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DESIGN OF PRESTRESSED CONCRETE TANKS

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SYNOPSIS

In the past, the safe design of circular structures has been extremely difficult, mostly because exact knowledge concerning shrinkage and plastic flow of concrete has been lacking. But with the passage of time, more and more knowledge was achieved about those factors and thus it has become possible to go for large size circular structures, with the use of the high strength of cold-drawn steel wire. These developments have made it possible to design tanks and other large circular structures on a rational basis, with the assurance that an adequate prestress will be maintained to eliminate cracking of the concrete. This has further helped us in availing a great amount of saving in the quantity of materials required.

INTRODUCTION

In an effort to find suitable subdivisions in the expanding literature on prestressed concrete a distinction is drawn between post-tensioning and pre-tensioning of the reinforcing, although pre-tensioning is confined to a small category of work where only small prestressing forces are required. Actually, a far more general subdivision exists as between the prestressing of linear and circular structures. The first includes structures such as bridges, building frames and small pre-cast elements; the second includes tanks, pressure pipes and domes.

So much has been published during recent years about the work in linear prestressing that we tend to forget that by far the largest use of prestressed concrete up to this time has been in the construction of circular structures, notably tanks and silos, for which the methods of design and construction have been developed almost exclusively by American engineers. In the early times engineers could not design large size tanks with reinforced concrete. Even later the safe design of circular structures has been difficult, mostly because exact knowledge concerning shrinkage and plastic flow of concrete has been lacking. With time new data for evaluating these phenomena, and methods that make practicable the use of high strength cold-drawn steel wire were available. Such developments have made it possible to design tanks and other large circular structures on a rational basis, with the assurance that an adequate prestress will be maintained to eliminate cracking of concrete and that important reductions will be made in the weight and in the quantities of critical raw materials required for their construction.

In the early development of prestressed concrete designers soon found that the methods of linear prestressing then developed were not competitive in the high priced labor economy of North America, but that it had inescapable structural advantages in the design of tanks and pipes. Therefore, they applied themselves to the task of developing the methods of design and machinery for construction of tanks which have now become standardized and put to use in most countries of the free world. With the increase in demand for large size circular structures, more and more use of prestressed construction for various purposes has become imperative. The author in this report will deal with the design, construction and maintenance of prestressed circular tanks used for storing water.

LITERATURE REVIEW

Probably the first to conceive of prestressed containers to hold liquids was the cooper who first curved the staves of a barrel so as to be able to force metal hoops around them from opposite ends. By so stressing the reinforcing he developed a compressive force between the staves greater than the tensile force developed in filling the barrel and so made a liquid-tight container. In the annals of his military exploits before the birth of Christ, Julius Caesar reports that he found the natives of western Europe making barrels in this fashion using hoops of copper and bronze.[1*] It may be interesting to note that Caesar made use of these barrels for building pontoon bridges, probably the first use of prestressing principles in bridge construction. It is exactly the same principle we follow today in building large prestressed concrete tanks.

The first engineer to apply the principle of prestressing to concrete tanks was William S. Hewett of Chicago.[2] Having noticed that concrete tanks reinforced with unstressed steel bars frequently developed shrinkage and loading cracks sufficient to cause leakage and severe frost damage in the Northern States, he attempted as early as 1918 to correct this condition by placing around circular

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*Superscripts refer to references listed in Bibliography.

concrete walls steel rods, joined by turnbuckles, of sufficient area that when tensioned to 18,000 psi (the maximum which could be used with intermediate steel bars tightened in this manner) he could create a compressive stress in the concrete greater than the tensile stress in filling the tank. But all the prestress in such bars could be lost in the course of time as a result of shrinkage and flow of concrete. Maunter, of England, employed high-strength steel wires wrapped around precast concrete units, between which were inserted jacks which, when extended, stretched the steel wires.[3] Openings left for the jacks were eventually filled with concrete to maintain the compression in the walls.

In 1930 Mr. J. M. Crom, who was then associated with Hewett, noticed that while these early tanks gave excellent performance so long as they were kept constantly filled with liquid, they often developed cracks and leakage almost as great as in conventional design if left empty for any considerable period of time.[2] This led him to investigate the reason for the loss of original compression in the concrete under these conditions. Shortly before, the same phenomena had been observed in prestressed concrete beams by the noted French engineer, Mr. Eugene Freyssinet.[1] As the structures behaved exactly as predicted when first constructed, both Crom and Freyssinet concluded that the subsequent loss of compression in the concrete could only be accounted for by three factors — (a) shrinkage deformation of concrete, (b) plastic flow of the concrete, (c) creep in the steel.

It was realized that to overcome the loss of compression in the concrete it would be necessary to isolate and measure the magnitude of these factors for various qualities of materials and conditions of loading, and at the same time find some way to use much higher unit stresses in the reinforcing than was possible with rods tightened by turnbuckles. By using a prestress in the reinforcing in excess of 100,000 psi, substantial plastic deformations in the concrete and steel could be absorbed by a relatively small percentage loss in the initial unit tension in the steel and a correspondingly small percent loss in the unit compression in the concrete. High carbon wire of a small diameter was the only form of steel having physical properties which could sustain the high stresses necessary to overcome the relaxation of the compression in the concrete.

In 1935 Preload Company was formed, with Crom as one of the founders. Arrangements were made to undertake a series of tests at M.I.T. to isolate and determine the magnitude of the factors causing a loss of compression in the concrete. At the same time Mr. Crom undertook the task of designing machinery which could install high strength wire under high stress around circular structures and of solving the attendant problems of joining and anchoring the wire at its ends.

The M.I.T. tests were conducted over a period of 900 days starting in the year 1941. The method of testing and partial results at the end of 400 days were published in 1946. Similar tests were carried out in England, France, Switzerland and Belgium.[1]

While the findings on all these tests do not exactly correspond due to differences in loading and methods of testing, the results agree in the following conclusions that are of prime importance in the design of tanks:

(a) The extent of shrinkage and plastic flow in concrete is influenced by many factors and varies over wide limits.

(b) The shrinkage coefficient for high strength concrete varies from 0.0003 to 0.0006 inches per inch, and shrinkage is defined as its contraction due to drying and chemical changes dependent on time and moisture conditions, but not on stresses.

(c) Plastic flow in similar concrete varies almost directly with the sustained stress. For a range of 1,000 to 2,000 psi the coefficient varies from .00025 to .00065 inches per inch.

(d) For an E for steel of 28×10^6 psi the combined shrinkage and plastic flow coefficients are equal to a change in steel stress of 15,400 to 35,000 psi.

(e) Under average construction conditions 25 percent of the losses occur within 20 days, 50 percent within 60 days and 75 percent within 130 days after prestressing. Thereafter the rate of loss approaches a horizontal straight line condition.

While the M.I.T. tests were in progress several experimental machines for stressing wires were tried and abandoned as being too slow and cumbersome. The problem was to devise a machine which could apply wire without friction at a predetermined stress around a circular structure with provision to adjust the spacing of the wire to conform with the decreasing ring tension as the machine ascended the tank wall.[1]

In linear prestressing where the cables run in straight lines or slightly parabolic lines to terminate at the center of gravity of the section at the ends, the problem of friction resistance during stressing of the cables is small and can be overcome by adequate lubrication. However, in stressing wire around a circular wall it is essential to eliminate friction by having the wire under final stress before it comes in contact with the concrete.

A machine, as described later, was finally evolved and used for the first time in 1941 on a two million gallon storage tank for the Bureau of Yards and Docks of the U. S. Navy at Indian Head, Maryland.[1] While it took many times as long as it does today to install the 20,000 pounds of wire required for a tank of this size, this first application demonstrated the possibility of economically building fully prestressed tanks in which the steel would remain permanently in tension and the concrete permanently in compression under maximum assumed operating conditions.

Since that time many hundreds of large tanks have been built using the so-called "Merry-Go-Round" machine for placing the principal prestressing reinforcing. Prestressed tanks have been built having diameters up to 320 feet, heights up to 120 feet, and with capacities of as much as 11 million gallons.[4]

Types of structures utilizing circumferential prestressing are primarily those with thin cylindrical shells, either cast-in-place or shotcreted. Prestressed cylindrical structures recently have been constructed using precast segments. Present construction includes incorporation of a steel diaphragm in shotcrete and precast concrete walls. Prestressed tension rings are used in circular storage structures with vertical spanning members in the form of barrel shells between upper and lower rings.

Circular prestressed structures have been used predominantly for water storage. Other applications include waste water tanks, silos, chemical storage, cryogenic vessels and pressure vessels.[5]

THEORY AND DESIGN CONSIDERATIONS

As the circular concrete tanks are put to many uses they can be classified by the following considerations:

(a) Placement of tank, whether (i) underground, (ii) surface, or (iii) elevated;

(b) Capacity of tank, whether (i) small capacity, or (ii) large capacity;

(c) Whether reinforced concrete or prestressed concrete;

(d) Storage usage whether (i) liquid storage materials such as water, waste water, process liquids, cement slurry, petroleum and other products. (ii) Gas storage such as gaseous by-products of waste treatment processes, cryogenic storage and others. (iii) Dry materials such as grain, cement, sugar, and various other granular products.

This report will be concerned with the design of a prestressed concrete tank on a ground surface for water storage. The following components of the tank require study and consideration for tank design.

