Independent Structural Calculations

- Upper Canopy of Berryessa Station

REGISTERED PROFESSIONAL ENGINEER
JUN JI
No. C 72794
EXP. 6/30/2014
CIVIL
STATE OF CALIFORNIA

Kal Krishnan Consulting Services, Inc

May 06, 2013

TABLE OF CONTENTS

Independent Structural Calculation – Upper Canopy

1. Executive summary

KKCS conducted independent structural calculations for the upper canopy for the Berryessa Station. A 3D SAP2000 model was constructed. The ARS response spectrum curves provided by AVA were used as the inputs for seismic calculation. The ASCE7-05 procedure was followed to calculate the wind load. The structural analysis and design check by KKCS bring up the following comments regarding the current design.

a. The design check indicated that the current design is sufficient in terms of member size under different load combination per ASCE 07-05 2.3.2 and 12.4.3.2.

b. The maximum inelastic story drift is 3.6” per the approach in ASCE 07-05 12.9.2. Therefore, it meets the required story drift limit per ASCE 07-05 Table 12.12-1.

c. However, more structural details including connection details between web struts and chords, column bases welding details, roof connections to the framing etc need to be developed. Assumptions have been made in the absence of these information.

d. Though capacity proves sufficient, the designed high strength 1.5" rod may still have brittle failure potential. Recommend changing to larger size and more ductile material.

2. Assumptions

Since some structural details are not available at the time of this analysis, some assumptions are made based on common engineering practice:

a. The connection of web struts to the chords under the roof and the legs in the column are assumed to be hinged;
b. Each of the four legs of the splayed columns has hinged connection at the location of the base plates;
c. The cover plate above the base plate of the columns are ½ inch thick;

In addition, some of details are not shown in the current drawings but were confirmed to be included in the future are:

a. two additional HSS 16X12X5/16 running parallel with Axis A and B along the edge of the metal decking roof;
b. The tension rods are 1.5 inch in diameter and are made of ASTM A449;

Other assumptions include:

a. Wind load is calculated per ASCE 07-05 Fig. 6-18B and 6-18C.
b. The upper canopy is treated as Ordinary Moment Frame based on the instruction email from BART [2].

3. The SAP2000 model

The upper canopy is modeled as a 3D steel structure in SAP2000The major chords of horizontal and vertical trusses are modeled as frame element. Web struts are modeled as truss element. The tension rods are modeled as tension-only member.

LRFD approach is adopted for the design check. The Acceleration Response Spectrum (ARS) curves used in the Response Spectrum Analysis (RSA) is from the "response spectra for design the berryessa station canopy structure" by T.Y. Lin in 2012 [1].

4. Loads and load combinations

Following loads have been included in the calculation:

a. Dead load;

b. Live load of 20 psf (BFS 2.0 sec 5.1.1) and concentrated load of 1,000 lbf anywhere on roof framing (RFP addendum 6 sec 5.2);
c. Wind load (see attachment for detailed calculation);
d. Earthquake load;

The load combinations are as specified in ASCE 07-05 sec 2.3.2 and 12.4.3.2;

1. 1.4D

2. $1.2D+0.5L_r$

3. $1.2D+1.6L_r+0.8W$

4. $1.2D+1.6W+0.5L_r$

5. 0.9D+1.6W

6. $(1.2+0.2S_{DS})\ D+E+L_r$

7. $(0.9-0.2S_{DS})\ D+E$

5. Analysis results and design check

The modes, base shear from RSA, maximum story drift, and design check of typical structural members are presented in orders.

Summary Tables (page 9)

a. Modal period and modal mass participation factor
b. Base shear from response spectrum analysis
c. Maximum story drift

Design Checks (see the attachment)

a. Main OMF beam design check
b. Main OMF column design check
c. Typical web strut design check
d. 1.5” diameter tension rod design check

