C.E. Thurin

A

THESIS

entitled

STORAGE AND POWER POSSIBILITIES ON THE AUSABLE RIVER

by

Albert F. Kinzey

and

Arthur E. Larratt

Albert F. Kinzey

Arthur E. Larratt

M.I.T. Dormitories

Cambridge Mass. May 26, 1926

Prof. Allyne L. Merrill,

Secretary of Faculty,

Mass. Inst. of Tech.,

Dear Sir:

In compliance with the requirements of the Civil Engineering Department we beg to submit the following thesis entitled "Storage and Power Possibilities on the Ausable River" as partial fulfillment of the requirements for the degree of Bachelor of Science.

Very truly yours,

Signature redacted

Signature redacted

ACKNOWLEDGEMENT

We wish to express our appreciation of the helpful assistance and kindly advice given us by Prof.Barrows and also to extend our thanks to Mr.Daggett of the S.Morgan Smith Company and to Mr.Savage of the General Electric Company for the turbine and generator cost data obtained.

Arthur E.Larratt

Albert F.Kinzey

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INTRODUCTION

I N T R O D U C T I O N

The Ausable River, rising on the eastern slope of the Adirondack Mountains in New York State, flows in two branches to Ausable Forks and thence to its mouth in Lake Champlain a total distance of approximately sixty miles. This river is characterized by its rapid rate of fall and also by its small amount of natural storage. Due to the variable climatic conditions in this section of the country the run off is very irregular, ranging from 15000 second feet in March, after the early spring thaws, to a minimum of 150 second feet in the dry months of summer and early fall.

History

In the year 1915 the New York State Conservation Commission madea thorough study of the river to determine where it would be advisable to locate storage reservoirs since the necessity of storage for future power development of any size is plainly evident.

Detailed surveys and studies were made of three sites on the river proper, at Ausable Lakes and Keene Valley on the East Branch, and Cherrypatch Pond on the West Branch.

It was found that both sites on the East Branch were impracticable because of excessive costs. The site at Cherrypatch Pond however was found to be suitable for the construction of a controlling resevoir of two and one half billion cubic feet capacity.

At the present time there is a wheel capacity of 6426 horse power installed on the river. This power is consumed for the most part by factories and towns near the sites. With the proposed regulation at Cherrypatch Pond the annual available horse power could be increased from 5540 HP to 24070 HP.

Black Brook, one to the largest and most important of the tributaries, lies north of the West Branch and joins the river about four miles above Ausable Forks. This brook, at Black Brook Village, drains an area of 52.6 square miles. Of this area 17.9 square miles are controlled by a dam about a mile and a quarter below Taylor Pond. An additional 3.4 square miles are controlled by a dam across the outlet from Fern Lake. The remaining 31.3 square miles are uncontrolled.

Our problem was, first to determine the location, size, and approximate cost of erecting a dam to most economically control as large a portion of this area as seems advisable and, second, to determine the power that may be developed at the site and the cost of harnessing said power.

A study of the Black Brook drainage area was made from contour maps of the U.S. Geological Survey which for large scale work are sufficiently accurate. Stream flow records of the U.S. Geological Survey have been made at Ausable Forks since August 1910 and at Newmans on the West Branch since July 1919. These records as published in the Water Supply Papers have been our only resource of estimating the rate of run off for this region.

Scope

Part 1

Part 1

HYDROLOGY

The determination of the advisable size to build a storage resevoir is controlled primarily by long time records of either rainfall or run off. As the watershed is in a very sparsely settled country the United States Government has not deemed it advisable to locate a rainfall station within the bounds of the river. Hence our only resource was to approximate the precipitation by the triangulation method as suggested by Prof. Barrows in his Hydrology Notes. By this method we obtained an average value of 32 inches a year. It should be noted however that as there is an elevation difference of over 2000 feet on the Ausable River the precipation would likewise vary, being larger in amount the higher the elevation. Also all records in this region are less than 40 years in extent which is the value given by Prof. Henry of the Weather Bureau to give an accuracy within 15%.

Although we found very meagre rainfall data, we were able to obtain flow records of the Ausable River at Ausable Forks, a town four miles below the junction of Black Brook and the West Branch of the Ausable River, and at Newmans, a farmstead about 25

Miles up the river. The published records of run off at Ausable Forks extend from August 1910 to September 1922 and at Newmans continuously from April 1920 to September 1922. It is at once evident that the records are not of sufficient duration to afford accurate determination but are adequate for close estimation. From the Ausable Forks records we find there has been a maximum flow of 25000 second feet and a minimum of 50 second feet. As can be expected the maximum flow comes in the spring and, according to the tabulation given in Appendix A, 50% of the run off in the average year occurs in two spring months, this excessive run off being due primarily to the depletion of the large snow storage in the mountainous up-country. Within five months of this high water period the river reaches its lowest flow, being very slight in the summer months.

As the Ausable Forks records were much more complete, we neglected considerations using the records at Newmans. With this knowledge of the average flow of the river and by consultation with Prof. Barrows it was decided that it was necessary to store a minimum of 20 million cubic feet per square mile to a maximum of 35 million to attempt to control the flow in this watershed, the large permissable range being due to the large range of annual run off

(2.03 sec. ft./sq. mile to 1.03 sec. ft./sq. mile) and the run off for which the economics of the project seemed most favorable.

The geology and topography of the drainage area favor rapid run off as the region is very rough and rocky. However, most of the hills are wooded, a fact which materially cuts down the rate of run off. This run off varies from 14.7 inches to 29.9 inches with an average of 22.0 inches according to the Report of the New York State Conservation Commission.

DRAINAGE AREA

After outlining the drainage area of Black Brook we were able to subdivide it into the component creek drainage areas and find the relative importance of each tributary. This is shown in the following table.

Taylor Pond Resevoir ----------	8.82 sq.mi.
Little Black Brook diversion ---	9.10 " "
Blake Brook --------------------	14.80 " "
Fern Lake ----------------------	3.64 " "
Black Brook proper -------------	19.52 " "
Total	55.88 sq.mi.

