THIN SHELL CONCRETE WATER TOWER CONSTRUCTED WITH BALLOON FORMWORK

A Project

Presented to the

Department of Civil Engineering

Brigham Young University

In Partial Fulfillment

of the Requirements for the Degree

Master of Engineering

by

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August 1983

This project, by Kurt A. Warren, is accepted in its present form by the Department of Civil Engineering Science of Brigham Young University as satisfying the project requirement for the degree of Master of Engineering.

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ACKNOWLEDGMENTS

I give thanks for the support, encouragement and love my wife has given me throughout my graduate work.

I also give thanks to Dr. Arnold Wilson for his enthusiasm, help and guidance on this project.

Chapter 1

INTRODUCTION

Bids were recently opened for the design of two water towers to be built in the New Orleans, Louisiana area. The only specifications given were for its capacity and elevation (head). The shape and materials of construction were left open to the designer.

Many shapes have been used for elevated water tanks. The most efficient shape, in terms of inside surface area of tank per capacity, is the sphere. This shape should, therefore, be the most economical.

Choosing the material of construction is the next consideration. Relatively few reinforced concrete water towers are built due to the high cost of formwork and labor. However, a completely new construction technique has been developed which provides great savings in construction, labor and materials. The technique brings the cost of reinforced concrete as a construction material for water tanks to a very competitive level with that of steel.

The technique, developed by Mr. David South, Contractor from Idaho, involves the use of a balloon for the formwork. A basic explanation of his technique is as follows. A heavy vinyl material is sewn into the desired shape, anchored down, and inflated by means of an air compressor. Next, urethane foam is sprayed on the inside of the balloon. This stiffens the balloon and provides a place to fasten the reinforcing steel. Shotcrete methods are then used to apply the concrete on the inside of the tank. A thin layer of a rubber material sprayed on the inside of the tank provides water proofing. The balloon can then be stripped away and reused or left on the structure for an aesthetic covering and protection of the urethane foam.

The cost of materials is further lowered by making the tank a thin shell structure. A thin shell is a shell whose thickness is small relative to its other dimensions. The thin shell has the advantage that it maintains structural soundness while using relatively small quantities of materials.

The purpose of this project is to provide a working design of a thin shell concrete water tower, spherical in shape, and according to the specifications of the proposed water towers in Louisiana. The technique of balloon formwork makes this design practical.

Chapter 2

DESIGN EXAMPLE

Given Information

The water tower will be designed entirely of reinforced concrete. It will consist of a spherical tank supported by a hollow cylindrical tower resting on a square spread footing. Dimensions of the tank and tower are shown in Figure 1.

The design parameters will be defined as required in each of the succeeding design sections.

Wind Analysis

The wind analysis is based upon wind pressure acting on the vertical projected area of the water tower. The allowable resultant wind pressure on exterior surfaces of ordinary square buildings thirty feet above ground is 50 psf (*Uniform Building Code*,4:147) in Louisiana. Wind pressures for various height zones above ground are shown in Table 1 (*UBC*,4:140) along with their corresponding corrected values obtained by multiplying them by a shape factor of 0.6 (*UBC*,4:141).

Figure 2 shows the wind pressure zones as they relate to the structure, along with corresponding projected

