EXHIBIT B.

STRUCTURAL CALCULATIONS

FOR

Awning Replacement

Double Dragon Restaurant

39131 Pioneer Blvd.
Sandy, OR 97055

FOR

Keystone Architecture

12020 SE Idleman Rd.
Portland, OR 97266

ALL COMPUTATION AND STRUCTURAL ENGINEERING FOR THIS PROJECT HAVE BEEN PERFORMED BY MYSELF OR UNDER MY DIRECT SUPERVISION. THESE CALCULATIONS ARE FOR THE ABOVE REFERENCED PROJCT SITE AND FOR THE CURRENT PHASE OF CONSTRUCTION ONLY. THESE CALCULATIONS DO NOT APPLY TO SAME OR SIMILAR CONFIGURATIONS AT THIS SITE OR AT A DIFFERENT SITE AND SHOULD BE VOID WITHOUT WET SIGNATURE.

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
18,138
OREGON
December 6, 1995
BABRAK AMIRI
05/14/2025
EXPIRES: 12-31-2025

Associated Consultants, Inc. Structural Engineers

100 East 13th Street • Suite 10 • Vancouver, WA 98660
Phone: (503) 384-0460 • (360) 699-0607

DESIGN CRITERIA

DESIGN STANDARDS	
BUILDING CODE:	2022 Oregon Structural Specialty Code
SNOW	
DESIGN ROOF SNOW LOAD	35.3 psf
SNOW DRIFT	PER SEAO SNOW LOAD ANALYSIS
GROUND SNOW LOAD	P_g = 42 psf
MIN. ROOF SNOW LOAD	P_f = 30 psf
SNOW EXPOSURE FACTOR	C_e = 1.0
SNOW IMPORTANCE FACTOR	I_s = 1.0
THERMAL FACTOR	C_t = 1.2
WIND	
RISK CATEGORY	II
DESIGN WIND SPEED	V_{ult} = 97 mph ULTIMATE (3-SECOND GUST)
EXPOSURE CATEGORY	B
GUST / INTERNAL PRESSURE	GC_{pi} = 0

SCOPE

The scope of this project is strictly limited to structural design of awnings attached to existing building.

Dead Load **1.0** psf

There are six 4'-0" wide **a**wnings. Frame spacing is 2-9" (max) on center.

Material is assumed to be 6063-T6 aluminum

All joints are assumed to be welded all around that matches material thickness

SNOW LOAD:

$P_g = 42^{PSF}$

$P_f = 30^{PSF}$

DRIFT

$l_u = 60^{FT}$

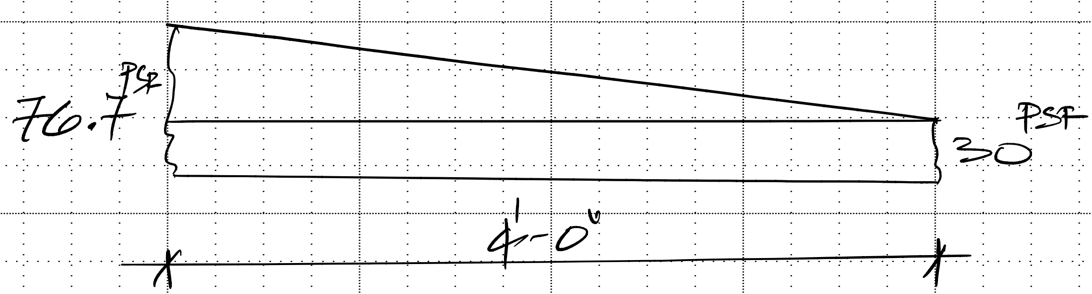
$h_d = 3^{FT}$ ← ASCE-7 FIG 7.6-1

$0.6 \times 4 = 2.4^{FT} < h_d$

↑ USE

$\gamma = 0.13 P_g + 14 = 19.5^{PCF}$

$P_d = 2.4 \times 19.5 = 46.7^{PSF}$

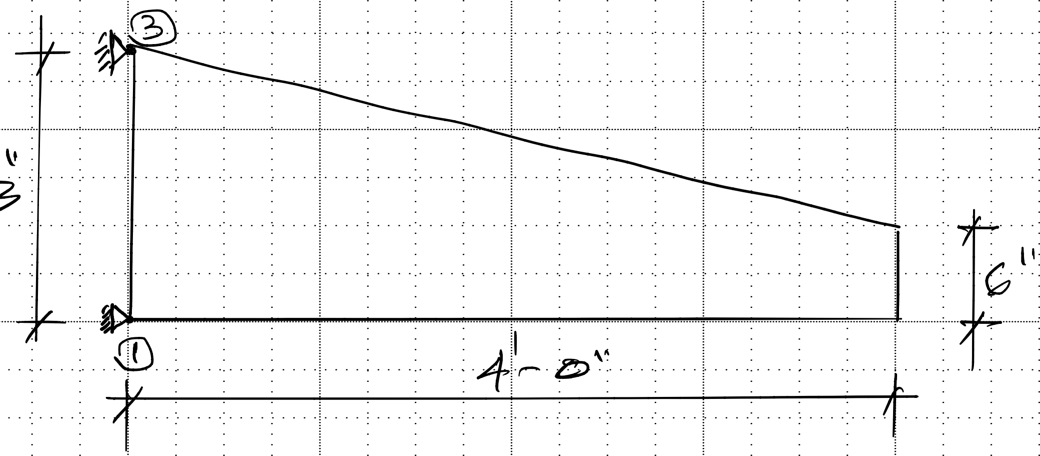


By: ________

Associated Consultants, Inc. Structural Engineers 100 East 13th Street • Suite 10 • Vancouver, Washington 98660 Portland: (503) 384-0460 • Vancouver: (360) 699-0607	Project Awning Replacement - Double Dragon Restaurant	Job No. 25-111
	Location 39131 Pioneer Blvd. - Sandy, OR 97055	Date
	Client Keystone Architecture Planning and Project Managment	Sheet No.

