ENGINEERING 525.526

DESIGN OF HEADWORKS

FOR

KAZMIEH IRRIGATION SCHEME

Thesis presented by GIRAYR KHACHADURIAN for the B.Sc. Degree in Civil Engineering under the supervision of Professor Romansky.

Academic Year 1945-1946

REFERENCES

Notes of Prof. Romansky	Eng. 505 - 506
Daugherty	Hydraulics
Bligh	Dams and Weirs
Etcheverry	Irrigation practice and Engineering
Union des Ingenieurs et techniciens Français au Levant	25 annees d'efforts Français au Levant
Kidder - Parker	Architects' and Builders' Handbook
Shedd	Structural Design in Steel
Service Hydraulique of Lebanese Government	Location Map; scale 1/10000 Contour Map; scale 1/200.

C O N T E N T S.

I. IRRIGATION IN SYRIA AND LEBANON

A - GENERAL PRINCIPLES

Syria and Lebanon are considered as being ary countries. Except in a few spots, water is seldom seen. A large fraction of the lands seem to be as arid as the Sahara desert.

This belief is untrue. In the contrary Syria and Lebanon possess huge amounts of water for a great part not yet utilized. Out of the 18 million hectares, which constitute the total area of these two countries, at least four million hectares can be cultivated. Those cultivated now do not exceed one and a half million hectares, which means less than 10% of the total area and the 3/8 of the agricultural possibilities.

The rains that fall in abundance from December till March over the mountains of Lebanon, Anti-Lebanon, the Amanus, and the Taurus, due to the geological formation of the subsoil of these mountains, are stored in huge amounts, thus giving birth to perennial springs which keep running several permanent rivers.

B - HISTORY

Ancient people living in Syria and Lebanon have been aware of the possibilities offered to them by Nature. Since the highest antiquity, many irrigation schemes have been planned and executed with a maximum of ingeniosity, and getting results remarkable for their period. In Syria we still find the canals of the Khabour near Abou Kemal. The underground water was caught in many places by faggoras, a kind of gallery dug through the rock at a depth of 15 or 20 meters. Recent archeological surveys have thrown more light over the irrigation scheme of the Ancients. Thus we know about the dam of Hardaka near Palmyre, the dam of Homs, the cisterns of Resafé, the canal of Heracles over the Euphrate, the aqueducts of Nahr-Beirut, known as those of Zbeidè, the aqueducts of Nahr Ibrahim and the collecting basins of Salomon which used to collect the artesian water near Tyre, and which are used up to now.

The aqueduct of Nahr Beirut. On the background: Headworks of new irrigation scheme around Beirut.

Unfortunately, this set of very interesting irrigation schemes have later on been discontinued. Up to ten years ago, irrigation has been practised with very primitive methods. Every agriculturer used to make his own irrigation, without whatsoever knowledge of the subject. This has led to poor output and at the same time created an eternal subject of dispute among the agriculturers.

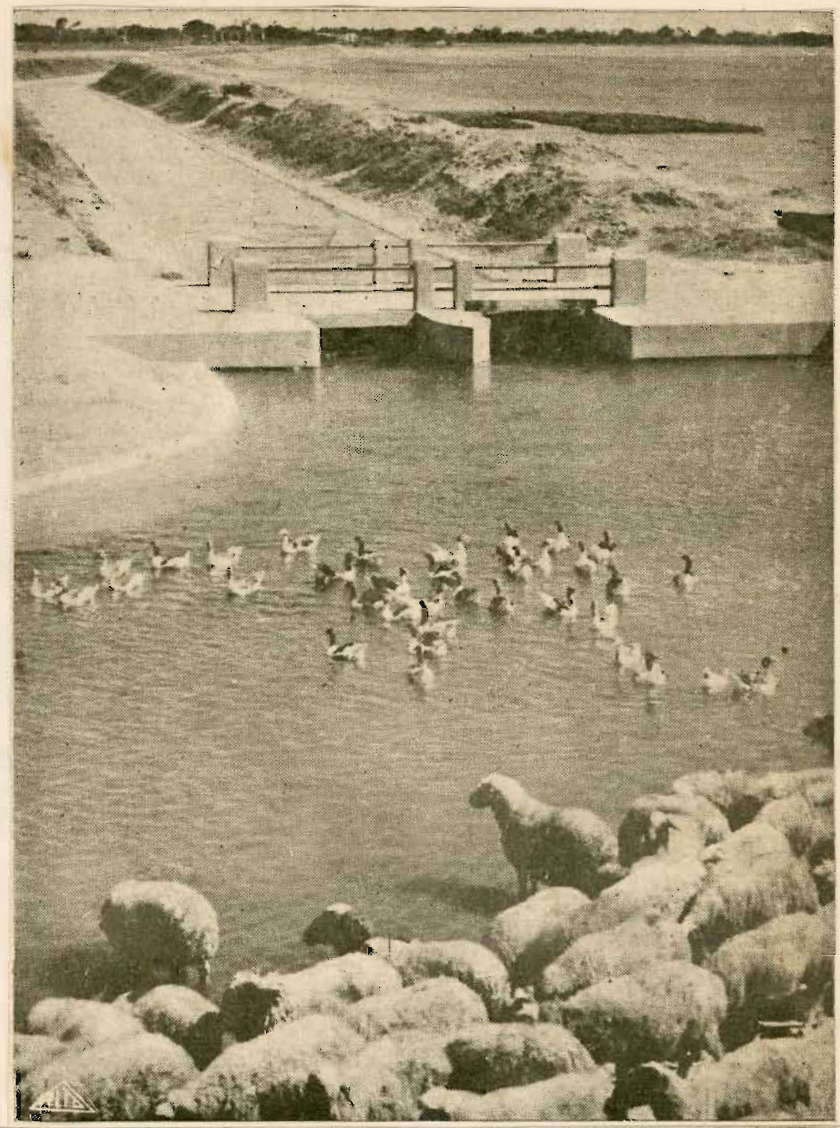
The same headworks seen from upstream.

