CALCULATIONS FOR

Job: 18'-0" x 18'-2" x 10'-0" Max Height - 4 Post Canopy - State of Utah

Address : 2006 International Building Code

(20 psf Red., 90 mph, Seismic Design Category D)

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

Client: Western States Decking, Inc

Index to Calculations

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

 Project Engineer G Starks

Job # 1304-09

S.E. CONSULTANTS, INC.

BASIS FOR DESIGN

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

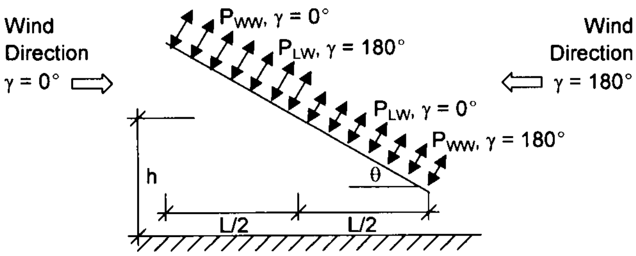
STRESSES OF MATERIALS

CONCRETE			
Footings		f'c =	2500 psi.
STEEL			
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
SOIL			
	Allow. Soil Bearing 1500 psf		Passive Pressure 150 psf / ft
	Soil Class = D	Material Class =	5 [Table 1802.4]
Structural	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

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post canopy Date: 11/09 By: GCS Job No.: 1304-09 SH.: 1

S.E. CONSULTANTS, INC.



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 = qzGCN	=	**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 °)			*Load Case A (γ = 180 °)*		
P, windward roof	=	**15.28** psf	P, windward roof	=	**15.28** psf
P, leeward roof	=	**3.82** psf	P, leeward roof	=	**3.82** psf
Load Case B (γ = 0 °)			*Load Case B (γ = 180 °)*		
P, windward roof	=	**-14.01** psf	P, windward roof	=	**-14.01** psf
P, leeward roof	=	**-1.27** psf	P, leeward roof	=	**-1.27** psf

International Building Code - 2006

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

Ss = **178.7 %**
S1 = **75.0 %**

Site Class	Table 1615.1.2 (1) (Fa) Mapped spectral response acceleration at short periods (Ss)				
	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	-	-	-	-	-

Fa = 1.000 (interpolated) S_{MS} = Fa * Ss = 1.787

Site Class	Table 1615.1.2 (2) (Fv) Mapped spectral response acceleration at 1 sec. periods (S1)				
	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	-	-	-	-	-

Fv = 1.5 (interpolated) S_{M1} = Fv * S_1 = 1.125

S_{DS} = 2/3 * S_{MS} = 1.191 **S_{D1} = 2/3 * S_{M1} = 0.750**

Sds	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

Sdi	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

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 Post Canopy Date: 11-09 By: GCS Job No.: 1304-09 SH.: 3

S.E. CONSULTANTS, INC.

Earthquake Lateral/Longitudinal Load to Column - 2006 IBC

Seismic Use Group	=	I
Seismic Design Category	=	D
I	=	1.00
Ss	=	178.68 %
S_1	=	75.03 %
S_{MS}	=	1.787
S_{M1}	=	1.125
R	=	2.50
Height	=	10 ft
T = 0.02 * (hn)^0.75	=	0.112
r Reliability/redundancy Factor	=	1.00
S_{DS}	=	1.191
S_{DI}	=	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

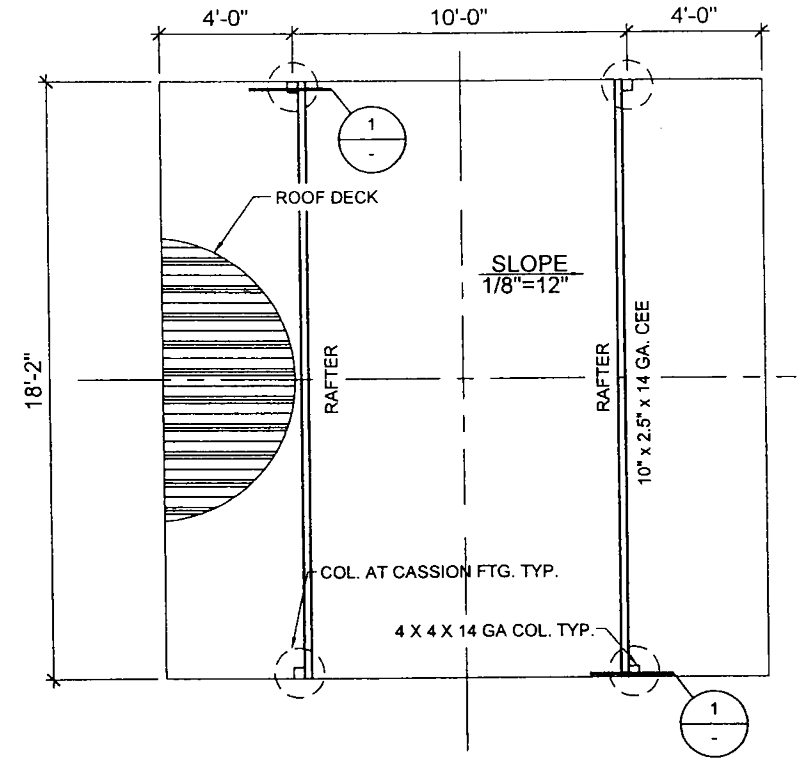
(16-38)
$V = 0.5 * S_1 / [R/I]$ = 0.150 W

