A PUBLIC WATER SUPPLY SYSTEM FOR FOUNTAIN CITY, TENNESSEE

By

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Chapter I.

Preface.

The relatively small town of Fountain City has long been in need of a public water supply system.

During the past three years, no less than six disastrous fires have made five families homeless and destroyed a large three-story sanatorium, because of lack of fire protection.

It seems that everybody that I asked for opinions or data was more than glad to help me and I wish to take this occasion to thank Professor H. B. Aikin, Professor J. A. Switzer, Professor Augustus Sisk, Mr. B. F. Johnson, Mr. Stewart Fonde, Mr. Summers Hinshaw, Mr. H. F. Davis, Mr. E. T. Manning, and Mr. Berney Slagle for their help and advice.

Herbert I. Scheitlin.

Chapter II.

Purpose of this Paper.

It is the purpose of this paper to plan and design a water supply system that is capable of economically and efficiently serving the people who live in Fountain City with adequate water for consumption and for fire protection purposes.

Chapter III.

Consideration of the present and the probable future demand for service.

At the present time, May 1926, there are some 727 buildings in the immediate vicinity of Fountain City. Of these 727 buildings, 88 of them have been built during the past five years. This shows an average increase of 17.6 buildings per year. If Fountain City continues to grow during the next 15 years as it has in the last five years, there will be 727 + 88 x 3 = 991 buildings in the section of Fountain City shown by Plate I, at the end of 15 years from the present date.

If it is assumed that there are on the average of 4.5 persons per building, then at the present date there are 727 x 4.5 = 3,270 persons living in Fountain City. If it is assumed that the rate of increase of the number of buildings is the same during the next 15 years as it has been during the past five years, then there will be 991 buildings and with 4.5 persons per building the population will be 991 x 4.5 = 4451 persons.

But, with the large number of vacant building lots and the ideal location of Fountain City with respect to school, church and pure fresh air facilities, there is every reason to believe that Fountain City will grow much more rapidly per year during the next 15 years than it has per year, during the past five years.

Fountain City is primarily a residential section and it will remain so, because nobody wants to start a manufacturing plant a mile and a half from the nearest railroad.

At present the only industrial plants in Fountain City are a couple of lumber mills.

Available Water Supply.

First Creek starts from the Fountain City spring in Fountain City Park. Just outside of the park another spring runs into a lake and the lake discharges its overflow into the same branch with the park spring. These two streams flow along together and just above Maple Avenue three more branches run into the first streams. By this time the branch is a young creek which is discharging between 800,000 and 900,000 gallons of water per 24 hours.

At Greenway, which is about three-fourths of a mile below Maple Avenue on the same creek, there is a discharge of more than twice the discharge at Maple Avenue.

Beaver Creek, which is about two and one-half miles north of Fountain City, is discharging just about the same quantity of water that

First Creek is discharging at Greenway. The physical difficulties that must be considered in the Beaver Creek supply are two ridges, Black Oak and Beaver, which are each about 350 or 400 feet high. It would be entirely feasible to pump the water over the two above mentioned ridges if it becomes necessary to do so.

There is a creek between Black Oak Ridge and Beaver Ridge, running north into Beaver Creek near the Jacksboro Pike. It would be possible to change the direction of this creek and let it run between Black Oak and Beaver Ridges until it is just north of Fountain City and then pump it over Black Oak Ridge to Fountain City. It would probably be advisable under such circumstances to locate the filter plant in the valley between Black Oak and Beaver Ridges and pump the filtered water into a reservoir on Black Oak Ridge for distribution in Fountain City.

In the topographical map shown on plate II a curved line was drawn which enclosed practically all of the useful drainage area in the Fountain City section. The closed curve was planimetered and found to contain 0.7 square inch. Since one square inch on the topographical map represents 4 ± square miles, then the useful drainage area in the Fountain City section is 4 x 0.7 = 2.8 square miles.

The last and probably the surest source of water supply is from the Tennessee River thru the Knoxville Water Department. In this case it would only be necessary to lay about one mile of cast iron pipe from the Knoxville main, at a point about 60 feet north of the Southern Railway crossing on North Broadway, to the Fountain City main which is about 250 feet north of Greenway.

Another source of water supply which is being considered after the last one, is from wells. I am not familiar with the geological construction of the rock formation which is from 75 to 150 feet below the ground surface at Fountain City. If the geological conditions are such that there are sufficient beds of sand and gravel, or other previous formations to yield water enough to keep the wells filled, then it may be that groups of wells drilled in various sections of Fountain City would supply enough water for all purposes.

Construction of the weir with which the discharge of First Creek at Fountain City was measured.

The weir was of the rectangular type. It was constructed of tongued and grooved four inch wide, nine-sixteenths thick boards which were nailed on a frame that was constructed of one inch thick boards. The opening of the weir thru which the water flowed was nine feet wide and three feet deep. A strip of straight sheet metal was filed until it had a wire edge, then the strip was screwed against the downstream face of the weir. The ends of the weir frame were set one and one-half feet into the bank on each side of the stream, the bottom edge of the wier frame was imbedded eight inches below the natural bed of the stream. In installing the wier I was very careful to see to it that the wire edge of the metal strip was level. This was done by means of a spirit level.

A sharp ended board about three inches wide and about three feet long was driven into the bed of the stream about three and one-half feet up stream from the wire edge of the metal strip. This board was driven in until the spirit level showed it to be level with the wire edge of the wier.

Another board was bolted onto the board that was driven into the stream bed, a yard stick was screwed to the latter board so that its lower end was on the upper end of the board that was driven into the stream.

I realize that this method of measurement is not nearly so accurate as the hook-gage method. But due to the lack of a good hook-gage and to the publicity of the wier location, I used the yard stick method. There was probably some error in the readings due to the capulary attraction between the water and the yard stick, which caused the water to climb slightly up the yard stick. The yard stick was well shellaced. A small allowance was made for each reading, because of the capulary attraction.

Readings and Calculations for determining the rate of discharge of First Creek in Fountain City at Maple Avenue. (Width of wier = 9')

Date of reading	H; inches	H 2/3	Q; cu.ft./sec.	Gallons per 24 hrs.
Sept. 12, 1925.	1.25	1.39	1.006	650,146.6
" 14, "	1.5	1.84	1.321	853,722.3
" 15, "	1.625	2.07	1.492	964,237.8
" 16, "	1.625	2.07	1.492	964,237.8
" 18, "	1.5	1.84	1.321	853,722.3
" 19, "	1.5	1.84	1.321	853,722.3
" 21, "	1.5	1.84	1.321	853,722.3
" 22, "	7.0	18.55	13.31	8,600,880.6
" 23, "	1.5	1.84	1.321	853,722.3
" 24, "	1.625	2.07	1.492	964,237.8
" 28, "	1.5	1.84	1.321	853,722.3
" 29, "	1.5	1.84	1.321	853,722.3
Oct. 1, "	1.5	1.84	1.321	853,722.3
" 2, "	1.875	2561	1.850	1,195,603.2
" 3, "	1.625	2.07	1.492	964,237.8
" 4, "	1.625	2.07	1.492	964,237.8
" 5, "	1.625	2.07	1.492	964,237.8
" 6, "	1.625	2.07	1.492	964,237.8
" 9, "	1.625	2.07	1.492	964,237.8

Pumping System.

Whenever motion is transmitted to a body, an expenditure of energy takes place. A portion of this energy is expended in producing velocity head, part of it is used in overcoming friction, and part of it is used to do useful work, in lifting the water. If the pump lifts the water to a height Z, it has to overcome the resistance of the suction pipe as well as the discharging pipe. The friction head thus produced is equal to some added lift so that the effect is just the same as if the pump were lifting the water to a height of $Z + H'$, in which H' is the total head lost due to frictional resistances. Therefore the power delivered to the column of water by the pump is $W(Z + H')$, in which W is the weight of one cubic weight of water. For ordinary fresh water W may be taken as 62.4. The weight of one cubic foot of ocean water is approximately 64. The power required to drive the pump itself is greater than $W(Z + H')$, depending upon the efficiency of the pump which may vary from about 15 % to 40 % for an air lift pump with an air compressor, about 60 % to 85 % for reciprocating pumps, about from 50 % to 80 % for centrifugal pumps, and about from 50 % to 80 % for rotary pumps.

The actual head against which the pump has to work is h =

$h = Z + \frac{V^2}{2g} + H'$, in which Z is the height which the water is lifted, $\frac{V^2}{2g}$ is the velocity head given to the water, and H' is the total head lost due to friction. The power which must be delivered to the water by the pump is $W\left(Z + \frac{V^2}{2g} + H'\right)$ foot pounds. The horse power required is equal to $\frac{Wq\left(Z + \frac{V^2}{2g} + H'\right)}{550}$ foot pounds per second, in which q is the quantity of water delivered by the pump in cubic feet per second.

$$\text{Or H.P.} = \frac{62.4 \times q \times \left(Z + \frac{V^2}{2g} + H'\right)}{550} = \frac{q \times \left(Z + \frac{V^2}{2g} + H'\right)}{8.82}$$

According to Mr. Daugherty,* centrifugal pumps should nut pump against more than 100 to 200 feet per stage.

In the Fountain City pumping station it is proposed that a five foot dam shall be built across First Creek at a distance of thirty feet south of the Maple Avenue bridge. A pump will lift the water from behind the dam for a distance of seven feet up into a coagulation basin.

* Page 302 of Daugherty's "Hydraulics"

As the water enters, a part of it will pass thru a mixing chamber where alum and lime will be added in the correct quantities. From the coagulation basin the water will run, by gravity, to and thru two rapid sound filters. From the filters the water will collect in a sump, from which it will be pumped by a two-stage, electrically driven, direct connected centrifugal pump to the storage reservoir near the top of Black Oak Ridge.

A part of the filtered water will be pumped into an elevated storage tank at the pumping station and will be used only to wash the filter beds with.

Filtration and Filter Design.

A filter consists of a layer of filtering material, generally sand, thru which the water is passed for the purpose of removing bacteria and other suspended matter. The layer or layers, of sand are supported by an underdrain system which is sufficiently large to carry off the water as fast as it comes thru the filter bed. The filters act as strainers and the smallest particals of silt together with the largest bacteria are a great deal smaller than the smallest grain of sand in the filter bed. The removal of the suspended matter including bacteria, is accomplished partly by the absorptive power of the film which surrounds each grain of surface sand. Another thing that helps in removing the suspended matter is the film that forms around the body of the suspended matter. The films on the suspended matter tend to adhere to the films on the grains of sand and the suspended matter is halted on or near the surface of the filter bed.

From practical experience it has been shown that thicker beds are more steady in operation than thinner ones, because of this fact it is now believed that each and every partical of sand plays a more or less important part in removing the suspended matter. It will readily be admitted, however, that the largest part of the suspended matter is always removed in the topmost surface of the sand bed.

Rapid, or mechanical filters may be of either pressure or gravity type. The rapid filters are washed with either raw or filtered water, preferably filtered water. The rapid filters differ from the slow filters in as much as the sand is washed in place by reversing the direction of flow of water thru the filter bed of the rapid sand filter and the rate of filtration is as great as 50 times that employed in slow sand filters.

It is very necessary that the concrete work in rapid filters be absolutely tight in order to prevent the mixture of the filtered and the unfiltered water. As a rule the sand should have an effective size of 0.30 to 0.50 millimeters.

In choosing the type of filters to be used it should be kept in mind that for a water having a turbidity less than 30 parts per million or a color less than 20 parts per million, slow sand filters without coagulation give very good results. For waters having a turbidity of more than 50 parts per million or a color of more than 30 parts per million, the mechanical filters with coagulation will give the best results. The mechanical filter with coagulation produces not only an equally safe water, but it also produces a water of far better appearance than does the slow sand filter without coagulation.

From an analysis of a sample of water taken from First Creek at the Maple Avenue bridge on May 17, 1926, the water showed a turbidity of 25 parts per million and a color of 15 parts per million. This would indicate that a slow sand filter should be used. But due to the fact that the turbidity and color are not always as low as the sample indicated and due to the fact that a great deal of organic material tends to remain in suspension, it is proposed to install a rapid sand filter with coagulation, in the Fountain City plant.

Design of Filter Plant.

The filter beds shall have a capacity of 1,000,000 gallons per 24 hours. There will be two separate filter units so that the plant may continue to operate while one of the filter beds is being washed.

"For the area of beds for mechanical filters, allow a rate of two gallons per square foot per minute plus about 8 % for wash water." *

Number of gallons per minute that must pass thru filter bed =

$$\frac{1,000,000}{24 \times 60} = 694.9 \text{ gallons}$$

Number of gallons excess for wash water = 694.9 x 0.08 = 55.59

Total number of gallons of water that must pass thru the filter bed per minute = 694.9 + 55.59 = 750.49 gallons. But since two gallons per square foot of filter bed per minute is filtered, then 750.49 ÷ 2 = 375.24 square feet of filter bed that will be required to filter 750.49 gallons at the rate of two gallons per minute per square foot of filter bed.

Depth of filtering material: Gravel = 6", sand = 30". Size of filtering material = 0.41 millimeters.

The washing of mechanical filters, which is done by reversing the current of water thru the filter bed, may be most efficiently done by using a wash-water velocity 18" per minute for more vertical velocity, depending upon the size of the grains of sand. Where water is not used by itself, the use of air or of some mechanical agitating device may be used. The wash-water should be applied gradually at first and then

* Page 749 Flinn, Weston and Bogert; "Water Works Handbook."

shut off gradually when the washing has been completed in order to avoid mixing the gravel with the sand. The velocity of the wash water ought to be just enough to hold the particles of sand in suspension and not enough to wash them away.

The very small amount of iron in the sample of water that was taken from First Creek at Fountain City near the Maple Avenue bridge on May 17, 1926 does not warrant aeration.

There will be a small chemical laboratory in the filter plant in order that the standard of purity of the water may be kept up.

