DESIGN COMPARISON

AN WIRELESS TOWER AND THE TRYLON TITAN TOWER

TEP # 03190

August 8, 2003

Prepared For:

AN Wireless
RF Communications Towers

TOWER
ENGINEERING
PROFESSIONALS

TOWER ENGINEERING PROFESSIONALS

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Subject: **Design Comparison**
TEP # 03190

Dear Mr. Simmonds:

Tower Engineering Professionals (TEP) completed its comparison of the Trylon Titan tower and AN Wireless (ANW) tower.

Investigation included the following:

1) Internet research of the Trylon Titan tower
2) Correspondence with Trylon
3) Review of the ANW tower design
4) Observations and measurements of the tower section number 4 of the Trylon Titan tower, provided by ANW

Based on its work TEP offers the following:

1) The Titan tower was designed using a different method than the ANW tower.

A safety factor is the ratio of load capacity to applied load. Usually this load capacity is based on the yield stress, Fy, of the material used to fabricate the tower. If the applied load is kept below this yield stress, the structure will not experience permanent deformation during a design load event. Since there is a set of unknowns in design, manufacture, erection, and service of the structure, engineers usually apply safety factors to their designs.

For instance, a structure may be specified so that the applied loads are a certain fraction, less than one, of the load capacity. The amount of stress is limited to be below the yield stress via the structure selection. This is the allowable stress design (ASD) method. It is the current approach of the ANSI/TIA/EIA-222-F-1996, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures (TIA).

Another method is to increase the anticipated loads by a number greater than or equal to one. This is called overload factor. The structure is designed with a capacity for that overload. This is ultimate strength design (USD). It was the approach used in the 1970's by the steel transmission tower industry.

TOWER ENGINEERING PROFESSIONALS

To illustrate the difference between the two design methods, TEP prepared calculations. For comparison purposes, calculations are based on section #4 of the Titan tower and section #7 of the ANW tower. Each of these sections would be located at the top of the heaviest 80-ft modular tower available.

TEP attempted to identify the standard or code used by Trylon for the design of the Titan tower. No reference was available from Trylon. However, it is apparent that the tower was designed using a form of the USD method. The ANW tower was designed to meet the TIA.

From our calculations, the following observations were made:

a) The ultimate load capacity of the ANW tower leg was 32.4 kips. The allowable load was 22.6 kips.
b) The ultimate load of the Titan tower leg was 6.5 kips. The allowable load was 4.5 kips.
c) The allowable load was 70 percent of the ultimate load for each tower leg.
d) The wind load on the bare ANW tower section per TIA was 25.5 pounds per linear foot (plf).
e) The wind load on the bare Titan tower section per TIA was 18.6 pounds per linear foot (plf).
f) The wind load per the TowerCalc program was 13.1 plf. This is about 70 percent of the load required by TIA.
g) The Titan tower was designed for less wind load than the ANW tower.
h) The Titan leg member has less capacity than the ANW leg member for the sections considered.

2) Description of tower members and connections

Section #4 of the Titan tower is comprised of the following:

a) Members
   i) Leg members are 60-degree bent plate, approximately 4 inches by 3/32 inches before forming. Members are fabricated using 32 ksi yield-strength material. No specification was available from Trylon.
   ii) Bracing members are 90-degree bent plate, 2 inches by 5/64 inches before forming. Members are fabricated using 32 ksi yield-strength material. No specification was available from Trylon.
b) Connections
   i) Leg connections at the base of the section are (2) 3/8-in diameter bolts in single shear. Connections are SAE Grade 5 material.
   ii) Bracing connections are 1/4-in diameter bolts in single shear. Connections are SAE Grade 5 material.

Section #7 of the ANW tower section is comprised of the following

a) Members
   i) Leg members are 60-degree bent plate, 5 inches by 3/16 inches before forming. Members are fabricated of ASTM A572 Grade 50 (50 ksi yield strength) material.
   ii) Bracing members are 90-degree bent plate, 3 inches by 11 gage (about 1/8 inches) before forming. Members are fabricated of ASTM A572 Grade 50 (50 ksi yield strength) material.
b) Connections
   i) Leg connections at the base of the section are (2) 1/2-in diameter bolts in double shear. Connections are SAE Grade 5 material.

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ii) Bracing connections are 3/8-in diameter bolts in single shear. Connections are SAE Grade 5 material.

3) The panel points for the Titan tower section are eccentric. There are 6 inches between the diagonal to leg connections (panel points). This connection requires (2) bolts per brace. It also introduces bending stresses in the tower legs. The ANW tower section uses concentric panel points. This allows less bracing connections. It also eliminates bending stress in the tower legs.

4) The Titan tower section is composed of pre-galvanized steel sheets. This results in the sheared edges of members and puched holes for connections being exposed to the elements. The ANW tower is composed of steel plates that are cold formed, punched for connections and then hot-dipped galvanized. This results in all edges having a layer of surface protection.

5) The ANW tower was designed to meet the TIA structural standards. The Trylon Titan tower was not.

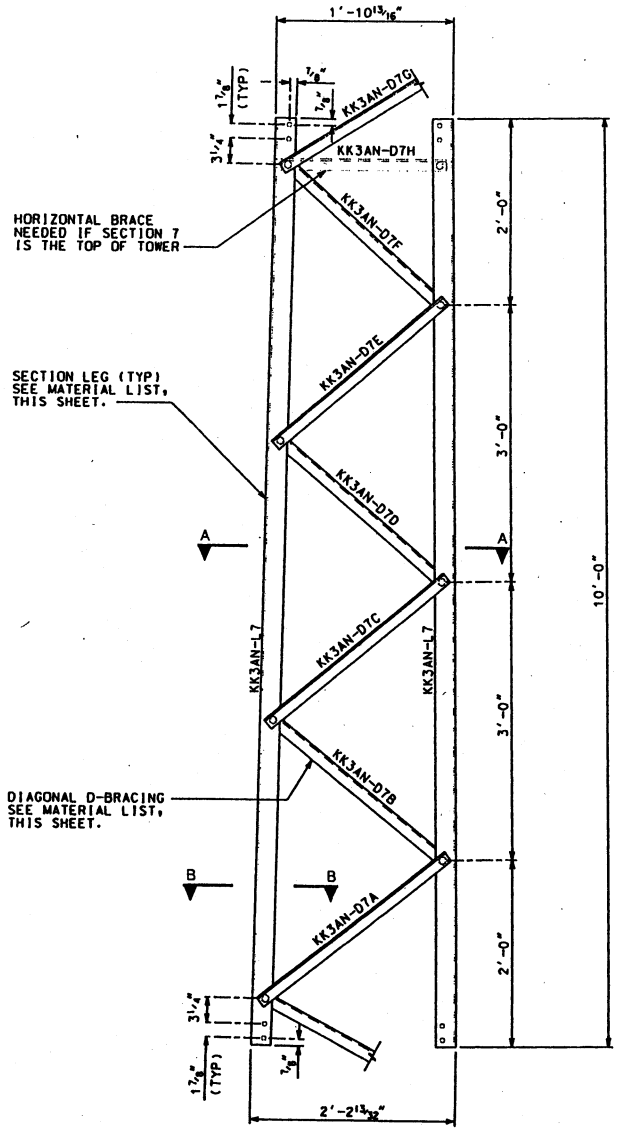
Thank you for the opportunity to perform this service for you. We hope this letter illustrates the differences in the designs and physical characteristics of the two towers. If you have questions or comments, please contact our office.

