PRELIMINARY REPORT OF WATER POWER
DEVELOPMENTS ON THE
HIWASSEE RIVER
BETWEEN
MURPHY and APPALACHIA, N.C.

by

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and

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Water Power Developments on the Hiwassee River between Murphy and Appalachia, N.C.

The work done in this thesis is similar to that required by any party or company or corporation which is considering developing the power of a stream. However, all information on this project was secured from government bulletins, reports and former surveys and not by personal survey. Therefore, this report could not be accepted as a basis for actual and definite conclusions. It is, nevertheless, a sample of what is required before any steps are taken to develop a stream's power. Methods of calculation and procedure are standard. The sources of information are given in the bibliography.

The method of procedure was this: U.S. Geological Topographical maps of the vicinity were secured and the nature of the terrain studied. Cross sections of the river were also secured at several places. From this material two dam sites were selected, one at Coleman where a dam of 170 ft. will back water to Murphy and one at or near the mouth of Shoal Creek where a dam of 117 ft. will back water to the Coleman dam. The back water curve was assumed a straight line, horizontal, from the crest of the imaginary dams up stream to its intersection with the bed of the river. A gaging station is located at Murphy but there is none near Coleman or Shoal Creek. From these readings as recorded in government bulletins the mean monthly flow at Murphy for the period 1900 - 1920 was figured and a hydrograph plotted. It is assumed that the flow of the river at different points along the river varies directly as the drainage areas for these points. The drainage area for Murphy is given in the government bulletins and that for the

Coleman and Shoal Creek sites was measured by planimeter on the topographical maps. From this material the hydrograph or rather the flow at Coleman was determined. The primary power is determin mined by the minimum flow reached in any month during the entire twenty year period. This minimum flow occured in 1904. See Fig. 2. Another dry year was 1913. Drawdown curves were plotted between acres covered and head on plant. The data for acres covered covered at different heads was secured from a former survey as the contours on the topographical maps were off badly. The curve between acre feet drawdown and head on dam was then plotted and it was determined how much continuous flow of water was available the year round and 24 hours a day. Mass curves and another method, which will be explained in detail later, were used in this determination. The secondary power was figured from duration curves and normal head on the plant. The enstalled capacity of the project was based on a 50% load factor and 1.6 times the total of primary and secondary power. From a study of Mr. Wegmann's " Design and Construction of Dams" a profile was selected and calculations made for the volume of the dam. The cross section of the river at Coleman showed that the dam would be 950 ft. long. As this territory is rocky it was assumed that excavations would not have to exceed 10 ft. to reach rock. So the cubic yards of excavation was figured on this basis. The length and height of spillway had to be figured to care for the maximum flood which might ever occur. From the government bulletin it was found that in 1899 a flood of 50,000 sec, ft, occured. To be safe a factor of 2 is used and the conclusion is that the spillway must care for 100,000 sec. ft./ day. Also from a curve in Turneaure -Russell's "Public Water Supplies" it was found that a flood might possibly occur to equal 96,000 sec. ft./ day. The size of tainter gates

gates was next figured and the volume to be occupied by them was subtracted from the volume of the solid dam as figured. On account of so great a possible flood the spillway extends from one bank to the middle of the stream. This makes it necessary to figure aprons for one side and the bottom of the stream. As it was possible to put the power house near the middle of the stream it was figured to be built into the dam itself. An estimate was made by a local contractor and architect at 20¢ per cubic foot of volume. This estimate may seem high but was taken so on account of so much solid work and complicated forms. The cost of turbines, generators, switching, transformers and auxiliaries was figured from handbooks on the installed capacity basis. The last item to figure was the cost of water rights. The acres covered has been determined. But the acres which the stream actually occupies before a dam has been erected do not have to be purchased. Only the land flooded has to be bought. So by determining the length of the river affected by back water and subtracting this from the acres covered the acres to be bought is estimated. Mr. R.W.Coward gave an estimate of $50.00 per acre as a possible average cost of this land. After all the above calculations and determinations are made and the costs totaled, about 15% of this sum is added for engineering, contingences and legal advice. Since it would take about two years to construct and get into operation such a project interest for two years must be added in as part of the capitol cost. This total capitol cost divided by the installed capacity gives the cost per K.W. This final figure determines whether or not a project is practical. There would be no trouble about marketing power developed in this vicinity so that a cost of $125.00 per K.W. would be very reasonable. Even a higher cost than this might not be exemplitory.

SELECTING THE DAM SITES

From a study of the United States Geological Topographical Maps of the Hiwassee River below Murphy, N.C. and "Water Powers of North Carolina" Bulletin No. 20 together with cross sections of the river at several points, two good sites for dams were selected. The upper one, Coleman, will afford a dam 170 feet high and 950 feet long which will back water to the dam at Murphy. The lower site, Shoal Creek, will afford a dam of 117 ft. height and 610 ft. length. The lower dam will back water to the Coleman dam. The terrain in this country is hilly and rocky. This condition insures good foundation and plenty of rough material. The back water from the low lower sight floods out a considersble portion of the highway and also the village of Ogreeta. But these are small matters and will not mean much to so large a project. The Louisville and Nashville R.R. comes to Appalachia, within 5 miles of the lower site, and roads can be built from this point on. The lower site could be developed first and the power generated by its plant used in constructing the Coleman project. A much more detailed discussion could be made on this topic had a personal survey been made of the stream. See Fig. 1.

COMPARATIVE HYDROGRAPH

The flow of a stream undergoes great changes. From year to year conditions are very different. From month to month the fluctuations in flow are very great. In order to get any kind of conception of the water power value of a stream a study must be made of its flow over a considerable period of time. There were no records of flow of the river at Coleman or Shoal Creek, but daily readings were taken at Murphy and from the government bulletins of 1900 - 1920 gage readings and rating tables furnished data to make a hydrograph for this place. The mean monthly flow for the entire twenty years was plotted against time. The flow was expressed in cubic feet per second as ordinates and the time in months as abscissae. While this does not give the conditions at Coleman or Shoal Créek, it does give comparative results. It is assumed that the flow at Coleman will equal the product of the flow at Murphy times the ratio of the drainage area at Coleman to that at Murphy. The bulletin gives 410 sq. mi. drainage area at Murphy. By drawing off the drainage area at Coleman on the topographical map and planimetering it, its drainage area was found to be 410 plus 505 or 915 sq. mi. Every reading taken at Murphy multiplied by 915/410 gave the comparative reading at Coleman. The drainage area at Shoal Creek was found to be 47.7 sq. mi. greater than that at Coleman. Since the flow of water in the stream below the Coleman dam is going to be regulated to a large extent by the discharge from the turbines at that site, a comparative hydrograph could not be made for Shoal Creek but another method, which will be discussed later, was used. Sample tables and calculations for the hydrograph will be shown on page 6.

Daily Gage Height, in Feet, of Hiwassee River at Murphy, N.C.

1900 (Day)	Jan	Feb	Mar	Apr	May	June	July	Aug	etc.
1	5.50	5.12	7.00	5.90	6.10	5.40	6.80	5.80	

etc. etc. etc.

Rating Table for Hiwassee River at Murphy, N.C., for 1900.

Gage Height (Feet)	Discharge (Second-feet)	Gage Height (Feet)	Discharge (Second-feet)	Gage Height (Feet)	Discharge (Second-feet)	etc.
4.80	310	5.60	861	6.40	2,025	
4.90	362	5.70	977	6.50	2,200	
5.00	415	5.80	1,093	6.60	2,375	
5.10	481	5.90	1,209	6.70	2,550	
5.20	547	6.00	1,325	6.80	2,725	
5.30	613	6.10	1,500	6.90	2,900	
5.40	679	6.20	1,675	7.00	3,075	
5.50	745	6.30	1,850	7.20	3,425	

Take for example the 1st. day of July 1900. The gage height was 6.80 feet. For this height the rating table shows a discharge of 2,725 cubic feet per second. By adding the values for each day and dividing by the number of days in the month we get the mean flow in sec. ft. for the month. These are the values plotted as ordinates on the hydrograph.

Other tables in the government bulletins give the maximum, minimum and mean flow for the month already figured out. The same table gives the run-off in sec.-ft. per sq. mi. so that the flow at any point along the river can be figured by multiplying this value by the area of the drainage for the point. The hydrograph plotted from these results is the Comparative Hydrograph.

PRIMARY POWER

On account of the storage afforded by the reservoir, the primary power of a hydro-electric plant does not depend upon the minimum flow of the stream, but on that minimum flow plus a certain amount which may be economically drawn out of the reservoir.

The method used in determining the primary flow of the stream is as follows:

The draw-down curve Fig. 4, having been plotted, is carefully considered to determine the economical limit to which the water in the reservoir may be drawn down in order that the highest possible efficiency of operation may be realized. When this draw-down limit has been decided upon, the amount of water which would have been drawn out of the reservoir may be read from the curve. This reading is usually in acre feet (i.e. an acre of water 1 ft. deep). In this method of determining primary flow, the available storage is expressed in terms of the average flow in second feet per square mile of drainage area which that amount of water could maintain for a period of one month. For example, take the most economical limit of draw-down to be 25ft. on a 150 ft dam and the amount of water in the reservoir above the 125 ft. level is 50,000 acre feet. The available storage is then 50,000 acre feet. Considering this amount of water to flow during the period of one month the average flow in acre feet per day is 50,000/30.4 and in cubic feet per second 50,000/30.4 x 1.9835, 30.4 being the average number of days in a month and 1.9835 the conversion factor of acre feet per day to cubic feet per second. If the result is divided by the drainage area in square miles the quotient will be the flow in cu. ft. sec. per sq. mi. For a drainage area of 1,000 sq. mi. this flow would be 0.868 sec. ft. per sq. mi. This is the rate, over and above the rate of inflow, at which water might be drawn from the reservoir over a period of one month without drawing down the water level more than 25 ft.

Now to consider the natural flow of the stream. This is also given in terms of the average monthly flow in sec. ft. per sq. mi. Suppose the monthly flows for the driest year on record are as follows:

Jan.	6.09	May	2.57	Sept.	0.59
Feb.	6.83	June	1.70	Oct.	0.63
Mar.	5.23	July	0.82	Nov.	0.89
Apr.	2.17	Aug.	0.76	Dec.	2.67

The dry period of the year extends from July to December, and the average flow for that period will be 0.738 sec. ft. per sq. mi. However, if the available storage is added to the natural flow, the average rate at which water might be used would be 0.9116 sec. ft. per sq. mi., an increase of 0.1736 sec. ft. per sq. mi. The availabe storage would be divided up as follows:

Month	Natural Flow	Drawn from Storage	Regulated Flow
July	0.82	0.0916	0.9116
Aug.	0.76	0.1516	0.9116
Sept.	0.59	0.3216	0.9116
Oct.	0.63	0.2816	0.9116
Nov.	0.89	0.0216	0.9116
Mean	0.738	Total 0.8680	

At the end of the dry season, the reservoir is drawn down to its limit and must be again filled. This is done by the surplus flows of the succeeding months. In the year under discussion it could be completely filled during December and still the average flow let thru the turbines would not go below the regulated flow of the preceeding month. If this were not the case, the reservoir would be only partially filled in December and completely filled during the following month or months. In the present case, the natural flow of December would be divided so that a flow of 0.868 sec. ft. per sq. mi. would go to filling the reservoir and the remaining 1.802 sec. ft. per sq, mi. allowed to flow thru the turbines.

From a study of the hydrograph it was found that the driest years in the 20 yr. period were 1913-14. The data recorded below is that actually used in figuring the primary flow and power. Since this data is used also for determining the regulated flow at Shoal Creek, the Shoal Creek calculations

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will also be recorded and referred to when the Shoal Creek project is discussed in part two.

Explanation of the table.