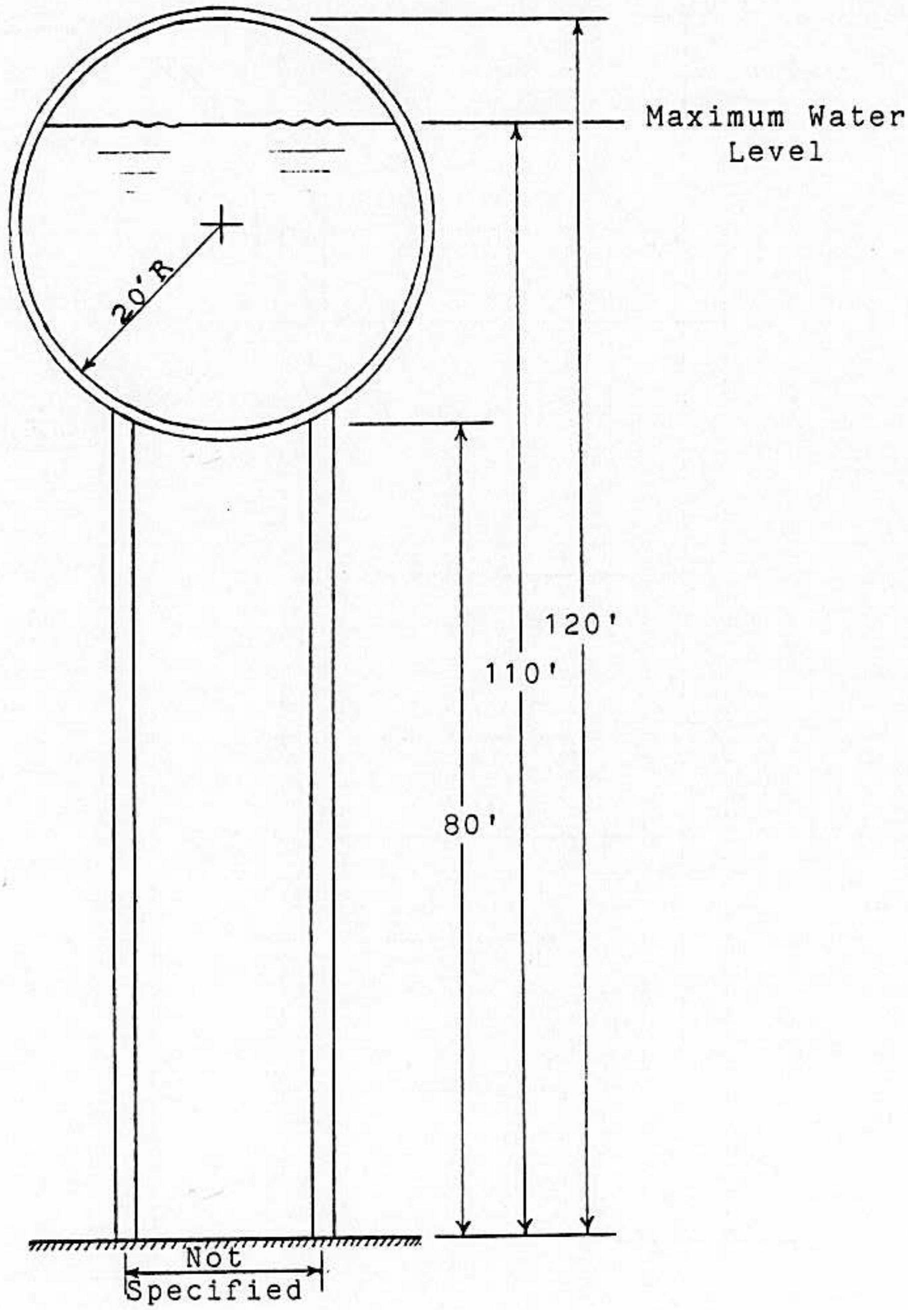

Figure 1. Dimensions

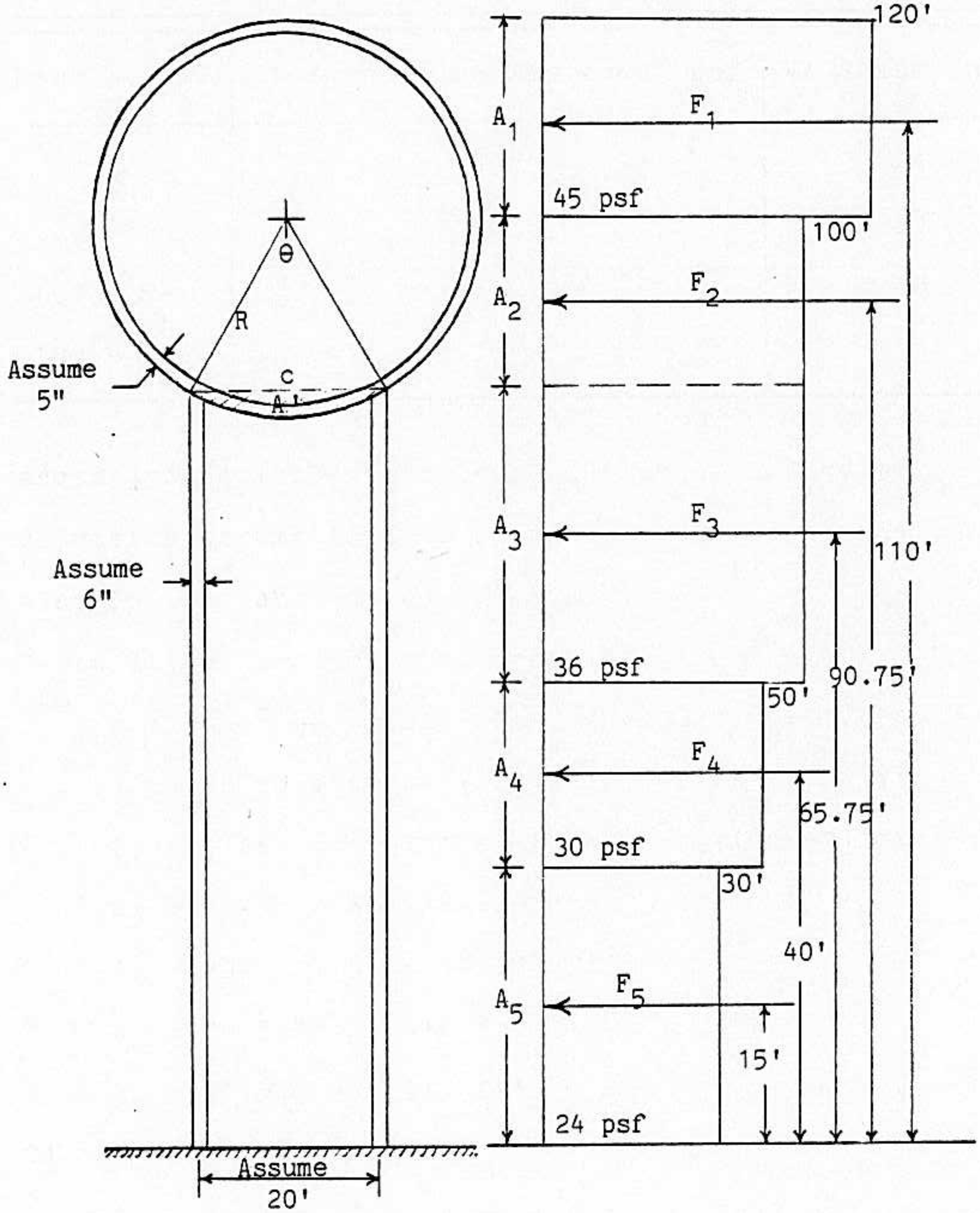

Figure 2. Wind Pressure Zones with Equivalent Forces

Table 1

Wind Pressures for Various Height Zones Above Ground

Height, ft.	Wind-Pressure Map Area, psf	W/ Shape Factor
< 30	40	24
30 - 49	50	30
50 - 99	60	36
100 - 499	75	45

areas and equivalent forces. Included in Figure 2 are dimension assumptions to be used in this analysis and also in the earthquake analysis.

Calculating the projected areas:

$$\theta = 2\sin^{-1}\frac{c}{2R} = 2\sin^{-1}[20.5/2(20.42)] = 1.0519 \text{ rad}$$

$$A' = \tfrac{1}{2}R^2(\theta - \sin\theta) = \tfrac{1}{2}(20.42)^2(\theta - \sin\theta) = 38 \text{ ft}^2$$

$$A_1 = \tfrac{1}{2}(\text{Area of Circle}) = \tfrac{1}{2}\pi R^2 = \tfrac{1}{2}\pi(20.42)^2 = 655 \text{ ft}^2$$

$$A_2 = A_1 - A' = 655-38 = 617 \text{ ft}^2$$

$$A_3 = 20.5(31.5) = 645.75 \text{ ft}^2$$

$$A_4 = 20.5(20) = 410 \text{ ft}^2$$

$$A_5 = 20.5(30) = 615 \text{ ft}^2$$

Calculating the forces:

$$F_1 = A_1(\text{Wind Pressure}) = 655 \text{ ft}^2(.045 \text{ psf}) = 29.5^k$$

$$F_2 = 617(.036) = 22.2^k$$

$$F_3 = 645.75(.036) = 23.2^k$$

$$F_4 = 410(.030) = 12.3^k$$

$F_5 = 615(.024) = 14.8^k$

Calculate overturning moment:

$$O.M. = 29.5(110)+22.2(90.75)+23.2(65.75)+12.3(40)$$
$$+14.8(15) = 7{,}499'k$$

According to the UBC(4:126), the overturning moment due to wind is not to exceed two-thirds of the dead load resisting moment.

Dead load without water:

$$W = 150 \text{ pcf}[4/3\pi((20+5/12)^3-(20)^3)$$
$$+80\ ((20+3/12)^2-(20-3/12)^2)] = 1084.4^k$$

The resisting moment becomes:

$$1084.4(10.25)(2/3) = 7410'k$$

The resisting moment is almost equal to the overturning moment. This comparison is not complete in that it does not include the effects of the footing in resisting overturning moments. The footing will be designed in a later section.

Earthquake Analysis

The earthquake analysis of this section is based upon formulas and procedures as outlined in the UBC (4:126). The forces calculated by this procedure will be assumed to act at the same locations as those for the wind analysis for easy comparison.

Weight of water: $W_w = 62.4[\pi(4/3(20)^3-1/3(10)^2(3(20)-10))]$
$= 1764.3^k$

Total dead load: $W = 1084.4+1764.3 = 2848.7^k$

The minimum total lateral seismic forces: $V = ZIKCSW$ (UBC,4:128).

The water tanks will be built in a Zone 1 earthquake zone from which Z is taken as 3/16 (UBC,4:128). From Table No. 23-I (UBC,4:142), K=2.0. From Table No. 23-K (UBC,4:144), I=1.5. The combined value of CxS will be assumed to be 0.14 (UBC, 4:128).

Therefore, $V = (3/16)(1.5)(2.0)(0.14)(2848.7) = 224.3^k$.

The value of F_t will be assumed negligible.

The distributed forces (Figure 3) are calculated by the equation

$$F_x = \frac{Vw_xh_x}{\sum_{i=1}^{n} w_ih_i} \quad \text{(UBC,4:130).}$$

$$\sum_{i=1}^{n} w_ih_i = 81.02(110)+1677.57(90.75)+2221.3(65.75)$$
$$+2467.02(40)+2705.59(15) = 446,466.8$$

$$F_1 = \frac{224.3(81.02)(110)}{446,466.8} = 4.5^k$$

$$F_2 = 76.5^k$$

$$F_3 = 73.4^k$$

$$F_4 = 49.6^k$$

$$F_5 = 20.4^k$$

The earthquake analysis forces control except for F_1. For the worst possible case, the overturning moment will be calculated using F_1 from the wind analysis and

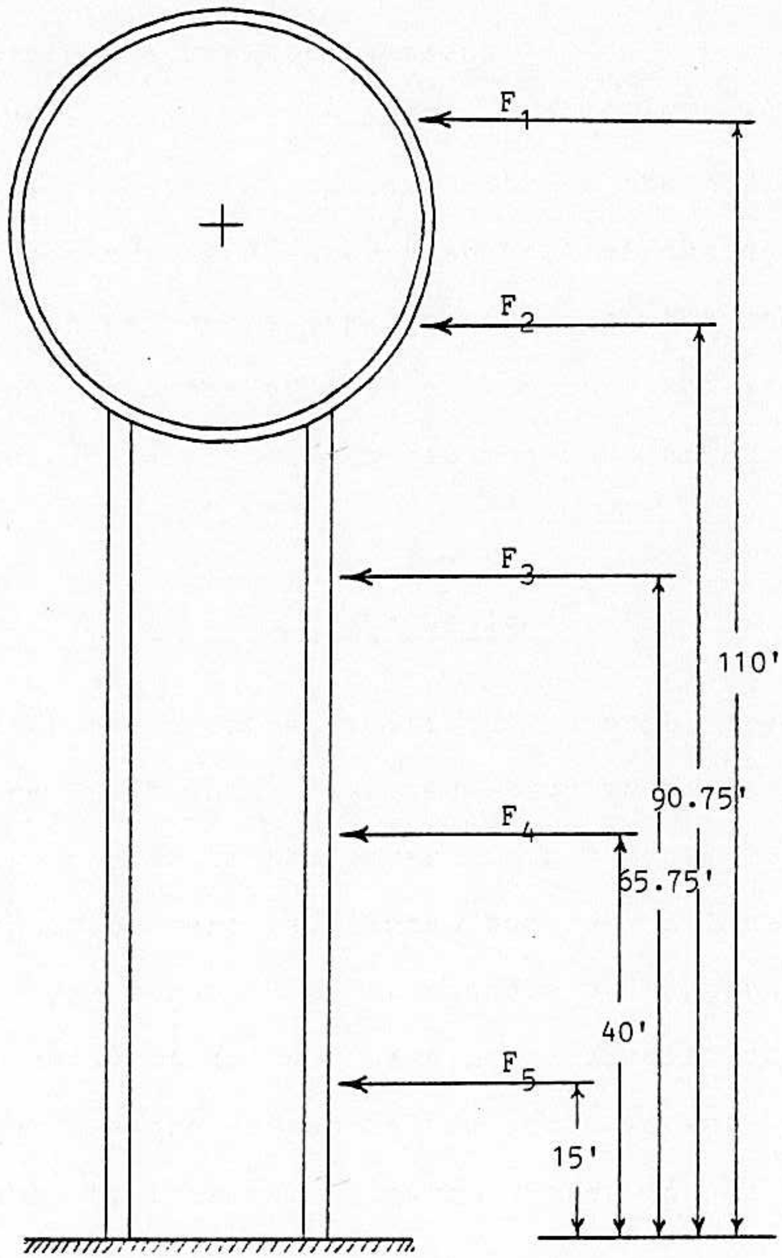

Figure 3. Earthquake Forces

forces F_2, F_3, F_4, and F_5 from the earthquake analysis.

$$O.M. = 29.5(110)+76.5(90.75)+73.4(65.75)$$
$$+49.6(40)+20.4(15) = 17{,}303'k$$

Calculating the resisting moment:

$$R.M. = 2/3(2848.7)(10.25) = 19{,}466'k$$

The resisting moment, without the added effects of the footing, is greater than that of the overturning moment. Therefore, a tank and tower of the approximate dimensions as those of this preliminary analysis should be adequate against either maximum wind or earthquake forces.

Tower Design

After an extensive library search, several articles were found which analyzed structures similar in dimension ratios to the tower of this problem. The structures analyzed were reinforced concrete chimneys or similar type structures not capable of supporting large axial loads. It did not take many calculations to determine that the procedures in the articles were not applicable to this water tower. Refer to the bibliography for the list of articles researched.