V (Controls) = 0.476 W kips

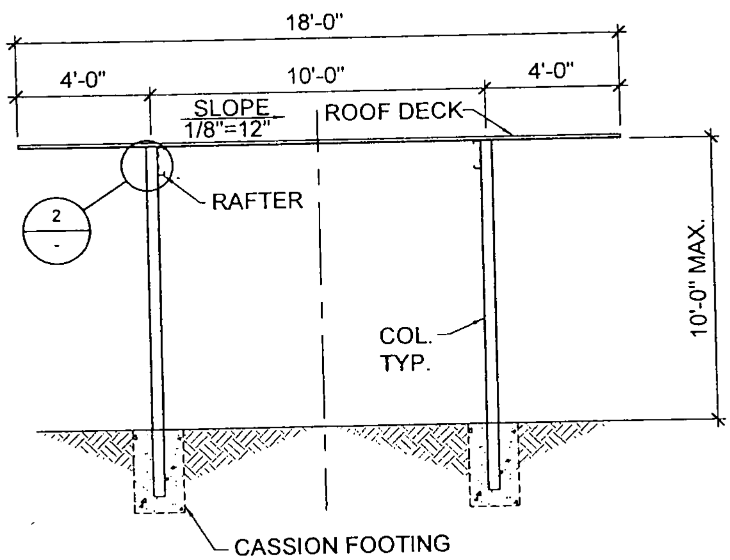
PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4post Canopy Date: 11-08 By: GCS Job No.: 1304-08 SH.: 4

S.E. CONSULTANTS, INC.



ROOF FRAMING & FOUNDATION PLAN



FRONT ELEVATION

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post Canopy Date: 11-09 By: GCS Job No.: 1304.09 SH.: 5

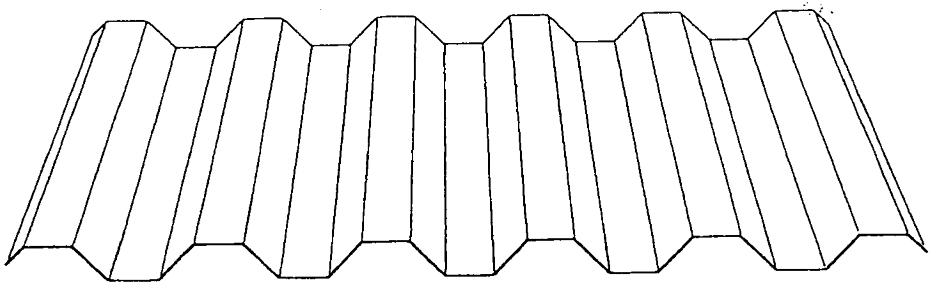
S.E. CONSULTANTS, INC.

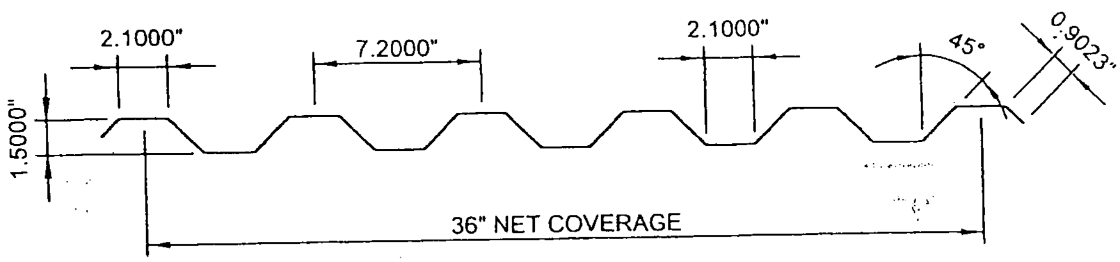
26 ga Western Span

F_y, yield strength	=	90.0 ksi
F_u, tensile strength	=	60.0 ksi
I_x, moment of inertia	=	0.07 in^4/ft
$S_x(t)$, section modulus	=	0.07 in^3/ft
$S_x(b)$, section modulus	=	0.06 in^3/ft

See Manufacture's Specifications.

