UNITED
STRUCTURAL DESIGN LLC

NSG2 - Colusa

Structural Calculations

Project Adress: 1017 Bridge Street
Engineer: DG

Issue Date: June 27, 2016

REGISTERED PROFESSIONAL ENGINEER
JOHN B. ELDER
C 83746
06/27/2016
CIVIL
STATE OF CALIFORNIA

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UNITED
STRUCTURAL DESIGN LLC

PROJECT NAME: NSG2 - Colusa
PROJECT LOCATION: 1017 Bridge Street
ENGINEER: DG
REVIEWER:
DATE: 6/21/2016

Project Name: NSG2 - Colusa
Job Number:

CODE

2013 California Building Code / ASCE 7-10

STRUCTURAL NARRATIVE

Gravity loads are supported by steel purlins which are supported by steel beams and steel columns. Lateral loads are transmitted through the steel purlin framing directly into the steel beams and from there, into the steel columns conforming to the Special Cantilever Column System (SCCS) requirements. Drilled piers distribute gravity and lateral loads to the soil.

LOADS

Roof:

Dead Load

DL :	8.0 psf

Roof Live Load

RLL:	0.0 psf

Wind

Occupancy Category	I
V :	100 MPH
Exposure Category :	C
Importance Category :	1.00
Mean Roof Height :	15.0 ft
G :	0.85
K_d :	0.85
K_{zt} :	1.0
K_z :	0.85
Enclosure Classification :	Open Structure

Seismic

Occupancy Category	I
I_e :	1
Seismic Site Class:	D
Seismic Design Category:	D
S_s :	0.792
S_1 :	0.331
S_{DS} :	0.625
S_{D1} :	0.384
R :	1.25
Ω :	1.25
C_d :	1.25
C_s :	0.500

UNITED STRUCTURAL DESIGN LLC

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Dead Load

Solar Panels :	3.0 psf
Purlins :	1.0 psf
Beams:	2.0 psf
Misc. :	2.0 psf
Total Dead Load :	8.0 psf

Roof Live Load

The IBC defines Live Loads (Roof) as "Those loads produced (1) during maintence by workers, equipment and materials; (2) during the life of the sturcture by movable objects such as planters or other similar small decorative appurtenances that are not occupacy related; and (3) use and occupancy of the roof such as for roof gardens or assembly areas."
The code also reads "All roof surfaces subject to mainenance worker" be designed for 300# point loads or 20 psf roof live load.
The solar panels for this structure are not to be loaded ontop nor are they intended to support a person as defined by the solar panel manufacterer nor do they need to be re-roofed nor do they need to support any mechanical equipment. Therefore, Roof Live Loads do not apply to this structure. Lr = 0.

Material Strengths

Concrete:

Assumed f'c:	2500 psi

Steel:

Rebar:	ASTM A615, Fy = 60ksi
	ASTM A706, Fy = 60ksi
Bolts:	ASTM A325N
Anchor Rods:	ASTM F1554 Gr. 55
W Section:	ASTM A992, Fy = 50ksi
M, S, C, MC, L Sections:	ASTM A36, Fy = 50ksi
HSS Rect. Section:	ASTM A500 Gr. B, Fy = 46ksi
HSS Round. Section:	ASTM A500 Gr. B, Fy = 42ksi
Light Gage Steel:	Fy = 55ksi

Soil:

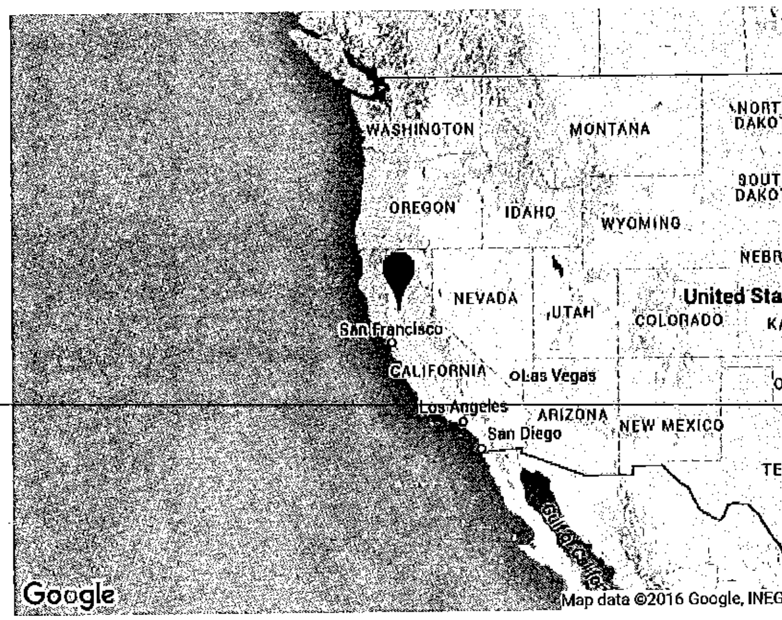
Allowable Soil Bearing:	1500 psf
Allowable Lateral Bearing:	150 psf/ft

Applied Technology Council

WINDSPEED BY LOCATION

ASCE 7 Windspeed | ASCE 7 Ground Snow Load | Related Resources | Sponsors | About ATC | Contact

Search Results



Query Date: Tue Jun 21 2016
Latitude: 39.2043
Longitude: -122.0026

ASCE 7-10 Windspeeds (3-sec peak gust in mph*):

Risk Category I: 100
Risk Category II: 110
Risk Category III-IV: 115
MRI 10-Year:** 72
MRI 25-Year:** 79
MRI 50-Year:** 85
MRI 100-Year:** 91

ASCE 7-05 Windspeed:
85 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
75 (fastest mile in mph)

*Miles per hour
**Mean Recurrence Interval

Users should consult with local building officials to determine if there are community-specific wind speed requirements that govern.

Print your results

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UNITED STRUCTURAL DESIGN LLC

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DATE: 6/21/2016

Open Building Wind Load

EXPOSURE C WIND LOADS

Code:	2012 IBC \ ASCE 7-10	(2013 IBC)
Mean Roof Height z :	15.00 ft	
Basic Wind Speed V :	100 mph	
Occupancy Category :	I	
Exposure Category :	C	
Directionality K_d :	0.85	
Topographic Factor K_{zt} :	1.00	
Type of Windflow :	Clear Wind Flow	
Gust Factor G :	0.85	
Importance Factor I_w :	1	
Structure Inputs		
Fascia Thickness T_1 :	1.0 ft	
Roof Angle :	5.0 deg	
Total Structure Width B :	37.0 ft	
Total Structure Length L :	132.0 ft	

Wind Loads - Open Buildings: $0.25 \leq h/L \leq 1.0$

[Code Ref]

[ASCE 7, Table 26.9-1]	z_g =	900.0 ft
[ASCE 7, Table 26.9-1]	α =	9.5
	K_z =	0.85
	K_{zt} =	1.00
	K_d =	0.85
	I_w =	1.00
[ASCE 7, eq. 27.3-1]	q_h =	18.5 psf

Main Wind Force Resisting System for Monoslope Roof

G = 0.85
q_h = 18.5 psf

Roof pressures - Wind Normal to Ridge

Wind Flow	Load Case		Wind Direction X = 0 deg		Wind Direction γ = 180 deg	
			C_{nw}	C_{nl}	C_{nw}	C_{nl}
Clear Wind Flow	A	Cn =	1.20	0.30	1.20	0.30
		p =	18.8 psf	4.7 psf	18.8 psf	4.7 psf
	B	Cn =	-1.10	-0.10	-1.10	-0.10
		p =	-17.3 psf	-1.6 psf	-17.3 psf	-1.6 psf

NOTE: 1). Cnw and Cnl denote combined pressures from top and bottom roof surfaces.
2). Cnw is pressure on windward half of roof. Cnl is pressure on leeward half of roof.
3). Positive pressures act toward the roof. Negative pressures act away from the roof.

Roof pressures - Wind Parallel to Ridge, X = 90 deg

Wind Flow	Load Case		Horizontal Distance from Windward Edge		
			≤ h	>h ≤ 2h	> 2h
Clear Wind Flow	A	Cn =	-0.80	-0.60	-0.30
		p =	-12.6 psf	-9.4 psf	-4.7 psf
	B	Cn =	0.80	0.50	0.30
		p =	12.6 psf	7.9 psf	4.7 psf

h = 15.0 ft
2h = 30.0 ft

NOTE: 1). Cn denotes net pressures from top and bottom roof surfaces.

Fascia Panels -Horizontal pressures

qp = 18.5 psf

Windward fascia: +/-27.71 psf (GCpn = +1.5)
Leeward fascia: +/-18.47 psf (GCpn = -1.0)

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Open Building Wind Load

Components and Cladding Pressures

a = 3.7 ft

	Effective Wind Area	Clear Wind Flow					
		zone 3		zone 2		zone 1	
		positive	negative	positive	negative	positive	negative
C_N	≤ 13.69 sf	2.93	-3.90	2.20	-1.97	1.47	-1.30
	>13.69, ≤ 54.76 sf	2.20	-1.97	2.20	-1.97	1.47	-1.30
	> 54.76 sf	1.47	-1.30	1.47	-1.30	1.47	-1.30
$P_{wc\&c}$	≤ 13.69 sf	46.1 psf	-61.2 psf	34.5 psf	-30.9 psf	23.0 psf	-20.4 psf
	>13.69, ≤ 54.76 sf	34.5 psf	-30.9 psf	34.5 psf	-30.9 psf	23.0 psf	-20.4 psf
	> 54.76 sf	23.0 psf	-20.4 psf	23.0 psf	-20.4 psf	23.0 psf	-20.4 psf

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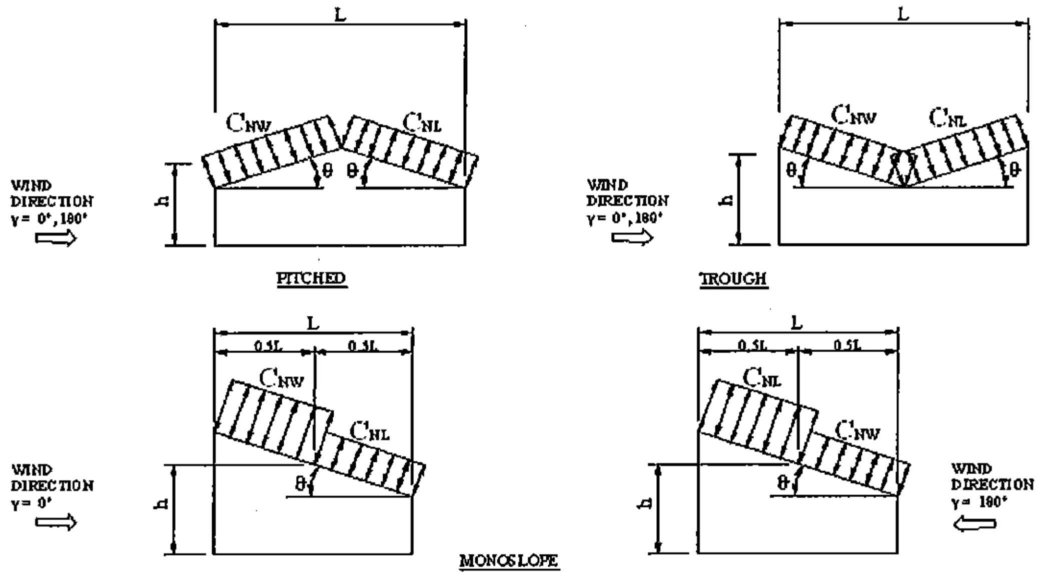
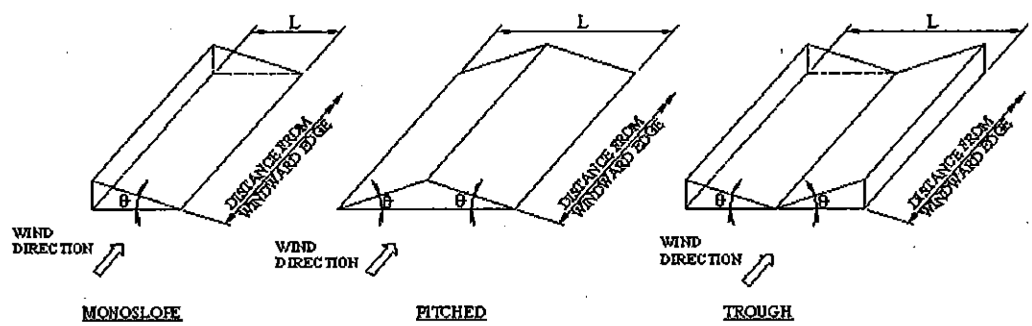
Open Building Wind Load

Location of Wind Pressure Zones

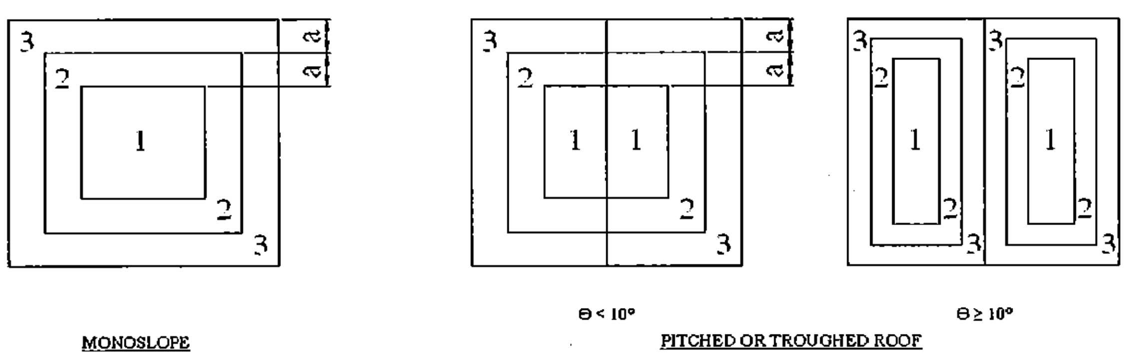


WIND DIRECTION $\gamma = 0°, 180°$



WIND DIRECTION $\gamma = 90°$

MAIN WIND FORCE RESISTING SYSTEM



COMPONENTS AND CLADDING

USGS Design Maps Summary Report

User–Specified Input

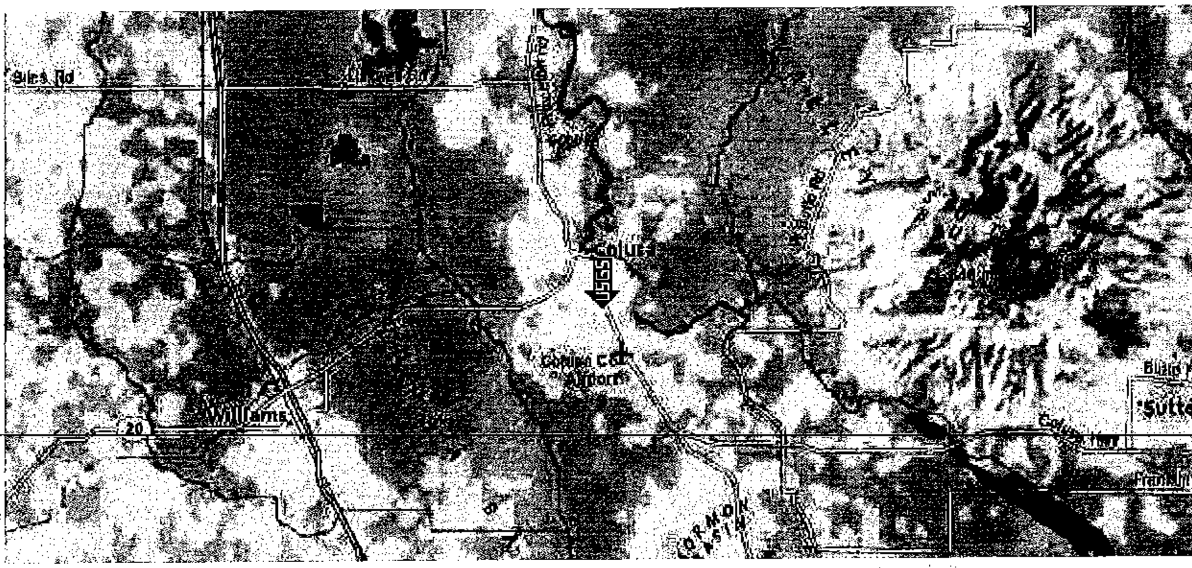
Report Title NSG2 - Colusa
Tue June 21, 2016 22:31:57 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 39.2043°N, 122.00261°W

Site Soil Classification Site Class D – "Stiff Soil"

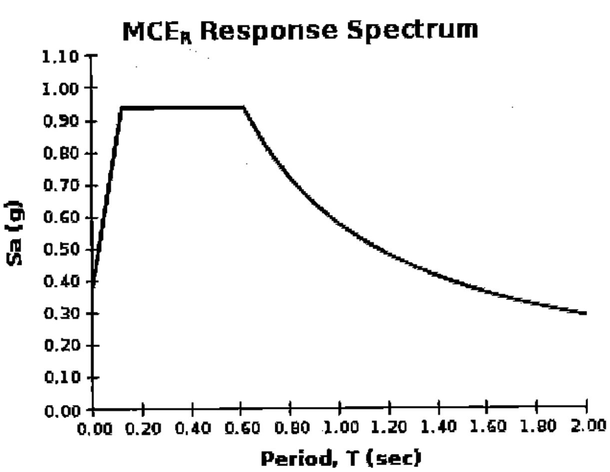
Risk Category I/II/III

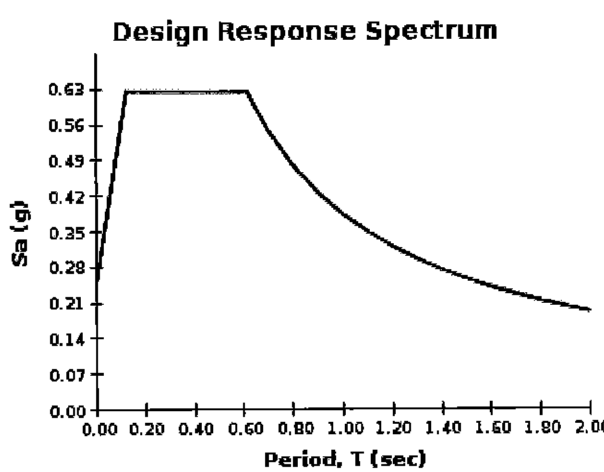


USGS–Provided Output

$S_S =$	0.792 g	$S_{MS} =$	0.937 g	$S_{DS} =$	0.625 g
$S_1 =$	0.331 g	$S_{M1} =$	0.575 g	$S_{D1} =$	0.383 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

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JOB TITLE NSG2 - Colusa

JOB NO. SHEET NO.
CALCULATED BY DG DATE 6/21/16
CHECKED BY DATE

Seismic Loads: ASCE 7- 10

Strength Level Forces

Risk Category : I
Importance Factor (I) : 1.00
Site Class : D

Ss (0.2 sec) = 79.20 %g
S1 (1.0 sec) = 33.10 %g

Fa =	1.183	Sms =	0.937	S_{DS} =	0.625	Design Category =	D
Fv =	1.738	Sm1 =	0.575	S_{D1} =	0.384	Design Category =	D

Seismic Design Category = **D**
Number of Stories: 1
Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **error**
Seismic resisting system: **Steel ordinary cantilever column system**
~~System Structural Height Limit:~~ **System not permitted for this seismic design category**
Actual Structural Height (hn) = 16.1 ft
See ASCE7 Section 12.2.5 for exceptions and other system limitations

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 1.25
Over-Strength Factor (Ωo) = 1.25
Deflection Amplification Factor (Cd) = 1.25
S_{DS} = 0.625
S_{D1} = 0.384

