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Project # 22268

CALCULATION COVER SHEET

Calculations Prepared For:

AMERICAN PATIO & FIREPLACE
618 NW 60TH SUTIE F
GAINESVILLE , FL
(352) 332-4433

Project:

JENKINS RESIDENCE
5310 SE COUNTRY CLUB RD
LAKE CITY, FL

Subject:

CANOPY CALCULATIONS

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

COA # 35662

ANDREW McCANN
LICENSE
No 92471
STATE OF FLORIDA
PROFESSIONAL ENGINEER

SEAL EXP:
02/28/2023

Andrew McCann

Engineer's Seal Valid For Pages
1 Through 42

FL

07/22/22

ANDREW McCANN
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Cert Auth 35662

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Project # 22268

American Patio & Fireplace
Jenkins Residence

Wind Loading Criteria (ASCE 7-16)

Basic Wind Speed	120	MPH	ASCE	7-16
Wind Velocity (Vasd)	93	MPH		ASD
Risk Category	II			Residential
Importance Factor	1.00			
Exposure Category	C			

Snow Loading Criteria (ASCE 7-16)

Ground Snow Load	0	PSF
Flat Roof Snow Load	5.00	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	5	PSF	Azenco

Seismic Load Criteria (ASCE 7-16)

Site Class	D	Host Attached?	N
Occupancy Category	II	Host Supported?	N

Mapped Spectral Response Accelerations:

S_S	0.084
S_1	0.050

Spectral Response Coefficients:

S_{DS}	0.090
S_{D1}	0.080
P	1.0
SDC	B
TL	8

Load Combinations (ASCE 7-16)

Gravity D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)

Uplift 0.6D + 0.6W

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Project # 22268 - Jenkins Residence

American Patio & Fireplace
Jenkins Residence

DESIGN CRITERIA:

Enter custom loads:

Vult =	120 mph		
Exposure:	C		
Ground Snow Load:	0.00 psf		
Live Load:	20.00 psf	Type of project:	Residential
Dead Load:	5.0 psf		
Wind Porosity:	50%		
Roof Type:	Louvered		

These are the loads that this calculator will utilize:

Vult =	120 mph		
Exposure:	C	Deflection criteria:	L / 180
Ground Snow Load:	0.00 psf		
Design Live Load:	20.00 psf		
Design Dead Load:	5.00 psf		
Wind Porosity:	50%		For seismic design, see column calculations
Critical positive grav comb. (+):	25.65 psf		
Critical negative uplift comb. (-):	- 4.46 psf		
Critical lateral pressure (+):	19.95 psf		

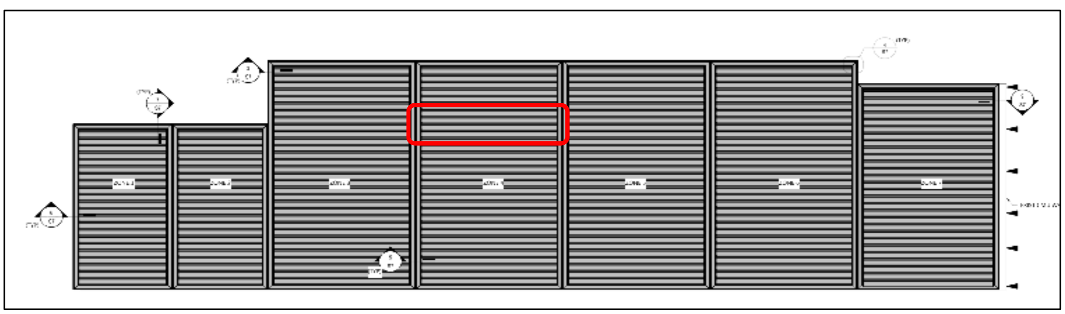
SYSTEM CONFIGURATION:

Louvers:

Overall Canopy Length:	15.3 ft
Overall Canopy Width:	15.0 ft
Roof Slope:	0.0 º

LOUVER BLADES OPEN CHECK 6063-T6

Sx=	0.7137	in^3	Mmax	189.70	**lb-ft**		
Sy=	1.26438	in^3	Stress	3.19	**ksi**	Stress Check:	29.5%
Length of Longest Louver Blade:	15 ft	0 in				Louver Length:	15.0 ft



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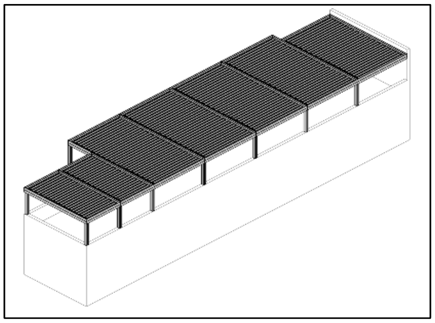
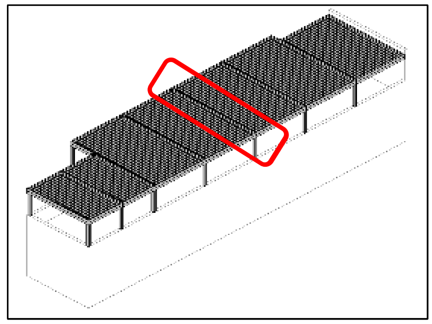
Louver Support Beam

Check intermediate or edge? Edge

Edge Louver Beam Configuration



Intermediate Louver Beam Configuration



Louver Support Beam - Edge Condition Analysis

Support Spacing (Louver Beam Length): 15 ft | 4 in

Reinf: N

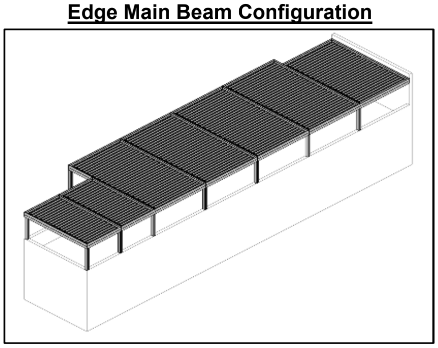
Single/Double/Triple/Quad: Single

Louver Beam Span:	15.3 ft
Louver Beam Trib:	7.5 ft
Shear at Ends:	1475 lb
Moment Check	48%
Deflection Check	29%

Main Beam

Check intermediate, edge, or none? Edge

Edge Main Beam Configuration



Main Beam - Edge Condition Analysis

Post Spacing (Main Beam Length): 15 ft | 0 in

Reinf: N

Single/Double/Triple/Quad: Single

Quantity of louver between a set of posts: 0

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

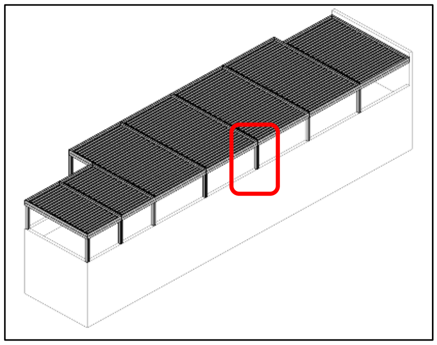
Main Beam Span:	15.0 ft
Load from Louver Beam:	0 lb
Shear at Ends:	0 lb
Moment Check	0%
Deflection Check	0%

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

Support Posts

Check intermediate, edge, or none? Edge

Edge Support Post Configuration



Mounting Height Above Grade: 0.0 ft F+ RQP P P P P H

Height of Posts: 10.0 ft

Attached to host? N

Total Mean Roof Height:	10.0 ft

OPEN

Support Posts - Edge Condition Analysis

Post Size: 6.5" x 6.5" x 0.125"

P,P RP P P_P khxk

P,P RP P P_P kh Q

^Wz P_P z2hzM 3F P P P PW z3P P P P H

	Max Moment/Axial/Shear	39%
	Moment/Axial Check:	39%
	Shear Check	3%
ASD Method	Required Tension:	257 lb
	Required Compression:	1475 lb
	Required Shear:	405 lb
	Required Moment	2.34 kip-ft

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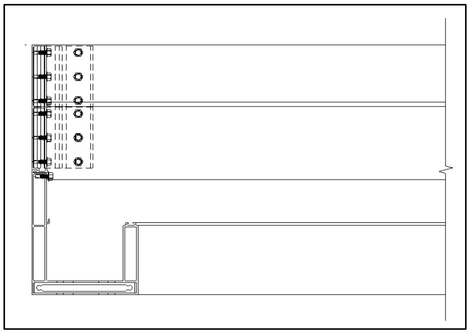
Beam to Beam (Clip in Shear)



6	Anchor Qty at Connection per Beam

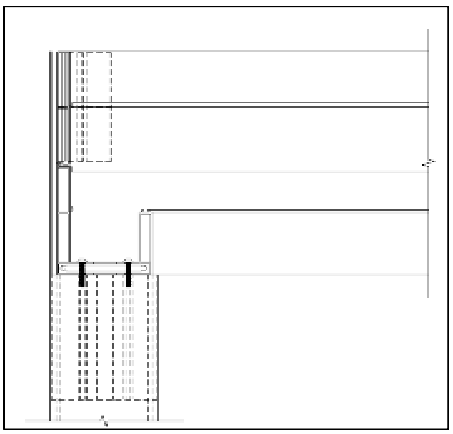
Required Tension:	0 lb
Required Shear:	1475 lb

