

THESIS

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GRAVITY WATER SUPPLY FOR KNOXVILLE

FROM THE GREAT SMOKY MOUNTAINS.

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Submitted

for

a

Degree

of

Bachelor of Science

in

MECHANICAL ENGINEERING

AND

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by

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INTRODUCTION.

At the present time there are plans under way to attempt locate the government armour plate plants neare Knoxville Tenn. In the event that the plant does locate here the site will be about four miles above the present intake of the city water works pumping station. The waste products of the plant will then make it necessary to move the intake.

It then becomes a question whether it will be cheaper to move the intake up the river above the proposed site or whether a mountain supply may be obtained cheaper.

It is not the object of this discussion to determine which of the above propositions will be the cheaper but to select the most favorable mountain supply:to design the works complete to a connection to the present city mains:to estimate the cost of such a plant and the probable operating and maintainance ~~ex~~expense.

THE GENERAL REQUIREMENTS OF THE SYSTEM.

It is of the greatest importance that a city be supplied with pure, wholesome water. It is also essential that there be an abundant water supply. In order that a city may develop to the limit of its resources, it must have a water supply large enough to furnish water to whatever industries that region is capable of developing and also to supply the requirements of the inhabitants which such a condition and enterprises naturally attract to the community.

It may be well to enumerate some of the chief requirements of a public water supply and apply the test to determine the sufficiency of the stream to be considered as a suitable source of supply.

The water must be free from mineral salts that might prove poisonous. The mountain here considered flows through a region that is free of any harmful mineral formations and as the water is now used continuously for domestic purposes without bad results this condition is fulfilled.

The water must be free from disease germs. As the river basin is to be used for a watershed all inhabitants may be taken off this basin and in this way all chance of disease germs entering into the water may be eliminated. However, with the few inhabitants that at the present time occupy the basin there would be very little danger from this source.

Furthermore the water has the advantage of being in every way a satisfactory supply. The rapids in the rough stream beds add an aeration that no artificial means can duplicate. To these advantages may be added the consider-

ation that the water supply comes from a mountain stream. This of itself offers a mental satisfaction and security to the consumer and those that may be entering as visitors, into the city supplied.

The works for delivering a continuous supply to the city consist essentially of three parts. First; the impounding reservoir which is located, near to or on, the stream which is the source of supply. The reservoir may serve two functions; one, the storage of water to equalize the flow of the stream and give maximum discharge; the other, as a purification reservoir. As the stream is very pure in its original condition the primary object of the reservoir is for storage.

This reservoir has as its essential parts a dam either for diverting the water, or for closing some natural gorge, thereby forming a lake. Second, an intake where the water may be taken from different levels and permitted to discharge into the conduit which is to convey the water to the distribution reservoirs.

The third factor in the works is the conduit, which is to convey the water from the distribution reservoirs. The conduit may be made of various materials such as steel, cast iron, wood and etc. It must be of such size so as to deliver the required amount of water to the distribution reservoirs with the available head.

At intervals of ten or fifteen miles along this conduit, and where it approaches very closely the hydraulic gradient, there should be placed intermediate reservoirs. These reservoirs should be of about 10,000,000 gallons capacity for an automatic control of the system. These reservoirs are to act as equalizing basins the purpose of

which is to take care of the fluctuations in flow along the pipe line.

The conduit should be equipped both with stop valves and air valves. Each dip in the line should contain a blow off valve, in order that the line may be drained. The air valves are to prevent the occurrence of vacuua, and the stop valves are to cut-off the flow off the certain sections of the pipe line. The conduit should terminate at some distribution center, whence the water may be distributed to the different reservoirs and thence to the mains.

The distribution system may be more or less elaborate. there may be one or more reservoirs as the requirements and the topography of the city may demand. If it be deemed necessary that the water be sterilized, equipment for this purpose could be placed on the conduit as it enters the city. It would be of advantage to have located at this point the remote control of the system, also such offices and laboratories as may be deemed necessary.

The distribution reservoirs should be located at such elevations, that the highest points in the city may be supplied with water under the required pressure. These reservoirs will take care of the hourly fluctuations and fire demand. They should be of such size as to supply the city with water pending the worst possible break repair in the conduit, which might reasonably be expected to occur. This requires a storage of from five to ten days' supply.

These reservoirs should be connected to the supply system of the city, and they should feed directly into the trunk lines of the distribution system. Proper arrangement should be made so that these reservoirs may be cleaned

at required intervals without endangering a continuous supply.

The distribution system from the reservoir to the consumer may be designed in several different ways. In designing the distribution system, the fact must be kept in mind throughout that the greatest demand for water comes at the time of fire. Calculations should be made in designing this system so that at no time the velocity exceeds the maximum allowable. Valves should be so arranged that any desired section may be cut out for repairs without inconveniencing too many patrons.

Above all things, the system should be so designed throughout that, whatever may occur, there will be an uninterrupted supply both for ordinary consumption and for fire protection.

DEMAND FOR WATER.

Population Study.

The quantity of water required by any city depends upon the population. The consumption however may be influenced by the proper installation of meters. Population being the important factor, it is essential that we study the growth of the city in order to obtain some definite information as to what the requirements will be in the future.

By a study of the reports of the various "Water Commissions we find that all distribution systems and distribution reservoirs are calculated on a basis of approximately twenty-five years and that all sources must be capable of supplying the demand for a period of about fifty years. Similar conditions prevail in the case of Knoxville, for it would not be profitable to construct conduits for a demand of less than twenty-five years or select a source of supply that would not meet the demands for fifty years or more.

From these considerations it may be seen that the important problems are centered about the population at the end of twenty-five and fifty years. There are several methods for arriving at estimates of future population, three of the most important being as follows.

By comparison of percentage increase of population of different cities with the city under consideration.

By determination of a constant ratio factor of increase or the percent increase of the city for one year.

By a population study of other cities.

Each of the foregoing methods will be discussed in the order as they are tabulated. A detail explanation will be given of the method and how applied to the problem under consideration.

An analysis by the first method is shown on Plate I. Platted on this plate is the percentage increase of population of fourteen different cities of the United States, with the percentage increase of Knoxville to the present and the assumed increase for fifty years hence.

In the center of the plate will be found a heavy division line. The cities to the left represent the growth of cities to a population of 90,000, present population of Knoxville, and cities to the right a growth from 90,000 upward for a period of fifty years. Summarized, we have a chart of the percentage increase of cities from a period fifty years before they reached a population of 90,000, to a period of fifty years after they attained this population. The heavy line presents the conditions in Knoxville as they existed to the period of 90,000 population and as they are assumed to exist for a period of fifty years hence.

The portion of the curve to the 90,000 population mark was calculated from the U.S. census, which was as follows:

Year	Population	Gain in Population	Percentage Increase
1850	4692		
1860	9465	4836	103.0
1870	19625	10156	107.3
1880	21906	2285	11.6
1890	50929	29023	132.0
1900	73760	22831	44.8
1910	82142	8382	11.4

That part of the curve which lies beyond the 90,000 population mark was constructed after a study of the past conditions of Knoxville with reference to the other cities platted and the general resemblance of percentage increase curves during the given period of growth. At a glance it seems as if the curves are very irregular and follow no de-

finite law, but it will be shown that Knoxville and other cities follow some, but not a well defined law.

Consider first the cities for a period of fifty years before the 90,000 mark as shown by the curves to the left of the heavy line. They are very irregular with a general tendency toward a higher increase in percentage as they approach the mark. Nothing definite can be said of the curves as a whole.

Consider now the curves to the right, from the 90,000 mark, increasing for fifty years. We find a marked similarity. From this mark there is an increase in the percentage increase and the curves tend to concentrate between fifteen and sixty-five percent. The period starts with a high value which gradually decreases and more nearly approaches a straight line.

With the above facts well in mind study the ages of the different cities. In the left group we find cities comparatively young. They have not matured and are therefore fluctuant in their growth. Of these cities Knoxville is one of the elder. In the right hand group we find matured cities, past an age upon which Knoxville is now entering. From the above data this conclusion may be derived; as a city grows older the percentage increase in population becomes less and approaches a more uniform rate.

The assumed curve obtained from this foregoing data. By a study of the table of Knoxville's growth, given on page 8, it may be estimated that the percentage increase will not be more than twenty percent, unless something very unusual happens in the three years remaining before the year 1920. This is not very likely as anything of such a magnitude would be under consideration for several years before it

would be executed. Considering this point established let us turn to the curves. Note the peak twenty years before the 90,000 mark in the city of Knoxville. Now note the peaks of the other cities, they fall on the heavy line or about the 90,000 population period. This fact coupled with the knowledge that Knoxville is the oldest city in the group, tend to show that She has reached, twenty years before reaching the 90,000 population mark, the peak percentage increase. This above assumption being true, which seems very evident from the data as we have it, we would produce our curve keeping a constant distance below the mean. However in order not to possibly underestimate the value we will assume that the curve follows the mean. The first point beyond the heavy line is determined from the percentage increase assumed at 1920 and the points for the remaining forty years follow the mean.

An analysis by the second or constant ratio method is purely mathematical in nature and consists in applying a determined yearly percentage increase to a geometric progression.

To determine the fixed ratio of increase or the constant of increase we will study the population of Knoxville since 1910. This is as follows:

Year	Population	Gain	Percent Gain
1910	82288		
1911	82800	512	0.61
1912	84200	1420	1.72
1913	88467	4267	5.08
1914	91498	3031	3.44
1915	92703	1205	1.32
1916	93703	1000	1.07
		Mean	2.23

This would indicate a probable increase of 22.3% per ten years as a minimum. Consider also a ratio of 4.00 %

nearly twice as much, which would approach very nearly a maximum or an increase of 40% every ten years.

These two percentages will, upon investigation, be found to apply very closely to the cities shown on Plate I.

Now apply these ratios to the equation:

$$\log b - \log a = (N-1) \log r$$

b population at the future date.

a population at the present date.

N number of years that intervene between.

r ratio of increase (percentage plus one)

Ratio	Year	Population
1.0223	1970	303100
1.0400	1970	832200
	Mean	567700 population in 1970.

The above method depends absolutely upon the proper choice of a value for r.

The third and last method seems to offer the most most valuable method of all, as it is a study of actual conditions as applied to other cities bearing in mind the definite relations to the city under consideration. Plate II shows the growth of the cities of Plate I as related to population increase. Knoxville is again shown by the heavy line. As before the cities to the left represent cities in a period fifty years before the 90,000 population mark and those to the right cities developed for a period fifty years past this mark. The population of Knoxville before is platted from actual data and that after from the condition met. The shaded area shows the probable maximum and minimum increase for the time considered as chosen by a

study of the cities that have grown beyond the 90,000 population mark, shown by the heavy line. There is no definite rule for drawing these lines, only a matter of choice as the data from the other cities may be interpreted by the drawer.

In order to see how the three methods compare, they have all been plotted on Plate II, each being labeled. The heavy dotted line shows the increase of population by the percentage method, the dot and dash line the constant ratio method. It may be noted that these three methods have approximately the same value at fifty-five years, the first and second method curves crossing on the mean of the maximum and minimum shaded area.

From the foregoing considerations assumed population may be taken as follows:

Years past the 90,000 population mark (heavy line)	Year Represented	Population
25	1945	210,000
50	1970	460,000

From the above estimated population will be calculated the probable demand for the years given and all works will be designed accordingly. To obtain the demand for any time the probable consumption per capita must be considered, so the next step will be the study of the per capita consumption for the years of which the population has been determined.

Per Capita Consumption.

The consumption for Knoxville as given by John A. and Ewd. S. Cole, in 1912, was 105 gallons per capita per day with 42% of the taps metered. There is no other information available as no records are kept by the city which owns the present plant. The above data is correct as to percent of

metered taps, but the per capita consumption was probably obtained from the pump register as there is no other means at the present plant for determining the amount of water going to the distribution mains. As a whole the 105 gallons per capita per day consumption is probably low.

Geo. I. Bailey in a very extended study of water consumption per capita of the cities of the United States reached the conclusion that 137 gallons per capita per day was the average consumption, for cities that do not use meters. There is a marked improvement if meters are used and the per capita consumption is decreased. The left half of Plate III shows a study of 150 cities of the United States with a population over 25,000.

From the study of the three curves, the maximum, the minimum and the average we find that it is true that with an increase of metered taps we find a decrease in the consumption per capita per day. On the same side is plotted analysis by two engineers as follows:

Analysis by Wm. S. Johnson M. Am. Soc. C. E.

Percent of Metered Taps	Cities over 25,000 Consumption	Cities Under 25,000
1 to 10	146	77
11 to 25	97	77
26 to 50	89	63
51 to 75	89	45
76 and over	72	48

Analysis by Geo. I. Bailey M. Am. Soc. C. E.

Percent of Metered taps	Consumption per capita per day
10 to 25	110
less than 10	153
25 to 50	104
more than 50	62

Each of these two analysis is plotted on the left half of plate III, and is marked. It may be seen that a close relation exists between each of these analysis and

the curves obtained by a study of the various cities. Johnson's curve, for cities under 25,000 population, follows with slight variation the minimum average of the six lowest cities; while his curve for the maximum consumption for cities over 25,000 and Bailey's do not vary much from the average.

The right portion of Plate III is a study of the influence of the increase of population on the per capita consumption under approximately the same meter conditions. Cities having from 100 to 80 percent of their taps metered were chosen, giving an average of 90 percent. The consumption of forty-five such cities was plotted against their population and a curve approximating the points drawn. The curve is not accurate enough to be used for obtaining the consumption but it shows that there is an increase in per capita consumption per day as the population increases, even if the same number of metered taps are used. That this factor should be taken into consideration in the design of a water supply is essential. It shows that even with a very accurate account, of the now existing conditions and consumptions, there would not be a basis upon which to calculate the consumption at some future date, the percentage of taps remaining a constant.

The percentage of taps to be metered depends upon a great many variables. The basis for a calculation is to install as many metered taps as possible without having the interest on the investment amounting to more than the value of the water saved. Some few cities have been able to satisfactorily install a meter for every tap and many cities have 90 percent of their taps metered. It is of economic

value for every city to meter as many taps as possible, it has even at times saved cities, where the consumption was high from installing a new plant.

The probable economic number of metered taps for Knoxville would be 90 percent. At the present 42 percent of metered taps there are indications of a very large per capita consumption. For a gravity supply which is carried over a long distance, the design and cost being proportional to the amount of water conveyed, it is essential that the consumption be reduced to a minimum.

The Demand.

Having determined the conditions of population growth and the probable water consumption we may determine the demand.

Year	Population	Probable consumption 90% of metered taps	Demand gallons
1945	210,000	85 ga. per day	¹⁷ 41 ,850,000
1970	466,000	100 ga. per day	46,600,000

This gives the actual demand made upon the works by the population and manufacturing establishments. To this must be added the amount lost by leakage and other sources which cannot be accounted for. This factor is very large considered per capita. Various authorities differ on the amount to allow, they vary from 15 to 30 gallons per capita there being few leaks at first and as the pipe settles and the leakage increases the loss may become as high as 30 gallons per capita per day. The average is approximately 23 gallons per capita per day. Apply this to the above problem and we find that, for at the end of twenty-five years:

Population	210,000
Consumption per capta	17,850,000 gallons
Loss due to leakage etc.	4,860,000 gallons
Demand	22,710,000 gallons
Year	1945

At the end of fifty years:

Population	466,000
Consumption per capta	46,600,000 gallons
Loss due to leakage etc.	10,720,000 gallons
Demand	57,320,000 gallons
Year	1970

Having the demand it is a simple matter to calculate the size of the source of supply which will be solved on the assumption that it takes 1.55 cu.ft. per sec. flow to give 1,000,000 gallons per day. This is the value used by the U.S. Geological survey in their water supply papers.

Year	Million Gallon Demand	Discharge in second feet.
1945	22.71	35.2 14.7
1970	57.32	37.0

Upon the above figures will be based the design of a gravity water supply for the city of Knoxville.

Proper Location of Impounding Reservoir.

In locating the impounding reservoir several things should be kept in mind. These may be summarized as follows:

First;that the basin must have such an elevation that the water may be delivered from the same to the distribution reservoirs by gravity,and at minimum cost for conduit. This requires that the elevation be as great as possible in order that pipe of relative small diameter may be used.

Second;the discharge of the stream must be ample to meet the demands at all times,for a period of fifty years to come. This does not necessarily mean that the minimum stream flow must be large enough to supply the demand,but that there must be storage provided to equalize the flow, and that the average flow shall fullfill the requirements.

Third;the water obtained must stand the test as to purity and wholesomeness,or be of such a nature that it lends itself readily to purification. Hence there should be but very little turbidity,and the watershed should be free from habitation. If there be any inhabitants on the watershed the greatest care should be taken with the disposal of the sewage.

Fourth;the topography of the water shed and of that portion of the country through which the conduit runs should have the following characteristics:one,the watershed should contain a favorable damsite,so located that ample storage may be obtained without to costly a dam;second, the topography of the route for the conduit should be such that a good grade for the conduit may be obtained,free from any excessive head.

Fifth: the cost, must be within reason. This includes the property damage, and land costs. The overall cost of the works, the difficulty of making the installation and the ease with which the material may be delivered to the location.

Keeping the above facts in mind we are now ready to select the watershed. This is readily accomplished by a study of the topographic maps of the U.S. Geological Survey. The sheets numbered 33, 25, 75, 16, 27 and the Mt. Guyot, were assembled in their proper relation. The topographic map, with Knoxville as a center, extends twenty-eight north, south and west, approximately fifty-six miles, east. A study of this map will locate the most available watershed for this city.

Such a study finds the desirable site on the upper Little River region. This region contains several promising catchment basins marked on the map by green lines. A comparison of these basins will show the one which is the most favorable.

The basin selected is on the East Prong of Little River, with a damsite about one fourth of a mile down stream from Shield's Branch. A map of the watershed is shown on plate V. The advantages offered by this basin are as follows:

The dam will be located, by air line, only twenty-eight miles from the city. It will give a fall of about 1000 feet and a water supply, as estimated from the mean flow, of four or five times the probable demand for at least fifty years to come.

Located in the heart of the Great Smoky Mountains, the basin is practically free, except in summer, from habitation there being only a few logging camps located on the watershed. In summer two summer resorts are open in the foothills, and some adjustment would have to be made for these.

The water is clear and becomes turbid only after very violent storms, the turbidity lasting only a few hours.

The damsite selected is one where a 100 foot dam could easily be built affording the proper amount of storage. At the same time the river valley offers an easy exit for the conduit.

The cost of the flow rights would not be prohibitive. The greater part of the basin has been timber cut, leaving land valued at seven or eight dollars per acre. There would be no objection to cutting the remainder of the timber, as the vines and bushes would hold the soil in place. The total basin is 45 square miles in area. A rail road runs past the damsite through the storage basin to the camps above. The presence of this rail road offers a problem not easily solved. They are two alternatives; first, the purchase of the whole basin as it stands, thereby removing the need of the rail road above the damsite; second, relocate the rail road for about five miles and follow the mountains above the proposed flow line. During the construction of the dam the rail road would be of service to deliver material.

Though this basin has several marked advantages, we should not fail to those of some of the other watersheds in this basin. A combination of the Laurel Creek and Middle Prong would offer a very interesting and possibly a very profitable study. The Middle Prong would be rather expensive, while on the other hand the Laurel Creek Prong would be very inexpensive, since the fires have burnt that side of the mountain and the whole watershed is barren. These two basins combine, have not the area of the East Prong watershed; therefore not a discharge sufficient for the fifty year demand. It would not be out of place to buy the Laurel Creek Prong, as the land at present is very cheap, and practic-

ally uninhabited. This watershed at some future date, when the yield of the water from the East Prong shall be insufficient, will be very usefull to aid in taking care of the demand cheaply.

The Knoxville topographic sheet (No.16) is the only one included in this report. This sheet is numbered Plate IV. The red line encloses the chosen water shed, the green lines indicate other sites. On the sheet the gravity works are shown in black.

When a stream is under consideration as a source of water supply, knowledge of the peculiarities of flow are essential. The minimum, maximum and total flow for various periods must be determined.

The most accurate and direct method used for determining these values is by direct measurements at the damsite, covering a relative long period of time. As such data is rarely available, either a station must be established or some other method must be adopted.

This method is by means of comparison. A very close estimate of the stream flow may be obtained by comparing the watershed with similar sheds, making allowances for differences in climate, rainfall and other characteristics. Where the differences are great results are only approximate, but on the other hand where the conditions are similar and the streams located close to each other, a fairly reliable basis for estimates is found.

