HYDRO ELECTRIC DEVELOPMENT

OF

FAWWAR (SPRING) ANTELYAS

ANTELYAS LEBANON

PART I

BY

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MAY 1950

BEIRUT LEBANON

Epsn 84

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

O F

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

A N T E L Y A S - L E B A N O N

P A R T   I

D A M - H E A D W O R K S - C A N A L

Thesis presented by

SAMIR GEORGE BARDAWIL

May 1950

" This thesis submitted to the Civil Engineering Faculty in Partial fulfillment of the requirements for the Degree of Bachelor of Science in Civil Engineering." A.U.B.

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

The Candidate herein would like to express his deep appreciation and gratitude to Prof~~f~~essor F. Antippa for his valuable assistance throughout the preparation of this thesis. He is also thankfull to Prof~~f~~essor K. Yaramian and Prof~~f~~essor Rubinsky for their helpfull advice~~s~~ and suggestions.

TABLE OF CONTENTS

FIGURES & DRAWINGS

Figures included in the text:

represent ?

INTRODUCTION

The subject of this thesis was suggested by the increasing need of our country for industrial development. The essential of all industries is an adequate provision for energy to drive its machinery. Therefore the problem becomes how to get and supply this energy: This is done by various ways.

One of the most common forms of power is Electricity and one way to generate it is by Water power. Therefore, as the title of this thesis indicates, we will be trying hereafter to design a small hydroelectric system to generate the necessary power to supply a private electrochemical plant; the design of this plant is not included in the project which is divided into two parts: each part is a thesis by itself. Part I is this thesis dealing with the Dam, the Headworks and the Canal. Part II is a different thesis. (1)

Lebanon is full with hydroelectric possibilities one of which is at "Fawar Antelyas", a very convenient place for such a small project.

cap (?)

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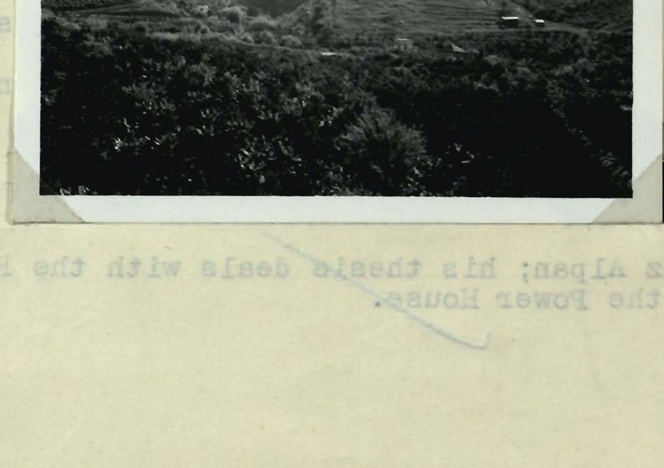
(1) by Mr. Yavuz Alpan; his thesis deals with the Reservoir, the Penstock and the Power House.



CHAPTER I

SITE & TOPOGRAPHY

(a) Description.

"Fawar Antelyas" is a well known spring situated at about 7 km. north of Beirut, and 2 km. from the sea shore. Water springs out from the ground between two rocky ridges and finds its way down to the sea into a shallow stream meandering into orange and banana plantations. At the source is a coffee-house surrounded by trees. Shade, continuous flow of water, and the beauty of the landscape make of the coffee-house a place frequented regularly all through the year.

At a small distance downstream from the source, right in the center of the café, is a drop of some 3 meters in the bed of the stream: An ideal place for a dam, especially that in the following 300 meters downstream the general slope of the land is relatively steep, allowing a drop of about 15 meters between the dam crest and the river bed at that point.

The only use made of the water at present is to irrigate the nearby orchards, and to drive a mill.

(b) Economical Considerations.

Considering that all the water of the source is required for power during summer time, and that summer is the only season of the year in which the water is used for irrigation, the result is that all the land between the dam and the power house will have to be deprived of its irrigation rights. The limit of this non agricultural land in the cross wise direction stops at the contour line of the water level behind the dam. The area of this parcel of land is estimated to be 50,000 m^2. Supposing the net income to be 300 L.L. per 1,000 m^2 per year (from the agricultural point of view), the total income of the land will be 15,000 L.L. per year and with an interest of 10% per annum the total sum to purchase the water rights of the 50,000 m^2 is 150,000 L.L.

This consideration does not include the mill which is run by part of the source water at some distance downstream; water being conveyed in a concrete canal which will be designated hereafter as the north canal. It is obvious that the mill has to stopped during the dry season which usually extends over a period of five months from June to October. The indemnity to be paid in compensation for this idleness is worked out according to the following data obtained from a dependable source:

Wheat milled per day = 300 kg. at 5 P.L. per kg. net

Working days per month = 26

Total income in 5 months = 5 x 26 x 5 x 300 = 1950 L.L.

With an interest of 10% per annum, the sum to be paid for the mill is : 10 x 1950 = 19,500 L.L.

Total initial sum to be paid = water rights + indemnity for the mill over a period of 5 months:

150,000 + 19,500 = 169,500 L.L.

CHAPTER II

HYDROLOGY

The essential data required in a hydroelectric project are as follows:

1- Monthly record of flow over a period of years, the longer the better; the necessity of this is to estimate the average stream flow, and its distribution during the year.

2- Minimum or low water; this will be the basis for the amount of dependable power.

3- Maximum or flood; this will govern the design of the dam and the provision for an adequate spillway.

The above mentioned are the minimum required data.

The following are the data available:

1- No record whatsoever concerning the monthly flow over a period of years.

2- Measurements taken in September 1949[(1)] showed that the total flow was a little more than 0.30 m^3 per second.[(2)] (In September-october the flow of almost all sources is considered to be a minimum for the year, being due exclusively to ground water.) Therefore a discharge of 0.30 m^3 per second was assumed to be permanently available, thus limiting the output of the plant to the net power produced by this quantity of water.

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(1)- By Prof. Antippa, Y. Alpan, S. Bardawil
(2)- See appendix A

3- The maximum flow of the river and discharged by the spillway depends upon the ground water and also and mainly upon surface water.

Unfortunately no records of flood flow are available so that a flow probability diagram could be drawn. For such a diagram a value for flood magnitude for the design can be selected depending upon a number of considerations, the most important of which are:

1- Use made of the water

2- Importance of plant

3- Cost of the structure

The discharge actually used in the design was based on a measurement taken in March 1950(1) giving a total flow of about 4.0 m^3 per second.(2) However this discharge could not be taken as the maximum which was estimated to be 5.0 m^3 per second upon informations given by the owner of the café concerning the level reached by the river during winter.

Therefore the flow used in the design of the spillway was 5.0 m^3 per second.

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(1)- By Y. Alpan, S. Bardawil
(2)- See appendix A

C H A P T E R III

S T R U C T U R E S

This chapter deals with the design of the structures involved in the first Part of the project namely the Dam, the Headworks and the Canal.

A- The Dam

a- Location and dimensions:

For the reasons stated in chapter I, the dam will be erected at the coffee-house thus adding to the landscape the beauty and noise effect of the falling water. There the rock is flush with the river bed thus allowing easy foundations and preventing any uplift. Also a good anchorage of the sides of the structure will be warranted by the rocky banks, which are some 9 m. apart. The hight of the dam will be about 3 m. from base to crest.

b- Type:

Since only part of the available water is going to be used for power, the dam will obviously be of the overflow gravity type, the gravity type being the most economical for the conditions at hand: small hight, rock foundation.

To fix the type we still have to decide upon the material for construction: whether rock or plain concrete.

The former is available in any size and quantity at the site thus making its use more economical.(1) Only one objection stands against it: the spillway, which has to be of the overflow type for technical and aesthetic reasons, so it is unwise to have an overflow rock fill dam. Therefore concrete has to be used. Therefore the dam will be of the overflow gravity type made of concrete.

In order to give to the structure a more attractive appearance, a roadway will be built over the spillway thus forming a bridge which joins the two parts of the café.(2) The floor of the roadway consists of a slab, supported on beams cantilevered from the piers that divide the spillway. Therefore all parts above the spillway will have to be built of reinforced concrete.

c- The spillway:

1- Head of water on crest

The spillway will be divided into 4 sections of 2 m. each thus making a total length of 8 m.; the sections being separated by small piers supporting the roadway.

The formula used for discharge over a spillway, considering the velocity of approach is:

$$Q = m\, l \sqrt{2g}\,(h + h_a)^{3/2}$$

where:

Q = total discharge, 5 m^3 per second.

m = a coefficient, for round crested spillway = 0.4

l = length of spillway = 8 meters.

(1)- Although economy here doesn't have much value because of the small scale project.
(2)- Two bridges, rather unsightly, already exist; the projected one will fall in between the two.

h = height of water above crest measured a distance upstream

h_a = head due to velocity of approach = $\frac{v_a^2}{2g} = \frac{1.5^2}{2g} = 0.115$ m. (1)

therefore:

$$\frac{5}{0.4 \times 8 \times 4.43} = h + 0.115$$

$$\text{and } h = 0.38$$

It can be proved that for a maximum discharge $d = \frac{2}{3} h$ (2)

where d = height of water above crest. Therefore, $d = \frac{2}{3}\, 0.38 = 0.254$ m

2- Shape of crest and downstream face

It can be prooved that the center line of the sheet of falling water over the crest follows a parabolie curve whose equation is $y = \frac{k}{h} x^2$, where k = constant. This equation is based on the average velocity of the water cross section at the crest of the dam. Therefore in order to prevent the formation of a vacuum between the structure and the sheet of water, and to minimise erosion, the crest and downstream face will be shaped to a curve that enters well into the lower nappe of water.

This curve will be constructed according to Table I (1) which is based on experiments by Warnock, (3) taking into consideration the velocity of approach.

To prevent the impact of the water on the foundation a reverse curve will be made at the lowest end of the downstream face, thus discharging the water in a direction parallel to the stream bed. The radius of this deflection curve will be taken

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(1)- See appendix A

(2)- C. F. Flamant, Hydraulique.

