

# CALCULATIONS FOR

Job: 18'-0" x 18'-2" x 10'-0" Max Height - 6 Post Canopy - State of Utah

Address : 2006 International Building Code

(40 psf Snow, 90 mph, Seismic Design Category D, Frost Depth 30")

**(These calculations apply to the job at this address only.)**

Client: Western States Decking, Inc

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THESE STRUCTURAL CALCULATIONS MUST BE SUBMITTED WITH **WET SEAL** DATED NOT OVER **180 DAYS** PRIOR TO PERMIT APPLICATIONS.

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Project Engineer G Starks

Job # 1306-09

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**S.E. CONSULTANTS, INC.**

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**BASIS FOR DESIGN**

<u>DEAD LOADS</u>		actual
<u>LIVE LOADS</u>		
Roofs		20.0 psf.,
<u>LATERAL</u>		
Wind	Wind Load Basis: Exposure:	90 mph (3 sec. Gust) C
Seismic Use Group: Seismic Design Cat:		I D
<u>CODE:</u>	2006 International Building Code 2001 Edition of Cold-Formed steel design manual W/ 2004 supplement	

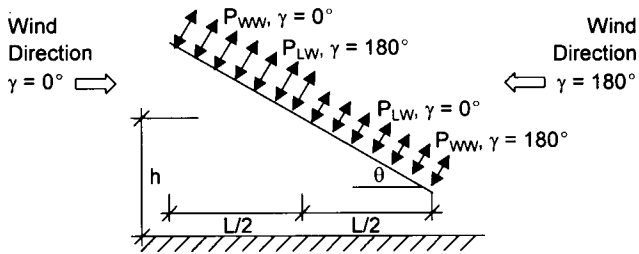
**STRESSES OF MATERIALS**

<u>CONCRETE</u>			
Footings	f'c =	2500 psi.	
<u>STEEL</u>			
Reinforcing	fy =	40000 psi. A-615, Grade 40	
Weldable Reinforcing	fy =	40000 psi. A-706, Grade 40	
Roof Deck	fy =	80000 psi. A-653, Grade 80	
Cold formed Steel	fy =	55000 psi. A-653, Grade 55	
<u>SOIL</u>			
	Allow. Soil Bearing	Passive Pressure	
	1500 psf	150 psf / ft	
	Soil Class = D	Material Class =	5 [Table 1802.4]
<u>Structural</u>	Wide Flange	fy =	36000 psi. A-36 or
		fy =	50000 psi. A-572
	Tube	fy =	46000 psi. A-500
	Pipe	fy =	36000 psi. A-501

<b>PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET</b>			
Job:	<u>6 post</u>	Date:	<u>11/09</u>
	<u>Canopy</u>	By:	<u>GCS</u>
		Job No.:	<u>1706-09</u>
		SH.:	<u>2</u>
<b>S.E. CONSULTANTS, INC.</b>			

**DESIGN WIND LOADS FOR MONOSLOPE FREE ROOFS**

**ASCE 7 - 6.5.13**



V, wind speed = 90 mph  
 Exposure Category = C  
 θ, roof angle = 0.6 °  
 h, mean roof height = 10.0 ft

**Main Wind Force Resisting System**

$K_z$ , exposure coeff. = 0.85 ASCE7 6.5.6.6, Table 6-3  
 $K_{zt}$ , topography factor = 1.00 ASCE7 6.5.7.2, Figure 6-4  
 $K_d$ , directionality factor = 0.85 ASCE7 6.5.4.4, Table 6-4  
 $I_w$ , wind factor = 1.00 ASCE7 6.5.5, Table 6-1  
 G, gust effect factor = 0.85 ASCE7 6.5.8  
 P =  $qzGC_N$  = 12.73 ( $C_N$ ) psf  $0.00256K_zK_{zT}K_DV^2I_wGC_N$ , Eq. 6-25

Roof Angle, θ	Load Case	Wind Direction, γ = 0°				Wind Direction, γ = 180°			
		Clear Flow		Obstructed Flow		Clear Flow		Obstructed Flow	
		$C_{NW}$	$C_{NL}$	$C_{NW}$	$C_{NL}$	$C_{NW}$	$C_{NL}$	$C_{NW}$	$C_{NL}$
0°	A	1.2	0.3	-0.5	-1.2	1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.9	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3
22.5°	A	-1.5	-1.6	-1.5	-1.7	1.7	1.8	0.5	-1
	B	-2.4	-0.3	-2.3	-0.9	2.2	0.7	1.3	0
30°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.1	0.6	-1
	B	-2.5	-0.5	-2.3	-1.1	2.6	1	1.6	0.1
37.5°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.2	0.7	-0.9
	B	-2.4	-0.6	-2.2	-1.1	2.7	1.1	1.9	0.3
45°	A	-1.6	-1.8	-1.3	-1.8	2.2	2.5	0.8	-0.9
	B	-2.3	-0.7	-1.9	-1.2	2.6	1.4	2.1	0.4

Linearly interpolated coefficients from Figure 6-18A shown below.

0.6	A	1.20	0.30	-0.50	-1.20	1.20	0.30	-0.50	-1.20
	B	-1.10	-0.10	-1.10	-0.60	-1.10	-0.10	-1.10	-0.60

Is flow obstructed (Yes/No)? NO

Load Case A (γ = 0 °)

P, windward roof = 15.28 psf  
 P, leeward roof = 3.82 psf

Load Case B (γ = 0 °)

P, windward roof = -14.01 psf  
 P, leeward roof = -1.27 psf

Load Case A (γ = 180 °)

P, windward roof = 15.28 psf  
 P, leeward roof = 3.82 psf

Load Case B (γ = 180 °)

P, windward roof = -14.01 psf  
 P, leeward roof = -1.27 psf

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Jbb: 6 post canopy Date: 11-09 By: BCG Jbb No.: 1306-09 SH.: 2

**S.E. CONSULTANTS, INC.**

**International Building Code - 2006**

Soil Site Class = D  
 Seismic Use Group = I O.K.

S<sub>s</sub> = 178.7 %  
 S<sub>1</sub> = 75.0 %

Site Class	Table 1615.1.2 (1) (F <sub>a</sub> )				
	Mapped spectral response acceleration at short periods (S <sub>s</sub> )				
	0.25	0.50	0.75	1.00	1.25
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.20	1.20	1.10	1.00	1.00
D	1.60	1.40	1.20	1.10	1.00
E	2.50	1.70	1.20	0.90	0.90
F	-	-	-	-	-

F<sub>a</sub> = 1.000 (interpolated)      S<sub>M</sub>S = F<sub>a</sub> \* S<sub>s</sub> = 1.787

Site Class	Table 1615.1.2 (2) (F <sub>v</sub> )				
	Mapped spectral response acceleration at 1 sec. periods (S <sub>1</sub> )				
	0.10	0.20	0.30	0.40	0.50
A	0.80	0.80	0.80	0.80	0.80
B	1.00	1.00	1.00	1.00	1.00
C	1.70	1.60	1.50	1.40	1.30
D	2.40	2.00	1.80	1.60	1.50
E	3.50	3.20	2.80	2.40	2.40
F	-	-	-	-	-

F<sub>v</sub> = 1.5 (interpolated)      S<sub>M1</sub> = F<sub>v</sub> \* S<sub>1</sub> = 1.125

S<sub>Ds</sub> = 2/3 \* S<sub>M</sub>S = 1.191      S<sub>DI</sub> = 2/3 \* S<sub>M1</sub> = 0.750

S <sub>Ds</sub>	Seismic Use Group		
	I	II	III
0.000	A	A	A
0.167	B	B	C
0.330	C	C	D
0.500	D	D	D

S <sub>DI</sub>	Seismic Use Group		
	I	II	III
0.000	A	A	A
0.067	B	B	C
0.133	C	C	D
0.200	D	D	D

Seismic Design Category = D

<b>PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET</b>			
Job: <u>6 post images</u>	Date: <u>11-09</u>	By: <u>GCS</u>	Job No.: <u>1306-09</u> SH.: <u>3</u>
<b>S.E. CONSULTANTS, INC.</b>			

# Earthquake Lateral/Longitudinal Load to Column - 2006 IBC

Seismic Use Group	=	I
Seismic Design Category	=	D
I	=	1.00
S <sub>s</sub>	=	178.68 %
S <sub>1</sub>	=	75.03 %
S <sub>M<sub>s</sub></sub>	=	1.787
S <sub>M<sub>1</sub></sub>	=	1.125
R	=	2.50
Height	=	10 ft
T = 0.02 * (hn) <sup>0.75</sup>	=	0.112
r Reliability/redundancy Factor	=	1.00
S <sub>Ds</sub>	=	1.191
S <sub>DI</sub>	=	0.750