Collection of Sample.

The sample was collected under clean and aseptic conditions. In getting the sample the bottles were held by their bottom with their mouth upstream so that the hand did not come in contact with the mouth of the bottle. The mouth of each bottle was held under the surface of the water until it was filled and the bottle was corked by handling the cork with a piece of sterilized gauze.

The time between the collection and the analysis was too long to get a true test for carbonic acid (H_2CO_3). In order to get an accurate test for carbonic acid, the test should be made in the field at the time at which the sample is collected.

"Bacillus coli is one of the normal types of bacteria of the intestinal tracts of man and of higher animals. Its presence is usually the result of human or of animal pollution. To make presumptive test for B. coli, inoculate fermentation tubes containing dextrose broth and lactose bile, respectively, with varying volumes of water, 0.1 to 10 cc., incubate for 24 to 48 hours at 37^0 and observe formations of gas. With the production of 10 % or more of gas, B. coli is presumed to be present. Lactose-litmus-agar plates inoculated with gas forming cultures may be used to isolate B. coli which form acid, red colonies."

* Page 795, Flinn, Weston and Bogert, "Water Works Handbook."

WATER COLLECTED FROM FIRST CREEK AT MAPLE AVENUE ON MAY 17, 1926.

Bacteriological Examination. *

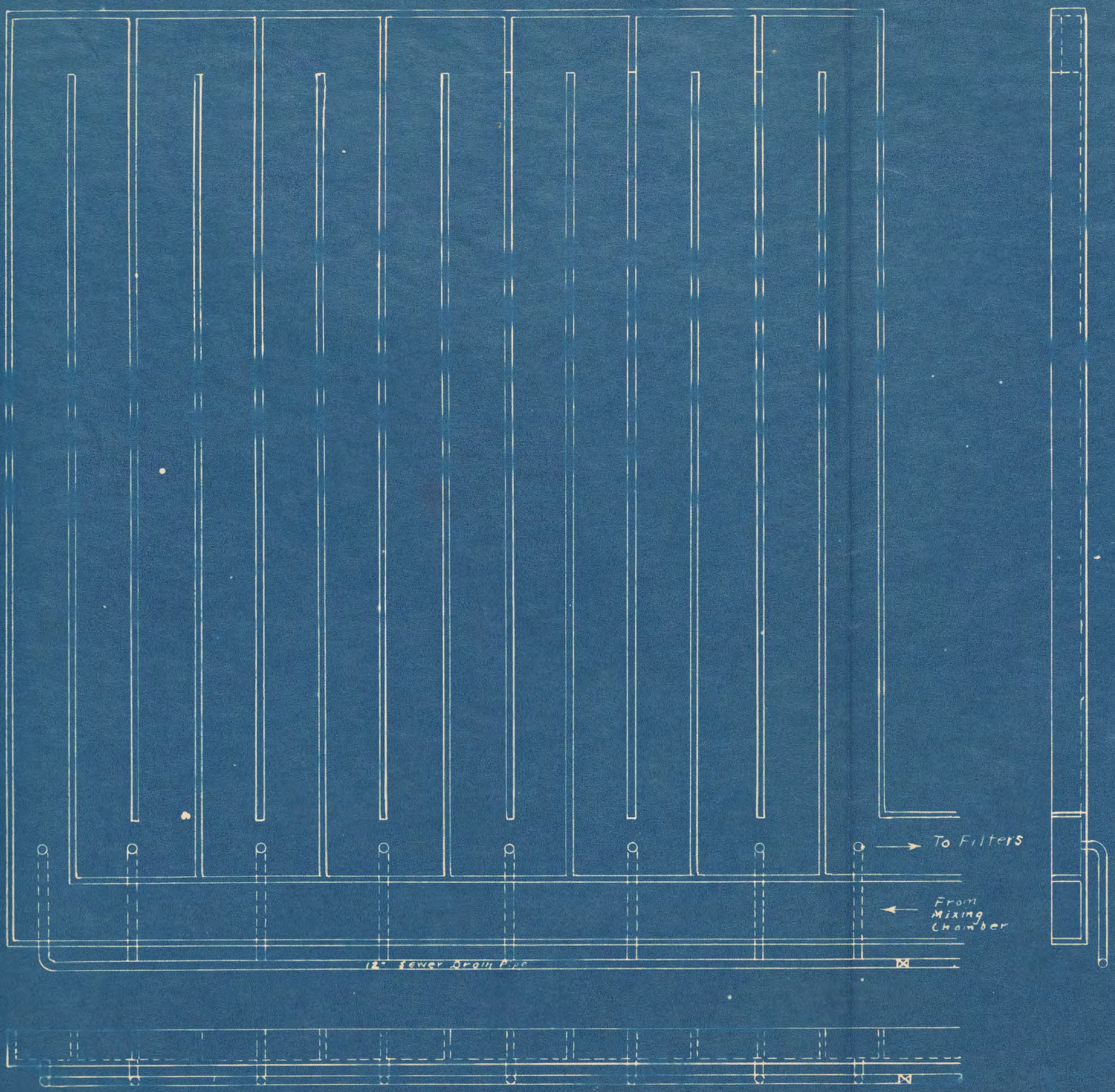
No. of cc	Media	Bacteria per cc at 37° C for 24 hours	Bacteria per cc at 37° C for 48 hours	B-coli Determination Per cent gas 24 hours	48 hours
1	Neutrient Agar	1000			
1	Lactose Broth			0	15
10	Lactose Broth			0	10
10	Lactose Broth			0	10
10	Lactose Broth			0	15
		B-coli			-

(- test shows that no harmful B-coli present)

Mineral Analysis Expressed in Parts per Million. Per gal

Turbidity	25.0	
Color	15.0	
Alkalinity (HCO_2)	14.3	
Hardness (soap) (Based on total hardening agents)	133.0	7.8
Hardness (calculated) (Based only on Ca.and Mg.)	116.0	6.8
Chlorine (combined)	15.0	
Chlorine (free)	0.0	
Sulphate (SO_4)	6.2	
Total Solids (on evaporation)	151.0	8.9
Loss on Ignition	63.0	
Silica ($Al_2O_3 + Fe_2O_3$)	12.8	
R_2O_3 (Al and Fe oxides) (Mostly Al_2O_3)	12.8	
Calcium	33.7	
Magnesium	5.3	
Sodium (Sodium and Potassium)	0.34	

* Analysis by Mr. B.F.Johnson of the Knoxville Water Department



Sedimentation Basin Lay Out

Scale 1" = 20'

FIGURE 1

SEDIMENTATION BASIN LAY OUT

Isometrical Sketch of Filter Plant Layout

Scale 1" = 50'

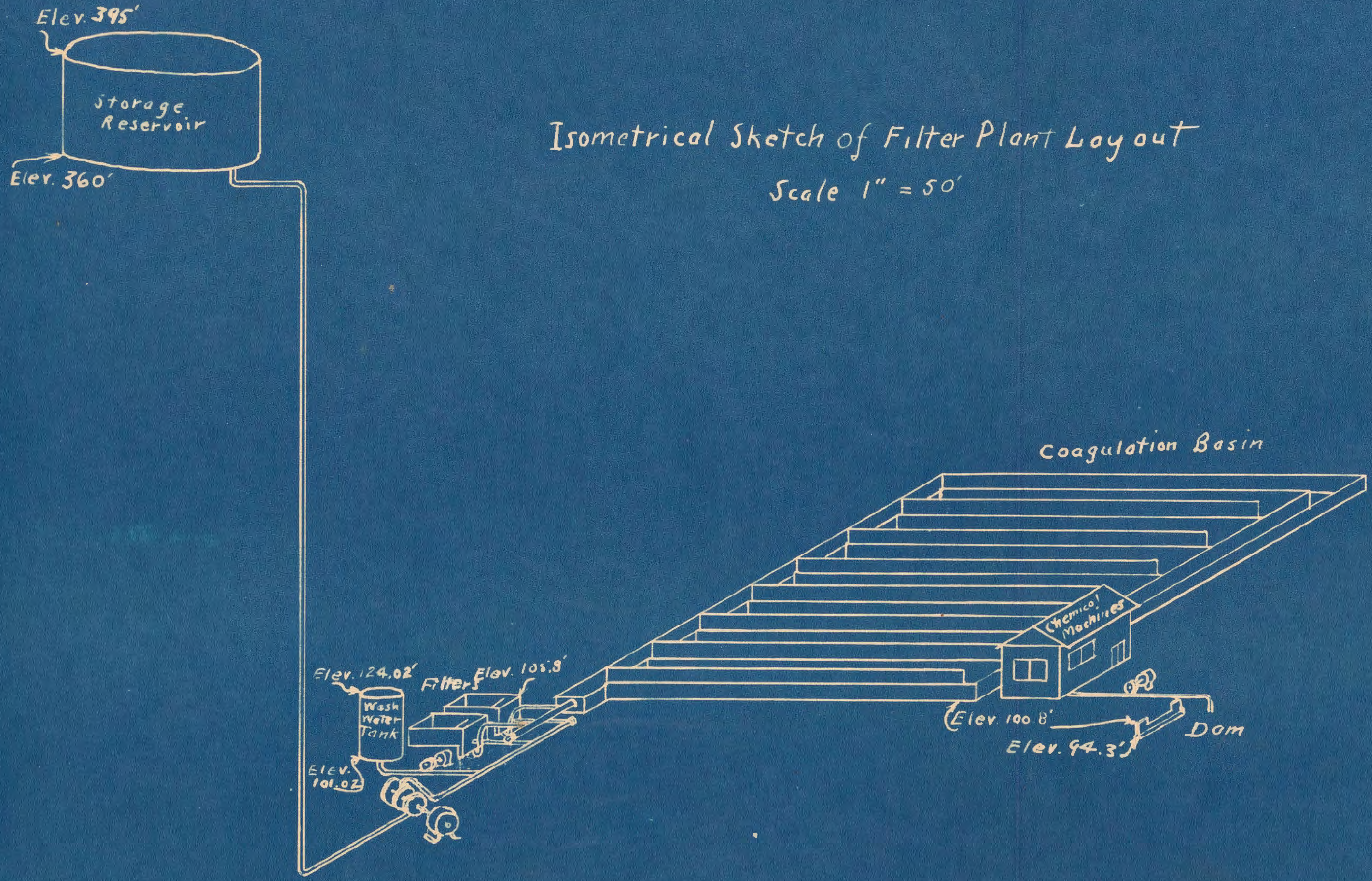
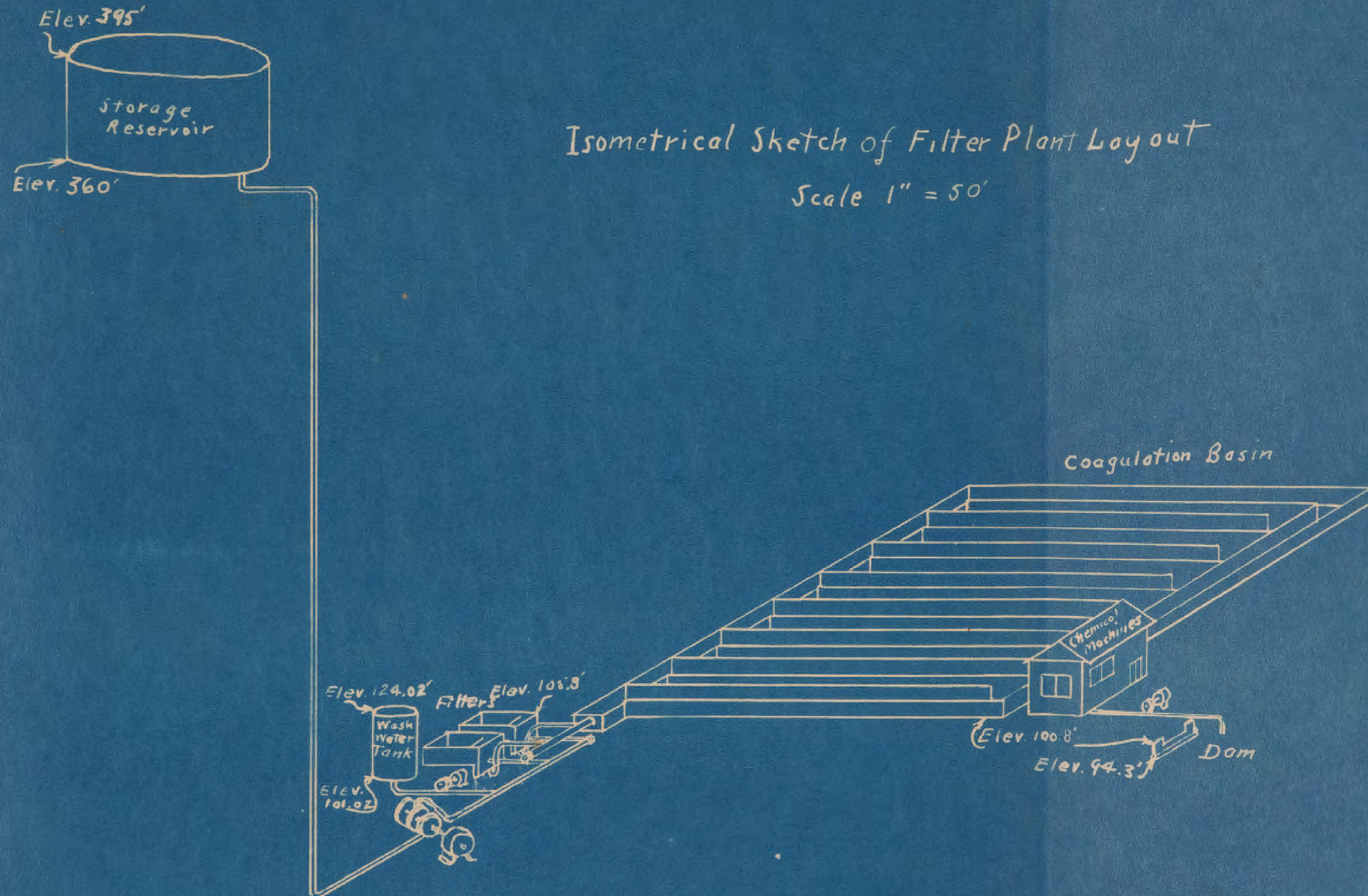


FIGURE 2

ISOMETRICAL SKETCH OF FILTER PLANT LAY OUT

Isometrical Sketch of Filter Plant Layout

Scale 1" = 50'



Calculation for slope and length of sedimentation basin bed.