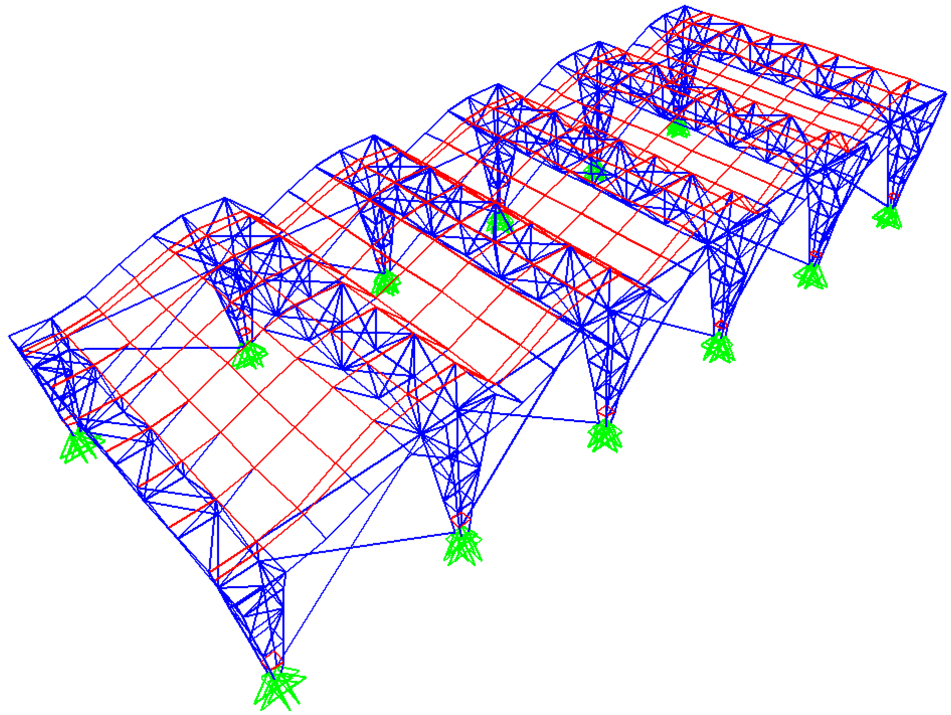
6. Conclusion

The current design is sufficient in terms of strength and deflection. But the structural details shown on the drawings are not sufficient for a complete independent calculation. Certain assumptions as listed above have been made.

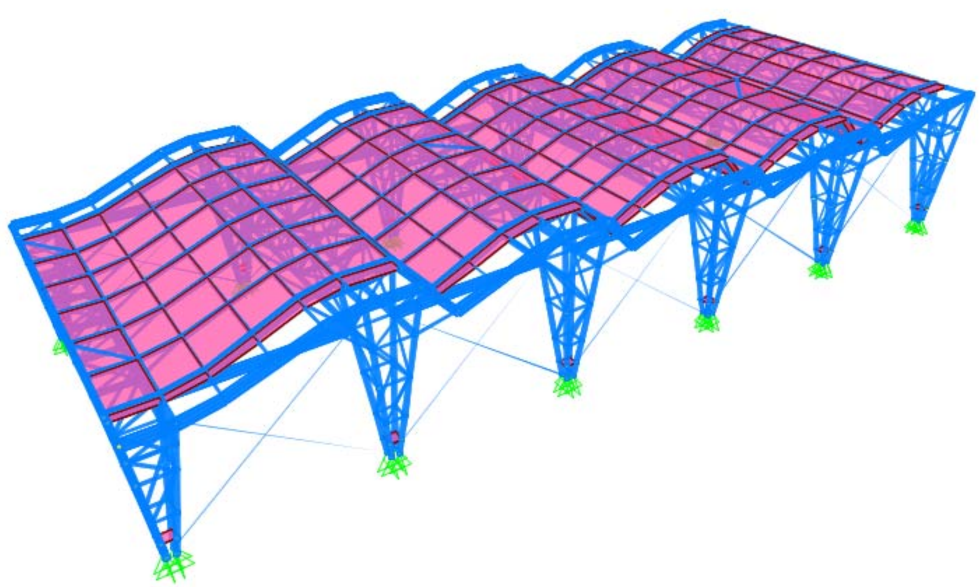
The tension rods' DCR ratio is relatively high. Therefore, larger size and more ductile tension rods are recommended.

