HYDRO-ELECTRIC DEVELOPMENT
AT
FRENCH'S MILLS, NEW YORK

BY
A. C. CRAMER
J. T. LUCAS
J. A. WOOD

ARMOUR INSTITUTE OF TECHNOLOGY

1913
Cramer, August C.
Proposed hydro-electric development at French's
PROPOSED HYDRO-ELECTRIC DEVELOPMENT

AT

FRENCH'S MILLS, N. Y.

A Thesis
Presented by
August C. Cramer
John Wood
John T. Lucas
to the
President and Faculty of
Armour Institute of Technology
For the Degree of
Bachelor of Science in Civil Engineering
Having Completed the Prescribed Course of Study
In Civil Engineering

1913

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Photograph No. 1

Showing Location of Dam Site and Horizontal Rock Strata
INTRODUCTORY

In submitting this thesis for the approval of the Faculty of the Civil Engineering Department of the Armour Institute of Technology, we have endeavored to present a complete and intelligent design of a modern hydro-electric plant as proposed for the installation at French's Mills, N. Y. It has not been the intention of the authors to treat this subject from an economical standpoint, although economy in design was sought after, nor to prepare a complete set of working drawings, inasmuch as lack of time and available data did not warrant such a deep research.

It has been our intention, however, to work out a preliminary design with the data at hand, such as might serve as an invaluable aid in promoting the scheme from a financial standpoint, and also to incorporate in a set of specifications in securing contractual bids for such an enterprise. Originality has been our chief aim, and we have earnestly sought after a practical application of the engineering principles pertinent to hydro-electric design.
As a result of this action, the second year was taken up by the study of the liberal sciences, with a view to securing a place in one of the schools of higher education. This was a period of great toil, and a considerable amount of money was spent in securing the necessary books and apparatus.

The third year was devoted to the study of the art of medicine, and a considerable amount of time and money was spent in this pursuit. The fourth year was spent in the study of the science of surgery, and a considerable amount of money was spent in securing the necessary apparatus and books.

The fifth year was devoted to the study of the art of midwifery, and a considerable amount of money was spent in securing the necessary apparatus and books. The sixth year was spent in the study of the science of obstetrics, and a considerable amount of money was spent in securing the necessary apparatus and books.
Photograph No. 2

The Falls of the Normanskill.
LOCATION OF SITE AND ITS ADVANTAGES

French's Mills, the site of this hydraulic installation, is a small manufacturing center situated about 10 miles northwest of Albany, and 7 miles due south of Schenectady, N. Y. Its industries are few, yet sufficient to make use of the available electric power as developed in this installation.

The advantages of location are excellent. A branch of the N. Y. W. A. & B. R. R. extends through this section of the country within a short distance of the site, and thus affords a convenient means of communication with Albany or Schenectady, from which all construction material and miscellaneous equipment could be shipped. Moreover from a standpoint of economy, within a radius of 5 miles of the plant site are situated the towns of Fuller, Meadowdale, Guilderland, Guilderland Center, Altamont, Dunnsville and Schenectady, thus affording an excellent opportunity of further distribution of power, should the power developed not be entirely consumed by the local demand at French's Mills.
SOURCE OF POWER AND MEASUREMENTS
OF STREAM FLOW

The source of power for this installation lies in the Normanskill Basin, which contains a small stream having its rise at a point 12 miles directly west of Schenectady and flowing in a southeasterly direction, emptying into the Hudson River below Albany. Its tributaries are Bozen Kill, Black Creek, Bonny Brook, Island Creek and several other small streams in the vicinity of French's Mills.

It is somewhat unfortunate that a complete set of gagings for this stream in question is not available, since for an intelligent and economical design such information would be absolutely necessary. The accompanying letter from the State Engineer of New York contains a set of gagings of the Normanskill for the year 1891 which we are informed are the only measurements on record.

Complete reliance, however, could not be placed in this record, and a set of gagings was fortunately procured from Mr. Robert E. Horton,
a consulting hydraulic engineer, of Albany. His values for the mean monthly discharge are considerably higher than the Albany City Gagings. In view of the fact that Mr. Horton personally investigated this location and was especially interested in the project as is evidenced from the substance of his letter, we have concluded that his gagings are the more reliable and have prepared our designs accordingly.

It must be understood, however, that in actual practice a set of stream measurements extending over a period of at least five years is absolutely necessary for an intelligent design of any water power development. Neither were gagings to be had on any similar stream in the vicinity flowing under the same conditions, nor could any comparison be made with a stream the gagings of which had been recorded, that might afford any information whatsoever upon the flow of the Normanskill at French's Mills. With the only available data at hand then, a rating curve was plotted making use of the stream measurements recorded by Mr. Horton, and a graph of
the discharges in second feet throughout the year thus obtained.

Reference was made to this record of yearly flow in estimating the storage necessary to carry the plant over dry season periods. It is to be noted especially that during the summer months the flow runs as low as 3.5 second feet. Whether this represents an average yearly condition of the flow of the Normanskill, or whether it may have been due to an extremely dry period during this particular year of 1931, is a question open to considerable discussion. In accepting this low value, however, we are providing against any similar emergency that the installation may be called upon to meet during the seasons of future drought.
Dear Sir:

Your letter of October 10th to the State Engineer has been referred to me and in reply would say that I am having sent to you State Engineer's reports for the past five years with the exception of that for 1906, the supply of which has been exhausted.

The stream which you refer to as running through French's Mills near Voorheesville is the Normanskill. The only gagings ever taken on this stream were made in 1891 by the City of Albany. These records appear in the 1901 State Engineer's report, the supply of which has been exhausted, but I am sending you a print of the mean daily discharge record which appears in this report.

Yours very truly,

(Signed) Alex. E. Kastf
Special Deputy State Engineer
ALBANY CITY GAGINGS

Discharge in Second Feet of Normanskill at
French's Mills, Albany Co., N.Y.,
for 1891

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Dear Sir:

As per your request I send you herewith some preliminary data and estimates relative to the Normanskill Water Power at French's Mills.

First: As to the power obtainable I am satisfied that the Albany City Gagings of 1891 are too low. This is no mere guess, but is based on my personal observations of the stream practically every week during the past five years.

I have long and accurate records of Schoharie Creek, Catskill Creek, and other streams, and other data from which, without going into details, I am satisfied the Normanskill would yield at least the following horsepower under 56 feet head.