The number of gallons that must pass thru the filters was found to be 750 gallons per minute. Since the sedimentation basin must furnish water as fast as it is filtered then the cross section of the channel and the slope of the channel must be such that 750 gallons of water will be delivered to the filters per minute.

Assume the channel to be 5 feet deep and 10 feet wide. The cross sectional area is 50 square feet. The hydraulic mean depth $= m = \frac{50}{10+2\times5} = 2.5$. Assume $c = 154$, and $v = \frac{750}{60 \times 50} = 0.25$ because $q = AV$; and $V = \frac{q}{a}$ in which q is the discharge in cubic feet per second, A is the cross-sectional area of the channel in square feet, and V is the velocity in feet per second.

In the formula $V = C\sqrt{ms}$

C is a quantity which is determined by m (the hydraulic radius), by s (the slope of the channel), and by N (which is a function of the nature of the surface of the channel).

Therefore from the formula $V = C\sqrt{ms}$

$$0.25 = 154\sqrt{2.5\times5}$$
$$0.25 = 154\sqrt{2.5}\times\sqrt{s}$$
$$0.25 = 154\times1.58\times\sqrt{s}$$
$$\sqrt{s} = \frac{0.25}{154\times1.58} \quad 0.001029$$
$$s = (0.001029)^2 = 0.00000105$$

or a slope of 1.05 ft. in 1,000,000 feet of length, or a slope of 0.000105 ft. in 100 feet of length. Velocity x time = distance.

Assume that $2\frac{1}{2}$ hours is required for water to pass from one end of the sedimentation channel to the other end of the channel.

Therefore there are 3600x2.5=9000 seconds in 2.5 hours and the total length of the sedimentation channel equals 9000x0.25=2250 feet.

Take a section of the wall 1 foot wide and 5 feet deep. The total pressure on the section is equal to $P = W\bar{Z}h$ in which

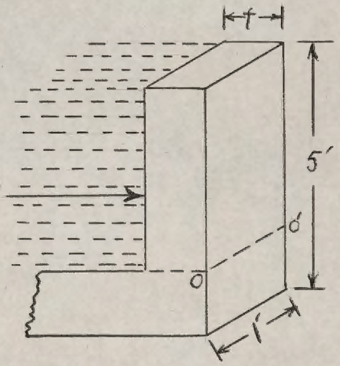
P is the total pressure in lbs.
W is the weight of 1 cubic ft. of water
$\bar{Z}$ is the depth to the center of gravity of the section measured downward from the surface of the water.

FIGURE 3

SECTION OF SEDIMENTATION BASIN.

and h is the total depth of the section.
Then $P = 62.4 \times 2.5 \times 1 \times 5 = 780\#$.
To find the thickness of the wall take moments about the base of the wall.

Depth from the surface to the center of pressure $= y' = \bar{y} + \dfrac{kg^2}{\bar{y}}$



in which $\bar{y}$ is the depth to the center of gravity.

$$kg^2 = \frac{Ig}{A} = \frac{\text{moment of inertia}}{\text{area of section}}$$

$$kg^2 = \frac{\frac{bd^3}{12}}{A} = \frac{1 \times (5)^3}{12 \times 1 \times 5} = \frac{125}{60} = 2.081$$

$$\text{Then } y' = 2.5 + \frac{2081}{2.5} = 2.5 + 0.834 = 3.334' \text{ from surface}$$

$5 - 3.334 = 1.666'$ from bottom.

Taking moments about the base line OO' we get
$1.666 \times 780 = 1300$ ft. lbs.

Assume that the allowable tension in concrete is 150# per square inch.

Total area of section 12 inches wide and t inches thick equals to $\dfrac{1300 \times 12}{150} = 104$ square inches

Thickness of wall $= t = \dfrac{104}{12} = 8.67$ or better 9 inches.

Volume of concrete in sedimentation basin

$153 \times 5 \times 0.75 =$	576 cu. ft.
$142.5 \times 5 \times 0.75 =$	535 cu. ft.
$155 \times 5 \times 0.75 =$	581 cu. ft.
$160 \times 5 \times 0.75 =$	600 cu. ft.
$11 \times 140 \times 5 \times 0.75 =$	5782 cu. ft.
$3(140 \times 5 \times 0.75 - 10 \times 2 \times 0.75) =$	1530 cu. ft.
$162.5 \times 153 \times 0.5 =$	12431.25 cu. ft.
	22035.25 cu. ft.

$$\frac{22035}{27} = 815 \text{ cu. yds.}$$

Use the following mixture:-
1 part cement
$2\frac{1}{2}$ parts sand
5 parts stone (less than 1-3/4" in diameter)

Then there will be required:-

2,592.3 cu. ft. of cement	.2592 bags @ 9.70=	$1,814.
6,480.75 cu. ft. of sand	@ $1.50 per yd.=	360.
12,961.5 cu. ft. of stone	@ 1.50 per yd.=	720.
		$2,894.

Estimated cost of labor to build sedimentation basin -------------------$1,500.
Estimated cost of lumber for forms ------------- 500.
Total cost ---------------4,894.

Calculation for reservoir for wash water.
Velocity of water upward thru filter bed = 18" per minute =

$$\frac{18}{12x60} = 0.025 \text{ ft. per second.}$$

The quantity of water passing upward thru the filter bed= $q = av = 375.24x0.025 = 9.38$ cu. ft. per second.

Assume the duration of the wash is 6 minutes. Then the total amount of water required for washing = 9.38 x 6 x 60 = 3380 cubic feet.

If the wash should have to be continued for 8 minutes then 9.38 x 8 x 60 = 4500 cubic feet will be required.

Therefore the wash water reservoir or storage tank should have a capacity of 5000 cubic feet, or 5000x7.48 = 37400 gal loms.

Assume the diameter of the tank to be 16 feet.

Area of base of tank $= \pi r^2 = \pi (8)^2 = 201.1$ square feet.

Area of base x height of tank = volume of tank
201.1 x height of tank = 4500 cubic feet
height of tank $= \frac{4500}{201.1} = 22.4$ or 23 ft.

Use a wood stowe tank with steel hoops.

Calculations for vertical distance from the bottom of the filter bed to the bottom of the storage tank.

Assume diameter of pipe which connects bottom of filter bed to base of storage tank as 12 inches and assume the length of the pipe as 100 feet.

Height of base of tank $= H' = ke\frac{v^2}{2g} + f\frac{l}{d}\frac{v^2}{2g} + kd\frac{v^2}{2g}$

Assume ke = 0.5
and kd = 0.9

Area of a 12" pipe $= \frac{\pi(6)^2}{144} = 0.786$ square feet.

$$v = \frac{q}{A} = \frac{4500}{8x60x0.786} = 11.91 \text{ feet per second.}$$

$$H' = (1.4 + 2.16)\ \frac{(11.91)^2}{2g} = 3.56x2.21 = 7.87 \text{ or } 8 \text{ feet.}$$

Then the height of the base of the tank above the bottom of the filter bed is 8 feet.

It is very probable that a pipe of larger than 12" in diameter would deliver the required quantity of water with a lower head, thus saving the work that would otherwise be used in raising the water up to the tank.

With this thought in mind assume a pipe which is 18" in diameter.

$$v = \frac{q}{a} \frac{4500}{8x60x\pi(9)^2} = 5.3 \text{ feet per second.}$$

$$f = 0.002 + \frac{0.02}{18} = 0.02111$$

$$H' = (1.4 + 0.02111x\frac{100x12}{18}\ \frac{(5.3)^2}{2g}$$

$$= (1.4 + 1.405)0.437$$

$$= 2.805 \times 0.437 = 1.225 \text{ feet}$$

The difference is $7.87 - 1.22 = 6.65$ feet lower by using an an 18" pipe instead of a 12" pipe.

The saving in work each time the tank was filled would be $6.65 \times 62.4 \times 4500 = 1,863,500$ foot-pounds.

Calculations for pump for lifting raw water from the creek to the coagulating basin.

Height thru which the water must be lifted is 17.39 feet.

Capacity of pump must be 1,000,000 gallons per 24 hours. Allowing for a 25% overload, the capacity of the pump must be 1,250,000 gallons per 24 hours.

$$\frac{1,250,000}{60x60x24} = 14.47 \text{ gallons per second.}$$

$$\frac{14.47}{7.48} = 1.935 \text{ cubic feet per second.}$$

Horse power of electric motor drive for raw water pump $= \frac{Wxqxh}{550xeff.}$

in which W is the weight of one cubic foot of water,
q is the quantity of water delivered in cu. ft. per second,
h is the total head (actual head + friction loss.)
550 ft. pounds of work per second is equivalent to one HP.
Eff. is the combined efficienty of pump and motor.

Assume $l = 60$ feet, Eff. = 65%

$$q = AV; \; V = \frac{q}{a} = \frac{1.935}{0.349} = 5.55 \text{ feet per second.}$$

H' = head lost due to friction =

$$\left(ke + f\frac{l}{d} + kd\right)\frac{V^2}{2g}$$

For value of ke and kd see Appendix II.

$$H' = \left(0.5 + \left(0.02 + \frac{0.02}{8}\right)\frac{60 \times 12}{8} + 0.93\right)\frac{(5.55)^2}{2g}$$

$$= (1.43 + 0.225 \times 90)\ 0.479$$

$$= (1.43 + 20.22)\ 0.479 = 21.65 \times 0.479 = 10.39 \text{ feet.}$$

$$h = 7 + 10.39 = 17.39$$

$$HP = \frac{1.935 \times 17.39}{8.82 \times 0.65} = 5.86 \text{ or 6 horse power.}$$

Calculations for pump and motor to lift filtered water to wash water tank.

The water is to be lifted from 3 feet below the bottom of the filter bed. Assume $d = 8"$, $V = 1.5$, eff. = 65%, and $l = 55$

$$f = 0.02 + \frac{0.02}{8} = 0.0225$$

From Appendix II ke = 0.5 and kd = 1

$$H' = \left(1.5 + 0.0225 \times \frac{55 \times 12}{8}\right)\frac{(1.5)^2}{2g}$$

$$= (1.5 + 1.856)0.035 = 3.356 \times 0.035 = 0.1172 \text{ feet}$$

$$h = 1.22 + 23 + 0.117 = 24.33$$

The capacity of the pump should be 338.400 gallons per 24 hours.

Horse power of electric motor drive $= \frac{q \times h}{8.82 \times eff.}$

$$= \frac{(0.349 \times 1.5) \times 24.33}{8.82 \times 0.65} = 2.22 \text{ or 3 horse power.}$$

Calculation for pump to deliver water from filter plant on Maple Avenue to the storage reservoir near the top of Blackoak Ridge.

Use a 10" cement-lined cast iron pipe. Length of pipe required = 6150 feet.

$$q = AV$$

$$V = \frac{q}{a}$$

Area of a pipe whose diameter is 8" = 0.349 sq. feet.

$$V = \frac{1.935}{0.349} = 5.55 \text{ feet per second.}$$

Assume ke = 0.5 and kd = 0.93 (See Appendix II).

$$H' = (0.5 + f\frac{l}{d} + 0.93)\frac{V^2}{2g} \; ; \quad f = 0.02 + \frac{0.02}{10} = 0.022$$

$$= (1.43 + 0.022 \times \frac{6150 \times 12}{10}) \frac{(5.549)^2}{2g}$$

$$= (1.43 + 162.2)\ 0.4765$$

$$= 163.63 \times 0.4765 = 78.0 \text{ feet lost due to friction.}$$

Total height of top of reservoir on Blackoak Ridge about 3 feet below the bottom of the filter bed = 298.2'

Total head which the pump must do work against =
h = 298.2 + 78 = 376.2 feet.

Assume that the combined efficienty of the electric motor and the direct connected centrifugal pump is 65%.

$$HP = \frac{q \times h}{8.82 \times eff.} = \frac{1.935 \times 376.2}{8.82 \times 0.65} = 127$$

A Venturi type control will control the rate of flow of the water from the coagulating basin to the filter bed.

Calculations for filters.

Total area of filter beds = 375.24 square feet. Since the filter bed is to be in two sections, then $\frac{375.24}{2} = 187.62$ square feet, area of one filter bed.

If the lemgth of the filter bed is 20 feet, then the width $= \frac{187.62}{20} = 9.38$ feet.

At a filtration rate of 2 gallons per square foot per minute, 187.62 x 2 = 3.75.24 gallons per minute per filter bed.
375.24 x 2.28 x 10^{-3}* = 0.8555 cu. ft. per sec.

* From page 122 of Hudson & Lypka's "Manual of Mathematics" multiply gallons per minute by 2.28 x 10^{-3} to obtain cu. ft. per second.

If a section of the filter bed one foot square is taken, then two gallons of water will require one minute to pass thru the 3.5 feet thickness of the filter bed.

$$\text{Velocity} = \frac{q}{a} = \frac{0.8555}{1} \text{ ft. per second}$$

Therefore the pipe, which drains the filtered water from beneath the filter bed must have a cross sectional area of one sq. ft.

$$A = \pi r^2;\ r = \sqrt{\frac{A}{\pi}} = \sqrt{\frac{1}{\pi}} = \sqrt{0.318} = 0.565 \text{ ft.} \times 12 = 6.8''$$

The inside diameter of pipe to carry away the filtered water to the pool $= 6.8 \times 2 = 13.6''$ or better 14" wrought iron.