7. Reference

1. Response spectra for design the berryessa station canopy structure, by T.Y. Lin International, October 8, 2012.
2. Email from Eric Folk dated 02/11/2013, subject: SVBX - Bereyessa Station Roof Truss framing systems.
3. ASCE 07-05 Minimum Design Loads for Buildings and Other Structures.

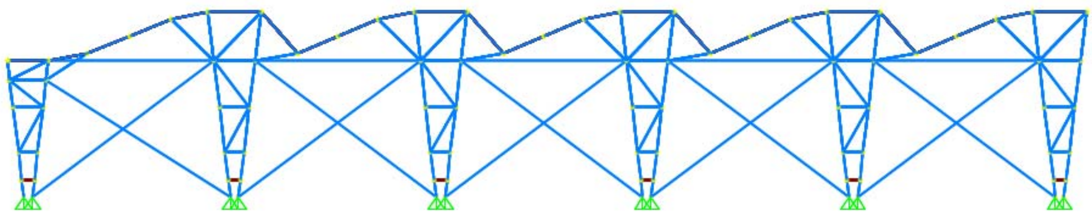


(a) Framing

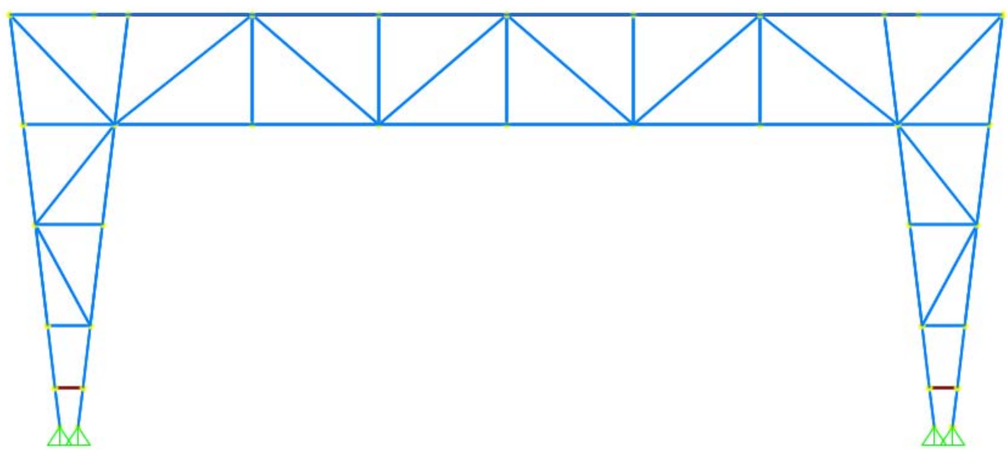


(b) Complete model

Figure 1: Finite element model

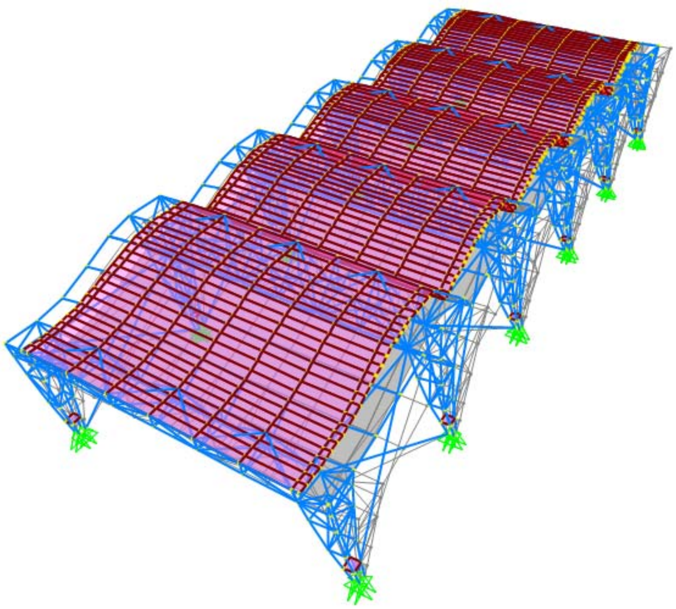


(a) Longitudinal side view

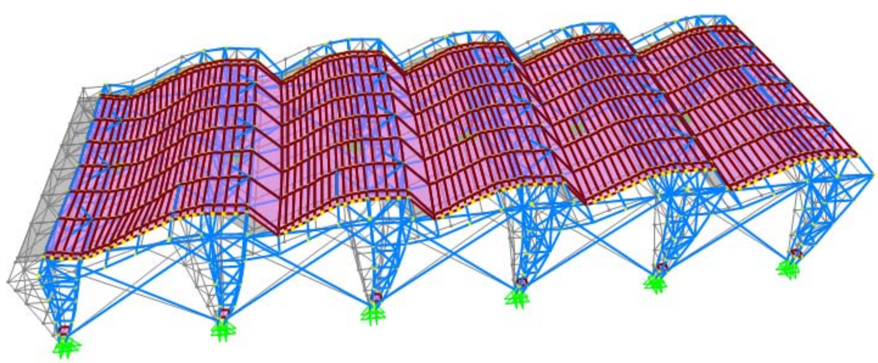


(b) Transverse side view

Figure 2: Side view of the 3D model



(a) First mode (swaying in transverse direction)



(b) Sixth mode (swaying in longitudinal direction)

Figure 3: Primary modes

Table 1 Modal period and modal mass participation factor

Modes	Period	UX (Longitudinal)	UY (Transverse)
Unitless	Sec	Unitless	Unitless
1	0.331023	2.325E-12	**0.95509**
2	0.309306	4.22E-12	0.00425
3	0.270443	1.966E-13	0.00467
4	0.235632	9.985E-12	0.00055
5	0.213912	1.048E-10	0.00007272
6	0.200649	**0.925**	6.36E-12
7	0.165313	2.035E-10	0.00262
8	0.132062	0.00274	1.281E-12
9	0.116197	0.03365	7.929E-12
10	0.099462	1.068E-11	0.00021
11	0.096448	0.00099	3.466E-12
12	0.093982	0.00051	4.114E-11

Table 2 Base shear

	Longitudinal	Transverse
Base Shear (kips)	561	902

Table 3 Maximum inelastic drift

	Longitudinal	Transverse
Maximum drift (in)	1	3.6

Note: the factor used in the calculation is per ASCE 07-05 sec 12.9.2.