Column 1 - Natural flow of stream in sec. ft./sq.mi.
" 2 - Drawn from storage "
" 3 - Total draw-down to date "
" 4 - Stored in reservoir "
" 5 - Regulated flow from Coleman "
" 6 - " " " in sec. ft. per sq. mi. difference in drainage area.
" 7.- Total flow to Shoal Creek in sec. ft. per sq. mi. difference in drainage area.
" 8.- Total flow to Shoal Creek sec.ft.per sq.mi.

" $6 = 5 \times \frac{915}{47.7}$ $7 = 6 + 1$ $8 = 7 \times \frac{47.7}{962.7}$

The drainage area at Coleman is 915 sq. mi. That at Shoal Creek is 47.7 sq. mi. more or a total of 962.7 sq. mi.

1913 1914	1	2	3	4	5	6	7	8
J	2.56	0	0	1.63	2.65			2.56
F	2.93	0	0		2.93			2.93
M	7.15	0	0		7.15			7.15
A	2.73	0	0		2.73			2.73
M	2.05	0	0		2.05			2.05
J	1.89	0	0	1.63	1.89			1.89
J	1.09	0.03	0.03	1.60	1.12	21.50	22.59	1.118
A	0.937	0.183	0.213		1.12	21.50	22.437	1.113
S	0.777	0.343	0.556		1.12	21.50	22.277	1.107
O	0.738	0.382	0.938		1.12	21.50	22.238	1.103
N	0.714	0.406	1.344		1.12	21.50	22.214	1.101
D	0.945	0.175	1.519		1.12	21.50	22.445	1.112
J	1.010	0.110	1.629	0.001	1.12	21.50	22.510	1.114
F	1.720			0.601	1.12	21.50	23.220	1.151
M	1.830			1.311	1.12	21.50	23.330	1.155
A	2.66			1.630	2.341	45.00	47.660	2.360
M	1.26				1.26			1.260
J	0.822	0.249	0.249	.	1.071	20.58	21.402	1.062
J	.727	0.344	0.593		1.071	20.58	21.307	1.056
A	0.698	0.373	0.966		1.071	20.58	21.278	1.054
S	0.507	0.564	1.530		1.071	20.58	21.087	1.045
O	0.971	0.100	1.630		1.071	20.58	21.551	1.068
N	1.250			0.179	1.071	20.58	21.830	1.082
D	4.100			1.630	2.649	50.80	54.900	2.720

Note: The lowest value in column 5 determines the primary flow. This is 1.071 sec. ft. per sq. mi. of drainage area. The flow will never go below this.

A table similar to the one above was made for 1903-1904 as it was also a dry season. But the period covered by

the above was the driest

the above was the driestmand, therefore, governed what the primary flow should be. The product of 1.071 x 915(the drainage area at Coleman) gives 980 sec. ft. as the minimum total flow for this development. It would be proper to call this 1,000 sec. ft. The theoretical horse power will be the product of 1,000 x 140 (minimum head) divided by 8.8 = 15,800

Primary Power for 100% load factor 15,800 H.P.

50% load factor 31,600 H.P.

Turbines required 2 15,000 H.P.

Alternators @ 95% eff. 2 10,650 K.W.

Rated in K.V.A. 2 13,000 K.V.A. @ 80% p.f.

The actual primary power which could be guaranteed 12 hours a day with 90% turbine and 95% alternator efficiencies would be 31,600 x .9 x .95 x 746 = 20,150 K.W.

SECONDARY POWER

Secondary power is the difference between the total power the stream can afford and the primary power. The total power developed is figured from a duration cürve (see fig. 3) This cürve is made by plotting the monthly flows against time. The mean monthly flows in sec. ft. were arranged in order of magnitude and not in order of occurance. The entire 20 yr. period is used. The total flow available for 50% of the time is taken as that shown on the curve in the middle of the period. This was done and the value found to be 1,900 sec. ft. Assuming that the head would be normal (170 ft.) the power for 100% load factor figures 1,900x 170 x 1/ 8.8 = 36,700 h.p.

For 50% load factor this figures 73,400 H.P. The secondary is, therefore, 73,400 - 31,600 = 41,800 H.P.

INSTALLED CAPACITY

The total theoretical horse power which is available in the stream for 50% of the time has been figured at 73,400. It is not practical to install merely for this H.P. Assuming 90% efficiency for the turbines, penstock etc., the value changes to 66,000 H.P. It has been found that this value should be multiplied by 1.6 to give the practical capacity to install for. In extremely wet seasons a great amount of power would go to waste for want of machinery to convert it to electri-energy. Practice has founfd that 1.6 is the economical factor to use. The installed capacity would be, therefore, 73,400 x 1.6 = 105,000 H.P. Seven 15,000 H.P. turbines would care for this. Assuming an alternator efficiency of 95% the total K.W. capacity would be $\frac{105,000 \times 746 \times .95}{1,000}$ = 74,400 K.W. This would be 93,000 K.V.A. at 80% p.f, Seven 13,000 K.V.A. alternators would work with the seven 15,000 H.P. turbines.

Design of Dam

The type of dam to be used was decided upon by a study of Mr. Wegmann's "Design and Construction of Dams". Practical Profile No. 2 was selected (see fig. 7). All dimensions were converted to those for 170 ft. dam. For this profile, the width varies directly with the height. All principles of dam construction are met with in this type. Since this section of country is very rocky there will be no trouble in securing a good foundation and bond with the banks of the stream. From a cross section of the stream at Coleman (see fig. 6) the maximum length of dam was found to be 950 ft. It was decided that a safe estimate on the depth necessary to dig for rock foundation would average 10 ft. The cross section was, therefore, outlined with a margin of 10 ft. and vertical lines drawn at convenient intevals to divide the dam into sections. The altitude for each section was taken at the center of the section and from the table accompanying the profile the width of dam for this height was figured. The product of this width times the length of section and 10 (depth of excavation) gave the volume of excavation in cu. ft. for this section. This was figured for each section and the total excavation for the dam found to be 30,054 cu. yds. The volume of a solid dam was next to be figured. The table mentioned above gave volumes of dam at different distances down from the top. Using the same sections mentioned above the volumes for the corresponding heights were figured from the table (volumes for 1 ft. length of dam) and this times the length of section gave the total volume of section. The total of these sections was found to be 130,581 cu. yds. From this will have to be taken the volume of tainter gate space and other construction other than solid dam.

Before the length of spillway could be figured a study of flood flow of this and other streams was made. From the government bulletins it was found that in 1899 the largest flood for many years occured. It was 50,000 sec. ft. From a study of "Public Water Supplies" by Turneaure and Russell it was figured that it was very slightly possible and not at all probable that a flood flow of 105 sec. ft. per sq. mi. might occur at some time. This figured 96,000 sec. ft. Therefore, it is safe to take 100,000 sec. ft. as the maximun flood ever possible. Using the equation $q = C\ 2/3 \sqrt{2g}\ h^{3/2}$ where q is the discharge is sec. ft. per foot of spillway, C is the coefficient for the spillway (weir) and h is the head on the spillway. Setting C = .85, h= 13 ft. and solving, the q = 212 sec. ft. per ft. of spillway. Dividing the 100,000 sec. ft. flood flow by 212 gives 472 ft. of spillway with 1.00 for the coefficient of contraction. Using 93% for this coefficient the length of spillway required for the flood flow is 507 ft. Taking 20 ft. as the width of tainter gates and dividing 507 by 20 gives 25.3 tainter gates. This means that 26 tainter gates 20 ft. wide and 13 ft. high will take care of the flood flow. See fig. 8. It will be necessary to have a four foot pier between each pair of tainter gates, so that the total length of spillway will be 26 x 20 plus 25 x 4 = 620 ft. leaving 330 ft. for power house. Data from the Knoxville Boiler and Iron Works showed that 5/8" boiler plate weighed 23# per sq. ft. and came in 14 ft. widths. On account of the slight curvature in tainter gates the 14 ft. width would work fine for the 13 ft. gates. The cost of the boiler plate alone for 26 gates would be 26 x 20 x 14 x 23 x $0.045 (cost per #). = $7,540 The brace and frame work was estimated at about $7,500 and the extra concrete work and hoisting mechanism at $10,000 making a total of about $25,000 for tainter gates. The volume occupied by the tainter gates to be taken from that of the solid dam will be the area of cross section of the dam for 13 ft. down times the length of spillway devided by 27. = $\frac{203 \times 620}{27}$ = 4,670 cu. yds.

Volume of solid dam	130,581 cu. yds.
Tainter gate space	4,670 cu. yds.
Volume of concrete	125,911 cu. yds.
Cost @ $10.00	$1,259,110

Since the spillway will extend from one bank to the middle of the stream an apron will have to be constructed on that bank and in the bed of the stream. It was estimated from the cross section of the stream that the apron at the side will take the form of section of cone 400 ft. in altitude and 10 ft. upper base with 70 ft. lower base. The section is so near that of a trapezoid that its area may be figured as such. Assuming that 10 ft. excavation will have to be made to strike good rock foundation we figure the excavation for this apron $\frac{400 \times (10 \text{ plus } 70) \times 10}{2 \times 27} = 59,300$ cu. yds. It is only necessary that the apron be 3 ft. thick reinforced for heat and compression. Therefore, the concrete required for it will be 5,930 x 3 = 17,790 cu. yds. The bottom apron should be 300 ft. wide by 50 ft. long. The excavation would be $\frac{300 \times 50 \times 10}{27} = 55,500$ cu. yds. and the concrete 5,550 x 3 = 16,650 cu. yds.

Cost of concrete for aprons reinforced for heat and compression is $12.00 per cu. yd. Total cu. yds. 17,790 plus 16,650 = 34,440 cu. yds. Cost 12 x 34,440 = $413,000

The total excavation for dam and aprons is

for dam	30,054	cu. yds.
side apron	59,300	'' ''
bottom apron	55,500	'' ''
	144,854	'' ''

At $2.00 per cu. yd. the cost of excavation will be

$289,708

DESIGN OF PLANT

Since the spillway occupies only 620 ft. of the 950 ft. dam, the power house can be placed near the middle of the stream. See fig. 9. The total length of the power house was figured to be 180 ft. to accommodate seven turbines. The width of the turbine room should be 30 ft. and the transformer room should be the same. The height of the down stream side would be about 100 ft.

and the average width may taken (for the purpose of figuring volume) as 50 ft. The total volume of the building would be 100 x 50 x 180 = 940,000 cu. ft. A cost of 20¢ per cu. ft. of volume was given by Barber and McMurry, Architects as a safe cost of such a building. This figured $188,000. The cost of the building with finished woodwork and offices should be $200000 .

From a handbook on costs of mechanical and electrical equipment the following data was secured:

Water wheel, gates and governor	$4.35 per K.W.
Alternators and exciters	4.20
Switching and wiring	2.63
Transformers	2.36
Heating and miscellaneous	.26
Total cost per K.W. installed capacity	$13.80

Cost of Mechanical and Electrical equipment for 74,400 K.W. plant would be 13,80 x 74,400 = $1,027,000.00

WATER RIGHTS

The agricultural value of land in this section is very small, but should a farmer hear that his land was wanted for the development of power, its value would greatly increase. However, it is right and just that the owners of such land be paid more than the actual agricultural value for it. There is a water power value also. A price of $50.00 per acre would be safe enough, according to Professor R.W.Coward, University of Tennessee. Only land which would be flodded should be bought. The total drainage area less the area of the natural bed of the stream will be the land which has to be purchased. From fig. 4 the total acres covered is 3,670. To be safe for flodds this should be taken as 4,000 acres. The bed of the stream is 300 ft. wide and 16 mi.

or $\frac{300 \times 16 \times 640}{5,280} = 581$ acres. Therefore, the acres to be purchased would be 4,000 - 581 = 3,419 At $50.00 per acre this figures $ 170,950.00 as cost of water rights.