WESTERN SPAN





Section Properties			Top in Compression			Bott. in Compression		
gauge	fy	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:

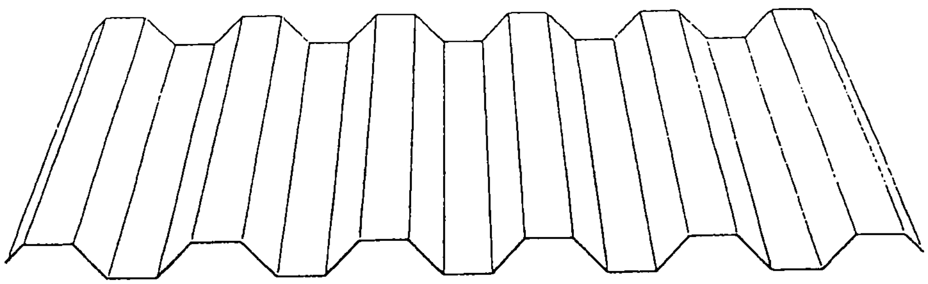
1. Effective section properties are calculated in accordance with the 2004 North American Specifications for the design of Cold-Formed Steel Structural Members

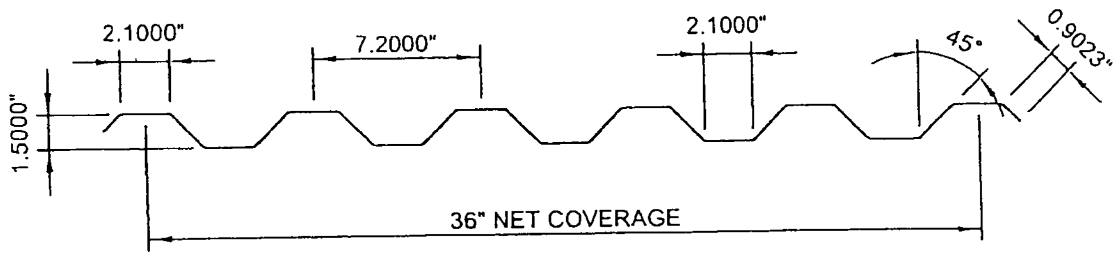
2. Ix is for the determination of deflection

3. Sx and Ma 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.
(fastner 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

NOTE: Shear Governed By Base Metals.

Allowable Tension per Screw = 251.7 lbs/screw

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.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post Canopy Date: 11-09 By: GCS Job No.: 130409 SH.: 9

S.E. CONSULTANTS, INC.

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

WHEN t2/t1 <= 1.0			WHEN t2/t1 >= 2.5		
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/screv
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.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post Canopy   Date: 11-09   By: GCS   Job No.: 1304.09   SH.: 10

S.E. CONSULTANTS, INC.

SCREW CAPACITIES IN LIGHT GAUGE COLD FORMED STEEL

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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

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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

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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)

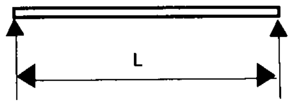
NOTES:
1. Minimum Spacing of screws shall not be less than 3d
2. Minimum Edge Distance of screws shall not be less than 3d
(May be 1.5d in direction perpendicular to force, when connection is subject to shear in one direction.)
3. The head of the screw or the washer shall have a diameter, dw of not less 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: 4 post canopy Date: 11-09 By: GCS Job No.: 1304-09 SH.: 11

S.E. CONSULTANTS, INC.

PURLIN



Code Requirements = IBC

Structures Basic Geometry

Roof slope	=	0.125 /12
Length	=	18.17 feet
Tributary Width	=	9.00 feet
Total Tributary Area	=	163.50 Sq. Ft.
Total Live Load	=	20.00 psf
Dead Load	=	2.00 psf
Live Load (reduced)	=	20.00 psf
Snow Load	=	0.00 psf
Total Gravity Load	=	22.00 psf
Wind Uplift	=	14.01 psf
w (dead) = D.L. x Trib. Lgth.	=	18.00 lbs/ft.
w (live) = L.L. x Trib. Lgth.	=	180.00 lbs/ft.
w (snow) = S.L. x Trib. Lgth.	=	0.00 lbs/ft.
w (wind) = w.L. x Trib. Lgth.	=	126.09 lbs/ft.
w (T.L.) = Uniform Gravity Load	=	198.00 lbs/ft.
w (U.L.) = Uniform Uplift Load	=	108.09 lbs/ft.

