DESIGN OF A 20,000 H. P.
HYDRO-ELECTRIC DEVELOPMENT
ON THE GENESEE RIVER FORTAGE, N. Y.
BY
L. I. GOLDBERG

ARMOUR INSTITUTE OF TECHNOLOGY
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hydroelectric development
DESIGN OF A 20,000 H. P. HYDRO-ELECTRIC DEVELOPMENT ON THE GENESSEE RIVER AT PORTAGE, N. Y.

A THESIS

PRESENTED BY

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CHICAGO, IL 60616

Professor of Civil Engineering

Dean of Engineering Studies

Dean of Cultural Studies
Preface.

The data upon which this design is based was secured from the Department of the U. S. Geological Survey. As the contour interval given upon the maps published by this department is twenty feet, an accurate description of the site of the proposed development could not be secured. The data given is no doubt fairly accurate, but should be carefully checked before any development involving considerable sums of money is attempted. The design has been made upon the same principles and with the same degree of exactness, as though all of the data used was known to be correct.

The calculations made in the design of the tunnel, canal, pipe line and surge tank have been omitted from this volume. Their general features have been described, however, and detailed drawings given upon the Plates included in this Design.
I desire to express my gratitude to my instructor, Professor Stanley Dean, whose interest and cooperation have been of great assistance in solving the problems encountered in connection with this design.

Louis I. Goldberg.
THE DESIGN OF 20,000 H.P. HYDRO-ELECTRIC DEVELOPMENT

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It is the function of the Draft Tube to utilize by suction
PART 1

GENERAL CONSIDERATIONS.
THE DESIGN OF A 20,000 H.P. HYDRO-ELECTRIC DEVELOPMENT

PART 1

GENERAL INFORMATION ON LOCATION, TOPOGRAPHY AND STREAM FLOW.

The site of the proposed development is on the Genesee River near the village of Portageville, Livingston County, N. Y. It is about forty miles southwest of Rochester and thirty-five miles north of the Pennsylvania boundary line. Here the river plunges over a series of three large falls within a distance of about two miles. It is proposed to develop the head due to these three falls and to the rapids between them, utilizing a total head somewhat in excess of 300 feet. On Map No. 1 Plate (1) is shown a general view of the site, showing the alignment of the river, the location
of the falls, and the village of Portageville.

A general idea of the topography of the district may be obtained from Map No. 1, Plate (1), where contours are shown at 100 foot intervals. Map No. (2), Plate (1), shows the topography of the natural basin just east of Portageville, the contour lines being drawn at four foot intervals. Both of these maps were obtained by enlarging portions of the U.S. Geological Survey Map, the four foot contours being interpolated.

Since the year 1909 a gaging station has been located at St. Helena, a village situated about 5 1/2 miles below Portageville. In the year 1913 a self-recording stage was established a short distance below the chain stage at the station. The channel at this point is cut in gravel of a fairly permanent nature, and the rating curve is well defined. Current meter measurements are taken by wading, or from a steel bridge.
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TABLE (1).

MONTHLY DISCHARGE OF THE GENESEE RIVER

AT ST. HELENA, N.Y.

<table>
<thead>
<tr>
<th>Month</th>
<th>Discharge in Second-Feet</th>
<th>Run-off PPer sq mi.</th>
<th>Depth&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan.</td>
<td>14600 1140 4800 4.56</td>
<td></td>
<td>5.37</td>
</tr>
<tr>
<td>Feb.</td>
<td>1300 300 578 0.561</td>
<td></td>
<td>0.58</td>
</tr>
<tr>
<td>Mar.</td>
<td>31100 424 5330 5.17</td>
<td></td>
<td>5.96</td>
</tr>
<tr>
<td>April</td>
<td>19000 449 2610 2.53</td>
<td></td>
<td>2.32</td>
</tr>
<tr>
<td>May</td>
<td>3790 296 917 0.89</td>
<td></td>
<td>1.03</td>
</tr>
<tr>
<td>June</td>
<td>1000 120 406 0.394</td>
<td></td>
<td>0.44</td>
</tr>
<tr>
<td>July</td>
<td>188 76 119 0.116</td>
<td></td>
<td>0.13</td>
</tr>
<tr>
<td>Aug.</td>
<td>104 41 66.6 0.065</td>
<td></td>
<td>0.07</td>
</tr>
<tr>
<td>Sept.</td>
<td>---- -- 50 0.049</td>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>Oct.</td>
<td>1170 20 208 0.202</td>
<td></td>
<td>0.23</td>
</tr>
<tr>
<td>Nov.</td>
<td>4360 222 832 0.208</td>
<td></td>
<td>0.90</td>
</tr>
<tr>
<td>Dec.</td>
<td>588 273 373 0.362</td>
<td></td>
<td>0.42</td>
</tr>
</tbody>
</table>

TABLE (1) -- Discharge data obtained under the direction of the N.Y. State Conservation Commission and U.S. Geol. Survey.
Table (1) gives the monthly discharge data for the year 1913, and Figure (2) shows the curve of mean monthly flow for a period of several years. Table (2) gives the result of a series of discharge measurements taken during 1913. The area of the drainage basin above Portageville, shown in Figure (1), is 995 square miles, while that above St. Helena is 1030 square miles, but for our purpose we may use the same values of stream flow.

DETERMINATION OF HEAD CAPABLE OF ECONOMIC DEVELOPMENT.

It may be seen from the contours shown on Map No. 1, Plate (1), that there is a total drop of head of (1095-765) or 330 feet between the natural basin opposite Portageville and the base of the fall farthest down stream. The construction of a shallow diversion dam of such height as to
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TABLE (2).

<table>
<thead>
<tr>
<th>Date</th>
<th>Mean</th>
<th>Meter No.</th>
<th>Lateral Interval</th>
<th>Submergence</th>
<th>Area</th>
<th>Discharge sq.ft tharge</th>
</tr>
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<tbody>
<tr>
<td>1913</td>
<td>Gage</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F. 13</td>
<td>4.61</td>
<td>897</td>
<td>10'</td>
<td>0.2 &amp; 0.8</td>
<td>579</td>
<td>233 417</td>
</tr>
<tr>
<td>21 4.81</td>
<td>896</td>
<td>10</td>
<td>&quot;</td>
<td>674</td>
<td>280</td>
<td>1220</td>
</tr>
<tr>
<td>M. 12 4.96</td>
<td>797</td>
<td>5</td>
<td>&quot;</td>
<td>892</td>
<td>265</td>
<td>3315</td>
</tr>
<tr>
<td>28 8.41</td>
<td>896</td>
<td>10</td>
<td>&quot;</td>
<td>2220</td>
<td>314</td>
<td>13400</td>
</tr>
<tr>
<td>23 7.73</td>
<td>896</td>
<td>10</td>
<td>&quot;</td>
<td>1980</td>
<td>309</td>
<td>10300</td>
</tr>
<tr>
<td>A. 2 4.59</td>
<td>896</td>
<td>10</td>
<td>&quot;</td>
<td>1060</td>
<td>293</td>
<td>2270</td>
</tr>
<tr>
<td>Je. 23 2.62</td>
<td>996</td>
<td>5</td>
<td>&quot;</td>
<td>199</td>
<td>152</td>
<td>300</td>
</tr>
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<td>23 2.53</td>
<td>896</td>
<td>5</td>
<td>&quot;</td>
<td>210</td>
<td>152</td>
<td>256</td>
</tr>
<tr>
<td>A. 14 1.82</td>
<td>764</td>
<td>4</td>
<td>&quot;</td>
<td>55</td>
<td>94</td>
<td>33</td>
</tr>
<tr>
<td>14 1.82</td>
<td>764</td>
<td>4</td>
<td>&quot;</td>
<td>55</td>
<td>94</td>
<td>33</td>
</tr>
<tr>
<td>C. 30 2.93</td>
<td>897</td>
<td>10</td>
<td>&quot;</td>
<td>501</td>
<td>282</td>
<td>487</td>
</tr>
</tbody>
</table>

TABLE (2) -- Current Meter discharge measurements taken at St. Helena, N.Y., under the direction of the N.Y. State Conservation Commission and the U.S. Geological Survey.
raise the elevation of the water in the basin to El. 1115 will increase the total head to 350 feet, and will at the same time recreate a large impounding reservoir for storage purposes.