F* Fzv2,z2P ρ 3MzxP 3P F
FP FQz/'5P FQz/'5P P
P H

48%

OK, (6) anchors sufficient

Beam to Post Insert Connection



4	Anchor Qty at Connection

F	5148.59 lb	Ω = 4
Fsu	60000.00	
Ats	0.34 in^2	
n	20.00 threads/in	
LE	1.57 in	
Dsmin	0.31 in	
Enmax	0.28 in	

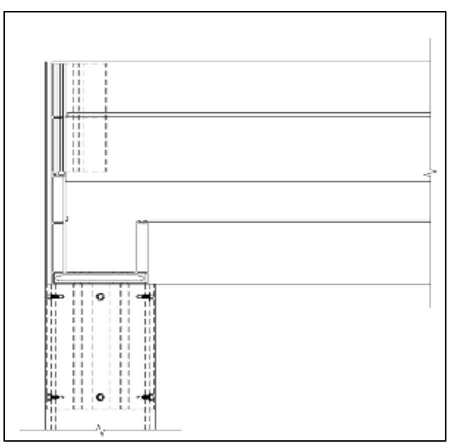
Required Tension:	257 lb
Capacity Shear:	20594 lb

* FΩ P P FR P FF2HP
ρ 8 2QFρ P

1%

OK

Post Insert to Post Connection



4	Anchor Qty at Connection

Required Tension:	0 lb
Required Shear:	405 lb

F* Fzv2,z2P ρ 3MzxP 3P F
FP FQz/'5P FQz/'5P P
P H

20%

OK, (4) anchors sufficient

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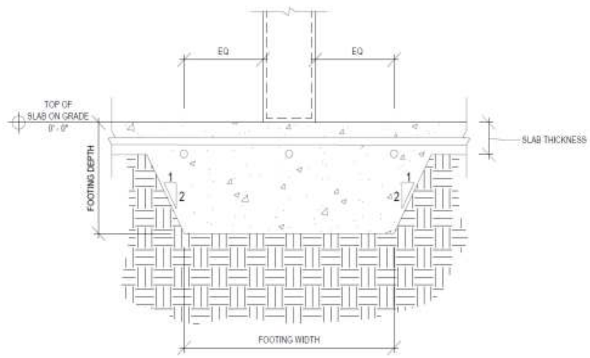
Foundation

Isolated Footing

Isolated Footing

Square	Footing Dimensions:	Wx = 36 in	Wy = 36 in	D = 4 in
No Slab				



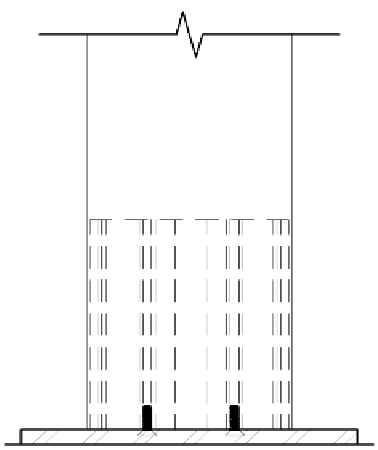
Gravity	1500 psf	Min Soil Bearing Pressure (to be verified by General Contractor)
	1475.1 lb	Gravity load at column
Uplift	450.0 lb	Total Weight
	256.5 lb	Uplift load at column

OK SQUARE OR RECTANGLE, soil allowable bearing pressure (1500 psf) not exceeded at

OK, factor of safety FOS = 1.75 >1.0

Baseplate	4-Bolt		**Spacing:**	8.625	in	86%	**Okay**
Plate Thickness:	0.75	in	**Width:**	10.625	in	23%	**Okay**

Post Insert to Base Plate Connection



4 Anchor Qty at Connection

F	5148.59 lb	Ω = 4
Fsu	60000.00	
Ats	0.34 in^2	
n	20.00 threads/in	
LE	1.57 in	
Dsmin	0.31 in	
Enmax	0.28 in	

Required Tension:	11826 lb
Capacity Shear:	20594 lb

* RΩ P P RR P RR2HP
ρ 8 2QRρ P

57%

OK

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Work Prepared For:	American Patio & Fireplace
Project:	22268 - Jenkins Residence

DESIGN CRITERIA:

H =	10.00	ft, Mean Roof Height	ASCE:	7-16
Θ =	0.0 º	Roof Slope F= 0.0000	Exposure:	C
Vult =	120	mph, Wind Velocity (3-Second Gust)	Building Category:	II
Kd =	0.85	Directionality Factor		
G =	0.85	Gust Effect Factor	Snow:	N
Kz =	0.85	Velocity Pressure Coefficient	Ground Snow Load:	0.00 psf
Kzt =	1	Topographic Factor	Design Snow Load:	0.00 psf
			Design Live Load:	20.00 psf
			Design Dead Load:	5.0 psf
	Wind Flow:	Clear	Wind Porosity:	50%
L =	15.33	ft, Overall Canopy Length	Method:	ASD
W =	15.00	ft, Overall Canopy Width	Live Load Lr:	19.40 psf
a =	3.00 ft		Reduction Per Lo:	20.00 psf
			IBC R1:	0.97
			1067.13.2.1 R2:	1

LOADS ON COMPONENTS & CLADDING:

(Roof Decking and Decking Fasteners)

L1 =	15.00	ft, Effective Deck Panel Length	
W1 =	5.00 ft	Effective Deck Panel Width	
A =	75.00 ft^2	Effective Wind Area, L1*W1	A > 4.0*a^2
CNp =	1.2	Positive Pressure Coefficient	
CNn =	-1.1	Negative Pressure Coefficient	
qz =	13.30 psf	Velocity Pressure w/ Porosity	
WLp =	13.57 psf	Positive Wind Load, = qz*G*CNp	
WLn =	-12.44 psf	Negative Wind Load, = qz*G*CNn	
Grav =	**25.65 psf**	**D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)**	**Critical positive DP**
Uplift =	**-4.46 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 =	1.2	Critical Positive Pressure Coefficient	
CNn =	-1.1	Critical Negative Pressure Coefficient	
WLp =	13.57 psf	Critical Positive Wind Load, = qz*G*CNp	
WLn =	-12.44 psf	Critical Negative Wind Load, = qz*G*CNn	
Grav =	**25.65 psf**	**D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)**	**Critical positive DP**
Uplift =	**-4.46 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 =	**19.95 psf**	**Average Wind Load on Fascia, qz*GCpn*.06**

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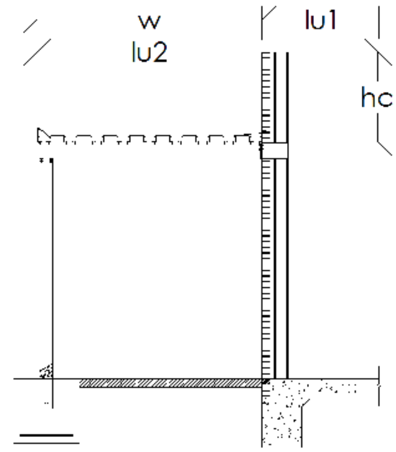

Snow Loads

Pg =	0	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 =	14.00 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 =	5.00 psf	Snow load on flat roofs (slope < 5°): Pf = max[(I)(20),(0.7)(Ce)(Ct)(I)(Pg)]
Ps =	5.00 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=	10.00	ft, Height from top of lower roof to top of eave

hb=	0.36 ft	Height of balanced snow: Ps/(γ)
hd1=	0.58 ft	Height of snow drift (Fig 7-9): 0.43(lu)^(1/3)(Pg+10)^(1/4)-1.5 (Leeward)
hd2=	0.29 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)

Is Pg 20 PSF or less? NO

Unreducible Snow Load: No
Include Uniform Dist. Ice Load? Yes

Snow Porosity: 30%

hd=	0.58 ft	Governing drift height
w=	2.30 ft	Governing drift width
hend=	0.00 ft	Drift height at edge of lower roof
pd=	0.00 psf	Surcharge load Uniform Distribution Over Drift Width
	0.00 psf	Surcharge Load Distributed over Tributary Area

SL= 0.00 psf Total snow load (balanced + drift snow distribution) * (1 - Snow Porosity)

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

Acounting for Accumulating Ice on Louver Blades

t_i =	0.00	Nominal Ice Thickness (in.)		
K_{zt} =	1.0	Topographic Factor		
Z =	10.00 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.	#REF!
Width (bf)	1.866 in.	#REF!
Thickness	.065 in.	#REF!
Length	15.00 ft	15.33 ft

