisting System:

(Per ASCE 7-16 Chapter 27.3-4 & 27.3-7 - MWFRS Directional Methodology)

Wind Direction, γ = 0º

Windward Coefficient, Load Case A, C_{NWa} = 1.2

Windward Coefficient, Load Case B, C_{NWb} = -1.1

Leeward Coefficient, Load Case A, C_{NLa} = 0.3

Leeward Coefficient, Load Case B, C_{NLb} = -0.1

Wind Direction, γ = 180º

CNWa = 1.2

CNWb = -1.1

CNLa = 0.3

CNLb = -0.1

Wind Direction, γ = 90º (Critical Values at Windward Fascia)

Windward Coefficient, Load Case A, C_{Na} = -0.8

Load Case B, C_{Nb} = 0.8

Gravity & Uplift Loads On Monoslope, Host Attached Main Wind Force Resisting System:

(Per ASCE 7-16 Chapter 30.11- MWFRS Methodology)

Effective Wind Area, A_{EF} = 400 ft²

h_c / h_e = 0.42

+ Coefficient, GC_{pn+} = 0.6

- Coefficient, GC_{pn-} = -0.5

Critical Positive Coefficient, C_{Np} = 0.6

Critical Negative Coefficient, C_{Nn} = -0.5

Roof Drag Factor (Lateral Pressures)		
Flat Roof	**Trellis**	**Open Louvers**
1.0	1.1	1.25

MWFRS Gravity Wind Load, WL_p = 6.55 psf = qz * Roof Porosity * G * CNp

MWFRS Uplift Wind Load, WL_n = -5.24 psf = qz * Roof Porosity * G * CNn

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Design Loading from Structure Classificaition & Wind**

Lateral Wind Loads on Open or Partially Enclosed Buildings with Transverse Frames and Pitched Roofs
(ASCE 7-16 MWFRS - Ch 28.3.5)

For Open Structures, The Following Lateral Pressure Equation Shall Apply:

$$p_{open\ walls} = q_h\ [(GC_{pf})_{Windward} - (GC_{pf})_{Leeward}] * K_B * K_S * \text{Roof Drag Factor}$$

Where The Gcpf Values Are The Average Of The Load Case B Values For The Edge And Wall Conditions:

$GC_{pf\ Windward}$ =	0.463	
$GC_{pf\ Leeward}$ =	-0.332	
Building Width, B =	20.00 ft	
K_B = Frame Width Factor =	1.600	(= 1.8 - 0.01B) (Minimum 0.8)
Effective Solid Area, A_S =	35.3 ft²	Open Walls
Total End Wall Area, A_E =	220.0 ft²	
Solidity Ratio, ϕ =	0.161	(= A_S / A_E)
K_S = Shielding Factor =	0.646	(=0.6+ 0.073*(# Frames(min 3) - 3) + (1.25* ϕ^ 1.8))
Roof Drag Factor	1.25	
Open Frame Lateral Pressure, p =	26.42 psf	

Roof Drag Factor		
Flat Roof	**Trellis**	**Open Louvers**
1.00	1.1	1.25

MWFRS Gravity, Uplift, & Lateral Pressures For Enclosed And Partially Enclosed Low Rise Structures & Host Atachment Directions
(Per ASCE 7-16 CH 28.3.1 - MWFRS Envelope Methodology)

Enclosue Classification	Open Building	(Host Attached Flow)
External Coefficient, GCpf =	See Below	(ASCE 7-16 Figure 28.3-1)
Internal Coefficient, GCpi =	± 0.00	(ASCE 7-16 Table 26.13-1)
Lateral Roof Drag Factor	1.25	

Critical GCpf Values Per Load Case & Surface Location

	Max GCpf - Windward		Min GCpf - Leeward	
	Roof	Wall	Roof	Wall
Load Case A	-0.37	0.40	-0.69	-0.29
Load Case A (Edge)	-0.53	0.61	-1.07	-0.43
Load Case B	-0.37	0.40	-0.69	-0.45
Load Case B (Edge)	-0.53	0.61	-1.07	-0.48

Applied Wind Pressure, p = qz * (GCpf - GCpi)

Envelope Gravity Load, WLep =	**0.00 psf**	= qz*G*(Cpf - Cpi) (Max +)*
Envelope Uplift Load, WLnp =	**-27.50 psf**	= qz*G*(Cpf - Cpi) (Min -)
Envelope Lateral Load, WL_L =	**19.60 psf**	= qz*G*(Cpf - Cpi) (Max ±)

*(Envelope Procedure Results in Only Uplift On Windward And Leeward Roof Surfaces When Slope is Low)

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Snow Loading**

Calculation of Design Snow Loading

Structure Type =	Host Attached	
Ground Snow Load, Pg =	30.0 psf	
Snow Loading Unreducible Per Local Codes?	FALSE	
Exposure Factor , Ce =	1.0	Partially Exposed
Thermal Factor, Ct =	1.2	Unheated & Open Air Structure
Importance factor, Is =	1.0	Risk Category II
Roof Slope =	0.00 °	Flat Roof (Slope < 5°)
Width (From Eave To Ridge), W =	20.0 ft	
Roof Style =	Louvers	
Roof Snow Porosity =	0%	
Snow Density, γ =	17.90 pcf	= 0.13* Pg +14 < 30 psf
Slope Factor, Cs =	1.00	(Figure 7.4-1)

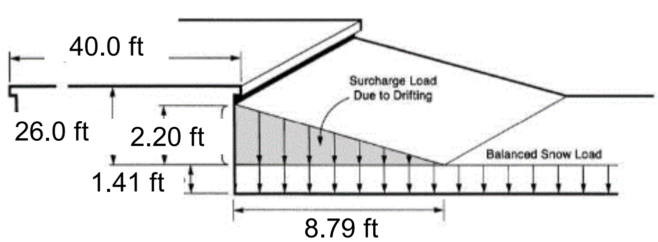
Balanced Snow Loads

Snow Load On Flat Roof (Slope < 5°), P_f =	**25.2 psf**	= Max(I *20),(0.7 *Ce *Ct* I* Pg),(5)
Snow Load On Sloped Roof (Slope < 5°), P_s =	**25.2 psf**	= Cs * Pf

Rain-On-Snow Surcharge Required? (Ch 7.10)	FALSE	
	0.00 psf	

Drifts on Lower Roofs (Aerodynamic Shade)

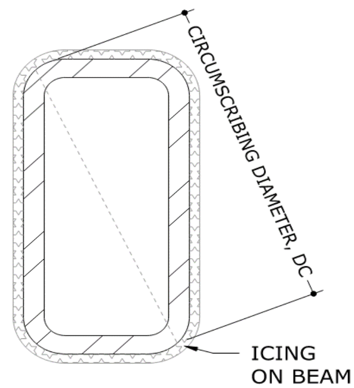
Include Surcharge Due To Drift Loading? (Structure Shall Experience Snow Drift)	TRUE	
Assumed Length Of Upper Roof, lu1 =	40.0 ft	
Attached Structure Total Projection X, lu2 =	20.0 ft	
Height From Top Of Lower Roof To Top Of Eave, hc =	26.0 ft	
Height of Balanced Snow, h_b =	1.41 ft	= Pf / γ
Height Of Leeward Snow Drift, h_{d1} =	2.20 ft	= $0.43 * lu^{1/3} * (Pg + 10)^{1/4} - 1.5$
Height Of Windward Snow Drift, h_{d2} =	1.08 ft	= $0.43 * lu^{1/3} * (Pg + 10)^{1/4} - 1.5$
Governing Drift Height, h_d =	2.20 ft	
Governing Drift Width, W =	8.79 ft	
Drift Height At Edge Of Lower Roof, h_{end} =	0.00 ft	
Surcharge Load Distributed Over Drift Width, p_d =	19.67 psf	
Surcharge Load Distributed Over Tributary Area, p_d =	**8.65 psf**	
Design Snow Load, S =	**33.9 psf**	**= Balanced Load + Distributed Drift Surcharge**



Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Ice Loading Calculations**

Ice Load Due to Freezing Rain (per ASCE 7-16 - Chapter 10)
Acounting for Accumulating Ice on Louver Blades

Nominal Ice Thickness, t_i =	1.00 in	
Risk Category =	II	
Topographic Factor, K_{zt} =	1.0	
System Height, Z =	11.00 ft	
Importance Factor for Icing, I_i =	1.00	
Ice Density, I_d =	56.0 pcf	(56 pcf default)
Snow Density, g =	17.90	



Member Properties

	Louver Blade	Louver Beam
Depth, d =	5.0 in	8.0 in
Width, bf =	5.1 in	2.0 in
Length, l =	10.00 ft	19.50 ft
Spacing, s =	8.0 in O.C.	

Ice Thickness Increasing Factor, F_z =	0.8960	$= (Z/33)^{0.1}$
Design Ice Thickness, t_d =	0.90	$= t_i * I_i * f_z * (K_{zt})^{0.35}$
Weight of Ice (per td), W_i =	4.18 psf	$= (td / 12) * I_d$

Ice Loading on Individual Members

Louver Blade Ice Loading (Single Member)

Circumscribing Diameter Of Member, D_{c1} =	7.14 in	$= \sqrt{d^2+bf^2}$
Area of Ice, A_{i1} =	22.61 in^2	$= \pi * t_d * (D_c + t_d)$
Uniform Distributed Ice Load, W_{i1} =	8.79 plf	$= A_i * I_d$

Louver Beam Ice Loading

Circumscribing Diameter Of Member, D_{cBeam} =	8.25 in	$= \sqrt{d^2+bf^2}$
Area of Ice, A_{iBeam} =	25.73 in	$= \pi * t_d * (D_c + t_d)$
Uniform Distributed Ice Load, W_{iBeam} =	10.01 plf	$= A_i * I_d$

Louver Blade Ice Loading Acting On Louver Beam

Ice Load On First Single Member, W_{i1} =	8.79 plf	
Tributary Width of Louver Blade, Trib =	10.00 ft	
Additional Ice Load on Beam, $W_{i(Beam)}$=	11.0 plf	$= W_{i1}$ * Trib / Spacing

$\mathbf{W_{i(Louver)}}$ =	**8.79 plf**	Uniform Linear Ice Load (Louver Blade)
$\mathbf{W_{i(Beam)}}$ =	**10.01 plf**	Uniform Linear Ice Load (Ice on Beam Only)
$\mathbf{W_{i(Beam\ Total)}}$ =	**21.00 plf**	Total Additional Loading On Beam

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Seismic Design Criteria & Loading**

Seismic Design Criteria

Max Considered Response Acceleration For 0.2 S, S_s =	1.462	
Max Response Acceleration At 1 S, S_1 =	0.504	
Overall Width or Projection X, W =	20.00 ft	
Overall Length Y, L =	20.00 ft	
Total Area, A =	400.0 ft²	
Height of Structure, H =	11.00 ft	
Attached to Host Structure?	TRUE	
Laterally Supported by Host in Both Directions?	FALSE	
Structure Dead Load =	5 psf	
Ground Snow Load =	30 psf	≤ 30 PSF - Not Considered in Seismic
Site Class =	D	
Short Period Amplification Factor, F_a =	1.2	
Long Period Amplification Factor, F_v =	1.5	
Modified Spectral Response Acceleration At 0.2 S, S_{MS} =	1.754	F_a*S_s
Modified Spectral Response Acceleration At 1.0 S, S_{M1} =	0.756	F_v*S_1