MOMENTS DUE TO GRAVITY LOADS

M (Simple) = w (T.L.) x L^2 / 8 = 8.168 ft-kips

MOMENTS DUE TO WIND LOADS

M (Simple) = w (W.L.) x L^2 / 8 = 4.459 ft-kips

MAXIMUM END REACTIONS DUE TO GRAVITY LOADS

R(Simple) = w(T.L.) x L / 2 = 1.799 kips

MAXIMUM END REACTIONS DUE TO WIND UPLIFT LOADS

R(Simple) = w(U.L.) x L / 2 = 0.982 kips

MINIMUM NUMBER OF SCREWS REQUIRED FOR END SUPPORT

Maximum capacity of #12 screw in shear	=	0.618 kips
Minimum number of screws required	=	3 - #12 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

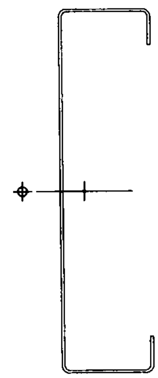
Job: 4 post Date: 11-09 By: GCS Job No.: 304-09 SH.: 12

S.E. CONSULTANTS, INC.

CFS Version 5.0.3
Section: 10x2.5- 14 Gage Cee- unstiffened.sct
BHP 10x2.5 - 14 Gage Cee

Steve Schaub, P.E.
S.E. Consultants, Inc.

Rev. Date: 10/15/2008 8:48:27 AM
By: Steve Schaub, P.E.



Section Inputs

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^6
Torsion Constant Override, J	0 in^4

Cee, Thickness 0.075 in
Placement of Part from Origin:

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

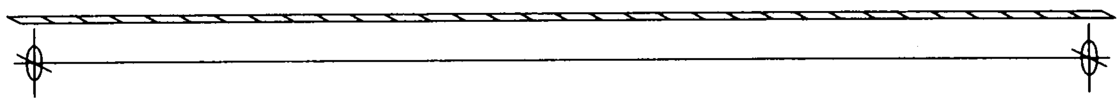
Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	1.0000	270.000	0.13600	None	0.000	0.0000	0.5000
2	2.5000	180.000	0.13600	Single	0.000	0.0000	1.2500
3	10.0000	90.000	0.13600	Single	0.000	0.0000	5.0000
4	2.5000	0.000	0.13600	Single	0.000	0.0000	1.2500
5	1.0000	-90.000	0.13600	None	0.000	0.0000	0.5000

Analysis: Analysis purlin - rafter IBC.anl
18'-2" FT PURLIN

Steve Schaub, P.E.
S.E. Consultants, Inc.

Rev. Date: 11/17/2009 11:05:56 AM
By: Steve Schaub, P.E.



Analysis Inputs

Members

	Section File	Revision Date and Time
1	10x2.5- 14 Gage Cee- unstiffened.sct	10/15/2008 8:48:27 AM

	Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	ex (in)	ey (in)
1	0.000	18.167	Top	0.4700	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	-0.01800	-0.01800	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	0.12600	0.12600	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	-0.18000	-0.18000	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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Rev. Date: 11/17/2009 11:05:56 AM
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)

```
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: 10x2.5- 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: 0.4700

Loads:             P          Mx          Vy          My          Vx
                  (k)       (k-ft)       (k)        (k-ft)       (k)
Total          0.0000      7.7286      0.0000      0.0000      0.0000
Applied        0.0000      7.7286      0.0000      0.0000      0.0000
Strength       8.2919      9.3885      3.9195      0.7618      6.4288

Effective section properties at applied loads:
Ae        1.23016 in^2    Ixe         17.424 in^4    Iye          1.021 in^4
                          Sxe(t)      3.4848 in^3    Sye(l)      1.5502 in^3
                          Sxe(b)      3.4848 in^3    Sye(r)      0.5545 in^3

Interaction Equations
NAS Eq. C5.2.1-1  (P, Mx, My)  0.000 + 0.823 + 0.000 = 0.823 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.000 + 0.823 + 0.000 = 0.823 <= 1.0
NAS Eq. C3.3.1-1      (Mx, Vy)     Sqrt(0.678 + 0.000)= 0.823 <= 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)

```
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: 10x2.5- 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: 0.4700

Loads:             P          Mx          Vy          My          Vx
                  (k)       (k-ft)       (k)        (k-ft)       (k)
Total          0.0000     -4.3077      0.0000      0.0000      0.0000
Applied        0.0000     -4.3077      0.0000      0.0000      0.0000
Strength       8.2919      4.4126      3.9195      0.7627      6.4288

