ENGINEERING EXPRESS®

Project # 21-45903

CALCULATION COVER SHEET

Calculations Prepared For:

STRUXURE OUTDOOR OF WASHINGTON
9116 E SPRAGUE AVE #547
SPOKANE, WA
509-928-0880

Project:

BAKER, BRIAN
4107 83RD AVE SE
MERCER ISLAND, WA

Subject:

CANOPY CALCULATIONS

REFERENCE SEALED DRAWING BY BELOW-SIGNED ENGINEER FOR ALL NOTES AND DETAILS INCORPORATED HEREIN

ENGINEERING EXPRESS®

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Project # 21-45903

StruXure Outdoor of Washington
Baker, Brian

Wind Loading Criteria (ASCE 7-16)

Basic Wind Speed	110	MPH	ASCE	7-16
Wind Velocity (Vasd)	86	MPH		ASD
Risk Category	II			Residential
Importance Factor	1.00			
Exposure Category	C			

Snow Loading Criteria (ASCE 7-16)

Ground Snow Load	25	PSF
Flat Roof Snow Load	30.00	PSF
Snow Exposure Factor	1.00	
Snow Thermal Factor	1.20	
Snow Importance Factor	1.00	

Live Loading Criteria (ASCE 7-16)

Roof Live Load	20	PSF

Dead Loading Criteria (ASCE 7-16)

Dead Load	3	PSF	StruXure

Seismic Load Criteria (ASCE 7-16)

Site Class	D	Host Attached?	Y	ONE SIDE
Occupancy Category	II	Host Supported?	N	

Mapped Spectral Response Accelerations:

S_S	1.419
S_1	0.493

Spectral Response Coefficients:

S_{DS}	1.135
S_{D1}	0.526
P	1.0
SDC	D
TL	6

Load Combinations (ASCE 7-16)

Gravity D + (Lr or S or R)

Uplift 0.6D + 0.6W

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Project # 21-45903 - Baker, Brian

StruXure Outdoor of Washington
Baker, Brian

DESIGN CRITERIA:

Enter custom loads:

Vult =	110 mph
Exposure:	C
Ground Snow Load:	25.00 psf
Live Load:	20.00 psf
Dead Load:	3.0 psf
Wind Porosity:	50%
Roof Type:	Louvered

Type of project: Residential

These are the loads that this calculator will utilize:

Vult =	110 mph
Exposure:	C
Ground Snow Load:	25.00 psf
Design Live Load:	20.00 psf
Design Dead Load:	3.00 psf
Wind Porosity:	50%
Critical positive grav comb. (+):	33.00 psf
Critical negative uplift comb. (-):	- 1.98 psf
Critical lateral pressure (+):	23.15 psf

Deflection criteria: L / 80

For seismic design, see column calculations

SYSTEM CONFIGURATION:

Louvers:

Overall Canopy Length:	24.0 ft
Overall Canopy Width:	22.0 ft
Roof Slope:	0.0 º

LOUVER BLADES OPEN CHECK 6063-T5

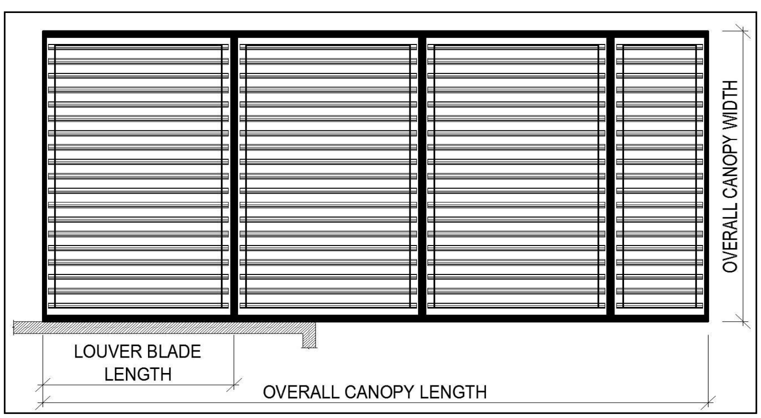
Sx=	0.34528	in^3		
Sy=	0.94568	in^3	**Fcy/Ω =**	**22 ksi**

Mmax	11230.3125 **lb-in**
Stress	11.8753833 **ksi**

Stress Check: 54%

Louver Length: 11.0 ft

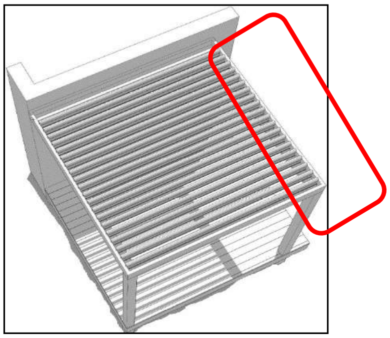
Length of Longest Louver Blade: 11 ft 0 in



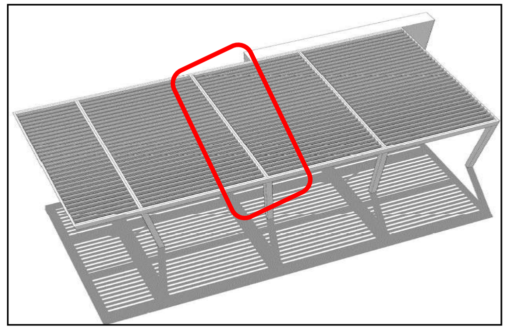
Purlin/Louver Support Beam

Check intermediate or edge? Intermediate *(Intermediate uses full louver blade tributary)*

Edge Louver Beam Configuration



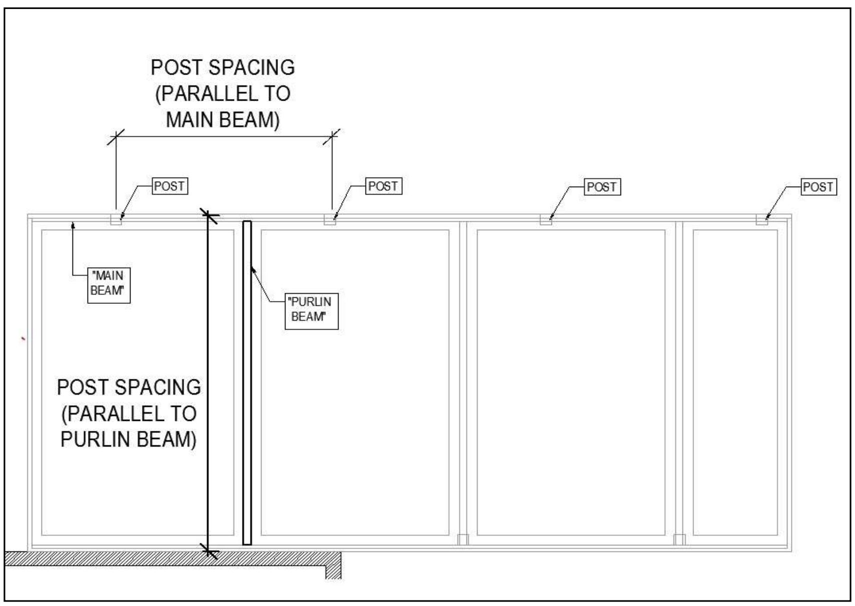
Intermediate Louver Beam Configuration



Purlin/Louver Support Beam - Intermediate Condition Analysis

Support Spacing (Louver Beam Length):	18 ft	8 in			
Single/Double/Triple/Quad:	Double				
Purlin Beam Size:	2"	x	8"	x	0.250"

(Analyzing Double 2" x 8" x 0.25" purlin beam, 18.6666666666667ft long)



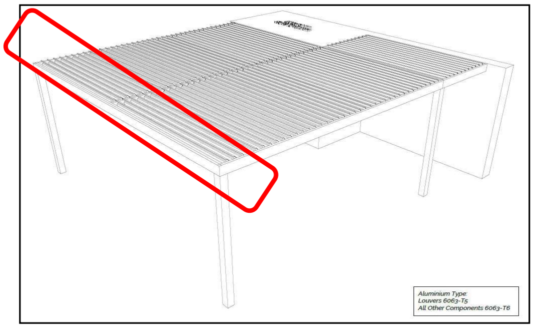
Note: Intermediate condition shown

Purlin Beam Span:	18.7 ft
Purlin Beam Trib:	11.0 ft
Shear at Ends:	4351 lb
Moment Check	99%
Deflection Check	69%

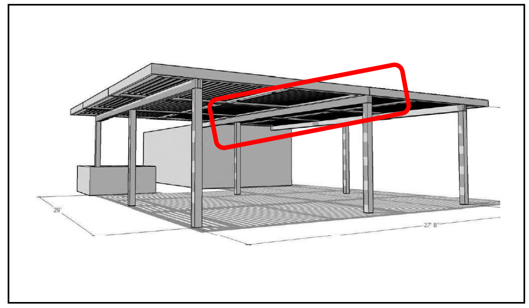
Main Beam

Check intermediate, edge, or none? **Edge** *(Intermediate doubles point loads from louver beams)*

Edge Main Beam Configuration



Intermediate Main Beam Configuration



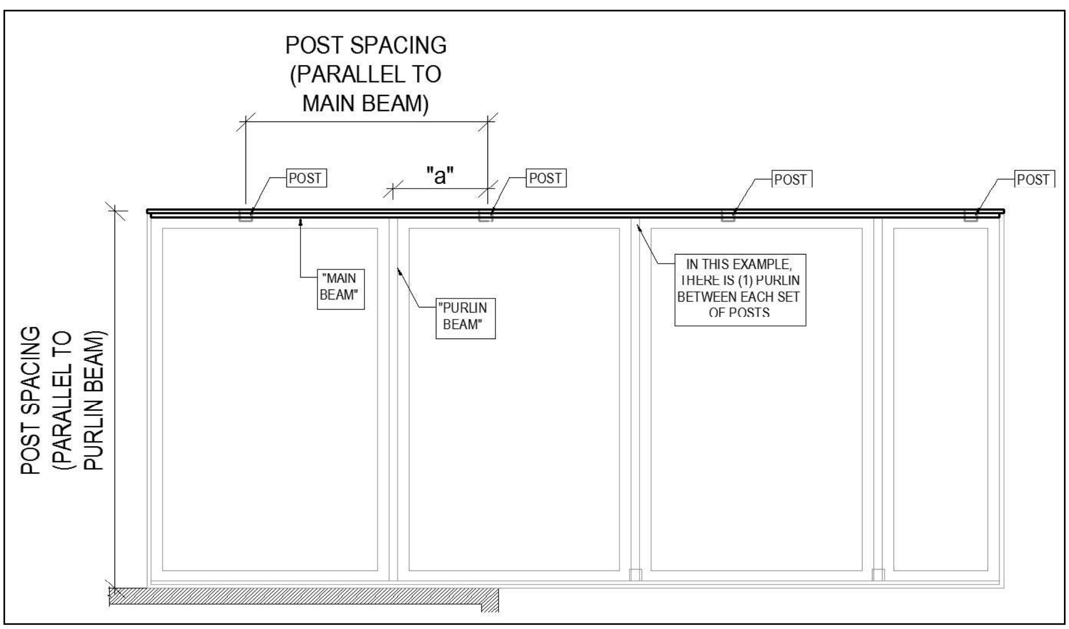
Main Beam - Edge Condition Analysis

Post Spacing (Main Beam Length):	12 ft	0 in				
Single/Double/Triple/Quad:	Single					
Main Beam Size:	2"	x	8"	x	0.125"	

(Analyzing Single 2" x 8" x 0.125" main beam, 12ft long)

Quantity of purlins between a set of posts: 0 *(0 indicates purlins line up directly over posts)*

Assumed offset distance "a" of purlin, measured from post (see diagram): 0.0 ft



Note: Edge condition shown

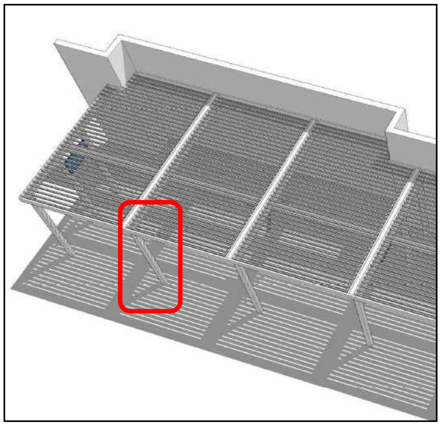
Purlin beams line up directly on posts. No vertical load acting on main beam. Self-weight negligible.