Spectral Response Acceleration Parameters

Design Spectral Response Acceleration At 0.2 S, S_{DS} =	1.170	(2/3)*S_{ms}
Design Spectral Response Acceleration At 1.0 S, S_{D1} =	0.504	(2/3)*S_{M1}

Structural Design Requirements

Approximate Fundamental Period (s), T_a =	0.121 s	C_t*h_n^x
Geographic Long Transition Period (s), T_L =	6 s	
Vertical Seismic Load Effect, E_V =	**0.82 psf**	Vertical Seismic Loads (PSF)
Response Modification Coefficient, Rp =	2.50	Structure Directly Supported by Host
Overstrength Factor, Ω =	2.00	Host Attached
Amplification Factor, ap =	2.500	
Min Seismic Response Coefficient, CS Min =	0.101	
Component Importance Factor, Ip =	1.00	
Seismic Importance Factor, Ie =	1.00	
Tributary Weight with Additional Snow Load, Wp =	2000 lb	Tributary Weight
Total Effective Seismic Design Force, Fp =	**1871 lb**	= 0.4* ap* SDS* Wp/ (Rp / Ip)* (1+ 2 (z/ h))
FpMAX=	3742.72 lbs	
ASD Service Factor =	0.7	
Redundancy Factor, ρ =	1.0	
Total Effective Seismic Moment, M_{SEIS} =	**14409 lb-ft**	= V * H
Loading from Horizaontal Seismic Forces, Q_E =	4.68 psf	= V / A
Horizontal Siesmic Load Effect, E_h =	**4.68 psf**	= QE * ρ (Eq. 12.4-3)

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **ASD Loading Combinations per ASCE 7-16, Chapter 2.4**
Formatted For Use With Freestanding or Host Attached Pergolas

Unfactored, Calculated, or Provided Loads

Loading From Structure			
Dead Load	5.0 psf	$D =$	5.0 psf
Live Load	0.0 psf	$L =$	0.0 psf
Reduced Roof Live Load	16.0 psf	$L_R =$	16.0 psf

Loading From Wind

Components & Cladding			
Gravity (+)	6.6 psf	$W_{CC+} =$	6.6 psf
Uplift (-)	-5.2 psf	$W_{CC-} =$	-5.2 psf
Main Wind Force Resisting System			
Gravity (+)	6.6 psf	$W_{MWF+} =$	6.6 psf
Uplift (-)	-27.5 psf	$W_{MWF-} =$	-27.5 psf
Lateral Force			
On Fascia & Roof Drag	26.4 psf	$W_{LAT\ FAC} =$	26.4 psf
On Walls & Posts	26.4 psf	$W_{LAT\ WALL} =$	26.4 psf

Loading from Snow

Ground Snow Load	30.0 psf		
Flat Roof Snow Load	25.2 psf	$p_f =$	25.2 psf
Sloped Roof Snow Load	25.2 psf	$p_s =$	25.2 psf
Unreducible Snow Load	33.9 psf		
Design Snow Load	33.9 psf	$S =$	33.9 psf

Loading from Icing

Area Ice Loading	8.8 psf	$D_i =$	8.8 psf
Reduced Wind Forces due to Ice Load			
Components & Cladding			
Gravity (+)	1.9 psf	$W_{CCice+} =$	1.9 psf
Uplift (-)	-1.6 psf	$W_{CCice-} =$	-1.6 psf
Main Wind Force Resisting System			
Gravity (+)	1.9 psf	$W_{MWFice+} =$	1.9 psf
Uplift (-)	-8.2 psf	$W_{MWFice-} =$	-8.2 psf
Lateral Force			
On Fascia	7.9 psf	$W_{iLAT} =$	7.9 psf
On Walls	7.9 psf	$W_{LAT\ WALL} =$	7.9 psf

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**ASD Loading Combinations per ASCE 7-16, Chapter 2.4**

Loading from Rain, Flood, and Additional Design Conditions

Rain Load	0.0 psf	R =	0.0 psf
Static Fluid Load	0.0 psf	F =	0.0 psf
Flood Risk (2.4.2)	Low	Factor	0
Flood Load	0.0 psf	F_a =	0.0 psf
Lateral Earth Pressure Load	0.0 psf		
LatEPr Adds or Resists?	Adds	H =	0.0 psf
Self-Straining Force	0.0 psf	T =	0.0 psf

Loading from Seismic Forces

Vertical Seismic Load	0.8 psf	E_v =	0.8 psf
Horizontal Seismic Load	4.7 psf	E_h =	4.7 psf

Allowable Stress Design (ASD) Load Combinations Per ASCE 7-16 Ch 2.4

Critical Design Load Combinations for Components & Cladding and Main Wind Force Resisting System:

Gravity Components & Cladding	**38.85 psf**	EQ # 3b.	D + S
Uplift Components & Cladding	**-10.00 psf**	EQ # 11 Min.	Min Requirement
Gravity Main Wind Force	**38.85 psf**	EQ # 3b.	D + S
Uplift Main Wind Force	**-13.50 psf**	EQ # 7.	0.6 D + 0.6 W
Lateral Components & Cladding	**15.85 psf**	EQ # 5.	D + 0.6 W
Lateral Main Wind Force	**15.85 psf**	EQ # 5.	D + 0.6 W

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**5.087"x5.006" 6063-T6 Standard Aluminum Louver - Louver Blade**

ALUMINUM DESIGN MANUAL (2015 EDITION)

Specifications for Aluminum Structures (Buildings)

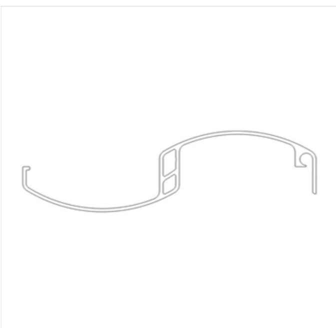
Allowable Stress Design

Design Check of 5.087"x5.006" 6063-T6 Standard Aluminum Louver

Per 2015 Aluminum Design Manu

Alloy: 6063 Temper: T6 Critically Welded: N

Member Properties **5.087"x5.006" 6063-T6 Standard Aluminum Louver**



(Louver In Open Position)

Base Width, b =	5.087"
Base Thickness, tb =	0.125"
Web Height, h =	5.006"
Web Thickness, th =	0.250"
Moment of Inertia About Axis || To Base, Ix =	2.454 in^4
Moment of Inertia About Axis | | To Web, Iy =	1.180 in^4
Section Modulus About The X-Axis, Sx =	1.062 in^4
Radius Of Gyration About Axis | | To Base, rx =	1.66 in
Radius Of Gyration About Axis | | To Web, ry =	1.15 in
Torsional Constant, J =	19.15 in^4
Cross Sectional Area, A =	0.89 in^2
Plastic Section Modulis, Z =	4.52 in^3
Warping Constant, Cw =	0.00 in^6

Member Spans

Unsupported Length (Max Span Between Supports), L =	10.0 ft
Unbraced Length For Bending (Against Side-Sway), Lb =	10.0 ft
Effective Length Factor, k =	1.0

Material Properties

Tensile Ultimate Strength, Ftu =	30 ksi
Tensile Yield Strength, Fty =	25 ksi
Compressive Yield Strength, Fcy =	25 ksi
Shear Ultimate Strength, Fsu =	18 ksi
Shear Yield Strength, Fsy =	15 ksi
Compressive Modulus Of Elasticity, E =	10,100 ksi

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**5.087"x5.006" 6063-T6 Standard Aluminum Louver - Louver Blade**

Buckling Constants

Compression In Columns & Beam Flanges (Intercept), Bc =	27.64 ksi
Compression In Columns & Beam Flanges (Slope), Dc =	0.14 ksi
Compression In Columns & Beam Flanges (Intersection), Cc =	78.38 ksi
Compression In Flat Plates (Intercept), Bp =	31.39 ksi
Compression In Flat Plates (Slope), Dp =	0.17 ksi
Compression In Flat Plates (Intersection), Cp =	73.55 ksi
Compressive Bending Stress In Solid Rectangular Bars (Intercept), Bbr =	46.12 ksi
Compressive Bending Stress In Solid Rectangular Bars (Slope), Dbr =	0.38 ksi
Shear Stress In Flat Plates (Intercept), Bs =	18.98 ksi
Shear Stress In Flat Plates (Slope), Ds =	0.08 ksi
Shear Stress In Flat Plates (Intersection), Cs =	94.57 ksi
Ultimate Strength Coefficient Of Flat Plates In Compression, k1c =	0.35
Ultimate Strength Coefficient Of Flat Plates In Compression, k2c =	2.27
Ultimate Strength Coefficient Of Flat Plates In Bending, k1b =	0.50
Ultimate Strength Coefficient Of Flat Plates In Bending, k2b =	2.04
Tension Coefficient, kt =	1.0

Member Strength Calculations

D.2 Axial Tension

Tensile Yielding - Unwelded Members	Fty_n =	25.00 ksi	
	Ω =	1.65	
	Fty_n/Ω =	15.15 ksi	
Tensile Rupture - Unwelded Members	*Ftu_n =*	30.00 ksi	
	Ω =	1.95	
	Ftu_n/Ωt =	15.38 ksi	

Axial Compression Members

E.2 Compression Member Buckling

Axial, Gross Section Subject To Buckling		Lower Slenderness Limit, λ1 =	18.23	
		Upper Slenderness Limit, λ2 =	78.38	
		Slenderness, λ(max) =	103.99	≥ λ2
	[0.85π²E/λ²]	*Fc_n =*	7.84 ksi	
		Ω =	1.65	
		Fc_n/Ω =	4.75 ksi	

E.3 Local Buckling

For Column Elements In Uniform Compression Subject To
Local Buckling, The Uniform Compressive Strength Is
B.5.4.2 - Flat Elements Supported On Both Edges (Base)
B.5.4.2 - Flat Elements Supported On Both Edges (Web)

E.4 Buckling Interaction

Per Table B.5.1	*[π²*E/ (1.6*b/tb)²]*	*Fe(flange) =*	28.92 ksi
	[Fc_n]	*Fc_n =*	7.84 ksi
	Fe(flange) > Fc_n (E.2 Member Buckling)	Ω =	1.65
		Fc_n/Ω =	4.75 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	107.59 ksi
	[Fc_n]	*Fc_n =*	7.84 ksi
	Fe(web) > Fc_n (E.2 Member Buckling)	Ω =	1.65
		Fc_n/Ω =	4.75 ksi

Flexural Members

F.2 Yielding And Rupture

Nominal Flexural Strength For Yielding And Rupture		Limit State Of Yielding	
	*[1.5*St*Fty]*	Mnp =	39.83 k-in
	[Mnp/Sx]	Fb_n =	37.50 ksi
		Ω =	1.65
		Fb_n/Ω =	22.73 ksi
		Limit State Of Rupture	
	*[Z*Ftu/kt]*	Mnu =	135.52 k-in
	[Mnu/Z]	Fb_n =	30.00 ksi
		Ω =	1.95
		Fb_n/Ω =	15.38 ksi

F.4 Lateral-Torsional Buckling

Square Or Rectangular Tubes Subject To Lateral-Torsional Buckling

Slenderness For Shapes Symmetric About The Bending Axis, λ F.4.2.1 =	11.81	
Slenderness For Closed Shapes, λ F.4.2.3 =	11.91	
Slenderness For Any Shape, λ F.4.2.5 =	11.81	
Maximum Slenderness, λ(max) =	11.91	< Cc