The area of one of the 6 branches $= \frac{1}{6} = 0.167$ sq. ft.

$$A = \pi r^2\ ;\ r = \sqrt{\frac{A}{\pi}} = \sqrt{\frac{0.167}{\pi}} = \sqrt{0.0531} = 0.23 \text{ feet.}$$

The inside diameter of a single branch pipe $= 0.23 \times 2 \times 12 = 5.52''$ or better 6".

Since the filter gravel is to be $\frac{1}{4}''$ in diameter, then the holes in the branches must be about 3/16" in diameter.

$$\text{Area of a } 3/16'' \text{ hole} = \left(\frac{3}{32 \times 12}\right)^2 \pi = \left(\frac{1}{128}\right)^2 \pi = (0.0078)^2 \pi = 0.000191 \text{ Sq. Ft.}$$

$$\text{Total number of } 3/16 \text{ diameter holes per branch} = \frac{0.167}{0.000191} = 875.$$

$Q_1 = A_1 V_1$, for quantity of water transversing the sedimentation or coagulation basin.

$Q_2 = A_2 V_2$, for quantity of passing thru the raw water pipe to the filter bed.

Since Q_1 must necessarily be equal to Q_2 then $A_1 V_1 = A_2 V_2$.

But $A_1 = 50$ sq. ft.; $V_1 = 0.25$ ft. per sec. $A_2 =$ unknown;

V_2 = assumed to be 4 ft. sec. $A_2 = \frac{A_1 V_1}{V_2} = \frac{50 \times 0.25}{4} = \frac{12.5}{4} = 3.12$ sq. feet

$$A = \pi r^2;\ r = \sqrt{\frac{A}{\pi}} = \sqrt{\frac{3.12}{\pi}} = \sqrt{0.995} = 0.996 \text{ ft.}$$

Inside diameter of raw water feed pipe $= 0.996 \times 2 = 1.992'$ or better 2', or 24".

Size of pipe from raw water feed pipe to filter.

Without making calculations, it can readily be seen that the size of this pipe must be approximately the same as the 14" pipe which removes the filtered water from beneath the filter bed. Therefore use a 14" (inside diameter) pipe the raw water feed pipe and the filter.

From the discussion of the reading under the head of "Design of Filter Plant" it will probably be remembered that the vertical wash water velocity was 1.5 feet per minute. There are two gutters, each is assumed to carry one-half of the wash water or overflow.

Area of one filter bed = 187.62 square feet,

$$Q_1 = 187.62 \times \frac{1.5}{60} = \frac{281}{60} = 4.7 \text{ cu. ft. per sec.}$$

Let Q_1 be the quantity of water passing upward thru the filter bed.

Let Q_2 be the quantity of water being discharged thru the gutter.

But since Q_2 must be equal to Q_1, when the filter bed is being washed, and $Q_2 = A_2V_2$ and $Q_1 = A_1V_1$, then $A_2V_2 = \frac{A_1V_1}{2}$

$$A_1 = 187.62 \text{ square feet,}$$

$$V_1 = 0.028 \text{ feet per second,}$$

Assume $V_2 = 2$ feet per second

$$\text{then } A_2 = \frac{A_1V_1}{2V_2} = \frac{187.62 \times 0.025}{2 \times 2} = 1.17 \text{ square feet}$$

$$A \ \frac{\pi r^2}{2}; \ r \ \sqrt{\frac{2A}{\pi}} = \sqrt{\frac{2 \times 1.17}{\pi}} = 0.745 = 0.864 \text{ feet.}$$

Diameter of the semi-circular gutter = 2 x 0.864 = 1.728 ft.

Determination of slope of gutter:-

$$V_2 = C\sqrt{ms}$$

$$V_2 = 2; \ m = \frac{r}{2} = \frac{0.864}{2} = 0.432$$

Assume $c = 117$

$$\text{then } 2 = 117\sqrt{0.432 \times s}$$

$$(2)^2 = (117)^2 \times 0.432 \times s$$

$$s = \frac{4}{(117)^2 \times 0.432} = \frac{4}{5900} = 0.00068$$

Determination of size of sewer pipe:-

The area of the sewer pipe should be large enough so that the pipe will carry the full discharge of the two gutters.

Discharge of the 2 gutters $= Q = \pi(0.864)^2 \times 2 = 4.68$ cu. ft. per second.

Therefore the cross sectional area of the sewerage pipe should be $A = \pi r^2 = \pi(0.864)^2 = 1.17$ sq. ft. The capable discharge of the sewerage pipe will be equal to $1.17 \times 2 \times 2 = 234 \times 2 = 468$ cu. ft. per second.

Since it is not possible to buy a pipe whose diameter is 1.728, it would be best to use a 2 foot pipe.

For sample of filter sand, see Plate III.

From the sample, 1 cu. inch of sand weighs 0.0625#.

1 cu. ft. = 0.0625# x 1728 = 108#

1 cu. yd. = 108# x 27 = 2916#

1 cu. yd. = $\frac{2916}{2000}$ = 1.458 tons

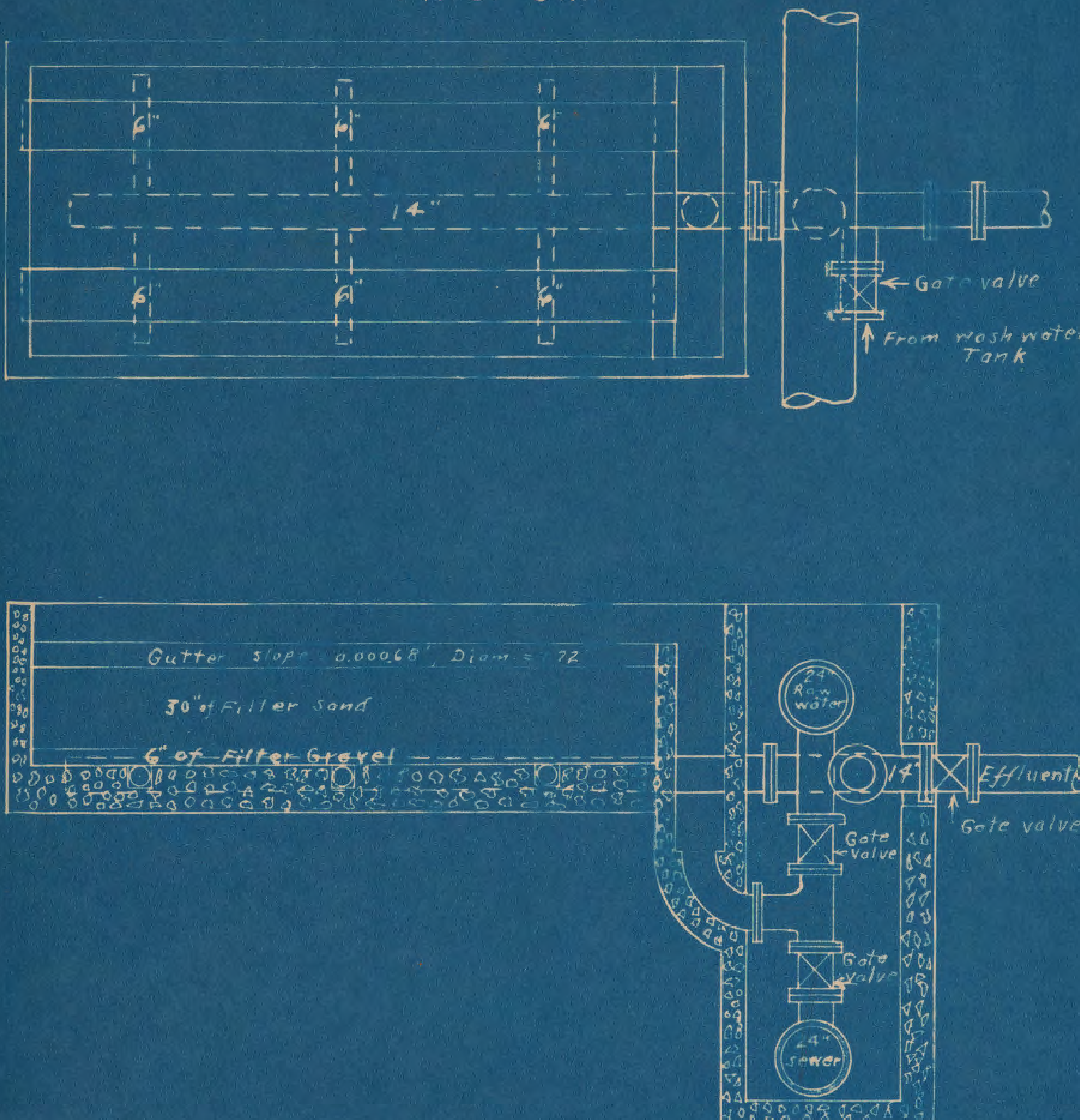
Cost of 1 cu. yd. @ \$3.50 per ton=

1.458 x \$3.50 = \$5.10

FIGURE 4

FILTER UNIT

Filter Unit



Scale 1" = 5'

With the gate valve to the sewer closed, the raw water passes in thru the raw water pipe and up into the top of the filter bed. It filters thru the bed of filter sand and is drawn off thru the effluent pipe. During this process the gate valve to the wash water tank is also closed.

When the layer of aluminum hydroxide and mud becomes so thick on the surface of the filter bed that the rate of filtration is appreciably reduced, then the filter bed must be washed.

Proceed as follows: Close effluent valve, close raw water valve, open sewer valve, and open valve to wash water tank. The filtered water from the wash water tank is thus passed upward thru the bed of filter sand, loosening and carrying the aluminum hydroxide and mud into the gutters and thus to the sewer.

When the water that is being discharged thru the gutters ceases to be laden with mud, close the gate valve to the wash water tank, close the gate valve to sewer, open effluent gate valve, and open the raw water gate valve.

Cost of a Filter Unit.

2 ends:	9' x 5' x 3/4'=	33.7 cu. ft.
2 sides:	23.5' x 5' x 3/4'=	88.0 cu. ft.
1 end:	3.5'x 5' x 3/4'=	13.1 cu. ft.
1 end:	15' x 9' x 1' =	135.0 cu. ft.
1 end:	10' x 3/4' x 9'=	67.5 cu. ft.
1 bottom:	22' x 10.75' x 1.5'=	355.0 cu. ft.
1 bottom:	5' x 9' x 3/4'=	33.7 cu. ft.
		726.0 cu. ft.

$$\frac{726}{27} = 27 \text{ cu. yds.}$$

Use a 1:2:4 mixture

103.7 cu. ft. of cement @ \$0.75 per bag =	\$78.
207.4 cu. ft. or 7.7 cu. yds. of sand @ \$1.50 per cu. yd.=	12.
414.8 cu. ft. or 15.3 cu. yds. of rock @ \$1.50 per cu. yd.=	23.
50' of 14" (inside diameter) W I pipe @ \$2.42 per ft.=	121.
24' of 6" W I pipe @ \$0.87 per foot.=	21.
40' of 24" (inside diameter) WI pipe @ \$3.08 per ft.=	123.
3 14" gate valves @ \$59.34 each =	175.
1 standard 90° 14" curve @ \$15.10 each=	15.
1 standard 14" tee branch @ \$19. each =	19.
42' of 1.72' diameter semi-circular gutter @ \$1.16 per ft=	49.
Total cost of one filter unit not allowing for labor --------------	\$636.

If \$1500.00 is assumed to be the cost of labor and transportation, then

1500 + 636 = \$2136.00, total cost of one filter unit.

Cost of Filter Plant.

Building (structure) (estimated)	\$2000.
Dam (assume 17 cu. yds. of concrete @ \$20. per cu yd)	340.
Sedimentation basin	4894.
Automatic alum adding machine	200.
Automatic lime adding machine	200.
Wood-stove wash water tank	600.
Two 1,250,000 gallon centrifugal pumps for pumping raw water (use the Union Steam Pump Company's 5" Class "A" pump, which must be operated at 1150 RPM. The efficiency under these conditions will be 55%)	498.
One 5 HP AC motor with speed of 1150 RPM	108.
One 4 HP gasoline engine	112.
Two 338,400 gallon centrifugal pumps (use the Union Steam Pump Company's 2" Class "A" pump, operated at 1150 RPM. The efficienty under these conditions will be 50%)	442.
One 5 HP AC motor with speed of 1150 RPM	108.
One 5 HP gasoline engine	125.
Two 1,250,000 gallon centrifugal pumps (use the Union	

Steam Company's 6" Class "U" two-stage pump to be operated at 1725 RPM. The efficiency will be 71%).	1646.
One 127 HP AC motor (estimated)	2100.
One 127 HP gasoline engine (estimated)	2200.
Chemical laboratory(estimated)	800.
Grading 55.5 cu. yds @ \$1.65 per cu. yd.	92.
Sand for filter bed 41.6 cu. yds. @ \$5.10 per cu. yd.	212.
Gravel for filter bed 7.0 cu. yds. @ \$1.60 per cu. yd.	11.
2 filter units @ \$2136. each	4272.
320 feet of 12" sewer pipe for sedimentation basin	579.
Total	\$21 539.

DISTRIBUTION SYSTEM

With respect to the distribution system, the uses of water may be divided into two separate and distinct classes: (1) the use for fire protection; and (2) for public and domestic purposes. In the latter the rate of consumption is relatively uniform over the entire system except in such a case that a manufacturing concern which uses a great deal of water be located in the community. There is always a peak in the rate of consumption about five or six o'clock in the morning when people start to get up to wash and to cook breakfast. There is usually another peak in the rate of consumption about ten to twelve in the morning and another from about five to six thirty at night. Near midnight the rate of consumption is practically zero, unless there is a leak some place in the distribution system or in a customer's house. If the leak is in the distribution, the water that leaks out is a total loss to the city. If the leak is in a customer's house and the water doesn't slip thru the service meter with causing it to register the city gets paid for the water and does not suffer any loss doe to its leakage.

