DESIGN OF ELEVATED STEEL TANKS

by

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THESIS

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1911
I have carefully examined the thesis prepared by MERLE JAY TREES entitled Design of Elevated Steel Tanks, and recommend that it be approved as fulfilling this part of the requirements for the degree of Civil Engineer.

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   (b) Wind Loads.
   (c) Moments.
   (d) Resisting Moment.
   (e) Posts.
   (f) Rods.
   (g) Struts.

2. Sections.
   (a) Tank.
   (b) Posts.
   (c) Rods.
   (d) Struts.
   (e) Anchor Bolts.
   (f) Foundations.

I. INTRODUCTION.

A. General Discussion of Uses and Advantages.

Water may be stored in natural or artificial lakes, reservoirs, tanks, or elevated tanks. The system used is largely determined by local conditions and the purpose for which the storage is wanted.

For irrigation purposes where a large volume of water is required, it is the usual custom, in the absence of a natural body of water, to build dams forming an artificial lake. These lakes are ordinarily at sufficient elevation to enable the distribution of the water by gravity flow. They are filled during the wet season from the surface water coming from higher elevations, and are built large enough to furnish a supply over the dry period of the year.

When a smaller volume of water is required as the reserve supply to a water works plant or fire protection system, it may be stored in steel or concrete reservoirs. The size of the steel reservoir is limited however, on account of the plates becoming so thick that they cannot be rolled and punched economically. Concrete is being used considerably for the construction of large reservoirs; but so far has not proved very satisfactory when built above ground, on account of not remaining water tight. This causes cracks from freezing and a discoloring of the outside walls, which is very displeasing to the eye.

Standpipes, very small in diameter and of great height, were formerly used for water storage when a volume of water was required at a fixed minimum pressure, such as municipal water works plants. This style of construction has practically been abandoned however, on account of the excessive cost for the efficiency obtained, and the extreme variation of pressure. In case there is a natural elevation centrally located that will give the required pressure, a standpipe should be used. It should be made of large diameter and shallow depth so as to obtain a small variation of pressure. If the selected site is not near the center of distribution, investigation should be made as to whether or not the extra cost of pumping, pipe laying, etc., does not exceed the initial cost of building an elevated tank near the center of the system, or at the pumping station. In the absence of a natural elevation, the modern method of storing water to obtain a certain pressure for fire protection or domestic service is to construct a framework and place upon this a tank. This form of structure is usually built of wood, concrete or steel. The concrete construction is very costly, has the disadvantage of being difficult to make water tight, and is not a success from an architectural point of view.

The question of superiority of steel over wood for building tanks, is the same that applies to bridges, buildings and many other structures. Formerly they were all built of wood, but now are being built almost entirely of steel. The principal
advantages of steel construction are greater durability and greater strength of material. The steel tank remains absolutely water tight, when properly caulked, while the wooden tank will continually give trouble by leaking. The life of the steel tank is more than double that of the wooden tank, and the cost of the two is practically the same.

It is the object of this article to give in detail a discussion of the design of elevated steel tanks for water storage.

B. General Description.

An elevated steel tank may be supported on three or more posts and the bottom may be hemispherical, segment of a hemisphere, conical, elliptical, or flat. It must be designed to withstand the maximum gravity and wind stresses. The maximum gravity stresses are those due to the weight of the water in the tank plus the weight of the structure. The wind stresses are those caused by the wind blowing on the tank and the exposed surfaces of the tower. The first data necessary in designing an elevated tank are the capacity and minimum pressure required.

The dimensions of the tank will depend somewhat upon the purpose for which the storage is required, the local conditions, and the kind of bottom used. For tanks with hemispherical or conical bottoms, the diameter is usually made a little less than the vertical height of the cylinder, and this gives a well proportioned tank which is pleasing to the eye.

Railroad tanks for locomotive service are usually proportioned to give the smallest practicable variation in pressure for the volume of water, and this necessitates using a tank of very large diameter and shallow depth. Railroads use the flat and elliptical bottom type of tank almost entirely. Until a few years ago, practically all railroad tanks were built with flat bottoms; but there has lately been introduced a patented elliptical bottom tank, with a special washout arrangement, that is fast replacing the old style of tank.

The height to the extreme bottom of the tank is determined by establishing the minimum pressure that will be required.

The location of the tank sometimes governs whether the posts are vertical or battered; but generally battered posts are used where the tower is at much of an elevation to give stability against the wind forces. Elliptical and flat-bottom tanks, used for railway service, are usually built on vertical posts.

The following information is necessary for designing and bidding on elevated tank work, and should always be supplied to the manufacturer in an inquiry for prices.
6.

1. - Capacity of the tank in gallons.

2. - Height of tower from top of foundation to extreme bottom of tank.

3. - Maximum pressure required.

4. - Accessories contractor is to furnish, such as riser pipe, frost casing, overflow pipe, indicator, pressure gauges, heaters and heater house, foot elbows, gate valves, ladders, foundations, etc.

5. - Sizes of riser and overflow pipes.

6. - If tank is for sprinkler purposes, the name of the company inspecting same.

7. - Give peculiar local conditions and state whether the proposed location is such as to hinder using common standards.

8. - Give exact location as to city in which tank is to be erected.

9. - Transportation facilities and necessary haul to reach site.
II. TABLE OF DIMENSIONS AND PROPERTIES.

Following are given tables of dimensions and properties of a cylinder and the different style tank bottoms:

A - Dimensions.

<table>
<thead>
<tr>
<th>Style</th>
<th>Volume in Cubic Feet</th>
<th>Capacity in Gallons</th>
<th>Contents in Pounds of Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder</td>
<td>$0.7854D^2H$</td>
<td>$5.875D^2H$</td>
<td>$49.10D^2H$</td>
</tr>
<tr>
<td>Hemisphere</td>
<td>$0.2618D^3$</td>
<td>$1.958D^3$</td>
<td>$16.36D^3$</td>
</tr>
<tr>
<td>Segmental Hemisphere</td>
<td>$0.5236H^2\left(\frac{3D^2}{2} - H^2\right)$</td>
<td>$7.84H^2\left(\frac{3D^2}{2} - H^2\right)$</td>
<td>$65.43H^2\left(\frac{3D^2}{2} - H^2\right)$</td>
</tr>
<tr>
<td>Cone</td>
<td>$0.2618D^2H'$</td>
<td>$1.958D^2H'$</td>
<td>$16.36D^2H'$</td>
</tr>
<tr>
<td>Ellipse</td>
<td>$0.131D^3$</td>
<td>$0.960D^3$</td>
<td>$8.188D^3$</td>
</tr>
</tbody>
</table>

B - Properties.

<table>
<thead>
<tr>
<th>Style</th>
<th>Position of Center of Gravity</th>
<th>Area of Diametrical Plane</th>
<th>External Surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder</td>
<td>$\frac{H}{2}$</td>
<td>$HD$</td>
<td>$3.14DH$</td>
</tr>
<tr>
<td>Hemisphere</td>
<td>$0.213D$</td>
<td>$0.3227D^2$</td>
<td>$1.571D^2$</td>
</tr>
<tr>
<td>Segmental Hemisphere</td>
<td>$\frac{d^3}{12A}$</td>
<td>$\frac{1}{3}(acb\frac{D}{2}) - d(H - H'\frac{D}{2})$</td>
<td>$3.14D'H'$</td>
</tr>
<tr>
<td>Cone</td>
<td>$\frac{H'}{5}$</td>
<td>$\frac{DH'}{2}$</td>
<td>$1.57DS'$</td>
</tr>
<tr>
<td>Ellipse</td>
<td>$0.213D$</td>
<td>$0.1964D^2$</td>
<td>$1.24D^2$</td>
</tr>
</tbody>
</table>
III - STRESSES.

A. - Dead Load Stresses.

1. Tower Posts.

\[ N = \text{Number of posts.} \]
\[ W = \text{Total weight of water and structure above point considered.} \]
\[ V = \text{Vertical load per post.} \]
\[ \phi = \text{Angle post makes with a vertical.} \]
\[ \Omega = \text{Angle horizontal diagonal of tower makes with the adjoining strut.} \]
\[ P = \text{Dead load stress in post.} \]

(a) Vertical Posts.

The stress in a vertical post equals the total load divided by the number of posts.

\[ P = \frac{W}{N} \]

(b) Battered Posts.

The stress in a battered post equals the total vertical load multiplied by the secant of the angle the post makes with a vertical.

\[ P = \frac{W}{N} \sec \phi \]

2. Rods.

There is no dead load stress in the rods.

3. Struts.

(a) Vertical Posts.

There is no dead load stress in the strut, when the post is vertical.

(b) Battered Posts.

There is no dead load stress in the strut with a battered post, except when the post changes its angle of inclination. Then the horizontal thrust at the strut line is equal to the difference between the horizontal components of the post stresses above and below the strut.

That is: \[ H_2 = H_1 - H. \]

\[ = (V_1 \tan \phi_1 - V \tan \phi ) \]
4. Anchor Bolts.

(a) Vertical Posts.

There is no horizontal shear on the anchor bolts for vertical posts.

(b) Battered Posts.

For battered posts the horizontal shear on the anchor bolt equals the total vertical load multiplied by the tangent of the angle of inclination the post makes with the vertical.

\[ H = V \times \tan \phi \]

5. Foundations.

In designing the foundation piers it should be noted that the dead load weight of the masonry itself must be considered along with the total load from the post.

\[ W = \text{Total weight of structure above section considered.} \]
\[ H = \text{Height of cylinder in feet.} \]
\[ D = \text{Diameter of cylinder in feet.} \]
\[ d = \text{Diameter of cylinder in inches.} \]
\[ r = \text{Radius of cylinder in inches.} \]
\[ p = \text{Pressure in pounds per square inch.} \]
\[ S = \text{Allowable unit stress on plate.} \]

The forces to be considered in determining the dead load stresses in a cylinder, are the water pressure, weight of the metal, and weight of the ice adhering to the sides of the tank. The total compression on a section of the cylinder cut by a horizontal plane is equal to (1) the weight of the structure above the point under consideration (ordinarily, roof plates and framing, tank plates, cornice work, etc.), (2) the compression due to the wind blowing on the tank, and (3) in cold climates where the tank is not heated, an addition must be made for ice adhering to the shell. The wind stress is usually neglected on account of being very small for ordinary size tanks; but on tall standpipes and smokestacks, it is sometimes large, and must always be investigated.

(a) Compression on Horizontal Joint.

Consider the section cut from cylinder by passing a horizontal plane.

Total load at this point = \( W \).

Horizontal compression per lineal inch equals

\[ C = \frac{W}{3.14 \times D \times 12} \]

(b) Tension on Vertical Joint.

The stress on a vertical section of the cylinder is that produced by hydrostatic pressure.

Pass two horizontal planes through the cylinder a unit distance apart, and normal to this a diametrical plane intersecting the cylinder at points \( X \) and \( Y \). Let the tension at these points equal \( T_X \) and \( T_Y \) per unit height, and, because of symmetry, they are equal to each other, that is, \( T_X \) equals \( T_Y \). The total force tending to rupture the cylinder at points \( X \) and \( Y \) is equal to \( p \times D \times 12 \).