FOUNDATIONS

To minimize bending stresses from unequal loading in surface tanks, it is desirable to design for a uniform unit loading over the entire ground area covered by the tank under maximum hydraulic load. In surface tanks covered with a dome roof the unit pressure under the wall is about 2.5 times the hydraulic pressure on the floor. It is therefore necessary to provide a ring foundation under the wall to distribute the load. In many cases the allowable soil loading at the surface is inadequate to carry the desired head of liquid and special foundations are required. Where an adequate stratum exists within 25 feet of the surface the best foundation is a prestressed ring wall extending to the stratum to support the fluidity of the contained soil and transmit the load of the tank to the chosen stratum.[1] This system has been successfully used in many tanks over a period of years. Pile foundations are sometimes used but the floor must be then designed to span between the piles for the full liquid load.

In elevated tanks it is often necessary to use a reinforced concrete mat foundation in which wind loads on the structure must be considered. The load of the tank is usually transferred to the mat by concentric supporting towers spaced at a maximum of 20 feet.

Monolithic base tanks normally employ a thickened and reinforced edge region of the floor to distribute wall loads to the foundation.

FLOORS

The ideal floor for a surface tank is a flexible liquid-tight membrane capable of yielding uniformly to transmit pressure directly to prepared sub-base.[1] The uniformity of the yielding is more important than its extent. Virgin clay and sandy soils, free from boulders, form a good sub-base even though they may settle several inches under load or by loss of original moisture content or by frost upheaval. When the original soil has unequal bearing value, as with boulders occurring near the surface causing "pressure points," it is necessary to provide a compacted sand or gravel "cushion," preferably stabilized with road oil. Where possible, roofs carried on columns passing through the floor should be avoided as they interrupt the ability of the floor to follow the movements of the sub-base.

Those accustomed to designing floors on ground for unequal and concentrated loads frequently make the mistake of using thick slabs heavily reinforced in tanks. Such rigid floors are incapable of following normal movement of the sub-base without cracking. A tank floor must be independently self-supporting under full loading or it must transfer the load to the sub-base with minimum restraint. One need only consider the load imposed in a 100 foot diameter tank filled with 50 feet of water to realize that a flexible floor is the only practical solution for tanks. The best type of water-tight flexible floor developed so far is two inches of gunite reinforced with at least 0.5 percent of steel in each direction.

In some special cases tank floors have been prestressed throughout their area to overcome shrinkage stresses if the tanks were to be left empty and allowed to dry out over long periods. The cost of such prestressing is high, and it has been found to be unnecessary if the floors are kept flooded with water to minimize shrinkage until put into service. Actually, some prestress is imparted to the periphery of the floor when the wall is stressed. This tends to prevent radial edge cracks which are usually the most troublesome in floors on non-prestressed tanks in which some tension is often transferred to the floor in the expansion of the wall under load.

WALLS

Design Method. Wall design is normally based on elastic cylindrical shell analysis for stresses and deformations due to prestressing, fluid load and backfill load. Effects of shrinkage, temperature change, temperature gradient and creep should be taken into account using empirical methods, or a combination of these approaches which have been demonstrated by experience to result in satisfactory structures.

End Restraint. Restraint of displacement of the wall at the base and roof causes significant bending stresses which should be evaluated by rational analysis.

Base Joint Detail. All of the various base details in use involve some restraint to radial horizontal movement under circumferential prestressing and lateral wall loads.

In general, these details are classified as given below.

(a) Monolithic and fully restrained against translation before and after prestressing (see Fig. 1).

(b) Monolithic, hinged with limited restraint against translation during prestressing, and monolithic, fully restrained against translation after prestressing. (see Fig. 2).

(c) Separated, to allow translation and/or rotation (i.e., elastomeric bearing pads)(see Fig. 3).

Monolithic, fully restrained base joints may be fully or partially "fixed" against rotation or they may be "hinged" to permit rotation.[5]

In smaller tanks up to 1 million gallons capacity it is usually cheaper to use a fixed joint with sufficient vertical prestress force to neutralize the higher moment. In larger tanks the vertical moment becomes so great it is necessary to provide for partial sliding at the joint to relieve the restraint. The wall is made with a key in the footing coated with heavy asphalt to reduce friction. For large diameters it is necessary to slope the bearing surface of the key inwards by 15 degrees or more to assist the wall in sliding during prestressing. When the wall has moved, the key on each side is filled with a jointing compound capped with Ironite mortar to prevent outward movement when filled. As slight variations in movement at the base cause wide variations in the vertical moments, extreme care is essential in forming the joint and measuring movements in

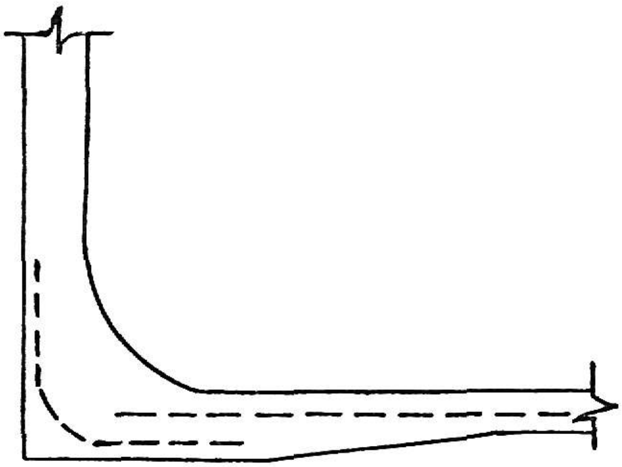


Fig. 1

Monolithic base joint; monolithic and fully restrained against translation before and after wire winding.

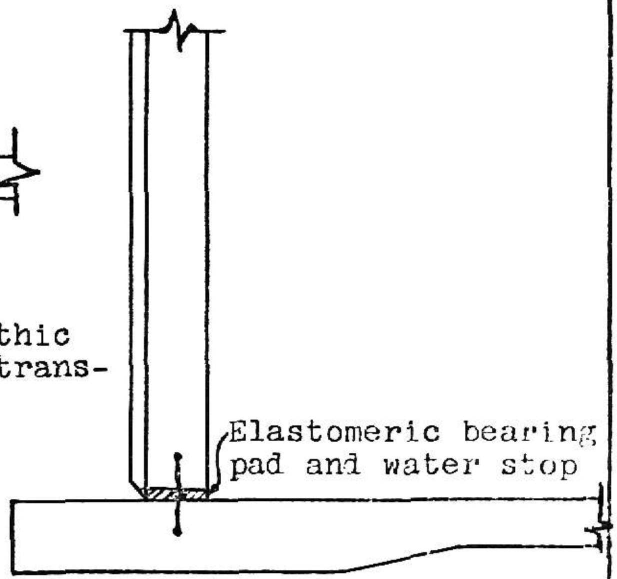


Fig. 3

Separated base joint; separated to allow translation, rotation or both.

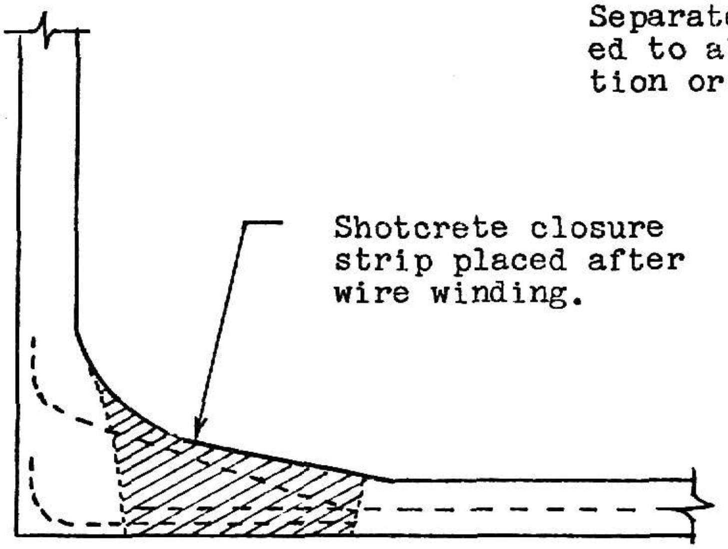


Fig. 2

Monolithic base joint; hinged and limited restraint against translation during wire winding and monolithic, fully restrained against translation after wire winding.

the base. If the movement is less than the calculated, additional prestressed wire must be placed at the base of the tank.

ROOF JOINT DETAIL

The various roof details in use for covered tanks are as follows.

(a) Separated connection (see Fig. 4).

(b) Monolithic connection (see Fig. 5).

THEORY OF CYLINDRICAL SHELLS

In practical applications we frequently encounter problems in which a circular cylindrical shell is submitted to the action of forces distributed symmetrically with respect to the axis of the cylinder. The stress distribution in cylindrical boilers submitted to the action of the steam pressure and the stresses in cylindrical containers having a vertical axis and submitted to internal pressure are examples of such problems.