To supply water for domestic purposes requires the wide distribution of moderate quantities. While fire protection demands that a large volume shall be available and concentrated in a given district. The distribution pipe must be large enough to furnish a stream of about 500 gallons per minute per fire hydrant with a pressure which may vary from 50 pounds per square inch to 125 pounds per square inch, depending upon the height of the buildings and the number of fire streams that must be concentrated at a given locality. For low buildings the former pressure may be used and for very tall structures the latter pressure may be used. In general, pressures greater than 130 pounds per square inch have been found to give a great deal of extra trouble in breakage of mains.

The National Board of Fire Underwriters say that the number of gallons which must be available per minute=

$$1020\sqrt{P}\,(1-0.01\sqrt{P})$$

in which P is the number of thousand of population.

$$G = \text{gallons per minute} = 1020\sqrt{3.27}(1-0.01\ 3.27)$$

$$= 1020 \times 1.81(1-0.01 \times 1.81) = 1849 \times 0.9819 = 1810.$$

In designing a new water works there is a sharp distinction drawn in many problems between the period of serviceability of some portions of a plant and the period during which it will be adequate for the demands upon it, without extension or reinforcement. The period of adequacy is determined by the rate of increase of consumption and of the general character of the whole water works plant. The words taken as a whole are estimated to have an average life of adequacy of twenty to thirty years by most of the officials furnishing information on the subject, the highest estimate being fifty years. In laying mains in a distribution system that can be readily reinforced, the general opinion seems to be that the added capacity should provide for about ten years growth, additional pumping equipment should be adequate for fifteen years growth and stand pipes for twenty-five years growth.

Several officials who advocated making certain of an ample supply for many years in advance, also expressed the opinion that there had been a tendency to keep the excess capacity of distribution systems and pumping equipment above current demands somewhat larger than was necessary.*

Fire hydrants:- the principle requirements of a fire hydrant are that it shall:

1- Be of simple and rugged construction.
2- Operate freely and positively.
3- Drain when closed and not leak when open.
4- Discharge with a minimum loss of pressure.
5- Permit withdrawal of working parts from the top of the barrel.

The hydrant most commonly used is designed for two hose streams, has a 6" main connection valve opening 5" or more in diameter, discharges 500 gallons per minute with a loss of pressure of not more than 2 pounds per square inch, and is equipped with two $2\frac{1}{2}$ inch hose nozzles and one steamer nozzle which is usually about $4\frac{1}{2}$ inches in diameter. The use of smaller hydrants with 4 inch main connection is thought to be a questionable policy except in high pressure areas.

As a general rule, a hydrant should be located at each street intersection, with intermediate hydrants at hazardous locations. The spacing should never exceed 500 feet.

Gate valves should be closely spaced so as to permit segregating a part of the system with a minimum interruption of local service. The valves should be so placed that not more than 4 to 6 will have to be closed to segregate a given section.

* Page 568, "Water Works Practice Manual."

It is better still, to be able to cut out a given section by closing only two valves. It is customary to gate all branches from the feeder mains, and to provide not less than two valves at any street intersection. Cross connections between feeders should be gated at each end. All gate valves ought to be systemmatically located with reference to street lines, trees, or telephone poles.

It is quite common to use Class "C" cement lined cast iron pipe in distribution systems in which the maximum pressure is anything less than 300 pounds per square inch.

The cement lining stands up much better than is commonly believed. The pipe can be drilled and tapped, and broken with a surprisingly small amount of shattering. The cement lining in the pipes is not ordinary Portland cement, because Portland cement would set too quickly, would crack, and would not adhere to the cast iron surface. No doubt the cement that is used in the pipe contains a large portion of Gypsum to retard the setting of the cement.

Mains should, as far as possible, be located in a standard position with respect to the center of the street so that they may be conveniently located.

In the following calculations, the assumption is made that the reservoir is constantly half full of water. That is, the elevation of the surface of the water in the reservoir is assumed to be constantly 377.5 feet above the Bench mark by the Fountain City Lake which was assumed to be 100 feet.

CALCULATIONS FOR SIZE OF PIPE
FOR DISTRIBUTION.

Loss of head in feet $= H' = ke\frac{V^2}{2g} + f\frac{l}{d}\,\frac{V^2}{2g} + kd\frac{V^2}{2g}$

In the above equation ke = coefficient of loss at entrance
kd = coefficient of loss at discharge
f = friction factor $0.002 + \frac{0.02}{d''}$ (Darcy)
l = length of pipe in feet
d = diameter of pipe in feet
$\frac{V^2}{2g}$ = velocity head at discharge.

It is safe to assume that ke = 0.5
and kd = 1.0

It is desired to deliver 500 cubic feet of water per minute at the intersection of Tennessee Avenue and Jacksboro Pike.

$l = 10{,}630'$ $f = 0.02 + \frac{0.02}{10}$

Assume $d = 10''$ $= 0.02 + 0.002 = 0.022$

$$192.9' = H' = (1.5 + 0.022 \times \frac{10{,}630 \times 12}{10})\frac{V^2}{2g}$$

$$192.9 = (1.5 + 281)\frac{V^2}{2g}$$

$$\frac{192.9}{282.5} = \frac{V^2}{2g}$$

$$V^2 = \frac{192.9 \times 2g}{282.5} = 43.9$$

$$V = 6.62$$

Area of 10" pipe $= \pi(\frac{5}{12})^2 = \pi(0.416)^2 = 0.543$ sq. feet.

$q = AV = 0.543 \times 6.62 = 3.49$ cu. ft. per second.
$q = 3.49 \times 60 \times 7.48 = 1566$ gallons per minute.
Therefore a 10" pipe is too large.

Assume $d = 8"$ $\qquad f = 0.02 + \frac{0.02}{8} = 0.0225$

$$192.9 = (1.5 + 0.0225 \times \frac{10630 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 359.2)\frac{V^2}{2g}$$

$$V^2 = \frac{192.9 \times 2g}{360.7} = 34.4$$

$V = 5.86$ ft. per second.

Area of 8" pipe $= \pi(\frac{4}{12})^2 = \pi(0.333)^2 = 0.3485$ sq. ft.

$q = 0.3485 \times 5.86 = 2.041$ cu. ft. per sec.
$q = 2.041 \times 60 \times 7.48 = 918$ gallons per min.
Therefore it will be safe to use an 8" pipe.

At the intersection of Wautauga Avenue and Third St.

$$150.3 = (1.5 + 0.0225 \times \frac{8917 \times 12}{8} + 0.0233 \times \frac{550 \times 12}{6})\frac{V^2}{2g}$$

$$= (1.5 + 301.1 + 25.63)\frac{V^2}{2g}$$

$$V^2 = \frac{150.3 \times 2g}{328.23} = 29.41$$

$V = 5.42$ ft. per second.
$q = 0.1962 \times 5.42 = 1.062$

$q = 1.062 \times 60 \times 7.48 = 477$ gallons per minute.

This is not enough, so use an 8" pipe from Tennessee Avenue to Wautauga on Third Street.

$$150.3 = (1.5 + 0.0225 \times \frac{9467 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 319.8)\frac{V^2}{2g}$$

$$V^2 = \frac{150.3 \times 2g}{321.3} = 30.11 \;;\; V = 5.5$$

$q = 0.3485 \times 5.5 = 1.92$ cu. ft. per sec.

$q = 1.92 \times 60 \times 7.48 = 860$ gallons per minute.

$f = 0.02 + \frac{02}{6} = 0.0233$ $\qquad$ area of a 6" pipe $= \pi(\frac{3}{12})^2 = 0.1962.$

At intersection of Tennessee Avenue and Second Street.

$$204.3 = (1.5 + 0.0225 \times \frac{8317 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 280.9)\frac{V^2}{2g}$$

$$V^2 = \frac{204.3 \times 2g}{282.4} = 46.5 \ ; \ V = 6.82 \text{ ft. per second.}$$

$q = 0.3485 \times 6.82 = 2.38$ cu. ft. per second.

$q = 2.38 \times 60 \times 7.48 = 1069$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

At intersection of Tennessee Avenue and Broadway.

$$238.6 = (1.5 + 0.0225 \times \frac{7,917 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 267.5)\frac{V^2}{2g}$$

$$V^2 = \frac{238.6 \times 2g}{269} = 57.1$$

$V = 7.57$ ft. per sec.

$q = 0.3485 \times 7.57 = 2.64$ cu. ft. per sec.

$q = 2.64 \times 60 \times 7.48 = 1185$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

At intersection of College Street and Broadway.

$$259.7 = (1.5 + 0.0225 \times \frac{6642 \times 12}{8} + 0.0233 \times \frac{650 \times 12}{6})\frac{V^2}{2g}$$

$$= (1.5 + 224.0 + 30.3)\frac{V^2}{2g}$$

$$V^2 = \frac{259.7 \times 2g}{255.8} = 65.4$$

$V = 8.1$ ft. per second.

$q = 0.1962 \times 8.1 = 1.589$ cu. ft. per second.

$q = 1.589 \times 60 \times 7.48 = 714$ gallons per minute

area of 6" pipe $= 0.1962$ sq. ft.

At intersection of Church Street and Broadway.

$$277.0 = \left(1.5 + 0.0225 \times \frac{5825 \times 12}{8} + 0.0233 \times \frac{1000 \times 12}{6}\right)\frac{V^2}{2g}$$

$$= (1.5 + 196.8 + 46.6)\frac{V^2}{2g}$$

$$V^2 = \frac{277 \times 2g}{244.9} = 72.8$$

$V = 8.53$ ft. per second.

$q = 0.1962 \times 8.53 = 1.676$ cu. ft. per sec.

$q = 1.676 \times 60 \times 7.48 = 752$ gallons per minute.

area of 6" pipe = 0.1962 sq. ft.

At intersection of Hotel Avenue and Broadway.

$$276.9 = \left(1.5 + 0.0225 \times \frac{5825 \times 12}{8} + 0.0233 \times \frac{1066 \times 12}{6}\right)\frac{V^2}{2g}$$

$$= (1.5 + 196.8 + 49.75)\frac{V^2}{2g}$$

$$V^2 = \frac{276.9 \times 2g}{248.05} = 71.7$$

$V = 8.46$ ft. per sec.

$q = 0.1962 \times 8.46 = 1.66$ cu. ft. per second.

$q = 1.66 \times 60 \times 7.48 = 745$ gallons per minute.

area of 6" pipe = 0.1962 sq. ft.

At intersection of "C" Street and Third Avenue.

$$237.2 = \left(1.5 + 0.0225 \times \frac{4675 \times 12}{8} + 0.0233 \times \frac{675 \times 12}{6}\right)\frac{V^2}{2g}$$

$$= (1.5 + 157.9 + 31.5)\frac{V^2}{2g}$$

$$V^2 = \frac{237.2 \times 2g}{190.0} = 80$$

$V = 8.95$ ft. per sec.

$q = 0.1962 \times 8.95 = 1.755$ cu. ft. per second.

$q = 1.755 \times 60 \times 7.48 = 786$ gallons per minute.

area of 6" pipe = 0.1962 sq. ft.

Intersection of "B" Street and Third Avenue.

$$247.4 = (1.5 + 0.0225 \times \frac{4050 \times 12}{8} + 0.0233 \times \frac{675 \times 12}{6})\frac{V^2}{2g}$$

$$= (1.5 + 136.9 + 31.5)\frac{V^2}{2g}$$

$$V^2 = \frac{247.4 \times 2g}{169.9} = 93.6$$

$V = 9.67$ ft. per second

$q = 0.1962 \times 9.67 = 1.9$ cu. ft. per sec.

$q = 1.9 \times 60 \times 7.48 = 852$ gallons per minute.

area of 6" pipe $= 0.1962$ sq. ft.

Intersection of "A" Street and Third Avenue

$$240.8 = (1.5 + 0.0225 \times \frac{4075 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 137.6)\frac{V^2}{2g}$$

$$V^2 = \frac{240.8 \times 2g}{139.1} = 111.2$$

$V = 10.55$ ft. per second.

$q = 0.3485 \times 10.55 = 3.68$ cu. ft. per second.

$q = 3.68 \times 60 \times 7.48 = 1650$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

Intersection of "A" Street and Fourth Avenue.

$$203.0 = (1.5 + 0.0225 \times \frac{3750 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 126.6)\frac{V^2}{2g}$$

$$V^2 = \frac{203 \times 2g}{128.1} = 101.9$$

$V = 10.1$ ft. per second.

$q = 0.3485 \times 10.1 = 3.52$ cu. ft. per sec.

$q = 3.52 \times 60 \times 7.48 = 1579$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

Intersection of "A" Street and Fifth Avenue.

$$182.8 = (1.5 + 0.0225 \times \frac{3400 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 114.9)\frac{V^2}{2g}$$

$$V^2 = \frac{182.8 \times 2g}{116.4} \quad 100.8$$

$V = 10.04$ ft. per sec.

$q = 0.3485 \times 10.04 = 3.51$ cu. ft. per sec.

$q = 3.51 \times 60 \times 7.48 = 1571$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

Intersection of "A" Street at Seventh Avenue.

$$162.8 = (1.5 + 0.0225 \times \frac{2750 \times 12}{8})\frac{V^2}{2g}$$

$$= (1.5 + 92.9)\frac{V^2}{2g}$$

$$V^2 = \frac{162.8 \times 2g}{94.4} = 111.0$$

$V = 10.54$ ft. per sec.

$q = 0.3485 \times 10.54 = 3.67$

$q = 3.67 \times 60 \times 7.48 = 1649$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

Intersection of Virginia Avenue and Broadway.

$$275.1 = (1.5 + 0.0225 \times \frac{6410 \times 12}{8} + 0.0233 \times \frac{600 \times 12}{6})\frac{V^2}{2g}$$

$$= (1.5 + 216.3 + 28.0)\frac{V^2}{2g}$$

$$V^2 = \frac{275.1 \times 2g}{245.8} = 72.0$$

$V = 8.5$ ft. per sec.

