

T H E S I S .

A PROPOSED POWER DEVELOPMENT .

Of the

NOLICHUCKY RIVER .

At

EMBREEVILLE , TENNESSEE.

For the Degree of —

BACHELOR OF SCIENCE IN ELECTRICAL ENGINEERING.

Under

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POWER DEVELOPMENT OF THE NOLICHUCKY RIVER.

The Nolichucky river flows for a great part of its course through the wild and mountainous region of the Unaka Range, and a glance at a government contour map will show that its descent from the source to the level land bordering the mountains is a steep one, so that the character of the stream through this descent partakes more of the nature of a mountain torrent than of a river.

The river passes within three-fourths of a mile of the town of Erwin, and at a point almost opposite the town it disappears into a deep gorge. It was at this point that we first viewed the stream and we were at once struck with the favorable conditions for power development, and determined to investigate the vicinity as a probable site for this development.

Just before the river enters the gorge it flows through a considerable valley, offering ample place for storage, and if it were not for the fact that the tracks of the C.C. & O. Ry. also pass through this valley, the amount of available storage would be almost unlimited.

The region in this immediate neighborhood has been several times surveyed by different parties for prospective power development, but so far as we know there have as yet been no active steps taken toward power development, although it was rumoured at the time we made our survey that a large corporation had made arrangements for buying up the land around the gorge for the purpose of eventually developing power, this power to be generated for running several proposed cotton mills. However rumour is not to be relied upon, and

we began our investigation about the first of March, 1912, and this paper contains the results of the investigation.

THE SURVEY .

The location of our proposed hydro-electric plant is on the Nolichucky river between Erwin, Tenn. and Embreeville, Tenn., about twenty miles south of Johnson City.

This river between Erwin and Embreeville affords an almost ideal location for power development in flowing through the gorge spoken of in the preceding paragraph.

We began a survey at a point one mile south of Erwin, at the mouth of Martin's Creek, selecting as a bench mark, a marking on an old concrete bridge pier, three feet above the surface of the water on that date. The elevation of this B. M. we called 250, the elevation of the C. C. & O. Ry. Tracks here, being 15 feet above the B. M. These tracks run southward, parallel with the river and along its bank, for several miles, but the grade is ascending from point of B. M., and as there is also an ascending grade from this point northward, this is the point of lowest elevation of the tracks within the limits which concern us.

Plate I. is a map of our survey and shows the features mentioned above. It is seen from this map that in order to secure any considerable storage it will be necessary to raise the storage elevation as near to 265 as possible, because the area flooded is small and the depth of layer of water stored must be

correspondingly deep. Upon completion of the survey we found a drop of 133 feet from B. M. to a point approximately four and one half miles down stream, marked site of power house on the map. This point is situated directly across the river from Embreeville. Here we decided to discontinue the survey because any obstruction of the stream below this point would flood the town, and because examination of government maps shows that the slope of the stream decreases decidedly below Embreeville, so that the advantage gained by running a line of penstocks along the river bank would not warrant the extra cost entailed.

There are two or three good locations for a dam between Erwin and Embreeville, but the site chosen was selected because of the natural features available only at this point, there being an outcrop of limestone on both sides of the river extending entirely across the river bed. The lines of cleavage making an angle of about 115° to a line drawn horizontally in direction of stream flow. The sides of the gorge rise almost vertically to a height of about 150 feet above the river bed, and are only 500 feet apart, but gradually diverge as one passes down stream until a point is reached about 600 or 700 feet below the outcrop where the walls spread rapidly apart, and the gorge itself becomes a valley of moderate dimensions. Any site, therefore, below the outcrop would necessitate a higher and much longer dam. There would be no advantage in placing the dam further up stream except in the resulting decrease in height of dam, but this advantage would be offset by two things, first, the decrease of storage, and secondly, the necessity of

extensive excavation for the dam foundation, because there is no other visible outcrop of rock occurring simultaneously on both banks of the stream.

After carefully comparing the increase in height and length of dam with head gained, it was found utterly out of the question to select a site lower down stream, and we therefore decided to place the dam at the point marked on our map. The consideration which determined the height of the dam was the maximum elevation to which we could back our storage water, and as we found from the survey that we had a rather small storage basin, we determined to utilize every possible opportunity for increasing the storage. We therefore decided to select the 265 foot contour as the crest of our dam, and to protect the C. C. & O. tracks at the B. M., by raising the tracks 5 feet for a distance of about one half mile, and furthermore, protecting same by constructing a concrete wall for a like distance along the tracks, the height of which we determined later from our headwater curve to be 5 feet.

The storage obtainable with a dam at this point and with a maximum storage elevation of 265 will of course depend upon how low we wish to drain the basin. After examination of numerous hydro-electric plants already in existence, and taking into consideration the effect of varying head on the different makes of turbines, we found a twenty foot variation of head to be permissible, but owing to the additional variation of friction head in the penstocks which we use and which will be discussed later, we took fifteen feet as the depth of layer

available for storage. The volume of this water was found from the map to be approximately 223,000,000 cubic feet, and it covers an area of 348 acres, part of which is above the 250 foot contour, thus making the average depth of layer stored 13 feet.

Having found a location for the dam, and decided upon its height, the next question was to determine the flow of the stream, and see what could be expected in the way of steady discharge, and this brings us to the—

HYDROGRAPH.

"The flow of a stream is constantly changing and any single measurement of that flow will not furnish sufficient data upon which to base an intelligent estimate of the extent of its possible, or even, probable economical development of power. A knowledge of the economical possibilities of such development must be based upon a much broader knowledge of the variations that take place in the flow of the stream. In order to fully appreciate the power value of the stream, the character and extent of its daily fluctuations must be known or estimated. Averages for the year, monthly averages and estimates for the average power have been ordinarily taken as a basis for water power estimates, but they are more or less misleading, unsatisfactory and uncertain, for the reason that such averages include extremes, the maximum of which are often unavailable for water power purposes without more extensive storage than is ordinarily available. The maximum and minimum flow which affect

the power of the stream not only through the quantity flowing, but also through the head as well, are of the utmost importance for a broad consideration for water power. So also is the knowledge of the various stages of the flow, and the length of time that each will prevail. Such knowledge demands daily observations or estimates of daily flow, which can be represented in graphical form by the hydrograph."

Since no hydrograph of the river at this point existed, it was necessary for us to construct what is known as a comparative hydrograph, based upon the assumption that the flow per unit of area at different points on the same stream, or at points on different streams under similar circumstances is essentially the same. We measured the discharge of the river at Section A-A, Plate I., this section being located at a point comparatively free from eddies and cross currents. This measurement was conducted by dividing the cross section of the stream into 5 foot sections, soundings being taken at the edge of each section and the average of each section being determined by the use of a Price current meter. Plate II. shows the method of procedure and the results obtained thereby. This discharge we found to be 2,400 second feet on the 8th of March, and on the same date the government gaging station at Bird's Bridge, about 30 miles below Embreeville, and on the Nolichucky, showed a discharge there of 3,300 second feet. The ratio of discharges at Embreeville and Bird's Bridge is then 0.73 . As a check upon this method we found the ratios of the drainage areas at the two places to be $780 \div 1,099 = 0.71$. Taking a mean of the two

ratios , or 0.72, we were now able to plot a hydrograph of the river at Embreeville by multiplying the discharges at Bird's Bridge station by the factor 0.72 . The Bird's Bridge discharges we first plotted from rating curves and daily observations, obtained through the kindness of Mr. Hall, U. S . G. S., of Atlanta, these observations extending over a period of nine years. Plate III. is the hydrograph of the Nolichucky at Embreeville for the same period, i.e., nine years.

THE RELATION OF STREAM FLOW AND STORAGE TO POWER DEVELOPED.

As stated before the storage is 223,000,000 cubic feet with a 15 foot layer of water available for use. On examination of the hydrograph from 1903 to 1912 it is seen that 1903 and 1904 are the lowest years, 1904 being an exceptionally low year the discharge remaining at 250 second feet during the entire month of October. Going back to 1903 and supposing that we wish to take 600 second feet as our minimum average daily flow, the average flow during July is 800 second feet, hence our basin would be full at the end of July. The average daily flow for August is 650 second feet up until the 24th of the month, when the discharge falls to 400 second feet, so that for a period of 39 days we must supply 200 second feet from the amount previously stored. $223,000,000 \div 200 \times 86,000 = 13$, i.e., the number of days that our storage would supply the deficit, which carries us to October the 4th. From October the 4th to November 15th, the average discharge is 350 second feet, therefore equivalent aux-

iliary power must be supplied to cover the deficit of 250 second feet for a period of 43 days. From November 15th to Dec. 17th the average discharge is 450 second feet, and since there is no water stored, the equivalent power of 150 second feet must be supplied by the auxiliary plant for a period of 33 days. From December the 17th to January the 20th, 1904 the average discharge is 500 second feet, and there is yet no storage the auxiliary plant must supply power that is equivalent to 100 second feet for a period of 33 days. From January 20th to September 5th 1904 the flow is at no time below 600 second feet, except for three or four days, and on the 5th of September the basin is full. From September 5th to November 15th the average discharge is 300 second feet and 300 second feet must be drawn from basin which will last nine days, or until September 15th. Then 300 second feet equivalent auxiliary power must be supplied for a period of 60 days, i.e., until November 15th when the equivalent of 100 second feet must be supplied for 20 days. Again on September 15th, 1905, the discharge drops to 500 second feet and remains there for a period of 76 days, but as the basin is full on September the 15th, the water stored will supply deficit of 100 second feet for 28 days, so that the auxiliary plant will be called on for power equivalent to 100 second feet for a period of 48 days. From December 2nd, 1905, until September 20th, 1910, the flow is at no time below 600 second feet for a period greater than 15 days which deficit can be supplied from storage. On September 20th, 1910, the basin is full, the discharge drops to 500 second feet and remains there until October 15th, a period

of 25 days, during which time the storage must supply 100 second feet, which it does, and on October 15th the storage supply being exhausted the auxiliary power must again supply equivalent power of 200 second feet for a period of 45 days. In August, 1911, for a period of 30 days, power equivalent of 200 second feet will be demanded of auxiliary. The total demand for auxiliary power from January, 1903, until January, 1912, is 300 days, the demand varying from 100 to 400 second feet equivalent power, so that the auxiliary plant will have been in operation for 9.4% of the time during the nine years. The above calculations are based on unity load factor, i.e., we have supplied an output of power equivalent to 600 second feet discharge continuously, day and night, for nine years, with the aid of auxiliary power supplied for length of time as already stated.

Daily load curves were plotted for five large cities, and an average curve taken from these, this curve being shown on Plate V. The load factor derived from this average was 48 %, while the load factor of the Knoxville Railway and Light Co. was 46 %. The question now arises whether it is more advisable to take the average of these two, i.e., 47 % of the load factor and with the available 600 second feet solve for peak load on the assumption that the load factor will never go beyond 47 %, or whether it is better to take a lower load factor, thus obtaining a lower peak load and allowing for future sale of off-peak power. We decided to take 50 % as our load factor, which value is extremely high for the locality in which we shall market our power. On this basis our peak load would rise to power

equivalent to 1200 second feet.

This amount of power can be, with the assistance of the auxiliary plant, generated the entire year, however the summer demand will be lighter than the winter demand by about 20 to 25 %, and the winter discharge would be sufficient to generate a great deal more power than the average power as calculated above. Further examination of hydrograph shows that the summer season must extend to the middle or last of December, and from then until first of July secondary power could be obtained. It would be entirely unusual and extremely unwise to take care of all of this additional power at the outset, nevertheless, we finally determined to install one additional unit as a reserve and as a means of supplying some secondary power. However, we will base all of our calculations as to output on the 600 second feet average daily discharge, with a peak demand of 1200 second feet.

If the power house were located at the site of the dam, the theoretical peak output would be $1200 \times 71 \div 8.8 = 9,700$ H. P. (Since the available head at that point would be 71 feet, neglecting friction losses.) But if the power house location were carried down stream by use of penstocks, the head would rapidly increase, thereby increasing the output. The topography of the locality makes it possible to construct a pipe line as far as Embreeville in an almost straight line, this pipe line following the river bank for a distance of 8,500 feet, then leaving the course of the river to cross a neck 1500 feet wide, as noted on Plate I., making a total length of line of 10,000

feet, the head gained in this length being 61 feet. The question might be asked, why extend the line across the neck, thereby demanding a much longer penstock, but this is answered by the fact that 40 feet of the 61 feet drop lies between the opposite sides of the neck. With a power house located at Embreeville the theoretical output would be $1200 \times 132 \div 8.8 = 18,000$ H.P. (Since the head here is 132 feet, neglecting the losses due to friction.) We have then the choice of two alternatives, the location of the power house at the site of dam, developing 9,700 T.H.P., and the location of the power house at Embreeville, developing 18,000 T.H.P.; in the latter case we must construct in addition to the power house and dam, an expensive penstock line, 10,000 feet in length, and it is very probable^{cost} of this line would exceed the cost of the dam itself. (Which we later found to be the case.) The only wise and rational course to pursue in the selection of the proper alternative is to allow the issue to be governed by the conditions of the —

MARKET FOR THE POWER.

Since there are no large towns in the immediate vicinity of the proposed plant it will be necessary in any case to construct long transmission lines to reach points where the total sale of power would amount to more than 3,000 H.P., and if we choose the site of the power house at the dam, thus developing 6,700 H.P.-actual (Under the assumption of an overall efficiency of 70 %.) We would be compelled to run the trans-

mission lines to Knoxville in order to sell all the power generated, unless we count upon future growth in immediate vicinity eventually demanding the excess of power. If we did this our first several years of sales would be exceedingly low and the investment would be a losing one during those years as the returns from the sale of 3,000 H.P. would not pay dividends enough to more than cover the interest on the money invested. Under these conditions we thought it advisable to extend transmission lines to Knoxville, and by so doing we could find a market for our power which would take a large percent of the total amount generated, even when the power house is located at Embreeville.

Assuming that we could sell 50 % more power in the towns considered than is sold by steam-electric plants, we prepared the following table showing the probable distribution and sale of power.

TABLE I.

Erwin.....	200 H.P.
Johnson City.....	1500 H.P.
Greenville.....	800 H.P.
Morristown.....	1250 H.P.
Jefferson City.....	250 H.P.
Knoxville.....	<u>7250 H.P.</u>
Total	11250 H.P.

Of course the allowance of 50 % beyond the present steam-electric output of the various towns gives room for growth of at least two or three years, as it would take that long, or longer, to raise the demand to the figures cited above, but there

is no doubt that this can be done in this length of time, provided that we can afford to sell power at rates reasonable enough to induce factory owners to demand additional power. From the above figures we think it is safe and advisable to develop as large an output as possible, and therefore conclude to select the Embreeville alternative as the proper location of the power house. Having now determined the conditions governing the selections of the site of dam and power house, the height of dam and the theoretical output of plant, we are ready to begin calculations as to size and strength and cost of the installation.

DESIGN AND COST OF FLOOD WALL FOR PROTECTING C.C. & O. TRACKS.

The cost of raising the double line of C.C. & O. tracks for a distance of one half mile to a maximum height of 5 feet, and an average height of $2\frac{1}{2}$ feet, is approximately as follows:-

Filling— $2,640 \times 2\frac{1}{2} \times 23 = 150,200$ cubic feet, or 5,600 cubic yards @ \$0.25 per cubic yard = \$ 1,270.00, cost of track alignment to be borne by railroad company.

A concrete wall 5 feet high will need to be 3 feet thick at base and 1 foot at top in order to safely withstand the pressure that would result in case water should rise to the top. These dimensions were calculated on the basis that the specific gravity of the concrete was 2.00, and were calculated to provide against overturning when water is at top of wall. This wall, one half mile long will contain 880 cubic yards of concrete @ \$ 4.50 per cubic yard--including setting--gives \$3,950.00

Excavating 150 cubic yards @ \$ 1.00 per cubic yard = \$ 150.00

Total cost of track raising and protection.....\$ 5,370.00

COST OF LAND AND WATER RIGHTS.

Total land required is 360 acres, including pipe line right of way, which is 50 feet wide, and power house site, but the area of the river bed itself is 55 acres, leaving 315 acres to be purchased. The average cost of land in this neighborhood is \$ 38.00 per acre, making a total cost of land...\$ 12,000.00
The right of way for pole setting will be included under the transmission department.

Since the stream is not navigable the United States water rights are free to whosoever chooses to utilize them, and so far, the state of Tennessee has been content to blindly give away any and all water rights, so that our water rights will cost us nothing.

DESIGN, CONSTRUCTION AND COST OF DAM.