Design Check

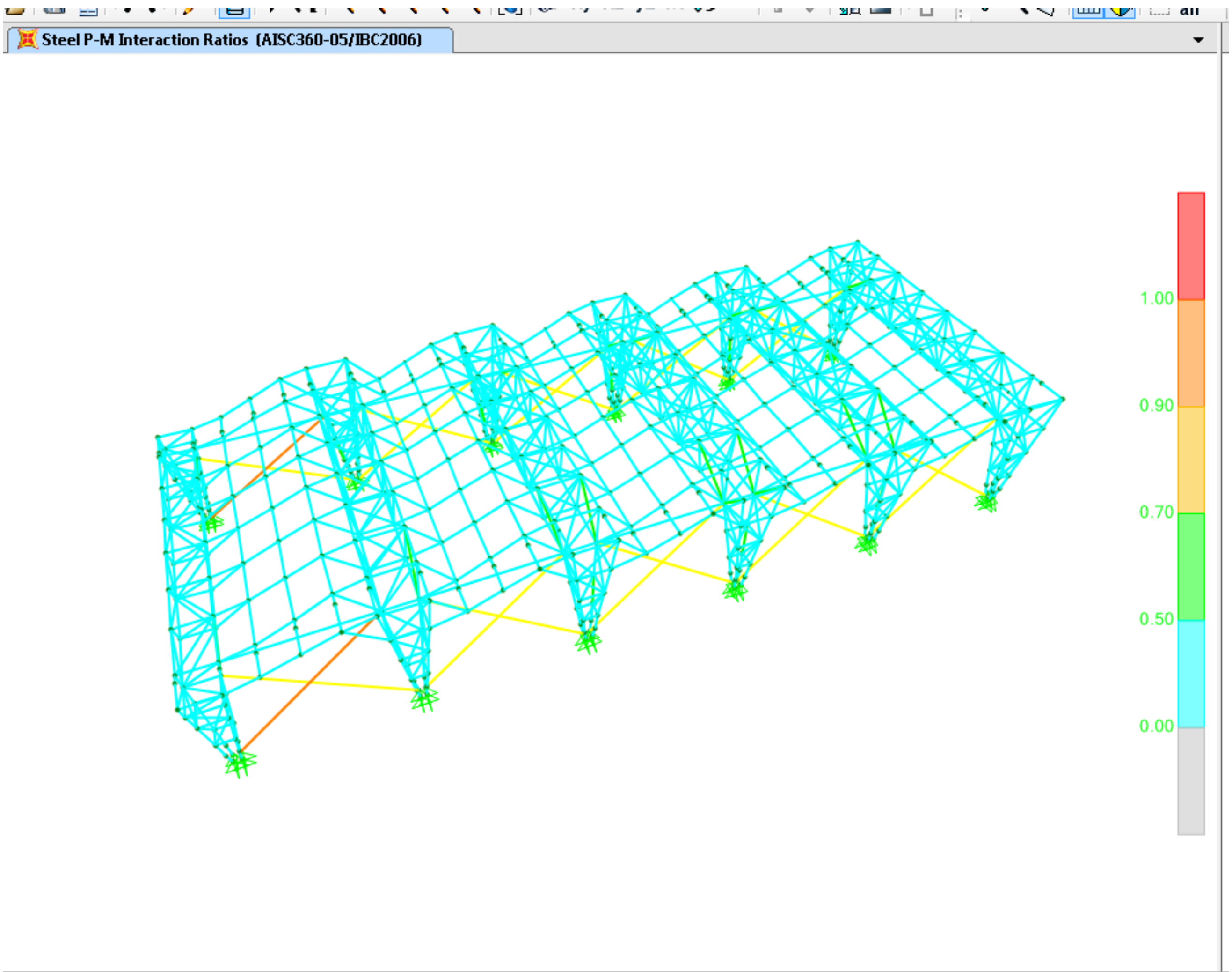


Figure 4: Design Check of the canopy structure in SAP2000

AISC360-05/IBC2006 STEEL SECTION CHECK (Summary for Combo and Station)

Units : Kip, ft, F

Frame : 1270	X Mid: 260.401	Combo: 1.4D-E1+Lr	Design Type: Beam
Length: 28.035	Y Mid: -357.796	Shape: HSS16X12X5/16	Frame Type: Ordinary Moment Fram
Loc : 13.083	Z Mid: 6.164	Class: Slender	Princpl Rot: 0.000 degrees

Provision: LRFD	Analysis: Direct Analysis			
D/C Limit=0.950	2nd Order: General 2nd Order		Reduction: Tau-b Fixed	
AlphaPr/Py=0.068	AlphaPr/Pe=0.051	Tau_b=1.000	EA factor=0.800	EI factor=0.800
Ignore Seismic Code? No		Ignore Special EQ	Load? No	D/P Plug Welded? Yes

SDC: D	I=1.500	Rho=1.000	Sds=0.500
R=3.500	Omega0=3.000	Cd=3.000	
PhiB=0.900	PhiC=0.900	PhiTY=0.900	PhiTF=0.750
PhiS=0.900	PhiS-RI=1.000	PhiST=0.900	

A=0.109	I33=0.029	r33=0.513	S33=0.043	Av3=0.049
J=0.035	I22=0.019	r22=0.412	S22=0.037	Av2=0.065
E=4176000.000	fy=6624.000	Ry=1.100	z33=0.051	
RLLF=1.000	Fu=8352.000		z22=0.042	

HSS Welding: ERW Reduce HSS Thickness? No

STRESS CHECK FORCES & MOMENTS (Combo 1.4D-E1+Lr)

Location	Pu	Mu33	Mu22	Vu2	Vu3	Tu
13.083	-49.385	7.316	0.000	-0.070	0.000	-0.310

PMM DEMAND/CAPACITY RATIO (H1-1b)

D/C Ratio: 0.088 = 0.059 + 0.030 + 0.000
= (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN (H1-1b)

Factor	L	K1	K2	B1	B2	Cm
Major Bending	1.000	1.000	1.000	1.000	1.000	1.000
Minor Bending	1.000	1.000	1.000	1.000	1.000	1.000