$q = 0.1962 \times 8.5 = 1.669$ cu. ft. per sec.

$q = 1.669 \times 60 \times 7.48 = 749$ gallons per minute.

area of 8" pipe $= 0.1962$ sq. ft.

At intersection of Virginia Avenue and Fountain Avenue.

$$274.9 = \left(1.5 + 0.0225 \times \frac{6410 \times 12}{8} + 0.0233 \times \frac{350 \times 12}{6}\right)\frac{V^2}{2g}$$

$$= (1.5 + 216.3 + 16.3)\frac{V^2}{2g}$$

$$V^2 = \frac{274.9 \times 2g}{234.1} = 75.5$$

$V = 8.7$ ft. per second.

$q = 0.1962 \times 8.7 = 1.709$

$q = 1.709 \times 60 \times 7.48 = 766$ gallons per minute.

area of 6" pipe $= 0.1962$ sq. ft.

At intersection of Virginia Avenue and Central Avenue.

$$272.9 = \left(1.5 + 0.0225 \times \frac{6260 \times 12}{8}\right)\frac{V^2}{2g}$$

$$= (1.5 + 211.9)\frac{V^2}{2g}$$

$$V^2 = \frac{272.9 \times 2g}{213.4} = 82.3$$

$V = 9.09$ ft. per sec.

$q = 0.3485 \times 9.09 = 3.168$ cu. ft. per second.

$q = 3.168 \times 60 \times 7.48 = 1421$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

At intersection of Maple Avenue and Central Avenue.

$$274.2 = \left(1.5 + 0.0225 \times \frac{6810 \times 12}{8}\right)\frac{V^2}{2g}$$

$$= (1.5 + 230)\frac{V^2}{2g}$$

$$V^2 = \frac{274.2 \times 2g}{231.5} = 76.35$$

$V = 8.75$ ft. per second.

$q = 0.3485 \times 8.75 = 3.048$ cu. ft. per second.

$q = 3.048 \times 60 \times 7.48 = 1366$ gallons per minute.

area of 8" pipe $= 0.3485$ sq. ft.

Intersection of Jackson Boulevard and Broadway.

$$\text{Area of 8" pipe} = 0.3485 \text{ sq. ft.}$$

$$271.9 = \left(1.5 + 0.0225 \times \frac{8010 \times 12}{8}\right)\frac{V^2}{2g}$$

$$= (1.5 + 270.7)\frac{V^2}{2g}$$

$$V^2 = \frac{271.9 \times 2g}{272.2} = 64.1$$

$$V = 8.02 \text{ ft. per sec.}$$

$$q = 0.3485 \times 8.02 = 2.795 \text{ cu. ft. per sec.}$$

$$q = 2.795 \times 60 \times 7.48 = 1256 \text{ gallons per minute.}$$

450' West of Melville Avenue on Jackson Boulevard.

$$\text{Area of 8" pipe} = 0.3485 \text{ sq.ft.}$$

$$237.4 = \left(1.5 + 0.0225 \times \frac{9060 \times 12}{8}\right)\frac{V^2}{2g}$$

$$= (1.5 + 305.9)\frac{V^2}{2g}$$

$$V^2 = \frac{237.4 \times 2g}{307.4} = 49.6$$

$$V = 7.05 \text{ ft. per sec.}$$

$$q = 0.3485 \times 7.05 = 2.46 \text{ cu. ft. per sec.}$$

$$q = 2.46 \times 60 \times 7.48 = 1102 \text{ gallons per minute.}$$

450' East of Melville Avenue on Jackson Boulevard.

$$\text{Area of 8" pipe} = 0.3485 \text{ sq. ft.}$$

$$223.0 = \left(1.5 + 0.0225 \times \frac{10{,}010 \times 12}{8}\right)\frac{V^2}{2g}$$

$$= (1.5 + 338)\frac{V^2}{2g}$$

$$V^2 = \frac{223 \times 2g}{339.5} = 42.25$$

$$V = 6.5 \text{ ft. per sec.}$$

$$q = 0.3485 \times 6.5 = 2.265 \text{ cu. ft. per sec.}$$

$$q = 2.265 \times 60 \times 7.48 = 1018 \text{ gallons per minute.}$$

TABULATIONS

Street Intersection	Elevation, ft.	From Reservoir to Street Intersection Length of Pipe	Size of Pipe	Average Available Head: ft.	Average Velocity at Discharge: ft. per sec.	Average Discharge: cu. ft. per sec.	Average Discharge: gallons per min.	Average Pres.: lbs.per sq.in
Tenn. Ave. & Jacksboro Pk.	184.6	10,630'	8"	192.9	5.86	2.041	918	83.71
Wautauga Ave. & Third St.	227.2	9,467'	8"	150.3	5.50	1.920	860	65.2
Tenn. Ave. & Second St.	173.2	8,317'	8"	204.3	6.82	2.380	1,069	88.7
Tenn. Ave. & Broadway	138.9	7,917'	8"	238.6	7.57	2.640	1,185	103.5
College St. & Broadway	117.8	6,642' 650'	8" 6"	259.7	8.10	1.589	714	112.4
Church St. & Broadway	100.5	5,825' 1,000'	8" 6"	277.0	8.53	1.676	752	120.0
Hotel Ave. & Broadway	100.6	5,825' 1,066'	8" 6"	276.9	8.46	1.660	745	120.1
"C" St. & Third Avenue	140.3	4,675' 675'	8" 6"	237.2	8.95	1.755	786	102.9
"B" St. & Third Ave.	130.1	4,050' 675'	8" 6"	247.4	9.67	1.900	852	107.0
"A" St. & Third Ave.	136.7	4,075'	8"	240.8	10.55	3.680	1,650	104.2
"A" St. & Fourth Ave.	174.5	3,750'	8"	203.0	10.10	3.520	1,579	88.0
"A" St. & Fifth Ave.	194.7	3,400'	8"	182.8	10.04	3.510	1,571	79.2
"A" St. at Seventh Ave.	214.7	2,750'	8"	162.8	10.54	3.670	1,649	70.5
Virginia Ave. & Broadway	102.4	6,410' 600'	8" 6"	275.1	8.50	1.669	749	119.1
Virginia Ave. & Fountain Ave.	102.6	6,410' 350'	8" 6"	274.9	8.70	1.709	766	119.1
Virginia Ave. & Central Ave.	104.6	6,260'	8"	272.9	9.09	3.168	1,421	118.2
Maple Ave. & Central Ave.	103.3	6,810'	8"	274.2	8.75	3.048	1,366	119.0
Jackson Boulevard & Broadway	105.6	8,010'	8"	271.9	8.02	2.795	1,256	117.8
450' West of Melville Ave on Jackson Boulevard.	140.1	9,060'	8"	237.4	7.05	2.460	1,102	102.9
450' East of Melville Ave. on Jackson Boulevard.	154.5	10,010'	8"	223.0	6.50	2.265	1,018	96.7
Jackson Boulevard & Jacksboro Pike	144.6	11,060'	8"	232.9	6.34	2.206	990	101.0
Hickory Ave. & Jacksboro Pk.	154.5	11,885'	8"	223.0	5.98	2.082	935	96.7
Conner Ave. & Jacksboro Pike	140.9	12,335'	8"	236.6	6.04	2.102	945	102.8

Intersection of Jackson Boulevard and Jacksboro Pike.

Area of 8" pipe $= 0.3485$ sq. ft.

$$232.9 = (1.5 + 0.0225 \times \frac{11,060 \times 12}{8})\frac{V^2}{2g}$$
$$= (1.5 + 373)\frac{V^2}{2g}$$
$$V^2 = \frac{232.9 \times 2g}{374.5} = 40.0$$
$$V = 6.34 \text{ ft. per sec.}$$

$q = 0.3485 \times 6.34 = 2.206$ cu. ft. per sec.

$q = 2.206 \times 60 \times 7.48 = 990$ gallons per minute.

Intersection of Hickory Avenue and Jacksboro Pike.

Area of 8" pipe $= 0.3485$ sq. ft.

$$223.0 = (1.5 + 0.0225 \times \frac{11,885 \times 12}{8})\frac{V^2}{2g}$$
$$= (1.5 + 401.0)\frac{V^2}{2g}$$
$$V^2 = \frac{223 \times 2g}{402.5} = 35.6$$
$$V = 5.98 \text{ ft. per sec.}$$

$q = 0.3485 \times 5.98 = 2.082$ cu. ft. per sec.

$q = 2.082 \times 60 \times 7.48 = 935$ gallons per minute.

Intersection of Conner Avenue and Jacksboro Pike.

Area of 8" pipe $= 0.3485$ sq. ft.

$$236.6 = (1.5 + 0.0225 \times \frac{12,335 \times 12}{8})\frac{V^2}{2g}$$
$$= (1.5 + 417)\frac{V^2}{2g}$$
$$V^2 = \frac{236.6 \times 2g}{418.5} = 36.4$$
$$V = 6.04 \text{ ft. per sec.}$$

$q = 0.3485 \times 6.04 = 2.102$ cu. ft. per sec.

$q = 2.102 \times 60 \times 7.48 = 945$ gallons per minute.

Cost of Distribution System

63,678 ft. of 8", cement lined, tar coated pipe at $1.53 per ft. _	$97,427.
50,705 ft. of 6", cement lined, tar coated pipe at $0.80 per ft. _	40,564.
162 6" fire hydrants at $130 each (installed) _	21,060.
373 6" gate valves at $25.62 each _	9,556.
189 8" gate valves at $34.10 each _	6,445
561 gate valve boxes at $3.50 each _	1,964
115 3 way branches 2 bells 8", 1 bell 6" at $11.60 each _	1,334
21 3 way branches 3 bells 8" at $11.60 each _	244
94 3 way branches 3 bells 6" at $10.80 each _	1,015
10 4 way branches 4 bells 6" at $12.60 each _	126
4 4 way branches 4 bells 8" at $13.80 each _	55
2 4 way branches 2 bells 8", 2 bells 6" at $13.60 each _	27
688 3/4" service meters at $18.00 each _	12,384
3 $1\frac{1}{2}$" service meters at $54.00 each _	162
688 3/4" corporation cocks at $2.19 each _	1,507
3 $1\frac{1}{2}$" corporation cocks at $6.60 each _	20
691 service valve boxes at $2.50 each _	1,728
691 (D&R) service boxes at $5.00 each _	3,455
43,530 ft. of 3/4" service pipe at $0.07 per ft. _	3,047
500 ft. of $1\frac{1}{2}$" service pipe at $0.12 per ft. _	60
6 8" 90° curves at $5.93 each _	36
5 6" 90° curves at $4.15 each _	21
16 8" 45° curves at $5.93 each _	95
19 6" 45° curves at $4.15 each _	79
18 8" $22\frac{1}{2}^\circ$ curves at $5.93 each _	107
15 6" $22\frac{1}{2}^\circ$ curves at $4.15 each _	62
16 8" $11\frac{1}{4}^\circ$ curves at $5.93 each _	95
11 6" $11\frac{1}{4}^\circ$ curves at $4.15 each _	46
15 standard reducers from 8" to 6" at $5.65 each _	85
2 standard reducers from 6" to 4" at $3.45 each _	7
400 ft. of 4" cement lined, tar coated iron pipe at $0.25 per ft. _	100
2 4" $22\frac{1}{2}^\circ$ curves at $1.65 each _	3
2 4" gate valves at $14.00 each _	28
Main ditching 29,350 cu. yds. at $1.62 per cu. yd. _	47,500
Service pipe ditching 3,260 cu. yds. at $1.62 per cu. yd. _	5,290
Pig Lead 117,164# at $0.110 per lb. _	12,888
Yarn 4,214# at $0.105 per lb. _	442
Pipe Derrick _	300
Lead kettle _	16
Topping machine _	80.
Total _ _ _ _ _ _ _ _ _	$269,460.00

RESERVOIRS

The location of a reservoir is largely determined by the distance of the reservoir from, and the elevation above, the point of distribution. Long distances require very heavy expenditures for pipe lines, but these expenditures are relatively less the larger the quantity of water dealt with. For large cities, therefore, it will be practical to go much farther than it would be in small cities. The elevation of the reservoir should be such that as many as possible of the customers will be served by gravity alone.

Fountain City is so located that it may all be served by gravity except two or three large country homes on top of Black Oak Ridge, which is about 65 feet above the top of the reservoir. If the three above homes are to be supplied with water for domestic and for fire protection purposes from the Fountain City works it will be necessary to build a special reservoir on the top of Black Oak Ridge, and to lay the pipe-lines along the crest of the ridge.

To the author of this paper, it is indeed a questionable policy to build an extensive reservoir and piping lay out just to accomodate three houses. As the three houses already have their own private water supply for domestic purposes, it would probably be a good idea to lay a pipe line from the main reservoir on up the side of the ridge to an elevation 65 feet above the main reservoir and set a fire plug there. Then a fire-engine could pump the water from the main reservoir in case of fire. One of the houses will be about 900 feet west of the hydrant, one house will be 400 feet east of the hydrant, and the other house will be about 1,000 feet east of the hydrant.

Calculations for Thickness of Reservoir Walls

$$S_h t = P'r$$

in which S_h is the allowable tensile stress in pounds per sq. in.

t is the thickness of the wall.

P is the total pressure in pounds per square inch.

r is the radius of the tank in inches.

Assume that 1/16 of the total area of a vertical cross section of the wall is steel whose tensile strength is 10,000 pounds per square inch and that the allowable tensile stress in the concrete is 50 pounds per square inch. That is, for every 15/16 square inch of concrete there is 1/16 square inch of steel.