TABULATION OF COSTS FOR COLEMAN

Water Rights....3,419 acres @ $50.00 per acre...			$ 170,950.00
Dam			
Excavation...144,854 cu. yds. @ $2.00 /cu.yd...			289,708.00
Concrete...125,911 cu. yds. @ $10.00/cu.yd...			1,259,110.00
Aprons(concrete)..34,440 cu.yds./$12.00........			413,000.00
Tainter Gates..................................			25,000.00
Total for dam....1,986,818.000			
Power House			
Including crane, offices and interior work.			
2,000,000 cu. ft. volume @ 20¢ per cu. ft..			200,000.00
Equipment			
Turbines, gates and governors..	$4.35	per K.W.	
Alternators and exciters.......	4.20	'' ''	
Switching and wiring...........	2.63	'' ''	
Transformers...................	2.36	'' ''	
Heating and miscellaneous......	0.26	'' ''	
74,400 K.W. installed cap. @	$ 13.80	'' ''	1,027,000.00
Dwellings, shops etc.........................			50,000.00
Roads and Railroads..........................			150,000.00
			$ 3,583,768.00
Engineering, contingencies and legal advice 15%..........................			537.564.20
			4,121,332.20
Interest two years @ 4%			329,706.58
Total capitol cost			$ 4,451,038.00
Cost per K.W. installed capacity of 74,400			$ 59.90

SHOAL CREEK PROJECT

The selection of the dam site at Shoal Creek was made at the same time as that at Coleman, but no other work was done on this project until the first development was finished, that is, intil all calculations had been completed. The height of dam was found to be 117 ft. and the length to be 607 ft.

The drainage area for this site was found to be 47.7 sq.mi. ~~feet~~ greater than that for the upper site, or a total of 915 plus 47.7 = 962.7 sq. mi.

REGULATED FLOW

In case of the development of several power sites on the same stream, the flow into all reservoirs below the upper site will be the regulated flow from the plant immediately above plus the natural flow from the drainage area between the two dams. At all times, accept during the dry period of the year, the regulated flow from the upper dam is at the same rate as the natural flow of the stream, and the combination of the regulated and natural flow into the lower reservoir is at the same rate as the natural flow. In other words, regulated flow differs from natural flow only in dry weather when the water in the reservoir is being drawn down.

The flow in sec. ft. per sq. mi. from the upper plant must be multiplied by the ratio of the upper drainage area to the difference in drainage areas of the upper and lower sites. The result is the flow in sec. ft. per sq. mi. of drainage area between the two sites. To the flow thus obtained is added the natural flow of the river the sum being the total flow into the lower reservoir in sec. ft. per sq. mi. of drainage area between the sites. The product of this flow times the 47.7 (difference in drainage area) divided by the total drainage area of the lower site gives the flow in sec. ft. per sq. mi. of drainage area.

Example:-

Given	D.A. of upper site	1,000 sq. mi.
	D.A. of lower site	1,100 sq. mi.
	Difference of D.A.	100 sq. mi.

To determine the average flow in sec. ft. per sq. mi. to the lower site for the month of August (see discussion of primary flow on pages 7 and 8) .

Regulated flow from upper plant 0.9116 sec.ft./sq. mi.

Sec. ft. / sq. mi. difference in drainage area $= \frac{0.9116 \times 1000}{100} = 9.116$

Natural flow for Aug. 0.760

Total flow to lower site 9.876 sec. ft. per sq. mi. of difference in D.A.

$9.876 \times \frac{100}{1,000} = 0.897$ sec. ft. per sq. mi. of D.A.

The following figures for acres covered are based on a survey of Professor John A. Switzer on the Coleman site. The figures for the same on Shoal Creek site are reduced from those at Coleman in proportion to the respective areas of reservoirs.

H. of Water	Acres covered
1520 ft.	3,670
1510	3,200
1500	2,800
1490	2,350
1480	2,000
1470	1,770
1460	1,530
1450	1,300

AREA COVERED
Shoal Creek

H. of Water	Acres Covered
1342	1,990
1336.575	1,735
1331.150	1,518
1325.725	1,275
1320.3	1,085
1314.875	960
1309.450	830
1304.025	705

DRAWDOWN

H. of Water	Acres	Acre Feet.	
1342	0	0	
1340	3,895	3,895	
1330	16,775	20,670	
1320	12,670	33,340	
1310	9,620	42,960	
1300	7,250	50,210	See Fig. 10.

From a study of the table on page 9, it is easily seen that the driest period extends from June to November, inclusive, of the year 1914. Taking the limit of drawdown for one month to be 22 ft. we see from the above data that the acre feet available is 33,340. This converted to cu. ft. per sec./sq.mi.is 0.575 . Averaging this flow available from the reservoir with values in column 8 page 9 for June thru Nov. 1914 gives the regulated primary flow at Shoal Creek to be 1.157 cu. ft. / sq. mi.

Month	Flow in sec.ft./sq.mi.	Drawn from reserve
June	1.062	0.095
July	1.056	0.101
Aug.	1.054	0.103
Sept.	1.045	0.112
Oct.	1.068	0.089
Nov.	1.082	0.075
	0.575	0.575
	6.942	
Mean	1.157 sec. ft./ sq. mi. D.A.	

Primary Flow 1.157 x 962.7 = 1103.84 cu. ft. / sec.

Primary Power at 80% efficiency $\frac{1103.84 \times 95 \times .8}{8.8}$ = 9,533.25 H.P. The head being 117 - 22 = 95 ft.

SECONDARY POWER

The flow at Coleman for 50% of the time was found to be 1900 sec. ft. On the comparative basis, the flow at Shoal Creek would be $\frac{962.7}{915.0} \times 1900 = 2000$ sec. ft. The total power at 115 ft. average head and 80% efficiency would be $\frac{2000 \times 115}{11}$ 20,909 H.P. The secondary power will be 20,909 - 9,533 =11,376 H.P.

For 50% load factor 80% efficiency	Primary Power	19,066
	Secondary	22,752
	Total	41,818
For 100% efficiency	Primary	23,800
	Secondary	28,500
	Total	52,300
Less 5% loss in head gates, penstocks etc.		2,615
Total H.P. to turbines		49,685

Install 1.6 x 49,685 x 0.9 = 71,546.4 H.P. (turb. eff. 90%)

Or five 15,000 H.P. turbines = 75,000 H.P.

Assume alternator efficiency 95% the total K.W. capacity would be $\frac{75{,}000 \times 746 \times .95}{1{,}000}$ = 53,200 K.W. This would require five 13,000 K.V.A. alternators at 80% pf.

DESIGN OF DAM

The same type dam can be used here as at Coleman. See Fig. 7. All dimensions were converted to those for 120 ft. dam. The volume of solid dam was figured as in the first case and found to be 92,473 cu. yds. The excavation figured 13,072 cu. yds. See Fig. 12 for cross section of stream.

The maximum flood flow may be taken as 105 x 47.7 = 5,000 sec. ft. more that that at Coleman. See page 12.

The total flood flow possible is, then, 105,000 sec. ft. The spillway will, no doubt, have to extend the full length of the dam; that length being only 607 ft. This means that practically the same size and number of tainter gates will be necessary as was figured for Coleman. This is 26 gates 14 ft. x 20 ft. with 25.4 ft. piers at a cost of 25,000 dollars. The volume displaced by these gates, to be subtracted from that of the solid dam, will be different, however. It will be $\frac{14 \times 12 \times 20 \times 26}{27}$ = 3,240 cu. yds. The factor 12 being the width of dam. This leaves a volume of solid dam 92,473 - 3,240 = 89,233 cu. yds. at $10.00 = $ 892,330.00

Side aprons will be necessary for both banks. These figure $\frac{10 \times 30 \times 145}{27}$ = 16,100 and $\frac{10 \times 50 \times 300}{27}$ = 55,500 cu. yds. The bottom apron figured 55,500 cu. yds. and the second side apron figured $\frac{10 \times 30 \times 196}{27}$ = 21,800 cu. yds. The total excavation for dam will then be:-

	Dam proper	13,072 cu. yds.	
	Aprons	16,100	
		21,800	
		55,500	
Cost at $ 2.00 per		106,472 cu. yds.	= $ 212,944.00

Concrete for aprons $\frac{93,400 \times 3}{10}$ = 28,000 cu. yds. This concrete has to be reinforced for compression and temperature. The cost is given as $ 12.00 per cu. yd. Total cost of concrete for aprons 28,000 x 12 =$336,000.00

DESIGN OF PLANT

Since the spillway was designed for the entire length of dam, there is no place for the power house other than down stream about 150 ft. This means that the water will have to be conducted this distance thru conduit of some description.

The construction of a concrete pipe line at Pento Arizona in 1906 as described in the American Civil Engineers Handbook presents conditions somewhat similar to those at Shoal Creek. The conduit was designed to carry a maximum internal head of 31ft. at a velocity of discharge of 250 cu. ft. per sec. and an external head of 10 ft. It was 10.5 ft. in diameter with a minimum wall thickness of 6 in. and was reinforced by 5/8 in. steel hoops 6 in. from center to center and 6 longitudinal rods. It was built without expansion joints.

The Shoal Creek conduit must be capable of carrying a maximum of 6,400 sec. ft. At an average velocity of 9 ft. per sec. this would require a conduit of 711.1 sq. ft. cross section, which requirement would be fulfilled by two pipes each 21.4 ft. in diameter. A modified cross section of the Pento conduit could be used for these pipes. See Fig. 14.

It will be necessary that the top of the Shoal Creek conduit be below the maximum drawdown level, which is about 22 ft. below the top of the dam. Allowing about 33% for water hammer, the maximum internal pressure on the pipe will be 1.33(22 plus 21.33) = 58 ft. approximately. This increase in pressure over that in the Pento conduit may be overcome by increasing the thickness of the walls to 7 inches and by using larger reinforcements placed closer together.

The Pento conduit cost $10.00 per linear foot in 1906. Increasing the cost in proportion to the size and allowing 50% for increase in labor cost and material, the Shoal Creek conduit (both pipes) would cost about $70.00 p.l.f. The total cost of conduit would be 150 x 70 = $10,500.00

The conduit is designed to stand an external pressure of about 10 ft. of earth. Referring to fig. 14, the amount of excavation per linear foot is 50 x 30 = 150 cu. ft. or 50 cu. yds. The conduit plus forebay is 225 ft. long, making a total excavation of 11,250 cu. yds. At $2.00 per cu. yd. this will cost $22,500.00.

On account of the fact that the conduit is not to be covered it is necessary to build a 40 ft. retaining wall for the portion of the conduit not protected by the apron below the dam. The wall should be about 100 ft. long and 2 ft. thick and will contain approximately 300 cu. yds. of concrete. At $10.00 per cu. yd. this cost will be $3,000.00.

The enclosed forebay of reinforced concrete at the end ot the conduit just above the power house must be long enpugh to accomodate five 13.5 ft. penstocks. Allowing for rounded approaches, it should be 75 ft. long. Its height is 21 ft. 4 in. and the width tapering from about 45 ft. at the upper end to about 9 ft. at the lower end. Basing an estimate of cost at 15¢ per cu. ft. of space occupied (about 45,000 cu. ft.) the total will be $ 6,750.00

From the forebay five steel penstocks will conduct the water to the five units installed in the power house below. Each of these must be capable of carrying a maximum of 1280 cu. ft. per sec. which, at a velocity of 9 ft. per sec. requires that they shall be 13.5 ft. in diameter. The static head above the lower end of the penstocks is not more than 100 ft. Allowing 50% for water hammer and 10% for corrosion* the total working head is 160 ft. The following formula from Gillette's Handbook of Construction Costs gives the thickness

*Gillette's Handbook of Construction Costs.

of pipe walls in terms of the diameter, the head, the working stress of the material and the efficiency of the joints.

$$t = \frac{0.217\ D\ H}{S\ E}$$

S for steel pipes is taken as 16,000# per sq. in. Using a double rivited joint, 72% eff., the required thickness of pipe is 1/2 in. while with the lock bar joint, 90% eff., the thickness would be 13/32 in. This comparison does not show much to the favor of the lock bar joint and the double rivited joint being easier and cheaper to install is chosen. According to formula the weight p.l.f. of pipe = (12,5 D t) plus 10# = 1,022.5# . The penstocks will be approximately 100 ft. long. The five will weigh, then, 511,250# . In 1912 the cost of steel pipe in place (without anchors, extra fittings etc.) was $3.39 per cwt.* Allowing for a 50% increase in cost of materials, labor etc., the five penstocks would cost approximately $25,500.00 . Allowing $1.00 per ft. for foundations and anchorage there would be an additional cost of $500.00, making a total of $26,000.00 as cost of penstocks.