Hence \( T_X + T_Y = p \times D \times 12 \).

\[ T_X = T_Y = \frac{p \times D \times 12}{2} \]

\[ = p \times r. \]
11.

There is no bending moment in the shell, and as we have no horizontal shear, there is tension only on the vertical joint, which is equal to \( T = p x r \).

Pressure per square inch at any point \( p = 0.434 x H \).

Stress per vertical lineal inch of cylinder \( T = \frac{p x D x 12}{2} \)

\[ = \frac{0.434 x H x D x 12}{2} \]

\[ = 2.604 x H x D \]

Net section of plate required \( = \frac{T}{S} \)

\[ = \frac{2.604 x H x D}{S} \]

(c) Stress at Joint of Bottom and Cylinder.

The connecting angle must carry the total weight of the cylinder and the compression due to wind forces. It is therefore necessary to have a sufficient number of rivets in the vertical leg of the angle to carry the total vertical load. The horizontal leg of the angle must be wide enough to give sufficient bearing surface to carry the total load due to the weight of the shell and wind forces. This angle is of sufficient thickness to give bearing pressure for rivets, and is usually a little lighter than the bottom ring of the cylinder.

7. Curved Tank Bottoms.

![Diagram of curved tank bottom](image_url)

- \( D = \) Diameter of cylinder in feet.
- \( D_1 = \) Diameter of section considered in feet.
- \( R_1 = \) Radius of section considered in feet.
- \( p = \) Pressure in pounds per square inch.
- \( \Delta = \) Angle between tangent to curve at point considered and a vertical.
- \( H = \) Head in feet from section considered to top of tank.
- \( H_1 = \) Effective head considering weight of water and metal below section.

(a) Stresses on Horizontal Joint.

Pass a horizontal plane \( x - x \). The total load tending to rupture the bottom on a horizontal joint at the section cut is the weight of the cylinder of water above section \( x - x \), whose base is the section cut from the bottom, plus the weight of the water and metal below the section considered.
Total area of section $x - x = 3.14 \times R_i^2$

Neglecting the weight of metal and water below section considered, total load on $x - x = p \times 3.14 \times R_i^2 \times 144$.

The component tangent to curve $= p \times 3.14 \times R_i^2 \times 144 \times \sec \Delta$

Stress per inch of circumference $= \frac{p \times 3.14 \times R_i^2 \times 144 \times \sec \Delta}{2 \times 3.14 \times R_i \times 12}$

$= \frac{p \times R_i \times 12 \times \sec \Delta}{2}$

$= 0.434 \times H \times \frac{D_i}{2} \times 12 \times \sec \Delta$

$= 1.302 \times H \times D_i \times \sec \Delta$

Considering the weight of water and metal below $x - x$, and substituting the effective head $H$, which we will assume equal to the distance from the section considered to the top of the tank, plus $2/3$ the distance from section $x - x$ to extreme lowest point of bottom; then the stress per lineal inch of circumference on the horizontal joint of a curved tank-bottom equals

$T = 1.302 \times H \times D_i \times \sec \Delta$

(1) For a sphere $\sec \Delta$ reduces to $\frac{D_i}{D_i}$, and the stress per lineal inch of circumference on a horizontal joint equals

$T = 1.302 \times H \times D_i$

(2) For a cone stress on horizontal joint equals

$T = 1.302 \times H \times D_i \times \sec \Delta$

(3) For ellipse stress on a horizontal joint equals

$T = 1.302 \times H \times D_i \times \sec \Delta$

(b) Stresses on Vertical Joint.

Water pressure $= 0.434 \times H$.

Tension in tank plates per lineal inch is equal to the total weight supported by the section $AA$, divided by the circumference
in inches, multiplied by the secant of the angle a tangent at point A makes with the vertical. The resultant of the tensions at A and B acts inward and tends to give compression. The difference between this compression and the outward pressure of the water gives the stress in the shell.

\[ R = \text{Intersection of tangents SP and TQ.} \]
\[ W, = \text{Weight supported at Q.} \]
\[ W, = \text{Weight supported at P.} \]
\[ \alpha = \text{Angle radius of curvature makes with a horizontal.} \]
\[ \Delta = \text{Angle tangent to the curve makes with a vertical.} \]
\[ GP = \text{Radius of curvature, at } P \]

**Fig. 17**

Tension at point P = \[ W, \times \frac{PS}{PM} = \frac{W, \times SR}{RO} \]

Tension at point Q = \[ W, \times \frac{QT}{QN} = \frac{W, \times RT}{RO} \]

If we assume points P and Q one inch apart, the weights \( W, \) and \( W, \) may be taken as equal.

The above tensions are therefore proportional to \( RS \) and \( RT, \) and their resultant is proportional to \( TS. \)

Resultant = \[ W \times \frac{TS}{RO} = \frac{W}{RO} (TO - SO) \]
\[ = W \left( \frac{TN}{QN} - \frac{SM}{PM} \right) \]
\[ = W \left( \frac{NQ}{NG} - \frac{PM}{GM} \right) \]
\[ = W \left( \frac{NQ \times GM - NG \times PM}{NG \times GM} \right) \]

\( NQ \times GM = \text{Rectangle } GKLM. \)
\( NG \times PM = \text{Rectangle } NJHG. \)
\( (GKLM - NJHG) = HJQK + QJPQ + MNJP. \)
\[ = 2 (QJG + QJP + PJG) \]
\[ = 2 GPQ, \text{ for small arc.} \]
\[ \text{Resultant} = \frac{W \times PQ \times GP}{NG \times GM} \]

This resultant multiplied by \( \frac{r}{6.28r} \) will give the compression in the ring of width QP.

\[ \text{Compression per inch} = \left( \frac{W}{6.28} \times \frac{PQ \times GP}{NG \times GM} \right) \div PQ \]

\[ = \frac{W}{6.28} \times \frac{GP}{NG \times GM} \]

When P is close to Q, \( GN = GM \); and we have

\[ \text{Compression per inch} = \frac{W}{6.28} \times \frac{GP}{GM^2} \]

\[ = \frac{W}{6.28} \times GP \sec^2 \Theta \]

\[ = \frac{W \times \sec^2 \Theta}{6.28 \times R} \]

Subtracting from this compression the tension due to the outward pressure of the water we have

\[ \text{Tension} = 0.434 \times H \times r \times \sec \Delta \]

\[ \text{Net Result} = \frac{W \times \sec^2 \Theta}{6.28 \times R} - 0.434 \times H \times r \times \sec \Delta \]

If the result is positive, the stress is compression; and if negative, it is tension.

\( \square \) (1) For a Cone the first term reduces to zero, and the resultant is tension equal to \( 0.434 \times H \times r \times \sec \Delta = 2.604 \times H \times D \times \sec \Delta \)

\( \square \) (2) For a Sphere the first term reduces to half the second, and the resultant is tension equal to \( \frac{W \times r}{2} = 1.302 \times H \times D \).
(c) Stresses at Joint of Bottom and Cylinder.

- $H$ = Head in feet to top of tank from point considered.
- $D$ = Diameter of the cylinder in feet.
- $r$ = Radius of cylinder in inches.
- $\Delta$ = Angle between tangent to curved bottom at joint and a vertical.
- $W$ = Total weight of water in tank plus weight of metal in bottom.
- $h$ = Height of ring around joint.

![Diagram of stresses at joint of bottom and cylinder]

The total vertical load at joint equals the weight of all the water, plus the weight of the metal in the bottom.

Tension per lineal inch in bottom plates $= \frac{W}{6.28r} \sec \Delta$

Horizontal component of tension in bottom plates $= \frac{W \sec \Delta \sin \Delta}{6.28r}$

$$= \frac{W}{6.28r} \tan \Delta$$

Total compression around joint $= \frac{W \tan \Delta \times r}{6.28r}$

$$= \frac{W}{6.28r} \times \tan \Delta$$

Resultant compression deducting internal pressure equals

$$C = (0.159 W \tan \Delta) - (p \times r \times h)$$

$$= (0.159 W \tan \Delta) - (2.604 \times H \times D \times h)$$

Vertical component of tension in bottom plates $= \frac{W \sec \Delta}{6.28r} \times \cos \Delta$

$$= \frac{W}{6.28r}$$

(1) Hemisphere.

Horizontal component of tension in bottom plates $= 0$

Resultant is tension due to outward pressure $= 2.604 \times H \times D \times h$

Vertical component of tension in bottom plates $= \frac{W}{6.28r}$
(2) Segment of a Hemisphere and Cone.

Horizontal component of tension in bottom plates = \( \frac{W}{6.28 r} \tan \Delta \)

Resultant compression on joint deducting internal pressure equals

\[ C = (0.159 W \tan \Delta) - (2.604 x H x D x h) \]

Vertical component of tension in bottom plates = \( \frac{W}{6.28 r} \)

(3) Ellipse.

The outline of an elliptical bottom is formed of curves having different radii. The extreme bottom is the segment of a hemisphere, and where this joins the next curve, there is a horizontal component or thrust that must be taken care of. This compression inward usually increases toward the joint, and must be provided for by using extra heavy plates in the upper part of the tank bottom, or by using a horizontal girder around the joint. For the smaller size tanks sufficient sectional area can be obtained by using an angle or other shape around the joint, but for the larger size tanks, it is necessary to use a girder made up of a web plate and flanges, or to increase the thickness of the bottom plates to obtain the required sectional area. In laying out elliptical bottoms, it is better to use a number of short radii, rather than fewer long ones. This will increase the compression at the connection, but eliminates the bending stresses that would result from a sudden change of curvature.

(1) Hemisphere and Segment of a Hemisphere. (Conclusions)

The maximum stresses on both the horizontal and vertical joints occur at the extreme bottom of the hemisphere, and they are equal at this point. At other points the stress on the horizontal joint is greater than on the vertical joint, on account of the weight of water and metal below the point considered.

(2) Conical Bottom. (Conclusions)

The stress on the horizontal joint reaches a maximum at the joint between the cylinder and the bottom, and decreases to zero at the apex. Neglecting the weight below any point considered, the stress on the horizontal joint is just one-half that on the vertical joint. The stress on the vertical joint also varies from zero at the apex to a maximum at joint of bottom and cylinder.

The cylinder is a special case of the cone in which secant \( \Delta = 1 \) and the stress for the vertical joint reduces to \( T = 2.604 x H x D \).

(3) Elliptical Bottom. (Conclusions)

Let \( R \) equal radius of curvature in the vertical plane and \( R \) the radius in a plane at right angles to the vertical plane. When \( R = 2R \), there is neither tension or compression on vertical joint. When \( R > 2R \), the resultant stress on vertical joint is compression and when \( R < 2R \), there will be tension.
The following tables give the formulae for finding stresses in curved tank bottoms.