Per Table 10-1

t_d = 0.00 in, Design Ice Thickness — $t_d = t_i{*}I_i{*}f_z{*}(K_{zt})^{0.35}$

W_i = 0.00 psf Weight of Ice (for td) — $W_i = (td/12){*}I_d$

F_z = 0.8875 — $F_z = (Z/33)^{0.1}$

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 = .00 in^2 Area of Ice = $\pi\, t_d{*}(D_c+t_d)$

$W_{i(Louver)}$ = 0.00 plf Uniform Distributed Ice Load (Single Louver Blade)

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

Louver Beam Ice Loading from Louver Blades

$W_{i(Louver)}$ = 0.00 plf Distributed Ice Load on Louver Blade

L= 15.00 ft Length of Longest Louver Blade

$W_{i(Beam)}$= 0.0 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)

$Wi_{(Louver)}$= 0.00 plf Uniform Linear Ice Load (Louver Blade)

$W_{i(Beam)}$= 0.00 plf Uniform Linear Ice Load (Louver Beam)

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

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Seismic Loads Criteria

S_s =	0.084	Max considered response acceleration for a period of 0.2 s	
S_1 =	0.050	Max response acceleration at period of 1 s	

Height of Structure = 10.00 ft   Attached to host structure? N

Site Class	D		
F_a =	1.6	short period amplification factor	
F_v =	2.4	long period amplification factor	
S_{MS} =	0.134	modified spectral response acceleration at a period of 0.2 s	F_a*S_s
S_{M1}=	0.120	modified spectral response acceleration at a period of 1.0 s	F_v*S_1

Spectral Response Acceleration Parameters

S_{DS} =	0.090	Design spectral response acceleration at a period of 0.2 s	(2/3)*S_{ms}
S_{D1}=	0.080	design spectral response acceleration at a period of 1.0 s	(2/3)*S_{M1}

Structural Design Requirements

T_a =	0.112		approximate fundamental period (s)	C_t*h_n^x
T_L =	8.0		Long Transition Period (s)	
E_V=	0.063		Vertical Seismic Loads (PSF)	
R=	1.25		G.2 Steel Ordinary Cantilever Column System	
Cs=	0.072	SDS/(R/Ie)	Seismic Response Coefficient	
CSMax=	0.569	SD1/(Ta*(R/Ie)	CSMin= 0.020	0.5*S1/(R/Ie)
Ie=	1			
W=	287.50 lbs		Tributary Weight	
V=	20.61 lbs		Seismic Base Shear (Cs*W)	

		Ω=	1.25
P=	1	SERVICE =	0.7
144.26 lb-ft	Effective Seismic Moment		(H*V)

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Project: 22268 - Jenkins Residence

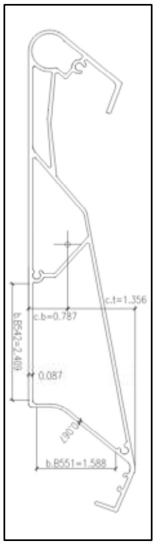
ALUMINUM DESIGN MANUAL (2020 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

LOUVER BLADE CHECK

Design Check of Azenco Louver Blade0.087"x0.087"x9.9627"/17.3769" 6063-T6 Aluminum Tube

Alloy:	6063	Temper:	T6	Critically Welded:	N

MEMBER PROPERTIES



Flange width	2.143"
Flange thickness	0.087"
Web height	9.963"
Web thickness	0.087"
Moment of inertia about axis parallel to flange	17.38 in^4
Moment of inertia about axis parallel to web	1.04 in^4
Section modulus compression x-axis	3.16 in^3
Section modulus tension x-axis	3.95 in^3
Section modulus tension y-axis	0.78 in^3
Section modulus compression y-axis	1.31 in^3
Radius of gyration about centroidal axis parallel to flange	2.92 in
Radius of gyration about centroidal axis parallel to web	0.71 in
Torsion constant	1.26 in^4
Cross sectional area of member	2.04 in^2
Plastic section modulus	5.19 in^3
Warping constant	5.73 in^6

MEMBER SPANS

Unsupported member length (between supports)	VP	15.0 ft
Unbraced length for bending (between bracing against side-sway)	V P	15.0 ft
Effective length factor	P	1.0
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)	Ω P	27.64 ksi	
Compression in columns & beam flanges (Slope)	Z P	0.14 ksi	
Compression in columns & beam flanges (Intersection)	> P	78.38 ksi	
Compression in flat plates (Intercept)	Ω P	31.39 ksi	
Compression in flat plates (Slope)	Z P	0.17 ksi	
Compression in flat plates (Intersection)	> P	73.55 ksi	
Compressive bending stress in solid rectangular bars (Intercept)	Ω P	46.12 ksi	
Compressive bending stress in solid rectangular bars (Slope)	Z P	0.38 ksi	
Shear stress in flat plates (Intercept)	Ω P	18.98 ksi	
Shear stress in flat plates (Slope)	Z P	0.08 ksi	
Shear stress in flat plates (Intersection)	> P	94.57 ksi	
Ultimate strength coefficient of flat plates in compression (slenderness limit λ2)	z P	0.35	
timate strength coefficient of flat plates in compression (stress for slenderness > λ2)	/ P	2.27	
Ultimate strength of flat plates in bending (slenderness limit λ2)	z P	0.50	
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	/ P	2.04	
Tension coefficient	P	1.0	

D.2 Axial Tension

Tensile Yielding - Unwelded Members	^	Fty_n =	25.00 ksi	
		Ω =	1.65	
		Fty_n/Ω =	15.15 ksi	
Tensile Rupture - Unwelded Members	^	^ P	30.00 ksi	
		Ω =	1.95	
		Ftu_n/Ω t =	15.38 ksi	

AXIAL COMPRESSION MEMBERS

E.2 Compression Member Buckling

Axial, gross section subject to buckling	Lower slenderness limit	zP	18.23	
	Upper slenderness limit	/ P	78.38	
	Slenderness	F HP	252.21	≥ λ2
	Q'8' +w	^ P	1.33 ksi	
		Ω =	1.65	
		Fc_n/Ω =	0.81 ksi	

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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.

ΩΉ ΉΖΉ ΡΌΡ^ P P P P P ΡΓ^

ΩΉ ΉΖΉ ΡΌΡ^ P P P P P ΡΓ H

E.4 Buckling Interaction

Per Table B.5.1 P 1+ΨΓΖhx1 w H ^ F	HP	76.04 ksi	
P^ ^	P	1.33 ksi	
Fe(flange) > Fc_n (E.2 Member Buckling)	P	1.65	
	Fc_n/□ =	0.81 ksi	
P 1+ΨΓΖhx1 w H ^ F	HP	3.08 ksi	
P^ ^	P	1.33 ksi	
^ F HΑP^ ΓΓ+hΉρ ΓΩ H	P	1.65	
	Fc_n/□ =	0.81 ksi	

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture — Limit State of Yielding			
1^	Mnp =	78.88 k-in	
ρ w	Fb_n =	25.00 ksi	
	Ω =	1.65	
	Fb_n/□ =	15.15 ksi	
Limit State of Rupture			
1^ w	Mnu =	155.83 k-in	
ρ w	Fb_n =	30.00 ksi	
	Ω =	1.95	
	Fb_n/□ =	15.38 ksi	

F.4 Lateral-Torsional Buckling

Unsymmetric shape subject to lateral-torsional buckling

Slenderness for any close shape about the bending axis	P^ΉΖΉ Ή P	51.18	
Maximum slenderness	F HP	51.18	< Cc
Nominal flexural strength - lateral-torsional buckling			
ρ Γz,Fw HHF 1+1 1 w MH	Mnmb =	60.80 k-in	
ρ w	Fb_n =	19.27 ksi	
	Ω =	1.65	
	Fb_n/□ =	11.68 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	zP	22.8	
Upper slenderness limit	/P	39.2	
Flange Slenderness	w P	22.63	≤ λ1
Web Slenderness	w P	112.51	≥ λ2
P^ ^	zP	25.00 ksi	
	Ω =	1.65	
	Fc_n1/□ =	15.15 ksi	
P / 1 ΓΩ 1+HΓzhx1 w H ^	/P	7.10 ksi	
	Ω =	1.65	
	Fc_n2/□ =	4.30 ksi	

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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	zP	34.73	
Upper slenderness limit	/ P	92.95	
Slenderness	w P	112.51	≥ λ2
P / 1 FΩ 1+HF 1 w H	^ P	19.04 ksi	
	Ω =	1.65	
	Fb_n/ =	11.54 ksi	

SHEAR

G.2 Shear Supported on Both Edges - Web

Members with flat elements supported on both edges

Lower slenderness limit	zP	38.73	
Upper slenderness limit	/ P	75.65	
Slenderness	w P	112.51	≥ λ2
+wFzh' 1 w H	^ P	5.04 ksi	
	Ω =	1.65	
	Fv_n/ =	3.05 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	10.81 ksi
Allowable axial stress, compression	Fac =	0.81 ksi
Allowable shear stress; webs	Fv =	3.05 ksi