	Lltb	Kltb	Cb
LTB	1.000	1.000	1.136

	Pu Force	phi*Pnc Capacity	phi*Pnt Capacity
Axial	-49.385	420.961	649.980

	Mu Moment	phi*Mn Capacity	phi*Mn No LTB
Major Moment	7.316	246.283	246.283
Minor Moment	0.000	178.661	

	Tu Moment	Tn Capacity	phi*Tn Capacity
Torsion	-0.310	245.999	221.399

SHEAR CHECK

	Vu Force	phi*Vn Capacity	Stress Ratio	Status Check
Major Shear	0.070	218.689	0.000	OK
Minor Shear	0.000	160.862	0.000	OK

CONNECTION SHEAR FORCES FOR BEAMS

	VMajor Left	VMajor Right
Major (V2)	1.048	1.048

Main OMF beam design check

```
AISC360-05/IBC2006 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kip, ft, F

Frame : 143        X Mid: 280.923    Combo: 1.4D-E2+Lr        Design Type: Brace
Length: 7.777      Y Mid: -364.300   Shape: HSS 12.25X0.625   Frame Type: Ordinary Moment Fram
Loc   : 7.777      Z Mid: 2.330      Class: Seismic           Princpl Rot: 0.000 degrees

Provision: LRFD    Analysis: Direct Analysis
D/C Limit=0.950    2nd Order: General 2nd Order         Reduction: Tau-b Fixed
AlphaPr/Py=0.157   AlphaPr/Pe=0.012  Tau_b=1.000        EA factor=0.800   EI factor=0.800
Ignore Seismic Code? No              Ignore Special EQ Load? No          D/P Plug Welded? Yes

SDC: D             I=1.500           Rho=1.000          Sds=0.500
R=3.500            Omega0=3.000      Cd=3.000
PhiB=0.900         PhiC=0.900        PhiTY=0.900        PhiTF=0.750
PhiS=0.900         PhiS-RI=1.000     PhiST=0.900

A=0.159            I33=0.019         r33=0.343          S33=0.037         Av3=0.079
J=0.037            I22=0.019         r22=0.343          S22=0.037         Av2=0.079
E=4176000.000      fy=6048.000       Ry=1.100           z33=0.049
RLLF=1.000         Fu=8352.000                          z22=0.049

HSS Welding: ERW   Reduce HSS Thickness? No


STRESS CHECK FORCES & MOMENTS  (Combo 1.4D-E2+Lr)
    Location              Pu        Mu33        Mu22         Vu2         Vu3          Tu
    7.777            150.897     -19.342       3.662       3.481      -1.191       2.064

PMM DEMAND/CAPACITY RATIO   (H1.2,H1-1b)
    D/C Ratio:    0.161 = 0.087 + 0.073 + 0.014
                        = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1.2,H1-1b)
    Factor                 L          K1          K2          B1          B2          Cm
    Major Bending      1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending      1.000       1.000       1.000       1.000       1.000       1.000

                        Lltb        Kltb          Cb
    LTB                1.000       1.000       1.985

                          Pu     phi*Pnc     phi*Pnt
                       Force    Capacity    Capacity
    Axial            150.897     835.993     862.809

                          Mu      phi*Mn      phi*Mn
                      Moment    Capacity      No LTB
    Major Moment     -19.342     266.314     266.314
    Minor Moment       3.662     266.314

                          Tu          Tn      phi*Tn
                      Moment    Capacity    Capacity
    Torsion            2.064     278.615     250.754

SHEAR CHECK
                          Vu      phi*Vn      Stress      Status
                       Force    Capacity       Ratio       Check
    Major Shear        3.481     258.843       0.013          OK
    Minor Shear        1.191     258.843       0.005          OK

BRACE MAXIMUM AXIAL LOADS
                           P           P
                        Comp        Tens
    Axial                N/C     150.897
```