Although in developments utilizing a much lower head, the surface elevation of the water in the reservoir can be lowered only a comparatively small amount without impairing the efficient operation of the plant, in the present case we may drain the reservoir from El. 1115 to El. 1096 without causing a change in effective head greater than 6%. This change in head is well within the maximum allowable variation of 25%. (Lyndon—"Hydro-Electric Power"). The change in head will never, except in case of accident to the dam, be as great as that calculated above, for at such times when the length of draught allows the level of the impounded water to be reduced to El. 1096, the elevation of the tail water will be considerably lower than under normal conditions.
DETERMINATION OF SIZE OF DEVELOPMENT
JUSTIFIED BY FALL OF STREAM, QUANTITY
OF FLOW AND STORAGE POSSIBILITIES.

The determination in advance, of the actual amount of power that may be obtained continuously over a long period of years is one of the most important factors to be considered.

The power which can be developed from the stream depends not only upon the head which may be maintained and utilized at the plant but upon the continuous flow as well. As the maximum variation in head has been found to be less than 6% of the average gross head, it may be seen that it is the variation in flow which must be investigated to determine the maximum power capable of economic development. The quantity of flow available depends upon the total annual rainfall upon the drainage area, and its distribution throughout the year.
Figure (2) shows the mean monthly discharge of the Genesee River, in second-feet for the years 1908 to 1915 inclusive, with the exception of 1911, for which year no data was available. A careful survey of the stream flow data proves that 1913 is the critical year which determines the limiting value of the power for which the stream may be developed economically.

Figure (3) shows the hydrograph of mean monthly flow for the low-water months of the year 1913, and Table (1) gives the discharge data for the entire year. The entire daily flow can be utilized in any number of hours desired, perfect pondage being furnished by the basin which acts as storage reservoir. By this means the portions of the stream flow not utilized during periods of light load can be used at some later time.

During the months of August and Sept., the quantity of stream flow was far less than that during the remainder of the season.
CURVE SHOWING
MEAN MONTHLY FLOW
OF
GENESEE R.

- Indicates quantity which may
  be used continuously.
- Indicates additional amount which
  may be installed for on a
  12-hour basis.
- Indicates quantity of Aug. flow
  utilized in Sep.
- Indicates additional quantity
  obtained from reservoir.

Flow in Cubic Feet
The continuous quantity of flow during these two months averaged 58.3 cubic feet per second. If we assume that the average power developed is to be used for twelve hours daily, a quantity of 116.6 cubic feet per second is available as shown on Figure (3).

The maximum gross head which is to be developed has been ascertained to be 350 feet, and the minimum 330 feet. Allowing a drop or "loss of head" of 10 feet in the controlling works, a minimum net head of 320 feet can be maintained and utilized at the hydraulic turbines.

The power represented by a drop of $(Q)$ cusecs from a height of $(H)$ feet is expressed as

$$H \cdot P = \frac{Q \times 62.4 \times H}{550}$$

Assuming that the fall is developed by turbines of 80% efficiency, the actual power at the turbine shaft is

$$A \cdot H \cdot P = \frac{Q \times 62.4 \times H \times 0.8}{550}$$

$$\frac{QH}{11}$$
Therefore the power developed by turbines of 80% efficiency, utilizing 116.6 cusecs under a head of 320 feet is

\[ \text{A.H.P.} = \frac{116.6 \times 320}{11} = 3390 \text{ H.P.} \]

As the above value of power is obtained by utilizing a flow equal to the average available during August and September, the stream can beyond doubt be developed for a much greater power. A thorough investigation of the storage possibilities has therefore been made.

It was found that, due to the very gradual gradient of the river above the falls, no basin of appreciable size can be utilized without necessitating the relocation of a considerable length of railroad, except the one opposite Portageville. The latter basin has been previously been referred to.

Several locations for a dam to convert this basin into a storage reservoir of considerable capacity were investigated, the one shown on Plate (1) being finally selected.
It has been decided to install a diversion dam at this location of sufficient height to raise the surface of the water in the basin to El,1115. A dam of greater height would necessitate the re-location of an excessive amount of railway and highway property. The area which will be flooded when the reservoir is completely filled is indicated upon the maps of Plate (1). Two sections of earth road are included within the area to be flooded. A relocation of one of them has been shown on Map No.(2).

In estimating the volume of the storage reservoir created by the dam it was assumed that the slope of the ground was regular between contours. The topographic map of the entire district to be flooded was divided into sections, and the various areas obtained with a planimeter. The volumes of the various sections were obtained by multiplying the areas by their respective mean depths. The total capacity was found to be 8.61 mile feet or 240,000,000 cubic feet.
As the reservoir is filled to capacity at the beginning of August the above quantity of water is available for the development of power during the two months of minimum flow. The water in storage is equivalent to a mean average flow of 45 cusecs, or to 90 cusecs if average power is used but twelve hours per day. By adding this quantity to that obtained from the stream flow during the low water period, we find that an average quantity of $116.6 + 90$ or 206.6 cusecs can be utilized, as illustrated in Figure (3).

The actual power which may be developed at the turbine shaft using 206.6 cusecs at a net head of 320 feet is

$$QH \times A.H.P. = 206.6 \times 320$$

$$A.H.P. = \frac{206.6 \times 320}{11} = 6010 \text{ H.P.}$$

If the average load is to be carried but 10 hours per day 7212 H.P. may be developed continually during that time. To develop this power at a plant load factor of 40% over this period will necessitate a total installation of 18,030 H.P.
As the flow upon which the foregoing calculations have been based is much less than the average, and remains at such a low value but a comparatively short time, it seems highly desirable that the power of the stream be developed to a much greater extent than that previously indicated. This will necessitate the installation or purchase of a steam power plant to act as an auxiliary to the development.

The installation of hydraulic equipment of sufficient capacity to develop 18,000 - 19000 H.P., enable and of a steam auxiliary of 7,500 H.P., will us to contract for a connected load of 30,000 H.P. Primary power, and Secondary ten-month load of 10,000 H.P.

During ordinary stages of flow there will be a sufficient quantity of water to enable the hydraulic units to care for the entire load, with the exception of maximum peak. As the Diversity factor can be safely relied upon to be greater than 1.5 as a minimum, it may be assumed that the peak due
to the total connected load will never exceed a value of 26,700 H.P., which can be carried by the above installation with a slight overload upon the steam auxiliary.

The location and design of the steam installation is not properly a part of this volume and has therefore been omitted. It is highly desirable, however, that a steam plant located in the vicinity of the center of power distribution be acquired by the purchase of an existing installation. If this cannot be done, the steam auxiliary equipment should be installed at the Terminal Station.
GENERAL LAYOUT OF THE PROPOSED DEVELOPMENT.

A general view of the entire development is shown upon Map No. (1), Plate No. (1). The essential features of the proposed development are as follows:—A diversion dam, which will create a storage reservoir—A power plant—A canal, tunnel and pipe line, which will convey the water from the reservoir to the power plant, penstocks replacing the pipe line for the last 500′—Tunnel intake, forebay, and surge tank located along the line of diversion.