Elastic buckling stress	^ =	0.80 ksi
Weighted average allowable compressive stress (per Section E.3.1)	^ P	6.12 ksi

Mmax	189.70	lb-ft	
Stress	3.19	ksi	
Fb	10.81 ksi		29.51%

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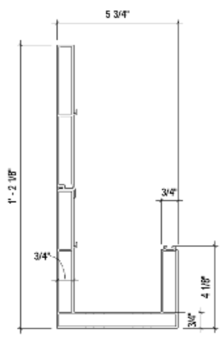
Work Prepared For: American Patio & Fireplace
Project: 22268 - Jenkins Residence
Detail/Member: Louver Beam

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

Design Check of 5.75"x14.125"x0.125"/0.125" 6005A-T6 Aluminum Tube

Alloy: 6005A Temper: T6 Critically Welded: N

MEMBER PROPERTIES



Flange width	P	5.750"
Flange thickness	P	0.125"
Web height	P	14.125"
Web thickness	P	0.125"
Moment of inertia about axis parallel to flange	RP	80.77 in^4
Moment of inertia about axis parallel to web	RP	17.42 in^4
Section modulus compression x-axis	P	8.94 in^3
Section modulus tension x-axis	P	15.88 in^3
Section modulus tension y-axis	P	4.34 in^3
Section modulus compression y-axis	P	10.03 in^3
Radius of gyration about centroidal axis parallel to flange	P	4.36 in
Radius of gyration about centroidal axis parallel to web	P	2.02 in
Torsion constant	JP	1.90 in^4
Cross sectional area of member	* P	4.25 in^2
Plastic section modulus	P	16.60 in^3
Warping constant	Cw =	5.75 in^6

MEMBER SPANS

Unsupported member length (between supports)	VP	15.33 ft
Unbraced length for bending (between bracing against side-sway)	V P	15.33 ft
Effective length factor	P	1.0

MATERIAL PROPERTIES

Tensile ultimate strength	^ P	38 ksi
Tensile yield strength	^ P	35 ksi
Compressive yield strength	^ P	35 ksi
Shear ultimate strength	^ P	23 ksi
Shear yield strength	^ P	21 ksi
Compressive modulus of elasticity	+P	10,100 ksi

BUCKLING CONSTANTS

Compression in columns & beam flanges (Intercept)	Ω P	39.37 ksi
Compression in columns & beam flanges (Slope)	Z P	0.25 ksi
Compression in columns & beam flanges (Intersection)	> P	65.67 ksi
Compression in flat plates (Intercept)	Ω P	45.00 ksi
Compression in flat plates (Slope)	Z P	0.30 ksi
Compression in flat plates (Intersection)	> P	61.42 ksi
Compressive bending stress in solid rectangular bars (Intercept)	Ω P	66.82 ksi
Compressive bending stress in solid rectangular bars (Slope)	Z P	0.67 ksi
Shear stress in flat plates (Intercept)	Ω P	27.24 ksi
Shear stress in flat plates (Slope)	Z P	0.14 ksi
Shear stress in flat plates (Intersection)	> P	78.95 ksi
Ultimate strength coefficient of flat plates in compression (slenderness limit λ2)	z P	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	/ P	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	z P	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	/ P	2.04
Tension coefficient	P	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	^	Fty_n =	35.00 ksi
		Ω =	1.65
		Fty_n/Ω =	21.21 ksi
Tensile Rupture - Unwelded Members	^ w	^ P	38.00 ksi
		Ω =	1.95
		Ftu_n/Ω t =	19.49 ksi

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AXIAL COMPRESSION MEMBERS

E.2 Compression Member Buckling

Axial, gross section subject to buckling	Lower slenderness limit	zP_	17.76	
	Upper slenderness limit	/P_	65.67	
	Slenderness	F HP_	90.9	≥ λ2
	Q/8' +w	^ P_	10.25 ksi	
		Ω =	1.65	
		Fc_n/ =	6.21 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.

ΩI I2I /,PP^ P P P P P PP^ H

ΩI I2I /,PP^ P P P P P PF H

E.4 Buckling Interaction

Per Table B.5.1	P 1+wPzhx1 w H	^ F HP_	20.11 ksi	
	P^	^ P_	10.25 ksi	
	Fe(flange) > Fc_n (E.2 Member Buckling)	P_	1.65	
		Fc_n/ =	6.21 ksi	
	P 1+wPzhx1 w H	^ F HP_	3.16 ksi	
	PQ/8' +wF H zwMPP^ /wM	^ P_	4.68 ksi	
	^ F HPjP^ PF+I/Pρ PΩ H	P_	1.65	
		Fc_n/ =	2.84 ksi	

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture	Limit State of Yielding			
	zI 1 1^	Mnp =	312.73 k-in	
	ρ w	Fb_n =	35.00 ksi	
		Ω =	1.65	
		Fb_n/ =	21.21 ksi	
	Limit State of Rupture			
	1^ w	Mnu =	839.60 k-in	
	ρ w	Fb_n =	38.00 ksi	
		Ω =	1.95	
		Fb_n/ =	19.49 ksi	

F.4 Lateral-Torsional Buckling

Unsymmetric shape subject to lateral-torsional buckling				
	Slenderness for shapes any shape about the bending axis	P^I2I I P_	43.02	
	Maximum slenderness	F HP_	43.02	< Cc
Nominal flexural strength - lateral-torsional buckling				
	ρ Fz,Fw HHF 1+1 1 w MH	Mnmb =	243.16 k-in	
	ρ w	Fb_n =	27.21 ksi	
		Ω =	1.65	
		Fb_n/ =	16.49 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	zP_	20.8	
	Upper slenderness limit	/P_	32.8	
	Flange Slenderness	w P_	44.0	≥ λ2
	Web Slenderness	w P_	111.0	≥ λ2
	P / 1 FΩ 1+HFzhx1 w H	^ zP_	21.74 ksi	
		Ω =	1.65	
		Fc_n1/ =	13.17 ksi	
	P / 1 FΩ 1+HFzhx1 w H	^ /P_	8.62 ksi	
		Ω =	1.65	
		Fc_n2/ =	5.22 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	zP	33.10	
Upper slenderness limit	/ P	77.22	
Slenderness	w P	111.00	≥ λ2
P / 1 FΩ 1+HF 1 w H	^ P	23.23 ksi	
	Ω =	1.65	
	Fb_n/Ω =	14.08 ksi	

SHEAR

G.2 Shear Supported on Both Edges - Web

Members with flat elements supported on both edges

Lower slenderness limit	zP	35.29	
Upper slenderness limit	/ P	63.16	
Slenderness	w P	111.00	≥ λ2
+vFzh' 1 w H	^ P	5.18 ksi	
	Ω =	1.65	
	Fv_n/Ω =	3.14 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	16.49 ksi
Allowable axial stress, compression	Fac =	3.79 ksi
Allowable shear stress; webs	Fv =	3.14 ksi

Elastic buckling stress	^ =	6.19 ksi
Weighted average allowable compressive stress (per Section E.3.1)	^ P	7.48 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	ρ P	5.65 kip-ft	
Bending stress developed in member	fb =	7.59 ksi	
Allowable bending stress of member	Fb =	16.49 ksi	< 1.0

Axial Loads

Axial load developed in member	^ P	0 lb	
Axial stress developed in member	fa =	0.00 ksi	
Allowable compressive axial stress of member	Fac =	3.79 ksi	< 1.0

Shear Loads

Shear load developed in member	P	1,475 lb	
Shear stress developed in member	fv =	0.43 ksi	
Allowable shear stress of member webs	Fv =	3.14 ksi	< 1.0

Interaction Equations

	√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.48	< 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

Support Type	Beam =	Simple
Beam Length	L =	15.33 ft
Tributary Width	W =	7.50 ft
Load on Tributary (LL, WL, DL, etc)	RL =	25.65 psf
Additional Beam Load (Weight or Service Loads)	DL =	0.00 lb/ft
Total Loading on Beam	w =	192.41 lb/ft
Shear Loading at End of Beam	Vy =	1475 lbs
CALCULATED MOMENT	**Mmax =**	**5.7 kip-ft**

Deflection Check

	P	Simple	
Z FV	P	L / 180	
	P	192.41 lb/ft	
ALLOWABLE DEFLECTION	**∆Allow =**	**1.02 in**	
MAXIMUM DEFLECTION	**∆Max =**	**0.29 in**	29%

Simple Max Deflection = 5wl^4/384EI

OK, Allowable Deflection Sufficient

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Work Prepared For:	American Patio & Fireplace
Project:	22268 - Jenkins Residence
Detail/Member:	Main Beam

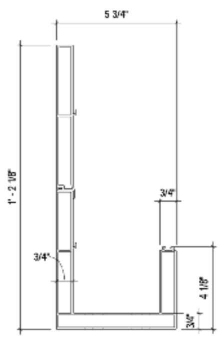
ALUMINUM DESIGN MANUAL (2020 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 5.75"x14.125"x0.125"/0.125" 6005A-T6 Aluminum Tube