Practically all of the land in this area is


Whistle Pond
Tolman Mtn
Signal Pk
Sugar Hill
Disco
Military Pond
Hogback
Green Mtn
Slush Pond Mtn
Slush Pond
BLACK BROOK
Blake Brook
SILVER LAKE MTS
Hawkeye
SILVER LAKE
Silver Lake
Mud Pond
Goodrich Mill
Black Brook
Fremont Hill
Taylor Pond
Fern Lake
East Kilns
BLACK
BLACK BROOK
Catamount
Middle Kilns
Black Brook
Little Black Brook
CLINTON CO.
ESSEX CO.
BOUNDARY LINE
WEST BRANCH
Rome
West Kilns
MOUNTAIN
Morgan Pond
New Bridge Brook
Browns
Clark Mountain

wooded as it is much too rocky for cultivation. Also there is very little habitation being outside the belt of small manufacturing towns which thrive on the cheap power of the region. The village of Black Brook on the lower part of Black Brook is the only place where there is a community. All of the roads in the Black Brook area are either county or township roads whose removal would not incur large expenditures.

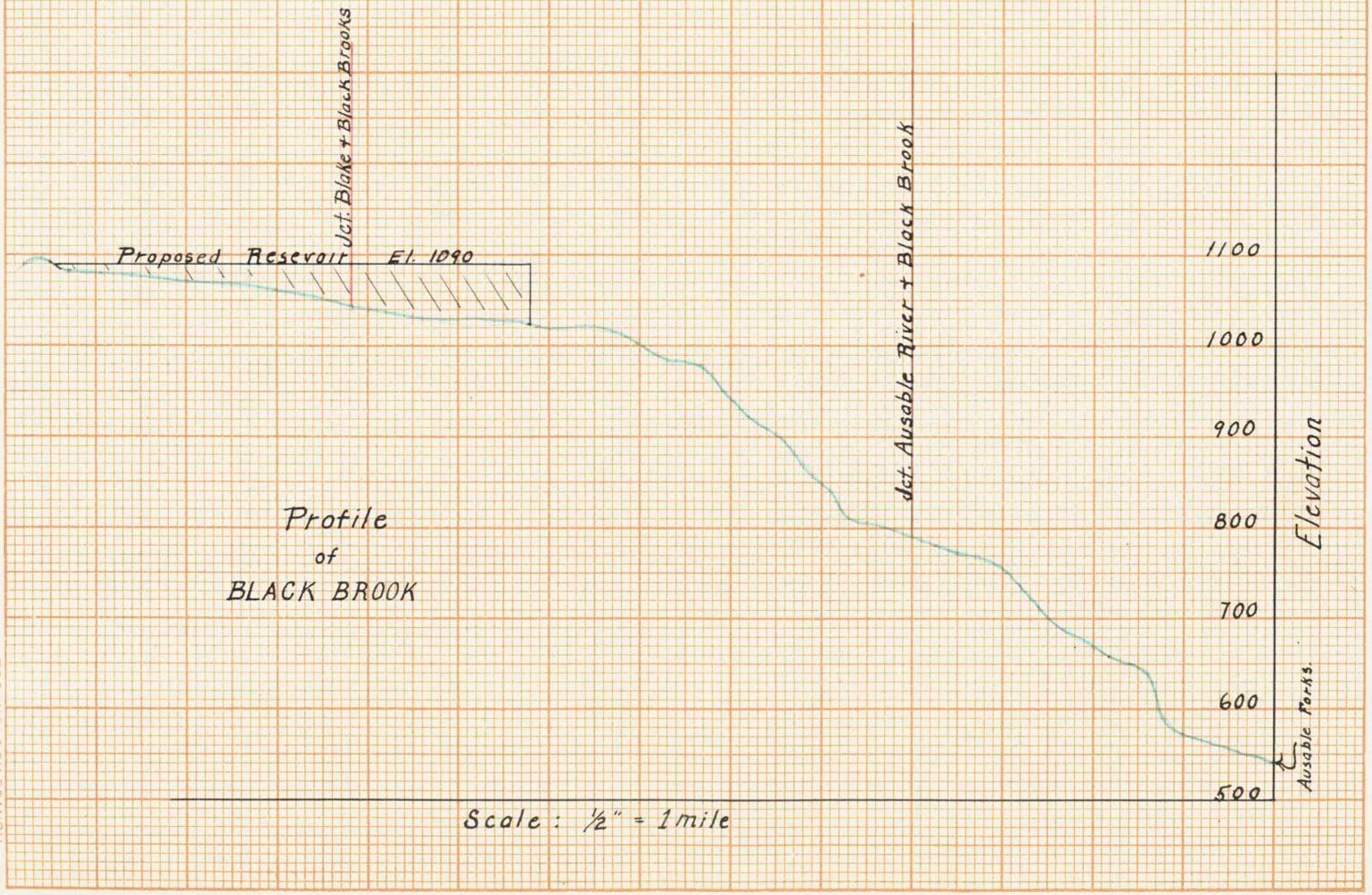
STORAGE

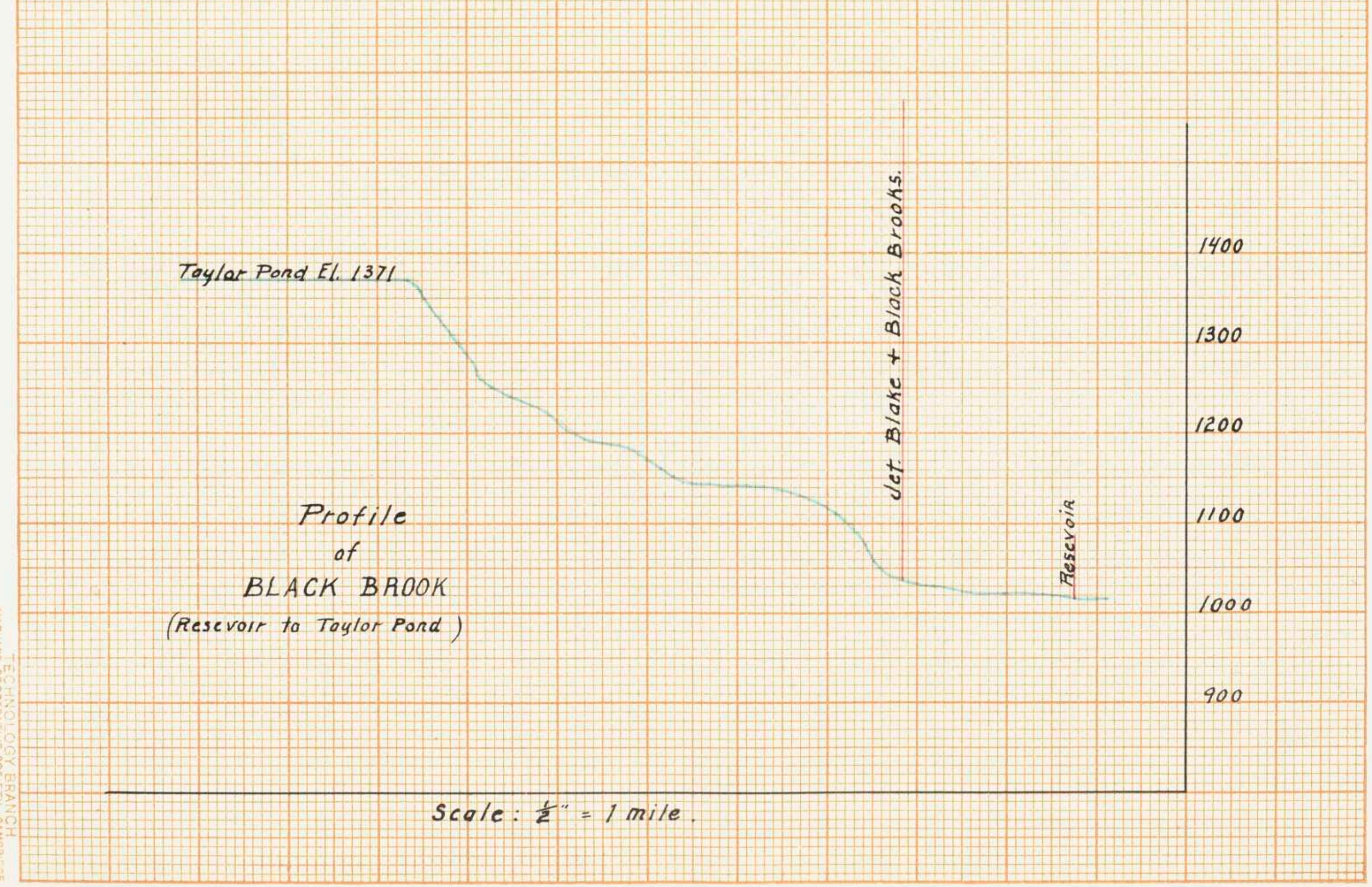
Originally there was 1.8 square miles of water surface in the Black Brook drainage area and approximately 1.2 square miles of swamp land. But since the development of Taylor Pond and Fern Lake into controlling reservoirs the water area has increased to over 2 square miles. These developments at Taylor Pond and Fern Lake are Owned by the J. J. Rogers Company and control four tenths (4/10) of the total drainage area of the stream. This relatively high percentage of the drainage area already controlled, being in the upper reaches of the river, makes future developments on the lower brook especially favorable, as the flow into such a reservoir would be very steady. The storage on this brook would also increase the available power on the Ausable River which is in needof higher flow in the dry season.

A study of the topographical maps of the Black Brook watershed and of the profile of the river show very few places where storage resevoirs are possible because of the rapid drop in the river. Below the village of Black Brook the river drops steadily down to the Ausable River. Above the village there is an immediate rise and then a long stretch with a low gradient. It is here that there is a five mile stretch with a drop of only sixty feet. In the lower part of this section the river passes through a fairly narrow gorge that seemed suitable for a dam location. The upper