HYDRO-ELECTRIC DEVELOPMENT OF FAWWAR SPRING ANTILI

ALPAN, YAVUZ S. 1950

H Y D R O - E L E C T R I C D E V E L O P M E N T

OF

F A W W A R (S P R I N G) A N T E L I A S

A N T E L I A S , L E B A N O N

P A R T II

BY

Y A V U Z S. A L P A N

M A Y , 1 9 5 0

Thesis Submitted to the Civil Engineering Faculty in Partial Fulfillment of the Requirements for the Bachelor of Science in Civil Engineering.

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Beirut, Lebanon

A C K N O W L E D G E M E N T

The candidate wishes to acknowledge the valuable and constant assistance rendered to him by Prof. F. Antippa during the preperation of this thesis. The candidate is also thankfull to Prof. Rubinsky and Prof. Yeremian, who helped much in solving some of the problems encountered.

T A B L E   O F   C O N T E N T S

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

Hydro-electric developments play a vital part in the economic life of a country because they constitute one and, in most cases, the most important source of electric power, the availability of which determines to a large extent the degree of progress of a community. Inexpensive electric power is essential for the industrial and agricultural development of any country.

The advantages of hydro-electric developments reside in the fact that they use water and thus do not depend on a supply of coal or oil, both of which are subject to interruptions due to strikes, war situations or an increasing scarcity of these materials in the world. To this advantage can be added the cleanliness and comparative simplicity of hydro-electric plants.

This does not imply that water is the only source for the economic development of electric power. Indeed, it has several disadvantages, such as large investment, distance to load centers, requirement for a high load factor etc., that might make thermal energy more economical or even necessary in the absence of adequate water power. Moreover, the development of atomic energy might before long supersede all other forms of energy.

Lebanon at present is experiencing a shortage of electric power which started with the end of World War II and is being felt more acutely ever since. The choice as to the source of energy to use for the development of electric power is in most cases an economic problem and should be treated as such. However, the fact that Lebanon is relatively rich in potential water power should not be overlooked and every effort be put to study the possibility of a co-ordinated development of these resources.

The tables given in Appendix A indicate the possibilities for hydro-electric developments in Lebanon.

Having thus considered the importance of hydro-electric developments in general and their significance in Lebanon, the candidate decided to chose this subject for his thesis with the idea of getting acquainted with this field of engineering as much as possible. Hence in studying and designing the project the main stress was laid on the method of design and procedure to be followed rather than on any great accuracy in the data for design. However, the project as finally submitted conforms well with the actual conditions existing in place.

The project prepared in collaboration with

Mr. Samir Bardawil, B.A. 1949, B.Sc. candidate, comprises the use of the waters of Fawwar(Spring) Antelias for the purpose of developing electric energy to be consumed in a neighboring private electrochemical factory.

After a first survey of the whole site it was decided that a concentrated head development, where the whole head is created by building a dam across a river course and raising the level of the water, would be impossible due to the lack of any suitable site for a dam. Hence it was found that the best procedure would be to capture the waters of the spring near its highest point by building an overflow weir and divert these waters into a side canal that will follow the hill side almost parallel with the main stream but at a much flatter grade until it reaches a suitable site for a forebay. The head thus gained would then be utilized for the generation of power by leading the water through a penstock from the forebay to the turbine in the powerhouse, and back to the stream by a short tailrace canal.

The design submitted in the subsequent pages deals with partII of the project and includes the following sections:

1) Forebay

2) Penstock

3) Powerhouse and tailworks

CHAPTER I

PRELIMINARY SURVEY AND DATA

Preceding any study of the project a reconnaissance survey of the whole site was made on Sept.28, 1949. The data collected during this survey consisted of the following:

A) Stream flow measurements : This was made by the velocity-area method of measurement-- the surface velocity being determined by means of floats placed at various points of the cross-section. Measurements were taken for the discharge in the north canal and main stream(1), the combined discharge giving the total flow from the spring. In addition the flow in the south canal was also measured as a check. The data taken and the calculations made are given in Appendix B. Since during the time these measurements were taken the spring flow was at a minimum and since the minimum flow is sustained during the greater part of the year, and in the absence of any other reliable flow

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(1) See Plan No. I

measurements for any length of time the available discharge for the design of the project was taken as 0.3 m^3/sec.

B) <u>Topographic Study</u>: During the reconnaissance survey a study of the region was made with the idea of deciding on the type of development and thence on the location of the dam, canal, forebay, penstock, and powerhouse. Following this, an approximate stadia traverse was run from the spring to the location of the forebay. The drop between the forebay level and the level of the water in the stream near the powerhouse site was found to be 15 meters. The slope of the hill along whose side the forebay was to be constructed was found to be 28°-$31'$. The nature of the ground at this place was rocky.

These data were later supplemented by a cadastral map of the region obtained from the Government Offices, and a few phtographs shown at the end of the chpter.

CHAPTER II

FOREBAY

Location, capacity, and dimensions

A) The location of the forebay was chosen along a straight stretch of the hill side because of the inclined rocky nature of the ground(except on the downhill side where the terraces indicated in Plans No.II and IV start), and the convenient location with respect to the powerhouse and stream. Its orientation was thus fixed by the direction of the hill side and in this way some economy could be realized because the forebay could be created by building a retaining wall on the downhill side and excavating behind it to the width required for the capacity of the forebay. The broken stone thus obtained from the excavation in rock would be used for constructing the dabsh masonary retaining wall.

B) In choosing the capacity of the forebay the following points had to be considered:

1) The intake of the penstock at the forebay

should have a sufficient water seal of about 4 to 5 ft.(1) to prevent the formation of whirlpools and the consequent carrying of air into the penstock. Thus the normal water level should never be allowed to fall below this hight. To provide for this water seal and at the same time not to lose all the capacity of the forebay to a depth of 4 ft., the bottom of the forebay at the location of the penstock intake is to be excavated 2.00 meters lower than the general level of the forebay bottom and the penstock centerline placed 70 cms above the bottom of the cavity. This provides about 1.00 meter of water seal during the lowest allowable level of water in the forebay. Although not in exact conformity with the above requirements, this much water seal was considered sufficient due to the temporary occurance of such conditions and the relative insignificanee of efficiency at such a stage.

2) The capacity of the forebay is not supposed to provide any storage or pondage, the required to store the discharge for any length of time (say a few hours) proves to be too large to be economical and justifiable for such a project. Hence the forebay capacity was chosen with the idea of taking care only of temporary accidental fluctuations in flow without causing great variation in head. Moreover, the dimensions of the forebay had to be reduced to the minimum compatible with convenient functioning

(1) Water Power Engineering- Barrows p. 374

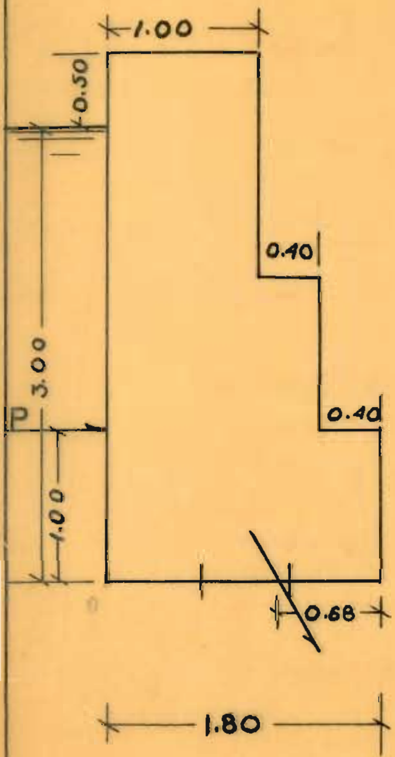
1.00
0.50
0.40
0.40
3.00
P
1.00
0.68
1.80

Fig. 1

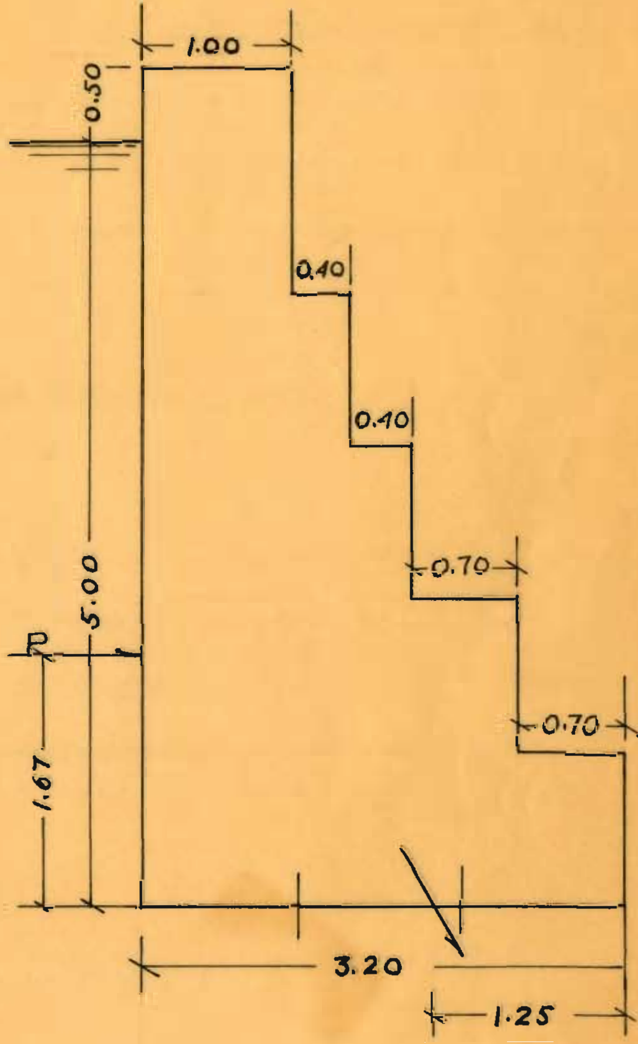
1.00
0.50
0.40
0.40
0.70
0.70
5.00
P
1.67
3.20
1.25

Fig. 2

so as to reduce the cost of the structure and the land(partly agricultural land with fruit trees) to a minimum.

With these considerations in view the capacity of the forebay was made sufficient to hold 450 m^3 of discharge, equivalent to a flow of 25 minutes(0.3x25x60= 450) . This does not include the cavity at the intake to the penstock. In addition, a freeboard of 50 cms. was provided above the highest level of water to take care of any accidental rise in the level of the water.

C) Calculations for retaining wall

Data assumed

Weight of masonary..........2400 kgs./m^3

Bearing power of rock ......4-6 kgs./cm^2

Coefficient of sliding

friction(dry masonary on rock).........0.7

1) Low side (see fig.1)

Taking moments about 0,

$$Px1 = \frac{1000x9x1}{2} = 4500 \text{ kg-m}$$

3.50x1.00x2400=	8400x1.30	= 10920	kg-m
2.00x0.40x2400=	1920x0.60	= 1152	"
1.00x0.40x2400=	960x0.20	= 192	"
W=	11280kgs.	12264	"

Overturning $\quad F.S. = \dfrac{12264}{4500} = 2.72$

Sliding $\quad F.S. = \dfrac{11280 \times 0.7}{4500} = 1.75$ (can be increased by a rough base

Base pressure

$$\begin{array}{r} 12264 \\ -\ 4500 \\ \hline 7764 \text{ kg-m} \end{array}$$

$X = \dfrac{7764}{11280} = 0.68$ (within middle third

$$S = \frac{Mc}{I} + \frac{W}{A}$$

$$= \frac{11280 \times 0.22 \times 0.90}{(1.80)^3/12} + \frac{11280}{1.80}$$

$= 1.09 \text{ kg/cm}^2$ O.K.

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2) High side (Penstock intake cavity) (see fig. 2)

Taking moments about 0,

$$P \times 1.67 = \frac{1000 \times 25 \times 1.67}{2} = 20875 \text{ Kg-m}$$

		W			M
1.00x5.50x2400	=	13200	x 2.70	=	35700
0.40x4.00x2400	=	3840	x 2.00	=	7680
0.40x3.00x2400	=	2880	x 1.60	=	4600
0.70x2.00x2400	=	3360	x 1.05	=	3530
0.70x1.00x2400	=	1680	x 0.35	=	590

Hence, -

$W = 24960$ kgs.