The power house is figured at 130 ft. length, 60 ft. width and 100 ft. height, making a volume of 780,000 cu. ft. at a cost of 20¢ per cu. ft. is $156,000.00 . This sounds reasonable, since the Coleman power house was to accomodate seven units of the same size as the five for this plant.

The installed capacity is 53,200 K.W. and the cost per K.W. installed capacity for electrical and mechanical equipment has been given as $13.80 . The total cost will be, therefore, $ 735,000.00 .

WATER RIGHTS

The total acres covered will be 2,000 . The area of the river bed 8 miles long and 220 ft. wide gives 213 acres. Total to be purchased 2,000 - 213 = 1,787 at \$50.00 per acre gives \$ 89,350.00 .

TABULATION OF COSTS FOR SHOAL CREEK

Water Rights 1,787 acres @ \$50.00 per acre..		\$ 89,350.00
Dam		
Excavation 166,472 cu.yds. @ \$2.00per..		212,944.00
Concrete 89,233 cu. yds. @ \$10.00 per..		892,330.00
Aprons (concrete) 28,000 cu.yds. @ \$12.00		336,000.00
Tainter Gates............................		25,000.00
Power House 780,000 cu.ft. volume @ 20¢ ...		156,000.00
Conduit two 21.4ft. concrete pipe(diameter) 150 ft. long at \$70.00 p.l.f....		10,500.00
Excavation for coudiut and forebay 11,250 cu.yds. @ \$2.00..........		22,500.00
Retaining Wall 300 cu. yds. concrete at \$10.00 per cu. yd............		3,000.00
Forebay 45,000 cu.ft. volume @ 15¢......		6,750.00
Penstocks five 13.5 ft. diameter steel pipe 511,250# @ \$3.39 per cwt...		26,000.00
Equipment		
Turbines, gates and gov.	\$4.35 per W.K.	
Alternators and exciters	4.20 " "	
Switching and wiring	2.63 " "	
Transformers	2.36 " "	
Heating and miscellaneous	0.26 " "	
53,200 K.W. installed @	\$13.80 " "	735,000.00
Dwellings. shops etc........................		50,000.00
Roads and Railroads.........................		150,000.00
		2,715,374.00
Engineering, contingencies and legal advice 15%..........................		407,306.00
		3,122,680.00
Interest two yrs. @ 4%		249,814.40
Total capitol cost		\$ 3,372,494.40
Cost per K/W installed capacity 53,200 K.W.		\$ 63.35
Cost per K.W. 127,600 K.W. installed cap. of both projects..........................		\$61.63

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Other information was secured from previous surveys made on the river and from individuals and business concerns in Knoxville, Tenn. These are mentioned in the text.

FIGURES AND DRAWINGS

By A. B. Wood.

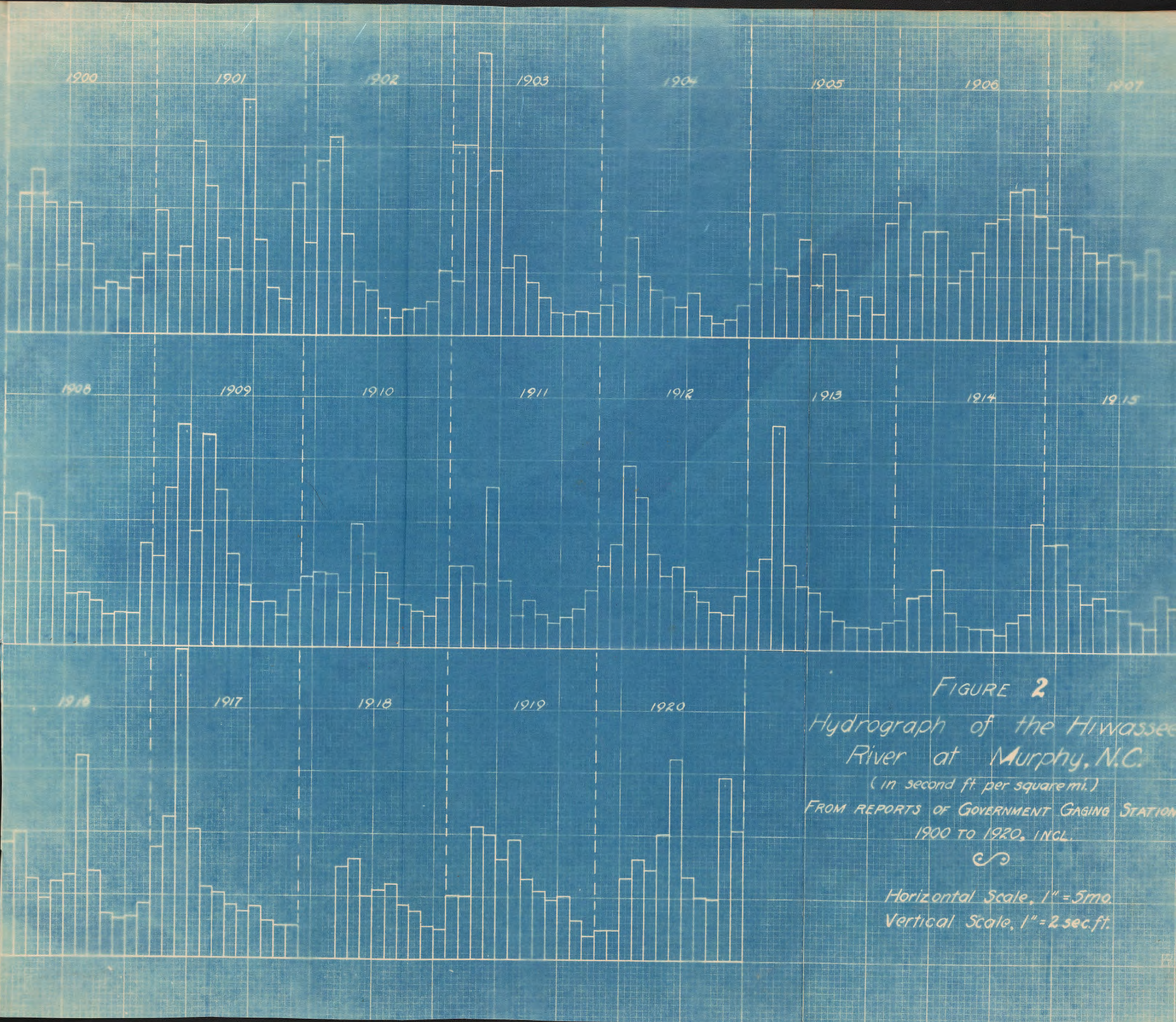


FIGURE 2

Hydrograph of the Hiwassee River at Murphy, N.C.

(in second ft. per square mi.)

FROM REPORTS OF GOVERNMENT GAGING STATION 1900 TO 1920, INCL.

Horizontal Scale, 1" = 5 mo.

Vertical Scale, 1" = 2 sec. ft.

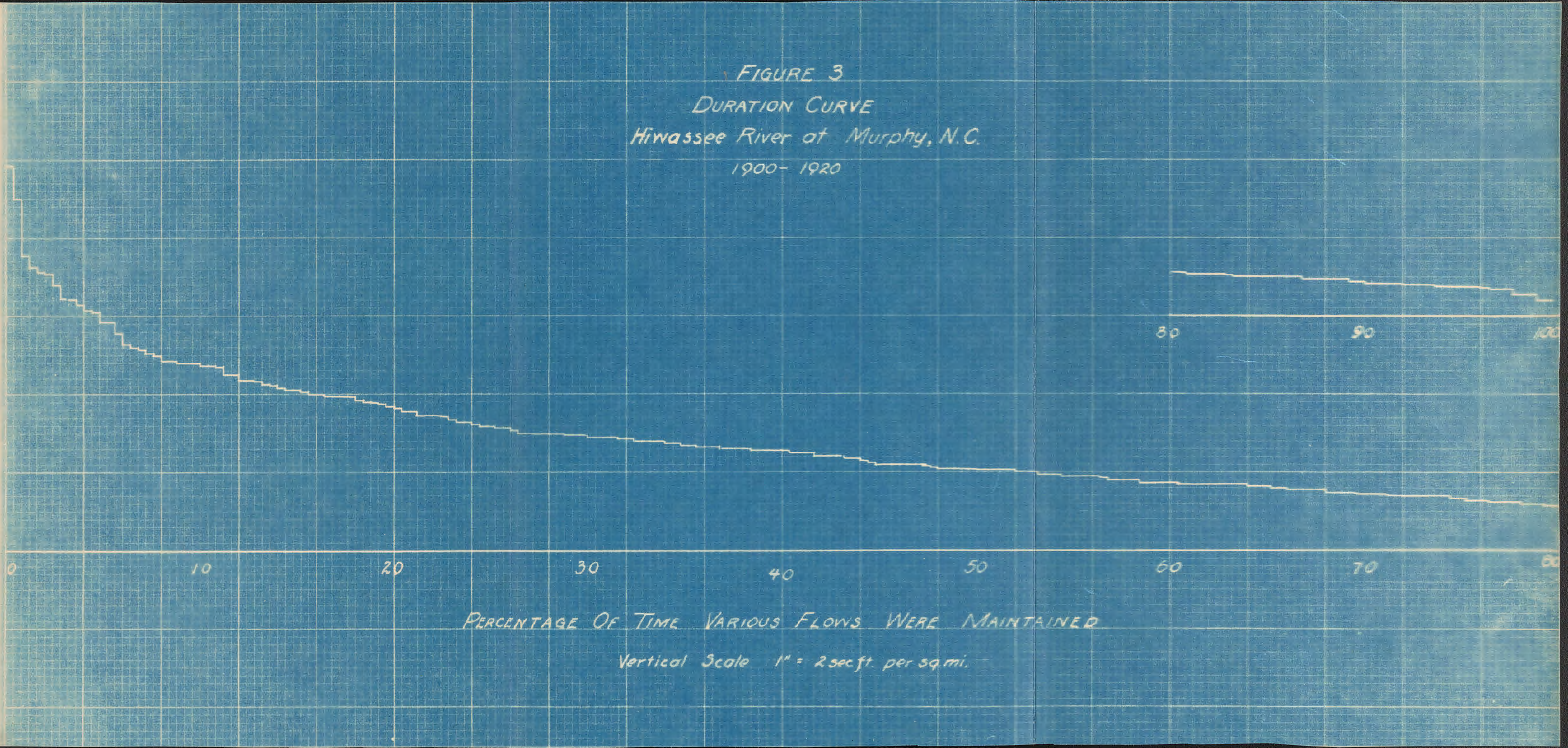
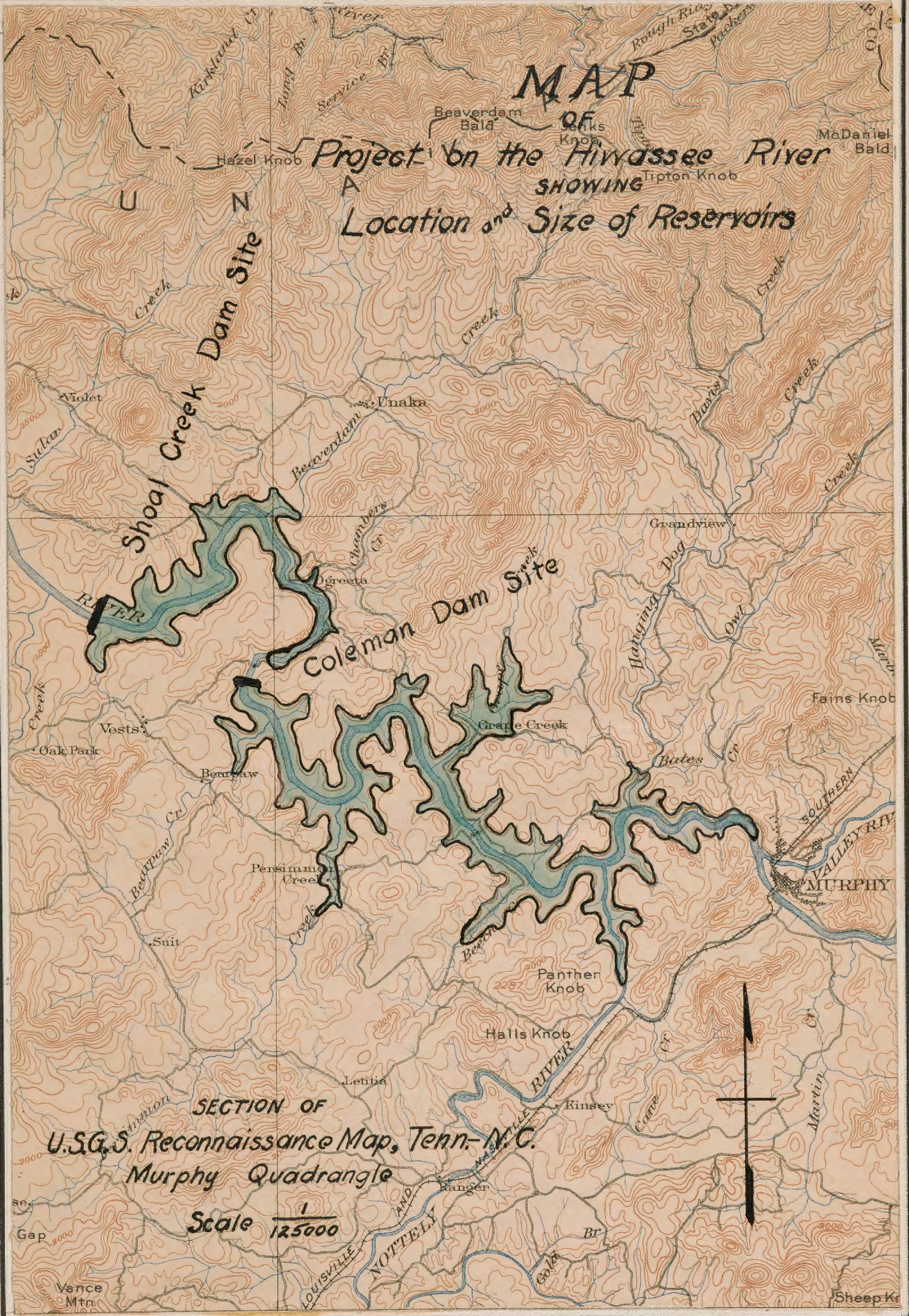