Nominal Flexural Strength - Lateral-Torsional Buckling			
	*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	36.39 k-in
	[Mnmb/Sx]	Fb_n =	34.27 ksi
		Ω =	1.65
		Fb_n/Ω =	20.77 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**5.087"x5.006" 6063-T6 Standard Aluminum Louver - Louver Blade**

Uniform Compression Elements

B.5.4.2 Flat Elements Supported On Both Edges - Web & Flange

Uniform Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	22.8	
	Upper Slenderness Limit, λ2 =	39.2	
	Flange Slenderness, b/tb =	36.7	λ1 - λ2
	Web Slenderness, h/th =	19.02	≤ λ1
*[Bp-1.6*Dp*b/tb]*	*Fc_n1* =	21.11 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	12.80 ksi	
[Fcy]	*Fc_n2* =	25.00 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	15.15 ksi	

Flexural Compression Elements

B.5.5.1 Flat Elements Supported On Both Edges - Web

Flexural Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	34.73	
	Upper Slenderness Limit, λ2 =	92.95	
	Slenderness, h/th =	19.02	≤ λ1
*[1.5*Fcy]*	*Fb_n* =	37.50 ksi	
	Ω =	1.65	
	Fb_n/Ω =	22.73 ksi	

Shear

G.2 Shear Supported On Both Edges - Web

Members With Flat Elements	Lower Slenderness Limit, λ1 =	38.73	
Supported On Both Edges	Upper Slenderness Limit, λ2 =	75.65	
	Slenderness, h/th =	19.02	≤ λ1
[Fsy]	*Fv_n* =	15.00 ksi	
	Ω =	1.65	
	Fv_n/Ω =	9.09 ksi	

CALCULATED ALLOWABLE STRESSES

Allowable Bending Stress, F_b =	15.38 ksi
Allowable Axial Stress, Compression, F_{ac} =	4.75 ksi
Allowable Shear Stress; Webs, F_v =	9.09 ksi

Elastic Buckling Stress, Fe =	4.73 ksi
Weighted Average Allowable Compressive Stress (Per Section E.3.1), Fao =	14.39 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**5.087"x5.006" 6063-T6 Standard Aluminum Louver - Louver Blade**

Member Loading & Capacity Calculation

Dimensions & Loading Inputs

Layout Style =	**Layout # 1 Louver**
Beam Use =	C&C
Beam Total Length, L =	10.00 ft
# Spans =	1
Max Beam Span (Between Supports), l =	10.00 ft
Beam Overhang Left, OhL =	0.00 ft
Beam Overhang Right, OhR =	0.00 ft
Beam Location =	Intermediate
Point Load At Left Overhang, PohL =	0 lb
Point Load At Right Overhang, PohR =	0 lb
Point Load #1 (Left) On Span, P1 =	0 lb
Point Load #1 Offset, a =	0.00 ft
Point Load #2 (Right) On Span, P2 =	0.0 lb
Point Load #2 Offset, b =	0.00 ft
Resultant Weight Loading On Tributary, RL =	38.9 psf
Tributary Width, W =	0.67 ft
Additional Beam Loading (Icing, Service, Ect), AL =	8.79 lb/ft
Linear Loading On Beam, w =	34.7 lb/ft

Shear In Member And Compression / Tension Reactions At Supports

Max Reaction From Span Point Loads, Vsp =	0 lb
Left Reaction From Overhang Point Loads, VopL =	0 lb
Right Reaction Right Overhang Point Loads, VopR =	0 lb
Max Reaction From Span Weight, Vsw=	173 lb
Reaction From Weight Adjustment Factor For Multi-Span, Vwaf =	1
Adjusted Reaction From Span & OH Weight, Vsw'=	173 lb
Left Reaction From Overhang Weight, VowL=	0 lb
Right Reaction From Overhang Weight, VowR=	0 lb
Max Tension At Supports, Tmax =	0 lb
Max Compression At Supports, Cmax =	**0.17 Kip**

Bending Moment Calculations

Moment From Span Point Loads, Msp =	0 lb-ft
Moment From Point Loads Adjustment Factor For Multi-Span, Mpaf =	1.000
Adjusted Moment From Span Point Loads, Msp' =	0 lb-ft
Moment From Left Overhang Point Loads, MohpL =	0 lb-ft
Moment From Right Overhang Point Loads, MohpR =	0 lb-ft
Moment From Span Weight, Mw=	434 lb-ft
Moment From Weight Adjustment Factor For Multi-Span, Mwaf =	1.00
Adjusted Moment From Span & OH Weight, Mw'=	434 lb-ft
Moment From Left Overhang Weight, MohwL =	0 lb-ft
Moment From Right Overhang Weight, MohwR =	0 lb-ft
Total Max Moment At x, Mmaxx =	0.4 Kip-ft
Total Max Moment At Supports, Mmaxs =	0.0 Kip-ft
Absolute Max Moment On Beam, Mmax =	**0.4 Kip-ft**

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**5.087"x5.006" 6063-T6 Standard Aluminum Louver - Louver Blade**

Deflection Calculations

Deflection From Span Point Loads At x, Δspx =	0.00 in	
Location Of Max Moment From Weight Between Spans, x =	5.00 in	
Deflection From Overhang Point Loads At x, Δopx =	0.00 in	
Deflection From Span & Overhangs Weight At x, Δwx =	0.65 in	
Point Load Deflection At Left Overhang End, ΔowL =	0.00 in	
Point Load Deflection At Right Overhang End, ΔopR =	0.00 in	
Weight Deflection At Left Overhang End, ΔowL =	0.00 in	
Weight Deflection At Right Overhang End, ΔopR =	0.00 in	
Span Max Deflection, Δsp =	0.65 in	
Overhang Max Deflection, Δoh =	0.00 in	
Total Max Deflection, Δmax =	**0.65 in**	

Note: Negative Deflection Values Indicate Upward Deflection

Member Capacity Equations

Bending Stress

Bending Moment Developed In Member, Mz =	**0.4 Kip-ft**	
Bending Stress Developed In Member, fb =	4.90 ksi	
Allowable Bending Stress Of Member, Allowable Bending Stress, Fb =	15.38 ksi	
Bending Moment Capacity =	32%	< 100%

Axial Stress

Axial Load Developed In Member, Fx =	**0.00 Kip**	
Axial Stress Developed In Member, fa =	0.00 ksi	
Allowable Axial Stress, Compression, Fac =	4.75 ksi	
Axial Stress Capacity =	0%	< 100%

Shear Stress

Shear Load Developed In Member, Vz =	**0.17 Kip**	
Shear Stress Developed In Member, fv =	0.07 ksi	
Allowable Shear Stress Of Member Webs, Fv =	9.09 ksi	
Shear Capacity =	1%	< 100%

Interaction Equations

Reduced Bending And Shear Interaction	√ [(fb/Fb)^2 + (fv/Fv)^2] =	32%	< 100%
Axial And Bending Interaction	fa/Fa + fb/Fb =	0%	< 100%
Axial With Reduced Bending And Shear Interaction	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0%	< 100%

Capacity Less than 100% - OK, Member Is Sufficient For Applied Loading

Deflection Check

Deflection Limit =	L / 80	
Allowable Deflection, ΔAllow =	1.50 in	
Maximum Deflection, ΔMax =	**0.65 in**	
Deflection Capacity =	44%	< 100%

OK, Allowable Deflection Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

ALUMINUM DESIGN MANUAL (2015 EDITION)

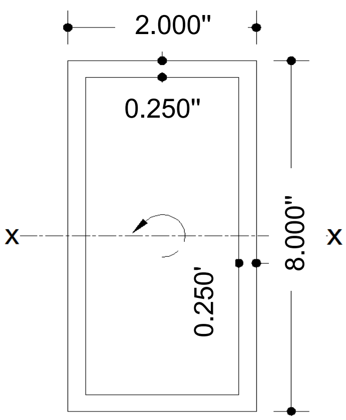
Specifications for Aluminum Structures (Buildings)

Allowable Stress Design

Design Check of Standard Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube
Per 2015 Aluminum Design Manu

Alloy: 6063 Temper: T6 Critically Welded: N

Member Properties **Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube**



# of Parallel Beams in Section # Beams =	1
Base Width, b =	2.000"
Base Thickness, tb =	0.250"
Web Height, h =	8.000"
Web Thickness, th =	0.250"
Moment of Inertia About Axis || To Base, Ix =	32.599 in^4
Moment of Inertia About Axis | | To Web, Iy =	3.224 in^4
Section Modulus About The X-Axis, Sx =	8.150 in^4
Radius Of Gyration About Axis | | To Base, rx =	2.62 in
Radius Of Gyration About Axis | | To Web, ry =	0.82 in
Torsional Constant, J =	9.68 in^4
Cross Sectional Area, A =	4.75 in^2
Plastic Section Modulis, Z =	10.91 in^3
Warping Constant, Cw =	0.00 in^6

Member Spans

Unsupported Length (Max Span Between Supports), L =	19.5 ft
Unbraced Length For Bending (Against Side-Sway), Lb =	2.0 ft
Effective Length Factor, k =	1.0

Material Properties

Tensile Ultimate Strength, Ftu =	30 ksi
Tensile Yield Strength, Fty =	25 ksi
Compressive Yield Strength, Fcy =	25 ksi
Shear Ultimate Strength, Fsu =	18 ksi
Shear Yield Strength, Fsy =	15 ksi
Compressive Modulus Of Elasticity, E =	10,100 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

Buckling Constants

Compression In Columns & Beam Flanges (Intercept), Bc =	27.64 ksi
Compression In Columns & Beam Flanges (Slope), Dc =	0.14 ksi
Compression In Columns & Beam Flanges (Intersection), Cc =	78.38 ksi
Compression In Flat Plates (Intercept), Bp =	31.39 ksi
Compression In Flat Plates (Slope), Dp =	0.17 ksi
Compression In Flat Plates (Intersection), Cp =	73.55 ksi
Compressive Bending Stress In Solid Rectangular Bars (Intercept), Bbr =	46.12 ksi
Compressive Bending Stress In Solid Rectangular Bars (Slope), Dbr =	0.38 ksi
Shear Stress In Flat Plates (Intercept), Bs =	18.98 ksi
Shear Stress In Flat Plates (Slope), Ds =	0.08 ksi
Shear Stress In Flat Plates (Intersection), Cs =	94.57 ksi
Ultimate Strength Coefficient Of Flat Plates In Compression, k1c =	0.35
Ultimate Strength Coefficient Of Flat Plates In Compression, k2c =	2.27
Ultimate Strength Coefficient Of Flat Plates In Bending, k1b =	0.50
Ultimate Strength Coefficient Of Flat Plates In Bending, k2b =	2.04
Tension Coefficient, kt =	1.0

Member Strength Calculations

D.2 Axial Tension

Tensile Yielding - Unwelded Members		Fty_n =	25.00 ksi	
		Ω =	1.65	
		Fty_n/Ω =	15.15 ksi	
Tensile Rupture - Unwelded Members		*Ftu_n =*	30.00 ksi	
		Ω =	1.95	
		Ftu_n/Ωt =	15.38 ksi	

Axial Compression Members

E.2 Compression Member Buckling

Axial, Gross Section Subject To Buckling		Lower Slenderness Limit, λ1 =	18.23	
		Upper Slenderness Limit, λ2 =	78.38	
		Slenderness, λ(max) =	89.32	≥ λ2
	[0.85π²E/λ²]	*Fc_n =*	10.62 ksi	
		Ω =	1.65	
		Fc_n/Ω =	6.44 ksi	