Seismic Load Effect (E) = $\rho\, Q_E$ +/- $0.2S_{DS}$ D = $\rho\, Q_E$ +/- 0.125D
Special Seismic Load Effect (Em) = $\Omega o\, Q_E$ +/- $0.2S_{DS}$ D = 1.3 Q_E +/- 0.125D

ρ = redundancy coefficient
Q_E = horizontal seismic force
D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.020
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.161 sec x= 0.75
User calculated fundamental period (T) = 0 sec
Long Period Transition Period (TL) = ASCE7 map = 8
Seismic response coef. (Cs) = $S_{DS}I/R$ = 0.500
need not exceed Cs = Sd1 I /RT = 1.909
but not less than Cs = 0.044SdsI = 0.027
USE Cs = 0.500
Design Base Shear V = 0.500W

Cu = 1.40
Tmax = CuTa = 0.225
Use T = 0.161

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift = 0.020hsx where hsx is the story height below level x

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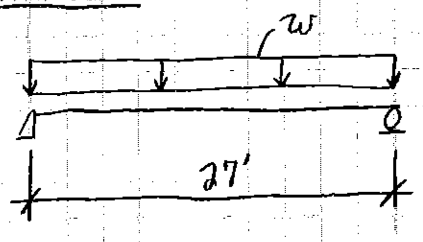
PROJECT NSG2 - Colusa	
ENGINEER DJG	DATE

PURLIN DESIGN

Typical Purlin



LOADS

$w_{DL} = 4\,PSF(3.25') = 13\,PLF$

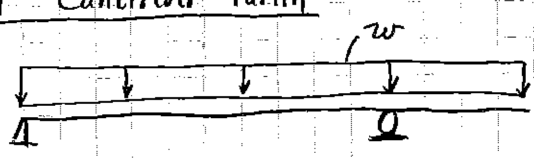
$w_{WL} = 23\,PSF(3.25') = 75\,PLF$

$w_{UPLIFT} = -20.4\,PSF(3.25') = 67\,PLF$

See Attached Calc

$\boxed{\text{USE } 10 \times 3\tfrac{1}{2} \times 14 \text{ GA.}}$

Typical Cantilever Purlin



Same loads as above

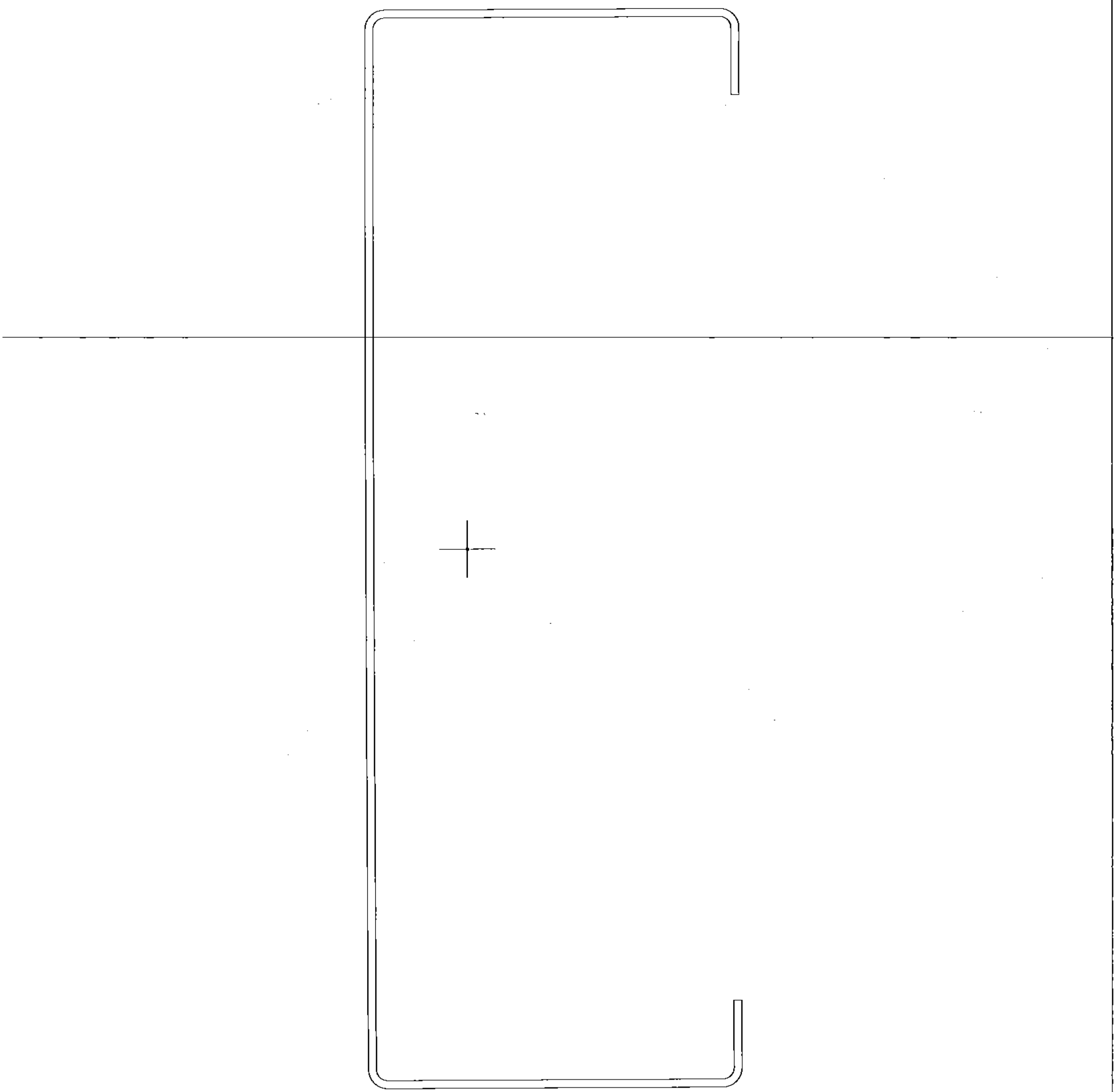
See Attached Calc

$\boxed{\text{USE } 10 \times 3\tfrac{1}{2} \times 14 \text{ GA.}}$

CFS Version 9.0.4
Section: 10x3.5x14 Gauge.cfss
Channel 10x3.5x0.8-14 Gage

Rev. Date: 6/21/2016

Printed: 6/21/2016



CFS Version 9.0.4

Section: 10x3.5x14 Gauge.cfss
Channel 10x3.5x0.8-14 Gage

Rev. Date: 6/21/2016

Printed: 6/21/2016

Section Inputs

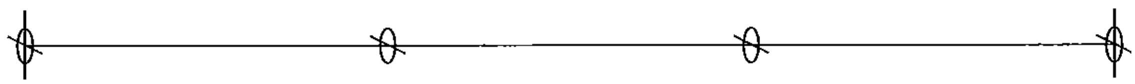
```
Material: A653 SS Grade 55
No strength increase from cold work of forming.
Modulus of Elasticity, E               29500 ksi
Yield Strength, Fy                        55 ksi
Tensile Strength, Fu                      70 ksi
Warping Constant Override, Cw              0 in^6
Torsion Constant Override, J               0 in^4

Stiffened Channel, 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	0.800	270.000	0.10690	None	0.000	0.000	0.400
2	3.500	180.000	0.10690	Single	0.000	0.000	1.750
3	10.000	90.000	0.10690	Cee	0.000	0.000	5.000
4	3.500	0.000	0.10690	Single	0.000	0.000	1.750
5	0.800	-90.000	0.10690	None	0.000	0.000	0.400

CFS Version 9.0.4
Analysis: Typical Purlin.cfsa
27 ft Span Simple Beam

Rev. Date: 6/21/2016

Printed: 6/21/2016



Analysis Inputs

Members

	Section File	Revision Date and Time
1	10x3.5x14 Gauge.cfss	6/21/2016 4:02:12 PM

	Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	kϕ (k)	Lm (ft)
1	0.000	27.000	None	0.0000	0.0000	30.000

	ex (in)	ey (in)
1	0.000	0.000

Supports

	Type	Location (ft)	Bearing (in)	Fastened	K
1	XYT	0.000	2.00	No	1.0000
2	XT	9.000	1.00	No	1.0000
3	XT	18.000	1.00	No	1.0000
4	XYT	27.000	2.00	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	27.000	-0.013000	-0.013000	k/ft

Loading: Wind Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1	Distributed	90.000	0.000	27.000	-0.075000	-0.075000	k/ft

Loading: -Wind

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1	Distributed	90.000	0.000	27.000	0.067000	0.067000	k/ft

Rev. Date: 6/21/2016

Printed: 6/21/2016

Load Combination: D
Specification: 2012 North American Specification - US (ASD)
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000

Load Combination: D+0.6W
Specification: 2012 North American Specification - US (ASD)
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	1.000
2	Dead Load	1.000
3	Wind Load	0.600

Load Combination: 0.6D+0.6W
Specification: 2012 North American Specification - US (ASD)
Inflection Point Bracing: No

	Loading	Factor
1	Beam Self Weight	0.600
2	Dead Load	0.600
3	-Wind	0.600

Member Check - 2012 North American Specification - US (ASD)

Load Combination: D+0.6W
Design Parameters at 13.500 ft:

Lx	27.000 ft	Ly	9.000 ft	Lt	9.000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000

Section: 10x3.5x14 Gauge.cfss
Material Type: A653 SS Grade 55, Fy=55 ksi

Cbx	1.0135	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kφ	0 k		
Red. Factor, R: 0		Lm	30.000 ft		

Loads:	P (k)	Mx (k-in)	Vy (k)	My (k-in)	Vx (k)
Total	0.000	68.457	0.000	0.000	0.000
Applied	0.000	68.457	0.000	0.000	0.000
Strength	11.481	90.651	3.896	25.723	9.703

Effective section properties at applied loads:

Ae	1.35391 in^2	Ixe	20.708 in^4	Iye	2.106 in^4
		Sxe(t)	4.1415 in^3	Sye(l)	2.2093 in^3
		Sxe(b)	4.1415 in^3	Sye(r)	0.8268 in^3

Interaction Equations

NAS Eq. C5.2.1-1	(P, Mx, My)	0.000 + 0.755 + 0.000 = 0.755 <= 1.0
NAS Eq. C5.2.1-2	(P, Mx, My)	0.000 + 0.755 + 0.000 = 0.755 <= 1.0
NAS Eq. C3.3.1-1	(Mx, Vy)	Sqrt(0.425 + 0.000)= 0.652 <= 1.0
NAS Eq. C3.3.1-1	(My, Vx)	Sqrt(0.000 + 0.000)= 0.000 <= 1.0

CFS Version 9.0.4
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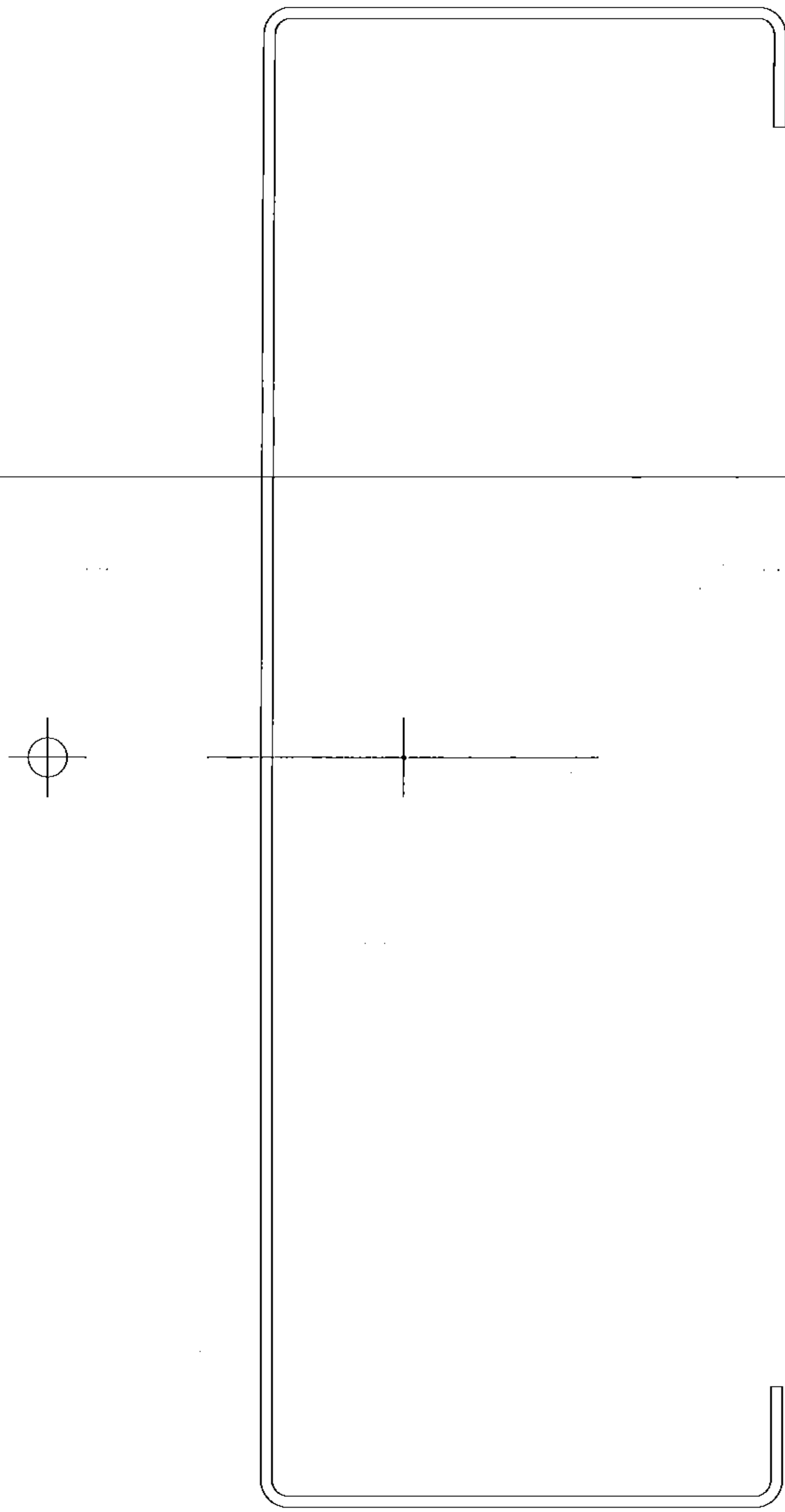

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Yield Strength, Fy	55 ksi
Tensile Strength, Fu	70 ksi
Warping Constant Override, Cw	0 in^6
Torsion Constant Override, J	0 in^4

Stiffened Channel, 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	0.800	270.000	0.10690	None	0.000	0.000	0.400
2	3.500	180.000	0.10690	Single	0.000	0.000	1.750
3	10.000	90.000	0.10690	Cee	0.000	0.000	5.000
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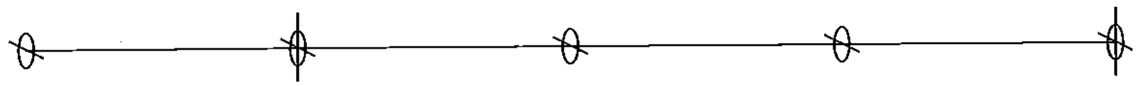

Analysis Inputs

Members

	Section File	Revision Date and Time
1	10x3.5x14 Gauge.cfss	6/21/2016 4:02:12 PM

	Start Loc. (ft)	End Loc. (ft)	Braced Flange	R	kϕ (k)	Lm (ft)
1	-9.000	27.000	None	0.0000	0.0000	30.000

	ex (in)	ey (in)
1	0.000	0.000

Supports

	Type	Location (ft)	Bearing (in)	Fastened	K
1	XT	-9.000	1.00	No	1.0000
2	XYT	0.000	2.00	No	1.0000
3	XT	9.000	1.00	No	1.0000
4	XT	18.000	1.00	No	1.0000
5	XYT	27.000	2.00	No	1.0000

Loading: Dead Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1	Distributed	90.000	-9.000	27.000	-0.013000	-0.013000	k/ft

Loading: Wind Load

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1	Distributed	90.000	-9.000	27.000	-0.075000	-0.075000	k/ft

Loading: -Wind

	Type	Angle (deg)	Start Loc. (ft)	End Loc. (ft)	Start Magnitude	End Magnitude	
1	Distributed	90.000	-9.000	27.000	0.067000	0.067000	k/ft

```
Load Combination: D
Specification:    2012 North American Specification - US (ASD)
Inflection Point Bracing: No
    Loading                     Factor
  1 Beam Self Weight             1.000
  2 Dead Load                    1.000

Load Combination: D+0.6W
Specification:    2012 North American Specification - US (ASD)
Inflection Point Bracing: No
    Loading                     Factor
  1 Beam Self Weight             1.000
  2 Dead Load                    1.000
  3 Wind Load                    0.600

Load Combination: 0.6D+0.6W
Specification:    2012 North American Specification - US (ASD)
Inflection Point Bracing: No
    Loading                     Factor
  1 Beam Self Weight             0.600
  2 Dead Load                    0.600
  3 -Wind                        0.600
```

Member Check - 2012 North American Specification - US (ASD)

```
Load Combination: D+0.6W
Design Parameters at 15.000 ft:
Lx        27.000 ft     Ly        9.000 ft      Lt          9.000 ft
Kx        1.0000        Ky        1.0000        Kt          1.0000

Section: 10x3.5x14 Gauge.cfss
Material Type: A653 SS Grade 55, Fy=55 ksi
Cbx       1.0303        Cby       1.0000        ex          0.0000 in
Cmx       1.0000        Cmy       1.0000        ey          0.0000 in
Braced Flange: None     kϕ             0 k
Red. Factor, R: 0       Lm        30.000 ft

Loads:          P         Mx          Vy         My         Vx
               (k)      (k-in)       (k)       (k-in)       (k)
Total         0.000     54.089      0.000      0.000      0.000
Applied       0.000     54.089      0.000      0.000      0.000
Strength     11.481     90.651      3.896     25.723      9.703

Effective section properties at applied loads:
Ae       1.35391 in^2   Ixe      20.708 in^4    Iye         2.106 in^4
                        Sxe(t)   4.1415 in^3    Sye(l)     2.2093 in^3
                        Sxe(b)   4.1415 in^3    Sye(r)     0.8268 in^3

Interaction Equations
NAS Eq. C5.2.1-1  (P, Mx, My)  0.000 + 0.597 + 0.000 = 0.597 <= 1.0
NAS Eq. C5.2.1-2  (P, Mx, My)  0.000 + 0.597 + 0.000 = 0.597 <= 1.0
NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.265 + 0.000)= 0.515 <= 1.0
NAS Eq. C3.3.1-1     (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```

Member Check - 2012 North American Specification - US (ASD)

Load Combination: D+0.6W
Design Parameters at 15.000 ft:

Lx	27.000 ft	Ly	9.000 ft	Lt	9.000 ft
Kx	1.0000	Ky	1.0000	Kt	1.0000

Section: 10x3.5x14 Gauge.cfss
Material Type: A653 SS Grade 55, Fy=55 ksi

Cbx	1.0303	Cby	1.0000	ex	0.0000 in
Cmx	1.0000	Cmy	1.0000	ey	0.0000 in
Braced Flange: None		kϕ	0 k		
Red. Factor, R: 0		Lm	30.000 ft		

Loads:	P (k)	Mx (k-in)	Vy (k)	My (k-in)	Vx (k)
Total	0.000	54.089	0.000	0.000	0.000
Applied	0.000	54.089	0.000	0.000	0.000
Strength	11.481	90.651	3.896	25.723	9.703

Effective section properties at applied loads:

Ae	1.35391 in^2	Ixe	20.708 in^4	Iye	2.106 in^4
		Sxe(t)	4.1415 in^3	Sye(l)	2.2093 in^3
		Sxe(b)	4.1415 in^3	Sye(r)	0.8268 in^3