THE SPILLWAY:- See Section B-B Plate VIII. The type of spillway most widely in use in present day construction seems to be the Ogee, or a modification of the Ogee. After a careful consideration of the many different types, we thought it best to use the Ogee spillway, first, because the downstream face is curved in such a manner as to insure that the water will always cling to the surface, and will prevent the formation of a

vacuum under the falling sheet, and as it is generally conceded by engineers that the formation of a vacuum on the down stream side is responsible for the trembling often felt in the vicinity of an overfall dam; secondly, because the crest on the upstream side is sloped in such a manner as to prevent any undue shocks caused by floating bodies striking against the crest, this slope at the same time materially increasing the effective discharge of the spillway. Mr. Merriman states (Page 160— Hydraulics) that the coefficient M in the equation for discharge, as determined by experiment by government engineers for this particular type of spillway, has the value, $M = (3.78 - 0.16s) \times H^{1/20}$, where s is the ratio of the horizontal to the vertical projection of the straight inclined part of the crest. A good value for s is 1.73, making the included angle 30° , an angle which will permit logs of usual diameter to ride easily over the crest. With this value of s , $M = 3.51 H^{1/20}$, and the formula of discharge, $q = MbH^{3/2}$, becomes, $q = 3.51 H^{1.55}$, because b is taken as unity. The discharge as found per linear foot of spillway is as shown below:-

TABLE II.

$H =$ Head on Crest.	q Sec. Ft.	$H =$ Head on Crest.	q Sec.Ft.
0.5	1.22	3.0	19.25
1.0	3.51	3.5	24.50
1.5	6.56	4.0	30.10
2.0	10.25	4.5	35.90
2.5	14.50	5.0	42.60

TABLE II. Con.

H= Head on Crest	q Sec.Ft.	H= Head on Crest	q Sec.Ft.
5.5	49.25	9.0	105.45
6.0	56.40	10.0	123.20
7.0	71.50	11.0	144.85
8.0	88.20	12.0	165.00

The only way of determining the proper length of spillway is to draw a number of curves for different lengths, and noting which one comes nearest to fulfilling the desired conditions. We drew discharge-head curves for 80, 100, 120, 140, and 300 foot spillways, as shown in Plate VI. There were two conditions governing the selection of any one of the above lengths, first, the maximum height of flood water must not rise above elevation 272. From the hydrograph it is seen that the maximum discharge did not exceed 20,000 second feet during the last ten years, and a discharge of 20,000 second feet would raise the water elevation to 272 with a 300 foot spillway, as is shown from the 300 foot spillway curve. It is therefore necessary to use at least a 300 foot spillway, since a shorter one will not meet the above requirement, and as this ~~this~~ length is sufficient to meet this requirement, there will be nothing gained by using a longer one. The second consideration is that the variation of head with variation of river discharge shall be as small as possible. To determine this variation of head it is first necessary to draw the tailwater curve of the river at the

power house site. This curve was drawn from a few points obtained as follows:- The elevation of water surface at power house site was noted on certain date, later, from the hydrograph we were able to find the discharges corresponding to the heads measured. We were fortunate enough to obtain one measurement during a flood when the head, or depth, reached 7 feet. Since one bank of the river is very steep and the other extends backward with a gentle uniform slope above the 7 foot mark, we assumed that the curve would have almost the same slope beyond the 7 foot mark as it has between the 5 and 7 foot marks, and drew our tailwater curve accordingly, as is shown on Plate VI. This curve might be in error from 20 to 30 % and yet such an error would not effect the results obtained by its use more than $1\frac{1}{2}$ %. With all turbines running, or 1200 second feet discharging through the turbines, the maximum variation of head with the 300 foot spillway as found from curve, is 0.9 foot with river discharges ranging from the zero ,or no flow point, to the maximum. For constant river discharge and varying turbine discharge , the maximum variation of head is 1.6 feet as found from the short dotted curves of Plate VI., which curves are the headwater curves under varying turbine discharge. The total maximum variation is then, only 2.5 feet, which is very satisfactory indeed. The next step is to work out the curve for the down stream face . Every one knows that the curve taken by a jet of water issuing from an orifice is a parabola, undoubtedly then , the surface of the spillway must be a parabola if the friction is to be a minimum, only it must be a somewhat flatter

parabola than the one calculated for the maximum head in order that water shall not shoot clear of the surface during times of flood. The equation for the parabola necessary to meet our demands was found to be:-

$$y = \frac{gx^2}{2v^2}, \text{ and since } v = Mxh^{\frac{1}{2}}, y = \frac{gx^2}{(2 \times 3.51)^2} \times H^{-1/20}$$

We took H in the above equation to be 6 feet, although the maximum height taken by the water during the last ten years was 6.5 feet, but we may take this value of H = 6.0, since the force of adhesion of water to the concrete surface will more than the difference due to a head of 0.5 foot. Placing H = 6.0 in the above equation, it reduces to $y = 0.183x^2$. This parabola we extend from the crest of the spillway to a distance 43 feet below the elevation of the crest, the remaining length of the curve being a circle of radius = $65\frac{1}{2}$ feet, this circle being tangent to the parabola at the point of junction, and extending far enough so that water leaves it at lower end parallel with river bed and at a height of about $1\frac{1}{2}$ feet above same. The $1\frac{1}{2}$ foot space being filled with rip-rap, extending for some 30 or 40 feet down stream as an additional protection against scouring of the river bed. Plate VIII.-"C" shows dimensions of the cross section of the spillway. Calculations of this section for strength and stability were deferred until after calculation had been made for the Section C-C, and were then determined by comparison with the latter section.

SEC.C-C. "The following general principles of dam construction must be observed:-

- (1)...The dam must have suitable foundations to sustain the pressure transmitted through them, which must be either impervious or rendered practically so.
- (2)...It must be stable against over-turning.
- (3)...It must be safe against sliding.
- (4)...It must have a sufficient strength to withstand the strains and shocks to which it is subjected.
- (5)...It must be practically water tight.
- (6)...It must have essentially water tight connections with its bed and banks, and if bed or banks are pervious, with some impervious strata below the bed and within the banks of the stream."

Since the foundations are to be laid directly upon a stratum of limestone, and since the lines of cleavage of this stratum make such an angle with the river bed as to make it impervious, grouting will be unnecessary, and it is only essential that a water tight joint be obtained to prevent leakage. Having satisfied all of the above stated principles except the 2nd, 3rd and 4th, we will now turn our attention to these.

The possibility of the dam sliding on its base we safeguarded against by excavating three trenches in the solid rock, to an average depth of 5 feet, and for an average width of 12 feet, also extending same from bank to bank. The detail of this method of anchorage is shown on Plate VIII., Section "C".

Mr. Mead states that for masonry and concrete dams not over 100 feet in height, built on rock foundations, the design can be based on the condition that the lines of pressure must lie within the middle third of the profile, and he gives the following formulae for the calculations of such dams:-

$$b = 4.0 + 0.07 \times H \dots\dots\dots(1)$$

$$y_0 = 1.8 + 0.05 \times H \dots\dots\dots(2)$$

where b is the width of the top, y_0 the height above water level, and H the greatest depth of water. These two empirical formulae are due to Coventry. Taking $H = 86$ feet (Allowing a factor of safety.) $y = 6.5$ feet = Distance BX, Section C-C, Plate VIII. - "C", and $b = 10$ feet = Distance AB of same section and plate. Also, $y_1 = b \times s^{\frac{1}{2}} \dots\dots\dots(3)$

where y_1 = Height of the rectangular portion = Distance BD, Plate VIII.--"C", and s = Specific gravity of the material used. Kent gives 2.20 as an average value of the specific gravity of concrete. Since our dam is to be of cyclopean concrete its specific gravity will be even higher, but we will use the figure, $s = 2.00$ in order to provide an additional factor of safety. With this value of s , we get $y_1 = 14$ feet.

The down stream side of dam is now to be sloped so that the resultant pressure, with water at the top of dam, shall fall within the middle third of length of any joint. We divided the height of the dam into ten foot sections and calculated the length of each joint from the formula below.

Taking for example section GH, Plate VIII.—"C".

$$GH = (B - C^2)^{\frac{1}{2}} - C \dots\dots\dots(4)$$

where,

$$B = 6 \text{ m } \frac{(\text{Area ABFE})}{FH} + \frac{(BH)^3}{8 FH} + E, \text{ and where}$$

$$C = \frac{1}{2} \frac{(\text{Area ABFE} + EF)}{FH}$$

m being the distance from F to the line of action of weight of masonry above EF. Substituting in this formula we get GH = 20 feet.

"When m becomes greater than one third the length of the joint it becomes necessary to batter the upstream face, also so that the lines of pressure with reservoir full and empty both lie at the limits of the middle third of the length of any joint." The lengths of the joints, as NP may be found from the formula:-

$$NP = \left(\frac{BO^3}{8 LO} + \left(\frac{KM}{2} + \frac{\text{Area ABMK}}{LO} \right)^2 \right)^{\frac{1}{2}} - \frac{\text{Area ABMK}}{LO} - \frac{KM}{2} \dots\dots\dots(5)$$

and the value of OP is,

$$OP = \frac{2(\text{Area ABMK})(NP - 3 \text{ m}) - (LO \times AM^2)}{6(\text{Area ABMK}) + LO(2KM + NP)},$$

where m is the same as already explained. Substituting in this equation gives us, NP = 46½ feet, and OP = 3 feet. The other sections were calculated by the same formula. As a check upon these formulae the following calculations were also made:-

The center of pressure is known to be at a distance one third the depth from the bottom, which is 29 feet. P was found by the formula $P = w \int_0^H y^2 dy$ to be 230,000 lbs. per linear foot. The center of gravity of the dam lies in a line marked W, in Section C-C, Plate VIII.—"C", found by a simple process of mechanics, 28 feet from what may be called the acting toe of the dam, i.e., a point, where, if the dam really overturned, the concrete

would hold as a fulcrum, and the value of W must be equal to $P \times 29+28 = 235,000$ lbs. per linear foot. The actual weight of the dam with dimensions as used, and with $s = 2.00$ is 434,300 lbs. per linear foot, giving a factor of safety of 2 against overturning.

The shear along any section is equal to the algebraic sum of all the loads above that section, or, $V = \frac{wx}{h}$, where V is the shear in pounds per square foot, x the distance in feet from top of dam, and h the length of section. For example, along section EF, $V = \frac{62.4 \times 24}{13} = 110$ lbs. per square foot, and the remaining sections all show an even less value for V, while shearing stress of concrete is commonly taken at 100 lbs. per square inch.(Merriman.)

Returning now to the spillway, we find that section C-C is not as thick as the spillway must be if we are to use the curve previously derived for same, and desiring by all means to use this curve, we increased the thickness of section C-C, as shown by the solid line drawing of Section C-c, Plate VIII.-"C"; dotted line drawing shows this section as calculated by formulae. The reason for so increasing the thickness of Section C-C is twofold, to give additional strength, and to provide a flanking wall for spillway. Even then we had to build a low flanking wall at the lower part of the spillway, as is shown in the plate. The calculation of the strength of the spillway portion now takes care of itself, as it has already been stated that it is thicker than section C-C as derived from

the preceding formulae.

The cross sectional area of Section B-B is 3,070 square feet, its length being 300 feet, giving us 34,200 cubic yards of concrete.

The cross sectional area of Section C-C is 2,900 square feet, and referring to Plates IV, and VIII--A, and VIII--B, the average length is found to be 200 feet, giving us 21,500 cubic yards. From these figures is to be deducted the volume of two sluiceways, two penstock intakes and a passage way, totaling 800 cubic yards as shown by figures based on dimensions of Plates VIII. and IX. This leaves $34,200 - 21,500 - 800 = 54,900$ cubic yards of concrete to be used in the structure, @ \$ 4.50 per cubic yard (Gillette's "Handbook of Cost Data" says average cost of concrete laid, is \$ 4.85 per cubic yard, but we can procure all necessary stone for the cost of digging.) gives \$ 247,000.00 for cost of material used in the dam, and laying of the same.

Excavation for anchorage, both on banks and in river bed, of 5,000 cubic yards @ \$ 1.00 per yard = \$ 5,000.00 .

Total cost of dam, not including cost of engineering, is then\$ 252,000.00 .

The calculations for sluiceways, penstock intakes, gates and racks, and passage ways are now to be taken up in the order named.

SLUICeways.

The sluiceways are intended as an additional safeguard against the excessive rising of water above the crest of the spillway, and are designed to discharge 3,000 second feet when the depth of water on the crest is 5 feet. At time of maximum past discharge of 20,000 second feet the water level would be at 272, and a possible further rise of 3 feet would flood valuable property, namely the C.C. & O. tracks. Any further rise beyond the 3 feet could be handled by the opening of these sluiceways. The head on the sluiceways being approximately 75 feet, $q = C a (2gh)^{\frac{1}{2}}$, taking $C = 0.80$ (Merriman, page 187.) then $a = 54$ square feet. If we use two 9 foot compound tubes with central section 4 feet in diameter and ratios of lengths of sections (See Merriman) as shown in Plate VIII-"C", we can discharge 3,000 second feet. The object of reducing the central section is to decrease the size of valve required to close the sluiceways. Two motor-operated 4 foot valves, designed for a pressure of 50 lbs. per square inch, which is about the pressure to which they will be subjected, can be purchased from the Crane Co. for \$ 1,700.00 each, total \$ 3,400.00.

Two 2 H.P. motors required for the operation of these valves can be had for \$ 172.00 .(Since the price of valves as quoted above does not include motor.)

Total cost of sluiceway equipment.....\$ 3,572.00.

THE PENSTOCK INTAKES.

Plate IX. gives details of the intakes. Each intake is divided into two openings, each opening having a well rounded approach. This arrangement almost excludes entrance losses, and at the same time requires two gates and racks, each 19' x 9½', instead of a very much larger gate, as would be required if there were no division. This larger gate would be extremely difficult to design owing to the great pressure it is forced to bear. The intake of penstock No. 1 was placed approximately 28 feet from the bank, since much trouble is caused by swirling if the intake be placed nearer to the bank than two diameters. The principle reason for placing the intake so low, is, that a penstock intake passing through the dam from outside bottom to inside, near water surface, is a source of great weakness to the structure, and would call for much greater dimensions at this section. Again, in time of low river discharge, it may be found advantageous to draw the water level in the basin to a lower depth than the 250 contour, in which case, the upper edge of the intake should be at least 10 feet below the new level. If this level were 240, the upper edge would properly be at elevation 230, whereas it is now at elevation 215, or, 15 feet lower than is necessary. This difference of elevation does not increase the pressure—and consequently the size of the gates—enough to warrant the abandoning of the lower position in favor of the higher one. The interior of the down stream portion of the intake is 11 feet in diameter, allowing the wooden stave

penstocks to be properly connected with the intake. The concrete column, or rather division, between the gates of each intake was designed to bear the pressure exerted upon one gate, but is to be of re-inforced concrete.

GATES AND RACKS.

The area of each gate is 180 square feet— to be built of $\frac{1}{2}$ inch plate steel, the weight of this plate being 3,700 lbs. The pressure of water at this depth is 3,700 lbs. per square foot; using twelve cross braces, the load per brace is 45,000 lbs., and a brace of sufficient strength, and at the same time of the smallest possible size, is best made of two 8"x 8" angles, placed back to back, and rivited to the plate. Total length of braces per gate is 216 feet, weight is 12,300 lbs. The vertical braces are to be four in number, each consisting of one 12" channel, the weight of the four being 1,560 lbs. Total weight of gate is 17,000 lbs., @ $3\frac{1}{2}$ cents per pound, gives cost of \$ 620.00 . The total cost of the four gates, including the material and construction, is then \$ 2,500.00 . Since the weight of each gate is 17,000 lbs., and the total pressure on the gate is $8 \times 19 \times 3,700 = 666,000$ lbs., the coefficient of friction of iron on iron being 0.50, then the load of the motor is $17,000 + 0.50 \times 666,000 = 350,000$ lbs. Raising the gate two feet per minute gives us 700,000 foot lbs. per minute, so that it will take a 21 H.P. motor to do this work. We prepose raising the two gates belonging to one penstock by the use of one 50 H.P. motor.

Hoisting machinery capable of standing this service may be

purchased from the James Leffel Co. for \$95.00 per hoist.
 One 50 H.P. motor (D.C.Motor) will cost \$670.00 ,so that the
 total cost of the four gates,two hoists, and two motors will
 amount to\$4,030.00 .

The operation of these gates is discussed later.

TRASH RACKS.

These racks are designed to stand two thirds of the
 gate pressure, and are to be built of $\frac{1}{2}$ " x 3" iron rods, spaced
 horizontally 2 inches apart. It takes 10 cubic feet of rods
 per rack, or 5,610 lbs. @ $3\frac{1}{2}$ cents per lb. = \$ 204.00 .
 Allowing \$ 280.00 for braces and framing, the cost per rack is
 \$ 484.00 , making the cost of four racks \$ 1,930.00, this, plus
 the cost of two sluiceway racks ,which is approximately \$780.00,
 gives \$ 2,710.00 for all racks. Cost of bracing \$ 65.00.
 As the weight of each rack is 5,600 lbs., and the total pres-
 sure (allowing two thirds pressure of gate) is 444,000 lbs.,
 it will require 455,200 foot lbs. of work per minute to raise
 rack at rate of 2 feet per minute. This equals to 13.8 H.P.,
 and we will use one 30 H.P. motor for raising the two racks be-
 longing to the same penstock, the cost of a 30 H.P. D.C. motor
 being \$560.00 . Hoisting machinery @ \$75.00 per hoist.
 Sluiceway racks are to remain permanently in position. Total
 cost of the six racks, two hoists and two motors :-

Total.....\$4,045.00.