P Q QP PZ Pp

Alloy:	6005A	Temper:	T6	Critically Welded:	N

MEMBER PROPERTIES



Flange width	P	5.750"
Flange thickness	P	0.125"
Web height	P	14.125"
Web thickness	P	0.125"
Moment of inertia about axis parallel to flange	RP	80.77 in^4
Moment of inertia about axis parallel to web	RP	17.42 in^4
Section modulus compression x-axis	P	8.94 in^3
Section modulus tension x-axis	P	15.88 in^3
Section modulus tension y-axis	P	4.34 in^3
Section modulus compression y-axis	P	10.03 in^3
Radius of gyration about centroidal axis parallel to flange	P	4.36 in
Radius of gyration about centroidal axis parallel to web	P	2.02 in
Torsion constant	JP	1.90 in^4
Cross sectional area of member	* P	4.25 in^2
Plastic section modulus	P	16.60 in^3
Warping constant	Cw =	5.75 in^6

MEMBER SPANS

Unsupported member length (between supports)	VP	15.0 ft
Unbraced length for bending (between bracing against side-sway)	V P	15.0 ft
Effective length factor	P	1.0

MATERIAL PROPERTIES

Tensile ultimate strength	^ P	38 ksi
Tensile yield strength	^ P	35 ksi
Compressive yield strength	^ P	35 ksi
Shear ultimate strength	^ P	23 ksi
Shear yield strength	^ P	21 ksi
Compressive modulus of elasticity	+P	10,100 ksi

BUCKLING CONSTANTS

Compression in columns & beam flanges (Intercept)	Ω P	39.37 ksi
Compression in columns & beam flanges (Slope)	Z P	0.25 ksi
Compression in columns & beam flanges (Intersection)	> P	65.67 ksi
Compression in flat plates (Intercept)	Ω P	45.00 ksi
Compression in flat plates (Slope)	Z P	0.30 ksi
Compression in flat plates (Intersection)	> P	61.42 ksi
Compressive bending stress in solid rectangular bars (Intercept)	Ω P	66.82 ksi
Compressive bending stress in solid rectangular bars (Slope)	Z P	0.67 ksi
Shear stress in flat plates (Intercept)	Ω P	27.24 ksi
Shear stress in flat plates (Slope)	Z P	0.14 ksi
Shear stress in flat plates (Intersection)	> P	78.95 ksi
Ultimate strength coefficient of flat plates in compression (slenderness limit λ2)	z P	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	/ P	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	z P	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	/ P	2.04
Tension coefficient	P	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	^	Fty_n =	35.00 ksi
		Ω =	1.65
		Fty_n/Ω =	21.21 ksi
Tensile Rupture - Unwelded Members	^ w	^ P	38.00 ksi
		Ω =	1.95
		Ftu_n/Ω t =	19.49 ksi

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AXIAL COMPRESSION MEMBERS

E.2 Compression Member Buckling

Axial, gross section subject to buckling	Lower slenderness limit	zP_	17.76	
	Upper slenderness limit	/P_	65.67	
	Slenderness	F HP_	88.92	≥ λ2
	Q/8' +w	^ P_	10.72 ksi	
		Ω =	1.65	
		Fc_n/Ω =	6.49 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.

Ω/ /2/ P,P^ P P P P P P^ H

Ω/ /2/ P,P^ P P P P P P^ H

E.4 Buckling Interaction

Per Table B.5.1	P 1+wPzhx1 w H	^ F HP_	20.11 ksi	
	P^	^ P_	10.72 ksi	
	Fe(flange) > Fc_n (E.2 Member Buckling)	P_	1.65	
		Fc_n/Ω =	6.49 ksi	
	P 1+wPzhx1 w H	^ F HP_	3.16 ksi	
	PQ/8' +wF H zMPP^ /wM	^ P_	4.75 ksi	
	^ F HpP^ PF+/ Pp PΩ H	P_	1.65	
		Fc_n/Ω =	2.88 ksi	

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture	Limit State of Yielding			
	z/ 1 1^	Mnp =	312.73 k-in	
	ρ w	Fb_n =	35.00 ksi	
		Ω =	1.65	
		Fb_n/Ω =	21.21 ksi	
	Limit State of Rupture			
	1^ w	Mnu =	839.60 k-in	
	ρ w	Fb_n =	38.00 ksi	
		Ω =	1.95	
		Fb_n/Ω =	19.49 ksi	

F.4 Lateral-Torsional Buckling

Unsymmetric shape subject to lateral-torsional buckling				
	Slenderness for shapes any shape about the bending axis	P^/2/ / P_	42.65	
	Maximum slenderness	F HP_	42.65	< Cc
Nominal flexural strength - lateral-torsional buckling				
	ρ Fz,Fw HHF 1+1 1 w MH	Mnmb =	243.75 k-in	
	ρ w	Fb_n =	27.28 ksi	
		Ω =	1.65	
		Fb_n/Ω =	16.53 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	zP_	20.8	
	Upper slenderness limit	/P_	32.8	
	Flange Slenderness	w P_	44.0	≥ λ2
	Web Slenderness	w P_	111.0	≥ λ2
	P / 1 PΩ 1+Hzhx1 w H	^ zP_	21.74 ksi	
		Ω =	1.65	
		Fc_n1/Ω =	13.17 ksi	
	P / 1 PΩ 1+Hzhx1 w H	^ /P_	8.62 ksi	
		Ω =	1.65	
		Fc_n2/Ω =	5.22 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	zP	33.10	
Upper slenderness limit	/P	77.22	
Slenderness	w P	111.00	≥ λ2
P / 1 FΩ 1+HF 1 w H	^ P	23.23 ksi	
	Ω =	1.65	
	Fb_n/Ω =	14.08 ksi	

SHEAR

G.2 Shear Supported on Both Edges - Web

Members with flat elements supported on both edges

Lower slenderness limit	zP	35.29	
Upper slenderness limit	/P	63.16	
Slenderness	w P	111.00	≥ λ2
+vFzh' 1 w H	^ P	5.18 ksi	
	Ω =	1.65	
	Fv_n/Ω =	3.14 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	16.53 ksi
Allowable axial stress, compression	Fac =	3.90 ksi
Allowable shear stress; webs	Fv =	3.14 ksi

Elastic buckling stress	^ =	6.46 ksi
Weighted average allowable compressive stress (per Section E.3.1)	^ P	7.48 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	ρ P	0.0 kip-ft	
Bending stress developed in member	fb =	0.00 ksi	
Allowable bending stress of member	Fb =	16.53 ksi	< 1.0

Axial Loads

Axial load developed in member	^ P	0 lb	
Axial stress developed in member	fa =	0.00 ksi	
Allowable compressive axial stress of member	Fac =	3.90 ksi	< 1.0

Shear Loads

Shear load developed in member	P	0 lb	
Shear stress developed in member	fv =	0.00 ksi	
Allowable shear stress of member webs	Fv =	3.14 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 =	15.00 ft
Tributary Width	W =	0.00 ft
	P Load=	1475.1 lb
Load on Tributary (LL, WL, DL, etc)	RL =	0.00 psf
Additional Beam Load (Weight or Service Loads)	DL =	0.00 lb/ft
Total Loading on Beam	w =	0.00 lb/ft
Shear Loading at End of Beam	Vy =	0 lbs
CALCULATED MOMENT	**Mmax =**	**0.00 kip-ft**

Deflection Check

	P	Simple	
Z FV	P	L / 180	
	P	0.00 lb/ft	
ALLOWABLE DEFLECTION	**∆Allow =**	**1.00 in**	
MAXIMUM DEFLECTION	**∆Max =**	**0.00 in**	0%

OK, Allowable Deflection Sufficient

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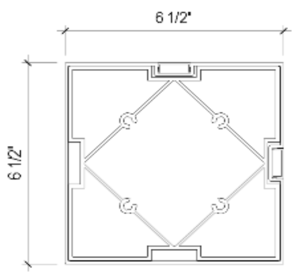
Work Prepared For:	American Patio & Fireplace
Project:	22268 - Jenkins Residence
Detail/Member:	Post Design

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

Design Check of 6.5"x6.5"x0.125"/0.125" 6063-T6 Aluminum Tube
P Q' QP PZ Pb

Alloy:	6063	Temper:	T6	Critically Welded:	N

MEMBER PROPERTIES



Flange width	P	6.500"
Flange thickness	P	0.125"
Web height	P	6.500"
Web thickness	P	0.125"
Moment of inertia about axis parallel to flange	RP	40.50 in^4
Moment of inertia about axis parallel to web	RP	40.50 in^4
Section modulus about the x-axis	P	12.46 in^3
Radius of gyration about centroidal axis parallel to flange	P	2.33 in
Radius of gyration about centroidal axis parallel to web	P	2.33 in
Torsion constant	JP	32.39 in^4
Cross sectional area of member	* P	7.44 in^2
Plastic section modulus	P	6.05 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	VP	10.0 ft
Unbraced length for bending (between bracing against side-sway X-Axis)	V P	10.0 ft
Unbraced length for bending (between bracing against side-sway Y-Axis)	V P	10.0 ft
Effective length factor	P	2.0
	P	2.0