Interaction Equations

```
NAS Eq. C5.2.1-1  (P, Mx, My)  0.000 + 0.597 + 0.000 = 0.597 <= 1.0
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NAS Eq. C3.3.1-1     (Mx, Vy)     Sqrt(0.265 + 0.000)= 0.515 <= 1.0
NAS Eq. C3.3.1-1     (My, Vx)     Sqrt(0.000 + 0.000)= 0.000 <= 1.0
```
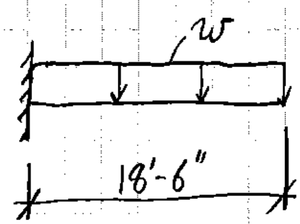

UNITED STRUCTURAL DESIGN LLC

PROJECT NSG2 Colusa	
ENGINEER DJG	DATE

STEEL BEAM DESIGN (STRUCTURE 1,2,3,4,5,+6)



trib = 21'

$w_{DL} = 8PSF(21') = 170\ PLF$

$w_{w} = 23PSF(21') = 485\ PLF$

See Attached Calc

$\boxed{W12X30}$

STEEL MEMBER ANALYSIS & DESIGN (AISC 360)

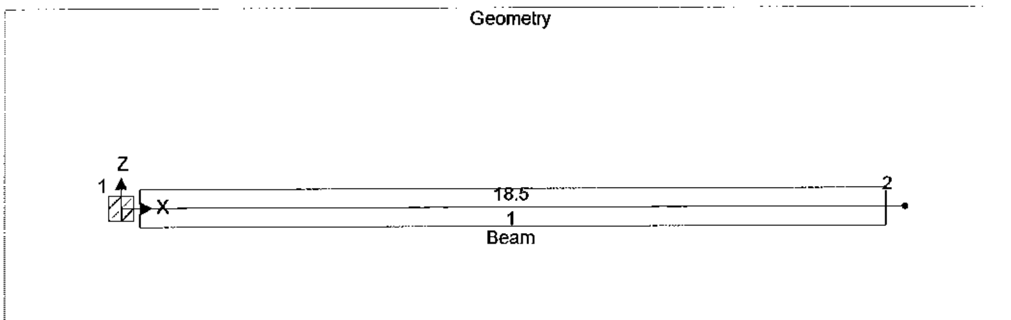
In accordance with AISC360 14th Edition published 2010 using the LRFD method

Tedds calculation version 4.1.00

ANALYSIS

Tedds calculation version 1.0.13

Geometry



Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (ft)	Z (ft)	X	Z	Rot.	Name	Angle (°)	X (kips/ft)	Z (kips/ft)	Rot. kip_ft/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	18.5	0	Free	Free	Free		0	0	0	0

Materials

Name	Density (lbm/ft³)	Youngs Modulus ksi	Shear Modulus ksi	Thermal Coefficient °C⁻¹
Steel (AISC)	490	29000	11200	0.000012

Sections

Name	Area	Moment of inertia		Shear area	
		Major	Minor	A_y	A_z
	(in²)	(in⁴)	(in⁴)	(in²)	(in²)
W 12x30	9	238	20	5	33

Elements

Element	Length (ft)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	18.5	1	2	W 12x30	Steel (AISC)	Fixed	Fixed	Fixed	

Members

Name	Elements	
	Start	End
Beam	1	1

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	Section Structures			Sheet no./rev. 2	
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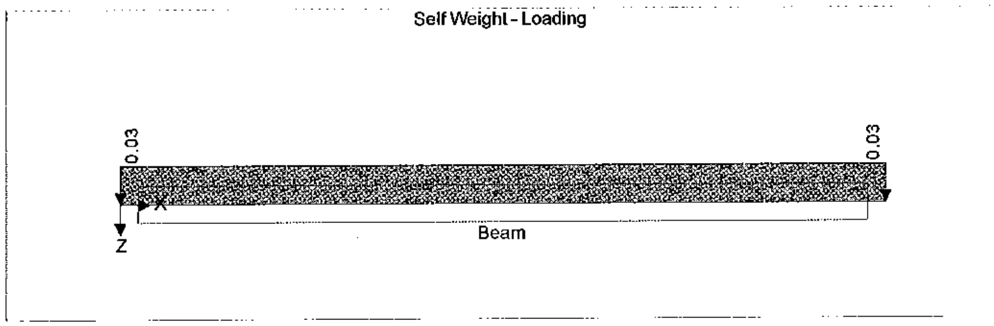
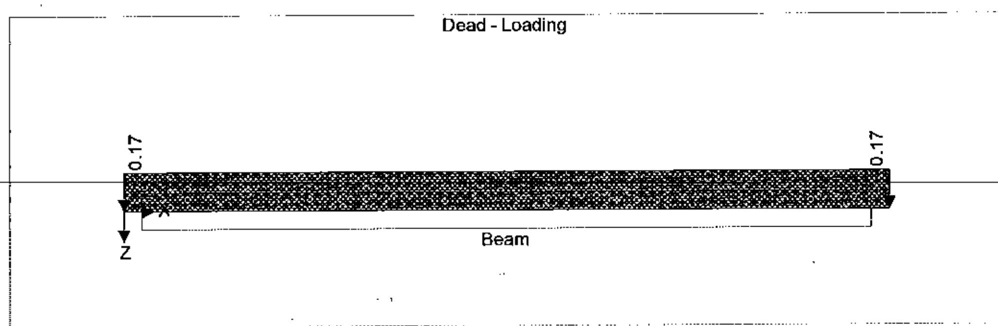
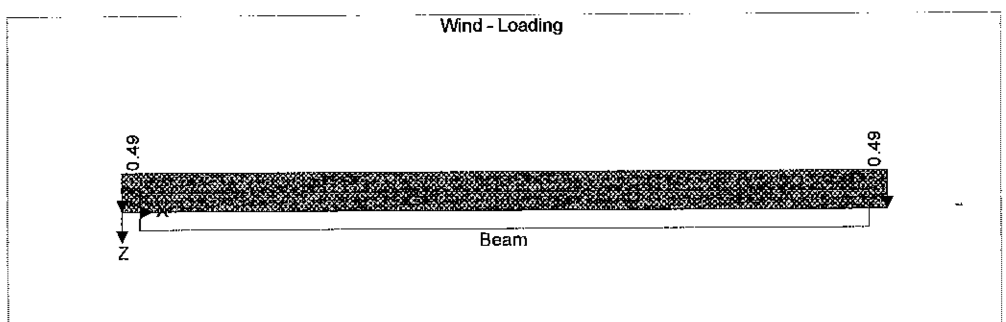
Loading







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	Section Structures				Sheet no./rev. 3	
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Load cases

Name	Enabled	Self weight factor	Patternable
Self Weight	yes	1	no
Dead	yes	0	no
Wind	yes	0	no

Load combinations

Load combination	Type	Enabled	Patterned
1.2D+1.0W	Strength	yes	1.2D+1.0W
D	Service	yes	D
0.42W	Service	yes	0.42W

Load combination: 1.2D+1.0W (Strength)

Load case	Factor
Self Weight	1.2
Dead	1.2
Wind	1

Load combination: D (Service)

Load case	Factor
Self Weight	1
Dead	1

Load combination: 0.42W (Service)

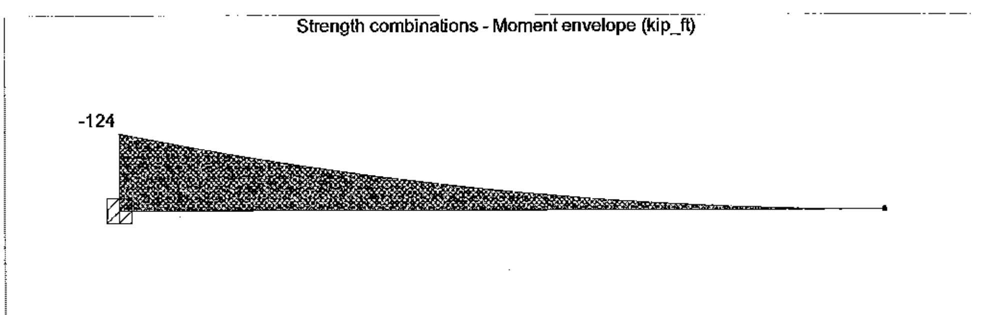
Load case	Factor
Wind	0.42

Member UDL loads

Member	Load case	Position			Load	Orientation
		Type	Start	End	(kips/ft)	
Beam	Dead	Ratio	0	1	0.17	GlobalZ
Beam	Wind	Ratio	0	1	0.49	GlobalZ

Results

Forces



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	Section Structures				Sheet no./rev. 4	
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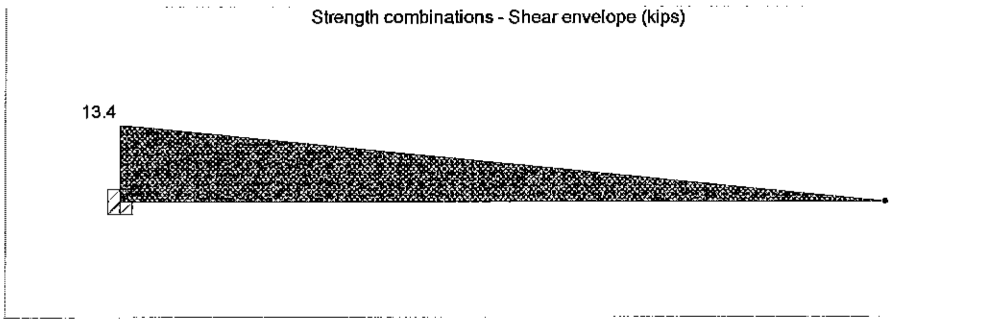

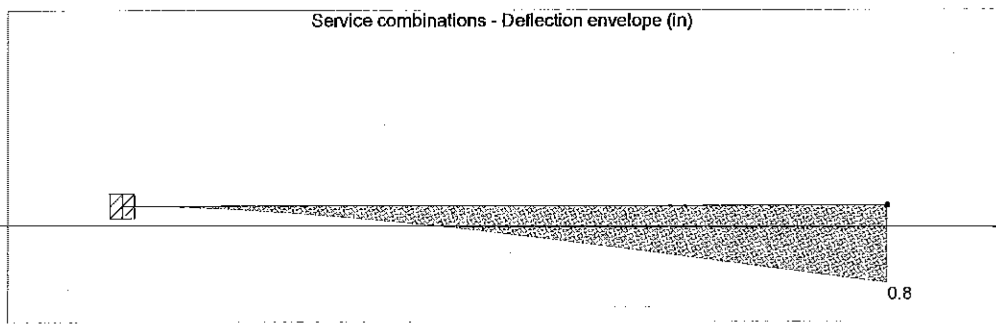

Resistance factors

Shear	ϕ_v = **1.00**
Flexure	ϕ_b = **0.90**
Tensile yielding	$\phi_{t,y}$ = **0.90**
Tensile rupture	$\phi_{t,r}$ = **0.75**
Compression	ϕ_c = **0.90**

Beam - Span 1

Section details

Section type	W 12x30 (AISC 14th Edn 2010)
ASTM steel designation	A992
Steel yield stress	F_y = **50** ksi
Steel tensile stress	F_u = **65** ksi
Modulus of elasticity	E = **29000** ksi

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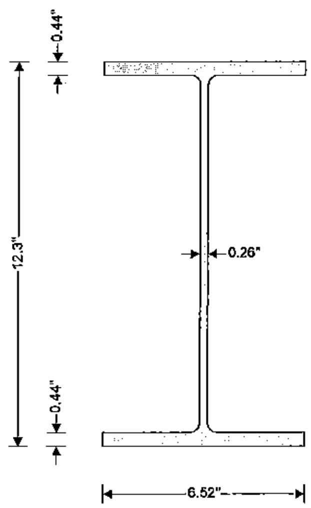
	Section Structures				Sheet no./rev. 5	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date



W 12x30 (AISC 14th Edn 2010)
Section depth, d, 12.3 in
Section breadth, b_f, 6.52 in
Weight of section, Weight, 30 lbf/ft
Flange thickness, t_f, 0.44 in
Web thickness, t_w, 0.26 in
Area of section, A, 8.8 in²
Radius of gyration about x-axis, r_x, 5.21 in
Radius of gyration about y-axis, r_y, 1.52 in
Elastic section modulus about x-axis, S_x, 38.6 in³
Elastic section modulus about y-axis, S_y, 6.24 in³
Plastic section modulus about x-axis, Z_x, 43.1 in³
Plastic section modulus about y-axis, Z_y, 9.56 in³
Second moment of area about x-axis, I_x, 238 in⁴
Second moment of area about y-axis, I_y, 20.3 in⁴

Lateral restraint

Both flanges have lateral restraint at supports plus 5ft 1.6in & 11ft 9.7in

Consider Combination 1 - 1.2D+1.0W (Strength)

Classification of sections for local buckling - Section B4

Classification of flanges in flexure - Table B4.1b (case 10)

Width to thickness ratio	$b_f / (2 \times t_f) =$ **7.41**	
Limiting ratio for compact section	$\lambda_{pff} = 0.38 \times \sqrt{[E / F_y]} =$ **9.15**	
Limiting ratio for non-compact section	$\lambda_{rff} = 1.0 \times \sqrt{[E / F_y]} =$ **24.08**	Compact

Classification of web in flexure - Table B4.1b (case 15)

Width to thickness ratio	$(d - 2 \times k) / t_w =$ **41.62**	
Limiting ratio for compact section	$\lambda_{pwf} = 3.76 \times \sqrt{[E / F_y]} =$ **90.55**	
Limiting ratio for non-compact section	$\lambda_{rwf} = 5.70 \times \sqrt{[E / F_y]} =$ **137.27**	Compact

Section is compact in flexure

Check design at start of span

Design of members for shear - Chapter G

Required shear strength	$V_{r,x} =$ **13.4** kips
Web area	$A_w = d \times t_w =$ **3.198** in²
Web plate buckling coefficient	$k_v =$ **5**
	$(d - 2 \times k) / t_w <= 2.24 \times \sqrt{(k_v \times E / F_y)}$
Web shear coefficient - eq G2-2	$C_v =$ **1.000**
Nominal shear strength - eq G2-1	$V_{n,x} = 0.6 \times F_y \times A_w \times C_v =$ **95.9** kips
Design shear strength	$V_{c,x} = \phi_v \times V_{n,x} =$ **95.9** kips
	$V_{r,x} / V_{c,x} =$ **0.14**

PASS - Design shear strength exceeds required shear strength

Design of members for flexure - Chapter F

Required flexural strength	$M_{r,x} =$ **124** kips_ft

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	Section Structures				Sheet no./rev. 6	
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Yielding - Section F2.1

Nominal flexural strength for yielding - eq F2-1 $M_{n,yld,x} = M_{p,x} = F_y \times Z_x =$ **179.6** kips_ft

Lateral-torsional buckling - Section F2.2

Unbraced length $L_b = L_{m1\_s1\_seg1\_B} =$ **5.135** ft

Limiting unbraced length for yielding - eq F2-5 $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} =$ **5.369** ft

Distance between flange centroids $h_o =$ **11.9** in

$c =$ **1**

$r_{ts} =$ **1.77** in

Limiting unbraced length for inelastic LTB - eq F2-6 $L_r = 1.95 \times r_{ts} \times E / (0.7 \times F_y) \times \sqrt{((J \times c / (S_x \times h_o)) + \sqrt{((J \times c / (S_x \times h_o))^2 + 6.76 \times (0.7 \times F_y / E)^2)})} =$ **15.604** ft

$L_b <= L_p$ - Lateral-torsional buckling does not apply

Design flexural strength - F1

Nominal flexural strength $M_{n,x} = M_{n,yld,x} =$ **179.6** kips_ft

Design flexural strength $M_{c,x} = \phi_b \times M_{n,x} =$ **161.6** kips_ft

$M_{r,x} / M_{c,x} =$ **0.767**

PASS - Design flexural strength exceeds required flexural strength

Consider Combination 3 - 0.42W (Service)

Check design at end of span

Design of members for x-x axis deflection

Maximum deflection $\delta_x =$ **0.754** in

Allowable deflection $\delta_{x,Allowable} = L_{m1\_s1} / 90 =$ **2.467** in

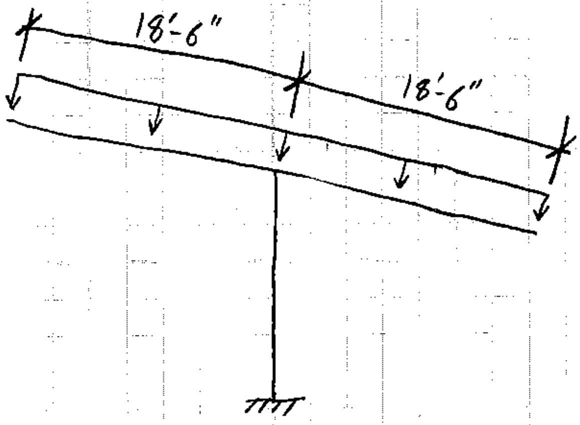
$\delta_x / \delta_{x,Allowable} =$ **0.306**

PASS - Allowable deflection exceeds design deflection

UNITED
STRUCTURAL DESIGN LLC

PROJECT	
ENGINEER	DATE

STRUCTURES 1,2,3,4, + 6 2D ANALYSIS (Double-Cant.)