THE PASSAGE WAY.

This passage way gives access to the sluiceway valves, the motors for operating these valves being placed within it. The passage also gives access to the penstock intakes by means of a manhole. The passage floor is at a sufficient height above the top of the intakes to insure against the pressure in intakes breaking through into it. The manhole opening will be closed by a heavy iron plate, suitably fastened into position in such a manner as to be easily removed. The dimensions of the passage are made large enough to allow handling of all material that will necessarily pass through it. Outlet of passage is at convenient point at bank of stream.

THE PENSTOCK LINE.

The penstock line is to be 10, 000 feet in length, and as this length is great enough to cause a large loss of head due to friction, unless the line is of large diameter, it will be seen at once that the cost of constructing such a line will be a very heavy one. As the cost of pipe increases rapidly with a slight increase of diameter—when the diameter is above 5 or 6 feet—it became necessary to decide upon what would be the best course to adopt, namely, to purchase smaller pipe, and thereby lose a large head, or to install a larger line and save several feet of head. The problem was solved in this fashion.

At peak load we have 1200 second feet to discharge. From catalogues furnished us by different manufacturers we found

the coefficient of friction for the different sizes of penstocks as manufactured by them, and obtained the loss of head in a 10,000 foot line discharging 1200 second feet. This loss at peak load being 12 feet when using two 11 foot inside diameter steel lines. These lines as manufactured by the Dayton Globe Iron Works will cost \$ 44.80 per foot , not including rivets or construction. The following data were furnished us by the Pacific Coast Pipe Co.: - " A 12 foot inside diameter wooden stave line, discharging 1200 second feet , would have a loss of 1.62 feet per 1000 foot length. The same diameter, discharging 600 second feet, would have a loss of 0.45 feet per 1000 feet. A 10 foot inside diameter wooden stave line, discharging 600 second feet, would have a loss of 1.08 feet per 1000 foot length, and a 9 foot wooden stave line, discharging the same amount, would have a loss of 1.81 feet per 1000 foot length" No other manufacturers showed as favorable figures as those quoted above, therefore, the calculations are based upon their figures. Again quoting from the P.C.Co.: - " Assuming average head on line to be 90 feet, when reservoir is full of water, and under static conditions, a 12 foot wood stave pipe line of proper thickness, would cost \$ 23.00 per foot, a 10 foot line, about \$ 16.70 , a 9 foot line about \$ 14.25. These prices cover the delivering of all materials necessary for the construction of the pipe line, namely, staves, tongues, bands and shoes, f.o.b. cars, Knoxville, Tenn., and the assembling of these materials into acceptable pipe on a prepared right of way. No allowance is made for the transportation of the materials from Knoxville to the point of use, and the distribution of the same along the trench; neither is

any allowance made for earth work of any description, nor the providing and placing or permanent supports, such as sills, cradles, trestles, &, that may be required. A determination of whether one or two lines would make a better installation for the work could only be made by one thoroughly acquainted with all local conditions. As a general rule, two parallel lines cost about 35 % more than a single line of the same carrying capacity. The two pipe lines side by side, require considerable more excavation, if laid in a trench, than a single line, but some advantage is gained by having two independent pipe lines, in that, repairs might be made on one without throwing the plant entirely out of commission."

The cost of the first mentioned double 11 foot steel line, including construction and riveting, is \$ 468,000.00 , maximum loss of head being 12 feet. The cost of a single 12 foot wood stave line is \$ 230,000.00, laid in place, maximum loss of head 16.2 feet, minimum, 4.5 feet. The cost of double 10 foot wood stave pipe line is \$334,000.00 , laid in place, maximum loss of head 10.8 feet, minimum, 2.8 feet. Cost of double 9 foot line, \$ 285,000.00 , laid in place, maximum loss of head 18.1 feet, minimum 5.5 feet. The steel pipe may be here dropped out entirely, because of its excessive price, particularly, when we find from actual proof that the life of the wood stave line, when properly laid, is even longer than that of the steel line. A great advantage of the wood stave line over the steel , in addition to the greater durability, is that the former is never affected by tuberculation.

Let us take the 10 foot double stave line as the standard of comparison. Suppose that we were to use a single 12 foot line, then the loss of head exceeds that of the above chosen standard by 5.4 feet, the difference in cost of the two is \$ 104,000.00 in favor of the 12 foot line, but a loss of 5.4 feet head means a loss of 740 theoretic horse power, or, assuming an overall efficiency of 72 % we would actually lose 550 H.P., and at \$ 20.00 per Horse Power per year, this would amount to \$ 11,000.00 annual loss, so that in nine years time, the sale of power due to the extra head saved by using the double 10 foot line would pay the difference in initial cost between the double 10 and the single 12 foot lines.

Since a single 9 foot line is out of the question, owing to excessive loss of head, we will next examine the double 9 foot line. The loss of head exceeds that of the chosen standard by 7.9 feet, meaning a loss of 800 actual H.P., which at \$ 20.00 per H.P. per year equals \$ 16,000.00 . The difference in cost of the double 10 and the double 9 foot lines would be \$ 49,000.00. The sale of power due to head saved by using a 10 system would pay the difference of initial costs in three years.

These figures point conclusively to the selection of the double 10 foot line as the proper one to be used, since a double line larger than this would cost more than the small increase of gained power is worth. Penstock lines will then cost us a total of\$ 334,000.00.

These penstocks are to be tapped at their lowest elevation by 24" drain valves, one to each penstock. These valves

afford a ready means of draining pipe in case of necessity for repairing. The cost of these two hand-operated valves, as quoted by the Crane Co., will be\$ 1,020.00.

From experimental data furnished us by the manufacturers we found that the head loss due to bends would be 0.70 feet for the 20° horizontal curve—shown on Plate I. This curve was selected because it was the smallest one we could use without running line below the high water mark; the loss for the vertical curve at the power house will be approximately 0.33 feet, making a total loss due to bends of 1.03 feet.

Plate VII. is a virtual profile of the penstock lines, showing the relative elevation of dam, power house, and the vertical curve of the penstocks.

PENSTOCK RIGHT OF WAY.

The cost of land for pipe line right of way has been already taken up under the heading " Cost of Land." The type of trestle, used to support the pipe, is shown in Plate IX. These trestles are to be placed every eight feet. It takes 80 board feet of lumber per trestle, and takes 1200 trestles per line, giving 95,000 feet lumber, which @ \$ 15.00 per M gives \$ 1,450.00. Each trestle rest on two concrete piers. Allowing 2 cubic feet per pier, it requires 180 cubic yards of concrete per pipe line, which @ \$ 4.50 per cubic yard = \$ 800.00. Cost of labor for erection—also allowing for necessary clearing — @ \$ 5.00 per M (Gillette) = \$ 500.00.

Excavation for piers, 2 cubic feet per pier = 180 cubic yards per line, and allowing 500 cubic yards for other necessary excavation, makes total excavation of 680 cubic yards @ \$ 0.75 per cubic yard (Excavation mostly in dirt) = \$ 510.00. Total cost of preparing double line of right of way\$ 6,520.00.

THE POWER HOUSE.

As previously mentioned, the power house is to be located at Embreeville. It is to be located at the foot of a bluff, which rises to a height of 100 feet immediately behind it, forming here a sort of saddle. The penstock passing through this saddle, enters the power house from above. The location is better shown than described and a reference to the map of Plate I. will make it clear. At the foot of the bluff is a narrow shelf of rock about 15 of 20 feet wide, and it is upon this shelf that the building will be erected. The rear portion of the building is to be built against the face of the cliff itself, the main portion, that is, the generator room will be out just beyond the present river bank, while the tail races will have to be excavated from the present bank and bed. Plate X. is a cross section of the power house and shows in detail the features just mentioned. The number and the size of the units to be installed, will of course be the chief factor in determining the power house dimensions. Having obtained these sizes and number of units by calculations which will be discussed later, and allowing liberally for extra floor space, we planned the building as shown by plate X. The overall dimensions being 150 feet by 66 feet. The height of the main building is 45 feet , that of the transform-

er and switch room department is $41\frac{1}{2}$ feet.

Our method of turbine installation is a rather novel one, namely, that of installing the turbines all in one casing, this casing, however, to be so arranged that it can be divided into two sections by means of a hydraulic-operated dividing gate, see Plate X. This scheme of installation was chosen because it renders one set of turbines accessible for repairs without shutting down the plant, and because it does entirely away with the loss of head entailed in running a separate pipe from the penstock header to each turbine, it also offers additional advantage of cutting down penstock loss when only one or two units are in operation.

The turbine casing, or, more properly speaking, the wheel chamber, is to be constructed of re-inforced concrete, and as the length of the turbines is 20 feet, it will be 20 feet in width. Allowing 20 feet between center lines of turbines, except the middle two, where we allowed 28 feet on account of dividing gate, the interior length of wheel chamber will be 86 feet. The thickness of the walls determined by calculating them as beams loaded to the extent of the pressure of the water under a head of 110 feet, this pressure is 7,200 lbs. per square foot. The rear chamber wall is 4 feet thick, the required thickness found as follows. The height of this wall is to be 9 feet, therefore we have a beam 9 feet long, supported at both ends loaded uniformly with a load of 7,200 lbs. per square foot. Taking a strip of one foot in width as the basis of our calculations, since the concrete is to be reinforced, the value of S is taken as that of steel, that is, 10,000 lbs. The formula for a beam loaded in this manner is :-

$\frac{SI}{C} = \frac{1}{24} w l^2$, where w is the load per linear foot, $\frac{I}{C} = \frac{bh^2}{6} = \frac{7,200 \times 81}{24 \times 10,000}$, from which $h = 3.8$ feet, and we took it as 4 feet.

For the front wall, which is to be 11 feet high, $h = 5$ feet, calculated as above, the end walls being same as the front, since they are of the same height. Ten iron tie rods, $1\frac{1}{2}$ " diameter, are to be placed in the chamber in addition to the re-inforeement.

The penstocks are made to enter this chamber in such a manner as to eliminate nearly all entrance losses, the curves being gradual, and there being at no place a sudden expansion, the wooden pipe being built into the concrete and the two coupled by a plate steel coupling. The dividing gate openings are two in number and are 8 feet by 8 feet, the contraction of the chamber here is also gradual. Behind the wheel chamber, and extending entirely along its length, is a passage way $5\frac{1}{2}$ by 10 feet excavated from the solid rock giving access to the rear turbine bearings.

The generator room is supported on five piers, under the front wall, the rear wall being a vertical extension of front wheel chamber wall, which latter rest upon solid concrete foundations. These piers are joined by 16 foot arches of re-inforced concrete, 4 feet thick at the center, this thickness being determined from Merriman's discussion of arches. The piers are also joined to the solid foundations by a 28 foot re-inforced concrete span, 4 feet thick at center, so that the generator floor rest upon a series of groined spans or arches.

The draft tubes are partially excavated from rock, and partially built of concrete, their size and shape being such as to eliminate all possible friction losses. The length of these tubes is such that the distance from the center of turbines to mean

water elevation is 17 feet. A longer tube was at first considered, but Mr. Mead and Prof. Switzer cited instances where plants were wrecked by using too long a tube, the water having a back hammer effect when turbines suddenly shut down, caused by the vacuum created in the tube. We took as long a tube as was deemed advisable by hydraulic experts, that is, 17 feet; the lower end of these tubes empty into the tail race at an elevation which provides for their upper edge being always below the water surface, thus insuring draft tube action. The tail races are combined into one large excavation as shown in Plate X., gradually extending from their lowest depth at inner wall to the river bed elevation somewhat beyond the line of piers.

It was necessary to place the transformer room at one end of the power house proper, owing to the narrow space available, and we chose the downstream end as the most suitable, both because it would require much less excavation for foundation, and because it could be more easily reached by the Southern Ry. The natural place for the high tension switch room is above the transformer room, and here we again allowed liberally for wiring space and extra room in the design of the department. The three foot wire space behind the transformer room where the wires coming from the low tension bus go up to the high tension switch room give ample room for this wiring.

The rear wall of the power house is to be of concrete, 2 feet thick, while the three remaining walls, and those of the transformer house are to be built of brick, whose thickness we took as one foot, or four brick, this being the usual thickness of walls of similar structures.

The roof of the wheel chamber is of course to be of the re-inforced concrete, calculated for the pressure to which it is subjected, that of the generator room is of the ordinary type, slate covered, while that of the transformer house is flat and made of gravelled roofing.

Especially was attention given to the detail of windows, they being provided of sufficient number and size large enough to insure pleasing and comfortable lighting.

The upstream end of the power house is protected against flood water damages by a solid concrete wall, the top of which wall encloses a space available for the storage of surplus material.

Provision is made for the installation of a large crane to handle the power house machinery, the railroad tracks are to be run into the north end of the generator room far enough to allow of the crane picking machinery off the cars.

This covers the description of the power house and we will now procede to the question of the cost of construction.

COST OF CONSTRUCTION OF POWER HOUSE.

EXCAVATION:- It has already been stated that the power house is to be located upon a shelf of rock at the bottom of a high cliff. This shelf will have to be excavated for the draft tube openings as shown by Plate X., and there will also be quite a large amount of excavation to be done in the building of the tail races. We measured the amount of the total excavation by drawing the cliff as it now exists and integrating the area taken out in Plate X. with the planimeter. We find it will be necessary to ex-

cavate a total of 3,300 cubic yards of rock @ \$ 1.00 per cubic yard gives\$ 3,300.00.

CONCRETE:- Total amount of concrete used in foundations, flooring, piers, wheel chamber walls and roofing, is 2,050 cubic yards @ \$ 4.50 per cubic yard\$ 9,000.00.

Part of this to be re-inforced as per description.

BRICK:- It will require 192,100 brick for the walls if built four brick thick. According to Gillette this will cost \$15.00 per M for material and labor. Total cost of the finished walls will then be\$ 2,880.00.

POWER HOUSE ROOF:- The roof will be supported by seven Fink trusses, costing approximately \$ 850.00. The roof itself is of slate and contains 4,880 square feet, @ \$ 10.00 per 100 , including the cost of labor, amounts to \$ 480.00. Total cost of all roofing.....\$ 1,330.00.

TRANSFORMER HOUSE ROOF:- This roof to be supported by four 8 " I-Beams @ \$ 22.00 each, cost of roofing and sheeting for same, \$500.00. Total cost of this roof, including the construction is\$ 588.00.

SWITCH ROOM FLOORING:- Area is 1,035 square feet , which gives approximately 1000 board feet, @ \$ 15.00 . Sills for same \$ 7.00. Total.....\$ 21.00.

WINDOWS:- Total area of all windows = 1,444 feet, and @ \$ 1.25 per square foot (Obtained from Mr. R.F.Graf, a local architect.) gives.....\$ 1810.00.

POWER HOUSE EQUIPMENT.

TURBINES:- "Great care and attention should be given to the selection of a turbine intended to operate under any given set of conditions, and these conditions should be carefully determined before the choice of a wheel is made. To determine the best installation of turbines to fullfill these conditions, the value of K_5 in the equation $K_5 = \frac{N^2 P}{H^{5/2}}$, where N is the number of revolutions per minute, P is the power to be developed, and H is the head, should first be determined, this having been done, a turbine should be selected having a value for K_5 not less than the amount determined, and if the turbine is to operate at its maximum efficiency, the value of K_5 for the wheel selected should not greatly exceed the value found by computation. If the computed value of K_5 greatly exceeds the value of K_5 for the various makes of turbines, then the power must be divided among two or more units in order that the conditions may be satisfied. As K_5 is in direct proportion to P, one half, or one third, or any other fraction of K_5 would give the value of K_5 for a wheel having a similiar fractional value of the power P, and will therefore show the type of wheel which must be selected in order that two, three or more will do the work in question."

From advise received from turbine manufacturers, we selected the speed of the machines at 360 R.P.M., the effective average operating head is 111 feet, and the power which we may therefore expect to develope is 12,000 H.P. Substituting these values in the equation for K_5 , we obtained as the value of K_5 , 12,000. Upon examination of a table showing the value of K_5 for

all the important makes of turbines, we found that our value greatly exceeded any of the given ones, and it was necessary that we use two or three units. We found that by using three Sampson Turbines we could most nearly approach our conditions, so of course this was the turbine to use. We received the following letter from the makers of these turbines, also advising us to install three of their turbines:-

"We note that you wish advice as to the number of turbines to be installed in a plant operating under a head varying from 105 to 125 feet , with an average flow of water of 600 second feet, and a maximum discharge of 1200 second feet.