(16-35)  $V = S_{Ds} / [R/I] = 0.476 W$

(16-36)  $V = S_{DI} / [(R/I)T] = 2.668 W$

(16-37)  $V = 0.044 * S_1 / [R/I] = 0.013 W$

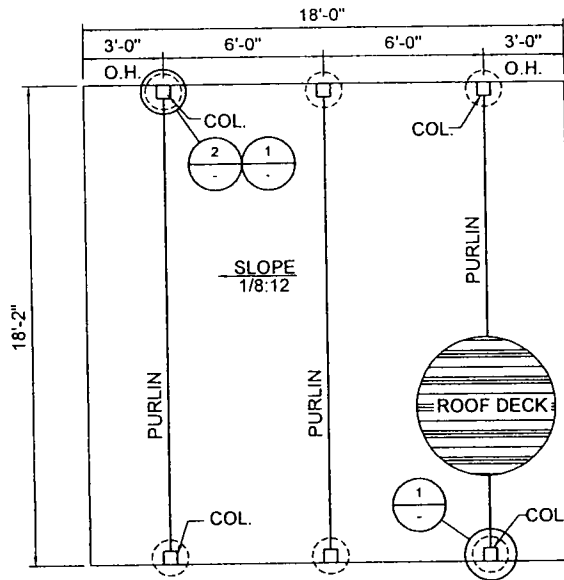
(16-38)  $V = 0.5 * S_1 / [R/I] = 0.150 W$

V (Controls) = 0.476 W kips

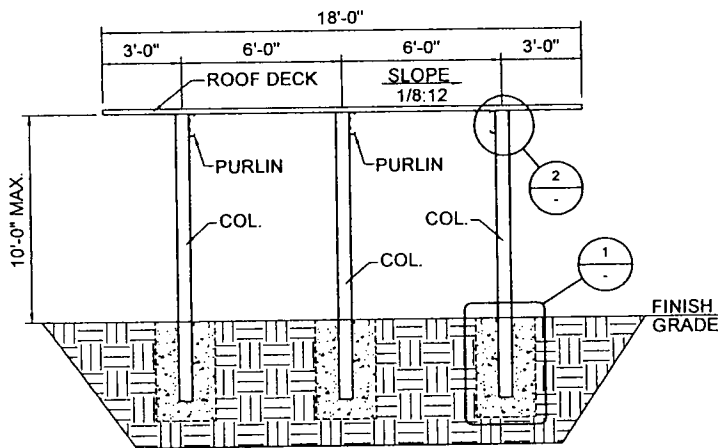
PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 6 post canopy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 4

**S.E. CONSULTANTS, INC.**



ROOF FRAMING PLAN  
& FOUNDATION PLAN



CROSS SECTION

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 6 post canopy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 5

**S.E. CONSULTANTS, INC.**

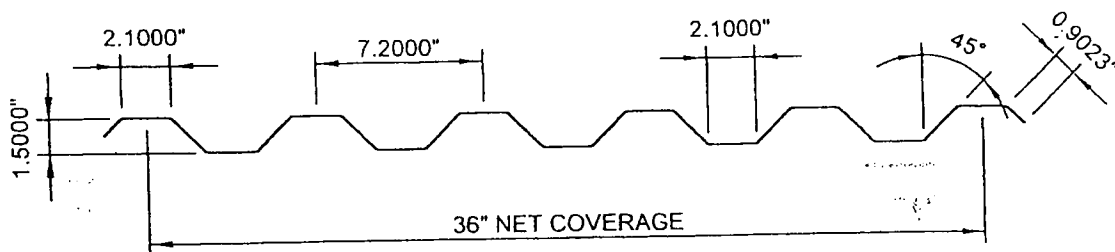
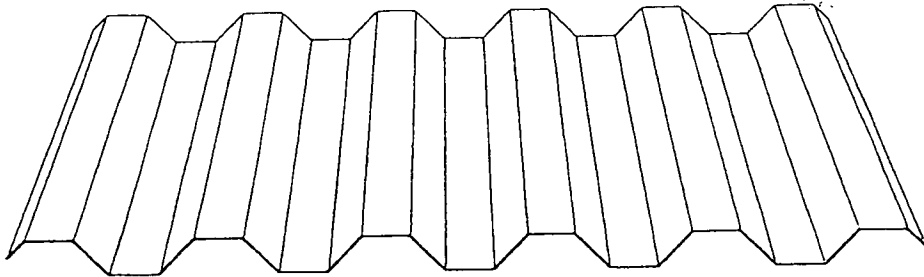
<b>.26 ga Western Span</b>		
F <sub>y</sub> , yield strength	=	90.0 ksi
F <sub>u</sub> , tensile strength	=	60.0 ksi
I <sub>x</sub> , moment of inertia	=	0.07 in <sup>4</sup> /ft
S <sub>x(t)</sub> , section modulus	=	0.07 in <sup>3</sup> /ft
S <sub>x(b)</sub> , section modulus	=	0.06 in <sup>3</sup> /ft

See Manufacture's Specifications.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Jbb: 6 post canopy Date: 11-09 By: GCS Jbb No.: 1306-09 SH.: 6

# WESTERN SPAN



Section Properties			Top in Compression			Bott. in Compression		
gauge	$f_y$	wt	ix	sx	ma	ix	sx	ma
29	80	0.62	0.0399	0.0407	1.4635	0.0372	0.0847	1.2481
26	80	0.86	0.0658	0.0743	2.6653	0.0621	0.0624	2.2419
24	50	1.10	0.0917	0.1090	3.9168	0.0911	0.0979	3.5208
22	50	1.40	0.1249	0.1521	5.4630	0.1249	0.1375	4.9392

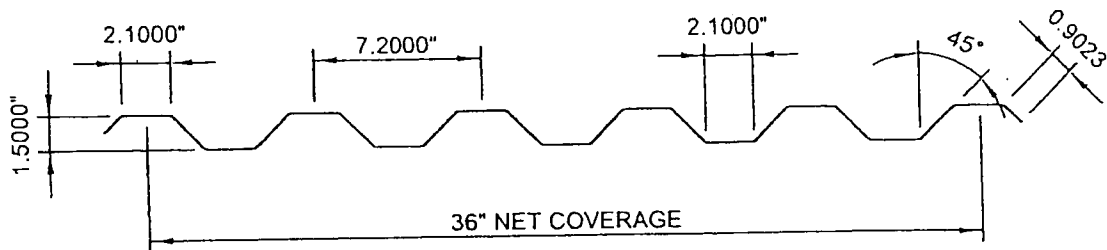
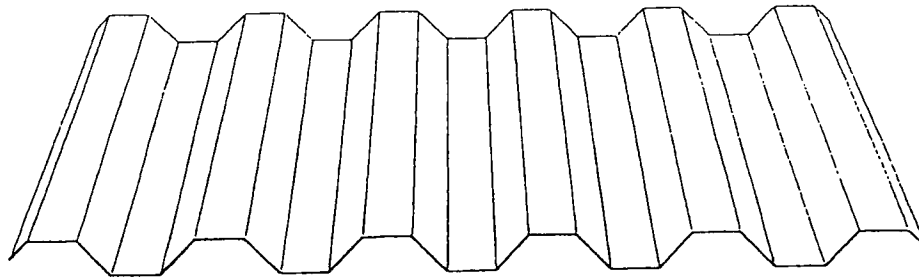
**NOTES:**

1. Effective section properties are calculated in accordance with the 2004 North American Specifications for the design of Cold-Formed Steel Structural Members
2.  $I_x$  is for the determination of deflection
3.  $S_x$  and  $M_a$  are for stress determination

ISSUED: APRIL 18, 2008



# WESTERN SPAN



		span type	load type	Gravity - Total Allowable Load in psf, (span in feet)									
				3	3.5	4	4.5	5	6	7	8	9	10
26 Gage (Fy=80ksi)	single span	stress	197.4	145.1	111.1	87.7	71.1	49.4	36.3	27.8	21.9	17.8	
		deflection	209.4	131.9	88.3	62.0	45.2	26.2	16.5	11.0	7.8	5.7	
	2 spans	stress	166.1	122.0	93.4	73.8	59.8	41.5	30.5	23.4	18.5	14.9	
		deflection	476.1	299.8	200.8	141.1	102.8	59.5	37.5	25.1	17.6	12.9	
	3 spans or more	stress	193.8	142.4	109.0	86.1	69.8	48.5	35.6	27.3	21.5	17.4	
		deflection	395.9	249.3	167.0	117.3	85.5	49.5	31.2	20.9	14.7	10.7	
		span type	load type	Uplift - Total Allowable Load in psf, (span in feet)									
				3	3.5	4	4.5	5	6	7	8	9	10
26 Gage (Fy=80ksi)	single span	stress	166.1	122.0	93.4	73.8	59.8	41.5	30.5	23.4	18.5	14.9	
		deflection	197.6	124.5	83.4	58.6	42.7	24.7	15.6	10.4	7.3	5.3	
	2 spans	stress	197.4	145.1	111.1	87.7	71.1	49.4	36.3	27.8	21.9	17.8	
		deflection	504.4	317.7	212.8	149.5	109.0	63.1	39.7	26.6	18.7	13.6	
	3 spans or more	stress	230.4	169.3	129.6	102.4	83.0	57.6	42.3	32.4	25.6	20.7	
		deflection	419.5	264.2	177.0	124.3	90.6	52.4	33.0	22.1	15.5	11.3	

**NOTES:**

1. Allowable loads are based on section properties in accordance with the 2004 North American Specification for the Design of Cold-Formed Steel Structural Members.
2. Load capacities do not include the panel self weight. Capacities are for the panel itself and do not include the capacities for the panel attachments or panel supports.  
(fastener pullout or panel pullover is not included in capacities)
3. A 1/3 stress increase for wind is not included in allowables shown.
4. Allowable loads are based on Fy = 60 ksi.
5. A ratio of L/180 was used for all deflection calculations.