reach is a flat valley with a swamp area of 0.60 sq. miles. As the brook rises steadily to the west above this point, we realized that any controlling storage project would have to include this large swamp. However the development of the swamp area is limited by the nearness of the Saranac watershed, development above the 1090 level being impossible without the construction of a diverting dam.

RIVER PROFILE

Before actually choosing the site for a dam one has to make test borings and get some actual information of the conditions under the surface. As we could not do this our choice of dam site is merely a guess and might have to be changed to suit existing conditions.


Jct. Blake + Black Brooks
Jct. Ausable River + Black Brook
Proposed Reservoir El. 1090
1100
1000
900
800
700
600
500
Elevation
Ausable Forks.
Profile
of
BLACK BROOK
Scale: 1/2" = 1 mile


Taylor Pond El. 1371
Jct. Blake + Black Brooks.
Resevoir
1400
1300
1200
1100
1000
900
Profile
of
BLACK BROOK
(Resevoir to Taylor Pond)
Scale: ½" = 1 mile.

As the water stored is to be used principally at the village of Black Brook it was deemed advisable to build the dam here if economical. At this location we could develope the head created by the dam and the falls at this place into a central power station. However, when we studied the topographical maps and the river profile it became evident that it was necessary to make the level of the reservoir such as to include the large swamp area at the Junction of Blake Brook and Black Brook if we were to obtain controlling storage. This condition required the construction of a dam at least eighty feet (80) high . A dam of such a height would have to be about one mile long due to the widening of the river valley at this point. With this information this dam site was at once ruled out.

The rejection of this site left the selection to the narrowest part of the aforementioned gorge. The location finally selected was three miles above Black Brook Village. By building the reservoir here we lose about 5 square miles of drainage area which would have been controlled by a dam at the village, but as 3.4 square miles of this area is already controlled at Fern Lake the difference in size of dam necessary greatly offsets the difference in areas controlled.

From the computations and curves shown in Appendix B it is evident that a sixty foot (60) dam

SIZE AND CAPACITY.