Column (2) gives the estimated flow in cubic feet per second and column (3) gives the net horsepower obtainable with water wheels of 60 cubic feet per second capacity; that is about the size wheels
I would suggest installing at the start. Under 56' head, one cubic foot per second gives precisely 5 H. P. net, from which you can easily estimate the power for water wheels of larger or smaller capacity.

Estimated flow and power of Normanskill at French's Mills in an ordinary dry year.

<table>
<thead>
<tr>
<th>Month</th>
<th>Flow Second Feet (1)</th>
<th>Second Feet (2)</th>
<th>Net Horsepower Second Feet (3)</th>
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<tr>
<td>January</td>
<td>- - 204</td>
<td>-</td>
<td>-300</td>
</tr>
<tr>
<td>February</td>
<td>- - 322</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>March</td>
<td>- - 391</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>April</td>
<td>- - 277</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>May</td>
<td>- - 277</td>
<td>-</td>
<td>300</td>
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<tr>
<td>June</td>
<td>- - 63</td>
<td>-</td>
<td>-300</td>
</tr>
<tr>
<td>July</td>
<td>- - 23</td>
<td>-</td>
<td>140</td>
</tr>
<tr>
<td>August</td>
<td>- - 3.3</td>
<td>-</td>
<td>41</td>
</tr>
<tr>
<td>September</td>
<td>- - 32</td>
<td>-</td>
<td>160</td>
</tr>
<tr>
<td>October</td>
<td>- - 119</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>November</td>
<td>- - 246</td>
<td>-</td>
<td>300</td>
</tr>
<tr>
<td>December</td>
<td>- - 230</td>
<td>-</td>
<td>300</td>
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The stream is easily good for 300 horse power in nine months of an ordinary year, about 150 H. P.
in two additional months, but perhaps only 40 to 60 H. P. in one month, usually August. However, August is the time when paper mills usually shut down for repairs.

Of course with the reservoir built affording 100 foot head or more and regulating the flow, this power will produce probably 1000 H. P. or more of continuous power. I understand, however, you are not figuring on developing storage at present.

I would suggest a concrete dam of about the form shown in the sketch below.

![Concrete Dam Sketch](image)

This must be well anchored into rock by iron pins and should be set back far enough from the crest of the upper falls to prevent undermining. The damsite is very favorable as you will see from
the accompanying photo. I would suggest a 4 foot diameter penstock and two turbines each of about 30 cubic feet per second capacity in horizontal cases. The cost of development would be roughly as follows:

<table>
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<tr>
<td>Concrete dam 200' long</td>
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<tr>
<td>Intake, racks and head gates</td>
<td>350</td>
</tr>
<tr>
<td>4' steel penstock 1300' long</td>
<td>6500</td>
</tr>
<tr>
<td>Two turbines in steel cases, erected</td>
<td>1000</td>
</tr>
<tr>
<td>Standpipe, penstock anchorage, etc.</td>
<td>250</td>
</tr>
<tr>
<td>Tail race excavation, etc.</td>
<td>500</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>$10000</strong></td>
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<tr>
<td>Add contingencies, etc.</td>
<td>$11500</td>
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</table>

The cost of the penstock could probably be reduced by using wood stave pipe, but the steel pipe and turbines proposed here would be adapted also to the higher head development.

This development would produce an average of 250 net H. P. and is an exceedingly low cost proposition.

Yours very truly,

(Signed) Robert E. Horton.
DRAINAGE AREA

The accompanying geological survey sheet shows in outlined form a complete plan of the drainage area tributary to the Normanskill Basin at French's Mills. By means of a planimeter this area was measured and determined as 117.44 square miles. After mature deliberation and a study of the prevailing conditions of rainfall, evaporation and the influencing topography of the surrounding country, an estimate of the maximum flood discharge was made at 40 second feet per square mile of drainage area. In accordance with this assumption the maximum flood run-off would equal 40 \times 117.5 = 4700 second feet. Making use of Francis' formula,

\[ q = 3.33LH^{2/3} \]

where \( q \) = discharge in second feet = 4700

\( L \) = length of spillway section = 300 feet.

This formula can then be solved for \( H \), which represents the head of water passing over the crest.
It is to be noted in passing that the above figure will determine the necessary height of flash boards required to take care of low water stages.

AVAILABLE HEADS

From an inspection of the accompanying drainage sheet it is seen that high water elevation lies on the 269 foot contour, whereas low water elevation lies on the 259 foot contour, thus limiting the drop in the reservoir to 10 feet between these extreme stages. Accordingly the elevation of the top of flash boards will be 269.00. Elevation of crest of spillway will then be 269 − 2.31 = 266.19 feet. By reference to the sketch on Plate II (Map of the Normanskill Basin) the elevation of tail race under low water is seen to be 130.00. Therefore the head under low water will equal 259 − 130 = 79.0 feet.

Under high water, the elevation of the water level in the tail race is 133.00, and in accordance with this elevation the available head under high water conditions will equal 269.0 − 133.0 = 36.0 feet.
STORAGE

An inspection of the monthly discharges of the Normanskill in second feet shows the gagings to vary from a maximum of 300 second feet during the winter months to a minimum of 3.3 second feet during the dry summer periods. To secure a constant source of power it is evident therefore that a storage reservoir must be resorted to during the months of June, July, August and September to carry the plant over the dry season without shutting down.

The area enclosed between the 269 foot contours is seen to be 662.9 acres, whereas an area of 457.5 acres is bounded by the 259 foot contour. Accordingly, assuming a uniform slope between the two contours, the storage capacity of the reservoir is

\[
662.9 - 457.5 \times 10/2 = 5602 \text{ acre feet.}
\]

Now 1 acre foot = 43560 second feet. Therefore the available storage under a 10-foot drop is equal to

\[
5602 \times 43560 = 244,003,120 \text{ second feet.}
\]

Again 1 second foot for 24 hours will represent 36,400 cubic feet, or 244,003,120/36,400 = 2320 second feet for a 24-hour day, or 2320 \times 24/20 = 3334
second feet for a 20-hour day. A tabulation of available storages may be thus enumerated on a 20-hour day basis, inasmuch as it will be later shown to have been decided to operate the plant continuously over a 20-hour day period. Accordingly, using the 20-hour day as the unit,

\[
\begin{align*}
3394 \text{ second feet for one day} &= 112.3 \text{ second feet for 30 days} \\
= 56.4 &\quad " \quad " \quad 60 " \\
= 37.6 &\quad " \quad " \quad 90 " \\
= 23.2 &\quad " \quad " \quad 120 "
\end{align*}
\]

By a process of interpolation we may so adjust these values that by drawing off all the available storage during the months of July, August and September we may obtain a constant source of power over this dry period. Accordingly, if we assume the monthly discharge during this season to be 60 second feet, we have the resulting tabulation.

\[
\begin{align*}
60 - 23 &= 32 \text{ second feet for 30 days in July} \\
60 - 3 &= 52 &\quad " \quad " \quad 30 &\quad " \quad " \quad \text{Aug.} \\
60 - 23 &= 23 &\quad " \quad " \quad 30 &\quad " \quad " \quad \text{Sept.}
\end{align*}
\]

\[= 112 \text{ second feet for 30 days.}\]
Thus it is evident that during these three summer months a maximum discharge of 60 second feet only can be obtained by exhausting the reservoir of its entire capacity.