TRIB = 21'

w_{DL} = 170 PLF

w_{WL} = See Output

See Attached Output

MAXIMUM COLUMN REACTIONS

$P_u = 18.5k$

$V_u = 1.8k$

$M_u = 77.1k\text{-}ft$

USE W12X30, SEE OUTPUT

UNITED STRUCTURAL DESIGN LLC

PROJECT NAME: NSG2 - Colusa
PROJECT LOCATION: 1017 Bridge Street
ENGINEER: DG
REVIEWER:
DATE: 6/24/2016

Design Forces

EXPOSURE C WIND LOADS

Code: 2012 IBC \ ASCE 7-10

Structure Inputs

Trib Width: 21.0 ft

Roof Angle : 5.0 DEG

Fascia Thickness T_1 : 1.0 ft

Wind Load Cases

Wind Flow	Load Case		Wind Direction X = 0 deg		Wind Direction y = 180 deg	
			C_{nw}	C_{nl}	C_{nw}	C_{nl}
Clear Wind Flow	A	p =	18.8 psf	4.7 psf	18.8 psf	4.7 psf
	B	p =	-17.3 psf	-1.6 psf	-17.3 psf	-1.6 psf

WIND 1

W_{nw} = 395.7 plf
W_{nl} = 98.9 plf

WIND 2

W_{nw} = 98.9 plf
W_{nl} = 395.7 plf

WIND 3

W_{nw} = -362.7 plf
W_{nl} = -33.0 plf

WIND 4

W_{nw} = -33.0 plf
W_{nl} = -362.7 plf

WIND 5

W_{nw} = 395.7 plf
W_{nl} = 98.9 plf

WIND 6

W_{nw} = 98.9 plf
W_{nl} = 395.7 plf

WIND 7

W_{nw} = -362.7 plf
W_{nl} = -33.0 plf

WIND 8

W_{nw} = -33.0 plf
W_{nl} = -362.7 plf

Fascia Shear

V_f = 1.0 k

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	Section 2 D Analysis - Double Cantilever				Sheet no./rev. 1	
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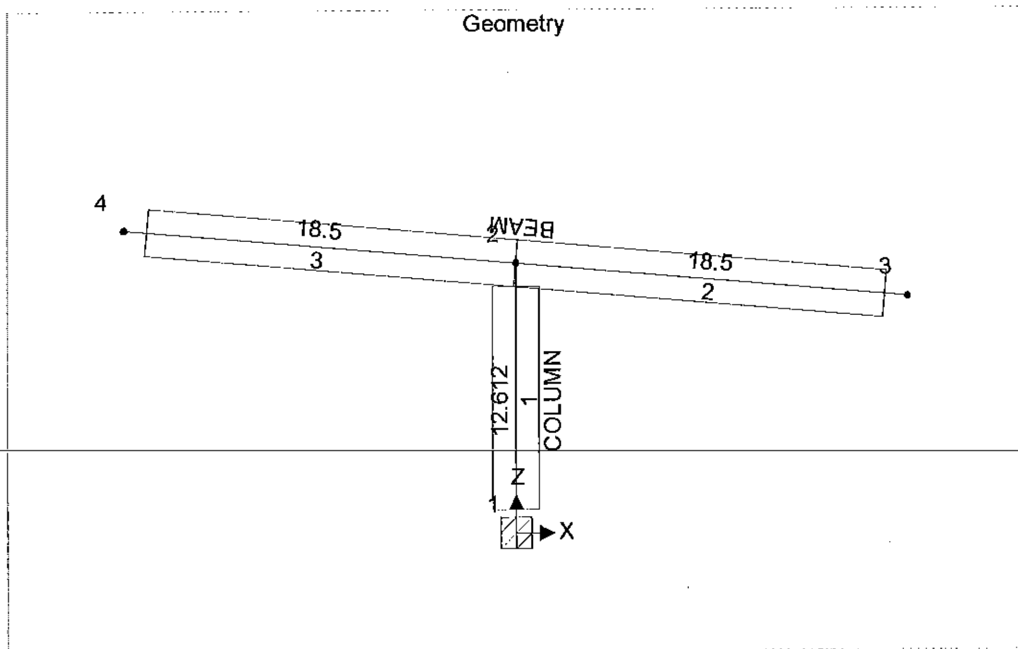
ANALYSIS

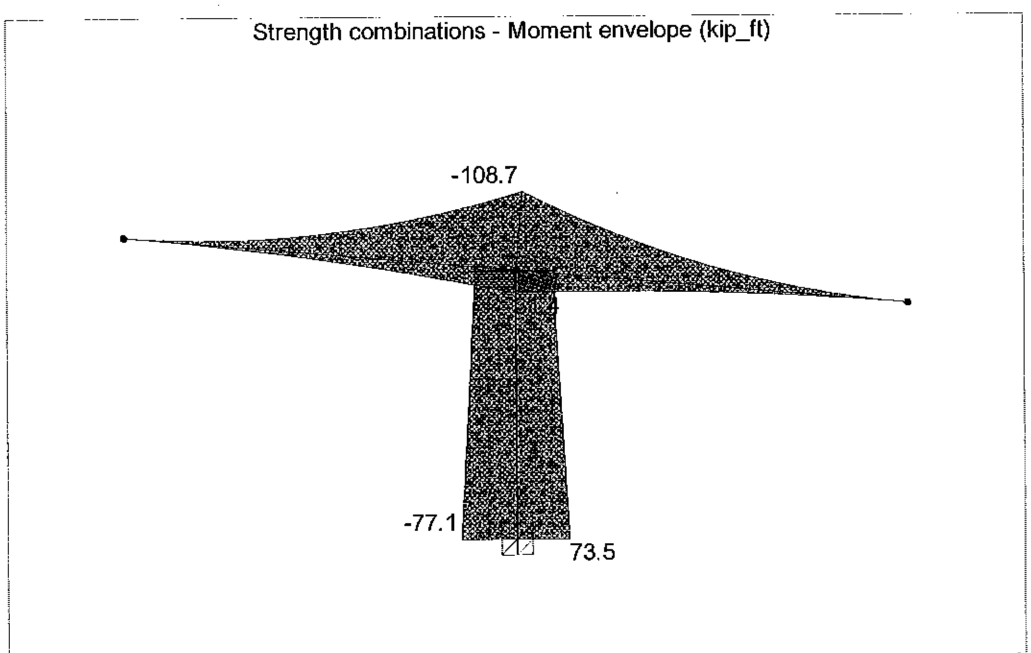
Tedds calculation version 1.0.13

Geometry



Results

Forces



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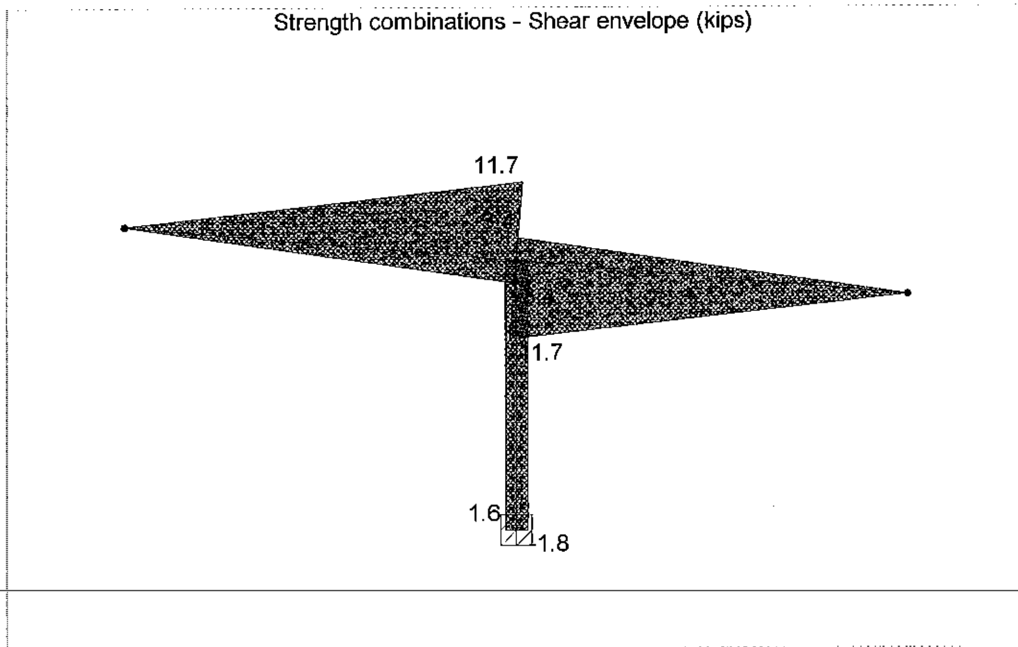
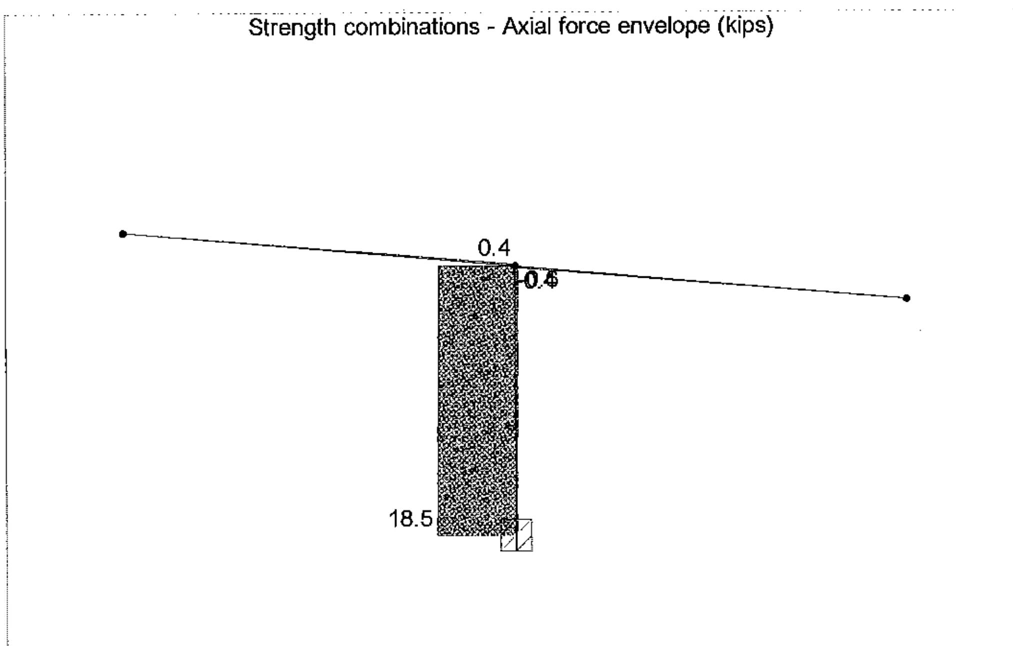




Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
BEAM	18.5	-11.7	18.5	31.4	18.5	-108.7
COLUMN	0	-1.8	0	73.5	0	-77.1

United Structural Design Phoenix, AZ (602) 888-1143	Project NSG2 - Colusa			Job Ref.	
	Section 2 D Analysis - Double Cantilever			Sheet no./rev. 3	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date

Envelope - Strength combinations

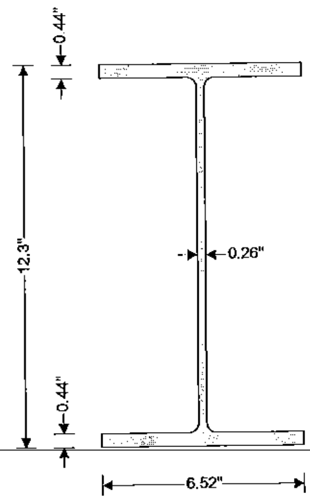
Member	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
BEAM	18.5	0.4	18.5	-0.4
COLUMN	0	18.5	12.61	-0.6

United Structural Design Phoenix, AZ (602) 888-1143	Project NSG2 - Colusa			Job Ref.	
	Section Steel Column - Double Cantilever Structure			Sheet no./rev. 1	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date

STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.06



Column and loading details

Column details

Column section — **W 12x30**

Design loading

Required axial strength — P_r = **19** kips (Compression)

Moment about x axis at end 1 — M_{x1} = **0.0** kips_ft

Moment about x axis at end 2 — M_{x2} = **77.1** kips_ft

Single curvature bending about x axis

Maximum moment about x axis — M_x = max(abs(M_{x1}), abs(M_{x2})) = **77.1** kips_ft

Moment about y axis at end 1 — M_{y1} = **0.0** kips_ft

Moment about y axis at end 2 — M_{y2} = **0.0** kips_ft

Maximum moment about y axis — M_y = max(abs(M_{y1}), abs(M_{y2})) = **0.0** kips_ft

Maximum shear force parallel to y axis — V_{ry} = **1.8** kips

Maximum shear force parallel to x axis — V_{rx} = **0.0** kips

Material details

Steel grade — **A992**

Yield strength — F_y = **50** ksi

Ultimate strength — F_u = **65** ksi

Modulus of elasticity — E = **29000** ksi

Shear modulus of elasticity — G = **11200** ksi

Unbraced lengths

For buckling about x axis — L_x = **150** in

For buckling about y axis — L_y = **150** in

For torsional buckling — L_z = **150** in

United Structural Design Phoenix, AZ (602) 888-1143	Project NSG2 - Colusa			Job Ref.	
	Section Steel Column - Double Cantilever Structure			Sheet no./rev. 2	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date

Effective length factors

For buckling about x axis K_x = **2.00**

For buckling about y axis K_y = **2.00**

For torsional buckling K_z = **1.00**

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2$ = **3.260** in

Width to thickness ratio of flange $\lambda_f = b / t_f$ = **7.409**

Depth between root radii $h = d - 2 \times k$ = **10.820** in

Width to thickness ratio of web $\lambda_w = h / t_w$ = **41.615**

Compression

Limit for nonslender flange $\lambda_{rf\_c} = 0.56 \times \sqrt{(E / F_y)}$ = **13.487**

The flange is nonslender in compression

Limit for nonslender web $\lambda_{rw\_c} = 1.49 \times \sqrt{(E / F_y)}$ = **35.884**

The web is slender in compression

The section is slender in compression

Flexure

Limit for compact flange $\lambda_{pf\_f} = 0.38 \times \sqrt{(E / F_y)}$ = **9.152**

Limit for noncompact flange $\lambda_{rf\_f} = 1.0 \times \sqrt{(E / F_y)}$ = **24.083**

The flange is compact in flexure

Limit for compact web $\lambda_{pw\_f} = 3.76 \times \sqrt{(E / F_y)}$ = **90.553**

Limit for noncompact web $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **137.274**

The web is compact in flexure

The section is compact in flexure

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x$ = **57.6**

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **197.4**

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m $C_{mx} = 0.6 + 0.4 \times M_{x1} / M_{x2}$ = **0.600**

Coefficient α α = **1.0**

Elastic critical buckling stress $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2$ = **3027.6** kips

P-δ amplifier $B_{1x} = \max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x}))$ = **1.000**

Required flexural strength $M_{rx} = B_{1x} \times M_x$ = **77.1** kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m $C_{my} = 0.6 + 0.4 \times M_{y1} / M_{y2}$ = **0.600**

Coefficient α α = **1.0**

Elastic critical buckling stress $P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2$ = **258.2** kips

P-δ amplifier $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y}))$ = **1.000**

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Required flexural strength $M_{ry} = B_{1y} \times M_y =$ **0.0** kips_ft

Shear strength

Shear parallel to the minor axis (cl. G2.1)

Shear area $A_w = d \times t_w =$ **3.198** in^2

Web plate buckling coefficient $k_v =$ **5.0**

Web shear coefficient $C_v =$ **1.000**

Nominal shear strength $V_{ny} = 0.6 \times F_y \times A_w \times C_v =$ **95.9** kips

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear $\phi_v =$ **1.00**

Design shear strength $V_{cy} = \phi_v \times V_{ny} =$ **95.9** kips

PASS - The design shear strength exceeds the required shear strength

Reduction factor for slender elements

Reduction factor for slender unstiffened elements (E7.1)

No slender unstiffened elements therefore $Q_s =$ **1.000**

Reduction factor for slender stiffened elements (E7.2)

Initial reduction factor $Q =$ **1.0**

For flexural buckling about x axis

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (K_x \times L_x / r_x)^2 =$ **86.3** ksi

Flexural buckling stress $F_{crx} = Q \times (0.658^{Q \times Fy/Fex}) \times F_y =$ **39.2** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{crx})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{crx})}])$

$b_e =$ **10.557** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.722** in^2

Reduction factor $Q_{ax} = A_e / A =$ **0.992**

Resultant reduction factor $Q_x = Q_s \times Q_{ax} =$ **0.992**

For flexural buckling about y axis

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (K_y \times L_y / r_y)^2 =$ **7.3** ksi

Flexural buckling stress $F_{cry} = 0.877 \times F_{ey} =$ **6.4** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{cry})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{cry})}])$

$b_e =$ **10.820** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.790** in^2

Reduction factor $Q_{ay} = A_e / A =$ **1.000**

Resultant reduction factor $Q_y = Q_s \times Q_{ay} =$ **1.000**

For torsional/flexural-torsional buckling

Torsional/flexural-torsional elastic buckling stress $F_{et} = [\pi^2 \times E \times C_w / (K_z \times L_z)^2 + G \times J] \times 1 / (I_x + I_y) =$ **55.3** ksi

Flexural buckling stress $F_{crt} = Q \times (0.658^{Q \times Fy/Fet}) \times F_y =$ **34.2** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{crt})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{crt})}])$

$b_e =$ **10.820** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.790** in^2

Reduction factor $Q_{az} = A_e / A =$ **1.000**

Resultant reduction factor $Q_z = Q_s \times Q_{az} =$ **1.000**

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Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2 =$ **86.3** ksi

Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y =$ **39.0** ksi

Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g =$ **342.8** kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2 =$ **7.3** ksi

Flexural buckling stress about y axis $F_{cry} = 0.877 \times F_{ey} =$ **6.4** ksi

Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g =$ **56.6** kips

Torsional and flexural-torsional buckling (cl. E4)

Torsional/flexural-torsional elastic buckling stress $F_{et} = [\pi^2 \times E \times C_w / (K_z \times L_z)^2 + G \times J] \times 1 / (I_x + I_y) =$ **55.3** ksi

Torsional/flexural-torsional buckling stress $F_{crt} = Q_z \times (0.658^{Q_z \times F_y / F_{et}}) \times F_y =$ **34.2** ksi

Nom. torsional/flex-torsional buckling strength $P_{nt} = F_{crt} \times A_g =$ **301.0** kips

Design compressive strength (cl.E1)

Resistance factor for compression $\phi_c =$ **0.90**

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny}, P_{nt}) =$ **51.0** kips

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F2.1)

Nominal flexural strength $M_{nx\_yld} = M_{px} = F_y \times Z_x =$ **179.6** kips_ft

Lateral torsional buckling limiting lengths (cl. F2.2)

Unbraced length $L_b =$ **150.0** in

Limiting unbraced length (yielding) $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} =$ **64.4** in

$L_b > L_p$ - Limit state of lateral torsional buckling applies

Effective radius of gyration $r_{ts} = \sqrt{(\sqrt{(I_y \times C_w)} / S_x)} =$ **1.770** in

Distance between flange centroids $h_o = d - t_f =$ **11.860** in

Factor c $c =$ **1.000**

Limiting unbraced length (inelastic LTB)

$$L_r = 1.95 \times r_{ts} \times E/(0.7 \times F_y) \times \sqrt{(J \times c / (S_x \times h_o))} \times \sqrt{[1 + \sqrt{(1 + 6.76 \times (0.7 \times F_y \times S_x \times h_o / (E \times J \times c))^2)}]}$$

$L_r =$ **187.3** in

Lateral torsional buckling modification factor (cl. F1)

Maximum moment in unbraced segment $M_{max} = M_x =$ **77.10** kips_ft

Moment at centreline of unbraced segment $M_B = abs((M_{x1} + M_{x2}) / 2) =$ **38.55** kips_ft

Moment at ¼ point of unbraced segment $M_A = abs((M_{x1} + M_B) / 2) =$ **19.27** kips_ft

Moment at ¾ point of unbraced segment $M_C = abs((M_{x2} + M_B) / 2) =$ **57.83** kips_ft

Lateral torsional buckling modification factor $C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_C)$

$C_b =$ **1.667**

Lateral torsional buckling (cl. F2.2)

Plastic bending moment $M_{px} = F_y \times Z_x =$ **179.6** kips_ft

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Nominal flexural strength $M_{nx\_ltb} = \min(M_{px}, C_b \times [M_{px} - (M_{px} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)])$

M_{nx_ltb} = **179.6** kips_ft

Design flexural strength about the major axis (cl. F1)

Resistance factor for flexure ϕ_b = **0.90**

Design flexural strength $M_{cx} = \phi_b \times \min(M_{nx\_yld}, M_{nx\_ltb})$ = **161.6** kips_ft

PASS - The design flexural strength about the major axis exceeds the required flexural strength

Combined forces

$M_{ry} / M_{cy} < 0.05$ - Moments exist primarily in one plane therefore check combined forces in accordance with clause H1.3.

