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Welding Design and Construction
By CHARLES L. SAMMONS and JOHN H. STEWART, C.E.

(Continued from the November issue)

FIGURE 3
Failure of Section with Shelton Web System

Total Length of the Cables When Stress in X is a Maximum

Total length—Initial length + stretch from wind stress
Z = 330.202 — .256 = 329.946 feet

Formula (1) is arranged in a slightly different form to make the solution of the deflection of point A possible. Dividing (1) by sec θ gives

\[
\frac{L}{\sec \theta} = a + \frac{(8)(a)(\Delta)^2}{(3)(a)^2(\sec \theta)} x (a) \cos \theta = a^2 + \frac{(8)(\Delta)^2}{3 \sec \theta}
\]

The horizontal distance from the ground attachment of the guy to the vertical projection of point A on the ground now becomes \(a + d\) as shown in figure 6.

Substituting \(a + d\) for \(a\) and transposing, and solving the quadratic gives the horizontal deflection \(d\).

\[
(3)(a + d)^2 - L \cos \theta (a + d) + \frac{8\Delta^2}{3 \sec \theta} = 0
\]

The fact that \(\Delta\) is now actually \(w(a + d)^2\) does not however make enough difference to warrant this refinement.

The component of the deflection parallel to guy X is \(d_x\).

\[
(a + d_x)^2 = 330.607 \cos \theta (a + d_x) + 8(2.23)^2
\]

Completing the square, \((270 + d_x) - 135.188 = 135.166 d_x = .354\) feet
The component of the deflection parallel to guy Y is \( d_y \approx 330.404 \cos \theta (a+d) + 8(3.56)^2 \)
\[
\frac{(270+d_y)}{3(\sec \theta)} - 135.105 = 135.050
\]
\( d_y = 0.155 \) feet

\[
d_z = 0.354 + 0.155 = 0.509 \) feet
\]
Equation (3) is solved for \( \Delta^2 \)

\[
\Delta^2 = \frac{3 \sec \theta (a+d) - (a+d) \cos \theta}{8}
\]
\[
\Delta_z = 3 \sec \theta (270 - 509) (329.946 \cos \theta)
\]
\[
\Delta_z = 6.711 (269.491) (0.345) = 7.80
\]
\( 8 \)
\( \Delta_z = 8.82 \) feet

\[
H_z = \frac{(w)(a)}{8} = 2.32(269.491)^2 = 2390 \text{ ft.}
\]
\[
\frac{8 \Delta}{8(8.82)} = 192 \times 12 = 43 \text{ lbs. per foot of tower}
\]

**FIGURE 5**

*Tower Moment and Shear Diagrams*

Horizontal Component of Initial Stress—\( H_x \) the Horizontal Component of the Relief of Initial Tension.

or \( 6870-2390=4480 \) lbs. horiz. comp. of relief of initial tension.

\[
4480 = 5480 \text{ lbs. relief of initial tension in guy Z.}
\]

\[
5480 \text{ lbs. compares very closely with the assumed relief of the initial tension, 5470 lbs., in guy Z.}
\]

\[
\tan \phi = \frac{(w)(a)}{2xH_x} = \frac{2.32 \times 270}{2 \times 4980} = 0.0331
\]

\( \phi = 1^\circ 54' \)

Maximum stress in guy X—\( \frac{H_x}{\cos (\theta + \phi)} \)

Max X—\( \frac{9480}{\cos (35^\circ 08' + 1^\circ 54')} = 11,870 \) lbs. tension.

The allowable working stress in a high strength \( \frac{7}{8} \)-inch cable is 13,200 lbs. Therefore a \( \frac{7}{8} \)-inch cable is safe for the wind loading specified (25 lbs/sq. ft.).

**Stresses in the Mast Above the Guy Attachment**

The approximate dead weight per panel of tower (54 inches) is calculated below.

1\( \frac{7}{8} \)-inch legs, \( \pi \times 1.875 \times 3 \times 4.5 \times 490 = 126 \) lb/panel

1-inch diameter webs, \( \pi \times 1 \times 3 \times 5 \times 490 = 40 \) lb/panel

\( \frac{7}{8} \)-inch diameter webs, \( \pi \times 3 \times 51 \frac{1}{4} \times 490 = 26 \) lb/panel

Total 192 lb/panel
For calculations 45 lbs. of dead weight per foot of tower is used.