AVAILABLE POWER

From the previous study of the prevailing conditions of stream flow it is found that a maximum discharge of 60 second feet only is available during July, August and September, and 63 second feet during the month of June. With the exception of these months of the dry period there is an abundance of flow as is attested by the monthly gaging recorded during the year. After a careful study of the most economical power output as warranted by various factors due to the location of the site and the nature of the surrounding industries, it was decided to develop a flow of 120 second feet.

In order to maintain this continuous power, which is greater than that due to the minimum flow of the stream, plus the storage during the months of
June, July, August and September, Some source of auxiliary power must be available. This would necessitate an increased first cost of the development, an additional increase in maintenance and operation, and a doubtful economical return in revenue, comparatively speaking. It was found, however, after a careful investigation of the nature of the industries situated in that part of the country, that it was customary for the large paper mills to shut down during the summer for repairs. This factor immediately offered a solution of the problem and it was decided to operate two units continuously during eight months of the year. During the dry period when the pondage was not sufficient to meet the demands of the turbines, it is proposed to shut down one unit and operate the plant at half capacity.

Had not this condition of affairs prevailed, inasmuch as at all other times water power would be available, the addition of steam or some other source of auxiliary power would apparently be warranted during the season of drought. The size of the plant needed to furnish such excess power would depend entirely upon the method of power utilization.
An investigation of the capacity and amount of auxiliary power needed, without pondage or storage, to maintain a given continuous power, could readily be made from a hydrograph as obtained by plotting the monthly flow in second feet or horsepower against the corresponding time units.

THE LOAD CURVE AND ITS INFLUENCE ON THE DESIGN OF THE POWER PLANT

All power plants are subjected to more or less change in load, and this continually changing load has an important bearing on the economy of the plant, and must accordingly be carefully considered in its design and construction. If the power output of any plant be ascertained by means of recording devices over a continuous period, and if the results obtained be graphically recorded, a curve will result showing the variation in power from time to time. This curve is known as the load curve, and its characteristics, due to certain demands, may be quite safely predicted, and to a certain degree of accuracy. The power install-
ation in question will carry a comparatively small continuous night load. This, in dark weather and in winter, will be increased by the early risers who are obliged to go early to shop and factory. These demands usually begin to affect the load curve about 5 a. m. and may cease wholly or in part by 7 a. m. depending on the weather and the season of the year. From 7 a. m. to 3 a. m. the motor load will begin to be felt and will reach a maximum about noon, suffering a slight decrease from 12 to 1. From 1 to 6 a maximum factory load will again come on the circuit, after which an appreciable falling off will be noticeable.

Between the hours of 7 a. m. and 6 p. m. then, it is proposed to supply the factories of French's Mills and those of the surrounding towns with power. From 5 a. m. to 7 a. m., the power is to be connected with mains delivering current for lighting purposes to the city of Schenectady. During the months of June, July, August and September when the demand for lighting current is not great, inasmuch as the plant will be running only at half load, all the available power can be utilized for factory loads. The suggestion
of charging storage batteries during periods of light loads had offered itself, but in view of the fact that alternating current is to be used exclusively, such a course would not be practicable. For the purposes of this power development it will be assumed that power will be supplied for a continuous 20-hour day, and during the months of June to September inclusive, the plant is to be operated at half capacity.

In actual practice every effort would be made to bring the load factor for all the hours of the day as near unity as possible. For this purpose power would be supplied to every conceivable kind of industry and consumer and the rates for consumption would be made especially favorable at times of low loads. In our development the factory and light loads overlap during the morning and evening hours giving a decided peak to the load curve but the alternators will be selected with this point in view that they will be able to carry a 50 to 75% overload for an hour or more. During the winter months water will be plentiful and the turbines can be operated at their maximum capacity.
HORSE POWER DEVELOPED

The maximum available head occurs when there is no water spilling over the flashboards, the entire discharge passing through the wheels. In this case the head is 39 feet. With the flashboards down in the flood period, the available head is less than this due to the decrease of head at the dam and the increase in elevation of the tail water. The minimum head is 79 feet which occurs during the dry season when the water in the reservoir has been drawn down 10 feet.

The losses in head occur at the intake, the penstock, the surge tank, the wheels and the tail race. Through the screens the velocity is calculated to be 2 feet per second, so the loss in head at this point is \((n-1)v^2/2g\), where "n" is the ratio of the cross-section of the channel to the meshes of the screen. Since this ratio is approximately 2:1 the loss in head is

\[
h_1 = \frac{v^2}{2g} = \frac{4}{64} = 0.06 \text{ feet}
\]

As the entrance to the penstock at the intake is bell-mouthed, no loss in head will be allowed here.
To discharge 120 second feet the penstock will have a diameter of 5 feet and a cross-sectional area of 19.65 feet$^2$.

Friction head = \( \frac{f}{d} \times \frac{v^2}{2g} \)

\( f = 0.02 \)

\( l = 1300 \) feet

\( h = \frac{0.02 \times 1300}{5} \times \frac{312}{64.4} = 3.0 \) feet

From a wood stave pipe catalogue, the loss of head for 5-foot pipe is 0.2 feet per 100', giving a total loss of 2.6 feet for the 1300 feet.

In the surge tank there are losses due to sudden contraction and expansion. By constructing a concrete channel at the bottom of the tank these losses are reduced to a minimum. The loss therefore will be taken as 0.05\( v^2/2g \).

Loss in surge tank = \( 0.05 \times 36/64 = 0.29 \) feet

From Meade's "Water Power Engineering" the loss in steel riveted pipe is 0.2 ft. per 100'. With a penstock 400 feet long the loss is 4 \( \times \) 0.2 = 0.80 feet. The loss in the tail race is \( v^2/2g = 16/64 = 0.25 \) feet.