FIGURE 3
DURATION CURVE
Hiwassee River at Murphy, N.C.
1900- 1920



MAP OF
Project on the Hiwassee River
SHOWING
Location and Size of Reservoirs
Shoal Creek Dam Site
Coleman Dam Site
SECTION OF
U.S.G.S. Reconnaissance Map, Tenn.-N.C.
Murphy Quadrangle
Scale 1/125000
Hazel Knob
Beaverdam Bald
Tipton Knob
McDaniel Bald
Violet
Unaka
Ogreeta
Grandview
Grape Creek
Vests
Oak Park
Persimmon Creek
Suit
Panther Knob
Halls Knob
Letitia
Kinsey
Ranger
MURPHY
Fains Knob
Gap
Vance Mtn
Sheep Kr
NOTTELY
LOUISVILLE AND NASHVILLE
SOUTHERN
VALLEY RIV
RIVER
Beaverdam
Chambers Cr
Hanging Dog
Owl Cr
Bates Cr
Martin Cr
Cane Cr
Bearpaw Cr

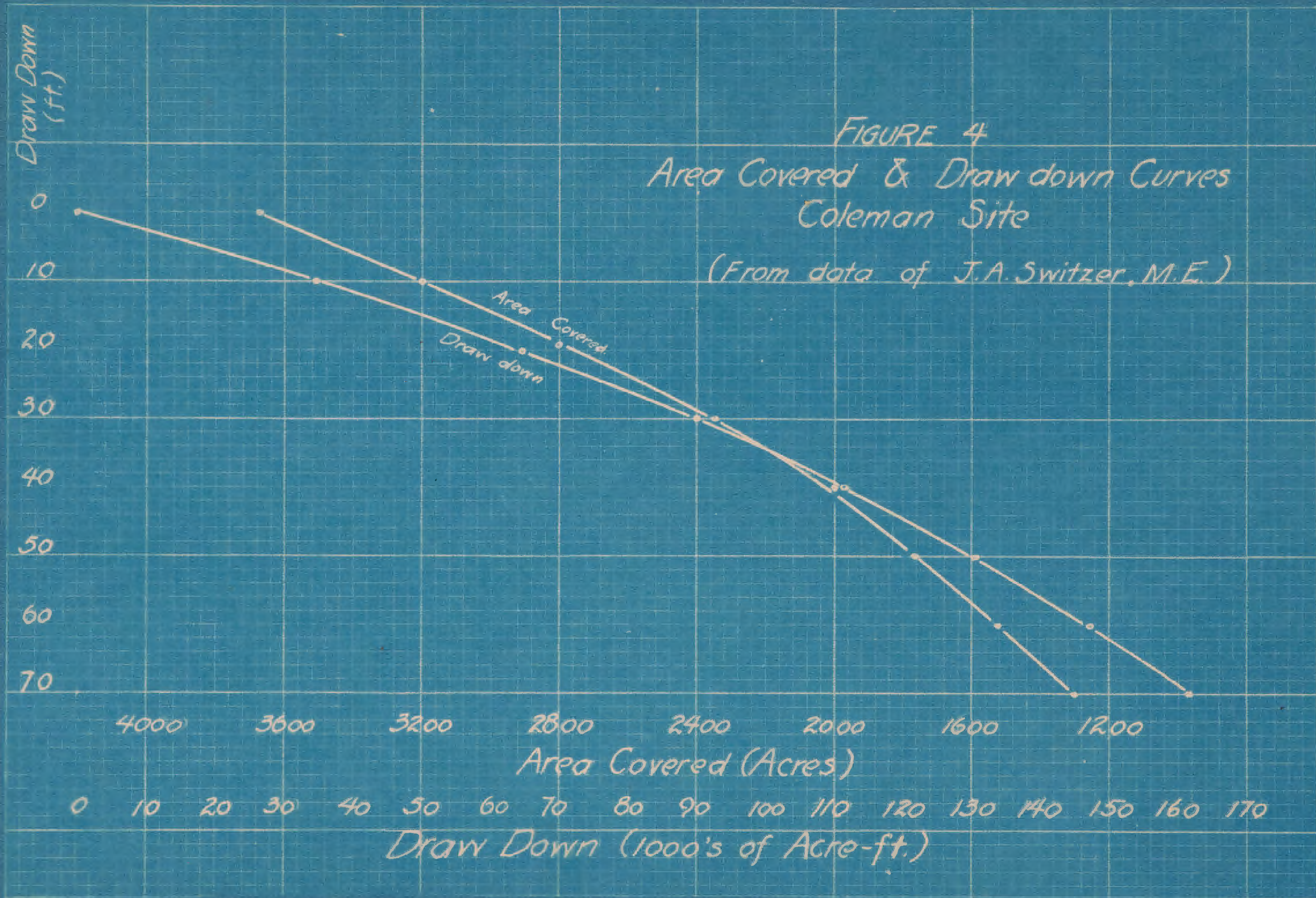


FIGURE 4
Area Covered & Draw down Curves
Coleman Site
(From data of J. A. Switzer, M.E.)
Draw Down (ft.)
0
10
20
30
40
50
60
70
Area Covered
Draw down
4000
3600
3200
2800
2400
2000
1600
1200
Area Covered (Acres)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
Draw Down (1000's of Acre-ft.)

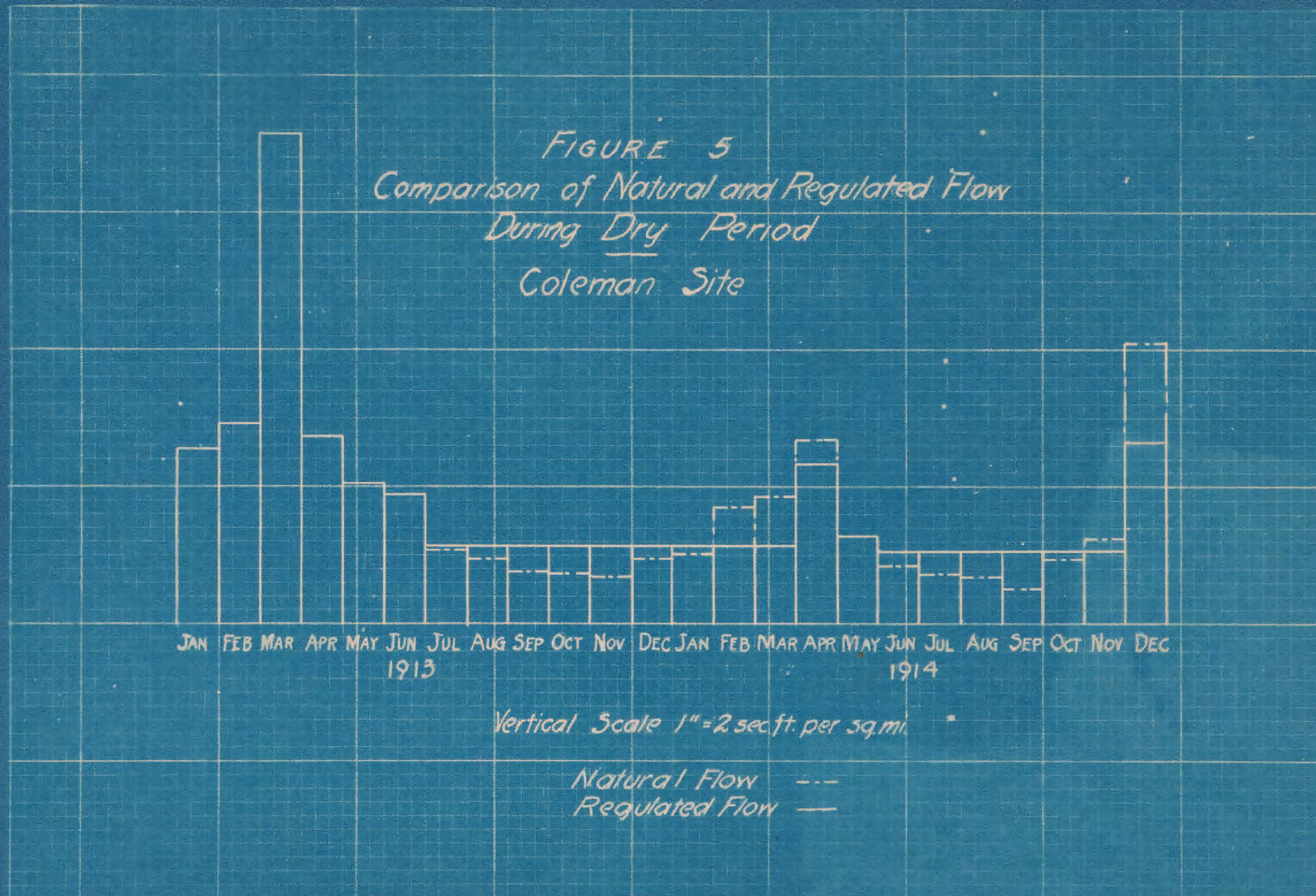


FIGURE 5
Comparison of Natural and Regulated Flow
During Dry Period
Coleman Site
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
1913
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
1914
Vertical Scale 1" = 2 sec ft. per sq. mi.
Natural Flow
Regulated Flow