It occurs to us that probably the best arrangement would be for you to install three units each of 3,800 H.P., each of these units would use 400 second feet. Two of these units running at $\frac{3}{4}$ th gate, which is the point of highest efficiency, would use 600 second feet, and the three units running at full gate would use 1200 second feet. This would seem about the best way to divide up this power, and as to the variable head, we would select a speed for you that would be best suited for this variable head conditions, as we understand that it is necessary to have these turbines running at a constant speed under the different heads. This we can take care of for you and would make the turbines to meet the conditions. It seems to us that a speed of 360 R.P.M. would be the most suitable to meet the conditions set forth, therefore we could furnish you these hydraulic units as follows:-

Each of these units, when operating under a net working head of 105 feet would develop 3,800 H.P., using 400 secondfeet

and running at a speed of 360 R.P.M. The gross shipping weight of each of these turbines would be 11,000 lbs., price net f.o.b. cars, Springfield, Ohio, \$ 10,080.00. Suitable governors for controlling the speed of this unit, would be a 45,000 foot-pound, Lombard, oil pressure, quick acting governor. Gross shipping weight, 9,000 lbs. Price net f.o.b. cars, Springfield, Ohio, \$ 2,500.00."

Although only three such units are necessary to handle the power to be developed, we decided to install four units at the outset, using one in case of breakdown as a reserve, and also in order to avail ourselves of secondary power during the winter months. As can be seen from the hydrograph, this secondary power may be obtained for a period of from five to seven months during most of the year covered by the hydrograph. The total cost of four of the above units, including Lombard governors for same, is

\$ 50,320.00.

POWER HOUSE ELECTRICAL EQUIPMENT.

A generator suitable for direct connection to the above turbine unit, generating 3,300 H.P., or, a 2,500 K.V.A., 2300 Volt, 3-phase, 60-cycle, 360 R.P.M. generator, can be purchased from the General Electric Company for \$ 11,400.00. Total weight of 80,000 lbs., this generator having a rated efficiency of 86%. Four of these units to be installed, total.....\$ 45,600.00.

One, 150 H.P., 3-phase, 2300 volt, induction motor, for driving excitors, will cost\$ 1,300.00.

One, 150 H.P., D.C., 720 R.P.M., 250 Volts, generator, direct connected to the above induction motor.....\$ 1,450.00.

One, 175 H.P., Francis Turbine, 720 R.P.M., for driving
exciter.....\$ 400.00.

One, 150 H.P., 720 R.P.M., 250 Volt, D.C. generator, to
be directly connected to the above turbine.....\$1,450.00.

One, 250 H.P., 2300-250 Volt, rotary converter, for
supplying the 250 D.C. power to motors operating hoists, valves,
and for supplying the current for lighting, remote control, etc.,
.....\$3,150.00.

Four, 2,500 K.W., 2300-66,000 volt, 3-phase, delta con-
nected, step up transformers, @ \$ 5,000.00.....\$ 20,000.00.

Eight, time limit relay, overload, oil switches, for
66,000 volts @ \$ 350.00\$ 2,800.00.

Eight, non-automatic, 66,000 volt, oil switches, arrang-
ed for remote control, @ \$ 270.00.....\$ 2,160.00.

Sixteen, 2300 volt, hand operated, oil switches, @
\$ 125.00\$ 1,000.00.

Switch board pannels for the above units, including
bus-bars, L.T.....\$ 1,500.00.

Four electrolytic lightening arresters, 66,000 volts,
@ \$ 50.00.....\$ 200.00.

High Tension Bus bars and cables for connecting H.T.
and L.T. bus bars,\$ 500.00.

Allowing 3 % of cost of machinery for installation, or
cost of installation.....\$ 2,450.00.

Total.....\$ 83,960.00.

HIGH TENSION AND LOW TENSION WIRING.

Plate XI. is a simplified diagram of wiring of the power house, showing only the 2300 and 66,000 volt wiring. This plate shows the method by which either of the two transmission lines can be operated from either set of generators, or the two lines can be operated from a single high tension bus, by closing the transfer, high tension oil switch. Two, 2300 volt, bus bars are provided in duplicate, and any generator can be thrown on either set, also any transformer can be thrown on either set. This makes almost any combination possible, and makes it possible to make repairs on any part of the equipment without danger to the workmen because that particular unit of the equipment may be thrown entirely out of use by merely opening or closing the proper switch, without making it necessary to shut down the entire plant or any other portion of the plant except the unit or units that may need the repairs.

The switchboard itself, is to be located in the generator room as is shown by Plate X. This switchboard being raised to an elevation such that all parts of the generating room are visible to the operator, and the board includes panels controlling high and low tension switches, both in generating room and in switch house, panels controlling exciter sets, rotary converter, and small panels equipped with the necessary wiring for controlling the motors operating gates, hoists, etc., at the dam. It is here on these latter panels that indicator lamps are to be placed

that will show the position of these gates, and etc. In the space immediately below the board, provision is made for placing of the disconnecting generator and disconnecting low tension transformer line switches, there being sixteen of these switches located in the small apartments provided for them.

The low tensions wires are to run from switchboard in conduits laid in the concrete flooring, into the space marked "Wire space and Arrester Chamber." Plate X., where they are led to their respective destinations through conduits. The cross section of switch house in Plate XI., shows the wires coming from the switchboard into the low tension side of the transformer. This cross section is intended to show the disposition of the high tension oil switches and the high tension bus bars. The conductors are led from the high tension side of the transformers to the high tension bus through a time relay and remote controlled oil switch, and are made of heavy copper tubing as is consistent with the best practice of today in handling problems of extreme high voltages. The bus bars are of heavy copper rods swung from the roof of the switch house upon built up insulators, each of these units of insulators, capable of standing 100,000 volts pressure. From the bus bars the copper tubing conductor runs to the time relay and the remote controlled hand operated oil switch through a choke coil, and from there through a specially constructed window of thick plate glass, in the center of which there is a large porcelain tube, and it is through this tube that the copper conductor passes to the outside. This window is arranged in such a manner as to obviate the possibilities

of grounding during a rainstorm. After passing through the window the conductor is received upon a substantial truss on the roof of the house, and it is at this point that the regular line wires are connected to the copper tubing conductor. The connection to the lightning arrester is made just outside of the window as may be seen from Plate XI. It was the object in designing this department to provide plenty of room between all high tension equipment, and we think that there will be little danger of possibilities of sparking or leaking from one unit to another, as we have taken most of our wiring dimensions from descriptions of 66,000 to 100,000 volt switch houses, and have showed a decided preference in selecting the dimensions as cited for the latter, thus providing extra guarding against mishap due to failure of insulation.

RELIEF VALVES.

Since it is impossible for us to construct a stand-pipe to absorb the water hammer owing to the undue height of such a pipe, and since a surge tank is impracticable on a high head installation, it was necessary that we take care of the water hammer by installing a set of relief valves. The largest relief valves manufactured by the Lombard Governor Co., is a 23 inch valve and this valve will discharge 116 second feet under a head of 100 feet, so that in order to properly take care of the hammer effect, we must provide for $1200 \div 116 = 11$ valves. These valves cost \$ 300.00 each, approximately, and we will in-

stall 12 of them, which will amount to\$ 3,600.00.

OPERATION OF PENSTOCK GATES AND DIVIDING GATES.

Since the water pressure on the penstock gates is 3,700 lbs. per square foot, it will take a positive force to close the gates as well as to open them, unless the operation of closing be as follows: suppose that we wish to close gates in penstock No. 1. Penstock No. 1 is full of water, either at static pressure, or slightly less than static pressure if water is flowing through it. We must first close down both turbines in the division of wheel chamber into which penstock No. 1 enters, and then close the dividing gate. This latter can be easily done because there is water in both divisions of the chamber. As soon as the dividing gate is closed, the penstock gate of No. 1 is closed, and the weight of the gate is sufficient to do the closing, as the penstock itself and the wheel chamber are near enough water tight to hold the water for the time required in closing the gate. To open the penstock gate, we merely start the motor in the required direction, whether the penstock is full or empty, as the size of motor is designed to raise the gate when penstock is entirely empty. The gates when raised are to be supported by an arrangement which can be projected into the space between gate guides, so that the weight of gate will not be thrown upon the hoisting chains and hoisting machinery, except during time of raising or lowering. These chains are of course, to be large enough to

stand the load due to raising the gates, plus ample factor of safety

To operate the dividing gates in wheel chamber, it is only necessary to open the valve H (See Plate X.) for opening the gate, and valve J for closing same, provided that both sides of wheel chamber are full of water, but when division No. 1 is empty, division No. 2 being full, it will be necessary to first open a 24" by-pass valve before gate can be opened. This by-pass valve is shown on Plate X. and will cost.....\$ 600.00.

The D.C. motors used in operating, valves, gates and racks at the dam are to be provided with a device for indicating when valve, gate, or rack is in closed or open position, this device operating a signal located on low tension switch board at power house.

Plate XII. is a series of curves usually made in connection with the design of a dam for power purposes, showing the total horse power of the river for any given discharge, and with a given head. It also shows curves giving the variation of head with all turbines running, height of crest of dam above tailwater, and gives scales of the theoretic and actual horse power of the turbines.

As has already been stated, the electrical output of the plant, with a generator efficiency of 86 %, is 10,000 H.P.

AUXILIARY POWER PLANT.

Referring to pages 7 and 8, it is seen that we will need auxiliary power equivalent to 400 second feet discharge as a maximum value, the power equivalent to this is 5,000 T.H.P., and assuming an efficiency of 72 % we will need a plant capable of

supplying 3,600 H.P., but we will take 4,000 H.P. as the desired size of plant, for if there should be a demand for auxiliary power beyond this amount, we can easily run the plant for a short while at 50 % overload.

The location of this plant is to be at Johnson City, for there is already a plant in operation at this point having a maximum capacity of 1200 K.W. output, and by slight changes the buildings of this plant could be altered sufficient to answer purpose. We would need to purchase this plant complete and add one 1800 K.W. turbo-generator, build a transformer house, installing two 66,000 volt, of 1000 K.W. capacity, a complete new switchboard, and add four boilers of 500 rated H.P., an enlargement of boiler room being also necessary. The estimated cost of this plant, including the price paid for the present installation, the required new units and changes, and of additions to building, amounting to approximately.....\$ 115,000.00.

TRANSMISSION LINES.

The question of transmission lines is a very important one indeed and should be the object of much careful thought and attention. It is becoming a common practice to erect steel towers for transmission lines, even though the line is a very long one, and the cost of erecting of such a line is not so excessive as would at first seem, because of the much greater distance apart at which steel towers may be placed. We quote below a few prices showing approximately the cost per mile of erecting

a transmission line suitable for a 66,000 volt circuit, both for wooden poles and steel towers. (Quoted from Thesis of W.W.Carson, who obtained prices from the manufacturer.)

WOODEN POLE LINE.

53 wooden poles-100 feet apart @ \$ 2.00.....	\$ 106.00.
3x53x4 insulators @ \$ 2.50 per set (4 sections= 1 set.)	397.00.
cross arms-metallic, and iron pins @ \$ 1.25 per pole..	\$ 66.00.
erection per pole \$ 1.25, total.....	\$ 66.00.
right of way @ \$ 3.00 per pole.....	\$ 159.00.
number 0, 7 strand B. & S. Wire @ 20 cts per lb-3 wires	990.00.
stretching wire.....	<u>\$ 20.00.</u>
Total cost per mile.....	\$ 1804.00.

STEEL TOWER LINE .

9 steel towers, 45 ft. high, with cross arms and pins complete, @ \$ 60.00.....	\$ 540.00.
3x9x4 insulators (4 making a set) @ \$ 2.50 per set..	\$ 69.00.
assembling and erection \$ 7.00 each.....	\$ 63.00.
right of way @ \$ 4.50 per tower.....	\$ 40.00.
No. 0 gage, 7 strand B&S wire @ 20 cts lb (3 wires)...	\$ 990.00.
stretching wire	<u>\$ 12.00.</u>
Total cost per mile.....	\$ 1814.00.

The above insulators are of the built up type, each insulating unit made of four insulators, hanging one below the other,

each section capable of standing 20,000 volts, so that the complete unit will stand 80,000 volts. Since there is only \$ 10.00 difference per mile in the cost of the two systems of transmission, and since the steel tower system is by all means the most desirable , we will use the steel tower line as the basis of our figures.

The transmission lines of a great many power companies are made in duplicate, yet we can cite instances where lines equally as long as ours and carrying as much power are not duplicated. We therefore decided to count upon a single line from the site of our power house to Knoxville, that is a single line of steel towers , but we propose to place another metallic arm upon the towers below the first arm, and to carry a second line upon these arms as is done in Ontario Power Co.'s transmission system. This second line will be of No. 2 B&S gage wire, the three wires of the circuit costing \$ 650.00 per mile. The cost of arms and pins for same being \$ 66.00 per mile, and the cost of insulators will be \$ 69.00 per mile, so that the cost per mile of the second line is \$ 785.00.

The nearest route from site of power house to Knoxville is the railroad line, that is, to go first from Embreeville to Johnson City, and direct from Johnson City to Knoxville, a total distance of 126 miles. The secondary line is to be discontinued at Morristown, so that there will be 84 miles of single tower, three No. 0, and three No. 2, line, and 42 miles of single tower, three wire No. 0.

84 miles double line @ \$ 2,599.00 per mile.....	\$ 218,100.00.
42 miles of single line @ \$ 1814.00 per mile.....	\$ 76,000.00.
Total cost of all lines.....	\$ 294,100.00.

SUB-STATIONS.

We will of course need a sub-station wherever we intend to supply power. We have not counted on running a 66,000 volt line by way of Erwin, so we can supply this town with current at 2300 volts and stepped down to 110 volts at the Erwin end of the line. We need then five sub-stations located as follows:-

Johnson City	Jefferson City
Greenville	Knoxville
Morristown	

The cost of equipping these stations will vary almost directly as the power to be delivered to the towns, and will be approximately as follows:-

Johnson City.....	\$ 1,900.00.
Greenville.....	\$ 1,200.00.
Morristown.....	\$ 1,500.00.
Jefferson City.....	\$ 800.00.
Knoxville.....	\$ 9,800.00.

Total cost of Sub-stations. **\$15,200.00.**

This cost covers all necessary equipment at the various sub-stations such as lightening arresters, switches, etc.

COST OF ENGINEERING.

Mr. Gillette says that the usual cost of engineering for any work ranges in value from 2 % to 5 % of the construction, we will take 2 %, as we have figured high in all previous cases.

At 2 % the cost for engineering and superintending is...\$ 7000.00.

TOTAL COST .

The probable error in calculation of total cost is probably \$ 20,000.00 to \$ 25,000.00, assuming it to be the latter figure, this will mean an error of about 1.7 % which is accurate enough for a preliminary investigation of the subject at hand. By total cost we mean the cost of everything covered in the preceding pages, and this amounts to.....\$1,211,266.00.

We are to develop 10,500 electrical H.P., so that the total cost of the completed plant, lines, sub-stations, etc., is, per..actual.electrical.H.P.....\$ 115.00.

This cost per H.P. does not include any reference to secondary power at all, but is based upon the continuous power which we will develop with three units, and secondary power, which we might be able to develop and sell to the extent of 3,300 H.P. will of course reduce the initial cost per H.P. if figured upon this basis. The cost per H.P. when thus figured being only \$ 87.00.

TABULATED COSTS.