ISSUED: APRIL 18, 2008

**THE FOLLOWING IS BASED ON THE 2001 EDITION OF THE COLD-FORMED STEEL DESIGN MANUAL**

**DEFINITIONS:**

- d = Nominal screw diameter  
 dw = Larger of Head or Washer Diameter, not larger than 1/2"  
 Factor of Safety = 3.0  
 Pas = allowable shear force per screw  
 Pns = nominal shear strength per screw  
 Pat = allowable tension force per screw  
 Pnt = nominal tension strength per screw  
 Pnot = pull-out per screw  
 Pnov = pull-over per screw  
 t1 = thickness of member in contact with the screw head  
 t2 = thickness of member not in contact with the screw head  
 Fu1 = tensile strength of member in contact with screw  
 Fu2 = tensile strength of member not in contact with screw

t1 =	0.0179 in. Approximate Gauge =	26	Fu1 =	90000 psi, Fy =	80000 psi
t2 =	0.0747 in. Approximate Gauge =	14	Fu2 =	65000 psi, Fy =	55000 psi
Screw No.	12				
Shank Diam. =	0.216 in.				

**SECTION E4.3.1 CONNECTION SHEAR GOVERNED BY BASE METALS**

t2/t1 =	4.1732	t2/t1 >= 2.5, USE EQUATIONS E4.3.4 & E4.3.5
WHEN t2/t1 <= 1.0		WHEN t2/t1 >= 2.5
Equation E4.3.1	= 2590.4 lbs/screw	Equation E4.3.4 = 939.5 lbs/screw
Equation E4.3.2	= 939.5 lbs/screw	Equation E4.3.5 = 2831.7 lbs/screw
Equation E4.3.3	= 2831.7 lbs/screw	
Pns (smallest of the above:)	= 939.5 lbs/screw	Pns (smallest of the above:) = 939.5 lbs/screw
Pas = Allow. Shear per SCREW = Pns/ (F.S. of 3.0)	= 313.2 lbs/screw	* Allow. Shear Capacity of Screw = 625.0 #/screw
		* Based on "HILTI" w/ F.S. of 3.0

**SECTION E4.4, TENSION**

- Equation E4.4.1.1, Pullout Force, Pnot = 891.5 lbs/screw  
 Equation E4.4.2.1, Pullover Force, Pnov = 755.2 lbs/screw  
 Pnt = nominal tension strength per screw = lesser of Pnot & Pnov  
 Pnt = 755.2 lbs/screw  
 Pat = Allow. Tension per SCREW = Pnt/ (F.S. of 3.0) = 251.7 lbs/screw

Allowable Shear per Screw =	313.2 lbs/screw	Allowable Tension per Screw =	251.7 lbs/screw
NOTE: Shear Governed By Base Metals.			

- NOTE: 1. Minimum Spacing of screws shall not be less than 3d  
 2. Minimum Edge Distance of screws shall not be less than 3d  
 (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)  
 3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.  
 4. Values may be increased 33% for wind or earthquake loads.

<b>PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET</b>			
Job:	<i>6 part canopy</i>	Date:	<i>11-09</i>
		By:	<i>ACS</i>
		Job No.:	<i>1306-09</i>
		SH.:	<i>9</i>
<b>S.E. CONSULTANTS, INC.</b>			

THE FOLLOWING IS BASED ON THE 2001 EDITION OF THE COLD-FORMED STEEL DESIGN MANUAL

DEFINITIONS:

- d = Nominal screw diameter  
 dw = Larger of Head or Washer Diameter, not larger than 1/2"  
 Factor of Safety = 3.0  
 Pas = allowable shear force per screw  
 Pns = nominal shear strength per screw  
 Pat = allowable tension force per screw  
 Pnt = nominal tension strength per screw  
 Pnot = pull-out per screw  
 Pnov = pull-over per screw  
 t1 = thickness of member in contact with the screw head  
 t2 = thickness of member not in contact with the screw head  
 Fu1 = tensile strength of member in contact with screw  
 Fu2 = tensile strength of member not in contact with screw

t1 =	0.0747 in. Approximate Gauge =	14	Fu1 =	65000 psi, Fy =	55000 psi
t2 =	0.0747 in. Approximate Gauge =	14	Fu2 =	65000 psi, Fy =	55000 psi
Screw No.	12				
Shank Diam. =	0.216 in.				

SECTION E4.3.1 CONNECTION SHEAR GOVERNED BY BASE METALS

t2/t1 =	1	t2/t1 <= 1.0, USE EQUATIONS E4.3.1 TO E4.3.3			
Equation E4.3.1	=	2590.4 lbs/screw	Equation E4.3.4	=	2831.7 lbs/screw
Equation E4.3.2	=	2831.7 lbs/screw	Equation E4.3.5	=	2831.7 lbs/screw
Equation E4.3.3	=	2831.7 lbs/screw			
Pns (smallest of the above:)	=	2590.4 lbs/screw	Pns (smallest of the above:)	=	2831.7 lbs/screw
Pas = Allow. Shear per SCREW = Pns/ (F.S. of 3.0)	=	863.5 lbs/screw	* Allow. Shear Capacity of Screw	=	625.0 #/screw
			* Based on "HILTI" w/ F.S. of 3.0		

SECTION E4.4, TENSION

- Equation E4.4.1.1, Pullout Force, Pnot = 891.5 lbs/screw  
 Equation E4.4.2.1, Pullover Force, Pnov = 2276.0 lbs/screw  
 Pnt = nominal tension strength per screw = lesser of Pnot & Pnov  
 Pnt = 891.5 lbs/screw  
 Pat = Allow. Tension per SCREW = Pnt/ (F.S. of 3.0) = 297.2 lbs/screw

Allowable Shear per Screw =	625.0 lbs/screw	Allowable Tension per Screw =	297.2 lbs/screw
NOTE: Governed By Shear Capacity Of Screws			

- NOTE: 1. Minimum Spacing of screws shall not be less than 3d  
 2. Minimum Edge Distance of screws shall not be less than 3d  
 (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)  
 3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.  
 4. Values may be increased 33% for wind or earthquake loads.

<b>PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET</b>			
Job:	<i>6 post canopy</i>	Date:	<i>11-09</i>
		By:	<i>GCS</i>
		Job No.:	<i>1306-09</i>
		SH.:	<i>10</i>
<b>S.E. CONSULTANTS, INC.</b>			

SCREW CAPACITIES IN LIGHT GAUGE COLD FORMED STEEL

=====
THE FOLLOWING IS BASED ON THE 2001 EDITION OF THE COLD-FORMED STEEL DESIGN MANUAL

DEFINITIONS:

- d = Nominal screw diameter
dw = Larger of Head or Washer Diameter, not larger than 1/2"
Factor of Safety = 3.0
Pas = allowable shear force per screw
Pns = nominal shear strength per screw
Pat = allowable tension force per screw
Pnt = nominal tension strength per screw
Pnot = pull-out per screw
Pnov = pull-over per screw
t1 = thickness of member in contact with the screw head
t2 = thickness of member not in contact with the screw head
Fu1 = tensile strength of member in contact with screw
Fu2 = tensile strength of member not in contact with screw

SECTION E4.3.1 CONNECTION SHEAR

WHEN t2/t1 <= 1.0

Equation E4.3.1 Pns = 4.2(t2^3 \* d)^0.5 \* Fu2

Equation E4.3.2 Pns = 2.7 t1 \* d \* Fu1

Equation E4.3.3 Pns = 2.7 t2 \* d \* Fu2

WHEN t2/t1 >= 2.5

Equation E4.3.4 Pns = 2.7 t1 \* d \* Fu1

Equation E4.3.5 Pns = 2.7 t2 \* d \* Fu2

Pas = ALLOW. SHEAR PER SCREW = Pns/ (F.S. of 3.0)

SECTION E4.4, TENSION

Equation E4.4.1.1, Pnot, Pull-out force = 0.85 tc d Fu2

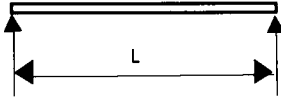
Equation E4.4.2.1, Pnov, Pull-over force = 1.5 t1 dw Fu1

Pnt = nominal tension strength per screw = lesser of Pnot & Pnov

Pat = ALLOW. TENSION PER SCREW = Pnt/ (F.S. of 3.0)

- NOTES: 1. Minimum Spacing of screws shall not be less than 3d
2. Minimum Edge Distance of screws shall not be less than 3d (May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)
3. The head of the screw or the washer shall have a diameter, dw of not less than 5/16 inch, washers shall be at least 0.050 inch thick.
4. Values may be increased 33% for wind or earthquake loads.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET
Job: 6 post canopy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 11
S.E. CONSULTANTS, INC.