$M = 52100$ kg-m

<u>Overturning</u> $F.S. = \frac{52100}{20875} = 2.49$

<u>Sliding</u> $F.S. = \frac{24960 \times 0.70}{12500} = 1.39$ (can be (increased by (a rough (base

<u>Base pressure</u>

$$\begin{array}{r} 52100 \\ -20875 \\ \hline 31225 \text{ kg-m} \end{array} \qquad X = \frac{31225}{24960} = 1.25$$

(within middle (third

$$S = \frac{Mc}{I} + \frac{W}{A}$$

$$= \frac{24960 \times 0.35 \times 1.60}{(3.20)^3/12} + \frac{24960}{3.20}$$

$$= 1.29 \text{ kgs/ cm}^2. \quad \text{O.K.}$$

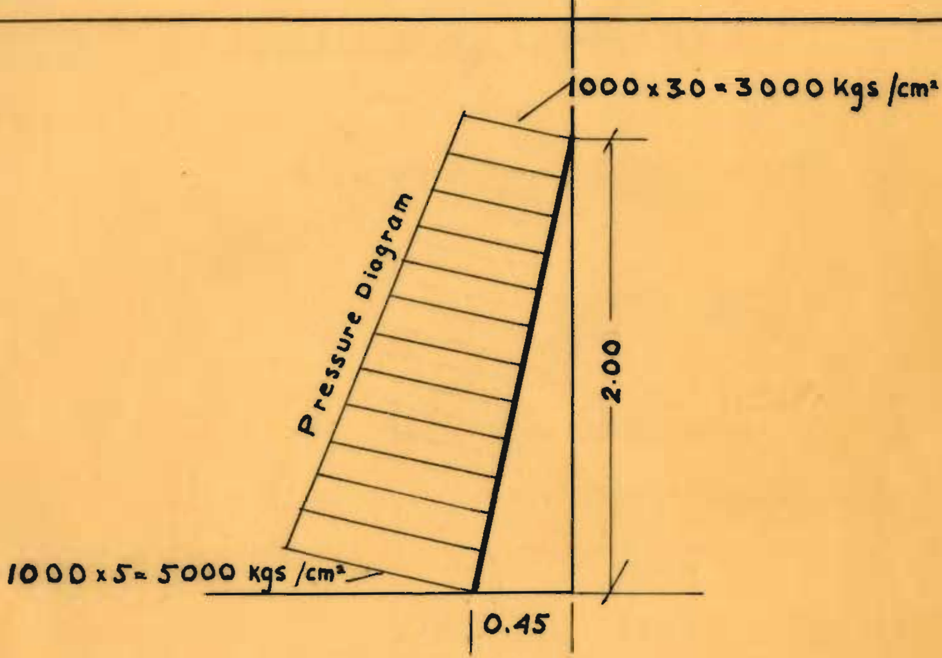
It should be observed that in the above calculations no provision has been made for the presence of a 24 in. I.D. pipe imbedded in the masonary. This condition, as found out by a seperate set of calculations reduces the apparent F.S. against sliding to about 1.28. Though this is low yet if we consider that the pipe is only at one point of the high-side retaining wall and that the high side cannot slide off without shearing off from the adjacent wall the F.S. obtained can considered satisfactory.

In this design it is assumed that the rock encountered in excavation to form the bottom and unbuilt walls of the forebay. In case the rock proves to be loose a 15-cms. cement-rubble masonary facing would be necessary. In any case a layer of plaster covering all the walls of the forebay is essential to prevent excessive leakage.

Accessories

A) Trash Rack The purpose of the trash rack being to prevent foreign bodies found in the water from reaching the turbine and damaging it, the important consideration in their design is the net opening to be left between the bars. If made too small head loss becomes appreciable and frequent glogging may result. If made too large harmful objects may reach the turbine. Usually the turbine manufacturer specifies the maximum size of object that is considered harmless to pass through the turbine. The trash rack will then be designed with the net opening between the bars equal or less than the dimension specified.

In the absence of such an information the recommendation that the clear space should be 1 to 3 inches or more(2) was followed. Moreover, the trash rack was designed to carry the total hydraulic pressure as occaisioned when the rack is completely clogged. Though this condition is rather rare yet it should be provided for in case of unforseen accidents.



1000 x 30 = 3000 Kgs /cm²
Pressure Diagram
2.00
1000 x 5 = 5000 Kgs /cm²
0.45

Fig. 3

It is observed also that contrary to usual practice the trash rack is not carried to the surface of the water but only to the top of the penstock intake cavity. It was found out to be unjustifiably expensive to carry the rack to the surface of the water. Thus at the expence of creating some inconvenience to cleaning operations considerable saving in material is achieved.

Design of Trash Rack (see fig. 3)

$$\text{Max. Moment} = \frac{5000(2.02)^2}{16}\left(1 + \frac{1}{k}\right) \qquad (3)$$

$$\left(k = \frac{5000}{3000}\right.$$

$$= 2040 \text{ kg-m}$$

$$= 177000 \text{ in.-lb.}$$

at 20000lbs./in.

$$I/c = 8.9 \text{ in.}^3$$

Using 3 x 7/16 in. flat bars

I/c per bar $= 0.65$ in.^{3}

Hence,

$$\frac{8.9}{0.65} = 13.7 \qquad \text{say 14 bars}$$

(2) Water Power Engineering- Barrows p. 514

(3) Aide-Mémoire Dunod-- Béton Armé p. 18

Using $2\frac{1}{2}$ in. c-to-c, a net space of 17/16 in. is obtained. On top and bottom use angles $5x3\frac{1}{2}x3/8$ in. Place one 2x3/8 flat bar at every third point for spacing. Attach anchorage bars to the bottom angle.

For ease in transportation and installation the trash rack is to be made in two pieces composed of 16-3x7/16 in. flat bars, giving a total width end to end of about 97 cms. The anchorage is to be made as indicated in Plan No. III.

For protection against corrosion the trash rack should be painted with lead oxide paint and black oil paint.

B) Steel ladder and railing. Weld-jointed pipe railing should be provided as indicated in plan No.III to insure the safety of the workers crossing the crest of the forebay or working at the trash rack.

The steel ladder provides a simple and easy access from and to the crest of the forebay wall--- used not only by workers cleaning the rack but also as a pathway by those going to or coming from the canal and headworks.

All railing is 1in. standard pipe and all posts $1\frac{1}{2}$in. standard pipe.

C H A P T E R III

P E N S T O C K

The function of the penstock is to lead the water from the forebay to the turbine casing in the most economic and safe manner. The penstock should follow the shortest route to the powerhouse while conforming more or less with the topography of the terrain. Any bends either vertical or horizontal will require anchorage blocks and cause head losses. With this idea in mind the powerhouse was oriented in such a way as to make possible a straight connection(in plan) between the penstock inlet and turbine casing. In profile, the penstock follows a straight line without excessive cut and fill.

A) Determination of material and diameter of pipe

The choice of the material and size of the penstock is an economic problem and hence varies with the time and place of design.

The procedure to be followed is outlined briefly in Appendix C.

The above procedure was applied to the penstock under consideration for three different materials: welded steel, cast iron, and R.C. pipe. Woodstave pipe was not considered because of the high price of timber and the skilled labour required for the work. Complete calculations for the three materials are found in Appendices D, E, and F.

From a comparison of the three graphs obtained it is seen that R.C. pipe indicates the lowest cost. However, the expenses involved in the difficult connections to be made between concrete pipe and valves and air vent together with the difficulty of obtaining easy bends rules out the choice of a R.C. pipe.

If adequate manufacturers' catalogues had been available the dimensions of cast iron pipe could have been chosen more accurately and a reduction of the cost could probably have been made-- resulting in the choice of a cast iron penstock.

Thus, under the conditions at hand and with the data available the choice left was the welded steel pipe. Considering that the project is a low head development and that the penstock is not subject to any shocks or vibrations under normal conditions, oxyacetelene or arc welded joints are considered perfectly safe and satisfactory-- specially if a good supervision is provided during execution.

The fact that a reduction in internal diameter is expected in a few years due to scale formation, and the fact that minor losses were neglected in calculating the annual value of power lost and the fact that such losses become more with smaller diameters indicate and justify the choice of a 24in. I.D. pipe instead of the 22in. indicated by the curve.

B) Penstock details and accessories

1) Pipe anchorage to forebay wall. The length of pipe imbedded in the forebay wall will be surrounded by a concrete shell about 10 cms. thick and lightly reinforced with 10 mm bars, both longitudinally and circumferentially.

The purpose of this shell is threefold:

a) Provide a protection for the thin steel pipe from any external load that might come on it.

b) Provide a good waterproof bond between pipe and wall.

c) Ease in constructing the rubble masonary wall around the pipe.

2) Air Vent. The function of the air vent is to admit air into the penstock when the latter is emptying. Otherwise a collapsing vacuum will form in the pipe. To accomplish this properly the air vent is connected as near the upstream valve as possible and on the side towards the powerhouse. It is made up of a pipe large enough to admit

sufficient air without reducing the internel pressure of the penstock much, and long enough to extend above the forebay water level. The top is covered with a protective cap to prevent any dirt from falling into the pipe and eventually obstructing the passage of air. It is essential that the air vent be always clean and ready to function.

Determination of vent diameter : The formula to be used to calculate the requisite diameter of air vent is the following:

Source

$$\overline{P}_1^2 - \overline{P}_2^2 = \left(\frac{fL}{d} + 2 \log_e \frac{V_2}{V_1} \right) \frac{W^2RT}{gA^2} \qquad (1)$$

The allowable difference in pressure between the inside and outside of the pipe depends on the diameter and plate thickness. According to tests by Carman and Carr (2) this difference should be taken as half the value given by the formula $\Delta p = 50200000 \times \left(\frac{t}{d} \right)^3$

t= plate thickness in.

d= Diameter in.

Δp= difference in pressure lbs./ in.^{2}

In the case under consideration t= 0.25 and d= 24 in. Hence,

p= 56.5 lbs./in.

Actually a difference in pressure of 2 lbs./in.

will be taken and N.T.P. conditions assumed.

Applying the formula,

$f = 0.015$

$L = 15$ ft.

$W = Q \times w = 10.6 \times 0.081 = 0.86$ lbs.

$R = 96$

$T = 273^\circ A$

$$(P_1 - P_2)(P_1 + P_2) = \frac{fL}{d} \times \frac{W^2RT}{gA^2}$$

$$d = \frac{21.9}{1135000} = 1.93 \times 10^{-4}$$

$d = 0.865$ in.

Use 2 in. vent pipe for ample safety.

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3) Expansion joint. An expansion joint is normally not necessary in such a short pipe, but the lack of proper anchorage that would take the stresses resulting from expansion and contraction, and the fact that the pipe is exposed necessitate the use of an expansion joint. Moreover, its use makes installation easier by adjusting or taking care of the slight inaccuracies that are bound to occur in the pipe dimensions.

The amount of expansion to be provided for can

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(1) See Appendix G

(2) Waterworks Handbook-- Flinn, Weston and Bogert--p.447

be determined by the following relation,

$$\Delta l = L \times 6.5 \times 10^{-6} \times \Delta T$$

Δl= expansion

L= length of exposed penstock= 65 ft.

ΔT= Range of tempreture variation= 54°F

Hence,

Δl= 0.023 ft.

= 0.7 cms.

As to the type of expansion joint to use there are several possibilities. The one chosen, however, is recommended for its simplicity.

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4) Valves. Two valves are used in the penstock. The upper valve is to shut off the water from the penstock during an accident or when repair works are to be undertaken in the pipe. In high head installations these valves are usually made automatic so as to close when the velocity of flow increases beyond a certain limit. Such a valve is not justified here. A hand operated through or gate valve will be used.

The lower valve shuts off the water from the turbine and is used in emergency closures when reaching the upper valve will take time , or prove inconvenient.

In pipes of large diameter the valves are made smaller than the pipe diameter and connected to it by a reducer and an enlarger. As to the choice of valve diameter it becomes an economic study to get the minimum annual cost obtained by adding the annual cost of valve and the annual value of power lost through the turbulence caused at the valve.

In the present case a reduction of diameter of the first valve is not justified because the penstock diameter of 24 in. is not excessive.

For the lower valve, however, a reduction of diameter is both desirable and necessary. It is desirable because a reduction in diameter increases the velocity and thus creates more steady conditions of flow just before the entrance to the turbine casing. It is necessary because a connection has to be made to the 18 in. I.D. inlet pipe attached to the turbine casing.