Main OMF column design check

SAP2000 Steel Design

Project

Job Number

Engineer

```
AISC360-05/IBC2006 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kip, ft, F

Frame : 330         X Mid: 308.915     Combo: 1.4D-E2+Lr        Design Type: Column
Length: 8.435       Y Mid: -337.612    Shape: HSS6X.500         Frame Type: Ordinary Moment Fram
Loc   : 4.218       Z Mid: 10.351      Class: Seismic           Princpl Rot: 0.000 degrees

Provision: LRFD     Analysis: Direct Analysis
D/C Limit=0.950     2nd Order: General 2nd Order          Reduction: Tau-b Fixed
AlphaPr/Py=0.019    AlphaPr/Pe=0.007   Tau_b=1.000        EA factor=0.800   EI factor=0.800
Ignore Seismic Code? No                Ignore Special EQ Load? No           D/P Plug Welded? Yes

SDC: D              I=1.500            Rho=1.000          Sds=0.500
R=3.500             Omega0=3.000       Cd=3.000
PhiB=0.900          PhiC=0.900         PhiTY=0.900        PhiTF=0.750
PhiS=0.900          PhiS-RI=1.000      PhiST=0.900

A=0.056             I33=0.002          r33=0.164          S33=0.006         Av3=0.051
J=0.003             I22=0.002          r22=0.164          S22=0.006         Av2=0.051
E=4176000.000       fy=6048.000        Ry=1.100           z33=0.008
RLLF=1.000          Fu=8352.000                           z22=0.008

HSS Welding: ERW    Reduce HSS Thickness? No


STRESS CHECK FORCES & MOMENTS  (Combo 1.4D-E2+Lr)
    Location              Pu        Mu33        Mu22         Vu2         Vu3          Tu
    4.218             -6.318       0.041       0.000       0.000       0.000      -0.036

PMM DEMAND/CAPACITY RATIO    (H1-1b)
    D/C Ratio:    0.013 = 0.012 + 0.000 + 0.000
                        = (1/2)(Pr/Pc) + (Mr33/Mc33) + (Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1b)
    Factor                 L          K1          K2          B1          B2          Cm
    Major Bending      1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending      1.000       1.000       1.000       1.000       1.000       1.000

                        Lltb        Kltb          Cb
    LTB                1.000       1.000       1.316

                          Pu     phi*Pnc     phi*Pnt
                       Force    Capacity    Capacity
    Axial             -6.318     259.760     305.802

                          Mu      phi*Mn      phi*Mn
                      Moment    Capacity      No LTB
    Major Moment       0.041      45.045      45.045
    Minor Moment       0.000      45.045

                          Tu          Tn      phi*Tn
                      Moment    Capacity    Capacity
    Torsion           -0.036      49.892      44.903

SHEAR CHECK
                          Vu      phi*Vn      Stress      Status
                       Force    Capacity       Ratio       Check
    Major Shear        0.000      91.741       0.000          OK
    Minor Shear        0.000      91.741       0.000          OK
```

Typical web strut design check

```
AISC360-05/IBC2006 STEEL SECTION CHECK  (Summary for Combo and Station)
Units  : Kip, ft, F

Frame : 1726      X Mid: 224.014    Combo: 1.4D+EQ1+Lr COM  Design Type: Brace
Length: 38.585    Y Mid: -359.183   Shape: 1.5 IN ROD       Frame Type: Ordinary Moment Fram
Loc   : 19.292    Z Mid: -5.368     Class: Compact          Princpl Rot: 0.000 degrees

Provision: LRFD   Analysis: Direct Analysis
D/C Limit=0.950   2nd Order: General 2nd Order        Reduction: Tau-b Fixed
AlphaPr/Py=0.520  AlphaPr/Pe=224.32 Tau_b=1.000       EA factor=0.800   EI factor=0.800

PhiB=0.900        PhiC=0.900        PhiTY=0.900       PhiTF=0.750
PhiS=0.900        PhiS-RI=1.000     PhiST=0.900

A=0.012           I33=1.198E-05     r33=0.031         S33=1.917E-04     Av3=0.009
J=2.397E-05       I22=1.198E-05     r22=0.031         S22=1.917E-04     Av2=0.009
E=4176000.000     fy=11664.000      Ry=1.000          z33=3.255E-04
RLLF=1.000        Fu=15120.000                        z22=3.255E-04

HSS Welding: ERW  Reduce HSS Thickness? No


DESIGN MESSAGES
    Warning: l/r > 300 (AISC D1)
```