FRAME ANALYSIS (EXISTING CONDITIONS)

MAXIMUM SPACING = 2'-9"



$M_{max} = 0.313^{K \cdot FT}$ ← N.G

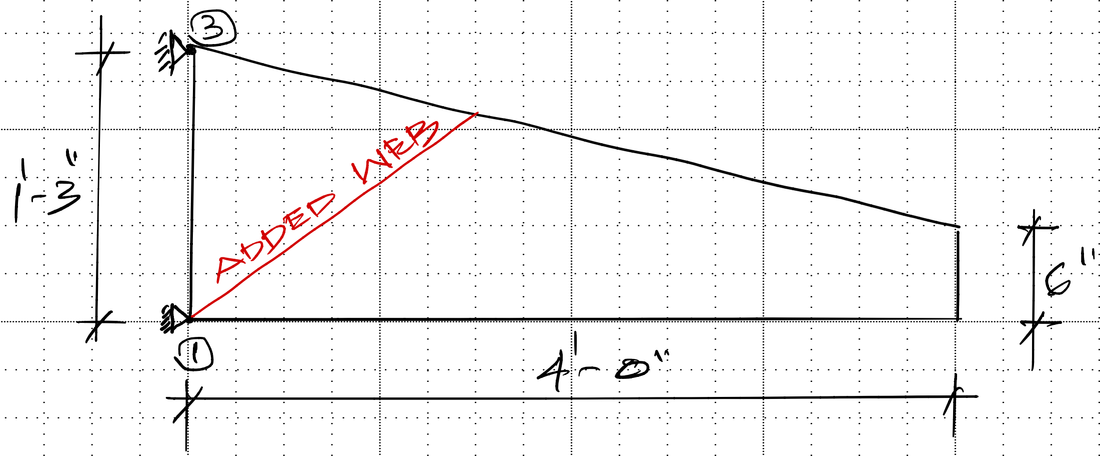
$P_{max} = 0.541^{K}$

$R_{Y} = 0.304^{K}$ T & B

$R_{T} = 0.835^{K}$

FRAME ANALYSIS (REINFORCED)

MAXIMUM SPACING = 2'-9"



$M_{max} = 0.125^{K-FT}$

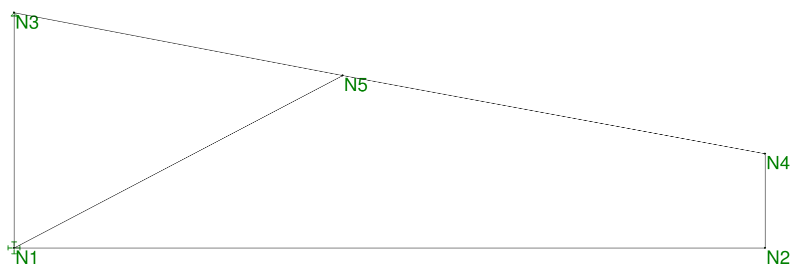
$P_{max} = -0.232^{K}$

$R_Y = 0.33^K$ (ASD)
$R_T = 0.88^K$ (ASD)
TOP & BOT

BY INSPECTION SNOW GOVERNS OVER WIND

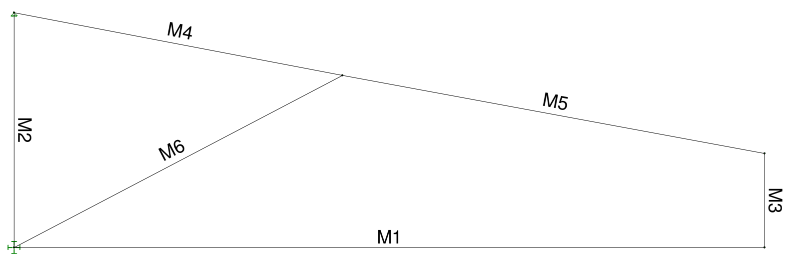
By: ________

Associated Consultants, Inc. Structural Engineers	Project: Awning Replacement - Double Dragon Restaurant	Job No. 25-111
100 East 13th Street • Suite 10 • Vancouver, Washington 98660	Location: 39131 Pioneer Blvd. - Sandy, OR 97055	Date
Portland: (503) 384-0460 • Vancouver: (360) 699-0607	Client: Keystone Architecture Planning and Project Managment	Sheet No.



Results for LC 3, DL + SL + SD

Associated Consultants Inc.	Double Dragon - Awning Frame	April 17, 2025
Babrak Amiri		9:49 AM
25-111		Awning - Revised.r3d



Results for LC 3, DL + SL + SD

Associated Consultants Inc.	Double Dragon - Awning Frame	April 17, 2025
Babrak Amiri		9:50 AM
25-111		Awning - Revised.r3d

Materials (General)

Material Label	Young's Modulus (ksi)	Shear Modulus (ksi)	Poisson's Ratio	Thermal Coef. (per 10^5 F)	Weight Density (k/ft^3)	Yield Stress (ksi)
ALUMINUM	10000	11154	.3	.65	.49	15

Joint Coordinates

Joint Label	X Coordinate (ft)	Y Coordinate (ft)	Z Coordinate (ft)	Joint Temperature (F)	Detach from Diaphragm
N1	0	0	0	0	No
N2	4	0	0	0	No
N3	0	1.25	0	0	No
N4	4	.5	0	0	No
N5	1.75	.916	0	0	No

Member Data

Member Label	I Joint	J Joint	K Joint	X-Axis Rotate (degrees)	Shape / Section Set	Material Set	Phys Memb	TOM	End Releases I-End xyz xyz	End Releases J-End xyz xyz	End Offsets I-End (in)	End Offsets J-End (in)	Inactive Code	Length (ft)
M1	N1	N2			SEC1	ALUMIN...	Y							4
M2	N1	N3			SEC1	ALUMIN...	Y							1.25
M3	N2	N4			SEC1	ALUMIN...	Y							.5
M4	N3	N5			SEC1	ALUMIN...	Y							1.782
M5	N5	N4			SEC1	ALUMIN...	Y							2.288
M6	N1	N5			SEC1	ALUMIN...	Y							1.975

Boundary Conditions

Joint Label	X Translation (k/in)	Y Translation (k/in)	Z Translation (k/in)	MX Rotation (k-ft/rad)	MY Rotation (k-ft/rad)	MZ Rotation (k-ft/rad)
N1	Reaction	Reaction	Reaction	Reaction	Reaction	
N3	Reaction	Reaction	Reaction			

Basic Load Case Data

BLC No.	Basic Load Case Description	Category Code	Category Description	Gravity X	Gravity Y	Gravity Z	Joint	Point	Direct Dist.	Area	Surf.
1	DL	DL	Dead Load						2		
2	SL	SL	Snow Load						2		
3	SD	SL	Snow Load						2		

Member Direct Distributed Loads, Category : DL, BLC 1 : DL

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M4	Y	-.003	-.003	0	0
M5	Y	-.003	-.003	0	0