(to elevation 1090) would have a storage capacity of 890 million cubic feet. This capacity is the equivalent of a control of 33 million cubic feet per square mile. If the dam were 10 feet lower it would have a capacity of 528 million cubic feet which amounts to a control of 19 million cubic feet per square mile. This shows that with a 20% increase in elevation the storage increases 68% and raises the control of the average annual run off from 37% to 65%. With similarly designed dams the 50 foot one would cost 33% less than the 60 foot. Also the cost per million cubic feet stored is $430 for the low dam and $380 for the high. (See Appendix C). If the elevation is raised from 1090 to 1100 storage is greatly increased but the percentage of the available flow stored becomes so high that it would only be under exceptional conditions that such a large reservoir would be needed. Also, as previously mentioned, water levels above 1090 would cause an overflow into the Saranac River system if a diverting dam were not constructed at Military Pond.

With the foregoing information in mind we selected 1090 as the limiting elevation of our storage.

The selection of a type of dam suitable for this location would require a more complete study than our time allows. However any dam would have to be either concrete, earth fill, or a combination of both.

Sugar Hill
1504
1218
Disco
K
Brook
BM 1131
1081 Military Pond
1098
1333
1381
2189 Hogback
Green Mtn
1822
1593
Slush Pond Mtn
B L A C K B
Slush Pond
Goodrich Mill
BM 1150
Brook
Black Brook
1737
1640
Fern Lake
1178
1462
Kilns
B L A C K
K
1065
1009
Black Brook
Little
Black
Brook
Black
CLINTON CO. BOUNDARY
ESSEX CO. LINE
BRANCH
WEST
Rome

Wooden construction is prohibitive because of the height, and a hollow concrete dam is not suitable for this part of the country as the temperature range is so great as to make frost action very destructive. In our computations we considered only a solid concrete dam(which usually costs the most) thus obtaining maximum values of cost.

As it is reasonable to expect to find solid rock in the floor of the valley and on the sides we would have no difficulty in placing a concrete dam on a good footing, a condition which is very important. In making the actual computations for a concrete dam in this part of the country a good allowance must be made for ice pressure. By comparison with other dams it is estimated that a spillway section of more than 100 feet is unnecessary and this value could be cut down to a considerably lower figure.

The shape of the valley has few of the characteristics that particularly favor a concrete spillway section with earth wing walls. A study of actual construction costs should be made before this combination can be compared with the solid concrete dam. With both this type of construction and with the solid concrete, flash-boards could be provided along the spillway section and the capacity of the reservoir temporarily increased.

Using a solid earth dam there are two plans available for controlling the flood waters. The first is to build a large tunnel through the east bank and divert the water through it down to the adjacent natural valley. The second method is to dig out and construct a concrete spillway at the boundary of the Ausable and Saranac Watersheds just east of Military Pond. This would divert the flood water into the Saranac Valley and thus diminish the flood conditions in the Ausable system. Much careful study of flood conditions would have to be made before attempting either of the last two projects, even though the expense of construction was less.

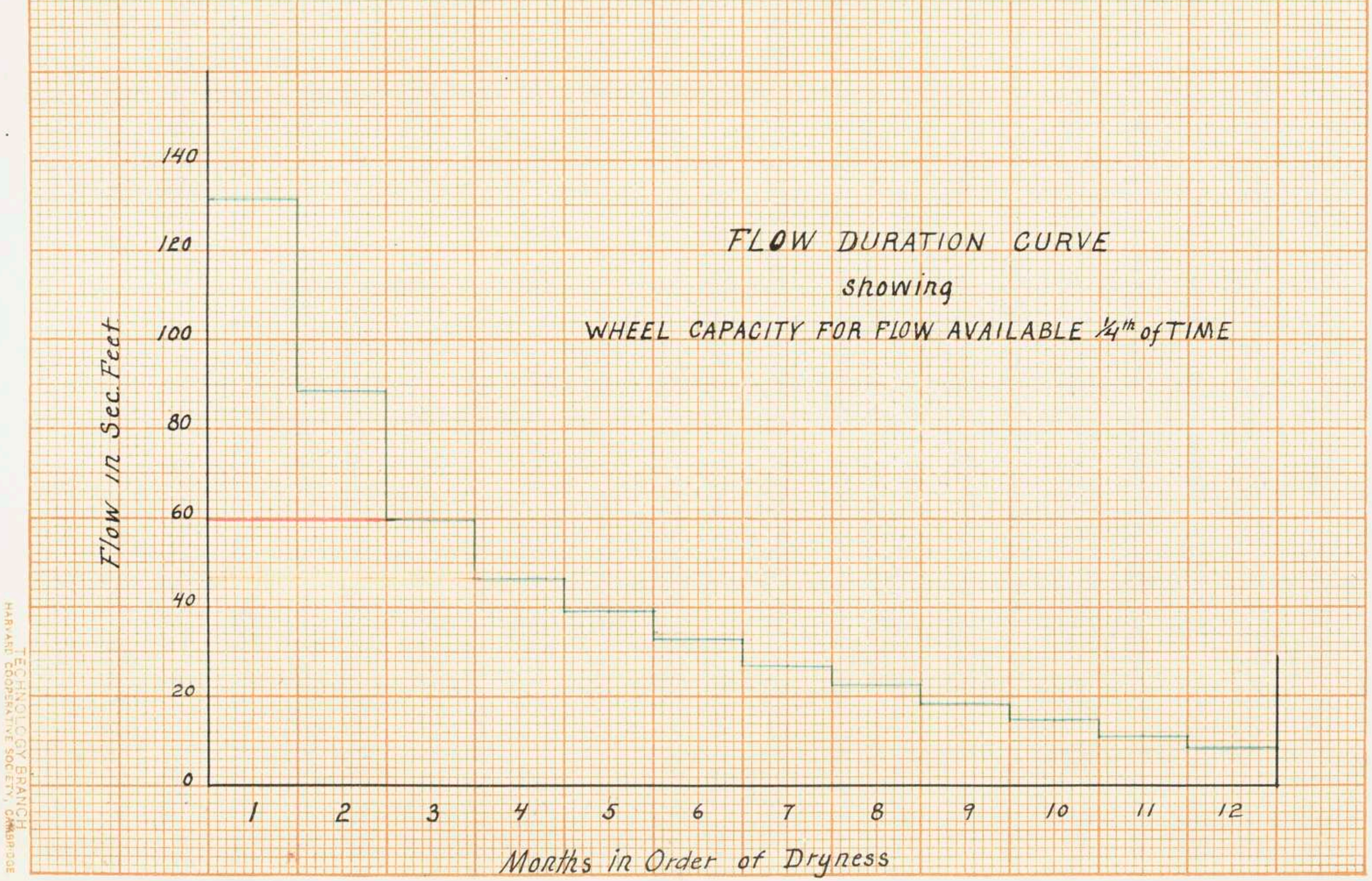
PART II

PART II

With the construction of the dam on Black Brook there is a head created which, if put to use, would develope some power. This power would be obtained by directing the water drawn in the dry period through turbine wheels. As the purpose of the reservoir is to provide power output when the natural flow is below normal, any incidental added means of obtaining this power would be very helpful in supplying the demand.