Regards,

Tower Engineering Professionals, Inc.
Michael L. Lassiter, P.E., S.E.
Division Manager, Structures

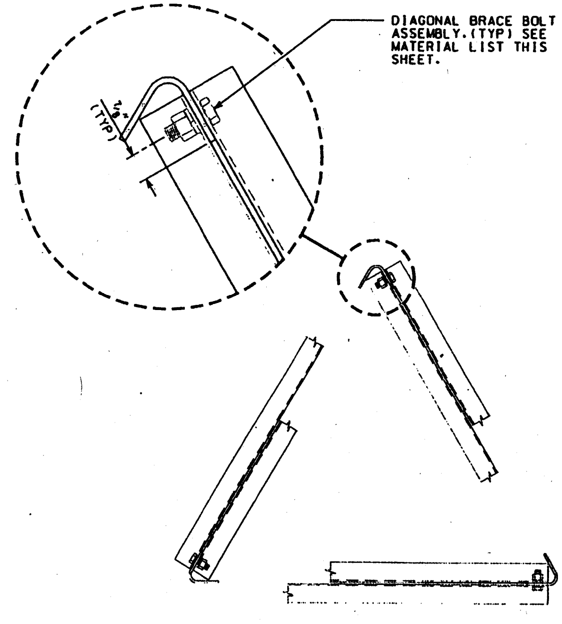
PORTABLE DOCUMENT FORMAT

Attachments:
1) ANW Tower Section #7 Drawing
2) Trylon Titan Section #4 Drawing
3) Calculations



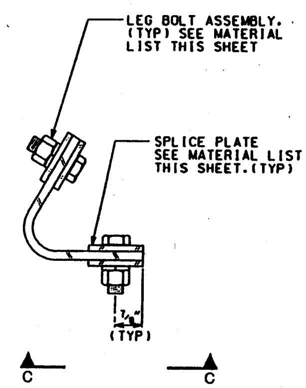
SECTION 7 ELEVATION

SCALE: 3/4" = 1'-0"



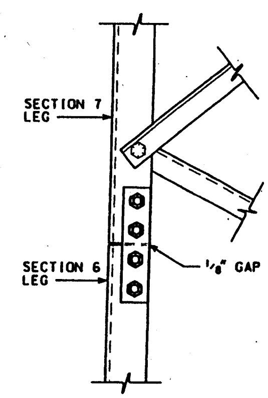
SECTION A-A

SCALE: N.T.S.



SECTION B-B

SCALE: 3" = 1'-0"



SECTION C-C

SCALE: 1 1/2" = 1'-0"

MATERIAL LIST

PIECE MARK	QTY.	DESCRIPTION
KK3AN-L7	3	PL 5" x 3/16" x 10'-0"
KK3AN-D7A	3	PL 3" x 11 GA. x 2'-6 9/16"
KK3AN-D7B	3	PL 3" x 11 GA. x 2'-6 1/8"
KK3AN-D7C	3	PL 3" x 11 GA. x 2'-5 7/8"
KK3AN-D7D	3	PL 3" x 11 GA. x 2'-5 5/16"
KK3AN-D7E	3	PL 3" x 11 GA. x 2'-4 15/16"
KK3AN-D7F	3	PL 3" x 11 GA. x 2'-4 1/2"
KK3AN-D7G	3	PL 3" x 11 GA. x 2'-0 5/8"
KK3AN-D7H	3	PL 3" x 11 GA. x 1'-9 5/16"
BRACE BOLT	21	3/8" ø x 1" BOLT ASSEMBLY
LEG BOLT	24	1/2" ø x 1 1/2" BOLT ASSEMBLY
SPLICE PLATE	12	PL 1 3/4" x 3/16" x 0'-7 3/8"

NOTES

1. FOR SECTION LOCATION AND ORIENTATION SEE KEY ASSEMBLY.
2. LOCKWASHERS TO BE INSTALLED AT ALL BOLTED CONNECTIONS.
3. THEORETICAL SECTION WEIGHT BEFORE GALVANIZING = 171#
4. BEND RADIUS FOR DIAGONAL D-BRACING = 3/16"
5. BEND RADIUS FOR LEG = 1/2"
6. STEEL GRADES :MEMBERS ASTM 572 GR.50
   :BOLTS SAE GR.5
   :WASHERS SAE GR.5
   :NUTS SAE GR.5
7. ALL MATERIALS SHALL BE HOT-DIP GALVANIZED PER ASTM A123 AND A153

REV	DATE	ISSUED FOR :
5	07-22-02	REVISION PER TEP
4	06-16-02	REVISION PER AN WIRELESS
3	12-14-01	REVISION PER AN WIRELESS
2	11-25-01	REVISION PER AN WIRELESS
1	11-09-01	FOR REVIEW

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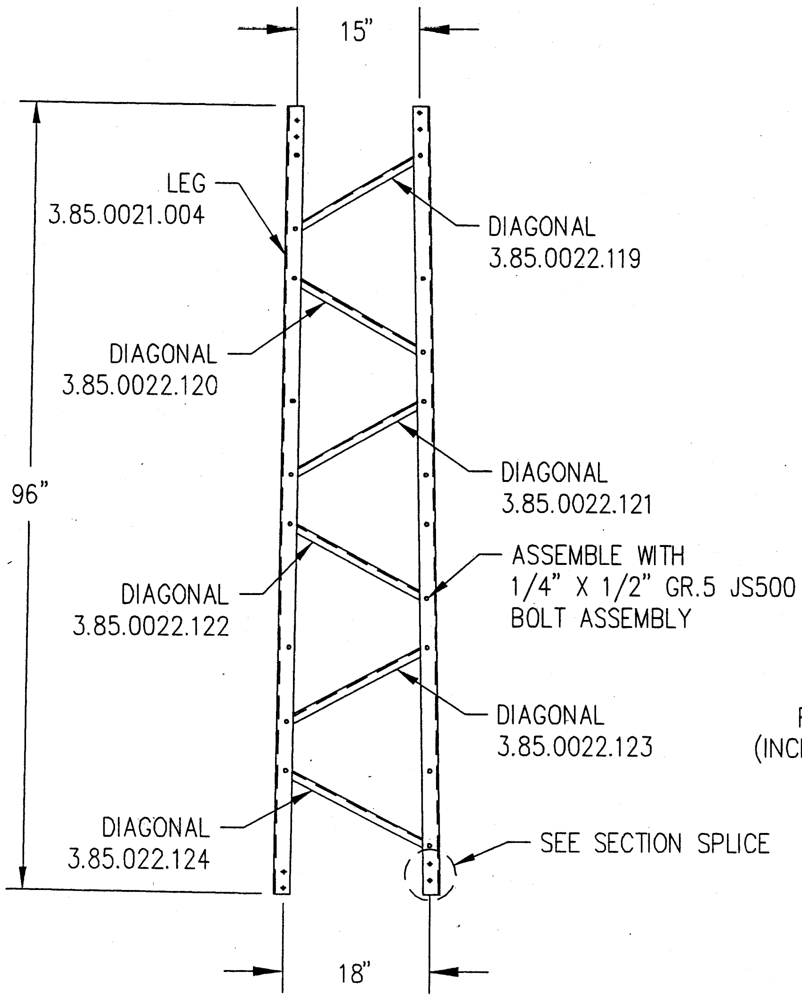
DRAWN BY : G.M.ANDRES
CHECKED BY : A.T.HALDANE