### Stresses on Horizontal and Vertical Joints

<table>
<thead>
<tr>
<th>Type of Bottom</th>
<th>Horizontal Joint</th>
<th>Vertical Joint</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hemisphere</td>
<td>$1.302 \times H \times D$</td>
<td>$1.302 \times H \times D$</td>
</tr>
<tr>
<td>Segment of Hemisphere</td>
<td>$1.302 \times H \times D$</td>
<td>$1.302 \times H \times D$</td>
</tr>
<tr>
<td>Cone</td>
<td>$1.302 \times H \times D \times \sec \alpha$</td>
<td>$2.604 \times H \times D \times \sec \alpha$</td>
</tr>
<tr>
<td>Ellipse</td>
<td>$1.302 \times H \times D \times \sec \alpha \left( \frac{W \sec \theta}{3.14 D} - 2.604 x H x D \times \sec \alpha \right)$</td>
<td></td>
</tr>
</tbody>
</table>

### Stresses at Joint of Bottom and Cylinder

<table>
<thead>
<tr>
<th>Type of Bottom</th>
<th>Horizontal Component</th>
<th>Vertical Component</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hemisphere</td>
<td>$0$</td>
<td>$W + 2 \pi r$</td>
</tr>
<tr>
<td>Segment of Hemisphere</td>
<td>$(0.159 W \tan \alpha - 2.604 x H x D x h)$</td>
<td>$W + 2 \pi r$</td>
</tr>
<tr>
<td>Cone</td>
<td>$(0.159 W \tan \alpha - 2.604 x H x D x h)$</td>
<td>$W + 2 \pi r$</td>
</tr>
<tr>
<td>Ellipse</td>
<td>(See preceding discussion)</td>
<td>$W + 2 \pi r$</td>
</tr>
</tbody>
</table>
8. Horizontal Girder.*

\[ n = \text{Number of load points.} \]
\[ M = \text{Variable moment.} \]
\[ M_A = \text{Resisting moment at A.} \]
\[ Q = \text{Total thrust at points.} \]
\[ r = \text{Radius of the center of the girder in inches.} \]
\[ d = \text{Diameter of the center of the girder in inches.} \]

In the case of inclined posts riveted directly to the shell of the tank, there is a horizontal thrust at the post connection. This thrust is usually taken care of by a horizontal or balcony girder made of a web plate with an angle connection to the tank, and angle or channel outer flange which also supports a hand railing. These angles act as the flanges, and the plate as a web of the circular girder. The inward horizontal thrust at the post connection is the horizontal component of the total dead and wind loads supported at this point, minus the outward forces due to the water pressure, acting over the surface covered by the connection. It is considered good practice, however, to neglect the

*Adopted from proof by W.A. Slater, University of Illinois.
outward pressure of the water and this method of design adds to the factor of safety at this point, which is the most important of the whole structure. The balcony girder is usually very narrow in proportion to the diameter of the tank, and for practical purposes, the solution of stresses in a circular ring with radial forces applied at points placed equidistant around the circumference, may be used. Then from this the flange and web stresses of the horizontal girder may be determined.

Consider a ring acted upon by any number of radial forces, Q in the plane of the ring so that all the angles a, b, c, d, etc. Fig. 19a are equal. Because of the symmetry of loading, the tangents at the points of application of the forces will all keep their original directions throughout the loading.

Let the number of load points be n, Fig. 19b. The angle formed between loads will be $\frac{2\pi}{n}$. Let the radius vector for $\phi = 0$ coincide with the line of action of one of the forces. Consider on the unstressed ring a point p, a radius through which makes an angle $d\phi$ with the base line through $\phi = 0$. If now the n loads are applied the ring will be deformed, the point p will take a new position p' and a radius through p' will make some angle $\Delta d\phi$ with the original radius through p. If $d\phi$ becomes the finite angle $\int d\phi$ (or $\phi$), $\Delta d\phi$ becomes $\int \Delta d\phi$ and since at the points of application of the loads, the tangents to the circle keep their original directions the radii of curvature will do the same, that is the angle $\int \Delta d\phi = 0$. This relation is of value later.

According to the common theory of flexure $\frac{M}{EI} = \frac{1}{R}$

where $M =$ moment at any section, $E =$ modulus of elasticity, $I =$ moment of inertia of the section and $R =$ radius of curvature.
\[
\frac{1}{R} = \frac{d\phi}{ds}
\] where \(d\phi\) = the angle between adjacent radii of curvature and 
\(ds\) = a differential length of the curve under consideration. 

\[
\frac{M}{EI} = \frac{d\phi}{ds}.
\]

Now \(d\phi\) for straight beams under load corresponds with 
\(\Delta d\phi\) of the preceding paragraph and while the relation 
\(\frac{M}{EI} = \frac{1}{R} = \frac{d\phi}{ds}\) is rigid for curved beams, only when the depth of beam is infinitely small it may with small error be applied to curved beams of considerable depth. Replacing \(d\phi\) and \(ds\) by their equivalents \(\Delta d\phi\) and 
\(rd\phi\), the following equation is then obtained. 
\(\frac{M}{EI} = \frac{\Delta d\phi}{rd\phi}\) from which 
\(\Delta d\phi = rd\phi \frac{M}{EI}\) But \(\int \Delta d\phi = 0\) as shown in the preceding paragraph. 

\[
\int \frac{2\pi}{n} d\phi = 0 \quad \text{and} \quad \int_{0}^{2\pi/n} d\phi = 0.
\]

This equation applies only to the case of a ring acted on symmetrically in its own plane by \(n\) radial forces, but can be made general by changing the upper limit of the integration to agree with the condition of the problem. This expression, however, is sufficient for the case in hand. Suppose that from 
the ring of Fig. 19b an arc subtending an angle \(\phi\) less than \(\frac{2\pi}{n}\) be considered as the free body of Fig. 19c. It will be in equilibrium under the forces \(P\) and \(V\) and the couples \(M\) and \(M_A\) as shown. Clockwise moment is considered positive. 
\(M_A\) and \(M\) represent the moments under a load point and at any other section within the limits considered. When an equation for \(M_A\) in terms of \(P\), \(V\) and \(n\) has been obtained the values of \(P\) and \(V\) for any value of \(n\) may be substituted and the desired moment obtained. For the special cases in which 
\(n\) is a multiple of 2 but not of 4, 
\[
P = \phi \left( \sin \frac{2\pi}{n} + \sin \frac{4\pi}{n} + \sin \frac{6\pi}{n} + \cdots + \sin \frac{n-2\pi}{n} \right)
\]
and 
\[
V = \frac{\phi}{2}
\]
p provided that the radius vector for \(\phi = 0\) is made to coincide with the line of action of one pair of forces \(\phi\).
Following is a solution for $M_A$.

In Fig. 19c take moments above point $A$ remembering that the moment of a couple is the same about all points in the plane of the couple.

$$M_A = Pr(1 - \cos \phi) + Vr \sin \phi + M = 0.$$  

$$M = Pr(l - \cos \phi) - Vr \sin \phi - M_A$$

$$\int_{0}^{2\pi} M d\phi = 0$$ for symmetrical loading where $n = \text{number of load points}$.

$$\int Pr(l - \cos \phi) - Vr \sin \phi - M_A d\phi = 0$$

$$\left[ Pr(\phi - \sin \phi) + Vr \cos \phi - M_A \right] \phi = 0.$$

$$Pr \left\{ (\frac{2\pi}{n} - 0) - (\sin \frac{2\pi}{n} - \sin 0) \right\} + Vr(\cos \frac{2\pi}{n} - \cos 0) - M_A(\frac{2\pi}{n} - 0) = 0$$

$$Pr \left( \frac{2\pi}{n} - \sin \frac{2\pi}{n} \right) + Vr(\cos \frac{2\pi}{n} - 1) - M_A \frac{2\pi}{n} = 0.$$

$$M_A = \frac{Pr \left( \frac{2\pi}{n} - \sin \frac{2\pi}{n} \right) + Vr(\cos \frac{2\pi}{n} - 1)}{\frac{2\pi}{n}}$$

$$= Pr \left( 1 - \sin \frac{2\pi}{n} \right) + Vr \left( \frac{\cos \frac{2\pi}{n} - 1}{\frac{2\pi}{n}} \right)$$

The maximum shear on the horizontal girder occurs at the load point and is equal to one-half the horizontal thrust.

The following table gives the maximum bending moment and shear in a horizontal girder:

<table>
<thead>
<tr>
<th>Number of Posts</th>
<th>Bending Moment Under Load</th>
<th>Shear Under Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>- 0.068 Qd</td>
<td>0.50 Q</td>
</tr>
<tr>
<td>6</td>
<td>- 0.045 Qd</td>
<td>0.50 Q</td>
</tr>
<tr>
<td>8</td>
<td>- 0.034 Qd</td>
<td>0.50 Q</td>
</tr>
<tr>
<td>10</td>
<td>- 0.027 Qd</td>
<td>0.50 Q</td>
</tr>
<tr>
<td>12</td>
<td>- 0.022 Qd</td>
<td>0.50 Q</td>
</tr>
</tbody>
</table>

The vertical component to be carried by a vertical girder is taken care of in different ways. In the case of masonry towers and sometimes steel towers, the shell of the tank extends below the joint of the bottom, and rests on top of the walls or posts. The bottom flange is usually made of two angles, and vertical stiffeners are placed at several points along the web of the girder, to stiffen up the web around the necessary holes cut out for riveting the bottom.

When the tower posts are riveted directly to the tank, the shell is figured as the web of the vertical girder, and only in exceptional cases will it be found necessary to use vertical stiffeners. In the case of a tank very large in diameter, shallow in depth and with posts spaced at considerable distance apart, it may be necessary to use stiffeners. In figuring the shearing stress on the cylinder, the total effective depth to be used is the distance between centers of gravity of the top and bottom angles of the cylinder. Vertical stiffeners are sometimes used with tanks supported on circular walls by an angle flange, to give stability to the outstanding leg of the shelf angle. It might be added that the vertical stiffeners are only necessary in what are classed as the freaks in tank construction. For standard work the plates necessary to withstand the water pressure, are of sufficient sectional area to take up all shearing stresses that come upon them.

A practical example of this is the 1,200,000 gallon elevated tank used in connection with the water works system for the City of Louisville, Kentucky. This large tank is supported on eight posts; its shell is fifty feet in diameter by sixty-five feet high. The total effective depth of the vertical girder was figured as the distance between the center of gravity of the top and bottom flanges. The concentrated loads on the vertical girder are equal to the total weight of the water, plus the weight of the tank, divided by the number of supporting posts. The maximum shear equals one-half the concentrated load on the vertical girder.
B. - Wind Load Stresses.

The wind stresses on a water tower are those due to the wind blowing on the tank and the exposed surfaces of the supporting structure. There is a great deal of difference of opinion regarding the actual conditions caused by wind forces; but it is generally conceded that the pressure on cylindrical surfaces, such as tanks, is something less than it is on flat surfaces. Some good authorities assume the maximum wind pressure at 30#/ per square foot, on 50% of the diametrical plane of the tank, while others maintain that 40#/ per square foot over 60% of the diametrical plane of the tank should be used. The writer recommends 30#/ per square foot over 60% of the diametrical plane of the tank, which, according to experiments made, represents a wind having a velocity of something over one hundred miles per hour. The total wind load blowing on the tank is assumed to be acting at the center of gravity of the diametrical plane of the tank, and the stresses therefrom to be transmitted through the shell to the top of the tower posts. We, therefore, have compression on the leeward side of the cylinder due to these wind forces. The supporting structure carries its stresses due to the wind load on the tank, and wind pressure on the exposed surfaces, to the foundations. The wind load acting on the tower is assumed to be applied at the panel points, and is usually taken at so many pounds per vertical foot of tower.