(3)- J.E. Warnock,"A study for the design of crest for overflow dams" thesis submited to the faculty of the Graduate School of the University of Colorado, 1939.

at 2/3 the height that is 2 m.

d- Design of roadway: 1- Slab and parapet

Assuming 10 cm. of reinforced concrete and 2 cm. of wearing surface total thickness 12 cm.

weight of slab..........300 kg./m.^{2}

live load...............300 kg./m.^{2}

total = ...600 kg./m.^{2}

total width = 1.40 m.

net width.. = 1.20 m.

span....... = 2.00 m.

moment.....(1) = $\frac{1}{10}$ x 600 x $\overline{2.0}^2$ = 24,000 kgcm.

required thickness t= 0.038 $\sqrt{24,000}$ = 5.9 cm.;thickness used=10 cm.

required steel section per m. s = 0.024 $\sqrt{24,000}$ = 3.72 cm.^{2}

use per m. length 5- 12 mm. ∅ (s = 5.65 cm.^{2})

temperature and parapet reinforcement use 10 mm. ∅ spacing 30 cm. c. to c. will be bent up vertically at each side to go into 10 cm. parapet, the longitudinal reinforcement of which will be 10 mm. ∅ at 20 cm. c. to c.

hight of parapet above slab = 0.90 m.

volume of reinforced concrete used in one span:

slab 0.12 x 1.4 x 2.25 = 0.378 m.^{3}

parapet 2 x 2.25 x 0.9 x 0.1= 0.405 m.^{3}

0.783 m.^{3}; in 4 spans = 3.12 m.^{3}

The intake of the canal situated on the south bank is separated from the left end of the dam by a rocky plateform 1.25 m. wide. (The intake itself is cut in rock and is 1.50 m. wide.)

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(1)- For all formulas used in design of reinforced concrete, C.F. Dunod, Béton Armé page 100-122, 19th edition, year 1948.

The bridge will extend over the plateform and the canal, thus the total length of the roadway will be 9.00 + 1.25 + 1.50 = 11.75 m.

No floor is required for the plateform, the parapet only being prolonged.

required volume of parapet over plateform:

$(0.9 \times 0.1 \times 1.25)\ 2 = 0.225\ m^3$ of reinforce concrete.

slab and parapet over canal intake:

$$\text{volume} \begin{cases} \text{slab: } 0.10 \times 1.4 \times 1.70 = 0.238\ m^3 \\ \text{parapet: } 2 \times 1.70 \times 0.9 \times 0.1 = \underline{0.306\ m^3} \end{cases}$$

$0.544\ m^3$

slab and parapet over dam $\underline{3.120\ m^3}$

total for slab and parapet $3.664\ m^3$ of reinforced concrete

2- <u>Beams and piers</u>

There will be 3 central and 2 end piers 25 x 70 cm. each. From each pier a beam will be cantilevered downstream for a span of 0.70 m. Widths of beams and piers are the same.

<u>Beams</u>: Concentrated load at end due to parapet

0.16 x 2,500 = 400 kg.

load from slab

2.0 x 0.6 x 600=720 kg.

assumed weight of beam = 50 kg.

total load 1170 kg.

Moment due to condentrated load

M = 400 x 0.70 = ...28,000 kgm.

Moment due to distributed load

$M = 770 \times \frac{0.70}{2}$ =..........................27,000 kgm.

total M = 55,000 kgm.

required height of beam at pier

$$H = \sqrt{0.06 \times 55{,}000} = 15 \text{ cm.}$$

section as governed by shear

$$A = \frac{1{,}170}{10} = 117 \text{ cm.}^2$$

Therefore take a beam 25 cm. width x $\begin{cases} 15 \text{ cm. height at pier} \\ 7 \text{ cm. height at end} \end{cases}$

required steel section

$$s = \frac{55{,}000}{0.88 \times 1{,}200 \times 11} = 4.75 \text{ cm.}^2$$

use 3- 16 mm. Ø (s = 6.03 cm.^{2})

bend down 2- 16 mm. Ø, 6 mm. Ø stirrups will be used.

Piers:

The cross-section of the piers is taken as 25 x 70 cm.; while the width 25 cm. is fixed by the length of the dam and that of the spillway, there is no special reason for the choice of the length 70 cm., the only consideration being aesthetics: Half of the roadway will be directly resting on the piers, the other half being cantilevered. Thus each pier can be considered as an excentrically loaded column; the center of gravity of the load falls at a distance of 35 cm. from the center of the pier thus coinciding with its downstream face.

Column formulas for the present conditions are very elaborate, but this is not the reason why they will not be used here. It is because these formulas do not reach to a final answer, and involve a series of trial problems to get the required steel section, this procedure being undesirable and unnecessary for such small load and structure.

It can be shown that there is tension in the upstream face of the pier due to the excentricity of the load;(1) the pier

(1)- See Appendix A

section being determined, the total steel area to be used as a reinforcement will be taken as 1% of the concrete section; this procedure is very questionable but practice has shown that this proportion of steel is safe, for ordinary cases. Here are some figures:

Total load: 2370 kg. eccentricity 35 cm.

Bending moment: $2370 \times 35 = 82,900$ kgcm.

Concrete section: $25 \times 70 = 1,750$ cm^2

Steel section: $\frac{1,750}{100}$ (1) $= 17.50$ cm^2

The concrete section is more than enough to take care of the fiber stress in compression which amounts to 5.40 kg. (2) per cm^2 Therefore more steel will be put in the tension side of the pier; the steel distribution being as follows:

Compresion side of pier: $1/3 \times 17.50 = 5.8$ cm^2

use 3- 16 mm. ø ($s = 6.03$ cm^2)

Tension side of pier: $2/3 \times 17.50 = 11.7$ cm^2

use 6- 16mm. ø ($s = 12.06$ cm^2) in two rows of 3 each; one row will be bent up horizontally to form the reinforcement of the cantilevered beam. The bars are tied by 6 mm. ø at 15 cm. intervals.

Volume of reinforced concrete used:

beams: $4 \times \frac{(0.15 + 0.07)}{2} \times 0.25 \times 0.7 = 0.077$ m^3

piers: $4 \times (0.70 \times 0.25 \times 0.80) = 0.560$ m^3

0.637 m^3

\+ total slab and parapet 3.664 m^3

total for roadway, reinforced concrete 4.301 m^3

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(1)- Although the corners of the piers have to be rounded off the total rectangular section of concrete is used in the design

(2)- See appendix A

Surcharge per m. length of dam.

Weight of 1 m. length of roadway:

slab and parapet: $\frac{3.12}{9}$ x 2,500 = ..865 kg.

pier and beam: $\frac{0.637}{4}$ x 2,500 = ..398 kg.

1,263 kg.

live load: 300 x 1.2 = 360 kg.

1,623 kg.

Weight of water on crest (total h=0.50m.)

0.75 x 0.5 x 1,000 = ... 375 kg.

1,998 kg.

\- Flash boards:

During the dry season of the year all the water is used for power production and no wastes can be afforded.

It is obvious therefore that the flow over the spillway should be stopped thus creating a reservoir behind the dam to take care of the water needs.

A cheap and effective way of cutting the flow is by the use of wooden flash boards.(1)

These will slide vertically into concrete grooves extending all along the upstream faces of the piers. One or two boards can be put in each groove either separately or together allowing in the latter case a total height of 50 cm. of water behind the dam. The boards will be of pine wood which is quite common and strong; size 210 x 25 x 5 cm. each,(2) (length x height x thick.) The boards will be put in place or taken out

(1)- No investigations are made for the possibility of having steel gates. Since it is clear that this is not the right solution for the conditions at hand

(2)- For the design of boards see Appendix A by means of an inverted Y formed with 10 mm. Ø. The ends of the legs are hooked up so they can fit into the upper side of the board.(1) The lifting force (maximum) necessary to remove the lower board is 75 kg.(2) This pull could be developed by one man, but two men may serve the purpose since this maximum pull will have to be exerted twice a year: at the begining of the seasonal rains and on occasional sudden storms.

f- Review of dam section:

The horizontal dimensions of the dam (width of crest and base) have been fixed by the spillway curve. The theoretical(3) base width being determined by the intersection of the base line with the vertical upstream face and with the tangent to the curve at the point: x = 141, y = 140.

The purpose of this article is to find out whether the imposed section complies with the requirements for overturning, sliding and crushing.

Height of dam = 3.00 m. (from base to crest)

Width of base = 2.17 m.

All calculations will be made on the basis of a dam section 1.0 m. in thickness including one pier.

1- Vertical presure:

Weight of surcharge per m. section: W_s = 1,263 kg. stabilizing force; W_s = 1,998 kg. crushing force

W_s will be acting at a distance of 70 cm. from upstream face.

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(1)- See detail drawing Plate IV
(2)- See Appendix A
(3)- Because the actual base extends from 40 cm. behind the heel to the end of the reverse curve added at the toe of dam.

Weight of dam (1 m. section), W_d = 10,150 kg.[1] acting at a distance of 80.44 cm.[1] from upstream face.

Resultant weight:

Stabilizing, W_R = 1,263 + 10,150 = 11,413 kg. acting at a distance of:

$$\frac{(10{,}150 \times 80.55) + (1{,}263 \times 70)}{11{,}413} = 79.0 \text{ cm.}$$

from upstream face.

Crushing, W_R = 1,998 + 10,150 = 12,148 kg. acting at a distance of:

$$\frac{(10{,}150 \times 80.55) + (1{,}998 \times 70)}{12{,}148} = 79.0 \text{ cm.}$$

W_R acting at 79.0 cm. from upstream face.

2- Horizontal pressure:

Earth pressure: It is assumed (and it usually happens) that earth and gravel deposits fill the upstream of the dam up to the crest.