There being no data available at the damsite or on the Little River, we must turn elsewhere for a basis of investigation. The Nottely River upon which have been made gaugings, will serve the purpose. A gauging station has been established at Ranger N.C., and records are obtainable from 1901 to 1905

The river is described by the Geological Survey as follows, "The bed of the stream is rough and rocky through out the entire course, and a section suitable for making discharge measurements could not be found. The drainage area is rough, mountainous and largely forest covered. The stream flows, for the most part in valleys which are very narrow and moderately steep." These words may be applied with equal accuracy to the East Prong above the damsite. The topography of the two streams are the same or very similar. The Nottely River is situated on the east slope of the Great Smokies,

while the East Prong is on the west, both however drain into the same river basin.

A study of the Weather Bureau's records shows the same temperature conditions, for the two basins, for the years 1901 to 1905. Plate VI shows a study of the rain fall on the different sides of the mountains. This study reveals the fact that the rainfall on the west slope of the range exceeds that on the east by about 0.6 inches a month, as a mean. The average of the rainfall for the period of five years was as follows:

	East	West
Average per month	3.21	3.815
Total precipitation for five years	192.65	228.91

as measured in inches .

Having thus determined the comparative conditions on the two watersheds, a comparison of areas of the two watersheds may be made in order to determine the proper ratio for rendering readings on the Nottely watershed applicable to the East Prong of Little River. The area above the gauging station at Ranger N.C. is 270 square miles, and by planimeter determination the area above the damsite on the East Prong is 45 square miles. This at once fixes a ratio of $1/6$, a value by which observations at Ranger must be multiplied to give corresponding values on the East Prong at the damsite.

In taking this ~~are~~ ^{area} the assumption is being made, that the runoff for each square mile is the same on each basin. As shown above the rainfall on the East Prong Watershed is the heavier; and inasmuch as runoff, all other things equal, is directly proportional to the rainfall, in assuming the same runoff per square mile we are erring on the side of safety. However when considering maximum flow, this fact

into account and add a certain percent on account of the differences in rainfall.

From the above considerations and the runoff data for the Nottely River, at Ranger, prepare the comparative hydrograph, Plate VII. In this curve the runoff for each six consecutive days is averaged, and platted as an ordinate, the coördinates of the curve being total discharge in second feet and time, in units of six days. Within the period, of 1901 to 1905, occurred both the highest and lowest water years for at least two decades. The year 1904, being the lowest since 1871 and 1901 the highest in 23 years. This period therefore gives the extremes for which our works should be constructed.

- A study of Plate VII shows that the stream of itself will adequately supply the city for about nine months out of the year without storage; but a storage of approximately 2000 acre feet would be required for the balance of the year to give the required 30 second feet discharge at the end of fifty years. Other data deduced from this study:

Mean discharge per square mile	1901	2.89 sec.ft.
	1902	1.87
	1903	2.16
	1904	1.14
	1905	1.61
	Mean	2.334 $\times 75 = 105$
Mean discharge in second feet (East Prong)		105
Minimum discharge in second feet (East Prong)		17
Maximum discharge in second feet (East Prong)		945
Max. discharge corrected for heavier rainfall on west (cor, 10%)		1040
Discharge of the six highest water months		150 sec.ft.
Possible discharge that could be economically used (given as 43% of the average)		46 sec.ft.

IMPOUNDING RESERVOIR.

The impounding reservoir, situated on the East Prong of the Little River, extends from a point 960 feet below Sheild's branch to a point 3000 feet above Laurel Branch. It lies in between a series of foot hills that run parallel to the stream.

Pictures 1,2,3 and 4 on Plate VIII give a general idea of the topography of the reservoir. These pictures, taken in connection with the map Plate IX, will be used in giving a discussion of the site.

Starting with picture 1; the hill in the foreground is a natural dam to the stream turning its course from east and west, to north and south. This hill forms one of the walls of the dam, which spans from the hill from which the picture was taken to the one in the foreground. About 1600 feet upstream may be seen the mouth of Shield's Branch. Up to this point steep hills rise on either side of the stream. At the mouth of the branch the valley suddenly opens into a flat through which Shield's Branch flows. Water will be backed up in this flat to a depth of about fifty feet over the farm house in the back ground. This is only property of much value in the impounding reservoir site. Water will be backed up Shield's Branch approximately 1200 feet.

The flat gradually narrows, going up stream, until it again flows between two steep hills. Beyond the hill, as shown back of the farm house, the stream runs through another flat. This flat is marked with a green cross in picture No.1 and is clearly shown in picture No.2. Water will stand approximately forty feet deep over this flat.

Around the bend as marked by the cross on picture No.2 is the mouth of Laurel Branch. Picture No.3 shows the hollow formed by this branch as marked by the cross. Water will be backed up this hollow about 800 feet and will stand at a depth of about thirty feet. From this point the valley begins to narrow and the rest of the reservoir lies between comparative deep slopes. The rise of the stream from this point is also very rapid there being rapids and swift water throughout the rest of the course, which lies in the reservoir site.

Picture No.4 takes up the basin from the point where the third picture leaves off and takes in the upper portion of the stream. The headwater of the reservoir lies just below the point marked on the picture. The hill on the right is very steep and heavily wooded, while the one on the left has a more moderate slope.

The rail road as seen in the four pictures is the Little River R.R. which will be submerged for approximately 6000 feet, under about 73 foot of water at the damsite. This railroad would have to be relocated, and in order to obtain this difference in elevation, the relocation would have to extend several miles below the site.

Except for two flats as shown the valley is uncultivated. For the most part it is covered blackberry bushes and other shrubbery. The foot hills have been cut of their timber, and all that remains is undergrowth, fallen trees, old logs and a very meager second growth of trees.

Over all, the reservoir will extend up stream about 7000 feet. The maximum depth of water will be 85 feet when water is even with the spillway and not more than 86.5 feet in the highest floods. The reservoir line includes

between its boundaries approximately 60 acres, and the impounding reservoir having a capacity of 2000 acre feet, enough water to take care of the deficiency of the lowest water year. At the rate of maximum demand it will take water twenty-eight days to flow through the reservoir or there is a twenty-eight day supply for the city in the reservoir if water did not enter into the same from any other source. Or with the flow that did exist in the lowest water year a continuous discharge of thirty-seven second feet for three months may be obtained.

STORAGE.

There are two methods of studying storage; from the hydrograph of the river, and by the Rippl method.

The required storage may be found by the first method from plate VII. Take that portion of the hydrograph where the discharge in second feet falls below the required discharge. This discharge is thirty-seven second feet. Draw the thirty-seven second foot ordinate, intersecting the hydrograph at the two points on each side of the interval of time that the discharge is below the required. Find the mean ordinate of this area between the curve and the thirty-seven second foot ordinate. From the time, found by measuring the length of the ordinate intercepted between the two points of the hydrograph to the scale used in platting the same, and the mean ordinate we can calculate the equivalent storage.

The storage desired will be the deficiency in the lowest water year. The application of this method, to Plate VII, gives the following:

Mean ordinate	13	second feet.	Time	87	days.
One second foot equals	1.983	acre feet	per	day.	
13 second feet	25.779	"	"	"	"
Then for 87 days storage will be	2143	second feet.			

From the above data it will be sufficient if we obtain a storage of approximately 2000 acre feet. This will give the extreme condition a continuous discharge of thirty-seven second feet.

By the second method the Rippl diagram. This method has the advantage of giving the extreme conditions during the period under consideration.

The method is as follows; from the measured or estimate flow of the stream for each month during the period considered, subtract the monthly loss. The result will be the net

yield for each month. Add together the yields from the beginning to each month in succession; then from these figures construct a curve as shown on plate XV, in which the abscissa of any point is the total time from the beginning of the selected period, and the ordinate is the total net flow during the time represented by the abscissa. The inclination of the curve at any point is thus equal to the rate of the net flow. A second curve may be drawn to represent the consumption, this will be a straight line as the small variation will not effect the flow, as a whole. The line OK represents the 37 second foot discharge, or consumption curve. An ordinate between the lines OK and AB will represent a surplus from the beginning at the 37 second foot rate of consumption. The greatest deficiency occurring during the dry period AB is found by drawing AB parallel to OK and tangent to the curve; and the amount is given by the maximum ordinate 12. This ordinate represents 2000 acre feet and corresponds with the value determined. The deficiencies for other dry periods are found in a similar manner, as for CD. It is found that the greatest deficiency is 2000 acre feet, therefore this is the storage required.

From this curve may be determined the discharge that could be obtained with proper storage. Trying at the different points on the curve, draw tangents until the one with the maximum slope is obtained which will intersect the curve again when drawn from any point on the curve. This line is GC and the consumption curve which it represents is OM, or a continuous discharge of 48.6 second feet. The storage required would be shown by the ordinate ⁵⁶56, which indicates ~~12,000~~ ~~20,000~~ acre feet.

Having determined the amount of storage needed it is necessary to determine the proper height of dam to give the

to give the required storage. This value is determined from the contour map Plate IX. The contours are at 10 foot intervals. To obtain the storage between any two contour intervals, find the area bounded by each contour line, take the mean of the two and multiply by the interval distance. This is accurate where the side slopes are steep, as would be the case in the basin under consideration. Determining the areas by means of a planimeter, the results are as follow:

Contour	Area in Acres	Mean Area Acres	Depth Feet	Storage acre feet
1970-60	1.83	0.92	10	9.20
1980-70	3.32	2.58	10	25.80
1990-80	9.25	6.34	10	63.40
2000-1990	18.53	13.89	10	138.90
2010-00	27.09	22.81	10	228.10
2020-10	33.21	30.15	10	301.50
2030-20	42.37	37.80	10	378.00
2040-30	55.44	47.96	10	479.60
2050-40	59.77	56.11	10	561.10
2045-40	56.61	54.53	5	272.65

Do not consider water available for drawdown below the 1990 contour. This represents 98.40 acre feet that are not available. If the spillway of the dam be such as to have an elevation of 2045 above sea level the storage will be 2000 acre feet. The evaporation from this body of water represents 80 acre feet during the dry season and no allowance has been made for this because there will be a flow from the ground waters, as the basin is surrounded by high mountains, which will more than balance the loss.

Besides the storage obtained at the impounding reservoir there will be 20,000,000 ~~xxxx~~ gallons stored in the equalizing reservoirs, 28,000,000 gallons in Luttrell Hill reservoir and 100,000,000 gallons in Sharpe's Ridge reservoir. The storage in the impounding reservoir amounts to

651,700,000 gallons. The total storage available is therefore 799,700,000 gallons. Twenty-five years hence this storage would furnish the city for a period of thirty-five days, if no water came from any other source. 19.6

THE DAM.

An idea of the damsite may be obtained from the topographical map and Profile Plate X, when studied with the pictures 1, 5, 6, 7 and 8. The damsite is located 960 feet from the mouth of Sheild's branch.

Picture No. 5 shows the west wall of the dam. This hill is rather steep and the rock formation is very often exposed to few. The summit of the hill is over two-hundred feet above the river bed. Pictures 6 and 7 show this hill marked with a green cross. The east bank is not as steep and offers one of the greatest objections to the site. The railroad is located on this side of the river. Picture 1 shows the topography of this hill the best, however it seems more flat than it really is, due to the fact that the picture was taken from a point far above it. Its summit is one-hundred feet above the stream bed. This hill is marked with a black cross on pictures 6 and 7.

Picture 6 and 7 give a general idea of the damsite as a whole, both these views are looking downstream, from a hill back of the damsite. The red cross locates the rock formation of picture 8. The strata is tilted upstream there by giving the desired water tightness required by the walls between which the dam is constructed. This formation would require the water to flow against gravity upward in order to follow the strata.

The dam will have its crest as shown by the dotted line on picture 7. The spillway will be located in the center, and the intake in the center of the spillway.

The general demensions of the dam will be as follows;

Length of crest of dam	526 feet
Elevation of crest above mean sea level	2053 feet
Spillway(overall length)	140 feet

Taken out for intake	20 feet.
Actual length of spillway	120 feet.
Portion of dam east of spillway	226 feet long.
Portion of the dam west of spillway	160 feet long.
Elevation of spillway above mean sea level	2045 feet.
The spillway will be 85 feet above river bed.	
The crest of the dam will be 93 feet above river bed.	

The Profile.

Designed by Wegmann's method of determining a profile. This method consists of determining at successive horizontal sections, beginning at the top, the necessary width of section to fulfill the condition of stability, assuming the area enclosed between successive sections to be trapezoidal. The degree of exactness required is obtained by choosing the sections sufficiently close.

For a dam 100 feet in height, there are three stages to be considered. First there is a certain top width, which when determined establishes the upper rectangular portion. This portion is of such a depth so as to bring the resultant pressure, with reservoir full, to the outer edge of the middle third, of the section. The expression for this condition is,

$$h = a\sqrt{g}$$

h the depth of section
a the width of the top
g the specific gravity of the masonry.

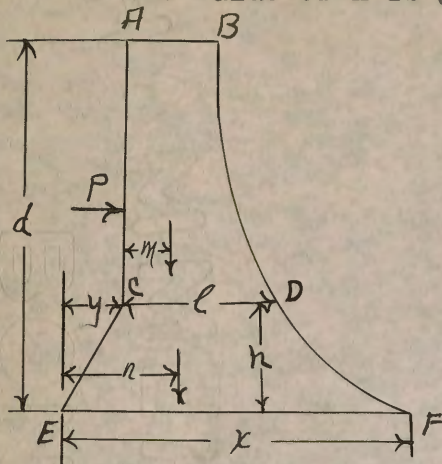
Second there is the section where the downstream face only, is battered and the upstream face continues vertical. This section continues until the resultant pressure, with reservoir empty, cuts the inner edge of the middle third. The expression for this condition is,

$$B = \frac{d^3}{gh} + \frac{6Am}{h} + 12$$

$$x = \sqrt{B + C^2} - C$$

$$C = \frac{1}{2} \left(\frac{4A}{h} + 1 \right)$$

The value of n is given by the equation,



$$n = \frac{(x^2 + lx + l^2) \frac{h}{6} + Am}{A + A'}$$

d depth of water at the section considered.

g specific gravity of masonry.

h depth of section taken.

A area of the total section above

the section considered.

m The line of action about the toe for the total section above considered section,

n The line of action for the total design

l Length of the last sectional width found.

A' Area of the section under consider-

ation.

x value of the width of section

considered.

Third, for this portion the face will receive a slight

batter. The resultant pressure must be such that the inner

third will be cut with the reservoir empty. An expression

for this condition is,

$$x = \sqrt{B + C^2} - C$$

$$B = \frac{d^3}{gh}$$

$$C = \left(\frac{1}{2} + \frac{A}{h} \right)$$

$$y = \frac{2A(x - 3m) - hl^2}{6A + h(2l + x)}$$

$$n = x/3$$

y is the amount of batter on the upstream face.

With these equations we may begin to design the dam profile. First however add two empirical equation to those obtained.

$$a = 4.0 + 0.07 H$$

$$y_0 = 1.8 + 0.05 H$$

a width of top.

H greatest depth of water.

y_0 height of crest above water line.

In order to determine H , the greatest depth of water on the spillway, must be determined. For the required height of spillway we must have 85 feet above the bed of the river to obtain the storage. Under runoff we find that the maximum discharge will be approximately 1040 second feet. How high will water stand on the Spillway ?

The discharge over a weir is expressed by the formula,

$$Q = c h l^{\frac{3}{2}}$$

Q the quantity of water is second feet.
 h the height of water on the weir.
 c a coefficient for different types of weirs.
 l the length of the weir.

The spillway will be of the parabolic type and tables for c may be found for weirs that have been calibrated. Results for l were found as follows for different assumed heights of water on the spillway.

h	c	Q	l
1.4	4.56	1040	138 feet approx.
1.5	4.58	1040	124 feet approx.

Make the spillway 140 feet long with twenty feet out for the intake, 129 feet remaining. As the intake will not have the effect of blocking the water completely we will consider the maximum head as 1.5 feet at maximum flow.

With this value, H is determined as 85 plus 1.5 which gives 86.5 feet. Substituting this value in the equation we have,

$$\begin{aligned} a &= 10.055 \text{ feet breath of top.} \\ y_0 &= 6.125 \text{ feet above high water.} \end{aligned}$$

and from these calculations assume a to be 10 feet and from the value of y_0 , which when added to H gives the crest as 92.6 feet above the bed of the stream. Make this 93 feet in order to have an even number of feet for the height of the dam.

From the above value of a, substituted in the equation for the first stage, the depth must be 15.81 in depth. Assume the value of g to be 2.5 corresponding to a mixture with a weight of 156 pounds per cubic foot.

Having determined this first stage, divide the dam into sections for design. Consider the rest of the first 33 feet as the first section. This gives 33-15.81 or 17.19 feet for

the first stage, section, and the rest will be divided into sections of twenty foot each.

Having these divisions and applying the different stages as reached, the results obtained are as tabulated.

Section	Distance Down	Width of section	y
15.81	15.81	10	
17.19	33.00	19.47	
20.00	53.00	32.18	1.78
20.00	73.00	45.60	3.21
20.00	93.00	58.56	3.38

These values agree in the third place, as checked, with Wegmann's standard profile, for a dam having a top width of 10 feet. A profile of the dam is shown in detail on Plate XI.

The Spillway.

The object of a curved form is to guide the water smoothly over the dam and at the bottom turn it tangentially to the stream bed. This turning takes care of the greater part of the energy contained in the falling water and decreases the effect of scouring of the river bed. If the volume where there would have to be some other means of taking care of this energy.

The profile in section is designed similar to the profile of the dam proper. The highest water on the crest will be, at maximum flow 1.5 feet, therefore the reduction, due to curving the top does not effect the strength of the spillway and no shocks have to be withstood as very little timber of any size would ever pass over the dam.

The convex top curve to be given to the dam, should be of such a nature, as to prevent the water from leaving the surface. This will give a parabolic curve, which the water would take if it were allowed to fall freely with a horizontal velocity corresponding to the depth of water on the weir. The average velocity of the water would be,

$$v = c \frac{2}{3} \sqrt{2gh}$$

v the velocity.

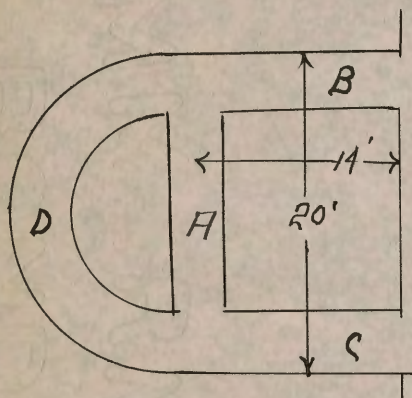
c coefficient as before determined.

h the head on the weir.

g the acceleration of gravity.

Consider different intervals of time t , and the abscissa of the curve will be, $x = vt$ and the ordinate $y = \frac{1}{2}gt^2$.

Having assumed several values for t plot the curve. Reduce the curve plotted to the scale of the drawing by means of proportional triangles. By scaling the reproduced curve we are able to determine the dimensions of the desired curve. Give the upstream side of the spillway a slight curvature, using the arc of a circle whose radius is 1.5 to 2 feet. The profile of the spillway is plotted with the profile of the dam on Plate XI.



The Intake.

The intake is to have the general shape as shown in the figure. The upstream face to be cylindrical, to offer as little resistance as possible to the flow of water. The intake is to be of such a size that the gate house may be

located upon it, and to provide a sufficient passage way for water.

The design of the intake may be divided into different portions. The two side walls B and C, these form beams fixed in the dam and resting on A. The pressure of this beam will be found to be,

$$P = wh = 62.5 \times 93 = 5812 \text{ lbs. per sq.ft.}$$

this is the lowest level or under a head of 93 feet. Consider a section one foot wide and apply the ordinary beam formula for concrete beams;

$$\text{Maximum moment} = \frac{1}{8} w l^2 \quad \text{or } 1708728 \text{ inch lbs.}$$

$$d = \sqrt{\frac{m}{12 K}}$$

$$A_s = 12 \text{ pd}$$

A_s steel area.
 d depth of slab.
 p a factor (1:2:4 mix) 0.005
 M maximum moment
 K a factor (1:2:4 mix) 71.5

the value obtained for d is 44.5" A_s 2.67 sq inches,
 This is the steel required per foot of width. Place in the
 slabs 5/8" rods on three inch centers, on both sides of the
 beam, to insure no outward collapse by drainage of the basin
 with the intake full.