To establish the equations required for the solution of these problems we consider an elementary segment of a cylinder of size dx dy dz, as shown in Fig. 6. The following relationship can be derived by use of the conditions of equilibrium. The direct stresses in the z-direction are usually small compared with tensile stresses in the wall and are neglected. Force N_β is constant circumferentially due to axial symmetry. Bending moment M_x is negligible when t

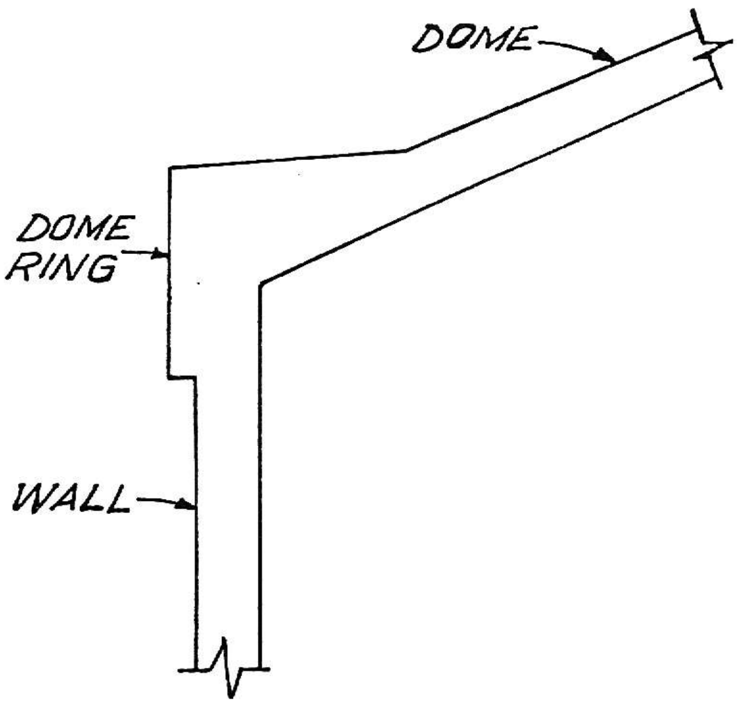


Fig. 5

Monolithic dome-wall connection.

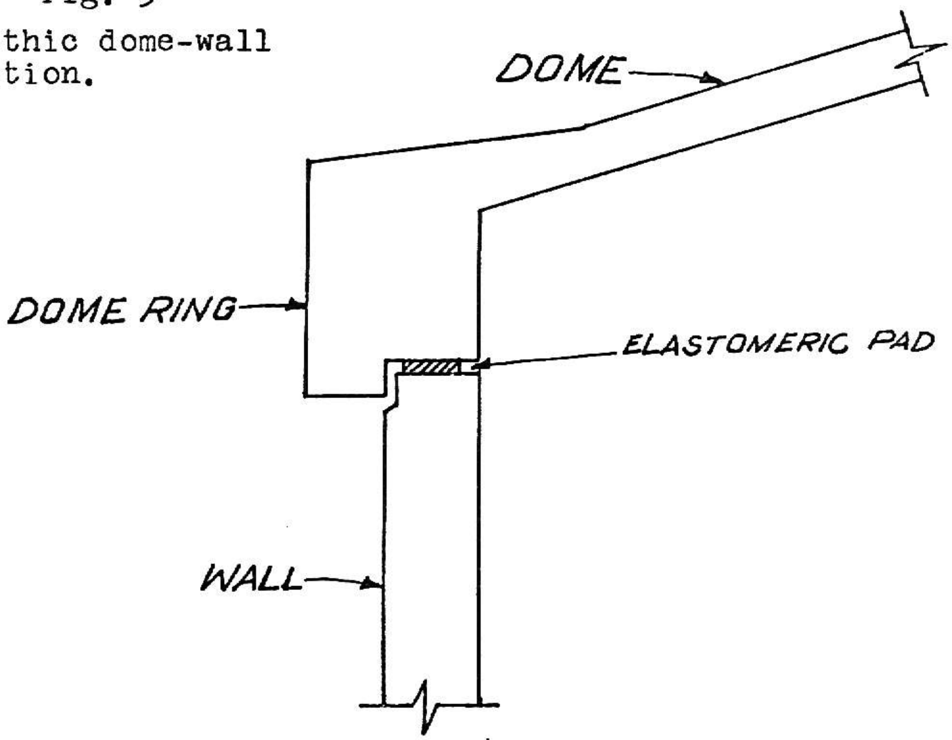


Fig. 4

Separated dome-wall connection.

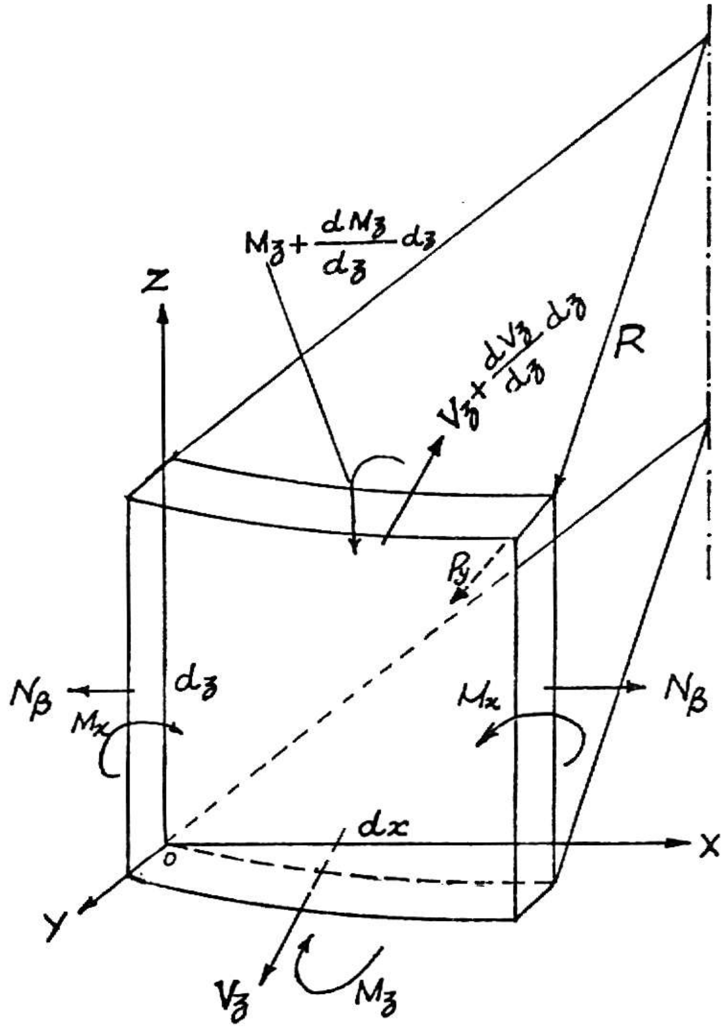


Fig. 6

Element of Cylinder with
Internal Tractions

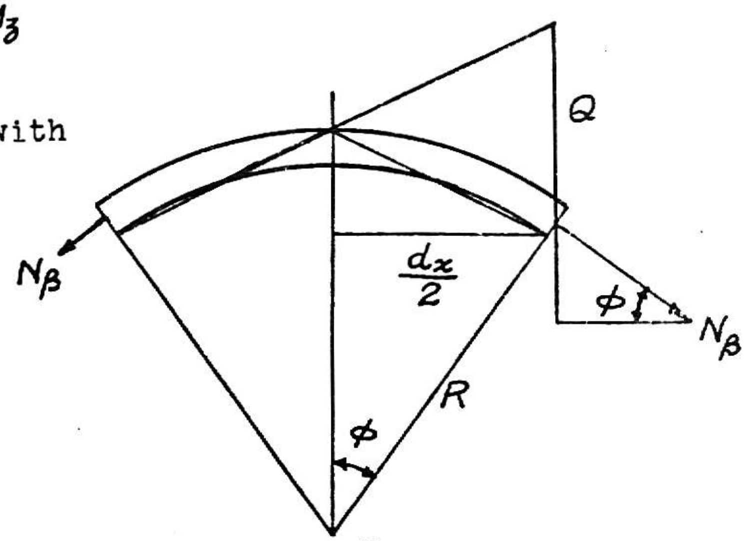


Fig. 7

Cross-section of Elemental Segment

is small compared to R. From the conditions of equilibrium,

$$\sum Fy = 0$$

i.e., $$\frac{dV_z}{dz} dzdx + \frac{N_\beta}{R} dzdx - P_y\, dzdx = 0^*$$

and $$\sum Mo = 0$$ about x-axis,

i.e., $$\frac{dMz}{dz} dzdx - V_z dzdx - \frac{dV_z}{dz} dx \frac{(dz)^2}{2} = 0\ .$$

If the second-order quantity, $(dz)^2$, is neglected in the second equation,

$$V_z = \frac{dMz}{dz}$$

and, from the first equation,

$$\frac{dV_z}{dz} + \frac{N_\beta}{R} - P_y = 0.$$

Combining these two equations yields

$$\frac{d^2Mz}{dz^2} + \frac{N_\beta}{R} = P_y$$

Letting u = deformation of the shell in the radial direction, the following relations hold:

the circumferential strain becomes,

$$\epsilon = \frac{u}{R} = \frac{N_\beta}{tE} \quad \text{or} \quad N_\beta = \frac{Etu}{R}\ .$$

---

*Referring to Fig. 7 N_β is usually in units of force per unit height

$$2\ N_\beta\ dz \sin \phi = 2\ N_\beta\ dz \frac{dx}{2R} = \frac{N_\beta}{R} dzdx.$$ By similar *triangles*, $$\frac{N_\beta}{R} = \frac{Q}{dx} \qquad Q = \frac{N_\beta}{R} dzdx.$$

From the moment-deformation relationship

$$Mz = D \frac{d^2u}{dz^2} .$$

Where D=flexural rigidity $= \dfrac{Et^3}{12(1-\nu^2)}$.

Then, by substitution,

$$\frac{d^2}{dz^2} \left(D \frac{d^2u}{dz^2}\right) + \frac{Et}{R^2} u = P_y .$$

For shells having constant thickness, D=constant and letting

$$\beta^4 = \frac{3(1 - \nu^2)}{(tR)^2}$$

the equation becomes

$$\frac{d^4u}{dz^4} + 4\beta^4 u = \frac{P_y}{D} . \qquad (1)$$

This non-homogeneous linear differential equation can be solved by separating the deformation into two parts, deflection due to internal pressure in a free shell, and deflection due to boundary loads, denoted by up and ui, respectively.