The thrust due to a submerged fill is given by :

$$P_E = K\ (p - 1{,}000)\ H^2 \quad [2]$$

where

$K = 1/2 \tan^2 (45° - \phi/2)$

ϕ = Angle of repose of fill considered = 35°

p = density of fill = 1,500

H = height of fill = 3.00 m.

for ϕ = 35° , K = 1.35

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(1)- See Appendix A
(2)- C. F. Dunod, Béton Armé page 230, 19th edition, 1948

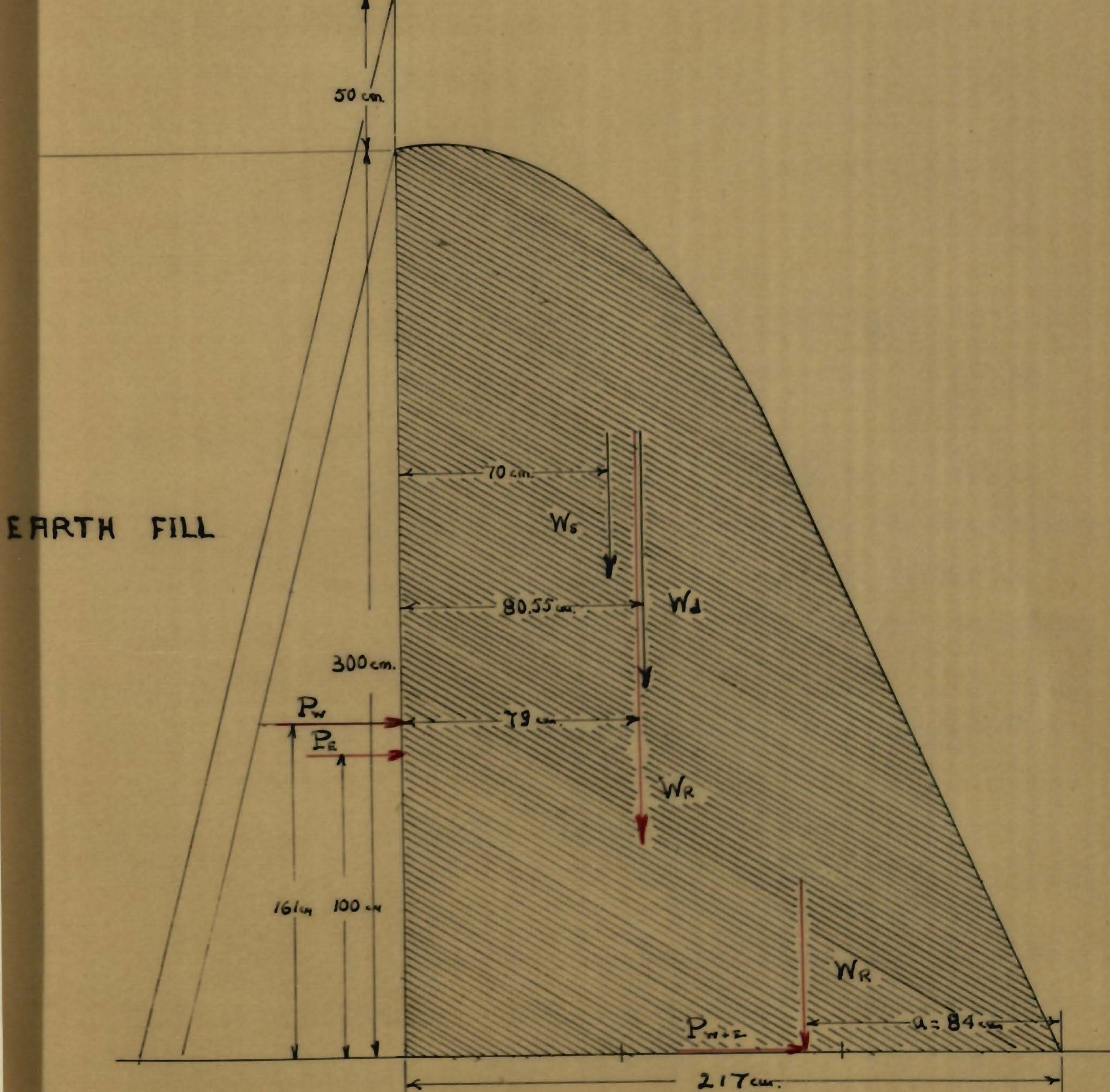

Fig. 1

Diagramatic Section Showing Forces On Dam

$P_E = 1.35\ (1,500 - 1,000)\ 3^2$

$P_E = 607$ kg.

The point of application of this force is at a height H/3 above the base that is 1.0 m.

Water pressure:

$P = 1/2\ w\ H^2$

where $w = 1,000$ kg./m.^{3}

(1)
$H = 3.50$ m.

$P_w = 500 \times 12.20 = 6,100$ kg.

This will also act at a height $H/3 = \frac{3.50}{3} = 1.16$ m.

Total horizontal pressure: 6,100 + 607 = 6.707 kg.

3- Overturning:

Taking moments about the toe of the dam of all the forces acting on the section, the ratio between the stabilizing moment and the overturning moments is called the factor of safety (F. S.) it should not be less than 2 for the structure to be safe.

$$F.\ S. = \frac{11,413 \times (2.17 - 0.79)}{6,100 \times 1.16 + 607 \times 1.0} = \frac{11,413 \times 1.38}{7,707} =$$

$$= \frac{15,780}{7,707} = 2.05$$

Which is just beyond the minimum allowable.

If a larger factor of safety is desired, the straight part of the downstream face could be pushed forward thus heightening its point of tangency with the spillway curve and increasing the base width.

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(1)- When flash boards are in place, maximum static head = 50 cm. when spillway is free, maximum total head = 49.5 cm.

4- <u>Sliding</u>:

Due to the horizantal thrust the concrete section tends to slide over the foundations if the stabilizing forces are not enough to maintain it. The ratio between the product of the stabilizing force times coefficient of friction and the thrust is called the factor of safety for sliding.

The coefficient of friction between concrete and rocks is taken as 0.7.

$$F.S. = \frac{11,413 \times 0.7}{6,707} = 1.2$$

This factor is not safe enough; to increase the F.S. a key of concrete 20 cm. wide will be run all along the length of the dam base, protruding some 15 to 20 cm. into the rock. Also the surface of contact between the rock and the concrete will be made rough.

Shearing stress of concrete = 4 kg./cm.^{2}

Under these conditions the new factor of safety will be:

$$F.S. = \frac{11,413 \times 0.7 + 2,000^{*} \times 4}{6,707} = 2.39$$

Volume of excavated rock for key: 0.2 x 0.2 x 9 = 0.36 m.^{3}

5- <u>Crushing</u>:

The maximum pressure on the foundations occurs at the toe of the dam. In order not to produce tension between the rock and the concrete the resultant of all the forces should fall within the middle third of the base.

Let a = distance between the toe and the point where the resultant cuts the base.

---

* This is the section area of 1 m. length of key in cm.^{2}

If M_1 = resultant moment about toe we have: $M_1 = W_R \times a$

$$a = \frac{M_1}{W_R} = \frac{12{,}148 \times 1.38 - 7{,}707}{12{,}148} = 0.84 \text{ m.}$$

$$\text{eccentricity of } W_R = \frac{2.17}{2} - 0.84 = 0.245 \text{ m.}$$

Crushing prèssure on foundation = $\frac{W_R}{A} \pm \frac{Mc}{I}$

where:

A = base sectional area = 2.17 m^2.

M_2 = moment due to eccentricity = 12,148 x 0.245 = 2,980 kgm.

c = half base width = 1.085 m.

I = moment of inertia of section about axis of base =

$= \frac{1}{12} \times 1 \times \overline{2.17}^3 = 0.852 \text{ m}^4.$

$$\text{Pressure at toe: } P_1 = \frac{12{,}148}{2.17} + \frac{2{,}980 \times 1.085}{0.852} = 5{,}600 + 3{,}720 =$$

$$= 9320 \text{ kg./m}^2 = 0.932 \text{ kg./cm}^2$$

Pressure at heel: P_2 = 5,600 - 3,720 = 1,880 kg./ m^2 = 0.188 kg./cm^2

These pressures are within very safe limit, the allowable being 7 kg./cm^2

g- Downstream protection of foundations:

The sheet of water falling over the spillway, leaves the deflection curve, at the toe of the dam, with appreciable velocity,(1) "and would easily erode the river bed due to the action of the sand and gravel carried by the current even though the

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(1)- This velocity can be easily calculated: It is composed of the initial velocity upstream plus the velocity due to gravity $v = \sqrt{2gh}$ where h= total fall at point considered.= = 3.0 m. initial velocity head = 0.15 m.
velocity at end of deflection curve:
$v = \sqrt{9.8 \times 3.15} = 7.85$ m./sec.

river bed is of rock unless that rock be as strong as the concrete apron."(1)

In the present case the rock is not as strong as concrete, but there is almost no fear of erosion by gravel action on the foundation, because the source is quite near to the dam. Yet it is unwise to leave the place unprotected.

The deflection curve will be produced to form an apron sloping upward. At the end of the apron pieces of rock of preferably sharp edges will be put. This is the most economical means of protection and yet a very effective one.

h- Expansion and contraction:

Like any other material concrete is particularly affected by temperature variation. If a concrete dam be poured one piece between two rocky banks with no joints whatsoever, the rock will have to sustain, for an increase of temparature, a compressive thrust in the direction of the dam axis.(2) It is necessary therefore to provide a joint that allows free displacement of the mass in the longitudinal direction, preventing at the same time any escape of water. This joint will be made between the rock and the dam at the left bank: the left pier (70 cm. wide) will protrude 20 cm. into the rock while the remaining part of the dam section will be stopped at the face of the bank.(3) A space of 2 cm. will be left all along the joint between the dam and the rock. To prevent any seepage, liquid cement mortar will

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(1)- C.F. René Koechlin, Mecanisme de L'eau Vol. II page 46, 1926
(2)- It seems theoretically that this thrust should be considerable in magnitude since for every increase of 10° c. in temperature it will reach a value of 20 to 25 kg./cm^2.
(3)- See detail drawing plate IV

be forced under pressure in the joint, this operation will take place preferably during winter when the temperature of the dam is minimum.

Volume excaveted in rock for the tongue forming joint:

$$0.7 \times 0.2 \times 3.8 = 0.53\ m^3$$

i- Construction cost:

		Unit cost L.L.	total cost L.L.
Excavation in rock:			
key at base..................	0.360		
tongue for joint............	0.530		
total....	0.890 m^3	at 5.00	= 4.45
Plain concrete for dam:			
Theoretical section 9x4.065	=36.600		
additional heel..............	1.130		
apron.........................	0.540		
key at base..................	0.360		
tongue for joint............	0.530		
total....	39.160 m^3	at 60.00	=2,380.00
Reinforced concrete superstructure	4.301 m^3	at 90.00	= 388.00
Flash boards (2.10x0.25x0.05)8 =	0.210 m^3	at140.00	= 29.40

total for dam and bridge.... 2,801.85 L.L.

B- The Headworks

a- Intake:

The intake of the canal conveying water to the reservoir[(1)] will be located on the left bank and will be cut in rock to a depth of about 90 cm. and a width of 150 cm. Its direction

(1)- See part II, a thesis submitted by Mr. Yavuz Alpan.

is almost perpendicular to the dam axis. Under normal conditions the depth of water will not exceed 35 cm. unless one or more parts of the spillway are blocked by flash boards; this depth is considered to be permanently available. The major importance of this last consideration lies in the fact that the water depth in the canal proper has to be 65 cm.(1) Therefore the bottom of the canal has to be lower than the bottom of the intake by a minimum of 65 - 35 = 30 cm.

The length of the intake is about 3.0 m. It will be noticed that no scouring sluices are provided, the assumption being that the silt deposit is null or almost so, the main reasons for that are the quality of the water and the nearness of the source.

b- Headgate:

As will be seen later, the water section in the canal is 130 x 65 cm. this will fix the minimum width of the gate opening b,= 130 cm. The gate will be placed at the end of the intake canal just at the section where the width is reduced from 150 to 130 cm.