A plan and profile along the entire length of the center line of proposed hydraulic transmission is shown upon Plate (2). A short study of this map will give a clearer idea of the general layout of the proposed development than could be given by a written explanation.

The location and general characteristics of the diversion dam have been discussed in another part of this volume, and will not be considered here.

The canal is to have a length of about
1400', its bottom at Sta.(1) being at El.1092. It will be designed to carry 600 cubic feet of water per second. Most of the excavation will be in earth or gravel, the last 200' requiring some rock removal. The slope of the canal bottom is to be .001, its elevation at the tunnel intake being 1090.6. The detailed design will be considered later under "Design of Canal".

The tunnel intake is located at the rear of a forebay. The latter is enclosed by reinforced concrete walls. The detailed design of the intake is shown on Plate(5). Controlling gates are here provided as are also screening devices. The design of this feature of the development will be taken up under "Design of Tunnel Intake".

A concrete-lined tunnel will be designed to extend from the tunnel intake at Sta.14 - 30 to a forebay at Sta.42 - 10. The capacity of the tunnel is to be 500 cubic feet per second at full load. Its total length will be 3700'. The cross-section designed for the tunnel is shown Plate(5).
A wood-stave pipe line 10' in diameter will be used to convey the water from the forebay to the site of the Surge Tank. Its total length is to be 1800'. For the first 400' of its length it runs down a hillside. The remainder of the distance it follows the contour at El. 1000, which runs almost on a straight line to a point above the power house site. At the Surge Tank the pipe discharges into a header which supplies the penstocks. The Surge Tank riser extends upwards from this header.

The site of the proposed power house is on the bank of the river, which is 600' from the Surge Tank. The fall in head from the tank to the tail water level is 250'. Four penstocks of 5' diameter extend down the bluff from the tank header to the plant. The detailed design of the various elements of the plant and its equipment is given in Part IV of this volume, and the drawings comprised in this design on Plates(4), (7), (3) and (9).
PART 2

DESIGN OF DAM.
DETERMINATION OF HEIGHT
AND LENGTH OF SPILLWAY.

The maximum flow that has been recorded at St. Helena occurred in March of the year 1913, at which time a discharge of 37,800 cusecs was observed. The spillway of the diversion dam will be designed to discharge 50,000 cusecs, thus allowing a liberal factor of safety. As the spillway is essentially a weir it will be designed as such.

The general formula for the quantity of water discharged by a weir may be expressed as

\[ Q = \frac{3}{2} C L H \]

where \( Q \) is the number of cubic feet discharged per second, \( C \) is a constant determined by experiment, \( L \) is the length of the weir in feet, and \( H \) is the height of the water in feet over the crest of the weir.

Francis gives a value of 3.33 for the constant \( C \), his equation being

\[ Q = \frac{3}{2} \times 3.33 L H \]

for suppressed weirs, neglecting the velocity of approach. There are many other constants given by various experimenters which take
into consideration this last item. However, as the dam is to be located at the lower end of the large storage basin, this velocity will be relatively small, and need not be here considered.

For preliminary calculations a spillway 500 feet in length was assumed. Substituting in the above formula we obtain

\[ \frac{3}{2} \times 50,000 = 3.33 \times 500 \times H \]

\[ H = 9.66 \text{ feet}. \]

As the dam itself is to be very shallow it was decided to limit the height over the crest to 9 feet. This will necessitate a dam having a length of clear spillway of

\[ \frac{3}{2} \times L = \frac{50,000}{9} \times 3.33 = 556' - 0". \]

The lowest point of the stream bed on the line of the proposed dam is at El. 1090, which is 25' below the maximum established water level for the basin. As the crest of the dam must be 9'-0" below this maximum, a dam will be designed the crest of which at this point is
16' - 0" above the gravel bottom, and which will be provided with gates for impounding water to a depth of 9' - 0" over the crest.

DETERMINATION OF THE TYPE OF DAM TO BE USED.

From the very limited information secured regarding the character of the stream bed at the site of the proposed dam it appears that the dam will have to be founded upon gravel of a permanent character.

The nature of the foundation thus confines the selection of the type of dam to be used to one of either the gravity type or the hollow reinforced concrete type.

The use of the latter type of construction has been increasing very rapidly during the last few years, especially at locations such as the one at hand, where the materials of construction
can be delivered directly to the site by rail.

The essential differences between this type of dam and the solid masonry gravity dam is that it depends upon the weight of the water upon its inclined upstream face, to hold it against overturning. The principal advantages of this type of dam over the solid gravity dam may be enumerated as follows:

Sliding and overturning are practically eliminated.

It has a higher factor of safety against crushing.

No uplift pressure exists beneath it.

It is suitable for any foundation which will sustain two tons or more per square foot.

Its girder-like action prevents failure of the dam when a portion of the foundation fails.

The cost is low.

The time of construction is short.

In view of the advantages possessed by
this type it has been used for the spillway section of the diversion dam. The remainder of the dam, as may be seen from Plates (1) and (3) is very long and shallow. Due to the location of the village of Portageville just below the dam, the latter must at all points be perfectly water-tight. The extreme length necessitates using some inexpensive type of section. An earth fill gravity section with a concrete core wall fulfills the requirements, and has been designed for the retaining sections of the dam.
THE DESIGN OF A 20,000 H. P. HYDRO-ELECTRIC DEVELOPMENT.

DESIGN OF HOLLOW REINFORCED CONCRETE SPILLWAY OF DAM.

The general features of this section as designed are patterned after the Ambersen Hollow Reinforced Concrete Dam. Several dams based upon the patents held by this concern have been built recently, and have been giving excellent service.

The outlines of a section through the spillway have been shown in Fig. (4), and the complete section, as finally designed, on Plate (4).

The section consists essentially of a reinforced concrete slab, shaped as shown in the figure, supported by reinforced concrete buttresses spaced 16' 0" center to center. These buttresses extend from the front face to the spillway deck of the dam, and upwards for a distance of 12' 2" above the crest of the dam.
A reinforced concrete apron extends from the toe of the dam for a distance of 75' 0" downstream at the highest section of the dam, and a lesser distance for the other sections. The purpose of this apron is two-fold: first, it prevents scouring at the base of the dam from the action of the falling water; second, it prevents seepage water from rising until it has passed a considerable distance below the dam.

A matting of reinforced concrete 12" in thickness runs the entire length of the dam. Openings are provided at intervals to allow any water which may find its way under the dam to escape.

Automatic Crest Gates 9' 0" are to be provided. These are supported between buttresses, and ride upon bascules. The total head behind the dam is raised from 16' 0" to 25' 0" by the use of these gates.

The design of the several parts of the dam will be taken up separately.
DESIGN OF DECK SLAB.

The deck slab will be supported at intervals of 16' - 0" by reinforced concrete buttresses set vertical and perpendicular to the center line of the dam. The slab will be poured in sections extending over two panels, so that it may be analyzed as a simple continuous beam.

For preliminary calculations the thickness of buttresses will be assumed as 1' 6". Then the clear span is 16 - 1' 6" or 14' 6".

The formula

\[ M = \frac{wL}{10} = 1.2 \times \frac{W}{1} \text{ in. lbs.} \]

will be used in finding the maximum bending moment, which is positive at the middle of the span and negative at every point where the beam is continuous. For purposes of analysis the slab will be considered as being divided into a series of beams, 12" in width, and supported at the buttresses.