Mathematically, up corresponds to the particular solution of equation (1) and may be found, by standard methods to be

$$up = \frac{P_y R^2}{Et} .$$

Boundary forces consist of shear Vo, or a combination of moment Mo and shear Vo. Then ui satisfies the homogeneous differential equation or

$$\frac{d^4 ui}{dz^4} + 4\beta^4 ui = 0$$

whose solution may be expressed in the following form

$$ui = e^{-\beta z}\ (C_1 \mathrm{Cos}\beta z + C_2\ \mathrm{Sin}\beta z) + e^{\beta z}\ (C_3 \mathrm{Cos}\beta z + C_4 \mathrm{Sin}\beta z).$$

The first half indicates a damped function, i.e., the deflection of the shell is largest at its loaded edge. The second half of the solution is not a damped function, but when we substitute the variable $-z_1$ and the constant $-C_4'$ for z and C_4, respectively,

$$ui = e^{-\beta z_1}\ (C_3 \mathrm{Cos}\beta z_1 + C_4' \mathrm{Sin}\beta z_1)$$

which now is also a damped function. It indicates the deflection due to loads at the other edge or where $z_1 = 0$ (Fig. 8). The distance, L, between the origins is arbitrary, and the constants induced may be absorbed by C_3 and C_4. If L is sufficiently large, these two half solutions do not influence each other and only half of the solution needs to be examined. Using the relationships,

$$\frac{dui}{dz} = \text{slope} = \alpha$$

$$D\ \frac{d^2 ui}{dz^2} = \text{moment} = M$$

$$D\ \frac{d^3 ui}{dz^3} = \text{shear} = V$$

for a cylindrical shell subjected to a given moment Mo and shear Vo at edge x=0, the boundary values are:

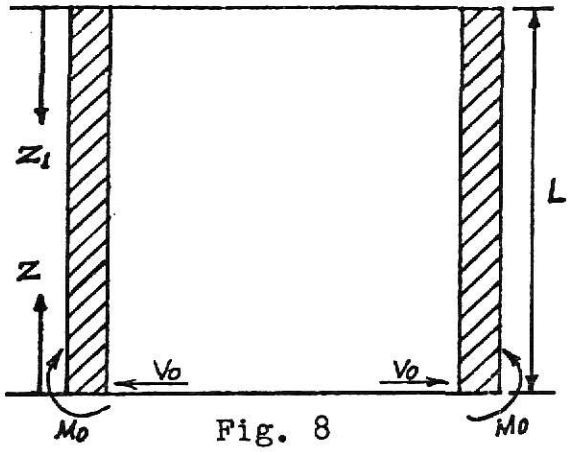


Fig. 8
Vertical Section of the Cylinder

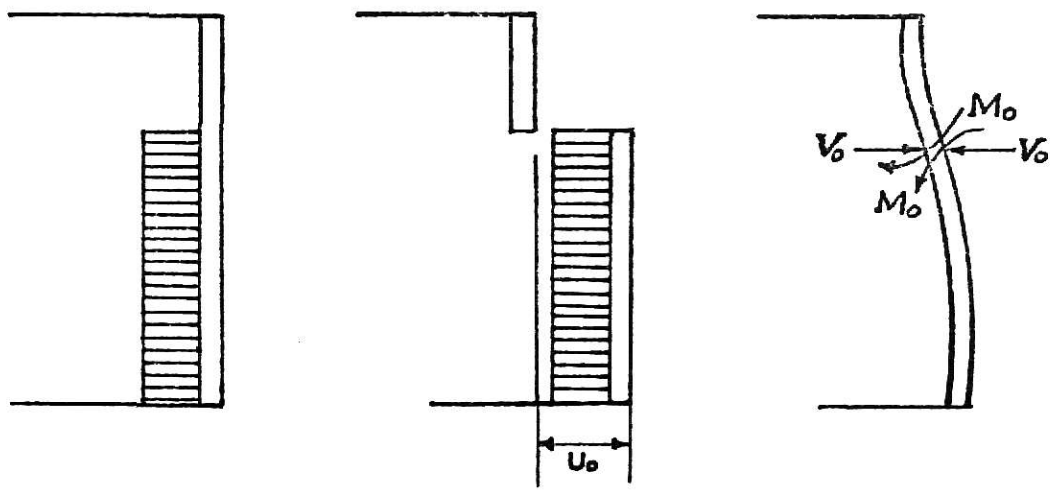


Fig. 9
Behavior of partly loaded tank wall built in at top and free at bottom.

$$D \frac{d^2ui}{dz^2}\bigg|_{z=0} = Mo$$

$$D \frac{d^3ui}{dz^3}\bigg|_{z=0} = Vo \ .$$

If the boundary loads are equated to the respective derivatives of ui, then C_1 and C_2 can be found in terms of known quantities Mo and Vo. The deflection u and slope α of a shell anywhere due to these boundary loads can be easily calculated if Mo and Vo are known. If the boundary loads are unknown, they can be found by the following conditions. For instance,

For hinged base	For fixed base
$Vo \neq 0$	$Vo \neq 0$
$Mo = 0$	$Mo \neq 0$
$\alpha o \neq 0$	$\alpha o = 0$
$uo = 0$	$uo = 0$

Superimposing the displacement due to liquid pressure in a free shell on the deformation due to boundary forces, the resulting deformation must satisfy the above conditions. For a shell subjected to loading on part of its height (Fig. 9), moment and shear may be induced at the point of discontinuity; by use of the foregoing procedure, moment and shear can be evaluated. It is sufficient to illustrate this method considering uniform load only. First divide the shell at the edge of loading; the two parts become free shells. The uniform deflection of the loaded part can be computed. In order to satisfy the continuity condition, a shearing force

Vo and a moment Mo must exist at the dividing edge to bring the edges of the two shells to meet at the same point and have the same slope. Derivation of expressions for u, α, M and V in a long shell due to end loads Vo and Mo:

$$ui = e^{-\beta z}\,(C_1 \mathrm{Cos}\beta z + C_2\ \mathrm{Sin}\beta z)$$

$$\frac{dui}{dz} = e^{-\beta z}\left[(-C_1 \mathrm{Sin}\beta z + C_2 \mathrm{Cos}\beta z) + (-C_1 \mathrm{Cos}\beta z - C_2 \mathrm{Sin}\beta z)\right]\beta$$

$$= \beta e^{-\beta z}\left[(C_1 + C_2)\ \mathrm{Sin}\beta z + (C_1 - C_2)\mathrm{Cos}\beta z\right]$$

$$\frac{d^2ui}{dz^2} = 2\beta^2 e^{-\beta z}\left[C_1 \mathrm{Sin}\beta z - C_2 \mathrm{Cos}\beta z\right]$$

$$\frac{d^3ui}{dz^3} = 2\beta^3 e^{-\beta z}\left[(C_1 - C_2)\mathrm{Sin}\beta z - (C_1 + C_2)\mathrm{Cos}\beta z\right]$$

$$Vo = D\left.\frac{d^3ui}{dz^3}\right|_{z=0} = -2D\beta^3\left[C_1 + C_2\right]$$

$$Mo = D\left.\frac{d^2ui}{dz^2}\right|_{z=0} = -2D\beta^2 C_2$$

$$C_2 = \frac{-Mo}{2D\beta^2}$$

$$C_1 = \frac{1}{2D\beta^3}\,(\beta Mo + Vo)$$

Therefore, the expressions for deflection, slope, moment, and shear are:

$$U = \frac{e^{-\beta z}}{2D\beta^3}\left[\beta Mo(\mathrm{Cos}\beta z - \mathrm{Sin}\beta z) + Vo\mathrm{Cos}\beta z\right]$$

$$\alpha = \frac{e^{-\beta z}}{2D\beta^2}\left[Vo(\mathrm{Sin}\beta z + \mathrm{Cos}\beta z) + 2\beta Mo\mathrm{Cos}\beta z\right]$$

$$M = \frac{e^{-\beta z}}{2\beta}\left[2\beta Mo(\mathrm{Sin}\beta z + \mathrm{Cos}\beta z) + 2\ Vo\mathrm{Sin}\beta z\right]$$

$$V = \frac{e^{-\beta z}}{2}\left[4\beta Mo(\mathrm{Sin}\beta z) + 2Vo(\mathrm{Sin}\beta z - \mathrm{Cos}\beta z)\right]$$

The above equations are useful for analyzing high tanks for uniform thickness of wall and with built-in edges. For hinged bases, only Vo is present, and Mo in the above equation is set equal to zero.[4]

It is observed from the above expressions that u, α, M and V become negligible when z is large. It can easily be verified that their values for $\beta z = 7$ are less than 0.2 percent of their values for $\beta z = 0$; in particular, moment and shear for $\beta z = 3$ are reduced to approximately 5 percent of their original values which, for engineering purposes, may be neglected. For this reason tank walls longer than $3/_\beta$ may be considered as infinitely long without producing appreciable error. Because of its importance, $1/_\beta$, which has a dimension of length, is sometimes called the "characteristic length." "Long shell" length then, should be at least three times the characteristic length.

CIRCUMFERENTIAL PRESTRESSING IN WALLS

Circumferential prestress in tanks is designed to resist hoop tension produced by liquid pressure. Hence, essentially, each horizontal slice of the wall forms a ring subject to uniform internal pressure. In several senses, such a ring can be regarded as a prestressed-concrete member under tension.