ok for tension rods

```
STRESS CHECK FORCES & MOMENTS  (Combo 1.4D+EQ1+Lr COM)
    Location              Pu        Mu33        Mu22         Vu2         Vu3          Tu
    19.292            74.424       1.256       0.000       0.000       0.000   4.707E-04

PMM DEMAND/CAPACITY RATIO   (H1.2,H1-1a)
    D/C Ratio:     0.904 = 0.578 + 0.327 + 0.000
                         = (Pr/Pc) + (8/9)(Mr33/Mc33) + (8/9)(Mr22/Mc22)

AXIAL FORCE & BIAXIAL MOMENT DESIGN   (H1.2,H1-1a)
    Factor                 L          K1          K2          B1          B2          Cm
    Major Bending      1.000       1.000       1.000       1.000       1.000       1.000
    Minor Bending      1.000       1.000       1.000       1.000       1.000       1.000

                        Lltb        Kltb          Cb
    LTB                1.000       1.000      19.738

                          Pu     phi*Pnc     phi*Pnt
                       Force    Capacity    Capacity
    Axial             74.424       0.262     128.825

                          Mu      phi*Mn      phi*Mn
                      Moment    Capacity      No LTB
    Major Moment       1.256       3.417       3.417
    Minor Moment       0.000       3.417

                          Tu          Tn      phi*Tn
                      Moment    Capacity    Capacity
    Torsion        4.707E-04       2.684       2.416

SHEAR CHECK
                          Vu      phi*Vn      Stress      Status
                       Force    Capacity       Ratio       Check
    Major Shear        0.000      38.647       0.000          OK
    Minor Shear        0.000      38.647       0.000          OK
```

1.5" diameter tension rod design check

Wind Load Calculation

Wind Load Calc (ASCE 07-10)

1. Risk Category IV
2. Basic Wind speed $V = 85$ mph
3. Wind directionality factor

$K_d = 0.85$ (ASCE 07-10 Table 26.6-1)

Velocity pressure exposure coefficients
Assume height above ground is 50 ft

$K_z = 1.09$ (ASCE 07-10 Table 27.3-1)

Topographic factor

$K_{zt} = (1 + K_1 \cdot K_2 \cdot K_3)^2$

$= 1$ (ASCE 07-10 Sec 26.8.2)

Therefore velocity pressure

$q_h = 0.00256\, K_z \cdot K_{zt} \cdot K_d \cdot V^2$

$= 0.00256 \cdot 1.09 \cdot 1 \cdot 0.85 \cdot 85^2$

$= 17.1$ psf

4. Gust effect factor

$G = 0.85$ (ASCE 07-10 Sec 26.9.1)

5. Net pressure coefficient

Pitched free roof (ASCE 07-10 Fig 27.4-5)

Rise-to-span ratio

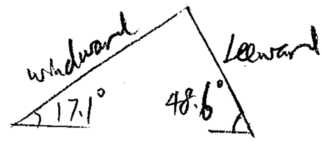
$r = \frac{8.3}{2.5 - 2.5} = 0.263$

$\theta_1 = \arctan\left(\frac{8.5}{35 - 3.5 - 4.5}\right) = 17.1^\circ$ use 15° in the table

$\theta_2 = \arctan\left(\frac{7}{6.17}\right) = 48.6^\circ$ use 45°

Load case A

$C_{NW} = 1.1$; $C_{NL} = -0.4$ (ASCE 07-10 Fig 27.4-5)



6. Net design pressure

$p_w = q_h \cdot G \cdot C_{NW}$

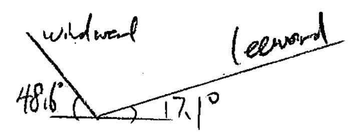
$= 17.1 \cdot 0.85 \cdot 1.1$

$= 16$ psf windward

$P_L = q_h \cdot G \cdot C_{NL}$

$= 17.1 \cdot 0.85 \cdot (-0.4)$

$= -5.8$ psf Leeward.



$C_{NW} = -1.1$; $C_{NL} = -0.9$

Therefore, use $p_w = 16$ psf windward to be conservative

$P_L = -16$ psf Leeward.