Main Beam Span:	12.0 ft
Load from Purlin:	0 lb
Shear at Ends:	0 lb
Moment Check	0%
Deflection Check	0%

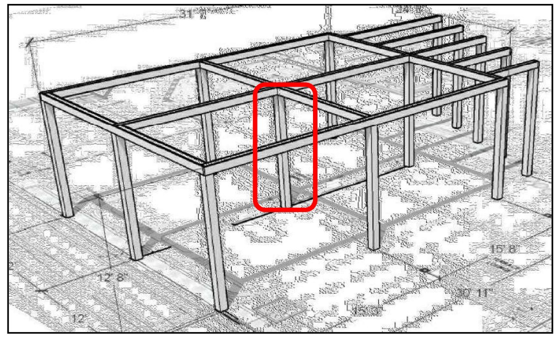
Support Posts

Check intermediate, edge, or none? Edge

Edge Support Post Configuration



Intermediate Support Post Configuration



Mounting Height Above Grade:	0.0 ft	*(Enter 0 for installations at ground level)*		
			Total Mean Roof Height:	20.0 ft
Height of Posts:	20.0 ft			
Attached to host?	Y	Roof Eave Height=	25.0 ft	
			SCREEN	

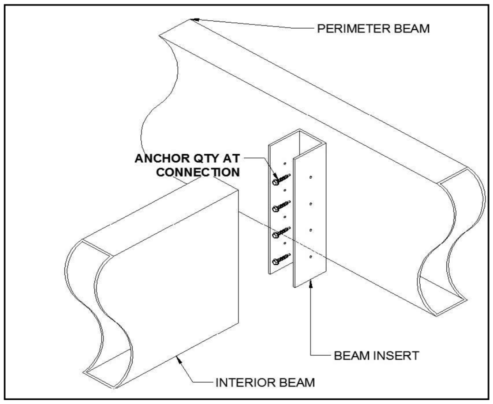
Support Posts - Edge Condition Analysis

Post Size: 8" x 8" x 0.188"

X - AXIS Post Trib =	*6.00*	*ft*
Y - AXIS Post Trib =	*10.00*	*ft* Host Attached
FH1 =	*8.00*	*in, (side fascia height at HT1, normal to lateral windload)*

	Max Moment/Axial/Shear	52%
	Moment/Axial Check:	51%
	Shear Check	11%
ASD Method	Required Tension:	119 lb
	Required Compression:	1980 lb
	Required Shear:	2731 lb
	Required Moment	6.37 kip-ft

Purlin to Perimeter Beam (Clip in Shear)



Qty 6 Anchor Qty at Connection per Beam

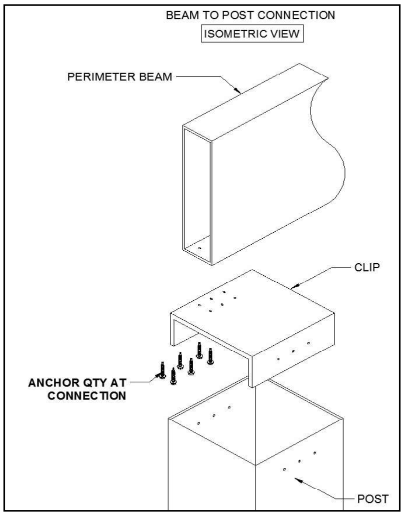
Required Tension:	0 lb
Required Shear:	2175 lb

(Analyzing 1/4-14 SMS, 316 SS, Steel Screw to 0.125" x 0.125" connecting parts thicknesses)

70%

OK, (6) anchors sufficient

Beam to Clip Connection



Qty 8 Anchor Qty at Connection

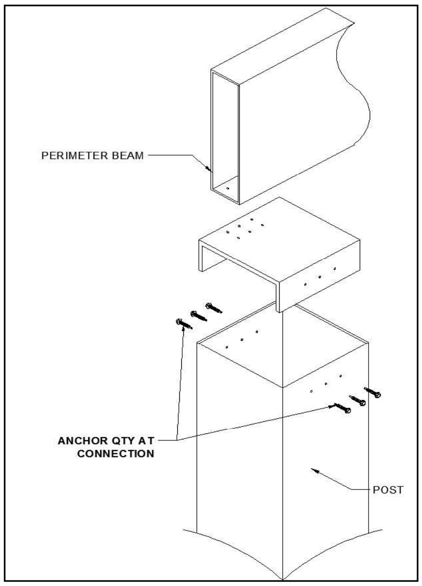
Required Tension:	119 lb
Required Shear:	2731 lb

(Analyzing 1/4-14 SMS, 316 SS, Steel Screw to 0.125" x 0.125" connecting parts thicknesses)

71%

OK, (8) anchors sufficient

Clip to Post Connection



Qty 6 Anchor Qty at Connection

Required Tension:	0 lb
Required Shear:	2731 lb

(Analyzing 1/4-14 SMS, 316 SS, Steel Screw to 0.188" x 0.125" connecting parts thicknesses)

88%

OK, (6) anchors sufficient

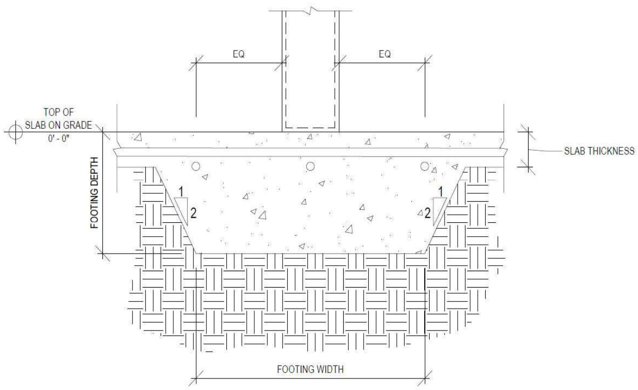
Foundation Isolated Footing

Isolated Footing

Square

No Slab

Footing Dimensions: Wx = 48 in Wy = 48 in D = 24 in



Gravity	2000 psf	Min Soil Bearing Pressure (to be verified by General Contractor)
	1980.0 lb	Gravity load at column
Uplift	4800.0 lb	Total Weight
	118.7 lb	Uplift load at column

OK SQUARE OR RECTANGLE, soil allowable bearing pressure (2000 psf) not exceeded at

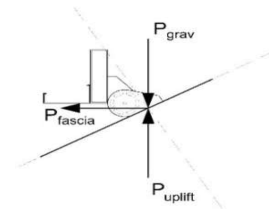
OK, factor of safety FOS = 40.43 >1.0

Plate Thickness: 0.375 in **Width:** 14 in 17% **Okay**

Ledger Connection

Ledger Connection? N

Roof Mount pivot? Y



Roof pitch: 3 / 12

S= 4.00 ft Roof Pivot Mount Spacing

Va = 379 lb

Ta = 72 lb

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Work Prepared For:	StruXure Outdoor of Washington	
Project:	21-45903 - Baker, Brian	

DESIGN CRITERIA:

H =	20.00	ft, Mean Roof Height	ASCE:	7-16
Θ =	0.0 °	Roof Slope F= 0.0000	Exposure:	C
Vult =	110	mph, Wind Velocity (3-Second Gust)	Building Category:	II
Kd =	0.85	Directionality Factor		
G =	0.85	Gust Effect Factor	Snow:	Y
Kz =	0.90	Velocity Pressure Coefficient	Ground Snow Load:	25.00 psf
Kzt =	1.3	Topographic Factor	Design Snow Load:	30.00 psf
			Design Live Load:	20.00 psf
			Design Dead Load:	3.0 psf
	Wind Flow:	Clear	Wind Porosity:	50%
L =	24.00	ft, Overall Canopy Length	Method:	ASD
W =	22.00	ft, Overall Canopy Width	Live Load Lr:	13.44 psf
a =	3.00 ft		Reduction Per Lo:	20.00 psf
			IBC R1:	0.672
			1067.13.2.1 R2:	1

LOADS ON COMPONENTS & CLADDING:

(Roof Decking and Decking Fasteners)

L1 =	11.00	ft, Effective Deck Panel Length	
W1 =	3.67 ft	Effective Deck Panel Width	
A =	40.33 ft^2	Effective Wind Area, L1*W1	A > 4.0*a^2
CNp =	0.6	Positive Pressure Coefficient	
CNn =	-0.5	Negative Pressure Coefficient	
qz =	15.44 psf	Velocity Pressure w/ Porosity	
WLp =	7.87 psf	Positive Wind Load, = qz*G*CNp	
WLn =	-6.30 psf	Negative Wind Load, = qz*G*CNn	
Grav =	**33.00 psf**	**D + (Lr or S or R)**	**Critical positive DP**
Uplift =	**-1.98 psf**	**0.6D + 0.6W**	**Critical negative DP**

LOADS ON MAIN WIND FORCE RESISTING SYSTEM:

(Beams, Columns, Foundations)

Wind Direction, γ = 0º			Wind Direction, γ = 180º		
CNWa =	1.2	Cnw value, load case A	CNWa =	1.2	Cnw value, load case A
CNWb =	-1.1	Cnw value, load case B	CNWb =	-1.1	Cnw value, load case B
CNLa =	0.3	Cnl value, load case A	CNLa =	0.3	Cnl value, load case A
CNLb =	-0.1	Cnl value, load case B	CNLb =	-0.1	Cnl value, load case B

Wind Direction, γ = 90º

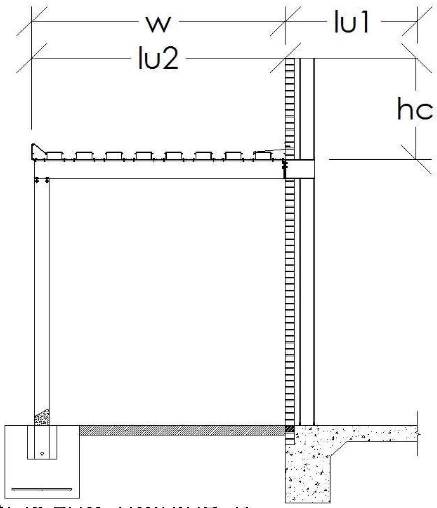
CNa =	-0.8	Cn value, load case A	CNb =	0.8	Cn value, load case B

CNp =	0.6	Critical Positive Pressure Coefficient	
CNn =	-0.5	Critical Negative Pressure Coefficient	
WLp =	7.87 psf	Critical Positive Wind Load, = qz*G*CNp	
WLn =	-6.30 psf	Critical Negative Wind Load, = qz*G*CNn	
Grav =	**33.00 psf**	**D + (Lr or S or R)**	**Critical positive DP**
Uplift =	**-1.98 psf**	**0.6D + 0.6W**	**Critical negative DP**

LOADS ON CANOPY FASCIA:

GCpn1 =	1.5	Combined Net Pressure Coefficient on windward fascia
GCpn1 =	-1	Combined Net Pressure Coefficient on leeward fascia
WL =	**23.15 psf**	**Average Wind Load on Fascia, qz*GCpn*.06**

Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian



Snow Loads

Pg =	25	psf, Ground snow load
Ce =	1.0	Exposure factor (Table 7-2)
Ct =	1.2	Thermal factor (Table 7-3)
Is =	1.0	Importance factor (Table 7-4)
Evs =	1.00 °	Eave slope
S =	57.29	Roof slope run for a rise of one
W =	15.00	ft, Horizontal distance from eave to ridge
γ =	17.25 pcf	Snow density Eq. 7-3: 0.13(Pg)+14 < 30 psf
Cs =	1.00	Slope factor at 1° (Figure 7-2)