5) Protective measures against corrosion.

a) Exposure of pipe. The penstock was laid exposed for the following reasons:

1) Buried pipes are subject to corrosion more than exposed pipes and hence require better protection.

2) Exposed pipes can be under easy and constant inspection.

3) Steel pipes are not efficient in carrying backfill pressure.

b) Protective coatings

Adequate protection of penstock lines against corrosion is becoming more and more important as the nature of corrosion is being understood more definitely and its detrimental effects on unprotected pipes are more clearly realized.

The nature and mechanism of corrosion is electrochemical and require the presence of an electrolyte, Oxygen, and two different metals. It is usually divided into atmospheric, hydraulic, and soil corrosion. In the case under consideration only the first two types are important because the pipe is not buried underground.

a) Atmospheric corrosion. The factors affecting this type of corrosion are foreign matter found in the air, and purity of the metal. An adequate protection consists of one coat of lead oxide paint and two coats of white oil paint. The white color will, in addition, protect the pipe from excessive tempreture variations.

b) Hydraulic corrosion. This type of corrosion occurs mainly in the interior of the penstock and depends on the purity of the metal and the composition of the water flowing in it. If the water flowing in it contains a lot of calcium bicarbonate this might deposit as calcium carbonate on the interior of the pipe and thus form a natural protective covering. However, a bituminous coating is usually considered necessary not only because it protects the pipe but also improves the hydraulic conditions. When applied not less than 1/16 in. thick to the interior of the water pipe, while the pipe is being rapidly revolved, these materials give satisfactory protection. Coal-tar pitch, properly refined seems to be less subject to deterioration in water or soil than the asphalts.(3)

c) Drainage

To provide a good base for the pipe and the same time drain away the rain water that will fall on the pipe and around it, a 15-cms layer of broken stone will be placed below the pipe as indicated in Plan No. IV. Proper drainage is essential for adequate protection of the penstock against corrosion.

On top of this layer of broken stone a layer of sand sufficient to smooth up the surface will be put. The purpose in putting this layer is to provide a smooth surface on which the pipes can be placed and eventually slide(due to expansion and contraction) without the protective paint being damaged by the rough broken stone surface. Special attention should be given so that no spot on the pipe remains unprotected.

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(3) Corrosion- Causes and Protection. Frank Speller p.348

6) Investigations for water hammer and force at bends.

a) Water hammer . The pressure variations due to a sudden closure of the turbine gates can be computed by an equation derived by L. Allievi(4).

$$h = \frac{N\,H}{2} \pm \sqrt{\frac{N^2}{4} + N} \qquad \begin{cases} \text{use } + \text{ for closing} \\ \text{use } - \text{ for opening} \end{cases}$$

h= pressure rise or drop beyond static head, in feet

H= normal static head

$$N = (LV/\,gTH)^2 \qquad \begin{cases} L = \text{length of penstock} \\ V = \text{velocity of flow, ft./sec} \\ T = \text{time of closure, sec.} \end{cases}$$

For the conditions under consideration,

$$V = \frac{10.6}{A} = \frac{10.6}{3.14} = 3.38 \text{ ft./sec}$$

T= 7.5 sec.(from manufacturer's (specifications.

H= 34 ft.

L= 90 ft.

Hence,

$$N = \left(\frac{90 \times 3.38}{32.2 \times 7.5 \times 34}\right)^2 = 13.7 \times 10^{-4}$$

(4) Water Power Engineering- Barrows p. 557

Therefore,

$$h = \frac{13.7 \times 10^{-4} \times 34}{2} + \sqrt{\frac{(0.037)^4}{4} + (0.037)^2}$$

$$= 0.060 \text{ ft.}$$

Obviously, the rise in pressure is completely negligible and no provision for pressure relief is required.

b) Forces acting at bends.. In the alignment of the penstock there is only one vertical bent with an angle of $16\frac{1}{2}°$, as indicated in Plan No. IV.

The usual formulae used in calculating the forces at a bend are the following (see fig. 4):

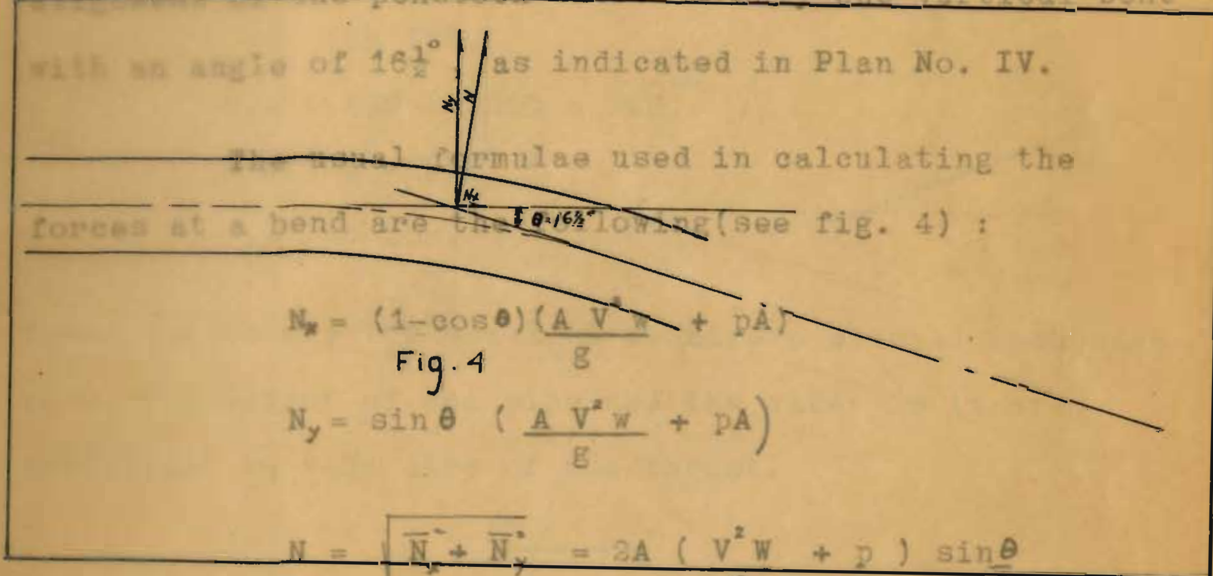

Fig. 4

$$N_x = (1-\cos\theta)\left(\frac{A V^2 w}{g} + pA\right)$$

$$N_y = \sin\theta \left(\frac{A V^2 w}{g} + pA\right)$$

$$N = \sqrt{\overline{N}_x^2 + \overline{N}_y^2} = 2A\left(\frac{V^2 W}{g} + p\right)\sin\frac{\theta}{2}$$

A= cross-section of pipe ft.

w= specific weight lbs/ft^3

p= pressure lbs/ft^2.

V= velocity of flow ft./sec

Hence for the conditions under study,

$A = 3.14\ ft^2$

$V = 3.38\ ft/sec$

$w = 62.5\ lbs./ft^3$

$p = 16\ ft. = 1000\ lbs./\ ft^2$

$\theta = 16\frac{1}{2}^\circ \qquad \sin\theta = 0.284$

$\cos\theta = 0.959$

$$N_x = (1-\cos\theta)\left(\frac{3.14 \times (3.38)^2 \times 62.5}{32.2} + 1000 \times 3.14\right)$$

$$= 0.041 \times 3210$$

$$= 132\ lbs.$$

$$N_y = 0.284 \times 3210 = 928^{\#}$$

$$N = (\overline{132}^2 + \overline{928}^2)^{1/2}$$

$$= 940\ lbs.$$

These forces are too small to require a special anchorage mass. The weight of the pipe and the water in it are sufficient to take care of the thrust.

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7) <u>Pathway</u>. As indicated in plan No.III a pathway about 1.00 meter in width is leveled along the penstock to serve for pipe inspection and access for workers going from the powerhouse to the forebay, canal or headworks.

C H A P T E R IV

POWERHOUSE AND TAILWOKS

I- Powerhouse

The powerhouse forms the nerve center of a hydroelectric system because it houses the hydraulic and electrical equipment and the main control mechanism. Economy, safety and convenience in its location and design are, therefore, of prime importance.

A) General plan and structures

In the arrangement of the powerhouse units the main aim was economy- through compactness and simplicity of layout, and convenience. Hence all the secondary units were compacted to one side of the large turbine room, thus providing on top a space for a small living quarter for the station operator. Moreover, this arrangement provides a large degree of flexibility to the plan because any future extension to the turbine room, although beyond probability for the project under consideration, can be done just by removing the northern wall and extending the building.

The turbine room dimensions are ample to accomodate the turbine-generator unit with its governer and switchboard. The main door of the turbine room should be large enough to pass the equipment easily, and the ceiling high enough to provide for raising the turbo-generator unit with a crane during installation or repairs. Ample light being essential, the windows were made as large as possible exept in the eastern wall, where the presence of the penstock and the switchboard necessarily reduced the window size.

In the secondary units of the powerhouse the office was put towrds the road for easy access, and adjacent to the turbine room and central among the other units for convenient control. The workshop, serving the turbine room mainly, was put adjacent to it. The lavatory facilities are adequate-- considering that the maximum number of workers in the powerhouse would not exceed 3 or 4. No urinary was considered necessary. ? Why ?

In planning the living quarters for the station operator, provision was made for a family made up of two adults and one or two children at most. The main two aims of the plan was, first, to provide a large bedroom, and, second, to seperate the toilet facilities from the bath to facilitate laundry washing. The Living-Dining room is small but this disadvantage is balanced by the large terrace provided in front of the Living-Dining room and which can be used for similar purposes.

As to the general location of the powerhouse it was made with the idea of obtaining a straight penstock, a short tailrace channel, convenient access from the road, and a sufficient setback from the river to facilitate construction operations.

The building was put in two levels to avoid excessive cut and at the same time provide easy access to the roof of the turbine room from the living quarters of the operator.

The materials to be used for constructing the powerhouse could be summarized as follows:

1) Foundations and frame : R. Concrete
2) Outside walls : Limestone, or sandstone
3) Inside walls: Cement blocks
4) All windows : Steel sash
5) Interior and exterior doors: timber
6) Turbine room main entrance: Sliding door.

B) <u>Equipment- Hydraulic, electrical and miscellaneous</u>

1) <u>Turbine Room</u>

Choosing the hydraulic and electrical equipment for any hydro-electric project is a delicate work for which a civil engineer is not wholly qualified.

However, before any preliminary selection of the units can be made, the total capacity for which the development is to be designed should be determined. In cases where the flow varies through a wide range the whole question becomes a complicated problem involving the determination of the most economic capacity for which the system is to be designed. Such a problem does not arise in the present case because the system is designed for the minimum flow which endures during the major part of the year. Any consideration of seasonal storage is out of the question-- due to the absence of any favourable site for storage.

Having thus determined the total capacity, the next step is to determine the number of units. In large developments this also becomes an economic problem involving the comparison of the costs of machinary, powerhouse, foundations, etc., together with the provision for permanence of service.

In the project under consideration a single unit was chosen because the power developed is small and continuous service is not vital. Production in the electro-chemical factory can easily be interrupted for some time without much harm.

Once the total capacity and the number of units have been decided upon, the final procedure is to make up a tentative selection of the units and then contact several turbine and generator manufacturers for the final selection and design of the suitable units. For small developments, as the one under consideration, the units-- hydraulic and electrical-- are usually selected from the standard sizes.

Through the procedure outlined above the following results were obtained:

a) Generator: Considering that the electric power generated would be supplied to a nearby electrochemical plant, it was decided to produce electricity at a low voltage-- 110/190 volts. Hence the tentative generator unit selected was a 110/190 volt, 3 phase , 50 cycles/sec. , 4 poles, 1500 r.p.m. , A.C. generator, with a capacity of 40 kva. (1)

b) Turbine : With the approximate speed of the turbine being known(considering a direct coupling with generator unit) a tentative selection for the suitable type of turbine was made by the Specific Speed Method.

---

(1) Available power = $\frac{0.65 \times 10.6 \times 62.5 \times 49.5}{550}$ = 39 h.p.
= 29 kw.

0.65 = Overall efficiency

10.6 = Discharge, $ft.^3$ / sec

49.5 = Gross head, ft.

40 kva at 0.80 P.F. = 32 kw.