Member Direct Distributed Loads, Category : SL, BLC 2 : SL

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M4	Y	-.082	-.082	0	0
M5	Y	-.082	-.082	0	0

Member Direct Distributed Loads, Category : SL, BLC 3 : SD

Member Label	Direction	Start Magnitude (k/ft, F)	End Magnitude (k/ft, F)	Start Location (ft or %)	End Location (ft or %)
M4	Y	-.128	-.086	0	0
M5	Y	-.086	0	0	0

Company	: **Associated Consultants Inc.**		April 17, 2025
Designer	: **Babrak Amiri**		9:51 AM
Job Number	: **25-111**	**Double Dragon - Awning Frame**	Checked By:_____

Load Combinations

Num	Description	Env	WS	PD	SRSS	CD	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	DL	y				1	1							
2	DL + SL	y				1	1	1	2	1				
3	DL + SL + SD	y				1	1	1	2	1	3	1		

Reactions, By Combination

LC	Joint Label	X Force (k)	Y Force (k)	Z Force (k)	X Moment (k-ft)	Y Moment (k-ft)	Z Moment (k-ft)
3	N1	.878	.336	0	0	0	0
3	N3	-.878	.301	0	0	0	0
3	Totals:	0	.637	0			
3	COG (ft):	X: 1.723	Y: .924	Z: 0			

Member Section Forces, By Combination

LC	Member Label	Section	Axial (k)	Shear y-y (k)	Shear z-z (k)	Torque (k-ft)	Moment y-y (k-ft)	Moment z-z (k-ft)
3	M1	1	.185	.02	0	0	0	.035
		2	.185	.02	0	0	0	.015
		3	.185	.02	0	0	0	-.005
		4	.185	.02	0	0	0	-.025
		5	.185	.02	0	0	0	-.046
3	M2	1	0	-.033	0	0	0	-.017
		2	0	-.033	0	0	0	-.007
		3	0	-.033	0	0	0	.003
		4	0	-.033	0	0	0	.013
		5	0	-.033	0	0	0	.024
3	M3	1	.02	-.185	0	0	0	-.046
		2	.02	-.185	0	0	0	-.023
		3	.02	-.185	0	0	0	0
		4	.02	-.185	0	0	0	.024
		5	.02	-.185	0	0	0	.047
3	M4	1	-.887	.137	0	0	0	.024
		2	-.869	.046	0	0	0	-.017
		3	-.853	-.041	0	0	0	-.018
		4	-.837	-.123	0	0	0	.018
		5	-.822	-.2	0	0	0	.09
3	M5	1	-.232	.236	0	0	0	.125
		2	-.215	.145	0	0	0	.016
		3	-.201	.067	0	0	0	-.044
		4	-.188	0	0	0	0	-.063
		5	-.178	-.053	0	0	0	-.047
3	M6	1	.732	-.026	0	0	0	-.018
		2	.732	-.026	0	0	0	-.005
		3	.732	-.026	0	0	0	.008
		4	.732	-.026	0	0	0	.021
		5	.732	-.026	0	0	0	.034

Joint Displacements, By Combination

LC	Joint Label	X Translation (in)	Y Translation (in)	Z Translation (in)	X Rotation (radians)	Y Rotation (radians)	Z Rotation (radians)
3	N1	0	0	0	0	0	-6.041e-4
3	N2	-.002	-.228	0	0	0	4.747e-3

Company : **Associated Consultants Inc.**
Designer : **Babrak Amiri**
Job Number : **25-111** **Double Dragon - Awning Frame**

April 17, 2025
9:51 AM
Checked By:_____

Joint Displacements, By Combination, (continued)

LC	Joint Label	X Translation (in)	Y Translation (in)	Z Translation (in)	X Rotation (radians)	Y Rotation (radians)	Z Rotation (radians)
3	N3	0	0	0	0	0	-1.58e-3
3	N4	-.037	-.228	0	0	0	4.671e-3
3	N5	.002	-.012	0	0	0	-4.724e-3

Member Deflections, By Combination

LC	Member Label	Section	x-Translation (in)	y-Translation (in)	z-Translation (in)	x-Rotation (radians)	(n) L/y Ratio	(n) L/z Ratio
3	M1	1	0	0	0	0	NC	NC
		2	0	-.05	0	0	7347.478	NC
		3	-.001	-.146	0	0	1494.969	NC
		4	-.002	-.225	0	0	877.604	NC
		5	-.002	-.228	0	0	NC	NC
3	M2	1	0	0	0	0	NC	NC
		2	0	0	0	0	NC	NC
		3	0	.002	0	0	8196.224	NC
		4	0	.003	0	0	5192.802	NC
		5	0	0	0	0	NC	NC
3	M3	1	-.228	.002	0	0	NC	NC
		2	-.228	.01	0	0	NC	NC
		3	-.228	.019	0	0	NC	NC
		4	-.228	.029	0	0	NC	NC
		5	-.228	.037	0	0	NC	NC
3	M4	1	0	0	0	0	NC	NC
		2	.001	-.01	0	0	2869.894	NC
		3	.002	-.012	0	0	3216.891	NC
		4	.003	-.006	0	0	6844.129	NC
		5	.004	-.012	0	0	NC	NC
3	M5	1	.004	-.012	0	0	NC	NC
		2	.004	-.086	0	0	1391.689	NC
		3	.005	-.181	0	0	460.63	NC
		4	.005	-.235	0	0	462.068	NC
		5	.005	-.23	0	0	NC	NC
3	M6	1	0	0	0	0	NC	NC
		2	0	.001	0	0	5439.335	NC
		3	-.002	.006	0	0	1941.646	NC
		4	-.003	.005	0	0	1698.674	NC
		5	-.004	-.012	0	0	NC	NC

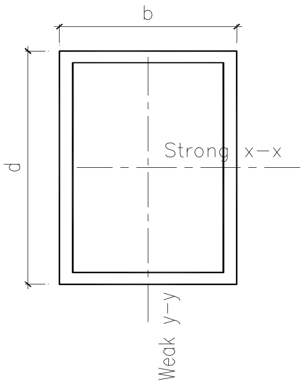
Associated Consultants, Inc.	PROJECT :	Double Dragon - Awning		PAGE :	
	CLIENT :	Keystone Arch		DESIGN BY :	Babrak Amiri
	JOB NO. :	25-111	DATE : 4/9/2025	REVIEW BY :	

Aluminum RT Member Capacity Based on Aluminum Design Manual 2010 (ASD)