Balanced Snow Loads

Pf =	25.00 psf	Snow load on flat roofs (slope < 5°): Pf = max[(I)(20),(0.7)(Ce)(Ct)(I)(Pg)]
Ps =	25.00 psf	Sloped roof snow loads (slope > 5°): Ps = (Cs)(Pf)

Drifts on Lower Roofs (Aerodynamic Shade)

lu1=	20.00	ft, Length of upper roof
lu2=	15.00 ft	Length of lower roof projection
hc=	5.00	ft, Height from top of lower roof to top of eave

Drift snow required, hc/hb>0.2

hb=	1.45 ft	Height of balanced snow: Ps/(γ)
hd1=	1.34 ft	Height of snow drift (Fig 7-9): 0.43(lu)^(1/3)(Pg+10)^(1/4)-1.5 (Leeward)
hd2=	0.81 ft	Height of snow drift (Fig 7-9): 0.43(lu)^(1/3)(Pg+10)^(1/4)-1.5 (Windward)

ASCE 7-10/7-16 - Rain-On-Snow Surcharge (7.10)

Unreducible Snow Load	Yes
Include Uniform Dist. Ice Load?	Yes
Include surcharge load?	Yes

Is Pg 20 PSF or less? NO

5 PSF Rain on Snow Surcharge

hd=	1.34 ft	Governing drift height
w=	5.36 ft	Governing drift width
hend=	0.00 ft	Drift height at edge of lower roof
pd=	11.55 psf	Surcharge load Uniform Distribution Over Drift Width
	4.12 psf	Surcharge Load Distributed over Tributary Area

SL= 30.00 psf Unreducible Roof Snow Load

Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian

Ice Load Due to Freezing Rain (per ASCE 7-16 - Chapter 10)

Acounting for Accumulating Ice on Louver Blades

t_i =	1.00	Nominal Ice Thickness (in.)
K_{zt} =	1.3	Topographic Factor
Z =	20.00 ft	System Height
I_i =	1.00	Importance Factor
I_d =	56.00	Ice Density (56 pcf default)
	II	Occupancy Category

Member Properties

	Louver (6" O.C.)	Louver Beam
Depth (d)	6.000 in.	8.000 in.
Width (bf)	1.866 in.	4.000 in.
Thickness	.065 in.	.250 in.
Length	11.00 ft	18.67 ft

Per Table 10-1

t_d =	1.04	in, Design Ice Thickness	$t_d = t_i{*}I_i{*}f_z{*}(K_{zt})^{0.35}$
W_i =	4.87 psf	Weight of Ice (for td)	W_i= (td/12)*I_d
F_z =	0.9512		$F_z = (Z/33)^{0.1}$

Ice Loading Ch 10.4

Louver Ice Loading

D_c = 6.32 in Circumscribing Diameter of Louver $D_c = \sqrt{d^2+bf^2}$

A_i = 24.11 in^2 Area of Ice = $\pi t_d{*}(D_c+t_d)$

$W_{i(Louver)}$ = 9.38 plf Uniform Distributed Ice Load (Single Louver Blade)

$W_i = (A_i/144){*}I_d$

Louver Beam Ice Loading from Louver Blades

$W_{i(Louver)}$ = 9.38 plf Distributed Ice Load on Louver Blade

L= 11.00 ft Length of Longest Louver Blade

$W_{i(Beam)}$= 51.6 plf Calculated Ice Load on Louver Beam

$W_{i(Beam)}$= $W_{i(Louver)}$*Louver Length*(1.866"/6")

(6" O.C. Louvers In Open Position)

$Wi_{(Louver)}$= 9.38 plf Uniform Linear Ice Load (Louver Blade)

$W_{i(Beam)}$= 103.13 plf Uniform Linear Ice Load (Louver Beam)

($W_{i(Beam)}$ doubled for intermediate Louver Beams)

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Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian

Seismic Loads Criteria

S_s =	1.419	Max considered response acceleration for a period of 0.2 s
S_1 =	0.493	Max response acceleration at period of 1 s

Height of Structure = 20.00 ft Attached to host structure? Y

Site Class	D		
F_a =	1.2	short period amplification factor	
F_v =	1.6	long period amplification factor	
S_{MS} =	1.703	modified spectral response acceleration at a period of 0.2 s	F_a*S_s
S_{M1}=	0.789	modified spectral response acceleration at a period of 1.0 s	F_v*S_1

Spectral Response Acceleration Parameters

S_{DS} =	1.135	Design spectral response acceleration at a period of 0.2 s	$(2/3)*S_{ms}$
S_{D1}=	0.526	design spectral response acceleration at a period of 1.0 s	$(2/3)*S_{M1}$

Structural Design Requirements

T_a =	0.189	approximate fundamental period (s)	$C_t*h_n^x$
T_L =	6.0	Long Transition Period (s)	
E_V=	0.477	Vertical Seismic Loads (PSF)	
Rp =	2.50		
ap=	2.500		
Ip=	1.000		

Wp=	180.00 lbs	Tributary Weight	
Fp=	81.73 lbs	Seismic Design Force	0.4ap*SDS*Wp/(Rp/Ip)*(1+2(z/h))
FpMAX=	326.94 lbs		
FpMIN=	61.30 lbs	Ω=	2.00
P=	1	SERVICE =	0.7
1144.28 lb-ft		Effective Seismic Moment	(H*Fp)
		SDF OK?	OK

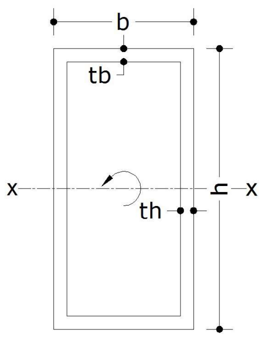
Work Prepared For:	StruXure Outdoor of Washington
Project:	21-45903 - Baker, Brian
Detail/Member:	Purlin Beam

ALUMINUM DESIGN MANUAL (2015 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 2"x8"x0.25"/0.25" 6063-T6 Aluminum Tube
Per 2015 Aluminum Design Manual

Alloy:	6063	Temper:	T6	Critically Welded:	N

MEMBER PROPERTIES



Flange width	*b* =	2.000"
Flange thickness	*tb* =	0.250"
Web height	*h* =	8.000"
Web thickness	*th* =	0.250"
Moment of inertia about axis parallel to flange	*Ix* =	32.60 in^4
Moment of inertia about axis parallel to web	*Iy* =	3.22 in^4
Section modulus about the x-axis	*Sx* =	8.15 in^3
Radius of gyration about centroidal axis parallel to flange	*rx* =	2.62 in
Radius of gyration about centroidal axis parallel to web	*ry* =	0.82 in
Torsion constant	*J* =	9.68 in^4
Cross sectional area of member	*A* =	4.75 in^2
Plastic section modulus	*Z* =	10.91 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	*L* =	18.67 ft
Unbraced length for bending (between bracing against side-sway)	*Lb* =	0.1 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

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 (slenderness limit λ2)	*k1c* =	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	*k2c* =	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	*k1b* =	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	*k2b* =	2.04
Tension coefficient	*kt* =	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	*[Fty]*	Fty_n =	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	*[Ftu/kt]*	*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) =*	85.51	≥ λ2
[0.85π²E/λ²]	*Fc_n =*	11.59 ksi	
	Ω =	1.65	
	Fc_n/Ω =	7.02 ksi	

E.3 Local Buckling

For column elements in uniform compression subject to local buckling, the uniform compressive strength is addressed in Section B.5.4 calculated below.

B.5.4.2 - Flat elements supported on both edges (Flange)

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 =*	11.59 ksi	
Fe(flange) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65	
	Fc_n/Ω =	7.02 ksi	
*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	43.27 ksi	
[Fc_n]	*Fc_n =*	11.59 ksi	
Fe(web) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65	
	Fc_n/Ω =	7.02 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 =*	9.46	
Slenderness for closed shapes	*λ F.4.2.3 =*	3.04	
Slenderness for any shape	*λ F.4.2.5 =*	9.46	
Maximum slenderness	*λ(max) =*	9.46	< Cc

Nominal flexural strength - lateral-torsional buckling

*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	255.71 k-in	
[Mnmb/Sx]	Fb_n =	31.38 ksi	
	Ω =	1.65	
	Fb_n/Ω =	19.02 ksi	

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 supported on both edges

Lower slenderness limit	*λ1* =	38.73	
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	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	15.15 ksi
Allowable axial stress, compression	Fac =	7.02 ksi
Allowable shear stress; webs	Fv =	9.09 ksi

Elastic buckling stress	*Fe* =	6.99 ksi
Weighted average allowable compressive stress (per Section E.3.1)	*Fao* =	14.14 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	*Mz* =	10.15 kip-ft	
Bending stress developed in member	fb =	14.95 ksi	
Allowable bending stress of member	Fb =	15.15 ksi	< 1.0

Axial Loads

Axial load developed in member	*Fx* =	0 lb	
Axial stress developed in member	fa =	0.00 ksi	
Allowable compressive axial stress of member	Fac =	7.02 ksi	< 1.0

Shear Loads

Shear load developed in member	*Vz* =	2,175 lb	
Shear stress developed in member	fv =	0.58 ksi	
Allowable shear stress of member webs	Fv =	9.09 ksi	< 1.0

Interaction Equations

	√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.99	< 1.0
Eq H.1-1	fa/Fa + fb/Fb =	0.00	< 1.0
Eq H.3-2	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.00	< 1.0

CONFIGURATION AND MOMENT TABULATION TOOLS

# of beam= 2

Support Type	Beam =	Simple
Beam Length	L =	18.67 ft
Tributary Width	W =	11.00 ft
Load on Tributary (LL, WL, DL, etc)	RL =	33.00 psf
Additional Beam Load (Weight or Service Loads)	DL =	0.00 lb/ft
Total Loading on Beam	w =	466.13 lb/ft
Shear Loading at End of Beam	Vy =	4351 lbs
CALCULATED MOMENT	**Mmax =**	**20.3 kip-ft**

Deflection Check

	Support =	Simple	
	Deflection Limit =	L / 80	
	w =	466.13 lb/ft	
ALLOWABLE DEFLECTION	**ΔAllow =**	**2.80 in**	
MAXIMUM DEFLECTION	**ΔMax =**	**1.93 in**	69%

Simple Max Deflection = 5wl^4/384EI

OK, Allowable Deflection Sufficient

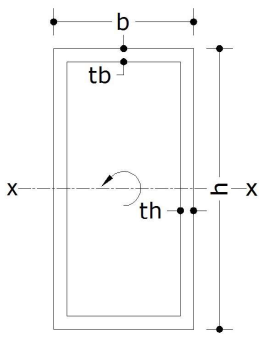
Work Prepared For:	StruXure Outdoor of Washington
Project:	21-45903 - Baker, Brian
Detail/Member:	Main Beam

ALUMINUM DESIGN MANUAL (2015 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 2"x8"x0.125"/0.125" 6063-T6 Aluminum Tube
Per 2015 Aluminum Design Manual

Alloy: 6063 Temper: T6 Critically Welded: N

MEMBER PROPERTIES



Flange width	*b* =	2.000"
Flange thickness	*tb* =	0.125"
Web height	*h* =	8.000"
Web thickness	*th* =	0.125"
Moment of inertia about axis parallel to flange	*Ix* =	17.45 in^4
Moment of inertia about axis parallel to web	*Iy* =	1.87 in^4
Section modulus about the x-axis	*Sx* =	4.36 in^3
Radius of gyration about centroidal axis parallel to flange	*rx* =	2.68 in
Radius of gyration about centroidal axis parallel to web	*ry* =	0.88 in
Torsion constant	*J* =	5.59 in^4
Cross sectional area of member	*A* =	2.44 in^2
Plastic section modulus	*Z* =	5.72 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	*L* =	12.0 ft
Unbraced length for bending (between bracing against side-sway)	*Lb* =	12.0 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

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 (slenderness limit λ2)	*k1c* =	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	*k2c* =	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	*k1b* =	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	*k2b* =	2.04
Tension coefficient	*kt* =	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	*[Fty]*	Fty_n =	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	*[Ftu/kt]*	*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) =*	164.31	≥ λ2
	[0.85π²E/λ²]	*Fc_n =*	3.14 ksi	
		Ω =	1.65	
		Fc_n/Ω =	1.90 ksi	

E.3 Local Buckling

For column elements in uniform compression subject to local buckling, the uniform compressive strength is addressed in Section B.5.4 calculated below.