1. Tower Posts.

The first step in determining post stresses due to wind loads is to ascertain the center of gravity on the diametrical plane of the tank, and then find the total load acting at this point. The effect on the structure caused by the wind forces, is a tendency to rotate about some axis. In case we have an absolutely rigid structure resting on its foundations without being anchored, the tendency would be either to slide horizontally, or to rotate about an axis passing through the extreme leeward post. It would be just as logical, however, to assume the axis of rotation passing through the extreme windward post, in which case we would have compression in all the posts, instead of tension. Neither of the above assumptions is practical and if the structure is designed so that all members take their proportional stress, and the tower is anchored to properly designed foundations, the axis of rotation will pass through the center of the anchor bolt circle. In the following discussion we will consider all three cases.

\[ R = \text{Radius of section considered in feet.} \]
\[ D = \text{Diameter of section considered in feet} \]
\[ P = \text{Pressure in pounds acting at the center of gravity on tank.} \]
\[ d = \text{Distance of center of gravity of tank to point being considered.} \]
\[ M = \text{Total moment of wind forces at point being considered.} \]
\[ M' = \text{Moment of wind forces at panel point above the one considered.} \]
\[ D' = \text{Diameter in feet at panel point above.} \]
Moment at top of post $M = P \times d$.

Assume the wind blowing in the direction of a horizontal diagonal on a four-post tower, and the axis of rotation $Y - Y$ through the leeward post. The post stresses will be proportional to their distance from the axis $Y - Y$ and we will have tension at points $A$ and $C$ equal to one-half the tension at point $D$.

$$M = (V_d \times 2R) + \left( \frac{V_A + V_C}{2} \times R \right)$$
$$= \frac{3}{2} \times V \times D$$
$$V_d = \frac{2}{3} \times \frac{M}{D} \text{ uplift on windward foundation.}$$
$$V_B = 2V_d = \frac{4}{3} \times \frac{M}{D} \text{ compression on leeward post.}$$

Assume the axis of rotation $Z - Z$ through point $D$ and possibility of failure by the buckling of posts $A, B, C$, then

$$M = (V_B \times 2R) + \left( \frac{V_A + V_C}{2} \times R \right)$$
$$V_c = \frac{2}{3} \times \frac{M}{D} \text{ compression on leeward post.}$$
$$V_d = 2V_b = \frac{4}{3} \times \frac{M}{D} \text{ uplift on windward foundation.}$$

Assume the axis of rotation $X - X$ through the center of the tower then we have tension at $D$ equal to the compression at $B$.

$$M = (V_o + V_b) \times R$$
$$V_o = \frac{M}{D} \text{ uplift on windward foundation.}$$
$$V_B = V_D = \frac{M}{D} \text{ compression on leeward post.}$$

$\text{Assume the wind normal to the side } A - D \text{ and the axis of rotation } Y - Y \text{ through the leeward posts.}$

$$M = (V_A + V_D) \times 1.414 \times R$$
$$V_A = \frac{M}{1.414 \times D} = 0.707 \times \frac{M}{D} \text{ uplift on windward foundation.}$$
$$V_B = V_A = 0.707 \times \frac{M}{D} \text{ compression on leeward post.}$$

Assume the wind normal to the side $A - D$ and the axis of rotation $X - X$ through the center of the tower.
\[ M = (V_A + V_8 + V_c + V_d) \times 0.707 \times R \]
\[ V_A = \frac{M}{1.414 D} = 0.707 \times \frac{M}{D} \text{ uplift on windward foundation.} \]
\[ V_d = V_c = V_8 = V_A = 0.707 \times \frac{M}{D} \text{ compression on leeward post.} \]

As stated above, if the structure is properly designed, the only practical axis of rotation to assume would be one passing through the center of the tower. Then the maximum uplift on the foundation, and compression in the post occur when the wind is blowing in the direction of a horizontal diagonal.

To get the tension in the windward post it would be necessary to subtract the vertical components of the rod stresses. Therefore, the direct tension in the windward post would be equal to the compression in the leeward post in the panel above.

(a) Vertical Posts.

Then in four-post towers having vertical columns with the axis of rotation passing through the center of the tower, we have the maximum post stresses occurring when the wind is blowing in the direction of a horizontal diagonal.

\[ a = \text{Angle between rod and vertical.} \]
\[ \Theta = \text{Angle between rod and adjoining strut.} \]
\[ \phi = \text{Angle post makes with the vertical.} \]
\[ \omega = \text{Angle horizontal diagonal of the tower makes with the adjoining strut.} \]
\[ W = \text{Wind load at panel point.} \]
\[ P = \text{Maximum wind stress on post.} \]
\[ R = \text{Rod stress in panel above strut considered.} \]

Post Compression = \[ \frac{M}{D} \]
Uplift on Foundation = \[ \frac{M}{D} \]
Post Tension = \[ \frac{M'}{D'} \]

(b) Dattered Posts.

For inclined posts the above expressions reduce to the following:

Post Compression = \[ \frac{M}{D} \sec \phi \]
Uplift on Foundation = \[ \frac{M}{D} \sec \phi \]
Post Tension = \[ \frac{M'}{D'} \sec \phi \]
2. Rods.

Assuming the wind in the direction of a horizontal diagonal and the axis of rotation through the center of the tower.

The difference in the vertical components of the wind stresses \( P_A - P_A' \). This difference is equal to the vertical components of the rod stresses.

Because of symmetry \( R_1 = R_2 \)

The vertical components of the rod stresses equal \( 2R_1 \cos \alpha \) when \( \alpha \) equals the angle a rod makes with a vertical.

\[
2R_1 \cos \alpha = P_A - P_A'
\]

\[
R_1 = (P_A - P_A') \frac{\sec \alpha}{2}
\]

Assume the wind normal to the side of the tower and the axis of rotation through its center.

Vertical components of rod stresses \( = (P_A - P_A') \cos \omega \) when \( \omega \) = angle a horizontal diagonal of tower makes with the adjoining strut and \( (P_A - P_A') \) equals the maximum post compression when the wind is taken in the direction of a horizontal diagonal of the tower.

The vertical components of the rod stresses \( = R_1 \cos \alpha \)

\[
R_1 \cos \alpha = (P_A - P_A') \frac{\cos \omega}{\cos \alpha}
\]

\[
= (P_A - P_A') \frac{\sec \alpha}{1.414}
\]

Therefore the latter assumption gives the maximum rod stress for a four-post tower.

3. Struts.

The maximum strut stress equals the horizontal component of the rod stresses in the panel above plus the direct load in the direction of the wind applied at the strut line minus the horizontal component of the wind stress in the post. The maximum strut stresses occur under the same assumption as was made for the maximum rod stresses.
(a) Vertical Posts.

In the case of vertical posts there is no horizontal component of the post stresses, and the strut stress would be

\[ S = W + R \cos \theta. \]

(b) Battered Posts.

For battered posts the stress would be

\[ S = (W + R \cos \theta) - \left(\frac{P_a - P_{a'}}{1.414}\right) \sin \phi \cos \omega \]

4. Anchor Bolts.

(a) Vertical Post.

For vertical post the horizontal shear equals the horizontal component of the rod stress, plus the proportional wind load on the lower panel of the tower, that is carried to the foundations.

\[ H = R \cos \theta + W. \]

(b) Battered Post.

For battered posts the maximum horizontal shear equals the horizontal components of both the dead and wind load stresses in the post.

\[ H = P_a \sin \phi. \]

5. Foundations.

The foundation pier must be designed to carry the total dead and wind load on the post, plus the additional weight of the pier itself.

\[ L = \text{Total dead and wind load on the post plus the weight of the masonry.} \]

\[ S = \text{Allowable bearing pressure on soil.} \]

\[ \text{Base of Pier Required} = \frac{L}{S} \]

The bearing pressure assumed, depends upon the bearing qualities of the soil. The size of the foundations also depends upon the maximum uplift, and the depth of frost line. In some cases of very high towers, where the total load on the foundation is small compared to the uplift, it is necessary to add to the quantity of the foundations to obtain sufficient anchorage. It is
always considered best practice to have the base of the foundations below the frost line, and this would require deep foundations in cold climates adding to the quantity of masonry.


![Diagram of Cylinder](image)

- $D_1 =$ Inside diameter of tank in inches.
- $D =$ Outside diameter of tank in inches.
- $P =$ Pressure in pounds per square foot.
- $H =$ Height of cylinder in feet.
- $T =$ Thickness shell of tank.
- $S =$ Allowable unit stress.

**Compression Tension.** The wind forces acting on the cylinder are assumed at a certain pressure per square foot, over a percentage of the diametrical plane, and to be applied at the center of gravity. Assume a loading of 30# per square foot over sixty per cent of the diametrical plane of the tank. The wind blowing normal to the shell of the tank causes compression on the leeward side, and tension on the windward side of the shell.

The bending moment of the wind is as follows:

Wind Pressure on the Cylinder = $P \times H \times \frac{D_1}{12} \times 0.6$

Bending Moment = \[ \left( \frac{P \times H \times D \times 0.6}{12} \right)^{\frac{H}{2}} \]

\[ = 7.2 \frac{H^2}{24} D^4 \]

\[ = 0.3 \frac{H^2}{24} D^4 (0) \]

The general formula for the moment of resistance to bending of a tubular section is

Resisting Moment \[ = 0.0982 \left( \frac{D^4 - D_1^4}{D} \right) S \]

Hence \[ (D^4 - D_1^4) = D^4 - (D - 2T)^4 \]

\[ = 8 D^3 T \text{ approximately for cases where } T \text{ is very small compared to } D. \]

Therefore \[ R = 0.0982 \left( \frac{8 D^3 T}{D} \right) S \]

\[ = 0.7856 D^2 T S (2) \]

Equating 1 and 2 then \[ T = \frac{0.3 \frac{H^2 D}{D^3}}{0.7856 D^2 S} \]

\[ = 0.382 \frac{H^2 D}{D^3} \]
7. Tables of Wind Stresses.