Two procedures will be used to analyze the tower. The first is an approximation assuming the tower acts as a beam with stresses calculated using $-P/A \pm Mc/I$. From this, the dimensions of the tower will be found. The second procedure assumes a balanced condition of the steel and concrete from which the reinforcing steel will be determined.

First Procedure

For all concrete in the structure, the compressive strength, f'_c will be assumed to be 4000 psi. The reinforcing steel strength, f_y will be assumed to be 60,000 psi. A live load of 40 psf over the horizontal projected area of the tank will be assumed.

The working stress method will be used in this procedure. From this, $f_c = .45f'_c = 1800$ psi and $f_s =$ 24,000 psi for Grade 60 steel. To keep the strains due to creep in the concrete down, the stress in the concrete should be designed for about one-half of f_c, or about 900 psi.

A program was written for the HP 41CV hand calculator which calculates the positive and negative stresses at the bottom of the tower due to the combined earthquake and wind forces previously found and due to the total dead and live loads, including the water. The program allows the user to input the outside diameter of the tower (in feet) and the shell thickness (in inches). Several tries can be made very quickly and a design chosen for the desirable values of stresses. Figure 4 shows a listing of the program.

The results of several tries are listed in Table 2. From this, an outside diameter of 20' and thickness of wall of 8" were chosen.

Table 2

Results From Calculator Program

Dia., ft.	t, in.	-P/A-Mc/I	-P/A+Mc/I
10	6	-4466	+2310
15	6	-2190	+720
20	6	-1379	+243
15	8	-1723	+567
20	8	-1086	+186
15	10	-1445	+478
20	10	-911	+153

Second Procedure

The reinforcing steel for the tower will now be determined. For a thin shell section in tension, the reinforcement is not to be less than 0.0035 times the gross cross-sectional area of the shell (ACI Code,2:75). Thus, the min. A_s = .0035(40.49 ft^2)(144) = 20.41 in^2. Maximum spacing is 18 in. (ACI Code,2:75). The circumference of the tower at the location of steel is

```
LBLᵀST
LBL 01
0
STO 99
SF 21
161151.68
STO 99
2137.7
STO 50
ᵀDIA<FT>=?
PROMPT
6
*
STO 01
ᵀTHICK<IN>=?
PROMPT
STO 02
-
X↑2
CHS
RCL 01
X↑2
+
PI
*
144
/
STO 03
2.138
*
RCL 50
+
65.75
*
ST+ 99
STO 20
RCL 03
6
*
RCL 50
+
40
*
ST+ 99
STO 21
RCL 03
9.75
*
RCL 50
+
15
*
ST+ 99
STO 22
RCL 03
12
*
STO 04
2137.7
+
.08438
*
RCL 99
/
STO 98
RCL 20
*
65.75
*
RCL 21
RCL 98
*
40
*
+
RCL 22
RCL 98
*
15
*
+
10314.43
+
STO 08
RCL 04
RCL 50
+
1000
*
RCL 03
/
144
/
STO 05
RCL 01
RCL 08
12
*
*
RCL 01
4
Y↑X
RCL 01
RCL 02
-
4
Y↑X
-
PI
*
4
/
STO 07
/
1000
*
STO 09
RCL 05
+
FIX 0
ᵀFX=-
ARCL X
ᵀ├ PSI
AVIEW
RCL 09
RCL 05
-
ᵀFc=+
ARCL X
ᵀ├ PSI
AVIEW
GTO 01
END
```

Figure 4. HP 41CV Program to Calculate Stresses

$2\pi(10-(4/12))12 = 728.85$ in.

The number of bars is

$728.85/18 = 40.49$ bars. Try 44 #7 bars. $A_s = 26.4$ in^2

The actual spacing around the circumference is

$(360/44)(2\pi/360)(9.67')(12) = 16.56"$.

Figure 5(a) shows relative placement of the steel. Coordinates of the bars are shown in Table 3.

Table 3

X and Y Coordinates of Steel
Taken From Center of Tower

Bar	X	Y
1	9.67	0.00
2	9.57	1.38
3	9.28	2.72
4	8.79	4.02
5	8.13	5.23
6	7.31	6.33
7	6.33	7.31
8	5.23	8.13
9	4.02	8.79
10	2.72	9.28
11	1.38	9.57
12	0.00	9.67

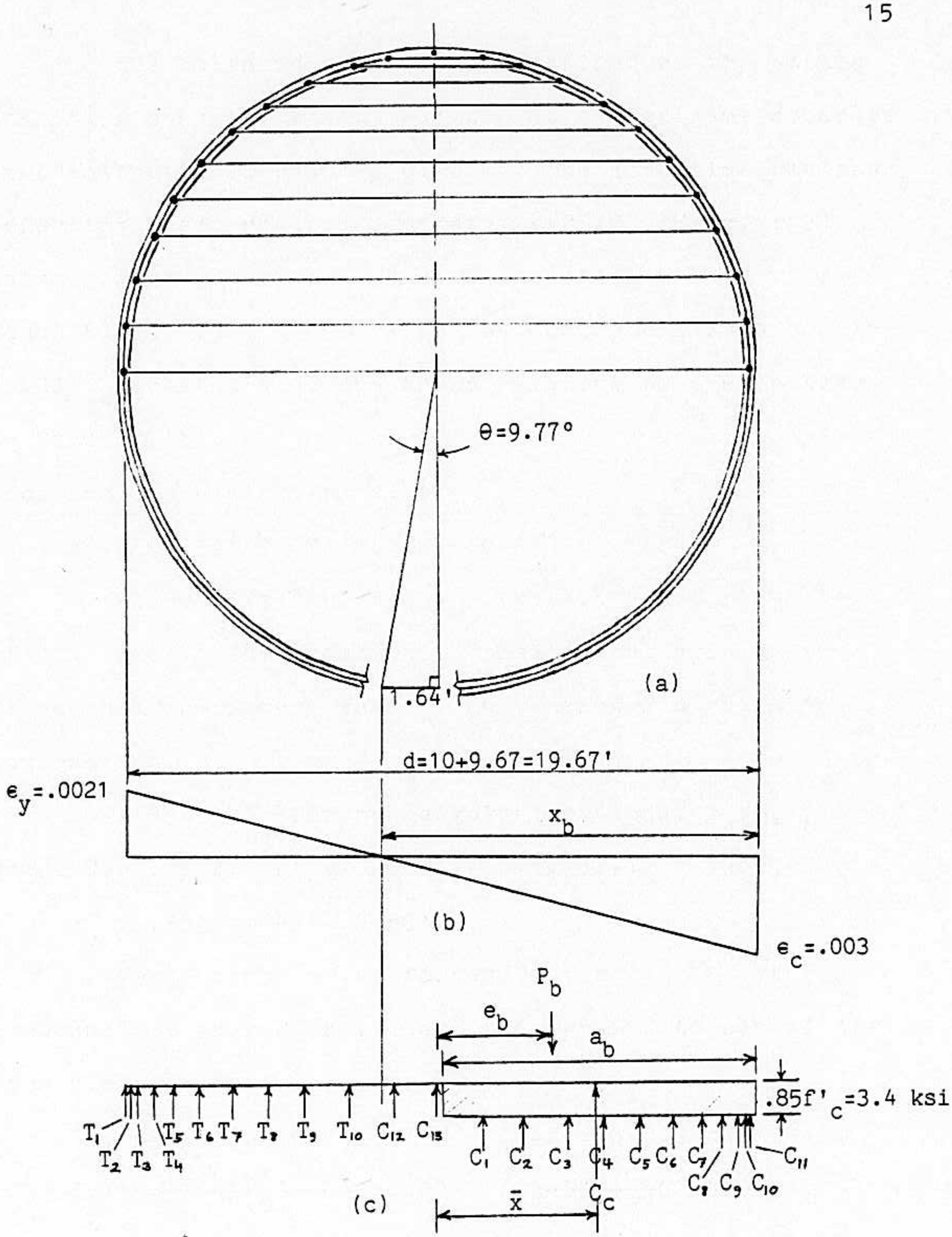

Figure 5. (a) Plan View of Tower Showing Steel (b) Strain Distribution at Balanced Condition (c) Cut Side View of Tower Showing Forces in Steel and Concrete

The balanced condition is defined as the strain condition when the maximum strain at the extreme concrete compression fiber reaches 0.003 at the same time the reinforcing steel reaches a strain, $e=f_y/E_s$ (Wang,4:42). Assuming this condition exists, the steel strain will equal 60/29,000 = 0.0021 when the concrete strain equals 0.003. These strains are shown relative to one another in Figure 5(b).

Locating the neutral axis:

$$x_b = [.003(19.67)]/[.003+.0021] = 11.64'$$

Figure 5(c) shows the forces in a cut side view of the tower. The location of the neutral axis determined which bars were assumed in tension and which in compression.