Neglecting the losses in the wheels which can be charged up against their efficiency, the total losses are found as follows:
Total Losses

<table>
<thead>
<tr>
<th>Component</th>
<th>Loss (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Screens</td>
<td>0.06</td>
</tr>
<tr>
<td>Wood stave pipe</td>
<td>2.60</td>
</tr>
<tr>
<td>Surge tank</td>
<td>0.23</td>
</tr>
<tr>
<td>Steel riveted pipe</td>
<td>0.30</td>
</tr>
<tr>
<td>Tail race</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Total losses = 3.99 feet

Using the maximum head of 89 feet, the available head at the wheels is 89 - 3.99 = 85.0 feet. With a discharge of 120 second feet under this head the theoretical horse power developed at the wheels is

\[ \text{H.P.} = 35 \times \frac{120}{3.3} = 1100 \]

The turbines to be used develop 463 H. P. under an 85-foot head at a speed of 750 R. P. M. Using 90% as the maximum efficiency of the turbines the maximum power developed = 933 H.P. = 700 K.W. Considering this power as the highest point on the efficiency curve of the turbines, the rating of the generators is selected as 5/6 of the maximum so they will be running at 20% overload when the turbines are running at their maximum efficiency. With a generator efficiency of 90% the switch board power available is 630 K. W.
LOCATION OF DAM AND RETAINING SECTION

In selecting a suitable site for the location of the dam and retaining section, several important factors were duly taken into consideration.

1) The overflow of valuable lands.
2) The interference with water power rights above the point of development.
3) The interference with other vested or public rights.
4) The cost of the structure.

The amount of property that would be affected by the flooding resulting from the backwater is very slight, and this factor may be considered negligible. The above factors presented no serious problem with the exception of the second one for several mills and factories at present hold riparian rights above the point of development. It is proposed to supply these industries with power and thereby eliminate any objections arising from that source.

The location of the structure itself was largely determined by the previous location of an
old timber dam, the remains of which can be observed by reference to the accompanying photograph. The foundation at this point on the Normanskill is almost ideal inasmuch as the bed of the stream is composed of hard horizontal limestone strata. This condition of the substrata makes possible the construction of a more stable and impervious structure, and eliminates the use of sheet piling which would otherwise greatly add to the total cost of the development. From this point, as herein described, the river is subject to a series of falls downstream as depicted in the small sketch on Plate . It is proposed to carry a wood stave penstock from the retaining section, as thus located, to a point 1300 feet downstream to the surge tank. It is unnecessary to explain further in regard to this phase of the project, as reference to the accompanying designs will render these points clear to the reader.
DESIGN OF THE RETAINING SECTION

In the design of the retaining section for this proposed installation, the following conditions of stability have been considered.

1) The maximum compressive strength, due to the external forces acting on the section with the reservoir either empty or full, shall not exceed safe limits at the toe or heel of said section. Allowable bearing pressure on limestone strata shall be assumed at 6 tons per square foot, which provides for a considerable factor of safety against crushing.

2) There shall be no tension at any point of the section, concrete being considered as incapable of withstanding tensile stresses. This requires that the resultant of all the forces at any section shall fall within the middle third of the base of section.

3) The resistance to shearing and sliding shall be greater than the total horizontal force at the level of the joint or section investigated.

The conditions of imperviousness have not been extensively treated since the proposed mixture of con-
crete (1:2 1/2:5) to be used in construction will in all probability be as impermeable as required for the purposes intended.

In determining the profile of the retaining section, Wegmann's method has been employed, which consists in computing at successive horizontal sections beginning at the top, the necessary width of base to fulfill the conditions affecting the stability of the structure, assuming the area enclosed between adjacent sections to be trapezoidal. As it is necessary that a dam shall have a certain top width, six feet in this case, the upper portion will consist of a rectangle until such a depth is reached as will bring the line of pressure with reservoir full at the outer edge of the middle third. In order to provide for extreme conditions, namely ice and wave action, the water may be assumed to rise to the top of the retaining section. Furthermore, at the lower sections of the retaining wall where the back face becomes slightly inclined, the vertical component of the water pressure is neglected, the error arising therefrom being on the side of safety.
The top of the retaining section is assumed to be 5 feet above high water, or at elevation 274.00. The elevation of the stream bed is 235.00, thus limiting the total height of the structure to 39 feet. For the purposes of design, the wall is divided into one rectangular section, height to be determined, and three trapezoidal sections of equal depth. In all we shall consider four stages as explained in the following treatises.

**First Stage.** Depth of rectangular portion.

By using the formula, \( h = a \sqrt{g} \)

where \( h \) = height or depth of section
\( a \) = top width
\( g \) = specific gravity of concrete

we may solve for \( h \). Thus

\[ h = 6 \sqrt{2.4} = 9.27 \text{ feet}. \]

For our purpose we shall call \( h \) equal to 9 feet, thus dividing the remaining portion of the structure into three 10-foot trapezoidal sections.
Second Stage

The back of the section shall be battered 1" in 12", and front sloped at an inclination to be determined in the following computations. Reference is herein made to "Public Water Supplies" by Turneaure and Russell, Fig. 92 on page 392, in which all the nomenclature employed in the design is thoroughly explained.

Accordingly solving for the width of base of the second section, which may be represented by "x",

\[ x = \sqrt{\frac{d^3}{gh} + \left(\frac{1}{2} + \frac{A}{h}\right)^2} - \left(\frac{1}{2} + \frac{A}{h}\right) \]

where
\[
\begin{align*}
d & = \text{depth of water on base} \\
g & = \text{specific gravity of concrete} \\
h & = \text{height of section} \\
l & = \text{length of joint of overlying section} \\
A & = \text{area of overlying section}
\end{align*}
\]

Substituting in the above formula, we find

\[ x = \sqrt{\frac{193}{2.4 \times 10} + \left(\frac{6}{2} + \frac{5.4}{10}\right)^2} - \left(\frac{6}{2} + \frac{5.4}{10}\right) \]

In our design we have made \( x = 3.5 \) feet, as by trial we found the above width of 10.5 feet to have provided for too large a factor of safety.
The content of the page is not legible due to the quality of the image. It appears to be a page from a book or a document discussing a topic, possibly related to mathematics or a scientific field, but the text is not discernible.
Third Stage

\[
x = \sqrt{\frac{293}{2.4 \times 10} \times \left( \frac{8.5 + 12.65}{10} \right)^2 - \left( \frac{8.5 + 12.65}{10} \right)}
\]

\[x = 19.1 \text{ feet.}\]

In our design we have made the width of base of third section 15.16 feet.