Thus ,

$$N_s = \frac{N\sqrt{h.p.}}{h^{5/4}}$$

N_s = specific speed

h.p. = horse-power output

h = head

N = r.p.m. of turbine

Hence,

$$N_s = \frac{1500 \times (39)^{1/2}}{(49)^{5/4}} = 72 \text{ r.p.m.}$$

Obviously this indicates a reaction turbine. (2) Moreover, the N_s obtained agrees almost exactly with the recommended relation between N_s and the head with regard to pitting. (3)

Having determined the type of turbine what remains is to select the wheel setting, whether horizontal or vertical.

---

(2)

Type of turbine	N_s	
Impulse	3-6	Water Power Engineering--- Barrows p. 230
Reaction	10- 110	
Propeller	110- 225	

(3) Water Power Engineering- Barrows Table 73

Some of the advantages and dis-advantages of the two types of setting are given below:

Vertical	Horizontal
1) Efficient draft tube	1) Losses at elbow
2) Inaccessibility to turbine parts, and bearing trouble.	2) Easy accessibility, and less bearing trouble.
3) Small floor area, but requires more excavation and a higher building.	3) Requires a large floor area.

Considering that floor space is not as important as an expensive foundation, and that easy access to all parts of the turbine is essential, and that turbine-generator unit can be set at the same level without any harm, considering all these points a horizontal setting was chosen.

With the above preliminary data at hand several turbine manufacturers were asked for dimensions of a suitable horizontal reaction turbine.

Only the Fitz Water Wheel Co. of Hanover, Pa., U.S.A. was able to supply a detail drawing for a suitable turbine unit. (4)

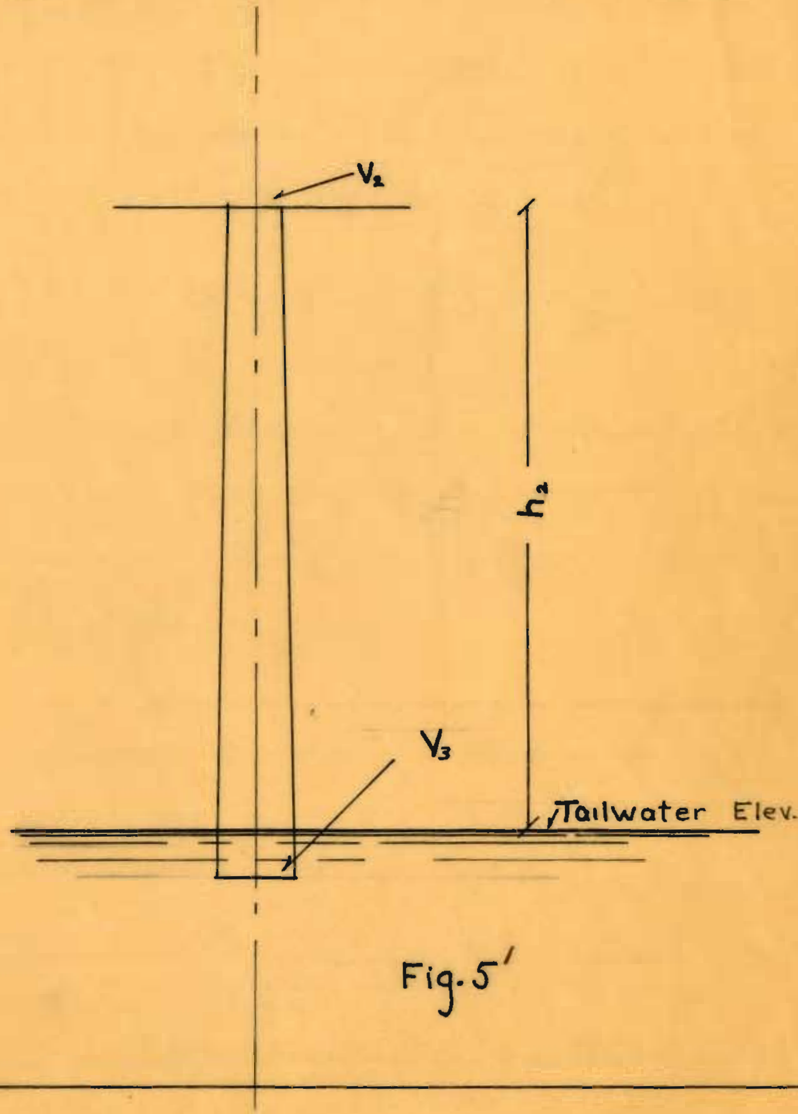

Fig. 5'

draft tube indicated in the detail drawing is too short for the conditions at hand and hence it was lengthened to about 16 ft. with an end submergence below tailwater level of about 2 to 3 ft.(5) That such lengthening of the draft tube is not objectionable can be shown by the following calculations: (see fig. 5′)

$$h_2 = -\frac{P_2}{w} - \frac{\bar{V}_2^2}{2g} + h_f + \frac{\bar{V}_3^2}{2g} \qquad (6)$$

Taking for the entrance diameter 12 in. and and to the exit diameter 22 in. (thus keeping the same flare as for the drafttube submitted by the manufacturer) we obtain the following velocities:

---

(4) 12 in. Fitz-Hanover Turbine (Bulletin No.100 p. 14)

Head = 48 ft.

h.p. = 62.9

Q = 823 ft^3/ min

N = 1121 r.p.m.

<u>It is specified that max. efficiency is obtained at 80% of indicated capacity.</u>

$$V_2 = \frac{Q}{A_2} = \frac{10.6 \times 144 \times 4}{\pi\ (\ 12\ )^2} = 15.1 \text{ ft./sec.}$$

$$\frac{\overline{V_2}^2}{2g} = 3.6 \text{ ft.}$$

$$V_3 = \frac{Q}{A_3} = \frac{10.6 \times 144 \times 4}{\pi\ (\ 22\ )^2} = 4.01 \text{ ft./ sec}$$

$$\frac{\overline{V_3}^2}{2g} = \theta.\ 25 \text{ ft.}$$

h_f = The friction loss plus the velocity head loss at exit, do not exeed 1 ft. as a max.

$\frac{P_2}{W}$ = This cannot be less than the vapour pressure of the water at the tempreture considered and should be at least 2 to 4ft. of water more. A good limit is-25 ft.below atmospheric pressure (this being taken as 0)

Hence,

$$h_2 = -(-\ 25\ ) - 3.6 + 1$$
$$= 25 - 4.6 = 20.4 \text{ ft.}$$

Since the desirable limit to the height of the draft tube is between 15 and 20 ft. the use of the16 ft. draft tube will be safe in the case considered. Moreover, the proportions agree with the recommended velocity ratios at the various points of the turbine.

The above selection cannot be considered final because such selections are made after a wider contact with various manufacturers. However, the turbine mentioned can be considered as suitable for the conditions at hand.

Together with the turbine is supplied a compensating governer, which, according to manufacturer's drawings has a time of closure of $7\frac{1}{2}$ sec. This time was the one used in calculating the pressure rise consequent to full closure of gates.

The switchboard is placed on the generator side for ease in electrical connections. It is also sufficiently far from the wall to allow a workman to go behind and make any necessary repairs.

A permenant crane in the turbine room was not considered necessary because the equipment is rather light and not sufficiently numerous to justify a permenant crane. In case of necessity a temporary crane can be used.

2) Workshop.

Usually a workshop equipment is added to little by little as time goes on. However, a tentative list of the necessary equipment can be made as follows:

1) Drill
2) Lathe
3) Vise
4) Working table
5) Tools & small closets

3) <u>Storage Battery Room</u>

This room is for the accomodation of 6 storage batteries and a rectifier. They are kept charged constantly and are used to light important points in the station(i.e. turbine room, workshop, storage battery room,etc.) at times when the generator is not running.(7)

C) <u>Engine bases</u>

The dimensions of the turbo-generator unit base were made to fit the drawings submitted by the turbine manufacturer. It consists mainly of a mass of concrete, lightly reinforced, on which rests the steel I-beam frame carrying the turbine and the generator. The concrete mass should be carried down to firm ground to prevent future settlement. However, a very large mass is not warranted because vibration is not a serious problem in a turbine. Details of the base are given in plan No. VI.

---

(7) This secondary lighting system being operated at 36 volts, it reqires a seperate wiring system and special lamps.

II. Tailworks

The aim of the tailworks is to lead the water from the point of discharge at the end of the draft tube back to the stream.

This part consists of a canal whose section is chosen according to the same economic principles outlined for the penstock. However, a detailed economic study in this case is not justified because at H. W. level the loss of power due to reduced head becomes appreciable as compared with the friction head loss.

Nevertheless, computations for the economic size of cement-lined rubble masonary canal was made and is given in Appendix H. The final section adopted is 80 x 40 cms. water section. A side thickness of 20 cms is sufficient because the canal is wholly in cut of rock or firm ground. A short wing wall at the junction of the canal with the stream is necessary for protection. The inside of the canal is plastered to a height of 60 cms. above normal water level to take care of high water conditions.

The possibility of using other materials for the canal was investigated but found uneconomical. As to the possibility of substituting a pipe for the canal it was also studied and found uneconomical. This latter variation has , in addition, the disadvantage of being inaccessible.

CHAPTER V

ECONOMIC CONSIDERATIONS

The economic aspect of a hydro-electric development is of vital importance because it determines the feasibility of the project.

In this chapter an attempt is made to estimate the price of one kwh (at the switchboard) of electric energy produced by the system under consideration. Since exact data were not used in design and since an exact price list could not be obtained for the equipment required, the estimate given can at best be taken as an approximation, which, nevertheless, gives an idea as to the economic feasibility of the project.

Price List Used for Estimating the cost of the project (1)

Rubble Masonary (Dabsh) per cubic meter

Stone	L.L. 3.00
Sand	L.L. 1.00
Cement($1\frac{1}{2}$)	L.L. 6.00
Labour	L.L. 5.00
	L.L.15.00

Plaster (9 bags of cement / M^3, without lime)

L.L. 1.50/ M^2

Iron work (in place)

L.L. 1.50 / Kg.

R. Concrete per cubic meter

Cement 6 bags	= L.L. 24
Sand & gravel	= L.L. 6
Steel 60 kgs.	= L.L. 20
Labour	= L.L. 15
	L.L. 65

Excavation

Earth........... L.L. 3.00 / M^3

Rock .... L.L. 4.00 / M^3

---

(1) Obtained from Prof. Yeremian

Rail Pipe

1 in. & 1½ in. ..........L.L. 4.00/ MR

Sand L.L. 3.00 / M^3

Broken stone L.L. 4.00 / M^3

FOREBAY

	Quantity	Unit Price	Total Price
Excavation & removal of loose earth	590 M^3	L.L.3.00	L.L.1770
Excavation in rock	530 M^3	L.L.4.00	L.L.2120
Rubble masonary	235 M^3	L.L.15.0	L.L.3500
Plaster	135 M^2	L.L.1.50	L.L. 210
R.C.	1.00M^3	L.L.65.0	L.L. 65
			L.L.7665

plus 5% accidentals L.L.8000

Interest, insurance & taxes 10%.......... L.L.800

Depreciation 40 years L.L.200

Maintenance 0.5% ........................L.L. 40

Total .... L.L. 1040

Trash Rack	520 Kgs.	L.L.1.50	L.L. 780
1½ in. pipe	18 MR	L.L.4.00	L.L. 72
1 in. pipe	20 MR	L.L.4.00	L.L. 80

Painting of trash rack 8.50 M^2 . L.L.2.00. L.L. 17

L.L. 949

Take as L.L. 950

Interest , insurance & taxes 10% L.L.95

Depreciation 20 years L.L.47.50

Maintenance 0.5% .... L.L. 4.75

L.L. 146.25

Take as L.L. 146

T O T A L for F O R E B A Y ......L.L. 1186 per annum

PENSTOCK

	Quantity	Unit Price	Total Price
Excavation of loose earth fob pathway & pipe bed	70 M^3	L.L.3.00	L.L. 210
Broken stone for drainage	7 M^3	L.L.4.00	L.L. 28
Sand	2.5M^3	L.L.3.00	L.L. 8
24 in. I.D. steel pipe	2.6T	L.L.420	L.L.1100
			110
			52
			35
Internal & external protective coating	47 M^2	L.L.4.00	L.L. 188

Ok

2 in. vent pipe				
with cap	4.50 MR	L.L.4.00	.L.L.	20.
Guy wires	20 MR		L.L.	20
Johnson coupling joint			L.L.	20
Total			L.L.1791	
plus 5% accidentals				
Take as			L.L.1880	