Assume, as for rectangular sections, $a_b=\beta_1 x_b$, where $\beta_1=0.85$ for $f'_c=4000$ psi (Wang,5:41). Thus,

$$a_b = .85(11.64) = 9.89'.$$

Next, compute the forces T_1 through T_{10} and C_1 through C_{13}. But first, check the strains to see if the bars yield or not:

T_1:	$e=e_y=.002069$	$f_s=f_y=60{,}000$ psi	
T_2:	$e=\frac{(9.57-1.64)e_y}{8.03}=.0200$	$f_s=\frac{.0020}{.002069}60{,}000$	$=59{,}253$
T_3:	$e= \quad =.0020$	$f_s=$	$=57{,}086$
T_4:	$e= \quad =.0018$	$f_s=$	$=53{,}425$
T_5:	$e= \quad =.0017$	$f_s=$	$=48{,}493$

T_6: $\epsilon=$	$=.0015$	$f_s=$	$=42,366$
T_7: $\epsilon=$	$=.0012$	$f_s=$	$=35,044$
T_8: $\epsilon=$	$=.0009$	$f_s=$	$=26,824$
T_9: $\epsilon=$	$=.0006$	$f_s=$	$=17,783$
T_{10}: $\epsilon=$	$=.0003$	$f_s=$	$=8,070$
C_{12}: $\epsilon=.003\frac{.26}{19.67}$	$=.00004$	$f_s'=$	$=1,150$
C_{13}: $\epsilon=$	$=.00025$	$f_s'=$	$=7,254$
C_1: $\epsilon=$	$=.0005$	$f_s'=$	$=13,357$
C_2: $\epsilon=$	$=.0007$	$f_s'=$	$=19,284$
C_3: $\epsilon=$	$=.0009$	$f_s'=$	$=25,034$
C_4: $\epsilon=$	$=.0010$	$f_s'=$	$=30,385$
C_5: $\epsilon=$	$=.0012$	$f_s'=$	$=35,251$
C_6: $\epsilon=$	$=.0014$	$f_s'=$	$=39,585$
C_7: $\epsilon=$	$=.0015$	$f_s'=$	$=43,212$
C_8: $\epsilon=$	$=.0016$	$f_s'=$	$=46,131$
C_9: $\epsilon=$	$=.0017$	$f_s'=$	$=48,298$
C_{10}: $\epsilon=$	$=.0017$	$f_s'=$	$=49,581$
C_{11}: $\epsilon=$	$=.0017$	$f_s'=$	$=50,023$

Now compute the forces:

$$T_1=A_sf_y=0.6(60,000)=36.0^k$$

$$T_2= \quad =1.2(59,253)=71.1$$

$$T_3= \quad =68.5$$

$$T_4= \quad =64.1$$

$$T_5= \quad =58.2$$

$$T_6= \quad =50.8$$

$T_7=$ $=42.1$

$T_8=$ $=32.2$

$T_9=$ $=21.3$

$T_{10}=$ $= 9.7$

The $.85f'_c$ in calculating the compressive forces is a correction for the concrete displaced by steel.

$C_{12}=A_s'f_s'=1.2(1150)=1.4^k$

$C_{13}=A_s'(f_s'-.85f'_c)=1.2(7254-3400)= 4.6$

$C_1=$ $=11.9$

$C_2=$ $=19.1$

$C_3=$ $=26.0$

$C_4=$ $=32.4$

$C_5=$ $=38.2$

$C_6=$ $=43.4$

$C_7=$ $=47.8$

$C_8=$ $=51.3$

$C_9=$ $=53.9$

$C_{10}=$ $=55.4$

$C_{11}=$ $=0.6(50,023-3400)=28.0$

The following procedure for calculating C_c and its point of application was taken from Wang(5:412). A method of coefficients is used to obtain values of area and first moment of circular segments from charts presented in the above reference.

Calculating η:

for outside diameter, $\eta_o=a/h=9.89/20 = .49$

for inside diameter, $\eta_i = 9.22/18.66 = .49$

From the charts:

the area coefficient = .385 for both diameters

the Q coefficient = .083 for both diameters

Calculating the areas:

$$A_o = .385(h)^2 = .385(20)^2 = 154.00$$

$$A_i = \quad = .385(18.67)^2 = \underline{134.15}$$

$$\text{difference} = \quad 19.85 \text{ ft}^2$$

Calculating Q:

$$Q_o = .083(h)^3 = .083(20)^3 = 664.00$$

$$Q_i = \quad = .083(18.67)^3 = \underline{539.86}$$

$$\text{difference} = 124.14 \text{ ft}^3$$

A check to compare with the method of coefficients is to multiply the area of the segment, assuming it is correct as calculated from coefficients, by the distance to the mass center. The distance to the mass center is

$$2r/\pi = 2(9.33)/\pi = 5.94 \text{ ft}$$

Checking Q:

$$Q = A(\text{dist.}) = 19.85(5.94) = 117.94 \approx 124 \qquad \underline{\text{OK}}$$

Calculate the point of application of C_c:

$$\bar{x} = Q/A = 124.14/19.85 = 6.25 \text{ ft}$$

Calculating C_c:

$$C_c = .85f'_c(\text{A of segment}) = .85(4)(19.34)(144)$$

$$= 9718.6^k$$

The load to cause the balanced condition, $P_b = C_c + C - T = 9678^k$

To find the point of application of P_b, first sum the moments of the forces around the centerline of the tower. From Table 4, the total moment is 67,400.5'k. Calculate e_b:

$$e_b = M_b/P_b = 67,400.5/9678 = 6.96 \text{ ft}$$

Summary of the Two Procedures

The first procedure used allowable stresses in the concrete as design criteria; the second used minimum required steel assumed to be in a balanced condition.

The moment caused by a combination of earthquake and wind forces with load factors applied is 24,322'k. The moment required to cause the tower system to reach a balanced condition is 67,400.5'k. This means the tower system will never reach a balanced condition.

In the worst loading condition, stresses at any point in the tower will be well within maximum allowable stress ranges. The addition of steel helps take care of tensile stresses. These tensile stresses will be low, as shown in Figure 6. A reduction in moment-causing loads from the worst case will tend to decrease both the tension and compression values, since the moment is the controlling factor in the stress equation, for this problem. Therefore, the tower dimensions and reinforcing steel are adequate as designed.

Calculate the horizontal or hoop steel:

Table 4

Moments of Forces Around the Centerline

Force, k	Arm, ft	Moment, 'k
T_1 = 36.0	9.67	348.1
T_2 = 71.1	9.57	680.4
T_3 = 68.5	9.28	635.7
T_4 = 64.1	8.79	563.4
T_5 = 58.2	8.13	473.2
T_6 = 50.8	7.31	371.3
T_7 = 42.1	6.33	266.5
T_8 = 32.2	5.23	168.4
T_9 = 21.3	4.02	85.6
T_{10} = 9.7	2.72	26.4
C_c = 9718.6	6.25	60,741.3
C_{12} = 1.4	-1.38	-1.9
C_{13} = 4.6	0.00	0.0
C_1 = 11.9	1.38	16.4
C_2 = 19.1	2.72	52.0
C_3 = 26.0	4.02	104.5
C_4 = 32.4	5.23	169.5
C_5 = 38.2	6.33	241.8
C_6 = 43.4	7.31	317.3
C_7 = 47.8	8.13	388.6
C_8 = 51.3	8.79	450.9
C_9 = 53.9	9.28	500.2
C_{10} = 55.4	9.57	530.2
C_{11} = 28.0	9.67	270.8

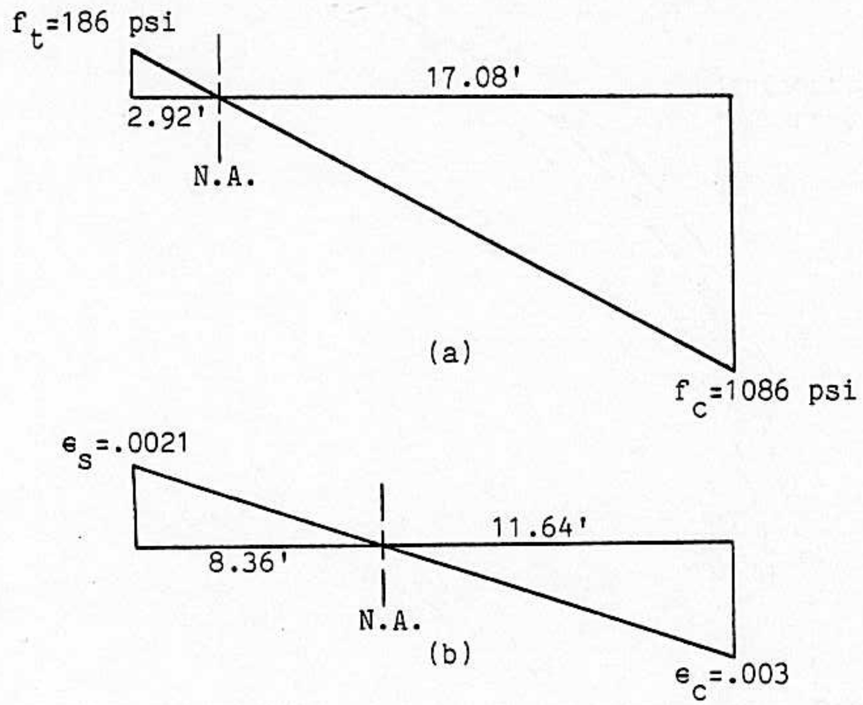

Figure 6. (a) Stresses due to Earthquake, Wind, Dead and Live Loads (b) Strains at Balanced Condition

A_s = .002(8")(12") = .19 in²/ft

use #4 bars @ 12" o.c.

Summary of the tower design specifications (Figure 7):

Outside Diameter = 20 ft

Thickness = 8 in

f'_c = 4000 psi

Longitudinal reinforcing steel: 44 #7 bars equally spaced; use Class B splice which is that no more than ½ of the steel will be spliced at any section.

Horizontal reinforcing steel: #4 bars @ 12" o.c.