INPUT DATA & DESIGN SUMMARY

MEMBER SIZE	RT 1 × 1 × 0.125	d	b	A	I_x	I_y	E	Wt (lbs/ft)
		1	1	0.438	0.1	0.1	10100	0.52

TENSILE ULTIMATE STRESS (T5 to T9, Table A.3.4) F_{tu} = **30** ksi
TENSILE YIELD STRESS (T5, T6, T7, T8, or T9) F_{ty} = **25** ksi
COMPRESSIVE YIELD STRESS (T5 to T9) F_{cy} = **25** ksi



AXIAL COMPRESSION FORCE P = **0** kips, ASD
STRONG GEOMETRIC AXIS EFFECTIVE LENGTH kL_x = **4** ft
WEAK GEOMETRIC AXIS EFFECTIVE LENGTH kL_y = **4** ft

STRONG GEOMETRIC AXIS BENDING MOMENT M_{rx} = **0.125** ft-kips, ASD
STRONG GEOMETRIC AXIS BENDING UNBRACED LENGTH L_{bx} = **2.5** ft
STRONG DIRECTION SHEAR LOAD, ASD V_{strong} = **0.2** kips

WEAK GEOMETRIC AXIS BENDING MOMENT M_{ry} = **0** ft-kips, ASD
WEAK DIRECTION SHEAR LOAD, ASD V_{weak} = **0** kips

THE DESIGN IS ADEQUATE.

ANALYSIS
CHECK COMPRESSION STRESS IN AXIAL FORCE

$$F_{a1}=\frac{P_n}{A_g\Omega_c}=\begin{cases} Min\left(\dfrac{\left(B_c-\dfrac{D_c kL}{r}\right)}{n_u}\ ,\ \dfrac{F_{cy}}{n_y}\right)\ ,\ for\ \dfrac{kL}{r}<S_2 \\ \dfrac{\pi^2 E}{n_u\left(\dfrac{kL}{r}\right)^2}\ ,\ for\ \dfrac{kL}{r}\geq S_2 \end{cases} = 2.90$$ ksi, (compression in column, ADM-I E3)

Where r_x = 0.361 in, (Page V-34 to V-37)
r_y = 0.361 in, (Page V-34 to V-37)
$kL / r = Max(kL_x / r_x , kL_y / r_y)$ = 132.96
E = 10100 ksi, (ADM-I Table A.3.4)
$B_c = F_{cy} [1 + (F_{cy} / 2250)^{0.5}]$ = 27.64 , (ADM-I Table B.4.2)
$D_c = (B_c / 10) (B_c / E)^{0.5}$ = 0.14 , (ADM-I Table B.4.2)
$C_c = 0.41 (B_c / D_c)$ = 78.38 , (ADM-I Table B.4.2)
$n_u = \Omega_c / 0.85$ = 1.941 , (ADM-I E.1, E.3-2 & E.3-3)
$n_y = \Omega_c$ = 1.65 , (ADM-I B.3-2, E.1, E.3-2 & E.3-3)
$S_2 = C_c$ = 78.38 , (ADM-I E.3-4)

$$F_{a2}=\frac{P_n}{A_g\Omega_c}=\begin{cases} \dfrac{F_{cy}}{n_y}\ ,\ for\ \dfrac{b}{t}\leq S_1 \\ \dfrac{\left(B_p-\dfrac{1.6D_p b}{t}\right)}{n_u}\ ,\ for\ S_1<\dfrac{b}{t}<S_2 \\ \dfrac{k_2\sqrt{B_p E}}{n_u\dfrac{1.6b}{t}}\ ,\ for\ \dfrac{b}{t}\geq S_2 \end{cases} = 15.15$$ ksi, (compression in web, ADM-I B.5.4.2)

Where b / t = 6.000 , (ADM-I Figure B.5.2)
k_1 = 0.35 , (ADM-I Table B.4.3)
k_2 = 2.27 , (ADM-I Table B.4.3)
$B_p = F_{cy} [1 + F_{cy}{}^{(1/3)} / 11.447]$ = 31.39 , (ADM-I Table B.4.2)

(cont'd)

$D_p = (B_p / 10) (B_p / E)^{0.5} =$ 0.17 , (ADM-I Table B.4.2)
$S_1 = (B_p - n_u F_{cy} / n_y) / (1.6 D_p) =$ 7.05 , (ADM-I B5.4.2)
$S_2 = k_1 B_p / (1.6 D_p) =$ 39.24 , (ADM-I B5.4.2)

$f_a = P / A =$ 0.00 ksi < $F_a = Min (F_{a1} , F_{a2}) =$ 2.90 ksi
[Satisfactory]

CHECK TENSION STRESS IN BENDING MOMENTS

$F = P_n/(A_g \Omega_t) = Min (F_{ty}/n_y , F_{tu}/k_t n_u , 1.3F_{ty}/n_y , 1.42F_{tu}/k_t n_u) =$ 15.15 ksi, (tension bending, ADM-I D.1, D.2 & F.8.1.2)
Where $k_t =$ 1.00 , (ADM-I Table A.3.3)

$f = Min (M_{rx} / S_x , M_{ry} / S_y) =$ 0.00 ksi < $F =$ 15.15 ksi
[Satisfactory]
Where $S_x =$ 0.11 in, (Page V-34 to V-37)
$S_y =$ 0.11 in, (Page V-34 to V-37)

CHECK COMPRESSION STRESS IN STRONG GEOMETRIC AXIS, x - x, BENDING MOMENT

$$F_{bx1} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \frac{F_{cy}}{n_y} \ , \ for \ \frac{L_b S_c}{C_b \sqrt{I_y J / 2}} \le S_1 \\ \frac{1}{n_y}\left(B_c - 1.6 D_c \sqrt{\frac{L_b S_c}{C_b \sqrt{I_y J / 2}}}\right) \ , \ for \ S_1 < \frac{L_b S_c}{C_b \sqrt{I_y J / 2}} < S_2 \\ \frac{\pi^2 E}{2.56 n_y \left(\frac{L_b S_c}{C_b \sqrt{I_y J / 2}}\right)} \ , \ for \ \frac{L_b S_c}{C_b \sqrt{I_y J / 2}} \ge S_2 \end{cases} = 15.15$$
ksi, (compression in beam, ADM-I F.3.1)