Assuming the axis of rotation through the center of the tower, the following tables are derived for Post Rod and Strut Stresses, due to wind forces. For towers having an even number of supports, the maximum post compression is obtained by assuming the wind blowing in the direction of a horizontal diagonal of the tower. The maximum Rod and Strut Stresses occur when the wind is taken in a direction parallel to two sides of the tower.

\[
a = \text{Angle between the rod and the vertical.} \\
\phi = \text{Angle between the rod and the strut.} \\
\phi' = \text{Angle post makes with a vertical.} \\
w = \text{Angle the horizontal diagonal of tower makes with the adjoining strut.}
\]

**Posts Vertical.**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>M + 2R</td>
<td>(F_\text{A} - F_\text{B}) 0.707 seca</td>
<td>W + R \cos \phi</td>
</tr>
<tr>
<td>6</td>
<td>M + 3R</td>
<td>(F_\text{A} - F_\text{B}) seca</td>
<td>W + R \cos \phi</td>
</tr>
<tr>
<td>8</td>
<td>M + 4R</td>
<td>(F_\text{A} - F_\text{B}) 1.307 seca</td>
<td>W + R \cos \phi</td>
</tr>
<tr>
<td>10</td>
<td>M + 5R</td>
<td>(F_\text{A} - F_\text{B}) 1.618 seca</td>
<td>W + R \cos \phi</td>
</tr>
</tbody>
</table>

**Posts Battered.**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4 (M + 2R)seca</td>
<td>(F_\text{A} - F_\text{B}) 0.707 seca (W + R \cos \phi) - 0.707(F_\text{A} - F_\text{B}) \sin \phi \cos w</td>
<td>W + R \cos \phi</td>
<td></td>
</tr>
<tr>
<td>6 (M + 3R)seca</td>
<td>(F_\text{A} - F_\text{B}) seca (W + R \cos \phi) - 0.50(F_\text{A} - F_\text{B}) \sin \phi \cos w</td>
<td>W + R \cos \phi</td>
<td></td>
</tr>
<tr>
<td>8 (M + 4R)seca</td>
<td>(F_\text{A} - F_\text{B}) 1.307 seca (W + R \cos \phi) - 0.38(F_\text{A} - F_\text{B}) \sin \phi \cos w</td>
<td>W + R \cos \phi</td>
<td></td>
</tr>
<tr>
<td>10 (M + 5R)seca</td>
<td>(F_\text{A} - F_\text{B}) 1.618 seca (W + R \cos \phi) - 0.32(F_\text{A} - F_\text{B}) \sin \phi \cos w</td>
<td>W + R \cos \phi</td>
<td></td>
</tr>
</tbody>
</table>

---

**Figures:**

- **Four Post.**
- **Six Post.**
- **Eight Post.**
- **Ten Post.**
SPECIFICATION FOR STEEL WATER TOWERS.

GENERAL.

The structure to be built, shall consist of a steel water tank, cylindrical in form, supported on a steel tower, to conform to the general dimensions given on the accompanying sketch.

LOCATION.

The structure shall be located on the site selected by the owner or his authorized agent.

PLANS.

Contractors shall submit with their proposal complete strain sheets, and such detail drawings as shall clearly show the dimensions of all parts, modes of construction, connections, sizes of members, thicknesses of material, etc. Upon the acceptance of the proposal and execution of the contract, all working drawings required by the Engineer must be furnished by the contractor, without additional cost. The contractor shall be required to submit with his bid the weight of the structure including the tower and tank. All detail drawings shall be subject to the approval of the purchaser or his authorized agent.

LOADING.

The structure shall be proportioned to carry the following loads:
1. The weight of the structure itself.
2. The weight of the water in the tank.
3. A wind pressure of not less than thirty pounds per square foot over sixty per cent of the diametrical plane of the tank, and a uniform load of 200 pounds per each vertical foot of tower. The wind forces shall be assumed acting in any direction and members must be proportioned for that direction which shall give the maximum stresses.

UNIT STRESSES.

Compression.

Members not exceeding ninety radii of gyration between supports shall be proportioned for a unit stress of 14000 lbs. per square inch. For members exceeding the above limit use the following reduction formula:

\[ P = 18500 - 50 \frac{L}{R} \]

P = Allowable unit stress in pounds per square inch.
L = Length of member between supports in inches.
R = Least radius of gyration in inches.
No compression member, however, shall have a length exceeding 120 times its least radius of gyration for main posts, or 150 times its least radius of gyration for struts or laterals.

Tension.

Tension members shall be proportioned for a unit stress of 12000 lb. per sq. in. of net section in tank plates and bracing.

Shearing.

Rivets shall be so spaced that the shearing strain shall not exceed 7500 lb. per sq. in. Pins shall be so proportioned that the shearing strain shall not exceed 7500 lb. per sq. in.

Bearing.

The bearing pressure on rivets and pins shall not exceed 15000 lb. per sq. in.

The foundations shall be so proportioned that the pressure shall not exceed the following values:
400 lb. per sq. in. for metal or concrete caps.
125 lb. per sq. in. on concrete through the mass.
28 lb. per sq. in. concrete on earth to be modified as local conditions demand. All the above unit stresses may be increased twenty five per cent for wind loads.

QUALITY OF MATERIAL.

All metal in the structure except the rods which require welding or forging shall be steel. All steel comprising the tank plates and principal parts of the columns must be made by the open hearth process. Other steel may be made by the open hearth or Bessemer process.

The ultimate strength of the structural steel shall be between 55000 and 65000 lb. per sq. in.; elastic limit not less than one half the ultimate strength and the minimum percentage of elongation in eight inches =\((140000 \div \text{ultimate strength})\). Rivet steel shall show an ultimate tensile strength between 48000 and 58000 lb. per sq. in.; otherwise same requirements as above for structural steel.

BENDING TEST.

The bending test specimens must stand bending to a curve whose inner radius is one and a half times the thickness of the sample, without showing any signs of fracture on the convex side of the bent portion. All tests and inspection of material shall be made at the place of manufacture prior to shipment.
CHEMICAL ANALYSES.

The chemical analysis of each heat or melt shall be made by experienced chemists under the direction of the Engineer or his inspector. In steel made by the acid process the phosphorous limit shall be .08 percent, and made by the basic process .04 per cent.

ADDITIONAL TESTS.

The plates must also admit of cold hammering and scarfing to a full edge at the laps without cracking. The failure of said test specimens when taken at random from the finished product of any heat or melt to conform with any or all the above requirements shall be sufficient cause for the rejection of the entire product of such heat or melt.

PLATES.

The plates shall be free from laminations, cinder and any other surface defects. They shall be fully up to the required thickness at the edges and any plates which shall be found more than 2 1/2 per cent short of the required thickness at any point shall be rejected.

INSPECTION.

The engineer or his authorized representative shall have the right at all times to inspect the manufacture of any and all sheets, plates or shapes and all work connected therewith, and the contractor must furnish, free of charge, all necessary appliances to the engineer for the proper performance of his duty, in the carrying out of the requirements of these specifications.

WROUGHT IRON.

Wrought iron used in making rods shall be first class refined iron, known as best bridge iron. The surface must be free from slivers, blisters, cinder spots or other injurious defects, and must be welded together without seams or ragged and torn edges.

DETAILS OF CONSTRUCTION.

The facilities of the manufacturer for handling work shall be investigated before the material is ordered. Compression members shall be of the open type; no closed section to be used. Bearing plates for distributing the pressure over the foundations must be attached to the bottoms of the main posts. They shall be so attached as to distribute the load throughout their entire area. In order to avoid eccentric loading on tower columns, the connections between the columns and the sides of the tank shall be made so that the center of gravity of the column section intersects the center of the connection between the bottom and the sides.
of the tank. All joints in main posts to be made as near as practical to a horizontal strut. Splices are to be made with plates on all sides of the columns with sufficient rivets to thoroughly hold parts together. Batten plates at the end of the members shall not have a length less than the distance between rivet lines connecting them to the channels and the pitch of the rivets in them shall not exceed four diameters of the rivet used. The distance between connections of the lattice or lacing bars to the flange of the channel shall not exceed two times the depth of the member, nor shall they be inclined to the axis of same less than 45°. The thickness of the lacing bars to be not less than 1/50 the distance of center to center of rivets connecting the same to channels. The thickness of lattice bars shall not be less than 1/30 of the distance. Lattice bars shall be riveted at their intersections. The width of lattice and lacing bars shall not be less than 2 1/2 times the diameter of the rivet used. For high towers the columns shall have a batter of 1/5 to 12. The height of the tower shall be the distance from the top of the foundations to the extreme bottom of the tank.

The size of rivets for various size channels and thicknesses of plates are as follows:

\[ \frac{1}{8} \text{ in. rivets for 4 and 5 in. channels.} \]
\[ \frac{1}{8} \text{ " } \frac{3}{8} \text{ " } \frac{5}{16} \text{ " } \frac{3}{16} \text{ " } \frac{1}{4} \text{ " } \frac{1}{4} \text{ in. to 1 in.} \]

In angles and other shapes the diameter of the rivet will generally not be less than 1/4 of the leg of the angle used in the flange. In work that does not require caulking the pitch of the rivets shall never exceed 6 inches or 16 times the thickness of the thinnest outside plate nor be less than three diameters of the rivet. In work that requires caulking the maximum pitch shall never exceed 10 times the thickness of the thinnest plate connected, and shall not be less than three diameters of the rivet. The distance between the edge of any piece and the center of the rivet hole must never be less than 3 1/4 times the diameter of the rivet, except for bars less than 3 1/2 times the diameter of the rivet in width. Plates 1/8 in. thick and over shall be subpunched 3/16" smaller than the diameter of the rivet to be used, and reamed to a diameter 1/16 in. larger than the rivet. No plates shall be used of less thickness than 1/4 in. All plates must be bevel-sheared or planed on the caulking edges. Where butt straps are used they must also be bevel-sheared on the caulking edges. The bottom plates shall always be connected to the inside of the shell of the tank. A "Z" bar or other shape shall be provided around the inside and outside of the shell of the tank near the top to serve as a stiffener to the tank shell, and also as a painters trolley. A curved girder must connect the main posts together where they connect to
the tank. This girder shall be connected to the tank by means of rivets pitched not greater than four diameters, and shall be sufficiently strong to stand the thrust and bending moments induced by the horizontal component of the stress in the posts. Its outer flange shall be supported by a vertical latticed girder, which will also act as a railing about the balcony.

All rods shall be provided with some adjustment for length, and where they are threaded the ends shall be upset to make up for the decreased area, or else the rod shall be of enough greater size throughout its entire length to allow for this deduction. At proper intervals, horizontal rods shall run out from the main posts and connect to the inlet pipe to hold same securely in position. All pipes entering the tank shall have expansion joints (excepting those having a diameter smaller than four inches). This joint is usually fastened at the bottom of the tank with bolts. The tank plates shall be reinforced where the pipes enter the tank.

WORKMANSHIP.

The structure shall be built in accordance with the above plans and the workmanship must be first class, equal to the best in modern shop practice. Plates heated for scarfing must not be hot enough to ignite a piece of dry wood when applied to it. All abutting surfaces of compression members must be planed to an even bearing, so that they will have an equal bearing over their entire area. At the joint between the bearing plate and the column above, the plate need not be planed. It must be carefully straightened however by methods which will not injure it. The diameter of the punch shall not exceed the diameter of the rivet to be used by more than 1/16 of an inch. All holes must be clean cut without torn or ragged edges. Rivet holes must be accurately spaced. The use of drift pins will be allowed only for bringing the several parts together and they must not be driven with such force as to disturb the metal surrounding the holes. Rivets must be driven by pressure tools wherever possible. All rivet heads must be concentric with the holes. The rivets must completely fill the holes and have a height of not less than 5/16 the diameter of the rivet and shall be in full contact with the surface, or be countersunk when required. Built members must when finished be true and free from twists, kinks and open joints between component pieces. The tank is to be made absolutely water tight by caulking only. No foreign substance of any description is to be put into the joints between the plates. All caulking shall be done with a round nosed tool. Caulking around rivets shall not be allowed. Leaky rivets must be cut out and replaced by new ones. The contractor must guarantee the tank against leakage for a period of one year after acceptance of same. If practicable the tank should be tested for leaks before the inside has been painted. Any leaks appearing during the test must be marked and recaulked. All bends in steel except those noted below must be made cold. This particularly applies to the plates for the sides and bottom of the tank.
Detail pieces if necessary may be bent hot, without annealing. If a steel piece in which the full strength is required has been partially heated the whole must subsequently be annealed. All parts shall be adjusted to a perfect fit and shall be marked before leaving the shop. The engineer shall have the right to require driven rivets cut out at random in order to inspect the equality of the workmanship.