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

E.3 Local Buckling

For Column Elements In Uniform Compression Subject To Local Buckling, The Uniform Compressive Strength Is

B.5.4.2 - Flat Elements Supported On Both Edges (Base)

B.5.4.2 - Flat Elements Supported On Both Edges (Web)

E.4 Buckling Interaction

Per Table B.5.1	*[π²*E/ (1.6*b/tb)²]*	*Fe(flange) =*	1081.63 ksi
	[Fc_n]	*Fc_n =*	10.62 ksi
	Fe(flange) > Fc_n (E.2 Member Buckling)	Ω =	1.65
		Fc_n/Ω =	6.44 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	43.27 ksi
	[Fc_n]	*Fc_n =*	10.62 ksi
	Fe(web) > Fc_n (E.2 Member Buckling)	Ω =	1.65
		Fc_n/Ω =	6.44 ksi

Flexural Members

F.2 Yielding And Rupture

Nominal Flexural Strength For Yielding And Rupture		Limit State of Yielding	
	*[Z*Fcy]*	Mnp =	272.66 k-in
	[Mnp/Z]	Fb_n =	25.00 ksi
		Ω =	1.65
		Fb_n/Ω =	15.15 ksi
		Limit State Of Rupture	
	*[Z*Ftu/kt]*	Mnu =	327.19 k-in
	[Mnu/Z]	Fb_n =	30.00 ksi
		Ω =	1.95
		Fb_n/Ω =	15.38 ksi

F.4 Lateral-Torsional Buckling

Square Or Rectangular Tubes Subject To Lateral-Torsional Buckling

Slenderness For Shapes Symmetric About The Bending Axis, λ F.4.2.1 =	15.14	
Slenderness For Closed Shapes, λ F.4.2.3 =	13.61	
Slenderness For Any Shape, λ F.4.2.5 =	15.14	
Maximum Slenderness, λ(max) =	15.14	< Cc

Nominal Flexural Strength - Lateral-Torsional Buckling

*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	245.53 k-in
[Mnmb/Sx]	Fb_n =	30.13 ksi
	Ω =	1.65
	Fb_n/Ω =	18.26 ksi

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

Uniform Compression Elements

B.5.4.2 Flat Elements Supported On Both Edges - Web & Flange

Uniform Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	22.8	
	Upper Slenderness Limit, λ2 =	39.2	
	Flange Slenderness, b/tb =	6.0	≤ λ1
	Web Slenderness, h/th =	30.0	λ1 - λ2
[Fcy]	*Fc_n1 =*	25.00 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	15.15 ksi	
*[Bp-1.6*Dp*h/th]*	*Fc_n2 =*	22.99 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	13.93 ksi	

Flexural Compression Elements

B.5.5.1 Flat Elements Supported On Both Edges - Web

Flexural Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	34.73	
	Upper Slenderness Limit, λ2 =	92.95	
	Slenderness, h/th =	30.00	≤ λ1
*[1.5*Fcy]*	*Fb_n =*	37.50 ksi	
	Ω =	1.65	
	Fb_n/Ω =	22.73 ksi	

Shear

G.2 Shear Supported On Both Edges - Web

Members With Flat Elements	Lower Slenderness Limit, λ1 =	38.73	
Supported On Both Edges	Upper Slenderness Limit, λ2 =	75.65	
	Slenderness, h/th =	30.00	≤ λ1
[Fsy]	*Fv_n =*	15.00 ksi	
	Ω =	1.65	
	Fv_n/Ω =	9.09 ksi	

CALCULATED ALLOWABLE STRESSES

Allowable Bending Stress, F_b =	15.15 ksi
Allowable Axial Stress, Compression, F_{ac} =	6.44 ksi
Allowable Shear Stress; Webs, F_v =	9.09 ksi

Elastic Buckling Stress, Fe =	6.41 ksi
Weighted Average Allowable Compressive Stress (Per Section E.3.1), Fao =	14.14 ksi

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

Member Loading & Capacity Calculation

Dimensions & Loading Inputs

Layout Style =	**Layout # 1**
	Beam #1 - Louver Beam
Beam Use =	MWF
Beam Total Length, L =	19.50 ft
# Spans =	1
Max Beam Span (Between Supports), Span =	19.50 ft
Beam Overhang Left, OhL =	0.00 ft
Beam Overhang Right, OhR =	0.00 ft
Beam Location =	Edge
Point Load At Left Overhang, PohL =	0 lb
Point Load At Right Overhang, PohR =	0 lb
Point Load #1 (Left) On Span, P1 =	0 lb
Point Load #1 Offset, a =	0.00 ft
Point Load #2 (Right) On Span, P2 =	0.0 lb
Point Load #2 Offset, b =	0.00 ft
Resultant Weight Loading On Tributary, RL =	38.9 psf
Tributary Width, W =	5.00 ft
Additional Beam Loading (Icing, Service, Ect), AL =	21.00 lb/ft
Linear Loading On Beam, w =	215.2 lb/ft

Shear In Member And Compression / Tension Reactions At Supports

Max Reaction From Span Point Loads, Vsp =	0 lb
Left Reaction From Overhang Point Loads, VopL =	0 lb
Right Reaction Right Overhang Point Loads, VopR =	0 lb
Max Reaction From Span Weight, Vsw=	2099 lb
Reaction From Weight Adjustment Factor For Multi-Span, Vwaf =	1
Adjusted Reaction From Span Weight, Vsw'=	2099 lb
Left Reaction From Overhang Weight, VowL=	0 lb
Right Reaction From Overhang Weight, VowR=	0 lb
Max Tension At Supports, Tmax =	**0.00 Kip**
Max Compression At Supports, Cmax =	**2.10 Kip**

Bending Moment Calculations

Moment From Span Point Loads, Msp =	0 lb-ft
Moment From Point Loads Adjustment Factor For Multi-Span, Mpaf =	1.000
Adjusted Moment From Span Point Loads, Msp' =	0 lb-ft
Moment From Left Overhang Point Loads, MohpL =	0 lb-ft
Moment From Right Overhang Point Loads, MohpR =	0 lb-ft
Moment From Span Weight, Mw=	10231 lb-ft
Moment From Weight Adjustment Factor For Multi-Span, Mwaf =	1.00
Adjusted Moment From Span Weight, Mw'=	10231 lb-ft
Moment From Left Overhang Weight, MohwL =	0 lb-ft
Moment From Right Overhang Weight, MohwR =	0 lb-ft
Total Max Moment Along Span, Mmaxspan =	10.2 Kip-ft
Total Max Moment At Supports, Mmaxsup =	0.0 Kip-ft
Absolute Max Moment On Beam, Mmax =	**10.2 Kip-ft**

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #1, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Louver Beam**

Deflection Calculations

Deflection From Span Point Loads At x, Δspx =	0.00 in	
Location Of Max Moment From Weight Between Spans, x =	9.75 in	
Deflection From Overhang Point Loads At x, Δopx =	0.00 in	
Deflection From Span & Overhangs Weight At x, Δwx =	2.13 in	
Point Load Deflection At Left Overhang End, ΔowL =	0.00 in	
Point Load Deflection At Right Overhang End, ΔopR =	0.00 in	
Weight Deflection At Left Overhang End, ΔowL =	0.00 in	
Weight Deflection At Right Overhang End, ΔopR =	0.00 in	
Span Max Deflection, Δsp =	2.13 in	
Overhang Max Deflection, Δoh =	0.00 in	
Total Max Deflection, Δmax =	**2.13 in**	

Note: Negative Deflection Values Indicate Upward Deflection

Member Capacity Equations

Bending Stress

Bending Moment Developed In Member, Mz =	**10.2 Kip-ft**	
Bending Stress Developed In Member, fb =	15.06 ksi	
Allowable Bending Stress Of Member, Allowable Bending Stress, Fb =	15.15 ksi	
Bending Moment Capacity =	99%	< 100%

Axial Stress

Axial Load Developed In Member, Fx =	**0.00 Kip**	
Axial Stress Developed In Member, fa =	0.00 ksi	
Allowable Axial Stress, Compression, Fac =	6.44 ksi	
Axial Stress Capacity =	0%	< 100%

Shear Stress

Shear Load Developed In Member, Vz =	**2.10 Kip**	
Shear Stress Developed In Member, fv =	0.56 ksi	
Allowable Shear Stress Of Member Webs, Fv =	9.09 ksi	
Shear Capacity =	6%	< 100%

Interaction Equations

Reduced Bending And Shear Interaction	√ [(fb/Fb)^2 + (fv/Fv)^2] =	100%	< 100%
Axial And Bending Interaction	fa/Fa + fb/Fb =	0%	< 100%
Axial With Reduced Bending And Shear Interaction	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0%	< 100%

Capacity Less than 100% - OK, Member Is Sufficient For Applied Loading

Deflection Check

Deflection Limit =	L / 80	
Allowable Deflection, ΔAllow =	2.93 in	
Maximum Deflection, ΔMax =	**2.13 in**	
Deflection Capacity =	73%	< 100%

OK, Allowable Deflection Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Beam #2, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Main Beam**

ALUMINUM DESIGN MANUAL (2015 EDITION)

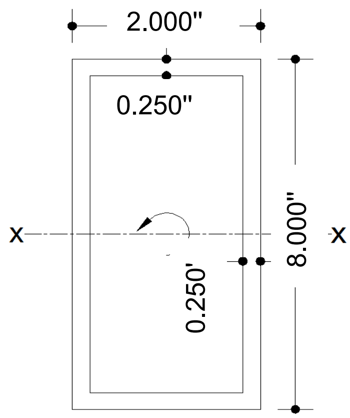
Specifications for Aluminum Structures (Buildings)

Allowable Stress Design

Design Check of Standard Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube
Per 2015 Aluminum Design Manu

Alloy: 6063 Temper: T6 Critically Welded: N

Member Properties



Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube

# of Parallel Beams in Section	# Beams =	1
	Base Width, b =	2.000"
	Base Thickness, tb =	0.250"
	Web Height, h =	8.000"
	Web Thickness, th =	0.250"
	Moment of Inertia About Axis || To Base, Ix =	32.599 in^4
	Moment of Inertia About Axis | | To Web, Iy =	3.224 in^4
	Section Modulus About The X-Axis, Sx =	8.150 in^4
	Radius Of Gyration About Axis | | To Base, rx =	2.62 in
	Radius Of Gyration About Axis | | To Web, ry =	0.82 in
	Torsional Constant, J =	9.68 in^4
	Cross Sectional Area, A =	4.75 in^2
	Plastic Section Modulis, Z =	10.91 in^3
	Warping Constant, Cw =	0.00 in^6

Member Spans

Unsupported Length (Max Span Between Supports), L =	10.0 ft
Unbraced Length For Bending (Against Side-Sway), Lb =	10.0 ft
Effective Length Factor, k =	1.0

Material Properties

Tensile Ultimate Strength, Ftu =	30 ksi
Tensile Yield Strength, Fty =	25 ksi
Compressive Yield Strength, Fcy =	25 ksi
Shear Ultimate Strength, Fsu =	18 ksi
Shear Yield Strength, Fsy =	15 ksi
Compressive Modulus Of Elasticity, E =	10,100 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #2, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Main Beam**