SHEET TITLE :

KK3AN SECTION 7 ASSEMBLY

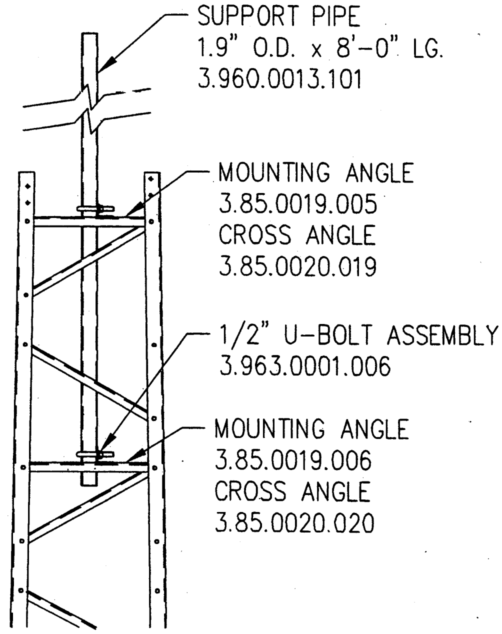
SHEET NUMBER : **KK3AN-S7**

REVISION: **5**



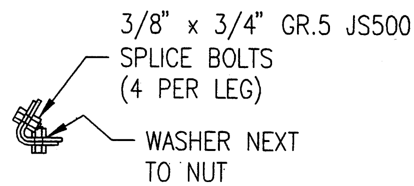
SECTION #4

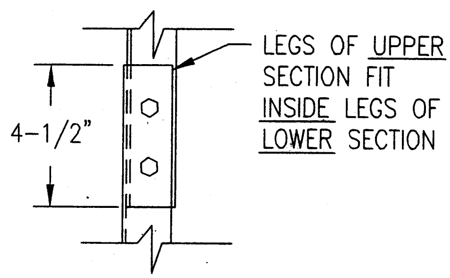
KNOCK-DOWN P/N: 4.94.0041.000
PRE-ASSEMBLED P/N: 4.95.0041.000



TOP SECTION #4

KNOCK-DOWN P/N: 4.94.0042.000
PRE-ASSEMBLED P/N: 4.95.0042.000
(INCLUDES ALL PARTS SHOWN IN SECTION #4)





SECTION SPLICE

REV.	REV. BY:	CHK. BY:	DESCRIPTION	DATE

NOTES:

1) ALL BRACING MEMBERS INSIDE LEGS WITH FLANGE FACING TOP END.

TRYLON TSF

CUSTOMER:	SITE:	SCALE: 15.000
DATE: 26 JAN 00	BY: BCP	CHK:	APP:

TITLE: ASSEMBLY OF SECTION #4

DRAWING NO. 000001.610.0004

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Project Name DESIGN COMPARISON
Project # 03190
Date 08/07/2003
Design M. LASSITER
Check JRH 8/7/2003
Page 1 of 5

WIND LOAD PER TIA

ANW SECTION #7

FLAT SHAPE AREA

ITEM	QTY	b (IN)	EXPOSED L (IN)	A_T (IN²)
LEG	2	2.5	120	600
BRACE G1	1	1.5	13	19.5
BRACE A	1	1.5	26.5	39.8
BRACE B	1	1.5	25.8	38.7
BRACE C	1	1.5	25.5	38.3
BRACE D	1	1.5	24.8	37.2
BRACE E	1	1.5	24.5	36.8
BRACE F	1	1.5	24.0	36.0
BRACE G2	1	1.5	8.5	12.8

859.1"□ → 5.97'□ = A_F

$$A_G = \left(\frac{1}{144}\right)(120)\left(\frac{1}{2}\right)(26.4063 + 22.8125) = \underline{20.5'}\square$$

$F = 255^{\#}$ [A-01]

$$\rightarrow w = \frac{255^{\#}}{10'} = \underline{\underline{25.5}}^{PLF}$$

TOWER ENGINEERING PROFESSIONALS

LOAD CAPACITY

Project Name DESIGN COMPARISON
Project # 03190
Date 08/04/2003
Design M. LASSITER
Check JRH 9/9/2003
Page 2 of 5

ANW TOWER SECTION #7

LEG PL 3/16 x 5 ASTM A572 GRADE 50

TIA

$$\frac{KL_x}{r_x} = \frac{(1.0)(36)}{(0.57)} = \underline{63.2}$$

$$\frac{KL_y}{r_y} = \frac{(1.0)(36)}{(0.87)} = \underline{41.4}$$

$$\left(\frac{kL}{r}\right)_{FTB} = 81.0 \leftarrow \text{CONTROLS}$$

$\rightarrow P'_{ALL} = \underline{22.6^k}$ [A-02 TO A-04]

ULTIMATE LOAD BASED ON $F_a A_g$

$$P_{ALL} = \frac{23}{12}(18.8^{ksi})(0.90''^{\square}) = \underline{32.4^k}$$

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WIND LOAD PER TIA

TITAN SECTION #4

FLAT SHAPE AREA

ITEM	QTY	b (IN)	EXPOSED L (IN)	A_T (IN^2)
LEG	2	2	96	384
BRACE 01	1	1	17.875	17.9
BRACE 02	1	1	17.5	17.5
BRACE 03	1	1	17	17.0
BRACE 04	1	1	16.625	16.6
BRACE 05	1	1	16.125	16.1
BRACE 06	1	1	15.625	15.6

$484.7 \text{ in}^2 \rightarrow 3.37 \text{ ft}^2 = A_F$

$$A_G = \frac{1}{144}(96)(1/2)(20.375 + 17.188) = 12.52 \text{ ft}^2$$

$F = 149^{\#}$ [A - 05]

$$\rightarrow w = \frac{149^{\#}}{8'} = 18.6^{PLF}$$

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WIND LOAD PER TOWER CALL

TITAN SECTION #4

PROCEDURE

1) SELECT HEIGHT = 96^{FT}

2) SELECT T200 MODEL

3) ENTER 70^{MPH} MAXIMUM WIND CONDITION

4) ENTER "TEST" FOR ANTENNA NAME

0 FOR PROJECTED AREA

"ROUND" FOR SHAPE

1^{FT} FOR HEIGHT

1^{LB} FOR WEIGHT

96^{FT} FOR ELEVATION ABOVE GRADE

0^{FT} FOR OFFSET

5) EXECUTE PROGRAM

6) SEE RESULTS

$F_{SECTION \#4}$ = ΔV BETWEEN NOMINAL HEIGHTS 80' & 72'

$= 0.26^k - 0.16^k = \underline{\underline{0.10^k}}$ [A-06 & A-07]