The resultant allowable tensile stress $= S_h =$

$$1/16 \times 10{,}000 \quad 15/16 \times 50$$

$$= 625 \quad 469 = 1094 \text{ pounds per square inch.}$$

When the reservoir is full up to the top the total pressure on the bottom and the walls at the bottom will be $P' = 35 \times 0.434 = 15.2$ pounds per square inch.

The radius of the tank or reservoir $= r = 35 \times 12 = 420$ inches.

Solving for t we get

$$t = \frac{P'r}{S_h} = \frac{15.2 \times 420}{1094}$$

$$= \frac{6390}{1094} = 5.84 \text{ inches thick.}$$

Using a factor of safety of 1.7, we get

$$t = 5.84 \times 1.7 = 10 \text{ inches.}$$

The base of the reservoir will also be 10 inches thick.

The soil on which the reservoir is to be built is hard clay. The safe bearing load for hard clay is 4 tons per square foot.*

Weight of water in the reservoir $= \dfrac{(35)^2 \times 35 \times 62.4}{2000} =$

$$\frac{8{,}400{,}000}{2000} = 4200.0 \text{ tons.}$$

Assume the weight of the concrete as 150 pounds per cubic foot.

* From page 322 of Biggs & Woolrich, "Handbook of Steam Engineering."

Total weight of concrete in reservoir $=$

$$(35.834)^2 - (35)^2 \quad 35 \qquad (35.834)^2 \times 0.834 \quad 150$$

$$= \qquad 190 \times 35 \quad 4050 \times 0.834 \quad 150$$

$$= \qquad 6650 \qquad 3380 \quad 150 = 10,030 \times 150 = 1,507,000 \text{ pounds.}$$

$$\frac{1,507,000}{2000} = 754.0 \text{ tons.}$$

Total weight of reservoir when it is full of water $=$

$$4200.0 \quad 754.0 = 4954.0 \text{ tons.}$$

$\frac{4954}{4} = 1240$ square feet of bearing surface will be required for one reservoir when it is filled full of water.

This radius is 15.14 feet less than the radius of the reservoir itself. Therefore, it will be perfectly safe to allow the reservoir to rest upon its own base without extending the area of the base.

Area of cross_section of a wall of reservoir $= 0.834 \times 35 = 29.19$ square feet. But since 1/16 of the total area was assumed to be steel, $29.19 \div 16 = 1.82$ square feet of steel in the cross-section of the wall.

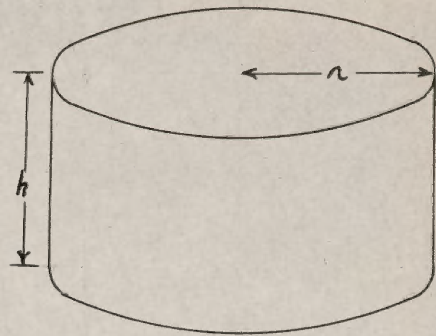
$$1.82 \times 144 = 262 \text{ square inches.}$$

Area of a $1\frac{1}{2}$ inch steel cable if it were solid would be equal to $(0.75)^2 = 1.77$ sq.in.

Number of turns of steel cable required $= \frac{262}{1.77} = 148.$

Pitch of cable $= \frac{35 \times 12}{148} = 2.82$ or 3 inches between centers.

CALCULATIONS FOR RESERVOIR



A = Area $= \pi r^2 + 2\pi r h$

V = Volume $= \pi r^2 h$ = a constant

Substitute value of h in the first equation

$$A = \pi r^2 + 2\pi r \left(\frac{V}{\pi r^2}\right)$$

$$A = \pi r^2 + \frac{2V}{r}$$

Differentiating the area with respect to the radius. $\frac{dA}{dr} = 2\pi r - \frac{2V}{r^2}$

The above derivative is equal to zero when $r = \left(\frac{V}{\pi}\right)^{\frac{1}{3}}$ and we have a maximum volume with a minimum area.

$$\frac{dA}{dr} = 2\pi\left(\frac{V}{\pi}\right)^{\frac{1}{3}} - \frac{2V}{\left(\frac{V}{\pi}\right)^{\frac{2}{3}}} = \pi\left(\frac{V}{\pi}\right)^{\frac{1}{3}} - V\left(\frac{V}{\pi}\right)^{-\frac{2}{3}} = \pi^{\frac{3}{3}}\pi^{-\frac{1}{3}}V^{\frac{1}{3}} - V^{\frac{3}{3}}\frac{V^{-\frac{2}{3}}}{\pi^{-\frac{2}{3}}}$$

$$= \frac{V^{\frac{1}{3}}}{\pi^{-\frac{2}{3}}} - \frac{V^{\frac{1}{3}}}{\pi^{-\frac{2}{3}}} = 0$$

Substitute some value of r greater than $r = \left(\frac{V}{\pi}\right)^{\frac{1}{3}}$ such as $r = \left(\frac{V}{\pi}\right)^{\frac{1}{2}}$. Then

$$\frac{dA}{dr} = \pi\left(\frac{V}{\pi}\right)^{\frac{1}{2}} - \frac{V}{\left(\frac{V}{\pi}\right)^{\frac{2}{2}}} = \sqrt{\pi V} - \pi$$

Substitute some value of r less than $r = \left(\frac{V}{\pi}\right)^{\frac{1}{3}}$ such as $r = \left(\frac{V}{\pi}\right)^{\frac{1}{4}}$
Then

$$\frac{dA}{dr} = \pi\left(\frac{V}{\pi}\right)^{\frac{1}{4}} - \frac{V}{\left(\frac{V}{\pi}\right)^{\frac{2}{4}}} = \pi^{\frac{4}{4}}\pi^{-\frac{1}{4}}V^{\frac{1}{4}} - V^{\frac{2}{2}}V^{-\frac{1}{2}}\pi^{\frac{1}{2}} = \pi^{\frac{3}{4}}V^{\frac{1}{4}} - \sqrt{\pi V}$$

Therefore when $r = \left(\frac{V}{\pi}\right)^{\frac{1}{3}}$ the tank will have a maximum volume with a minimum area.

Substituting this value of r in the equation $V = \pi r^2 h$ we get $V = \pi\left(\frac{V}{\pi}\right)^{\frac{2}{3}}h$

$$= \pi^{\frac{3}{3}}\pi^{-\frac{2}{3}}V^{\frac{2}{3}}h = \pi^{\frac{1}{3}}V^{\frac{2}{3}}h \quad \text{and } h = \frac{V^{\frac{3}{3}}}{\pi^{\frac{1}{3}}V^{\frac{2}{3}}} = \left(\frac{V}{\pi}\right)^{\frac{1}{3}}$$

$$A = \pi\left(\frac{V}{\pi}\right)^{\frac{2}{3}} + 2\pi\left(\frac{V}{\pi}\right)^{\frac{1}{3}}h = \pi^{\frac{1}{3}}V^{\frac{2}{3}} + 2\pi^{\frac{2}{3}}V^{\frac{1}{3}}h$$

Assuming that there are 4.5 persons living in each of the 727 buildings in Fountain City, the reservoir must be capable of supplying 100 gallons per person per day which is 327,150 gallons per day.

Since the maximum recorded flow was 8,000,000 gallons per 24 hours it is proposed to build 2 one million gallon reservoirs.

$$\frac{2,000,000}{7.48} = 267,600 \text{ cubic feet.}$$

Volume of one reservoir equals $\frac{267,600}{2} = 133,800$ cu. ft.

$$h = \sqrt[3]{\frac{V}{\pi}} = \sqrt[3]{\frac{133800}{\pi}} = \sqrt[3]{42,500} = 34.85$$

$$V = \pi r^2 h; \; r = \sqrt{\frac{V}{\pi h}} = \sqrt{\frac{133,800}{\pi \times 34.85}} = \sqrt{1220}$$

$$r = 35'$$

$$\begin{aligned} A &= \pi r^2 + 2\pi r h \\ &= \pi (35)^2 + 2\pi \times 35 \times 34.8 \\ &= 3850 + 7651 = 11,501 \text{ square feet.} \end{aligned}$$

To prove that a reservoir with $r = 35'$ and $h = 34.8$ has a maximum volume with a minimum area, assume some other value of h and r that will give the desired volume, such as $h = 47.2'$ and $r = 30'$. Then the area will be

$$A = \pi r^2 + 2\pi r h = \pi (30)^2 + 2\pi \times 30 \times 47.2$$

$$= 2831 + 8900 = 11,631 \text{ square feet}$$

which is 130 sq. ft. more area than the differential method gave.

Cost of Reservoir.

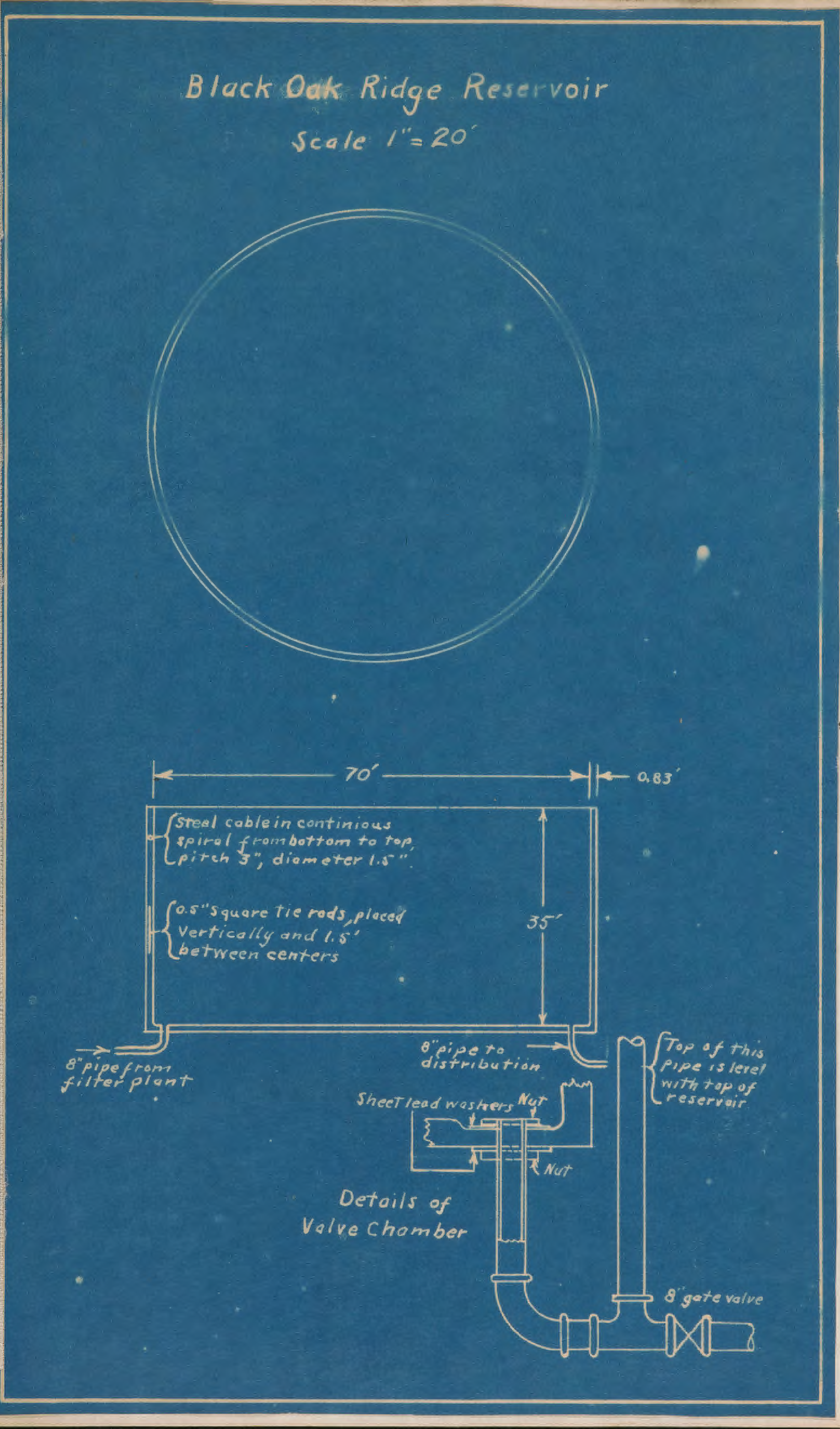
Volume of concrete in reservoir = 10,030 cu. ft.
Use a 1 : 2 : 4 mixture.

1430 cu. ft. or 1430 bags of cement at .75 per bag	– $1,073
2860 cu. ft. or 106 cu. yds. of sand at $1.50 per cu. yd.	– 159
5720 cu. ft. or 212 cu. yds. of rock at $1.50 per cu. yd.	– 318
148 x x 68 = 31600 ft. of $1\frac{1}{2}$" cable at .15 per ft.	– 4,740
One 10" 90° curve	– 14
One 8" 90° curve	– 12
One 3 way 8" branch	– 12
One 8" gate valve	– 34
5005 ft. of 1/2" square concrete reenforcement steel at .04$\frac{1}{2}$ per ft.	– 225
No. sq. ft. of lumber required = 70 x x 35 71.66 x x 35 = 15,600 sq. ft.	
15,600 sq. ft. of 3/4" X 4" pine at $0.11 per sq. ft.	– 1,716
Total cost of one reservoir (Not including labor and transportation costs)	– $8,303

If the labor and transportation cost is assumed to be $3,000 then the total cost of the reservoir will be $11,303.

FIGURE 5

BLACK OAK RIDGE RESERVOIR


Black Oak Ridge Reservoir
Scale 1"=20'
70'
0.83'
Steel cable in continious spiral from bottom to top, pitch 3", diameter 1.5".
0.5" Square tie rods, placed Vertically and 1.5' between centers
35'
8" pipe from filter plant
8" pipe to distribution
Top of this pipe is level with top of reservoir
Sheet lead washers
Nut
Nut
Details of Valve Chamber
8" gate valve

CHAPTER 4

Cost of Filtered Water

From previous calculations it was found that 127 horse power would be required to deliver 1.935 cubic feet per second against a head of 376 feet. But 1 horse power = 746 watts. Therefore, 127 x 746 = 94,742 watts.

$$\frac{94742}{1000} = 94.74 \text{ K. W.}$$

$94.74 \times 24 = 2273.76$ K. W. Hr.