Besides being under a beam action the beams C and B
 are under the action of the arch, on the upstream side this
 arch delivers to the compressive members Band C a load of,

$$\frac{\text{Pressure per square foot} \times 20 \text{ feet}}{2} = 58120 \text{ lbs.}$$

Assume the allowable stress in compression in concrete as
 500 pounds per square inch, an extra area of 116 square in-
 ches must be added. As the slab is 12 inches wide it must
 be increased by 10 inches in depth to take care of the ex-
 tra load.

The thickness of the reinforced concrete at the base
 will then be 44.5 plus 10 or 54.5 inches. Assume four foot
 six inches. The decrease of 0.5 of an inch will not effect
 the strength, for by adding the additional area for the com-
 pression, the strength of the beam has been increased. The
 section at the top is assumed two foot wide so as to with-
 stand the shocks imposed upon this portion. The sections
 were drawn so that they fell on a straight line drawn from
 the top of the intake to the bottom leaving the inside walls
 vertical. Several sections were investigated and it was
 found that they were of the required thickness.

The wall A acts as a compression member between the
 walls B and C. The load on each side is balanced by the
 hydrostatic head which exists on each side of the intake.

The wall will not have to bear the load on any one section a foot wide. Design for a load which would be distributed on one of these foot wide slabs under a 93 foot head.

d will be 13.5".

Make the thickness 18 inches throughout so that there will be no column action. An investigation at the base for crushing strength of the concrete shows that the limit is not exceeded. There should be openings placed in this wall so as to equalize the pressure as the intake fills up.

The arch is made the same thickness as the rest of the structure and the same steel added. This makes the design uniform and eliminates difficulties in construction. This arch must be tested for the steel required so as to make sure that the amount added was enough.

$$\begin{aligned} 2ts &= dR & s &= 16000 \text{ lbs. per sq. in.} \\ & & R &= 0.434 \times 93 \text{ or } 40.362 \\ & & d &= 20 \times 12 \text{ or } 240 \text{ inches} \end{aligned}$$

solve for $t = 3/40$ steel required. (thickness)

section 12 inches wide then $3/40 \times 12$ gives approximately 1 square inch.

as 2.67 square inches of steel have been added the construction is safe. The steel for the arch requires that only that part which is on one side of the slab be carried through and this is placed in the middle.

The openings in the intake tower and the pipe to carry the water to the pipe line, require consideration. As found in the section on runoff the possible economical amount of water that may be used will be 46 second feet.

Water will be taken from three stages; near the surface, half way to the drawdown point and at the draw down. The drawdown will be 30 feet above the bed of the river as discussed in storage. There may be another intake at the point just above the outlet pipe, which need only be used in the case of an emergency, but never otherwise, because of the

sanitary quality of the water.

The area of the openings will be calculated from the equation for the flow through a square orifice.

$$Q = ca\sqrt{2gh}$$

Q the quantity of water discharged

a area of the orifice.

c coefficient of discharge (approx. 0.61 for heads)

g the acceleration of gravity

h the head on the orifice.

The different openings are tabulated below and the required size of square orifice needed. Both the size calculated and the size to be used.

Depth of orifice below water	Area Calculated	Size square	Size to use
16 feet	3.31 sq. ft.	1.8	2 x 2
36 feet	1.59 sq. ft.	1.26	1.5 x 1.5
56 feet	1.27 sq. ft.	1.13	1.5 x 1.5

Emergency not calculated (made as others) 1.5 x 1.5

The outlet as obtained from a chart with reasonable loss due to friction, to take care of the ultimate discharge should be 36 inches in diameter. This value being calculated from the chart given for cast iron pipe.

A complete detail study was not made of the gate and gate control apparatus. The gates should be so located that the pressure will tend to make them water tight. Plate XII shows a complete drawing of the intake and its arrangement.

and
Plate XIII shows a plan/elevation of the dam and intake. As the dip of the rock strata, at the damsite is very steep, it is probable that in order to do trenching the foundation would have to be stepped, as shown in the drawing.

Foundations.

The dam can be built on rock foundation which can be found near the surface. The damsite is virtually a cliff of rock covered with a thin coating of decayed matter and loose soil.

The Wegmann method takes care of the stability of the dam and the crushing of the foundation of the dam proper. There still remains the question of gaurding against sliding on the foundation, and the crushing thereof. That the foundation is safe against failure by crushing, is evident from the fact that the stone is capable of sustaining a greater crushing stress than the concrete itself.

To gaurd against sliding on the foundation, we must compute the stability from the coefficient of friction and the weight of the structure. The total weight of the structure is 283820 pounds per linear foot, the coefficient of friction being assumed at 0.7, the total resisting force is 198678 pounds. The total force tending to move the mass when the reservoir is full is 278270 pounds. It therefore appears that the dam is unstable. In order to make the dam stable there are added two keys, each 5 feet deep and 10 feet wide. To shear either of these would require more force than the difference of the resisting and opposed forces.

This covers in general the requirements of the impounding reservoir dam.

THE CONDUIT

In order to locate the conduit, a study is made of the topographic map, Plate IV. East of the Chilhowee Mountains the conduit will necessarily follow the river, as this gives the most convenient route for an outlet to the impounding reservoir. West of this range the shortest course may be taken, thereby following very closely a straight line, for the most part through a comparatively level valley.

The conduit will be divided into three large sections. Section No. 1 will extend from the impounding reservoir at the damsite, to the equalizing reservoir No. 1, which is located at the mouth of the Middle Prong. This reservoir is to be of such an elevation that, when the time arrives, the water from Laurel Creek Prong is to be utilized, a conduit from the same may empty into it. The elevation of the water level will be 1500 feet. Assuming the water to be at drawdown head, we will have a fall in the conduit to this point of 500 feet. It is considered economical to utilize this head for the delivery of the water, and have the pipe line so arranged, that there will never be hydrostatic pressure on the line. With this arrangement we are able to place pipe used for low pressures, and at the same time utilize the high head. Another reason for having so large a fall in the first part of the line is, in order that we may not have an excessive head in the valley that must be crossed between this point and the gap in the Chilhowee Mountains. For the delivery of the demand for the ensuing 25 years, a pipe 18" in diameter will suffice; and to deliver the demand for 50 years a pipe 26 inches in diameter. The proper one to be used will be determined later by an analysis of conditions for the maximum economy.

Section No.2 extends from the mouth of the Middle Prong to the Gap of the Chilhowee Mountains where equalizing reservoir No.2 will be located. This section has a fall of 150 feet. The fall from reservoir No.1 to the reservoir on Sharpe's Ridge is equally divided, between this and reservoir No.2. This division has the advantage of freedom from excessive at any one of the dips, which must appear in the pipe line. With the head as taken, there must be a 28 inch pipe to deliver the 25 year demand and one 38 inches in diameter to deliver the fifty year demand.

Section No.3 extends from the Gap to the distribution reservoirs. The difference in head is 150 feet, and in this section will be found the greatest pressures on the pipe line. To deliver the 25 year demand the pipe must be 32 inches in diameter, and for the 50 year demand 46 inches in diameter.

The above diameters have been determined on an assumed length of pipe determined from the topographic map. The length of pipe assumed in section one was 8 miles, in section two was 12 miles and in section three 25 miles, which would give a total of 45 miles. All calculations will be carried on upon the basis that this assumption determines the profile of the line. When this is determined, and in connection with the plan drawn on the topographic map the actual length is determined, it must check with the assumed length; and if it fails, another value must be assumed and the same process followed until the proper diameters and location is found.

Determination of Size of the Conduit.

To determine whether the pipe for the fifty year or twenty-five year demand shall be used, the money invested must be considered. If the interest on the invested money invested

on the extra size pipe to provide for the fifty year demand exceeds, the cost difference that would be gained by an installation of one pipe line, the project is not economical. On the other hand if it does not, then it will be cheaper to install the one pipe to supply the demand for fifty years to come. This calculation will be carried on, on the basis of the average head in any section of the pipe line, as may be determined by a study of the topographic map. Cast iron pipe is considered at \$40.00 per ton the present price. The conditions calculated will answer for any form of pipe as the variation will be in some ratio, there being some variation but not considerable. Consider section No. 1,

Head available per 1000 ft. of pipe	12.8 feet
Velocity of the water in the pipe	8 feet/sec.
To deliver 25 year demand (diameter)	18 inches
To deliver 50 year demand (diameter)	26 inches,
Extra size to install at end of 25 yr.	22 inch dia.

Cost of 8 miles of 26 inch pipe	\$ 292000
" " " " " 18 " "	109000
Difference in cost.	<u>183,000</u>

Cost of 8 miles of 22 " pipe	220,000
------------------------------	---------

Gross gain (by present installation)	37000
Capital invested	183000

Interest at 6% would exceed	250,000
-----------------------------	---------

therefore the investment is not economical and pipe for the 25 year demand should be installed.

By the same method it is found that neither section 2 nor 3 should be built for a demand covering a period greater than 25 years. The diameter of pipe to use in each section will be,

Section	Present installation for 25 yr. demand		Future installation for 50 yr. demand.	
1	18 inch	8.2 ft/sec	22 inch	9.2 ft/sec.
2	28	3.7	32	4.2
3	32	3.3	38	2.6

The velocities in each of these pipes is large enough to prevent deposition of silt occurring in the conduit.

The Profile.

Having determined the sizes of pipe the profile of the line may be determined. This determination is only a differentiation between the cost of different locations. The question in most cases is whether it is cheaper to go through the valley or to follow a contour. To decide this question we must know the cost of a definite length of pipe under different heads. The following table gives the cost of a 100 foot length, under various heads that will occur in the line. The calculations are made for cast iron pipe, with the metal at \$40.00 per ton. All other pipeing will vary in some ratio though not a constant proportion. The profile would be approximately be the same whatever the material of construction.

Head	Cost of pipe, per 100 feet, in dollars diameters.		
	18 "	28"	32"
100	258.33	525.60	690.20
200	300.00	630.20	842.00
300	350.00	745.00	945.00
400		835.50	1125.60
500		962.40	1309.00
600		1100.20	1510.00

With this table we now turn to the topographic map and determine the cheapest location for the conduit. Having the cost of any two different routes compared by using the above table. We may also determine the point at which to cross a valley most economically. This having been determined the profile Plate XIV is platted. A summary of the same is as follows:-

Section No.1	Head on pipe	Miles of pipe used.
	100	7.4
Section No.2	100	0.1
	200	1.8

Section No.3	Head on Pipe	Miles of pipe used.
	300	9.1
	400	10.2
	500	4.6
	600	0.3
Total		26.1 miles

Section No.3		
	100	5.2
	200	3.2
	300	3.4
Total		12.0 miles

Summary of Conduit.

Section	Diameter of Pipe	Miles of pipe
1	18 inch	7.4
2	28 inch	12.0
3	32 inch	26.1
	Total	45.5 miles.

The distance finally determined from the profile and plan gives 45.5 miles and this compares favorable with the assumed distance. Therefore the profile may be accepted as representing an economical lay out.

The line will be equipped with valves as shown on the profile. Special provision is made that at no time will there be a hydrostatic head at any point on the line. Each dip is equipped with a blowoff valve and each peak with an air valve. Three way valves are located at different points on the line so that a portion of the line may be cut off for repairs without emptying the whole section. Stop valves are located at the outlet of each reservoir and a check valve at each inlet. The three way valves and blowoff valves lead to drains that empty into some nearby stream.

There are two equalizing reservoirs on the line each of 10,000,000 gallons capacity. These reservoirs will take care of the fluctuation in demand. Each is equipped with an over-

flow. Besides taking care of the fluctuations, they offer some storage which may be drawn upon in emergency.

The cost and maintenance will be taken up in another section.

Summary of Special Fixtures.

Section No.1.

- 7.4 miles of 18" pipe
- 4 air valves
- 1 three way valve
- 3 blow off valves
- 3 blow off specials

Section No.2.

- 12.0 miles of 28" pipe.
- 10 air valves
- 2 three way valves
- 10 blowoff valves
- 1 stop valve
- 10 blowoff specials

Section No.3.

- 21 air valves
- 2 three way valves
- 22 blowoff valves
- 1 stop valve
- 22 blowoff specials
- 26.1 miles of 32" pipe.

THE DISTRIBUTION SYSTEM.

The pipe line must terminate in one or more distribution reservoirs. There are two sites worth consideration in Knoxville. One is Luttrell Hill in south Knoxville, just south of the river bridge, and the other is Sharpe's Ridge situated directly across town to the north. The choice between them, if each of these would accommodate reservoirs having sufficient capacity for the demand, is a matter of the better location.

The elevation of the water level in the Sharpe's Ridge reservoir could be 1200 feet above mean sea level, while Luttrell Hill would admit of not more than 1150 feet. The first is such that the pressure in the lower parts of the city would be excessive, while the later would not furnish sufficient pressure for the higher parts of the city. A very good arrangement would be the use of both these reservoirs, each to supply the needs of the two zones in the city; this arrangement will be discussed later.

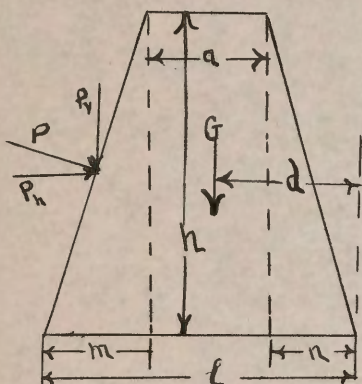
Let us consider the location of the several parts of the system. This is shown on Plate XVI. At the Water Works Building, a line may be turned up the hill to the Luttrell Hill reservoir; the building being shown in Plate XVII. At this building may also be located the sterilizing apparatus for the treatment of the water if deemed necessary. Remote control apparatus may also be installed. The Plate XVI shows the relative location of the present filter and pumping plants, and the proposed reservoirs. If using the Luttrell Hill reservoir, all the feeders would have to cross the river; and with the Sharpe's Ridge the conduit would be carried across.

To investigate the capacity of the Luttrell Reservoir is in order. The topography of the site is shown in Plate

XVIII, with the dotted line showing the proposed reservoir outline. The reservoir would have to be 20 feet in depth and a large amount of excavation must be done to secure the maximum for the site. This reservoir is divided into two equal sections for cleaning.

In order to determine the cheapest form of wall, both a gravity section and a reinforced section for a head of twenty feet, have been designed and compared.

The gravity section shown on Plate XIX was designed as follows:



The problem is to find such a value for l so that the resultant pressure will cut the middle third of the base.

$$P_h = wh^2/2;$$

$$P_v = wnh/2;$$

$$G = w \left(a + \frac{l}{2} \right) h$$

Dividing the moment of partial areas by total area about the toe D.

$$d = \frac{2/3 \quad n^2 + a^2 + 2an + 1/3 \quad m^2 + am + mn}{a + l}$$

Equating the moments of P_v , P_h and G about the outer middle edge.

$$P_h \cdot \frac{h}{3} - P_v \left(\frac{2}{3} l - \frac{m}{3} \right) - G \left(d - \frac{l}{3} \right) = 0$$

Substituting the forces above and substituting the forces above and the value for d we obtain an expression for l , m , n and with the expression below we can eliminate either m or n .

$$l = \frac{h^2 - (a+m)^2 (g-1) + n^2 g + \frac{n^2 g^2}{w}}{\frac{ng}{2}} - \frac{ng}{2}$$

If m where given,

$$l = \sqrt{A + B^2} - B$$

$$A = a^2 + 2am + \frac{h^2}{g} + \frac{m^2}{g}$$

$$B = \frac{m}{g} - \frac{m}{2} + \frac{a}{2}$$

l the bottom width
 a top width
 m batter upstream face
 n batter downstream face
 h height
 g specific gravity of masonry
 G weight
 P water pressure, subscript designates component
 w weight of unit volume of water
 w' weight of unit volume of masonry
 d moment arm about D, or toe.

From the empirical equations given in the design of the impounding reservoir, we find that the value for a will be 5.4 feet and for y₀, 2.8 feet. Therefore make the top width 3 5 feet 6 inches, and the distance of the top above water 3 feet.

Assume a batter on the inside of the reservoir of 2 feet so that it will aid in placing any water proofing that may be desired. Now having the value of a and m, l is found to be 13.5 feet. The results summarized;

a equals 5 feet 6 inches.

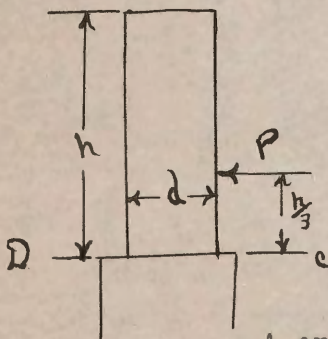
m equal to 2 feet.

n equal to 6 feet.

l equal to 13 feet 6 inches.

Plate XIX shows details of construction.

The reinforced section is designed by stages. The wall was considered a cantilever beam to the section considered, loaded by the total pressure at a point $2/3$ from the top. The section was designed with enough steel and concrete as a slab to withstand the stresses. Neglect the resistance due to weight of the structure.



$$\text{Max. } M = \frac{Ph}{3}$$

$$P = \frac{1}{2}wh^2$$

1:2:4 mix

$K = 71.5$

$p = 0.005$

$$\text{Max. } M. = 1/6(wh^3)$$

$$d = \sqrt{\frac{\text{Max. } M}{12 \cdot K}}$$

$$A_s = 12 \cdot pd$$

Practice.

A one to two inch covering of concrete over steel

Lateral spacing less than $1\frac{1}{2}$ times the diameter, center to center

Lateral spacing greater than $2\frac{1}{2}$ times the diameter, c. to c.

M moment at the section
 d thickness of the section
 h height of the section
 P water pressure on the section
 A_s area of the steel.
 K and p constants for different mixes of concrete.
 w weight of unit volume of water

The wall is divided into four sections and the design will be as follows:

Depth of Section	d	A _s	Spacing of Steel
5 feet	4.25	0.255	3/4" bars 6" centers
10 feet	12.08	0.72	3/4" bars 6" centers
15 feet	22.18	1.33	3/4" bars 4" centers
20 feet	34.14	2.05	3/4" bars 3" centers

The bottom section was made 3 feet wide and the very bottom is widened for anchorage. The 8" top is assumed so that the other sections will fall within^a/straight line connecting the points of the top and bottom. The complete profile in detail is given on Plate XX.

The foundation for the two alternative designs must be considered. The gravity section will of itself be stable. The section will be located on a clay foundation, which must support the weight and resist sliding. The weight for any foot width will be 39375 pounds. To carry this load extra foundation must be provided, inasmuch as this weight one ton per square foot. The coefficient of friction being assumed 0.33, the resisting force is 13125 pounds. The force tending to move the section is 13125 pounds, hence the section is stable. To make the section more secure, the base is given a slope in order to offer greater resistance to sliding.

The length of the reinforced concrete base depends on the stress to be resisted. The design is that of a slab for thickness, and of a beam of unknown length.

$$M = \frac{sI}{6}$$

$$I = \frac{bd^3}{6}$$

s pounds per square inch bearing stress of the clay.

b the depth of a one foot section
d the required length.

Solving, the length must be 19.4 feet, make the same 19 feet 6 inches.

By the formulae for a concrete slab we find that the slab must be 41.8 inches thick or approximately 42" and that the steel area must be 2.52 square inches.

Spacing as follows:

Five bars per foot width $3/4"$ bars $2\frac{1}{2}"$ centers.

Four bars per foot width $1/2"$ bars 3 " centers.

This gives the base the required moment arm for stability. The area of the base is the required amount to support the weight without exceeding, the allowable one ton per square foot. The investigation for stability against sliding shows that the force tending to cause the wall to slide is 13125 pounds. The resisting force is 18900×0.33 or 6300 pounds. The base is therefore strengthened by means of a key, two by four feet, to compensate the difference of force. Additional strength is obtained from the extended reinforcement and the slope on the back side. The stem and foundation are well tied together.

The division wall, Plate XXI, is designed on the same principle as the reinforced concrete wall. The wall is equipped with two gates for connecting the two halves of the reservoir when they are operating, and permitting free circulation. The gate chambers are designed by the methods used for concrete slabs. Plates XXIV and XXV show details of the Inlet and Outlet. The outlet is equipped with a waste weir for over flow, and the valves for the control of the water are located at these two points. The design of the partitions is dealt with in the same ~~manner~~ manner as the outer wall. The design of the weir and gates is similar to that shown in the design of the intake at the impounding

reservoir. Plate XXIII shows a system of drainage for the prevention of upthrust. These drains are to be carried so that

leakage, if any occur, will be apparent. A well drained clay is a better bearing surface than a wet clay, therefore the drainage is essential.