Effective section properties at applied loads:
Ae        1.23016 in^2    Ixe         17.424 in^4    Iye          1.021 in^4
                          Sxe(t)      3.4848 in^3    Sye(l)      1.5502 in^3
```

```
                         Sxe(b)     3.4848 in^3   Sye(r)     0.5545 in^3

Interaction Equations
NAS Eq. C5.2.1-1  (P, Mx, My)  0.000 + 0.976 + 0.000 = 0.976 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.000 + 0.976 + 0.000 = 0.976 <= 1.0
NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.211 + 0.000)= 0.459 <= 1.0
NAS Eq. C3.3.1-1     (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 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	=	2
Tributary Area to Column	=	81.75 sq. ft.
Dead Load	=	2.00 psf
Live Load	=	20.00 psf
Live Load (reduced)	=	20.00 psf
Total Dead Load to Column	=	0.164 kips
Total Live Load to Column	=	1.635 kips
Total Gravity Load to Column	=	1.799 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	=	1.145 kips
Total Wind Uplift to Column	=	0.982 kips

LATERAL WIND LOAD

Wind, Lateral	=	14.01 psf
Height of Vertical Surface	=	1.23 feet
Wind Load per Column	=	0.078 kips
Wind Moment to Column	=	0.782 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	=	0.164 kips	(1617-5-1)	
$T = 0.02 * hn^{(3/4)}$ =	=	0.112		
S_S	=	178.7 %		
S_1	=	75.0 %		
S_{MS} = 1.787 S_{DS}	=	2/3 * S_{MS}	=	1.191
S_{MI} = 1.125 S_{DI}	= =	2/3 * S_{MI}	=	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)	=	0.078 kips		
M_h= V*h	=	0.779 ft-kips		

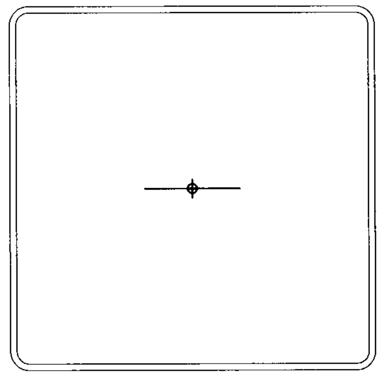
Wind Governs   See computer run for design size

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post Canopy Date: 11-09 By: GCS Job No.: 1304-09 SH.: 17

S.E. CONSULTANTS, INC.

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



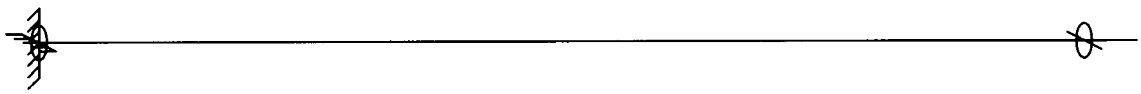
Section Inputs

```
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^6
Torsion Constant Override, J            0 in^4

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 11:18:09 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	0.1640	0.1640	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	1.6350	1.6350	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	-1.1450	-1.1450	k
2	Concentrated	90.000	9.958	NA	0.0780	NA	k
				Bearing Length		1.000	in
3	Concentrated	90.000	9.958	NA	0.0000	NA	k
				Bearing Length		1.000	in

Rev. Date: 11/17/2009 11:18:09 AM
By: Steve Schaub, P.E.

Loading: Earthquake Load

Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1 Concentrated	90.000	9.958	NA	0.0780	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

Rev. Date: 11/17/2009 11:18:09 AM
By: Steve Schaub, P.E.

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

```
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          Mx          Vy          My          Vx
                 (k)       (k-ft)        (k)       (k-ft)        (k)
Total          1.799     0.0000       0.000      0.0000       0.000
Applied        1.799     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^2   Ixe        2.7997 in^4   Iye         2.7997 in^4
                        Sxe(t)     1.3998 in^3   Sye(l)      1.3998 in^3
                        Sxe(b)     1.3998 in^3   Sye(r)      1.3998 in^3