2273.76 at \$0.09 per K. W. Hr. = \$204.63 to pump 1,000,000 gallons per 24 hours.

With a turbidity of 25 parts per million, 25 pounds of aluminum sulphate per million gallons will be required.*

25 lbs. at \$0.078 per lb. = \$1.95

For raw water pump and wash water pump 2 five-horse power motors.

$746 \times 10 = 7460$ watts

$$\frac{7460}{1000} = 7.46 \text{ K. W.}$$

$7.46 \times 24 = 179.04$ K. W. Hr.

179.04 at \$0.09 per K. W. Hr. = \$16.11

If two attendants at \$1400 each per year are employed at the filter plant, then $\frac{2800}{365}$ = \$7.66 per day for labor.

* From plate III, page 225 of Stern's "Water Purification Plants and Their Operation."

Cost of Purification and Pumping per Million Gallons.

Aluminum Sulphate	$1.95
Power	220.74
Labor	7.66
Total =	$230.35

If it is assumed that the life of the entire waterworks system is 75 years, then $\frac{302,302}{75} = \$4,030$ per year.

$\frac{4030}{365} = \$11.02$ per day or per million gallons of water.

If it is assumed that 2 meter readers, 3 office men, and 7 laborers to keep up main and service repairs, are each paid $80.00 per month, then $80 \times 12 = \$960$ per month.

$\frac{960}{30} = \$32.00$ per day.

The total amount that must be charged off per day is as follows:

For purification and pumping	$230.35
For depreciation on entire water works	11.02
For office and maintenance labor	32.00
Total =	$273.37

$\frac{273.37}{1,000,000} = \$0.000273$ per gallon.

$0.000273 \times 1000 = \$0.27$ per thousand gallons.

The Knoxville Water Department charges $1.00 per 500 cubic feet of water.

$7.5 \times 500 = 3750.0$ gallons for $1.00.

$\frac{1.00}{3750} = \$0.00026$ per gallon.

$0.00026 \times 1000 = \$0.26$ per thousand gallons.

Therefore, under the above conditions the Fountain City water works would have to charge one cent per thousand gallons more than the Knoxville Water Department is charging.

CHAPTER 5

Summation of Cost Data.

Cost of filter plant _ _ _ _ _ _ _ _ _ _ _ _ _ _ _	$21,539.00
Cost of distribution system _ _ _ _ _ _ _ _ _ _ _ _	$269,460.00
Cost of one storage reservoir _ _ _ _ _ _ _ _ _ _ _ _	11,303.00
Total cost of the water works system _ _ _ _ _ _ _ _	$302,302.00

CHAPTER 6

Bibliography

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APPENDIX I

Historical

The first idea of purification of water by filtration was brought back to the United States from Europe in 1866 by a civil engineer named Kirkwood.

Shortly before 1866, in 1842, the Croton works in New York were put into service. The water from the Croton River was guided forty-one miles through an 8 by 7 foot aqueduct into the city. This aqueduct was capable of carrying 95,000,000 gallons of water per day. A few years later a series of storage reservoirs were built by placing dams across the mouths of the tributaries of the Croton River. When New York outgrew the source of supply offered by the Croton River, the Catskill Mountain catchment area was taken under consideration. The Ashokan Reservoir was built on Esopas Creek in the Catskill Mountain about 100 miles from New York. This reservoir is capable of holding 132,000,000,000 gallons of water from a drainage area of 255 square miles.

Boston, Massachusetts, obtained its first water supply from Lake Cochituate in 1848. In 1898, Mystic Lake was abandoned because of the over populated catchment area. Mystic Lake was capable of supplying 9,000,000 gallons of water per day. The Sudbury River was then added to the Cochituate supply and further enlargement was made in the Wachusett Reservoir on the Nashua River. This reservoir is capable of storing 63,000,000,000 gallons of water.

Lynn, Massachusetts, and Norfolk, Virginia, both pump water from a river up into a reservoir that is too large to be filled from its own catchment area and the cities are fed by gravity.

Denver, Colorado; San Francisco, California; and Oakland, California, are supplied from large impounding reservoirs. Several years supply can be held in reserve in the above reservoirs.

Chicago, Detroit, Cleveland, Buffalo, and many smaller cities get their water supply from the Great Lakes.

London, England pumps water from the Thames River during the flood season into the Stains reservoirs where it is stored until needed.

Present State of Development and a General Survey of the Waterworks Field, Including a General Discussion of Purification, Storage, and Distribution.

At the present date the purification, storage, and distribution of water has reached a high degree of perfection. Water that is taken from muddy rivers must be purified. The process of purification generally used is as follows: coagulation with alum, sedimentation by gravity, filtration by gravity through beds of sand, and sterilization by means of liquid chlorine, ozone, ultra-violet rays, or copper sulphate. The liquid chlorine method is the most widely used. The copper sulphate method is not much used except in very large reservoirs in which there is not much danger of the copper sulphate remaining in solution. The copper sulphate must not be added in such large quantities that some of it will remain in solution because it is poison to the human body. When the raw river is inclined to be colloidal as it sometimes is, the sedimentation may be hastened by adding pulverized rock alum. This causes the impurities in the water to be carried down by the aluminum hydroxide precipitate which is formed when the alum is added. Unless it is added in large overdoses, the alum will not appreciably change the taste of the water.

A great deal of iron in ground water gives it an inky taste, a disagreeable odor, and upon standing it gives a very repulsive yellowish appearance. Fresh water that contains iron is usually clear when first drawn from the ground. It is said that a taste can be cultivated for this kind of water, especially if it is thought to have medical properties. The yellow sediment is caused by the ferrous iron in the water being oxidized to the ferric condition. Since the iron in the water is undesirable for cooking and drinking purposes, the iron must be removed. This may be accomplished by aeration, that is, by spraying the water through the air so that the iron can be freely oxidized and can settle out. All of the iron cannot be removed in this manner because some of the fine particles of iron oxide will remain in suspension. Therefore, the water must be filtered to rid it of the iron oxide.

Water that contains calcium or magnesium bicarbonate is said to have temporary hardness.

Water that contains magnesuim sulphate or calcium sulphate, or both of them, is said to have permanent hardness. The bad effect of this permanent hardness may be appreciably reduced by the addition of "soda ash" ($Na_2 CO_3$).

$$Ca\,SO_4 \quad Na_2CO_3 = \underline{Ca\,CO_3} \quad Na_2\,SO_4$$

The white calcium carbonate is precipitated out.

$$Mg\,SO_4 \quad Na_2CO_3 = \underline{Mg\,CO_3} \quad Na_2\,SO_4$$

The white magnesium carbonate is precipitated out.

Some (about 0.2% to 0.01%) undesirable, minute organisms and bacteria pass through the filters under certain conditions. These conditions being that there are not enough desirable bacteria on the surface of the filter bed to eat up or to destroy the undesirable bacteria. To get rid of these undesirable bacteria and at the same time not change the quality of the water by poisoning it, either liquid chlorine or hypochlorite of lime may be used. These two chemicals react with the water, liberating nascent oxygen which oxidizes the organic matter and kills practically all of the harmful bacteria.

Storage of water always depends upon local conditions. That is, concerning the location of rivers, mountains, and natural lake formations. It is generally always desirable to convert a natural lake into a storage or impounding reservoir whenever it is possible to do so where a supply of pure water is available. Because, unless the water is inclined to be colloidal, practically all of the suspended matter will settle out and the water will not require filtration. Then, too, it is cheaper than building an artificial reservoir, that is, if it is within an economical distance of the town to be supplied. It is much cheaper than purifying muddy river water because in the lake the sunshine kills most of the harmful bacteria and they settle to the bottom. There are many types of microscopic plant life that float on the surface of the water, thriving upon the elements of the air and the sunshine. To kill these it is necessary to treat the water with copper sulphate. This is done by hanging a bag of lump copper sulphate over the side of a boat and by rowing the boat back and forth across the lake or reservoir. About one part of copper sulphate in four million parts of reservoir water is sufficient to destroy algae and other objectionable plant life. The treatment with copper sulphate is by no means a permanent "cure" for the algae trouble, because the microscopic spores of the algae are carried by the wind.

When the algae and other forms of plant life are killed, they sink to the bottom of the reservoir. There is a limited amount of free oxygen in the water and until this supply is used up the aerobic bacteria, which acts only in the presence of oxygen, decomposes the dead algae and other dead foreign animals and plants that are present. After the supply of oxygen has been used up, the anerobic bacteria, which acts only when oxygen is not present, decomposes the dead algae and the other dead foreign animals and plants that are present. The anerobic bacteria produces a putrifactive decomposition. This decomposition produces very disagreeable tastes and odors.

The wind stirs the surface probably to a depth of fifteen or twenty feet, causing the surface water to come in contact with the air and to become oxidized and to remain practically tasteless. If the lake or reservoir is more than twenty feet deep, the water in the lower regions is quiet and never comes to the surface except during the Spring and the Fall turnover. In the Spring the surface water begins to warm up. When it gets to be the same temperature as the water in the lower regions, the water in the lower regions mixes with the surface water and with the aid of the wind there is a general turnover or mixing of the water. In the Fall the surface water begins to cool, as it cools it becomes denser than the warm water and sinks to the bottom, displacing and forcing the warm water to the surface. As the warm water comes to the surface it brings the undesirable tastes and odors to the surface with it.

The distribution system depends all together upon local conditions. As has already been suggested, it is generally advisable to make use of any natural lake that happens to be within an economical piping distance from the town. It is especially desirable that the lake be so situated that the town may be furnished by gravity. If one part of the town is a great deal higher than the remainder of the town it would probably be advisable to have a special storage reservoir and a separate piping system for the high part of the town. This will prevent the low part of town from being under excessive pressure when furnishing the high part of town with water. If a natural lake is not available then a storage reservoir must built. In selecting a sight for the storage reservoir it should be borne in mind that the reservoir must be placed at a height which will give the maximum desirable pressure. If it is placed higher than this, the excessive pressure and water-hammer may burst the main, and the expense of pumping the water to the reservoir will have been increased because of the increased head which the pump will have been pumping against. If the distribution lines are very long and a very large pipe is required to deliver the water, then it may be found more economical to install an equalizing reservoir with a smaller size pipe line.

APPENDIX II

Values of Kd with respect to the velocity, in feet per second.

Velocity	3.16	5	10	15	20	40	60
Values of Kd	1.00	0.96	0.91	0.88	0.86	0.81	0.79

Valves of Ke for certain shaped entrances:

For a bell-mouth entrance _ _ _ Ke = 0.04
For a conical entrance _ _ _ Ke = 0.18
For a non-projecting pipe _ _ _ Ke = 0.47 to 0.56
For a projecting pipe _ _ _ Ke = 0.62 to 0.93

Valve commonly taken for bell-mouth _ _ Ke = 0
Valve commonly taken for non-projecting _ Ke = 0.5
Valve commonly taken for projecting pipe Ke = 1.0

From pages 127 and 129 of Daugherty's "Hydraulics."

APPROVAL.

The foregoing thesis is hereby approved as a creditable study of an engineering subject, carried out and presented in a manner sufficiently satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is to be understood that by this approval the undersigned does not necessarily endorse or approve any statement made, opinions expressed, or conclusions drawn therein, but approves the thesis only for the purpose for which it is submitted.

Name J. A. Switzer

Title Prof. Hyd & San. Eng.

DEPARTMENT OF THE INTERIOR
JOHN BARTON PAYNE, SECRETARY
U.S. GEOLOGICAL SURVEY
GEORGE OTIS SMITH, DIRECTOR

TOPOGRAPHY

TENNESSEE
MAYNARDVILLE

Henry Gannett, Chief Topographer.
Gilbert Thompson, Chief Geographer.
Triangulation by S.S.Gannett.
Topography by F.M.Pearson and C.G.Van Hook.
Surveyed in 1884-85 and 1890.

Scale $\frac{1}{125000}$

Contour interval 100 feet.
Datum is mean sea level.

Edition of Oct. 1901, reprinted Oct. 1915

KNOXVILLE


MAP OF
FOUNTAIN CITY
PROPERTY OF
THE FOUNTAIN CITY COMPANY
SCALE 1"=200
APRIL 1907
Revised by Thomas & Jones
Civil Engineers
Tracing by Sehorn & Kennedy - June - 1920
EIGHTH AVE
CHEROKEE AVE
SEVENTH AVE
FOURTH AVE
SECOND AVE
UNION SQUARE
FRANKLIN SQUARE
BELMONT SQUARE
MARIAH AVE
BROADWAY
AVENUE
Telephone Office
MARKED GUM

MAP OF
FOUNTAIN CITY
SOUTH WEST SECTION
SCALE 1" = 200'
TRACING BY H.I Scheitlin - APRIL 1926
MAGNETIC NORTH
MAR. 1913
N
E
S
W
Creek and branches
8" Pipe
6" Pipe
4" Pipe
Fire Hydrant
4 Way Branch
3 Way Branch
Gate Valve
CEDAR AVENUE
FAIR AVENUE
CHARLES AVENUE
MAPLE AVENUE
HIGHLAND AVENUE
MAPLE AVE
HIGHLAND AVENUE
WOODROW AVENUE
RANDOLPH ST
BROADWAY
SMITHWOOD STATION
VALLEY ROAD
FRANKLIN SQUARE
SECOND AVE
CONNER AVE
HUMPHREY AVE
J. H. Williams


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BOULEVARD
HICKORY AVE.
CONNER AVE.
JACKSBORO PIKE
LAND ADDITION
Church
SMITHWOOD
School
TAZEWELL PIKE
2209