MATERIAL PROPERTIES

Tensile ultimate strength	^ P	30 ksi
Tensile yield strength	^ P	25 ksi
Compressive yield strength	^ P	25 ksi
Shear ultimate strength	^ P	18 ksi
Shear yield strength	^ P	15 ksi
Compressive modulus of elasticity	+P	10,100 ksi

BUCKLING CONSTANTS

Compression in columns & beam flanges (Intercept)	Ω P	27.64 ksi
Compression in columns & beam flanges (Slope)	Z P	0.14 ksi
Compression in columns & beam flanges (Intersection)	> P	78.38 ksi
Compression in flat plates (Intercept)	Ω P	31.39 ksi
Compression in flat plates (Slope)	Z P	0.17 ksi
Compression in flat plates (Intersection)	> P	73.55 ksi
Compressive bending stress in solid rectangular bars (Intercept)	Ω P	46.12 ksi
Compressive bending stress in solid rectangular bars (Slope)	Z P	0.38 ksi
Shear stress in flat plates (Intercept)	Ω P	18.98 ksi
Shear stress in flat plates (Slope)	Z P	0.08 ksi
Shear stress in flat plates (Intersection)	> P	94.57 ksi
Ultimate strength coefficient of flat plates in compression (slenderness limit λ2)	z P	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	/ P	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	z P	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	/ P	2.04
Tension coefficient	P	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	^	Fty_n =	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	^ w	^ P	30.00 ksi
		Ω =	1.95
		Ftu_n/Ω t =	15.38 ksi

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AXIAL COMPRESSION MEMBERS

E.2 Compression Member Buckling

Axial, gross section subject to buckling	Lower slenderness limit	zP_	18.23	
	Upper slenderness limit	/P_	78.38	
	Slenderness	F HP_	102.89	≥ λ2
	Q8' +w	^ P_	8.00 ksi	
		Ω =	1.65	
		Fc_n/Ω =	4.85 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.

ΩH h2h P,P^ P P P P P FF^ H

ΩH h2h P,P^ P P P P P FF H

E.4 Buckling Interaction

Per Table B.5.1	P 1+wFFzhx1 w H	^ F HP_	15.58 ksi	
	P^	^ P_	8.00 ksi	
	Fe(flange) > Fc_n (E.2 Member Buckling)	P_	1.65	
		Fc_n/Ω =	4.85 ksi	
	P 1+wFFzhx1 w H	^ F HP_	15.58 ksi	
	P^	^ P_	8.00 ksi	
	^ F HRP^ FF+h/Fp FΩ H	P_	1.65	
		Fc_n/Ω =	4.85 ksi	

FLEXURAL MEMBERS

F.2 Yielding and Rupture

Nominal flexural strength for yielding and rupture	Limit State of Yielding			
	1^	Mnp =	151.29 k-in	
	ρ w	Fb_n =	25.00 ksi	
		Ω =	1.65	
		Fb_n/Ω =	15.15 ksi	
	Limit State of Rupture			
	1^ w	Mnu =	181.55 k-in	
	ρ w	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	P^h2h hzP_	15.13	
	Slenderness for closed shapes	P^h2h HVP_	14.78	
	Slenderness for any shape	P^h2h h P_	15.13	
	Maximum slenderness	F HP_	15.13	< Cc
Nominal flexural strength - lateral-torsional buckling				
	ρ Fz,Fw HHF 1+1 1 w MH	Mnmb =	161.12 k-in	
	ρ w	Fb_n =	12.93 ksi	
		Ω =	1.65	
		Fb_n/Ω =	7.84 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	zP_	22.8	
	Upper slenderness limit	/P_	39.2	
	Flange Slenderness	w P_	50.0	≥ λ2
	Web Slenderness	w P_	50.0	≥ λ2
	P / 1 FΩ 1+HFzhx1 w H	^ zP_	15.98 ksi	
		Ω =	1.65	
		Fc_n1/Ω =	9.68 ksi	
	P / 1 FΩ 1+HFzhx1 w H	^ /P_	15.98 ksi	
		Ω =	1.65	
		Fc_n2/Ω =	9.68 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	zP	34.73	
Upper slenderness limit	/ P	92.95	
Slenderness	w P	50.00	λ1 - λ2
PΩ , 1Z 1 w	^ P	33.71 ksi	
	Ω =	1.65	
	Fb_n/ =	20.43 ksi	

SHEAR

G.2 Shear Supported on Both Edges - Web

Members with flat elements supported on both edges

Lower slenderness limit	zP	38.73	
Upper slenderness limit	/ P	75.65	
Slenderness	w P	50.00	λ1 - λ2
Ω ,zh' Z 1 w	^ P	13.84 ksi	
	Ω =	1.65	
	Fv_n/ =	8.39 ksi	

ALLOWABLE STRESSES

Allowable bending stress	Fb =	6.54 ksi
Allowable axial stress, compression	Fac =	4.85 ksi
Allowable shear stress; webs	Fv =	8.39 ksi
Allowable axial stress, Tension	Fat =	15.15 ksi
Elastic buckling stress	^ =	4.83 ksi
Weighted average allowable compressive stress (per Section E.3.1)	^ P	9.68 ksi

MEMBER LOADING

Bending Moments

Bending moment developed in member	ρ P	2.34 kip-ft	
Bending stress developed in member	fb =	2.25 ksi	
Allowable bending stress of member	Fb =	6.54 ksi	< 1.0

Compression Loads

Compression load developed in member	P	1,475 lb	
Compression stress developed in member	fc =	0.20 ksi	
Allowable compressive axial stress of member	Fac =	4.85 ksi	< 1.0

Tension Loads

Tension load developed in member	P	257 lb	
Tension stress developed in member	ft =	0.03 ksi	
Allowable Tension axial stress of member	Fat =	15.15 ksi	< 1.0

Shear Loads

Shear load developed in member	P	405 lb	
Shear stress developed in member	fv =	0.26 ksi	
Allowable shear stress of member webs	Fv =	8.39 ksi	< 1.0

Interaction Equations

√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.35	< 1.0
fa/Fa + fb/Fb =	0.39	< 1.0
fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.39	< 1.0

CONFIGURATION AND MOMENT TABULATION TOOLS

Mx =	28.09 kip-in	Applied Moment Per Member	26 PSF	Total Gravity Load
My =	27.62 kip-in	Applied moment Per Member	7.7 FT	Post Trib Area in X-Axis
Tn =	0.11 kip-in	Applied Torsion Per Member	7.5 FT	Post Trib Area in Y-Axis
Vx =	288 lbs	Applied Shear Load Per Member	4 PSF	Uplift
Vy =	284 lbs	Applied Shear Load Per Member	20 PSF	Lateral Load
V =	405 lbs	Applied Resultant Shear Load Per Member		
P =	1,475 lbs	Applied axial compression load		
T =	257 lbs	Applied axial tension load	1.73 kip-in	Seismic Moment

ΛM | STRUCTURES

Work Prepared For:	American Patio & Fireplace
Project:	22268 - Jenkins Residence
Member/Detail:	Beam To Beam

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 in^2	Tensile Stress Area (*Table 20.1,20.2)
Ar =	0.027 in^2	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
Le1 =	0.500''	Minimum Depth of Full Thread Engagement Into S*crew Boss If Applicable* (Not Including Tapping/Drilling Point)

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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	6	Anchor Qty at Connection
Treq	0 lb	Required Tensile Loading on Connection
Vreq	1475 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)

$$\frac{\square_{\square}}{\square_{\square\square\square}} + \frac{\square_{\square}}{\square_{\square\square\square}} = \quad 0.48$$

OK, (6) anchors sufficient

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ΛM | STRUCTURΞS

Work Prepared For:	American Patio & Fireplace
Project:	22268 - Jenkins Residence
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 =	0.125''	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
Le1 =	0.500''	Minimum Depth of Full Thread Engagement Into S*crew Boss If Applicable* (Not Including Tapping/Drilling Point)

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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	4	Anchor Qty at Connection
Treq	0 lb	Required Tensile Loading on Connection
Vreq	405 lb	Required Shear Loading on Connection
n	1.00	Exponent factor
Tcap	1250 lb	Tensile capacity of connection (Qty * Rz)
Vcap	2069 lb	Shear capacity of connection (Qty * Rx)

$$\frac{\square_{\square}}{\square_{\square\square\square}} + \frac{\square_{\square}}{\square_{\square\square\square}} = \quad 0.20$$

OK, (4) anchors sufficient

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AM | STRUCTURES

Work Prepared For:	Lake City
Project:	22268 - Jenkins Residence
Detail/Member:	Base Plate Design