Interaction Equations
NAS Eq. C5.2.1-1  (P, Mx, My)  0.288 + 0.000 + 0.000 = 0.288 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.074 + 0.000 + 0.000 = 0.074 <= 1.0
NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.000 + 0.000)= 0.000 <= 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)

```
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          Mx          Vy          My          Vx
                 (k)       (k-ft)        (k)       (k-ft)        (k)
Total         -1.047     0.7800      -0.078      0.0000       0.000
Applied       -1.047     0.7800      -0.078      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^2   Ixe        2.7997 in^4   Iye         2.7997 in^4
                        Sxe(t)     1.3998 in^3   Sye(l)      1.3998 in^3
                        Sxe(b)     1.3998 in^3   Sye(r)      1.3998 in^3

Interaction Equations
NAS Eq. C5.1.1-1  (Mx, My, T)  0.203 + 0.000 + 0.029 = 0.232 <= 1.0
NAS Eq. C5.1.1-2  (Mx, My, T)  0.244 + 0.000 - 0.029 = 0.215 <= 1.0
NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.060 + 0.000)= 0.244 <= 1.0
NAS Eq. C3.3.1-1     (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```

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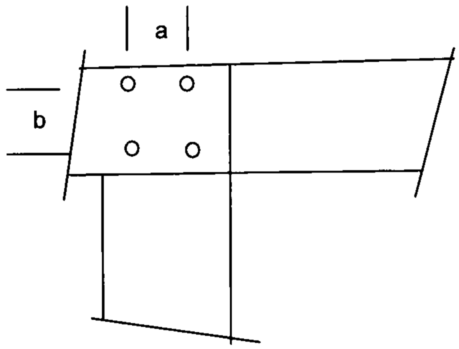
Rev. Date: 11/17/2009 11:18:09 AM
By: Steve Schaub, P.E.

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

```
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          Mx          Vy          My          Vx
                   (k)       (k-ft)        (k)       (k-ft)        (k)
Total             0.164      0.3399      -0.015      0.0000      0.000
Applied           0.164      0.3399      -0.015      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^2     Ixe         2.7997 in^4    Iye         2.7997 in^4
                           Sxe(t)      1.3998 in^3    Sye(l)      1.3998 in^3
                           Sxe(b)      1.3998 in^3    Sye(r)      1.3998 in^3

Interaction Equations
NAS Eq. C5.2.1-1  (P, Mx, My)  0.026 + 0.109 + 0.000 = 0.135 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.007 + 0.106 + 0.000 = 0.113 <= 1.0
NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.011 + 0.000)= 0.106 <= 1.0
NAS Eq. C3.3.1-1     (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```

ECCENTRIC 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
Ip	=	polar moment of inertia = Ixx + Iyy
Ixx	=	[nb^2(n^2-1)/12] x no. of vertical rows
Iyy	=	[mb^2(m^2-1)/12] x no. of horizontal rows
f1	=	P/(mxn)
f2	=	(moment)xb/2Ip
f3	=	(moment)x(n-1)b/2Ip
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	=	1.799 kips	Total Seismic Load	=	0.164 kips
Total Gravity Moment	=	0.000 k-ft	Total Seismic Moment	=	0.779 k-ft
Total Uplift Load	=	1.145 kips			
Total Uplift Moment	=	0.782 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 rows	=	2
Vertical Dimension	=	1.833 inches
Horizontal Dimension	=	4 inches
Ixx	=	33.599
Iyy	=	32
Ip	=	65.599

Use:

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

		Gravity	Seismic	Wind
f1	=	0.112 kips	0.010 kips	0.072 kips
f2	=	0.000 kips	0.143 kips	0.143 kips
f3	=	0.000 kips	0.196 kips	0.197 kips
fr, (total)	=	0.112 kips	0.248 kips	0.291 kips

Allowable Shear 0.625 kips

rv, Allowable w/ 33% increase 0.625 kips

Gravity: o.k.
Wind: o.k.
Seismic: o.k.

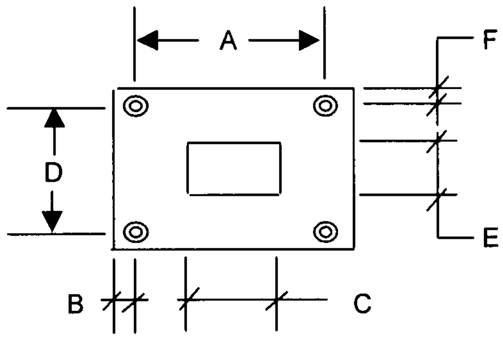
Increase Aloud (y, n) N

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4post Canopy Date: 11-09 By: BCS Job No.: 1304-09 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	=	1.80 kips	
Total Gravity Moment	=	0.00 k-ft	
Total Uplift Load	=	-1.15 kips	
Total Uplift Moment	=	0.78 k-ft	
Total Seismic Load	=	0.16 kips	
Total Seismic Moment	=	0.78 k-ft	

Tension in Bolts

Due to Gravity	=	-0.45 kips	
Due to Uplift	=	0.81 kips	**Wind Controls**
Due to Seismic	=	0.48 kips	

Checking Plate

Bending Moment	=	4.04 k-in	
t, Thickness of material	=	0.50 in	
Fy, of base plate	=	50.0 ksi	
Sx, Section Modulus	=	0.50 in^3	
Fb	=	37.5 ksi	
fb	=	8.1 ksi	o.k.