LADDER.

There shall be a ladder extending from a point about eight feet above the ground to the balcony line, and from there to the manhole on the roof and down the inside of the tank to the bottom. There shall also be a walk from the post on which the ladder is built to the expansion joint at the bottom of the tank.

ROOF.

The tank is to be provided with a conical steel roof made of plates not less than $\frac{1}{4}$ in. thick. In tanks of diameter up to twenty-five feet these roofs are self supporting, but for a tank having a larger diameter than this, the roof will be supported on radial rafters of light angle or channel shapes. A space shall be left between the top edge of the tank and the roof. The roof shall project over the edge of the tank the width of the balcony girder. A trap door shall be provided in the roof near the ladder, and there shall be a cast iron finial at the top of the tank securely fastened to same.

PAINTING.

Before it leaves the shop, all steel work shall be cleaned of mill scale, dirt, rust and oil, and receive one coat of graphite or equally good paint, except the contiguous surfaces of the tank plates. This portion shall not be painted. All other parts inaccessible after erection shall receive two coats of the above paint before erection. After the tank has been erected and as soon as all leaks have been stopped, all seams are to be repainted. After retouched places have dried, the entire surface except the inside of the tank shall be painted with one coat of the same paint. The final coat on the inside of the tank shall be composed of a heavy mixture of coal tar, Portland cement and kerosene oil or some well known brand of heavy asphalt coating. The paint used is not to be mixed more than twenty-four hours before applying. It shall be carefully applied and thoroughly brushed out; no succeeding coat is to be applied until the previous coat has become thoroughly hard and dry.
INLET PIPE.

Standard cast iron flanged pipe is recommended for the inlet pipe; however, standard hub and spigot flanged or screwed wrought pipe may be used. At proper intervals it is to be stayed to the main post and held securely in position. It is connected to the tank with an expansion joint to allow for changes in the height of the tower.

FROST CASING.

Where the climate is such as to demand it, a frost proof casing generally circular in form shall be built of one or more thicknesses of lumber, with dead air spaces of about two inches between each thickness. The outside course shall be built of \( \frac{3}{4} \) in. matched and dressed flooring. One inch strips covered with tarred or building paper shall comprise each inside course. The frost casing shall receive two coats of good paint after erection.

ERECTION.

The builder of the water tower shall erect the work, shall put in place the inlet pipe, build the frost casing, if one is to be built, and shall complete the work in all particulars unless otherwise specified.

He shall assume the responsibility of building the structure to the satisfaction of the engineer and shall at all times have a competent man in charge of the work. Rubbish and other unsightly material caused by his operations must be removed or disposed of on completion of the work.

FOUNDATIONS.

The foundations shall be built according to plans furnished by the manufacturer of the tank. They shall consist of concrete piers with the necessary anchor rods, and shall be capped with a reinforced concrete cap. They shall be of sufficient weight to be capable of resisting the uplifting due to the wind forces with a factor of safety of at least one and a half. Excavation shall be carried well below the frost line to a firm footing, deeper than shown on plans if so ordered by the engineer, but at an increased cost to be agreed upon. Wooden forms to bring the foundations to the shape indicated on the drawings shall be built for receiving the concrete which shall be mixed as follows. For the lower part of the foundations the concrete shall be made of one part Portland cement, three parts sand and five parts broken stone or gravel. The mixing of the concrete is to be done by hand on platforms. The cement and sand shall first be thoroughly mixed by turning over and over until it is of uniform color. After this is done, the stone or gravel, thoroughly moistened shall be added. The whole shall then be thoroughly mixed by turning over with shovels; sufficient water being added to make the whole mass a tenacious and quaking mixture. The concrete so mixed shall be immediately
deposited in the foundations, in layers not exceeding six inches thick, each layer to be thoroughly and compactly tamped until the whole mass is perfectly solid and free mortar appears on the surface. No concrete shall be put into the foundations which from any cause has been allowed to set or partially set. Concrete for the capstones shall be proportioned as follows:— one part Portland cement, two parts sand and three parts broken stone or gravel. The anchor bolts shall be firmly held in place so they cannot be moved while depositing concrete.

The cement shall be a true Portland cement of the best quality, dry and free from lumps and all foreign material. Its weight per cubic foot shall not be less than 100 pounds. After an exposure of one day in the air and six days under water, it shall develop a tensile strength of not less than 400 pounds per square inch.

The sand shall be coarse, sharp, clean and free from clay or loam.

The broken stone or gravel shall not be larger than 2 inches in any direction and shall be entirely free from dirt or other foreign substance.

In foundations for towers with inclined legs, care shall be taken that the piers are so constructed that the resultant of the vertical and horizontal forces passes through the center of gravity of the pier.

The top of the foundation will generally be at least twelve inches above the ground level, unless otherwise specified.
A. Four-post Hemispherical Bottom Tank (Posts Battered)

Capacity 100,000 gallons. Height 111 feet to balcony.

1. Stresses.

(a) Center of Gravity on Tank.

To determine the center of gravity on tank, take moments about the balcony line.

Roof: 9.95x13.25 = 131.8x29.63 = 3910
Shell: 26.31x22.0 = 580.0x13.16 = 7620
Bottle: \(0.7854 \times 22^2 = 190.0 \times 42 \times 11 = 880\)

Total area = 901.8 Moment = 10650#/ft

Distance of center of gravity above balcony line = \(\frac{10650}{901.8}\) = 11.85 feet.

(b) Wind Loads.

Total wind pressure on tank at 0.30#/ per square foot over 60% diametrical plane = 901.8 x 30 x 0.60

= 16200#/ft

Assume wind load on tower at 200 pounds per vertical foot.

Wind load at balcony line = 200 x 18

= 3600#

Wind load at first strut line

= 200 x 36.5

= 7500#

Wind load at second strut line

= 200 x 37.5

= 7500#

(c) Moments.

Determine moment of the wind forces at the balcony line, first strut, second strut and base of tower respectively.

Moment at the balcony line \(M_o = 16200 \times 11.85\)

= 192,000 foot pounds
Moment at 1st panel \( M_1 = 16200 \times 47.85 = 775,000 \)
\[
\begin{align*}
3600 \times 36.0 &= 130,000 \\
905,000 \text{ foot pounds}
\end{align*}
\]

Moment at 2nd panel \( M_2 = 16200 \times 84.85 = 1,370,000 \)
\[
\begin{align*}
3600 \times 73.0 &= 263,000 \\
7300 \times 37.0 &= 270,000 \\
1,905,000 \text{ foot pounds}
\end{align*}
\]

Moment base of tower \( M_3 = 16200 \times 122.85 = 1,990,000 \)
\[
\begin{align*}
3600 \times 111.00 &= 400,000 \\
7300 \times 75.00 &= 550,000 \\
7500 \times 38.00 &= 285,000 \\
3,225,000 \text{ foot pounds}
\end{align*}
\]

(d) Resisting Moment.

\[
V = \frac{M}{2R \sec \phi}
\]

That is, the compression in a battered post due to wind forces is equal to the total moment of the wind divided by twice the radius of the post centers multiplied by the secant of the angle between the post and a vertical.

(e) Posts.

Stress at point \( M_0 \) = \( \frac{192000}{2 \times 11} \times 1.0065 \)
\[
= 8740\#
\]

Stress at point \( M_1 \) = \( \frac{905,000}{2 \times 15.12} \times 1.0065 \)
\[
= 29900\#
\]

Stress at point \( M_2 \) = \( \frac{1,903,000}{2 \times 19.38} \times 1.0065 \)
\[
= 49,500\#
\]

Stress at point \( M_3 \) = \( \frac{3,225,000}{2 \times 23.78} \times 1.0065 \)
\[
= 68,240\#
\]

(f) Rods.

For battered posts, the rod stress is equal to the vertical shear in the panel considered multiplied by the secant of the angle the rod makes with the vertical.

\[
R = 0.707 (P_A - P_A') \sec \alpha
\]
Top Rods.

\[ R = 0.707 \left( 29900 - 8740 \right) \frac{40.6}{36} \]
\[ = 16850\# \]

Intermediate Rods.

\[ R = 0.707 \left( 49500 - 29900 \right) \frac{44.4}{37} \]
\[ = 16650\# \]

Bottom Rods.

\[ R = 0.707 \left( 68240 - 49500 \right) \frac{48.8}{38} \]
\[ = 17000\# \]

(g) Strut.

The strut stress in a tower with battered posts is equal to the directly applied wind load plus the horizontal component of the rod stress in the panel above, minus the horizontal component of the wind stress in the post multiplied by the cosine of the angle between the horizontal diagonal of the tower and the adjoining strut. 

\[ S = W + R \cos \theta - 0.707 \left( P_A - P_A' \right) \sin \phi \cos \omega \]

Top Struts.

Wind load = 7300 \div 2 = 3650\#

Hor. Component Rod Stress = 16850 \times \frac{18.52}{40.6} = 7700\#

Post (49500 - 29900) 0.707 \times 0.114 \times 0.707 = 1120\#

11350\#

10230\#

Bottom Struts.

Wind load = 7500 \div 2 = 3750\#

Hor. Component Rod Stress = 16650 \times \frac{24.44}{44.4} = 9180\#

12930\#

Post (68240 - 49500) 0.707 \times 0.114 \times 0.707 = 1070\#

11860\#

(h) Anchor Bolts.

Horizontal thrust on anchor bolts is equal to the total dead and wind load in the post, multiplied by the sine of the angle the post makes with the vertical.
Dead Load Water = 208300#
Dead Load Metal = 24000#
Wind Load = 67800#
Total = 300100#

Total Load Post = 300100 x 1.0065 = 302040#

Hor. Shear on Anchor Bolt = 302040 x 0.114
= 34430#

2. Sections.

(a) Tank.

<table>
<thead>
<tr>
<th>Number</th>
<th>Section Req'd.</th>
<th>Thickness Plate.</th>
<th>Section Used.</th>
<th>Size</th>
<th>Row</th>
<th>Joint Efficiency.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0335</td>
<td>1/4</td>
<td>0.098</td>
<td>1/2</td>
<td>1</td>
<td>Lap 0.393</td>
</tr>
<tr>
<td>2</td>
<td>0.0670</td>
<td>1/4</td>
<td>0.098</td>
<td>1/2</td>
<td>1</td>
<td>Lap 0.393</td>
</tr>
<tr>
<td>3</td>
<td>0.1005</td>
<td>1/4</td>
<td>0.122</td>
<td>5/8</td>
<td>1</td>
<td>Lap 0.490</td>
</tr>
<tr>
<td>4</td>
<td>0.1540</td>
<td>5/16</td>
<td>0.207</td>
<td>5/8</td>
<td>2</td>
<td>Lap 0.663</td>
</tr>
<tr>
<td>5</td>
<td>0.0930</td>
<td>1/4</td>
<td>0.175</td>
<td>5/8</td>
<td>2</td>
<td>Lap 0.700</td>
</tr>
</tbody>
</table>

See Fig.

\[ T = \frac{2.604 \times H \times D}{S} \]
will give net section of plate in cylinder.

\[ T = \frac{1.302 \times H \times D}{S} \]
will give net section of plate in bottom.