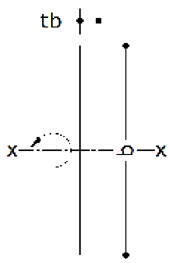
ALUMINUM DESIGN MANUAL (2020 EDITION)
Specifications for Aluminum Structures (Buildings)
Allowable Stress Design

Design Check of 10.625"x0.75" 6063-T6 Aluminum Flat Plate

P Q QP PZ Pb

Alloy: 6063 Temper: T6 Critically Welded: N

MEMBER PROPERTIES



Flat Plate Height	P	10.625"
Flat Plate Thickness	P	0.750"
Moment of inertia about axis parallel to flange	RP	74.97 in^4
Moment of inertia about axis parallel to web	RP	0.37 in^4
Section modulus about the x-axis	P	14.11 in^3
Radius of gyration about centroidal axis parallel to flange	P	3.07 in
Radius of gyration about centroidal axis parallel to web	P	0.22 in
Torsion constant	JP	1.49 in^4
Cross sectional area of member	* P	7.97 in^2
Plastic section modulus	P	21.17 in^3
Warping constant	Cw =	0.00 in^6

MEMBER SPANS

Unsupported member length (between supports)	VP	0.72 ft
Unbraced length for bending (between bracing against side-sway)	V P	0.72 ft
Effective length factor	P	1.0

MATERIAL PROPERTIES

Tensile ultimate strength	^ P	30 ksi
Tensile yield strength	^ P	25 ksi
Compressive yield strength	^ P	25 ksi
Shear ultimate strength	^ P	18 ksi
Shear yield strength	^ P	15 ksi
Compressive modulus of elasticity	+ P	10,100 ksi

BUCKLING CONSTANTS

Compression in columns & beam flanges (Intercept)	Ω P	27.64 ksi
Compression in columns & beam flanges (Slope)	Z P	0.14 ksi
Compression in columns & beam flanges (Intersection)	> P	78.38 ksi
Compression in flat plates (Intercept)	Ω P	31.39 ksi
Compression in flat plates (Slope)	Z P	0.17 ksi
Compression in flat plates (Intersection)	> P	73.55 ksi
Compressive bending stress in solid rectangular bars (Intercept)	Ω P	46.12 ksi
Compressive bending stress in solid rectangular bars (Slope)	Z P	0.38 ksi
Compressive bending stress in solid rectangular bars (Intersection)	> P	80.56 ksi
Shear stress in flat plates (Intercept)	Ω P	18.98 ksi
Shear stress in flat plates (Slope)	Z P	0.08 ksi
Shear stress in flat plates (Intersection)	> P	94.57 ksi
Ultimate strength coefficient of flat plates in compression (slenderness limit λ2)	z P	0.35
Ultimate strength coefficient of flat plates in compression (stress for slenderness > λ2)	/ P	2.27
Ultimate strength of flat plates in bending (slenderness limit λ2)	z P	0.50
Ultimate strength of flat plates in bending (stress for slenderness > λ2)	/ P	2.04
Tension coefficient	P	1.0

D.2 Axial Tension

Tensile Yielding - Unwelded Members	^	^ P	25.00 ksi
		Ω =	1.65
		Fty_n/Ω =	15.15 ksi
Tensile Rupture - Unwelded Members	^ w	^ P	30.00 ksi
		Ω =	1.95
		Ftu_n/Ω =	15.38 ksi

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AM | STRUCTURES

AXIAL COMPRESSION MEMBERS				
E.2 Compression Member Buckling				
Axial, gross section subject to buckling	Lower slenderness limit	zP_	18.23	
	Upper slenderness limit	/P_	78.38	
	Slenderness	F HP_	39.84	< λ2
	FΩ ,Z 1 HFQ/8' 4Q/z' 1FF> , HF> , zHH	^ P_	20.70 ksi	
		Ω =	1.65	
		Fc_n/ =	12.54 ksi	
FLEXURAL MEMBERS				
F.2 Yielding and Rupture				
Nominal flexural strength for yielding and rupture	Limit State of Yielding			
	1^	Mnp =	529.17 k-in	
	ρ w	Fb =	25.00 ksi	
		P_	1.65	
		Fb_n/ =	15.15 ksi	
	Limit State of Rupture			
	1^ w	Mnu =	635.01 k-in	
	ρ w	Fb =	30.00 ksi	
		P_	1.95	
		Fb_n/ =	15.38 ksi	
F.4 Lateral-Torsional Buckling				
Rectangular bars subject to lateral-torsional buckling				
	Slenderness for shapes symmetric about the bending axis	P^ H2H/ HzP_	41.54	
	Slenderness for rectangular bars	P^ H2H/ H2P_	29.36	
	Slenderness for any shape	P^ H2H/ H P_	41.54	
	Maximum slenderness	F HP_	41.54	< Cc
Nominal flexural strength - lateral-torsional buckling				
	ρ Fz,F w> HHF 1+1 1 w> MH	Mnmb =	370.09 k-in	
	ρ w	Fb_n =	26.23 ksi	
		Ω =	1.65	
		Fb_n/ =	15.89 ksi	
G.2 Shear Supported on Both Edges				
Members with flat elements supported on both edges				
	Lower slenderness limit	zP_	38.73	
	Upper slenderness limit	/P_	75.65	
	Slenderness	w P_	14.17	≤ λ1
	^	^ P_	15.00 ksi	
		Ω =	1.65	
		Fv_n/ =	9.09 ksi	
ALLOWABLE STRESSES				
	Allowable bending stress	Fb =	15.15 ksi	
	Allowable axial stress, compression	Fac =	12.54 ksi	
	Allowable shear stress	Fv =	9.09 ksi	
	Elastic buckling stress	^ =	32.21 ksi	
	Weighted average allowable compressive stress (per Section E.3.1)	^ =	12.54 ksi	

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MEMBER LOADING

Bending Moments

Bending moment developed in member	ρ P	3.99 kip-ft	
Bending stress developed in member	fb =	3.39 ksi	
Allowable bending stress of member	Fb =	15.15 ksi	< 1.0

Axial Loads

Axial load developed in member	^ P	405 lb	
Axial stress developed in member	fa =	0.05 ksi	
Allowable compressive axial stress of member	Fac =	12.54 ksi	< 1.0

Shear Loads

Shear load developed in member	P	1,475 lb	
Shear stress developed in member	fv =	0.19 ksi	
Allowable shear stress of member	Fv =	9.09 ksi	< 1.0

Interaction Equations

	√ [(fb/Fb)^2 + (fv/Fv)^2] =	0.22	< 1.0
Eq H.1-1	fa/Fa + fb/Fb =	0.23	< 1.0
Eq H.3-2	fa/Fa + (fb/Fb)^2 + (fv/Fv)^2 =	0.05	< 1.0

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Work Prepared For: American Patio & Fireplace
Project: 22268 - Jenkins Residence

CHECK SOIL BEARING PRESSURE FOR CRITICAL FOOTING

Footing Dimensions:		Wx = 36 in	Wy = 36 in	D = 4 in
		Sx = 0 in	Sy = 0 in	Thk = 0 in

1475 lb	Max Axial Gravity Load in Column
+ 450 lb	Weight of Footing (36" x 36" x 4" pad footer)
1925 lb	Total Load on Soil (gravity load + footing weight)
28.1 kip-in	Total Moment - X-Axis in Footing (column is assumed to be centered in footer)
27.6 kip-in	Total Moment - Y-Axis in Footing (column is assumed to be centered in footer)
1500 psf	Min Soil Bearing Pressure (to be verified by General Contractor)

$$q_{heel} = \frac{P_{total}}{B \cdot L} - \frac{6M_x}{B^2 \cdot L} - \frac{6M_y}{L^2 \cdot B} =$$ -817.7 psf — footing pressure at heel (along dimension "W1")

$$q_{toe} = \frac{P_{total}}{B \cdot L} + \frac{6M_x}{B^2 \cdot L} + \frac{6M_y}{L^2 \cdot B} =$$ 1245.5 psf — footing pressure at toe (along dimension "W1")

Max bearing pressure on soil = 1245.5 psf (at critical footing)
Frictional Resistance qf = 250.0 psf
Max Bearing Capacity of Footing = 1611.1 psf Square or Rectangle
Max Bearing Capacity of Footing = 1611.1 psf Circle

OK SQUARE OR RECTANGLE, soil allowable bearing pressure (1500 psf) not exceeded at critical footing

OK CIRCLE, soil allowable bearing pressure (1500 psf) not exceeded at critical footing

UPIFT RESISTANCE CALCULATION FOR CRITICAL FOOTING

Footing Dimensions:	W1 = 36 in	W2 = 36 in	D = 4 in
Slab Trib Dimensions:	S1 = 0 in	S2 = 0 in	Thk = 0 in
150 pcf	Concrete Density		
256.5 lb	Uplift load at column		

Conc Footing Weight = 450 lb
Conc Slab Weight = 0 lb
Total Gravity Weight = 450 lb