The section of the tower above A, the point of guy attachment, acts as a cantilever.

Moments are taken about point A.

\[
3.464F = (w) (1) = 25(100) = 25000 \text{ ft-lbs.}
\]

\[
F = \text{Max. comp.} = \text{Max. tension} = 36,150 \text{ lbs.}
\]

The dead load above point A, \[= 45 \times 100 = 4500 \text{ lbs. total.}
\]

Maximum compression = 36,150 + 1500 = 37,650 lbs/column.

Maximum tension = 36,150 - 1500 = 34,650 lbs/column.

Try a \(1\frac{7}{8}\)-inch round bar, the minimum allowed by L

\[
R = \text{diameter} = 1\frac{7}{8} = 15/32 \text{ inches}
\]

\[
L = 54/15/32 = 115
\]

Allowable compression stress = 10,590x1\frac{5}{8} = 14,110 lb/sq. in.

Required area = 37,650 = 2.67 sq. in.

Area furnished by a \(1\frac{7}{8}\)-inch bar = 2.76 sq. in.

Stresses in the Mast Below the Guy Attachment

The maximum compression occurs in column Z when the wind is blowing. Column Z is subjected to the wind compression, the dead weight, and the pull of guy Z.

\[
\phi = \tan^{-1}(w) = 2.32 \times 270 = 1311
\]

\[
2xH = 2000
\]

\[
\tan \phi = 2390
\]

\[
H_x = 2390 \text{ lbs.}
\]

\[
H_x \tan(42^\circ 37') = 2390 \times 0.92008 = 2200 \text{ lbs.}
\]

Compression due to wind on tower = 36,150 lbs.

One-third of dead load above A = 1,500 lbs.

Total Compression = 39,850 lbs.

Required area = 39,850 = 2.82 sq. in.

Area furnished by a \(1\frac{7}{8}\)-inch round bar = 2.76 sq. in.

Although a \(1\frac{7}{8}\)-inch bar is stressed slightly above the allowable it will be used here and throughout the tower because the use of a full panel length for L is a conservative measure as was determined in the experimental investigation, even though a full panel length was recommended. Furthermore, a \(1\frac{7}{8}\)-inch bar furnishes adequate cross sectional area at every other critical point in the tower and the use of a 2-inch round bar for the column members would constitute a needless expenditure of steel.

If a given size bar is satisfactory in compression it is not necessary to check it in tension for the maximum tensile stress is equal to the maximum compressive stress minus the dead weight and the allowable tensile stress is 20,000x1\frac{5}{8} lbs./sq. in. which exceeds considerably the allowable compressive stress.

From the shear diagram, figure 5, the maximum positive moment is seen to occur 68.80 feet above the ground.

(Continued on page 24)
WELDED RADIO TOWER
(Continued from page 11)

The Design of the Short Web Members

The webs in a given plane receive the maximum stress when the wind blows parallel to the plane of the webs and the shear in the plane is 2/3 of the total wind force. From the shear diagram, figure 5, the maximum shear = 3030 lbs.

Maximum shear in the webs = \( \frac{2}{3} \times 3030 = 2020 \) lbs.

Maximum stress in the webs = \( \frac{2020 \times 51.25}{48} = 2160 \) lbs. (t) or (c).

Try a \( \frac{1}{8} \)-inch web, the minimum allowed by the L ratio.

Area = 0.601 sq. in.

\[
\begin{align*}
R & = D = 7 \\
\frac{4}{7} & = 0.571
\end{align*}
\]

Allowable unit stress = 7.22 x 4/3 = 9.64 kips/sq. in.

Allowable compression in web = 9.64 x 601 = 5.79 kips.

Maximum compression in web = (see above) = 2.16 kips.

The Design of the Long Web Members

Maximum shear in the webs = \( \frac{2}{3} \times 3030 = 2020 \) lbs.

Maximum stress in the webs = \( \frac{2020 \times 60}{48} = 2525 \) lbs (t) or (c).

Try a 1-inch web, the minimum allowed by the L ratio.