(b) Posts.

The post must be designed to carry the total dead load of the water and structure itself, plus the compression in the leeward post due to the wind forces.

Weight of 100,000 gallons of water = 833300#

Weight of water per post = 833300 / 4 = 208300#

Total weight of metal = 66000#

Weight of metal per top post = 66000 / 4 = 16500#

The unit stresses for wind loads are increased twenty-five per cent over those allowed for dead loads.
Top Post.
Dead Load Water = 208300 x 1.0065 = 209650#
Dead Load Metal = 16500 x 1.0065 = 16560#
Wind Load on Post = 29700 x 1.0065 = 29900#
Section Required D.L. = $\frac{226210}{15800} = 16.40$ square inches.
Section Required W.L. = $\frac{29900}{17250} = 1.74$ square inches.

Use 2 - Channels 12" x 25# = 14.70 square inches.

1 - Cover Plate 14" x 5/16" = 4.38 square inches.

\[ P = 18500 - 50 \frac{432.0}{4.66} \]
\[ = 13850\# \text{ unit stress.} \]

Intermediate Post.
Dead Load Water = 208300 x 1.0065 = 209650#
Dead Load Metal = 20000 x 1.0065 = 20300#
Wind Load on Post = 49200 x 1.0065 = 49500#
Section Required D.L. = $\frac{229950}{13700} = 16.75$ square inches.
Section Required W.L. = $\frac{49500}{17125} = 2.89$ square inches.

Use 2 - Channels 12" x 25# = 14.70 square inches.

1 - Cover Plate 14" x 3/8" = 5.25 square inches.

\[ P = 18500 - 50 \frac{444.0}{4.66} \]
\[ = 13730\# \text{ unit stress.} \]

Bottom Post.
Dead Load Water = 208300 x 1.0065 = 209650#
Dead Load Metal = 24000 x 1.0065 = 24150#
Wind Load on Post = 67800 x 1.0065 = 68240#
Section Required D.L. = \( \frac{233800}{13600} \) = 17.15 square inches.

Section Required W.L. = \( \frac{66240}{17000} \) = 4.00 square inches.

\[
\begin{align*}
\text{Use} & \quad 2 - \text{Channels} \ 12" \times 25\# = 14.70 \text{ square inches.} \\
1 - \text{Cover Plate} \ 14" \times 1/2" & = 7.00 \text{ square inches.} \\
\text{P} & = 18500 - 50 \ 456.0 \ 4.66 \\
& = 13600\# \text{ unit stress.}
\end{align*}
\]

(c) Rods.

The rods are designed to carry the tension stresses due to wind forces, and where this stress is very small an initial stress of 3000\# should be assumed in order to limit the minimum section of rod used.

Top Rods.

Stress = 17200\#

Section Required = \( \frac{17200}{15000} \) = 1.145 square inches.

Use 1 - 1-1/4" = 1.23 square inches.

Intermediate Rods.

Stress = 16650#

Section Required = \( \frac{16650}{15000} \) = 1.11 square inches.

Use 1 - 1-1/4" = 1.23 square inches.

Bottom Rods.

Stress = 17000#

Section Required = \( \frac{17000}{15000} \) = 1.13 square inches.

Use 1 - 1-1/4" = 1.23 square inches.

(d) Struts.

The struts are designed for compression but the maximum length necessary is usually the governing element in determining the section to be used. No strut should exceed a length of
150 times it least radius of gyration, and in keeping within this limit there will ordinarily be ample section to take care of the maximum compression.

**Top Struts.**

Stress = 10230#

Section Required = \( \frac{10230}{14000} \)

= 0.73 square inches.

**Use**

4 - Angles 3-1/2" x 2-1/2" x 5/16" Lacing Bars 1-1/2" x 1/4"

Section = 7.12 square inches.

\[
\frac{L}{R} = \frac{258.0}{1.76} = 146.0
\]

**Bottom Struts.**

Stress = 11860#

Section Required = \( \frac{11860}{14000} \)

= 0.85 square inches.

**Use**

4 - Angles 5" x 3" x 5/16" Lacing Bars 2" x 5/16"

Section = 9.60 square inches.

\[
\frac{L}{R} = \frac{306.0}{2.51} = 122.0
\]

**Anchor Bolts.**

\[
\text{Uplift} = \frac{H}{2R} = \frac{3215000}{47.45} = 67800#
\]

Net uplift on anchor bolts equals uplift from the wind forces minus the dead load weight of the metal.

\[
\text{Net Uplift} = 67800 - \frac{96000}{4}
\]

= 43800#

Anchor Bolt Required = \( \frac{43800}{15000} \)

= 2.92 square inches.
Use

1 - 2\(\frac{1}{4}\)"\(\phi\) bolt = 3.023 square inches.

The maximum horizontal shear on the anchor bolt equals the horizontal component of both the dead and wind load stresses in the post.

\[ H = P\sin \phi \]

Max. Dead Load Water = 209650#
Max. Dead Load Metal = 24150#
Max. Wind Load Post = 68240# 302040#

\[ H = 302040\# \times 0.114 \]
\[ = 34500\# \]

(f) Foundations.

The foundations must be designed to carry the dead and wind loads in the post, plus the additional weight of the pier itself.

Total weight of water on one pier = 208300#
Total weight of metal on one pier = 24000#
Total wind load on one pier = 67800# 300100#

Approximate Base Required = \(\frac{300100}{4000}\) = 75.0 square feet.

Say

Base = 9.0\(\text{feet}\) square
Cap = 5.0\(\text{feet}\) square
Total Depth = 7.5\(\text{feet}\)

Approximate Quantity of Concrete per Pier = 14 cubic yards.
Approximate Weight of Concrete per Pier = 56000#
Total Load sustained by Earth = 300100# 56000#
Total = 356100#
47.

Actual Area of Base Required = \frac{356100}{4000} = 89.0 \text{ square feet.}

Use
- Base = 9.5 \text{ feet square}
- Cap = 5.0 \text{ square}
- Total Depth = 7.5 \text{ feet}

3. Graphic Solution of Wind Stresses.

Capacity 100,000 gallons.
Height 111 feet to balcony.

Stresses.

Posts.
- Top \( 42500 \times 0.707 = 30000 \text{#} \)
- Int. \( 70000 \times 0.707 = 49500 \text{#} \)
- Bot. \( 97500 \times 0.707 = 68800 \text{#} \)

Struts.
- Top \( 21000 \div 2.0 = 10500 \text{#} \)
- Bot. \( 24000 \div 2.0 = 12000 \text{#} \)

Rods.
- Top \( 34000 \div 2.0 = 17000 \text{#} \)
- Int. \( 33500 \div 2.0 = 16750 \text{#} \)
- Bot. \( 34500 \div 2.0 = 17250 \text{#} \)

Uplift = \( 97000 \times 0.707 = 68600 \text{#} \)

Scale.
- \( 1'' = 30' 0'' \)
- \( 1'' = 30000\)
(4) Horizontal Girder.

Total weight 100000 gallons water = 333300#
Total weight of tank and roof = 46700#

Vertical Loads at Post Connection to Tank.

- Dead Load Water = 208325#
- Dead Load Metal = 11675#
- Wind Load = 3700#
- Total = 228700#

Horizontal Thrust \( Q = 228700 \times 0.114 \)

\[ Q = 26100\# \]

Maximum Bending Moment = 0.68 \( Qd \)

\[ 0.68 \times 26100 \times 22 \times 12 = 472000\# \]

Assume Balcony 27" wide

Flange Stress in Girder = \( \frac{\text{Moment}}{\text{Effective Depth}} \)

\[ = \frac{472000}{25.73} \]

\[ = 18300\# \]

Flange Section Required = 18300 \div 10000

= 1.83 square inches

Use 1 - Channel 6" x 8\( \frac{1}{2} \)" = 2.33 square inches

Maximum Shear = \( \frac{Q}{2} \)

\[ = \frac{26100}{2} \]

\[ = 13050\# \]

Web Section Required = 13050 \div 7500

= 1.73 square inches

Thickness Plate Required = 1.73 \div 27.0

= 0.0642"

Use 27" x \( \frac{1}{2} \)" Plate. Thickness = 0.25"
B. - Ten-post Elliptical Bottom Tank (Posts Vertical)

Capacity 1,000,000 gallons. Height 68 feet to balcony

1. Stresses.

(a) Center of Gravity on Tank.

To determine the center of gravity of the diametrical plane of the tank, take moments about balcony line.

Roof 13.3x34.5 = 458x35.46 = 16300
Shell 31.03x64.0 = 1990x15.02 = 29800
\[
\text{Bott. } \frac{32x64x0.7854}{2} \times 4.42x16 = 5400
\]
Area =3253'Moment=40700'ft

Distance of center of gravity above balcony line = \( \frac{40700}{3253} \)

= 12.50 feet.

Assume the wind load on tank at 30\# per square foot over 60% of its diametrical plane.

Wind load on tank = 3253 x 30 x 0.60

= 58550\#

(b) Wind Loads.

Assume wind load on tower at 500 pounds per vertical foot.

Wind load on tower at Balcony line = 500 x 17

= 8500\#

Wind load at panel point = 500 x 34

= 17000\#

(c) Moments.

Determine moment of the wind forces at the balcony line, first panel point, and base of tower respectively.

Moment at balcony line \( M_o \) = 58550 x 12.5

= 730000 foot pounds.
Moment at 1st panel point $M_1 = 58550 \times 46.5 = 2725000$

$$8500 \times 34.0 = 288000$$

$$= 3013000 \text{ ft. pounds.}$$

Moment at base of tower $M_2 = 58550 \times 80.5 = 4700000$

$$8500 \times 68.0 = 580000$$

$$17000 \times 34.0 = 580000$$

$$= 5860000 \text{ ft. pounds.}$$

(a) Resisting Moment.

$$\begin{align*}
M &= \text{Total moment of wind forces at point considered.} \\
V &= \text{Vertical compression in leeward post} \\
R &= \text{Radius of section being considered.}
\end{align*}$$

Assume the wind in direction of a horizontal diagonal and the axis of rotation passing through the center of the tower. The vertical forces will be proportional to their distance from the axis of rotation.

Therefore Moment $M = (4 \times 0.309V) \times 0.309R = 0.38VR$

$$= (4 \times 0.809V) \times 0.809R = 2.62VR$$

$$= (2 \times 1.0V) \times R = 2.00VR$$

Total Moment $M = 5VR$

$$V = \frac{M}{5R}$$

Therefore, the compression in a vertical post due to wind forces is equal to the total moment of the wind, divided by five times the radius of the anchor bolt circle.

(e) Posts.

Stress at Point $M_0 = \frac{730000}{5 \times 32} = 4570\#$

Stress at Point $M_1 = \frac{3013000}{5 \times 32} = 18800\#$

Stress at Point $M_2 = \frac{5860000}{5 \times 32} = 36600\#$
(f) Rods.