Raising & protecting CC&O Tracks.....	\$ 5370.00
Land flooded.....	\$ 12000.00
Cost of Dam.....	\$252000.00
Sluiceway equipment.....	\$ 3572.00
Gates, racks and equipment.....	\$ 8075.00
Penstock Line.....	\$334000.00
Penstock drain valves.....	\$ 1020.00
Penstock right of way, clearing and excavation.....	\$ 6520.00
Construction of power house.....	\$ 18929.00
Hydraulic equipment at power house	\$ 50320.00
Electrical equipment at power house.....	\$ 83960.00
Relief valves.....	\$ 3600.00
Auxiliary plant.....	\$115000.00
By pass valves.....	\$ 600.00
Transmission lines.....	\$294100.00
Sub-stations.....	\$ 15200.00
Engineering.....	\$ 7000.00
Total	\$ 1,211,266.00

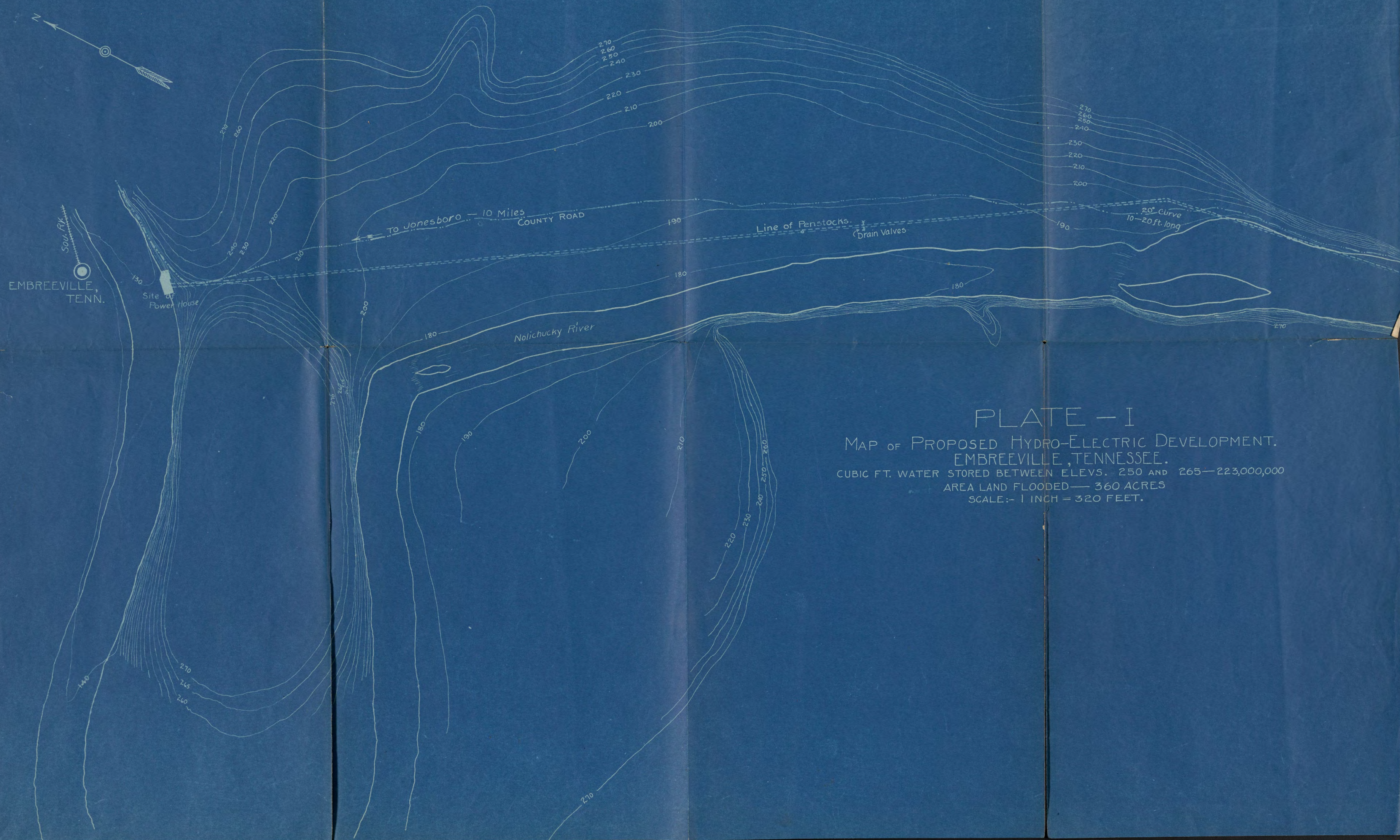


PLATE - I

MAP OF PROPOSED HYDRO-ELECTRIC DEVELOPMENT.
EMBREEVILLE, TENNESSEE.
CUBIC FT. WATER STORED BETWEEN ELEVS. 250 AND 265 - 223,000,000
AREA LAND FLOODED - 360 ACRES
SCALE - 1 INCH = 320 FEET.

11500
11000
10500
10000
9500
9000
8500
8000
7500
7000
6500
6000
5500
5000
4500
4000
3500
3000
2500
2000
1500
1000
500
0

RIVER DISCHARGE-SEC. FT.

HYDROGRAPH OF NOLICHUCKY RIVER.

AT

EMBREEVILLE, TENN.

1903

1904

PLATE-III



GRAPH OF NOLICHUCKY RIVER.
AT
EMBREEVILLE, TENN.
PLATE-III

1904

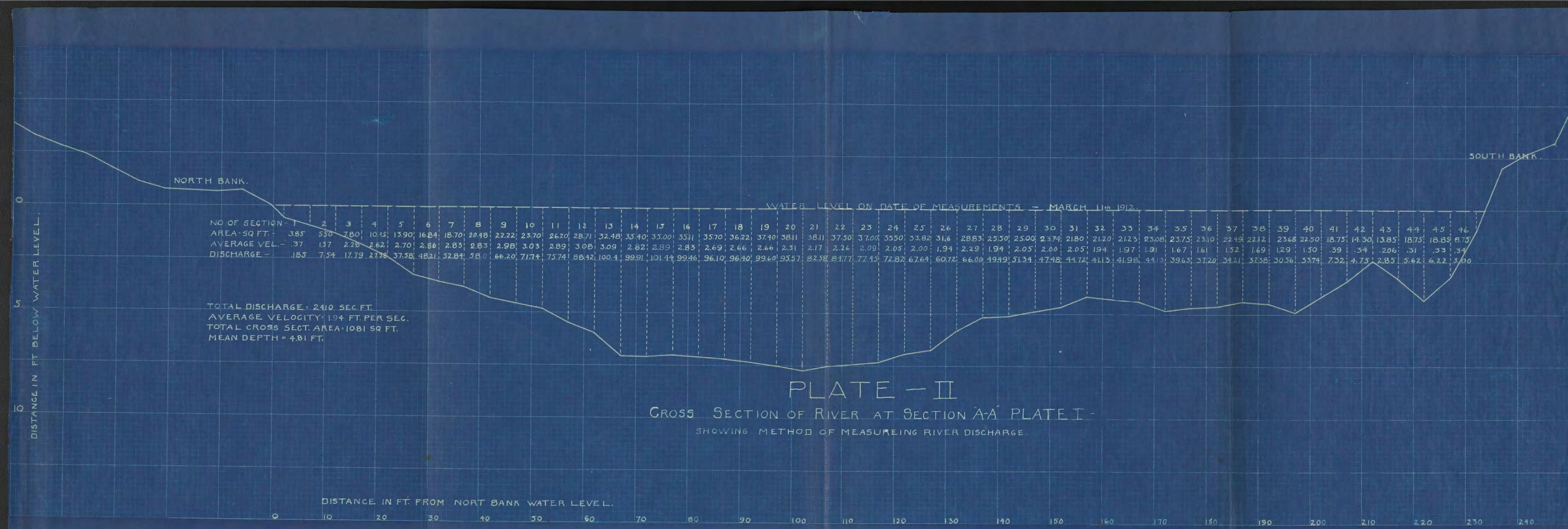


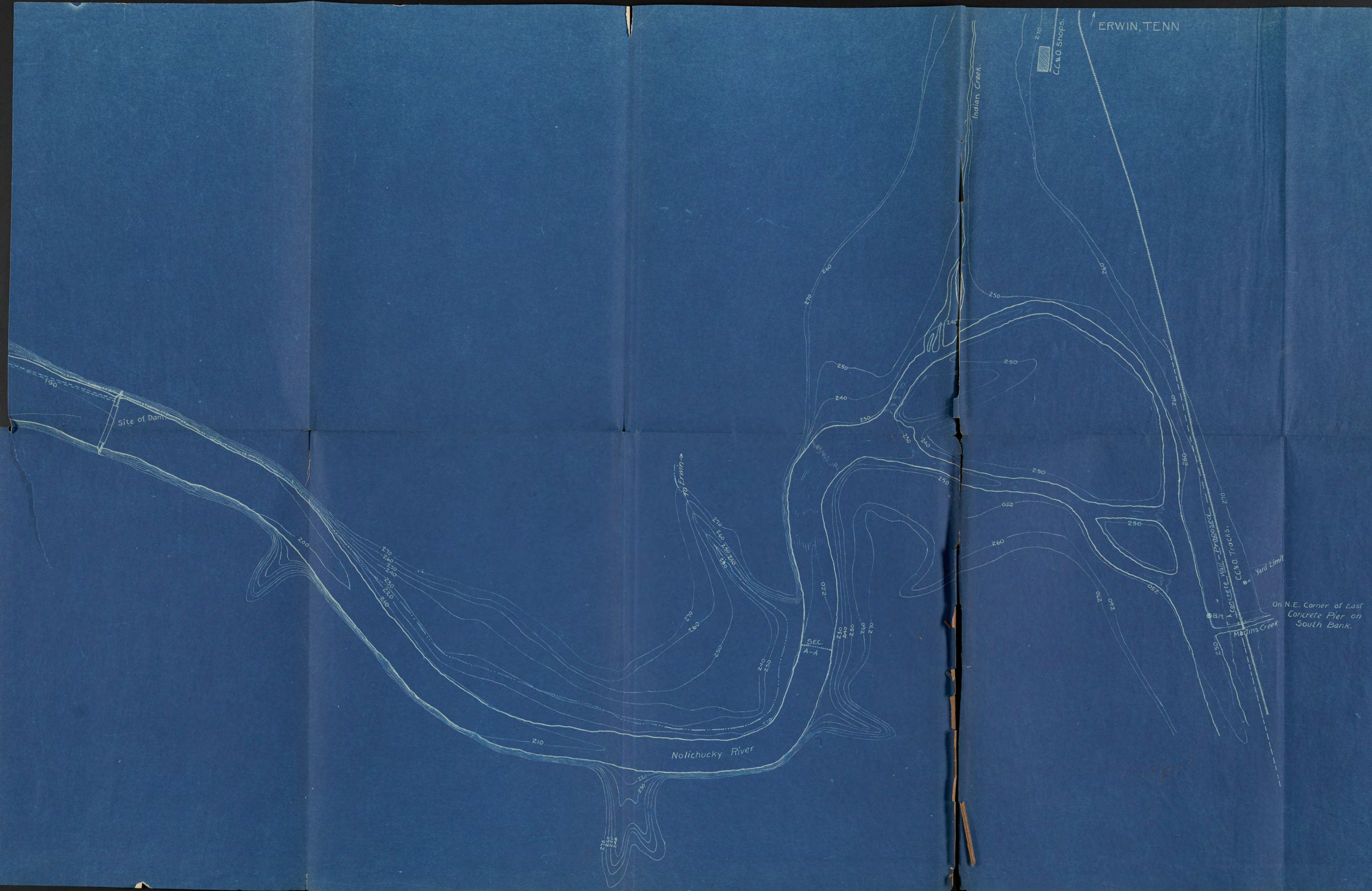
1905



1905







ERWIN, TENN

CC&O Shops

Indian Creek

Site of Dam

Nolichucky River

SEC. A-A

Concrete Wall - Proposed

CC&O Tracks

Yard Limit

On N.E. Corner of East Concrete Pier on South Bank

Martins Creek

1932

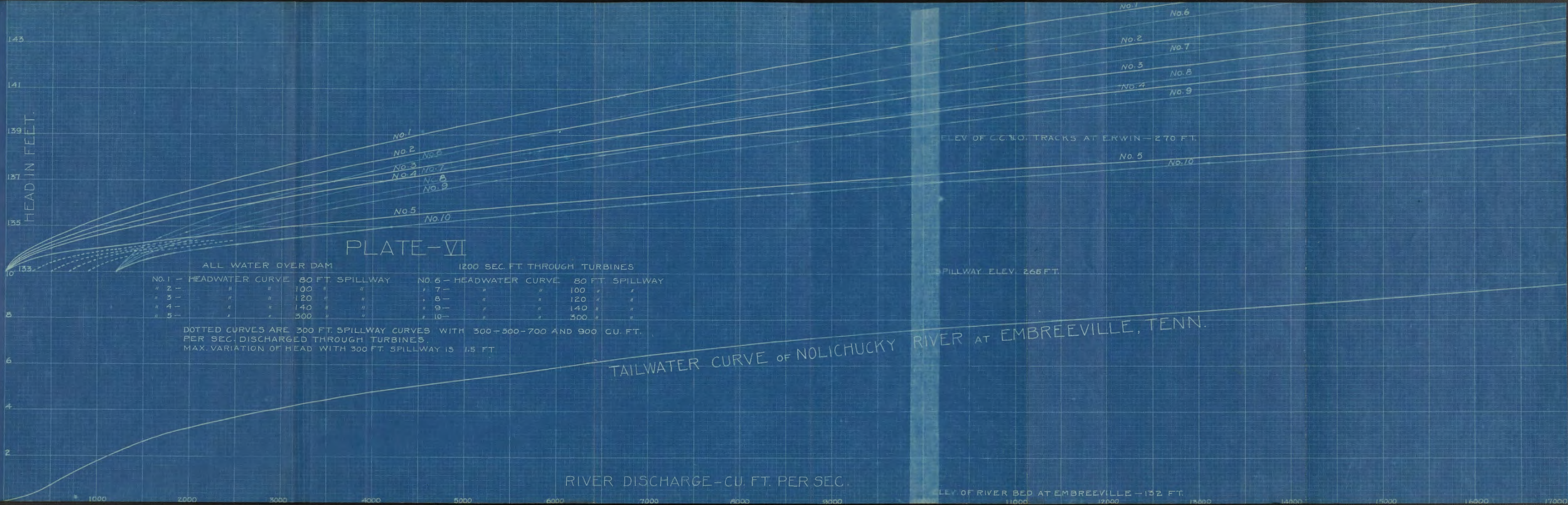


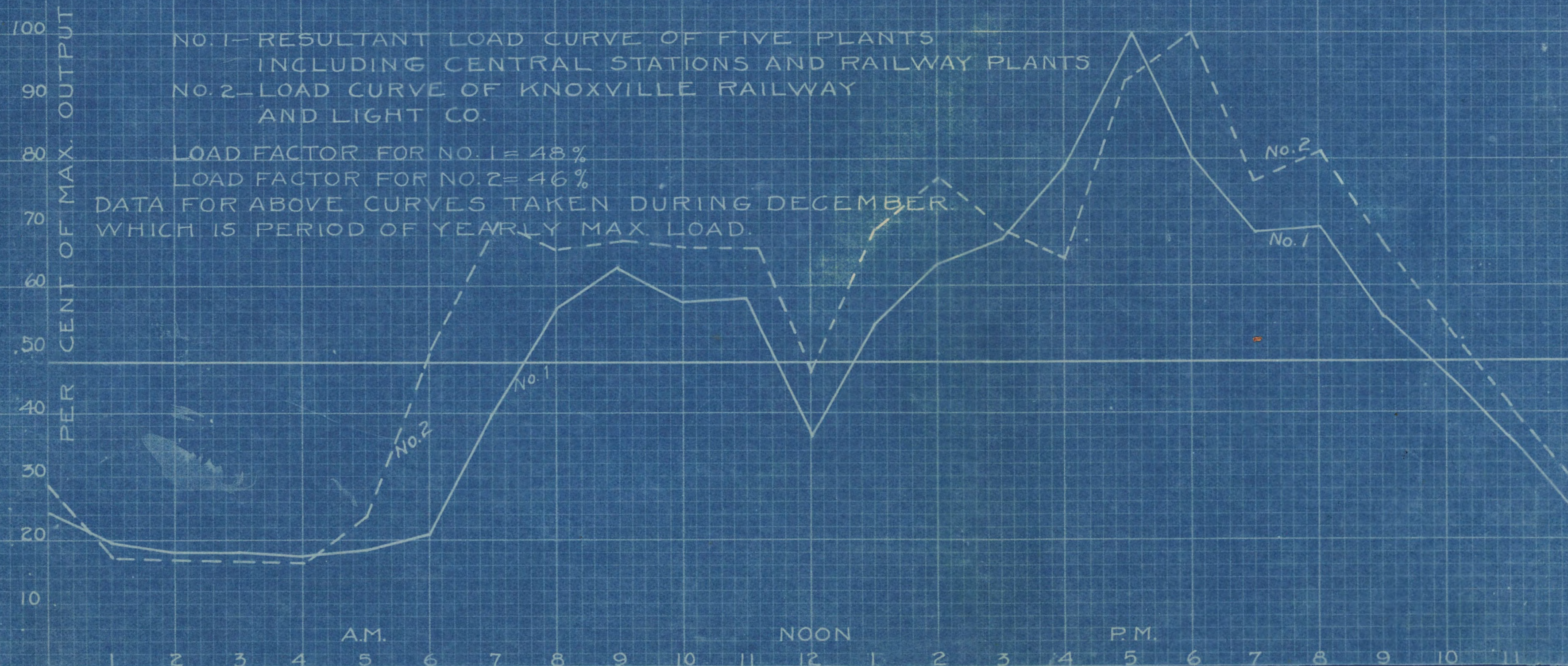
PLATE - V

NO. 1 - RESULTANT LOAD CURVE OF FIVE PLANTS
INCLUDING CENTRAL STATIONS AND RAILWAY PLANTS
NO. 2 - LOAD CURVE OF KNOXVILLE RAILWAY
AND LIGHT CO.

LOAD FACTOR FOR NO. 1 = 48%

LOAD FACTOR FOR NO. 2 = 46%

DATA FOR ABOVE CURVES TAKEN DURING DECEMBER
WHICH IS PERIOD OF YEARLY MAX. LOAD.



ELEVATION 265.

CREST OF SPILLWAY.