$\rightarrow w = \dfrac{100^{\#}}{[96-4.5]/12} = \underline{\underline{13.1}}^{PLF} < \underline{\underline{18.6}}^{PLF}$ ∴ LESS WIND PRESSURE THAN TIA

LOAD CAPACITY TITAN SECTION #4

LEG

TIA

$$\frac{kL_x}{r_x} = \frac{(1.0)(24)}{0.50} = \underline{\underline{48}}$$

$$\frac{kL_y}{r_y} = \frac{(1.0)(24)}{(0.71)} = \underline{\underline{33.8}}$$

$$\left(\frac{kL}{r}\right)_{FTB} = \underline{\underline{129.5}} \leftarrow \text{CONTROLS}$$

$$P'_{Au} = \underline{\underline{4.5^k}} \quad [A-08,\ A-09,\ A-10]$$

ULTIMATE CAPACITY BASED ON $F_a A_g$

$$P_{Au} = \frac{23}{12}(8.9^{ksi})(0.38''^{\square}) = \underline{\underline{6.5^k}}$$

HORIZONTAL FORCE APPLIED TO EACH SECTION (TRIANGULAR TOWER):
per section 2.3 of TIA/EIA-222-F.

TEP Job No.: 03190
Date: 08/02/2003
Calculated by: MLL

8/21/2003

Input - TOWER PARAMETERS: ANW Tower Section #7

$h := 100 \cdot ft$ = Total height of structure (ft)

$V := 70 \cdot mph$ = Basic wind speed for the structure location (mph)

$psf \equiv \frac{lbf}{ft^2}$

Input - SECTION PARAMETERS:

$z := 75 \cdot ft$ = Height above average ground level to midpoint of section, appurtenance or guy (ft)

$A_F := 5.97 \cdot ft^2$ = Projected area of flat structural components in one face (ft^2) $A_F = 0.555\ m^2$

$A_R := 0.00 \cdot ft^2$ = Projected area of round structural components in one face (ft^2) $A_R = 0\ m^2$

$A_G := 20.51 \cdot ft^2$ = Gross area of one tower face as if the face were solid (ft^2) $A_G = 1.905\ m^2$

$D_F := 1.0$ = Wind direction factor for flat structural components (Table 2)

$D_R := 1.0$ = Wind direction factor for round structural components (Table 2)

Input - APPURTENANCE PARAMETERS:

$C_A := 0.00$ = linear or discrete appurtenance force coefficient (Table 3)

$A_A := 0.00 \cdot ft^2$ = projected area of linear or dicrete appurtenance (ft^2)

Output - Exposure Coefficient, Kz (2.3.3):

$K_z := \left(\frac{z}{33 \cdot ft}\right)^{\left(\frac{2}{7}\right)}$ $K_z := if(K_z < 1.0, 1.0, K_z)$ $K_z := if(K_z > 2.58, 2.58, K_z)$ $K_z = 1.264$

Output - Velocity Pressure, qz (2.3.3):

$q_z := 0.00256 \cdot K_z \cdot \frac{V^2}{mph^2} \cdot psf$ $q_z = 15.86 \circ psf$

$q_z = 759.387 \circ Pa$

Output - Gust response factor for fastest-mile basic wind speed, Gh (2.3.4):

$G_H := 0.65 + \frac{0.60}{\left(\frac{h}{33 \cdot ft}\right)^{\left(\frac{1}{7}\right)}}$ $G_H := if(G_H < 1.00, 1.00, G_H)$ $G_H := if(G_H > 1.25, 1.25, G_H)$ $G_H = 1.162$

Output - Structure force coefficient, Cf (2.3.5):

$e := \frac{(A_F + A_R)}{A_G}$ $e := if(e > 1, 1, e)$ $e = 0.291$

$C_F := 3.4 \cdot e^2 - 4.7 \cdot e + 3.4$ $C_F = 2.32$

Output - Effective projected area of structural components in one face, Ae (2.3.6):

$R_R := 0.51 \cdot e^2 + 0.57$ $R_R := if(R_R > 1.0, 1.0, R_R)$ $R_R = 0.613$

$A_E := D_F \cdot A_F + D_R \cdot A_R \cdot R_R$ $A_E = 5.97 \circ ft^2$

Output - Horizontal force applied applied to a section of the structure, F (2.3.2):

$F := q_z \cdot G_H \cdot (C_F \cdot A_E + C_A \cdot A_A)$ $F_{limit} := 2 \cdot q_z \cdot G_H \cdot A_G$ $F := if(F > F_{limit}, F_{limit}, F)$ $F = 255 \circ lbf$

$F_{structure} := q_z \cdot G_H \cdot (C_F \cdot A_E)$ $F_{structure} = 255.281 \circ lbf$ $F_{limit} = 756.052 \circ lbf$ $F = 1136 \circ newton$

$F_{appurtenance} := q_z \cdot G_H \cdot (C_A \cdot A_A)$ $F_{appurtenance} = 0 \circ lbf$

TEP Job No.: 03190
Date: 08/02/2003
Calculated by: MLL
JRM 8/7/2003

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ALLOWABLE CONCENTRIC LOADS ON SINGLY SYMMETRIC SHAPES:
AISC ASD SPECIFICATION, 1989 NINTH EDITION & AISC LRFD, 1994

Input - PROPERTIES for 60-degree bent PL3/16x5

CONSTANTS: $kip \equiv 1000 \cdot lbf$ $ksi \equiv \frac{kip}{in^2}$ $G \equiv 11200 \cdot ksi$ $E \equiv 29000 \cdot ksi$

$K_x := 1.00$ = effective length factor in x direction
$K_y := 1.00$ = effective length factor in y direction
$K_z := 1.00$ = effective length factor, lateral torsional buckling
$L_x := 36 \cdot in$ = unbraced length in the x direction
$L_y := 36 \cdot in$ = unbraced length in the y direction
$L_z := 36 \cdot in$ = unbraced length, lateral torsional buckling

$F_y := 50 \cdot ksi$ = specified minimum yield stress

$I_y := 0.68 \cdot in^4$ = Moment of Inertia about y-y axis, major axis, axis of symmetry
$I_x := 0.30 \cdot in^4$ = Moment of Inertia about x-x axis, minor axis

$J := 0.01 \cdot in^4$ = Torsional constant
$A := 0.90 \cdot in^2$ = Cross-sectional area
$y_o := -1.16 \cdot in$ = Coordinate of shear center with respect to shear center
$x_o := 0.00 \cdot in$ = Coordinate of shear center with respect to shear center
$C_w := 0.01 \cdot in^6$ = Warping constant