B.5.4.2 - Flat elements supported on both edges (Flange)

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) =*	198.67 ksi
	[Fc_n]	*Fc_n =*	3.14 ksi
	Fe(flange) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65
		Fc_n/Ω =	1.90 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	10.13 ksi
	[Fc_n]	*Fc_n =*	3.14 ksi
	Fe(web) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65
		Fc_n/Ω =	1.90 ksi

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture	Limit State of Yielding		
	*[Z*Fcy]*	Mnp =	143.07 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 =	171.68 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 =*	32.22	
	Slenderness for closed shapes	*λ F.4.2.3 =*	32.05	
	Slenderness for any shape	*λ F.4.2.5 =*	32.22	
	Maximum slenderness	*λ(max) =*	32.22	< Cc
Nominal flexural strength - lateral-torsional buckling				
	*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	113.35 k-in	
	[Mnmb/Sx]	Fb_n =	25.98 ksi	
		Ω =	1.65	
		Fb_n/Ω =	15.75 ksi	

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 =*	14.0	≤ λ1
Web Slenderness	*h/th =*	62.0	≥ λ2
[Fcy]	*Fc_n1 =*	25.00 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	15.15 ksi	
[k2c√(Bp*E)/(1.6*h/th)]*	*Fc_n2 =*	12.88 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	7.81 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 =*	62.00	λ1 - λ2
*[Bbr-m*Dbr*h/th]*	*Fb_n =*	30.74 ksi	
	Ω =	1.65	
	Fb_n/Ω =	18.63 ksi	

SHEAR

G.2 Shear Supported on Both Edges - Web

Members with flat elements supported on both edges

Lower slenderness limit	*λ1 =*	38.73	
Upper slenderness limit	*λ2 =*	75.65	
Slenderness	*h/th =*	62.00	λ1 - λ2
*[Bs-1.25Ds*h/th]*	*Fv_n =*	12.61 ksi	
	Ω =	1.65	
	Fv_n/Ω =	7.64 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	15.15 ksi
Allowable axial stress, compression	Fac =	1.90 ksi
Allowable shear stress; webs	Fv =	7.64 ksi

Elastic buckling stress	*Fe =*	1.89 ksi
Weighted average allowable compressive stress (per Section E.3.1)	*Fao =*	9.16 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	*Mz =*	0.0 kip-ft	
Bending stress developed in member	fb =	0.00 ksi	
Allowable bending stress of member	Fb =	15.15 ksi	< 1.0

Axial Loads

Axial load developed in member	*Fx =*	0 lb	
Axial stress developed in member	fa =	0.00 ksi	
Allowable compressive axial stress of member	Fac =	1.90 ksi	< 1.0

Shear Loads

Shear load developed in member	*Vz =*	33 lb	
Shear stress developed in member	fv =	0.02 ksi	
Allowable shear stress of member webs	Fv =	7.64 ksi	< 1.0

Interaction Equations

	√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.00	< 1.0
Eq H.1-1	fa/Fa + fb/Fb =	0.00	< 1.0
Eq H.3-2	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.00	< 1.0

CONFIGURATION AND MOMENT TABULATION TOOLS

# of beam=	1
# P load=	0
a=	0.00 ft

Support Type	Beam =	Simple
Beam Length	L =	12.00 ft
Tributary Width	W =	0.17 ft
	P Load=	0.0 lb
Load on Tributary (LL, WL, DL, etc)	RL =	33.00 psf
Additional Beam Load (Weight or Service Loads)	DL =	0.00 lb/ft
Total Loading on Beam	w =	5.50 lb/ft
Shear Loading at End of Beam	Vy =	33 lbs
CALCULATED MOMENT	**Mmax =**	**0.00 kip-ft**

Deflection Check

	Support =	Simple	
	Deflection Limit =	L / 80	
	w =	5.50 lb/ft	
ALLOWABLE DEFLECTION	**ΔAllow =**	**1.80 in**	
MAXIMUM DEFLECTION	**ΔMax =**	**0.00 in**	0%

Simple Max Deflection = 5wl^4/384EI

OK, Allowable Deflection Sufficient

Work Prepared For:	StruXure Outdoor of Washington
Project:	21-45903 - Baker, Brian
Detail/Member:	Column Design

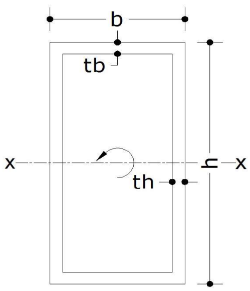
ALUMINUM DESIGN MANUAL (2020 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 8"x8"x0.188"/0.188" 6063-T6 Aluminum Tube

Per 2020 Aluminum Design Manual

Alloy: 6063 | Temper: T6 | Critically Welded: N

MEMBER PROPERTIES



Flange width	*b* =	8.000"
Flange thickness	*tb* =	0.188"
Web height	*h* =	8.000"
Web thickness	*th* =	0.188"
Moment of inertia about axis parallel to flange	*Ix* =	59.79 in^4
Moment of inertia about axis parallel to web	*Iy* =	59.79 in^4
Section modulus about the x-axis	*Sx* =	14.95 in^3
Radius of gyration about centroidal axis parallel to flange	*rx* =	3.19 in
Radius of gyration about centroidal axis parallel to web	*ry* =	3.19 in
Torsion constant	*J* =	89.63 in^4
Cross sectional area of member	*A* =	5.87 in^2
Plastic section modulus	*Z* =	17.21 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	*L* =	20.0 ft
Unbraced length for bending (between bracing against side-sway X-Axis)	*Lbx* =	20.0 ft
Unbraced length for bending (between bracing against side-sway Y-Axis)	*Lby* =	20.0 ft
Effective length factor	*kx* =	2.0
	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

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 (slenderness limit λ2)	*k1c* =	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	*k2c* =	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	*k1b* =	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	*k2b* =	2.04
Tension coefficient	*kt* =	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	*[Fty]*	Fty_n =	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	*[Ftu/kt]*	*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) =*	150.46	≥ λ2
	[0.85π²E/λ²]	*Fc_n =*	3.74 ksi	
		Ω =	1.65	
		Fc_n/Ω =	2.27 ksi	

E.3 Local Buckling

For column elements in uniform compression subject to local buckling, the uniform compressive strength is addressed in Section B.5.4 calculated below.

B.5.4.2 - Flat elements supported on both edges (Flange)

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) =*	23.68 ksi
	[Fc_n]	*Fc_n =*	3.74 ksi
	Fe(flange) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65
		Fc_n/Ω =	2.27 ksi
	*[π²*E/ (1.6*h/th)²]*	*Fe(web) =*	23.68 ksi
	[Fc_n]	*Fc_n =*	3.74 ksi
	Fe(web) > Fc_n (E.2 Member Buckling)	*Ω =*	1.65
		Fc_n/Ω =	2.27 ksi

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture	Limit State of Yielding		
	*[Z*Fcy]*	Mnp =	430.33 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 =	516.39 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 =*	16.13	
Slenderness for closed shapes	*λ F.4.2.3 =*	16.10	
Slenderness for any shape	*λ F.4.2.5 =*	16.13	
Maximum slenderness	*λ(max) =*	16.13	< Cc

Nominal flexural strength - lateral-torsional buckling

*[Mnp(1-(λ/Cc))+(π²*E*λ*Sx/Cc^3)]*	Mnmb =	391.67 k-in
[Mnmb/Sx]	Fb_n =	26.20 ksi
	Ω =	1.65
	Fb_n/Ω =	15.88 ksi

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 =*	40.55	≥ λ2
Web Slenderness	*h/th =*	40.55	≥ λ2
[k2c√(Bp*E)/(1.6*b/tb)]*	*Fc_n1 =*	19.70 ksi	
	Ω =	1.65	
	Fc_n1/Ω =	11.94 ksi	
[k2c√(Bp*E)/(1.6*h/th)]*	*Fc_n2 =*	19.70 ksi	
	Ω =	1.65	
	Fc_n2/Ω =	11.94 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 =*	40.55	λ1 - λ2
*[Bbr-m*Dbr*h/th]*	*Fb_n =*	36.06 ksi	
	Ω =	1.65	
	Fb_n/Ω =	21.85 ksi	

SHEAR
G.2 Shear Supported on Both Edges - Web
Members with flat elements supported on both edges

Lower slenderness limit	*λ1 =*	38.73	
Upper slenderness limit	*λ2 =*	75.65	
Slenderness	*h/th =*	40.55	λ1 - λ2
*[Bs-1.25Ds*h/th]*	*Fv_n =*	14.81 ksi	
	Ω =	1.65	
	Fv_n/Ω =	8.98 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	14.27 ksi
Allowable axial stress, compression	Fac =	2.27 ksi
Allowable shear stress; webs	Fv =	8.98 ksi
Allowable axial stress, Tension	Fat =	15.15 ksi
Elastic buckling stress	*Fe =*	2.26 ksi
Weighted average allowable compressive stress (per Section E.3.1)	*Fao =*	11.94 ksi

MEMBER LOADING
Bending Moments

Bending moment developed in member	*Mz =*	6.37 kip-ft	
Bending stress developed in member	fb =	5.11 ksi	
Allowable bending stress of member	Fb =	14.27 ksi	< 1.0

Compression Loads

Compression load developed in member	*P =*	1,980 lb	
Compression stress developed in member	fc =	0.34 ksi	
Allowable compressive axial stress of member	Fac =	2.27 ksi	< 1.0

Tension Loads

Tension load developed in member	*T =*	119 lb	
Tension stress developed in member	ft =	0.01 ksi	
Allowable Tension axial stress of member	Fat =	15.15 ksi	< 1.0

Shear Loads

Shear load developed in member	*Vz =*	2,731 lb	
Shear stress developed in member	fv =	0.95 ksi	
20	Fv =	8.98 ksi	< 1.0

Interaction Equations

√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.37	< 1.0
fa/Fa + fb/Fb =	0.51	< 1.0
fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.52	< 1.0

CONFIGURATION AND MOMENT TABULATION TOOLS
Member Loads:

Mx =	59.27 kip-in	Applied Moment Per Member	33 PSF	Total Gravity Load
My =	76.40 kip-in	Applied moment Per Member	6.0 FT	Post Trib Area in X-Axis
Tn =	1.35 kip-in	Applied Torsion Per Member	10.0 FT	Post Trib Area in Y-Axis
Vx =	401 lbs	Applied Shear Load Per Member	2 PSF	Uplift
Vy =	2,701 lbs	Applied Shear Load Per Member	23 PSF	Lateral Load
V =	2,731 lbs	Applied Resultant Shear Load Per Member		
P =	1,980 lbs	Applied axial compression load		
T =	119 lbs	Applied axial tension load	13.73 kip-in	Seismic Moment

Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian
Member/Detail: BEAM TO PURLIN

Steel Spaced Thread Tapping Screw to Aluminum Connections

†2020 Aluminum Design Manual, *AMMA TIR-A9-2014

Anchor: 1/4-14 SMS, 316 SS, Steel Screw

Size:	1/4-14 SMS	Nominal Anchor Size Designation
Alloy:	316 SS	Screw Material
Ftu=	100 ksi	Anchor Ultimate Tensile Strength
Fy =	65 ksi	Anchor Yield Strength
D =	0.250"	Nominal Screw Diameter (*Table 20.1,20.2)
Dmin =	0.185"	Basic Minor Diameter (*Table 20.1,20.2)
As =	0.027 in²	Tensile Stress Area (*Table 20.1,20.2)
Ar =	0.027 in²	Thread Root Area (*Table 20.1,20.2)
n =	14	Thread Per Inch
Dw=	0.625"	Washer Diameter ☐ Consider Washer?
Dws =	0.500"	Anchor Head Diameter
Dh =	0.250"	Nominal Hole Diameter
Screw Boss?	No	Is anchor placed in a screw boss/chase/slot?
Countersunk?	No	Yes or No?
CS Depth =		Countersink depth
de =	0.500"	Aluminum Edge Distance