This was the state of the affairs before the first set of works for the irrigation of the Homs district started.

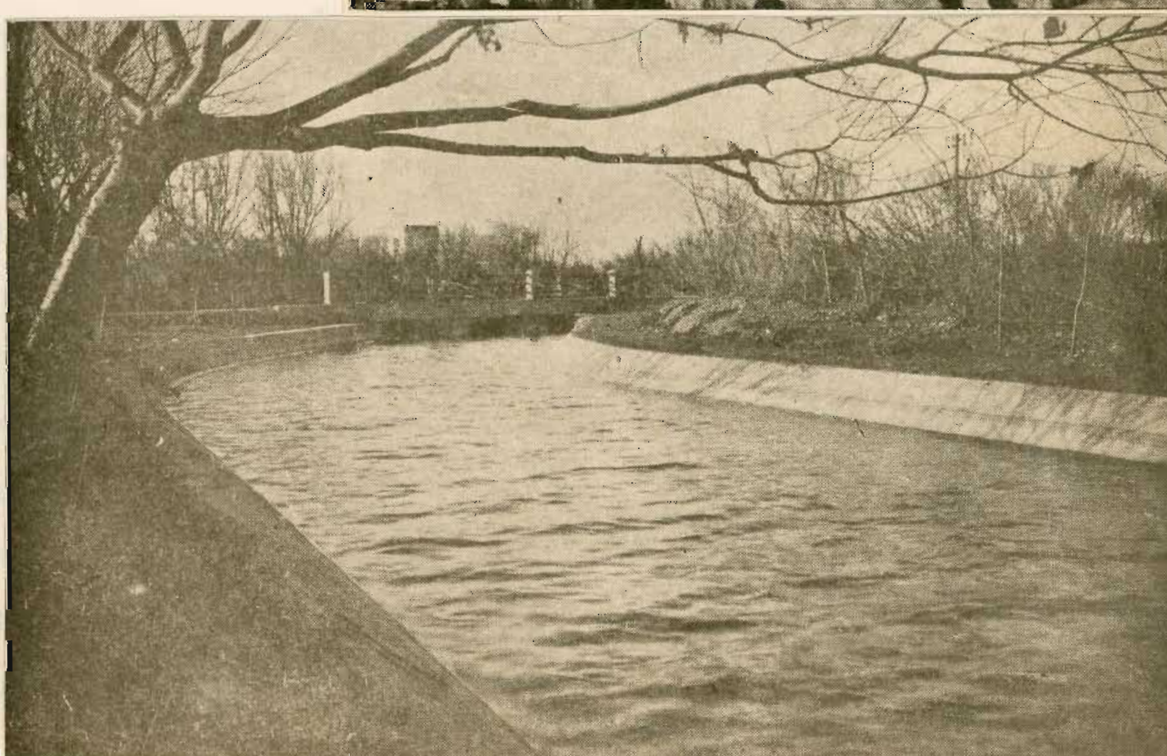
C - WORKS EXECUTED

Some fifteen years ago Syria and Lebanon were completely new in irrigation. Not only irrigation was inexisting, but also no record of the regime of the rivers was available. Projects of irrigation are