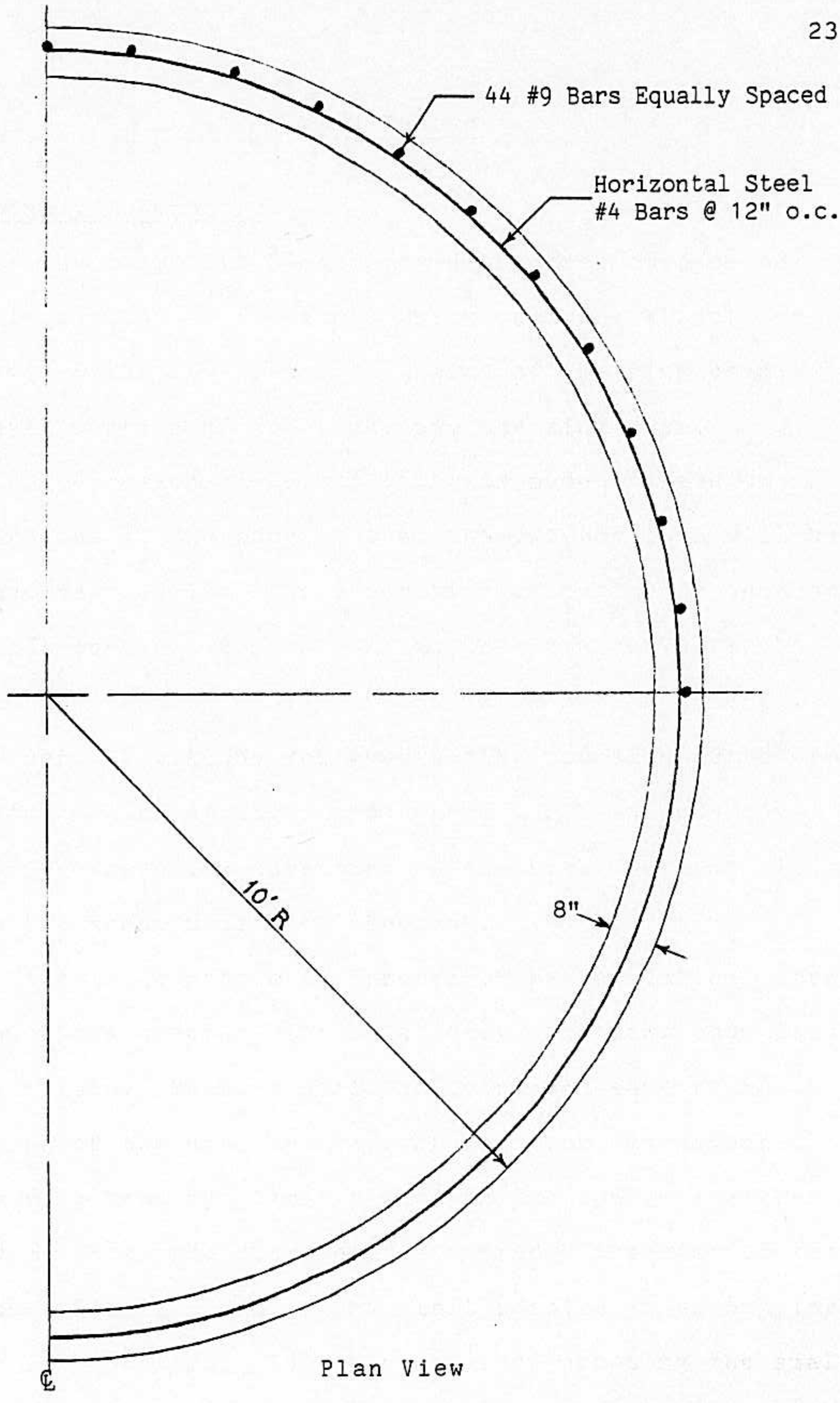

Figure 7. Tower Reinforcement Detail

Tank Design

Preliminary Information

The tank will be comprised of three components: the main portion of the tank, which includes all of the tank except the small section inside of the ring beam; the small portion of the tank; and the ring beam.

The reason the distinction is made between the two portions of the tank is that the two portions will be designed separately. The separation of design is because the small portion will not follow the same curve as the main portion (which would have made the tank a sphere). But rather, it will be designed having opposite curvature than it would have had if the sphere had been made complete. The opposite curvature of the small portion will aid in the construction of the tank.

In the construction process, the foundation, then the tower are erected. At this point, the ring beam is cast in place. Next, a small balloon, pre-sewn to match the shape of the small portion of the tank, is attached to the ring beam by clamps placed in the concrete either before or after the ring beam is poured. The rest of the procedure for fabricating the small portion is as outlined in the introduction. In the process of shooting the small portion, a hole can be blocked out. When this portion of the tank is strong enough, it can be used as a platform from which to work on the rest of the tank. Hoses can be passed through the hole. For the remainder of the tank, a balloon is attached to the ring beam and the same procedure is followed for its fabrication as for the small portion. Figure 8 represents a view of the tank showing its three components relative to the tower. It also shows the location of clamps and balloons previously mentioned.

A check is made to see how the curvature of the bottom portion of the tank affects the maximum elevation of the water. The initial capacity as calculated from the given information (Figure 1) is 28,274.33 ft^3. Keeping the capacity the same and solving for h (Figure 9),

$$28{,}274.33 = 4/3\pi(20)^3 - 2/3\pi(2.68)^2(3(20)-2.68)$$
$$-\pi(20)h^2 + \pi h^3/3$$

$$h^3 - 60h^2 + 13{,}121.22/\pi = 0$$

$$h = 9.05 \text{ ft}$$

The new maximum elevation of water = 120 - 9.05 = 110.95 ft. This change will not cause significant change in the earthquake analysis so the design thus far is still valid.

Design Procedure

The design of the tank will be accomplished through the use of a computer program developed by a

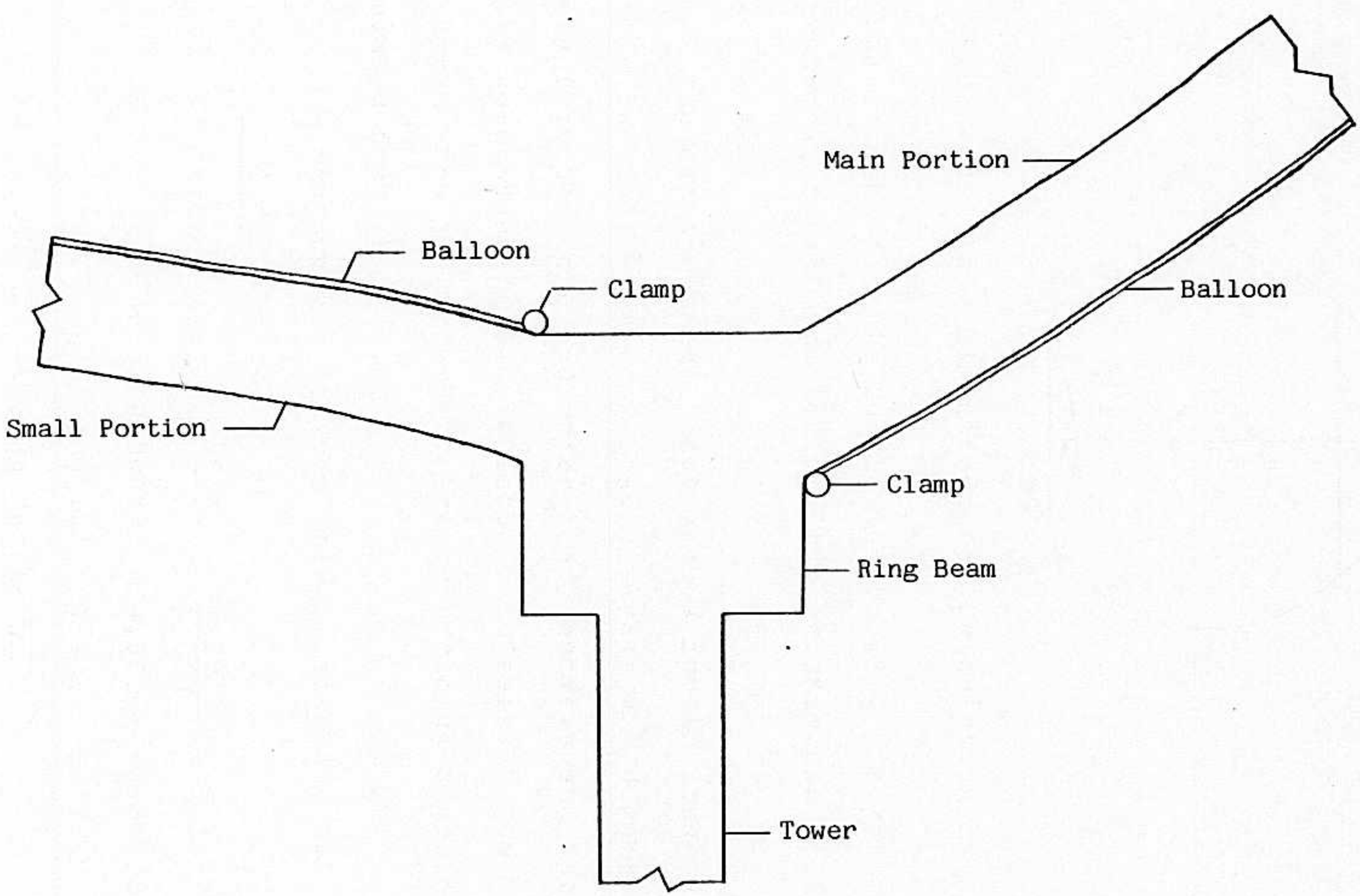

Figure 8. Cut Section of Ring Beam with Clamp Positioning

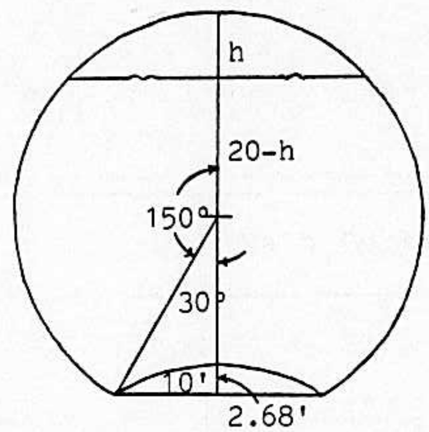

Figure 9. Tank Dimensions for Calculating h

graduate student for his thesis (Hoggan,3). Background information and verification for the program can be found in his thesis.