With the proposed height of dam there is a maximum head of 60 feet. This head is only available during the spring months when the flow is very large and the water in the reservoir is at the spillway level. However, as 70% of the capacity of the reservoir is controlled by the upper 20 feet of the dam, it is evident that we could consider a project using a minimum head of 40 feet. As the water is drawn from storage in such a way as to gradually lower the level, the turbine wheels selected would have to have a flat efficiency curve. As power is needed principally in the dry season, it would be advisable to have the maximum efficiency at a little less than one half the total drop of water level. For this reason we used a 45 foot head rather than a 50 foot head in our calculations.

From the accompanying flow duration curve it is


FLOW DURATION CURVE
showing
WHEEL CAPACITY FOR FLOW AVAILABLE 1/4th of TIME
Flow in Sec. Feet
140
120
100
80
60
40
20
0
1
2
3
4
5
6
7
8
9
10
11
12
Months in Order of Dryness
TECHNOLOGY BRANCH
HARVARD COOPERATIVE SOCIETY, CAMBRIDGE

seen that we considered it advisable to develop the head for a flow available 1/4 of the time or a discharge of 59.5 second feet. This is a slightly higher value than is usually taken , but when one considers the flow from Taylor Pond, it is not excessive. In the average year the average flow from the Taylor Pond resevoir would be 27.5 second feet and the flow from the Black Brook resevoir 41.5 second feet. Therefore our value of 59.5 sec.ft. is a very conservative one. If we should develope for the average annual flow of the whole drainage area of the upper brook, the power plant at Black Brook resevoir would be completely dependent upon the flow from Taylor Pond in the dry season. Therefore, as this plant is to be used principally in the dry season when natural flow is low, a dependence on any outside source would make this plant less reliable.

With the head of 45 feet and a discharge of 59.5 sec.ft. there is available 268 HP with an efficiency of 88%. As the head is comparatively low it is evident that with a discharge as large as this an impulse wheel is impractical. At the same time the head is too high and the discharge too great for a propeller type runner, hence an impulse wheel must be used. After a conference with Mr. Daggett, an engineer with the S. Morgan Smith Turbine Company, it was thought advisable to use one of their type NX 18 inch wheels. On test this size wheel developed 288 HP at 88% efficiency and the company was willing to guarantee a 270 HP output at 514 R.P.M.

As is customary, we would advise three phase current, and, with an R.P.M. of 514 it would have to be 60 cycle. When we inquired at the General Electric Company's office regarding a generator to fulfill the specifications, we were told that they did not stock such a vertical generator. However they did stock a horizontal generator with an output of 240 KW at 8/10 power factor and 514 R.P.M. The extra cost for redesigning this generator would be about 20%.

From the topographical maps it would seem advisable to build the power plant immediately below the dam and on the east bank of the river. The penstock built on this side of the river would have a better grade line than on the opposite side. With a maximum pressure head of 60 feet, a wooden penstock could be used. With a velocity of 7 feet per second this wooden penstock would be 3.4 feet in diameter.

The only estimates of cost we could make were on the turbine, which was $6000 in place, and the generator which was $3770 ($3140 for horizontal + 20%= $3770) . Before estimates of building costs could be made a design of the power plant would have to be drawn.

By having the resevoir on Black Brook the flow at the 125 foot development at Black Brook Village can be increased so that there will be a continuous output of 850 HP . Also the continuous output on the lower Ausable River can be appreciably increased.

If a penstock were run from the village of Black Brook down to the Ausable River, to a point just above the town of Ausable Forks there would be a head of 450 feet available. The line of such a penstock would be three miles long and could be made so that wood stave construction could be used to within a mile of the power house. If there were no unforseen difficulties we think this high head development would be more economical than the 125 foot development now at Black Brook Village.

APPENDIX A

STREAM FLOW RECORDS

At

Ausable Forks

(second feet per square mile)