The canal at Bab-Amar



The main canal near Homs



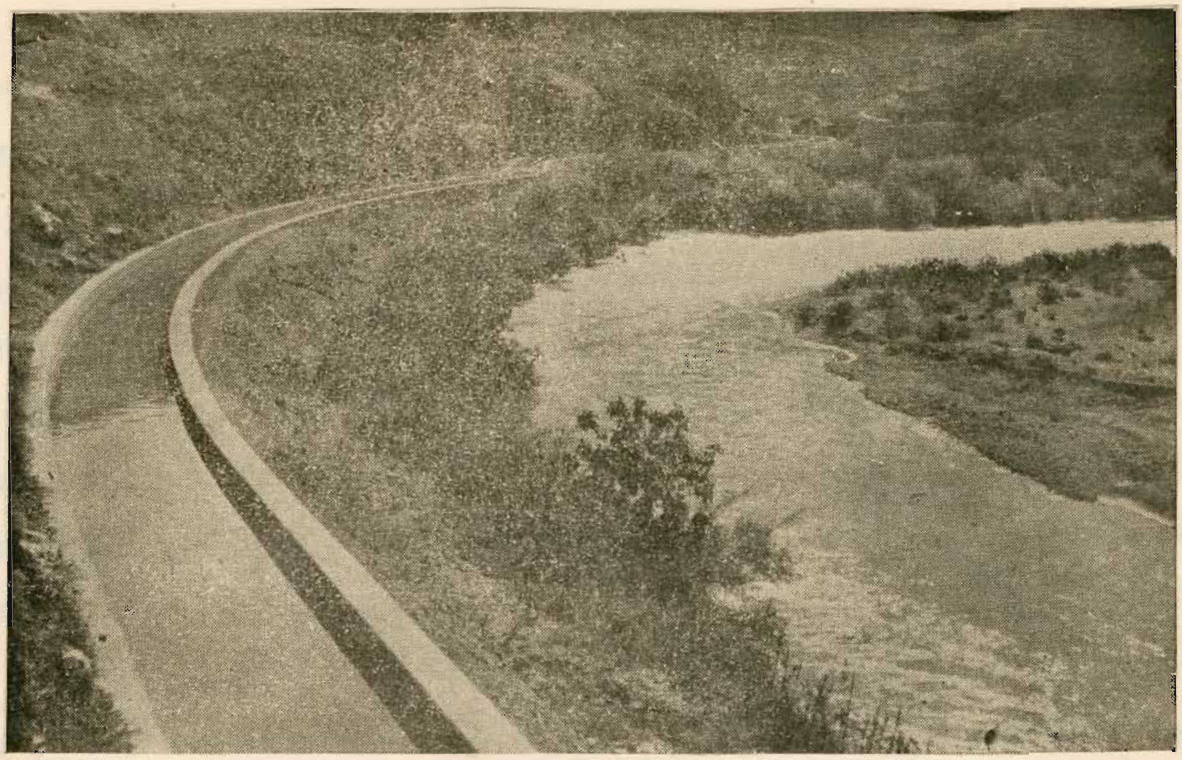
not like those of Structures. For long periods, sometimes running up to 50 years, the regime of the river and of its tributories have to be recorded, the nature of the subsoil has to be investigated, the rain carrying winds, the temperature changes, the snowfall, the amount of silt carried and many other factors have to be studied, and only then the design is to start.

But the country could not wait such a long period. In 1929 the "La Régie des Etudes Hydrauliques" was created and started immediately to study the Orontes river from the spring down to the mouth. Then they carried the following studies: the Yarmuk, small river near the Transjordan border; the Euphrate and its tributary the Khabour; irrigation of the Bekaa by the waters of the Yammouneh lake and the Nahr Kzayel; and that of the Batroun Valley, and the amelioration of the irrigation scheme existing around Beirut. All these projects were established from 1930-33.

During the year 1934, the execution of the first set of the irrigation of the Homs district started. Since then it did not stop. All the above-mentioned projects have been started and some completed. In addition in May 1943 has started the irrigation of the coastal valley from Nakoura to Saida by the waters of the Litani river and the Ras-el-Ain spring. This is known under the name "Kazmieh Irrigation Scheme".

D - POSSIBILITIES

The work executed during the last ten years is considerable if the difficulties and the short period be considered. But it is negligible compared to what could be done. Lebanon and Syria are essentially agricultural countries, and the effect of irrigation on agriculture cannot be overemphasized. Every drop of water, which can be utilized and is not, is a loss to the country. And we have scores of rivers and springs that run to the sea without being touched.



Kazmieh Irrigation Scheme
The river and the main canal just after the Headworks.

One of the many structures of the project
The aqueduct of Ras-el-Ain

II. THE KAZMIEH PROJECT

The Kazmieh project has been planned to irrigate the coastal region of the South Lebanon from Sidon to the Palestine border. Started in May 1943 it is still under construction. When completed, 4000 hectares could be irrigated. Already it is being utilized in some regions where the project is completed.

The modulus of irrigation is 1.25 liter/second/hectare. This seems too high, when we keep in mind the average modulus of 0.60 established for different countries of the Mediterranean basin. But when we consider the particular conditions existing in the area under question - relatively poor and pervious soil, plantations to be mostly orchards, and ignorance of the agriculturers for using the water economically - the above figure does not appear excessive.