Output - CALCULATE KL/rx:

$$r_x := \sqrt{\frac{I_x}{A}}$$

$$KLrx := \frac{K_x \cdot L_x}{r_x}$$

$r_x = 0.577 \cdot in$

$KLrx = 62.354$

Output - CALCULATE (KL/ry)eff:

$$r_y := \sqrt{\frac{I_y}{A}} \qquad KLry := \frac{K_y \cdot L_y}{r_y}$$

$KLry = 41.416$

$r_y = 0.869 \cdot in$

$$F_{ey} := \frac{\pi^2 \cdot E}{\left(\frac{K_y \cdot L_y}{r_y}\right)^2}$$

LRFD, EQ A-E3-11

$F_{ey} = 166.863 \cdot ksi$

$$r_o := \sqrt{x_o^2 + y_o^2 + \frac{I_x + I_y}{A}}$$ LRFD, EQ A-E3-8 $r_o = 1.56 \cdot in$

$$F_{ez} := \left[\frac{\pi^2 \cdot E \cdot C_w}{(K_z \cdot L_z)^2} + G \cdot J\right] \cdot \frac{1}{A \cdot r_o^2}$$ LRFD, EQ A-E3-12 $F_{ez} = 52.125 \cdot ksi$

$$H := 1 - \left(\frac{x_o^2 + y_o^2}{r_o^2}\right)$$ LRFD, EQ A-E3-9 $H = 0.447$

$$F_e := \frac{F_{ey} + F_{ez}}{2 \cdot H} \cdot \left[1 - \sqrt{1 - \frac{4 \cdot F_{ey} \cdot F_{ez} \cdot H}{(F_{ey} + F_{ez})^2}}\right]$$ LRFD, EQ A-E3-6 $F_e = 43.601 \cdot ksi$

$$KLry := \pi \cdot \sqrt{\frac{E}{F_e}}$$ $KLry = 81.022$

$KLr := if(KLrx > KLry, KLrx, KLry)$ $KLr = 81.022$

$$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}}$$ $C_c = 106.999$

$$F_{a1} := \frac{\left[1 - \frac{(KLr)^2}{2 \cdot C_c^2}\right] \cdot F_y}{\frac{5}{3} + \frac{3}{8} \cdot \left(\frac{KLr}{C_c}\right) - \frac{(KLr)^3}{8 \cdot C_c^3}}$$ ASD, EQ E2-1 $F_{a1} = 18.807 \cdot ksi$

$$F_{a2} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot (KLr)^2}$$ ASD, EQ E2-2 $F_{a2} = 22.748 \cdot ksi$

$F_a := if(KLr < C_c, F_{a1}, F_{a2})$

$F_a = 18.807 \cdot ksi$ = max. allowable compressive stress

Output - CALCULATION OF Pall & Tall:

$P_{all} := F_a \cdot A$ — $P_{all} = 16.927 \cdot kip$ = max. allowable compressive force

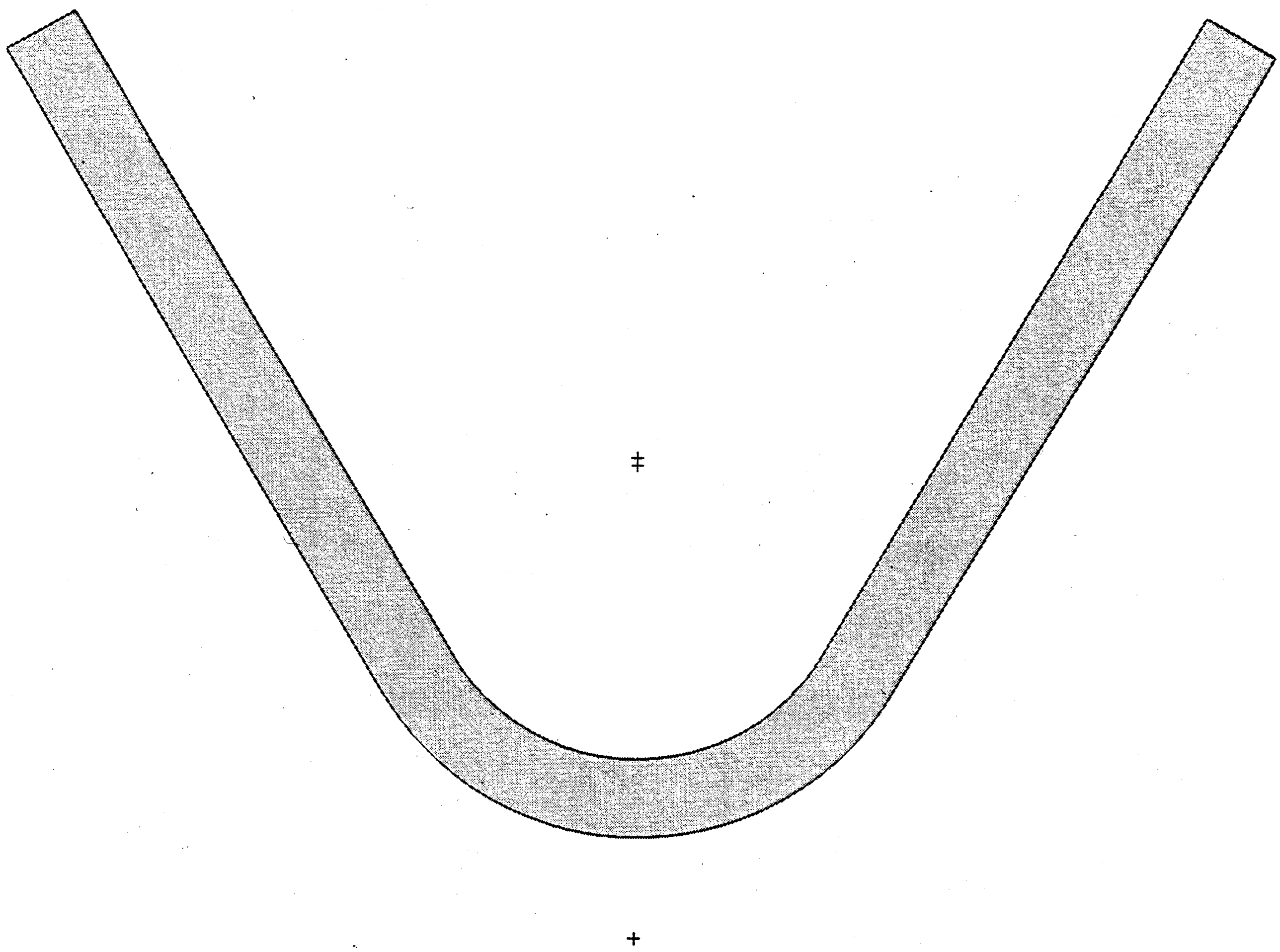
$P'_{all} := \frac{4}{3} \cdot P_{all}$ — $P'_{all} = 22.569 \cdot kip$ = max. allowable compressive force with 1/3 increase in allowable stress

$T_{all} := 0.6 \cdot F_y \cdot A$ — $T_{all} = 27 \cdot kip$ = max. allowable tensile force

$T'_{all} := \frac{4}{3} \cdot T_{all}$ — $T'_{all} = 36 \cdot kip$ = max. allowable tensile force with 1/3 increase in allowable stress

Michael L. Lassiter
Tower Engineering Profession.