Plate XXII shows a floor section. The under layer is reinforced at the points of intersection of the upper layers. The upper slabs are built with expansion joints, caulked with pitch. Plate XXVI gives the general layout of the entire basin, showing the relative positions of the different parts of the construction.

A comparison of costs showed that the reinforced concrete is the cheaper construction and is therefore adopted. The inlet is arranged with mushroom discharge.

The capacity of the reservoir is 28,000,000 gallons. This is insufficient to supply the wants of the city for some date 25 years hence, for more than a day. This being the case a second reservoir is essential. The Luttrell Hill may well be designed to supply the low pressure zone, or else the demand of south Knoxville. The Sharpe's Ridge Reservoir may be designed for a capacity of 100,000,000 gallons, at present, and addition may be made at any future date.

When using the two zones supplied by the two reservoirs as suggested, there could be valves placed at points in the city, so that at any time the two pressure systems could be connected. This would be of great advantage in the time of fires, as the friction losses would be greatly reduced. The conduit and feeders that would have to cross the river, if submerged would have to be equipped at both ends with automatic valves, which would close if the line were broken.

COST.

The cost of the construction will be calculated in each individual item, to the lowest \$ 100; and in each division to the cost \$ 1000. The cost is based on the present prices, which are exceedingly high.

Cost of the Watershed Rights.

	\$	\$
9600 acres of timber land	@ 60	576,000
19000 acres of cut land	@ 7	133,000
200 acres of cultivated land	@ 100	20,000
Elkmoor Tennessee		10,000
Wonderland Park Club		10,000
Appalachian Club		30,000
15 miles of rail road		<u>300,000</u>
Total		1,079,000

Impounding Reservoir Site and Necessities.

	\$	\$
30 acres of cultivated land	@ 100	3,000
30 acres of cut land	@ 7	200
Sewage disposal for population		10,000
Relocation of 10 miles of rail road	@ 20,000	<u>200,000</u>
Total		214,000

Dam and Intake at Impounding Reservoir.

	\$	\$
1000 cubic yards, of excavation at dam @ 2.00		2,000
37100 cubic yards, concrete at dam @ 6.00		222,600
2680 cubic yards, concrete at intake @ 12.00		33,000
150 feet of "36 inch" cast iron pipe @ 6.00		900
Valves, gates and etc.		<u>2,000</u>
Total		261,000

Pipe Line.

Section No. 1.	\$	\$
7.4 miles "18 inch" "Wood Stave" Class A @ 5810		43,000
4 air valves @ 10		100
1 three way valve (18 inch) @ 500		500
3 blowoff valves (6 inch) @ 33		100
3 blowoff special (603 pounds) @ 40		100
9 acres of right away @ 40		<u>400</u>
Total		45,000

Section No.2.		
5.2 miles"28 inch""Wood Stave" Class A @	7610	\$ 39,600
3.2 miles"28 inch""Wood Stave" Class B @	10880	\$ 34,900
3.4 miles"28 inch""Wood Stave" Class C @	17590	\$ 59,900
10 air valves @	10	\$ 100
2 three way valves(28 inch) @	600	\$ 1,200
10 blowoff valves(8 inch) @	54	\$ 600
1 stop valve @	600	\$ 600
10 blowoff specials @	64	\$ 700
73 acres right away @	100	\$ 7,300
Total		\$ 145,000

Section No.3.		
0.1 miles"32 inch""Wood Stave" Class A @	9610	\$ 1,000
1.8 miles"32 inch""Wood Stave" Class B @	14000	\$ 25,200
9.1 miles"32 inch""Wood Stave" Class C @	22380	\$ 203,700
10.2 " "32 inch" 3/8 inch Steel" @	56730	\$ 578,700
4.6 " "32 inch""7/16 inch Steel" @	79200	\$ 388,100
0.3 " "32 inch""7/16 inch Steel" @		
21 air valves @	10	\$ 200
2 three way valves @	1200	\$ 2400
22 blow off valves @	133	\$ 3,000
1 stop valve @	1075	\$ 1,100
22 blowoff specials @	80	\$ 1,800
200 acres of right away @	100	\$ 20,000
Total		\$ 1,226,000

Total cost of Conduit	\$	
Section No.1		\$ 45,000
Section No.2		\$ 145,000
Section No.3		\$ 1,226,000
Total Cost		\$ 1,416,000

Water Works Building and Control.

Cost of Building	\$	\$ 50,000
Signals and Remote Control		\$ 10,000
Sterilizing apparatus (15 sec.ft.)		\$ 5,000
Laboratory Equipment		\$ 5,000
Valves and etc.		\$ 3,400
Total		\$ 74,000

Luttrell Hill Reservoir.

223,000 cubic yards excavation @	0.30	\$ 66,900
1757 feet of reservoir wall @	92	\$ 162,000
66,700 sq.yd. of flooring @	1.20	\$ 80,000
66,700 sq.yd. of drainage @	0.10	\$ 700
Inlet and Valves etc.		\$ 10,000
Outlet and valves etc.		\$ 20,000
280 feet of division wall @	112	\$ 314,000
7 acres of land @	500	\$ 3,500
Total		\$ 658,000

Sharpe's Ridge Reservoir.

	\$	
2 Inlets	@ 20000	40,000
2 Outlets	@ 10000	20,000
4 division walls(300 feet each)	@ 112	134,400
223,000 sq.yd. of flooring	@ 1.20	267,600
5000 feet of reservoir wall	@ 92.00	460,000
223,000 sq.yd. of drainage	@ 0.10	2,300
370,000 cubic yards of excavation	@ 0.30	111,000
18 acres of land	@ 2000	3,600
Total		<u>1,039,000</u>

Cost of a Gravity Section Wall
at Luttrell Hill Reservoir.

1757 foot reservoir wall(gravity section)@ 145.80 256,200

Cheapest Arrangement of Works

Luttrell Reservoir	\$ 658,000
Conduit	1,416,000
Dam and Intake	261,000
Impounding Reservoir	214,000
Total	<u>2,549,000</u>

Complete Works

Watershed	\$ 1,079,000
Dam and Intake	261,000
Conduit	1,416,000
Water Works Building and etc.	74,000
Luttrell Hill Reservoir	658,000
Sharpe's Ridge Reservoir	1,039,000
	<u>4,527,000</u>

imp. res site & inc = 214,000
4,741,000

130

Cities

- ① Houston Tex.
- ② Schenectady N.Y.
- ③ Lawrence Mass.
- ④ Somerville Mass.
- ⑤ Youngstown Ohio
- ⑥ Springfield Mass.
- ⑦ Lynn Mass.

Cities

- ① Cleveland Ohio
- ② Buffalo N.Y.
- ③ Pittsburg Pa.
- ④ Milwaukee Wis.
- ⑤ Detroit Mich.
- ⑥ Jersey City N.Y.
- ⑦ Newark N.J.

120

110

100

90

80

70

60

50

40

30

20

10

0

Percentage.

60

50

40

30

20

10

0

10

20

30

40

50

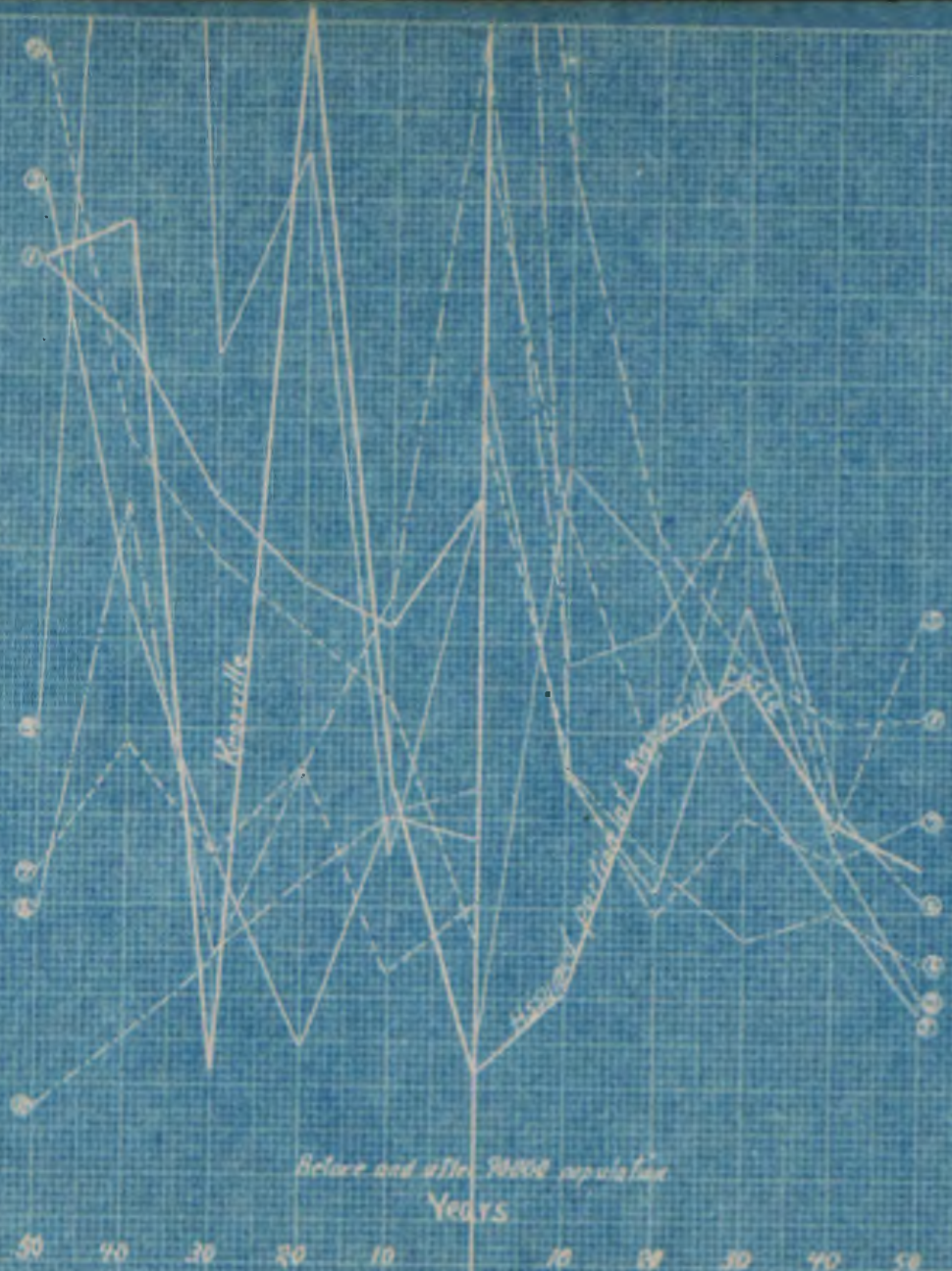
60

Before and after 1900 population

Years

(U.S. Census)

Percentage Growth of Cities



Cities

- ① Houston, Tex.
- ② Schenectady N.Y.
- ③ Somerville Mass.
- ④ Lawrence Mass.
- ⑤ Youngstown Ohio
- ⑥ Springfield Mass.
- ⑦ Lynn Mass.

Population Curves

with reference to
Knoxville

(U.S. Census)

Line of equivalent pop.
1970

Growth of cities to 20000-30000
Growth of cities to greater pop.

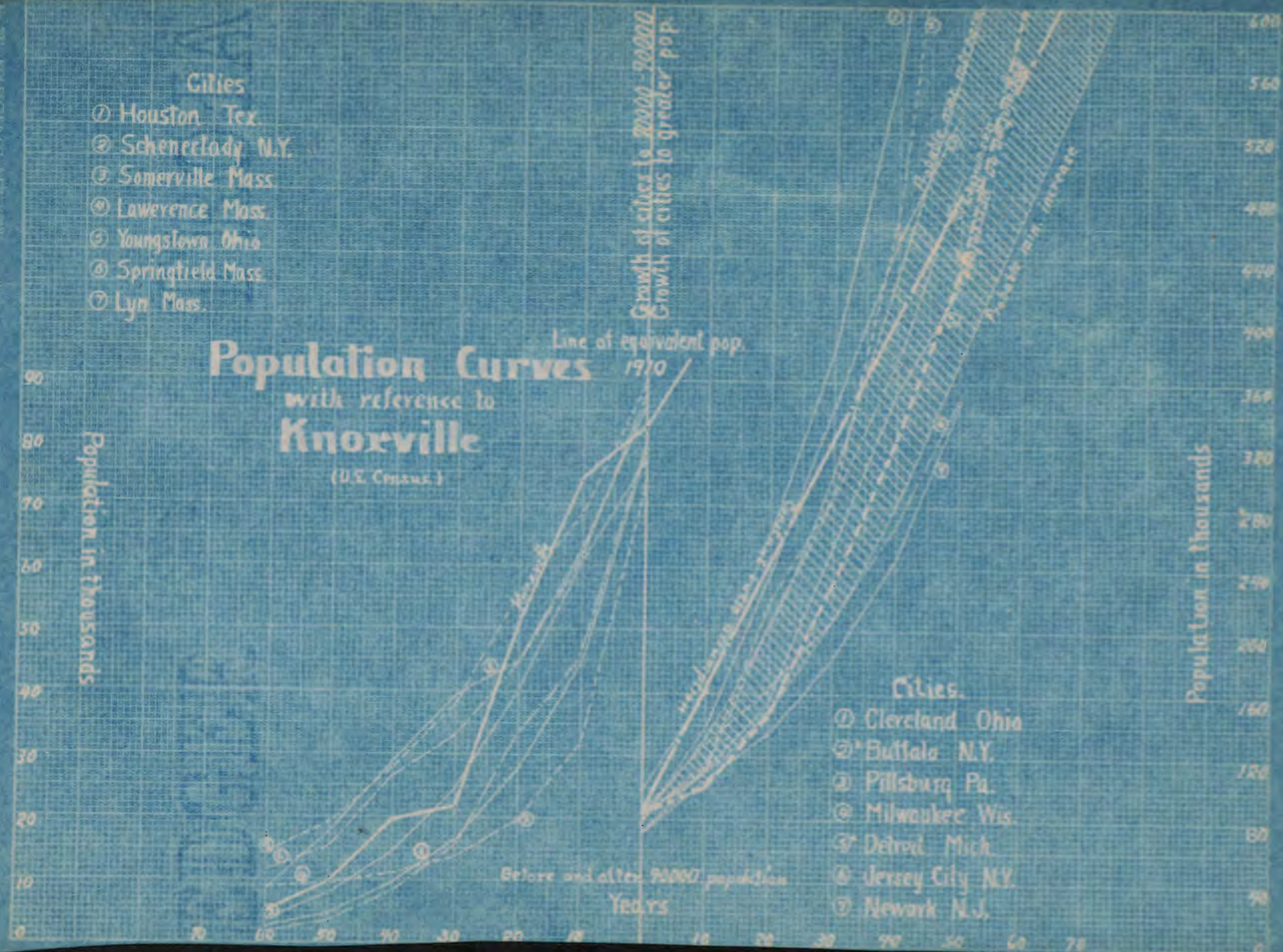
Cities

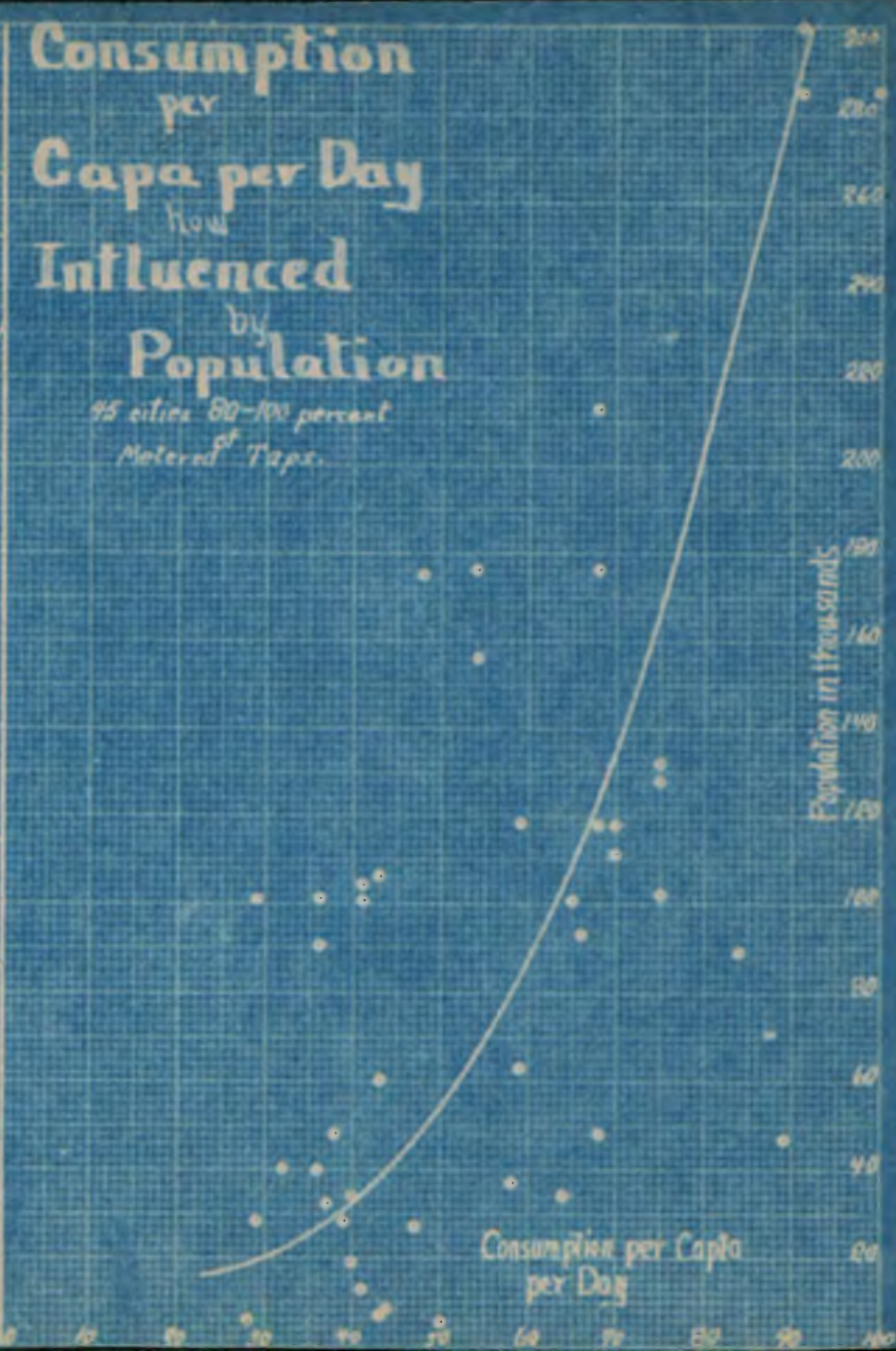
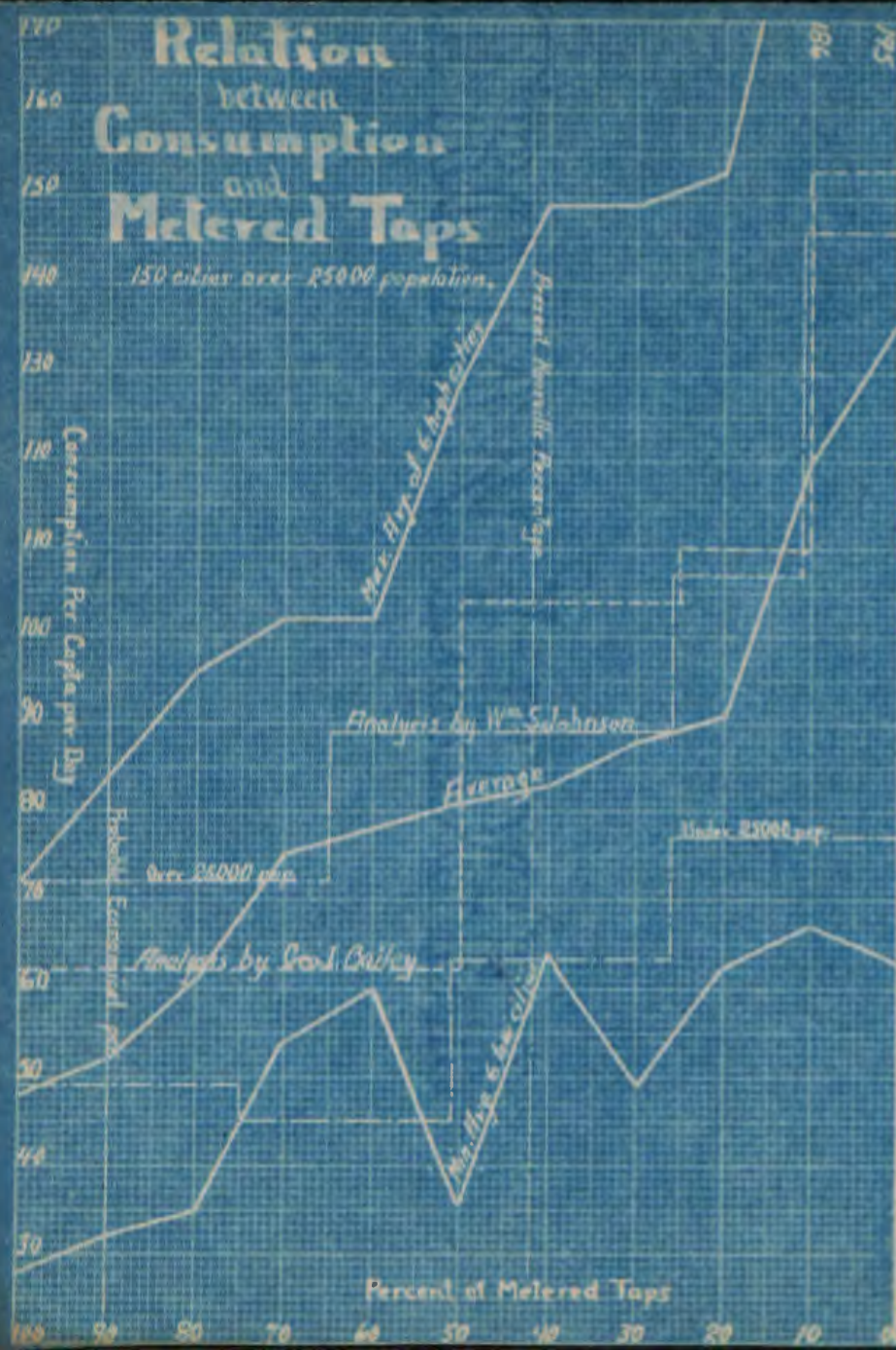
- ① Cleveland Ohio
- ② Buffalo N.Y.
- ③ Pittsburg Pa.
- ④ Milwaukee Wis.
- ⑤ Detroit Mich.
- ⑥ Jersey City N.Y.
- ⑦ Newark N.J.