The design of the Kazmieh project has been done very carefully and has necessitated the provision of many structures such as tunnels, aqueducts etc. In many respects, when completed, this project will be considered as one of the best irrigation schemes existing in Syria and Lebanon. Its main disadvantage is the lack of adequate headworks. The water is deviated into the canal by an old weir, and the regulating devices are inexisting. The purpose of this paper is to present a design of headworks for this project.

III. HEADWORKS

A - GENERAL CONDITIONS OF FLOW

The purpose of the headworks is to deviate the water into the canal. This water has to be so regulated that for all conditions of flow in the river, the discharge of the canal remains as near to 6.00 M.C./sec. as possible.

Before designing any irrigation project, the regime of the river has to be studied for several years, and the monthly and yearly variations recorded. In the case of the Litani only rough studies have been done, and the results are:

Maximum flow 200 M.C./sec

Minimum flow 30 M.C./sec

Average flow 60 M.C./sec

These figures cover only a few years. As the headworks are designed to last several decades, it would not be safe to base the design over such an incomplete data. The maximum flow will be assumed to be 500 M.C./sec. This figure may seem excessive. But as the available records are far from being complete, it is better to be on the safe side. For the same reason the minimum flow will be considered to be 10 M.C./sec.

The headworks will be designed so as to provide enough water in the canal at the flow of 10 M.C./sec and be strong enough to resist the flow of 500 M.C./sec. These extreme cases might never happen, nevertheless the design will be based upon them.

a) Choice of a weir formula

Among the many weir formulas the one given by Francis suits the best to our weir and conditions of flow.

For a weir with negligible velocity of approach, no end contraction and no submergence

$$Q = 1.875 BH^{3/2}$$

Q being the discharge of the river M.C./sec.

B " " opening of the weir, in meters.

H " " head, in meters.

For a weir with the velocity of approach considered

$$Q = 1.875B \left[(H + ho)^{3/2} - ho^{3/2}\right] \text{ for } \frac{H}{Z} < 0.218$$

in which $ho = \frac{Vo^2}{2g}$

Vo being the velocity of approach in the river measured some distance upstream of the weir, in meters,

Z being the height of the weir, in meters.

For a negligible velocity of approach or for a uniform velocity, Schoder and Turner at Cornell University have found the Francis formula to be about 7% too low when H = 0.03 meters, about 3% too low for H = 0.06 m., but to be accurate within 1 to 3 % for heads above 0.10 m. G.H. Rafter and others have conducted experiments at Cornell University with values of H up to 1.50 meters and found the formula to hold up to that head. (Schoder, E.W., and K.B. Turner, Precise Weir Measurements, Trans. A.S.C.E., Vol. 93, p. 199, 1929.)

As in the case of our weir the values of H will be within 0.10 m. and 1.50 m., the Francis formula will hold true within 1-3%, which is accurate enough.

For $\frac{H}{Z} > 0.218$ Francis recommends

$$Q = 1.875 BH^{3/2} (1 + 1.5\alpha \frac{ho}{H})$$

α varies from 1.00 to 2.00. It increases as $\frac{H}{Z}$ decreases.

b) Computation of the head.

Assume the weir opening to be 90 meters.

1-Q = 500 M.C./sec

Vo assumed 3.00 m./sec.

α " 1.15

$$ho = \frac{Vo^2}{2g} = \frac{3 \times 3}{19.6} = 0.46$$

$$500 = 1.875 \times 90 \times H^{3/2}(1 + \frac{1.5 \times 1.15 \times 0.46}{H})$$

$$2.96 = H^{3/2} + 0.78\ H^{1/2}$$

$\therefore$ H = 1.57 meters

2-Q = 400 M.C./sec

Vo assumed 2.50 m./sec.

α " 1.30

$$ho = \frac{2.5 \times 2.5}{19.6} = 0.32$$

$$400 = 1.875 \times 90 \times H^{3/2} (1 + \frac{1.5 \times 1.3 \times 0.32}{H})$$

$$2.37 = H^{3/2} + 0.625\ H^{1/2}$$

$\therefore$ H = 1.39 meters

3-Q = 300 M.C./sec

Vo assumed 2.00 m./sec.

α " 1.50

$$ho = \frac{2 \times 2}{19.6} = 0.20$$

$$300 = 1.875 \times 90 \times H^{3/2} (1 + \frac{1.5 \times 1.5 \times 0.20}{H})$$

$$1.78 = H^{3/2} + 0.46\ H^{1/2}$$

$\therefore$ H = 1.18 meters

4-Q = 200 M.C./sec

Vo assumed 1.50 m./sec.

α " 1.70

$$ho = \frac{1.5 \times 1.5}{19.6} = 0.115$$

$$200 = 1.875 \times 90 \times H^{3/2} \left(1 + \frac{1.5 \times 1.7 \times 0.115}{H}\right)$$

$$1.185 = H^{3/2} + 0.29\ H^{1/2}$$

$$\therefore\ H = 0.92 \text{ meters}$$

<u>5-Q = 100 M.C./sec.</u>

Vo assumed 1.00 m./sec.