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

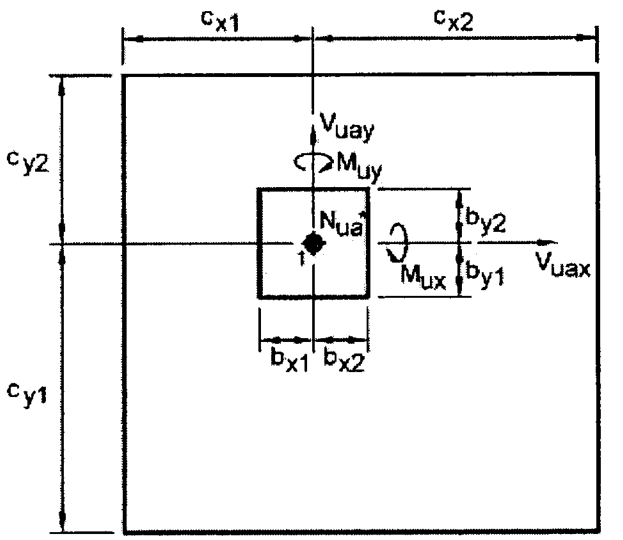
a) Layout

Anchor : 3/4" Titen HD

Number of Anchors : 1

Embedment Depth : 5.5 in

Built-up Grout Pads : No



1 ANCHOR

*Nua 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

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Concrete : Normal weight

Cracked Concrete : Yes

Condition : B tension and shear

Thickness, h : 24 in

Supplementary edge reinforcement : No

f'_c : 2500.0 psi

$\Psi_{c,V}$: 1.40

ϕF_p : 1381.3 psi

c) Factored Loads

Load factor source : ACI 318 Appendix C

N_{ua} : 4552 lb

V_{uay} : 0 lb

M_{uy} : 0 lb*ft

e_x : 0 in

e_y : 0 in

V_{uax} : 0 lb

M_{ux} : 0 lb*ft

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 in
Category = 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 lb

Sum of Anchor Tension ΣN_{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 ΣV_{uax} = 0.00 lb, ΣV_{uay} = 0.00 lb

e'_{Vx} = 0.00 in

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$e'_{Vy} = 0.00$ in

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

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

Number of anchors acting in tension, n = 1

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

$\phi = 0.70$ [D.4.5]

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

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

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

Number of influencing edges = 4

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

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

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

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

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

Note: Cracking shall be controlled per D.5.2.6

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

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

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

$k_c = 17$ [Sec. D.5.2.6]

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

$\phi = 0.75$ [D.4.5]

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

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

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

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

$\phi = 0.75$

$\phi N_{pn} = 4552.50$ 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]

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

ϕ = 0.65 [D.4.5]

ϕV_{sa} = 10946.00 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...

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

c_{a1} = 6.00 in

A_{vcx} = 108.00 in^2

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

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

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

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

l_e = 4.22 in

V_{bx} = 6293.26 lb

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

ϕ = 0.75

ϕV_{cbx} = 3964.75 lb (for a single anchor)

In y-direction...

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

c_{a1} = 6.00 in

A_{vcy} = 108.00 in^2

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

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

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

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

l_e = 4.22 in

V_{by} = 6293.26 lb

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

ϕ = 0.75

ϕV_{cby} = 3964.75 lb (for a single anchor)

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

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Case 3: Anchor checked for parallel to edge condition

Check anchors at c_{x1} edge

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

$c_{a1} = 6.00$ in

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

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

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

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

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

$l_e = 4.22$ in

$V_{bx} = 6293.26$ lb

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

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

$V_{cby} = 11747.42$ lb

$\phi = 0.75$

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

Check anchors at c_{y1} edge

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

$c_{a1} = 6.00$ in

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

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

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

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

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

$l_e = 4.22$ in

$V_{by} = 6293.26$ lb

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

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

$V_{cbx} = 11747.42$ lb

$\phi = 0.75$

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

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Check anchors at c_{x2} edge

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

c_{a1} = 6.00 in

A_{vcx} = 108.00 in^2

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

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

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

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

l_e = 4.22 in

V_{bx} = 6293.26 lb

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

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

V_{cby} = 11747.42 lb

ϕ = 0.75

ϕV_{cby} = 8810.57 lb (for a single anchor)

Check anchors at c_{y2} edge

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

c_{a1} = 6.00 in

A_{vcy} = 108.00 in^2

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

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

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

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

l_e = 4.22 in

V_{by} = 6293.26 lb

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

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

V_{cbx} = 11747.42 lb

ϕ = 0.75

ϕV_{cbx} = 8810.57 lb (for a single anchor)

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

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

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

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

$V_{cp} = 13600.00$ lb

$\phi = 0.75$ [D.4.5]

$\phi V_{cp} = 10200.00$ 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.Max(0) <= 0.2 and T.Max(1) <= 1.0 [Sec D.7.1]

Interaction check: PASS

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

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NON CONSTRAINED POLE TYPE FOOTING PER INTERNATIONAL BUILDING CODE

DESCIPTION 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	=	0.782 ft. kips
P, Lateral load = M / h	=	78 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 =		0.229

REQUIRED "d" = 1.70 FEET < 3 FEET
O.K.