Design of beam at base of deck slab:

The maximum depth of water at this point is 1115 - 1090 = 25'. The pressure per foot of
Beam is 62.5 x 25 x 1 = 1563#

The bending moment at the center is

\[ M = 1.2 \times 1563 \times 14.5 = 394,500 \text{ in.} \#

The beam will be designed for an allowed value of

\[ f = 500\# \text{ is unit compression in concrete.} \]

\[ f = 14000\# \text{ is unit tension in steel.} \]

The two formulae applying to this phase of rectangular reinforced concrete beam construction are:

\[ M = \frac{1}{2} f k j b d^2 \]

\[ M = p f j b d^2 \]

or \[ M = K b d^2. \]

For the values given above, Hool (Vol. 1 p. 194) gives

\[ K = 76.7 \quad k = .348 \]

\[ p = .0062 \text{ (ratio } a \text{ to } b d \text{.}) \]

\[ j = .884 \]

Therefore \[ M = 76.7 b d^2. \]
\[ M = 394,500 \text{ in.}\# \]
\[ b = 12" \]
\[
\begin{align*}
d &= \sqrt[6.7]{\frac{394,500}{76.7 \times 12}} \\
&= 20.7",
\end{align*}
\]
where \( d \) is the distance from the axis of reinforcement to the surface of compression. Allowing two inches of concrete over the steel gives a required depth of 22.7" required to safely resist bending.

Design of beam to resist shear:

Total shear at support is
\[
\frac{1563 \times (16 - 1.5)}{2} = 11,330\#
\]
\[ D = \frac{11330}{12 \times 40} = 23.6" \quad (\text{Allowed shear} = 40\#\text{s}^{\#}) \]

A depth of 24" will be taken and investigated, consideration being given to its dead weight.

\[ \text{Wt. of beam} = \frac{24 \times 150 \times 14.5}{12 \times 2} = 2175\# \]
per support or, 800# per foot.
Unit shear is

$$\frac{11330 - (2175 \times \cdot707)}{12 \times 24} = 44.7 \text{ #sq."}$$

As this figure is slightly in excess of the allowed value, a number of steel bars sufficient to reduce this figure, and to provide for diagonal tension, will be turned up.

Steel required:

The cross-sectional area of steel required is given by the expression

$$a = \frac{nbd.}{s}$$

For the assumed allowable stresses, and

$$n \text{ (the ratio of the modulae), } p = .0062.$$  

$$a = .0062 \times 12 \times (24 - 2) = 1.656 \text{ sq".}$$

The cross-sectional area of a 3/4" sq. rod is .5626 sq". Therefore the required maximum spacing is $12 / \frac{1.636}{.5625} = 4.12"$. A spacing of 4" will be used for this beam.

Safety in Bond:

The unit bond stress, as expressed by the symbol $u$, is equal to $\frac{V}{\sum j d}$. 
Substituting in the above formula we have

\[ u = 11330 - (2175 \times 0.707) = 73.4 \text{ # per sq.} \]

As this figure is well within the allowed value of 80# per sq., the above bars will be used. Therefore at the lowest section of the deck slab the total thickness \( D \) is 24", and the reinforcing steel to be used will be 3/4" sq. rods spaced on 4" centers.

Design of deck slab at top:–

The maximum depth of water acting on this section is 9'. The pressure per foot of beam will therefore be 9 \times 62 \frac{1}{2} = 564#.

\[ M = 1.2 \times 564 \times 14.5 = 142,400 \text{ in.} \#
\]

\[ d = \frac{\sqrt[3]{142,400}}{76.7 \times 12} = 12.4 \]

\[ D = 12.4 + 2 = 14.4 \]

\[ D = 15" \text{ will be used.} \]

Investigation of safety in shear:–

\[ W = \frac{15 \times 150 \times 14.5}{2} = 1360\# \text{ is dead load per support.} \]

Unit shear = \( \frac{564 \times 2.5}{12 \times 15} - (1360 \times 0.707) = 23.1 \text{ # per sq.}"
Investigation of safety in Bond.

\[ a = 0.0062 \times 12 \times (15-2) = .967 \text{ sq.}^2 \]

The required spacing is \( 12 / \frac{.967}{.5625} = 6.98" \)

If we use \( \frac{3}{4}" \) sq. rods on \( 7" \) centers the unit Bond stress will be

\[ u = \frac{V}{j \cdot d} = \frac{5051}{12/7 \times 3 \times .844} = 35.8 \text{# sq.}^2 \]

However, as the actual design only allows the deck slab to run up to within 10" of the crest the maximum spacing will be 6" in the slab proper, and 7" in a horizontal section at the crest. This will bring the Bond stress in the rods in the upper portion of the slab below the allowed maximum.

Therefore the slab as designed will vary in thickness, increasing uniformly from 15" at the top to 24" at the stream bed.

Investigation of stresses at other sections:

A section will be taken at a point 13' distant vertically below the maximum elevation of the surface of the water. This section is shown on Figure (4).
Investigation at section a - a:

Depth of water above section is 13' 0".

The pressure due to this head is

\[ P = 13 \times 62.5 = 812.5 \text{# per foot}. \]

\[ M = 1.2 \times 812.5 \times 14.5^2 = 204,600 \text{ inch#}. \]

\[ f_c = \frac{2 \times M}{k \times b \times d^2} = \frac{2 \times 204,600}{.348 \times .884 \times 12 \times 15.25} = 477\# \text{ per sq.".} \]

\[ f_s = f_c \times k/2p = \frac{477 \times .348}{2 \times .0062} = 13,400\# \text{ per sq.".} \]

\[ v_e = \frac{812.5 \times 7.25 + (17.5 \times 150 \times .707) \times 7.25/12}{12 \times 15.25} = 3\#.3\#. \]

\[ u = \frac{V}{\rho \times d} = \frac{7.005 \times 5 \times 12}{.324 \times 15.25} = 26.6\# \text{ per sq.".} \]

Investigation at section b - b.

Depth of water above section is 21' 0".

The pressure due to this head is

\[ P = 21 \times 62.5 = 1312.5 \text{# per foot}. \]

\[ M = 1.2 \times 1312.5 \times 14.5^2 = 331,000 \text{ inch#}. \]

\[ f_c = \frac{2 \times 331,000}{.348 \times .884 \times 12 \times 19.75^2} = 460\# \text{ per sq.".} \]

\[ f_s = \frac{460 \times .348}{2 \times .0062} = 12,900\# \text{ per sq."}. \]
\[ V = 1312.5 \times 7.25 \times (21.75 \times 150 \times 0.707) \times 7.25 / 12 = 10910 \text{#}. \]
\[ v_0 = \frac{10910}{1.2 \times 19.75} = 46 \text{# per sq."}. \]
\[ u = \frac{10910}{3 \times 12 \times 0.884 \times 19.75} = 78.1 \text{# per sq."}. \]

**DESIGN OF BUTTRESSES.**

The buttresses, which have been already described, conform in outline to the deck of the spillway below the crest, and are as shown in Plate (4) above. In the design of dams of this type it is customary to analyze the buttresses as columns. Due to the extreme shallowness of the dam being herein designed there will be very little chance for column action to exist. The buttresses have been designed for shear.

The shearing stress per sq. inch that will be set up in a 12" buttress at the base of the dam shall first be investigated, and the design based upon the value obtained.
A horizontal section through a buttress at the base of the dam takes a shearing stress due to a head of 25' 0" of water acting on a section of the dam 16' 0" in length.