Where $S_1 = (B_c - F_{cy})^2 / (1.6 D_c)^2 =$ 129.82 , (ADM-I F.3.1)
$S_2 = (C_c / 1.6)^2 =$ 2399.9 , (ADM-I F.3.1)
$J =$ 0.0837 , (Page V-34 to V-37)
$S_c = S_x =$ 0.11 in, (Page V-34 to V-37)
$C_b =$ 1.00 , (ADM-I F.1.1)
$L_b S_c / [C_b (I_y J)^{0.5} / 2] =$ 99.03

$$F_{bx2} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \frac{F_{cy}}{n_y} \ , \ for \ \frac{b}{t} \le S_1 \\ \frac{\left(B_p - \frac{1.6 D_p b}{t}\right)}{n_y} \ , \ for \ S_1 < \frac{b}{t} < S_2 \\ \frac{k_2 \sqrt{B_p E}}{n_y \frac{1.6 b}{t}} \ , \ for \ \frac{b}{t} \ge S_2 \end{cases} = 15.15$$
ksi, (compression in flanges, ADM-I B.5.4.2)

Where $b / t =$ 6.000 , (ADM-I Figure B.5.2)
$k_1 =$ 0.5 , (ADM-I Table B.4.3)
$k_2 =$ 2.04 , (ADM-I Table B.4.3)
$S_1 = (B_p - F_{cy}) / (1.6 D_p) =$ 22.81 , (ADM-I B5.4.2)
$S_2 = k_1 B_p / (1.6 D_p) =$ 56.06 , (ADM-I B5.4.2)

$$F_{bx3} = \frac{M_n}{S_x \Omega_b} = \begin{cases} \frac{1.3 F_{cy}}{n_y} \ , \ for \ \frac{h}{t} \le S_1 \\ \frac{\left(B_{br} - \frac{m D_{br} h}{t}\right)}{n_y} \ , \ for \ S_1 < \frac{h}{t} < S_2 \\ \frac{k_2 \sqrt{B_{br} E}}{n_y \frac{m h}{t}} \ , \ for \ \frac{h}{t} \ge S_2 \end{cases} = 26.28$$
ksi, (compression in web, ADM-I B.5.5.1)

Where $h / t = b / t =$ 6.000 , (ADM-I Figure B.5.2)
$c_c = c_o =$ 0.5 in

(cont'd)

m = 0.65 , (ADM-I B5.5.1)

$B_{br} = 1.3\,F_{cy}\,[1 + F_{cy}^{(1/3)} / 7]$ = 46.08 , (ADM-I Table B.4.2)

$D_{br} = (B_{br} / 20)\,(6\,B_{br} / E)^{(1/3)}$ = 0.69 , (ADM-I Table B.4.2)

$S_1 = (B_{br} - 1.3\,F_{cy}) / (m\,D_{br})$ = 0.45 , (ADM-I B5.5.1)

$S_2 = k_1\,B_{br} / (m\,D_{br})$ = 51.05 , (ADM-I B5.5.1)

$f_{bx} = M_{rx} / S_x$ = 13.16 ksi **<** $F_{bx} = Min\,(F_{bx1}, F_{bx2}, F_{bx3})$ = 15.15 ksi

[Satisfactory]

CHECK COMPRESSION STRESS IN WEAK GEOMETRIC AXIS, y - y, BENDING MOMENT

$$F_{by} = \frac{M_n}{S_y \Omega_b} = \begin{cases} \dfrac{F_{cy}}{n_y} \;, & for\;\; \dfrac{b}{t} \le S_1 \\ \dfrac{\left(B_p - \dfrac{1.6 D_p b}{t}\right)}{n_y} \;, & for\;\; S_1 < \dfrac{b}{t} < S_2 \\ \dfrac{k_2\sqrt{B_p E}}{n_y \dfrac{1.6b}{t}} \;, & for\;\; \dfrac{b}{t} \ge S_2 \end{cases} = 15.15$$

ksi, (compression in flanges, ADM-I B.5.4.2)

Where b / t = 6.000 , (ADM-I Figure B.5.2)

$f_{by} = M_{ry} / S_y$ = 0.00 ksi **<** F_{by} = 15.15 ksi

[Satisfactory]

CHECK COMBINED COMPRESSION AND BENDING (ADM-I H.1)

$$Max\begin{pmatrix} \dfrac{P_r}{P_c} + \dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}} = \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{by}}{F_{by}} \\ \left[\dfrac{f_a}{F_a} + \dfrac{C_{mx} f_{bx}}{F_{bx}\left(1 - f_a / F_{ex}\right)} + \dfrac{C_{my} f_{by}}{F_{by}\left(1 - f_a / F_{ey}\right)}\right] Check \end{pmatrix} = 0.87$$

< 1.33 , (1.33 if IBC/CBC 1605.3.2 apply)

[Satisfactory]

Where $Check$ = 1 Check if ASD 2005 Eq. 4.1.1-1 apply (1 = Yes, 0 = No)

C_{mx} = 0.85 C_{my} = 0.85

$F_{ex} = \pi^2 E / n_u (kL_x / r_x)^2$ = 2.90 ksi

$F_{ey} = \pi^2 E / n_u (kL_y / r_y)^2$ = 2.90 ksi

CHECK SHEAR STRESS (ADM-I G)

$$F_s = \frac{V_n}{A_w \Omega_v} = \begin{cases} \dfrac{F_{ty} / \sqrt{3}}{n_y} \;, & for\;\; \dfrac{h}{t} \le S_1 \\ \dfrac{\left(B_s - \dfrac{1.25 D_s h}{t}\right)}{n_y} \;, & for\;\; S_1 < \dfrac{h}{t} < S_2 \\ \dfrac{\pi^2 E}{n_y \left(\dfrac{1.25h}{t}\right)^2} \;, & for\;\; \dfrac{h}{t} \ge S_2 \end{cases} =$$

8.75 ksi, (for strong shear)
8.75 ksi, (for weak shear)
, (shear, ADM-I G.2)

Where $h / t = b / t$ = 6.000 , (for strong shear, Figure G.2)

$h / t = b / t$ = 6.000 , (for weak shear)

$B_s = (F_{cy} / 3^{0.5})\,[1 + (F_{cy} / 3^{0.5})^{(1/3)} / 9.3]$ = 18.21 , (ADM-I Table B.4.2)

$D_s = (B_s / 10)\,(B_s / E)^{0.5}$ = 0.08 , (ADM-I Table B.4.2)

$C_s = 0.41\,B_s / D_s$ = 96.55 , (ADM-I Table B.4.2)