Member in Contact with Screw Head:

Alloy 1:	6063-T6	
t1 =	0.125"	Thickness of Member 1
Ftu1 =	30 ksi	Tensile Ultimate Strength of Member 1
Fty1 =	25 ksi	Tensile Yield Strength of Member 1

Member not in Contact with Screw Head:

Alloy 2:	6063-T6	
t2 =	0.125"	Thickness of Member 2
Le =	0.125"	Depth of Full Thread Engagement Into *t2* (Not Including Tapping/Drilling Point)
Ftu2 =	30 ksi	Tensile Ultimate Strength of Member 2
Fty2 =	25 ksi	Tensile Yield Strength of Member 2
t3 =	0.125"	Screw Boss Wall Thickness
		Minimum Depth of Full Thread Engagement Into S*crew Boss If Applicable* (Not Including Tapping/Drilling Point)
Le1 =	0.500"	

Allowable Tension

C=	1.0	Coeff. Dependent On Screw Location (†Sect. J.5.4.2)
Ks=	1.2	Coeff. Dependent On Member 2 Thickness (†Sect. J.5.4.1.1b)
Rn_t1 =	937.5 lb	Nominal Pull-Out Strength Of Screw (†Sect. J.5.4.1.1b)
Rn_t2 =	937.5 lb	Nominal Pull-Over Strength Of Screw (†Sect. J.5.4.2)
Rn_t3 =	N/A	Nominal Pull-Out Strength From Screw Boss (if applicable) (†Sect. J.5.4.1.2)
Pnt =	896.0 lb	Allowable Tensile Capacity Of Screw (*Eqn. 10.4-10.7)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Tension = 313 lb

Allowable Shear:

Rn_v1 =	1875.0 lb	Bearing On Member 1 (†Sect. J.5.5.1)
Rn_v2 =	1875.0 lb	Bearing On Member 2 (†Sect. J.5.5.1)
Rn_v3 =	2784.2 lb	Screw Tilting (†Sect. J.5.5.2)
Rn_v4 =	N/A	Shear Capacity Of Screw Boss Wall
Pnv =	517.3 lb	Allowable Shear Capacity Of Screw (*Eqn. 7.5)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Shear = 517 lb

Alternate Options:

- ☐ 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)

Concentrated Shear & Tensile Reactions ☑ (Select this connection type)

Qty	6	Anchor Qty at Connection
Treq	0 lb	Required Tensile Loading on Connection
Vreq	2175 lb	Required Shear Loading on Connection
n	1.00	Exponent factor
Tcap	1875 lb	Tensile capacity of connection (Qty * Rz)
Vcap	3104 lb	Shear capacity of connection (Qty * Rx)

$$\frac{R_Z}{T_{CAP}} + \frac{R_X}{V_{CAP}} = \quad 0.70$$

OK, (6) anchors sufficient

Work Prepared For:	StruXure Outdoor of Washington
Project:	21-45903 - Baker, Brian
Member/Detail:	BEAM TO CLIP CONNECTION

Steel Spaced Thread Tapping Screw to Aluminum Connections

†2020 Aluminum Design Manual, *AMMA TIR-A9-2014

Anchor: 1/4-14 SMS, 316 SS, Steel Screw

Size:	1/4-14 SMS	Nominal Anchor Size Designation
Alloy:	316 SS	Screw Material
Ftu=	100 ksi	Anchor Ultimate Tensile Strength
Fy =	65 ksi	Anchor Yield Strength
D =	0.250"	Nominal Screw Diameter (*Table 20.1,20.2)
Dmin =	0.185"	Basic Minor Diameter (*Table 20.1,20.2)
As =	0.027 in²	Tensile Stress Area (*Table 20.1,20.2)
Ar =	0.027 in²	Thread Root Area (*Table 20.1,20.2)
n =	14	Thread Per Inch
Dw=	0.625"	Washer Diameter ☐ Consider Washer?
Dws =	0.500"	Anchor Head Diameter
Dh =	0.250"	Nominal Hole Diameter
Screw Boss?	No	Is anchor placed in a screw boss/chase/slot?
Countersunk?	No	Yes or No?
CS Depth =		Countersink depth
de =	0.500"	Aluminum Edge Distance

Member in Contact with Screw Head:

Alloy 1:	6063-T6	
t1 =	0.125"	Thickness of Member 1
Ftu1 =	30 ksi	Tensile Ultimate Strength of Member 1
Fty1 =	25 ksi	Tensile Yield Strength of Member 1

Member not in Contact with Screw Head:

Alloy 2:	6063-T6	
t2 =	0.125"	Thickness of Member 2
Le =	0.125"	Depth of Full Thread Engagement Into *t2* (Not Including Tapping/Drilling Point)
Ftu2 =	30 ksi	Tensile Ultimate Strength of Member 2
Fty2 =	25 ksi	Tensile Yield Strength of Member 2
t3 =	0.125"	Screw Boss Wall Thickness
		Minimum Depth of Full Thread Engagement Into S*crew Boss If Applicable* (Not Including Tapping/Drilling Point)

Le1 = 0.500"

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Allowable Tension

C=	1.0	Coeff. Dependent On Screw Location (†Sect. J.5.4.2)
Ks=	1.2	Coeff. Dependent On Member 2 Thickness (†Sect. J.5.4.1.1b)
Rn_t1 =	937.5 lb	Nominal Pull-Out Strength Of Screw (†Sect. J.5.4.1.1b)
Rn_t2 =	937.5 lb	Nominal Pull-Over Strength Of Screw (†Sect. J.5.4.2)
Rn_t3 =	N/A	Nominal Pull-Out Strength From Screw Boss (if applicable) (†Sect. J.5.4.1.2)
Pnt =	896.0 lb	Allowable Tensile Capacity Of Screw (*Eqn. 10.4-10.7)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Tension = 313 lb

Allowable Shear:

Rn_v1 =	1875.0 lb	Bearing On Member 1 (†Sect. J.5.5.1)
Rn_v2 =	1875.0 lb	Bearing On Member 2 (†Sect. J.5.5.1)
Rn_v3 =	2784.2 lb	Screw Tilting (†Sect. J.5.5.2)
Rn_v4 =	N/A	Shear Capacity Of Screw Boss Wall
Pnv =	517.3 lb	Allowable Shear Capacity Of Screw (*Eqn. 7.5)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Shear = 517 lb

Alternate Options:

☐ 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)

Concentrated Shear & Tensile Reactions ☑ (Select this connection type)

Qty	8	Anchor Qty at Connection
Treq	119 lb	Required Tensile Loading on Connection
Vreq	2731 lb	Required Shear Loading on Connection
n	1.00	Exponent factor
Tcap	2500 lb	Tensile capacity of connection (Qty * Rz)
Vcap	4138 lb	Shear capacity of connection (Qty * Rx)

$$\frac{R_Z}{T_{CAP}} + \frac{R_X}{V_{CAP}} = \quad 0.71$$

OK, (8) anchors sufficient

Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian
Member/Detail: CLIP TO POST CONNECTION

Steel Spaced Thread Tapping Screw to Aluminum Connections

†2020 Aluminum Design Manual, *AMMA TIR-A9-2014

Anchor: 1/4-14 SMS, 316 SS, Steel Screw

Size:	1/4-14 SMS	Nominal Anchor Size Designation
Alloy:	316 SS	Screw Material
Ftu=	100 ksi	Anchor Ultimate Tensile Strength
Fy =	65 ksi	Anchor Yield Strength
D =	0.250"	Nominal Screw Diameter (*Table 20.1,20.2)
Dmin =	0.185"	Basic Minor Diameter (*Table 20.1,20.2)
As =	0.027 in²	Tensile Stress Area (*Table 20.1,20.2)
Ar =	0.027 in²	Thread Root Area (*Table 20.1,20.2)
n =	14	Thread Per Inch
Dw=	0.625"	Washer Diameter ☐ Consider Washer?
Dws =	0.500"	Anchor Head Diameter
Dh =	0.250"	Nominal Hole Diameter
Screw Boss?	No	Is anchor placed in a screw boss/chase/slot?
Countersunk?	No	Yes or No?
CS Depth =		Countersink depth
de =	0.500"	Aluminum Edge Distance

Member in Contact with Screw Head:

Alloy 1:	6063-T6	
t1 =	0.188"	Thickness of Member 1
Ftu1 =	30 ksi	Tensile Ultimate Strength of Member 1
Fty1 =	25 ksi	Tensile Yield Strength of Member 1

Member not in Contact with Screw Head:

Alloy 2:	6063-T6	
t2 =	0.125"	Thickness of Member 2
Le =	0.125"	Depth of Full Thread Engagement Into *t2* (Not Including Tapping/Drilling Point)
Ftu2 =	30 ksi	Tensile Ultimate Strength of Member 2
Fty2 =	25 ksi	Tensile Yield Strength of Member 2
t3 =	0.125"	Screw Boss Wall Thickness
		Minimum Depth of Full Thread Engagement Into S*crew Boss If Applicable* (Not Including Tapping/Drilling Point)
Le1 =	0.500"	

Allowable Tension

C=	1.0	Coeff. Dependent On Screw Location (†Sect. J.5.4.2)
Ks=	1.2	Coeff. Dependent On Member 2 Thickness (†Sect. J.5.4.1.1b)
Rn_t1 =	937.5 lb	Nominal Pull-Out Strength Of Screw (†Sect. J.5.4.1.1b)
Rn_t2 =	1410.0 lb	Nominal Pull-Over Strength Of Screw (†Sect. J.5.4.2)
Rn_t3 =	N/A	Nominal Pull-Out Strength From Screw Boss (if applicable) (†Sect. J.5.4.1.2)
Pnt =	896.0 lb	Allowable Tensile Capacity Of Screw (*Eqn. 10.4-10.7)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Tension = 313 lb

Allowable Shear:

Rn_v1 =	2820.0 lb	Bearing On Member 1 (†Sect. J.5.5.1)
Rn_v2 =	1875.0 lb	Bearing On Member 2 (†Sect. J.5.5.1)
Rn_v3 =	2784.2 lb	Screw Tilting (†Sect. J.5.5.2)
Rn_v4 =	N/A	Shear Capacity Of Screw Boss Wall
Pnv =	517.3 lb	Allowable Shear Capacity Of Screw (*Eqn. 7.5)
Ω =	3.0	Safety Factor For Connections; Building Type Structures
Ω =	3.0	Safety Factor For Anchor

Allowable Shear = 517 lb

Alternate Options:

- [ ] 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)

Concentrated Shear & Tensile Reactions ☑ (Select this connection type)

Qty	6	Anchor Qty at Connection
Treq	0 lb	Required Tensile Loading on Connection
Vreq	2731 lb	Required Shear Loading on Connection
n	1.00	Exponent factor
Tcap	1875 lb	Tensile capacity of connection (Qty * Rz)
Vcap	3104 lb	Shear capacity of connection (Qty * Rx)

$$\frac{R_Z}{T_{CAP}} + \frac{R_X}{V_{CAP}} = \quad 0.88$$

OK, (6) anchors sufficient

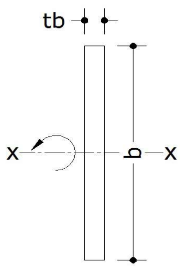
Work Prepared For: Mercer Island
Project: 21-45903 - Baker, Brian
Detail/Member: Base Plate Design

ALUMINUM DESIGN MANUAL (2020 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 14"x0.375" 6063-T6 Aluminum Flat Plate
Per 2020 Aluminum Design Manual

Alloy: 6063 Temper: T6 Critically Welded: N

MEMBER PROPERTIES



Flat Plate Height	*b* =	14.000"
Flat Plate Thickness	*tb* =	0.375"
Moment of inertia about axis parallel to flange	*Ix* =	85.75 in^4
Moment of inertia about axis parallel to web	*Iy* =	0.06 in^4
Section modulus about the x-axis	*Sx* =	12.25 in^3
Radius of gyration about centroidal axis parallel to flange	*rx* =	4.04 in
Radius of gyration about centroidal axis parallel to web	*ry* =	0.11 in
Torsion constant	*J* =	0.25 in^4
Cross sectional area of member	*A* =	5.25 in^2
Plastic section modulus	*Z* =	18.38 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	*L* =	0.46 ft
Unbraced length for bending (between bracing against side-sway)	*Lb* =	0.46 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