The chosen design input values for the tank are shown in Table 5. The snow load analysis was done separately from the dead and water load analysis and the results were superimposed in Table 6. Tables 6 and 7 show the output from the program. The reinforcing steel is calculated from this data.

The maximum and minimum steel limits are as follows (ACI Code,2:75):

For 4" thick: $\max. = 7.2\frac{hf'c}{f_y} = 7.2\frac{4(4000)}{60,000} = 1.92\ in^2/ft \leftarrow$

$\text{or} = 29,000h/f_y = 29,000\frac{4}{60,000} = 1.93\ \frac{in^2}{ft}$

$\min = 0.0035(4)(12) = .17\ in^2/ft$

For 6" thick: $\max = 2.88\ in^2/ft \leftarrow$

$\text{or} = 2.90\ in^2/ft$

$\min = 0.25\ in^2/ft$

Table 5

Input Data for Tank Design Program

Input Parameter	Main Portion	Small Portion
Radius of Curvature	20'	20'
Total Angle of Sphere	150°	30°
Thick. of Shell @ Apex	.333'	.667'
Thick. of Shell @ Edge	.667'	.667'
Unit Weight of Concrete	150 pcf	150 pcf
Poisson's Ratio	.2	.2
LL of Shell	40 psf(snow)	1764 psf(H_2O)
Comp. Strength of Concrete	4000 psi	4000 psi
Internal Pressure	0.0 psf	0.0 psf
Sp. Wt. of Fluid	62.4 pcf	0.0 pcf
Width of Ring	1.5'	1.5'
Depth of Ring	1.5'	1.5'

For 8" thick: max. = 3.84 in²/ft ←

or = 3.87 in²/ft

min. = 0.34 in²/ft

The development lengths for rebar are (Wang,5:193):

L_d for #4 bar = 12";

L_d for #5 bar = 15";

L_d for #6 bar = 18".

Table 6

Computer Output and Calculated Areas of Steel in Tank
Totals from Snow Load Analysis Combined with
Dead Load and Water Load Analysis
Main Portion

Angle From Dome Edge	N-Phi Force k/ft	N-Theta Force k/ft	N-Theta' Equiv. k/ft	M-Phi Moment ft-k/ft	N-Theta' A_s in^2/ft	M-Phi A_s in^2/ft
1.0	-39.729	-22.955	-17.761	-0.270	0.00	0.05
2.0	-39.198	-10.922	-8.451	-2.700	0.00	0.35
3.0	-38.347	0.949	0.734	-4.479	0.04	0.58
4.0	-37.224	12.246	9.475	-5.703	0.53	0.75
5.0	-35.876	22.676	17.546	-6.464	0.97	0.84
6.0	-34.350	32.046	24.796	-6.845	1.38	0.89
7.0	-32.694	40.248	31.142	-6.927	1.73	0.89
8.0	-30.952	47.239	36.551	-6.774	2.03	0.84
9.0	-29.158	53.028	41.030	-6.446	2.28	0.84
10.0	-27.349	57.663	44.617	-5.996	2.48	0.77
11.0	-25.549	61.216	47.366	-5.464	2.63	0.71
12.0	-23.783	63.785	49.354	-4.888	2.74	0.64
13.0	-22.068	65.472	50.659	-4.254	2.81	0.56
14.0	-20.417	66.390	51.369	-3.705	2.85	0.48
15.0	-18.840	66.648	51.569	-3.140	2.86	0.40
20.0	-12.209	61.541	47.617	-0.956	2.65	0.12
25.0	-7.561	52.553	40.663	0.058	2.26	0.01
30.0	-4.403	44.571	34.487	0.299	1.92	0.10
35.0	-2.241	38.716	29.957	0.226	1.66	0.09
40.0	-0.734	34.518	26.708	0.104	1.48	0.04
50.0	1.040	28.402	21.976	-0.008	1.22	0.00
60.0	1.762	23.199	17.950	-0.008	1.00	0.00
70.0	1.844	18.423	14.255	0.000	0.79	0.00
80.0	1.546	14.102	10.911	0.000	0.61	0.00
90.0	1.041	10.239	7.922	0.000	0.44	0.00
100.0	0.451	6.849	5.299	0.000	0.29	0.00
110.0	-0.136	3.976	3.076	0.000	0.17	0.00
120.0	-0.654	1.668	1.291	0.000	0.07	0.00
130.0	-1.058	-0.021	-0.016	0.000	0.00	0.00
140.0	-1.312	-1.052	-0.814	0.000	0.00	0.00
150.0	-1.400	-1.400	-1.083	0.000	0.00	0.00

Table 7

Computer Output and Calculated Areas of Steel in Tank
Small Portion

Angle From Dome Edge	N-Phi Force k/ft	N-Theta Force k/ft	N-Theta' Equiv. k/ft	M-Phi Moment ft-k/ft	N-Theta' A_s in²/ft	M-Phi A_s in²/ft
1.0	-15.159	16.169	12.511	-0.366	0.70	0.07
2.0	-15.814	12.589	9.741	0.244	0.54	0.05
3.0	-16.425	9.059	7.009	0.713	0.39	0.14
4.0	-16.988	5.653	4.374	1.059	0.24	0.20
5.0	-17.500	2.426	1.877	1.299	0.10	0.25
6.0	-17.959	-0.582	-0.450	1.451	0.00	0.28
7.0	-18.365	-3.347	-2.590	1.530	0.00	0.29
8.0	-18.719	-5.856	-4.531	1.550	0.00	0.30
9.0	-19.024	-8.105	-6.271	1.523	0.00	0.29
10.0	-19.281	-10.097	-7.813	1.460	0.00	0.28
11.0	-19.494	-11.842	-9.163	1.371	0.00	0.26
12.0	-19.666	-13.353	-10.332	1.264	0.00	0.24
13.0	-19.801	-14.647	-11.333	1.146	0.00	0.22
14.0	-19.903	-15.742	-12.180	1.023	0.00	0.19
15.0	-19.976	-16.657	-12.888	0.899	0.00	0.17
16.0	-20.023	-17.410	-13.471	0.777	0.00	0.15
17.0	-20.049	-18.021	-13.944	0.661	0.00	0.13
18.0	-20.056	-18.507	-14.320	0.553	0.00	0.11
19.0	-20.050	-18.886	-14.613	0.453	0.00	0.09
20.0	-20.032	-19.172	-14.834	0.363	0.00	0.07
25.0	-19.912	-19.653	-15.207	0.058	0.00	0.01
30.0	-18.640	-19.295	-14.930	-0.054	0.00	0.01

Calculate the edge ring steel:

Forces from the main portion analysis caused the ring beam to be in compression, C = -215.72^k.

$$f_c = 215.72(1000)/1.5(1.5)(144) = 665.8 \text{ psi}$$

This is well within tolerable limits.

Forces from the small portion analysis caused the ring beam to be in tension, T = 113.01^k.

A_s = 113.01/18 ksi = 6.28 in²

Use 8 #8 bars A_s = 6.32 in²

Use #4 U-stirrups @ 18" o.c.

A summary of the reinforcing steel in the tank is shown in Table 8 and diagrams of the details are shown in Figures 10 and 11.

Table 8

Summary of Tank Reinforcement

Dist. From Edge, ft	N-Theta' Horz. Steel	Dist. From Edge, ft	M-Phi Vert. Steel
Main Portion			
0-7'7"	#6 @ 3½" o.c. Double Mat Staggered 1" Clear Cover	0-5'	#6 @ 5½" o.c. Both Sides Extend L_d=18" Both Ends
7'7"-10'7"	#8 @ 4" o.c. Single Mat	5'-10'6"	#4 @ 5½" o.c. See Figure 10 Extend L_d=12" Both Ends
10'7"-20'11"	#7 @ 4" o.c. Single Mat		
20'11"-38'5"	#7 @ 7" o.c. Single Mat	10'6"-Top	#4 @ 12" o.c. Extend L_d=12" @ End
38'5"-Top	#4 @ 11½" o.c. Single Mat		
Small Portion			
0-1'3"	#5 @ 5" o.c. Single Mat	All the Way	#5 @ 10" o.c. L_d=15" into Tension Ring
1'3"-Top	#5 @ 10" o.c. Single Mat		