Code Requirements = IBC

**Structures Basic Geometry**

Roof slope	=	0.125 /12
Length	=	18.17 feet
Tributary Width	=	6.00 feet
Total Tributary Area	=	109.00 Sq. Ft.
Total Live Load	=	40.00 psf
Dead Load	=	2.00 psf
Live Load (reduced)	=	40.00 psf
Snow Load	=	0.00 psf
Total Gravity Load	=	42.00 psf
Wind Uplift	=	14.01 psf
w (dead) = D.L. x Trib. Lgth.	=	12.00 lbs/ft.
w (live) = L.L. x Trib. Lgth.	=	240.00 lbs/ft.
w (snow) = S.L. x Trib. Lgth.	=	0.00 lbs/ft.
w (wind) = w.L. x Trib. Lgth.	=	84.06 lbs/ft.
w (T.L.) = Uniform Gravity Load	=	252.00 lbs/ft.
w (U.L.) = Uniform Uplift Load	=	72.06 lbs/ft.

**MOMENTS DUE TO GRAVITY LOADS**

$$M (\text{Simple}) = w (\text{T.L.}) \times L^2 / 8 = 10.396 \text{ ft-kips}$$

**MOMENTS DUE TO WIND LOADS**

$$M (\text{Simple}) = w (\text{W.L.}) \times L^2 / 8 = 2.973 \text{ ft-kips}$$

**MAXIMUM END REACTIONS DUE TO GRAVITY LOADS**

$$R(\text{Simple}) = w(\text{T.L.}) \times L / 2 = 2.289 \text{ kips}$$

**MAXIMUM END REACTIONS DUE TO WIND UPLIFT LOADS**

$$R(\text{Simple}) = w(\text{U.L.}) \times L / 2 = 0.655 \text{ kips}$$

**MINIMUM NUMBER OF SCREWS REQUIRED FOR END SUPPORT**

$$\text{Maximum capacity of \#12 screw in shear} = 0.618 \text{ kips}$$

$$\text{Minimum number of screws required} = 4 - \#12 \text{ screws}$$

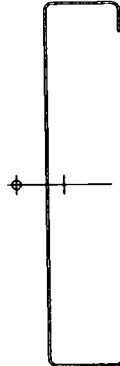
See detail for actual number of screws required.

See CFS computer run for design size

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 6 post canopy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 12**S.E. CONSULTANTS, INC.**

Rev. Date: 1/18/2006 5:24:14 PM  
 By: Steve Schaub, P.E.



**Section Inputs**

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Material: A607 Class 1 Grade 55

No strength increase from cold work of forming.

Modulus of Elasticity, E            29500 ksi  
 Yield Strength, Fy                    55 ksi  
 Tensile Strength, Fu                 70 ksi  
 Warping Constant Override, Cw       0 in<sup>6</sup>  
 Torsion Constant Override, J        0 in<sup>4</sup>

Cee, Thickness 0.075 in

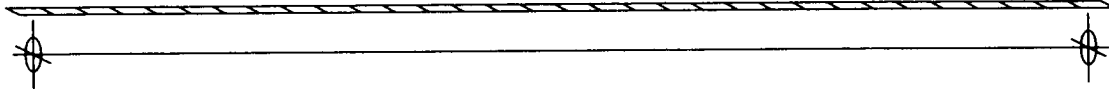
Placement of Part from Origin:

X to center of gravity                0 in  
 Y to center of gravity                0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	1.000	270.000	0.18750	None	0.000	0.000	0.500
2	2.500	180.000	0.18750	Single	0.000	0.000	1.250
3	12.000	90.000	0.18750	Single	0.000	0.000	6.000
4	2.500	0.000	0.18750	Single	0.000	0.000	1.250
5	1.000	-90.000	0.18750	None	0.000	0.000	0.500

Rev. Date: 11/17/2009 11:06:48 AM  
 By: Steve Schaub, P.E.



### Analysis Inputs

#### Members

Section File	Revision Date and Time					
1 12x2.5- 14 Gage Cee- unstiffened.sct	1/18/2006 5:24:14 PM					
Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	ex (in)	ey (in)	
1 0.000	18.167	Top	0.4000	0.0000	0.0000	

#### Supports

Type	Location (ft)	Bearing (in)	Fastened	K
1 XYT	0.333	1.000	No	1.0000
2 XYT	17.833	1.000	No	1.0000

#### Loading: Dead Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Distributed	90.000	0.000	18.167	-0.01200	-0.01200	k/ft

#### Loading: Wind Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Distributed	90.000	0.000	18.167	0.08400	0.08400	k/ft

#### Loading: Live Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Distributed	90.000	0.000	18.167	-0.24000	-0.24000	k/ft

Load Combination: ASD 2: D+L

Specification: 2004 North American Specification - US (ASD)

Inflection Point Bracing: No

Loading	Factor
1 Beam Self Weight	1.0000
2 Dead Load	1.0000
3 Live Load	1.0000

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By: Steve Schaub, P.E.

Load Combination: ASD 2:.6D+W  
Specification: 2004 North American Specification - US (ASD)  
Inflection Point Bracing: Yes

Loading	Factor
1 Beam Self Weight	0.6000
2 Dead Load	0.6000
3 Wind Load	1.0000

**Member Check - 2004 North American Specification - US (ASD)**

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Load Combination: ASD 2: D+L  
Design Parameters at 9.083 ft:

Lx	17.500 ft	Ly	17.500 ft	Lt	17.500 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000

Section: 12x2.5- 14 Gage Cee- unstiffened.sct

Cbx	1.1366	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: Top		Moment Reduction, R: 0.4000			

Loads:	P (k)	Mx (k-ft)	Vy (k)	My (k-ft)	Vx (k)
Total	0.0000	9.811	0.0000	0.000	0.000
Applied	0.0000	9.811	0.0000	0.000	0.0000
Strength	6.8910	10.444	3.2715	0.833	6.1102

Effective section properties at applied loads:

Ae	1.26483 in <sup>2</sup>	Ixe	25.897 in <sup>4</sup>	Iye	1.023 in <sup>4</sup>
		Sxe(t)	4.1500 in <sup>3</sup>	Sye(l)	1.6107 in <sup>3</sup>
		Sxe(b)	4.4962 in <sup>3</sup>	Sye(r)	0.5487 in <sup>3</sup>

Interaction Equations

NAS Eq. C5.2.1-1	(P, Mx, My)	0.000 + 0.939 + 0.000 = 0.939 <= 1.0
NAS Eq. C5.2.1-2	(P, Mx, My)	0.000 + 0.939 + 0.000 = 0.939 <= 1.0
NAS Eq. C3.3.1-1	(Mx, Vy)	Sqrt(0.883 + 0.000) = 0.939 <= 1.0
NAS Eq. C3.3.1-1	(My, Vx)	Sqrt(0.000 + 0.000) = 0.000 <= 1.0

**Member Check - 2004 North American Specification - US (ASD)**

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Load Combination: ASD 2:.6D+W  
Design Parameters at 9.083 ft:

Lx	17.500 ft	Ly	17.487 ft	Lt	17.487 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000

Section: 12x2.5- 14 Gage Cee- unstiffened.sct

Cbx	1.1364	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: Top		Moment Reduction, R: 0.4000			

Loads:	P (k)	Mx (k-ft)	Vy (k)	My (k-ft)	Vx (k)
Total	0.0000	-2.8286	0.0000	0.0000	0.0000
Applied	0.0000	-2.8286	0.0000	0.0000	0.0000
Strength	6.8910	4.1775	3.2715	0.8337	6.1102

Effective section properties at applied loads:

Ae	1.37353 in <sup>2</sup>	Ixe	26.838 in <sup>4</sup>	Iye	1.059 in <sup>4</sup>
		Sxe(t)	4.4731 in <sup>3</sup>	Sye(l)	1.8012 in <sup>3</sup>

(5)



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Sxe (b)      4.4731 in<sup>3</sup>      Sye (r)      0.5538 in<sup>3</sup>

Interaction Equations

NAS Eq. C5.2.1-1 (P, Mx, My)  $0.000 + 0.677 + 0.000 = 0.677 \leq 1.0$   
NAS Eq. C5.2.1-2 (P, Mx, My)  $0.000 + 0.677 + 0.000 = 0.677 \leq 1.0$   
NAS Eq. C3.3.1-1 (Mx, Vy)  $\text{Sqrt}(0.073 + 0.000) = 0.271 \leq 1.0$   
NAS Eq. C3.3.1-1 (My, Vx)  $\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$

**COLUMN LOADS - 2006 IBC**

**GRAVITY LOADS**

Roof Slope	=	0.125 /12
Length of Column	=	10 feet
Width of Building	=	18 feet
Average Bay Spacing	=	9.08333 feet
Number of Lateral Columns	=	3
Tributary Area to Column	=	54.50 sq. ft.
Dead Load	=	2.00 psf
Live Load	=	40.00 psf
Live Load (reduced)	=	40.00 psf
Total Dead Load to Column	=	0.109 kips
Total Live Load to Column	=	2.180 kips
Total Gravity Load to Column	=	2.289 kips

**WIND LOAD**

Wind Uplift	=	14.01 psf
Dead Load	=	2.00 psf
Net Wind Uplift (W.L. - D.L.)	=	12.01 psf
Total Wind Load	=	0.764 kips
Total Wind Uplift to Column	=	0.655 kips