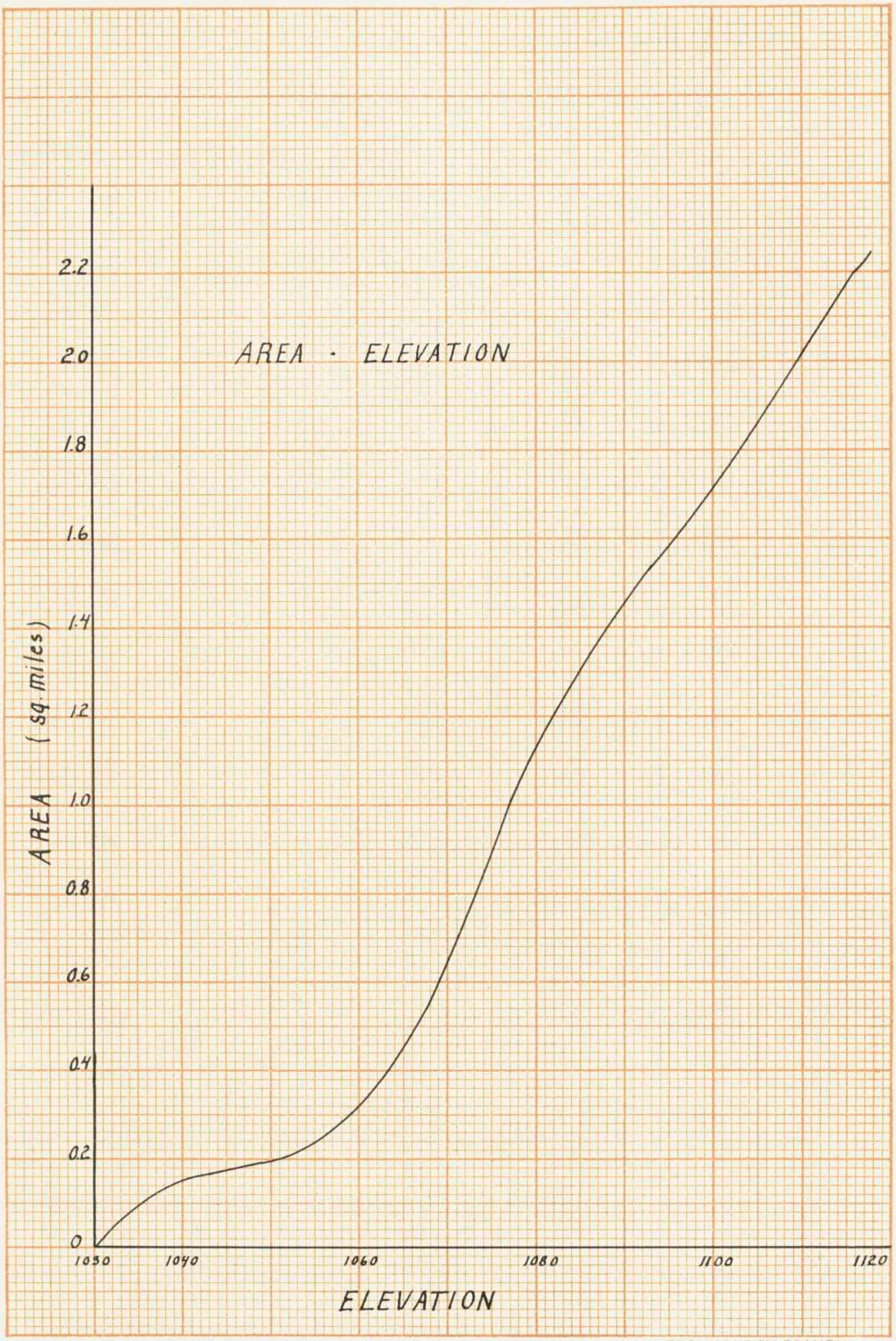
Arranged on Calandar Year Basis

Year	Max.	2	3	4	5	6	
1910-1911	3.53	2.75	1.05	.965	.897	.869	
1911-1912	5.11	3.47	1.85	1.33	1.00	.897	
1912-1913	5.95	4.03	1.92	1.85	1.60	1.39	
1913-1914	5.88	3.00	1.83	1.36	1.06	.959	
1914-1915	2.97	1.53	1.33	1.32	1.03	.856	
1915-1916	4.26	4.12	2.28	2.04	1.91	1.51	
1916-1917	4.68	3.63	3.04	1.53	1.49	1.07	
1917-1918	4.98	3.56	2.04	1.80	1.80	1.56	
1918-1919	4.12	3.78	3.69	3.06	2.55	2.16	
1919-1920	3.96	2.77	2.55	1.55	1.50	1.37	
1920-1921	7.41	3.74	2.43	1.29	1.22	1.08	
1921-1922	5.56	2.91	2.50	2.50	1.41	.876	
Totals	58.41	39.29	26.51	20.58	17.36	14.587	
Averages	4.87	3.27	2.21	1.72	1.45	1.22	
Total area Sec. Feet	131.7	88.5	59.75	46.5	39.2	33.0	

Year	7	8	9	10	11	Min.	Ave.
1910-1911	.786	.721	.526	.374	.337	--	
1911-1912	.864	.842	.823	.561	.460	.361	
1912-1913	.914	.563	.516	.410	--	--	
1913-1914	.759	.748	.507	.444	.410	.317	
1914-1915	.829	.662	.606	.577	.394	.363	
1915-1916	1.24	.745	.734	.673	.626	.482	
1916-1917	.905	.752	.704	.635	.629	.491	
1917-1918	1.39	1.38	.874	.806	.453	.298	
1918-1919	1.25	1.10	.883	.689	.507	.441	
1919-1920	1.11	.937	.732	.455	.444	.430	
1920-1921	1.02	.809	.498	.417	.288	.217	
1921-1922	.858	.703	.601	.545	.354	.336	
Totals	11.925	9.962	8.004	6.586	4.902	3.736	
Averages	.994	.830	.667	.549	.409	.311	
Total area Sec. feet	26.9	22.45	18.05	14.85	11.07	8.41	

APPENDIX B

HIGHT AND CAPACITY OF RESERVOIR


AREA · ELEVATION
AREA (sq. miles)
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
1030
1040
1060
1080
1100
1120
ELEVATION
TECHNOLOGY BRANCH
HARVARD COOPERATIVE SOCIETY, CAMBRIDGE

The following table gives the cubical contents of the resevoir for the different elevations to which the water would rise .

Elevation	Cubic Feet
1040	29,280,000
1045	51,580,000
1050	76,680,000
1055	107,330,000
1060	146,330,000
1065	200,730,000
1070	277,330,000
1075	386,130,000
1080	528,330,000
1085	698,430,000
1090	890,930,000
1095	1,104,330,000
1100	1,334,330,000

(Lowest point of dam at elevation 1030)

The area under the accompanying "height-elevation" curve gives the capacity for any height. The areas were obtained by planimetering the areas on the topographical map included within the different contour lines.