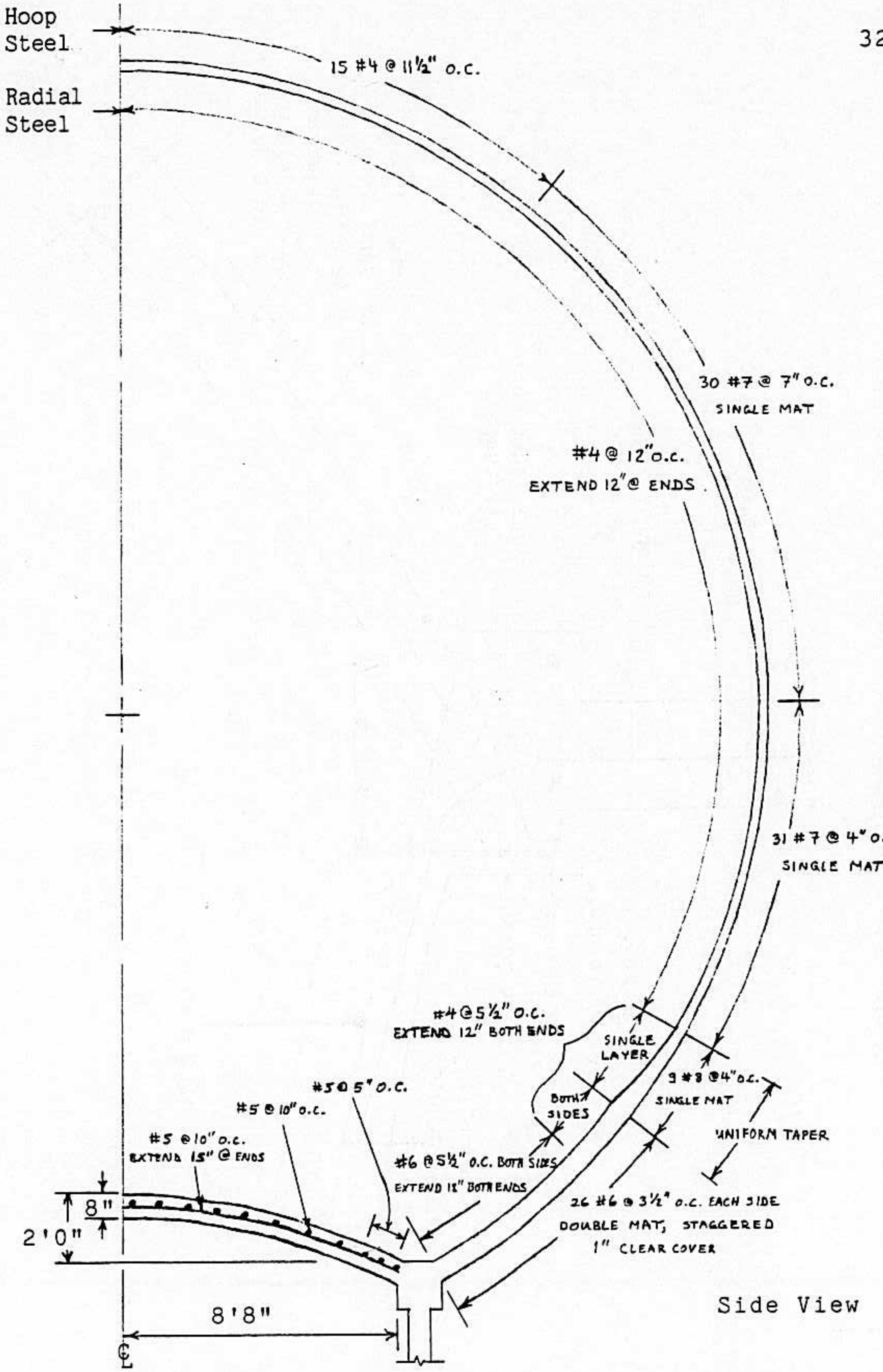

Figure 10. Reinforcement Detail in Tank

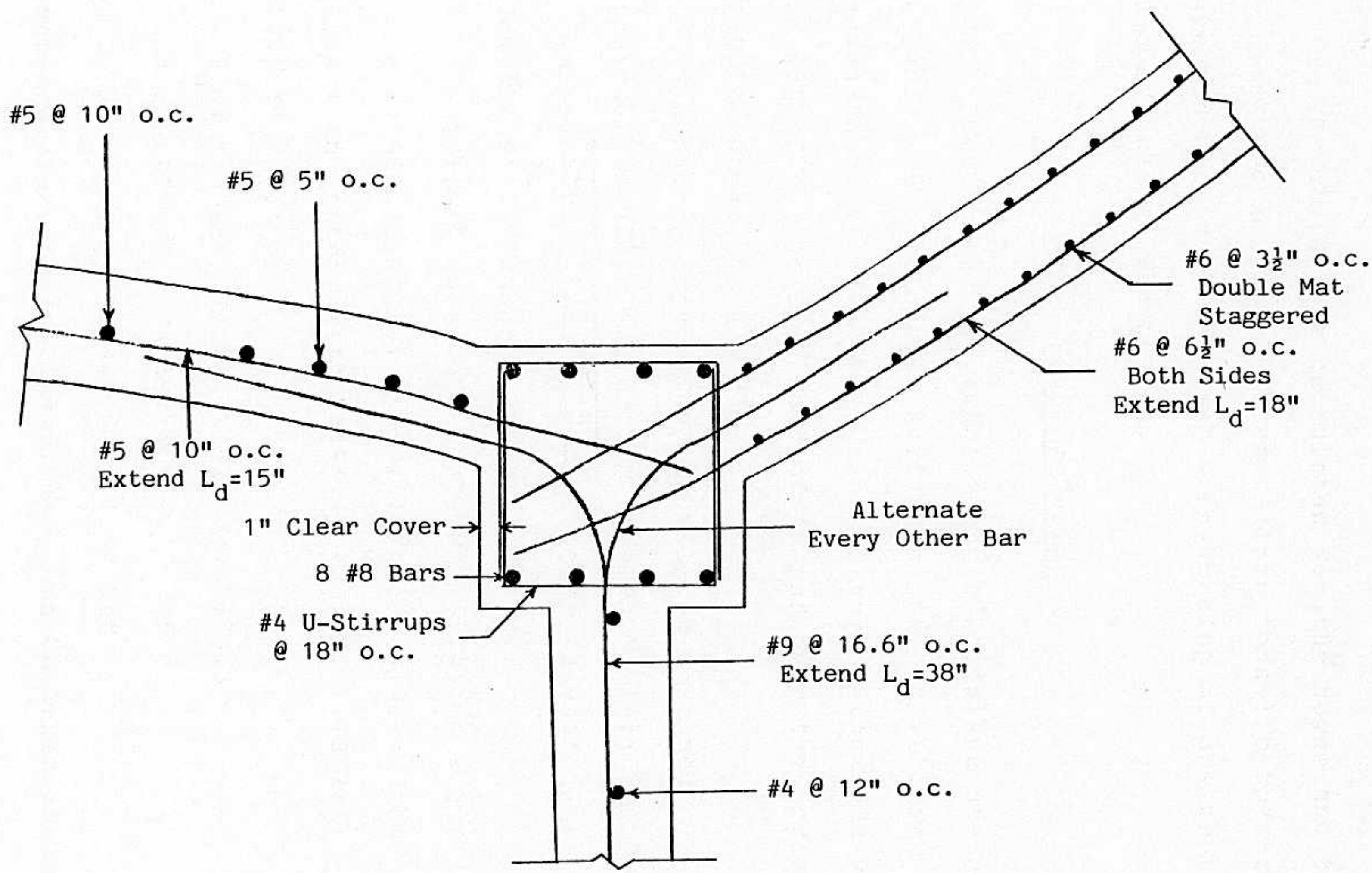

Figure 11. Reinforcement Detail in Ring Beam

The dimensions calculated for the tower and tank are close enough to the assumed dimensions for the earthquake and wind analyses that a re-working of the problem would not change the design.

Footing Design

The actual soil conditions in Louisiana are not available. The allowable soil pressure, q_a will be assumed to be 3 ksf. The weight of tower, tank and water, DL = 4817.3^k. The live load of 40 psf over the horizontal projected area, LL = 52.4^k.

A square spread footing will be designed.

Calculate the width of footing (Bowles,1:216):

$$B = \sqrt{\frac{DL+LL}{q_a}} = \sqrt{\frac{4817.3+52.4}{3}} = 40.3' \quad \text{say } 41'$$

Check the soil pressure (Bowles,1:256):

$$q = P/A(1 \pm 6e/B) \text{ where } e = M/P = \frac{17,558}{4869.7} = 3.61'$$

$$q = 4869.7/(41)^2(1 \pm 6(3.61)/41) = 4.43 > 3 \text{ ksf} \quad \text{N.G.}$$

By trial and error, with B=49', q = 2.92, 1.13 ksf.

Calculate q_{ult}:

$$P_u = 1.4(4817.3) + 1.7(52.4) = 6833.3^k$$

$$q = 2.92(6833.3/4869.7) = 4.10 \text{ ksf}$$

$$= 1.13(6833.3/4869.7) = 1.59 \text{ ksf}$$

Since the pressure is linear, q_{ult} can be taken as the average.

$$q_{ult} = (4.10 + 1.59)/2 = 2.85 \text{ ksf}$$

Now, find the depth for shear.

Calculate the diagonal-tension shear value (Bowles,1:212):

$$v_c = 4\phi\sqrt{f'_c} = 4(.85)\sqrt{4000} = 215.0 \text{ psi} = 30.97 \text{ ksf}$$

$$d^2(v_c + q/4) + d(v_c + q/2)a = q(B^2 - A_{col})/\pi$$

where a = diameter (Bowles,1:216)

$$v_c + q/4 = 30.97 + 2.85/4 = 31.68$$

$$(v_c + q/2)a = (30.97 + 2.85/2)(20) = 647.9$$

$$q(B^2 - A_{col})/\pi = (2.85)[(49)^2 - 314.16]/\pi = 1893.15$$

Substituting:

$$31.68d^2 + 647.9d = 1893.15$$

$$(d + 10.23)^2 = 59.76 + 104.55$$

$$d + 10.23 = \sqrt{164.31}$$

$$d = \pm 12.82 - 10.23 = -23.11,\ 2.59$$

use d = 2.67' = 2'8"

Next, find the bending moment for the design of A_s.