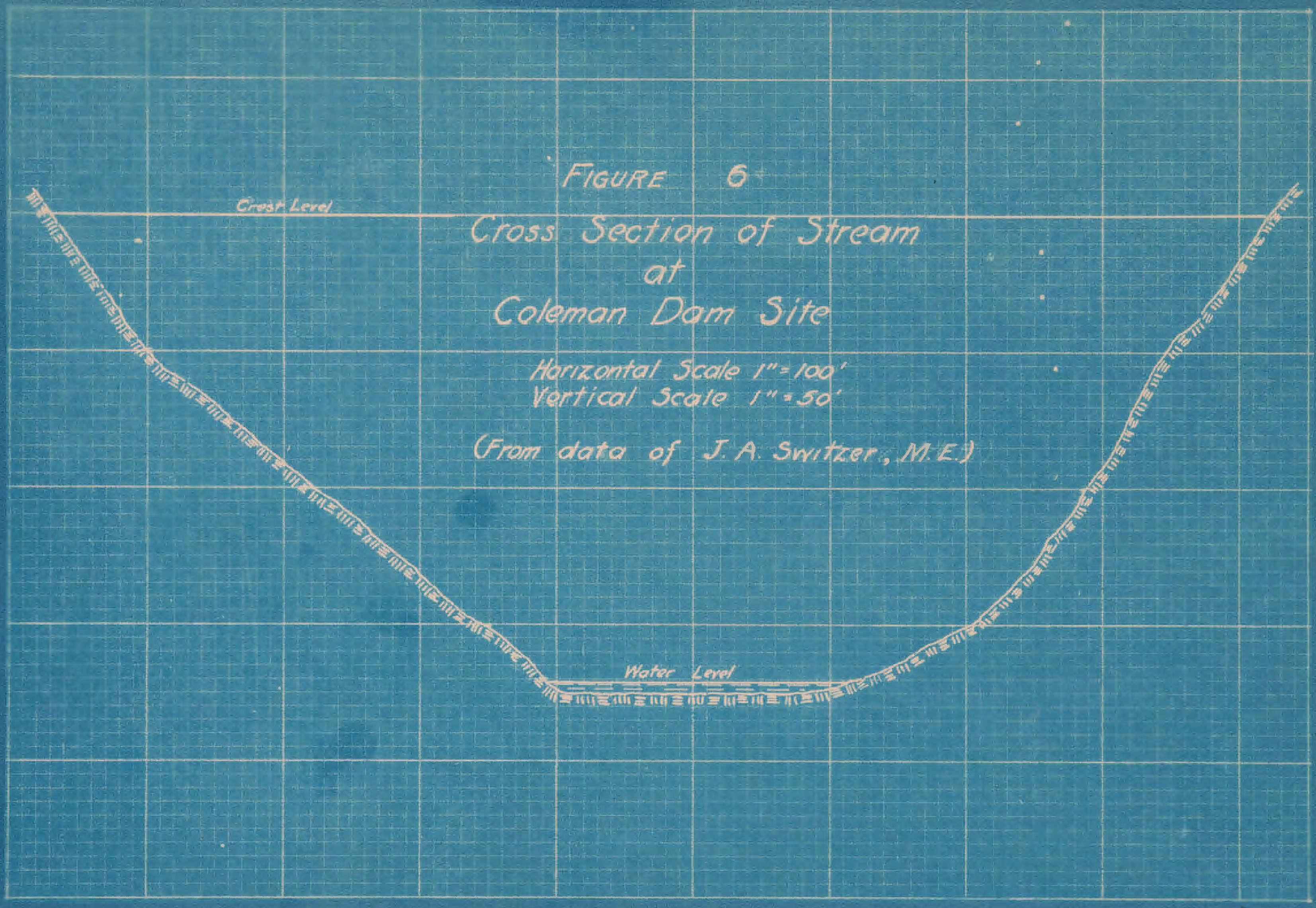


FIGURE 6
Cross Section of Stream
at
Coleman Dam Site
Horizontal Scale 1" = 100'
Vertical Scale 1" = 50'
(From data of J. A. Switzer, M.E.)
Crest Level
Water Level

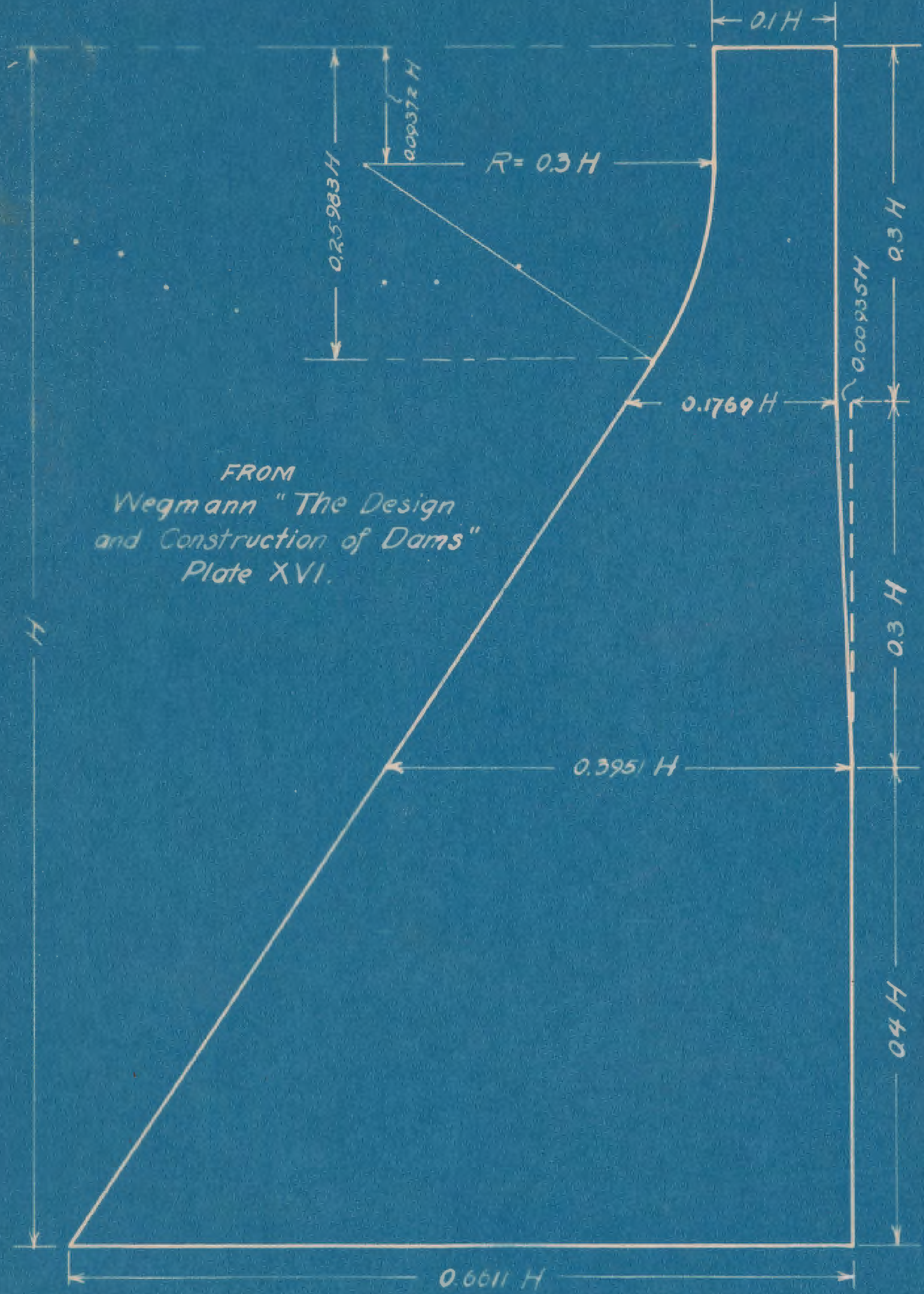


Figure 7 - Profile of Dam.