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
Compressive bending stress in solid rectangular bars (Intersection)	*Cbr* =	80.56 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 (slenderness limit λ2)	*k1c* =	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	*k2c* =	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	*k1b* =	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	*k2b* =	2.04
Tension coefficient	*kt* =	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	*[Fty]*	*Fty_n* =	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	*[Ftu/kt]*	*Ftu_n* =	30.00 ksi
		Ω =	1.95
		Ftu_n/Ω =	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) =*	50.81	< λ2	
*[(Bc-Dc*λ)(0.85+0.15*((Cc-λ)/(Cc-λ1))]*	*Fc_n =*	18.64 ksi		
	Ω =	1.65		
	Fc_n/Ω =	11.30 ksi		

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture

Limit State of Yielding			
*[Z*Fcy]*	Mnp =	459.38 k-in	
[Mnp/Z]	Fb =	25.00 ksi	
	Ω =	1.65	
	Fb_n/Ω =	15.15 ksi	
Limit State of Rupture			
*[Z*Ftu/kt]*	Mnu =	551.25 k-in	
[Mnu/Z]	Fb =	30.00 ksi	
	Ω =	1.95	
	Fb_n/Ω =	15.38 ksi	

F.4 Lateral-Torsional Buckling

Rectangular bars subject to lateral-torsional buckling

Slenderness for shapes symmetric about the bending axis	*λ F.4.2.1 =*	99.81	
Slenderness for rectangular bars	*λ F.4.2.4 =*	53.82	
Slenderness for any shape	*λ F.4.2.5 =*	99.81	
Maximum slenderness	*λ(max) =*	99.81	≥ Cc

Nominal flexural strength - lateral-torsional buckling

*[π²*E*Sx/λ²]*	Mnmb =	122.58 k-in	
[Mnmb/Sx]	Fb_n =	10.01 ksi	
	Ω =	1.65	
	Fb_n/Ω =	6.06 ksi	

G.2 Shear Supported on Both Edges

Members with flat elements supported on both edges

Lower slenderness limit	*λ1 =*	38.73	
Upper slenderness limit	*λ2 =*	75.65	
Slenderness	*b/tb =*	37.33	≤ λ1
[Fsy]	*Fv_n =*	15.00 ksi	
	Ω =	1.65	
	Fv_n/Ω =	9.09 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	6.06 ksi
Allowable axial stress, compression	Fac =	11.30 ksi
Allowable shear stress	Fv =	9.09 ksi

Elastic buckling stress	*Fe =*	19.80 ksi
Weighted average allowable compressive stress (per Section E.3.1)	*Fao =*	11.30 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	*Mz* =	0.8 kip-ft	
Bending stress developed in member	fb =	0.78 ksi	
Allowable bending stress of member	Fb =	6.06 ksi	< 1.0

Axial Loads

Axial load developed in member	*Fx* =	2,731 lb	
Axial stress developed in member	fa =	0.52 ksi	
Allowable compressive axial stress of member	Fac =	11.30 ksi	< 1.0

Shear Loads

Shear load developed in member	*Vz* =	2,731 lb	
Shear stress developed in member	fv =	0.52 ksi	
Allowable shear stress of member	Fv =	9.09 ksi	< 1.0

Interaction Equations

	√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.14	< 1.0
Eq H.1-1	fa/Fa + fb/Fb =	0.17	< 1.0
Eq H.3-2	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.07	< 1.0

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Work Prepared For: StruXure Outdoor of Washington
Project: 21-45903 - Baker, Brian

CHECK SOIL BEARING PRESSURE FOR CRITICAL FOOTING

Footing Dimensions: Wx = 48 in Wy = 48 in D = 24 in
Sx = 0 in Sy = 0 in Thk = 0 in

1980 lb	Max Axial Gravity Load in Column
+ 4800 lb	Weight of Footing (48" x 48" x 24" pad footer)
6780 lb	Total Load on Soil (gravity load + footing weight)
76.4 kip-in	Total Moment - X-Axis in Footing (column is assumed to be centered in footer)
76.4 kip-in	Total Moment - Y-Axis in Footing (column is assumed to be centered in footer)
2000 psf	Min Soil Bearing Pressure (to be verified by General Contractor)

$$q_{heel} = \frac{P_{total}}{B \cdot L} - \frac{6M_x}{B^2 \cdot L} - \frac{6M_y}{L^2 \cdot B} =$$ -770.1 psf footing pressure at heel (along dimension "W1")

$$q_{toe} = \frac{P_{total}}{B \cdot L} + \frac{6M_x}{B^2 \cdot L} + \frac{6M_y}{L^2 \cdot B} =$$ 1617.6 psf footing pressure at toe (along dimension "W1")

Max bearing pressure on soil = 1617.6 psf (at critical footing)
Frictional Resistance qf = 333.3 psf
Max Bearing Capacity of Footing = 2666.7 psf Square or Rectangle
Max Bearing Capacity of Footing = 2666.7 psf Circle

OK SQUARE OR RECTANGLE, soil allowable bearing pressure (2000 psf) not exceeded at critical footing

OK CIRCLE, soil allowable bearing pressure (2000 psf) not exceeded at critical footing

UPIFT RESISTANCE CALCULATION FOR CRITICAL FOOTING

Footing Dimensions: W1 = 48 in W2 = 48 in D = 24 in
Slab Trib Dimensions: S1 = 0 in S2 = 0 in Thk = 0 in

ρ_c 150 pcf Concrete Density

P 118.7 lb Uplift load at column

Conc Footing Weight = 4800 lb
Conc Slab Weight = 0 lb
Total Gravity Weight = 4800 lb

Total Uplift Load = (P+ M/d) = 119 lb

OK, factor of safety FOS = 40.43 >1.0

REQUIRED REINFORCEMENT

(10) #3 Horizontal Bars Top & Bottom Each Way
(6) #4 Horizontal Bars Top & Bottom Each Way
(4) #5 Horizontal Bars Top & Bottom Each Way
(3) #6 Horizontal Bars Top & Bottom Each Way

W/ (19) #3 Ties **OR** (11) #4 Ties

Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	21-45903 Baker, Brian	Date:	10/14/2021
Fastening point:			

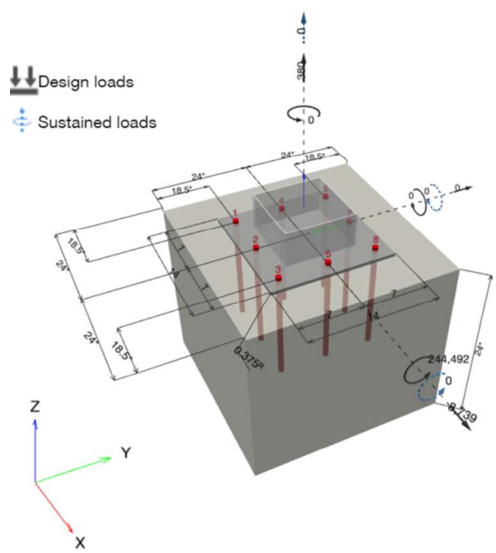
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-R 304/316 SS 5/8**
Item number:	not available (element) / 2022793 HIT-HY 200-R (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 12.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F 593
Evaluation Service Report:	ESR-3187
Issued I Valid:	5/1/2021 | 3/1/2022
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate^R :	l_x x l_y x t = 14.000 in. x 14.000 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.1875; (L x W x T) = 8.000 in. x 8.000 in. x 0.188 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 24.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: present, shear: present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

^{R} - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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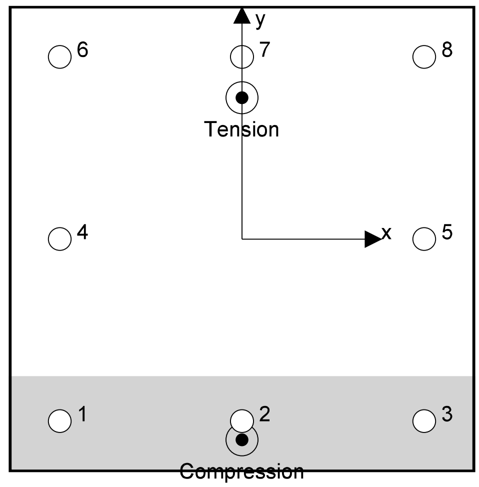
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 380; V_x = 8{,}739; V_y = 0;$ $M_x = 244{,}492; M_y = 0; M_z = 0;$	yes	100

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	1,092	1,092	0
2	0	1,092	1,092	0
3	0	1,092	1,092	0
4	2,671	1,092	1,092	0
5	2,671	1,092	1,092	0
6	6,182	1,092	1,092	0
7	6,182	1,092	1,092	0
8	6,182	1,092	1,092	0



max. concrete compressive strain: 0.27 [‰]
max. concrete compressive stress: 1,193 [psi]
resulting tension force in (x/y)=(0.000/4.270): 23,888 [lb]
resulting compression force in (x/y)=(-0.000/-6.062): 23,508 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	6,182	14,690	43	OK
Bond Strength**	23,888	26,924	89	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	23,888	32,121	75	OK

* highest loaded anchor **anchor group (anchors in tension)

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3.1 Steel Strength

N_{sa} = ESR value — refer to ICC-ES ESR-3187

$\phi\ N_{sa} \geq N_{ua}$ — ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	100,000

Calculations

N_{sa} [lb]
22,600

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
22,600	0.650	1.000	14,690	6,182

3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na}\ \psi_{ec2,Na}\ \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$	ACI 318-19 Eq. (17.6.5.1b)
$\phi\ N_{ag} \geq N_{ua}$	ACI 318-19 Table 17.5.2
A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)	
$A_{Na0} = (2\ c_{Na})^2$	ACI 318-19 Eq. (17.6.5.1.2a)
$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$	ACI 318-19 Eq. (17.6.5.1.2b)
$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.3.1)
$\psi_{ed,Na} = 0.7 + 0.3 \left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.4.1b)
$\psi_{cp,Na} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$	ACI 318-19 Eq. (17.6.5.5.1b)
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$	ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,261	0.625	12.500	18.500	1.000	1,192

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
0.000	0.970	28.666	1.000	0.990

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
8.920	673.10	318.25	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	0.902	1.000	28,952

Results

N_{ag} [lb]	ϕ_{bond}	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
55,228	0.650	0.750	1.000	26,924	23,888

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3\left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
12.333	0.000	0.970	18.500	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
28.666	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,040.00	1,369.00	1.000	0.950	1.000	1.000	40,330

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
57,104	0.750	0.750	1.000	32,121	23,888

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,092	5,695	20	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	8,739	95,025	10	OK
Concrete edge failure in direction x+**	8,739	24,644	36	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3187
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.23	100,000	0.700

Calculations

$V_{sa,eq}$ [lb]
9,492

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ V_{sa,eq}$ [lb]	V_{ua} [lb]
9,492	0.600	1.000	5,695	1,092

4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N} \; \psi_{ed,N} \; \psi_{c,N} \; \psi_{cp,N} \; N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi \; V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9 \; h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2 \; e_N'}{3 \; h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX} \left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c \; \lambda_a \; \sqrt{f_c'} \; h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	12.333	0.000	0.000	18.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	28.666	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
2,304.00	1,369.00	1.000	1.000	1.000	1.000	40,330

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi \; V_{cpg}$ [lb]	V_{ua} [lb]
135,750	0.700	1.000	1.000	95,025	8,739

4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e_v'}{1.5c_{a1}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
16.000	18.500	0.000	1.200	24.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
5.000	1.000	0.625	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
1,152.00	1,152.00	1.000	0.931	1.000	29,403

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
32,858	0.750	1.000	1.000	24,644	8,739

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.887	0.355	5/3	100	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2018, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

7 Installation data

Profile: Square HSS (AISC), HSS8X8X.1875; (L x W x T) = 8.000 in. x 8.000 in. x 0.188 in.