The pressure at the top of the deck is

\[ P = 62.5 \times 9 = 562.5 \text{# per foot}. \]

That at the base is

\[ P_1 = 62.5 \times 25 = 1563 \text{# per foot}. \]

The total pressure acting normally to the deck is

\[ \frac{562.5 + 1563}{2} \times 22.62 = 24,000 \text{# per}'. \]

Therefore the total pressure acting on the 16' section of dam is

\[ 16 \times 24,000 = 384,000 \text{#}. \]

The horizontal component of this force may be considered as producing shear upon a horizontal section of buttress 34' 0" in length. The unit shear produced in this section is

\[ \tau_0 = \frac{272,000}{12 \times 34 \times 12} = 55.7 \text{# per sq.″}. \]

A thickness of 18" has been used for all sections. The maximum shear will therefore be

\[ \tau = 55.7 \times 12 / 18 = 37.1 \text{# per sq.″}. \]
The remaining elements of the hollow section have been shown upon Plate (4). Their detailed design, however, will not be explained, nor will the calculations by which they were arrived at be given, in this volume.

* * * * * * * * * * *

DESIGN OF AUTOMATIC CREST GATES.

As the elevation of the crest of the spillway is 9' 0" below the maximum elevation to which the water in the reservoir is to be raised, some type of gate is necessary which will retain the water at this level.

Several types of gate have been investigated. It was found that any type of lift gate, as the ones in general use in connection with dams of the gravity or arch types, was impracticable, as it would involve an excessive amount of concrete in buttress extensions. Several gates of the automatic type were also considered. It was found that all of these, with the exception of the one shown upon Plate (4), also required the use of wide buttresses.
It has been decided, therefore, to install gates of the latter type. The use of the supporting buttresses as shown is not a weak point in the design, as, due to the very low head of water acting, they have a high factor of safety at every section.

The dimensions of each gate are 9' x 14' 6". They are made up of structural steel members. The facing of the gates for the upper 6' 0" is heavy planking. The lower third is filled with concrete.

The gates have been designed to commence tilting upon their bascules when the water head over the spillway crest reaches a value of 9' 0". In tilting the pins at the ends of the gate framework move in steel-lined grooves located in the sides of the adjoining buttresses. These pins prevent the gate from moving downstream when it has assumed its horizontal position. The quantity of concrete placed in the lower third is of such a value that the gate will be counterbalanced at every point in its travel upon the bascules. The form of the latter is shown in Plate (4).
DESIGN OF SLUICE GATES.

Two sluice gates have been provided at the bottom of the dam. The gates are necessary during construction, as they take care of the stream flow during this period. They are also of great value in case of the development of leaks, as they may be used to drain the reservoir.

The gates used are made up of heavy cast-iron slabs supported by 1 1/2" ribs. They slide in a strong structural steel frame faced with bronze strips. A longitudinal section through a portion of one of the sluiceways, showing the gate in place, is given on Plate(4A). Two views of the gate and sluiceway entrance have been shown.

The passageway at the entrance is moulded in solid concrete, and tapers from a cross-sectional area of 25 sq. ft at the gate to 16 sq. ft in a length of 2'.

The movement of the sluice gate is controlled from within the dam, the control shaft passing through the deck. The gates are operated by hand from the platform, as indicated in the figure.
DESIGN OF EARTH FILL SECTION AND ABUTMENTS.

An earth fill section has been designed for the portions of the diversion extending from the limits of the spillway abutments to the contour at El. 1120. This gives a height of 5' 0" above the maximum high water level in the reservoir. A concrete core-wall is to extend from a distance of 5' 0" below the bottom of the fill to within 4' 0" from the top. The earth fill section as designed is shown on Plate 13.

The empirical formula used in obtaining the width of the top of the fill is

\[ W = 5' \ 0" + 0.2H \]

This gives a value of 15' 0" for the width at the top of the 20' section. The side slopes used have been 1-2 and 1-3 for the upstream and downstream sides respectfully.

The upstream side of the embankment, which is to be subject to wash, has been protected by a 12" layer of rip-rap, placed upon a 16" layer of gravel.

The core wall, dimensioned, has been shown on the plate. A plan view of the dam is also shown.
PART 3

DESIGN OF CANAL, TUNNEL, AND SURGE TANK.
THE DESIGN OF A 20,000 H.P.
HYDRO-ELECTRIC DEVELOPMENT.

DESIGN OF CANAL, TUNNEL
AND SURGE TANK.

Design of Canal:-

The canal has been designed to carry a maximum of 600 cubic feet of water per second, at a velocity of 3' per second. A typical section near the beginning has a depth of 5', a base width of 30' 6", and side slopes of 1 - 1 1/2, the hydraulic gradient being 3.955. Length of canal is 1407', slope is .001. Reinforced concrete retaining walls are provided for the last 300'.

Design of Tunnel:-

A cross section of the tunnel as designed, with all dimensions, is given on Plate (5). Total length-2780'; capacity-600 cusecs; slope-.001; maximum velocity 7 1/2' per second. Tunnel intake as designed shown on Plate (5).

Design of Surge Tank:-

Diameter-15'; Height of tank-75'; Height of riser-55'; Diam. Riser-10.9'; Total height of tank-130'.
PART 4

DESIGN OF POWER PLANT.
THE DESIGN OF A 20,000 H.P. HYDRO-ELECTRIC DEVELOPMENT.

PART 4

DETERMINATION OF VOLTAGE, SPEED AND CAPACITY OF GENERATOR INSTALLATION.

The customary generating voltage for plants of this type is 6600 volts (A.C.). The power generated is to be transmitted a maximum distance of 66 miles. A rough rule for calculating the voltage of transmission desirable is to allow a voltage of 1000 volts per mile of line. It has therefore been decided to install generating units rated at 6600 volts, and step-up transformers rated at 6600-66000.

The generators have been so selected that their total capacity is great enough to supply the maximum demand while allowing one unit to be held in reserve. Only the generators located at the Hydro-Electric Plant are here considered, those run in connection with the steam auxiliary not being directly connected with this design.
Four units of 4,500 K. V. A. have been selected. The hydraulic supply system has been designed of sufficient capacity to feed the turbines driving but three generators at one time, the available generating capacity thus being 13,500 K. V. A. A fourth unit will thus be held in reserve at all times, and can be cut in whenever one of the other units is shut down. Any three of these units, having a rated capacity of about 13,000 H. P., can carry a peak of 20,000 K. W. at an overload on each machine of 11.1%.

An operating frequency of 60 Cycles per second is required, owing to the nature of the service supplied by the plant.

In selecting the operating speed a choice must be made from several which are within practical limits. As the cyclic frequency varies with the speed of rotation, and with the number of poles on the alternator, we may find the values of speed from the expression

\[ f = \frac{p \times n}{2 \times 60} \] cycles per second.
In this equation the symbols are as follows:

- \( f \) is the frequency in cycles per second,
- \( p \) is the number of poles, and
- \( n \) is the speed in Rev, per minute.

From the above equation it may be seen that a high rotative speed is required if the number of field poles is to be kept at a reasonable figure. By substituting various values for \( p \), and 60 for \( f \), values of \( n \) were obtained. These are listed in the following table.

<table>
<thead>
<tr>
<th>( f )</th>
<th>( p )</th>
<th>( n )</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>10</td>
<td>720</td>
</tr>
<tr>
<td>60</td>
<td>12</td>
<td>600</td>
</tr>
<tr>
<td>60</td>
<td>14</td>
<td>514</td>
</tr>
<tr>
<td>60</td>
<td>16</td>
<td>450</td>
</tr>
</tbody>
</table>

The speed at which the generators are to be run was therefore left undecided until a turbine was selected. The speed of rotation, as determined under "Selection of Hydraulic Turbine Units", is to be 514 R.P.M., and the generators will therefore be operated at this speed.
SELECTION OF HYDRAULIC TURBINE UNITS.

The turbines desired must each be of such capacity that they can deliver 6000 H.P. under a minimum net head of 320', while running at one of the speeds listed in the table on the preceding page.