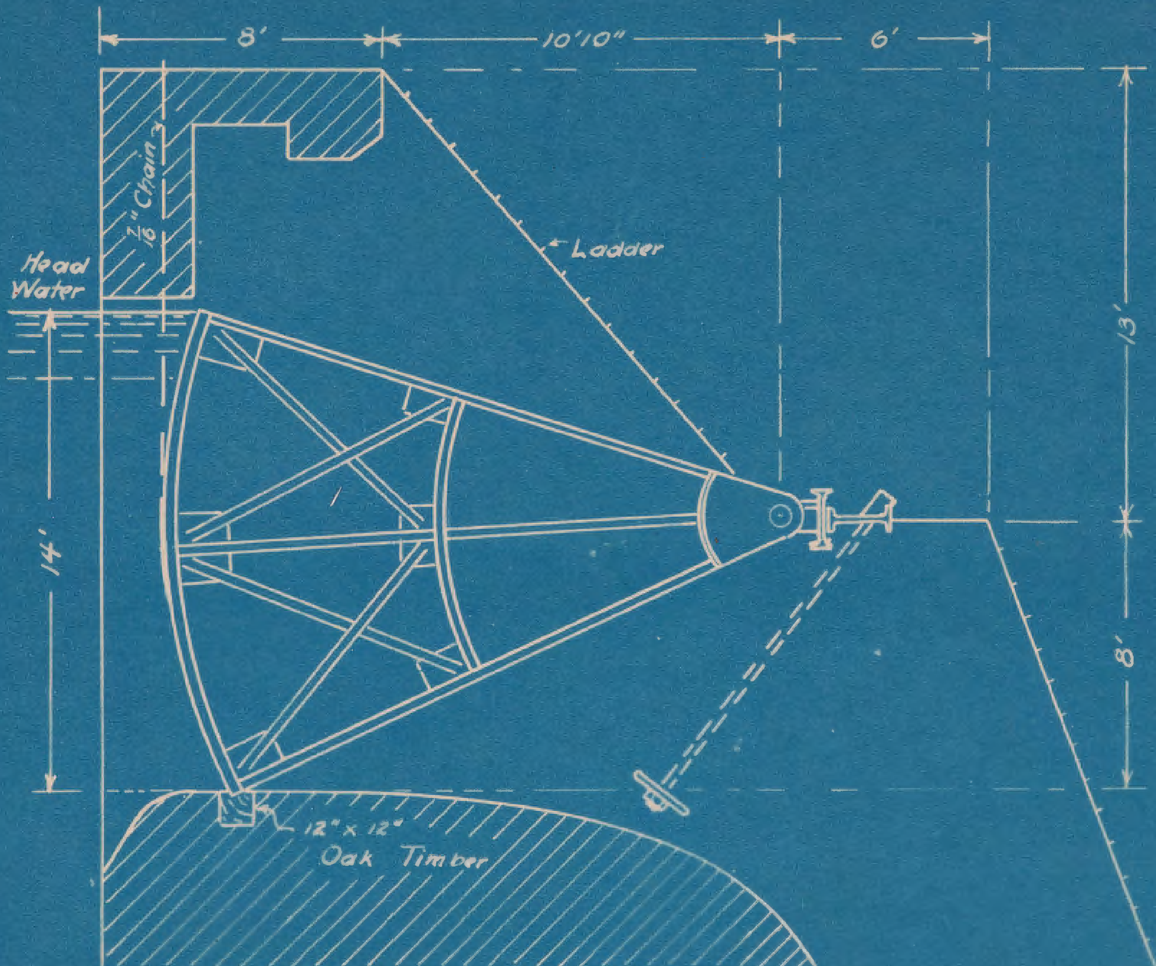


Figure 8

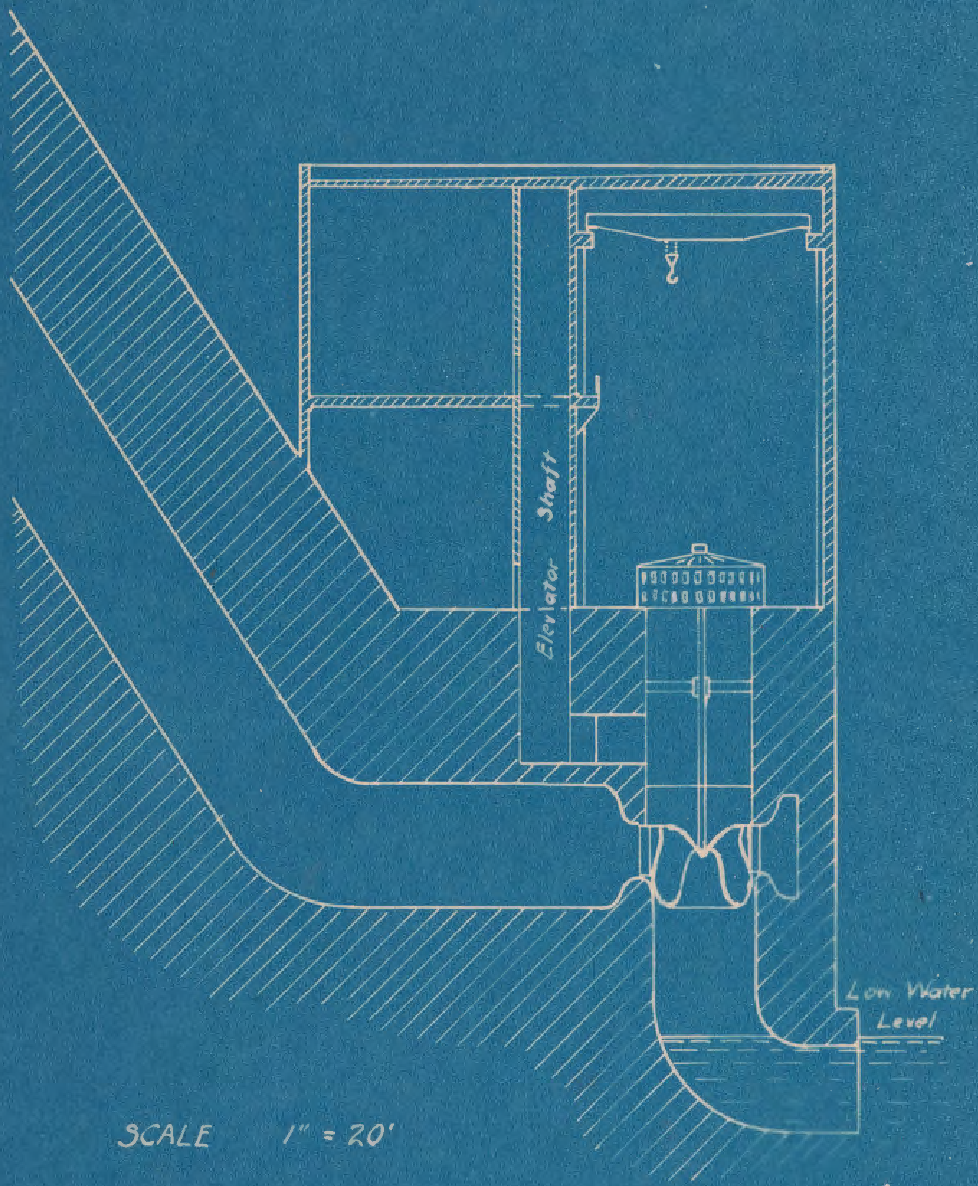
Tainter Gate Section

Length - 20'

Width of piers - 4'

Dimensions from
Taylor & Bramer
"American Hydro-Electric
Practice."

Figure 9
Plan of Power House
at Coleman Site



SCALE 1" = 20'

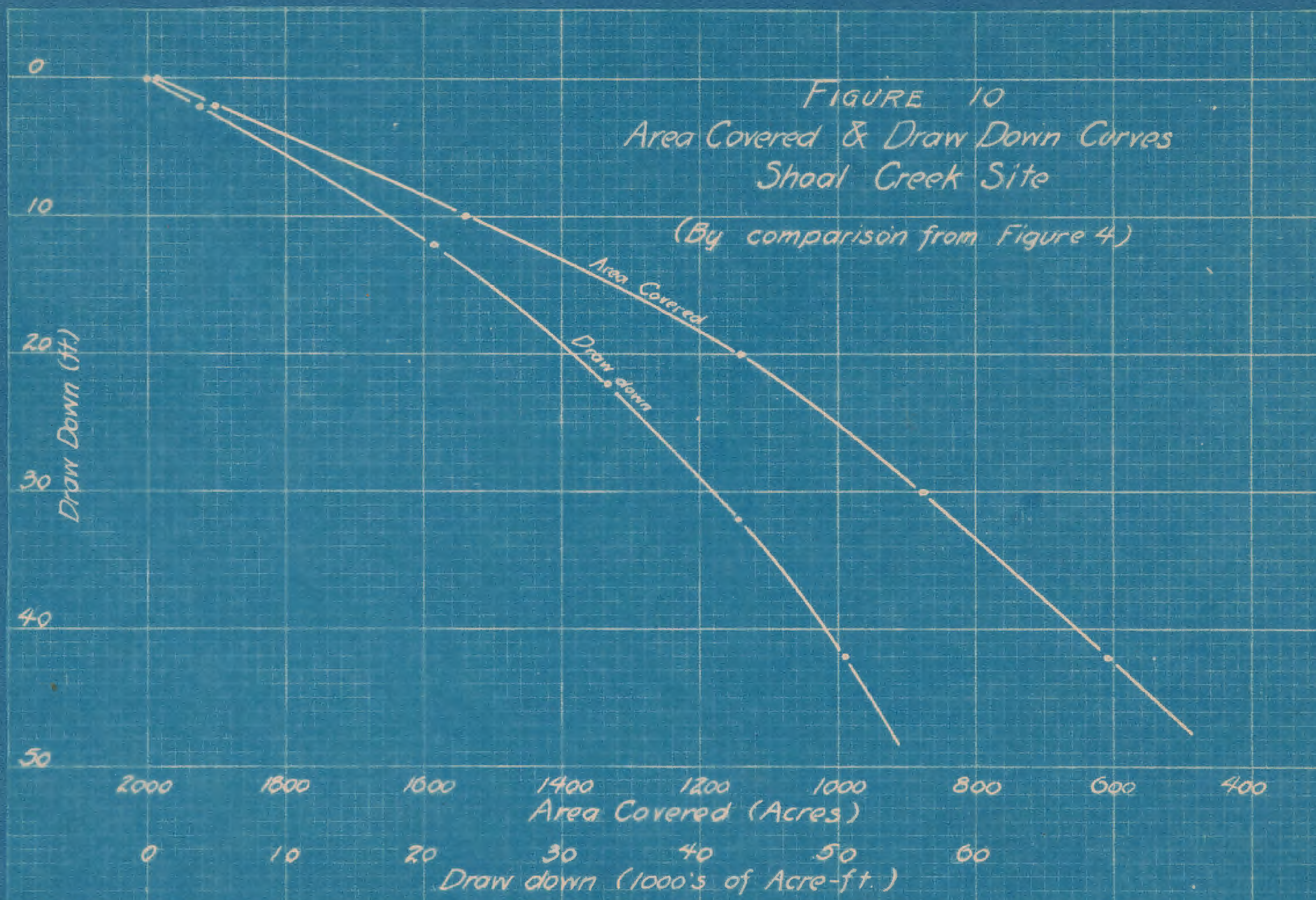


FIGURE 10
Area Covered & Draw Down Curves
Shoal Creek Site
(By comparison from Figure 4)
Area Covered
Draw down
Draw Down (ft.)
0
10
20
30
40
50
2000
1800
1600
1400
1200
1000
800
600
400
Area Covered (Acres)
0
10
20
30
40
50
60
Draw down (1000's of Acre-ft.)

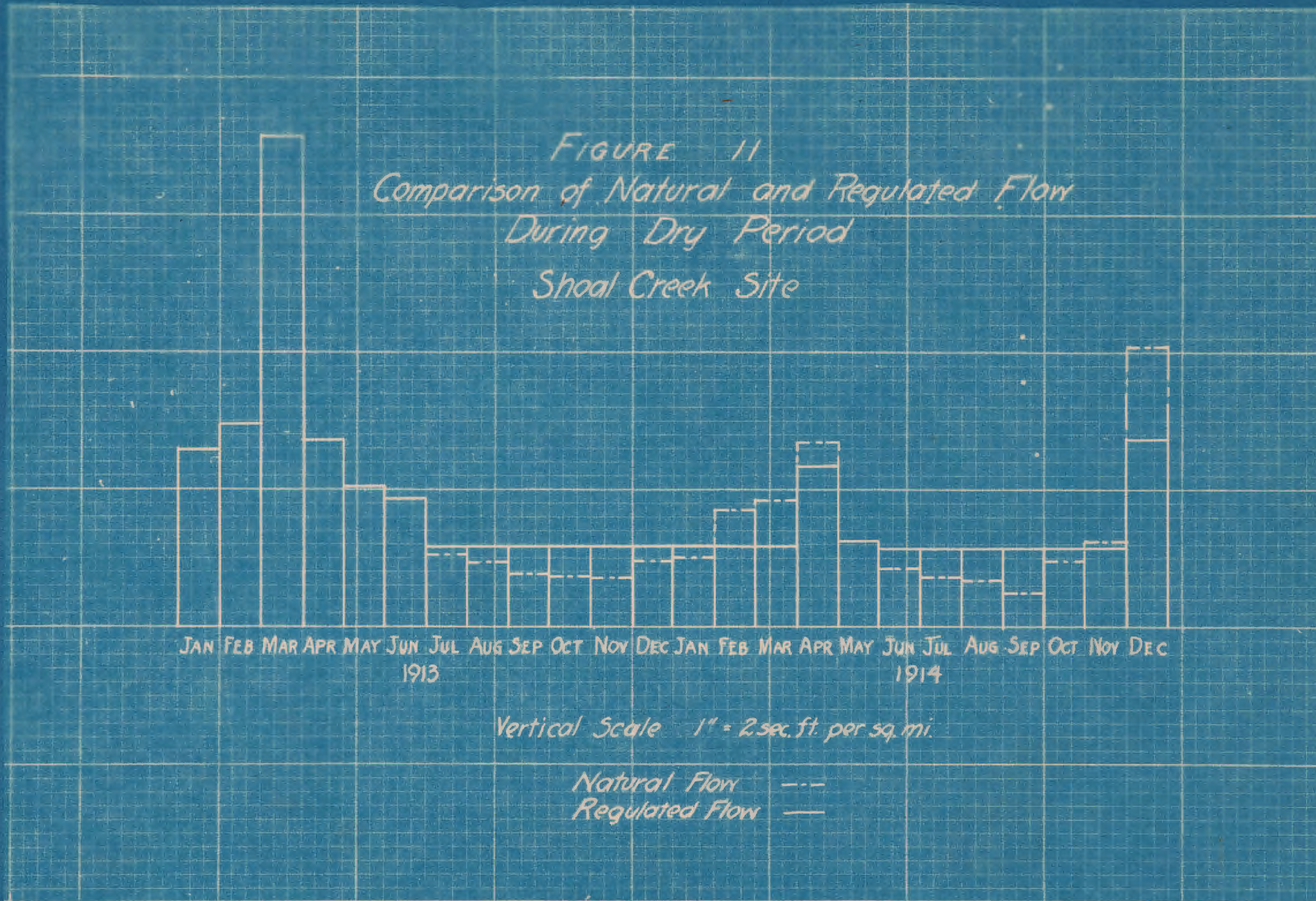


FIGURE 11
Comparison of Natural and Regulated Flow
During Dry Period
Shoal Creek Site
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
1913
1914
Vertical Scale 1" = 2 sec. ft. per sq. mi.
Natural Flow ---
Regulated Flow —