The rod stress is equal to the vertical shear in the panel considered, multiplied by the secant of the angle the rod makes with a vertical.

Top Rods.

Shear in panel AB = \( \frac{18800 - 4570}{2} = 7115\)#

Rod stress in panel AB = \( 7115 \times \frac{39.4}{34.0} \)

= \( 8250\)#

Rod stress in panel BC = \( [7115 + (14230 \times 0.809)] \frac{39.4}{34.0} \)

= \( 21600\)#

Rod stress in panel CD = \( [(7115 + 11500) + (14230 \times 0.309)] \frac{39.4}{34.0} \)

= \( 26700\)#

Maximum rod stress occurs in panel CD with wind assumed in direction of a horizontal diagonal of the tower or from table preceding the maximum rods stress is:

\[
R = (P_A - P_A') \times 1.618 \times \text{seca}.
\]

= \( (18800 - 4570) \times 1.618 \times \frac{39.4}{34.0} \)

= \( 26700\)##

Bottom Rods.

\[
R = (P_A - P_A') \times 1.618 \times \text{seca}.
\]

= \( (36600 - 18800) \times 1.618 \frac{39.4}{34.0} \)

= \( 33400\)##

(g) Struts.

The maximum strut stress for a vertical post tower equals the horizontal component of the rod stress in the panel above, plus the directly applied wind load at the strut line.

Strut \((A - B)\) = \( 8250 \times \frac{19.75}{39.40} = 4130\)##

\( (17000 + 20) \times 3.25 = 2750\)##

Strut \((B - C)\) = \( 21600 \times \frac{19.75}{39.40} \times 0.588 = 6350\)##

\( 1700 \times 0.951 = 1615\)##

\( 7965 \times 1.70 = 13550\)##
**Strut (C - D)**

- **Hor. Comp. of Rod = 13350#**
- **Panel Load = 1700#**
- **Total = 15050#**
- **Strut Stress = 13550#**
- **Hor. Comp. Lower Rod = 13500#**

Therefore, the maximum strut stress occurs in panel CD when the wind is taken in the direction of the horizontal diagonal of the tower or by preceding table maximum strut stress is:

\[ S = R \cos \theta + W = (26700 \times \frac{19.75}{39.40}) + 1700 = 15050# \]

**2. Sections.**

(a) **Tank.**

- **H** = Head in feet on section considered.
- **D** = Diameter of cylinder in feet.
- **S** = Allowable unit stress.

Net section of plate required = \( \frac{2.6 \times H \times D}{S} \)

From the above formula the following table is derived showing thicknesses of plates and riveting required for a unit stress of 12000#/ per square inch net section for tension in plates.

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<td>15/32</td>
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<tr>
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<td>0.445</td>
<td>17/32</td>
<td>0.445</td>
<td>7/8</td>
<td>4</td>
<td>Butt.</td>
<td>0.840</td>
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</tbody>
</table>

**See Fig.**

**Bottom.**

**Point A.**

- **Stress on Horizontal Joint.**
  \[
  \frac{8330000}{6.28 \times 32 \times 12} = 3450# \text{ Tension.}
  \]

- **Stress on Vertical Joint.**
  \[
  \text{Tension} = \frac{0.434 \times 32.8 \times 32.3 \times 12}{6.28\times 6.33\times 12} = 5520#
  \]
  \[
  \text{Compression} = \frac{8330000 \times (6.33)^2}{6.12} = \frac{18620#}{13100#}
  \]
Net section plate required = $\frac{13100}{12000} = 1.092$ sq. in.

Point B.

Stress on Horizontal Joint

Weight = $3.14 \times 30.5^2 \times 42.25 \times 62.5 = 7730000\#$

Tension = $\frac{7730000}{6.25 \times 30.5 \times 12} \times 35.5 = 3940\#$

Stress on Vertical Joint.

Compression = $\frac{7730000}{6.28 \times 12.16 \times 12} \times \left(\frac{12.16}{10.50}\right)^2 = 11300\#$

Tension = $0.434 \times 36.5 \times 30.56 \times 12 \times \frac{35.5}{30.5} = 6750\#$

Net section plate required = $\frac{4550}{12000} = 0.380$ sq. in.

Point C.

Stress on Horizontal Joint

Weight = $3.14 \times 25.5^2 \times 44.83 \times 62.5 = 5730000\#$

Tension = $\frac{5730000}{6.28 \times 25.5 \times 12} \times \frac{45}{25.5} = 5270\#$

Stress on Vertical Joint.

Compression = $\frac{5730000}{6.28 \times 23.75 \times 12} \times \left(\frac{23.75}{13.42}\right)^2 = 10000\#$

Tension = $0.434 \times 41.66 \times 25.5 \times 12 \times \frac{45}{25.5} = 9750\#$

Net section plate required = $\frac{5270}{12000} = 0.440$ sq. in.

Point D.

Stress on Horizontal Joint.

Weight = $3.14 \times 17.5^2 \times 46.75 \times 62.5 = 2815000\#$

Tension = $\frac{2815000}{6.28 \times 17.5 \times 12} \times 56.25 = 6350\#$
56.

Stress on Vertical Joint.

Compression = \( \frac{2315000}{6.28 \times 46.32} \times (40.82) \times 12.6 \approx 11600\)#

Tension = 0.434 x 45.5 x 56.25 x 12 = 13300#

Net section plate required = \( \frac{6800}{12000} = 0.570 \) sq.in.

Point E:

Stress on Horizontal Joint.

Weight = 3.14 x 6.5^2 x 48 x 62.5
= 398000#

Tension = \( \frac{398000}{6.28 \times 6.5 \times 12} \times 65.5 = 8200\)#

Stress on Vertical Joint.

Compression = \( \frac{398000}{6.28 \times 65.5 \times 12} \times (65.5)^2 = 8200\)#

Tension = 0.434 x 48 x 65.5 x 12 = \( \frac{16400\#}{8200\#} \) tension.

Net section plate required = \( \frac{8200}{12000} = 0.683 \) sq.in.

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<td>2           Lap</td>
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</tbody>
</table>

See Fig.

Compression = 12000\# square inches.
Tension = 12000\# square inches.

Use bottom curve plate 13/16".
Use top curve plate 1-11/32".
Diagram for 1,000,000 Gallon Tank Bottom.
(b) Posts.

The posts must be designed to carry the total dead load of the structure and contents of the tank, plus the compression due to wind load.

Weight of 1000000 gallons of water = 8333300#

Deduct eight foot column of water through center = 150300#

Net weight of water carried by posts = 8183000#

Weight of water per post = \( \frac{8183000}{10} = 818300 \) #

Top Post.

Total weight of metal at first panel point = 557000#

Weight of metal per post = \( \frac{557000}{10} = 55700 \) #

Weight of water = 818300#

Weight of metal = \( \frac{818300}{874000} = \frac{62.30}{63.38} \) square inches.

Wind load = 18800#

Section Required D.L. = \( \frac{874000}{14000} = 62.30 \) square inches.

Section Required W.L. = \( \frac{18800}{17500} = 1.08 \) square inches.

Use

2 - Web Plates 20" x 11/16" = 27.50 square inches.

2 - Flg. Angles 3-1/2" x 3-1/2" x 5/8" = 7.96 square inches.

2 - Flg. Angles 6" x 6" x 5/8" = 14.22 square inches.

1 - Cover Plate 28" x 1/2" = 14.00 square inches.

\( \frac{L}{R} = \frac{348.0}{7.37} = 47.3. \)

The length of column is less than ninety times the least radius of gyration; therefore, the unit stress of 14000#/ per square inch assumed above is correct. The unit stress is increased twenty-five per cent for wind stresses.
Bottom Post.

Weight of Water = 818300#
Weight of Metal = 64000# = 882300#
Wind Load = 36600#

Section Required for Dead Load = \( \frac{882300}{14000} = 63.00 \) square inches.

Section Required for Wind Load = \( \frac{36600}{17500} = 2.09 \) square inches.

Use

2 - Web Plates 20" x 11/16" = 27.50 square inches.
1 - Cover Plate 28" x 1/2" = 14.00 square inches.
2 - Angles 3-1/2" x 3-1/2" x 9/16" = 7.24 square inches.
2 - Angles 6" x 6" x 3/4" = \( \frac{16.88}{65.62} \) square inches.

\[ \frac{L}{R} = \frac{348.0}{7.37} = 47.3 \]

(c) Rods.

The rods are designed to carry the tension stresses due to the wind forces. In some cases these stresses are very small, and in order to limit the minimum section that may be used, most specifications call for an initial stress of some amount.

Top Rods.

Rod Stress in Top Story = 26,700#
Net Section of Rod Required = \( \frac{26,700}{15,000} = 1.78 \) square inches.

Use

2 - 1-1/16" ø Rods.

Net Section = 1.78 square inches.
60.

Bottom Rods.

Rod Stress in Bottom Story = 33400#

Net Section of Rod Required = \[ \frac{33400}{15000} \] = 2.23 square inches.

Use

2 - 1-3/16" Diam. Rods.

Net Section = 2.22 square inches.

(d) Struts.

The struts of a water tower are designed to carry the compression due to wind forces, but the governing element is the maximum length of the member. No strut should have an unsupported length greater than 150 times its least radius of gyration. In keeping within this limit there will usually be ample section to take care of the maximum compression.

Strut Stress = 15500#

Section Required = \[ \frac{15500}{14000} \] = 1.11 square inches.

Use

4 - Angles 3\(\frac{1}{2}\)" x 2\(\frac{1}{2}\)" x \(\frac{3}{8}\" Lacing Bars 1-1/2" x 1/4".

Section = 5.76 square inches.

\[ \frac{L}{R} = \frac{237.0}{1.76} = 135.0 \]

(e) Anchor Bolts.

The horizontal shear on an anchor bolt for a vertical post = the direct wind load, plus the horizontal component of the rod stress.

\[ H = W + R \cos \theta \]

\[ = 850 + 33400 \times \frac{19.8}{39.4} \]

\[ = 17650\# \]

Uplift on Anchor Bolt = 36600 - \[ \frac{640000}{10} \] = - 27400#

Use

2 - 1\(\frac{1}{2}\)" ø/Anchor Bolts.
(f) Foundations.

The foundations must be designed to carry the total dead and wind load in the post, plus the additional weight of the pier itself.

Total weight of 1000000 gallons of water = 8333300#

Total weight of water carried by posts deducting 8'Diam. = 8183000#

Total weight of water on one pier = 818300#

Total weight of metal on one pier = 64000#

Wind load on one pier = \( \frac{36600\#}{918900\#} \)

Approximate Base Required = \( \frac{918900}{4000} \)

Say

Base = 16.5' square

Cap = 7.0' square

Total Depth = 12.0'

Approximate Quantity of Concrete per Pier = 65 cubic yards.

Approximate Weight of Concrete per Pier = 260000#

Total Load sustained by Earth

Total = \( \frac{1178900}{4000} \)

Actual Area of Base Required = \( \frac{1178900}{4000} \)

Use

Base = 17.25' square

Cap = 7.0' square.

Total Depth = 12.0'

Fig.
Fig. 4