Buckling Constants

Compression In Columns & Beam Flanges (Intercept), Bc =	27.64 ksi
Compression In Columns & Beam Flanges (Slope), Dc =	0.14 ksi
Compression In Columns & Beam Flanges (Intersection), Cc =	78.38 ksi
Compression In Flat Plates (Intercept), Bp =	31.39 ksi
Compression In Flat Plates (Slope), Dp =	0.17 ksi
Compression In Flat Plates (Intersection), Cp =	73.55 ksi
Compressive Bending Stress In Solid Rectangular Bars (Intercept), Bbr =	46.12 ksi
Compressive Bending Stress In Solid Rectangular Bars (Slope), Dbr =	0.38 ksi
Shear Stress In Flat Plates (Intercept), Bs =	18.98 ksi
Shear Stress In Flat Plates (Slope), Ds =	0.08 ksi
Shear Stress In Flat Plates (Intersection), Cs =	94.57 ksi
Ultimate Strength Coefficient Of Flat Plates In Compression, k1c =	0.35
Ultimate Strength Coefficient Of Flat Plates In Compression, k2c =	2.27
Ultimate Strength Coefficient Of Flat Plates In Bending, k1b =	0.50
Ultimate Strength Coefficient Of Flat Plates In Bending, k2b =	2.04
Tension Coefficient, kt =	1.0

Member Strength Calculations

D.2 Axial Tension

Tensile Yielding - Unwelded Members	Fty_n =	25.00 ksi	
	Ω =	1.65	
	Fty_n/Ω =	15.15 ksi	
Tensile Rupture - Unwelded Members	*Ftu_n* =	30.00 ksi	
	Ω =	1.95	
	Ftu_n/Ωt =	15.38 ksi	

Axial Compression Members

E.2 Compression Member Buckling

Axial, Gross Section Subject To Buckling	Lower Slenderness Limit, λ1 =	18.23	
	Upper Slenderness Limit, λ2 =	78.38	
	Slenderness, λ(max) =	145.66	≥ λ2
[0.85π²E/λ²]	*Fc_n* =	3.99 ksi	
	Ω =	1.65	
	Fc_n/Ω =	2.42 ksi	

E.3 Local Buckling

For Column Elements In Uniform Compression Subject To
Local Buckling, The Uniform Compressive Strength Is
B.5.4.2 - Flat Elements Supported On Both Edges (Base)
B.5.4.2 - Flat Elements Supported On Both Edges (Web)

E.4 Buckling Interaction

Per Table B.5.1	*[π²*E/ (1.6*b/tb)²]*	*Fe(flange) =*	1081.63 ksi
	[Fc_n]	*Fc_n =*	3.99 ksi
	Fe(flange) > Fc_n (E.2 Member Buckling)	Ω =	1.65
		Fc_n/Ω =	2.42 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	43.27 ksi
	[Fc_n]	*Fc_n =*	3.99 ksi
	Fe(web) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65
		Fc_n/Ω =	2.42 ksi

Flexural Members

F.2 Yielding And Rupture

Nominal Flexural Strength For Yielding And Rupture		Limit State of Yielding	
	*[Z*Fcy]*	Mnp =	272.66 k-in
	[Mnp/Z]	Fb_n =	25.00 ksi
		Ω =	1.65
		Fb_n/Ω =	15.15 ksi
		Limit State Of Rupture	
	*[Z*Ftu/kt]*	Mnu =	327.19 k-in
	[Mnu/Z]	Fb_n =	30.00 ksi
		Ω =	1.95
		Fb_n/Ω =	15.38 ksi

F.4 Lateral-Torsional Buckling

Square Or Rectangular Tubes Subject To Lateral-Torsional Buckling

	Slenderness For Shapes Symmetric About The Bending Axis, λ F.4.2.1 =	30.71	
	Slenderness For Closed Shapes, λ F.4.2.3 =	30.43	
	Slenderness For Any Shape, λ F.4.2.5 =	30.71	
	Maximum Slenderness, λ(max) =	30.71	< Cc

Nominal Flexural Strength - Lateral-Torsional Buckling			
	*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	217.63 k-in
	[Mnmb/Sx]	Fb_n =	26.70 ksi
		Ω =	1.65
		Fb_n/Ω =	16.18 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #2, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Main Beam**

Uniform Compression Elements

B.5.4.2 Flat Elements Supported On Both Edges - Web & Flange

Uniform Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	22.8	
	Upper Slenderness Limit, λ2 =	39.2	
	Flange Slenderness, b/tb =	6.0	≤ λ1
	Web Slenderness, h/th =	30.0	λ1 - λ2
[Fcy]	*Fc_n1* =	25.00 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	15.15 ksi	
*[Bp-1.6*Dp*h/th]*	*Fc_n2* =	22.99 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	13.93 ksi	

Flexural Compression Elements

B.5.5.1 Flat Elements Supported On Both Edges - Web

Flexural Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	34.73	
	Upper Slenderness Limit, λ2 =	92.95	
	Slenderness, h/th =	30.00	≤ λ1
*[1.5*Fcy]*	*Fb_n* =	37.50 ksi	
	Ω =	1.65	
	Fb_n/Ω =	22.73 ksi	

Shear

G.2 Shear Supported On Both Edges - Web

Members With Flat Elements	Lower Slenderness Limit, λ1 =	38.73	
Supported On Both Edges	Upper Slenderness Limit, λ2 =	75.65	
	Slenderness, h/th =	30.00	≤ λ1
[Fsy]	*Fv_n* =	15.00 ksi	
	Ω =	1.65	
	Fv_n/Ω =	9.09 ksi	

CALCULATED ALLOWABLE STRESSES

Allowable Bending Stress, F_b =	15.15 ksi
Allowable Axial Stress, Compression, F_{ac} =	2.42 ksi
Allowable Shear Stress; Webs, F_v =	9.09 ksi

Elastic Buckling Stress, Fe =	2.41 ksi
Weighted Average Allowable Compressive Stress (Per Section E.3.1), Fao =	14.14 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Beam #2, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Main Beam**

Member Loading & Capacity Calculation

Dimensions & Loading Inputs

Layout Style =	**Layout # 1**
	Beam #2 - Unloaded Main Beam
Beam Use =	MWF
Beam Total Length, L =	10.00 ft
# Spans =	2
Max Beam Span (Between Supports), Span =	10.00 ft
Beam Overhang Left, OhL =	0.00 ft
Beam Overhang Right, OhR =	0.00 ft
Beam Location =	Edge
Point Load At Left Overhang, PohL =	2099 lb
Point Load At Right Overhang, PohR =	2099 lb
Point Load #1 (Left) On Span, P1 =	0 lb
Point Load #1 Offset, a =	0.00 ft
Point Load #2 (Right) On Span, P2 =	0.0 lb
Point Load #2 Offset, b =	0.00 ft
Resultant Weight Loading On Tributary, RL =	0.0 psf
Tributary Width, W =	0.00 ft
Additional Beam Loading (Icing, Service, Ect), AL =	10.01 lb/ft
Linear Loading On Beam, w =	10.0 lb/ft

Shear In Member And Compression / Tension Reactions At Supports

Max Reaction From Span Point Loads, Vsp =	0 lb
Left Reaction From Overhang Point Loads, VopL =	2099 lb
Right Reaction Right Overhang Point Loads, VopR =	2099 lb
Max Reaction From Span Weight, Vsw=	50 lb
Reaction From Weight Adjustment Factor For Multi-Span, Vwaf =	1.25
Adjusted Reaction From Span Weight, Vsw'=	63 lb
Left Reaction From Overhang Weight, VowL=	0 lb
Right Reaction From Overhang Weight, VowR=	0 lb
Max Tension At Supports, Tmax =	**0.00 Kip**
Max Compression At Supports, Cmax =	**2.16 Kip**

Bending Moment Calculations

Moment From Span Point Loads, Msp =	0 lb-ft
Moment From Point Loads Adjustment Factor For Multi-Span, Mpaf =	1.156
Adjusted Moment From Span Point Loads, Msp' =	0 lb-ft
Moment From Left Overhang Point Loads, MohpL =	0 lb-ft
Moment From Right Overhang Point Loads, MohpR =	0 lb-ft
Moment From Span Weight, Mw=	125 lb-ft
Moment From Weight Adjustment Factor For Multi-Span, Mwaf =	1.07
Adjusted Moment From Span Weight, Mw'=	134 lb-ft
Moment From Left Overhang Weight, MohwL =	0 lb-ft
Moment From Right Overhang Weight, MohwR =	0 lb-ft
Total Max Moment Along Span, Mmaxspan =	0.1 Kip-ft
Total Max Moment At Supports, Mmaxsup =	0.0 Kip-ft
Absolute Max Moment On Beam, Mmax =	**0.1 Kip-ft**

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Beam #2, Single 2"x8"x 0.25"/0.25" 6063-T6 Aluminum Tube - Main Beam**

Deflection Calculations

Deflection From Span Point Loads At x, Δspx =	0.00 in
Location Of Max Moment From Weight Between Spans, x =	5.00 in
Deflection From Overhang Point Loads At x, Δopx =	0.00 in
Deflection From Span & Overhangs Weight At x, Δwx =	0.01 in
Point Load Deflection At Left Overhang End, ΔowL =	0.00 in
Point Load Deflection At Right Overhang End, ΔopR =	0.00 in
Weight Deflection At Left Overhang End, ΔowL =	0.00 in
Weight Deflection At Right Overhang End, ΔopR =	0.00 in
Span Max Deflection, Δsp =	0.01 in
Overhang Max Deflection, Δoh =	0.00 in
Total Max Deflection, Δmax =	**0.01 in**

Note: Negative Deflection Values Indicate Upward Deflection

Member Capacity Equations

Bending Stress

Bending Moment Developed In Member, Mz =	**0.1 Kip-ft**	
Bending Stress Developed In Member, fb =	0.20 ksi	
Allowable Bending Stress Of Member, Allowable Bending Stress, Fb =	15.15 ksi	
Bending Moment Capacity =	1%	< 100%

Axial Stress

Axial Load Developed In Member, Fx =	**0.00 Kip**	
Axial Stress Developed In Member, fa =	0.00 ksi	
Allowable Axial Stress, Compression, Fac =	2.42 ksi	
Axial Stress Capacity =	0%	< 100%

Shear Stress

Shear Load Developed In Member, Vz =	**2.16 Kip**	
Shear Stress Developed In Member, fv =	0.58 ksi	
Allowable Shear Stress Of Member Webs, Fv =	9.09 ksi	
Shear Capacity =	6%	< 100%

Interaction Equations

Reduced Bending And Shear Interaction	√ [(fb/Fb)^2 + (fv/Fv)^2] =	6%	< 100%
Axial And Bending Interaction	fa/Fa + fb/Fb =	0%	< 100%
Axial With Reduced Bending And Shear Interaction	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0%	< 100%

Capacity Less than 100% - OK, Member Is Sufficient For Applied Loading

Deflection Check

Deflection Limit =	L / 80	
Allowable Deflection, ΔAllow =	1.50 in	
Maximum Deflection, ΔMax =	**0.01 in**	
Deflection Capacity =	0%	< 100%

OK, Allowable Deflection Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Post #1, Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube - Post**

ALUMINUM DESIGN MANUAL (2015 EDITION)

Specifications for Aluminum Structures (Buildings)