α " 1.85

$$ho = \frac{1.00}{19.6} = 0.05$$

$$100 = 1.875 \times 90 \times H^{3/2} \left(1 + \frac{1.5 \times 1.85 \times 0.05}{H}\right)$$

$$0.592 = H^{3/2} + 0.139\ H^{1/2}$$

$$\therefore\ H = 0.61 \text{ meters}$$

<u>6-Q = 60 M.C./sec.</u>

Vo assumed 0.80 m./sec.

" 1.90

$$ho = \frac{0.8 \times 0.8}{19.6} = 0.034$$

$$60 = 1.875 \times 90 \times H^{3/2} \left(1 + \frac{1.5 \times 1.9 \times 0.034}{H}\right)$$

$$0.356 = H^{3/2} + 0.096\ H^{1/2}$$

$$\therefore\ H = 0.45 \text{ meters}$$

For this value the ratio $\frac{H}{Z} = \frac{0.45}{2.15} = 0.21$ which is close to the critical value of 0.218 given by Francis.

With formula

$$Q = 1.875\ B \left[(H + ho)^{3/2} - ho^{3/2} \right]$$

$$60 = 1.875 \times 90 \left[(H + 0.034)^{3/2} - 0.034^{3/2} \right]$$

$$\therefore\ H = 0.47 \text{ meters}$$

And by neglecting the velocity of approach

$$Q = 1.875\ BH^{3/2}$$

$$60 = 1.875 \times 90 \times H^{3/2}$$

$$\therefore\ H = 0.50 \text{ meters}$$

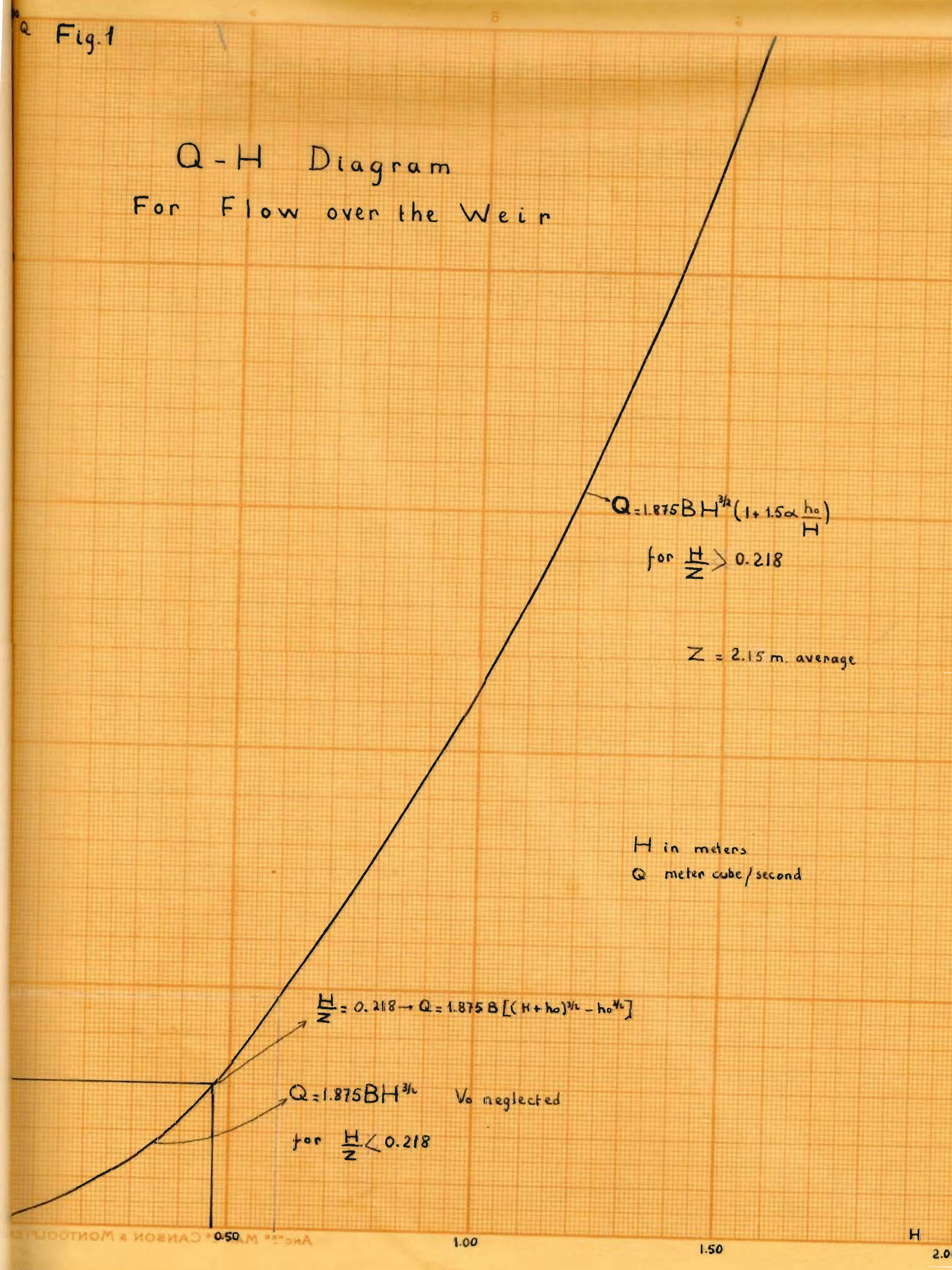


Fig. 1
Q
Q - H Diagram
For Flow over the Weir
$Q = 1.875 B H^{3/2} \left(1 + 1.5 \alpha \frac{h_0}{H}\right)$
for $\frac{H}{Z} > 0.218$
$Z = 2.15$ m. average
H in meters
Q meter cube / second
$\frac{H}{Z} = 0.218 \rightarrow Q = 1.875 B \left[(H + h_0)^{3/2} - h_0^{3/2}\right]$
$Q = 1.875 B H^{3/2}$ V_0 neglected
for $\frac{H}{Z} < 0.218$
0.50
1.00
1.50
H
2.0

It is seen that when $\frac{H}{Z}$ is near to 0.218 and with small velocity of approach the 3 formulas give close results. For values of Q $<$ 60 M.C./sec. as the velocity of approach becomes small, it will be neglected and the formula $Q = 1.875\ BH^{3/2}$ will be used. For Q = 60 M.C./sec. we will retain the value H = 0.47 meters.

7- Q = 40 M.C./sec.

$$40 = 1.875 \times 90 \times H^{3/2}$$

$$H^{3/2} = \frac{40}{1.875 \times 90} = 0.237$$

$$\therefore H = 0.38 \text{ meters}$$