$S_1 = (B_s - F_{cy} / 3^{0.5}) / (1.25\,D_s)$ = 39.09 , (ADM-I G.2)

$S_2 = C_s / 1.25$ = 77.24 , (ADM-I G.2)

$f_s = V_{strong} / A_w$ = 0.10 ksi **<** F_s = 8.75 ksi, (for strong shear)

$f_s = V_{weak} / A_w$ = 0.00 ksi **<** F_s = 8.75 ksi, (for weak shear)

[Satisfactory]

ATTACHMENT TO BLDG:

$R_Y = 0.33^k * 1.6 = 0.528^{lbs}$ (ULT)

$R_T = 0.88^k * 1.6 = 1.408^{lbs}$ (ULT)

EAST SIDE

CONNECTION TO (E) CONCRETE WALL

EXISTING ANCHORS : ASSUME 3/8" EXP ANCHORS

EXISTING ANCHORS W/ 2" EMBEDMENT

ARE ADEQUATE (SEE SAMPLE CALC

TEST EXISTING ANCHORS CAPACITY BY

TORQUING (25FT-lb)

SOUTH SIDE

CONNECTION TO (E) TUBE-STEEL

EXISTING ANCHORS : ASSUME 1/4" SCREWS

INTO EXISTING HSS HEADERS

By: ________

Associated Consultants, Inc. Structural Engineers
100 East 13th Street • Suite 10 • Vancouver, Washington 98660
Portland: (503) 384-0460 • Vancouver: (360) 699-0607

Project	Job No.
Awning Replacement - Double Dragon Restaurant	25-111
Location	**Date**
39131 Pioneer Blvd. - Sandy, OR 97055	
Client	**Sheet No.**
Keystone Architecture Planning and Project Managment	

SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.4.2504.1

Company:		Date:	5/2/2025
Engineer:		Page:	1
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Project description:
Location:
Design name: Design

Comment:

2. Input Data & Anchor Parameters

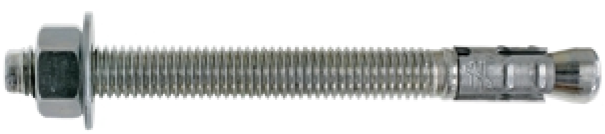
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Torque controlled expansion anchor
Material: Carbon Steel
Diameter (inch): 0.375
Nominal Embedment depth (inch): 1.875
Effective Embedment depth, h_{ef} (inch): 1.500
Code report: ICC-ES ESR-3037
Anchor category: 1
Anchor ductility: Yes
h_{min} (inch): 3.25
c_{ac} (inch): 6.50
C_{min} (inch): 6.00
S_{min} (inch): 3.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 6.00
State: Cracked
Compressive strength, f'_c (psi): 2500
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental edge reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: Not applicable
Build-up grout pad: No

Base Plate
Length x Width x Thickness (inch): 1.00 x 8.00 x 0.25

Recommended Anchor
Anchor Name: Strong-Bolt® 2 - 3/8"Ø STB2, hnom:1.875" (48mm)
Code Report: ICC-ES ESR-3037

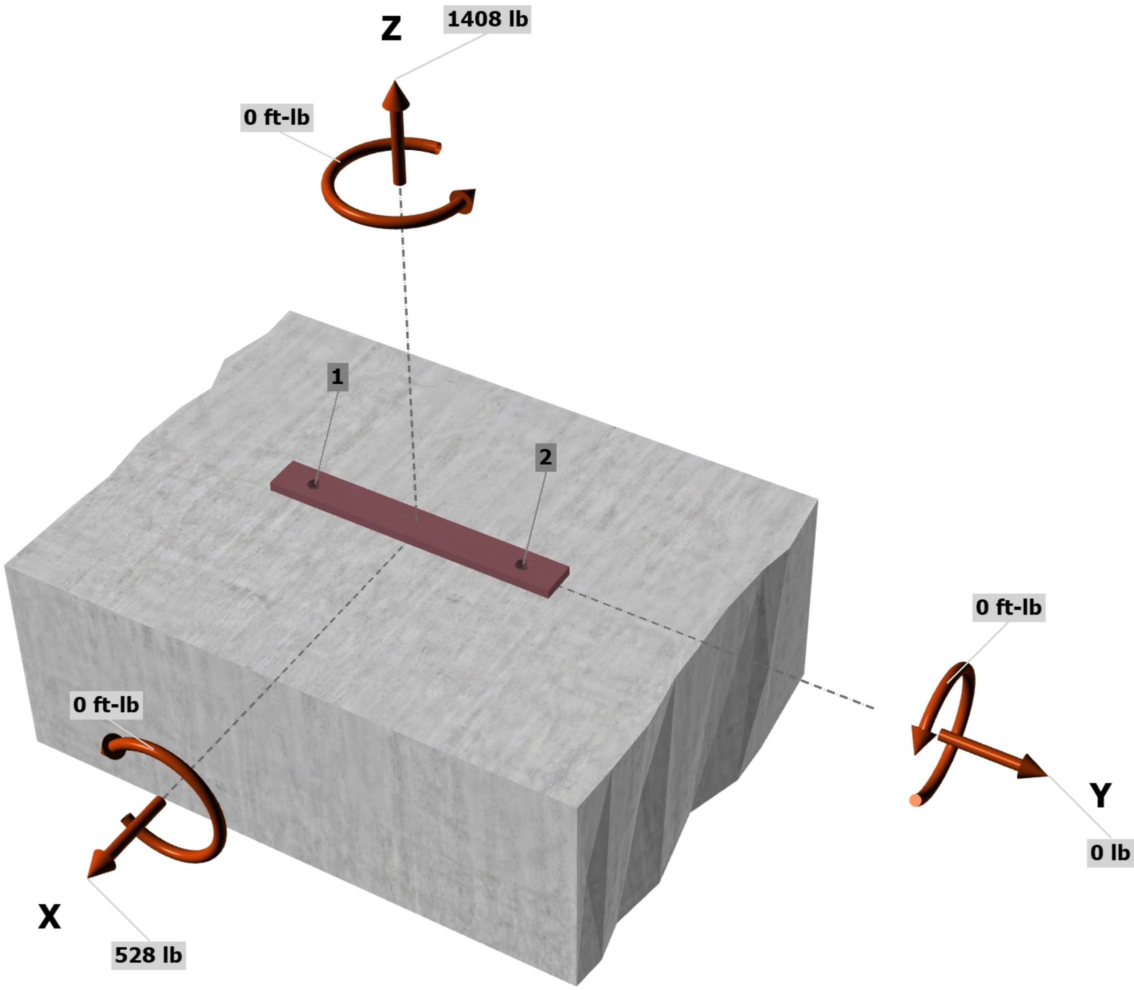


Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: Not applicable
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: No