In-plane instability (cl. H1.3(a))

Available comp. strength in plane of bending $P_{ci} = \phi_c \times \min(P_{nx}, P_{nt})$ = **270.9** kips

Member utilization (eqn H1-1) $UR_i = P_r / (2 \times P_{ci}) + M_{rx} / M_{cx}$ = **0.511**

Out-of-plane buckling and lateral-torsional buckling (cl. H1.3(b))

Available comp. strength out of plane of bending $P_{cy} = \phi_c \times \min(P_{ny}, P_{nt})$ = **51.0** kips

Available lat-torsional strength (C_b is 1.0) M_{cx_ltb} = **132.9** kip_ft

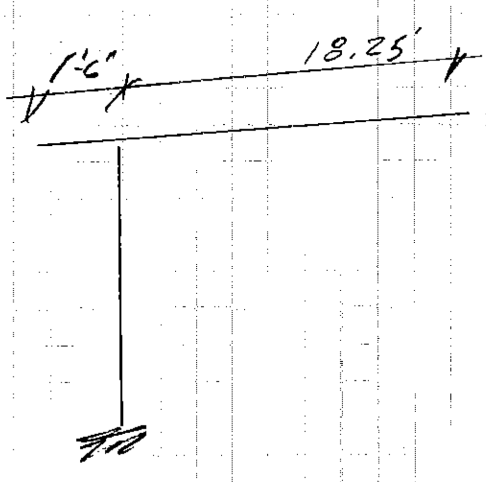
Member utilization (eqn H1-2) $UR_o = P_r / P_{cy} \times (1.5 - 0.5 \times P_r / P_{cy}) + (M_{rx} / (C_b \times M_{cx\_ltb}))^2$ = **0.600**

PASS - The member is adequate for the combined forces

UNITED STRUCTURAL DESIGN LLC

PROJECT	
ENGINEER	DATE

Semi Cant. Structure 2D analysis



TRIB = 16.5'

$W_{DL} = (8psf)(16.5')$

W_W = see output

Column Reactions

$P_u = 9.1k$

$V_u = 1.3k$

$M_u = 97.9 k.ft$

Use W12x30 Column

UNITED
STRUCTURAL DESIGN LLC

PROJECT NAME: NSG2 - Colusa
PROJECT LOCATION: 1017 Bridge Street
ENGINEER: DG
REVIEWER:
DATE: 6/24/2016

Design Forces

EXPOSURE C WIND LOADS

Code: 2012 IBC \ ASCE 7-10

Structure Inputs

Trib Width: 16.5 ft

Fascia Thickness T_1: 1.0 ft

Roof Angle: 5.0 DEG

Wind Load Cases

Wind Flow	Load Case		Wind Direction		Wind Direction	
			γ = 0 deg		γ = 180 deg	
			C_{nw}	C_{nl}	C_{nw}	C_{nl}
Clear Wind Flow	A	p =	18.8 psf	4.7 psf	18.8 psf	4.7 psf
	B	p =	-17.3 psf	-1.6 psf	-17.3 psf	-1.6 psf

WIND 1

W_{nw} = 310.9 plf

W_{nl} = 77.7 plf

WIND 5

W_{nw} = 310.9 plf

W_{nl} = 77.7 plf

Fascia Shear

V_f = 0.8 k

WIND 2

W_{nw} = 77.7 plf

W_{nl} = 310.9 plf

WIND 6

W_{nw} = 77.7 plf

W_{nl} = 310.9 plf

WIND 3

W_{nw} = -285.0 plf

W_{nl} = -25.9 plf

WIND 7

W_{nw} = -285.0 plf

W_{nl} = -25.9 plf

WIND 4

W_{nw} = -25.9 plf

W_{nl} = -285.0 plf

WIND 8

W_{nw} = -25.9 plf

W_{nl} = -285.0 plf

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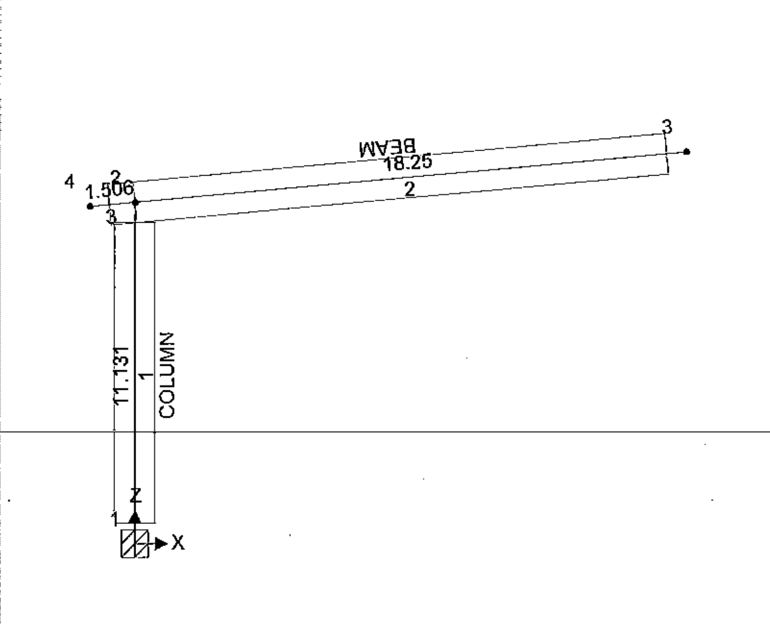
ANALYSIS

Tedds calculation version 1.0.13

Geometry

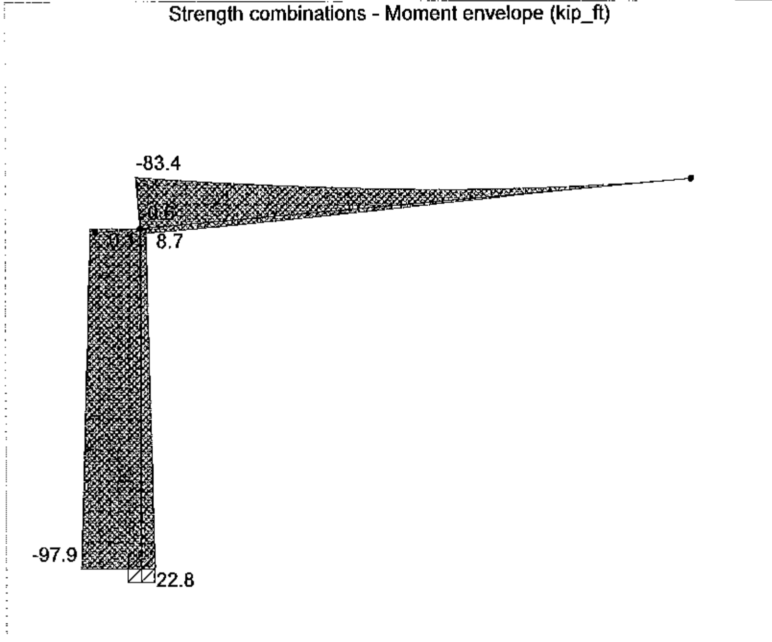
Geometry



Results

Forces

Strength combinations - Moment envelope (kip_ft)



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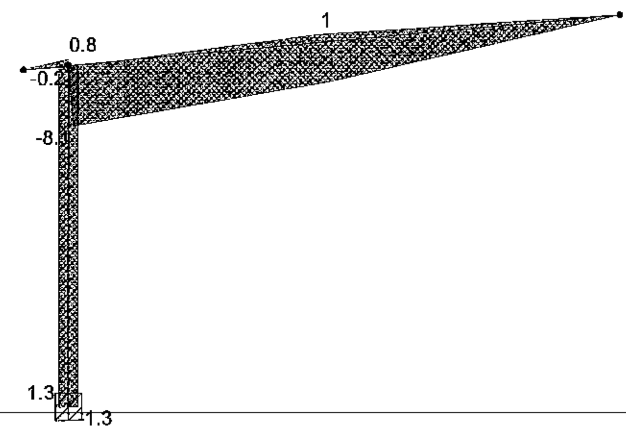
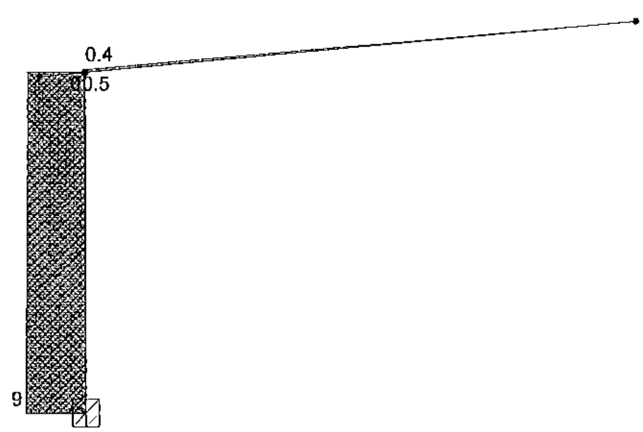
Strength combinations - Shear envelope (kips)



Strength combinations - Axial force envelope (kips)



Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
BEAM	18.25	-8.1	16.69	8.7	18.25	-83.4
COLUMN	0	1.3	0	22.8	0	-97.9

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Envelope - Strength combinations

Member	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
BEAM	18.25	0.4	18.25	0
COLUMN	0	9	11.13	0.5

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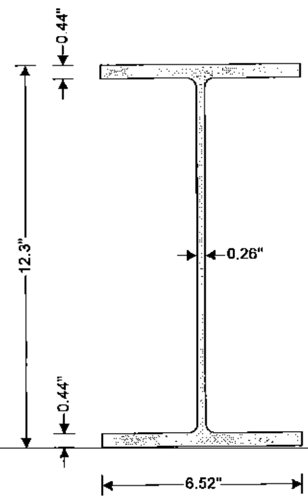
STEEL COLUMN DESIGN

In accordance with AISC360-10 and the LRFD method

Tedds calculation version 1.0.05



Column and loading details

Column details

Column section — **W 12x30**

Design loading

Required axial strength — P_r = **9** kips (Compression)

Moment about x axis at end 1 — M_{x1} = **0.0** kips_ft

Moment about x axis at end 2 — M_{x2} = **97.9** kips_ft

Single curvature bending about x axis

Maximum moment about x axis — M_x = max(abs(M_{x1}), abs(M_{x2})) = **97.9** kips_ft

Moment about y axis at end 1 — M_{y1} = **0.0** kips_ft

Moment about y axis at end 2 — M_{y2} = **0.0** kips_ft

Maximum moment about y axis — M_y = max(abs(M_{y1}), abs(M_{y2})) = **0.0** kips_ft

Maximum shear force parallel to y axis — V_{ry} = **1.3** kips

Maximum shear force parallel to x axis — V_{rx} = **0.0** kips

Material details

Steel grade — **A992**

Yield strength — F_y = **50** ksi

Ultimate strength — F_u = **65** ksi

Modulus of elasticity — E = **29000** ksi

Shear modulus of elasticity — G = **11200** ksi

Unbraced lengths

For buckling about x axis — L_x = **150** in

For buckling about y axis — L_y = **150** in

For torsional buckling — L_z = **150** in

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Effective length factors

For buckling about x axis K_x = **2.00**

For buckling about y axis K_y = **2.00**

For torsional buckling K_z = **1.00**

Section classification

Section classification for local buckling (cl. B4)

Critical flange width $b = b_f / 2$ = **3.260** in

Width to thickness ratio of flange $\lambda_f = b / t_f$ = **7.409**

Depth between root radii $h = d - 2 \times k$ = **10.820** in

Width to thickness ratio of web $\lambda_w = h / t_w$ = **41.615**

Compression

Limit for nonslender flange $\lambda_{rf\_c} = 0.56 \times \sqrt{(E / F_y)}$ = **13.487**

The flange is nonslender in compression

Limit for nonslender web $\lambda_{rw\_c} = 1.49 \times \sqrt{(E / F_y)}$ = **35.884**

The web is slender in compression

The section is slender in compression

Flexure

Limit for compact flange $\lambda_{pf\_f} = 0.38 \times \sqrt{(E / F_y)}$ = **9.152**

Limit for noncompact flange $\lambda_{rf\_f} = 1.0 \times \sqrt{(E / F_y)}$ = **24.083**

The flange is compact in flexure

Limit for compact web $\lambda_{pw\_f} = 3.76 \times \sqrt{(E / F_y)}$ = **90.553**

Limit for noncompact web $\lambda_{rw\_f} = 5.70 \times \sqrt{(E / F_y)}$ = **137.274**

The web is compact in flexure

The section is compact in flexure

Slenderness

Member slenderness

Slenderness ratio about x axis $SR_x = K_x \times L_x / r_x$ = **57.6**

Slenderness ratio about y axis $SR_y = K_y \times L_y / r_y$ = **197.4**

Second order effects

Second order effects for bending about x axis (cl. App 8.1)

Coefficient C_m $C_{mx} = 0.6 + 0.4 \times M_{x1} / M_{x2}$ = **0.600**

Coefficient α α = **1.0**

Elastic critical buckling stress $P_{e1x} = \pi^2 \times E \times I_x / (K_{1x} \times L_x)^2$ = **3027.6** kips

P-δ amplifier $B_{1x} = \max(1.0, C_{mx} / (1 - \alpha \times P_r / P_{e1x}))$ = **1.000**

Required flexural strength $M_{rx} = B_{1x} \times M_x$ = **97.9** kips_ft

Second order effects for bending about y axis (cl. App 8.1)

Coefficient C_m $C_{my} = 0.6 + 0.4 \times M_{y1} / M_{y2}$ = **0.600**

Coefficient α α = **1.0**

Elastic critical buckling stress $P_{e1y} = \pi^2 \times E \times I_y / (K_{1y} \times L_y)^2$ = **258.2** kips

P-δ amplifier $B_{1y} = \max(1.0, C_{my} / (1 - \alpha \times P_r / P_{e1y}))$ = **1.000**

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Required flexural strength $M_{ry} = B_{1y} \times M_y =$ **0.0** kips_ft

Shear strength

Shear parallel to the minor axis (cl. G2.1)

Shear area $A_w = d \times t_w =$ **3.198** in^2

Web plate buckling coefficient $k_v =$ **5.0**

Web shear coefficient $C_v =$ **1.000**

Nominal shear strength $V_{ny} = 0.6 \times F_y \times A_w \times C_v =$ **95.9** kips

Design shear strength (cl.G1 & G2.1(a))

Resistance factor for shear $\phi_v =$ **1.00**

Design shear strength $V_{cy} = \phi_v \times V_{ny} =$ **95.9** kips

PASS - The design shear strength exceeds the required shear strength

Reduction factor for slender elements

Reduction factor for slender unstiffened elements (E7.1)

No slender unstiffened elements therefore $Q_s =$ **1.000**

Reduction factor for slender stiffened elements (E7.2)

Initial reduction factor $Q =$ **1.0**

For flexural buckling about x axis

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (K_x \times L_x / r_x)^2 =$ **86.3** ksi

Flexural buckling stress $F_{crx} = Q \times (0.658^{Q \times Fy/Fex}) \times F_y =$ **39.2** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{crx})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{crx})}])$

$b_e =$ **10.557** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.722** in^2

Reduction factor $Q_{ax} = A_e / A =$ **0.992**

Resultant reduction factor $Q_x = Q_s \times Q_{ax} =$ **0.992**

For flexural buckling about y axis

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (K_y \times L_y / r_y)^2 =$ **7.3** ksi

Flexural buckling stress $F_{cry} = 0.877 \times F_{ey} =$ **6.4** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{cry})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{cry})}])$

$b_e =$ **10.820** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.790** in^2

Reduction factor $Q_{ay} = A_e / A =$ **1.000**

Resultant reduction factor $Q_y = Q_s \times Q_{ay} =$ **1.000**

For torsional/flexural-torsional buckling

Torsional/flexural-torsional elastic buckling stress $F_{et} = [\pi^2 \times E \times C_w / (K_z \times L_z)^2 + G \times J] \times 1 / (I_x + I_y) =$ **55.3** ksi

Flexural buckling stress $F_{crt} = Q \times (0.658^{Q \times Fy/Fet}) \times F_y =$ **34.2** ksi

Effective web width $b_e = \min(h, 1.92 \times t_w \times \sqrt{(E / F_{crt})} \times [1 - 0.34 / (h / t_w) \times \sqrt{(E / F_{crt})}])$

$b_e =$ **10.820** in

Effective area $A_e = A - t_w \times (h - b_e) =$ **8.790** in^2

Reduction factor $Q_{az} = A_e / A =$ **1.000**

Resultant reduction factor $Q_z = Q_s \times Q_{az} =$ **1.000**

Compressive strength

Flexural buckling about x axis (cl. E3)

Elastic critical buckling stress $F_{ex} = (\pi^2 \times E) / (SR_x)^2 = \mathbf{86.3}$ ksi

Flexural buckling stress about x axis $F_{crx} = Q_x \times (0.658^{Q_x \times F_y / F_{ex}}) \times F_y = \mathbf{39.0}$ ksi

Nominal flexural buckling strength $P_{nx} = F_{crx} \times A_g = \mathbf{342.8}$ kips

Flexural buckling about y axis (cl. E3)

Elastic critical buckling stress $F_{ey} = (\pi^2 \times E) / (SR_y)^2 = \mathbf{7.3}$ ksi

Flexural buckling stress about y axis $F_{cry} = 0.877 \times F_{ey} = \mathbf{6.4}$ ksi

Nominal flexural buckling strength $P_{ny} = F_{cry} \times A_g = \mathbf{56.6}$ kips

Torsional and flexural-torsional buckling (cl. E4)

Torsional/flexural-torsional elastic buckling stress $F_{et} = [\pi^2 \times E \times C_w / (K_z \times L_z)^2 + G \times J] \times 1 / (I_x + I_y) = \mathbf{55.3}$ ksi

Torsional/flexural-torsional buckling stress $F_{crt} = Q_z \times (0.658^{Q_z \times F_y / F_{et}}) \times F_y = \mathbf{34.2}$ ksi

Nom. torsional/flex-torsional buckling strength $P_{nt} = F_{crt} \times A_g = \mathbf{301.0}$ kips

Design compressive strength (cl.E1)

Resistance factor for compression $\phi_c = \mathbf{0.90}$

Design compressive strength $P_c = \phi_c \times \min(P_{nx}, P_{ny}, P_{nt}) = \mathbf{51.0}$ kips

PASS - The design compressive strength exceeds the required compressive strength

Flexural strength about the major axis

Yielding (cl. F2.1)

Nominal flexural strength $M_{nx\_yld} = M_{px} = F_y \times Z_x = \mathbf{179.6}$ kips_ft

Lateral torsional buckling limiting lengths (cl. F2.2)

Unbraced length $L_b = \mathbf{150.0}$ in

Limiting unbraced length (yielding) $L_p = 1.76 \times r_y \times \sqrt{(E / F_y)} = \mathbf{64.4}$ in

$L_b > L_p$ - Limit state of lateral torsional buckling applies

Effective radius of gyration $r_{ts} = \sqrt{(\sqrt{(I_y \times C_w)} / S_x)} = \mathbf{1.770}$ in

Distance between flange centroids $h_o = d - t_f = \mathbf{11.860}$ in

Factor c $c = \mathbf{1.000}$

Limiting unbraced length (inelastic LTB)

$$L_r = 1.95 \times r_{ts} \times E/(0.7 \times F_y) \times \sqrt{(J \times c / (S_x \times h_o))} \times \sqrt{[1 + \sqrt{(1 + 6.76 \times (0.7 \times F_y \times S_x \times h_o / (E \times J \times c))^2)}]}$$

$L_r = \mathbf{187.3}$ in

Lateral torsional buckling modification factor (cl. F1)

Maximum moment in unbraced segment $M_{max} = M_x = \mathbf{97.90}$ kips_ft

Moment at centreline of unbraced segment $M_B = abs((M_{x1} + M_{x2}) / 2) = \mathbf{48.95}$ kips_ft

Moment at ¼ point of unbraced segment $M_A = abs((M_{x1} + M_B) / 2) = \mathbf{24.48}$ kips_ft

Moment at ¾ point of unbraced segment $M_C = abs((M_{x2} + M_B) / 2) = \mathbf{73.43}$ kips_ft

Lateral torsional buckling modification factor $C_b = 12.5 \times M_{max} / (2.5 \times M_{max} + 3 \times M_A + 4 \times M_B + 3 \times M_C)$

$C_b = \mathbf{1.667}$

Lateral torsional buckling (cl. F2.2)

Plastic bending moment $M_{px} = F_y \times Z_x = \mathbf{179.6}$ kips_ft

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Nominal flexural strength $M_{nx\_ltb} = \min(M_{px}, C_b \times [M_{px} - (M_{px} - 0.7 \times F_y \times S_x) \times (L_b - L_p) / (L_r - L_p)])$

M_{nx_ltb} = **179.6** kips_ft

Design flexural strength about the major axis (cl. F1)

Resistance factor for flexure ϕ_b = **0.90**

Design flexural strength $M_{cx} = \phi_b \times \min(M_{nx\_yld}, M_{nx\_ltb})$ = **161.6** kips_ft

PASS - The design flexural strength about the major axis exceeds the required flexural strength

Combined forces

$M_{ry} / M_{cy} < 0.05$ - Moments exist primarily in one plane therefore check combined forces in accordance with clause H1.3.