Maximum head for gate design = 50 cm.

Dimensions of gate opening.. = 130 x 40 cm.

Dimensions of gate.......... = 138 x 45 cm.

The steel framing of the gate will consist of 4 steel channels, 2 verticals and 2 horizontals; the gate will slide vertically into fixed channels fitted in the rock, one on each side of the canal.

---

(1)- C.F. part C of this chapter dealing with the canal.

Size of channels section:[1]

	sliding channel	fixed channel
depth..........	60 mm.	120 mm.
flange width...	40 mm.	50 mm.
thickness......	3 mm.	5 mm.

The steel sheet, if supported only on the side channels, would require due to the span, a relatively large thickness of about 6 mm. It is preferable therefore to reduce the span by 1/3 by having two additional supports consisting of two vertical steel angles connected to the top and bottom horizontal channels.

Size of angles: 40 x 30 x 3 mm.[2]

Thus the sheet thickness would be reduced to 3.0 mm.[2]

The total force required to lift the gate is 107 kg.[3] The lifting device will consist of a simple rack and pinion system without any reducing gears, and will be inclosed in a steel box fixed on two horizontal channels which are supported by the vertical channels forming the grooves; the height of the lifting mechanism is 85 cm. above ground level. An operating plateform adjacent to the groove channels and consisting of a reinforced concrete slab will span the canal for a length of 80 cm.

The surfaces of contact between the sliding and the fixed channels will be covered by 15 mm. of timber to prevent any infiltration of the water. For the same purpose the bottom

---

(1)- These dimensions are arbitrary: in fact due to the low head the section required by the design would be insignificant
(2)- See Appendix B
(3)- See Appendix B

horizontal channel will enclose a piece of wood 40 x 50mm.[1]

c- Regulation gate:

This gate will be set across the north canal at some distance downstream the source where the width reduces to 1.0 m. It will serve two purposes:

First: To regulate the amount of water to be delivered to the mill.[2]

Second: To function as an emergency gate in case of flood.

The design and the sizes of this gate are exactly like those of the headgate save that its width is 100 instead of 130 cm.

d- Construction cost:

	Unit cost L.L.	total cost L.L.
Excavation for intake:		
$3.0 \times 1.5 \times 0.9 = 4.05\ m^3$........	at 5.00	= 20.20
Reinforced concrete for platforms:		
$2x\ 0.8 \times 0.1 \times 1.40 = 0.224\ m^3$....	at 80.00	= 17.90
Gates:		
$2 \times 85.35 = 170.70$ kg............	at 1.50	= 256.00
	total....	= 294.10 L.L.

C- The Canal

The canal will be run from the intake to the reservoir covering a total distance of 185 m.; for the first 35 m. the section will be cut in rock then in ordinary earth untill the last bend is reached at about 30 m. from the reservoir there again the section has to be cut in rock. As previously

---

(1)- All these details are shown on Plate V
(2)- C.F. chapter I page 2

stated the total flow in the canal will be of 0.3 m^3/sec. the section being designed accordingly.

a- Design of most economical section:

The canal section will be rectangular; this shape being given the priority for the following reasons:

1- Small flow.

2- Economy in area of land.

3- It has to be so in the rock cut part of the canal and it is preferable to keep the same section all through.

The materials the canal can be built of are either reinforced concrete or stone, wih cement lined or unlined surfaces. Each mode of construction will be considered separately. This consists in investigating a number of different size sections as to the construction cost per meter length and the cost of power lost per meter length per year.

If for each type of construction a graph be drawn with water depth in cm. as abcissas and cost in L.L. as ardinates, the most economical section can be readily deducted as being the abcissa (1) of that point having the least ordinate; and the most economical type of construction is the one whose curve gives that lowest point.

This procedure is worked out thoroughly in Appendix C. Results are as follows.

Most economical section for part of canal cut in rock:

80 x 160 cm., total annual cost per meter length = 1.34 L.L.

Most economical section for remaining part of canal cut in earth:

---

(1)- For best hydraulic section the hydraulic radius is one-half the water depth. That is to say for a rectangular canal the depth is one-half the height. Therefore by taking water depth as abcissas the section is readily determined.

Rubble masonry unlined, section: 60 x 120 cm.

Total annual cost ~~per~~ m. length = 2.90 L.L.

Thus we have two different sections for the same canal which is undesirable because of the loss of head and the work involved (the section will have to be modified twice: reduction from rock cut to earth cut then enlargement again from earth cut to rock cut near the reservoir.) Therefore a problem is imposed: to find a most economical common section through out the length of the canal. Considering curves A and B on figures 4 and 5 respectively, it may be noticed that these are parabolas (curve A represents only one branch.) whose axees are parallel to the Y axix.

Equation of parabola A is: $(x - 0.80)^2 = 0.0492\,(y - 1.34)$ (1)

$$\text{from which} \quad y = \frac{(x - 0.80)^2}{0.0492} + 1.34$$

Equation of parabola B is: $(x - 0.60)^2 = 0.0208\,(y - 2.9)$ (1)

$$\text{from which} \quad y = \frac{(x - 0.60)^2}{0.0208} + 2.9$$

If x = depth of common water section

Then y_A and y_B must be total annual costs of this section per meter length in rock cut and in earth cut respectively. And therefore we must have:

total cost of canal: $(120 \times y_B) + (65 \times y_A)$ = minimum.

$$\text{that is:} \quad 120\,\frac{(x - 0.60)^2 + 0.0603}{0.0208} + 65\,\frac{(x - 0.80)^2 + 0.066}{0.0492} = \text{minimum}$$

$$\text{that is:} \quad \frac{d}{dx}\,(7092x^2 - 9045x + 3356) = 0$$

$$14{,}184x = 9{,}045 \;; \quad x = 0.637 \text{ m.}$$

---

(1)- See Appendix C

Thus the most economical common section comes out with a depth of 63.7 cm.

The final section to be used will be: <u>65 x 130 cm.</u> and from curves A and B we get for $x = 65$ cm.

$y_A = 1.45$ L.L. ; $y_B = 2.95$ L.L.

b- <u>Loss of head</u>:

Slope of rock cut part: $s = 0.000454$ (1)

loss of head in 65 m. : $0.0454 \times 65 = 2.95$ cm.

Slope of masonery part: $s = 0.000167$ (1)

loss of head in 120 m. : $0.0167 \times 120 = 2.05$ cm.

loss of head in whole length.... 5.00 cm.

The only appreciable bend in the canal is the last one directly before the entrance to the reservoir; the loss of head in this bend may be taken as 3% of the velocity head that is:

$$\frac{\overline{0.357}^2}{2 \times 9.8} \times \frac{3}{100} = 1.95 \text{ cm.}$$

total loss of head in canal = 7.00 cm.

c- <u>Canal spillway</u>:

When water flowing in the canal is no more admitted to the penstock, it will overflow from the reservoir then from the canal, causing damage to the adjacent properties. To prevent flooding of the banks, a depression will be made in the right wall of the canal just before the last bend, where the canal is nearest to the stream flowing below. The spillway thus formed will have to accomodate 0.3 m^3 per second its crest being 9 cm. below the top of the wall or 1 cm. above normal water level.

---

(1)- See Appendix C

By working out the formula given on page 6 the length to be given to the spillway is:

$$l = \frac{0.3}{0.4 \times 4.44\ (0.09 + 0.065)^{3/2}} = \frac{0.3}{0.1075} = 2.80 \text{ meters}$$

The upper section of the spillway will be built of concrete and the crest rounded off to an appropriate shape.

D- Economy of the Project(-Conclusion-)

While working out this project the cost of 1 kwh. produced by the plant is assumed to be 0.10 L.L. It would be therefore of major interest to find out what would the real cost of 1 kwh. amount to. This will be done in Part II.

Here is the total annual cost for Part I:

			L.L.
Water rights and mill indemnity at 6% interest $169{,}500 \times \frac{6}{100}$ =			10,180
Dam: life assumed 50 years.	depreciation $\frac{2{,}802}{50}$(1) =	56.00	
	maintainance........	28.02	
	interest...........	280.20	
	annual cost..	364.22L.L.	364
Headworks: life assumed 25 years	depreciation $\frac{294}{25}$(2) =	11.80	
	maintainance....	2.94	
	interest.......	29.40	
	annual cost..	44.14L.L.	44
Canal:	(3) 1.45 x 65 = ...	94.00	
	(3) 2.95 x 120=	354.00	
		448.00L.L.	448
	total annual cost.......		11,036 L.L.

This result will appear again in Part II

(1)- See page 19 (2)- See page 22 (3)- See page 25

Appendix A

Measurements for minimum flow taken at Antelyas September 28, 1949.

Data

	l	h	s	t
Main stream.	2.50	0.55	8.0	30.0
	....	0.50	....	33.0
	....	0.45	....	40.0
	....	0.40	....	86.0
	....	0.30	....	42.0
	....	0.27	....	....
Average.....	....	0.40	....	42.6
North canal.	1.00	0.40	20.0	42.0

l = average width of canal or stream in m.

h = measured height of water in m. (taken at different points for the main stream)

s = measured distance in m. along stream or canal

t = time in seconds for a float to cover that distance.

Calculations

Main stream: $Q_1 = A_1 V_1$

$$A_1 \overset{(1)}{=} 2.50 \times 0.40 = 1.0 \text{ m.}^2$$

$$V_1 = \frac{s}{t} = \frac{8.0}{42.6} = 0.188 \text{ m. per second}$$

---

(1)- Although the width of the main stream is about 5.0 m. the effective width is only 2.5 m. because water is stationery between the middle of the stream and the left right bank.

mean velocity is 85% of surface velocity (1)

V_1 mean = 0.188 x 0.85 = 0.16 m. per second

Q_1 = 0.16 x 1.00 = 0.160 m^3 per second

North canal: $Q_2 = A_2 V_2$

(2)
A_2 = 1.00 x 0.40 = 0.40 m^2

$V_2 = \dfrac{20.0}{42}$ = 0.476 m. per second

V_2 mean = 0.476 x 0.85 = 0.404 m. per second

Q = 0.40 x 0.404 = 0.162 m^3 per second

total discharge

$Q = Q_1 + Q_2$ = 0.160 + 0.162 = 0.322 m^3 per second

Assumed minimum total discharge = 0.30 m^3 per second

Measurement for maximum flow taken at Antelyas March 1950.