As the first two speeds listed are rather high for reaction turbines of the type desired, a wheel was sought which would fulfill the other desired requirements, and run at one of the latter speeds.

The Type (C) Allis-Chalmers Turbine appeared to be the best suited for our purpose. The manufacturer's ratings for two wheels of this type are given as follows:

<table>
<thead>
<tr>
<th>Ns</th>
<th>D</th>
<th>H.P.</th>
<th>q</th>
<th>H</th>
<th>N1</th>
<th>Q</th>
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<tbody>
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<td>29.4</td>
<td>42&quot;</td>
<td>6900</td>
<td>1.016</td>
<td>514</td>
<td>320</td>
<td>29.10</td>
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<tr>
<td>29.4</td>
<td>46&quot;</td>
<td>6950</td>
<td>1.225</td>
<td>514</td>
<td>320</td>
<td>26.60</td>
</tr>
</tbody>
</table>

In this table $N_s$ represents the specific speed, which is defined as the number of r.p.m. of a wheel (of this series) of such size that it will develop 1 H.P. under a head of 1'. $N_1$ is the speed
of the wheel under a head of one foot, and H.P. the power under this condition. D is the nominal diameter of the wheel in inches; H.P. the horse power under a head (H) of 320', running at a speed of (n) r.p.m., and using a quantity of (Q) cubic feet of water per second.

The value of the specific speed desired may be obtained from the expression

\[ N_s = \frac{n \sqrt{P}}{H^{3/4}} \]

Where P is the H.P. of wheel.

\[ N_s = \frac{514 \times 6000^{1/2}}{320^{3/4}} = 29.4 \text{ r.p.m.} \]

This figure checks that of the Type C wheel very closely. The type of runner desired to give efficient results under the aforementioned conditions, and at this specific speed, lies between Types III and IV as listed by Gelpke and Van Cleve. (Tables-p 156-153: "Hydraulic Turbines". The Type C Allis-Chalmers runner appears to be of a type about midway between these, and was therefore selected.
The manufacturer's rating lists the power of the 42" wheel, when operating under a head of one foot as 1.016 H.P. The H.P. required of the turbine to be installed, under a 1' head is:

\[ H.P_1 = \frac{D_1}{H_1^{1/2}} \]

\[ = 6000 / 320^{3/2} = 1.050 \text{ H.P.} \]

Inasmuch as the power of a wheel varies as the square of its diameter we have that

\[ \frac{D_1^2}{D_2^2} = \frac{H.P_1}{H.P_2}, \]

and the required diameter of a wheel of the same type is

\[ D = \left( \frac{42^2 \times 1.050}{1.016} \right)^{1/2} = 42.75" \]

\[ \text{R.P.M.} = 28.65 \text{ by proportion.} \]

\[ n = 320^{1/2} \times 28.65 = 513 \text{ R.P.M.} \]

The quantity of water used at full gate by the 42.75" wheel running at 514 R.P.M. is obtained from the expression

\[ Q_1 = D_1^2 \times Q_2 \times H_1^{1/2} / D_2^2, \]

and equals

\[ Q = 42.75^2 \times 11.13 \times \frac{320^{1/2}}{42^2} = 207.2 \]

cubic feet per second.
A section through the scroll case is shown in Fig. (\textit{j}). A dimensioned section, drawn to a larger scale, is shown upon Plate (8). The dimensions and curves used in the design were based upon empirical design ratios given by "Gelode and Van Cleve".

Unfortunately, no test data on a wheel of the homologous series from which the above was chosen was obtainable. Therefore the detailed analysis of the wheel, which would form an important adjunct to the design, had to be omitted from this work.

\textbf{DESIGN OF TAPER PIECE.}

It is the function of the Taper piece, shown in Fig. (6), to increase the velocity of the water from that obtaining in the penstock to that desired in the scroll case.

The quantity of water taken per unit at full gate has already been ascertained to be 207.2 cubic feet per second. The diameter of
penstock is to be 5'-0". Thus the velocity of the water at the entrance to the taper piece will be

\[
V = \frac{0}{A} = \frac{207.2}{0.7354 \times 52} = 10.55' / \text{sec.}
\]

In general the velocity in the scroll case should be held constant at a value which varies from .15 to .20 of the spouting velocity, the lower value being that used for high head developments. The value of the spouting velocity may be found from the expression

\[
V = .15 \text{ to } .20 \sqrt{2gH}
\]

where \(H\) is the effective head. Substituting the lower value we obtain

\[
V = .15 \sqrt{2 \times 32.16 \times 320} = 21.5' / \text{sec.}
\]

The cross-sectional area at the smaller end of the Taper piece, which is the same as the entrance area of the Scroll Case, is to be

\[
A = \frac{0}{V} = \frac{207.2}{21.5} = 9.64 \text{ sq. feet.}
\]

\[
D = (\frac{9.64}{0.7354})^{1/2} = 3.53'.
\]

A value of 3' 6" shall be used for (D); for which the velocity is 21.73' / sec.

Thus it is the function of the Taper piece to
CURVES SHOWING VELOCITY OF FLOW, INTERNAL DIAM.
OF TAPER PIECE AT 1 FT INTERVALS.
increase the velocity of the water from \( 10.55' / \text{sec} \) to \( 21.75' / \text{sec} \). Before it is allowed to enter the turbine, the total increase being \( 11.19' / \text{sec} \).

In order that the velocity of the water might be increased \textit{without} shock, the taper piece has been designed so that the acceleration will be as shown in Curve (A) of Fig. (7). Its length is \( 12' 0'' \), and its sections such that the velocity will increase along a sine curve.

The \( X \)-axis of the curve passes through ordinate \( 16.14 \) on the diagram. The equation of the curve with reference to this axis is

\[
x = 5.59 \sin \theta,
\]

where point 0 is at \( \theta = 3\pi / 2 \) and 12 at \( \theta = 5\pi / 2 \).

The velocity \( (V) \) at any point on the curve is

\[
\]

For example, the velocity at a point distant \( 2' \) from the larger end of the tube is found as follows:

\[
V = 16.14 + 5.59 \sin \theta' = 16.14 - 4.55 = 11.62' / \text{sec}
\]
## TABLE (3).

### RESULTS OF COMPUTATIONS IN DESIGN OF TAPER PIECE.

<table>
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<td>18.36</td>
<td>4.84</td>
</tr>
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<td>10.74</td>
<td>19.30</td>
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</tr>
<tr>
<td>0</td>
<td>1.000</td>
<td>&quot;</td>
<td>10.55</td>
<td>19.64</td>
<td>5.00</td>
</tr>
</tbody>
</table>
The cross-sectional area of the Taper piece at this point is

\[ A = \frac{L}{V} = 207.2/11.29 = 18.36 \text{ sq. ft.} \]

The diameter is

\[ D = \left( \frac{18.36}{0.7854} \right)^{1/2} = 4.84' \]

The ordinates to curve (B), Fig. (c), give the internal diameter of the Taper piece at intervals of one foot for its entire length. The results of the computations made at these various points are given in Table (3).

Fig. (7) shows a longitudinal view of the tube as designed, together with its connections to the Penstock valve and the scroll case. A portion is also shown, dimensioned, upon Plate (3).

The tube is to be cast in one piece with flange couplings for connection to the wheel-case and penstock valve provided at the ends.

DESIGN OF SCROLL CASE.

The high head under which the turbines are to be operated necessitates the use of a very strong
pressure chamber for the wheel setting.

It has been found that the efficiency of water wheels is highest when they are set in spiral-shaped cases, the water entering the chamber tangentially from the taper piece, and flowing around the periphery of the wheel. A portion of the water entering the scroll case from the taper piece passes between the first guide vanes it encounters, the amount of water moving on to the remainder of the periphery being steadily diminished.