Fourth Stage

\[
x = \sqrt{\frac{393}{2.4 \times 10} \times \left( \frac{15.16 + 24.48}{10} \right)^2 - \left( \frac{15.16 + 24.48}{10} \right)}
\]

\[x = 27.2 \text{ feet.}\]

In our design we have made the width of base of the fourth section 23.5 feet. In the graphical solution, however, we have considered water at a point 5 feet below the top of the retaining wall, which accounts for the disparity between the corresponding values of "x".

The pressure at heel and toe of retaining section with reservoir full may now be computed as follows.

Total weight of section (1' wide) = 65,715#

Eccentricity of the resultant = 1.75 feet

\[S = \frac{P}{A} \left(1 \pm \frac{6e}{A} \right) = \frac{65715}{23.5} \left(1 \pm \frac{6 \times 1.75}{23.5} \right)\]

\[S (\text{toe}) = 4050\#, (\text{heel}) 1560\# \text{ per sq. ft.}\]
The elevation of the top of flash boards has been assumed as 269.00, which is high water elevation. Allowing 3 feet for height of flash boards (actual height as figured = 2.31 feet), the elevation of the crest of spillway will then be 266.00, thus making the total height of structure 31 feet. The crest of the dam is composed of an elliptical section comprising the lip, which has its semi-major axis equal to 1/4 H and its semi-minor axis equal to 1/3H, where H is the head of water on the lower edge of the lip, or 3/7 x 3 = 3.45 feet. The rollway section is made up of a parabola derived from the equation, \( x^2 = 2py \) where \( 2p = 1.93H \).

By drawing a line from the outer edge of effective base of structure, which is equal to about 0.7 height, tangent to the parabolic curve as plotted from the above equation, the body of the structure is determined. The form of apron is obtained by joining an arc of a circle to the tangent, the object of this design being to allow the falling water to
be deflected into a horizontal position on the apron with the least possible amount of friction.

Reference to Plate shows in detail the construction as above explained.

Pressures at heel and toe of spillway section with reservoir full may be computed as follows:

Total weight of section (1' wide) = 52,000#
Eccentricity of resultant = 4.13 feet
Effective base = 20.61 feet

Here the resultant falls just at the edge of the middle third of the base so that the maximum pressure will be twice the average, and the minimum pressure is equal to 0.

\[
S_{(\text{toe})} = \frac{2 \times 52000}{20.61} = .5030\# \text{ per sq. ft.}
\]
\[
S_{(\text{heel})} = 0.
\]
INTAKE SYSTEM

Complete details of the intake system are clearly indicated on Plate , so that an extensive discussion will not be necessary here. Suffice it to say at present that it is proposed to carry a wood stave penstock from elevation 245.00 in the retaining section to the surge tank and power house as previously stated.

Design of Gates and Openings

Diameter of penstock = 5 feet
Area of penstock = 19.63 sq. ft.
Discharge "Q" = 120 second feet.

\[ v = \frac{Q}{A} = \frac{120}{19.63} = 6.11 \text{ ft. per second.} \]

To reduce this velocity to 4 feet per second as the water passes through the screens, the required opening will have an area of 30.0 square feet. Or, allowing for two openings, the clear waterway of each shall have an area of 15 sq. ft.

It was proposed at first to employ wooden sluice gates to reduce the cost of construction of the intake
system. Owing to the complexity of such a scheme, however, and to the high head of water on the openings, such an arrangement was found impracticable. Instead it was decided to use two rectangular cast iron gates, each having a clear waterway 36" x 60", as manufactured by the Coffin Valve Co., Boston, Mass. In detailing these gates, the necessary information was extracted from catalogs furnished by the above mentioned manufacturers. At the entrance to the intake system it is proposed to reduce the velocity to 2 feet per second.

**Design of Screens**

In the design of the screens it is assumed that the section is a solid dam and that all the water behind it is drawn off. Screen bars 3 1/2" x 3/3" spaced at 1 1/2 centers shall be used, the angle of inclination with the horizontal being 60 degrees.

In determining the spacing of supports for screens, the following method has been employed. Assuming the screen bars to act as continuous beams, a bending moment of 1/12wl² has been allowed.
\[ M = \frac{S}{I/c} \quad \text{Assume } S = 17000\# \]

\[ I/c = \frac{1}{6}bd^2 = \frac{1}{6} \times 2.4 \times 49/4 = 4.9 \]

\[ SL/c = 35,300 \]

\[ M = \frac{1}{12}wl^2 = SL/c = 35,300 \]

Therefore, for a head of 25 feet, "w" will equal 25 \times 62.5 = 1562.5#. Solving for "l", the vertical distance between supports, we have

\[ l = \sqrt{\frac{12 \times 83300}{15625}} = 25' \]

Similarly for different heads the maximum spacing of supports has been determined as follows:

<table>
<thead>
<tr>
<th>Head</th>
<th>Distance between supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>29&quot;</td>
</tr>
<tr>
<td>20</td>
<td>32&quot;</td>
</tr>
<tr>
<td>15</td>
<td>33&quot;</td>
</tr>
<tr>
<td>10</td>
<td>46&quot;</td>
</tr>
<tr>
<td>5</td>
<td>64&quot;</td>
</tr>
</tbody>
</table>

The design of the I-beam supports may be illustrated as follows: Assuming that each I-beam support acts as a simple beam, an allowable resisting moment of \(1/3wl^2\) has been made.

Thus for a head of 25 feet,

\[ M = \frac{1}{3}wl = \frac{1}{3} \times 62.5 \times 25 \times 2.5 \times 36 \times 12 \]

\[ = 210,924 \text{ in. lbs.} \]
\[ M = \text{sI/c} \quad \frac{I}{c} = \frac{210924}{17000} = 12.4 \]

Therefore we shall use an 3\"-13\# I-beam.

Similarly for the other heads we find the following values:

<table>
<thead>
<tr>
<th>Head</th>
<th>M</th>
<th>I</th>
<th>Size I-beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>210924</td>
<td>92.4</td>
<td>3&quot;-13.00#</td>
</tr>
<tr>
<td>20</td>
<td>137000</td>
<td>3.06</td>
<td>6&quot;-17.25#</td>
</tr>
<tr>
<td>15</td>
<td>94500</td>
<td>5.56</td>
<td>5&quot;-14.75#</td>
</tr>
<tr>
<td>.10</td>
<td>51000</td>
<td>3.00</td>
<td>4&quot;- 8.50#</td>
</tr>
<tr>
<td>5</td>
<td>26974</td>
<td>1.53</td>
<td>3&quot;- 5.50#</td>
</tr>
</tbody>
</table>

See Plate IV for details of I-beam supports.