Allowable Stress Design

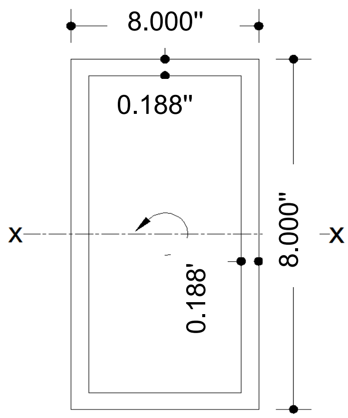
Design Check of Standard Single 8"x8"x 0.1875"/0.1875" 6063-T6 Aluminum Tube As Post

Per 2015 Aluminum Design Manu

Alloy:	6063	Temper:	T6	Critically Welded:	N

Member Properties

Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube



# of Parallel Beams in Section # Beams =	1
Base Width, b =	8.000"
Base Thickness, tb =	0.188"
Web Height, h =	8.000"
Web Thickness, th =	0.188"
Moment of Inertia About Axis || To Base, Ix =	59.639 in^4
Moment of Inertia About Axis | | To Web, Iy =	59.639 in^4
Section Modulus About The X-Axis, Sx =	14.910 in^4
Radius Of Gyration About Axis | | To Base, rx =	3.19 in
Radius Of Gyration About Axis | | To Web, ry =	3.19 in
Torsional Constant, J =	89.41 in^4
Cross Sectional Area, A =	5.86 in^2
Plastic Section Modulis, Z =	17.17 in^3
Warping Constant, Cw =	0.00 in^6

Member Spans

Unsupported Length (Max Span Between Supports), L =	11.0 ft
Unbraced Length For Bending (Against X-Side-Sway), Lbx =	11.0 ft
Unbraced Length For Bending (Against Y-Side-Sway), Lby =	11.0 ft
Effective Length Factor (X Direction), kx =	2.0
Effective Length Factor (Y Direction), ky =	1.0

Material Properties

Tensile Ultimate Strength, Ftu =	30 ksi
Tensile Yield Strength, Fty =	25 ksi
Compressive Yield Strength, Fcy =	25 ksi
Shear Ultimate Strength, Fsu =	18 ksi
Shear Yield Strength, Fsy =	15 ksi
Compressive Modulus Of Elasticity, E =	10,100 ksi

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Post #1, Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube - Post**

Buckling Constants

Compression In Columns & Beam Flanges (Intercept), Bc =	27.64 ksi
Compression In Columns & Beam Flanges (Slope), Dc =	0.14 ksi
Compression In Columns & Beam Flanges (Intersection), Cc =	78.38 ksi
Compression In Flat Plates (Intercept), Bp =	31.39 ksi
Compression In Flat Plates (Slope), Dp =	0.17 ksi
Compression In Flat Plates (Intersection), Cp =	73.55 ksi
Compressive Bending Stress In Solid Rectangular Bars (Intercept), Bbr =	46.12 ksi
Compressive Bending Stress In Solid Rectangular Bars (Slope), Dbr =	0.38 ksi
Shear Stress In Flat Plates (Intercept), Bs =	18.98 ksi
Shear Stress In Flat Plates (Slope), Ds =	0.08 ksi
Shear Stress In Flat Plates (Intersection), Cs =	94.57 ksi
Ultimate Strength Coefficient Of Flat Plates In Compression, k1c =	0.35
Ultimate Strength Coefficient Of Flat Plates In Compression, k2c =	2.27
Ultimate Strength Coefficient Of Flat Plates In Bending, k1b =	0.50
Ultimate Strength Coefficient Of Flat Plates In Bending, k2b =	2.04
Tension Coefficient, kt =	1.0

Member Strength Calculations

D.2 Axial Tension

Tensile Yielding - Unwelded Members	Fty_n =	25.00 ksi
	Ω =	1.65
	Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	*Ftu_n* =	30.00 ksi
	Ω =	1.95
	Ftu_n/Ωt =	15.38 ksi

Axial Compression Members

E.2 Compression Member Buckling

Axial, Gross Section Subject To Buckling		Lower Slenderness Limit, λ1 =	18.23	
		Upper Slenderness Limit, λ2 =	78.38	
		Slenderness, λ(max) =	82.75	≥ λ2
	[0.85π²E/λ²]	*Fc_n* =	12.37 ksi	
		Ω =	1.65	
		Fc_n/Ω =	7.50 ksi	

E.3 Local Buckling

For Column Elements In Uniform Compression Subject To
Local Buckling, The Uniform Compressive Strength Is
B.5.4.2 - Flat Elements Supported On Both Edges (Base)
B.5.4.2 - Flat Elements Supported On Both Edges (Web)

E.4 Buckling Interaction

Per Table B.5.1	*[π²*E/ (1.6*b/tb)²]*	*Fe(flange) =*	23.55 ksi
	[Fc_n]	*Fc_n =*	12.37 ksi
Fe(flange) > Fc_n (E.2 Member Buckling)		Ω =	1.65
		Fc_n/Ω =	7.50 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	23.55 ksi
	[Fc_n]	*Fc_n =*	12.37 ksi
Fe(web) > Fc_n (E.2 Member Buckling)		Ω =	1.65
		Fc_n/Ω =	7.50 ksi

Flexural Members

F.2 Yielding And Rupture

Nominal Flexural Strength For Yielding And Rupture		Limit State of Yielding	
	*[Z*Fcy]*	Mnp =	429.24 k-in
	[Mnp/Z]	Fb_n =	25.00 ksi
		Ω =	1.65
		Fb_n/Ω =	15.15 ksi
		Limit State Of Rupture	
	*[Z*Ftu/kt]*	Mnu =	515.08 k-in
	[Mnu/Z]	Fb_n =	30.00 ksi
		Ω =	1.95
		Fb_n/Ω =	15.38 ksi

F.4 Lateral-Torsional Buckling

Square Or Rectangular Tubes Subject To Lateral-Torsional Buckling

Slenderness For Shapes Symmetric About The Bending Axis, λ F.4.2.1 =	12.14		
Slenderness For Closed Shapes, λ F.4.2.3 =	11.94		
Slenderness For Any Shape, λ F.4.2.5 =	12.14		
Maximum Slenderness, λ(max) =	12.14	< Cc	

Nominal Flexural Strength - Lateral-Torsional Buckling

*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	400.23 k-in
[Mnmb/Sx]	Fb_n =	26.84 ksi
	Ω =	1.65
	Fb_n/Ω =	16.27 ksi

Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Post #1, Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube - Post**

Uniform Compression Elements

B.5.4.2 Flat Elements Supported On Both Edges - Web & Flange

Uniform Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	22.8	
	Upper Slenderness Limit, λ2 =	39.2	
	Flange Slenderness, b/tb =	40.67	≥ λ2
	Web Slenderness, h/th =	40.67	≥ λ2
[k2c√(Bp*E)/(1.6*b/tb)]*	*Fc_n1* =	19.64 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	11.90 ksi	
[k2c√(Bp*E)/(1.6*h/th)]*	*Fc_n2* =	19.64 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	11.90 ksi	

Flexural Compression Elements

B.5.5.1 Flat Elements Supported On Both Edges - Web

Flexural Compression Strength, Flat Elements Supported On Both Edges

	Lower Slenderness Limit, λ1 =	34.73	
	Upper Slenderness Limit, λ2 =	92.95	
	Slenderness, h/th =	40.67	λ1 - λ2
*[Bbr-m*Dbr*h/th]*	*Fb_n* =	36.03 ksi	
	Ω =	1.65	
	Fb_n/Ω =	21.83 ksi	

Shear

G.2 Shear Supported On Both Edges - Web

Members With Flat Elements	Lower Slenderness Limit, λ1 =	38.73	
Supported On Both Edges	Upper Slenderness Limit, λ2 =	75.65	
	Slenderness, h/th =	40.67	λ1 - λ2
*[Bs-1.25Ds*h/th]*	*Fv_n* =	14.80 ksi	
	Ω =	1.65	
	Fv_n/Ω =	8.97 ksi	

CALCULATED ALLOWABLE STRESSES

Allowable Bending Stress, F_b =	14.24 ksi
Allowable Axial Stress, Compression, F_{ac} =	7.50 ksi
Allowable Shear Stress; Webs, F_v =	8.97 ksi
Allowable Axial Stress, Tension, F_{at} =	15.15 ksi

Elastic Buckling Stress, Fe = 7.47 ksi

Weighted Average Allowable Compressive Stress (Per Section E.3.1), Fao = 11.90 ksi

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Post #1, Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube - Post**

Member Loading & Capacity Calculation

Post Dimensions And Geometry

Post Height, h =	11.00 ft
Post Location =	Edge
Post Trib Width in X-Axis (| | Projection), $W_{Trib\ X}$ =	9.75 ft
Post Trib Length in Y-Axis (⊥ Projection), $L_{Trib\ Y}$ =	10.00 ft
Total Tributary Roof Area, $A_{roof\ =}$	97.5 ft²
Fascia Height, h_{fac} =	0.67 ft
Wall Porosity, $\%_{Wall}$ =	100%
Wall / Screen / Post Effective Tributary Width (X Direction), W_{WallX} =	0.67 ft
Wall / Screen / Post Effective Tributary Length (Y Direction), W_{WallY} =	0.67 ft

Lateral Support from Host

Supported against Lateral Forces in X Direction =	TRUE
Supported against Lateral Forces in Y Direction =	FALSE
Roof Acts as Shear Diaphragm =	FALSE
Post Acting as (X Direction) =	Pinned - Fixed
Post Acting as (Y Direction) =	Cantilevered Column

Design Loading

Design Gravity Loading (MWFRS), P_{Grav} =	38.85 psf
Design Uplift Loading (MWFRS), P_{Uplift} =	-13.50 psf
Lateral Loading (MWFRS), $P_{Lateral}$ =	15.85 psf
Wind Force On Lateral Force System Per Post (X Direction) =	444 lb
Wind Force On Lateral Force System Per Post (Y Direction) =	560 lb

Local Seismic Loading (Acting on This Tributary Area)

Local Tributary Weight, W =	1148 lbs
Local Effective Seismic Design Force, Fp =	536.88 lbs
Redundancy Factor, ρ =	1.00
ASD Service Factor =	0.70
Max Seismic Shear, V_{Seis} =	**537 lb**
Max Seismic Moment, M_{Seis} =	**4134 lb-ft**

Axial Force Calculations

Compression Load From Gravity Loading On Tributary Area, F_C =	3788 lb
Tension Load From Uplift Loading On Tributary Area, F_T =	-1316 lb
Max Compression Loading From Loaded Beams, $F_{C\ Beam}$ =	2161 lb
Max Tensile Loading From Loaded Beams, $F_{T\ Beam}$ =	0 lb
Maximum Compressive Loading, F_{XC} =	**3.79 Kip**
Maximum Tension Loading, F_{XT} =	**-1.32 Kip**

Note: Negative Loading Values Indicate Uplift Or Tension

Shear Force Calculations

Lateral Shear (X Direction), V_X =	73 lb
Lateral Shear (Y Direction), V_Y =	222 lb
Resultant Shear (Magnitude), V =	234 lb
Maximum Design Shear, V_{max} =	**0.54 Kip**
Max Torsion due to 5% Eccentric Shear, Tn =	3.2 Kip-in

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Post #1, Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube - Post**