**LATERAL WIND LOAD**

Wind, Lateral	=	14.01 psf
Height of Vertical Surface	=	1.23 feet
Wind Load per Column	=	0.052 kips
Wind Moment to Column	=	0.521 ft-kips

**EARTHQUAKE LATERAL/LONGITUDINAL LOAD TO COLUMN - 2003 IBC**

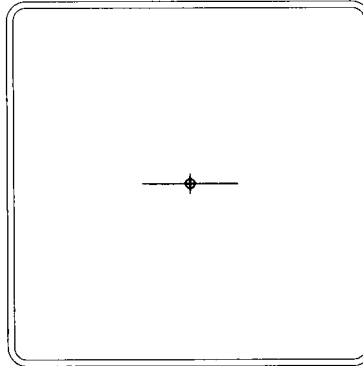
SEISMIC USE GROUP	=	I	(1616-3(1))
SEISMIC DESIGN CAT	=	D	(1616-3(1))
IMPORTANCE FACTOR	=	1.00	
SEISMIC BASE SHEAR V=	=	Cs*W	(1617-5)
R=Response modification factor	=	2.50	(1617-6) Cantilever Column Sys.
W=Effective Seismic Weight	=	0.545 kips	(1617-5-1)
T = 0.02 * hn^(3/4) =	=	0.112	
Ss	=	178.7 %	
S1	=	75.0 %	
Sms = 1.787	Sds =	2/3 * SMS =	1.191
Smi = 1.125	Sdi =	2/3 * SMI =	0.750
Fa	=	1.000	(1615-1-2(1))
Fv	=	1.500	(1615-1-2(2))
Cs = Sds / (R/I)	=	0.477	(Equation 16-35)
Cs = Sdi / [(R/I)T]	=	2.667	(Equation 16-36)
Cs = 0.044 * Sds * I	=	0.033	(Equation 16-37)
Cs = 0.5*S1 / (R/I)	=	N/A	(Equation 16-38)
V (controls)	=	0.260 kips	
Mh= V*h	=	2.597 ft-kips	

Seismic Governs

See computer run for design size

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET			
Job:	6 post canopy	Date: 11-09	By: GCS Job No.: 1306-09 SH.: 17
<b>S.E. CONSULTANTS, INC.</b>			

Rev. Date: 6/12/2006 8:52:01 AM  
By: Steve Schaub, P.E.



### Section Inputs

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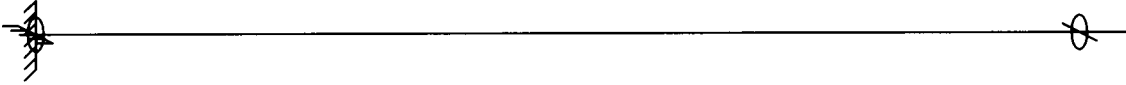
Material: A607 Class 2 Grade 55  
No strength increase from cold work of forming.  
Modulus of Elasticity, E           29500 ksi  
Yield Strength, Fy                   55 ksi  
Tensile Strength, Fu                 65 ksi  
Warping Constant Override, Cw       0 in<sup>6</sup>  
Torsion Constant Override, J         0 in<sup>4</sup>

Tube, Thickness 0.0713 in (14 Gage)  
Placement of Part from Origin:  
X to center of gravity               0 in  
Y to center of gravity               0 in

Outside dimensions, Closed shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	4.0000	0.000	0.13600	Single	0.000	0.0000	2.0000
2	4.0000	90.000	0.13600	Single	0.000	0.0000	2.0000
3	4.0000	180.000	0.13600	Single	0.000	0.0000	2.0000
4	4.0000	-90.000	0.13600	Single	0.000	0.0000	1.5000

Rev. Date: 11/17/2009 11:21:20 AM  
By: Steve Schaub, P.E.



**Analysis Inputs**

Members

Section File	Revision Date and Time					
1 TS 4x4x 14 ga.sct	6/12/2006 8:52:01 AM					
Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	ex (in)	ey (in)	
1 0.000	10.500	None	0.4000	0.0000	0.0000	

Supports

Type	Location (ft)	Bearing (in)	Fastened	K	
1 XYTRxRy	0.000	5.000	No	2.0000	
2 XT	10.000	1.000	No	1.0000	

Loading: Dead Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Axial	NA	0.000	10.000	0.1090	0.1090 k	
2 Concentrated	90.000	9.958	NA	0.0000	NA k	
			Bearing Length		1.000 in	

Loading: Roof Live Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Axial	NA	0.000	10.000	2.1800	2.1800 k	
2 Concentrated	90.000	9.958	NA	0.0000	NA k	
			Bearing Length		1.000 in	

Loading: Wind Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Axial	NA	0.000	10.000	-0.7640	-0.7640 k	
2 Concentrated	90.000	9.958	NA	0.0520	NA k	
			Bearing Length		1.000 in	
3 Concentrated	90.000	9.958	NA	0.0000	NA k	
			Bearing Length		1.000 in	

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Loading: Earthquake Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1 Concentrated	90.000	9.958	NA	0.2600	NA k
			Bearing Length		1.000 in

Loading: UNBAL

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude
1 Axial	NA	0.000	10.000	0.0000	0.0000 k
2 Concentrated	90.000	9.958	NA	0.0000	NA k
			Bearing Length		1.000 in

Load Combination: ASD 2: D+L+R

Specification: 2004 North American Specification - US (ASD)

Inflection Point Bracing: Yes

Loading	Factor
1 Dead Load	1.0000
2 Roof Live Load	1.0000

Load Combination: ASD 3: .6D+W

Specification: 2004 North American Specification - US (ASD)

Inflection Point Bracing: No

Loading	Factor
1 Dead Load	0.6000
2 Wind Load	1.0000

Load Combination: ASD 4: D + 0.7E

Specification: 2004 North American Specification - US (ASD)

Inflection Point Bracing: No

Loading	Factor
1 Beam Self Weight	1.0000
2 Dead Load	1.0000
3 Earthquake Load	0.7000

Load Combination: DL+UNBAL

Specification: 2004 North American Specification - US (ASD)

Inflection Point Bracing: No

Loading	Factor
1 Beam Self Weight	1.0000
2 Dead Load	1.0000
3 UNBAL	1.0000

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By: Steve Schaub, P.E.

**Member Check - 2004 North American Specification - US (ASD)**

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Load Combination: ASD 2: D+L+R

Design Parameters at 0.000 ft, Right side:

Lx	10.500 ft	Ly	10.000 ft	Lt	10.000 ft
Kx	2.0000	Ky	2.0000	Kt	2.0000

Section: TS 4x4x 14 ga.sct

Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		Moment Reduction, R: 0.0000			

Loads:	P (k)	Mx (k-ft)	Vy (k)	My (k-ft)	Vx (k)
Total	2.289	0.0000	0.000	0.0000	0.000
Applied	2.289	0.0000	0.000	0.0000	0.000
Strength	6.254	3.1973	10.545	3.1973	10.545

Effective section properties at applied loads:

Ae	1.09945 in <sup>2</sup>	Ixe	2.7997 in <sup>4</sup>	Iye	2.7997 in <sup>4</sup>
		Sxe(t)	1.3998 in <sup>3</sup>	Sye(l)	1.3998 in <sup>3</sup>
		Sxe(b)	1.3998 in <sup>3</sup>	Sye(r)	1.3998 in <sup>3</sup>

Interaction Equations

NAS Eq. C5.2.1-1	(P, Mx, My)	$0.366 + 0.000 + 0.000 = 0.366 \leq 1.0$
NAS Eq. C5.2.1-2	(P, Mx, My)	$0.094 + 0.000 + 0.000 = 0.094 \leq 1.0$
NAS Eq. C3.3.1-1	(Mx, Vy)	$\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$
NAS Eq. C3.3.1-1	(My, Vx)	$\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$

**Member Check - 2004 North American Specification - US (ASD)**

---

Load Combination: ASD 3: .6D+W

Design Parameters at 0.000 ft, Right side:

Lx	10.500 ft	Ly	10.000 ft	Lt	10.000 ft
Kx	2.0000	Ky	2.0000	Kt	2.0000

Section: TS 4x4x 14 ga.sct

Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		Moment Reduction, R: 0.0000			

Loads:	P (k)	Mx (k-ft)	Vy (k)	My (k-ft)	Vx (k)
Total	-0.699	0.5200	-0.052	0.0000	0.000
Applied	-0.699	0.5200	-0.052	0.0000	0.000
Strength	35.732	3.1973	10.545	3.1973	10.545

Effective section properties at applied loads:

Ae	1.09945 in <sup>2</sup>	Ixe	2.7997 in <sup>4</sup>	Iye	2.7997 in <sup>4</sup>
		Sxe(t)	1.3998 in <sup>3</sup>	Sye(l)	1.3998 in <sup>3</sup>
		Sxe(b)	1.3998 in <sup>3</sup>	Sye(r)	1.3998 in <sup>3</sup>

Interaction Equations

NAS Eq. C5.1.1-1	(Mx, My, T)	$0.135 + 0.000 + 0.020 = 0.155 \leq 1.0$
NAS Eq. C5.1.1-2	(Mx, My, T)	$0.163 + 0.000 - 0.020 = 0.143 \leq 1.0$
NAS Eq. C3.3.1-1	(Mx, Vy)	$\text{Sqrt}(0.026 + 0.000) = 0.163 \leq 1.0$
NAS Eq. C3.3.1-1	(My, Vx)	$\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$

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Rev. Date: 11/17/2009 11:21:20 AM  
By: Steve Schaub, P.E.