The Scroll case has been so designed that the cross-section is diminished in exact proportion to the quantity flowing at each point. Thus the velocity of the water, and the pressure produced within the case is constant at every point of the inner periphery.

The diameter of the entrance to the Scroll Case has already been ascertained as 3' 6", and the velocity of flow as 21.73' / sec.

In order to find the cross-sectional areas necessary to hold the velocity of flow at a constant
DESIGN OF A 20,000 H.P. HYDRO-ELECTRIC DEVELOPMENT.

TABLE (4)

RESULTS OF COMPUTATIONS IN DESIGN OF SCROLL CASE.

<table>
<thead>
<tr>
<th>Section</th>
<th>Velocity</th>
<th>Q</th>
<th>Area</th>
<th>Diam.</th>
</tr>
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<tr>
<td>1</td>
<td>21.73</td>
<td>207.2</td>
<td>9.65</td>
<td>3.50</td>
</tr>
<tr>
<td>2</td>
<td>&quot;</td>
<td>194.4</td>
<td>8.95</td>
<td>3.38</td>
</tr>
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<td>3</td>
<td>&quot;</td>
<td>181.4</td>
<td>8.35</td>
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<tr>
<td>4</td>
<td>&quot;</td>
<td>162.5</td>
<td>7.76</td>
<td>3.14</td>
</tr>
<tr>
<td>5</td>
<td>&quot;</td>
<td>155.5</td>
<td>7.16</td>
<td>3.02</td>
</tr>
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<td>&quot;</td>
<td>142.5</td>
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<td>7</td>
<td>&quot;</td>
<td>129.6</td>
<td>5.96</td>
<td>2.76</td>
</tr>
<tr>
<td>8</td>
<td>&quot;</td>
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<td>2.61</td>
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<td>&quot;</td>
<td>103.7</td>
<td>4.77</td>
<td>2.46</td>
</tr>
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<td>&quot;</td>
<td>90.7</td>
<td>4.17</td>
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<td>&quot;</td>
<td>77.8</td>
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<td>2.13</td>
</tr>
<tr>
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<td>&quot;</td>
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<td>2.98</td>
<td>1.95</td>
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<td>&quot;</td>
<td>51.8</td>
<td>2.38</td>
<td>1.74</td>
</tr>
<tr>
<td>14</td>
<td>&quot;</td>
<td>38.8</td>
<td>1.79</td>
<td>1.51</td>
</tr>
<tr>
<td>15</td>
<td>&quot;</td>
<td>25.9</td>
<td>1.19</td>
<td>1.23</td>
</tr>
<tr>
<td>16</td>
<td>&quot;</td>
<td>12.9</td>
<td>.60</td>
<td>.87</td>
</tr>
</tbody>
</table>
value, the circumference of the wheel ring was divided into 16 parts, the assumption being that 1/16 of the total quantity of water consumed passes through each section into the runner buckets. This construction has been indicated upon Plate (8).

The quantity of water entering the Scroll Case, as shown in Table is 207.2 cubic feet per second. To illustrate the manner in which the cross-sections of the tube were obtained, the calculations for Point No. 5, which is 1/4 of the distance around the circumference of the guide ring, shall be given in detail.

\[
A = \frac{Q}{V} = \frac{(1-1/4) \times 207.2}{21.73}=7.16 \text{ sq.} \\
Q = (1 - 1/4) \times 207.2 = 155.5 \text{ cubic feet.} \\
D = (7.16 / 0.7854)^{1/2} = 3.02'.
\]

The diameters were calculated for 16 different points on the speed ring, the results of these calculations being embodied in Table 4. The elevation of the scroll case as designed is shown on Fig. (7), and the plan on Fig. (a). Dimensioned drawings of the completed design are given on Plate (3).
DESIGN OF DRAFT TUBE.

It is the function of the Draft Tube to enable the turbine to utilize by suction the fall from the runner band to the tail-water level, and, by gradually decreasing the velocity of the discharged water, to enable the exit velocity head to be utilized. The tube, which is moulded in the concrete foundation, extends from the lower edge of the runner band to the front of the power house, discharging into the river below the minimum elevation of tail water.

As the success of the draft tube in utilizing the head depends upon its ability to maintain a solid, unbroken column of water, a comparatively short draft tube is desirable.

In order that the desired results might be satisfactorily secured, and to eliminate all chances of vibrations being set up in the power house sub-structure due to poor vacuum, the draft tube has been designed in accordance with the theories and equations advanced by Mr. A.S. Hillberg in "The
Electrical Review.

It is Mr. Hillberg's contention that the vibrations in the foundations of large power plants originate in the draft tube when the turbine discharge, which moves at a high velocity, is not sufficient to fill it. His design is based upon the theory that the change in velocity head must always be less than the corresponding change in elevation. This is essential if sufficient back pressure is to be maintained to cause the discharged stream to expand and fill the tube.

The total height of the tube should be less, by four feet or more, than the barometric limits of a perfect vacuum at the location in question. A margin must be allowed for governing purposes, as the sudden closing of the turbine gates might otherwise cause sufficient vacuum beneath the runner to bring the total draft head above the barometric column. A margin must also be left so that the absolute pressure of the
water below the turbine shall not be less than that at which water vaporizes. The formula given by Mr. Hillberg for the value of (D), the total draft head at the runner, is

$$D = E - \left( \frac{v^2}{2g} \right) - L$$

where $E$ is the difference in elevation between the top of the runner band and tail water level, $(v)$ is the velocity of flow through the runner band at full speed, $(g)$ is the acceleration due to gravity= 32.16, and $L$ is the loss due to friction, curvature, and outflow. Giving $(D)$ a value of 33.9, and allowing $(M)$ as a margin for governing and vapor tension, the equation may be written

$$E = 33.9 - (M - L) - \left( \frac{v^2}{2g} \right).$$

Values of 3 to 6 feet are usually taken for the quantity $(M - L)$, the former figure for very short tubes, and the latter for excessively long ones. A conservative value of 5' has been allowed for the quantity in this design.

The value of the velocity $(v)$ is obtained
from the expression \( v = Q / A \), where \((Q)\) is the quantity of discharge in cusecs and \((A)\) is the cross-sectional area at the top of the tube.

The value of \((A)\) is the same as that at the runner band. The diameter at this point of a Type 4 runner is given by "Gelpke and Van Cleve" as \( D = .855 \ D \), where \((D)\) is the nominal runner 1 diameter. Then

\[
D = \frac{.855 \times 42^{3/4}}{12} = 3.045',
\]

and \( A = \frac{3.045^2}{.7854} = 7.28 \text{ sq.} \)

As \( Q = 207.20 \), \( v = \frac{207.20}{7.28} = 28.45 \text{ feet per sec.} \)

Substituting in the equation for \((E)\)

\[
E = 33.7 - 4 - \frac{28.45}{2\pi} = 17.11' \text{ or } 17'.
\]

As the height of the lower end section is to be equal to \((D)\) by choice, the center of this section would be \(17 - \frac{3.045}{2} = 10' 6''\) below the runner band, allowing no water seal for the tube. The height of the tube, measured along the center line is to be 10' 2'', thus providing a seal
of a depth equal to $8'$ at times of minimum tailwater level.

For purposes of design the distance from the center line of the turbine to the front of the power house has been tentatively fixed at $16.0'$. 

**Layout of center line of tube:**

The center line of the draft tube should, whenever possible, be vertical immediately beneath the runner, and should be of such shape as to intersect the center point of the end section on a horizontal tangent.