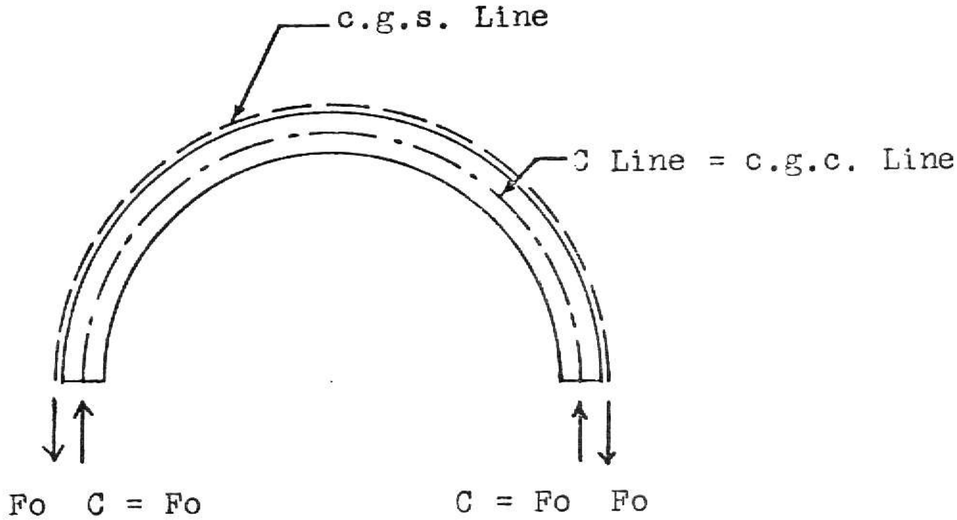
Consider one half of the thin horizontal slice of a a tank as a free body, (Fig. 10(a)). Under the action of the prestress, Fo in the steel, the total compression C in the concrete is equal to Fo. The location of the line of pressure or the C-line in the concrete does not usually coincide with the c.g.s. line. In a circular ring under circular prestress, the C-line always coincides with the c.g.c. line. This is because a closed ring is a statically indeterminate structure, and the theory of linear transformation for continuous beams is applicable to such a ring. A cable through the c.g.c. is a concordant cable; any other cable parallel to it is simply that line, linearly transformed, whose line of pressure will still remain through the c.g.c. This phenomenon can also be explained by the simple fact that the effect of circular prestress is to produce an initial hoop compression on the concrete, which is always axial irrespective of the point of application of the prestress. Hence, owing to circular prestress, the stress in the concrete is always axial and is given by the formula

$$fc = -\ {}^{Fo}/_{Ac}$$

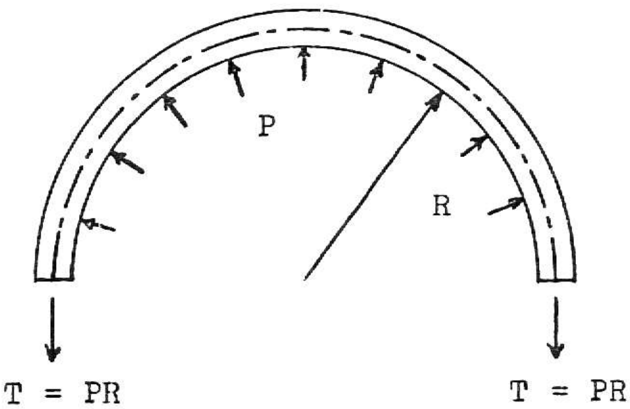
which reduces to

$$fc = -\ {}^{F}/_{Ac}$$

after the losses in prestress have taken place.



(a) Due to Prestress Fo



(b) Due to Internal Pressure P

Fig. 10

Forces in a horizontal slice of tank
(Half slice as free body)

With the application of internal liquid pressure, (Fig. 10(b)), the steel and concrete act together, and the stresses can be obtained by the usual elastic theory. Using the method of transformed section, we have

$$fc = PR/_{At}$$

where P=internal pressure intensity, R=internal radius of the tank, At=transformed area = Ac + (n - 1)As. The resultant stress in the concrete under the effective prestress, F, and the internal pressure, P, is

$$fc = - F/_{Ac} + \frac{PR}{At} \; . \qquad (2)$$

In order to be exact, the value of n has to be chosen correctly, considering the level of stress and the effect of creep. In practice, slight variation in the value of n may not affect the stresses very much, and an approximate value will usually suffice. If a coating of concrete or mortar is added after the application of prestress, then the area Ac under prestress may be the core area while the Ac sustaining the liquid pressure may include the additional coating. Such refinements in calculation may or may not be necessary, depending on the circumstances.

The criteria for designing prestressed tanks vary. The practice in this country has been to provide a slight residual compression in the concrete under the working pressure. This is accomplished by the following procedure of design.

Assume that the hoop tension produced by internal pressure is entirely carried by the effective prestress in the steel; we have

$$F = Asfs = PR \qquad (3)$$

thus the total steel area per foot required is

$$As = \frac{PR}{fs} . \qquad (4)$$

The total initial prestress is then

$$Fo = Asfo. \qquad (5)$$

For an allowable compressive stress, fc, in the concrete, the concrete area required to resist the initial prestress, Fo, is

$$Ac = - \frac{Fo}{fc} . \qquad (6)$$

From this value of required Ac, the thickness for the tank can be determined.

Corresponding to the adopted value of Ac, the stresses in the concrete and steel under the internal pressure, P, can be obtained by

$$\text{Stress in concrete} = F/_{Ac} + \frac{PR}{At} \qquad (7)$$

$$\text{Stress in steel} = fs + nfc \qquad (8)$$

Since F is equal and opposite to PR, and A_t is always greater than A_c, it can be seen from equation (7) that there will be some residual compression in the concrete under the working pressure. This residual compression serves as a margin of safety in addition to whatever tension may be taken by the concrete.

Since the serviceability of a tank is impaired as soon as the concrete begins to crack, it is of utmost importance that an adequate margin of safety be provided against cracking. Where overflow pipes are installed for tanks so that there can not exist any excessive pressure, a smaller margin of safety is required. Thus the English First Report on Prestressed Concrete recommends a factor of safety of 1.25 against cracking.[3]

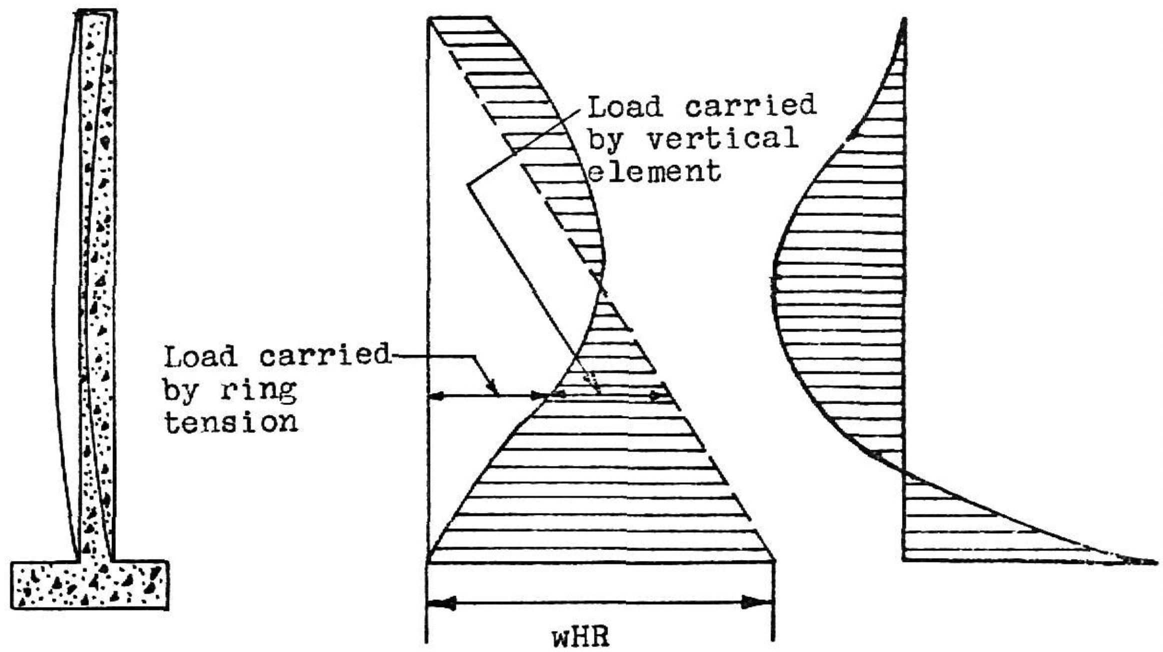
VERTICAL PRESTRESSING IN TANKS

The design of prestressed, concrete structures is based on a knowledge of the behavior of nonprestressed structures plus an understanding of the effect of prestressing. This is as true for the design of tanks as for beams and slabs. Before analyzing the stresses in a prestressed tank, let us consider an ordinary reinforced, concrete tank under the action of internal liquid pressure. It is well known that while the horizontal elements of the tank are subject to hoop tension, the vertical elements are under bending (Fig 11). The amount and variation of bending in the vertical elements will depend upon several factors.

1. The condition of support at the bottom of the wall, whether fixed, hinged, free to slide, or restrained by friction.

2. The condition of support at the top of the wall, whether fully or partially restrained or free to move.

3. The variation of concrete thickness along the height of the wall.



Vertical section of tank wall and deflection under load.

Load distribution between horizontal and vertical elements.

Moment in vertical element.

Fig. 11

Moment and deflection in vertical element of tank wall.