Area = 0.785 sq. in.

\[
\begin{align*}
R & = D = 1 \\
\frac{4}{4} & = 1
\end{align*}
\]

Allowable unit stress = 7.01 x 4/3 = 9.35 kips/sq. in.

Allowable compression in web = 9.35 x 785 = 7.34 kips.

Maximum compression in web = (see above) = 2.58 kips.

The Design of the Web Weld

For the maximum economy of steel the web weld is designed to develop the full allowable compressive strength of the web members.

The exact perimeter of the web weld cannot be determined because the weld is approximately elliptical in shape as can be seen in figure 7. It is on the safe side to take the length of the weld as the circumference of a circle whose diameter is the diameter of the web plus the distance to the center of gravity of the weld.

A weld is weaker in tension than in compression. Therefore the weld is designed for the greatest tension that is possible in the web without exceeding the allowable compression that is acting simultaneously in the two adjacent webs in the same plane.

The allowable tension or shear through the throat of a fillet weld is 13,600 lbs/sq. in. (A. W. S. Code). It is necessary to find the condition that will give the maximum tensile stress in a web member.

The allowable compression in the long web = 7.34 kips

The simultaneous tension in the adjoining short web is,

\[
\frac{7.34 \times 48 \times 51.25}{60} = 6.26 \text{ kips (t)}
\]

The allowable compression in the short web = 5.79 kips

The simultaneous tension in the adjoining long web is,

\[
\frac{5.79 \times 48}{60} = 6.78 \text{ kips web}
\]

A \( \frac{1}{4} \)-inch fillet weld is recommended as the minimum size weld for this type of outdoor structure. A \( \frac{1}{4} \)-inch weld is tried for the short web—diameter \( \frac{7}{8} \) inch.

The allowable tension per inch of weld, \( 0.707 \times 13,600 \times 1 \frac{1}{4} = 2400 \) lbs.

The diameter to center of gravity of the weld, \( (\frac{7}{8} + \frac{1}{4} + \frac{1}{4} + \frac{1}{4} x 1 \frac{1}{4} ) = 1.042 \) inches.

Circumference of the weld = (1.042) \( \pi = 3.28 \) inches.

Allowable tension on the weld, \( 3.28 \times 2400 = 7.88 \) kips in short web.

Max. tension on the weld = 6.26 kips in short web.

A \( \frac{1}{4} \)-inch weld is tried for the long web—diameter 1 inch.

The allowable tension per inch of weld, \( 0.707 \times 13,600 \times 1 = 2400 \) lbs.

The diameter to center of gravity of the weld, \( (1 + \frac{1}{4} + \frac{1}{4} + \frac{1}{4} x 1 ) = 1.167 \) inches.

Circumference of the weld = (1.167) \( \pi = 3.66 \) inches.

Allowable tension on the weld, \( 3.66 \times 2400 = 8.79 \) kips in long web.

Max. tension on the weld = 6.78 Kips in long web.

Design of the Connection Plate Weld

The connection plate weld is designed to develop the full tensile strength of the column.

The maximum tension that occurs in the tower leg is 34,650 lbs. The weld is designed for the
1. **Better Steels!** This country needs aircraft that fly high and far... and hit hard. It needs ships in great numbers. It needs tanks that can take it when the going gets tough. It needs equipment to outperform any on earth. All these things require many special steels. Such steels with needed properties are created through the use of alloys. Basic peacetime research by ELECTROMETALLURGICAL COMPANY, a Unit of UCC, has developed many important steels and the alloys to make them, such as chromium, silicon, manganese, vanadium, tungsten, calcium, and columbium... all vital today.

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Units of UCC do not make steel. They do make ferro-alloys used to purify and give special properties to steel. They also make nonferrous alloys which, because of their exceptional resistance to wear, heat, and corrosion, are used as cutting tools, hard-facing welding rods, and for other purposes. UCC research and developments mean ever-new and improved alloys for industry...and ever-better products for you.

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**Filling Prescriptions with Steel!**
maximum allowable tension which is 20,000 x 2.76 = 55,200 lbs.

A 7/8-inch fillet weld is tried. The column diameter is 1 7/8 inches. See figure 7.