8-Q = 20 M.C./sec.

$$H^{3/2} = \frac{10}{1.875 \times 90} = 0.06$$

$$\therefore H = 0.15 \text{ meters}$$

Figure 1 shows the Q-H diagram

c) Tail water

Past the weir, the water will drop to a new level. This is called the tail water. It is necessary to compute its height in order to have the effective head.

By the equation $Q = AV = LdV$

Q being the discharge cu.ft./sec.

A " " cross sectional area of the water, sq. ft.

V " " velocity of the water, ft.sec.

L " " the width of the river-bed, ft.

d " " the average height of water, ft.

Q and L are known. If V is determined, d can be computed, by Chezy's formula $V = C\sqrt{ms}$

C being a coefficient

m being the hydraulic mean depth in feet

S " the slope of the river bed

Kutter's Formula for C gives

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + (41.65 + \frac{0.00281}{S}) \frac{n}{\sqrt{m}}}$$

The slope of the river bed can be assumed to be 0.01. An average value of n for natural streams is 0.04. L is 200'. The width being much in excess of the height m can be taken as being equal to d.

$$C = \frac{41.65 + \frac{0.00281}{0.01} + \frac{1.811}{0.04}}{1 + (41.65 + \frac{0.00281}{0.01}) \frac{0.04}{\sqrt{d}}} = \frac{87.23\sqrt{d}}{1.68 + \sqrt{d}}$$

$$V = \frac{87.23\sqrt{d}}{1.68 + \sqrt{d}} \times \sqrt{dS} = \frac{87.23 \times 0.1 \times d}{1.68 + \sqrt{d}} = \frac{8.723d}{1.68 + \sqrt{d}}$$

$$Q = LdV = \frac{200\ d \times 8.723\ d}{1.68 + \sqrt{d}} = \frac{1744.6\ d^2}{1.68 + \sqrt{d}}$$

Keeping in mind the relation 1 M.C. = 35 cu. ft.

1. Q = 500 M.C./sec.

$$500 \times 35 = \frac{1744.6\ d^2}{1.68 + \sqrt{d}}$$

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{500 \times 35}{1744.6} = 10$$

d = 6.4' or ∴ 1.95 meters.

2. Q = 400 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{400 \times 35}{1744.6} = 8$$

d = 5.7' or ∴ 1.72 meters.