Before and after 90000 population
Years

Population in thousands

Population in thousands







THE TOPOGRAPHIC MAPS OF THE UNITED STATES

The United States Geological Survey is making a topographic atlas of the United States. This work has been in progress since 1882, and more than 38 per cent of the area of the country, excluding outlying possessions, has now been mapped. The areas mapped are widely distributed, every State being represented, as shown on the progress maps accompanying each annual report of the Director.

This atlas is being published in sheets of convenient size, about 16 1/2 by 20 inches. The four-sided area of land represented on an atlas sheet is bounded by parallels and meridians and is called a *quadrangle*. The quadrangles mapped cover 1° of latitude by 1° of longitude, 30' of latitude by 30' of longitude, 15' of latitude by 15' of longitude, or smaller areas, the size of the area mapped depending on the scale used. Several scales are employed. The smallest scale, that used for quadrangles covering 1°, is 1:250,000, or very nearly 4 miles to an inch—that is, 4 linear miles on the ground are represented by 1 linear inch on the map. This scale is used for maps of the desert regions and some other parts of the far West. For the greater part of the country, which is mapped by quadrangles covering 30', a larger scale, 1:125,000, or about 2 miles to an inch, is employed. A still larger scale, 1:62,500, or about a mile to an inch, is used for quadrangles covering 15', the unit selected for mapping thickly settled or industrially important areas. A fourth scale, 1:31,250, or one-half mile to an inch, is employed for maps that are to be used in connection with irrigation or drainage, and a few maps of mining districts are published on still larger scales.

A topographic survey of Alaska has been in progress since 1898 and nearly 30 per cent of its entire area has now been mapped. One-third of the area mapped, or 10 per cent of the Territory, has been covered only by reconnaissance work, the results of which have been mapped on a scale of about 10 miles to an inch. The maps of nearly all the remaining two-thirds of the surveyed area have been published on a scale of 1:250,000, or about 4 miles to an inch. These maps are large each representing 2° of latitude by 4° of longitude. A few areas that are of economic importance, aggregating about 3,000 square miles, have been surveyed in greater detail and mapped on a scale of 1:62,500, or about a mile to an inch.

A survey of the Hawaiian Islands was begun in 1910 and the resulting maps are being published on a scale of 1:62,500.

The features shown on these atlas sheets or maps may be classed in three groups—(1) *water*, including seas, lakes, rivers, canals, swamps, and other bodies of water; (2) *relief*, including mountains, hills, valleys, and other elevations and depressions; (3) *culture* (works of man), such as towns, cities, roads, railroads, and boundaries. The conventional signs used for these features are shown below, with explanations. Variations appear on some earlier maps.

All water features are printed in *blue*, the smaller streams and canals in full blue lines and the larger streams, lakes, and the sea in blue water-lining. Intermittent streams—those whose beds are dry at least three months in the year—are shown by lines of dots and dashes.

Relief is shown by contour lines in *brown*. A contour on the ground passes through points that have the same altitude. One who follows a contour will go neither uphill nor downhill but on a level. The contour lines on the map show not only the shapes of the hills, mountains, and valleys but also their elevations. The line of the sea coast itself is a contour line, the datum or zero of elevation being mean sea level. The contour at, say, 20 feet above sea level would be the shore line if the sea were to rise or the land to sink 20 feet. On a gentle slope this contour is far from the present coast; on a steep slope it is near the coast. Where successive contour lines are far apart on the map they indicate a gentle slope; where they are close together they indicate a steep slope; and where they run together in one line they indicate a cliff.

The manner in which contour lines express altitude, form, and grade is shown in the figure below.



The sketch represents a river valley between two hills. In the foreground is the sea, with a bay that is partly inclosed by a hooked sand bar. On each side of the valley is a terrace into which small streams have cut narrow gullies. The hill on the right has a rounded summit and gently sloping spurs separated by ravines. The spurs are truncated at their lower ends.

by a sea cliff. The hill on the left terminates abruptly at the valley in a steep scarp. It slopes gradually back away from the scarp and forms an inclined table-land, which is traversed by a few shallow gullies. On the map each of these features is indicated, directly beneath its position in the sketch, by contour lines.

The contour interval, or the vertical distance in feet between one contour and the next, is stated at the bottom of each map. This interval differs according to the character of the area mapped; in a flat country it may be as small as 5 feet; in a mountainous region it may be 250 feet. Certain contour lines, every fourth or fifth one, are made heavier than the others and are accompanied by figures stating elevation above sea level. The heights of many points, such as road corners, summits, surfaces of lakes, and bench marks, are also given on the map in figures, which express the elevations to the nearest foot only. More exact elevations of bench marks, as well as geodetic coordinates of triangulation stations, are published in bulletins issued by the Geological Survey. A bulletin pertaining to any State may be had on application.

The works of man are shown in *black*, in which color all lettering also is printed. Boundaries, such as those of a State, county, city, land grant, township, or reservation, are shown by continuous or broken lines of different kinds and weights. Public and through roads are shown by fine double lines; private and poor roads by dashed double lines; trails by dashed single lines.

Each quadrangle mapped for the topographic atlas is designated by the name of a principal town or of some prominent natural feature within the quadrangle, and on the margins of the maps are printed the names of adjoining quadrangles for which atlas sheets have been published or are in preparation. The sheets are sold at 10 cents each in lots of less than 50 copies or at 6 cents each in lots of 50 or more copies, whether of the same or of different sheets.

The topographic map is the base on which the geology and the mineral resources of a quadrangle are represented, the maps showing these features being bound together, with a description of the quadrangle, to form a folio of the Geologic Atlas of the United States. Circulars showing by index maps the published topographic atlas sheets and geologic folios covering any State or region will be sent free on application.

Applications for maps or folios should be accompanied by cash—the exact amount—or by post-office money order (not postage stamps), and should be addressed to—

THE DIRECTOR,

United States Geological Survey,
Washington, D. C.

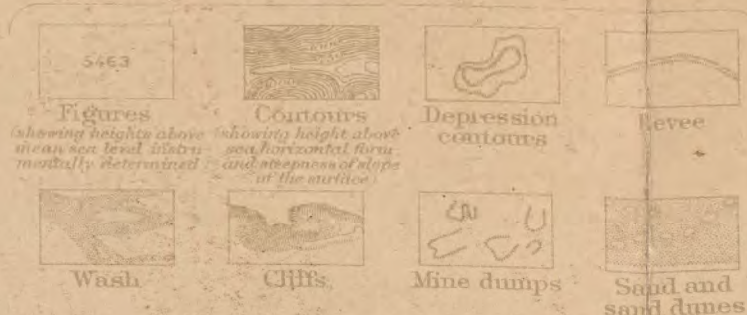
January, 1915.

CONVENTIONAL SIGNS

CULTURE (printed in black)



RELIEF (printed in brown)



WATER (printed in blue)



WOODS (when shown, printed in green)



WATER-SHED
of
WATER SUPPLY
for
KNOXVILLE
Area 45 sq. mi.

Scale



RAINFALL ANALYSIS

Tennessee River Basin

East and West Slope of Smoky Mts.

Excess of East Slope

Excess of West Slope

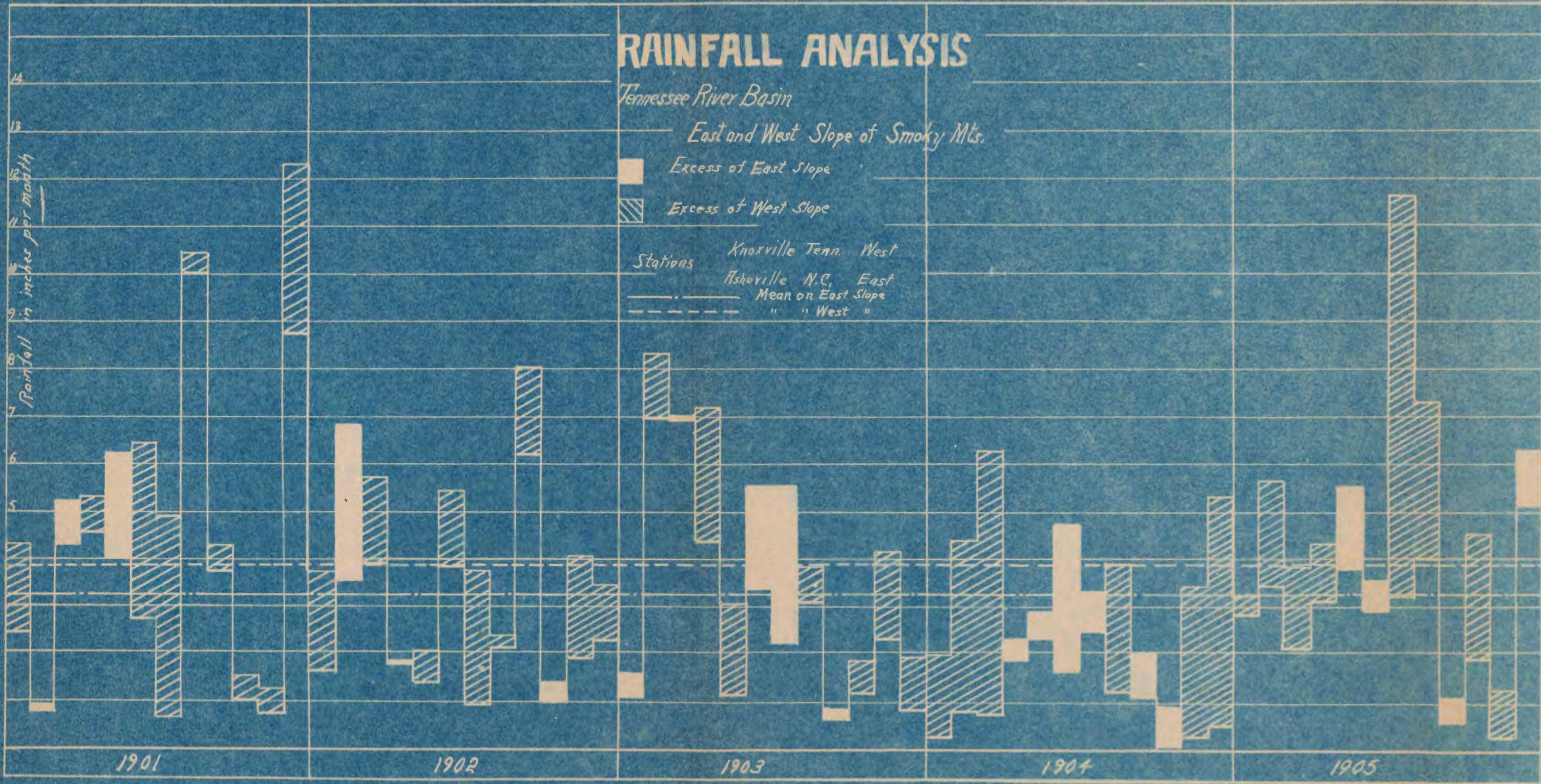
Stations Knoxville Tenn. West

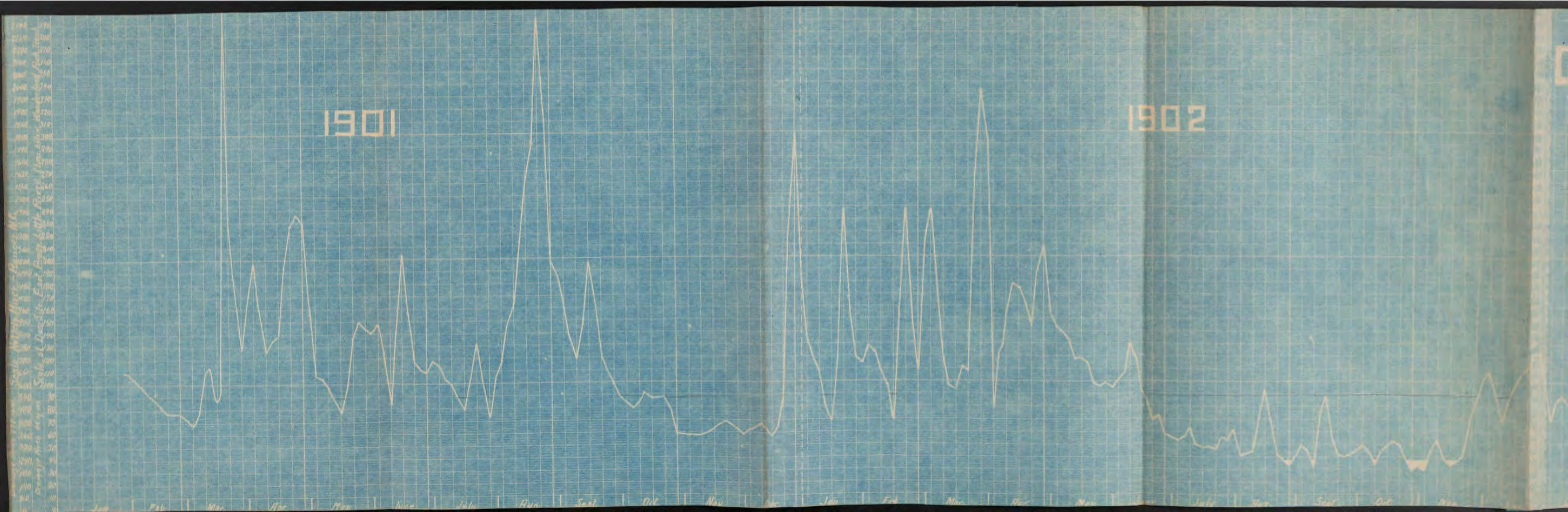
Ashville N.C. East

Mean on East Slope

" " West "

Rainfall in inches per month

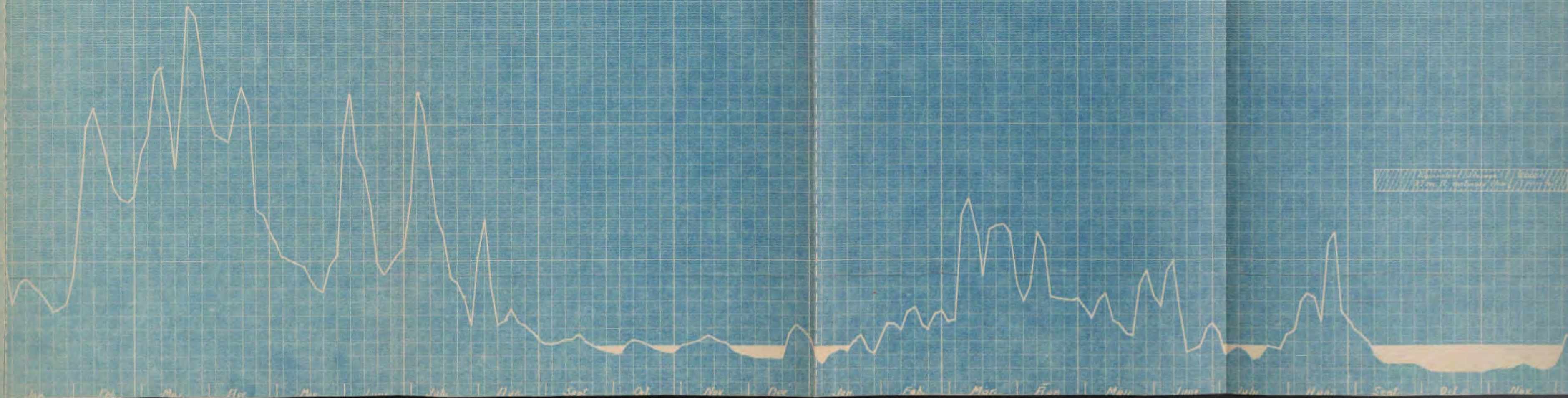




COMPARATIVE HYDROGRAPH

1903

1904



1905

Equivalent Storage 10000 ft.
37 sec. ft. continuous flow

Equivalent Storage 10000 ft.
37 sec. ft. continuous flow

Sent. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.



No 1.



No 2.



No 5.



No 6.



No 3.



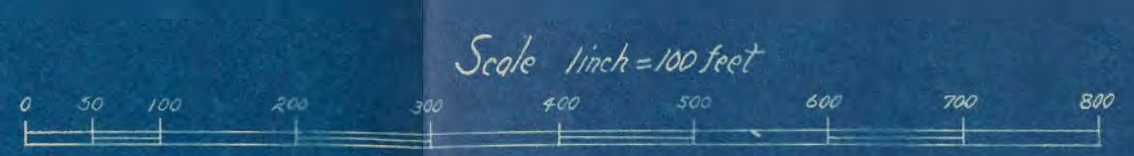
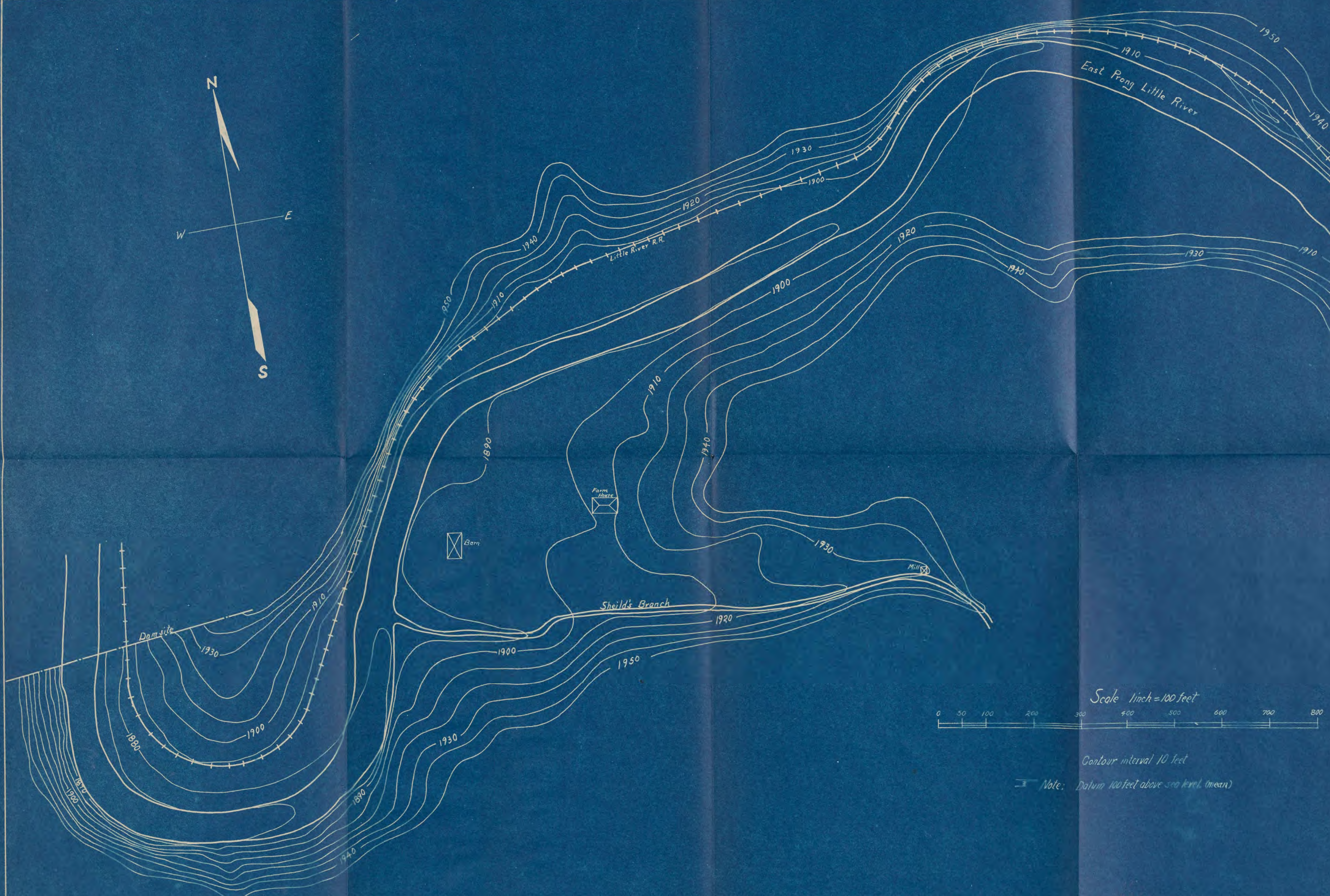
No 4.