In-plane instability (cl. H1.3(a))

Available comp. strength in plane of bending $P_{ci} = \phi_c \times \min(P_{nx}, P_{nt})$ = **270.9** kips

Member utilization (eqn H1-1) $UR_i = P_r / (2 \times P_{ci}) + M_{rx} / M_{cx}$ = **0.622**

Out-of-plane buckling and lateral-torsional buckling (cl. H1.3(b))

Available comp. strength out of plane of bending $P_{cy} = \phi_c \times \min(P_{ny}, P_{nt})$ = **51.0** kips

Available lat-torsional strength (C_b is 1.0) M_{cx_ltb} = **132.9** kip_ft

Member utilization (eqn H1-2) $UR_o = P_r / P_{cy} \times (1.5 - 0.5 \times P_r / P_{cy}) + (M_{rx} / (C_b \times M_{cx\_ltb}))^2$ = **0.445**

PASS - The member is adequate for the combined forces

UNITED STRUCTURAL DESIGN LLC

PROJECT	
ENGINEER	DATE

FOUNDATION

STRUCTURES 1,2,3,4,+6 (Double-Cant.)

Reactions From 2D Analysis

$P_{max} = 13.2k$

$V_{max} = 1.1k$

$M_{Max} = 46.3k \cdot ft$

Soil Value

Allowable Bearing = 1,500 PSF

Passive Pressure = 100 PSF

See Attached Calc

USE 24" DIA. X 11'-0"

STRUCTURE 5 (Semi-Cant.)

Reactions From 2D Analysis

$P_{max} = 6.6k$

$V_{max} = .8k$

$M_{max} = 68k \cdot ft$

See Attached Calc

USE 24" DIA. X 11'-6"

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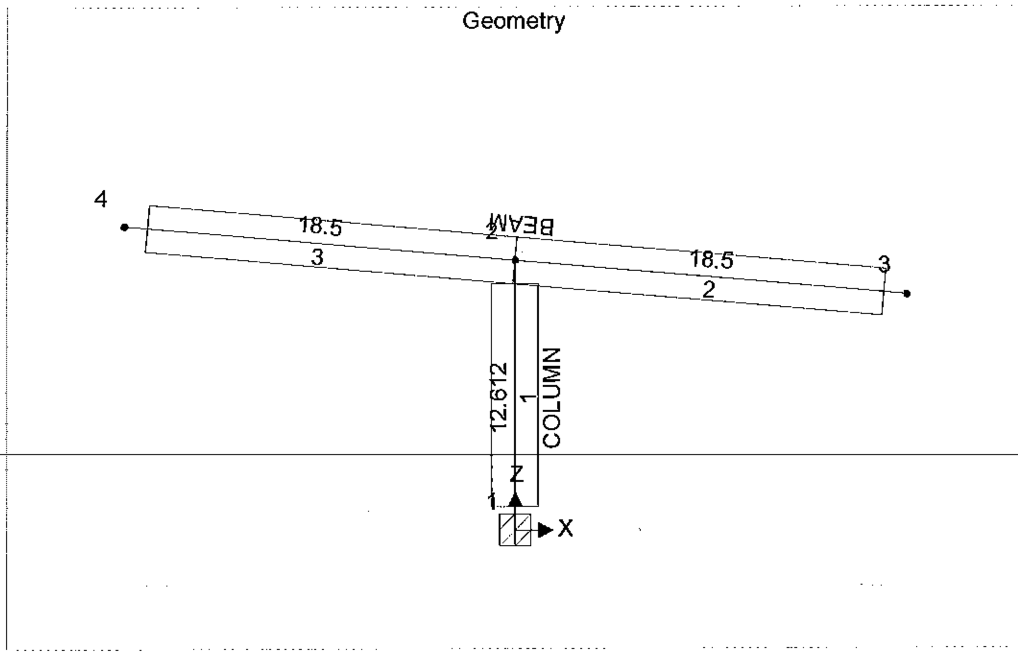
ANALYSIS

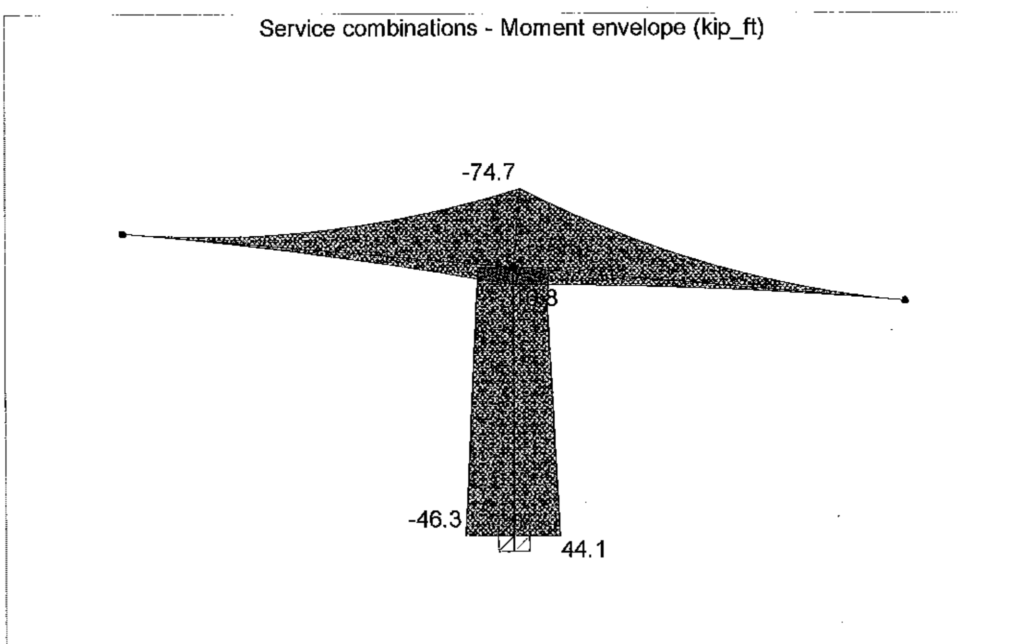
Tedds calculation version 1.0.13

Geometry



Results

Forces



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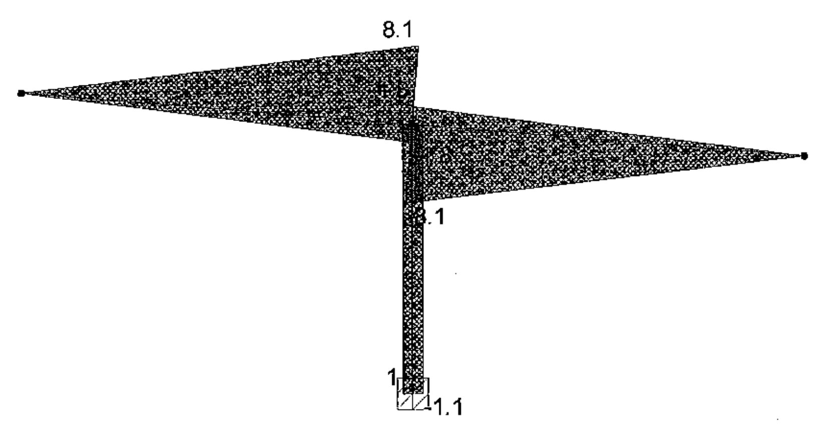
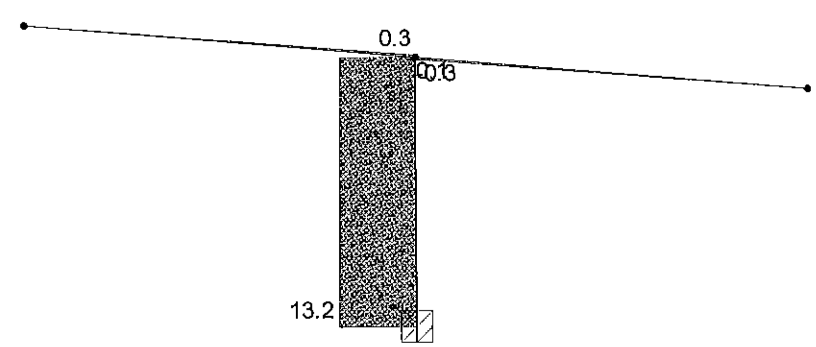
Service combinations - Shear envelope (kips)



Service combinations - Axial force envelope (kips)



Member results

Envelope - Service combinations

Member	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
BEAM	18.5	-8.1	18.5	16.8	18.5	-74.7
COLUMN	0	-1.1	0	44.1	0	-46.3

Envelope - Service combinations

Member	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
BEAM	18.5	0.3	18.5	-0.3
COLUMN	0	13.2	12.61	0.1

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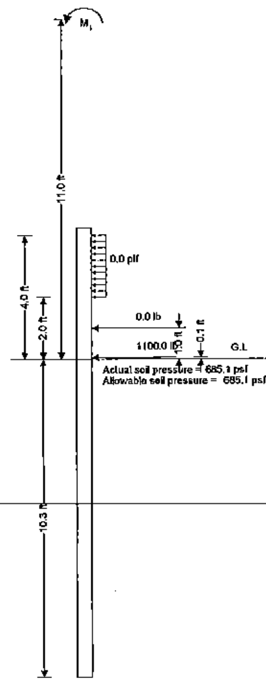
FLAGPOLE EMBEDMENT (IBC 2012)

TEDDS calculation version 1.1.01



Soil capacity data

Allowable passive pressure	L_{sbc} = **100** pcf
Maximum allowable passive pressure	P_{max} = **1500** psf
Load factor 1 (1806.1)	LDF_1 = **1.00**
Load factor 2 (1806.3.4)	LDF_2 = **2.0**

Pole geometry

Shape of the pole	**Round**
Diameter of the pole	Dia = **24** in
Laterally restrained	**No**

Load data

First point load	P_1 = **1100** lbs
Distance of P_1 from ground surface	H_1 = **0.1** ft
Second point load	P_2 = **0** lbs
Distance of P_2 from ground surface	H_2 = **1** ft
Uniformly distributed load	W = **0** plf
Start distance of W from ground surface	a = **2** ft
End distance of W from ground surface	a_1 = **4** ft
Applied moment	M_1 = **46300** lb_ft
Distance of M_1 from ground surface	H_3 = **11** ft

Shear force and bending moment

Total shear force	$F = P_1 + P_2 + W \times (a_1 - a)$ = **1100** lbs

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Total bending moment at grade — $M_g = P_1 \times H_1 + P_2 \times H_2 + W \times (a_1 - a) \times (a + a_1) / 2 + M_1 =$ **46355** lb_ft

Distance of resultant lateral force — $h = abs(M_g / F) =$ **42.14** ft

Embedment depth (1807.3.2.1)

Embedment depth provided — $D =$ **10.28** ft

Allowable lateral passive pressure — $S_1 = min(P_{max}, L_{sbc} \times min(D, 12\ ft) / 3) \times LDF_1 \times LDF_2 =$ **685.1** psf

Factor A — $A = 2.34 \times abs(F) / (S_1 \times Dia) =$ **1.9** ft

Embedment depth required — $D_1 = 0.5 \times A \times (1 + (1 + ((4.36 \times h) / A))^{0.5}) =$ **10.28** ft

Actual lateral passive pressure — $S_2 = (2.34 \times abs(F) \times ((4.36 \times h) + (4 \times D))) / (4 \times D^2 \times Dia) =$ **685.1** psf

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FOUNDATION ANALYSIS & DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Tedds calculation version 3.0.02

FOOTING ANALYSIS

Length of foundation L_x = **11** ft

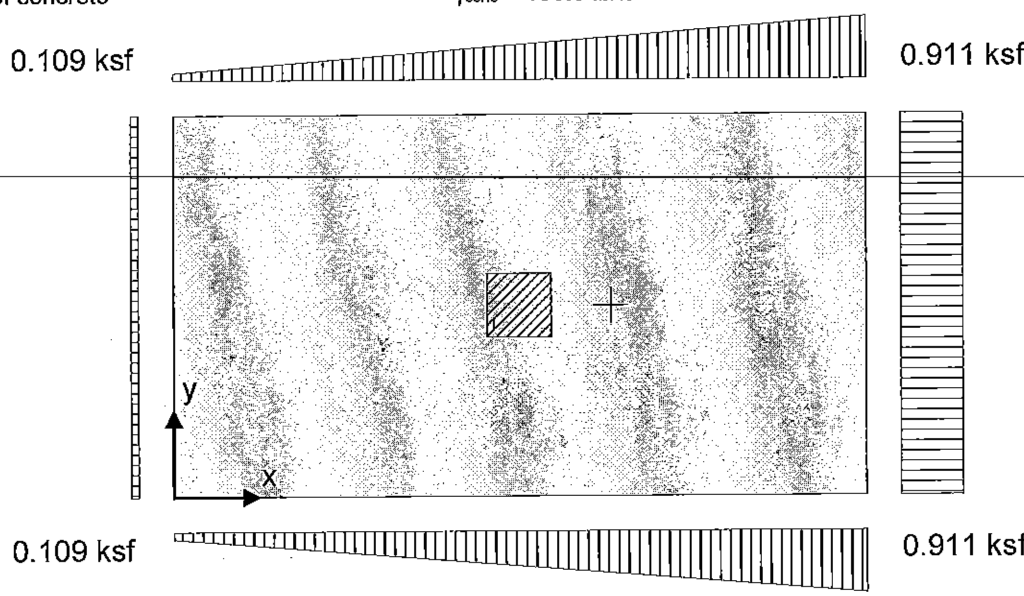
Width of foundation L_y = **6** ft

Foundation area $A = L_x \times L_y$ = **66** ft^2

Depth of foundation h = **24** in

Depth of soil over foundation h_{soil} = **1** in

Density of concrete γ_{conc} = **150.0** lb/ft^3



Column no.1 details

Length of column l_{x1} = **12.00** in

Width of column l_{y1} = **12.00** in

position in x-axis x_1 = **66.00** in

position in y-axis y_1 = **36.00** in

Soil properties

Gross allowable bearing pressure q_{allow_Gross} = **1.5** ksf

Density of soil γ_{soil} = **120.0** lb/ft^3

Angle of internal friction ϕ_b = **30.0** deg

Design base friction angle δ_{bb} = **30.0** deg

Coefficient of base friction $\tan(\delta_{bb})$ = **0.577**

Foundation loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **300** psf

Soil weight $F_{soil} = h_{soil} \times \gamma_{soil}$ = **10** psf

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Column no.1 loads

Dead load in z — F_{Dz1} = **13.2** kips

Dead load in x — F_{Dx1} = **1.1** kips

Dead load moment in x — M_{Dx1} = **46.3** kip_ft

Footing analysis for soil and stability

Load combinations per ASCE 7-10

1.0D (0.607)

Combination 1 results: 1.0D

Forces on foundation

Force in x-axis — $F_{dx} = \gamma_D \times F_{Dx1}$ = **1.1** kips

Force in z-axis — $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1}$ = **33.7** kips

Moments on foundation

Moment in x-axis, about x is 0 — $M_{dx} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h)$ = **233.6** kip_ft

Moment in y-axis, about y is 0 — $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1)$ = **101.0** kip_ft

Uplift verification

Vertical force — F_{dz} = **33.66** kips

PASS - Foundation is not subject to uplift

Stability against overturning in x direction, moment about x is L_x

Overturning moment — $M_{OTxL} = \gamma_D \times (M_{Dx1} + F_{Dx1} \times h)$ = **48.5** kip_ft

Resisting moment — $M_{RxL} = -1 \times (\gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2)) + \gamma_D \times (F_{Dz1} \times (x_1 - L_x))$ = **-185.13** kip_ft

Factor of safety — $abs(M_{RxL} / M_{OTxL})$ = **3.817**

PASS - Overturning moment safety factor exceeds the minimum of 1.00

Stability against sliding

Resistance due to base friction — $F_{RFriction} = max(F_{dz}, 0\ kN) \times tan(\delta_{bb})$ = **19.434** kips

Stability against sliding in x direction

Total sliding resistance — $F_{Rx} = F_{RFriction}$ = **19.434** kips

Factor of safety — $abs(F_{Rx} / F_{dx})$ = **17.67**

PASS - Sliding factor of safety exceeds the minimum of 1.00

Bearing resistance

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{dx} = M_{dx} / F_{dz} - L_x / 2$ = **17.291** in

Eccentricity of base reaction in y-axis — $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0** in

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.109** ksf

$q_2 = F_{dz} \times (1 - 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.109** ksf

$q_3 = F_{dz} \times (1 + 6 \times e_{dx} / L_x - 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.911** ksf

$q_4 = F_{dz} \times (1 + 6 \times e_{dx} / L_x + 6 \times e_{dy} / L_y) / (L_x \times L_y)$ = **0.911** ksf

Minimum base pressure — $q_{min} = min(q_1, q_2, q_3, q_4)$ = **0.109** ksf

Maximum base pressure — $q_{max} = max(q_1, q_2, q_3, q_4)$ = **0.911** ksf

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Allowable bearing capacity

Allowable bearing capacity — $q_{allow} = q_{allow\_Gross} =$ **1.5** ksf

$q_{max} / q_{allow} =$ **0.607**

PASS - Allowable bearing capacity exceeds design base pressure

FOOTING DESIGN (ACI318)

In accordance with ACI318-11 incorporating Errata as of August 8, 2014

Material details

Compressive strength of concrete — $f'_c =$ **2500** psi

Yield strength of reinforcement — $f_y =$ **60000** psi

Cover to reinforcement — $c_{nom} =$ **3** in

Concrete type — Normal weight

Concrete modification factor — $\lambda =$ **1.00**

Column type — Concrete

Analysis and design of concrete footing

Load combinations per ASCE 7-10

1.4D (0.042)

Combination 1 results: 1.4D

Forces on foundation

Ultimate force in x-axis — $F_{ux} = \gamma_D \times F_{Dx1} =$ **1.5** kips

Ultimate force in z-axis — $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} =$ **47.1** kips

Moments on foundation

Ultimate moment in x-axis, about x is 0 — $M_{ux} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_x / 2) + \gamma_D \times (F_{Dz1} \times x_1 + M_{Dx1} + F_{Dx1} \times h) =$ **327.1** kip_ft

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) =$ **141.4** kip_ft

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_{ux} = M_{ux} / F_{uz} - L_x / 2 =$ **17.291** in

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2 =$ **0** in

Pad base pressures

$q_{u1} = F_{uz} \times (1 - 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.153** ksf

$q_{u2} = F_{uz} \times (1 - 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **0.153** ksf

$q_{u3} = F_{uz} \times (1 + 6 \times e_{ux} / L_x - 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.275** ksf

$q_{u4} = F_{uz} \times (1 + 6 \times e_{ux} / L_x + 6 \times e_{uy} / L_y) / (L_x \times L_y) =$ **1.275** ksf

Minimum ultimate base pressure — $q_{umin} = \min(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **0.153** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1}, q_{u2}, q_{u3}, q_{u4}) =$ **1.275** ksf

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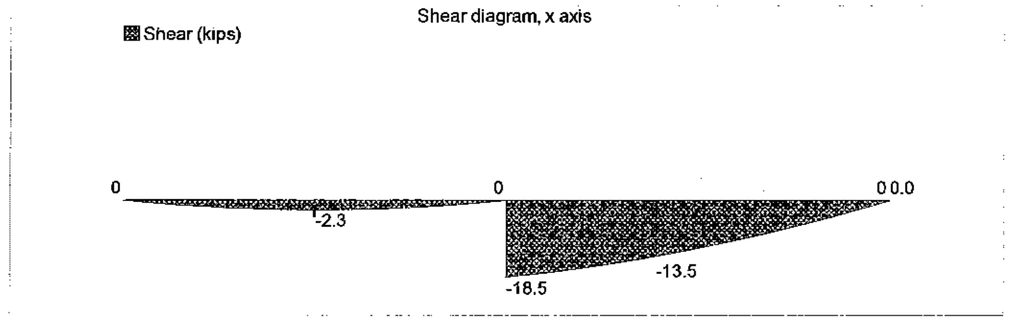