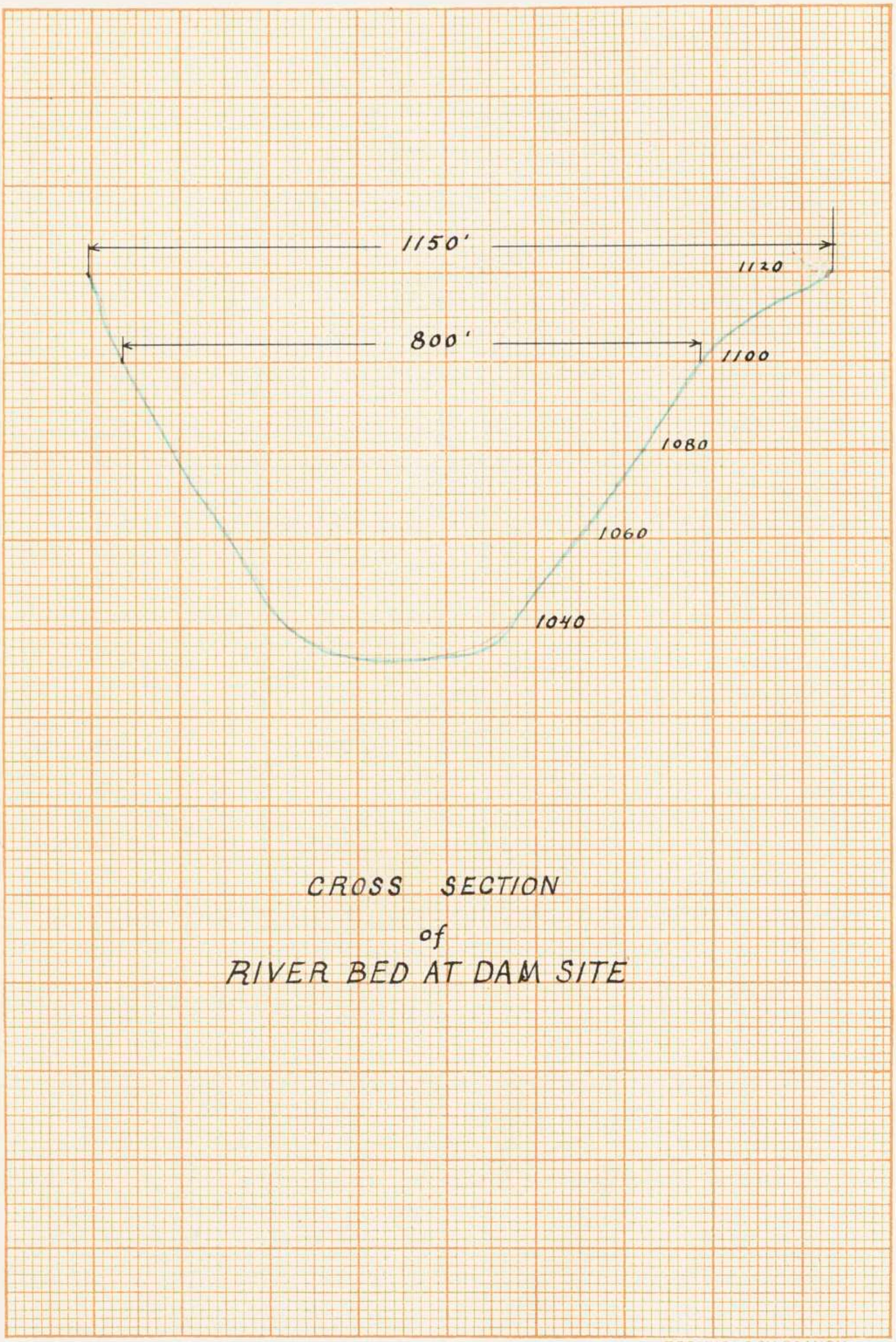
APPENDIX C

CHOICE AND COST OF DAM

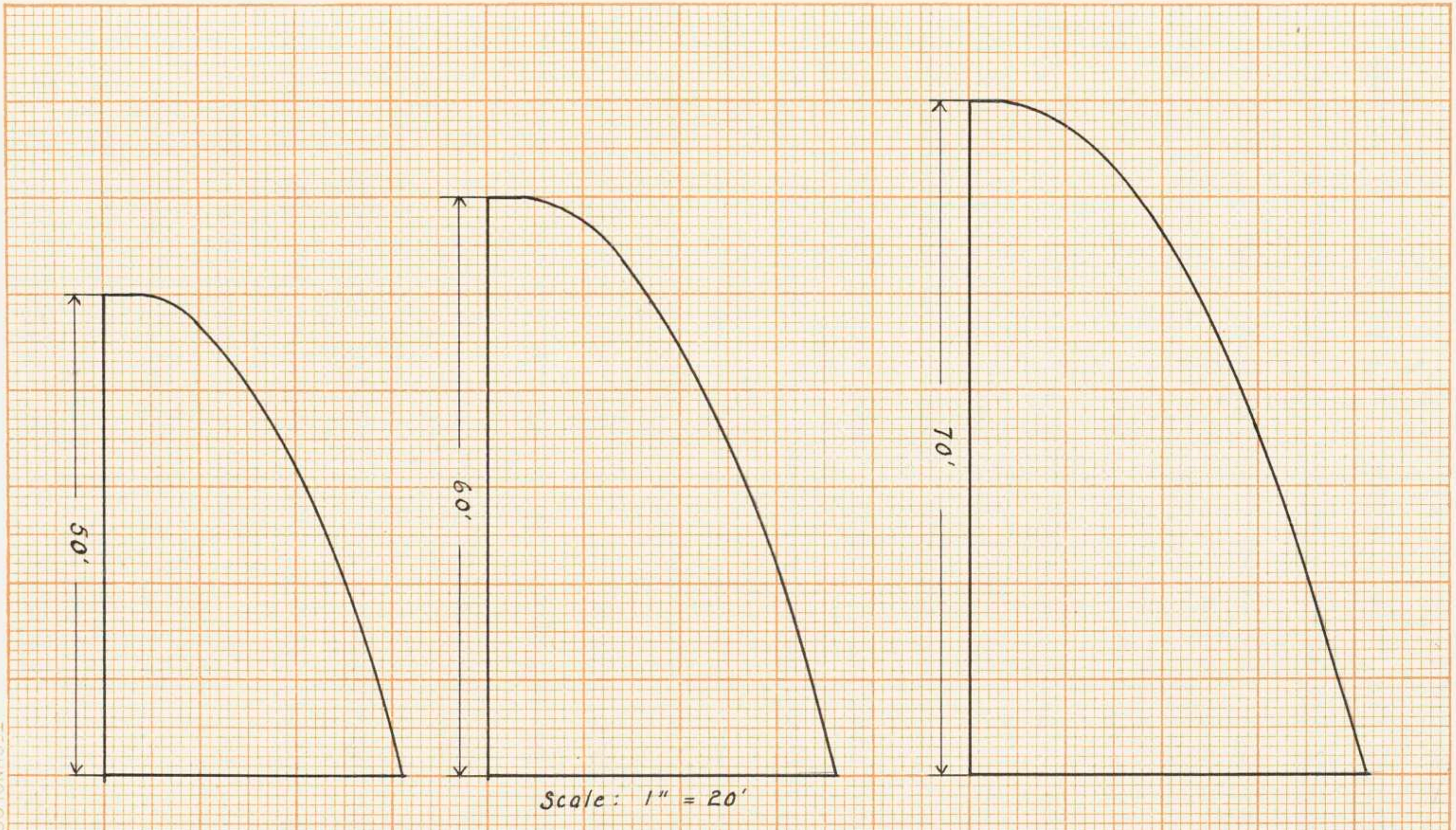
The question of choice of dam size is largely one of economics, the desireable size being the one that will afford the lowest cost per million cubic feet of storage. The following table shows the amount of storage required to control our area for the different values of run off (mil. cu. ft. per sq. mi.)