No 7.



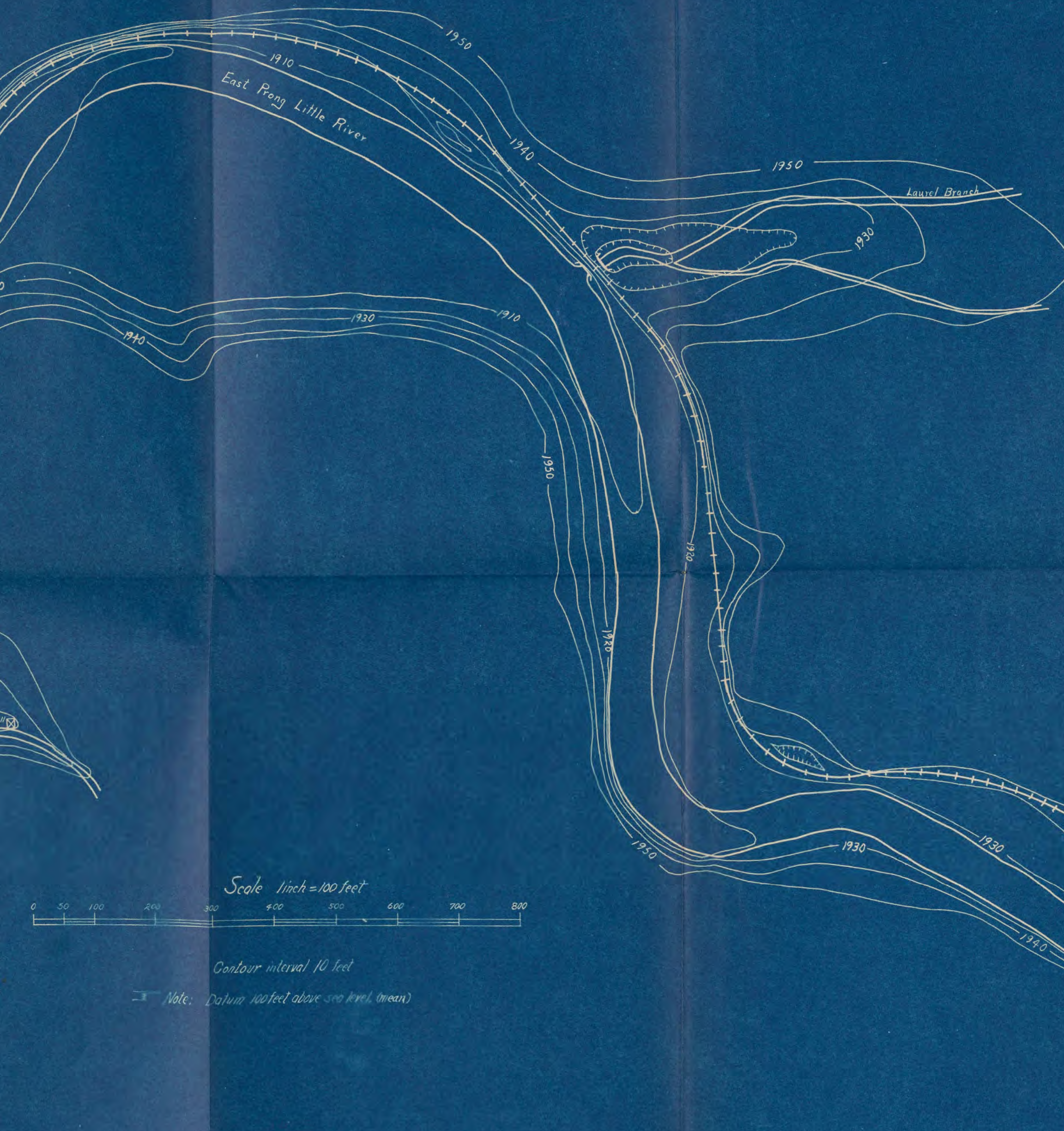
No. 8.



Scale 1 inch = 100 feet

Contour interval 10 feet

Note: Datum 100 feet above sea level (mean)



TOPOGRAPHY
OF
IMPOUNDING RESERVOIR
ON
EAST PRONG LITTLE RIVER

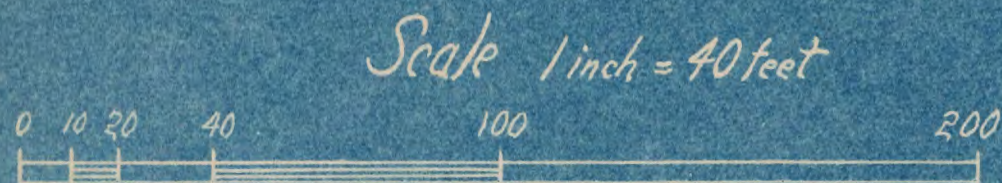
Proposed for gravity water supply for Knoxville

Scale 1 inch = 100 feet

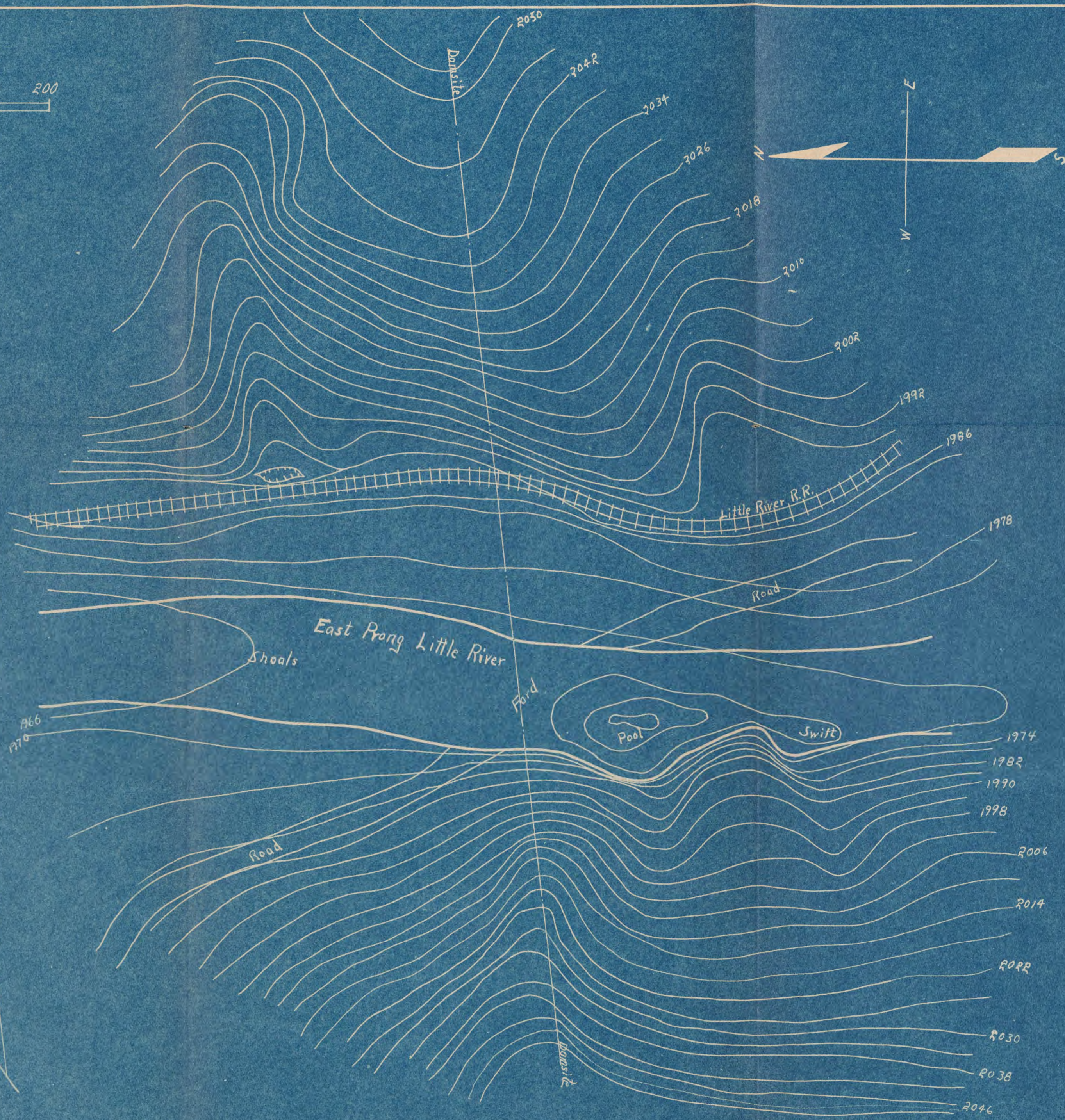
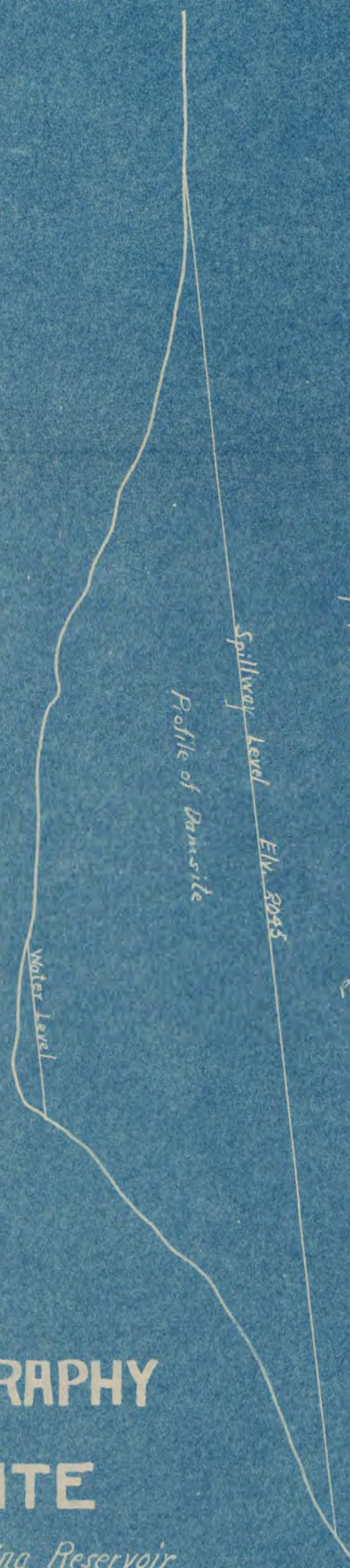


Contour interval 10 feet

Note: Datum 100 feet above sea level (mean)

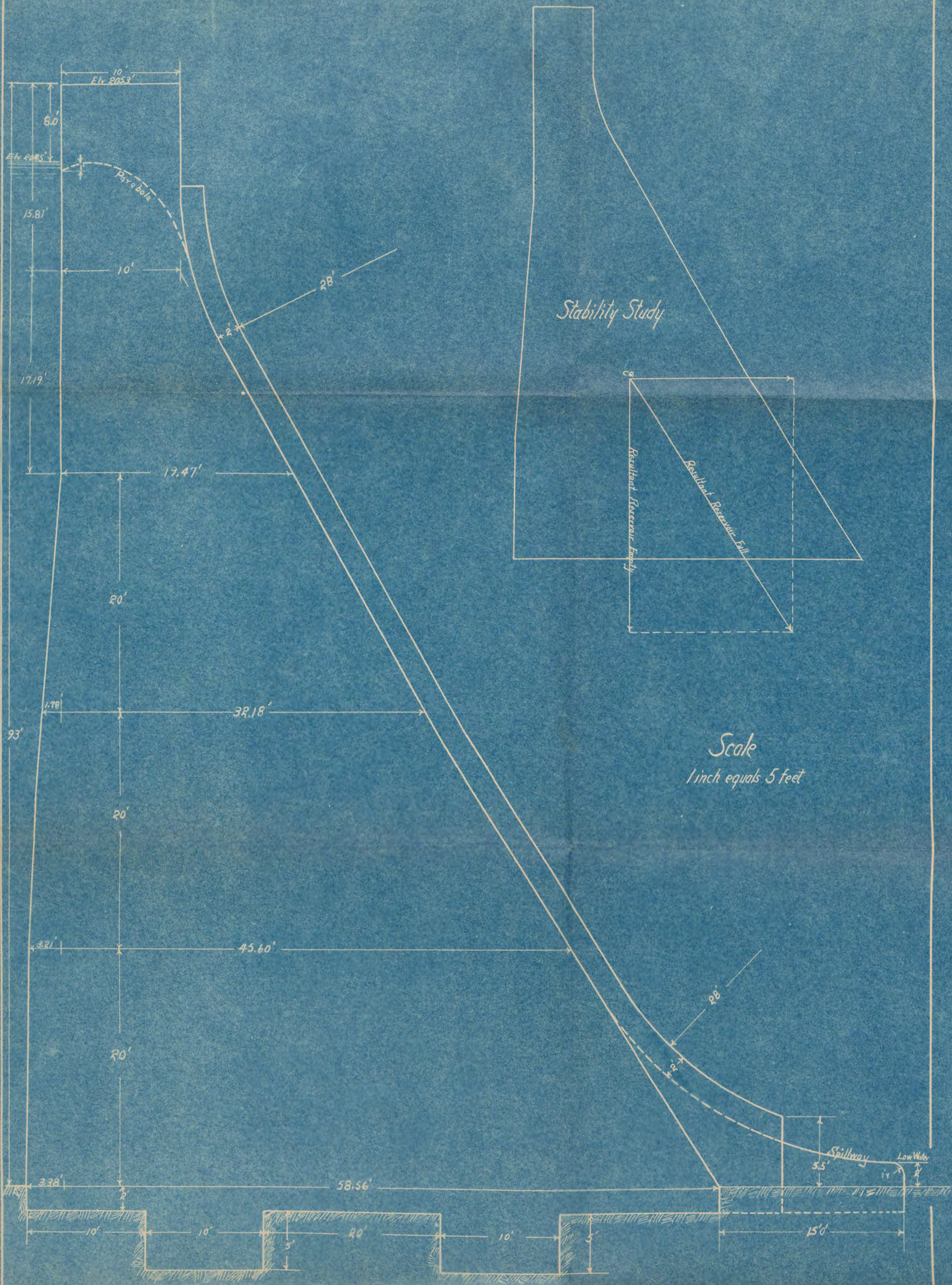


Contour interval 4 feet
Datum mean sea level



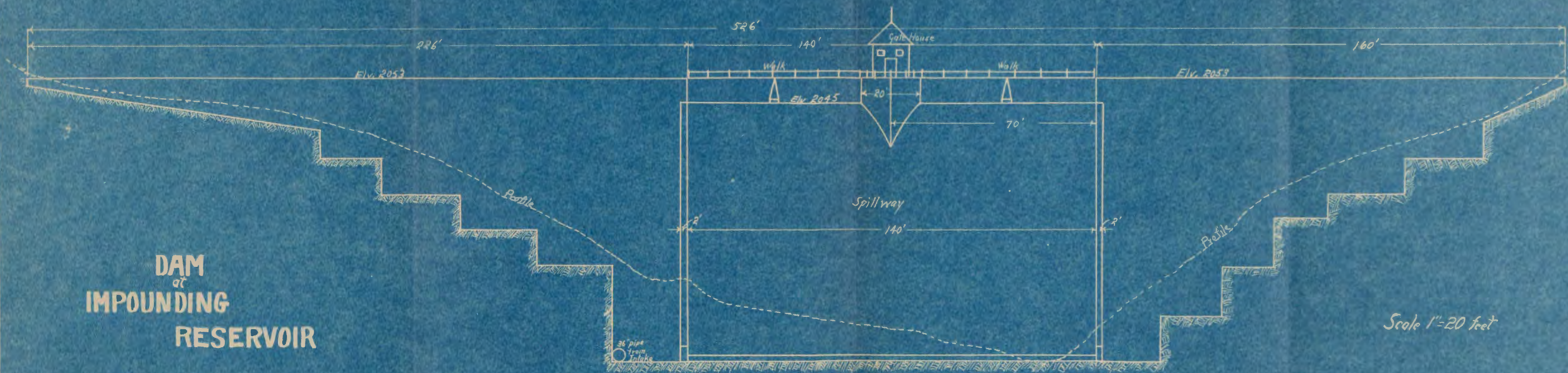
TOPOGRAPHY at DAM SITE

Impounding Reservoir



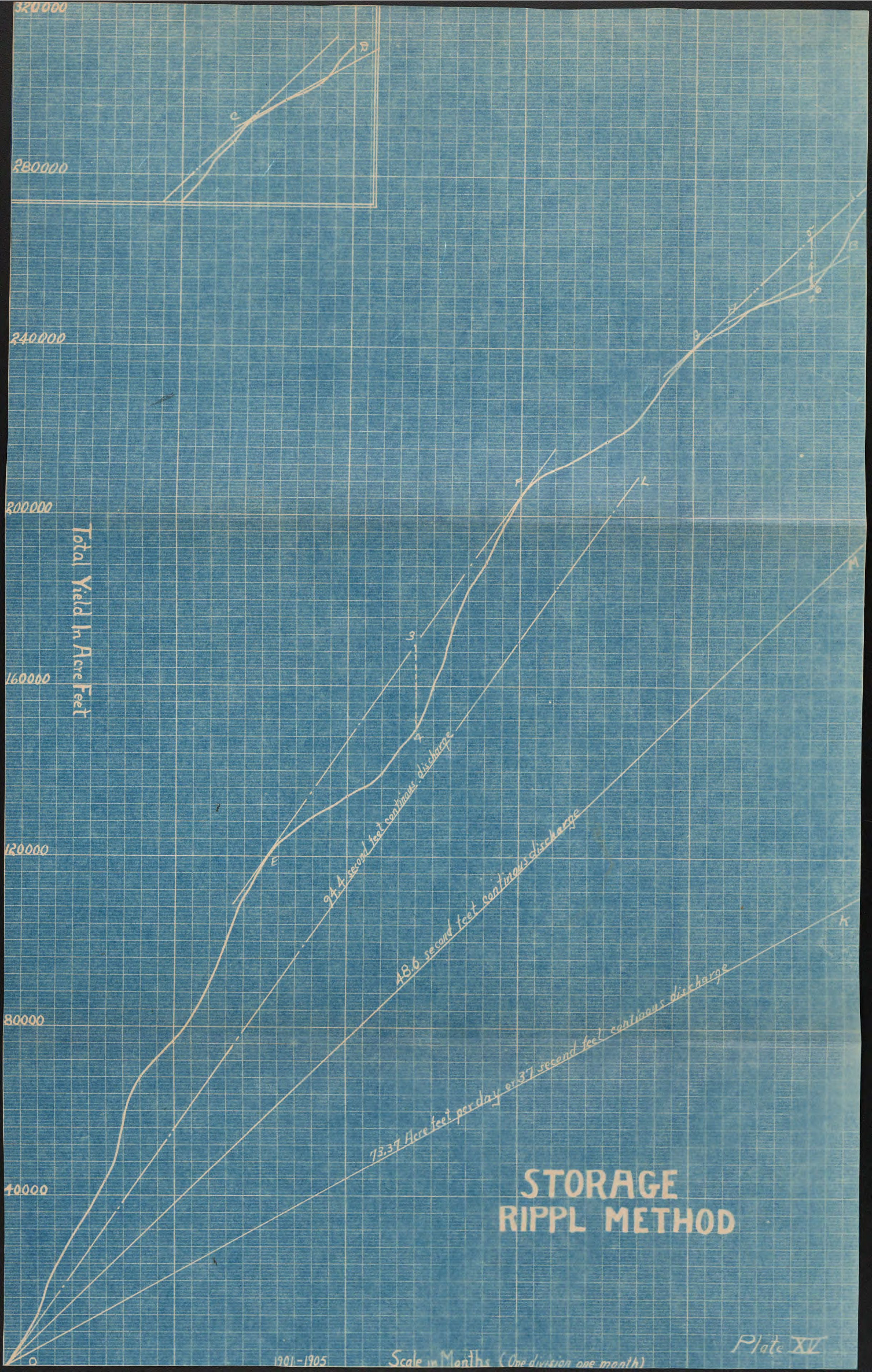
**CROSS-SECTION
IMPOUNDING RESERVOIR DAM**