PLATE-IV
CROSS SECTION
OF
RIVER CHANNEL
AT
SITE OF DAM

LOOKING SOUTH

HORIZONTAL SCALE

1" = 25'

VERTICAL SCALE

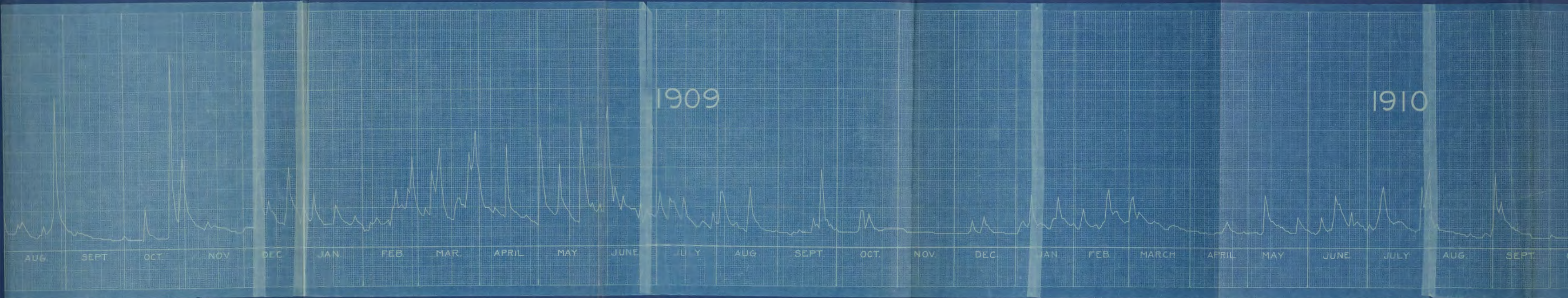
1" = 10'

COUNTY ROAD

ELEVATION 194.83.

WATER LEVEL

LIMESTONE FORMATION



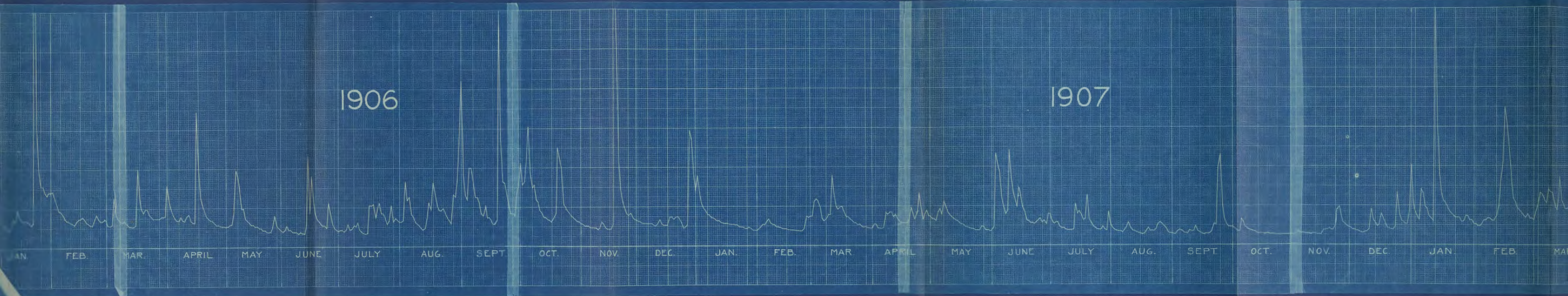
1909

1910



1906

1907

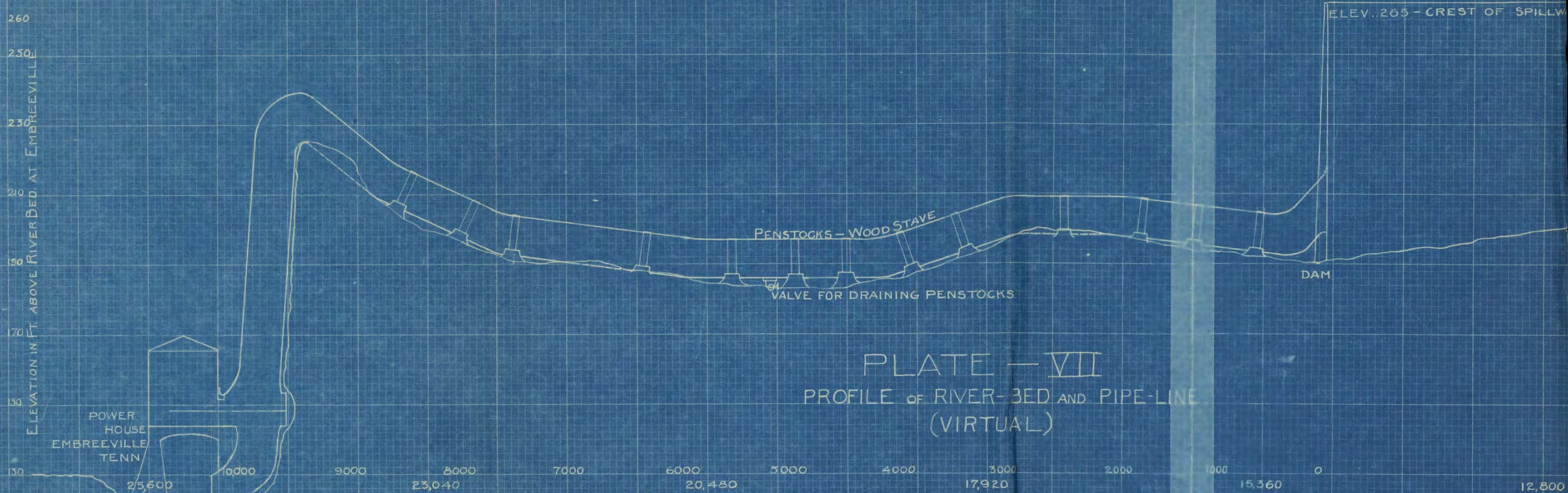


1906



1907

1908



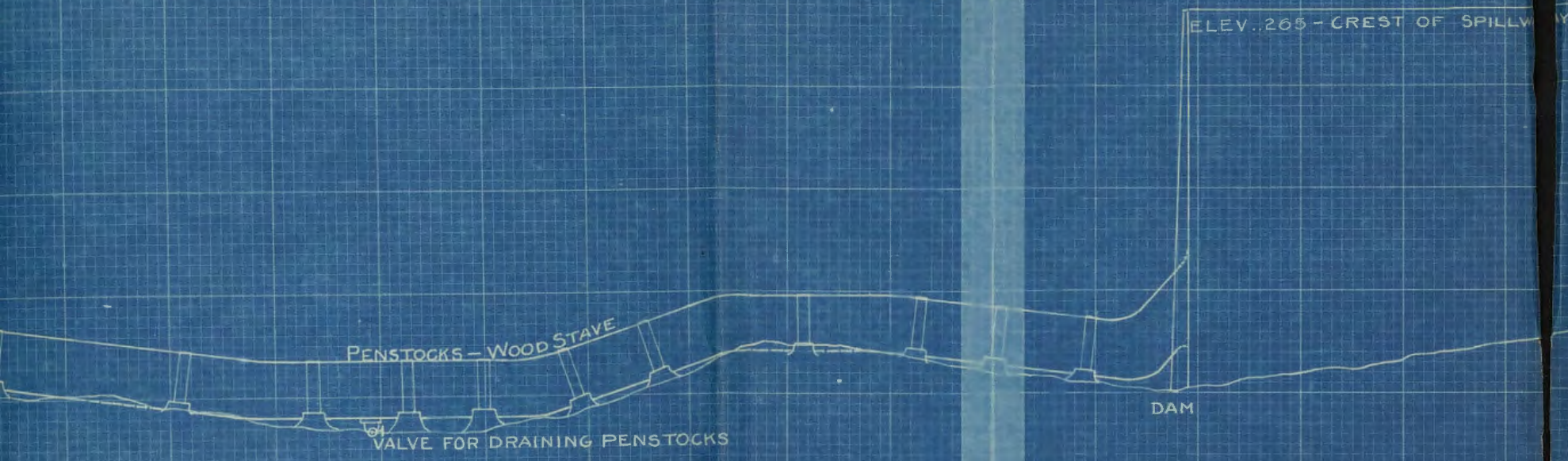
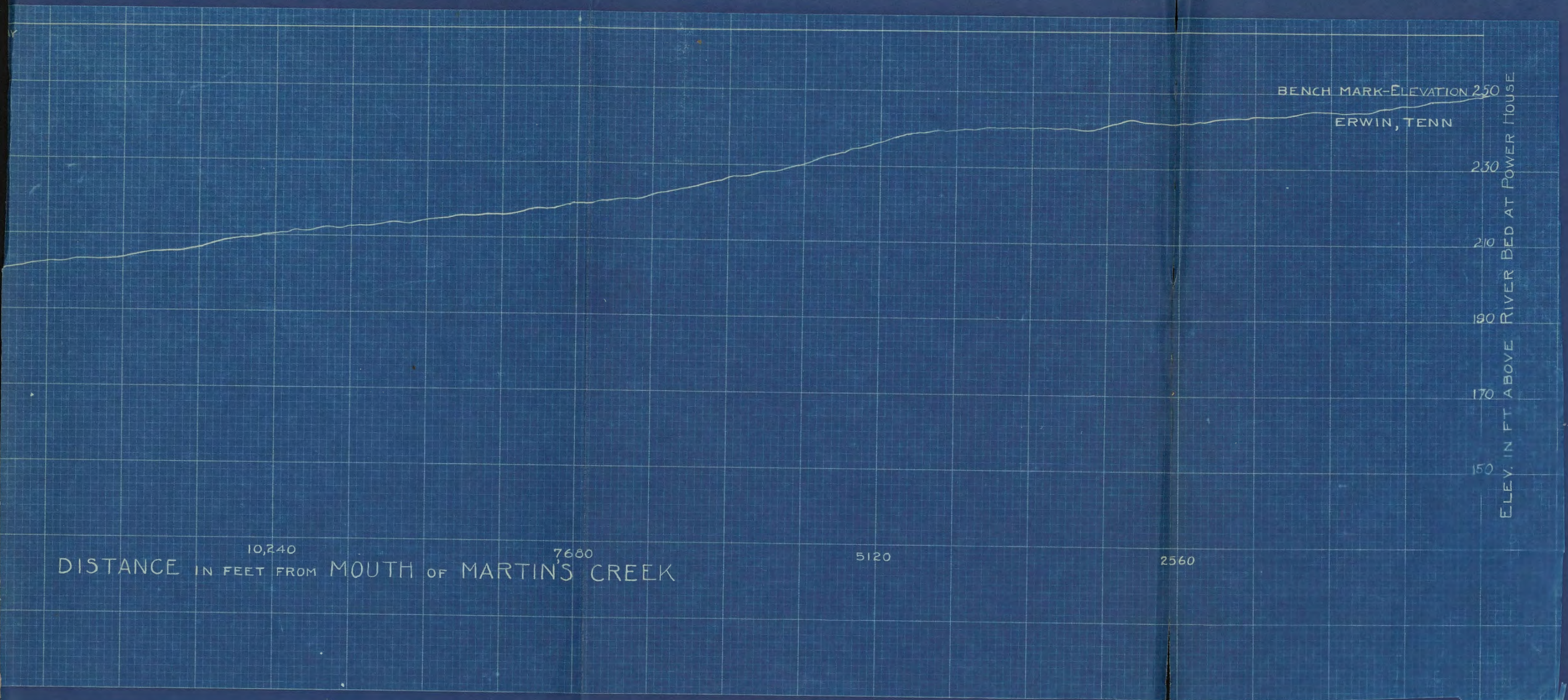


PLATE - VII
 PROFILE OF RIVER-BED AND PIPE-LINE
 (VIRTUAL)

7000 6000 5000 4000 3000 2000 1000 0
 20,480 17,920 15,360 12,800

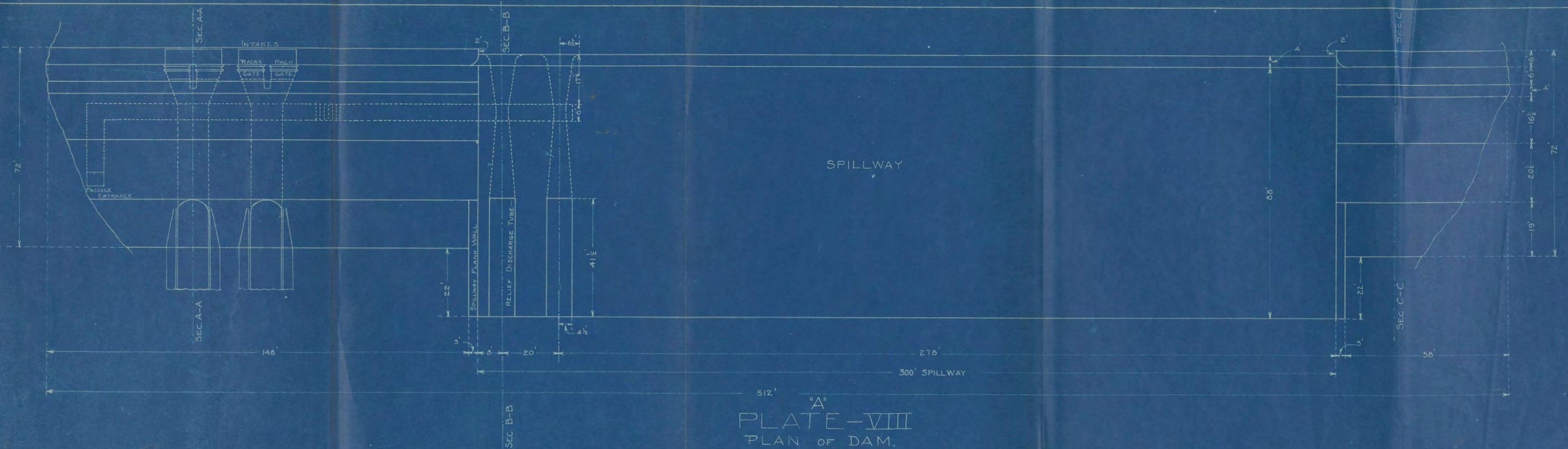
10,240 7,680 5,120
 DISTANCE IN FEET FROM MOUTH OF MARTIN'S CREEK



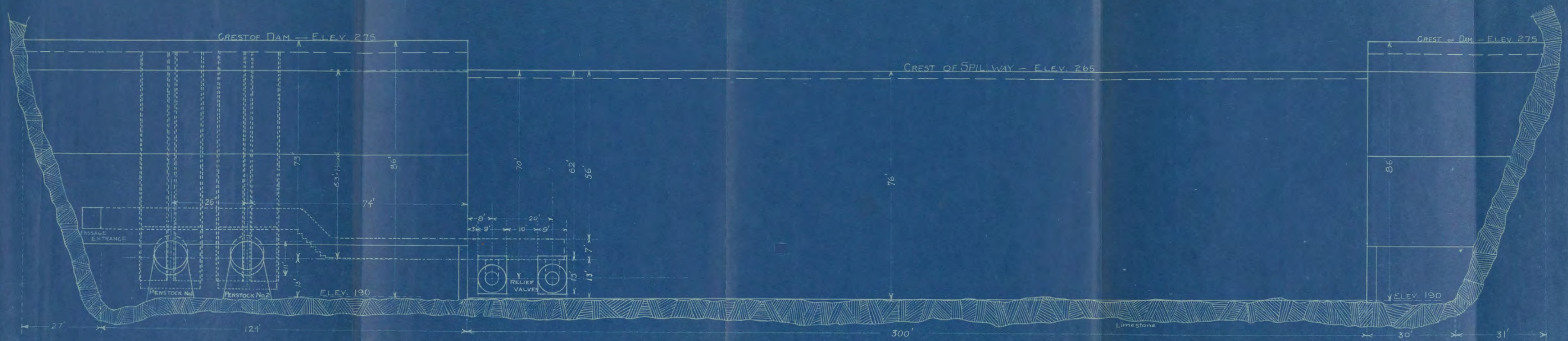
10,240 7,680 5,120 2,560
DISTANCE IN FEET FROM MOUTH OF MARTIN'S CREEK