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 0.375 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-R 304/316 SS 5/8

Item number: not available (element) / 2022793 HIT-HY 200-R (adhesive)

Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

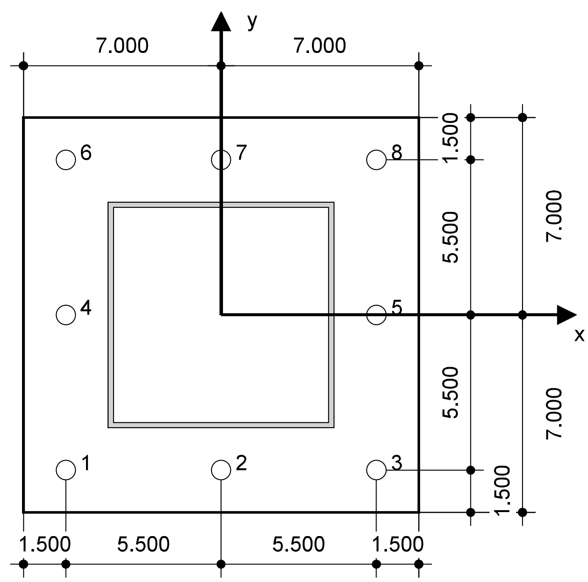
Hole depth in the base material: 12.500 in.

Minimum thickness of the base material: 14.000 in.

5/8 Hilti HAS Stainless steel threaded rod with Hilti HIT-HY 200 Safe Set System

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.500	-5.500	18.500	29.500	18.500	29.500
2	0.000	-5.500	24.000	24.000	18.500	29.500
3	5.500	-5.500	29.500	18.500	18.500	29.500
4	-5.500	0.000	18.500	29.500	24.000	24.000
5	5.500	0.000	29.500	18.500	24.000	24.000
6	-5.500	5.500	18.500	29.500	29.500	18.500
7	0.000	5.500	24.000	24.000	29.500	18.500
8	5.500	5.500	29.500	18.500	29.500	18.500

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8 Remarks; Your Cooperation Duties

- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

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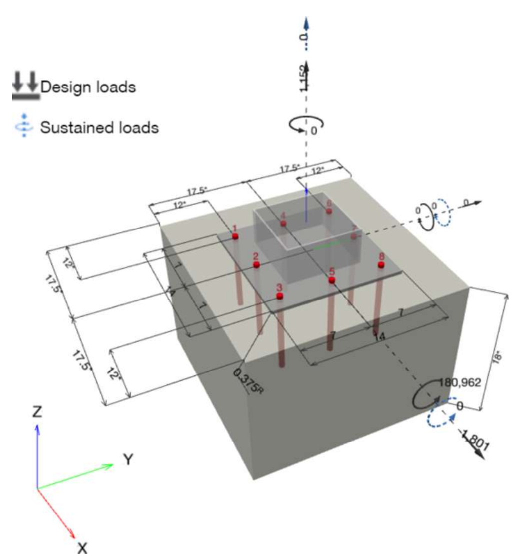
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 + HAS-R 304/316 SS 5/8**
Item number:	408988 HAS-R 316 SS 5/8"x12" (element) / 2022793 HIT-HY 200-R (adhesive)
Effective embedment depth:	$h_{ef,act}$ = 8.500 in. ($h_{ef,limit}$ = - in.)
Material:	ASTM F 593
Evaluation Service Report:	ESR-3187
Issued I Valid:	5/1/2021 | 3/1/2022
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate[R] :	l_x x l_y x t = 14.000 in. x 14.000 in. x 0.375 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS8X8X.1875; (L x W x T) = 8.000 in. x 8.000 in. x 0.188 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 18.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: present, shear: present; no supplemental splitting reinforcement present
	edge reinforcement: > No. 4 bar
Seismic loads (cat. C, D, E, or F)	Tension load: yes (17.10.5.3 (d))
	Shear load: yes (17.10.6.3 (c))

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



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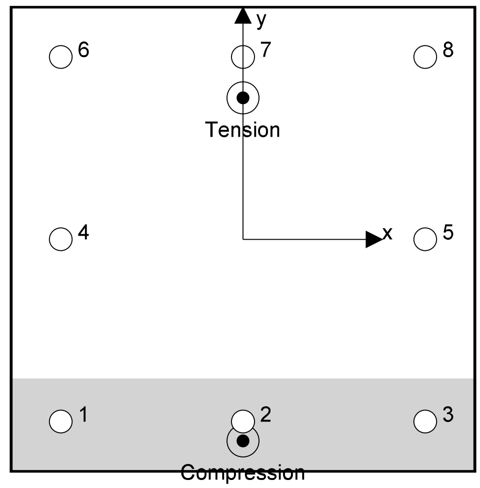
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 1,152$; $V_x = 1,801$; $V_y = 0$; $M_x = 180,962$; $M_y = 0$; $M_z = 0$;	yes	100

2 Load case/Resulting anchor forces



Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	0	225	225	0
2	0	225	225	0
3	0	225	225	0
4	2,044	225	225	0
5	2,044	225	225	0
6	4,695	225	225	0
7	4,695	225	225	0
8	4,695	225	225	0

max. concrete compressive strain: 0.20 [‰]
max. concrete compressive stress: 882 [psi]
resulting tension force in (x/y)=(0.000/4.262): 18,173 [lb]
resulting compression force in (x/y)=(0.000/-6.081): 17,021 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	4,695	14,690	32	OK
Bond Strength**	18,173	18,322	100	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	18,173	19,666	93	OK

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3187

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	100,000

Calculations

N_{sa} [lb]
22,600

Results

N_{sa} [lb]	ϕ_{steel}	$\phi_{nonductile}$	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
22,600	0.650	1.000	14,690	4,695

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3.2 Bond Strength

$N_{ag} = \left(\frac{A_{Na}}{A_{Na0}}\right) \psi_{ec1,Na}\ \psi_{ec2,Na}\ \psi_{ed,Na}\ \psi_{cp,Na}\ N_{ba}$ ACI 318-19 Eq. (17.6.5.1b)

$\phi\ N_{ag} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Na} see ACI 318-19, Section 17.6.5.1, Fig. R 17.6.5.1(b)

$A_{Na0} = (2\ c_{Na})^2$ ACI 318-19 Eq. (17.6.5.1.2a)

$c_{Na} = 10\ d_a \sqrt{\frac{\tau_{uncr}}{1100}}$ ACI 318-19 Eq. (17.6.5.1.2b)

$\psi_{ec,Na} = \left(\frac{1}{1 + \frac{e'_N}{c_{Na}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.3.1)

$\psi_{ed,Na} = 0.7 + 0.3\left(\frac{c_{a,min}}{c_{Na}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.4.1b)

$\psi_{cp,Na} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{c_{Na}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.5.5.1b)

$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \alpha_{N,seis} \cdot \pi \cdot d_a \cdot h_{ef}$ ACI 318-19 Eq. (17.6.5.2.1)

Variables

$\tau_{k,c,uncr}$ [psi]	d_a [in.]	h_{ef} [in.]	$c_{a,min}$ [in.]	$\alpha_{overhead}$	$\tau_{k,c}$ [psi]
2,261	0.625	8.500	12.000	1.000	1,192

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a	$\alpha_{N,seis}$
0.000	0.962	16.932	1.000	0.990

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
8.920	673.10	318.25	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	0.903	1.000	19,687

Results

N_{ag} [lb]	ϕ_{bond}	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
37,584	0.650	0.750	1.000	18,322	18,173

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
8.000	0.000	0.962	12.000	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
16.932	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,032.50	576.00	1.000	0.926	1.000	1.000	21,069

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
34,963	0.750	0.750	1.000	19,666	18,173

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	225	5,695	4	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Concrete Breakout Strength controls)**	1,801	62,732	3	OK
Concrete edge failure in direction x+**	1,801	15,040	12	OK

* highest loaded anchor **anchor group (relevant anchors)

4.1 Steel Strength

$V_{sa,eq}$ = ESR value refer to ICC-ES ESR-3187

ϕ $V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]	$\alpha_{V,seis}$
0.23	100,000	0.700

Calculations

$V_{sa,eq}$ [lb]
9,492

Results

$V_{sa,eq}$ [lb]	ϕ_{steel}	$\phi_{nonductile}$	ϕ $V_{sa,eq}$ [lb]	V_{ua} [lb]
9,492	0.600	1.000	5,695	225

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4.2 Pryout Strength (Concrete Breakout Strength controls)

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	12.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f_c' [psi]
1.000	16.932	17	1.000	3,000

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
1,225.00	576.00	1.000	1.000	1.000	1.000	21,069

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
89,616	0.700	1.000	1.000	62,732	1,801

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4.3 Concrete edge failure in direction x+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\ \psi_{ed,V}\ \psi_{c,V}\ \psi_{h,V}\ \psi_{parallel,V}\ V_b$ — ACI 318-19 Eq. (17.7.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ — ACI 318-19 Table 17.5.2

A_{Vc} see ACI 318-19, Section 17.7.2.1, Fig. R 17.7.2.1(b)

$A_{Vc0} = 4.5\ c_{a1}^2$ — ACI 318-19 Eq. (17.7.2.1.3)

$\psi_{ec,V} = \left(\frac{1}{1 + \frac{e_V'}{1.5c_{a1}}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.3.1)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ — ACI 318-19 Eq. (17.7.2.4.1b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ — ACI 318-19 Eq. (17.7.2.6.1)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2} \sqrt{d_a}\right) \lambda_a \sqrt{f_c'}\ c_{a1}^{1.5}$ — ACI 318-19 Eq. (17.7.2.2.1a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
12.000	12.000	0.000	1.200	18.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
5.000	1.000	0.625	3,000	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
630.00	648.00	1.000	0.900	1.000	19,098

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi_{seismic}$	$\phi_{nonductile}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
20,053	0.750	1.000	1.000	15,040	1,801

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.992	0.120	1.000	93	OK

$\beta_{NV} = (\beta_N + \beta_V) / 1.2 <= 1$

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2018, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- "An anchor design approach for structures assigned to Seismic Design Category C, D, E or F is given in ACI 318-19, Chapter 17, Section 17.10.5.3 (a) that requires the governing design strength of an anchor or group of anchors be limited by ductile steel failure. If this is NOT the case, the connection design (tension) shall satisfy the provisions of Section 17.10.5.3 (b), Section 17.10.5.3 (c), or Section 17.10.5.3 (d). The connection design (shear) shall satisfy the provisions of Section 17.10.6.3 (a), Section 17.10.6.3 (b), or Section 17.10.6.3 (c)."
- Section 17.10.5.3 (b) / Section 17.10.6.3 (a) require the attachment the anchors are connecting to the structure be designed to undergo ductile yielding at a load level corresponding to anchor forces no greater than the controlling design strength. Section 17.10.5.3 (c) / Section 17.10.6.3 (b) waive the ductility requirements and require the anchors to be designed for the maximum tension / shear that can be transmitted to the anchors by a non-yielding attachment. Section 17.10.5.3 (d) / Section 17.10.6.3 (c) waive the ductility requirements and require the design strength of the anchors to equal or exceed the maximum tension / shear obtained from design load combinations that include E, with E increased by ω_0.
- Installation of Hilti adhesive anchor systems shall be performed by personnel trained to install Hilti adhesive anchors. Reference ACI 318-19, Section 26.7.

Fastening meets the design criteria!

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7 Installation data

Profile: Square HSS (AISC), HSS8X8X.1875; (L x W x T) = 8.000 in. x 8.000 in. x 0.188 in.

Hole diameter in the fixture: d_f = 0.687 in.

Plate thickness (input): 0.375 in.

Recommended plate thickness: not calculated

Drilling method: Hammer drilled

Cleaning: Compressed air cleaning of the drilled hole according to instructions for use is required

Anchor type and diameter: HIT-HY 200 + HAS-R 304/316 SS 5/8

Item number: 408988 HAS-R 316 SS 5/8"x12" (element) / 2022793 HIT-HY 200-R (adhesive)

Maximum installation torque: 720 in.lb

Hole diameter in the base material: 0.750 in.