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

CHECK VERTICAL BEARING CAPACITY

P(t.l.), Gravity Load	=	1.799 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	=	572.639 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	=	1.145 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	=	3.94

CHECK STRESSES OF CONCRETE PIER

f'c, 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.^3
f, Stress in Plain Concrete Pier = M/S	=	7 psi.
ft, Allowable Tension = 1.6 x SQRT. f'c	=	80 psi.
ft, Increased for Wind or Seismic	=	106.67 psi.

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

Job: 4 post Canopy Date: 11-09 By: GCS Job No.: 1304-09 SH.: 32

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SPREAD FOOTING W/ COLUMN CENTERED IN FOOTING

DEAD LOAD MOMENT RESISTANCE AGAINST OVERTURNING SOIL PRESSURE

Applied Overturning Moment	=	0.782	ft-kips
D.L. of Structure	=	0.164	kips
Length of Footing	=	2	feet
Width of Footing	=	2	feet
Thickness of Footing	=	24	inches
Dead Load of Footing	=	1.2	kips
Volume of Earth in 30 degree Cone	=	9.2	cu. ft.
Slab on Grade (y or n)		N	
Length of Turndown	=	0	ft
Depth of Turndown	=	0	ft
Dead Load of Turndown	=	0.00	kips
Dead Load of Earth	=	0.925	kips
Total D.L.	=	2.289	kips
D.L. Resisting Moment	=	2.29	ft-kips
Required Factor of Safety	=	1.5	
F. of S. Against Overturning	=	2.927	

Allowable Soil Pressure	=	1500	psf
Increases for Width OR Depth	=	0	
Allowable S. P. w/ Increases	=	1500	
Loads Due to Wind or Seismic? (Y/N)		N	
S.P. w/ 33% Increase	=	N/A	psf
Applied Overturning Moment	=	0.782	ft-kips
D.L. of Structure	=	0.164	kips
Footing D.L.	=	1.2	kips
Total Vert. Load	=	1.364	kips
e' = (Mr - OTM) / Total Vert. Load	=	1.105	feet
e = l / 2 - e'	=	-0.105	feet
l / 6 =	=	0.333	feet
RESULTANT IN MIDDLE 1/3			
S.P. = P/A +/- M/S			
Soil Pressure	=	927.5	psf
		O.K.	

DEAD LOAD RESISTANCE AGAINST UPLIFT

Applied Uplift Load - Total Wind	=	0.78	kips
D.L. of Structure	=	0.164	kips
Volume of Earth in 30 degree Cone	=	9.249	cu. ft.
Dead Load of Earth	=	0.925	kips
Dead Load of Footing	=	1.20	kips
Dead Load of Earth	=	0.925	kips
Total Dead Load Resistance	=	2.289	kips
F. of S. Against Uplift	=	2.927	
		O.K.	

FOOTING REINFORCING

f'c	=	2500	psi
fy	=	40000	psi
d	=	21	inches
M, Moment	=	0.782	ft.-kips
Mu, Ultimate Moment = 1.7 M	=	1.329	ft.-kips
Assume a one foot strip			
Mu, = 1.7 M / Width of Footing	=	0.665	ft.-kips
Beta	=	0.85	
Phi	=	0.85	
rho, balanced	=	0.031	
m	=	18.824	
1.773 psi = Rn		0.002	ksi
rho, calc.	=	0.0000	
rho, min.	=	0.0000	
rho, Req'd.	=	0.0000	
Req'd. As	=	0.011	sq. in. / ft.
Total As Req'd.	=	0.022	sq. in.
Total As Req'd. (Wind or Seismic)	=	0.017	sq. in.
USE: 3 -# 5 As	=	0.920	sq.in.

LENGTH OF FOOTING: 2 feet
WIDTH OF FOOTING: 2 feet
THICKNESS OF FOOTING: 24 inches
REINFORCING: 3 - #5 Bars
Use #4's @ 12" o.c. lateral

PRELIMINARY UNLESS SEALED ON EACH SHEET OR ON COVER SHEET

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