DAM at IMPOUNDING RESERVOIR



Scale 1"=20 feet





Water Works
Building

Luttrell Reservoir
Elev. 1150

South
Knoxville

Tennessee River

Pump Station
Elev. 810

Stand
Pipe
Elev. 1151

West Knoxville

Knoxville

East Knoxville

RELATIVE LOCATION OF RESERVOIRS

Scale 1 inch = 1000 feet

0 250 500 1000 2000 3000 4000 5000

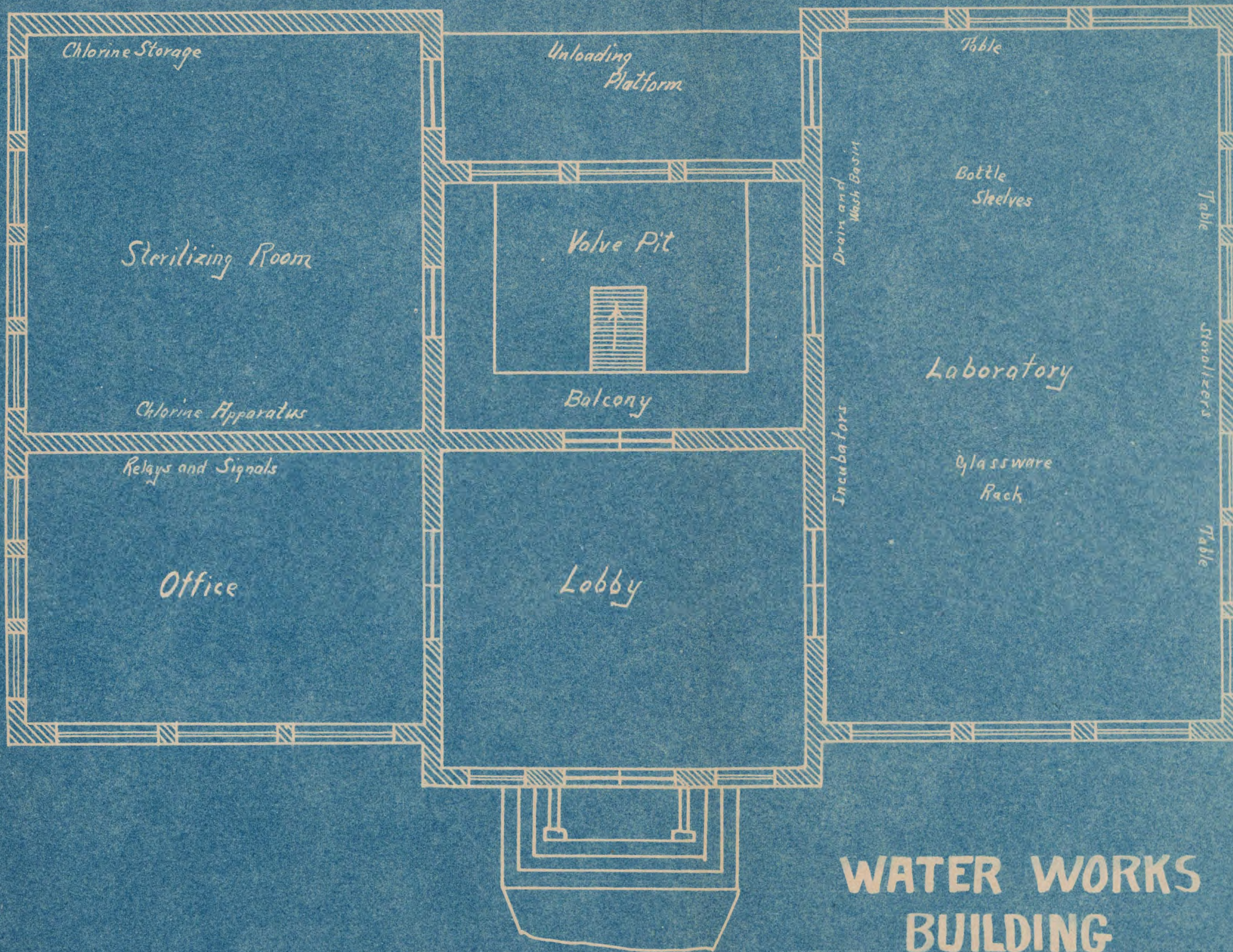
North Knoxville

Lansdale

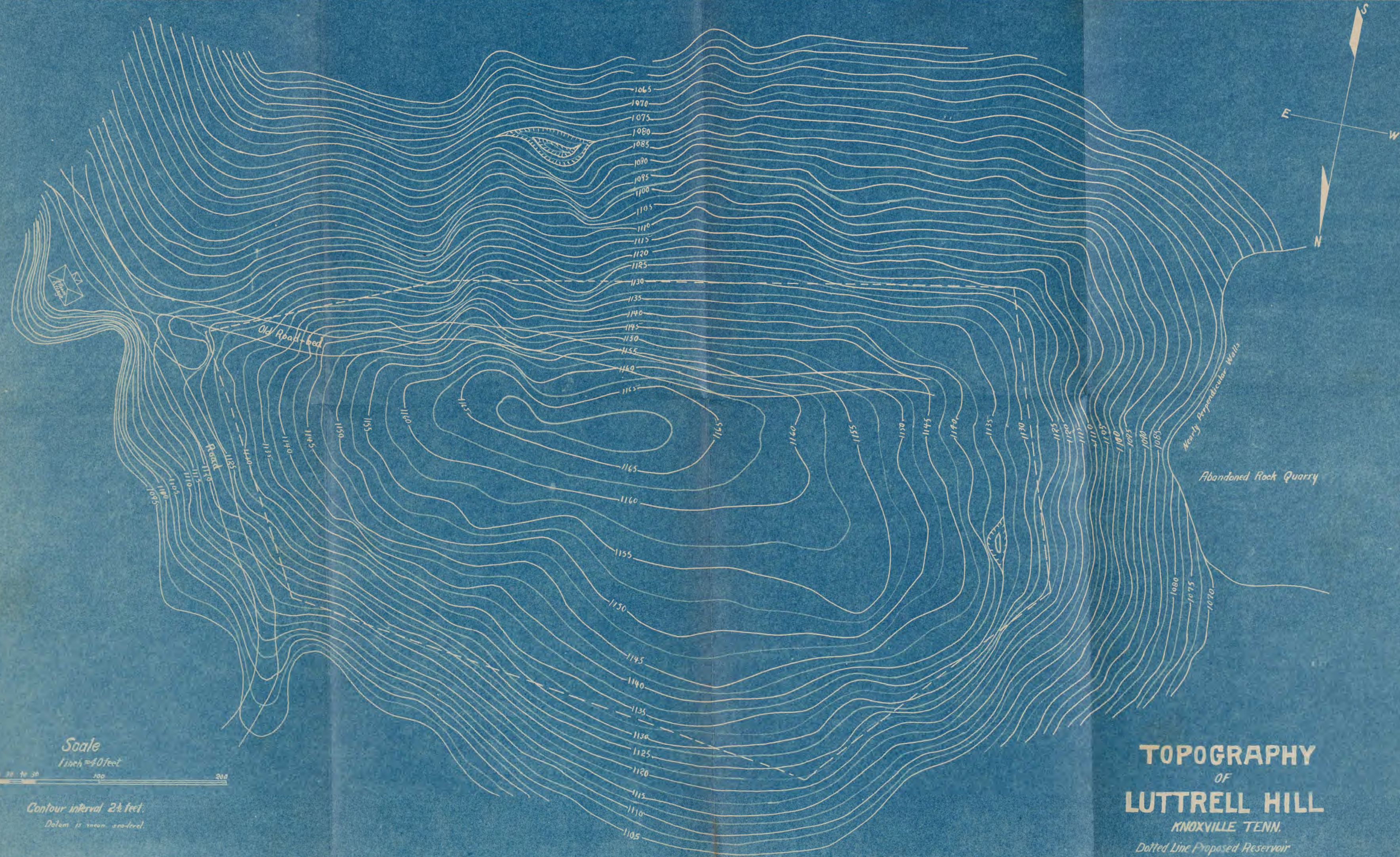
Shapley Reservoir
Oakwood
Elev.

Lincoln
Park

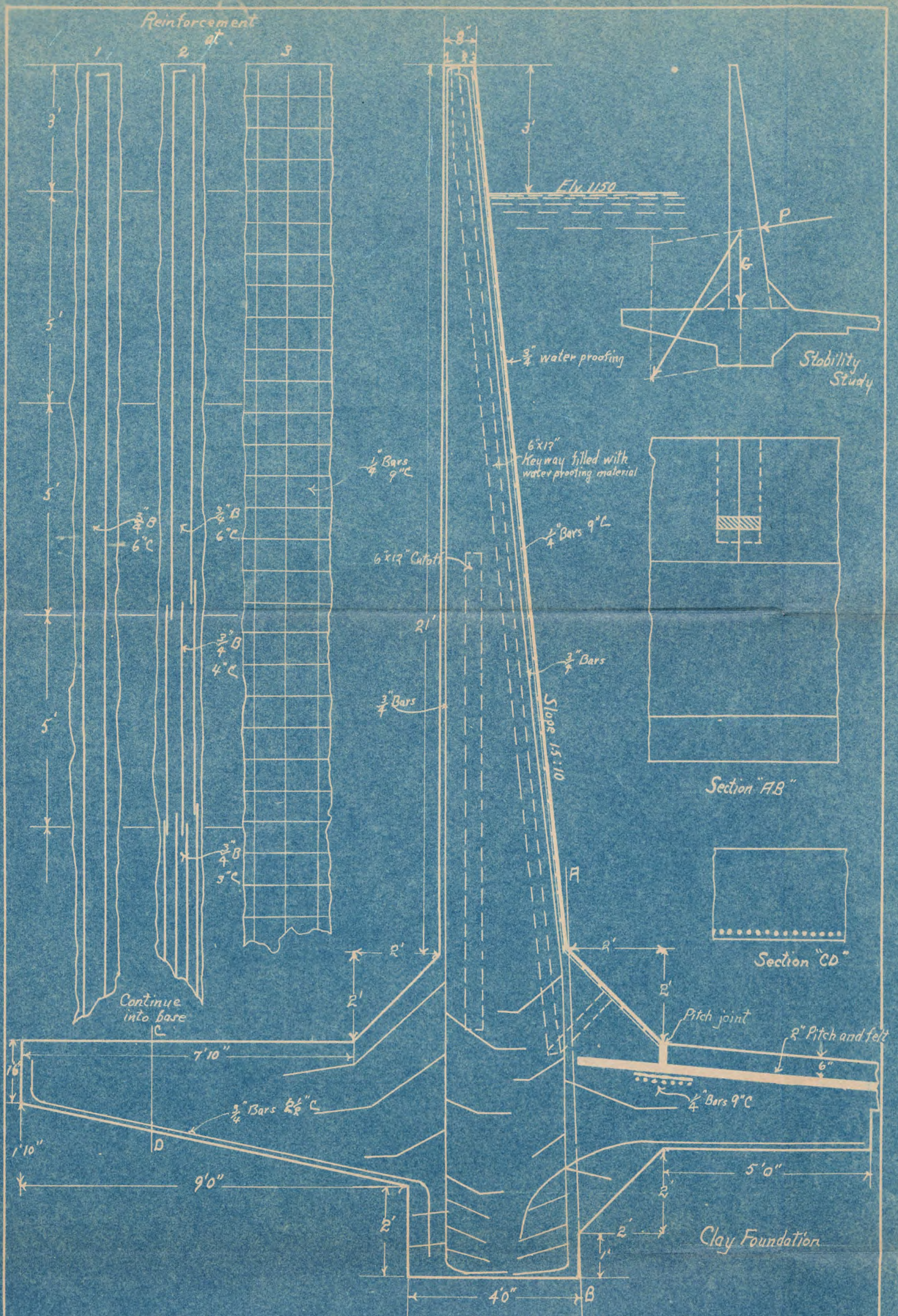
Park City



**WATER WORKS
BUILDING**



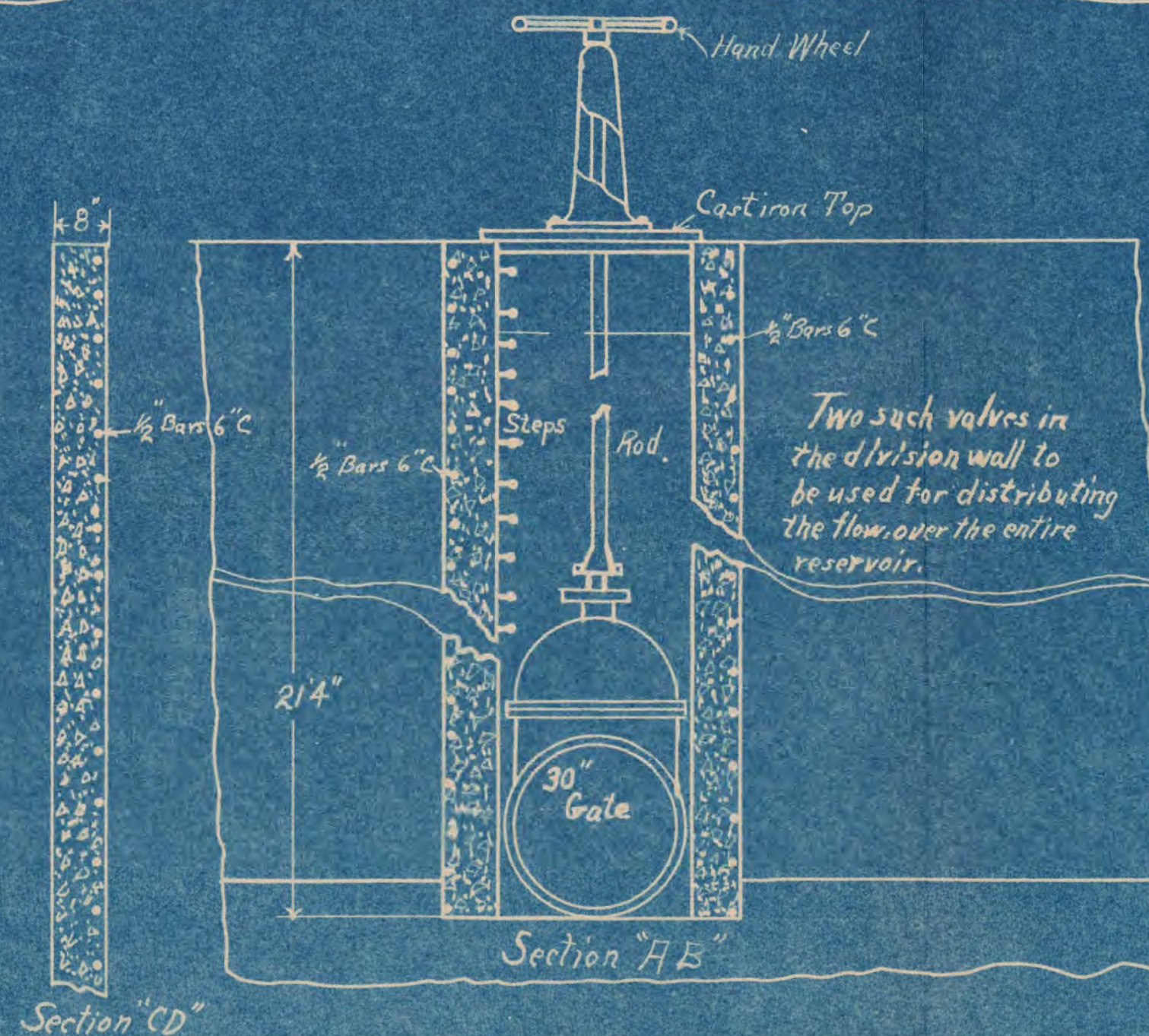
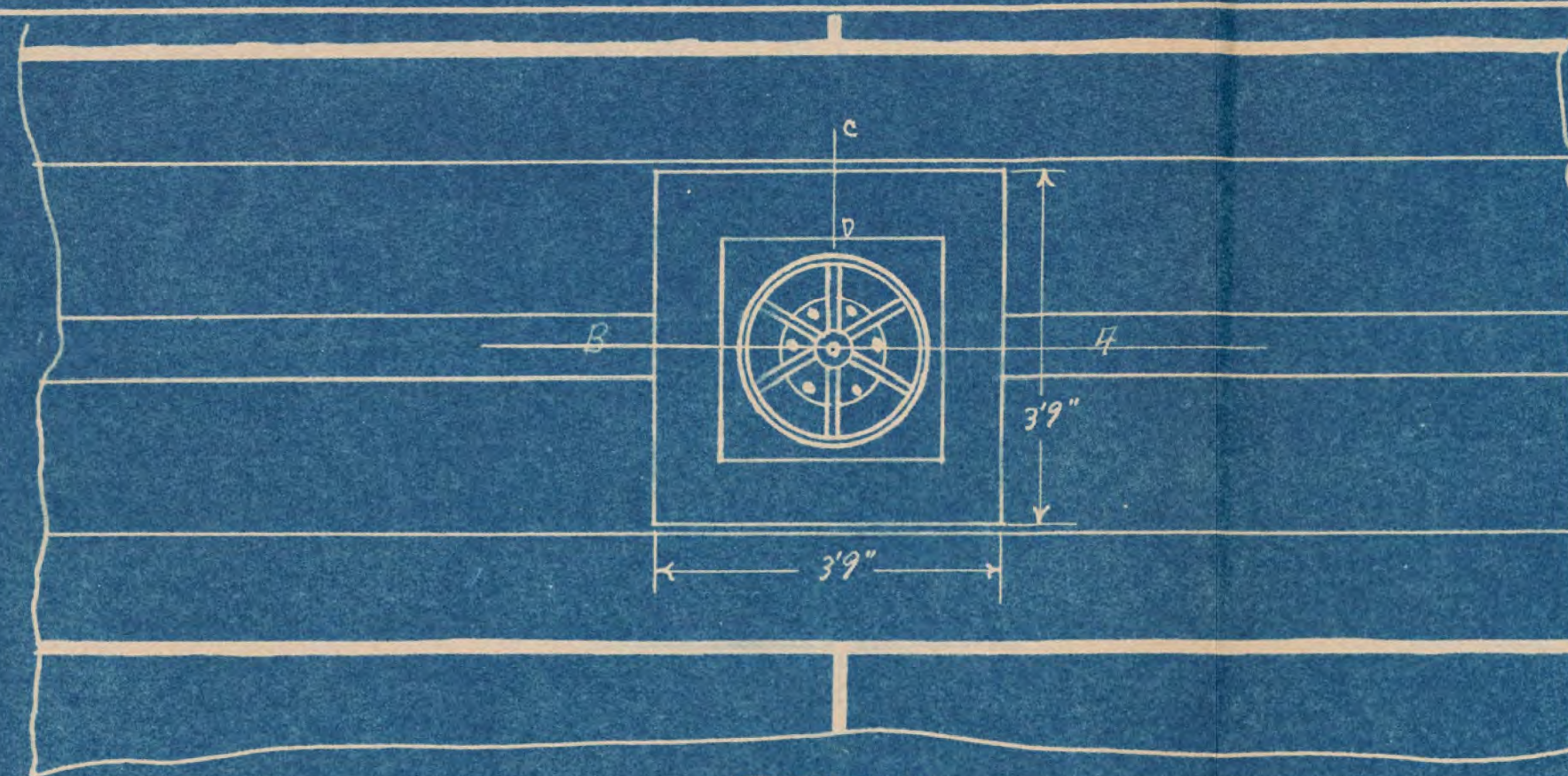
TOPOGRAPHY
OF
LUTTRELL HILL
KNOXVILLE TENN.
Dotted Line Proposed Reservoir