**Design of Flash Boards**

Flash boards are to be so designed that they will collapse when water reaches high water level which is at elevation 269.00.

Thus the height of flash boards being 3'-0", the resisting moment for which they are to be designed will equal \( 0.5 \times 62.5 \times 3^2 \times 1 \times 12 \)

\[ = 562.5 \times 12 = 6750 \text{ in. lbs.} \]

\[ M = \text{sI/c} \]
Assuming an ultimate breaking unit stress for timber of 2000# per sq. in.,

\[ \frac{I}{c} = \frac{6750}{2000} = 3.375 \]

\[ 3.375 = \frac{1}{6}bd^2 = 2d \]

\[ d^2 = 1.63 \]

\[ d = 1.3 \]

Therefore we shall use 12" x 1 1/4" timber 3'-6" in length. To support the flash boards iron pipe will be used 1 1/4" in diameter and spaced 3'-0" center to center. (See Plate III for details of flash boards.)

For the details of Intake System see Plate IV which contains complete information in regard to design.
Wood Stave Pipe

The elevation of the center line of the penstock at the intake is 245.00. From here the pipe runs along the 245-foot contour, following the surface of the ground as closely as possible, to the surge tank. At the surge tank it rises to elevation 251.00 in order to cut down the height of the tank. Wood stave pipe will be used from the intake to the surge tank and steel pipe from the surge tank to the power house.

Maximum head on pipe is 268—245 = 24 feet.
Maximum pressure = \(0.434 \times 24 = 10.4\) per sq. in.
From Table 76 on page 573 of Turneaure & Russell's "Public Water Supplies," for a nominal diameter of five feet,

Stock size of wood staves = 3" x 3".
Finished thickness of staves = 2 1/2".
Economic size of bands = 5/3" circular.
Working stress in bands = 4600# per sq. in.
Factor of safety for bands = 4.
The spacing of bands for wood stave pipe is obtained from the formula,

\[ d = \frac{S}{\rho R} + e't \]

where \( d \) is the spacing of bands in inches.

\( p \) is water pressure per square inch.

\( R \) is internal radius of pipe in inches.

\( e' \) is swelling force of wood per sq. in. (here assumed as 100# per sq. in.)

\( t \) is thickness of stave in inches.

Substituting these values in the above formula,

\[ d = \frac{4600}{10.4} \times (30 + 100) \times 2.5 = 7.9'' \]

By slightly decreasing the factor of safety the bands can be spaced 3" center to center. The wood staves are milled and tongued inorder to give an accurate fit, 24 staves being required for the circumference. The bands used are of wrought iron and are made in one piece. The forged coupling shoe is electrically welded to the band, the other end of the band being upset and threaded.

The joint between the wood stave and steel pipe is formed by building up a flange of plates and angles on the steel pipe and caulking with lead.
Steel Pipe

Steel pipe will be used from the surge tank to the power house because it is under a higher head here and also because the pipe must go below the ground where it crosses two public highways.

Diameter = 5 feet.
Maximum head = 35 feet.
Static water pressure = \(35 \times 434 = 37\) per sq.in.
Water hammer = 100% static pressure.
Total water pressure = 74# per sq. in.

For the calculation of the thickness of the plate to be used, the bursting pressure due to the water is equated to twice the thickness of the plate multiplied by the safe working strength of the steel.

\[
Rd = 2ts
\]

\[
t = \frac{74d}{2s} = \frac{74 \times 60}{2 \times 10000} =.222''
\]

1/4" plate could therefore be used here but specifications require a minimum thickness of 5/16".

At the top and bottom of the hill on the power house slope, the steel penstock will be firmly anchored by heavy concrete anchorages.
null
DESIGN OF SURGE TANK

To relieve excessive water hammer and to give better regulation of the wheels, a surge tank will be placed on the side of the hill toward the powerhouse. The height of the surge tank is determined by the piezometric head at the tank and the maximum fluctuation of the head in the tank. From page 623 of Meade's "Water Power Engineering," the maximum fluctuation is obtained from the equation,

$$y = \sqrt{\frac{A \times l}{F \times g}} (v - v_0)$$

where

- $A =$ area of penstock in square feet
- $l =$ length of penstock in feet
- $F =$ area of surge tank in square feet
- $g =$ acceleration due to gravity
- $v =$ velocity in penstock required for new load
- $v_0 =$ velocity at instant of gate change

A sudden load change of one-half the total is provided for. This would be the case where one wheel is in operation and the other wheel is thrown on, or when both wheels are in operation and one is shut off.
Penstock diameter = 5 feet.  \( A = 19.65 \text{ sq. ft.} \)

Surge tank diameter = 15 feet.  \( F = 176.5 \text{ sq. ft.} \)

\( v = \text{full load velocity} = \frac{120}{19.65} = 6.1 \text{ ft. per sec.} \)

\( v = \text{half load velocity} = \frac{60}{19.65} = 3.05 \) \( " \)

Substituting these values in the formula,

\[
Y = \pm \sqrt{\frac{1965 \times 1240}{176.5 \times 32.16} (6.1 - 3.05)}
\]

\[
= \pm \sqrt{4.28 \times 3.05} = 6.32 \text{'}
\]

Allowing 3 feet for loss of head in the penstock and surge tank, the maximum elevation of the hydraulic grade line at the surge tank is 266.00. Using this as the normal elevation of the water in the tank when both wheels are in operation, a sudden load change causes a drop of 6.32 feet, or approximately to elevation 259.50. A 6-foot seal over the penstock opening brings the center line of penstock to elevation 251.00. High water is at elevation 266 - 6.5 = 272.50, and the top of the tank is placed at 5 feet above this point. These figures give a total height of tank of 30 feet.

The tank will be covered with a sheet metal roof and will rest on a hexagonal concrete base 4 feet in thickness.
The forces to be considered in the design of a surge tank are water pressure, wind pressure, weight of the pipe and action of ice.

Let \( h \) = distance in feet of any point below water surface.

\( d \) = diameter of pipe in feet = 15 feet.
\( r \) = radius of pipe in feet = 7.5 feet.
\( t \) = thickness of shell in inches at given point.
\( s \) = stress in steel in pounds per sq. in.