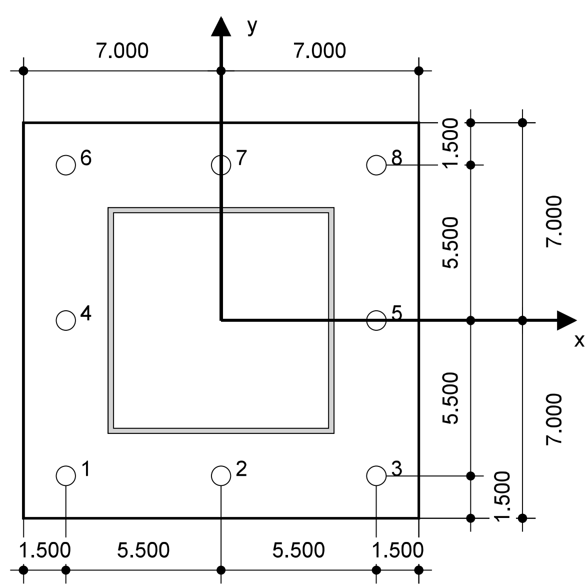
Hole depth in the base material: 8.500 in.

Minimum thickness of the base material: 10.000 in.

5/8 Hilti HAS Stainless steel threaded rod with Hilti HIT-HY 200 Safe Set System

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer • Properly sized drill bit	• Compressed air with required accessories to blow from the bottom of the hole • Proper diameter wire brush	• Dispenser including cassette and mixer • Torque wrench



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-5.500	-5.500	12.000	23.000	12.000	23.000
2	0.000	-5.500	17.500	17.500	12.000	23.000
3	5.500	-5.500	23.000	12.000	12.000	23.000
4	-5.500	0.000	12.000	23.000	17.500	17.500
5	5.500	0.000	23.000	12.000	17.500	17.500
6	-5.500	5.500	12.000	23.000	23.000	12.000
7	0.000	5.500	17.500	17.500	23.000	12.000
8	5.500	5.500	23.000	12.000	23.000	12.000

8 Remarks; Your Cooperation Duties

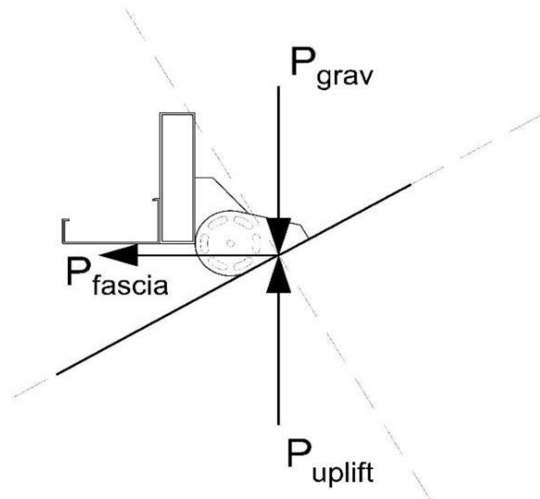
- Any and all information and data contained in the Software concern solely the use of Hilti products and are based on the principles, formulas and security regulations in accordance with Hilti's technical directions and operating, mounting and assembly instructions, etc., that must be strictly complied with by the user. All figures contained therein are average figures, and therefore use-specific tests are to be conducted prior to using the relevant Hilti product. The results of the calculations carried out by means of the Software are based essentially on the data you put in. Therefore, you bear the sole responsibility for the absence of errors, the completeness and the relevance of the data to be put in by you. Moreover, you bear sole responsibility for having the results of the calculation checked and cleared by an expert, particularly with regard to compliance with applicable norms and permits, prior to using them for your specific facility. The Software serves only as an aid to interpret norms and permits without any guarantee as to the absence of errors, the correctness and the relevance of the results or suitability for a specific application.
- You must take all necessary and reasonable steps to prevent or limit damage caused by the Software. In particular, you must arrange for the regular backup of programs and data and, if applicable, carry out the updates of the Software offered by Hilti on a regular basis. If you do not use the AutoUpdate function of the Software, you must ensure that you are using the current and thus up-to-date version of the Software in each case by carrying out manual updates via the Hilti Website. Hilti will not be liable for consequences, such as the recovery of lost or damaged data or programs, arising from a culpable breach of duty by you.

Work Prepared For:	StruXure Outdoor of Washington
Project:	21-45903 - Baker, Brian
Detail:	PIVOT ROOF MOUNT CONNECTION

Loading

Design Uplift ("Uplift") =	1.98 psf
Design Gravity Load ("Grav") =	33.00 psf
Design Fascia Load ("WL") =	23.15 psf

Roof pitch:	3 / 12	
Roof angle:	14°	
W =	9.33 ft	Tributary Width
S=	4.00 ft	Roof Pivot Mount Spacing



P_{grav} = 1232 lb
P_{uplift} = 74 lb
P_{fascia} = 82 lb

V_{grav} = 379 lb
V_{uplift} = 98 lb
T_{uplift} = 72 lb

V_u = 379 lb
T_u = 72 lb

Client: StruXure Outdoor of Washington		Job#: 21-45903
Project: Baker, Brian		Date: 10/14/21
Detail/member: Roof Mount Pivot Wood Connections	Calc. by: ZRV	Chk'd by: AEM

WOOD CONNECTION DESIGN — 2012 NDS - ASD

Load scenario: Single shear
Load duration: 1.6
Temperature: T<= 100°F
Exposure service: Wet Conditions
Moisture at fabric.: <= 19%
Moisture in service: <= 19%
Bending yield strength, Fyb: 70,000 psi

FASTENER

0.250 in	Nominal diameter
0.173 in	Root diameter
0.000 in	Washer thickness
1.094 in	Screw thread length
0.156 in	Length of tapered tip

1/4" Ø x1.5'' Lag Screws

MAIN MEMBER	SIDE MEMBER	
Wood	Aluminum	Material
Visually Graded Dimension Lumber	N/A	Wood type
Douglas Fir-Larch	6063-T6	Specie/Grade/Alloy
No. 2	N/A	Wood grade
2"x6"	N/A	Nominal size
0.50	N/A	Specific gravity, G
5.500 in	N/A	Member depth, d
1.500 in	N/A	Member thickness, ts, tm
	2.000 in	Member/plate depth
	0.125 in	Member/plate thickness
1,600 ksi	10,100 ksi	Modulus of Elasticity, E
90 °	0 °	Max. angle of load to grain
4,465 psi	31,000 psi	Actual dowel Bearing strength, Fe
5,600 psi	31,000 psi	Dowel Bearing strength, FeII
4,465 psi	31,000 psi	Dowel Bearing strength, FeT

Connection Geometry

LATERAL LOADING				WITHDRAWAL	ACTUAL	
MAIN MEMBER		SIDE MEMBER				
FULL VALUE	MIN.	FULL VALUE	MIN.	MIN.		
2 in (8Ø)	1 in (4Ø)	-	-	-	1.375 in	Penetration into main member
1 in (4Ø)	0.5 in (2Ø)	N/A	N/A	1 in (4Ø)	2.000 in	End distance, End
-	1 in (4Ø)	-	N/A	0.38 in (1.5Ø)	1.000 in	Loaded edge distance, Edl
-	0.38 in (1.5Ø)	-	N/A		1.000 in	Unloaded edge distance, Edu
-	-	-	-	-	1	Number of anchors in a row
-	-	-	-	-	3	Number of rows
1 in (4Ø)	0.75 in (3Ø)	N/A	N/A	1 in (4Ø)	1.000 in	Spacing for fasteners in a row, s
-	0.8 in (2<ls/D<6)	-	N/A	-		Spacing in between rows, Sr
lm/D = 4.88		ls/D = N/A				Bolt slenderness

Lateral Load Capacity

D =	0.173	*Root diameter, in*
lm =	1.219	*= Main member dowel bearing length, in*
ls =	0.125	*= Side member dowel bearing length, in*
$Fe\theta_m$ =	4,465 psi	*= Dowel bearing at an angle to Grain: $((FeII_m)*(FeT_m))/((FeII_m)*\sin^2\theta+(FeT_m)*\cos^2\theta)$*
$Fe\theta_s$ =	31,000 psi	*= Dowel bearing at an angle to Grain: $((FeII_s)*(FeT_s))/((FeII_s)*\sin^2\theta+(FeT_s)*\cos^2\theta)$*
Re =	0.144	*= Fem/Fes*
Rt =	9.750	*= lm/ls*
Kθ =	1.250	*= 1+ θ/360*
KD =	2.230	*= 10*D+0.5*
k1 =	0.565	*= (SQRT(Re+2*Re^2*(1+Rt+Rt^2)+Rt^2*Re^3)-Re*(1+Rt))/(1+Re)*
k2 =	0.600	*= -1+SQRT(2*(1+Re)+(2*Fyb*(1+2*Re)*D^2)/(3*Fem*lm^2))*
k3 =	6.668	*= -1+SQRT(2*(1+Re)/Re+(2*Fyb*(2+Re)*D^2)/(3*Fem*ls^2))*

Yield Mode	*Rd*	**Z** (single shear)		**Z** (double shear)	
I_m	2.79	338 lbs	*= D*lm*Fem/Rd*	338 lbs	*= D*lm*Fem/Rd*
I_s	2.79	240 lbs	*= D*ls*Fes/Rd*	481 lbs	*= 2D*ls*Fes/Rd*
II	2.79	136 lbs	*= k1*D*ls*Fes/Rd*		
III_m	2.79	157 lbs	*= k2*D*lm*Fem/((1+2*Re)*Rd)*		
III_s	2.79	108 lbs	*=k3*D*ls*Fem/((2+Re)*Rd)*	215 lbs	*=2k3*D*ls*Fem/((2+Re)*Rd)*
IV	2.79	145 lbs	*= D²/Rd*(2*Fem*Fyb/(3*(1+Re)))^½*	290 lbs	*= 2D^2/Rd*(2*Fem*Fyb/(3*(1+Re)))^0.5*

Z =	108 lbs	Minimum of yield mode values above
C_D =	1.60	Load duration factor
C_M =	1.00	Wet service factor for connections
C_t =	1.00	Temperature factor for connections
C_g =	1.00	Group action factor
C_Δ =	1.00	Geometric factor
C_{eg} =	1.00	End grain factor
Z =	172 lbs	Factored lateral load capacity (160%): Z'= (Z)(CD)(CM)(Ct)(Cg)(CD)(Ceg)(Cd)
ZT =	**517 lbs**	Total capacity of connection for lateral loads

Withdrawal load capacity for wood at main member:

Lag Screws or screws:

W =	225 lbs/in	Nominal design value in pounds per inch of penetration: 1800*(G)^(3/2)(d)^(3/4)
L =	1.219 in	Total length of lag screw into main member (shall not include length of tapered tip)
L' =	1.094 in	Total length of thread penetration into main member (all threads into main member)
W =	246 lbs	Nominal design value per fastener
C_D =	1.60	Load duration factor
C_M =	1.00	Wet service factor for connections
C_t =	1.00	Temperature factor for connections
C_{eg} =	1.00	End grain factor
C_{edge} =	1.00	Edge distance factor
W =	394 lbs	Factored withdrawal load capacity (160%): W'= (W)(CD)(CM)(Ct)(Ceg)(Cd)
WT =	**1,181 lbs**	Total capacity of connection for Withdrawal loads

Thru bolts:

F_{cp} =	N/A	Compression perpendicular to grain design value (Per NDS supplemental tables)
W_d =	1.000 in	Washer diameter
A_w =	0.785 in²	Washer bearing area
C_M =	0.97	Wet service factor
C_t =	1.00	Temperature factor
Ci =	0.80	Incising factor
C_b =	1.00	Bearing area factor
F'_{cp} =	N/A	Factored compression perpendicular to grain: (Fcp)(CM)(Ct)(Ci)(Cb)
Tcap =	N/A	Tension capacity per anchor
Tcap =	**N/A**	Wood capacity of connection for tension loads*

* Bolt tension capacity must be verified and the lesser value shall be adopted.