Data

	l	h	s	t
Main stream.	5.00	0.40	6.30	4.80
North canal.	2.50	0.80	5.40	5.00

Calculations

Main stream: $Q_1 = A_1 V_1$

(3)
A_1 = 5.00 x 0.40 = 2.00 m^2

$V_1 = \dfrac{s}{t} = \dfrac{6.30}{4.80}$ = 1.31 m. per second

velocity of approach for spillway design assumed = 1.5 m. per second

V_1 mean = 1.31 x 0.85 = 1.11 m. per second

Q_1 = 2.00 x 1.11 = 2.22 m^3 per second

---

(1)- Elementary surveying Vol. II Breed and Hosmer page 544
(2)- For some distance downstream the source, the north canal has a width of 2.50 m., then it narrows down suddenly to a width of 1.00 m.
(3)- See footnote (1) page 28 of Appendix A

North canal: $Q_2 = A_2 V_2$
(1)

$A_2 = 2.50 \times 0.80 = 2.00\ m.^2$

$V_2 = \frac{5.40}{50} = 1.08$ m. per second

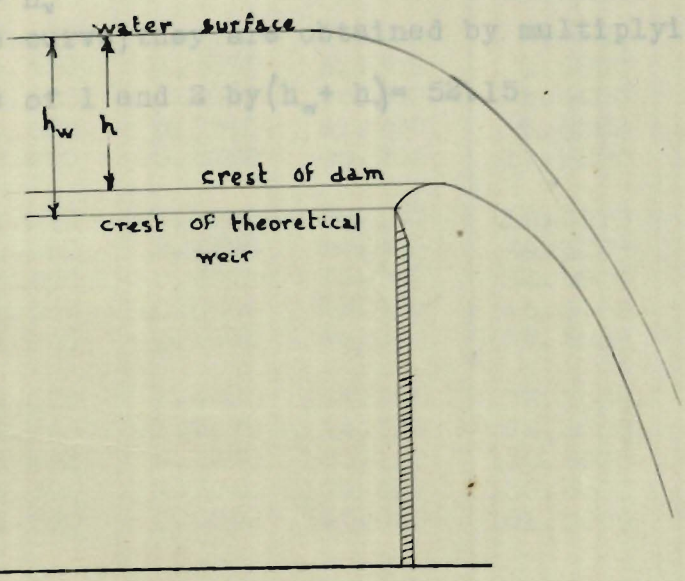
V_2 mean = 1.08 x 0.85 = 0.918 m. per second

$Q_2 = 2.00 \times 0.918 = 1.84\ m.^3$ per second. total $Q = 4.06\ m^3/sec$

This not taking care of flow due to rainfall. The maximum flow used in the design is assumed to be 5.0 $m.^3$ per second.

Table I
(2)

Columns 1 and 2 are coordinates of the curve, corresponding to a value of unity for $(h_w + h_v)$, h_w being the head on the theoretical crest of the sharp crested weir and h_v the head corresponding to the velocity of approach. The origin of this curve is at the crest of the theoretical sharp crested weir.



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(1)-This measurement was performed in the first part of the north canal, see footnote (2), page 28 of Appendix A
(2)-Taken from " Engineering for dams by Creager, Justin and Hind Vol. II page, 362 - 363; edition 1945. The y column is that for which $\frac{h_v}{h_w + h_v} = 0.22$

We have:

$h = 38$ cm.

$h_v = 11.5$ cm.

$$\frac{h_v}{h_v + h} = \frac{11.5}{49.5} = 0.233$$

The value of $\frac{h_w}{h_w + h_v}$ will be somewhat lest than this. Assume a trial value of 0.22 following down colum, 2, of table I, the maximum rise of crest above theoretical sharp crest is found to be $0.049\ (h_w + h_v)$.

Therefore:

$$h + h_v = h_w + h_v\ 0.049\ (h_w + h_v)$$

$$h + h_v = 0.951\ (h_w + h_v)$$

$$h_w + h_v = \frac{49.5}{0.951} = 52.15$$

Check: $\frac{h_v}{h_w + h_v} = \frac{11.5}{52.15} = 0.221$ which is sufficiently close to the assumed value. Therefore columns 1 and 2 will be values of $\frac{x}{h_w + h_v}$ and $\frac{y}{h_w + h_v}$. And columns 3 and 4 will give the true coordinates of the curve; they are obtained by multiplying corresponding values of 1 and 2 by $(h_w + h_v) = 52.15$

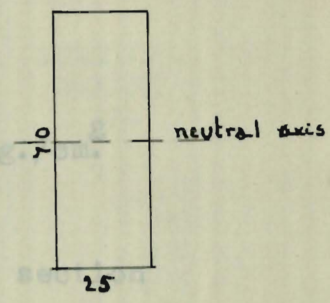
Table I

1	2	3	4
$\frac{x}{52.15}$	$\frac{y}{52.15}$	x	y
0.000	0.0000	0.000	0.0000
0.020	0.0150	1.043	0.7822
0.040	0.0265	2.086	1.3820
0.060	0.0355	3.130	1.6330
0.080	0.0415	4.173	2.1650
0.100	0.0450	5.215	2.3490
0.120	0.0475	6.258	2.4800
0.140	0.0485	7.301	2.5300
0.160	0.0490	8.344	2.5590
0.180	0.0485	9.387	2.5300
0.200	0.0480	10.430	2.5050
0.220	0.0465	11.473	2.4250
0.240	0.0450	12.516	2.3490
0.260	0.0425	13.559	2.2170
0.280	0.0400	14.602	2.0850
0.300	0.0365	15.645	1.9020
0.320	0.0330	16.688	1.7200
0.340	0.0290	17.731	1.5110
0.360	0.0250	18.774	1.3030
0.380	0.0200	19.817	1.0430
0.400	0.0150	20.860	0.7822
0.450	0.0000	23.490	0.0000
0.500	0.0180	26.100	0.9400
0.600	0.0590	31.300	3.0800
0.700	0.1080	36.500	5.6400
0.800	0.1640	41.700	8.5500
0.900	0.2280	46.900	11.9000
1.000	0.2980	52.100	15.5000
1.200	0.4660	62.600	24.3000
1.400	0.6610	73.000	34.5000
1.600	0.8860	83.500	46.3000
1.800	1.1460	94.000	59.6000
2.000	1.4400	104.200	75.1000
2.200	1.7670	114.500	92.2000
2.400	2.1250	125.100	110.5000
2.600	2.5040	135.500	130.6000
2.800	2.8930	146.000	151.0000

Values of y below separation line are measured from the x-axis downward.

Stress in upstream face of pier:

$$S = \frac{P}{A} - \frac{Mc}{I}$$



Where: P = total load = 2,370 kg.

A = concrete section = 1,750 $m.^2$

M = bending moment = 82,900 kgm.

c = 1/2 length of pier = 35 cm.

I = moment of inertia of section taken about axis of pier parallel to the dam

$$S = \frac{2,370}{1,750} - \frac{82,900 \times 35}{1/12 \times 25 \times 70'} = 1.35 - 4.05 = 2.70 \text{ kg./m.}^2 \text{ tension.}$$

Stress in downstream face of pier.

$$S = \frac{P}{A} + \frac{Mc}{I} = 1.35 + 4.05 = 5.40 \text{ kg./m.}^2 \text{ compression.}$$

Design of flash boards:

span = 200 cm.

height of board = 25 cm.

maximum head = 50 cm.

total pressure on board: $P = w\bar{z}A$

Where: w = 1,000 $kg./m.^3$

$\bar{z}$ = depth of center of gravity = 37.5 cm.

A = area of board = 0.25 x 2.0 = 0.5 $m.^2$

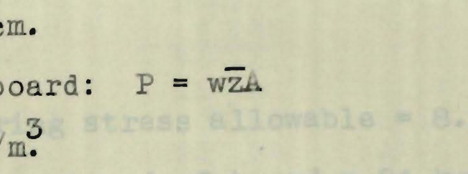
P = 1,000 x 0.375 x 0.5 = 187.5 kg.

Coefficient of friction between wood and concrete = 0.4

Required lifting force to pull up the board =

F = 187.5 x 0.4 = 75 kg.

Maximum moment: $M = (94 \times 1) - (94 \times \frac{1}{2}) = 47$ kgm. = 4,700 kgcm.



Allowable stresses:[1]

bending = s = 68.9 kg./cm.^{2}

shear = s = 8.45 kg./cm.^{2}

The bending stress is given by $S = \frac{Mc}{I}$

Where: S = unit bending stress allowable = 68.9kg./cm.^{2}

M = moment at the point considered

c = distance from the neutral axis of the section to outside fiber = b/2

I = moment of inertia of the whole section about the neutral axis

$$S = 68.9 = \frac{4{,}700 \times b/2}{1/12 \times 25 \times b^3}$$

$$b^2 = \frac{4{,}700 \times 12}{68.9 \times 25 \times 2} = 16.4$$

$$b = 4.05 \text{ cm.}$$

use b = 5 cm.

b as governed by shear:

$$S_s = \frac{VQ}{Ib}$$

Where: S_s = unit shearing stress allowable = 8.45 kg./cm.

V = total shear at end of board = 94 kg.

Q = statical moment of the part of the cross section between the neutral axis and the extreme fiber about the neutral axis = $\frac{ab^2}{8}$

I = moment of inertia (same as above)

For a rectangular section $\frac{Q}{Ib} = \frac{1.5}{A} = \frac{1.5}{ab}$

Where: A = total cross section

Therefore $S_s = 8.45 = 94 \times \frac{1.5}{25 \times b}$; $b = \frac{94 \times 1.5}{25 \times 8.45} = 0.67$ cm.

O.K.

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(1)- Making allowance for the fact that the wood is always wet thus the given stresses are 70% of their ordinary values.

Table II

Weight of dam and location of center of gravity.