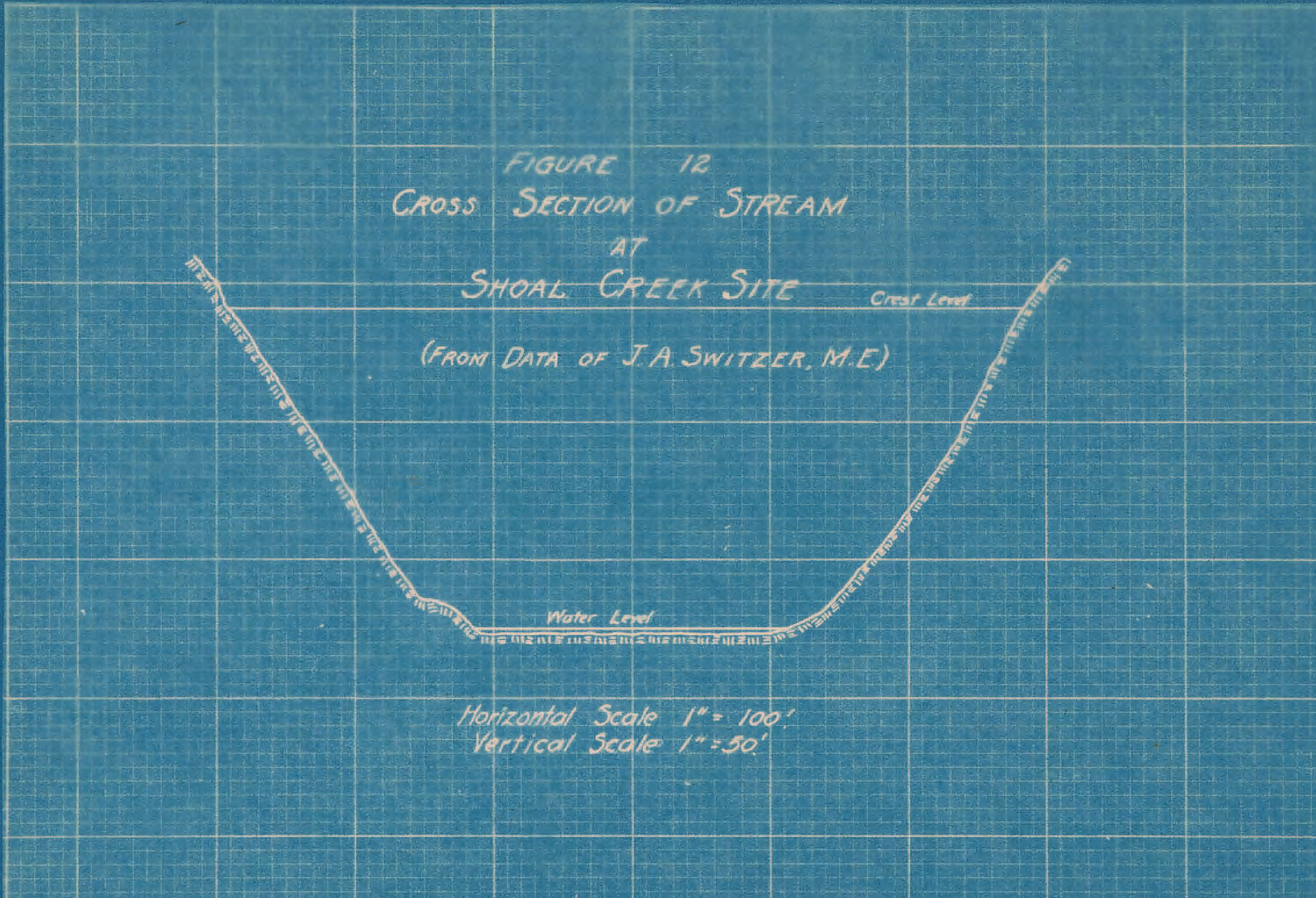


FIGURE 12
CROSS SECTION OF STREAM
AT
SHOAL CREEK SITE

(FROM DATA OF J. A. SWITZER, M.E.)

Horizontal Scale 1" = 100'
Vertical Scale 1" = 50'

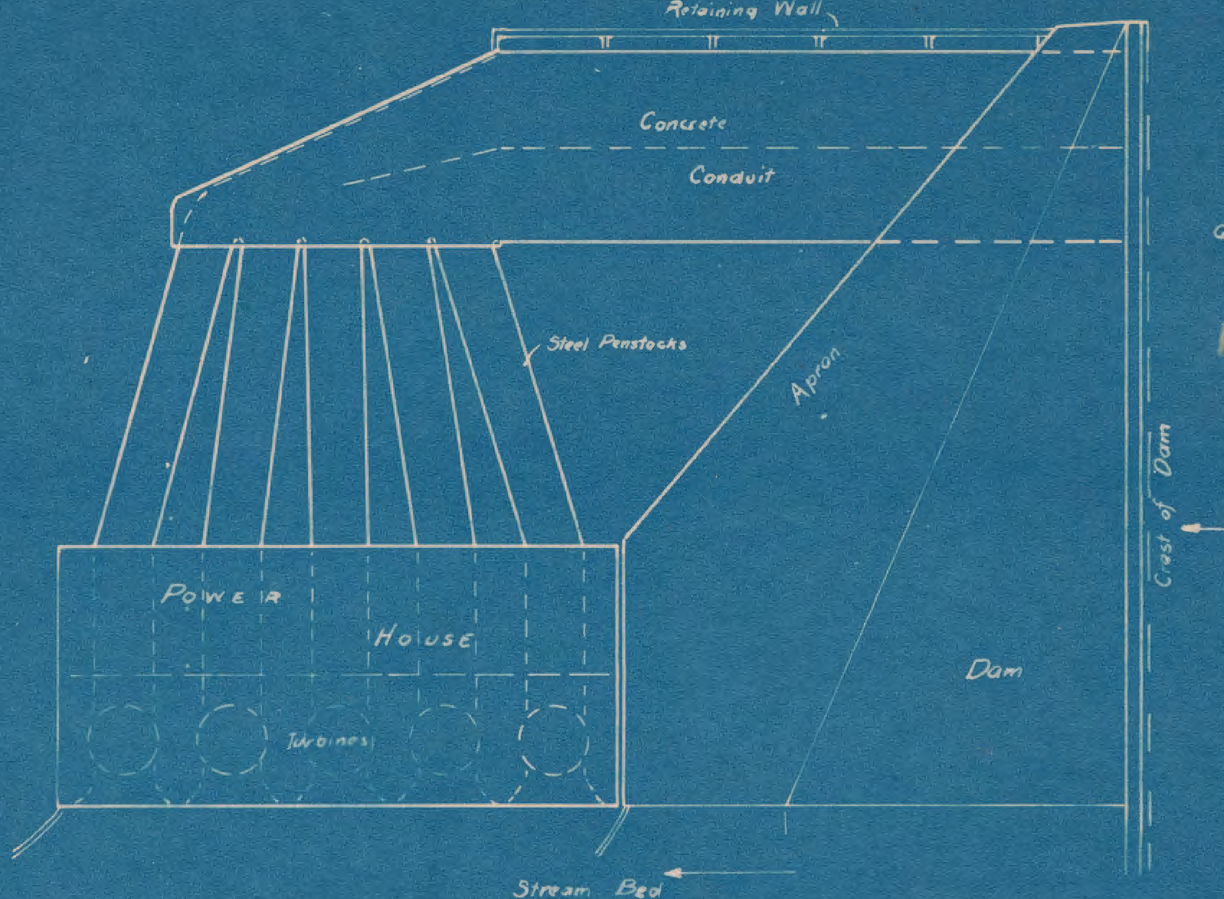


FIGURE 13
Arrangement of
Power House
and Penstocks at
Shoal Creek.

Plan View

SCALE - 1" = 40'

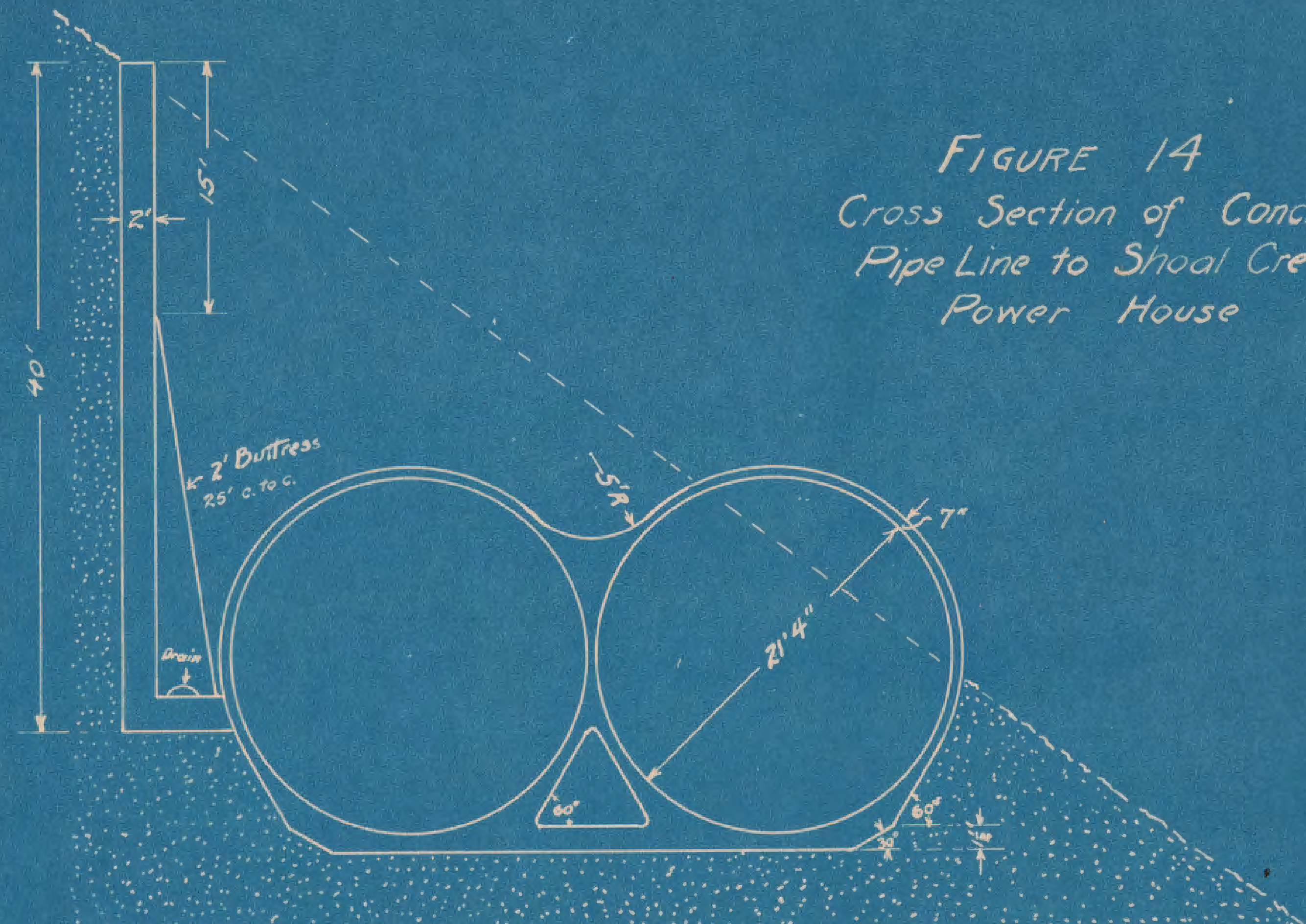


FIGURE 14
Cross Section of Concrete
Pipe Line to Shoal Creek
Power House

APPROVAL

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

Name J. A. Switzer

Title Prof. of Hyd. & San. Eng.