4. The variation of pressure along the depth, whether triangular or trapezoidal.

5. The ratio of the height of the tank to its diameter.

Theoretical solutions for several of these combinations are given by Timoshenko,[6] and numerical values, convenient for application, are tabulated in some pamphlets.[7] European books give solutions for additional cases, such as walls of varying thickness, and the results are plotted in some publications.[8]

For prestressed concrete tanks, an additional problem is introduced: the effect of prestressing, both circumferential and vertical. Since horizontal pressure will produce vertical moments M_z in the wall, it is evident that circumferential prestressing will also induce such moments. These vertical moments caused by circumferential prestressing will exist by themselves when the tank is empty and will act jointly with the moments produced by liquid pressure when the tank is filled. To reinforce the wall against these moments, vertical prestressing may be applied. If vertical prestressing is concentrically applied to the concrete, only discrete compressive stress is produced and the solution is simple. If the vertical tendons are bent or curved, the vertical prestress produces radial components which, in turn, influence the circumferential prestress. Hence the analysis can become quite complicated.

Let us investigate the effect of circumferential prestressing on the vertical moments. If the circumferential prestress varies triangularly from zero at the top to a maximum at the bottom, its effect is equal but opposite to the application of an equivalent liquid pressure. If the circumferential prestress is constant throughout the entire height of the wall, it is the same as the application of an equivalent gaseous pressure. For both cases, tables are available for the computation of vertical moments.[7] To obtain the optimum results, the circumferential prestress along the depth of the wall should be varied to suit the variation of the active pressure on the horizontal elements. However, the effect of such circumferential prestressing on vertical moments cannot be readily determined.

Vertical prestressing should be designed to withstand the stresses produced by various possible combinations of the following forces.

1. The vertical weight of the roof and walls themselves.

2. The vertical moments produced by internal liquid pressure.

3. The vertical moments produced by the applied circumferential prestress.

In addition to the above, stresses may be produced as a result of differential temperature between the inner and outer faces of the wall, and by shrinkage of the concrete walls unless they are entirely free to slide on the foundation. These forces cannot be easily evaluated and hence are often neglected and provided for indirectly in an overall factor of safety.

It must be noted that the maximum stresses in the concrete usually exist when the tank is empty, because then the circumferential prestress would have its full effect. When the tank is filled, the liquid pressure tends to counter-balance the effect of circumferential prestress and the vertical moments are smaller. Since it is convenient to use the same amount of vertical prestress throughout the entire height of the wall, the amount will be controlled by the point of maximum moment. By properly locating the vertical tendons to resist such moments, a most economical design can be obtained. However, efforts are seldom made to do so, and the amount of prestress as well as the location of the tendons is generally determined by empirical methods rather than by any logical method of design.[3]

Many designers have believed that no useful purpose would be served by prestressing tank walls in a vertical direction. Mr. Crom[9] inspected more than 100 conventionally designed reinforced concrete tanks and silos, and has found horizontal cracking in the walls almost invariably. Such cracks not only disfigure the structure, but also result in leakage which may cause early loss of the tank due to freezing and thawing of water in the fracture planes. His experience with tanks which had been prestressed circumferentially also strongly indicated the need for vertical prestressing. Tanks with vertical prestressing have been most satisfactory and no repairs have been necessary.

ROOFS

A spherical dome shell prestressed at the abutment ring is the most efficient roof for tanks not subject to heavy roof loads. Flat slabs require columns which are costly in deep tanks and which interrupt the floor system. These domes are usually designed for a one-eighth rise which makes an angle of 28 degrees between the tangent of the dome and the horizontal. This is about the steepest angle at which concrete can be placed without double forms and is within the angle at which reversals in stress occur in spherical shells.

The compression stress in such shells is very low and many have been built with only two inch thickness for spans up to 150 feet. In these the ratio of shell thickness to diameter is only one-fortieth of the same ratio for the average eggshell which is often used as an example of an efficient structural shape. However, such thin shells have a tendency to crack under severe changes in temperature or concentrated snow loading, and it is now the usual practice to use a thickness varying from two to six inches for diameters of 50 to 250 feet. Such domes when prestressed at the abutment ring actually lift up from the supporting forms by an amount proportional to the design live load.

Such domes have great strength in supporting uniform external loads such as earth covering for concealment and protection from attack. Mr. Dobell writes in his paper[1] that during the war he watched with alarm as a Caterpillar bulldozer pushed a load of earth over the top of a two-inch prestressed dome covering a 90 foot ammunition magazine but no damage occurred.

Domes of this kind have been used for the floors of elevated tanks up to 70 feet in diameter carrying a water load 50 feet in depth. The design of domes will not be discussed in this report as they constitute an independent topic.

MATERIALS

The materials used for the construction of prestressed tanks are concrete, shotcrete and steel.

CONCRETE

Whether regular concrete or shotcrete is used in constructing a prestressed concrete tank, careful planning must be assured in the construction phases to avoid cold joints. It has been found time and again that where cold joints exist in concrete tanks, they will probably leak.[10] A dense concrete which is homogeneous is more important than high concrete strength. Using a dry mix which would result in honeycombing and possible cold joints is not a good procedure for prestressed concrete tank walls. It is necessary to insure as much workability as possible to avoid cold joints and honeycombing.

STEEL

Wires used as prestressing steel should conform to ASTM A421 or ASTM A227, class 1 or 2. These wires have a minimum yield point of 200,000 psi, so they can be safely stressed to an initial stress of 140,000 psi. There are three factors which cause a loss in prestress. They are (a) elastic shortening of concrete, (b) shrinkage in concrete, (c) loss due to creep. On the average it is observed that these losses range from 20,000 to 40,000 psi. Thus after deducting these losses we get the working stress in the steel.

It is important to note that while the quantity of steel is a function of the working stress the quantity of concrete is a function of the initial stress in the steel. The quantities of both steel and concrete therefore vary directly as the working stress varies to the initial stress. Hence the use of high unit stress in the steel permits a reduction in concrete as well as a direct saving in steel.

The difference between the initial and working stress must be further increased for tanks containing hot or cold liquids, especially when rapidly filled or emptied, to compensate for thermal stresses between the two surfaces of the wall.

CORROSION OF STEEL

The item which seems to cause the greatest amount of concern, and has had the greatest publicity in recent years, is the problem of protecting the prestressing steel from corrosion. There have been various incidents of corrosion problems, both in prestressed concrete tanks and pipes. One tank in New York and one in Redwood City, California collapsed due to the corrosion of the prestressing wire of the tanks. Two tanks in Menlo Park, California were repaired when they appeared to be on the verge of collapsing as a result of extensive corrosion of the wire. Some corrosion of the wire of five tanks in Sacramento has been discovered. In this latter case, the extent of the corrosion was not so serious as to suggest immediate collapse of the tanks, but nevertheless, the installation of additional prestressing steel on the tanks, to insure against future collapse, was required. Of these nine tanks, eight have been used to store sludge and one was used to store cement slurry containing salt water.

For wire wrapped tanks, the method of protecting the wire has generally been the use of shotcrete. It is necessary to completely encase the prestressing wire with a rich dense mortar which is bonded to the previous layers in order to insure corrosion protection. If the wire is not completely protected it is likely that corrosion will occur. Corrosion of properly protected wires will generally not occur, even if a hairline crack exists through the shotcrete, and water from the inside of the tank leaks through. Proper protection of the wire seems to require a minimum dense mortar cover of ½ inch.

On one particular water tank inspected by Mr. Morris

Schupack,[10] it was found that where the shotcrete had spalled off, the wires were corroded on the outside face. By chipping into adjacent shotcrete it was found that whereever the shotcrete was less than ½ inch thick, corrosion existed. Where the shotcrete was more than ½ inch thick, the wires were bright and clean. That particular tank was a slipform tank which had suffered every catastrophe in the construction phases that can happen during a slipforming process. The tank is full of cold joints, and has been leaking for over 11 years. In areas where water seeped through the wall almost constantly, openings were made to expose the wire. It was found that where shotcrete was ½ inch thick or greater, and bonded to the under layers, the wires were bright and clean. This finding was similar to that found on other tanks examined.

Following are some interesting points regarding corrosion of steel reinforcement in concrete structures that have been advanced in several articles on the subject:

1. The presence of three elements simultaneously are necessary to support corrosion of steel: oxygen, moisture and ions. In a great many cases of corrosion, chlorides have provided the ions. The use of calcium chloride in reinforced concrete structures should be avoided.

2. The rate of corrosion of reinforcement decreases with an increase of cement content, decreases with an increase of concrete cover over the steel, increases with an increase of water in the mix, and increases with an increase

of the concentration chlorides present.

3. Some agencies, including East Bay Water, have adopted the practice of specifying a protective coating over the prestressing steel of tanks and of pipe consisting of a flush coat of neat cement slurry followed immediately by a coat of cement and sand mortar.[11] It is expected that inclusion of the slurry in the protective coating produces improved protection for the reinforcement by producing additional hydroxides and by minimizing voids in the coating in the vicinity of the steel.

A great many prestressed concrete tanks are in service in the United States and abroad. Only a few scattered cases of serious corrosion of the prestressing steel have been reported. These reported cases of corrosion have not been substantially correlated with any of the elements of environments to which the steel has been subjected. At this point, total field experience with prestressed concrete tanks indicates that there is some risk of corrosion failure connected with the decision to use these structures for storing liquids.