BENCH MARK-ELEVATION 250
ERWIN, TENN

ELEV. IN FT. ABOVE RIVER BED AT POWER HOUSE
250
230
210
190
170
150



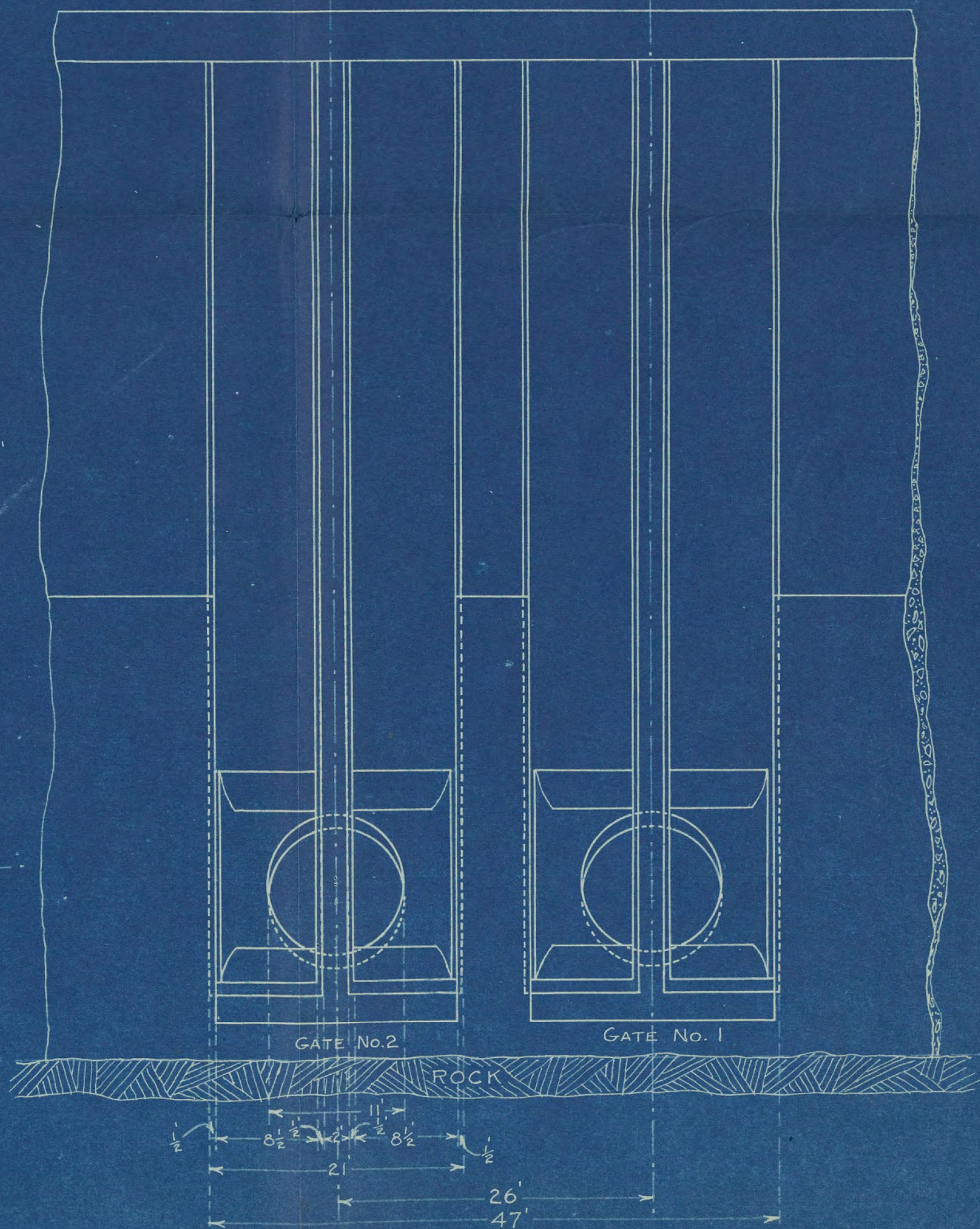
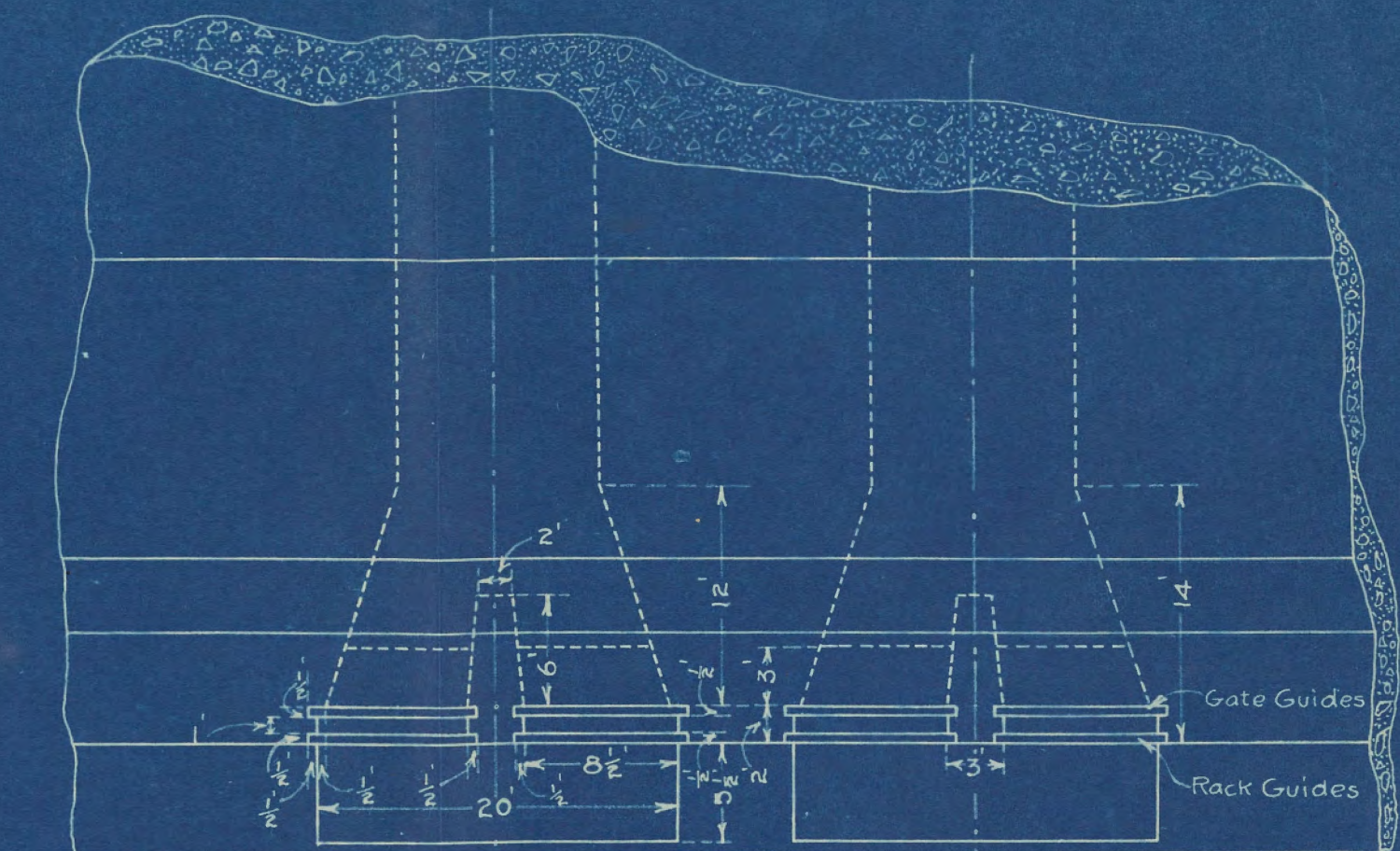
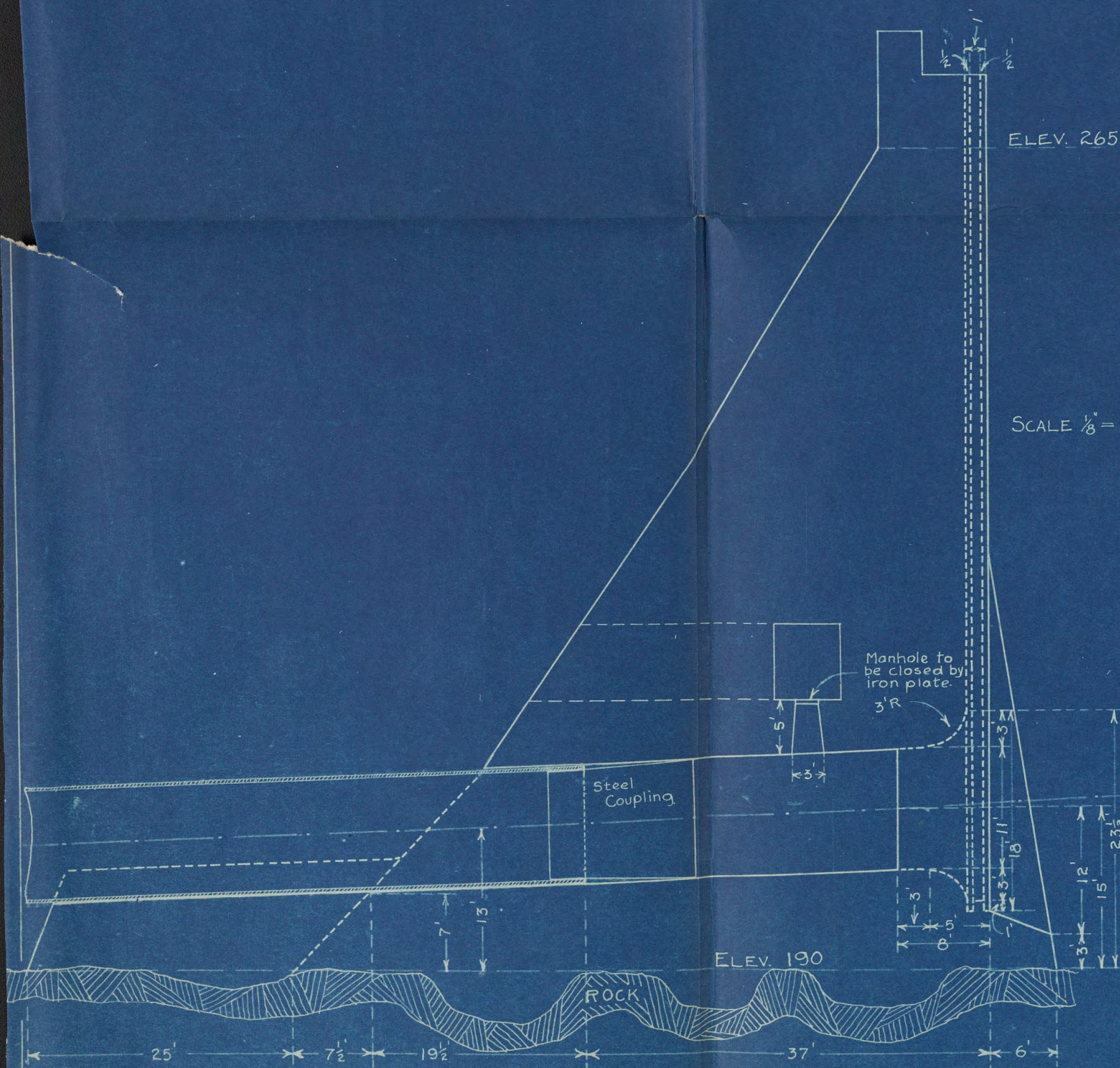
"A"
 PLATE-VIII
 PLAN OF DAM.
 SCALE:- 1/16" = 1'



PLATE—VIII
 ELEVATION OF DAM
 SCALE $\frac{1}{16} = 1'$
 "B"

PLATE-IX

Detail of Penstock Intakes, showing arrangement for raising
Gates and Trash Racks—Detail of Gates and Racks and Guides.
Gate Hoisting Mechanism Removed.
Scale $\frac{1}{8}" = 1'$



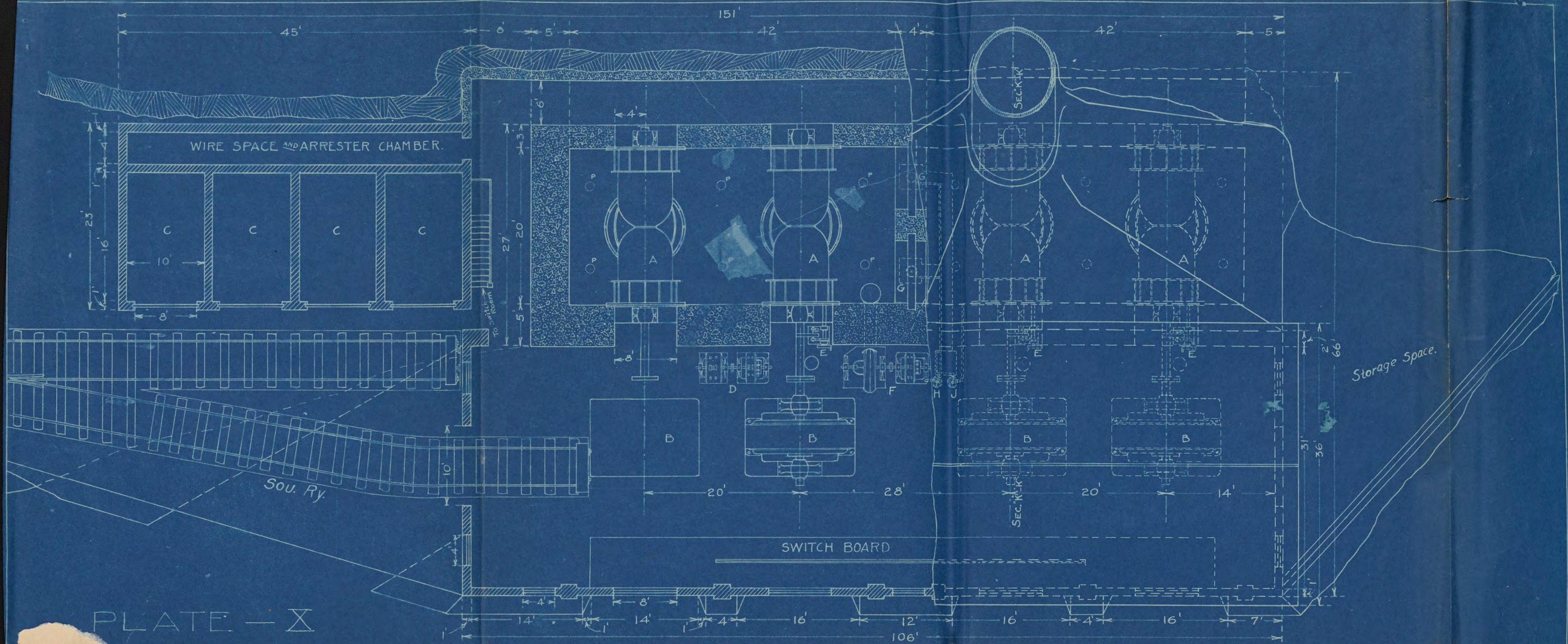
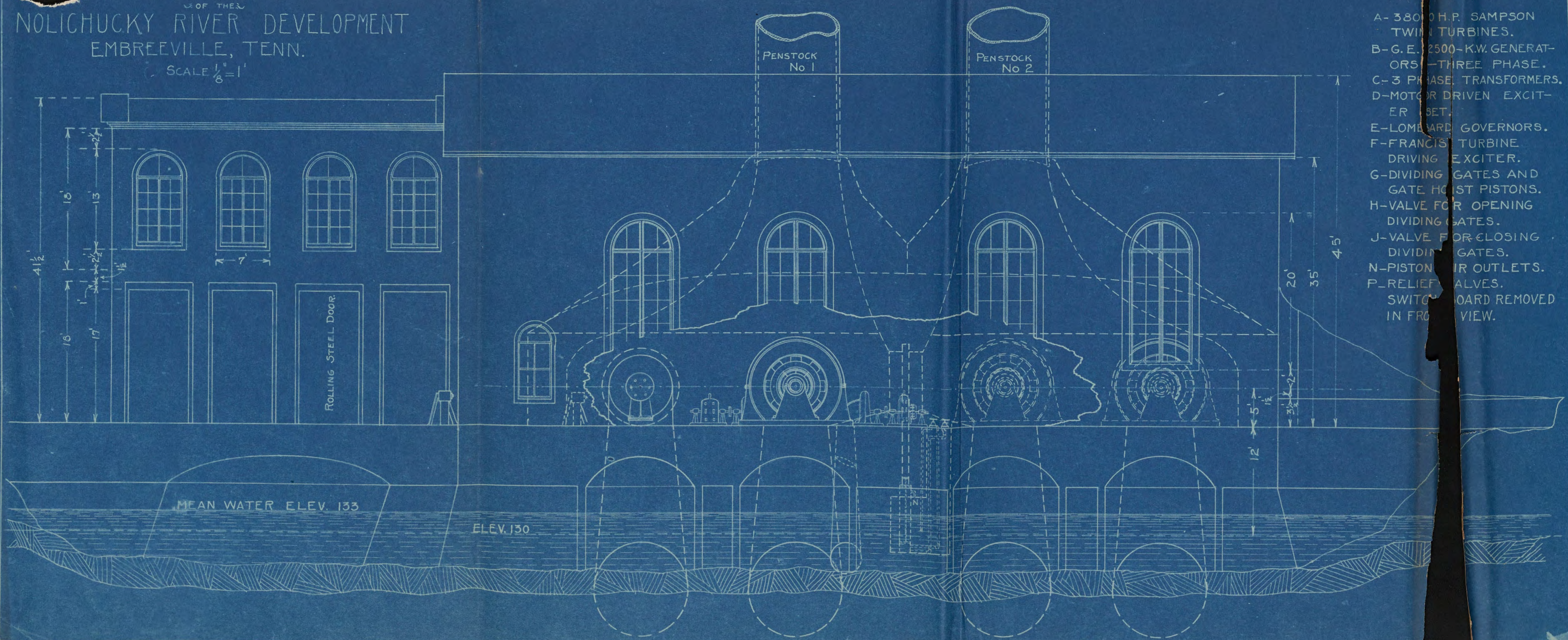