Two points on the center line of the tube have already been obtained, the one at the top being fixed, and the one at the lower end being assumed for the present.

It has been decided to design a tube having its center line vertical for a length of $5'~2''$, and parabolic, or as nearly so as possible, for the remainder of its length.
Assuming a set of rectangular coordinates passing through the point on the center line at which the vertical portion ends, the ordinates and abscissae of this point and the one at the lower end of the curve are (0, 0), and (-14, -16).

The elements of a parabola passing through these two points will be calculated. The general equation of the parabola is \( y^2 = a x \). Substituting the values of \( x \) and \( y \) as 16 and 14 in the above equation we obtain

\[
14^2 = a \times 16, \quad \text{and} \quad a = \frac{196}{16} = 12.25.
\]

Several points upon this curve were found and plotted, as shown upon Fig. (9). It may be seen from this figure that the curve is far from horizontal at the point (b). A circular arc of 14' radius has therefore been drawn which is tangent to the horizontal at the point (b), being centered at (-16, 0). A parabola passing through (0, 0), and tangent to this arc may be calculated from the expression \( y^2 = e x \).
\[ e = 2 \left( c \pm \sqrt{c(c-a)} \right), \]

where \( e \) is the horizontal distance from \((0, 0)\) to the center of the circle.

Substituting in this equation we have

\[ e = 2 \left( \frac{16 \pm \sqrt{16-12.25}}{16} \right) = 16.52. \]

and \( y^2 = 16.52x \).

The center line has accordingly been designed as shown on the lower curve of Fig. (9), being made up of a vertical, a parabolic, a circular and a horizontal section. Its total length, as scaled from dimensioned drawing shown in Plate (3), is 27.2'.

The cross-sectional areas at the different points on the center line have been designed in a manner such that the change in velocity head shall never exceed the change in static head. Reasoning along this line Mr. Hillberg has derived the following expression for the velocity at any point in the tube

\[ \frac{V^2_{a+1}}{V^2_a} = \frac{2g}{(A_{a+1}/A_a)^2} \left( h_{a+1} + h_a \right), \]
where \( h_a \) and \( h_{a-1} \) are the static heads at the corresponding points. This expression represents a parabola. The velocity in the tube should be retarded in accordance with a curve taken from the branch of a parabola, the equation of which is \( y = kx \).

To apply this curve it is necessary to shift the \( X \)-axis in such a manner that the apex of the parabola (of the parabola) will be located at \((a, b)\). If the axes of this system of coordinates are called \( Z \) and \( V \), \( z = x \), and \( v = b - y \).

\[(b - v)^2 = kx, \quad \text{and} \quad v = b - \sqrt{kx}\]

Designating the velocities at entrance and exit as \( v_1 \) and \( v_n \) respectively, and the abscissae of these points as \( z_1 \) and \( z_n \) we obtain the following:

Assuming \( v_1 \) and \( v_n \) equal to \( y_1 + y_n \)

(a) \[ v = v_1 + v_n \frac{y}{(v_1^2 + v_n^2)} z_1, \quad (l = \text{length of tube}) \]

(b) \[ z_1 = -\frac{1}{(v_1^2/v_n^2) - 1} \]

Substituting in equation (a) the values
already obtained for its symbols, and a value of 10' per second for the exit velocity, we have

\[ v = 28.45 + 10 - \frac{28.45^2 - 10^2}{28.45^2} \cdot z = 38.45 - 0.01/z \]

\[ z_1 = \frac{28.45^2 - 10^2}{10^2} = 0.01. \]

It should be noted that a value of 28.2' is taken for the length of the draft tube. It was found by carrying out the calculations for the 27.2' draft tube that the latter was too short to carry out the functions desired, if it were to live up to the rule regarding maximum change of velocity head. Accordingly, an extension one foot in length was made at the lower end, the sub-structure of the power house being extended this distance, as shown upon the plans of the power house. (Plates 6 and 7).

Upon Table (7) may be found the results of the complete calculations made at 15 sections of the tube. The curves of Velocity, Velocity head and Static head are shown upon Figure (11). Dimensined views of the draft tube as designed are given to scale upon Plate (8).
DESIGN OF A 20,000 H.P.
HYDRO-ELECTRIC DEVELOPMENT.

TABLE (5).

RESULTS OF COMPUTATIONS IN DESIGN OF
DRAFT TUBE.

<table>
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<th>d'</th>
<th>Z'</th>
<th>V</th>
<th>V''</th>
<th>h</th>
<th>h''</th>
<th>Area</th>
<th>Length'</th>
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</table>
DETAILED LAYOUT OF POWER PLANT.

HYDRAULIC EQUIPMENT

The detailed layout of the power plant is shown on Plates (6) and (9), and the wiring diagram on Plate (9).

The building is divided into two sections, one containing the turbines, generating units, and exciters, the other the transformers, switches, and lightning protective devices.

The turbine and generator units are placed 24' 0" center to center. The center line of the turbines is 17' 0" from the front of the powerhouse sub-structure, and 16' 0" from the front of the superstructure, the reason for the sub-structure extension being explained under "Design of Draft Tube".

A passageway has been provided for the inspection of the turbines and governors. The latter are of the Lombard type, and may be adjusted from the generator floor.

The detailed design of the various hydraulic features of the plant have already been given.
The penstocks pass from the rear of the plant, through arches in the foundations of the Switch House to the Penstock Valves, which are located as shown. These valves are operated from the generator floor. The concrete of the sub-structure is carried upwards as far as the middle of the Taper Piece for the section of the Generator House beneath the Switch Gallery, and to the Generator floor for the part of the plant between this and the river.

The scroll case rests directly upon the concrete of the sub-structure, the draft tube being moulded in the concrete beneath.

**ELECTRICAL EQUIPMENT.**

Four 4500 K.V.A. alternators are located upon the generator floor as shown, being mounted upon the same vertical shafts as their driving units. A 150 K.W. exciter is mounted upon the generator shaft above each unit. A motor driven exciter of the same capacity is located upon this floor.

The switchboard gallery is located over
a portion of the generator floor as shown. The rheostats are located beneath this balcony.

The Switch house has two floors. On the lower floor are located the low tension busses, low tension switches, and the transformers. The second floor contains the 66000 busses and switches, and the series transformers. The aluminum lightning arrester cells are located upon a balcony over the middle section of the plant. The spark gaps have been located upon the outside of the building, and are protected by an extension of the roof. Two high tension lines leave the plant at this point.

A wiring diagram of the plant has been given on Plate (9). The panels which have not been shown have been indicated so that a general idea of the whole system might be given.
Map No. 2

Topographic Map showing site of proposed Millipede Dam and Reservoir. Scale 1:4,000

Map No. 1

Map of district including development showing proposed construction. Scale 1:4,000

Note

The data for these maps was taken from U.S. topogaphic maps of Oregon. Surveying made by U.S. Geological Survey. Enlarged from scale of 1:50,000.

 помещение отображено при масштабе 1:4000

помещение отображено при масштабе 1:4000
Profile showing present ground line and proposed development.

DESIGN OF A 50000 kw HYDRO-ELECTRIC DEVELOPMENT
MENOMINEE INSTITUTE OF TECHNOLOGY
ENGINEERING DEPARTMENT

Drawn by: Monthly approved by: 
Section - A

Sectional view of earth embankment showing concrete core wall, and protection against washout

Earth Fill

Note

The data for the contours shown on this plate was obtained from US Geological Survey of Fortuna, NY, Burlington.

Van View of Shoreline Farm and Topography of area Scale 1:100

Earth embankment

Concrete core wall

Concrete retainage wall

Concrete wall

Note section

Earth fill

Concrete retaining wall