Summary Report
A-04



Basic Geometric Properties

Width	2.97 in
H	0.45
Height	1.98 in
Zy	0.71 in^3
Zx	0.45 in^3
I2	0.30 in^4
I1	0.68 in^4
Ixy	0 in^4
Ip	0.98 in^4
Iy	0.68 in^4
Ix	0.30 in^4
J	0.01 in^4
ro	1.56 in
NA-Y	-0.02 in
PNA-X	0 in
rp	1.04 in
ry	0.87 in
rx	0.57 in
Perimeter	9.98 in
A	0.90 in^2
SC Y	-1.16 in
SC X	0 in
SAy	0.90 in^2
SAx	0.90 in^2
Sy Right	0.46 in^3
Sy Left	0.46 in^3
Sx Bottom	0.33 in^3
Sx Top	0.28 in^3
Cw	0.01 in^6
Theta	90.0001 deg
Centroid Y	0 in
Centroid X	0 in

Applied Loads

VY	0 lb
VX	0 lb
MY	0 lb-in
MX	0 lb-in
P	0 lb
T	0 lb-in

Advanced Properties

Elements	792
Nodes	903

Extreme Stress Results

fn (min)	0 psi
fn (max)	0 psi
fvy (min)	0 psi
fvy (max)	0 psi
fvx (min)	0 psi
fvx (max)	0 psi
tau-y (min)	0 psi
tau-y (max)	0 psi
tau-x (min)	0 psi
tau-x (max)	0 psi

HORIZONTAL FORCE APPLIED TO EACH SECTION (TRIANGULAR TOWER):
per section 2.3 of TIA/EIA-222-F.

TEP Job No.: 03190
Date: 08/02/2003
Calculated by: MLL

'nput - TOWER PARAMETERS: Section #4 w/o OVERLAP

$h := 96 \cdot ft$ = Total height of structure (ft)

$V := 70 \cdot mph$ = Basic wind speed for the structure location (mph)

$psf \equiv \frac{lbf}{ft^2}$

Input - SECTION PARAMETERS:

$z := 76 \cdot ft$ = Height above average ground level to midpoint of section, appurtenance or guy (ft)

$A_F := 3.37 \cdot ft^2$ = Projected area of flat structural components in one face (ft^2) $A_F = 0.313\ m^2$

$A_R := 0.00 \cdot ft^2$ = Projected area of round structural components in one face (ft^2) $A_R = 0\ m^2$

$A_G := 12.52 \cdot ft^2$ = Gross area of one tower face as if the face were solid (ft^2) $A_G = 1.163\ m^2$

$D_F := 1.0$ = Wind direction factor for flat structural components (Table 2)

$D_R := 1.0$ = Wind direction factor for round structural components (Table 2)

Input - APPURTENANCE PARAMETERS:

$C_A := 0.00$ = linear or discrete appurtenance force coefficient (Table 3)

$A_A := 0.00 \cdot ft^2$ = projected area of linear or dicrete appurtenance (ft^2)

Output - Exposure Coefficient, Kz (2.3.3):

$K_z := \left(\frac{z}{33 \cdot ft}\right)^{\left(\frac{2}{7}\right)}$ $K_z := if(K_z < 1.0, 1.0, K_z)$ $K_z := if(K_z > 2.58, 2.58, K_z)$ $K_z = 1.269$

Output - Velocity Pressure, qz (2.3.3):

$q_z := 0.00256 \cdot K_z \cdot \frac{V^2}{mph^2} \cdot psf$

$q_z = 15.92 \cdot psf$

$q_z = 762.266 \cdot Pa$

Output - Gust response factor for fastest-mile basic wind speed, Gh (2.3.4):

$G_H := 0.65 + \frac{0.60}{\left(\frac{h}{33 \cdot ft}\right)^{\left(\frac{1}{7}\right)}}$ $G_H := if(G_H < 1.00, 1.00, G_H)$ $G_H := if(G_H > 1.25, 1.25, G_H)$ $G_H = 1.165$

Output - Structure force coefficient, Cf (2.3.5):

$e := \frac{(A_F + A_R)}{A_G}$ $e := if(e > 1, 1, e)$ $e = 0.269$

$C_F := 3.4 \cdot e^2 - 4.7 \cdot e + 3.4$ $C_F = 2.381$

Output - Effective projected area of structural components in one face, Ae (2.3.6):

$R_R := 0.51 \cdot e^2 + 0.57$ $R_R := if(R_R > 1.0, 1.0, R_R)$ $R_R = 0.607$

$A_E := D_F \cdot A_F + D_R \cdot A_R \cdot R_R$ $A_E = 3.37 \cdot ft^2$

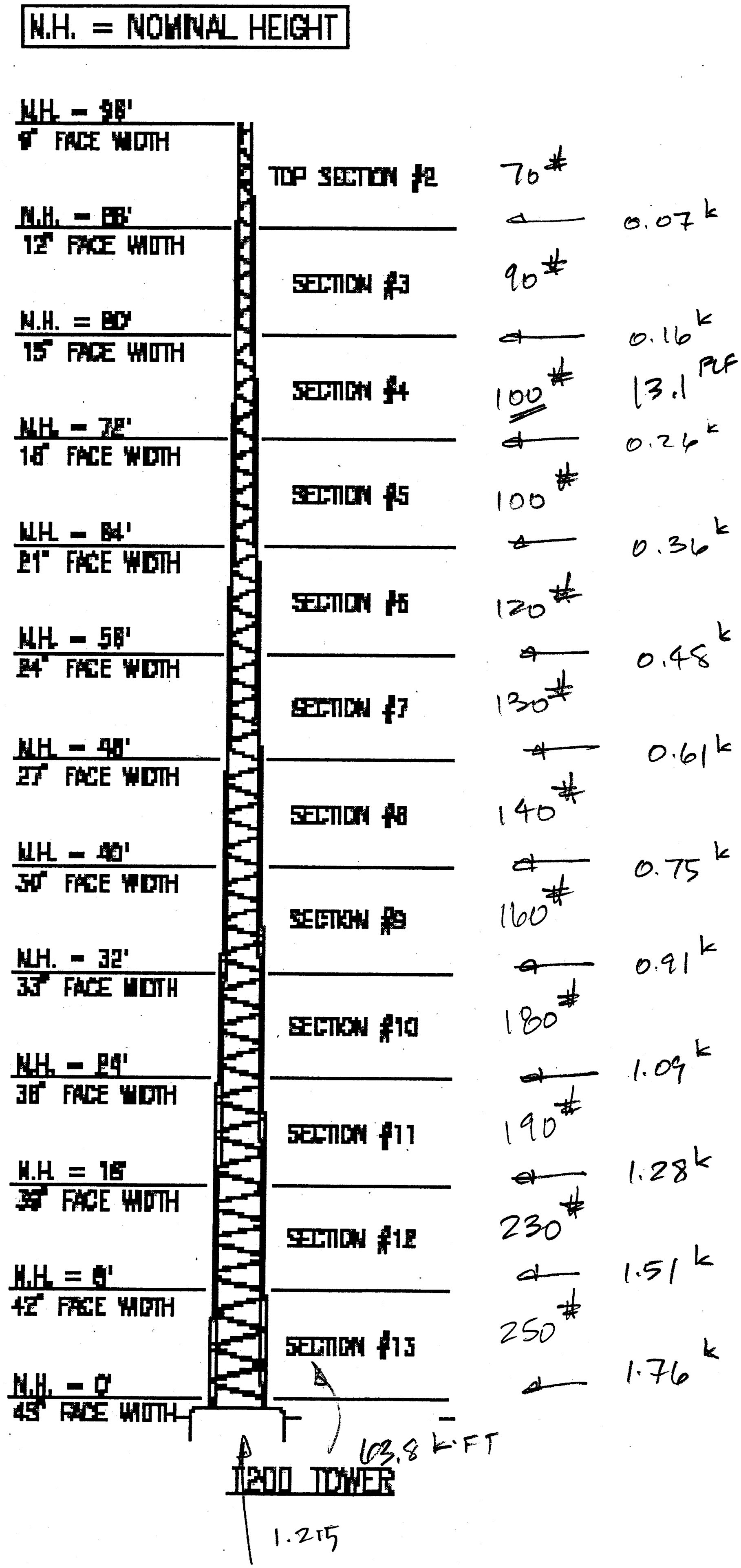
Output - Horizontal force applied applied to a section of the structure, F (2.3.2):