ITEMS (1)	DIMENSIONS cm.	AREA cm.^{2}	CENTER OF (2) GRAVITY cm.	STATICAL MOMENT cm.
1	0.5 x 4.2 x 2.2 =	4.62	2.8	13
2	4.2 x 2.2 =	9.24	6.3	58
3	5.2 x 2.2 =	11.42	10.9	125
4	5.2 x 2.2 =	11.42	16.1	185
5	0.5 x 4.7 x 1.3 =	3.06	20.3	62
6	23.5 x 3.1 =	72.80	11.7	852
7	0.5 x 7.8 x 3.1 =	12.10	26.1	316
		124.66		1,611
8	31.3 x 2.5 =	78.25	15.6	1,220
9	0.5 x 5.2 x 2.5 =	6.50	33.3	210
10	36.5 x 2.9 =	105.90	18.2	1,930
11	0.5 x 5.2 x 2.9 =	7.55	38.2	288
12	41.7 x 3.3 =	137.50	20.8	2,860
13	0.5 x 5.2 x 3.3 =	8.58	43.4	372
14	46.9 x 3.6 =	168.90	23.4	3,948
		637.84		12,439
15	0.5 x 5.2 x 3.6 =	9.37	48.6	455
16	52.1 x 8.8 =	458.00	26.0	11,810
17	0.5 x 10.4 x 8.8 =	45.75	55.5	2,540
18	62.6 x 10.2 =	639.00	31.3	20,000
19	0.5 x 10.4 x 10.2 =	53.10	66.1	3,510
20	73.0 x 11.8 =	862.00	36.5	31,416
21	0.5 x 10.4 x 11.8 =	61.45	76.5	4,700
		2,766.51		86,870
22	83.5 x 13.3 =	1,110.00	41.7	46,300
23	0.5 x 10.4 x 13.3 =	69.20	87.0	6,020
24	94.0 x 15.5 =	1,457.00	47.0	68,500
25	0.5 x 10.4 x 15.5 =	80.60	97.5	7,860
26	104.2 x 35.4 =	3,685.00	52.1	192,100
27	0.5 x 20.8 x 35.4 =	368.10	111.1	49,500
28	125.1 x 29.5 =	3,695.00	62.6	231,100
29	0.5 x 15.9 x 29.5 =	234.80	130.4	30,620
		12,466.21		718,870
30	141.0 x 157.4 =	22,200.00	70.5	1,566,800
31	0.5 x 76.0 x 157.4 =	5,990.00	167.3	1,000,000
	total	40,656.21		3,275,670

Center of gravity of total section =

$$= \frac{3,275,670}{40,656.21} = 80.55 \text{ cm. from upstream face}$$

Weight of 1 m. section: 4.065 x 2,500 = 10,150 kg.

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(1)- See diagram on next page.
(2)- Distance of center of gravity of area from upstream face of dam.

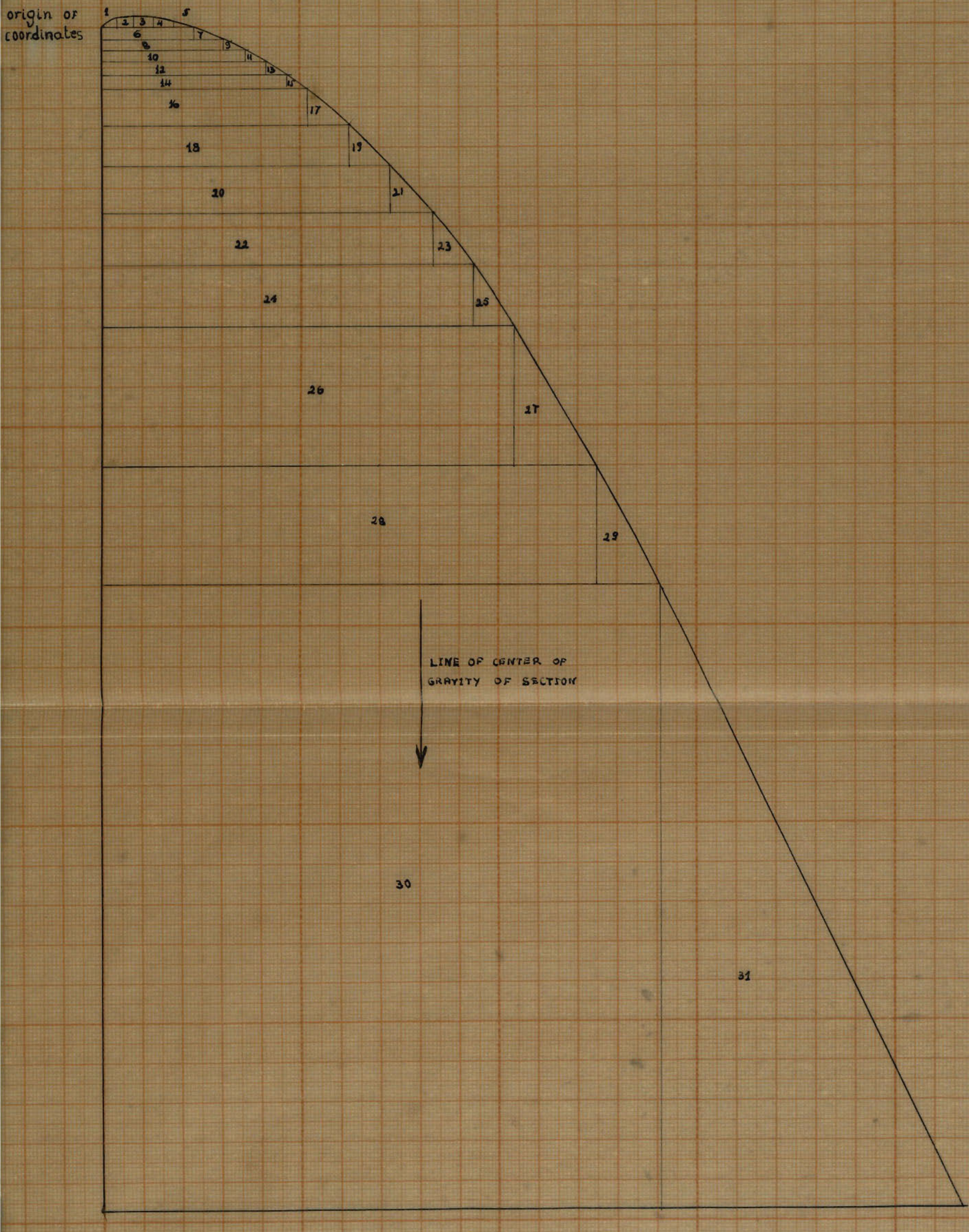


FIG. 2

SHAPE OF CREST AND DOWNSTREAM FACE

CURVE PLOTTED FROM COORDINATES ON TABLE I

NUMBERS ARE REFERED TO TABLE II

SCALE : 1/10

Appendix B

Design of vertical angles:

It is assumed that the total pressure is carried by the 2 central angles.

total pressure $P = 1,000 \times 0.25 \times 0.5 \times 1.38 = 173$ kg.

Each angle is subject to a triangular loading.

The formula for maximum moment is:

$M = 0.128\ P_1 l$

Where: $P_1 = P/2 = 86.5$ kg.

$l = 45$ cm.

$M = 0.128 \times 86.5 \times 45 = 50.0$ kgcm.

$$\frac{I}{c} = \frac{M}{s} = \frac{50.0}{1,200} = 0.0416 \text{ cm.}^3$$

use 40 x 30 x 3 mm. angles.

Design of steel sheet:

$$\text{span} = \frac{138}{3} = 46 \text{ cm.}$$

Unit pressure at lowest part of sheet =

$= p = 1,000 \times 0.5 \times 0.01 \times 0.01 = 0.05$ kg./cm.^{2}

Taking a strip 1 cm. heigh.

$$\text{Maximum moment: } M = \frac{0.05 \times \overline{46}^2}{8} = 13.2 \text{ kgcm.}$$

$$S = \frac{Mc}{I} \quad \frac{13.2 \times t/2}{1/12 \times 1 \times t^3} = 1,200$$

$$t^2 = \frac{13.2}{200} = 0.066$$

$t = 2.57$ mm.

use 3.0 mm. ~~iron~~ steel sheet

Weight of headgate:

a- Volume of sliding part.

Vertical channels..	2 x 45 (6.0 x 0.3 + 3.7 x 0.3 x 2) =	362 $cm.^3$
Horizontal channels	2 x 138(6.0 x 0.3 + 3.7 x 0.3 x 2) =	1,110 "
Vertical angles....	2 x 45 (4.0 x 0.3 + 3.7 x 0.3).. =	209 "
Steel sheet........	138 x 45 x 0.3 .... =	1,870 "
		3,551 $cm.^3$
Rack for lifting operation..	130.0 x 5.0 x 2.0 .... =	1,300 "
	total volume	4,851 $cm.^3$

total weight of gate:

7.8 x 4.851 = 37.85 kg.

b- Weight of fixed channels.

Verticals..	2 x 170 (12.0 x 0.5 + 4.5 x 0.5 x 2) =	3,570 $cm.^3$
Horizontals	2 x 120 (12.0 x 0.5 + 4.5 x 0.5 x 2) =	2,520 $cm.^3$
		6,090 $cm.^3$

Weight: 7.8 x 6.09 = 47.5 kg.

total weight of headgate: 37.85 + 47.50 = 85.35 kg.

Friction force = 173 x 0.4 = 69.3 kg.

Total force necessary to lift the gate:

37.85 + 69.3 = 107.15 kg.

Appendix C

Ganguillet and Kutter's formula for uniform flow in canals:

$$V = K\sqrt{RS}$$

Where: V = velocity in m./sec.

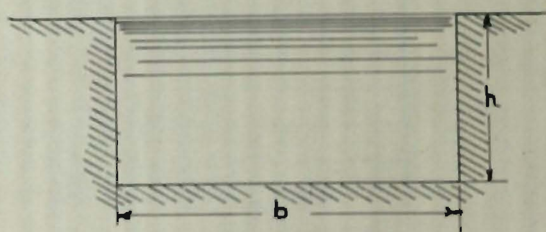
R = hydraulic radius = $\frac{\text{water section}}{\text{wetted perimeter}}$ in meters.

K = coefficient depending on R and on the coefficient of roughness n. of the canal (1)

S = slope of water surface or canal bottom.

For a rectangular section

$$R = \frac{bh}{b + 2h} = \frac{2h^2}{4h} = \frac{h}{2}$$



Values of K will be scaled off from figure 3 on the following page.

Values of A, R & V for different sections used in trials.

h	A	R	V
cm.	m^2	m.	m./sec.
40	0.32	0.200	0.937
50	0.50	0.250	0.600
60	0.72	0.300	0.417
65	0.84	0.325	0.357
70	0.98	0.350	0.306
80	1.28	0.400	0.234

---

(1)- Here are values of n. for different canal surfaces

Nature of surface	n.
neat cement.......	0.010
concrete..........	0.013
rubble masonry....	0.017
rock cut, smooth..	0.026

Graph for Coefficient K to be Used In Formula

$V = K\sqrt{Rs}$

From: "Mecanisme de l'Eau" vol. I by R. Koechlin.