**Member Check - 2004 North American Specification - US (ASD)**

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Load Combination: ASD 4: D + 0.7E

Design Parameters at 0.000 ft, Right side:

Lx	10.500 ft	Ly	10.000 ft	Lt	10.000 ft
Kx	2.0000	Ky	2.0000	Kt	2.0000

Section: TS 4x4x 14 ga.sct

Cbx	1.0000	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in

Braced Flange: None      Moment Reduction, R: 0.0000

Loads:	P (k)	Mx (k-ft)	Vy (k)	My (k-ft)	Vx (k)
Total	0.109	1.6139	-0.143	0.0000	0.000
Applied	0.109	1.6139	-0.143	0.0000	0.000
Strength	6.254	3.1973	10.545	3.1973	10.545

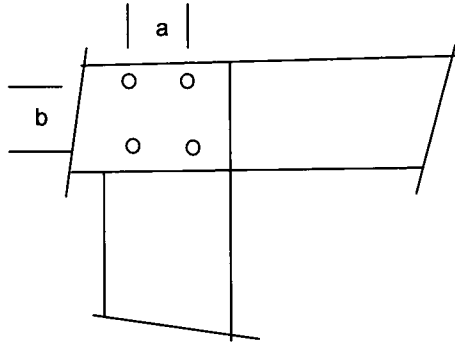
Effective section properties at applied loads:

Ae	1.09945 in <sup>2</sup>	Ixe	2.7997 in <sup>4</sup>	Iye	2.7997 in <sup>4</sup>
		Sxe(t)	1.3998 in <sup>3</sup>	Sye(l)	1.3998 in <sup>3</sup>
		Sxe(b)	1.3998 in <sup>3</sup>	Sye(r)	1.3998 in <sup>3</sup>

Interaction Equations

NAS Eq. C5.2.1-1	(P, Mx, My)	$0.017 + 0.513 + 0.000 = 0.530 \leq 1.0$
NAS Eq. C5.2.1-2	(P, Mx, My)	$0.004 + 0.505 + 0.000 = 0.509 \leq 1.0$
NAS Eq. C3.3.1-1	(Mx, Vy)	$\text{Sqrt}(0.255 + 0.000) = 0.505 \leq 1.0$
NAS Eq. C3.3.1-1	(My, Vx)	$\text{Sqrt}(0.000 + 0.000) = 0.000 \leq 1.0$

# ECENTRIC LOADS ON FASTENER GROUPS



- n = No. of fasteners in vertical row
- m = No. of fasteners in horizontal row
- P = applied load
- rv = allowable shear/bearing for one fastener
- lp = polar moment of inertia =  $I_{xx} + I_{yy}$
- $I_{xx} = [nb^2(n^2-1)/12] \times$  no. of vertical rows
- $I_{yy} = [mb^2(m^2-1)/12] \times$  no. of horizontal rows
- f1 =  $P/(m \times n)$
- f2 = (moment) $\times b/2lp$
- f3 = (moment) $\times (n-1)b/2lp$
- fr = actual load on bolts  $[(f3)^2 + (f1 + f2)^2]^{.5}$

Beam to Column Connection is Designed for Total Load and Moment

Total Gravity Load	= 2.289 kips	Total Seismic Load	= 0.109 kips
Total Gravity Moment	= 0.000 k-ft	Total Seismic Moment	= 1.818 k-ft
Total Uplift Load	= 0.764 kips		
Total Uplift Moment	= 0.521 k-ft		

Connection Design (half load to each side)

t, Thickness of material	= 0.0747 inches
Fu, of joined materials	= 70 ksi
Screw Size	= No. 12
No. of vertical rows	= 2
No. of fasteners in vertical row	= 4
No. of horizontal rows	= 4
No. of fasteners in horizontal row:	= 2
Vertical Dimension	= 1.833 inches
Horizontal Dimension	= 4 inches
$I_{xx}$	= 33.599
$I_{yy}$	= 32
$I_p$	= 65.599

**Use:**  
 Type of Screws: No. 12  
 Horizontal Spacing: 4 inches  
 Vertical Spacing: 1.833 inches  
 Number of Screws: 8

	<u>Gravity</u>	<u>Seismic</u>	<u>Wind</u>
f1 =	0.143 kips	0.007 kips	0.048 kips
f2 =	0.000 kips	0.333 kips	0.095 kips
f3 =	0.000 kips	0.457 kips	0.131 kips
fr, (total) =	0.143 kips	0.569 kips	0.194 kips

Allowable Shear	0.625 kips	Gravity: o.k.
rv, Allowable w/ 33% increase	0.625 kips	Wind: o.k.
Increase Aloud (y, n)	N	Seismic: o.k.

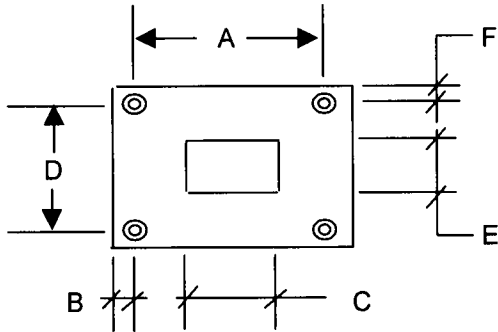
**PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET**

Job: 6 foot canopy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 23

**S.E. CONSULTANTS, INC.**



# Base Plate Design



- A= 9.000 inches
- B= 1.500 inches
- C= 4.000 inches
- D= 9.000 inches
- E= 4.000 inches
- F= 1.500 inches

Base Plate to Footing Connection is Designed for Total Load and Moment

Total Gravity Load	=	2.29 kips
Total Gravity Moment	=	0.00 k-ft
Total Uplift Load	=	-0.76 kips
Total Uplift Moment	=	0.52 k-ft
Total Seismic Load	=	0.11 kips
Total Seismic Moment	=	2.60 k-ft

Tension in Bolts

Due to Gravity	=	-0.57 kips
Due to Uplift	=	0.54 kips
Due to Seismic	=	1.70 kips

**Seismic Controls**

Checking Plate

Bending Moment	=	8.52 k-in
t, Thickness of material	=	0.50 in
Fy, of base plate	=	50.0 ksi
Sx, Section Modulus	=	0.50 in <sup>3</sup>
Fb	=	37.5 ksi
fb	=	17.0 ksi

o.k.

<b>PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET</b>			
Job:	<i>6 pool/structure</i>	Date:	<i>11-09</i> By: <i>GCS</i> Job No.: <i>1306-09</i> SH.: <i>24</i>
<b>S.E. CONSULTANTS, INC.</b>			

## Anchor Calculations

### Anchor Designer for ACI 318 (Version 4.2.0.1)

Job Name : Anchor Design

Date/Time : 11/16/2009 11:29:58 AM

#### 1) Input

Calculation Method : ACI 318 Appendix D For Cracked Concrete

Calculation Type : Analysis

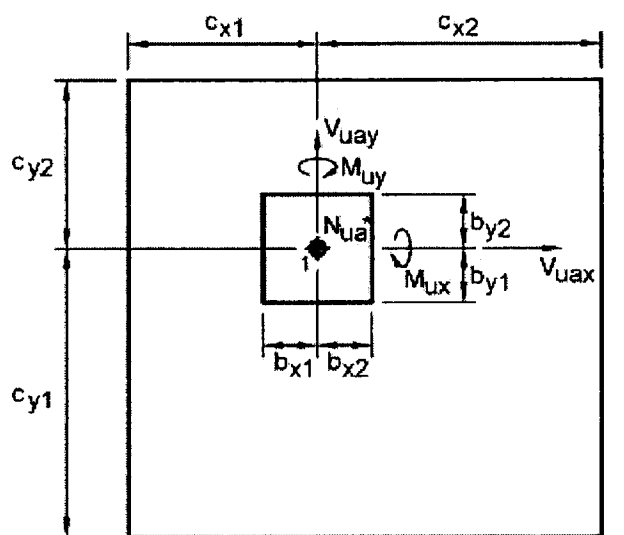
#### a) Layout

Anchor : 3/4" Titen HD

Number of Anchors : 1

Embedment Depth : 5.5 in

Built-up Grout Pads : No



1 ANCHOR

\* $N_{ua}$  IS POSITIVE FOR TENSION AND NEGATIVE FOR COMPRESSION.