Mil.cu.ft. per sq.mi.	Required Capacity Mil.cu.ft.
20	541
21	568
22	595
23	622
24	649
25	676
26	703
27	730
28	757
29	785
30	811
31	838
32	865
33	892
34	920

From a comparison of this table with the one in Appendix B it is clear that for control figured on the



CROSS SECTION
of
RIVER BED AT DAM SITE



DAM SECTIONS FOR 3 HEIGHTS

minimum value of 20(mil.cu.ft.per sq.mi) we would need a dam 50 feet high (capacity 528,330,000 cu.ft.). The maximum run off coefficient of 34 corresponds to a required capacity of 920,000,000 cu.ft. which shows that the 70 foot dam with a capacity of 1,334,330 is rather excessive. Let us consider, however, the costs of the 50,60, and 70 foot dams which, as we have seen, include the limits of required storage.

Cost of 50 foot dam

To estimate the cost of these different sized resevoirs we calculated the number of cubic yards of concrete necessary for each size and used the flat rate of $15 per cubic yard of concrete in place as the unit cost. The volumes were obtained by dividing the dam into 100 foot sections(as obtained from profile of valley at dam site) and finding the mean cross sectional area in each 100 feet. We used the spillway section as the typical shape. This section was considered as having a vertical back and a parabolic front. In obtaining the hydrostatic pressure we assumed a 10 foot head of water over the spillway (maximum conditions). No allowance was made for upward water pressure. These results are only rough estimates but are however consistent with the accuracy of our profile section.

Hydrostatic Pressure (section one foot wide)

$$50' \times 30' \times 62.5\# = 93,750\#$$

$$\text{Point of Action} = \frac{2}{3}\,\frac{h_2^3 - h_1^3}{h_2^2 - h_1^2} = \frac{216 - 1}{3600 - 100} = 41 \text{ feet}$$

" " " from bottom = (50'- 10') - 41' = 19'

Let X = Base width (Wgt. of Concrete = 150 #/cu.ft.)

$$\frac{\frac{2}{3} \times 50' \times 150\ \#/\text{cu.ft.}}{93,750\ \#} = \frac{19}{1\frac{1}{24}X}$$

$$X^2 = \frac{19 \times 93,750 \times 24}{50 \times 100 \times 11} = 778$$

$$X = 27.9 \text{ or } 28'$$

Using this value and the curves on preceding page we obtain a volume of 15,140 cu. yds.

15 x 15,140 = \$227,000 (estimated cost of dam)

$\frac{\$227,000}{528}$ = \$430 per million cubic feet of storage)

Cost of 60 foot dam

Force on Dam = 60 x 62.5 x 35 = 131,250#

Point of Action = $\frac{2}{3} \times \frac{343 - 1}{4800} = 47.5'$

$$X^2 = \frac{22.5 \times 24 \times 131{,}250}{11 \times 6000} = 1075$$

$$X = 32.8 \text{ or } 33'$$

Volume = 22,540 cu.yds.

Cost = " x $ 15 = $ 338,000

= $\frac{338{,}000}{890.93}$ = $379 per million cu.ft. of storage

---

Cost of 70 foot dam

Force on Dam = 70 x 62.5 x 40 = 175,00#

Point of Action = $\frac{2}{3} \times \frac{511}{6300} = 54'$

$$X^2 = \frac{26 \times 24 \times 175{,}000}{11 \times 7000} = 1420$$

$$X = 37.7 \text{ or } 38'$$

Volume = 33,590cu.yds.

Cost = 33,590 x $15 = 504,000

Small Dam at North end of Res. Cost = $31,700

Total cost = $\frac{504{,}000+31{,}700}{1{,}334}$ =$401 per million cu. ft. of storage.

From the foregoing figures we conclude that the 60 foot dam is the most economical size to build. In addition to the determination by this method it was shown before that the 70 foot dam was really higher than necessary and while the 50 foot dam would take care of a fairly large flow the cost of the 60 foot dam (per mil.cu.ft. of storage) was sufficiently lower to warrant its adoption.


Sugar Hill
1504
1218
Disco
K
Brook
BM 1131
Military Pond
1081
1098
1333
1381
Lake Bk
1191
2189 Hogback
Green Mtn
1822
Slush Pond Mtn
B L A C K B
Goodrich Mill
BM 1150
Brook
Slush Pond
Black Brook
1737
1640
Fern Lake
1178
East Kilns
1462
B L A C K
O K
Brook
1065
1009
Black Brook
Little
Black
CLINTON CO.
BOUNDARY
LINE
ESSEX CO.
BRANCH
Rome
Brook
WEST
WILMINGTON