Bending Moment Calculations

Max Y - Moment (At Base) (Bending Towards Host), My =	160 lb-ft
Max X - Moment (At Base) (Bending || To Host), Mx =	1802 lb-ft
X - Moment Reduction for Stiffness of Host Attached Members, $M_{X\text{-}Red}$	15%
Reduced X - Bending Moment, Mx' =	1531 lb-ft
Absolute Max Moment, Mmax =	**4.1 Kip-ft**

Deflection Calculations

Deflection in X - Direction, Δx =	0.00 in
Deflection in Y - Direction, Δy =	0.19 in
Max Deflection, Δmax =	**0.19 in**

Member Capacity Equations

Bending Stress

	Bending Moment Developed In Member, Mz =	**4.1 Kip-ft**	
	Bending Stress Developed In Member, fb =	3.33 ksi	
	Allowable Bending Stress Of Member, Allowable Bending Stress, Fb =	14.24 ksi	
	Bending Moment Capacity =	23%	< 100%
Axial Stress			
Compressive Stress	Compression Load Developed In Member, Fc =	**3.79 Kip**	
	Compression Stress Developed In Member, fac =	0.65 ksi	
	Allowable Axial Stress, Compression, Fac =	7.50 ksi	
	Compressive Stress Capacity =	9%	< 100%
Tensile Stress			
	Tension Load Developed In Member, F_T =	**-1.32 Kip**	
	Tension Stress Developed In Member, fat =	0.05 ksi	
	Allowable Axial Stress, Tension, Fat =	15.15 ksi	
	Tensile Stress Capacity =	0%	< 100%
Shear Stress			
	Shear Load Developed In Member, Vz =	**0.54 Kip**	
	Shear Stress Developed In Member, fv =	0.19 ksi	
	Allowable Shear Stress Of Member Webs, Fv =	8.97 ksi	
	Shear Capacity =	2%	< 100%

Interaction Equations

Reduced Bending And Shear Interaction	√ [(fb/Fb)^2 + (fv/Fv)^2] =	23%	< 100%
Axial And Bending Interaction	fa/Fa + fb/Fb =	32%	< 100%
Axial With Reduced Bending And Shear Interaction	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	14%	< 100%

Capacity Less than 100% - OK, Member Is Sufficient For Applied Loading

Deflection Check

Deflection Limit =	L / 80	
Allowable Deflection, ΔAllow =	1.65 in	
Maximum Deflection, ΔMax =	**0.19 in**	
Deflection Capacity =	12%	< 100%

OK, Allowable Deflection Sufficient

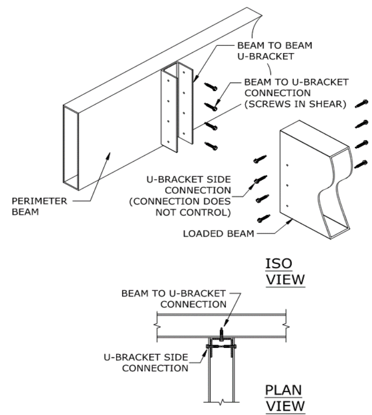
Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Loaded Beam To Perimeter Beam Screw Connection**

Design Of Steel Spaced Thread Tapping Screw to Aluminum Connections

† = 2020 Aluminum Design Manual ; * = AMMA TIR-A9-2014

Anchor To Be Analyzed: **#12-14 SMS, 316 SS, Steel Screws**

Nominal Anchor Size Designation, Size =	#12-14 SMS
Screw Material, (Alloy) =	316 SS
Anchor Ultimate Tensile Strength, Ftu =	100 ksi
Anchor Yield Strength, Fy =	65 ksi
Nominal Screw Diameter , D =	0.216"
Basic Minor Diameter, Dmin =	0.157"
Tensile Stress Area, As =	0.019 in²
Thread Root Area, Ar =	0.019 in²
# Thread Per Inch, n =	14
☐ Consider Washer? Washer Diameter, Dw =	0.625"
Anchor Head Diameter, Dws =	0.415"
Nominal Hole Diameter, Dh =	0.216"
Is anchor placed in a screw boss/chase/slot?	FALSE
Countersunk?	FALSE
Countersink depth, CS Depth =	0.000"
Minimum Aluminum Edge Distance, de =	0.43"



Member in Contact with Screw Head:

Alloy & Temper 1 =	6063-T6
Thickness of Member 1, t1 =	0.250"
Tensile Ultimate Strength of Member 1, Ftu1 =	30 ksi
Tensile Yield Strength of Member 1, Fty1 =	25 ksi

Member not in Contact with Screw Head:

Alloy & Temper 2 =	6063-T6
Thickness of Member 2, t2 =	0.250"
Depth of Full Thread Engagement Into t2, Le =	0.250"
Tensile Ultimate Strength of Member 2, Ftu2 =	30 ksi
Tensile Yield Strength of Member 2, Fty2 =	25 ksi
Screw Boss Wall Thickness, t3 =	0.125"
Min Depth of Full Thread Engagement Into Screw Boss, Le1 =	0.432"
Angle Defining Limits of Screw Engagement, In Screw Chase, a =	86.75
Ratio of Screw Boss Engaged Thread Area To Total Area, Re =	0.348

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Loaded Beam To Perimeter Beam Screw Connection**

Allowable Tension Calculation

Coeff. Dependent On Screw Location, C =	1.0	(† Sect. J.5.4.2)
Coeff. Dependent On Member 2 Thickness, Ks =	1.2	(† Sect. J.5.4.1.1b)
Nominal Pull-Out Strength Of Screw, Rn_t1 =	2494.8 lb	(† Sect. J.5.4.1.1b)
Nominal Pull-Over Strength Of Screw , Rn_t2 =	1492.5 lb	(† Sect. J.5.4.2)
›minal Pull-Out Strength From Screw Boss (if applicable), Rn_t3 =	N/A	(† Sect. J.5.4.1.2)
Allowable Pull-Out Strength From Screw Boss, Rn_t4 =	N/A	(* Sect. 14.0))
Allowable Tensile Capacity Of Screw , Pnt =	645.3 lb	(* Eqn. 10.4-10.7)
Safety Factor For Connections; Building Type Structures, Ω =	3.0	
Safety Factor For Anchor, Ω =	3.0	
Allowable Tension =	**498 lb**	

Allowable Shear Calculation

Bearing On Member 1, Rn_v1 =	3240.0 lb	(† Sect. J.5.5.1)
Bearing On Member 2 , Rn_v2 =	3240.0 lb	(† Sect. J.5.5.1)
Screw Tilting, Rn_v3 =	7319.9 lb	(† Sect. J.5.5.2)
Shear Capacity Of Screw Boss Wall, Rn_v4 =	N/A	
Allowable Shear Capacity Of Screw, Pnv =	372.6 lb	(* Eqn. 7.5)
Safety Factor For Connections; Building Type Structures, Ω =	3.0	
Safety Factor For Anchor, Ω =	3.0	
Allowable Shear =	**373 lb**	

Design Omissions:

Disregard The Limiting Allowable Capacities From Member 1 (Member In Contact With Screw Head) ☐

Disregard The Limiting Allowable Capacities From Member 2 (Member In Not In Contact With Screw Head) ☐

Connection Total Strength & Capacity Calculations

Anchor Qty at Connection, Qty =	6	
Required Tensile Loading on Connection, Treq =	0 lb	(Beam To Beam Connection Not
Required Shear Loading on Connection, Vreq =	2099 lb	Loaded in Tension)
Interaction Exponent factor, n =	1.00	
Tensile capacity of connection, Tcap =	2985 lb	(Anchor Qty* Allowable Tension)
Shear capacity of connection , Vcap =	2235 lb	(Anchor Qty* Allowable Shear)

$$\frac{R_Z}{T_{CAP}} + \frac{R_X}{V_{CAP}} =$$ 94% Maximum Capacity = 100%

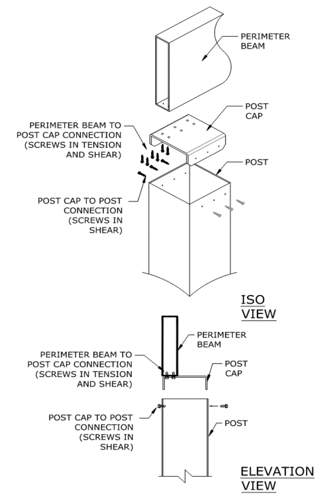
Capacity < 100% OK! - Connection Design Is Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Perimeter Beam To Post Screw Connection**

<u>Design Of Steel Spaced Thread Tapping Screw to Aluminum Connections</u>
† = 2020 Aluminum Design Manual ; * = AMMA TIR-A9-2014

Anchor To Be Analyzed: **#12-14 SMS, 316 SS, Steel Screws**

Nominal Anchor Size Designation, Size =	#12-14 SMS
Screw Material, (Alloy) =	316 SS
Anchor Ultimate Tensile Strength, Ftu =	100 ksi
Anchor Yield Strength, Fy =	65 ksi
Nominal Screw Diameter , D =	0.216"
Basic Minor Diameter, Dmin =	0.157"
Tensile Stress Area, As =	0.019 in²
Thread Root Area, Ar =	0.019 in²
# Thread Per Inch, n =	14
☐ Consider Washer? Washer Diameter, Dw =	0.625"
Anchor Head Diameter, Dws =	0.415"
Nominal Hole Diameter, Dh =	0.216"
Is anchor placed in a screw boss/chase/slot?	FALSE
Countersunk?	FALSE
Countersink depth, CS Depth =	0.000"
Minimum Aluminum Edge Distance, de =	0.43"



Member in Contact with Screw Head:

Alloy & Temper 1 =	6063-T6
Thickness of Member 1, t1 =	0.250"
Tensile Ultimate Strength of Member 1, Ftu1 =	30 ksi
Tensile Yield Strength of Member 1, Fty1 =	25 ksi

Member not in Contact with Screw Head:

Alloy & Temper 2 =	6063-T6
Thickness of Member 2, t2 =	0.188"
Depth of Full Thread Engagement Into t2, Le =	0.188"
Tensile Ultimate Strength of Member 2, Ftu2 =	30 ksi
Tensile Yield Strength of Member 2, Fty2 =	25 ksi
Screw Boss Wall Thickness, t3 =	0.125"
Min Depth of Full Thread Engagement Into Screw Boss, Le1 =	0.432"
Angle Defining Limits of Screw Engagement, In Screw Chase, a =	86.75
Ratio of Screw Boss Engaged Thread Area To Total Area, Re =	0.348

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Work Prepared For:	StruXure Outdoor of Washington
Project:	22-52551 - Shane McArthur
Calculations For:	**Perimeter Beam To Post Screw Connection**

Allowable Tension Calculation

Coeff. Dependent On Screw Location, C =	1.0	(† Sect. J.5.4.2)
Coeff. Dependent On Member 2 Thickness, Ks =	1.2	(† Sect. J.5.4.1.1b)
Nominal Pull-Out Strength Of Screw, Rn_t1 =	1579.5 lb	(† Sect. J.5.4.1.1b)
Nominal Pull-Over Strength Of Screw , Rn_t2 =	1492.5 lb	(† Sect. J.5.4.2)
Nominal Pull-Out Strength From Screw Boss, Rn_t3 =	N/A	(† Sect. J.5.4.1.2)
Allowable Pull-Out Strength From Screw Boss, Rn_t4 =	N/A	(* Sect. 14.0))
Allowable Tensile Capacity Of Screw , Pnt =	645.3 lb	(* Eqn. 10.4-10.7)
Safety Factor For Connections; Building Type Structures, Ω =	3.0	
Safety Factor For Anchor, Ω =	3.0	
Allowable Tension =	**498 lb**	