3. Q = 300 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{300 \times 35}{1744.6} = 6$$

d = 4.75' or ∴ 1.45 meters

4. Q = 200 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{200 \times 35}{1744.6} = 4$$

d = 3.8' or ∴ 1.13 meters

5. Q = 100 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{100 \times 35}{1744.6} = 2$$

d = 2.55' or ∴ 0.78 meters

6. Q = 60 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = 1.2$$

d = 1.9' or ∴ 0.58 meters

7. Q = 40 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{40 \times 35}{1744.6} = 0.80$$

d = 1.52' or ∴ 0.46 meters

8. Q = 20 M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{20 \times 35}{1744.6} = 0.40$$

d = 1.04' or ∴ 0.32 meters

9. $Q = 10$ M.C./sec.

$$\frac{d^2}{1.68 + \sqrt{d}} = \frac{10 \times 35}{1744.6} = 0.20$$

$d = 0.71'$ or $\therefore$ 0.21 meters

B. MAIN HEADGATES

a) Flow in the canal

Specifications of the canal

Bottom level 27.50 meters

Water line 29.00 meters

Discharge 6.00 M.C./sec.

The velocity of the water in the canal should not exceed 1.00 meters/sec. A higher velocity would, in the long run, damage the structure.

An undershot type regulator will be used. This is more economical in construction, and is easier to regulate. It is used on mostly all irrigation schemes. The head required is computed by the formula

$$V = \mu \sqrt{2gh}$$

Assume $\mu = 0.70$ The value of μ is relatively small because in the undershot regulator the water is in contact with the structure on all faces. Therefore

$$1.00 = 0.70 \sqrt{2 \times 9.8h}$$

$$\sqrt{h} = \frac{1.00}{0.7\sqrt{2 \times 9.8}} = 0.35$$

$h = 0.13$ meters say $\underline{h = 0.15 \text{ meters}}$

Assume a total width of the headgates of 6.00 meters

We have $Q = AV$

$A = 6d$

d being the height of the regulator

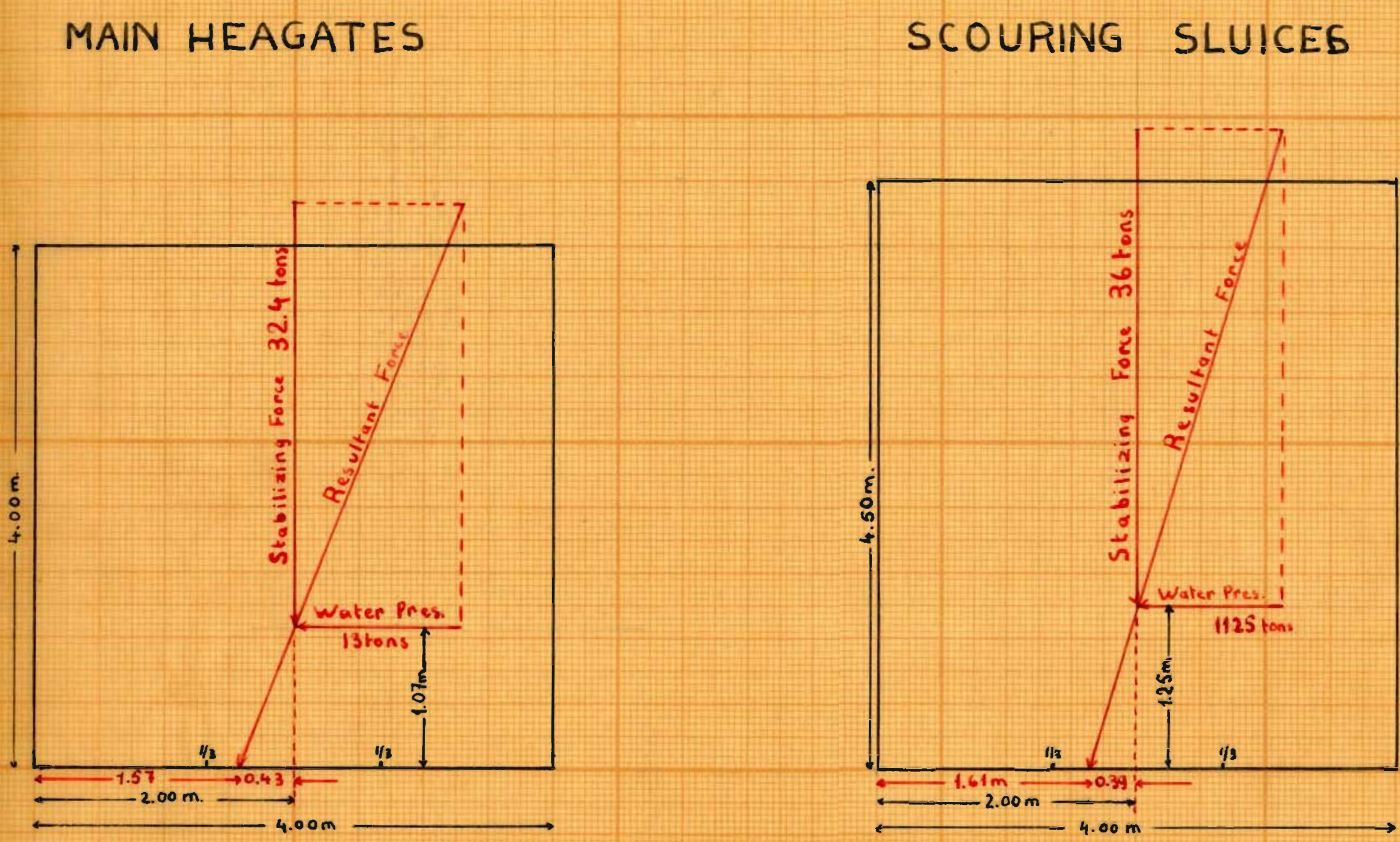
$$Q = 6dV = 6d \quad \text{or} \quad d = \frac{Q}{6}$$