PROTECTION OF PRESTRESSING STEEL

There is apparently a much greater need for a foolproof protection of the prestressing steel on sewage tanks than on water tanks. It is not felt that galvanized wire under a detrimental environment would necessarily give an appreciably extended life to the prestressing steel.[10]

It is the opinion of the author that prestressing steel for sewage tanks should be placed only under the most careful supervision. The wires should be accurately spaced and the protective covering should be greater than that required for water tanks. It is also necessary, evidently, to insure that the wall design takes into account all the environmental problems which may affect the cracking performance of the wall.

SPECIAL EQUIPMENT FOR CONSTRUCTION

The main piece of special equipment required for prestressed tanks is the "Merry-Go-Round" machine. It comprises two main parts: an overhead carriage which rides the top edge of the wall, from which is suspended the wire winding machine. The latter is a self-powered unit which propels itself around the tank wall by a power driven sprocket in engagement with an endless link belt spring tensioned around the tank. With each revolution of the machine the belt is raised to a new position on the wall equal to the required helical pitch of the prestressed wire. The cables on which the winding machine is suspended are would on drums which are slowly rotated by hydraulic motors to raise the machine at a uniform rate corresponding to the required pitch in the wire. The wire is carried on the machine in coils of about 300 pounds and is stressed by passing through dies which impart the required tension to the wire. Number 8 wire having a diameter of 0.162 inch is the size generally used. Placing wire is usually started from the bottom of the tank where the trailing end of wire is fastened to the base of the wall before setting the machine in motion. In some types of tanks the procedure is reversed to reduce the vertical moments. The machine is stopped at the end of each coil of wire while a splice is made to a new coil with spring-loaded "torpedo" splices which develop the full strength of the wire. Three sizes of the machines are used for various sizes of tanks. The smallest is driven by air-motors at a top running speed of three miles per hour. The intermediate machine, powered by a gas engine, has a top running speed of six miles per hour and the largest machine can place two strands of wire simultaneously at eight miles per hour. Each machine requires an operating crew of six men.

In addition to the wire winding machine special jacking machines are required for vertical stressing. One such machine is designed to stress loops of six strands of number 8 wire placed in slots on the interior face of the wall.

The loops are anchored by a pin at the top. The jack engages the lower loops and when the required elongation is reached at 1.5 inches diameter the pin is forced home by the jack to engage the wire loops to matching anchor loops set in the concrete. This machine is most efficient for high walls as it is capable of stressing and anchoring 100 units per day.

Other simpler jacks are used for stressing from the top of the wall wire strands contained in light metal tubes cast in the concrete with loop anchors at the bottom.

Numerous methods of building prestressed tanks were tried in arriving at the procedure described. Attempts are still made in some European countries to prestress circular tanks by jacking wires between anchor points while in frictional contact with the wall. However, the friction is so great that it is necessary to stress and anchor the individual wires at least eight points around the wall and such points must be staggered around the wall to achieve reasonable uniformity of stress. In a test conducted by Stevens Institute of Technology on a 110 foot diameter tank at the Owls Head Sewage Treatment Plant in New York City it was found that when a number 8 gauge wire was stressed to 140,000 psi against the concrete between anchor points 180^{o} apart, the loss of stress at the mid-point was 37,800 psi or 27 percent.

USES

The methods described have been used in the construction of many hundreds of tanks and silos in North and South America, Europe, Africa and the Near East. These tanks include sizes of 0.05 to 11.0 m.g. capacity.

A pilot tank of 0.2 m.g. capacity was constructed for the storage of liquid oxygen at -380^{o}F. Only by prestressing is it possible to accommodate the tremendous thermal contraction stresses occurring at this low temperature. One of the most effective uses of prestressed concrete tanks is in sludge digestion tanks. In the past years more than a hundred such tanks were built having a large capacity for the cities of New York, Philadelphia, Los Angeles, Oklahoma and smaller municipalities. The use of those tanks has resulted in a saving of millions of dollars to the taxpayers of those communities. The accompanying chart (Table 1) showing the relative quantities of steel and concrete required for prestressed and conventional design for four groups of the tanks is a good illustration of the savings in materials and resulting cost which is possible with fully mechanized methods of prestressed tank construction.

Elevated tanks also can be built in prestressed concrete in direct competition with steel construction, and as with surface tanks will save in maintenance cost as much as the initial cost during an operation period of 30 years.

Recently it has been suggested that the same methods could be used to construct vehicular tunnels in vertical sections of 60 feet or more in height which, after prestressing, would be laid end-to-end and longitudinally prestressed in lengths of 300 feet which could be towed with temporary bulkheads in place to position in the same manner as steel sections. Preliminary designs and cost estimates are most encouraging. The same principles are also being investigated for oil tankers and floating dry-docks.

The prestressed domes used to cover these tanks are also well suited as large economical roof areas for buildings such as hockey arenas, auditoriums and aircraft hangers. Such domes, in diameters up to 300 feet, covering an area of 71,000 square feet, can be built for a cost of about $3.00 per square foot, which is much less than any other form of roof construction for similar spans.

Table 1

QUANTITY COMPARISON - CONVENTIONAL VS. PRELOAD*					
Hunts Point Sewage Treatment Plant, Bronx, New York 8 Tanks					
ITEM	UNIT	CONVENTIONAL DESIGN	PRELOAD DESIGN	SAVING	% SAVING
Concrete	c.y.	17,840	11,060	6,780	38%
Steel	Ton	1,642	474	1,168	72%
North East Treatment Works, Philadelphia, Pennsylvania 8 Tanks					
Concrete	c.y.	18,800	12,080	6,720	36%
Steel	Ton	2,300	910	1,390	60%
Hyperion Sewage Treatment Works, Los Angeles, California 18 Tanks					
Concrete	c.y.	24,550	12,890	11,660	46%
Steel	Ton	2,510	561	1,949	78%
Owls Head Sewage Treatment Plant, Brooklyn, New York 8 Tanks					
Concrete	c.y.	8,824	4,741	4,083	46%
Steel	Ton	904	275	629	70%

*Preload company tanks have been prestressed concrete tanks.

APPENDIX I

DESIGN EXAMPLE

Water Storage Tank

Capacity = 1,000,000 gallons

Type: Surface Tank

Bearing capacity = 1,800/$\square'$

Wall connection: hinged with base slab

$f_s' = 200{,}000$ psi

$f_o = 140{,}000$ psi

$f_s = 105{,}000$ psi

$f_c' = 4{,}000$ psi

$f_c = 1{,}000$ psi

One gallon = 0.1337 cubic feet

Therefore volume of tank = 1,000,000 x 0.1337 = 133,700 cu.ft.

Weight of water = 133,700 x 62.4 = 8,340,000 lbs.

Base area of tank $= \dfrac{8{,}340{,}000}{1{,}800} = 4{,}633$ square feet.

Diameter of tank $= \sqrt{\dfrac{4{,}633 \times 4}{\pi}} = 76.7$ say 77 feet.

Thus height of tank $= \dfrac{133{,}700}{4{,}660} = 28.7$ say 29 feet.

Assuming the yield strength of high-carbon steel wires

$f_s' = 200{,}000$ psi

working stress initial $f_o = 0.7\ f_s' = 0.7 \times 200{,}000 =$ 140,000 psi.

Assuming prestress losses due to elastic shortening, shrinkage and creep to be 35,000 psi

Stress in steel after losses $f_s = 140{,}000 - 35{,}000 =$ 105,000 psi

Modulus of elasticity of concrete $E_c = 33\ w^{3/2}\sqrt{f_c'} = 3.82 \times 10^6$ where w is the unit weight of concrete.

Assuming $ES = 28 \times 10^6$

$$n = \frac{Es}{Ec} = 7.35$$

Hoop tension produced by water pressure inside the tank is taken by effective horizontal prestress in steel; using equation (3) we have

$$F = As\ fs = PR$$

$P = wh$ where h is effective depth of water

$$= 62.4 \times 28.7 = 1,790 \text{ psf}$$

Steel area required using equation (4)

$$As = \frac{PR}{fs} = \frac{1,790 \times 38.5}{105,000} = 0.655 \text{ square inches/foot.}$$

Initial prestressing force Fo using equation (5) is

$$Fo = As\ fo = 0.655 \times 140,000 = 91,700 \text{ lbs./foot.}$$

Area of concrete required using equation (6)

$$Ac = -\frac{Fo}{fc} = -\frac{91,700}{-1,000} = 91.7 \text{ square inches /foot.}$$

Wall thickness of tank $t = \frac{91.7}{12"/ft.} = 7.64"$ say 8".

Adopt wall thickness = 8 inches.

Checking the stress in concrete using equation (7)

$$fc = -\frac{F}{Ac} + \frac{PR}{At}$$

$$= -\frac{0.655 \times 105,000}{96} + \frac{1,790 \times 38.5}{96 + .655 \times 7.35}$$

$$= -716 + 685 = -31 \text{ psi.}$$

Using the tables for ring tension for hinged base and free top circular concrete tanks without prestressing in Portland Cement Association Bulletin[7], we have

$$K = \frac{H^2}{Dt} = \frac{(28.7)^2}{77 \times 8/_{12}} = 16.05 \text{ say } 16.$$

Looking into tables of the above reference we have values of ring tension and moments as shown in Table 2 and Table 3 respectively.

Fig. 12 shows variation of ring tension and moment along height.

$K = 16 \quad wHR = 6{,}900$ lbs./ft.

Ring tension = Coefficient x wHR

Ring Tension:

Table 2

POINT	0.0H	0.1H	0.2H	0.3H	0.4H
Coefficient Table II	0.002	0.100	0.198	0.299	0.403
Ring Tension	13.8	690	1,370	2,062	2,780
(Continued)					
POINT	0.5H	0.6H	0.7H	0.8H	0.9H
Coefficient Table II	0.521	0.650	0.764	0.776	0.536
Ring Tension	3,600	4,480	5,270	5,360	3,700
(Continued)					
POINT	1.0H				
Coefficient Table II					
Ring Tension					

$K = 16 \quad wH^3 = 62.4 \times 28.7^3 = 1,475,000$ ft. lb./ft.
+ ve sign tension outside.