REINFORCED CONCRETE SECTION RESERVOIR WALL

Scale 1/2" = 1 foot

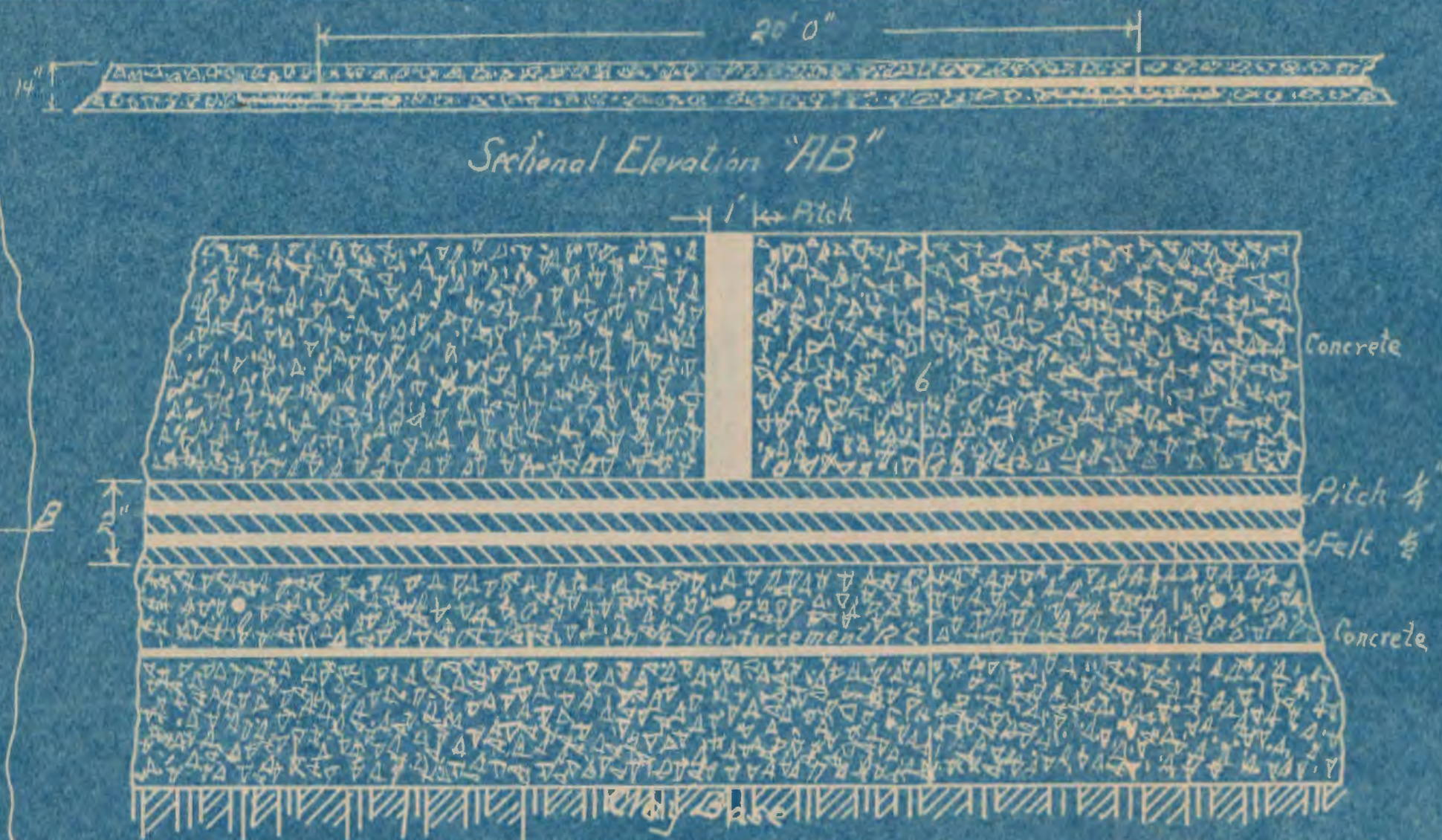
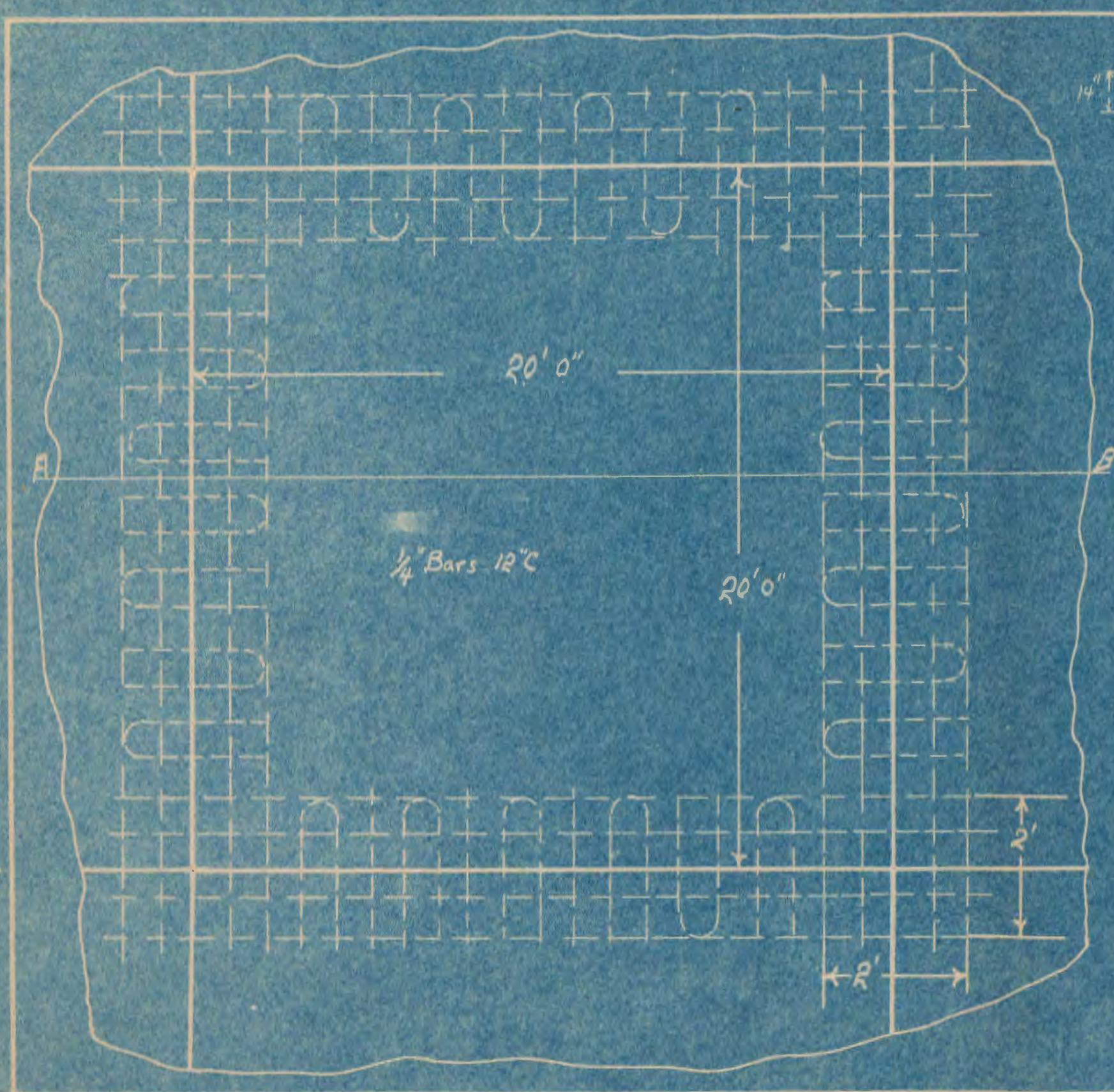
Proposed for Luttrell Hill Reservoir



Proposed for Lullrell Hill Reservoir

REINFORCED CONCRETE

Scale $\frac{1}{2}$ " = 1 foot

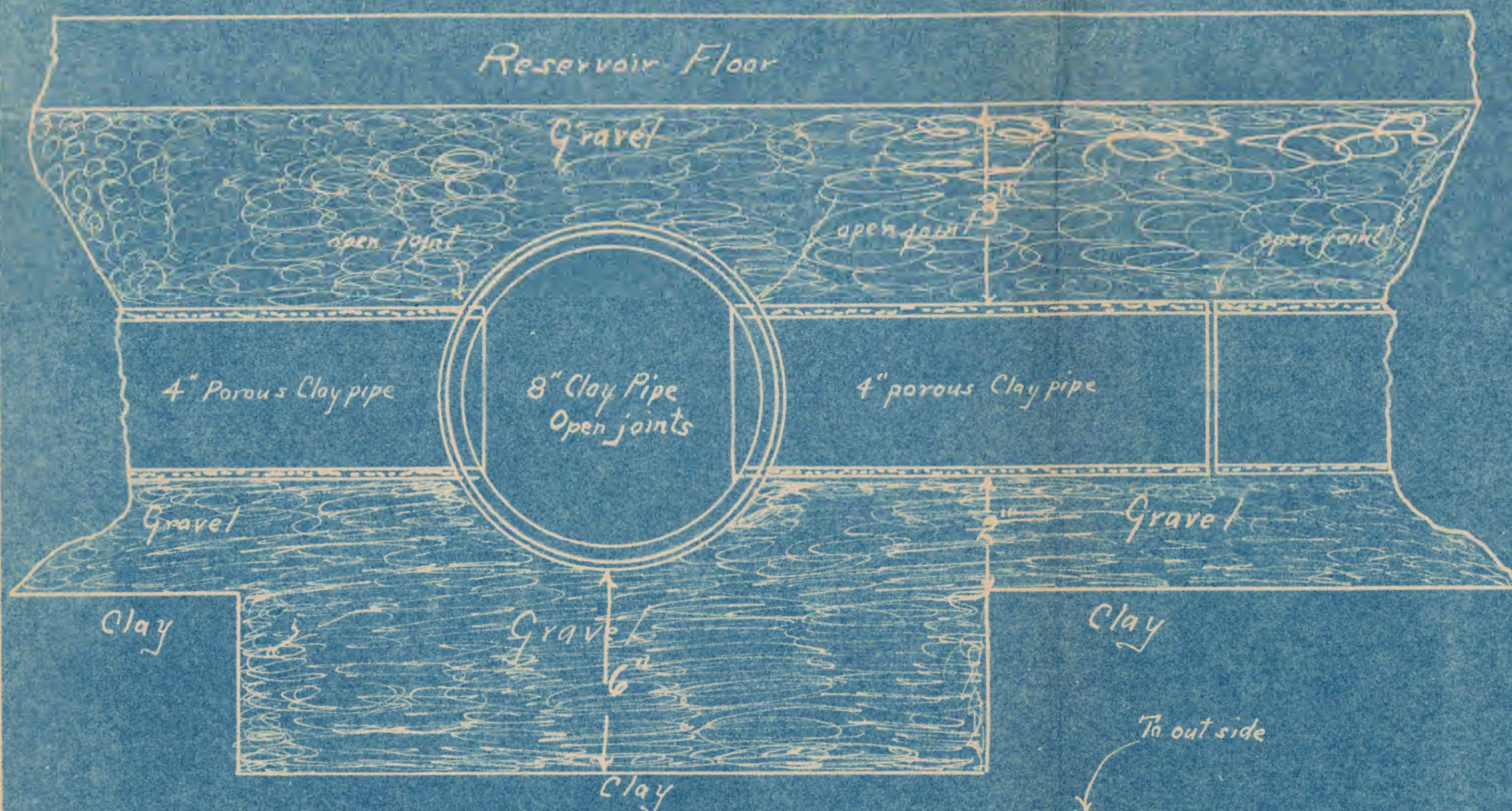


RESERVOIR FLOOR

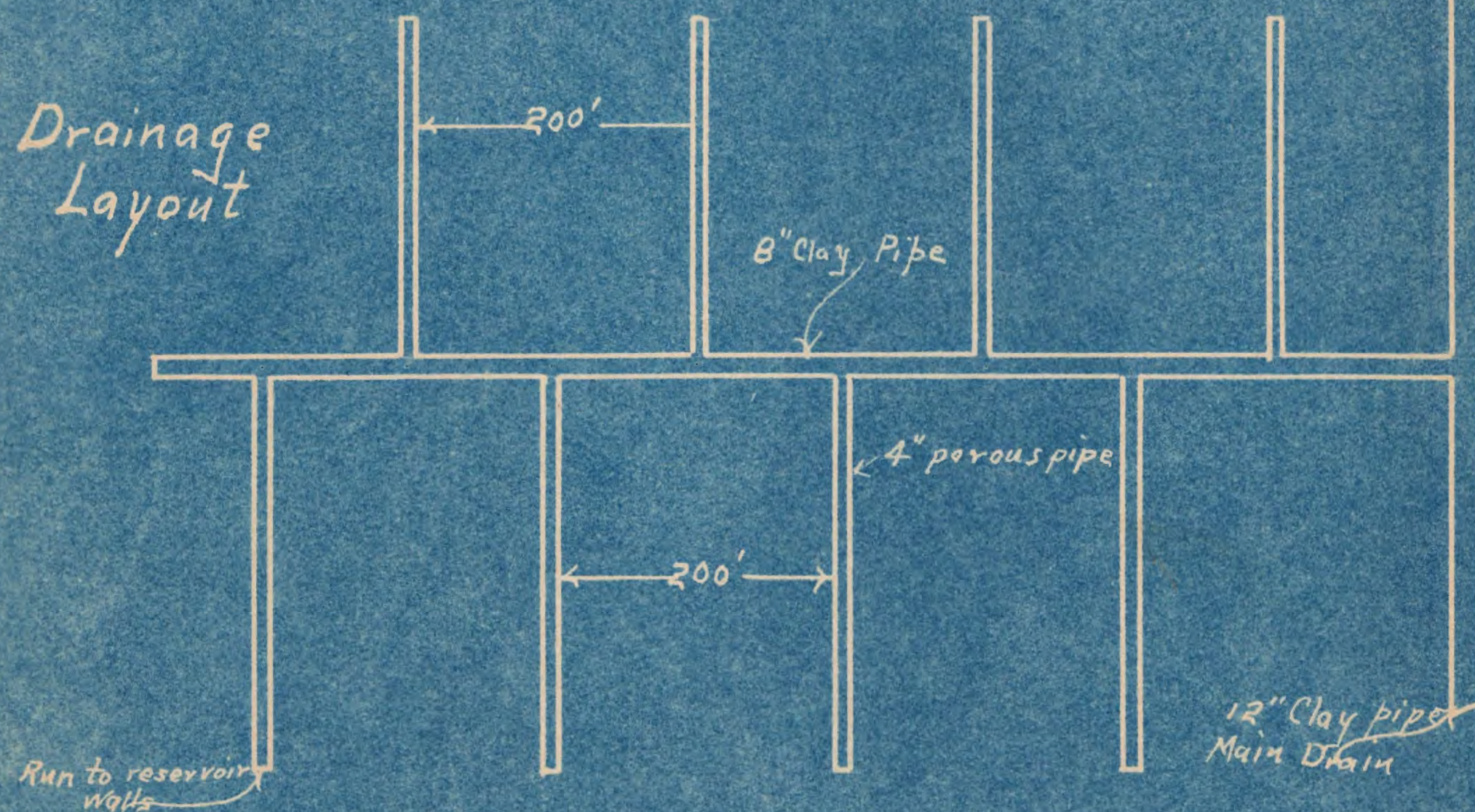
Proposed for Luttrell Hill Reservoir

Scale $\frac{1}{4}$ " = 1 foot

Section of Drainage System

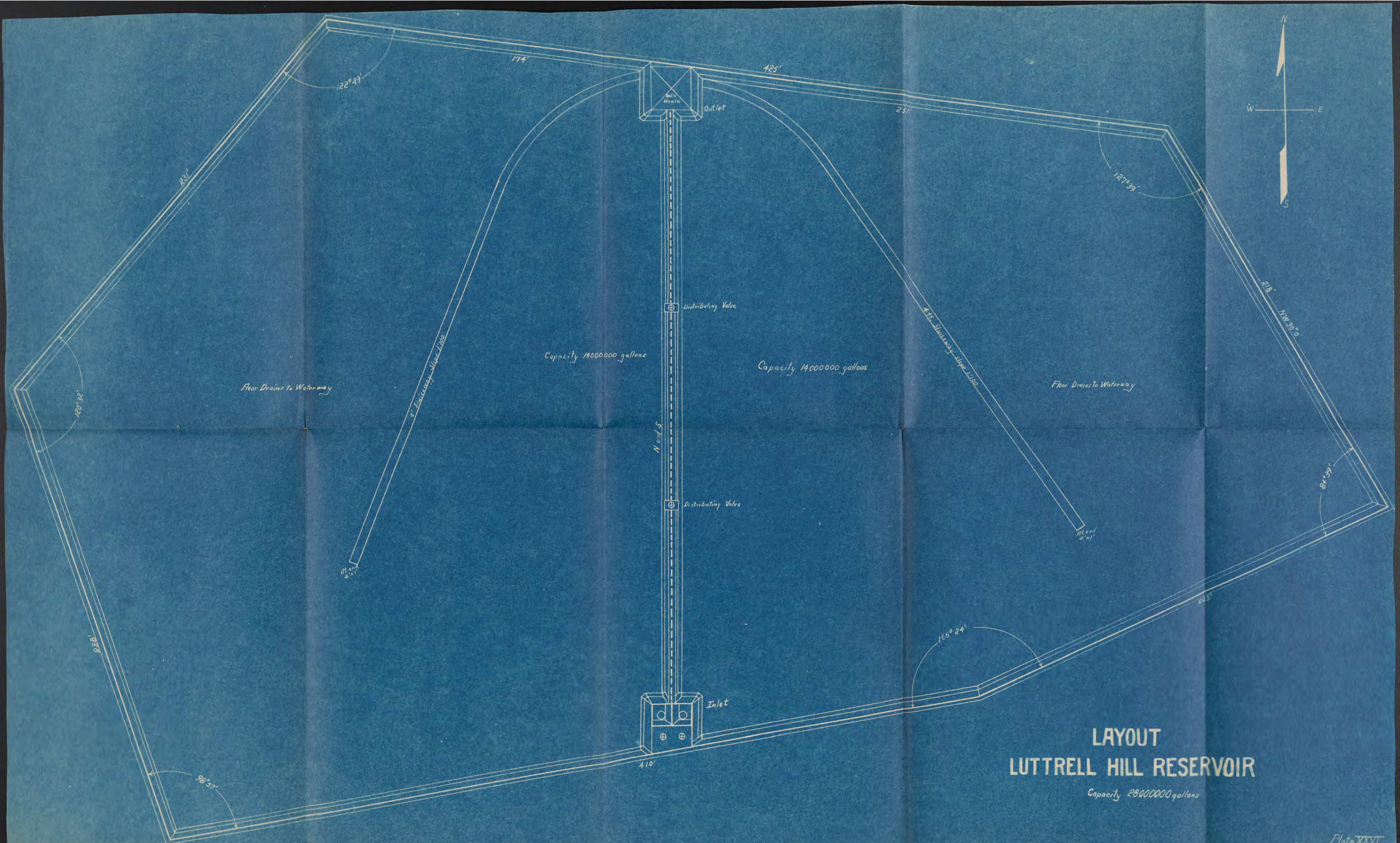


Drainage
Layout



DRAINAGE
Under Reservoir Floor
LUTTRELL HILL

To out side



LAYOUT
LUTTRELL HILL RESERVOIR
Capacity 28000000 gallons