$F := q_z \cdot G_H \cdot (C_F \cdot A_E + C_A \cdot A_A)$ $F_{limit} := 2 \cdot q_z \cdot G_H \cdot A_G$ $F := if(F > F_{limit}, F_{limit}, F)$ $F = 149 \cdot lbf$

$F_{structure} := q_z \cdot G_H \cdot (C_F \cdot A_E)$ $F_{structure} = 148.851 \cdot lbf$ $F_{limit} = 464.463 \cdot lbf$ $F = 662 \cdot newton$

$F_{appurtenance} := q_z \cdot G_H \cdot (C_A \cdot A_A)$ $F_{appurtenance} = 0 \cdot lbf$

A-06


N.H. = NOMINAL HEIGHT
N.H. = 96'
9" FACE WIDTH
TOP SECTION #2
70#
0.07k
N.H. = 88'
12" FACE WIDTH
SECTION #3
90#
0.16k
N.H. = 80'
15" FACE WIDTH
SECTION #4
100#
13.1 PLF
0.26k
N.H. = 72'
18" FACE WIDTH
SECTION #5
100#
0.36k
N.H. = 64'
21" FACE WIDTH
SECTION #6
120#
0.48k
N.H. = 56'
24" FACE WIDTH
SECTION #7
130#
0.61k
N.H. = 48'
27" FACE WIDTH
SECTION #8
140#
0.75k
N.H. = 40'
30" FACE WIDTH
SECTION #9
160#
0.91k
N.H. = 32'
33" FACE WIDTH
SECTION #10
180#
1.09k
N.H. = 24'
36" FACE WIDTH
SECTION #11
190#
1.28k
N.H. = 16'
39" FACE WIDTH
SECTION #12
230#
1.51k
N.H. = 8'
42" FACE WIDTH
SECTION #13
250#
1.76k
N.H. = 0'
45" FACE WIDTH
163.8 k·FT
1200 TOWER
1.215

A-07

Top Moment in Kip Feet : 0.00 **Top Shear in Kips :** 0.00

Height (feet)	88	80	72	64	56	48	40	32	24	16	8	0
Safety Factors	7.11	3.60	2.82	2.20	2.27	2.03	1.83	1.68	1.99	1.90	1.77	1.68
The following additional technical data may be useful to engineers.												
Shear Kips	0.07	0.16	0.26	0.36	0.48	0.61	0.75	0.91	1.09	1.28	1.51	1.76
Leg Load Kips	0.33	1.10	2.17	3.44	4.91	6.54	8.32	10.28	12.42	14.75	17.29	20.05
Face Width	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75
Point load (kips)	0.07	0.09	0.09	0.11	0.12	0.13	0.14	0.16	0.17	0.20	0.22	0.25

The Angular Deflection at the Tower top is:(in Degrees) **1.39**

TOWER REACTION AT BASE: Overturning Moment(Kip-Feet) **63.80**

Total Shear in Kips **1.76**

Total Torque in Kips-feet **0.00**

TEP Job No.: 03190
Date: 08/02/2003
Calculated by: MLL
JRL 8/7/2003

TOWER ENGINEERING PROFESSIONALS

ALLOWABLE CONCENTRIC LOADS ON SINGLY SYMMETRIC SHAPES: AISC ASD SPECIFICATION, 1989 NINTH EDITION & AISC LRFD, 1994

Input - PROPERTIES for 60-degree bent PL3/32x4 3/16

CONSTANTS: $kip \equiv 1000 \cdot lbf$ $\quad ksi \equiv \frac{kip}{in^2}$ $\quad G \equiv 11200 \cdot ksi$ $\quad E \equiv 29000 \cdot ksi$

$K_x := 1.00$ = effective length factor in x direction

$K_y := 1.00$ = effective length factor in y direction

$K_z := 1.00$ = effective length factor, lateral torsional buckling

$L_x := 24 \cdot in$ = unbraced length in the x direction

$L_y := 24 \cdot in$ = unbraced length in the y direction

$L_z := 24 \cdot in$ = unbraced length, lateral torsional buckling

$F_y := 32 \cdot ksi$ = specified minimum yield stress

$I_y := 0.19 \cdot in^4$ = Moment of Inertia about y-y axis, major axis, axis of symmetry

$I_x := 0.09 \cdot in^4$ = Moment of Inertia about x-x axis, minor axis

$J := 0.001 \cdot in^4$ = Torsional constant

$A := 0.38 \cdot in^2$ = Cross-sectional area

$y_o := -0.96 \cdot in$ = Coordinate of shear center with respect to shear center

$x_o := 0.00 \cdot in$ = Coordinate of shear center with respect to shear center

$C_w := 0 \cdot in^6$ = Warping constant

Output - CALCULATE KL/rx:

$$r_x := \sqrt{\frac{I_x}{A}}$$

$r_x = 0.487 \cdot in$

$$KLrx := \frac{K_x \cdot L_x}{r_x}$$

$KLrx = 49.315$

Output - CALCULATE (KL/ry)eff:

$$r_y := \sqrt{\frac{I_y}{A}} \qquad KLry := \frac{K_y \cdot L_y}{r_y}$$

$KLry = 33.941$

$r_y = 0.707 \cdot in$

$$F_{ey} := \frac{\pi^2 \cdot E}{\left(\frac{K_y \cdot L_y}{r_y}\right)^2}$$

LRFD, EQ A-E3-11

$F_{ey} = 248.454 \cdot ksi$

$r_o := \sqrt{x_o^2 + y_o^2 + \frac{I_x + I_y}{A}}$ LRFD, EQ A-E3-8 $r_o = 1.288 \cdot in$

$F_{ez} := \left[\frac{\pi^2 \cdot E \cdot C_w}{(K_z \cdot L_z)^2} + G \cdot J\right] \cdot \frac{1}{A \cdot r_o^2}$ LRFD, EQ A-E3-12 $F_{ez} = 17.772 \cdot ksi$

$H := 1 - \left(\frac{x_o^2 + y_o^2}{r_o^2}\right)$ LRFD, EQ A-E3-9 $H = 0.444$

$F_e := \frac{F_{ey} + F_{ez}}{2 \cdot H} \cdot \left[1 - \sqrt{1 - \frac{4 \cdot F_{ey} \cdot F_{ez} \cdot H}{(F_{ey} + F_{ez})^2}}\right]$ LRFD, EQ A-E3-6 $F_e = 17.072 \cdot ksi$