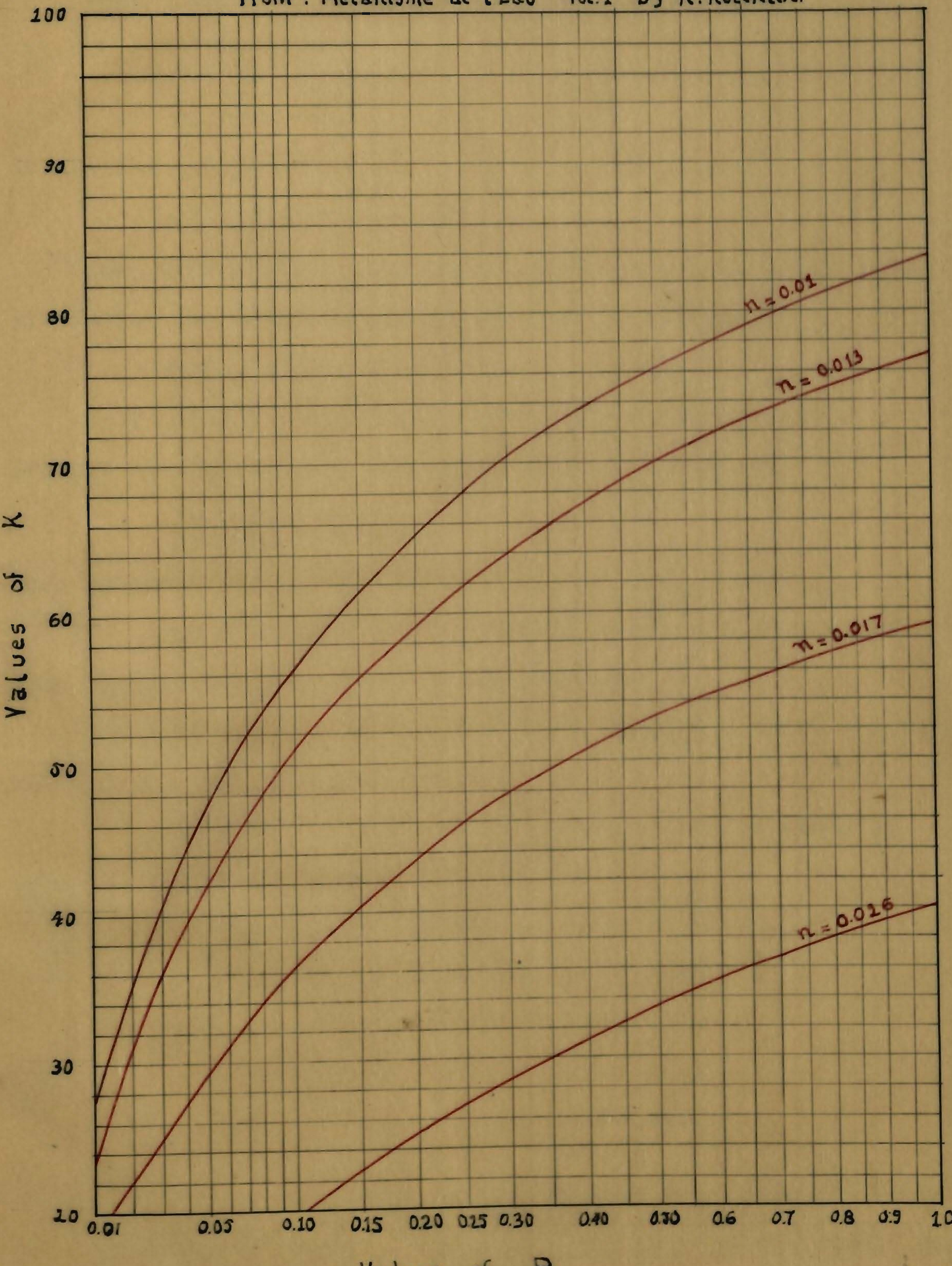


Fig: 3

<u>Canal section cut in rock</u>, $n = 0.026$

Take a water section: $h = 40$ cm., $b = 80$ cm.

$$K = 25.2$$

$$S = \frac{V^2}{K^2 R} = \frac{\overline{0.937}^2}{\overline{25.2}^2 \times 0.20} = 0.00693$$

Excavation per meter length in the first 35 meters.(1)

$$1.1 \times 0.80 = 0.88 \text{ m}^3$$

cost 0.88 x 5.0 = 4.4 L.L., x 35 = 154.0 L.L. total cost.

Excavation per meter length in the last 30 meters.(2)

$$0.50 \times 0.80 = 0.40 \text{ m}^3$$

cost 0.4 x 5 = 2.0 L.L., x 30 = 60 L.L. total cost

Average excavation cost per meter length of rock cut:

$$\frac{154 + 60}{35 + 30} = 3.30 \text{ L.L.}$$

Loss of power per meter length:

$$0.3 \times 1{,}000 \times 0.00693 \times \frac{0.746}{75} \times \frac{50}{100}^{(3)} = 1.49 \times 0.00693 \text{ kw.}$$

(4)

Cost of kwh. lost per meter length per year:

360 x 24 x 0.10 x 1.49 x 0.00693 = 1,288 x 0.00693 = 8.92 L.L.

Assumed life of canal = 30 years

The total annual cost per meter length of canal is made up of the following:

		L.L.
Depreciation...	3.30/30 .....=	0.11
Maintainance...	1% x 3.30 ...=	0.03
Interest.......(5)	10% x 3.30 ..=	o.33
Power lost.....	.............=	8.92

total annual cost per meter length = 9.39 L.L.

---

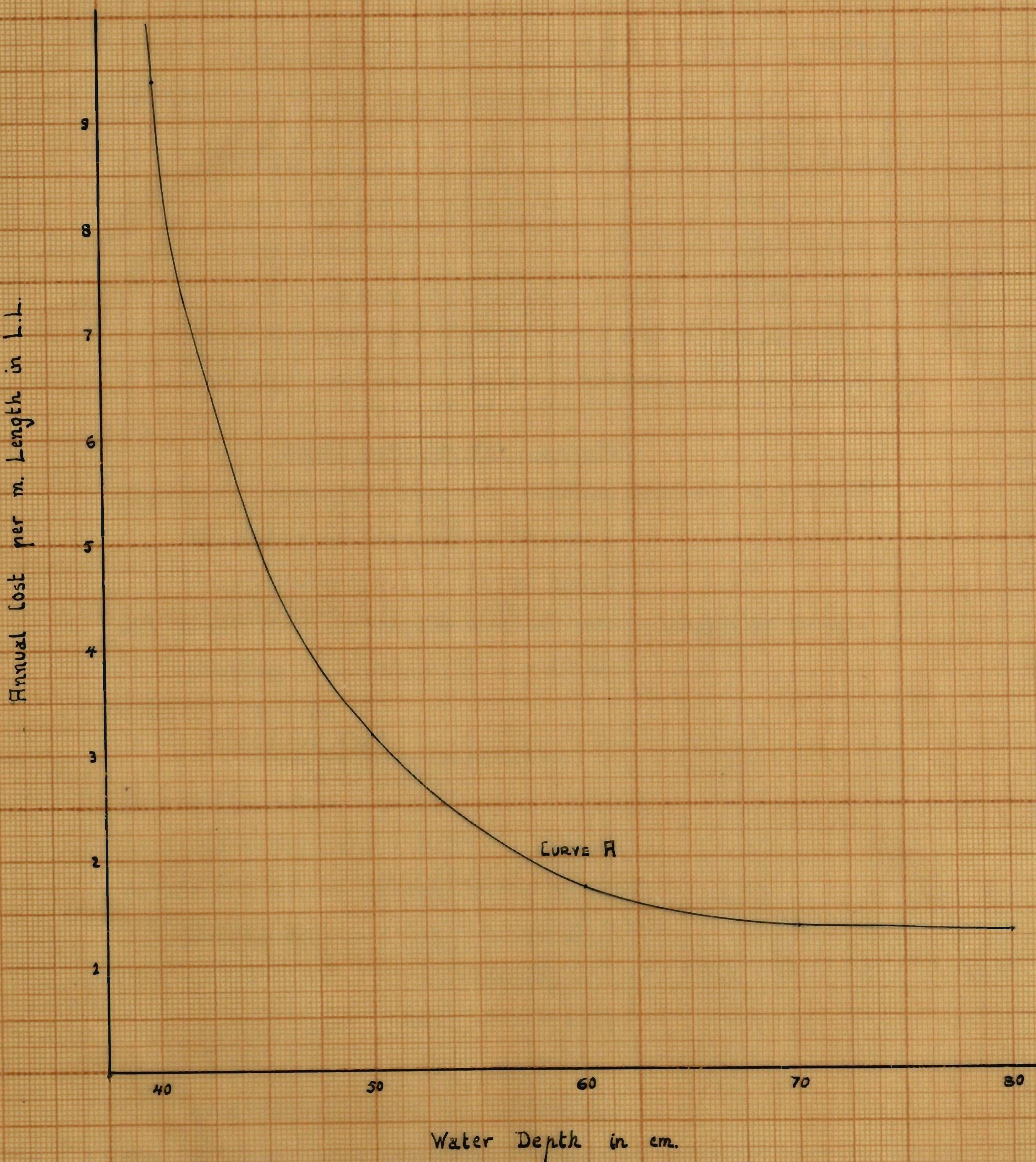
(1)- The depth of the canal in this stretch has to be of about 110 cm. due to the ground surface being almost level.

(2)- In that stretch and in the earth cut part of the canal, a free board of 10 cm. is allowed.

(3)- This is the assumed efficiency of the whole system at the switchboard.

(4)- The kwh. is assumed to cost 0.10 L.L.

(5)- Made of 6% interest proper + 2% taxes + 2% insurance



CURVE FOR MOST ECONOMICAL CANAL SECTION
CUT IN ROCK
FIG. 4

Following the same procedure as above for different sections the results are as follows:(1)

Canal section	K	S	annual cost per m. length
40 x 80 cm.	25.2	0.00693	9.39 L.L.
50 x 100 cm.	27.0	0.00198	3.17 L.L.
60 x 120 cm.	28.5	0.00072	1.71 L.L.
70 x 140 cm.	30.0	0.00030	1.35 L.L.
80 x 160 cm.	31.2	0.00014	1.34 L.L.

Canal section cut in earth,

Type I: Reinforced concrete section unlined, n = 0.013

Thickness of concrete = 10 cm.

Reinforcement........ = 40 kg./m^3.

All quantities and prices are per meter length of canal.

section: h = 40 cm., b = 80 cm.

K = 59

$$S = \frac{\overline{0.937}^2}{59^2 \times 0.20} = 0.00126$$

Excavation cost:

0.6 x 1.00 x 1 x 2.00 = 1.20 L.L.

Required quantity of reinforced concrete:

0.1 (0.80 + 2 x 0.6) = 0.20 m^3

x 80.00

cost 16.00 L.L.

\+ 1.20

construction cost 17.20 L.L.