PLATE - X

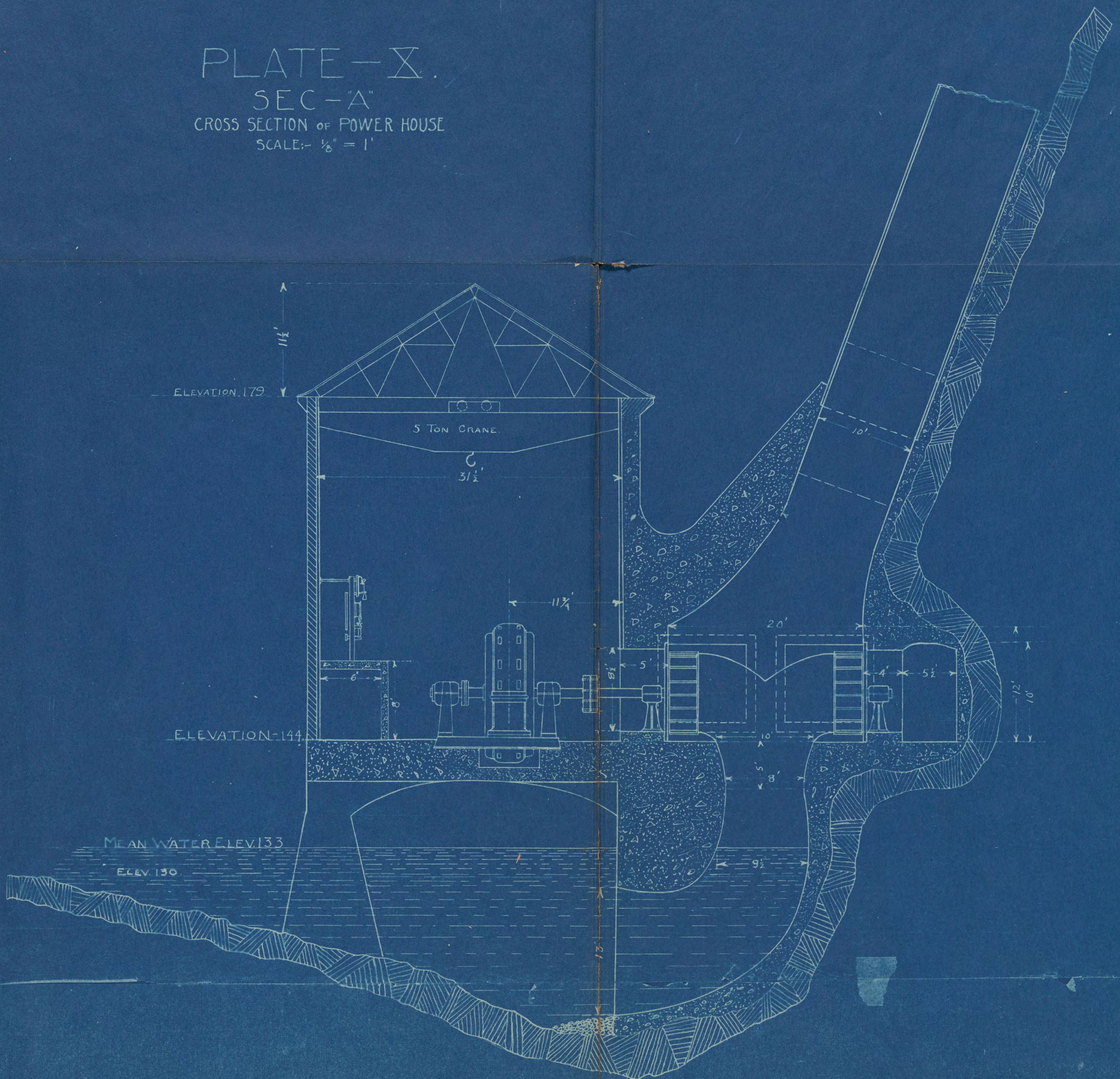
AND ELEVATION OF PROPOSED
POWER HOUSE
OF THE
NOLICHUCKY RIVER DEVELOPMENT
EMBREEVILLE, TENN.

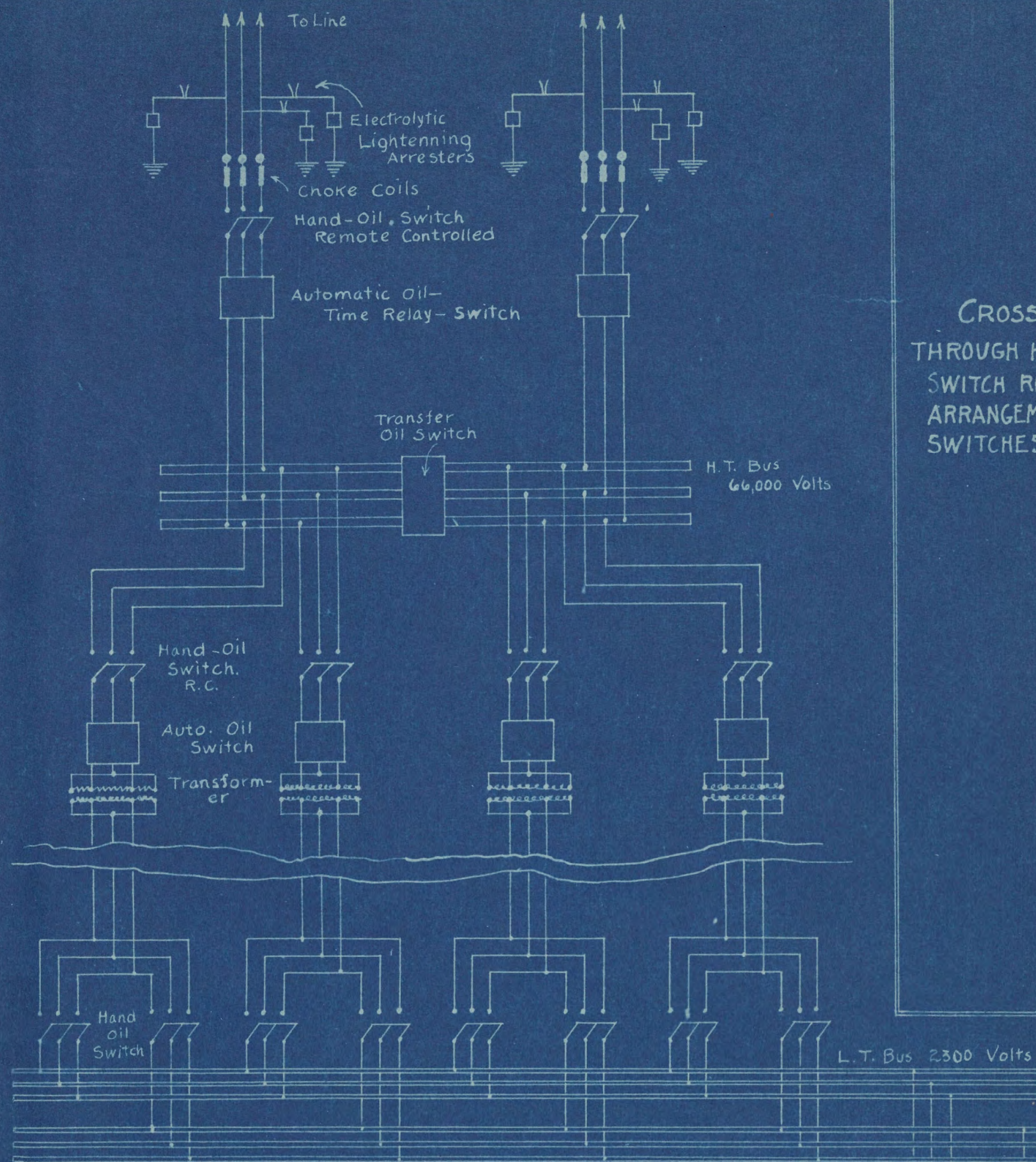
SCALE $\frac{1}{8}'' = 1'$



- A-3800 H.P. SAMPSON TWIN TURBINES.
- B-G.E. 2500-K.W. GENERATORS—THREE PHASE.
- C-3 PHASE TRANSFORMERS.
- D-MOTOR DRIVEN EXCITER SET.
- E-LOMBARD GOVERNORS.
- F-FRANCIS TURBINE DRIVING EXCITER.
- G-DIVIDING GATES AND GATE HOIST PISTONS.
- H-VALVE FOR OPENING DIVIDING GATES.
- J-VALVE FOR CLOSING DIVIDING GATES.
- N-PISTON AIR OUTLETS.
- P-RELIEF VALVES.
- SWITCH BOARD REMOVED IN FRONT VIEW.

PLATE-X.
 SEC-A"
 CROSS SECTION OF POWER HOUSE
 SCALE:- $\frac{1}{8}" = 1'$





CROSS SECTION
THROUGH HIGH TENSION
SWITCH ROOM, SHOWING
ARRANGEMENT OF THE OIL
SWITCHES, H.T. BUS, ETC.

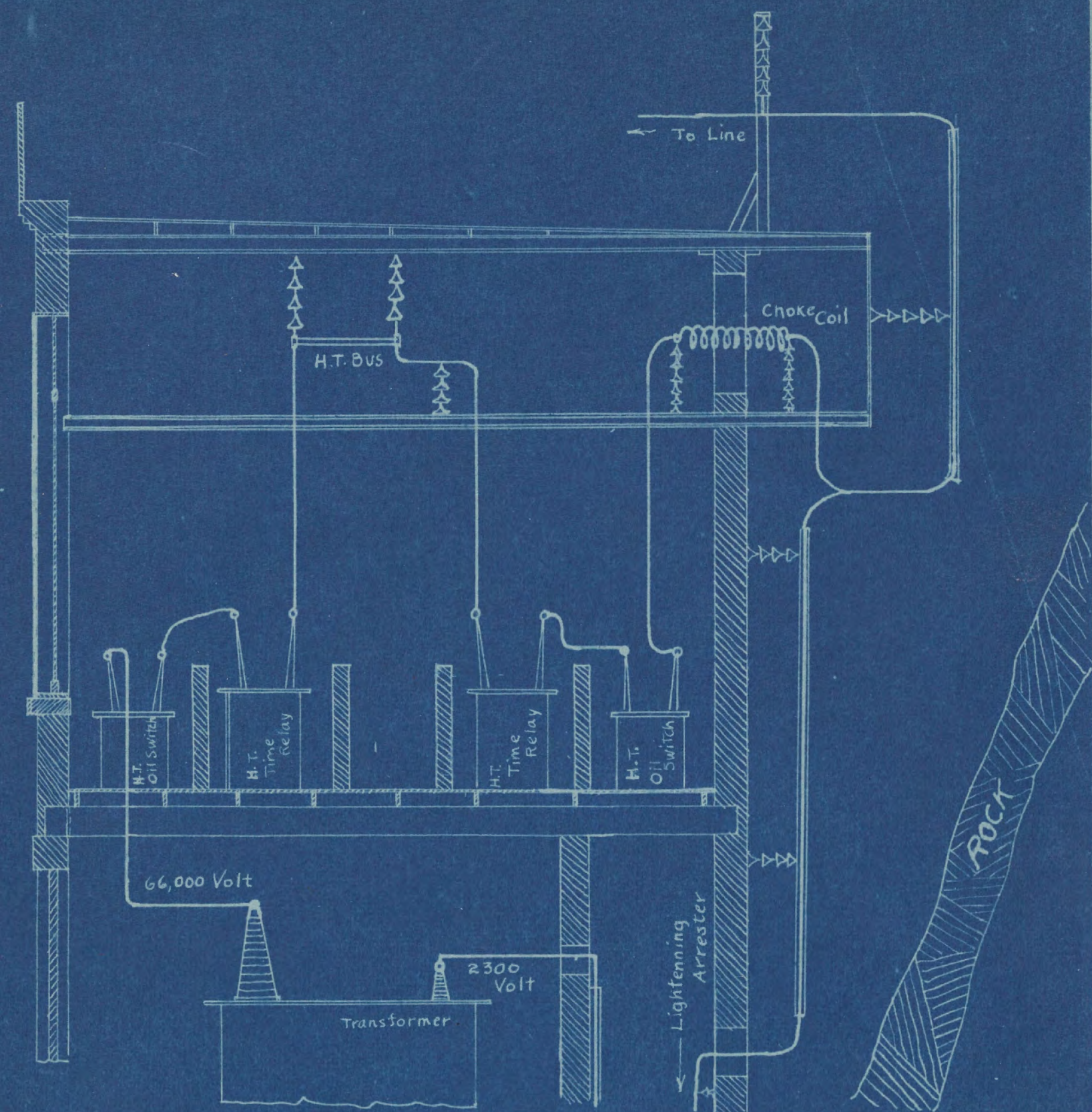
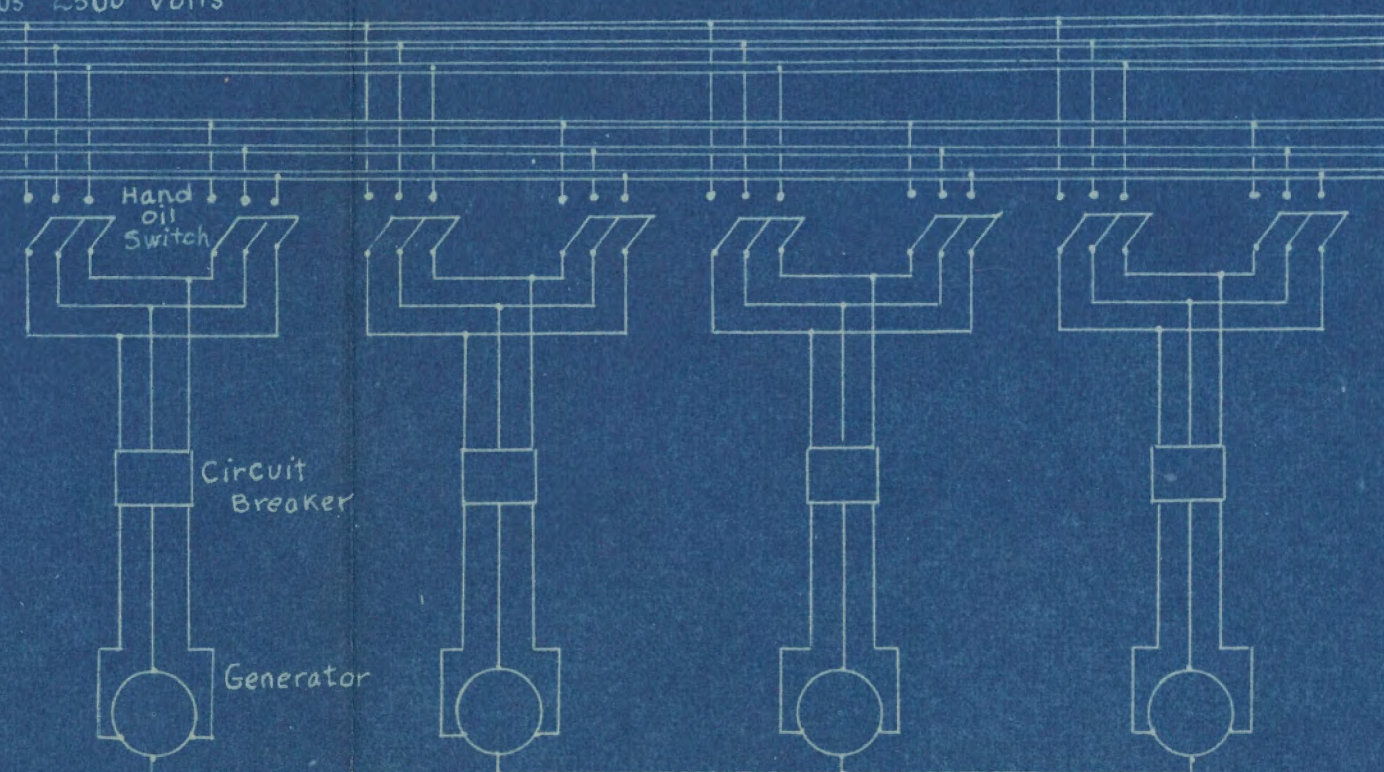


PLATE XI

HIGH AND LOW TENSION WIRING DIAGRAM
SHOWING BOTH SETS OF BUS BARS, BUT OMITTING
EXCITER BUS BARS.



RIVER DISCHARGE - SEC. FT.

PLATE - XII
HORSE POWER CURVES AND SCALES

70000 H.P.
65000 H.P.
60000 H.P.
55000 H.P.
50000 H.P.
45000 H.P.
40000 H.P.
35000 H.P.
30000 H.P.
25000 H.P.
20000 H.P.
15000 H.P.
10000 H.P.
5000 H.P.

-DISCHARGE 3-36" TURBINES-

Above Tailwater
Height of Crest of Dam

300' Spillway
Head - No Turbines Discharging -

Head - No Turbines Discharging - 140' Spillway

ELEVATION IN FT. ABOVE RIVER BED AT EMBREEVILLE

102 104 106 108 110 112 114 116 118 120 122 124 126 128 130 132 134 136 138 140 142 144 146 148 150

HORSE POWER OF RIVER SCALE.



THEORETIC HORSE POWER OF PLANT.



ACTUAL HORSE POWER OF PLANT. - TURBINES