Calculate an equivalent side of a square column (Figure 12):

$$w' = \sqrt{\pi(10)^2} = 17.72'$$

Figure 13 shows the dimensions of the footing with the corresponding soil pressures created by the loads.

Calculating bending for a 1' wide strip:

$$M = \int_0^x V dx = \int_0^{15.64} (4.1x - .0512x^2/2)dx$$
$$= [4.1x^2/2 - .0512x^3/6]_0^{15.64} = 468.79'k$$

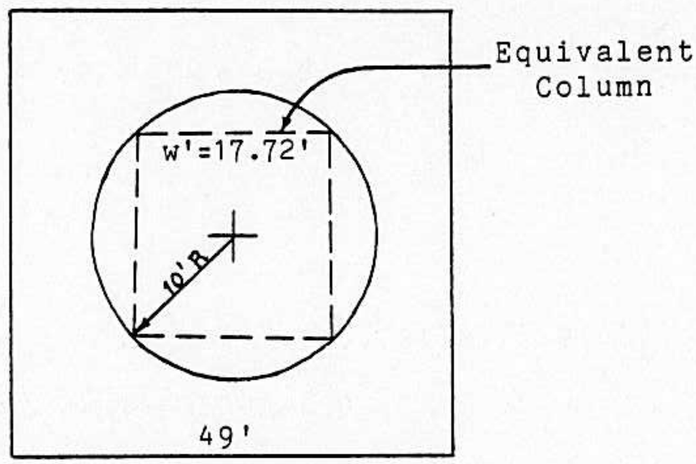

Figure 12. Plan View of Footing with Equivalent Square Column

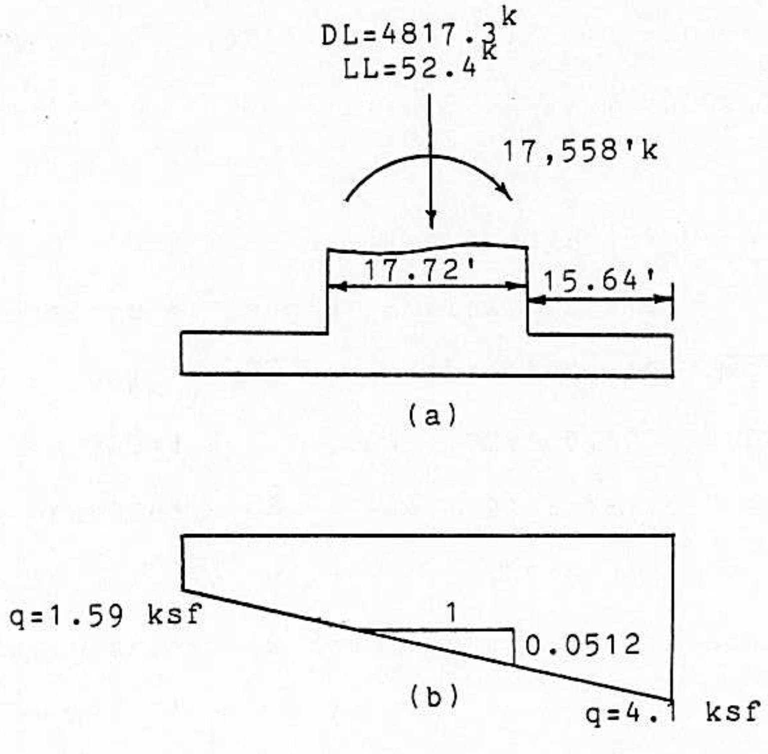

Figure 13. (a) Loads on the Tower (b) Allowable Stress Distribution in the Soil

Calculating A_s:

$M_u = \phi A_s f_y(d - a/2)$ where $\phi = .9$ and $a = \dfrac{A_s f_y}{.85f'_c b}$

(Bowles,1:214)

$a = A_s(60)/[.85(4)(12)] = 1.471A_s$

Substituting:

$A_s(2.67(12) - 1.471A_s/2) = 468.79(12)/[.9(60)]$

$A_s^2 - 43.5078A_s + 141.6391 = 0$

Solving:

$A_s = 3.54 \text{ in}^2/\text{ft}$

Minimum $\rho = .0018$ (Bowles,1:214)

Maximum $\rho = .0214$ (Bowles,1:211)

$\rho = A_s/bd = 3.54/(12)(2.67)(12) = .0092 > .0018$ OK

$< .0214$ OK

Use #9 bars @ 3" o.c. both ways $A_s = 4.00 \text{ in}^2/\text{ft}$

Check development length (Bowles,1:211):

$L_d = .04A_b f_y/\sqrt{f'_c} = .04(1.0)(60{,}000)/\sqrt{4000} = 37.95"$ ←

$= .0004d_b f_y = .0004(1.128(60{,}000) = 27.07"$

L_d provided $= \dfrac{49 - 17.72}{2}(12) = 187.68" - 3"$ cover

$= 184.68" > 37.95"$ OK

Check the bearing for dowel requirements (Bowles,1:213):

$A_1 = \pi(10)^2 = 314.16 \text{ ft}^2$

$A_2 = \pi/4[20 + 2(2.67)(2)]^2 = 739.27 \text{ ft}^2$

$\sqrt{A_2/A_1} = 1.53 < 2$ so use 1.53

$q_{brg} = f_c = .85\phi f'_c\sqrt{A_2/A_1}$ (Bowles,1:213)

$= .85(.7)(4)(1.53) = 3.65 \text{ ksi}$

Actual f_c = 6833.3/π(10²)(144) = .15 ksi < 3.65 ksi

Therefore, minimum dowels are required.

Calculate required dowels (Bowles,1:215):

$$A_s = .005A_g = .005\pi[(10)^2 - (10 - 8/12)^2](144)$$
$$= 29.15 \text{ in}^2$$

Use 30 #9 bars equally spaced A_s = 30.0 in²

Check the development length (Bowles,1:196):

$$L_d = .02(60{,}000)(1.128)/\sqrt{4000} = 21.40'' > 8'' \qquad \text{OK}$$
$$= .0003(60{,}000)(1.128) = 20.30'' < 21.40'' < 2.67' \text{OK}$$

Calculate the total depth of footing:

h = 2.67 + 1.128/(2)(12) + 3/12 = 2.96' Use 3'

Calculate the top steel, which is the minimum steel required for temperature and shrinkage (Bowles,1:213):

$$A_s = .0018[3(12)](12) = .78 \text{ in}^2/\text{ft}$$

Use #7 bars @ 9" o.c. both ways

Figure 14 shows placement of reinforcing steel in the footing.

Check the tower with footing against overturning:

Without water: R.M.=2/3(1084.4)24.5)=17,712'k
>7,499'k OK

With water: R.M.=2/3(2848.7)(24.5)=46,529'k
>17,303'k OK

Therefore, the footing is sufficiently large to aid in the resisting of overturning moments caused by wind and earthquake such that the resisting moments are within

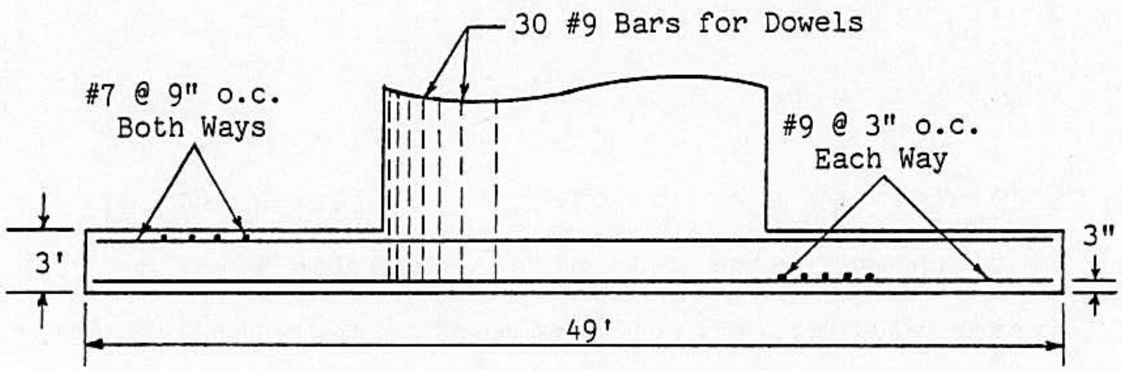

Figure 14. Reinforcement Detail in Footing

the standards set forth in the UBC (4:126).

Chapter 3

CONCLUSIONS

The result of this project is a working design fitting the specifications for two water towers to be built in Louisiana. This is only one possible design using the balloon formwork and reinforced concrete which would meet the design specifications.

For example, the tank might have been designed as a total sphere, as mentioned in the text. Or, for a more aesthetic look, the tower could be designed, instead of straight, as a hyperboloid of revolution about the vertical axis of the tower, concave out. A balloon could be sewn and used as the formwork. This, however, would prove to be very expensive to build.

The versatility of the balloon formwork is proven in this design example. Easily verifiable results were obtained for the design, and the reduced cost of formwork, materials and labor will render the design a feasible competitor in the bidding for construction.

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