Cost of power lost per meter length per year:

1,288 x 0.00126 = 1.62 L.L.

---

(1)- These results are illustrated in fig. 4

Depreciation... 17.20/30 .... = 0.57

Maintainance... .... = 0.17

Interest....... .... = 1.72

Power lost..... .... = 1.62

total annual cost...... = 4.08 L.L.

Results for other sections are as follows:

canal section	K	S	annual cost per m. length
40 x 80 cm.	59	0.00126	4.08 L.L.
50 x 100 cm.	62	0.00037	3.39 L.L.
60 x 120 cm.	64	0.00014	3.71 L.L.

Type II: Reinforced concrete lined with cement, n = 0.01

section: h = 40 cm., b = 80 cm.

K = 65.4

$$S = \frac{\overline{0.937}^2}{\overline{65.4}^2 \times 0.20} = 0.00103$$

Cost of plastering inside and top of canal:

(2 x 0.5 + 1.00) x 1.25 = 2.50 L.L.

Excavation cost............. = 1.20 L.L.

Reinforced concrete cost.... =16.00 L.L.

Construction cost.......... 19.70 L.L.

Cost of power: 1,288 x 0.00103 = 1.33 L.L.

Depreciation... 19.70/30 .... = 0.655

Maintainance... .... = 0.197

Interest...... .... = 1.970

Power......... .... = 1.330

total annual cost...... = 4.152 L.L.

Results for other sections are as follows:

canal section	K	S	annual cost per m. length
40 x 80 cm.	65.4	0.00103	4.15 L.L.
50 x 100 cm.	68.0	0.00031	3.82 L.L.
60 x 120 cm.	70.4	0.00012	4.18 L.L.

Type III: Rubble masonry canal. (unlined), $n = 0.017$

Thickness of walls and bottom = 30 cm.

cost of 1 m^3 of rubble masonry= 15 L.L.

section: $h = 40$ cm., $b = 80$ cm.

$K = 43.6$

$$S = \frac{\overline{0.937}^2}{\overline{43.6}^2 \times 0.20} = 0.00231$$

Excavation cost:

$0.80 \times 1.40 \times 1 \times 2.00 = 2.24$ L.L.

Cost of masonry:

$0.30\,(0.80 + 2 \times 0.80) \times 15 = 10.80$ L.L.

+ 2.24

construction cost.......... = 13.04 L.L.

Cost of power: $1,288 \times 0.00231 = 2.96$ L.L.

Depreciation... 13.04/30 .... = 0.435

Maintainance... .... = 0.130

Interest....... .... = 1.304

Power.......... .... = 2.960

total annual cost...... = 4.829 L.L.

Results for other sections:

canal section	K	S	annual cost per m. length
40 x 80 cm.	43.6	0.00231	4.83 L.L.
50 x 100 cm.	46.0	0.00068	3.10 L.L.
60 x 120 cm.	48.0	0.00025	2.90 L.L.
70 x 140 cm.	49.4	0.00011	3.10 L.L.

Type IV: Rubble masonry canal lined with cement, n = 0.010

This type of canal is a combination of types II and III. Therefore values of K, S and power cost will be similar to those in type II.

section: h = 40 cm., b = 80 cm.

Cost of plastering inside and top of canal:

(2 x 0.50 + 1.40) x 1.50 = 3.60 L.L.

Excavation cost.................. = 2.24

Cost of masonry.................. = 10.80

Construction cost........... = 16.64 L.L.

Depreciation... 16.64/30 .... = 0.555

Maintainance.... .... = 0.166

Interest....... .... = 1.664

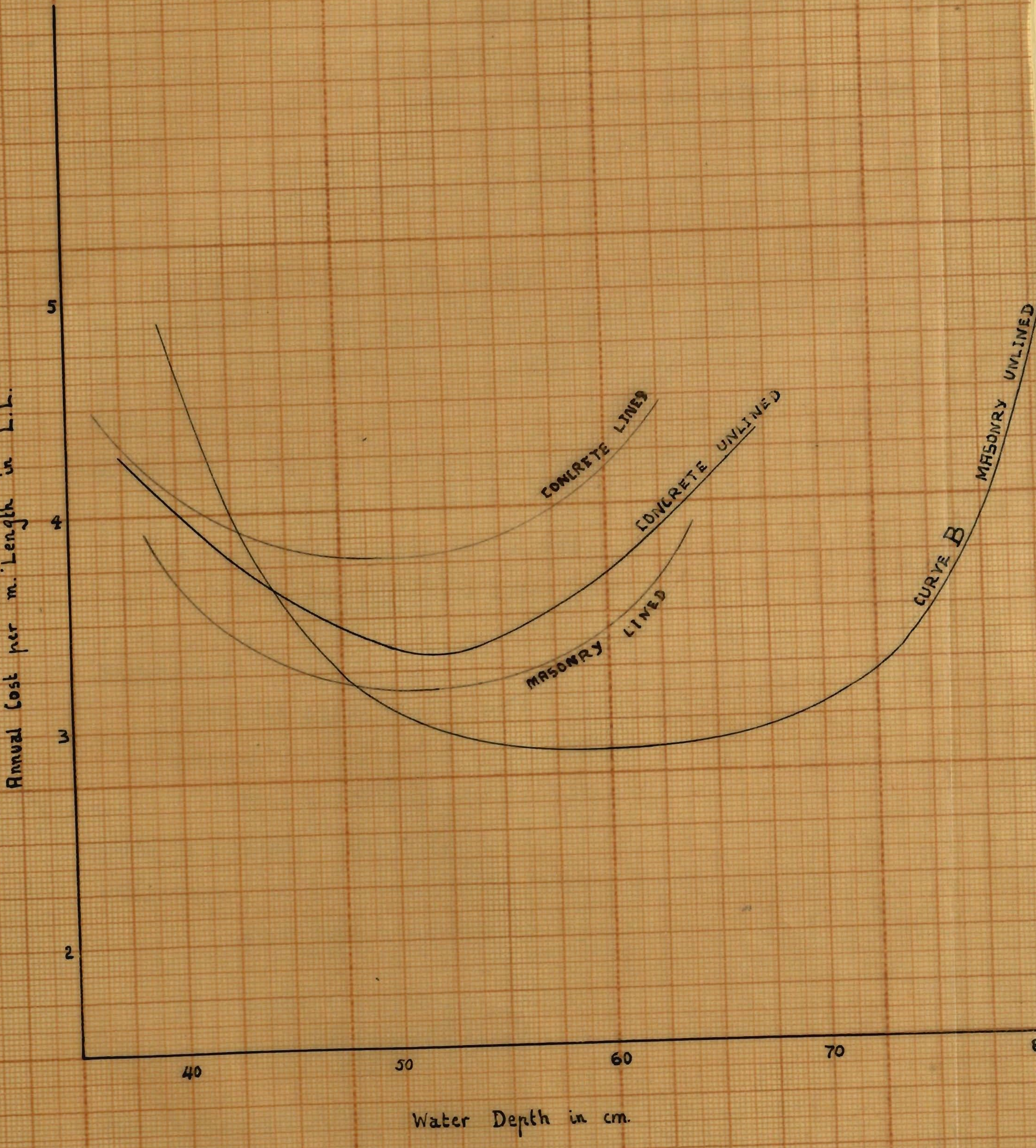
Power......... .... = 1.330

total annual cost........ = 3.715 L.L.

For other sections results are:

canal section	K	S	annual cost per m. length
40 x 80 cm.	65.4	0.00103	3.71 L.L.
50 x 100 cm.	68.0	0.00031	3.22 L.L.
60 x 120 cm.	70.4	0.00012	3.42 L.L.

These results are illustrated in fig. 5



CURVES FOR CANAL TYPES AND MOST ECONOMICAL SECTION

CUT IN EARTH

FIG. 5

To find the equations of parabolas A and B on figures 4 and 5 respectively.

The equation of a parabola whose axis is parallel to the y axis is:

$$(x - h)^2 = 4a\,(y - k)^2$$ [1]

Parabola A:

Vertex at point (0.80, 1.34)

Passing through point (0.50, 3.17)

$$(x - 0.80)^2 = 4a\,(y - 1.34)$$

When $x = 0.50$, $y = 3.17$

Therefore : $(0.50 - 0.80)^2 = 4a\,(3.17 - 1.34)$

$$4a = \frac{0.09}{1.83} = 0.0492$$

$$(x - 0.80)^2 = 0.0492\,(y - 1.34)$$

$$y = \frac{(x - 0.80)^2}{0.0492} + 1.34$$

Parabola B:

Vertex at point (0.60, 2.90)

Passing through point (0.40, 4.83)

$$(x - 0.60)^2 = 4a\,(y - 2.90)$$

When $x = 0.40$, $y = 4.83$

Therefore: $(0.40 - 0.60)^2 = 4a\,(4.83 - 2.90)$

$$4a = \frac{0.04}{1.93} = 0.02085$$

$$(x - 0.60)^2 = 0.0208\,(y - 2.9)$$

$$y = \frac{(x - 0.60)^2}{0.0208} + 2.9$$

Slope of canal water surface: section: 65 x 130 cm.

Masonry	Rock
$K = 48.5$	$K = 29.4$
$S = \dfrac{0.357}{48.5 \times 0.325} = 0.000167$	$S = \dfrac{0.357}{29.4 \times 0.325} = 0.000454$

(1)- c. f. Mathematics, by B. B. Low page 102 6th edition 1945

Appendix D

List of prices.

Taken from Proffessor K. Yaramian, May 11th, 1950

Cement	Cost per bag	4.00 L.L.
Sand	Cost per m^3	3.00 "
Gravel	" " "	4.00 "
Steel	Cost per ton	210.00 "
Iron for gates	Cost per kg.	1.50 "
Pine wood	Cost per m^3	140.00 "
Excavation in rock	" " "	5.00 "
" " earth	" " "	2.00 "
Rubble masonry	" " "	15.00 [1] "
Plain concrete for dam	" " "	60.00 [2] "
Reinforced concrete for bridge	" "	90.00 "
" " " canal	" "	80.00 "
Plastering on concrete surface	Cost per m^2	1.25 "
" " rubble masonry	" " "	1.50 "

---

(1)- This is found as follows:

Stone	1 m^3	at	3.00 L.L.
sand	1/3 -	"	1.00 L.L.
cement	$1\frac{1}{2}$ bags	"	6.00 L.L.
labor		"	5.00 L.L.
	total		15.00 L.L.

(2)-

Sand and gravel	0.8 m^3	at	5.00 L.L.
cement	5 bags	"	20.00 L.L.
labor		"	35.00 L.L.
			60.00 L.L.

Labor cost is high because involving construction of spillway.

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