Interest, insurance & taxes 10% .... L.L. 188

Depreciation 25 years .... L.L. 75

Maintenance 2% .... L.L. 34

Total L.L. 297

Annual value of power lost .... L.L. 116

Grand total .... L.L. 413/ year

---

24 in. through valve ................L.L. 700

18 in. through valve ................L.L. 500

L.L.1200

Interest, insurance & taxes 10% ......L.L. 120

Depreciation 35 years .... L.L. 35

Maintenance 2% .... L.L. 24

L.L. 179

Take as L.L. 180

T O T A L for P E N S T O C K = L.L.593/year

POWERHOUSE

	Quantity	Unit Price	Total Price
Building- Foundations			L.L. 3000
Building- Superstructure			
Turbine room	80 M	L.L.65.0	L.L. 5200
South wing	87 M	L.L.120	L.L.10440
	Total		L.L.18640

Interest, insurance & taxes 10% L.L.1864
Govt. tax on buildings...................... L.L. 60
Depreciation 40 years .... L.L. 466
Maintenance 1% .... L.L. 186

Total L.L.2576

Turbine - Generator unit .... L.L. 10,000

Interest, insurance & taxes 10% .... L.L. 1000
Depreciation 20 years ....................L.L. 500
Maintenance 3% .... L.L. 300

Total .... L.L. 1800

6 Storage batteries L.L.480.0

Interest, insurance & taxes 10%.......... L.L. 48.0
Depreciation 5 years .... L.L. 96.0
Maintenance 10% .... L.L. 48.0

Total .... L.L.192.0 / year

Rectifier .... L.L. 40.00

Interest , insurance & taxes 10% ......L.L. 4.00

Depreciation 15 years .... L.L. 3.00

Maintenance 0.5% .... L.L. 0.20

Total .... L.L. 7.20 / year

<u>Workshop equipment</u> .... L.L. 1000

Interest, insurance & taxes 10% .......L.L. 100

Depreciation 20 years .... L.L. 50

Maintenance 2% .... L.L. 20

Total .... L.L. 170 / year

<u>Office</u> ...........................L.L. 200

Interest, insurance & taxes 10% .......L.L. 20

Depreciation 15 years .... L.L. 14

Maintenance 2% .... L.L. 4

Total .... L.L. 38

<u>Annual cost for powerhouse</u>

Building .... L.L. 2576

Turbine -generator unit .... L.L. 1800

Storage batteries .... L.L. 192

Rectifier .... L.L. 7

Workshop equipment .... L.L. 170

Office ..........................L.L. 38

Grand total .....L.L 4783

P O W E R H O U S E Take as .......L.L.4800 / year

TAILRACE CANAL

Excavation in rock	55 M^3	L.L.4.00	L.L. 220
Excavation in loose earth	7 M^3	L.L.3.00	L.L. 21
Rubble masonary	23 M^3	L.L.15.0	L.L. 346
Plaster	32 M^2	L.L.1.50	L.L. 48
	Total		L.L. 635

Interest, insurance & taxes 10% ...........L.L. 63.5

Depreciation 30 years .... L.L. 22.0

Maintenance 2% ...........................L.L. 12.7

Total .... L.L. 98.2

Annual value of power lost ...... L.L. 10.0

TAILRACE CANAL L.L.108.2

Take as 110 / year

LAND

1670 M^2 .................L.L. 500

Interest 6% ..........L.L. 30

OPERATION

Day operator at L.L. 180/ month	L.L.	2160
Assistant at L.L. 60/ month	L.L.	720
Night operator at L.L.120 /month	L.L.	1440
Total	L.L.	4320

Total cost of project

Part I (taken from Mr. Samir Bardawil, B.A.)

Water rights	L.L.	10,180
Structures	L.L.	856
Total	L.L.	11,036

Part II

Forebay	L.L.	1,186
Penstock	L.L.	593
Powerhouse	L.L.	4,800
Tail-race	L.L.	110
Operation	L.L.	4,320
Total	L.L.	11,039

GRAND TOTAL ...... L.L. 22,075

Total amount of electric energy produced

At 65% gross efficiency, available power is 29 kw as indicated in the section on the selection of the turbine.

Hence,

$$\text{Energy} = 310 \times 22 \times 29 = 198{,}000 \text{ kwh}$$

Hence,

$$\text{Cost per kwh} = \frac{22{,}075}{198{,}000} = 0.111$$

$$= \underline{11.1 \text{ ps./ kwh}}$$

CONCLUSION

The price obtained per kwh is quite satisfactory for such a small development, inspite of the unusually high value of the water rights. It should be observed ,however, that the cost obtained depends on the amount of energy consumed annually. If for any reason the annual consumption is less than the amount assumed, the price per kwh will obviously jump up and might even prove uneconomical.

Nevertheless, the results obtained prove that even the smallest of hydro-electric projects might be developed economically, and thus the policy of not loosing even one drop of water is seen to be basically sound.

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A P P E N D I C E S

APPENDIX A

Statistical Tables Taken from L'Electricité au Liban by Mr Omar Ajam- 1950 .

TABLE I

Variation of power production in Lebanon

(1936-1947)

(p. 13)

Year	Thermal Energy	Hydraulic Energy	Total Energy
1936	905 572 kwh.	26 759 701 kwh	27 665 273 kwh
1937	1 345 336 »	31 904 375 »	33 249 711 »
1938	500 871 »	34 959 491 »	35 460 362 »
1939	1 510 157 »	36 479 160 »	37 989 323 »
1940	1 415 206 »	33 009 071 »	34 424 277 »
1941	1 414 916 »	20 562 181 »	21 977 097 »
1942	1 567 421 »	42 238 701 »	42 286 122 »
1943	2 563 025 »	39 723 205 »	42 286 230 »
1944	4 925 280 »	43 652 879 »	48 508 159 »
1945	9 261 240 »	47 935 000 »	57 196 240 »
1946	12 189 000 »	49 261 000 »	61 450 000 »
1947	22 611 556 »	50 272 000 »	72 883 556 »

T A B L E I I

Hydro-electric plants in operation (1948)

(p. 15)

Name of Plant	Installed Capacity kva	Production in 1947 - kwh
Safa	7900	28 345 000
Abou Ali	6800	15 220 000
Kadicha	2040	5 007 000
Kab Elias	170	180 000
Wadi El Araiche	700	1 100 000
Nahr-el-Kalb	200	360 000
Other small Inst.	50	60 000
T O T A L	17860 kva	50 272 000 kwh

T A B L E II

Hydro-electric plants in operation (1948)

p. 15

Name of Plant	Installed Capacity kva	Production in 1947- kwh
Safa	7900	28 345 000
Abou Ali	6800	15 220 000
Kadicha	2040	5 007 000
Kab Elias	170	180 000
Wadi El Araiche	700	1 100 000
Nahr-el-Kalb	200	360 000
Other small Instal.	50	60 000
T O T A L	17 860 kva	50 272 000 kwh

T A B L E III

Probable Hydro-electric Developments

Till 1955

(p. 17)

Name of plant	Installed Capacity	Usable Energy Million kwh	Approx. date of completion
Nahr Ibrahim	4160	20	1949
Al-Bared	9000	22	1952
Nahr-el-Joz	6000	15	1951
Kadich II	7000	17	1953
Taibe(Litani)	4000	6	1954
T O T A L			

T A B L E II

Hydro-electric plants in operation (1948)

p. 15

Name of Plant	Installed Capacity kva	Production in 1947- kwh
Safa	7900	28 345 000
Abou Ali	6800	15 220 000
Kadicha	2040	5 007 000
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Wadi El Araiche	700	1 100 000
Nahr-el-Kalb	200	360 000
Other small Instal.	50	60 000
T O T A L	17 860 kva	50 272 000 kwh

T A B L E III

Probable Hydro-electric Developments

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Kadich II	7000	17	1953
Taibe(Litani)	4000	6	1954
T O T A L			

APPENDIX B

Data and Measurements on Stream Flow taken At Antelias on Sept. 28, 1949

Main Stream				South Canal				North Canal			
l	h	S	t	l	h	S	t	l	h	S	t
2.50	0.55	8.00	30	0.80	0.37	17.00	45	1.0	0.40	20.0	42
	0.50		33								
	0.45		40								
	0.40		86								
	0.30		42								
	0.27										
	0.27										

l = width of channel , meters.

h = depth of water , meters

S = distance covered by float , meters

t = time to cover distance S , seconds

North Canal

$$Q = A\ V$$

$$A = 1.00 \times 0.40 = 0.40\ M^2$$

$$V = \frac{S}{t} = \frac{20.0}{42} = 0.476\ M/sec$$

To get the mean velocity in the middle vertical,

$V = 47.6 \times 0.85 = 40.4$ cms. /sec or 0.404 M/sec (1)

$Q = 0.40 \times 0.404 = \cancel{0.128}\ 162\ M^3/sec$

South Canal

$A = 0.80 \times 0.37 = 0.296\ M^2$

$V = \frac{17.00}{45} = 0.378$ M/sec

V mean $= 0.378 \times 0.85 = 0.321$ M/sec

$Q = 0.296 \times 0.321 = 0.095\ M^3/sec$

Main stream

$A = 2.50 \times 0.40 = 1.00\ M^2$ (2)

$V = 8.00 / 42.6 = 0.188$ M/sec

V mean $= 0.188 \times 0.85 = 0.160$ M/sec

$Q = 0.160 \times 1.00 = 0.160\ M^3/sec$

T O T A L F L O W : North Canal= $0.162\ M^3/sec$

Main Stream= $0.160\ M^3/sec$

$0.322\ M^3/sec$

Take: $0.30\ M^3/sec$

---

(1) Elementary Surveying Vol. II - Breed and Hoshmer p. 544

(2) It was observed that the water in the main stream became stationary at about 2.50 m from the left bank. Hence the effective width of the stream was taken as 2.50 meters.

A P P E N D I X   C

Procedure to be followed for the determination of the economic size of penstock

1) Determine the possible materials from which the pipe can be made(i.e. Steel, Cast Iron, R. Concrete, pre-stressed Concrete, wood, etc.).

2) For each material and for at least three different diameters, estimate, as closely as possible, the annual cost of pipe-- interest, depreciation , and maintenance-- and the annual value of power lost through friction and other minor losses.

3) For each material plot the two curves of value of power lost and cost of pipe and add them together to obtain the curve of total annual cost. The lowest point om this curve will determine the most economic diameter and the corresponding annual cost for the specific material considered.

4) Compare the most economic costs thus obtained for the different materials concerned and make a final choice as to the material and diameter to use. The final choice is not necessarily determined by the lowest cost obtained but by practical considerations.

APPENDIX D

Cast Iron Penstock

Advantages : Long life; can stand external pressure efficiently.

Disadvantages: First cost ; weight ; high freight rates; tuberculation by soft waters ; liability to corrosion by electrolysis ; external corrosion by acidic soils; under usual specifications a waste of material may be possible ; cast-iron pipe specials raise the cost very much.

Design . Since cast iron pipes can carry the external pressure effáciently their design involves investigation both for internal bursting pressure and external load.

Data assumed for design

Allowable tensile stress= 5000p.s.i.

Static head 35 ft. (15 p.s.i.)--- including any accidental allowance for water hammer.

Calculated thickness to be increased by 0.16 in. for foundary tolerance and corrosion allowance. (1)

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(1) Water Supply Engineering - Babbit and Doland p. 268

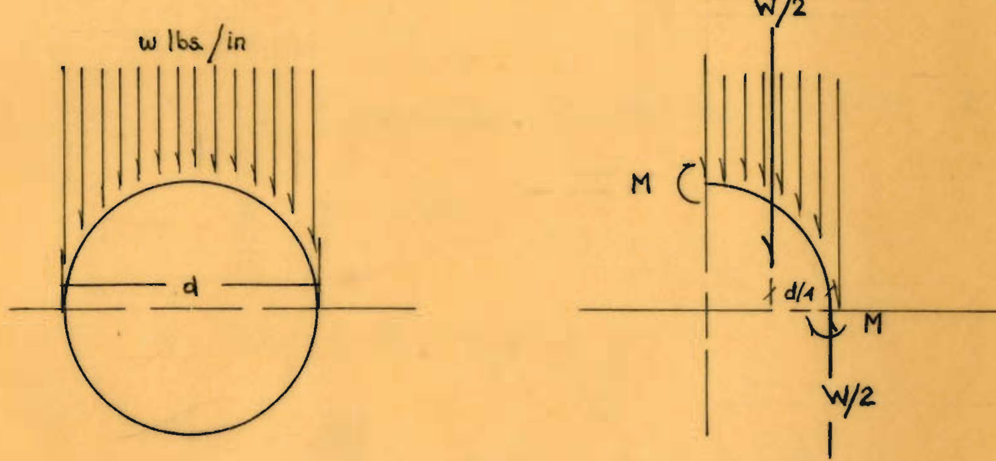

Fig. 5''

At least 2/3 weight of refill acts as uniformly distributed vertical load on pipe. (2)

Design Formulae

a) $S = \frac{P \times d}{2t}$

P= internal pressure, p.s.i.

d= internal diameter, in.

t= thickness of pipe, in.