The allowable tension per inch of weld, 
\[(.707 \times 13,600 \times \frac{7}{8}) = 8400 \text{ lbs.}\]

The diameter to the center of gravity of the weld,
\[\left(\frac{1}{7} + \frac{7}{8} \times \frac{7}{8}\right) = 2.458 \text{ inches.}\]

Circumference of the weld = \(2.458 \times \pi = 7.73 \text{ inches.}\)

Allowable tension on the weld,
\[7.73 \times 8400 = 64,900 \text{ lbs. tension.}\]

Allowable tension in the column = 55,200 lbs.

A 7/8-inch weld is therefore sufficient to develop the full tensile strength of the column and transfer this tension to the connection plate.

Design the bolts to develop the full strength of the tower column or 55,200 lbs. tension.

\[55,200 \div 2.07 = 26,500 \text{ sq. in. required root area.}\]

\[(20,000) \div 3 = 6666 \div 3 \text{ sq. in. required root area.}\]

Four 1-inch bolts with .551 root area = 2.204 sq. in.

Five 7/8-inch bolts with .419 root area = 2.095 sq. in.

Seven 3/4-inch bolts with .302 root area = 2.114 sq. in.

Use five 7/8-inch bolts with hex heads and hex nuts. The long dimension of a hex is 1 1/2 inches. A 7-inch connection plate will be required to provide clearance distance between the nut and the weld.

**Design of the Connection Plate Thickness**

The exact behavior of two connection plates bolted rigidly together around the edges and pulled apart at the centers defies exact analytical solution. The bending condition of the plate apparently varies somewhere between full fixity, and a free end condition. The bending moment is taken as the product of the tension in one bolt and the arm measured from the centerline of the bolt to the face of the column member. The bending moment for full fixity is one-half the moment for the free end condition.

For a working moment the average of the free and the fixed end moment is taken.

Tension per bolt = 55,200\(\div 5\) = 11,040 lbs.

\[\begin{align*}
\text{Moment arm} &= 2\frac{7}{8} \div 1\frac{9}{16} = 1 \div 1.562 \text{ inches.} \\
\text{Free end moment} &= 1.562 \times 11,040 = 17,245 \text{ in. lbs.} \\
\text{Fixed end moment} &= 1.562 \times 11,040 = 8,622 \text{ in. lbs.} \\
\text{Average of Moments} &= 12,934 \text{ in. lbs.} \\
\end{align*}\]

From mechanics, Stress = \(\frac{\text{Moment}}{\text{Section Modulus}}\) or \(M = \frac{S}{Z}\)

\[\begin{align*}
\text{Required } Z &= M = 12,934 \div 485 \text{ in.}^2 \\
S &= 20,000 \times \frac{1}{3} \\
\text{If } b &= \text{width, } d &= \text{depth, and } I = \text{Inertia then} \\
Z &= I - (b)(d)^3 - (b)(d)^3 \\
b &= \frac{12}{d} \\
b &= 6 \pi Z \\
I &= 6 \times 0.485 = 2.47 \\
d &= 1.57 \text{ inches.} \\
\text{Use a plate 7 inches in diameter by 1 1/8 inches thick.} \\
\end{align*}\]

It is now evident that the design of a welded steel tower is relatively straightforward mechanics of a not too complicated nature.

It is also evident that the use of welding cuts field work to an absolute minimum and relegates all of the construction work to the shop where the tools of construction are more readily available. The standardization of each section of the tower lends to simplicity of fabrication.

The welded joints give a permanent rigidity and fixity to the tower that is not obtainable with the bolted type of tower.

The Shelton web system, used in this design, does not reduce the L (free length) of the column members, but the twisting and bending in the columns is definitely less than in a similar tower having the conventional Warren web system, a finding that is borne out by the results of extensive testing of the two types of towers.
This is the Chemical Age and many people believe that Germany is the chemical nation. Yet history shows that the synthetic organic chemical industry really started in England, got much of its early impetus in France and has reached its greatest development right here in America.

Germany undoubtedly contributed vast research . . . and vast propaganda . . . but she made the mistake of trying to make it a German monopoly, through Government subsidies and control. There is a profound lesson for us in that.

The American chemical industry, operating on private capital, has pulled out of the test tube miraculous new medicines to save life, super-powerful explosives to overthrow dictators, marvelous new materials that Nature never dreamed of. America now has the greatest organic chemical industry in the world.

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