Moment = coefficient x wH^3

Table 3

POINT	0.0H	0.1H	0.2H	0.3H	0.4H
Coefficient Table VII	0.000	0.000	0.000	-.000	-.0001
Moment	0.0	0.0	0.0	0.0	-147.5
(Continued)					
POINT	0.5H	0.6H	0.7H	0.8H	0.9H
Coefficient Table VII	-.0002	-.0004	+.0008	+.0022	+.0029
Moment	- 295	-590.0	+1,180	+3,240	+4,280
(Continued)					
POINT	1.0H				
Coefficient Table VII	0				
Moment	0				

Table 3 shows that maximum moment induced by the band reinforcement is 4,280 ft. lbs.

VERTICAL REINFORCEMENT:

$$\text{Area of steel} = \frac{4,280 \times 12}{105,000 \times 0.9 \times 7}$$

$$= 0.0776 \text{ square inches}$$

Moment caused by vertical reinforcement

$$\text{Moment} = \frac{0.0776 \times 140,000 \times 3}{12} = 2,715 \text{ ft. lbs.}$$

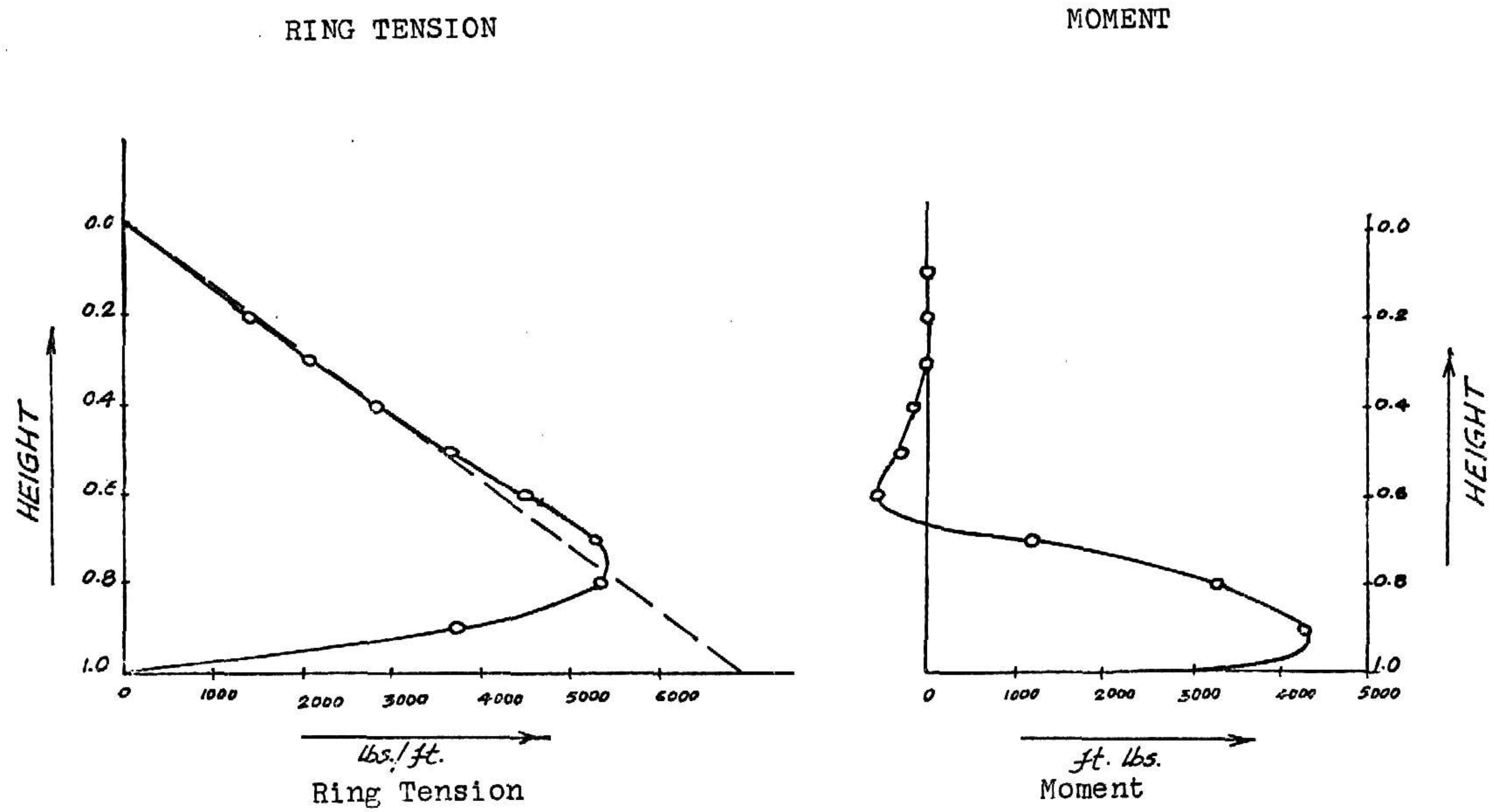


Fig. 12

Graph showing variation of ring tension and moment along height.

Table 4

COMPUTATIONS FOR STRESSES IN CONCRETE

CONDITIONS	Initial Concrete Stresses (psi)			
	Original		Revised	
	Inside	Outside	Inside	Outside
1. Assume due to weight of dome	- 30	- 30	- 30	- 30
2. Weight of Wall $\frac{29 \times 8 \times 1 \times 150}{12 \times 8 \times 12}$	- 31	- 31	- 31	- 31
3. Direct compression due to vertical reinforcement $\frac{.0776 \times 140,000}{12 \times 8}$	-113	-113	-197	-197
4. Eccentricity of vertical prestress $\frac{6M}{bd^2} = \frac{6 \times 2,715 \times 12}{12 \times 8 \times 8}$	+255	-255	+447	-447
5. Vertical moment due to circum. prestress $\frac{6M}{bd^2} = \frac{6 \times 4,280 \times 12}{12 \times 8 \times 8}$	-402	+402	-402	+402
Total for tank empty	-321	- 27	-213	-303
6. Full tank liquid load $\frac{6M}{bd^2} \times \frac{fs}{fo} = \frac{6 \times 4,280 \times 105,000}{12 \times 8 \times 8 \times 140,000}$	-302	+302	-302	+302
Total for tank full	-623	+275	-515	- 1

As we see here, there is tension in the outer fibre of tank wall which we can't afford. So increasing the vertical reinforcement to 1 3/4 times.

3. Revised direct compression vertical wires

$$= \frac{0.0776 \times 1.75 \times 140,000}{12 \times 8} = -977 \text{ psi}$$

4. Eccentricity of vertical prestress

$$= \frac{6 \times 2.715 \times 1.75 \times 12}{12 \times 8 \times 8} = \pm 447 \text{ psi}$$

APPENDIX II

NOTATION

α	Slope angle
β	Reciprocal of characteristic
ν	Poisson's ratio
ϵ	Circumferential strain
Ac	Area of concrete
As	Area of steel
At	Transformed area
b	Length of tank wall
C	Compressive force in concrete
c.g.c.	Center of gravity of concrete
c.g.s.	Center of gravity of steel
d	Tank wall thickness
D	Flexural rigidity
dx	Length of element along x-axis
dy	Thickness of element along y-axis
dz	Height of element along z-axis
E	Modulus of elasticity
Ec	Modulus of elasticity of concrete
Es	Modulus of elasticity of steel
Fy	Force in y-direction
Fo	Initial prestressing force in steel
F	Prestressing force after losses
fc	Compressive stress in concrete
f_c'	Ultimate compressive stress in concrete

fo	Initial prestress in steel
fs	Prestress in steel after losses
f_s'	Yield stress of high carbon steel
H	Height of water in tank
L	Height of tank in Figure 8
M	Bending moment
n	Ratio of modulus of elasticity of steel to concrete
N_β	Constant circumferential force
P	Intensity of water pressure inside tank
P_y	Internal pressure acting along y-axis
Q	Statical moment of area
R	Inside radius of tank
t	Tank wall thickness
T	Tensile force
u	Deformation in wall thickness
ui	Deformation due to boundary loads
up	Deformation due to internal loads
w	Unit weight of water; unit weight of concrete

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DESIGN OF PRESTRESSED CONCRETE TANKS

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AN ABSTRACT OF A MASTER'S REPORT

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ABSTRACT

This report deals with the theory and design of prestressed concrete circular tanks. In recent years the demand for bigger size circular structures has immensely increased, because they can be used for the storage of large quantities of solids, liquids and gaseous materials. Following are some of the major factors which have been responsible in the development of prestressed concrete tanks.

(a) They are competitive in initial cost with tanks in steel or standard reinforced concrete design and offer marked savings in maintenance cost.

(b) Prestressed concrete tank construction methods have been standardized and mechanized to a far greater degree than an other system of prestressed concrete structures of comparable size.

(c) Most of the designed dimensions for prestressed circular structures are based on actual tests and simple, direct computations that do not require the complicated assumptions necessary for conventional reinforced concrete.

(d) Larger size tanks can be made with prestressed concrete which was not possible with conventional reinforced concrete.

(e) There is a great saving in the quantity of materials used for construction as compared to the quantity of materials required for reinforced concrete tanks.