S= tensile stress, p.s.i.

b) Stress due to backfill (see fig. 5")

$\frac{W}{2} \times \frac{d}{4} = 2M$

Hence, $M = \frac{W\,d}{16}$

$$S = \frac{Wd}{16} \times \frac{t}{2} \times \frac{12}{t^3} + \frac{W}{2t}$$

$$= 3/8\,\frac{W\,d}{t^2} + \frac{W}{2t}$$

W = external load per in.

Design of pipe thickness

The above formulae will be used to

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(2) Waterworks Handbook - Flinn, Weston, & Bogert p. 384

calculate the necessary thickness of pipe for various diameters.

18 in. Diameter

Internal pressure:

$$t = \frac{15 \times 18}{2 \times 5000} = 0.027$$

$$\begin{array}{r} 0.16 \\ \hline 0.187 \end{array} \quad \text{say } 0.19 \text{ in.}$$

External load :

$$t^2 = 3/8 \times \frac{W \times 18}{5000}$$

$$W = \frac{4 \times 18}{12} \times \frac{1}{12} \times 100 \times 2/3 = 33 \text{ p.s.i.}$$

$$t = 0.21 \text{ in.}$$

Use standard thickness of 0.63 in. (3)

Proceeding in the same manner the thicknesses for the other diameters are determined as indicated below;

22 in. Diameter

Required for internal pressure: 0.20in.

Required for external load : 0.257in.

Use standard thickness of 0.68 in.

24 in Diameter

Required for internal pressure: 0.20in.

Required for external load: 0.28 in.

---

(3) Water Supply Engineering - Babbit and Doland p. 289

Use standard thickness of 0. 74 in.

Comparative cost study for various diameters

Data assumed

Price = L.L. 210 / ton CIF Beirut

Life = 40 years

C = 85

Hemp = 480 kgs/ m^3

L.L. 2.50 / kg.

Lead = L.L. 0.75 / kg.

Length of penstock = 90 ft.

I-- 22 in. I. D. pipe (56 cms) t= 0.68 in.

Cast Iron

$$\frac{\pi \times 22.6 \times 0.68 \times 90 \times 450}{144 \times 2.2} = 6.16 \text{ tons}$$

at L.L. 210 = L.L. 1290

Breakage and taxes 15% = L.L. 194

Transportation

2- Trucks ..................L.L. 40

Labour 10 men at L.L. 4 ....L.L. 40

Excavation

1.0 x 2.0 x 15 = 30 M3

at L.L. 3.00= L.L. 90

Coating

$$\frac{\pi \times 60 \times 27}{100} = 51 \text{ MS}$$

at L.L. 5.......L.L. 255

Jointing

Yarn $\pi(56+5) \times 1.0 \times 5.0 = 960$

$\div 10^4 = 0.096$

$\times 4.8 = 0.46$ kgs/joint

at L.L. 2.50= L.L. 1.15

Lead $\pi 61 \times 1.0 \times 7.0 = 1340$ cm.

$\div 10^4 = 0.134$

$\times 113.50 = 15.2$ kgs/joint

at L.L. 0.75

= L.L. 11.40

Labour

L.L. 10.

L.L. 5

L.L. 15 for 2 joints

Hence, L.L. 7.50 per joint

TOTAL PER JOINT = L.L. 20

x 5 = L.L. 100

Transport and porter = L.L. 10

Valves

2- Through valves at L.L. 600

= L.L. 1200

Grand Total .... L.L. 3219

<u>Annual Cost of pipe</u>

Interest 6%

Taxes & Insurance 4% ..................L.L. 322

Deprèciation (40 years) 2.5% .... L.L. 80.5

Maintenance 1% .... L.L. 32.2

L.L. 434.7

<u>Take as L.L. 450</u>

<u>Annual Value of power lost</u>

$Q = 10.6\ ft^3 / sec$

$D = 1.83$ ft.

$V = 4.01$ ft./sec

$A = 2.64\ ft^2$.

$C = 85$

Price of electric power = P.S. 10 per Kwh.

Using Hazen and William's Formula,

$$H = \frac{3030\ V^{1.85}}{C^{1.85}\ D^{1.17}}$$

$H = 5.3$ ft. / 1000 = 0.476 ft. / 90 ft.

at 70% efficiency,

$H = 0.333$ ft./ 90 ft.

$$\frac{62.5 \times 0.33 \times 10.6}{550} (0.746)(310) \times \frac{10 \times 10}{100} = \underline{L.L.\ 92.5}$$

Following the above procedure identically, a similar study was made for the other two diameters. Only the final results are given below:

<u>18 in. I.D. pipe</u>

Annual cost of pipe ...........L.L. 350

Annual value of power lost.... L.L. 246

<u>24 in. I.D. pipe</u>

Annual cost of pipe........... L.L. 500

Annual value of power lost ... L.L. 61.4

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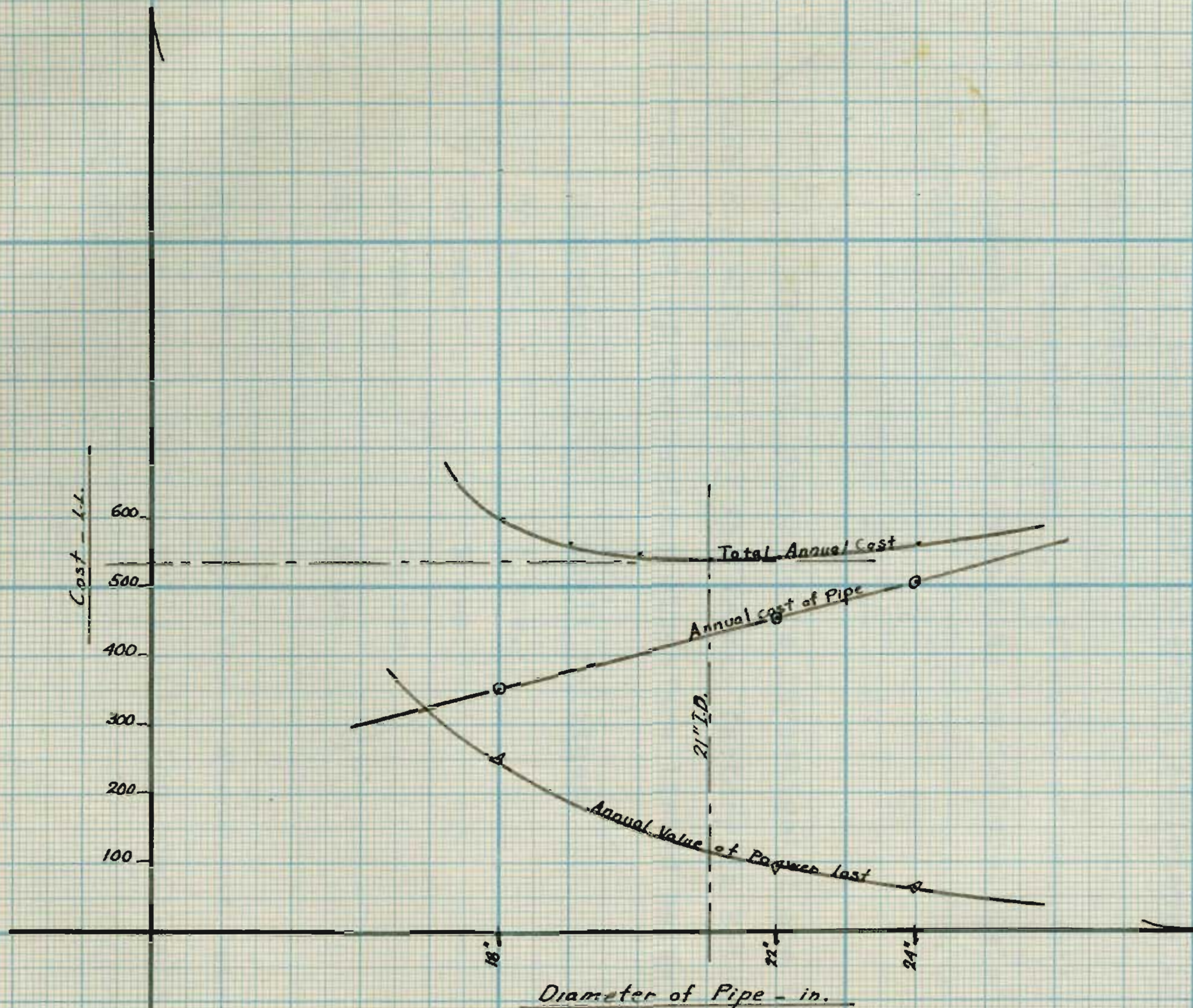


Economic Size For Cast Iron Penstock

APPENDIX E

R.C. Penstock

Advantages : Sustained carrying capacity; durability ; use of local material; efficiency in sustaining backfill pressure.

Disadvantages : Difficulty of connections; development of cracks resulting in failure; deterioration in alkali soils.

The R.C. pipes to be investigated are those cast centrifugally by the Araman factory in Beirut, Lebanon. Price list of the various sizes are listed below:

Diameter	Shell thickness	Price/MR
30 cms.	4 cms.	L.L. 16
40 cms.	5 cms.	L.L. 24
50 cms....	$5\frac{1}{2}$ cms.	L.L. 30
60 cms.	6 cms.	L.L. 38

Note: Pipes are desiged for 1 Atmos. of internal pressure-- which fits exactly with the conditions of the project.

I-- 30 cms. I.D. pipe

R. Concrete

27 MR at L.L. 16 per MR = L.L. 430

Breakage 10% = L.L. 43

Transportation

1- Truck .... L.L. 20

Labour 8 men at L.L. 4 .......L.L. 32

Excavation

30 MC at L.L. 3 .... L.L. 90

Jointing (Cement mortar at L.L. 37/MC)

10 joints at L.L. 0.15/joint.. L.L. 1.50

Labour L.L.15

Valves

2 -- through valves at L.L. 500

= L.L. 1000

Grand Total .... L.L. 1632

Annual Cost of Pipe

Interest 6%

Taxes and Insurance ...........L.L. 163

Depreciation (20 years) 5%...L.L. 82

Maintenance 1% .... L.L. 17

L.L. 262

Take as L.L. 275

Annual Value of power lost

$Q = 10.6\ ft.^3 / sec$

$d = 1.0\ ft.$

$V = 13.5\ ft./sec$

$H = 3.86\ ft./\ 90\ ft.$ (by Hazen and William's)

at 70% efficiency = 2.70 ft./ 90ft.

$$\frac{2.70 \times 10.6 \times 62.5}{550} \times 0.746 \times 310 \times \frac{10 \times 10}{100} = \text{L.L. } 755$$

Using identically the same procedure as indicated above, similar studies were made for the other diameters. Only the final results are given below:

40 cms. I.D. pipe

Annual cost of pipe ......L.L. 320

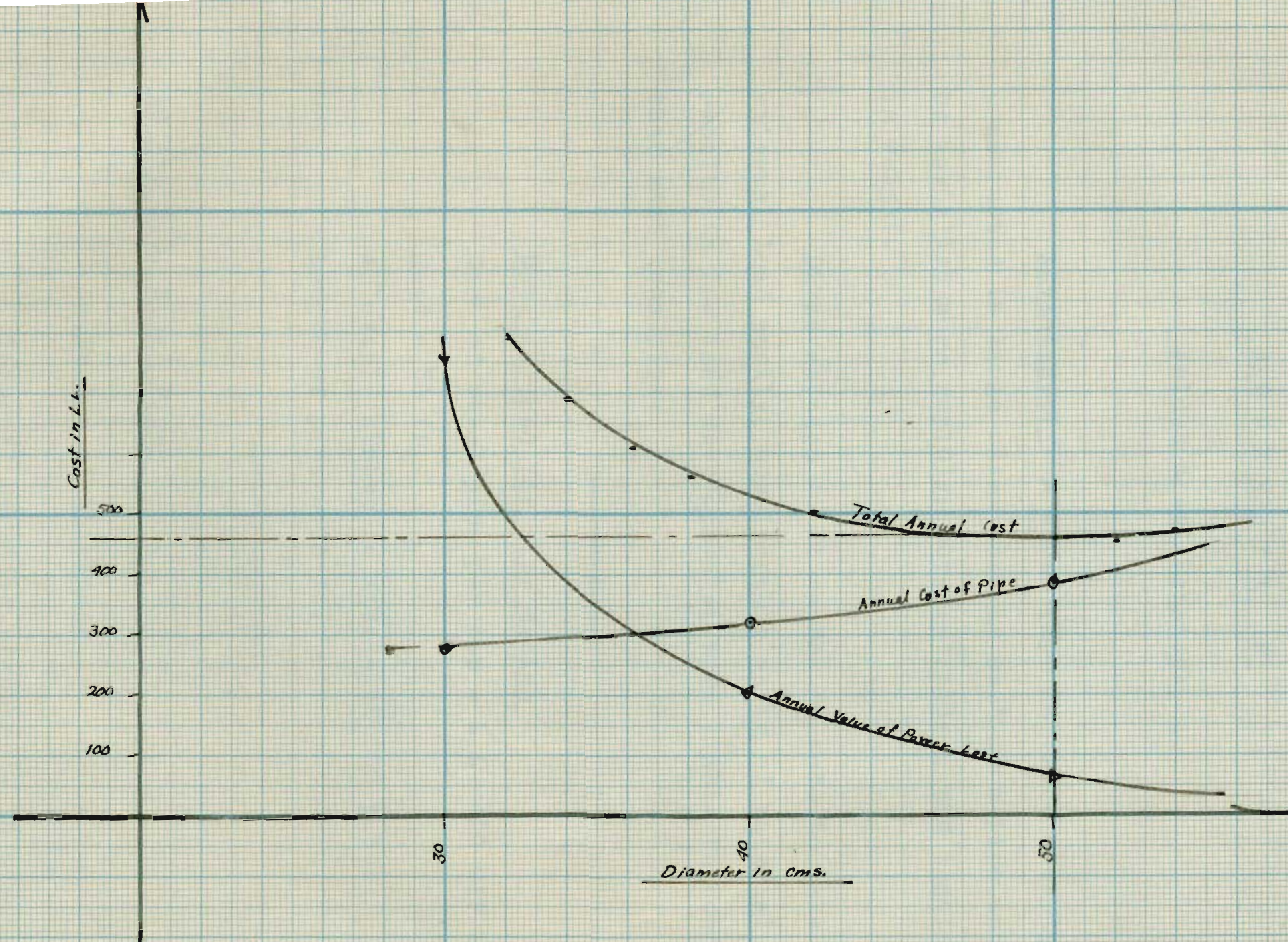
Annual value of power lost......L.L. 204

50 cms. I.D. pipe

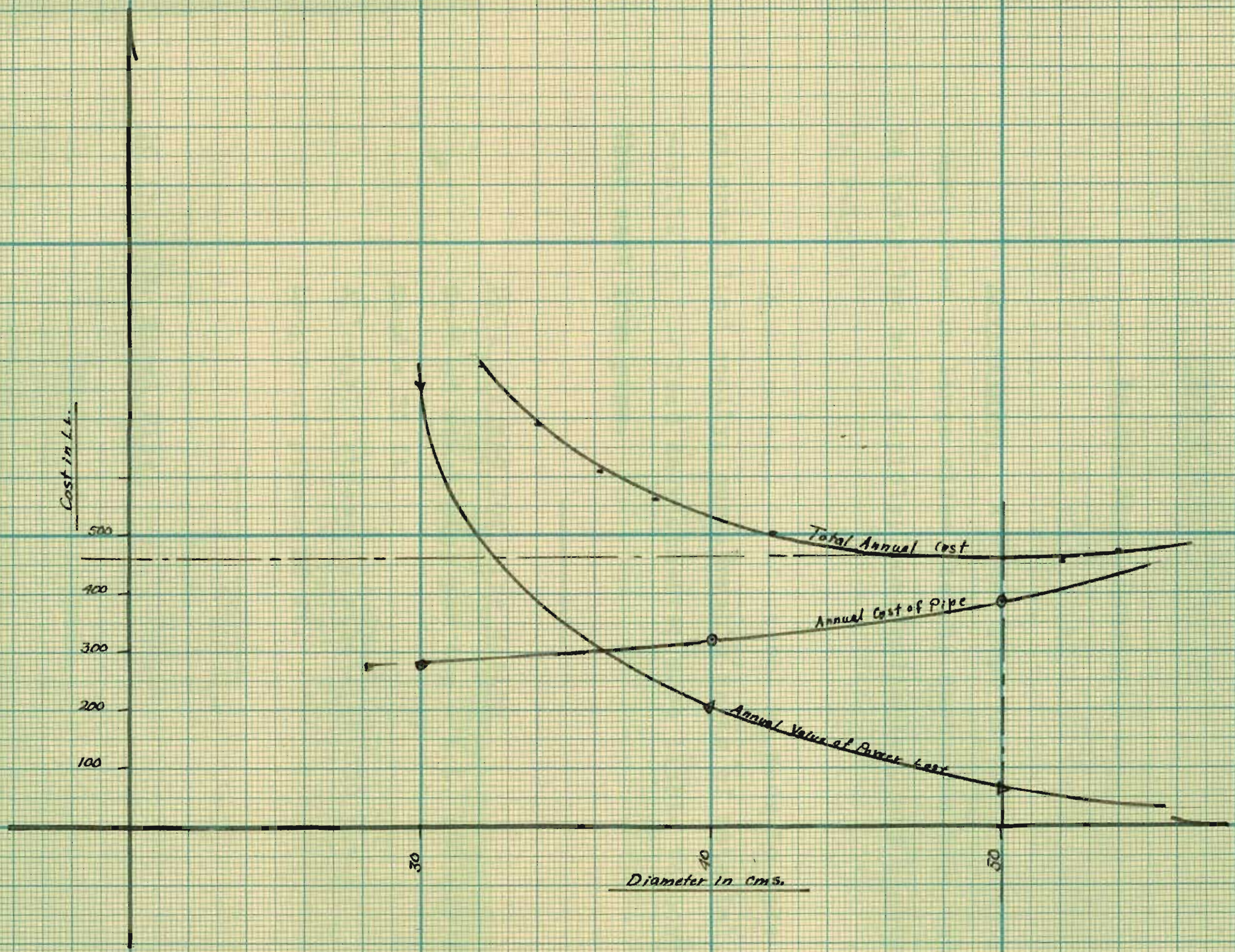
Annual cost of pipe ......L.L. 385

Annual value of power lost......L.L. 70

In all these calculations no consideration was given to the special and complicated connections required at valves, air vent and reducer. These will add considerably to the cost of the penstock.



Economic Size for R. Concrete Penstock



Economic Size for R.Concrete Penstock

APPENDIX F

Welded Steel Penstock

Advantages: A steel pipe is more cheaply constructed in large sizes than is cast iron pipe and can be installed more rapidly; more reliable , and lighter in weight.

Disadvantages : It cannot be made to take the external load easily; considerable waste of material results in low head developments; more liable to corrosion than cast iron pipe.

Data assumed for pipe design

Cost of steel pipe per ton = L.L. 420 CIF Beirut.

Head = 35 ft.

Allowable stress in steel= 16 000 p.s.i.

Length of pipe = 90 ft.

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16 in. I.D. Steel Pipe

Design of thickness

Internal bursting pressure = 15 p.s.i.

Hence,

$$t = \frac{15 \times 16}{2 \times 16000} = 0.0075 \text{ in.}$$

Use minimum allowable thickness = 0. 25 in.

Steel pipe

$$0.25 \times \frac{\pi \times 16.25}{144} \times 90 \times 490 = 3900 \text{ lbs.}$$

2200 = 1.77 tons

at L.L.420 ........L.L. 745

Breakage & taxes 10% ....L.L. 75

Transportation

1- Truck ..........................L.L. 20

Labour 6 at L.L. 4 .................L.L. 24

Excavation

30 MC at L.L. 3 ..................L.L. 90

Welding

$$\frac{\pi \times 41 \times 0.5 \times 8}{40 \times \pi (0.4)^2} = 25.8 \text{ say 26 rods}$$

at 0.70/rod ..L.L. 20

Transport .... L.L. 10

Porter .... L.L. 3

Time loss .... L.L. 3

Coating and painting

$$\frac{\pi \times 42 \times 27}{100} = 36 \text{ MS}$$

at L.L. 5 .........L.L. 180

Valves

2 -- Through valves at L.L. 500.....L.L.1000

Grand total L.L.2170

Annual cost of pipe

Interest 6%

Taxes & Insurance 4% ................L.L. 217

Depreciation(25 years) 4% ...........L.L. 87

Maintenance 2% ....L.L. 43

L.L. 347

Take as L.L. 365

Annual value of power lost

$Q = 10.6\ ft^3/sec$

$C = 90$

$d = 1.33\ ft.$

$A = 1.39\ ft.$

$V = 7.65\ ft./sec.$

Using Hazen and William's Formula,

$H = 22.2\ ft/1000 ft$

$= 2\ ft / 90\ ft$

At 70% efficiency , $H = 1.4\ ft. / 90\ ft.$

Hence,

$$\frac{10.6 \times 62.5 \times 1.4}{550} \times 0.746 \times 310 \times \frac{10 \times 10}{100}$$

$= L.L.\ 392$

Using exactly the same procedure, the cost for the other diameters were similarly calculated. Only the final results are given below:

<u>20 in. I.D. Steel pipe</u>

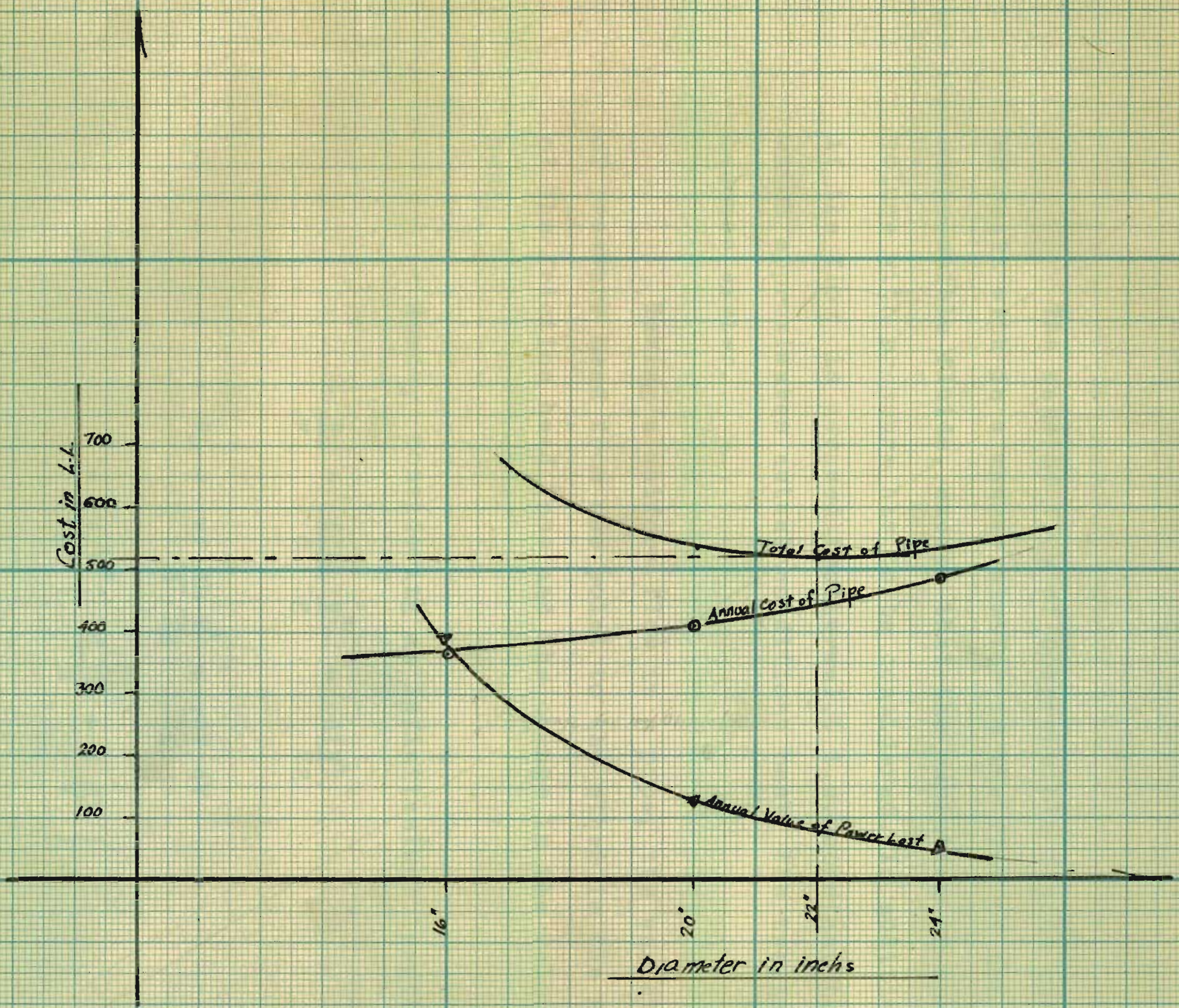
Annual cost of Pipe................L.L. 410

Annual value of power lost........ L.L. 128

<u>24 in. I.D. Steel pipe</u>

Annual cost of pipe................L.L. 485

Annual value of power lost.........L.L. 53



Economic Size for Steel Penstock

w = specific weight lbs./ $ft.^3$

V = velocity ft./ sec.

f = coefficient of friction for particular pipe considered

p = pressure lbs./ $ft.^2$

d = diameter of pipe , ft.