+ INDICATES CENTER OF THE ANCHOR

Anchor Layout Dimensions :

$c_{x1}$  : 6 in

$c_{x2}$  : 6 in

$c_{y1}$  : 6 in

$c_{y2}$  : 6 in

$b_{x1}$  : 1.5 in

$b_{x2}$  : 1.5 in

$b_{y1}$  : 1.5 in

$b_{y2}$  : 1.5 in

#### b) Base Material

28

Concrete : Normal weight

 $f'_c$  : 2500.0 psi

Cracked Concrete : Yes

 $\Psi_{c,v}$  : 1.40

Condition : B tension and shear

 $\phi F_p$  : 1381.3 psi

Thickness, h : 24 in

Supplementary edge reinforcement : No

**c) Factored Loads**

Load factor source : ACI 318 Appendix C

 $N_{ua}$  : 4552 lb $V_{uax}$  : 0 lb $V_{uay}$  : 0 lb $M_{ux}$  : 0 lb\*ft $M_{uy}$  : 0 lb\*ft $e_x$  : 0 in $e_y$  : 0 in

Moderate/high seismic risk or intermediate/high design category : No

Apply entire shear load at front row for breakout : No

**d) Anchor Parameters**

From C-SAS-2009:

Anchor Model = THD75  $d_o$  = 0.75 inCategory = 1  $h_{ef}$  = 4.22 in $h_{min}$  = 8.75 in  $c_{ac}$  = 6.375 in $c_{min}$  = 1.75 in  $s_{min}$  = 3 in

Ductile = No

**2) Tension Force on Each Individual Anchor**Anchor #1  $N_{ua1}$  = 4552.00 lbSum of Anchor Tension  $\Sigma N_{ua}$  = 4552.00 lb $a_x$  = 0.00 in $a_y$  = 0.00 in $e'_{Nx}$  = 0.00 in $e'_{Ny}$  = 0.00 in**3) Shear Force on Each Individual Anchor**

Resultant shear forces in each anchor:

Anchor #1  $V_{ua1}$  = 0.00 lb ( $V_{ua1x}$  = 0.00 lb ,  $V_{ua1y}$  = 0.00 lb )Sum of Anchor Shear  $\Sigma V_{uax}$  = 0.00 lb,  $\Sigma V_{uay}$  = 0.00 lb $e'_{Vx}$  = 0.00 in

$$e'_{vy} = 0.00 \text{ in}$$

#### 4) Steel Strength of Anchor in Tension [Sec. D.5.1]

$$N_{sa} = nA_{se}f_{uta} \text{ [Eq. D-3]}$$

Number of anchors acting in tension,  $n = 1$

$$N_{sa} = 45540 \text{ lb (for a single anchor) [C-SAS-2009]}$$

$$\phi = 0.70 \text{ [D.4.5]}$$

$$\phi N_{sa} = 31878.00 \text{ lb (for a single anchor)}$$

#### 5) Concrete Breakout Strength of Anchor in Tension [Sec. D.5.2]

$$N_{cb} = A_{Nc}/A_{Nco}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b \text{ [Eq. D-4]}$$

Number of influencing edges = 4

$$h_{ef} \text{ (adjusted for edges per D.5.2.3)} = 4.000 \text{ in}$$

$$A_{Nco} = 144.00 \text{ in}^2 \text{ [Eq. D-6]}$$

$$A_{Nc} = 144.00 \text{ in}^2$$

Smallest edge distance,  $c_{a,min} = 6.00 \text{ in}$

$$\Psi_{ed,N} = 1.0000 \text{ [Eq. D-10 or D-11]}$$

Note: Cracking shall be controlled per D.5.2.6

$$\Psi_{c,N} = 1.0000 \text{ [Sec. D.5.2.6]}$$

$$\Psi_{cp,N} = 1.0000 \text{ [Eq. D-12 or D-13]}$$

$$N_b = k_c \sqrt{f'_c} h_{ef}^{1.5} = 6800.00 \text{ lb [Eq. D-7]}$$

$$k_c = 17 \text{ [Sec. D.5.2.6]}$$

$$N_{cb} = 6800.00 \text{ lb [Eq. D-4]}$$

$$\phi = 0.75 \text{ [D.4.5]}$$

$$\phi N_{cb} = 5100.00 \text{ lb (for a single anchor)}$$

#### 6) Pullout Strength of Anchor in Tension [Sec. D.5.3]

$$N_{pn} = \Psi_{c,p}N_p$$

$$N_{pn} = 6070 \text{ lb } (f'_c/2,500 \text{ psi})^{0.5} = 6070.00 \text{ lb}$$

$$\phi = 0.75$$

$$\phi N_{pn} = 4552.50 \text{ lb}$$

#### 7) Side Face Blowout of Anchor in Tension [Sec. D.5.4]

Concrete side face blowout strength is only calculated for headed anchors in tension close to an edge,  $c_{a1} < 0.4h_{ef}$ . Not applicable in this case.

#### 8) Steel Strength of Anchor in Shear [Sec D.6.1]

$$V_{sa} = 16840.00 \text{ lb (for a single anchor) [C-SAS-2009]}$$

$$\phi = 0.65 \text{ [D.4.5]}$$

$$\phi V_{sa} = 10946.00 \text{ lb (for a single anchor)}$$

### 9) Concrete Breakout Strength of Anchor in Shear [Sec D.6.2]

Case 1: Anchor checked against total shear load

In x-direction...

$$V_{cbx} = A_{vcx}/A_{vcox} \Psi_{ed,V} \Psi_{c,V} V_{bx} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcx} = 108.00 \text{ in}^2$$

$$A_{vcox} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 0.9000 \text{ [Eq. D-27 or D-28]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})}^{1.5} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{bx} = 6293.26 \text{ lb}$$

$$V_{cbx} = 5286.34 \text{ lb [Eq. D-21]}$$

$$\phi = 0.75$$

$$\phi V_{cbx} = 3964.75 \text{ lb (for a single anchor)}$$

In y-direction...

$$V_{cby} = A_{vcy}/A_{vcoy} \Psi_{ed,V} \Psi_{c,V} V_{by} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcy} = 108.00 \text{ in}^2$$

$$A_{vcoy} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 0.9000 \text{ [Eq. D-27 or D-28]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})}^{1.5} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{by} = 6293.26 \text{ lb}$$

$$V_{cby} = 5286.34 \text{ lb [Eq. D-21]}$$

$$\phi = 0.75$$

$$\phi V_{cby} = 3964.75 \text{ lb (for a single anchor)}$$

Case 2: This case does not apply to single anchor layout

Case 3: Anchor checked for parallel to edge condition

Check anchors at  $c_{x1}$  edge

$$V_{cbx} = A_{vcx}/A_{vcox} \Psi_{ed,V} \Psi_{c,V} V_{bx} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcx} = 108.00 \text{ in}^2$$

$$A_{vcox} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 1.0000 \text{ [Sec. D.6.2.1(c)]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{bx} = 6293.26 \text{ lb}$$

$$V_{cbx} = 5873.71 \text{ lb [Eq. D-21]}$$

$$V_{cby} = 2 * V_{cbx} \text{ [Sec. D.6.2.1(c)]}$$

$$V_{cby} = 11747.42 \text{ lb}$$

$$\phi = 0.75$$

$$\phi V_{cby} = 8810.57 \text{ lb (for a single anchor)}$$

Check anchors at  $c_{y1}$  edge

$$V_{cby} = A_{vcy}/A_{vcoy} \Psi_{ed,V} \Psi_{c,V} V_{by} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcy} = 108.00 \text{ in}^2$$

$$A_{vcoy} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 1.0000 \text{ [Sec. D.6.2.1(c)]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{by} = 6293.26 \text{ lb}$$

$$V_{cby} = 5873.71 \text{ lb [Eq. D-21]}$$

$$V_{cbx} = 2 * V_{cby} \text{ [Sec. D.6.2.1(c)]}$$

$$V_{cbx} = 11747.42 \text{ lb}$$

$$\phi = 0.75$$

$$\phi V_{cbx} = 8810.57 \text{ lb (for a single anchor)}$$

Check anchors at  $c_{x2}$  edge

$$V_{cbx} = A_{vcx}/A_{vcox} \Psi_{ed,V} \Psi_{c,V} V_{bx} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcx} = 108.00 \text{ in}^2$$

$$A_{vcox} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 1.0000 \text{ [Eq. D-27 or D-28] [Sec. D.6.2.1(c)]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{bx} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{bx} = 6293.26 \text{ lb}$$

$$V_{cbx} = 5873.71 \text{ lb [Eq. D-21]}$$

$$V_{cby} = 2 * V_{cbx} \text{ [Sec. D.6.2.1(c)]}$$

$$V_{cby} = 11747.42 \text{ lb}$$

$$\phi = 0.75$$

$$\phi V_{cby} = 8810.57 \text{ lb (for a single anchor)}$$

Check anchors at  $c_{y2}$  edge

$$V_{cby} = A_{vcy}/A_{vcoy} \Psi_{ed,V} \Psi_{c,V} V_{by} \text{ [Eq. D-21]}$$

$$c_{a1} = 6.00 \text{ in}$$

$$A_{vcy} = 108.00 \text{ in}^2$$

$$A_{vcoy} = 162.00 \text{ in}^2 \text{ [Eq. D-23]}$$

$$\Psi_{ed,V} = 1.0000 \text{ [Sec. D.6.2.1(c)]}$$

$$\Psi_{c,V} = 1.4000 \text{ [Sec. D.6.2.7]}$$

$$V_{by} = 7(l_e/d_o)^{0.2} \sqrt{d_o} \sqrt{f_c(c_{a1})^{1.5}} \text{ [Eq. D-24]}$$