Allowable Shear Calculation

Bearing On Member 1, Rn_v1 =	3240.0 lb	(† Sect. J.5.5.1)
Bearing On Member 2 , Rn_v2 =	2430.0 lb	(† Sect. J.5.5.1)
Screw Tilting, Rn_v3 =	4754.4 lb	(† Sect. J.5.5.2)
Shear Capacity Of Screw Boss Wall, Rn_v4 =	N/A	
Allowable Shear Capacity Of Screw, Pnv =	372.6 lb	(* Eqn. 7.5)
Safety Factor For Connections; Building Type Structures, Ω =	3.0	
Safety Factor For Anchor, Ω =	3.0	
Allowable Shear =	**373 lb**	

Design Omissions:

Disregard The Limiting Allowable Capacities From Member 1 (Member In Contact With Screw Head) ☐

Disregard The Limiting Allowable Capacities From Member 2 (Member In Not In Contact With Screw Head) ☐

Connection Total Strength & Capacity Calculations

	Beam To Post Clip	**Post Clip To Post**
Anchor Qty At Connection, Qty =	6	6
Required Tensile Loading On Connection, Treq =	1316 lb	0 lb
Required Shear Loading On Connection, Vreq =	537 lb	1316 lb
Interaction Exponent Factor, n =	1.00	1.00
Tensile Capacity Of Connection, Tcap =	2985 lb	2985 lb
Shear Capacity Of Connection , Vcap =	2235 lb	2235 lb
$\frac{R_Z}{T_{CAP}} + \frac{R_X}{V_{CAP}} =$	68%	59%

Capacity < 100% OK! - Connection Design Is Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Isolated Footer Calculations**

Isolated Footing Design

Footing Dimensions

Isolated Footing	Width X =	36 in	Length Y =	36 in	Depth D =	30 in
Slab At Grade?	Trib Width =	0 in	Trib Length =	0 in	Thickness =	0 in

Required Reinforcement

(4) #5, Each Way, Top & Bottom

Check Resistance Against Uplift:

Concrete Unit Wt, γc = 150 pcf
Concrete Footing Weight = 3,375 lbs
Maximum Applied Uplift Load = 1,316 lbs
Uplift Resistance Capcity = 39%
Uplift Required FS = 100%
Capacity < FS - OK! - Uplift Resistance Sufficient

Check Resistance Against Sliding:

Coef. of Base Friction, μ = 0.35
Concrete Footing Weight = 3375.0 lb
Static Friction Force = 1,181 lbs
Maximum Applied Shear Load = 537 lbs
Sliding Resistance Capacity = 45% Sliding Required FS = 100%
Capacity < FS - OK! - Sliding Resistance Sufficient

Check Resistance Against Overturning:

Overturning Moment (X) = 4848 lb-ft (From Applied Uplift, Shear, and Overturning Forces)
Overturning Resistance (X) = 5063 lb-ft (From Concrete Weight Acting At Footing Center)
Overturning Resistance Capacity (X) = 96% OT (X) Required FS = 100%

Overturning Moment (Y) = 3476 lb-ft (From Applied Uplift, Shear, and Overturning Forces)
Overturning Resistance (Y) = 5063 lb-ft (From Concrete Weight Acting At Footing Center)
Overturning Resistance Capacity (Y) = 69% OT (Y) Required FS = 100%
Capacity < FS - OK! - Overturning Resistance Sufficient

Check Soil Bearing Capacity:

Min Soil Bearing Pressure = 1500 psf * To Be Verified By Others If Greater Than 1500 psf
Frictional Resistance = 250 psf * To Be Verified By Others If Greater Than 250 psf
Maximum Bearing Capacity of Footing = 2333 psf
Maximum Applied Gravity Loading = 3,788 lbs

Footing Pressure at Heel, q_{Heel} = 420 psf

$$q_{heel} = \frac{P_{total}}{W \cdot L} - \frac{6M_x}{W^2 \cdot L} - \frac{6M_y}{L^2 \cdot W}$$

Footing Pressure at Toe, q_{Toe} = 1172 psf

$$q_{toe} = \frac{P_{total}}{W \cdot L} + \frac{6M_x}{W^2 \cdot L} + \frac{6M_y}{L^2 \cdot W}$$

Bearing Pressure Capacity = 50% Bearing Required FS = 100%
Capacity < FS - OK! - Soil Bearing Capacity Sufficient

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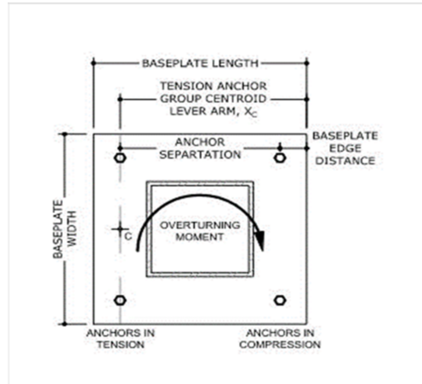
Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Baseplate Capacity Calculations**

Design Check Of A Fully Supported Aluminum - 6063-T6, 12" x 12" x 0.25" Baseplate For Bending And Punching Shear



<u>Member Properties</u>

Plate Length, l =	12.0 in	
Plate Width, b =	12.0 in	
Plate Thickness, tb =	0.250 in	
Moment of Inertia About Axis | | To Flange, Ix =	0.016 in^4	
Section Modulus (About X-Axis), Sc =	0.125 in³	
Baseplate Yield Stress, Fy=	15.0 ksi	

<u>Applied Loading</u>

Maximum Tension Applied To Baseplate, P =	1,316 lbs	
Maximum Moment Applied To Baseplate, M_{MAX} =	1.53 k-ft	

<u>Check Plate Thickness for Bending</u>

Tension/Compression At Either Side Of Plate (Located At Anchorline), T1 =	2.0 kip	(= Mreq / Sep)
Resultant Loading On Baseplate Considering Triangular Load Distribution, T_{Load} =	4.6 kip	(= 1/2 x (Sep/2) x T1)
Moment At Plate Section From Post Centerline To Anchor Centerline (L = 0 in), Mplate =	2.7 kip-in	(= 2 * W * L / 9*√3)
Determine The Value Of m :		
Plate Cantilever Dimension, m =	2.20 in	(= 0.5 (tb - 0.95 d))
Where The Depth of the Column Section, d =	8.00 in	

<u>Determine Thickness Of Base Plate:</u>

λ =	1	
n' = d / 4 =	2.00 in	
Max Plate Cantilever Dimension, c = MAX (m, λn') =	2.20 in	
Required Plate Thickness, tp =	0.100 in	(= 2* c* ([T1+ P/ 2]/ A1* Fy)* 0.5)

Plate Thickness OK! - Bending Resistance Is Sufficient

<u>Check Plate Thickness for Shear Punchout</u>

Vertical Load On Plate Due To Moment At Either Side Of Post (Located At Weld Throat), Vmax =	2.96 Kip	(= Mreq / b_post)
Shear Stress Developed In Plate, fa =	1.0 ksi	(= Vmax/ (Plate Thickness* Width))
Allowable Shear Stress Of Plate, Fac =	16.2 ksi	(= 0.6 * FyA)
Shear Punchout Capacity =	6%	

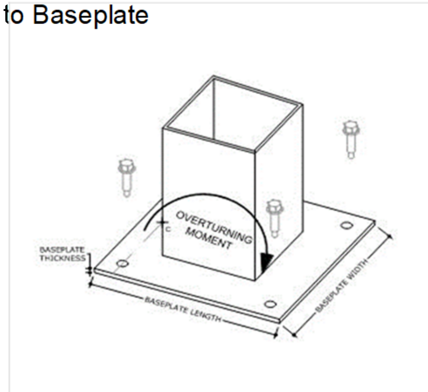
Plate Strength OK! - Shear Punchout Resistance Is Sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Baseplate Anchorage To Concrete Foundation**

Anchored Connection Design for 12" x 12" x 0.25" Baseplate With 4 Anchors, Equally Spaced
Considering (4) 3/8" Dia, Has Threaded Rods With Hy-200 Epoxy @ 4.5" Embed

Post & Baseplate Connection

Post = Single 8" x 8" x 0.1875" / 0.1875" 6063-T6 Aluminum Tube
Baseplate = 12" x 12" x 0.25" Baseplate
Connection = Post Mechanically Attached to Baseplate
Anchor Layout = 4 - Anchor Baseplate
Foundation Strength = 3000 psi Concrete



Anchor Layout & Spacing

Anchor To Plate Edge Distance, a = 1.5 in
Spacing Between Anchors, s = 9.0 in
Tension Anchor Group Centroid , Xc = 10.50 in
Anchor To Concrete Edge Distance = 13.50 in

Anchor Properties

Anchor Considered = 3/8" Dia, Has Threaded Rods With Hy-200 Epoxy @ 4.5" Embed
Anchor Diameter = 0.375 in
Embedment For Maximum Capacity = 4.5 in
Design Tensile Strength Of Anchor = 7,790 lb
Design Shear Strength Of Anchor = 16,780 lb
Concrete Safety Factor = 4

Anchor Strength Reduction Factors

Edge Distance	**Tension**	**Shear**
Edge Distance For Full Capacity =	14.00 in	14.00 in
Minimum Edge Distance Allowed =	1.75 in	1.75 in
Reduction At Min Edge Distance =	22%	5%
Edge Distance Considered =	13.50 in	13.50 in
Edge Distance Reduction Factor =	97%	96%

Anchor Spacing		
Spacing For Full Capacity =	14.00 in	48.00 in
Minimum Spacing Allowed =	1.88 in	1.88 in
Reduction At Min Spacing =	57%	52%
Spacing Considered =	9.00 in	9.00 in
Spacing Reduction Factor =	82%	59%

	Tension	**Shear**
Adjusted Anchor Design Strength =	3,715 lb	5,738 lb
Total Anchor Group Design Strength =	7,431 lb	22,954 lb

Work Prepared For: StruXure Outdoor of Washington
Project: 22-52551 - Shane McArthur
Calculations For: **Baseplate Anchorage To Concrete Foundation**

Applied Loading & Design Calculations

Per ACI 318-14 Ch 17.2.3.4.3 (d) For Tensile Loading And 17.2.3.5.3(c) For Shear Loading, Using Seismic Overstrength Factor, Ω, As Shown Below

Seismic Loading Overstrength Factor Considered?

Seismic Design Category =	D
Moment Due to Seismic Shear =	4.134 kip-ft
Moment Due To Wind Loading =	1.531 kip-ft

Seismic Design Category D - Overstrength Considered

Seismic Overstrength Factor, Ω = 2

Loading On Baseplate & Anchors

Applied Tension, Tmax =	1,316 lbs		
Applied Moment, Mmax =	4134.0 lb-ft		
Applied Tension Due to Moment =	4,725 lbs	= Applied Moment / Tension Anchors Centroid	
Applied Shear, Vmax =	537 lbs		
	Tension		**Shear**
Total Applied Design Loading =	6,041 lbs		537 lbs

Anchor Interaction Capacity

n = 1.00

$$\left(\frac{T_{Applied}}{T_{Strength}}\right)^n + \left(\frac{V_{Applied}}{V_{Strength}}\right)^n = \quad 84\%$$

Anchor Group Strength OK! - Anchors As Detailed Sufficient For Use