Total Uplift Load = (P+ M/d) = 257 lb

OK, factor of safety FOS = 1.75 >1.0

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FOOTER SLIDING CHECK

1925 lb	Total Load on Soil (gravity load + footing weight)
288 lb	Max Shear restisted by Column (V)
0.35	(Coefficient of Friction) μ
674 lb	Friction Force (F) = μ*V

$\text{F.S.} = \frac{F}{V} >= 1.5$ $\boxed{2.34}$ **FOOTER OK!**

REQUIRED REINFORCEMENT

(4) #3 Horizontal Bars Each Way
(2) #4 Horizontal Bars Each Way
(2) #5 Horizontal Bars Each Way
(2) #6 Horizontal Bars Each Way

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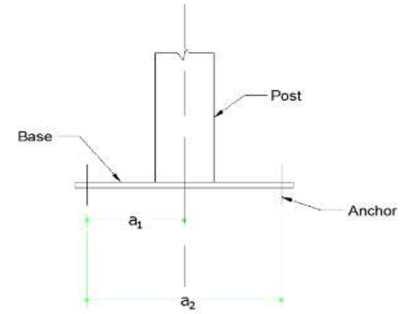
Work Prepared For: American Patio & Fireplace
Project: 22268 - Jenkins Residence
Mark/Detail: Post anchors to concrete

V = 702 lbs Required shear per anchor
T = 1,692 lbs Required tension per anchor
N Use gusset at beam to post?

Hilti/Powers Design Loads (LRFD)		
M	**44939**	**lb-in**
T	**410**	**lb**
C	**2360**	**lb**
V	**647**	**lb**



Resisting arm = a2

h
h h h h h h h h h h
h

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Fastening point:			

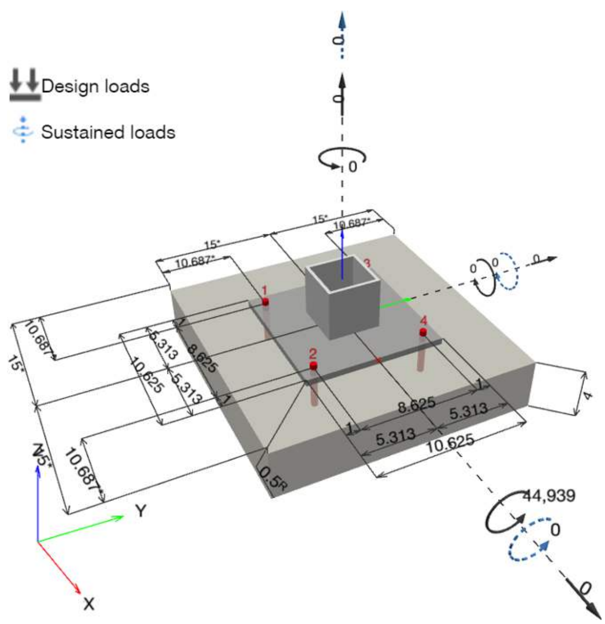
Specifier's comments:

1 Input data

Anchor type and diameter:	**HIT-HY 200 V3 + HAS-R 304/316 SS 1/2**
Item number:	2045003 HAS-R 316 SS 1/2"x6 1/2" (element) / 2334276 HIT-HY 200-R V3 (adhesive)
Effective embedment depth:	$h_{ef,opti}$ = 2.750 in. ($h_{ef,limit}$ = 2.750 in.)
Material:	ASTM F 593
Evaluation Service Report:	ESR-4868
Issued I Valid:	11/1/2021 | 11/1/2022
Proof:	Design Method ACI 318-19 / Chem
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate^R :	l_x x l_y x t = 10.625 in. x 10.625 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.
Base material:	cracked concrete, 3000, f_c' = 3,000 psi; h = 4.000 in., Temp. short/long: 32/32 °F
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: not present, shear: not present; no supplemental splitting reinforcement present
	edge reinforcement: none or < No. 4 bar

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

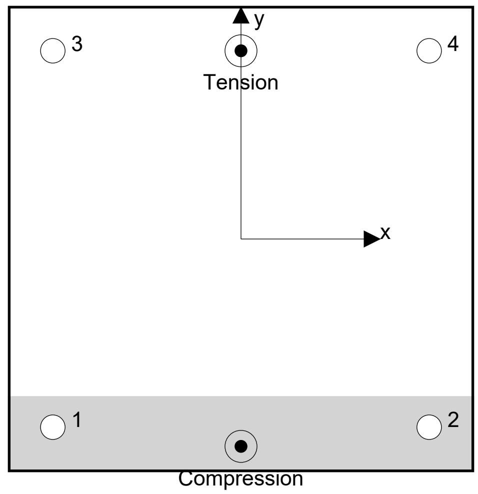
Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0$; $V_x = 0$; $V_y = 0$; $M_x = 44,939$; $M_y = 0$; $M_z = 0$;	no	96

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	0	0	0
2	0	0	0	0
3	2,479	0	0	0
4	2,479	0	0	0

max. concrete compressive strain: 0.13 [‰]
max. concrete compressive stress: 555 [psi]
resulting tension force in (x/y)=(0.000/4.313): 4,958 [lb]
resulting compression force in (x/y)=(0.000/-4.752): 4,958 [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*	2,479	9,223	27	OK
Bond Strength**	4,958	5,207	96	OK
Sustained Tension Load Bond Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Failure**	4,958	5,520	90	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-4868
$\phi\ N_{sa} \geq N_{ua}$		ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.14	100,000

Calculations

N_{sa} [lb]
14,190

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
14,190	0.650	9,223	2,479

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)
$N_{ba} = \lambda_a \cdot \tau_{k,c} \cdot \pi \cdot d_a \cdot h_{ef}$	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	0.500	2.750	10.687	1.000	1,156

$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	c_{ac} [in.]	λ_a
0.000	0.000	6.151	1.000

Calculations

c_{Na} [in.]	A_{Na} [in.^{2}]	A_{Na0} [in.^{2}]	$\psi_{ed,Na}$
7.136	326.77	203.68	1.000

$\psi_{ec1,Na}$	$\psi_{ec2,Na}$	$\psi_{cp,Na}$	N_{ba} [lb]
1.000	1.000	1.000	4,993

Results

N_{ag} [lb]	ϕ_{bond}	$\phi\ N_{ag}$ [lb]	N_{ua} [lb]
8,011	0.650	5,207	4,958

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.]	$\psi_{c,N}$
2.750	0.000	0.000	10.687	1.000

c_{ac} [in.]	k_c	λ_a	f'_c [psi]
6.151	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]
136.13	68.06	1.000	1.000	1.000	1.000	4,246

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
8,493	0.650	5,520	4,958

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength (Bond Strength controls)*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

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

5 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, 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/
- 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!

6 Installation data

Profile: Square HSS (AISC), HSS4X4X.25; (L x W x T) = 4.000 in. x 4.000 in. x 0.250 in.

Hole diameter in the fixture: d_f = 0.562 in.

Plate thickness (input): 0.500 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 V3 + HAS-R 304/316 SS 1/2

Item number: 2045003 HAS-R 316 SS 1/2"x6 1/2" (element) / 2334276 HIT-HY 200-R V3 (adhesive)

Maximum installation torque: 360 in.lb

Hole diameter in the base material: 0.562 in.

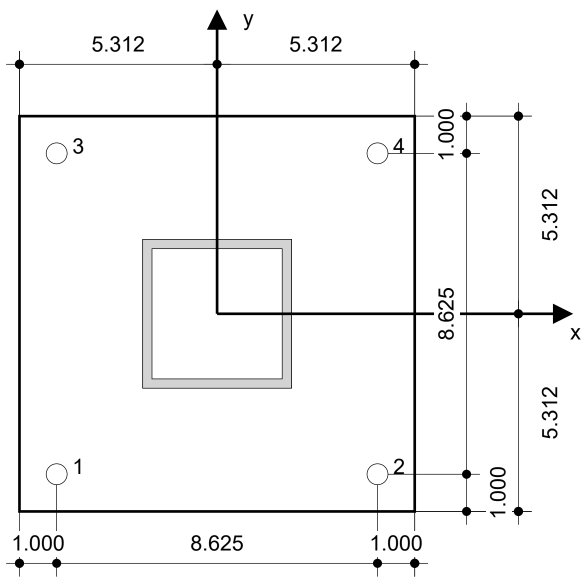
Hole depth in the base material: 2.750 in.

Minimum thickness of the base material: 4.000 in.

1/2 Hilti HAS Stainless steel threaded rod with Hilti HIT-HY 200 V3 Safe Set System

6.1 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	-4.312	-4.312	10.687	19.312	10.687	19.312
2	4.312	-4.312	19.312	10.687	10.687	19.312
3	-4.312	4.312	10.687	19.312	19.312	10.687
4	4.312	4.312	19.312	10.687	19.312	10.687

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7 Remarks; Your Cooperation Duties

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