$$l_e = 4.22 \text{ in}$$

$$V_{by} = 6293.26 \text{ lb}$$

$$V_{cby} = 5873.71 \text{ lb [Eq. D-21]}$$

$$V_{cbx} = 2 * V_{cby} \text{ [Sec. D.6.2.1(c)]}$$

$$V_{cbx} = 11747.42 \text{ lb}$$

$$\phi = 0.75$$

$$\phi V_{cbx} = 8810.57 \text{ lb (for a single anchor)}$$

### 10) Concrete Pryout Strength of Anchor in Shear [Sec. D.6.3]

$$V_{cp} = k_{cp} N_{cb} \text{ [Eq. D-29]}$$

$$k_{cp} = 2 \text{ [Sec. D.6.3.1]}$$

$$N_{cb} = 6800.00 \text{ lb (from Section (5) of calculations)}$$

$$V_{cp} = 13600.00 \text{ lb}$$

$$\phi = 0.75 \text{ [D.4.5]}$$

$$\phi V_{cp} = 10200.00 \text{ lb (for a single anchor)}$$

### **11) Check Demand/Capacity Ratios [Sec. D.7]**

#### Tension

- Steel : 0.1428
- Breakout : 0.8925
- Pullout : 0.9999
- Sideface Blowout : N/A

#### Shear

- Steel : 0.0000
- Breakout (case 1) : 0.0000
- Breakout (case 2) : N/A
- Breakout (case 3) : 0.0000
- Pryout : 0.0000

$$V_{\text{Max}}(0) \leq 0.2 \text{ and } T_{\text{Max}}(1) \leq 1.0 \text{ [Sec D.7.1]}$$

Interaction check: PASS

**Use 3/4" diameter Titen HD anchor(s) with 5.5 in. embedment**



**NON CONSTRAINED POLE TYPE FOOTING PER INTERNATIONAL BUILDING CODE**

DESCRIPTION OR LOCATION OF FOOTING: COLUMNS W/ D.L. & WIND  
 ALLOWABLE SOIL BEARING = 1500 PSF  
 ALLOWABLE LATERAL BEARING = 150 PSF/FT

**CHECK LATERAL BEARING CAPACITY**

$d = \frac{A}{2} \left( 1 + \left[ 1 + \frac{4.36 h}{A} \right]^{1/2} \right)$

$A = \frac{2.34 P}{S1 \times b}$

Load Due To Wind or Seismic? (Y/N) y

M, Moment in Column = 1.818 ft. kips  
 P, Lateral load = M / h = 182 lbs.  
 h Dist. in feet from ground to "P" = 10 feet  
 d Depth of Concrete Footing = 3.00 feet  
 b Diameter of round footing = 2.00 feet  
 Allowable Lateral Bearing = 150 lbs/sq.ft.  
 IBC, Section 1804.3.1

Will structure be adversely affected by a 1/2" motion at the ground surface? (Y/N) N

2 x Allowable Lateral Bearing, per sec. 1804.3.1 = 300.0 lbs/sq.ft.  
 S1 Allowable Lat. brg. based on 1/3 of "d" (limited to "d" = 12 feet) = 300.0 lbs/sq.ft.  
 Allowable Lat. Brg. w/ 33% Increase per footnote D = 400.0 lbs/sq.ft.

A = 0.532

REQUIRED "d" = 2.69 FEET < 3 FEET O.K.

**CHECK VERTICAL BEARING CAPACITY**

P(t.l.), Gravity Load = 2.289 kips  
 Allowable Bearing at Min. Depth = 1500 psf  
 Allowable Bearing w/ Increase for Depth = 1500 psf

Allowable Bearing Value at Bottom of Pier = 1500 psf  
 A, Bearing Area at Bottom of Pier = 3.142 sq. ft.  
 qs, Gravity Load Soil Pressure = 728.611 psf.

O.K. - WITHOUT SKIN FRICTION

Allow. Skin Friction = Allow. Bearing / 6 = 250.0 psf  
 Skin Surface (Excluding top one foot) = 12.57 sq. ft.  
 Skin Friction = 3.14 kips  
 NOT USED

**CHECK UPLIFT CAPACITY, IF APPLICABLE**

F, Total Wind Uplift Load = 0.765 kips  
 Allowable Bearing Value at 1' depth of Pier = 1500 psf  
 Allow. Skin Friction = Allow. Bearing / 6 = 250.0 psf  
 Skin Surface (Excluding top one foot) = 12.6 sq. ft.  
 Skin Friction = 3.1 kips  
 Remove Earth Loads from Uplift Capacity = N  
 Dead Load of Pier = 1.4 kips  
 Dead Load of Earth = 3.4 kips

Total Uplift Resistance = 4.5 kips O.K.  
 Safety Factor = 5.89

Diameter of Pier = 24.00 inches  
 Depth of Concrete Footing = 3.00 feet

**CHECK STRESSES OF CONCRETE PIER**

f<sub>c</sub>, Compressive Strength of Concrete = 2500 psi  
 Moment on Pier Due to Wind or Seismic? (Y/N) Y  
 S, Section Modulus of the Round Concrete Pier = 1357 in.<sup>3</sup>  
 f, Stress in Plain Concrete Pier = M/S = 16 psi.  
 ft, Allowable Tension = 1.6 x SQRT. f<sub>c</sub> = 80 psi.  
 ft, Increased for Wind or Seismic = 106.67 psi.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: *6 post canopy* Date: *11-09* By: *GCS* Job No.: *1306-09* SH.: *32*

**S.E. CONSULTANTS, INC.**

**SPREAD FOOTING W/ COLUMN CENTERED IN FOOTING**

**DEAD LOAD MOMENT RESISTANCE AGAINST OVERTURNING SOIL PRESSURE**

Applied Overturning Moment	= 1.8179 ft-kips	Allowable Soil Pressure	= 1500 psf
D.L. of Structure	= 0.109 kips	Increases for Width OR Depth	= 0
Length of Footing	= 2.5 feet	Allowable S. P. w/ Increases	= 1500
Width of Footing	= 2.5 feet	Loads Due to Wind or Seismic? (Y/N)	= N
Thickness of Footing	= 30 inches	S.P. w/ 33% Increase	= N/A psf
Dead Load of Footing	= 2.3438 kips	Applied Overturning Moment	= 1.8179 ft-kips
Volume of Earth in 30 degree Cone	= 18.1 cu. ft.	D.L. of Structure	= 0.109 kips
Slab on Grade (y or n)	= N	Footing D.L.	= 2.34375 kips
Length of Turndown	= 0 ft	Total Vert. Load	= 2.45275 kips
Depth of Turndown	= 0 ft	$e' = (M_r - OTM) / \text{Total Vert. Load}$	= 1.429 feet
Dead Load of Turndown	= 0.00 kips	$e = l/2 - e'$	= -0.179 feet
Dead Load of Earth	= 1.806 kips	$l/6 =$	= 0.417 feet
Total D.L.	= 4.259 kips	<b>RESULTANT IN MIDDLE 1/3</b>	
D.L. Resisting Moment	= 5.32 ft-kips	S.P. = $P/A \pm M/S$	
Required Factor of Safety	= 1.5	Soil Pressure	= 1090.5 psf
F. of S. Against Overturning	= 2.929		O.K.

**DEAD LOAD RESISTANCE AGAINST UPLIFT**

Applied Uplift Load - Total Wind	= 0.76 kips
D.L. of Structure	= 0.109 kips
Volume of Earth in 30 degree Cone	= 18.064 cu. ft.
Dead Load of Earth	= 1.806 kips
Dead Load of Footing	= 2.34 kips
Dead Load of Earth	= 1.806 kips
Total Dead Load Resistance	= 4.259 kips
F. of S. Against Uplift	= 5.575
	O.K.

**FOOTING REINFORCING**

$f_c$	= 2500 psi
$f_y$	= 40000 psi
$d$	= 27 inches
M, Moment	= 1.8179 ft.-kips
$M_u$ , Ultimate Moment = 1.7 M	= 3.090 ft.-kips
Assume a one foot strip	
$M_u = 1.7 M / \text{Width of Footing}$	= 1.236 ft.-kips
Beta	= 0.85
Phi	= 0.85
rho, balanced	= 0.031
$m$	= 18.824
2.494 psi = $R_n$	= 0.002 ksi
rho, calc.	= 0.0001
rho, min.	= 0.0001
rho, Req'd.	= 0.0001
Req'd. As	= 0.020 sq. in. / ft.
Total As Req'd.	= 0.051 sq. in.
Total As Req'd. (Wind or Seismic)	= 0.038 sq. in.
USE: 3 - # 5 As	= 0.920 sq.in.

**LENGTH OF FOOTING:** 2.5 feet  
**WIDTH OF FOOTING:** 2.5 feet  
**THICKNESS OF FOOTING:** 30 inches  
**REINFORCING:** 3 - #5 Bars  
 Use #4's @ 12" o.c. lateral

**PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET**

Job: 6 post Campy Date: 11-09 By: GCS Job No.: 1306-09 SH.: 33

**S.E. CONSULTANTS, INC.**