Strength level loads:

N_{ua} [lb]:	1408
V_{uax} [lb]:	528
V_{uay} [lb]:	0
M_{ux} [ft-lb]:	0
M_{uy} [ft-lb]:	0
M_{uz} [ft-lb]:	0

<Figure 1>

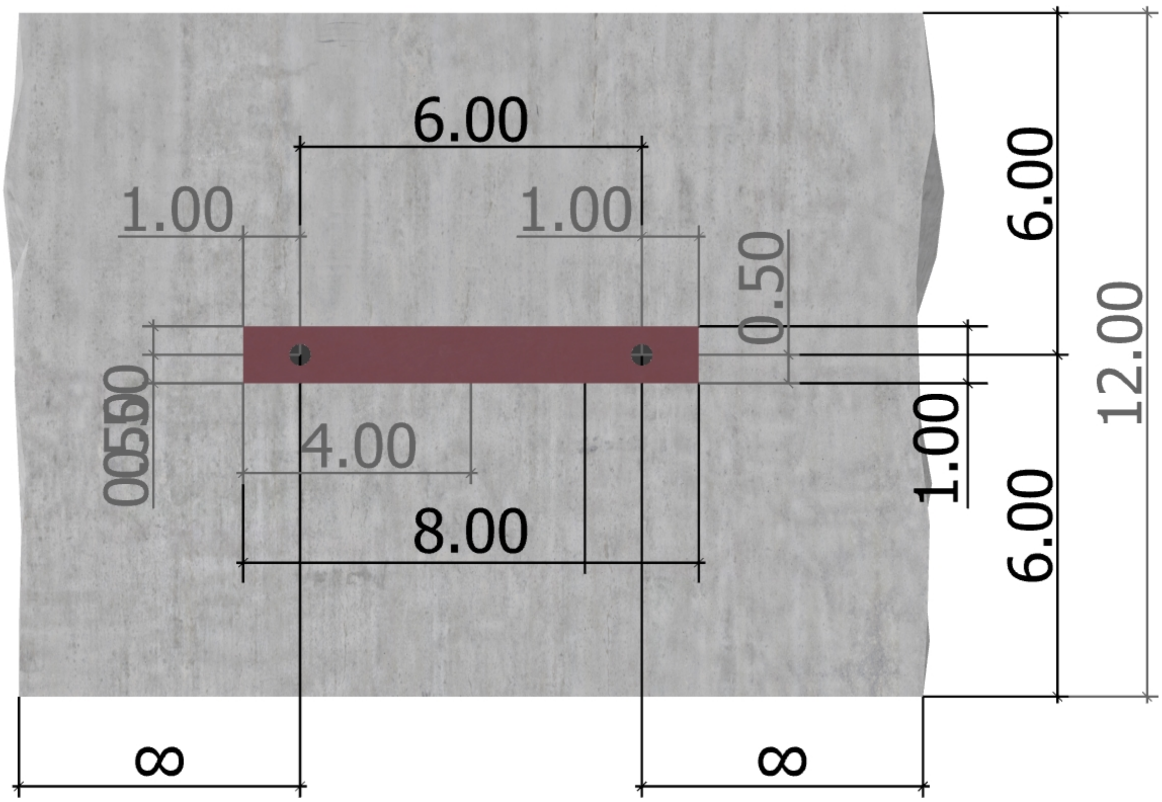


SIMPSON Strong-Tie

Anchor Designer™ for
Concrete Software
Version 3.4.2504.1

Company:		Date:	5/2/2025
Engineer:		Page:	3
Project:			
Address:			
Phone:			
E-mail:			

<Figure 2>

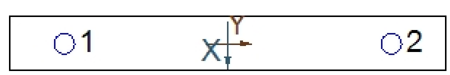


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	704.0	264.0	0.0	264.0
2	704.0	264.0	0.0	264.0
Sum	1408.0	528.0	0.0	528.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 1408
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



SIMPSON Strong-Tie

Anchor Designer™ for Concrete Software
Version 3.4.2504.1

Company:		Date:	5/2/2025
Engineer:		Page:	4
Project:			
Address:			
Phone:			
E-mail:			

4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
5600	0.75	4200

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = k_c \lambda_a \sqrt{f'_c} h_{ef}^{1.5}$ (Eq. 17.4.2.2a)

k_c	λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
17.0	1.00	2500	1.500	1562

$\phi N_{cbg} = \phi\,(A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕN_{cbg} (lb)
40.50	20.25	6.00	1.000	1.000	1.00	1.000	1562	0.65	2030

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$\phi N_{pn} = \phi \Psi_{c,P} \lambda_a N_p (f'_c / 2{,}500)^n$ (Sec. 17.3.1, Eq. 17.4.3.1 & Code Report)

$\Psi_{c,P}$	λ_a	N_p (lb)	f'_c (psi)	n	ϕ	ϕN_{pn} (lb)
1.0	1.00	1300	2500	0.50	0.65	845

7. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout} \phi V_{sa}$ (lb)
1800	1.0	0.65	1170

8. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e / d_a)^{0.2} \sqrt{d_a} \lambda_a \sqrt{f'_c} c_{a1}^{1.5};\ 9 \lambda_a \sqrt{f'_c} c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
1.50	0.375	1.00	2500	6.00	4156

$\phi V_{cbgx} = \phi\,(A_{Vc} / A_{Vco})\,\Psi_{ec,V}\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V} V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
144.00	162.00	1.000	1.000	1.000	1.225	4156	0.70	3167

9. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp} N_{cbg} = \phi k_{cp} (A_{Nc} / A_{Nco})\,\Psi_{ec,N}\,\Psi_{ed,N}\,\Psi_{c,N}\,\Psi_{cp,N} N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
1.0	40.50	20.25	1.000	1.000	1.000	1.000	1562	0.70	2186

10. Results

Interaction of Tensile and Shear Forces (Sec. R17.6)

Tension	Factored Load, N_{ua} (lb)	Design Strength, $øN_n$ (lb)	Ratio	Status
Steel	704	4200	0.17	Pass
Concrete breakout	1408	2030	0.69	Pass
Pullout	**704**	**845**	**0.83**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, $øV_n$ (lb)	Ratio	Status
Steel	264	1170	0.23	Pass
T Concrete breakout x+	528	3167	0.17	Pass
Pryout	**528**	**2186**	**0.24**	**Pass (Governs)**

Interaction check	$(N_{ua}/\phi N_{ua})^{5/3}$	$(V_{ua}/\phi V_{ua})^{5/3}$	Utilization Ratio	Permissible	Status

Sec. R17.6	0.74	0.09	83.1%	1.0	Pass

3/8"Ø STB2, hnom:1.875" (48mm) meets the selected design criteria.