$KLry := \pi \cdot \sqrt{\frac{E}{F_e}}$ $KLry = 129.481$

$KLr := if(KLrx > KLry, KLrx, KLry)$ $KLr = 129.481$

$C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}}$ $C_c = 133.748$

$F_{a1} := \frac{\left[1 - \frac{(KLr)^2}{2 \cdot C_c^2}\right] \cdot F_y}{\frac{5}{3} + \frac{3}{8} \cdot \left(\frac{KLr}{C_c}\right) - \frac{(KLr)^3}{8 \cdot C_c^3}}$ ASD, EQ E2-1 $F_{a1} = 8.874 \cdot ksi$

$F_{a2} := \frac{12 \cdot \pi^2 \cdot E}{23 \cdot (KLr)^2}$ ASD, EQ E2-2 $F_{a2} = 8.907 \cdot ksi$

$F_a := if(KLr < C_c, F_{a1}, F_{a2})$

$F_a = 8.874 \cdot ksi$ = max. allowable compressive stress

Output - CALCULATION OF Pall & Tall:

$P_{all} := F_a \cdot A$ $P_{all} = 3.372 \cdot kip$ = max. allowable compressive force

$P'_{all} := \frac{4}{3} \cdot P_{all}$ $P'_{all} = 4.496 \cdot kip$ = max. allowable compressive force with 1/3 increase in allowable stress

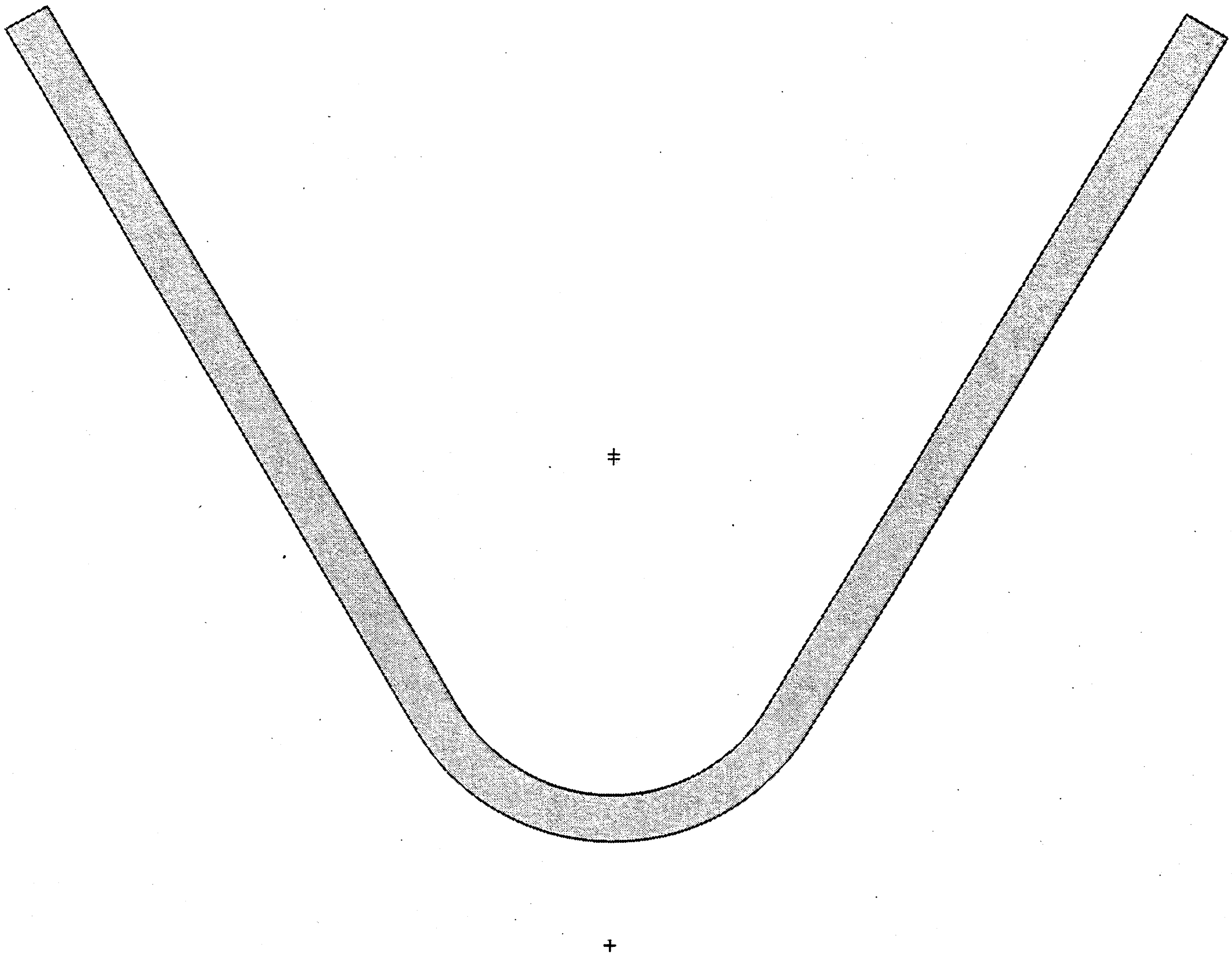
$T_{all} := 0.6 \cdot F_y \cdot A$ $T_{all} = 7.296 \cdot kip$ = max. allowable tensile force

$T'_{all} := \frac{4}{3} \cdot T_{all}$ $T'_{all} = 9.728 \cdot kip$ = max. allowable tensile force with 1/3 increase in allowable stress

Michael L. Lassiter
Tower Engineering Profession.

Summary Report

A-10



Basic Geometric Properties

Width	2.39 in
H	0.43
Height	1.68 in
Zy	0.24 in^3
Zx	0.17 in^3
I2	0.09 in^4
I1	0.19 in^4
Ixy	0 in^4
Ip	0.29 in^4
Iy	0.19 in^4
Ix	0.09 in^4
J	0.00 in^4
ro	1.32 in
PNA-Y	-0.01 in
PNA-X	0 in
rp	0.86 in
ry	0.71 in
rx	0.50 in
Perimeter	8.35 in
A	0.38 in^2
SC Y	-0.99 in
SC X	0 in
SAy	0.38 in^2
SAx	0.38 in^2
Sy Right	0.16 in^3
Sy Left	0.16 in^3
Sx Bottom	0.12 in^3
Sx Top	0.10 in^3
Cw	0 in^6
Theta	90 deg
Centroid Y	0 in
Centroid X	0 in

Applied Loads

VY	0 lb
VX	0 lb
MY	0 lb-in
MX	0 lb-in
P	0 lb
T	0 lb-in

Advanced Properties

Elements	750
Nodes	873

Extreme Stress Results

fn (min)	0 psi
fn (max)	0 psi
fvy (min)	0 psi
fvy (max)	0 psi
fvx (min)	0 psi
fvx (max)	0 psi
tau-y (min)	0 psi
tau-y (max)	0 psi
tau-x (min)	0 psi
tau-x (max)	0 psi