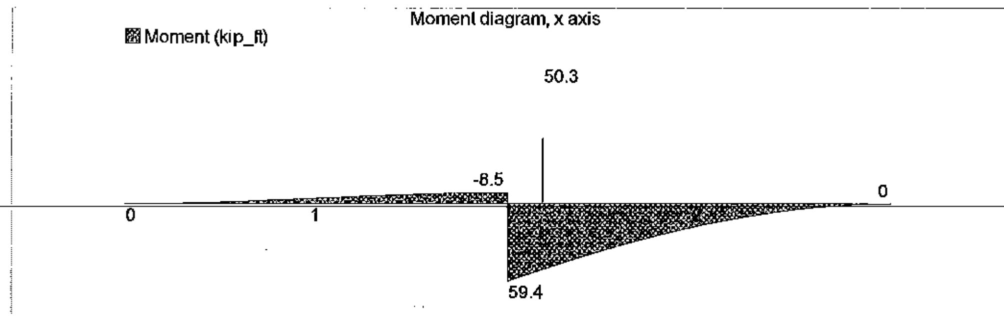

Moment design, x direction, positive moment

Ultimate bending moment	$M_{u.x.max}$ = **50.333** kip_ft
Tension reinforcement provided	8 No.6 bottom bars (9.3 in c/c)
Area of tension reinforcement provided	$A_{sx.bot.prov}$ = **3.52** in^2
Minimum area of reinforcement (10.5.4)	$A_{s.min} = 0.0018 \times L_y \times h$ = **3.11** in^2
	PASS - Area of reinforcement provided exceeds minimum
Maximum spacing of reinforcement (10.5.4)	s_{max} = min(3 × h, 18 in) = **18** in
	PASS - Maximum permissible reinforcement spacing exceeds actual spacing
Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.bot} / 2$ = **20.625** in
Depth of compression block	$a = A_{sx.bot.prov} \times f_y / (0.85 \times f'_c \times L_y)$ = **1.380** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.624** in
Strain in tensile reinforcement (10.3.5)	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03510**
	PASS - Tensile strain exceeds minimum required, 0.004
Nominal moment capacity	$M_n = A_{sx.bot.prov} \times f_y \times (d - a / 2)$ = **350.853** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **315.767** kip_ft
	$M_{u.x.max} / \phi M_n$ = **0.159**
	PASS - Design moment capacity exceeds ultimate moment load

Moment design, x direction, negative moment

Ultimate bending moment	$M_{u.x.min}$ = **-8.333** kip_ft
Tension reinforcement provided	8 No.6 top bars (9.3 in c/c)
Area of tension reinforcement provided	$A_{sx.top.prov}$ = **3.52** in^2

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Minimum area of reinforcement (10.5.4) $A_{s.min} = 0.0018 \times L_y \times h =$ **3.11** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (10.5.4) $s_{max} = \min(3 \times h, 18\text{ in}) =$ **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement $d = h - c_{nom} - \phi_{x.top} / 2 =$ **20.625** in

Depth of compression block $a = A_{sx.top.prov} \times f_y / (0.85 \times f'_c \times L_y) =$ **1.380** in

Neutral axis factor $\beta_1 =$ **0.85**

Depth to neutral axis $c = a / \beta_1 =$ **1.624** in

Strain in tensile reinforcement (10.3.5) $\varepsilon_t = 0.003 \times d / c - 0.003 =$ **0.03510**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity $M_n = A_{sx.top.prov} \times f_y \times (d - a / 2) =$ **350.853** kip_ft

Flexural strength reduction factor $\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9) =$ **0.900**

Design moment capacity $\phi M_n = \phi_f \times M_n =$ **315.767** kip_ft

$\text{abs}(M_{u.x.min}) / \phi M_n =$ **0.026**

PASS - Design moment capacity exceeds ultimate moment load

One-way shear design, x direction

Ultimate shear force $V_{u.x} =$ **13.453** kips

Depth to reinforcement $d_v = \min(h - c_{nom} - \phi_{x.bot} / 2, h - c_{nom} - \phi_{x.top} / 2) =$ **20.625** in

Shear strength reduction factor $\phi_v =$ **0.75**

Nominal shear capacity (Eq. 11-3) $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} \times L_y \times d_v =$ **148.5** kips

Design shear capacity $\phi V_n = \phi_v \times V_n =$ **111.375** kips

$V_{u.x} / \phi V_n =$ **0.121**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement $d_{v2} =$ **20.25** in

Shear perimeter length (11.11.1.2) $l_{xp} =$ **32.250** in

Shear perimeter width (11.11.1.2) $l_{yp} =$ **32.250** in

Shear perimeter (11.11.1.2) $b_o = 2 \times l_{xp} + 2 \times l_{yp} =$ **129.000** in

Shear area $A_p = l_{xp} \times l_{yp} =$ **1040.063** in^2

Surcharge loaded area $A_{sur} = A_p - l_{x1} \times l_{y1} =$ **896.063** in^2

Ultimate bearing pressure at center of shear area $q_{up.avg} =$ **0.714** ksf

Ultimate shear load $F_{up} = \gamma_D \times F_{Dz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p =$ **16.444** kips

Ultimate shear stress from vertical load $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\text{ psi}) =$ **6.295** psi

Column geometry factor (11.11.2.1) $\beta = l_{y1} / l_{x1} =$ **1.00**

Column location factor (11.11.2.1) $\alpha_s =$ **40**

Concrete shear strength (11.11.2.1) $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **300.000** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **413.953** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\text{ psi})} =$ **200.000** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc}) =$ **200.000** psi

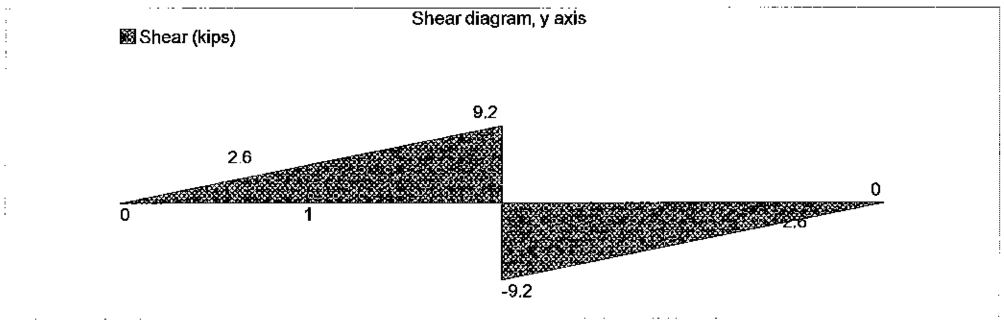
Shear strength reduction factor $\phi_v =$ **0.75**

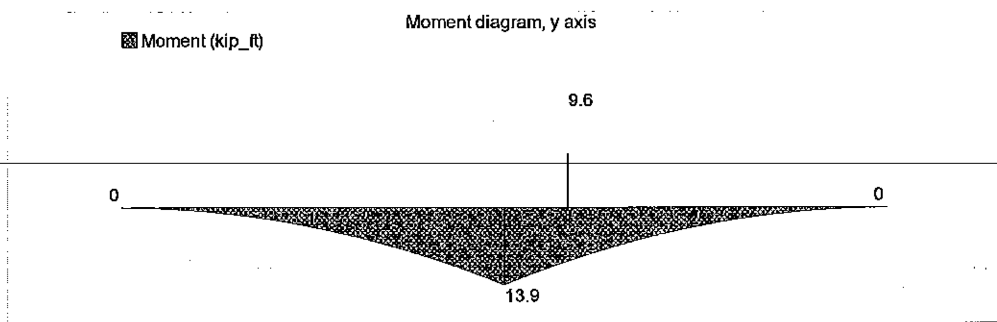
Nominal shear stress capacity (Eq. 11-2) $v_n = v_{cp} =$ **200.000** psi

Design shear stress capacity (Eq. 11-1) $\phi v_n = \phi_v \times v_n =$ **150.000** psi

$v_{ug} / \phi v_n =$ **0.042**

PASS - Design shear stress capacity exceeds ultimate shear stress load





Moment design, y direction, positive moment

Ultimate bending moment	$M_{u.y.max}$ = **9.625** kip_ft
Tension reinforcement provided	13 No.6 bottom bars (10.4 in c/c)
Area of tension reinforcement provided	$A_{sy.bot.prov}$ = **5.72** in^2
Minimum area of reinforcement (10.5.4)	$A_{s.min} = 0.0018 \times L_x \times h$ = **5.702** in^2

PASS - Area of reinforcement provided exceeds minimum

Maximum spacing of reinforcement (10.5.4)	s_{max} = min(3 × h, 18 in) = **18** in

PASS - Maximum permissible reinforcement spacing exceeds actual spacing

Depth to tension reinforcement	$d = h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2$ = **19.875** in
Depth of compression block	$a = A_{sy.bot.prov} \times f_y / (0.85 \times f'_c \times L_x)$ = **1.224** in
Neutral axis factor	β_1 = **0.85**
Depth to neutral axis	$c = a / \beta_1$ = **1.439** in
Strain in tensile reinforcement (10.3.5)	$\varepsilon_t = 0.003 \times d / c - 0.003$ = **0.03842**

PASS - Tensile strain exceeds minimum required, 0.004

Nominal moment capacity	$M_n = A_{sy.bot.prov} \times f_y \times (d - a / 2)$ = **550.929** kip_ft
Flexural strength reduction factor	$\phi_f = \min(\max(0.65 + (\varepsilon_t - 0.002) \times (250 / 3), 0.65), 0.9)$ = **0.900**
Design moment capacity	$\phi M_n = \phi_f \times M_n$ = **495.836** kip_ft
	$M_{u.y.max} / \phi M_n$ = **0.019**

PASS - Design moment capacity exceeds ultimate moment load

Footing geometry factor (15.4.4.2)	$\beta_f = L_x / L_y$ = **1.833**
Area of reinf. req. for uniform distribution (CRSI)	$A_{sreq} = (M_{u.y.max} / (\phi_f \times f_y \times (d - a / 2))) \times 2 \times \beta_f / (\beta_f + 1)$ = **0.144** in^2

PASS - Reinforcement can be distributed uniformly

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One-way shear design, y direction

Ultimate shear force — $V_{u.y}$ = **2.599** kips

Depth to reinforcement — $d_v = \min(h - c_{nom} - \phi_{x.bot} - \phi_{y.bot} / 2, h - c_{nom} - \phi_{y.top} / 2)$ = **19.875** in

Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear capacity (Eq. 11-3) — $V_n = 2 \times \lambda \times \sqrt{(f'_c \times 1\ psi)} \times L_x \times d_v$ = **262.35** kips

Design shear capacity — $\phi V_n = \phi_v \times V_n$ = **196.763** kips

$V_{u.y} / \phi V_n$ = **0.013**

PASS - Design shear capacity exceeds ultimate shear load

Two-way shear design at column 1

Depth to reinforcement — d_{v2} = **20.25** in

Shear perimeter length (11.11.1.2) — l_{xp} = **32.250** in

Shear perimeter width (11.11.1.2) — l_{yp} = **32.250** in

Shear perimeter (11.11.1.2) — $b_o = 2 \times l_{xp} + 2 \times l_{yp}$ = **129.000** in

Shear area — $A_p = l_{xp} \times l_{yp}$ = **1040.063** in^2

Surcharge loaded area — $A_{sur} = A_p - l_{x1} \times l_{y1}$ = **896.063** in^2

Ultimate bearing pressure at center of shear area — $q_{up.avg}$ = **0.714** ksf

Ultimate shear load — $F_{up} = \gamma_D \times F_{Dz1} + \gamma_D \times A_p \times F_{swt} + \gamma_D \times A_{sur} \times F_{soil} - q_{up.avg} \times A_p$ = **16.444** kips

Ultimate shear stress from vertical load — $v_{ug} = \max(F_{up} / (b_o \times d_{v2}), 0\ psi)$ = **6.295** psi

Column geometry factor (11.11.2.1) — $\beta = l_{y1} / l_{x1}$ = **1.00**

Column location factor (11.11.2.1) — α_s =**40**

Concrete shear strength (11.11.2.1) — $v_{cpa} = (2 + 4 / \beta) \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **300.000** psi

$v_{cpb} = (\alpha_s \times d_{v2} / b_o + 2) \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **413.953** psi

$v_{cpc} = 4 \times \lambda \times \sqrt{(f'_c \times 1\ psi)}$ = **200.000** psi

$v_{cp} = \min(v_{cpa}, v_{cpb}, v_{cpc})$ = **200.000** psi

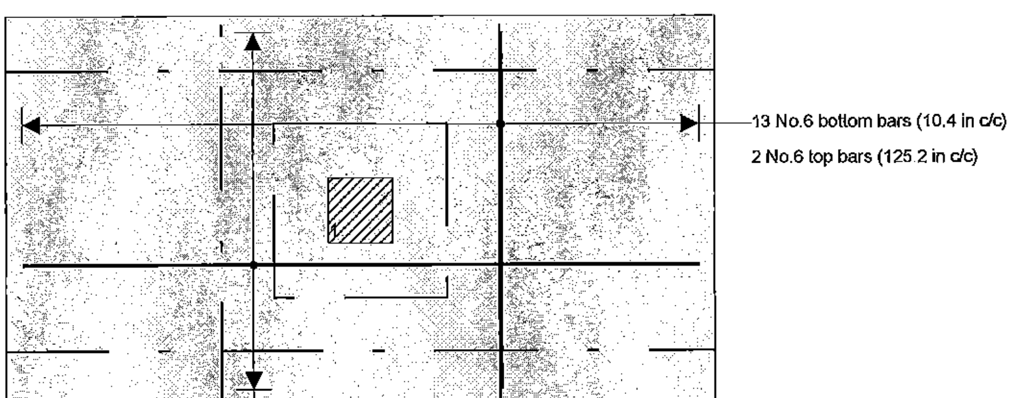
Shear strength reduction factor — ϕ_v = **0.75**

Nominal shear stress capacity (Eq. 11-2) — $v_n = v_{cp}$ = **200.000** psi

Design shear stress capacity (Eq. 11-1) — $\phi v_n = \phi_v \times v_n$ = **150.000** psi

$v_{ug} / \phi v_n$ = **0.042**

PASS - Design shear stress capacity exceeds ultimate shear stress load



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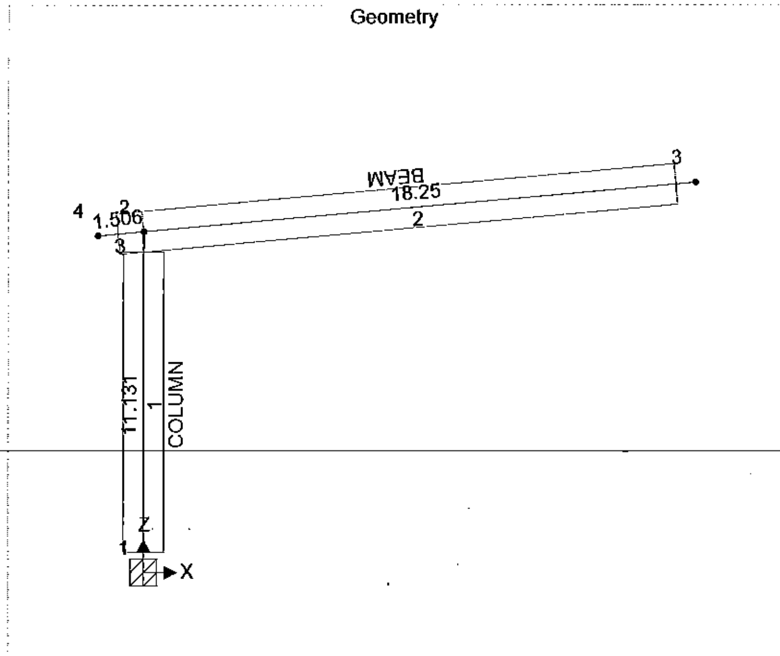
ANALYSIS

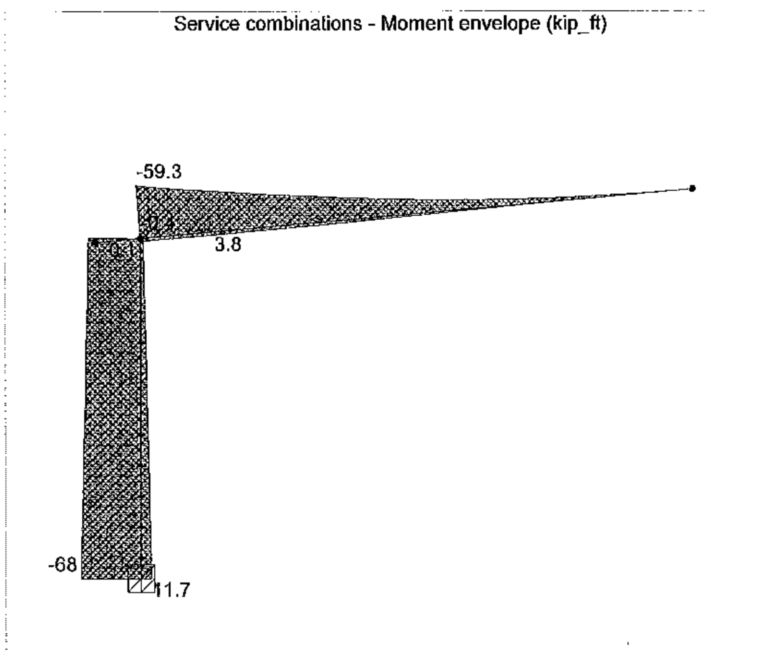
Tedds calculation version 1.0.13

Geometry



Results

Forces



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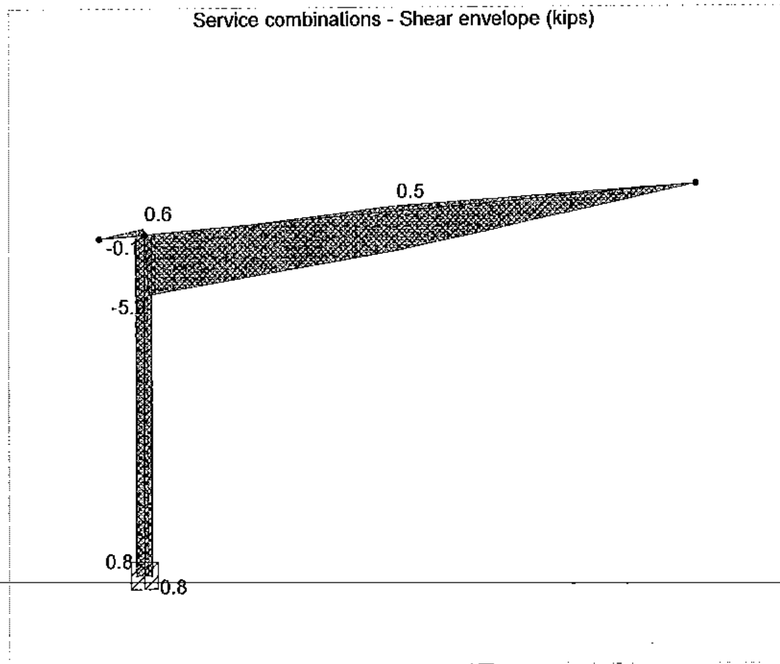