But,

$$pv = RT \quad \ldots\ldots\ldots\ldots 1$$

and

$$W = wVA \quad \ldots\ldots\ldots\ldots 2$$

Hence,

$$v = \frac{RT}{p} \quad \ldots\ldots\ldots\ldots 3$$

$$w = \frac{p}{RT} \quad \ldots\ldots\ldots\ldots 4$$

$$V = \frac{W}{wA} \quad \ldots\ldots\ldots\ldots 5$$

$$= \frac{WRT}{pA} \quad \ldots\ldots\ldots\ldots 6$$

p = pressure , lbs. / $ft.^2$

v = volume per unit weight

R = Universal gas constant

T = absolute temp.

W = weight of fluid flowing

A = cross-section of pipe

Substituting 4 & 6 in the original equation,

$$-dp = \frac{p}{RT}\left(\frac{WRT}{pA}\right)^2 \frac{1}{2g}\left(\frac{f\,dL}{d} + 2\frac{dV}{V}\right)$$

$$- \left(\frac{2gA^2}{WRT} \right) \int p \, dp = \int f \frac{dL}{d} + 2 \frac{dV}{V}$$

Hence,

$$\overline{P}_1^2 - \overline{P}_2^2 = \left(\frac{fl}{d} + 2 \log_e \frac{V_2}{V_1} \right) \frac{W^2 R T}{g A^2}$$

P_1 = External pressure lbs./ ft.

P_2 = Internal pressure lbs./ ft.

The second term in the above equation is small and it may be omitted with but small error. (2)

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Hydraulics-- Daugherty p. 254

$$-\left(\frac{2gA^2}{WRT}\right)\int p\,dp = \int f\,\frac{dL}{d} + 2\,\frac{dV}{V}$$

Hence,

$$\overline{P}_1^2 - \overline{P}_2^2 = \left(\frac{fl}{d} + 2\log_e\frac{V_2}{V_1}\right)\frac{W^2 R\,T}{g\,A^2}$$

P_1 = External pressure lbs./ ft.

P_2 = Internal pressure lbs./ ft.

The second term in the above equation is small and it may be omitted with but small error. (2)

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Hydraulics-- Daugherty p. 254

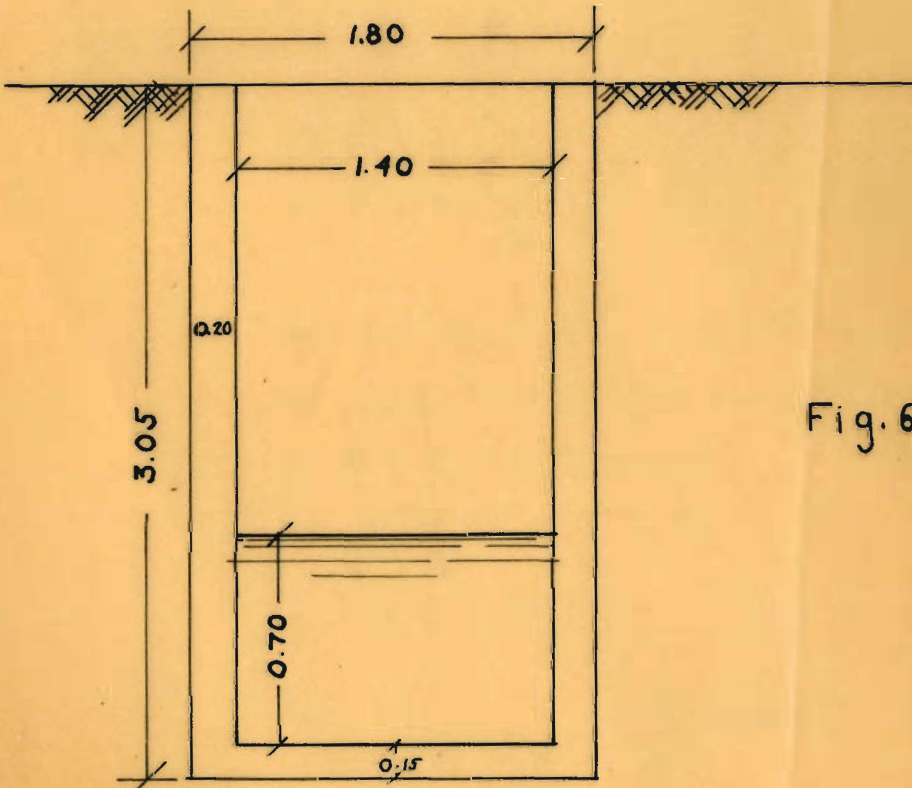

Fig. 6

A P P E N D I X   H

Calculations for the economic size of cement-lined rubble -masonary tailrace canal

Note: A rectangular water section in the canal is taken.

I-- Assume a velocity of flow of 1 ft./sec.

$V = 1$ ft. / sec.

$s = \text{slope} = \dfrac{V^2}{c^2 r}$ $\qquad c = \dfrac{1.486}{n} \times (r)^{1/6}$

$Q = 10.6$ ft.^{3}/ sec. (0.3 M^3/ sec.)

$A = \dfrac{10.6}{1} = 10.6$ ft^2. $= 2\ D^2$ ($b = 2D$)

$D = 2.3$ ft. (70 cms.)

$r = \dfrac{A}{(2D + b)} = \dfrac{D}{2} = 1.15$

$c = 114 \times (r)^{1/6}$

$s = \dfrac{1}{(114)^2 \times 1.20} = 0.000064$

Materials for construction (see fig. 6)

Excavation

2.00 x 3.90 = 7.80 at L.L. 4.........L.L. 31.2

Rubble Masonary

7.70 x 0.20 = 1.54

1.40 x 0.15 = 0.21

1.75 MC at L.L. 15....L.L. 26.2

Plaster

4.00 x 1.0 = 4.0 MS at L.L. 2.0

= L.L. 8

Grand total .... L.L. 65.4

Annual cost of canal per meter length

Interest 6%

Insurance & taxes 4% ......................L.L. 6.54

Depreciation (30 years) ................L.L. 2.18

Maintenance 1% .... L.L. 0.65

Total .... L.L. 9.37

Annual value of power lost

$$70\% \times \frac{0.3 \times 1000 \times 0.000064}{1000} \times 9.8 \times 310 \times \frac{10 \times 10}{100} =$$

L.L. 0.041

Following the same procedure as above, the cost fob the other assumed velocities were calculated. Only the final results are given below:

2 ft. / sec. velocity

Section : 1.00 x 0.50

Annual cost of canal/ meter ....L.L. 8

Annual value of power lost..... L.L. 0.26

3 ft./ sec. velocity

Section : 0.80 x 0.40

Annual cost of canal / meter.... L.L. 7.20

Annual value of power lost. .... L.L. 0.77

---

4 ft. / sec. velocity

Section : 0.70 x 0.35

Annual cost of canal/ meter.... L.L. 6.45

Annual value of power lost......L.L. 1.67

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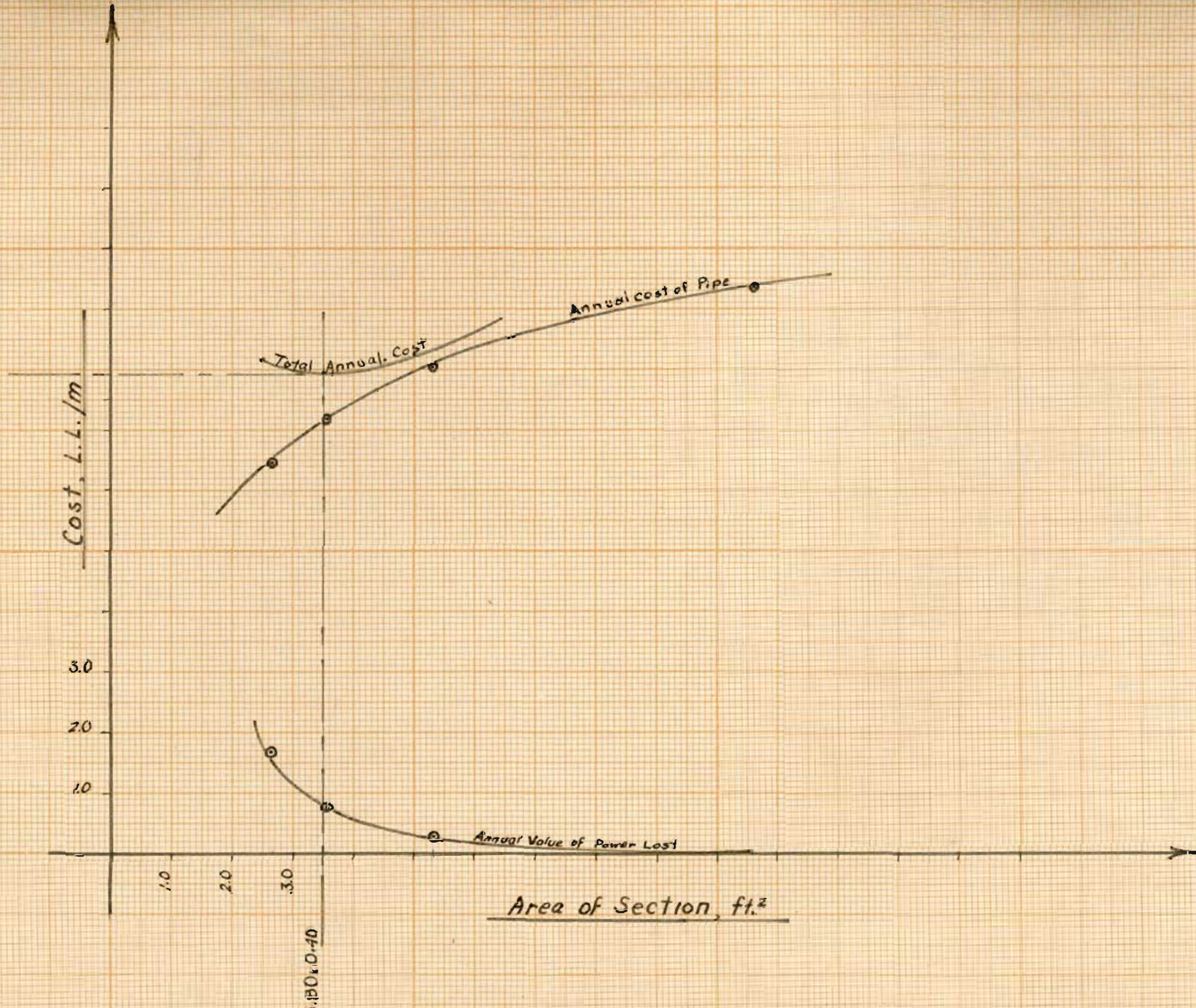

Economic Size for Cement-lined Rubble Masonary Tail-race Canal

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