Water pressure = 62.5 \( x \) \( h \) \( x \) \( d \).

This pressure is resisted by twice the thickness of the steel plate multiplied by the safe working strength of the steel.

Resisting strength = 2\( t \)\( s \) \( x \) 12.

Equating, 62.5\( h \)\( d \) = 24\( t \)\( s \).

\[ t = \frac{62.5hd}{24s} = 2.6hd/s \]

In calculating the thickness of plates, 5-foot sections will be used, beginning at the top. The efficiency of the vertical joints is taken as 70% and the safe working strength of the steel as 15000# per sq. in. This changes the above formula to

\[ t = \frac{2.6hd}{s} = \frac{2.6 \times h \times 15}{15000 \times 0.7} = 0.00372h \]
The thicknesses required resulting from substituting in this formula have been tabulated as follows:

<table>
<thead>
<tr>
<th>Section</th>
<th>&quot;h&quot;</th>
<th>&quot;t&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 feet</td>
<td>0.0000 inch</td>
</tr>
<tr>
<td>2</td>
<td>5 &quot;</td>
<td>0.0186 &quot;</td>
</tr>
<tr>
<td>3</td>
<td>10 &quot;</td>
<td>0.0572 &quot;</td>
</tr>
<tr>
<td>4</td>
<td>15 &quot;</td>
<td>0.0553 &quot;</td>
</tr>
<tr>
<td>5</td>
<td>20 &quot;</td>
<td>0.0744 &quot;</td>
</tr>
<tr>
<td>6</td>
<td>25 &quot;</td>
<td>0.0930 &quot;</td>
</tr>
</tbody>
</table>

Therefore the actual thickness required is only 0.1 inch, but to make the pipe safe against buckling and ice and wind action, the minimum thickness of 5/16" will be used. The base plate is 1/2" thick and is connected to the tank by a 3 1/2" x 3 1/2" angle. The top of the tank is stiffened against buckling by a 3" x 5" angle.

The maximum pressure per square foot on the foundation is due to the weight of the water plus the weight of the tank plus the weight of the concrete base. As this pressure does not exceed 2000# under any conditions it is seen that there is absolutely no danger of the tank settling.
The anchor bolts are designed to resist the stress due to wind pressure, which is taken as 25# per square foot on the vertical projection of the tank. The moment due to the wind is

\[ M = 25dh \times \frac{h}{2} = 12.5dh \]

The resisting moment of the steel shell is

\[ M = \frac{sl}{c} \]

Equating the two moments, substituting the proper values of I and c and solving for "s", we have

\[ s = \frac{1.33h}{td} \]

and the stress per lineal inch along a circumferential line is equal to

\[ S = \frac{1.33h}{d} \]

The stress on any anchor bolt is then equal to Sp where "p" is the distance in inches between bolts.

\[ S = \frac{1.33h}{d} = \frac{1.33 \times 900}{15} = 79.3\# \]

\[ p = \frac{3.1416d}{6} = 7.35' = 94.2" \]

\[ Sp = 79.3 \times 94.2 = 7500\# \]

As the anchor bolts are 1 1/2" in diameter, they have a very considerable factor of safety. The anchor bolts extend 3 feet into the concrete and are fastened to anchor bolts embedded therein.
The question as to whether a hydro-electric plant is feasible from a commercial viewpoint is largely a matter of the power plant efficiency. Hence great care should be taken in selecting the location for the power house and in its structural design. And since the plant efficiency is dependent to a great extent upon the efficiency of the various machines which are necessary for the proper operation of the plant, great care should be exercised in selecting them.

The site chosen for the proposed power house is at a point about a quarter of a mile downstream from the dam and is at the bottom of the falls which exist at this point. The water after passing through the turbines will be discharged into a tail race excavated in the rock and not directly into the bed of the stream. This tail race, which will be 10' x 3' in section in order to give the water discharged a velocity of 4 feet per second, will extend about 50 feet downstream to the foot of a small rapids. Such a tail
race will increase the head on the turbines about two feet. This tail race will be protected from stream currents for a short distance from the power house by a concrete cut-off wall.

Having decided upon the location, the determination of the size and general plan of the power house is the next step. In this plant conditions were most favorable for the installation of turbines with vertical shafts. The supply of water was such that it was decided to install two units, one of which might be shut down for a short period during the summer. The water is brought to the turbines in a 5-foot steel penstock which enters the power house at the same elevation as the turbines. A branch from this penstock supplies one turbine, while a continuation of the main penstock, decreased in diameter, supplies the other turbine. A hand operated gate valve is to be placed just where the water enters the spiral casing of the turbine. Each turbine discharges into a concrete draft tube which in turn discharges into the tail race. The draft tubes were so designed that the velocity of the water was gradually decreased after
leaving the turbines so that on entering the tail race its velocity was 4 feet per second.

The generator is to be mounted on the same shaft with the turbine and is to be on the main floor of the power house. The exciter generator is to be mounted on the same shaft with the main generator. The revolving portion of the generator will be carried on a special thrust bearing floor. The thrust bearings for the rotor are to be of the standard Allis-Chalmers oil-bath self-contained type. Such bearings need no supply of oil under pressure and have proved efficient on recent installations.

The governors are to be of the oil pressure type and are to be directly belted to the shaft of the turbine. Two motor driven pumps will supply the necessary pressure for the operation of the governors. No transformers will be required at the power house as it is intended to transmit the current at the voltage at which it is generated. A standard switchboard suited to local conditions will be required. A 10-ton crane will also be installed. Hand wheels operating the gate valves are on the generator floor.
The size of the power house, 20' x 40', was determined from a tentative layout of the apparatus selected. The superstructure is to be of brick while the substructure is to be of plain concrete. No great care need be exercised in the matter of foundation as a good hard limestone outcrops at the surface at the site of the power house.

The roof covering will be of 1/3" slate nailed on 1" boards which in turn rest on 4"-6.25# Z-bar pur- lins. The trusses which are to be spaced 10 feet center to center will be of 2 1/2"x 2 1/2"x 1/4" angles throughout. Two angles back to back will be used for the rafters and single angles for the remaining members. Knowing this data the roof may be read- ily detailed.

The brick walls will be made 13" thick. As shown in the calculations this is more than ample as regards pressures but practically they are not too thick. This statement also applies to the concrete sub- structure. The generator floor will be composed of a concrete slab which will rest on longitudinal I-beams which in turn will rest on transverse I-beams.
The bearing floor will be of concrete carried by I-beams which rest on the concrete partitions. The crane will be supported on I-beams which rest on pilasters.