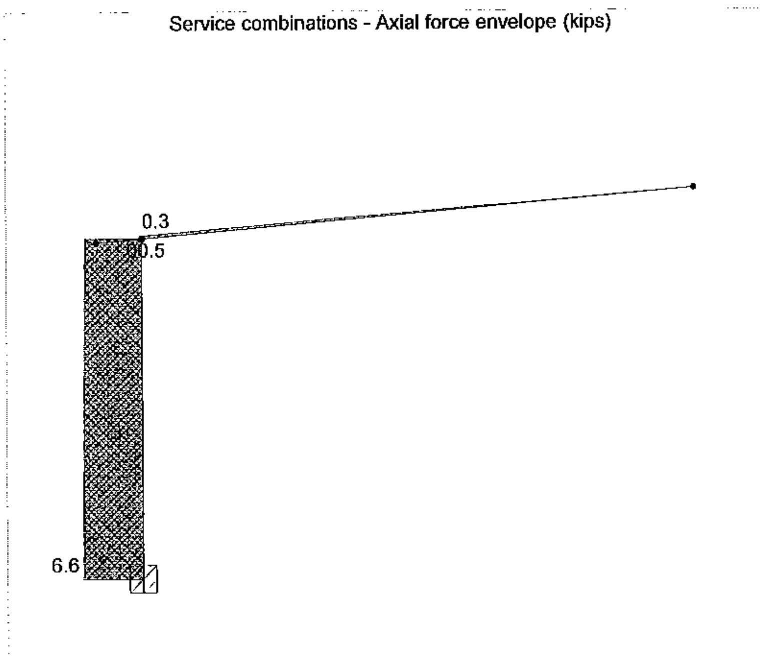

Member results

Envelope - Service combinations

Member	Shear force		Moment			
	Pos (ft)	Max abs (kips)	Pos (ft)	Max (kip_ft)	Pos (ft)	Min (kip_ft)
BEAM	18.25	-5.9	14.77	3.8	18.25	-59.3
COLUMN	0	0.8	0	11.7	0	-68

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Envelope - Service combinations

Member	Axial force			
	Pos (ft)	Max (kips)	Pos (ft)	Min (kips)
BEAM	18.25	0.3	18.25	0
COLUMN	0	6.6	11.13	0.5

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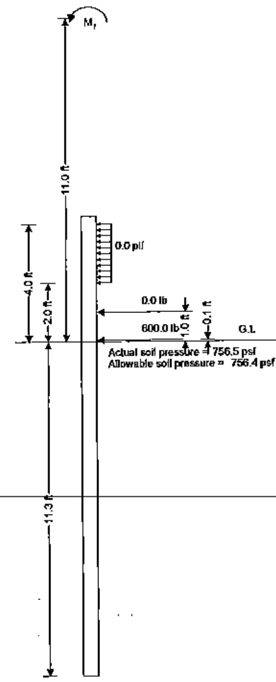
FLAGPOLE EMBEDMENT (IBC 2012)

TEDDS calculation version 1.1.01



Soil capacity data

Allowable passive pressure	L_{sbc} = **100** pcf
Maximum allowable passive pressure	P_{max} = **1500** psf
Load factor 1 (1806.1)	LDF_1 = **1.00**
Load factor 2 (1806.3.4)	LDF_2 = **2.0**

Pole geometry

Shape of the pole	**Round**
Diameter of the pole	Dia = **24** in
Laterally restrained	**No**

Load data

First point load	P_1 = **800** lbs
Distance of P_1 from ground surface	H_1 = **0.1** ft
Second point load	P_2 = **0** lbs
Distance of P_2 from ground surface	H_2 = **1** ft
Uniformly distributed load	W = **0** plf
Start distance of W from ground surface	a = **2** ft
End distance of W from ground surface	a_1 = **4** ft
Applied moment	M_1 = **68000** lb_ft
Distance of M_1 from ground surface	H_3 = **11** ft

Shear force and bending moment

Total shear force $F = P_1 + P_2 + W \times (a_1 - a) =$ **800** lbs

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Total bending moment at grade $M_g = P_1 \times H_1 + P_2 \times H_2 + W \times (a_1 - a) \times (a + a_1) / 2 + M_1 =$ **68040** lb_ft

Distance of resultant lateral force $h = abs(M_g / F) =$ **85.05** ft

Embedment depth (1807.3.2.1)

Embedment depth provided D = **11.35** ft

Allowable lateral passive pressure $S_1 = min(P_{max}, L_{sbc} \times min(D, 12\ ft) / 3) \times LDF_1 \times LDF_2 =$ **756.4** psf

Factor A $A = 2.34 \times abs(F) / (S_1 \times Dia) =$ **1.2** ft

Embedment depth required $D_1 = 0.5 \times A \times (1 + (1 + ((4.36 \times h) / A))^{0.5}) =$ **11.35** ft

Actual lateral passive pressure $S_2 = (2.34 \times abs(F) \times ((4.36 \times h) + (4 \times D))) / (4 \times D^2 \times Dia) =$ **756.5** psf

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STEEL COLUMN EMBEDMENT IN POLE FOOTING

User Inputs:

Axial Force;	Pu=18.5kips
Shear Force;	Vu=1.8kips
Moment;	Mu=97.9kip_ft
Pole Footing Diamter;	Dia=24in
Pole Footing Embedment Restrained;	EmbedR=11.5ft
Pole Footing Embedment Un-Restrained;	EmbedUR=11.5ft
Steel Column Embedment in footing;	Dsteel=4ft
Concrete Strength;	f'c=2500psi
Column Flange Width;	bfc=6.5in
Column Depth;	dc=12in
Beam Flange Width;	bfb=6.5in
Beam Depth;	db=12in
Pole Footing	
Restrained Soil Passive Pressure from Footing Analysis;	S2R=756.5psf
Un-Restrained Soil Passive Pressure from Footing Analysis;	S2UR=756.5psf
Bar Size;	BP=9
Area of Bar;	Ab=1.0in^2
Number of Bars on Each Side of Col;	Nbp=2
Spread Footing	
Bar Size;	Barsizes=9
Area of Bar;	Absp=1.0in^2
Number of Bars on Each Side of Col;	Nbs=3
Hodge Plate Connection;	
Plate Strength;	Fy=50ksi
Plate Width;	bpl=5.5in
Check Plate Width;	Checkp=if(bpl<=bfc-1in,"OK","REDUCE PLATE SIZE")=**"OK"**;
	Check2=if(bpl<bfb,"OK","REDUCE PLATE SIZE")=**"OK"**
Weld Size D (D/16");	D=5

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SUMMARY OF RESULTS

Embedment of Steel Column in Pole Footing;	Dsteel=getsectionvar(1,"Dsteel")=**4.00**ft
Embedment of Steel Column Check;	Check1=getsectionvar(6,"Check")=**"OK"**
Embedment of Steel Column DCR;	DCR1=getsectionvar(6,"DCR")=**0.57**
Pole Footing Reinforcing;	BarSize= getsectionvar(1,"BP")=**9.00**
Number of Bars on Each Side of Col;	Nbp= getsectionvar(1,"Nbp")=**2.00**
Pole Footing Reinforcing Check;	Check2=getsectionvar(2,"Check")=**"OK"**
Pole Footing Reinforcing DCR;	DCR2=getsectionvar(2,"DCR")=**0.32**
Un-Restrained Pole Footing Reinforcing Check;	Check3=getsectionvar(7,"Check")=**"OK"**
Un-Restrained Pole Footing Reinforcing DCR;	DCR3=getsectionvar(7,"DCR")=**0.32**
Spread Footing Column Reinforcing Bar Size;	BarSizes= getsectionvar(1,"Barsizes")=**9.00**
Number of Bars on Each Side of Col;	Nbs= getsectionvar(3,"Nbs")=**3.00**
Spread Footing Reinforcing Check;	Check4=getsectionvar(3,"Check")=**"OK"**
Spread Footing Reinforcing DCR;	DCR4=getsectionvar(3,"DCR")=**0.60**
Hodge Plate Strength;	Fy= getsectionvar(4,"Fy")=**50.00**ksi
Hodge Plate Width;	bpl= getsectionvar(4,"bpl")=**5.50**in
Minimum Hodge Plate Thickness;	tp= getsectionvar(4,"tp")=**0.47**in
Minimum Weld Size D/16;	D= getsectionvar(4,"D")=**5.00**
Minimum Weld Length;	L= getsectionvar(4,"L")=**16.72**in

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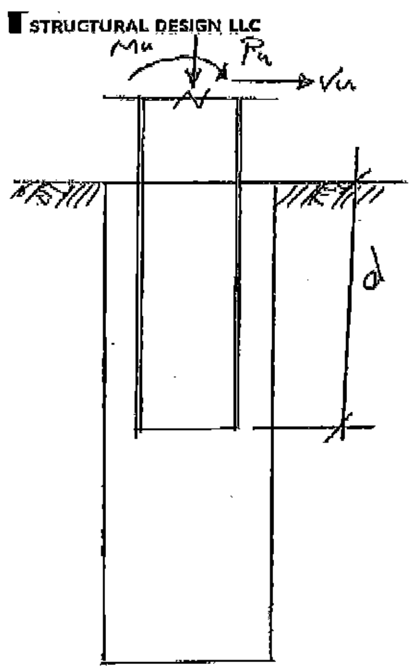
EMBEDMENT OF STEEL COLUMN IN POLE FOOTING

User Inputs:

Shear Force;	Vu= getsectionvar(1,"Vu")=**1.80**kips
Moment;	Mu= getsectionvar(1,"Mu")=**97.90**kip_ft
Steel Column Embedment in footing;	Dsteel= getsectionvar(1,"Dsteel")=**48.00**in
Concrete Strength;	f'c= getsectionvar(1,"f'c")=**2500.00**psi
Column Flange Width;	bfc=getsectionvar(1,"bfc")=**6.50**in

Allowable Bearing on Concrete;

Effective Column Flange Width;	effbfc=0.66*bfc=**4.29**in
Phi for Bearing on Concrete;	ϕ=0.6
Bearing Capacity;	ϕbn=ϕ×0.85×f'c=**1275.00**psi
Bearing Section Modulus;	Sb=effbfc×Dsteel^2/6=**1647.36**in^3
Bearing Force;	bu=Mu/Sb+Vu/(effbfc×Dsteel)=**721.88**psi
Check;	Check=if(ϕbn>bu,"OK","No Good")=**"OK"**
Demand to Capicity Ration;	DCR=bu/ϕbn=**0.566**



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POLE FOOTING REINFOCING (RESTRAINED)

User Inputs:	
Axial Force;	Pu=getsectionvar(1,"Pu")=**18.50**kips
Shear Force;	Vu= getsectionvar(1,"Vu")=**1.80**kips
Moment;	Mu= getsectionvar(1,"Mu")=**97.90**kip_ft
Pole Footing Diamter;	Dia= getsectionvar(1,"Dia")=**24.00**in
Pole Footing Embedment;	Embed= getsectionvar(1,"EmbedR")=**11.50**ft
Steel Column Embedment in footing;	Dsteel= getsectionvar(1,"Dsteel")=**4.00**ft
Column Depth;	dc=getsectionvar(1,"dc")=**12.00**in
Concrete Strength;	f'c= getsectionvar(1,"f'c")=**2500.00**psi
Bar Size;	Barsizep= getsectionvar(1,"BP")=**9.00**
Area of Bar;	Ab= getsectionvar(1,"Ab")=**1.00**in^2
Number of Bars on Each Side of Col;	Nbp= getsectionvar(1,"Nbp")=**2.00**
Section Modulus of Soil;	S2=getsectionvar(1,"S2R")=**756.50**psf
Section Modulus of Soil at S1;	S1=S2*Dsteel/Embed=**263.13**psf
Equivalent Force;	Peq=(S1+S2)/2×(Embed-Dsteel)×Dia=**7.65**kips
Design Moment;	Mud=Peq×2/3×(Embed-Dsteel)=**38.24**kip_ft
Effective Depth of Concrete Column;	d=min(Dia*2/3,dc+3in)=**15.00**in
Area of Steel;	As= Ab×Nbp=**2.00**in^2
a;	a=As×60ksi/(0.85×f'c×d)=**3.76**in
Flexural Phi;	ϕf=0.9
Concrete Foundation Capacity;	ϕMn=ϕf×As×60ksi×(d-a/2)=**118.06**kip_ft
Check;	Check=if(ϕMn>Mud,"OK","No Good")=**"OK"**
Demand to Capicity Ration;	DCR=Mud/ϕMn=**0.324**

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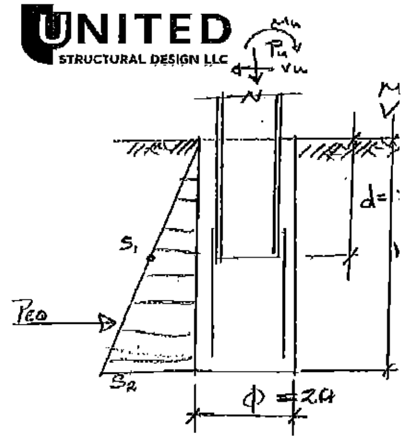

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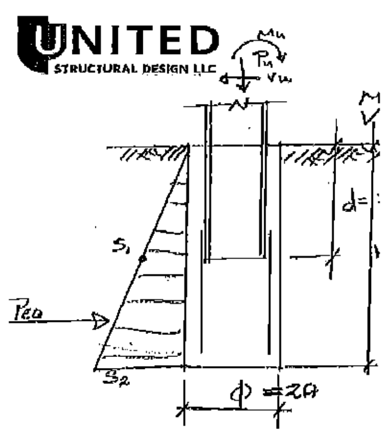
POLE FOOTING REINFOCING (UN-RESTRAINED CASE)

User Inputs:	
Axial Force;	Pu=getsectionvar(1,"Pu")=**18.50**kips
Shear Force;	Vu= getsectionvar(1,"Vu")=**1.80**kips
Moment;	Mu= getsectionvar(1,"Mu")=**97.90**kip_ft
Pole Footing Diamter;	Dia= getsectionvar(1,"Dia")=**24.00**in
Pole Footing Embedment;	Embed= getsectionvar(1,"EmbedUR")=**11.50**ft
Steel Column Embedment in footing;	Dsteel= getsectionvar(1,"Dsteel")=**4.00**ft
Column Depth;	dc=getsectionvar(1,"dc")=**12.00in**
Concrete Strength;	f'c= getsectionvar(1,"f'c")=**2500.00**psi
Bar Size;	Barsizep= getsectionvar(1,"BP")=**9.00**
Area of Bar;	Ab= getsectionvar(1,"Ab")=**1.00**in^2
Number of Bars on Each Side of Col;	Nbp= getsectionvar(1,"Nbp")=**2.00**
Section Modulus of Soil;	S2=getsectionvar(1,"S2UR")=**756.50**psf
Section Modulus of Soil at S1;	S1=S2*Dsteel/Embed=**263.13**psf
Equivalent Force;	Peq=(S1+S2)/2×(Embed-Dsteel)×Dia=**7.65**kips
Design Moment;	Mud=Peq×2/3×(Embed-Dsteel)=**38.24**kip_ft
Effective Depth of Concrete Column;	d=min(Dia*2/3,dc+3in)=**15.00**in
Area of Steel;	As= Ab×Nbp=**2.00**in^2
a;	a=As×60ksi/(0.85×f'c×d)=**3.76**in
Flexural Phi;	ϕf=0.9
Concrete Foundation Capacity;	ϕMn=ϕf×As×60ksi×(d-a/2)=**118.06**kip_ft
Check;	Check=if(ϕMn>Mud,"OK","No Good")=**"OK"**
Demand to Capicity Ration;	DCR=Mud/ϕMn=**0.324**



Tekla Tedds United Structural Design Phoenix, AZ (602) 888-1143	Project NSG2 - Colusa			Job Ref.	
	Section Connection Design			Sheet no./rev. 6	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date

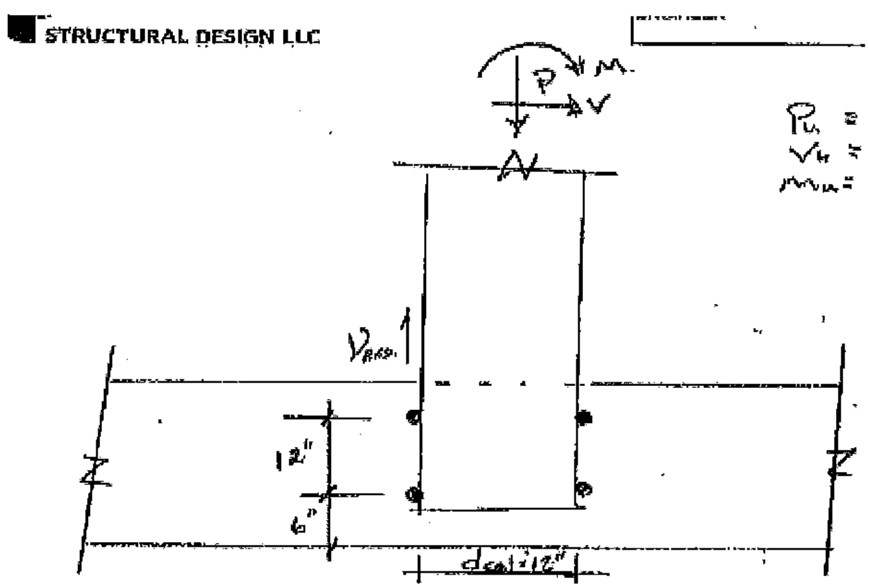
SPREAD FOOTING COLUMN ANCHORAGE

User Inputs:

Axial Force;	Pu=getsectionvar(1,"Pu")=**18.50**kips
Shear Force;	Vu= getsectionvar(1,"Vu")=**1.80**kips
Moment;	Mu= getsectionvar(1,"Mu")=**97.90**kip_ft
Column Depth;	dc=getsectionvar(1,"dc")=**12.00**in
Concrete Strength;	f'c= getsectionvar(1,"f'c")=**2500.00**psi
Area of Bar;	Absp= getsectionvar(1,"Absp")=**1.00**in^2
Number of Bars on Each Side of Col;	Nbs= getsectionvar(1,"Nbs")=**3.00**

Calculations;

Ultimate force Vres;	Vru=Pu+Mu/dc=**116.40**kips
Phi shear;	ϕ=0.9
Area of Shear Reinforcing;	Av=2×Absp×Nbs=**6.00**in^2
Capacity of Shear Reinforcing;	ϕVn=ϕ×0.6×60ksi×Av=**194.40**kips
Check;	Check=if(ϕVn>Vru,"OK","No Good")=**"OK"**
Demand to Capicity Ration;	DCR=Vru/ϕVn=**0.599**



Tekla Tedds United Structural Design Phoenix, AZ (602) 888-1143	Project NSG2 - Colusa				Job Ref.	
	Section Connection Design				Sheet no./rev. 7	
	Calc. by JE	Date 6/24/2016	Chk'd by	Date	App'd by	Date

HODGE PLATES STEEL CONNECTION AT TOP OF COLUMN

User Inputs:

Axial Force;	Pu=getsectionvar(1,"Pu")=**18.50**kips
Moment;	Mu= getsectionvar(1,"Mu")=**97.90**kip_ft
Column Flange Width;	bfc= getsectionvar(1,"bfc")=**6.50**in
Column Depth;	dc= getsectionvar(1,"dc")=**12.00**in
Beam Flange Width;	bfb= getsectionvar(1,"bfb")=**6.50**in
Beam Depth;	db= getsectionvar(1,"db")=**12.00**in
Plate Strength;	Fy= getsectionvar(1,"Fy")=**50.00**ksi
Plate Width;	bpl=getsectionvar(1,"bpl")=**5.50**in
Weld Size D (D/16");	D= getsectionvar(1,"D")=**5.00**

Calculations;

Maxium Tensile Foce on Connection;	Tu=Mu/dc+Pu=**116.40**kips
Phi Steel in Tension;	ϕ=0.9
Required Plate Thickness;	tp=Tu/(ϕ×Fy×bpl)=**0.47**in
Miniumum Weld Length;	L=Tu/(1.392kips/in×D)=**16.72**in