11. Warnings

- Designer must exercise own judgement to determine if this design is suitable.

- Refer to manufacturer’s product literature for hole cleaning and installation instructions.

CAPACITY OF 1/4" SCREW

$F_V = 1{,}016^{lbs} * 2 = 2{,}032^{lbs}$

$F_T = 572^{lbs} * 2 = 1{,}114^{lbs}$

$$\frac{330}{2{,}032} + \frac{880}{1{,}114} = 0.95 < 1.0 \rightarrow OK$$

By: ________

Associated Consultants, Inc. Structural Engineers	Project: Awning Replacement - Double Dragon Restaurant	Job No. 25-111
100 East 13th Street • Suite 10 • Vancouver, Washington 98660 Portland: (503) 384-0460 • Vancouver: (360) 699-0607	Location: 39131 Pioneer Blvd. - Sandy, OR 97055	Date
	Client: Keystone Architecture Planning and Project Managment	Sheet No.

Screw Capacities

Table Notes

1. Capacities based on AISI S100 Section E4.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values. Tabulated values assume two sheets of equal thickness are connected.
3. Capacities are based on Allowable Strength Design (ASD) and include safety factor of 3.0.
4. Where multiple fasteners are used, screws are assumed to have a center-to-center spacing of at least 3 times the nominal diameter (d).
5. Screws are assumed to have a center-of-screw to edge-of-steel dimension of at least 1.5 times the nominal diameter (d) of the screw.
6. Pull-out capacity is based on the lesser of pull-out capacity in sheet closest to screw tip or tension strength of screw.
7. Pull-over capacity is based on the lesser of pull-over capacity for sheet closest to screw header or tension strength of screw.
8. Values are for pure shear or tension loads. See AISI Section E4.5 for combined shear and pull-over.
9. Screw Shear (Pss), tension (Pts), diameter, and head diameter are from CFSEI Tech Note (F701-12).
10. Screw shear strength is the average value, and tension strength is the lowest value listed in CFSEI Tech Note (F701-12).
11. Higher values for screw strength (Pss, Pts), may be obtained by specifying screws from a specific manufacturer.

Allowable Screw Connection Capacity (lbs)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	#6 Screw (Pss = 643 lbs, Pts = 419 lbs) 0.138" dia, 0.272" Head			#8 Screw (Pss= 1278 lbs, Pts = 586 lbs) 0.164" dia, 0.272" Head			#10 Screw (Pss= 1644 lbs, Pts = 1158 lbs) 0.190" dia, 0.340" Head			#12 Screw (Pss= 2330 lbs, Pts = 2325 lbs) 0.216" dia, 0.340" Head			¼" Screw (Pss= 3048 lbs, Pts = 3201 lbs) 0.250" dia, 0.409" Head		
				Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over	Shear	Pull-Out	Pull-Over
18	0.0188	33	33	44	24	84	48	29	84	52	33	105	55	38	105	60	44	127
27	0.0283	33	33	82	37	127	89	43	127	96	50	159	102	57	159	110	66	191
30	0.0312	33	33	95	40	140	103	48	140	111	55	175	118	63	175	127	73	211
33	0.0346	33	45	151	61	140	164	72	195	177	84	265	188	95	265	203	110	318
43	0.0451	33	45	214	79	140	244	94	195	263	109	345	280	124	345	302	144	415
54	0.0566	33	45	214	100	140	344	118	195	370	137	386	394	156	433	424	180	521
68	0.0713	33	45	214	125	140	426	149	195	523	173	386	557	196	545	600	227	656
97	0.1017	33	45	214	140	140	426	195	195	548	246	386	777	280	775	1,016	324	936
118	0.1242	33	45	214	140	140	426	195	195	548	301	386	777	342	775	1,016	396	1,067
54	0.0566	50	65	214	140	140	426	171	195	534	198	386	569	225	625	613	261	752
68	0.0713	50	65	214	140	140	426	195	195	548	249	386	777	284	775	866	328	948
97	0.1017	50	65	214	140	140	426	195	195	548	356	386	777	405	775	1,016	468	1,067
118	0.1242	50	65	214	140	140	426	195	195	548	386	386	777	494	775	1,016	572	1,067

Weld Capacities

Table Notes

1. Capacities based on the AISI S100 Specification Sections E2.4 for fillet welds and E2.5 for flare groove welds.
2. When connecting materials of different steel thicknesses or tensile strengths, use the lowest values.
3. Capacities are based on Allowable Strength Design (ASD).
4. Weld capacities are based on E60 electrodes. For material thinner than 68 mil, 0.030" to 0.035" diameter wire electrodes may provide best results.
5. Longitudinal capacity is considered to be loading in the direction of the length of the weld.
6. Transverse capacity is loading in perpendicular direction of the length of the weld.
7. For flare groove welds, the effective throat of weld is conservatively assumed to be less than 2t.
8. For longitudinal fillet welds, a minimum value of EQ E2.4-1, E2.4-2, and E2.4-4 was used.
9. For transverse fillet welds, a minimum value of EQ E2.4-3 and E2.4-4 was used.
10. For longitudinal flare groove welds, a minimum value of EQ E2.5-2 and E2.5-3 was used.

Allowable Weld Capacity (lbs / in)

Thickness (Mils)	Design Thickness	Fy Yield (ksi)	Fu Tensile (ksi)	Fillet Welds		Flare Groove Welds	
				Longitudinal	Transverse	Longitudinal	Transverse
43	0.0451	33	45	499	864	544	663
54	0.0566	33	45	626	1084	682	832
68	0.0713	33	45	789	1365	859	1048
97	0.1017	33	45	1125	1269	-[1]	-[1]
54	0.0566	50	65	905	1566	985	1202
68	0.0713	50	65	1140	1972	1241	1514
97	0.1017	50	65	1269	1269	-[1]	-[1]

[1]Weld capacity for material thickness greater than 0.10" requires engineering judgment to determine leg of welds, W1 and W2.