The windows on the generator floor were so arranged as to secure the maximum amount of light. For this purpose "Fenestra" steel window frames will be used throughout. For convenience in installing the heavy machinery a rolling steel door has been located in the rear wall of the structure. A 10-foot concrete stairway leads from the main generator floor to the turbine floor and also affords a convenient means of access to the generator thrust bearings on the thrust bearing floor.

The structure has been so designed that in case conditions demand the installation of an auxiliary steam plant, connections with the generators can be conveniently made. Space is allowed on the generator floor for the erection of a 300 horse power engine.

For the calculations of the various beams and floors see the following calculations.
CALCULATION FOR POWER HOUSE DESIGN

Weight of Roof Covering (slate and boards) = 313#

Wind load = 10.5 x 10 x 30 = 3,150
Snow load = 10.5 x 10 x 20 = 2,100
Weight of truss and purlins = 2,300
Weight of masonry = 21,600

Total 29,683#

This load is distributed over an area of 13 x 120 = 1510 sq. ft. The pressure per sq. in. = \( \frac{29963}{1510} = 19.8\# \), whereas a pressure of 175# is permissible.

Total load at pilaster = 29963 - 11000 = 40963
Area = 22 x 120 = 2640 sq. in.
Pressure per sq. in. = \( \frac{40963}{2640} = 15.5\# \)

Design of generator floor

The point of maximum loading on this floor is at the generator. Hence the floor will be assumed to carry a unit load equal to that at the generator. The total weight of the alternator is about 22000#, (if this only the stationary part or about 3000# is carried by the generator floor. We may assume that this is distributed
over an area of 23.75 sq. ft. \((3.1416d^2/4 = \pi 55'-6''/4\)

where 5'-6'' = diameter of generator) thus giving a load of 3000/24 = 333# per sq. ft. We will use a system of I-beams such that the longitudinal ones are spaced five every four feet and the transverse ones every ten feet.

The concrete generator floor will rest on the longitudinal beams which in turn are supported by the transverse beams.

Thus the data on the concrete floor is: span 4'-0''; load 350# per sq. ft.; simple beam. By entering Table 21 of Turneaure & Maurer's "Principles of Reinforced Concrete" with this data we find that the floor required will be 3 1/2'' thick and will need a steel area of .277 sq. in. which may be supplied 7/16'' round rods spaced 6'' center to center. The distance from the outside of concrete to the rods is to be 3/4''.

**Design of Longitudinal I-Beams.**

The load to be carried per sq. ft. is equal to 333--45 or 376#. Hence the load per foot of beam will be

\[4 \times 376--20 = 1520\# \text{ per foot of beam.}\]
\[ M = 1520 \times 10 \div 3 = 19000\# = 223000\# \]

\[ M = 223000 = 16000 \text{ I/c} \]
\[ \text{I/c} = 223/16 = 14.2 \]

\[ \text{I/c of 9"-13# I-beam} = 14.2 \quad \text{C. K.} \]

**Transverse Beams**

Load = 376 \times 5 \times 30 = 1960\# per ft.
\[ M = 1960 \times 50/3 = 93000\# \]
\[ \text{I/c} = 73 \]

Use a 15"-60# I-beam.

**Bearing Floor**

Weight of rotor of generator = 7500\#
\[ M \text{ due to rotor} = 315000\# \]

\[ M \text{ due to floor} = 73600\# \]

\[ \text{Total } M = 333,600\# \]
\[ \text{I/c} = 333/16 = 24.2 \]

Use a 10"-35# I-beam. \quad \text{C. K.}

\[ M \text{ due to weight of crane} = 3,000,000\# \]
\[ I/c = 30000/16 = 190 \]

Use a 24"-90# I-beam. \quad \text{C. K.}
The first to be considered and perhaps the most important piece of apparatus is the turbine. At the present time turbine building has become so standardized that for most plants under ordinary conditions, the proper turbine may be selected by a careful study of the catalogs of the various turbine manufacturers. Special turbines may be designed for any plant the size of which warrants the extra cost or where local conditions necessitate it. The size of this proposed plant and existing conditions were such that a special turbine was not needed. The average head, 35 feet, indicated that a turbine of the reaction type could be used here. After a careful study of the catalogs of the various manufacturers it was decided that an Allis-Chalmers turbine fitted with a spiral casing best suited the conditions at this plant. Recent installations of water wheels with spiral casings have proved very efficient and for this reason they were selected for use in this plant. The casing is a true evolutionary spiral and so decreases in section
that the velocity of the water increases gradually until it has acquired the necessary velocity at its entrance to the guide vanes. The section is so decreased that nearly all losses due to eddy currents and shock such as are produced in ordinary cylindrical cases are eliminated. The entire casing is of cast steel.

The scarcity of water in this plant demanded a turbine runner with a maximum efficiency and with as small a sacrifice of power and speed as possible. The fact that the head also varied had to be considered in selecting the turbine. Type "E" runner of this firm's make seemed to meet the imposed conditions best and therefore was selected as the turbine to be used. The loading conditions as previously discussed were such that it was decided to deliver 25-cycle current. A Type "E" 25 cycle single runner turbine 20 inches in diameter satisfies these conditions. This turbine will deliver about 463 horsepower when under an 35 foot head and discharges about 60 second feet assuming an efficiency of 30%. However, under favorable conditions, an efficiency
of 33\% may be secured. This data was calculated upon the basis of 7/3 gate opening, thus allowing for a range of loads and of head without an appreciable change in efficiency. This turbine when properly regulated has a constant speed of 750 R. P. M.

The selection of the generator is dependent upon the operating data of the turbine. Thus the generator should be of the same capacity as the turbine and should have an efficient speed equal to that of the turbine. The efficiency of a generator for such work should be high over a considerable range of loads. The Allis-Chalmers Company builds a special water wheel generator which it was decided to use. This generator is 25 cycle 3 phase, having a capacity of 350 K. W., a speed of 750 R. P. M., and delivering current at 2200 volts. Excitation is secured by means of a 5 K. W. generator mounted on the same shaft with the generator.
MAP OF
Normans Kill Basin.

Showing contours 259 & 269 ft.
above Low Water Mark in the
Hudson River at Albany, N.Y.

Jan. 22nd 1891

Scale 1 inch = 500 ft.
Total area within 269 Contour 662.93 Acres
259
457.48