As $Q = 6.00$ M.C./sec

$$d = \frac{6.00}{6.00} = 1.00 \text{ meter}$$

Fig. 2

DESIGN OF PIERS



Space Scale 1/50

Forces Scale 1cm = 5tons

We will use 4 regulators each 1.50 x 1.00 meters.

The minimum required head is 0.15 meters. At the minimum flow of 10 M.C./sec. in the river, the height of the water above the weir is 0.15 meters. This would provide a sufficient head for the flow of 6.00 M.C./sec. in the canal. But considering also the effect of percolation and other losses, the weir will be raised 0.15 meters higher than the water line in the canal. The latter being at an elevation of 29.00 meters, the weir elevation will be 29.15 meters. This will provide sufficient head to keep a constant discharge of 6.00 M.C./sec. in the canal, even with no overflow of the weir.

b) Design of piers (fig. 2)

Maximum water level 29.15 + 1.57 =	30.72 m.
Top level of piers	31.50 m.
Bottom level of canal	27.50 m.
Opening of regulator	1.50 m.
Width of pier	1.00 m.

Width of sluiceway supported by one pier $1.00 + \frac{2 \times 1.50}{2} = 2.50$ m.

The pier is built of concrete having a specific weight of 2.25

The height of the pier above the bottom of the canal is

$31.50 - 27.50 = 4.00$ m.

The length of the pier is 4.00 meters.

1. Stabilizing forces

Average width of the pier is 0.90 meters.

Weight of pier $4.00 \times 4.00 \times 0.90 \times 2.25 = 32.4$ tons

2. Overturning forces

Water pressure $\frac{3.22 \times 3.22 \times 2.5}{2} = 13$ tons

3. Factor of safety $\frac{32.4}{13} = 2.5$

4. Check on overturning

Moments about right edge

$$\Sigma M = 32.4 \times 2 - 13 \times \frac{3.22}{3} = 64.8 - 13.8 = 51 \text{ ton meters}$$

Distance of point of application of the resultant force from the right edge $D = \frac{51}{32.4} = 1.57$ m.

The resultant force falls within the middle third

Eccentricity $\frac{4}{2} - 1.57 = 0.43$ m.

5. Maximum stress on foundation

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

$$S = \frac{32400}{400 \times 90} \pm \frac{32400 \times 43 \times 200 \times 12}{90 \times 400 \times 900 \times 400} = 0.9 \pm 0.60$$

S Max= 1.50 kg/cm^2

S Min= 0.30 kg/cm^2

The pier is safe against crushing

c) Design of walls

1. Panel wall

The regulator being of the undershot type and the gate being much lower than the maximum water line, walls are to be provided in between the piers in order to keep the water from entering into the canal otherwise than by the gates. This wall is to be of reinforced concrete. It will be designed as a slab supported on the 2 piers that it connects.

The width of the wall is 1.50 meters.

Intensity of water pressure at bottom $P = wh$

P being the pressure head in feet

w " " specific weight of water, 1.bs. per cu.ft.= 62.5

h " " height in feet

The average height is 3 meters or 3 x 3.28 = 9.84 ft.

$P = 62.5 \times 9.84 = 310$

Allow 50 % impact 155

465 l.b/sq.ft.

The slab being partly restrained use the formula $M = \frac{wl^2}{10}$

$$M = \frac{465 \times (1.5 \times 3.28)^2}{10} \times 12 = 13450 \text{ in-lb}$$

$$d = \sqrt{\frac{M}{K\ b}}$$

Using a 700 lb. concrete with n = 12

$$K = 106.7$$

$$d = \sqrt{\frac{13450}{106.7 \times 12}} = 3.25 \text{ in.}$$

Add 1 1/2" for cover on each side and 1/4" for reinforcement

depth = 3.25 + 3 + 0.25 = 6.5"

The use of reinforced concrete under water has only been lately introduced and the data available to its behaviour is incomplete. Therefore we will be conservative and use an 8" slab. The same will be applied to the reinforcement.

$$As = \frac{M}{fsjd} = \frac{13450}{16000 \times 0.885 \times 3.25} = 0.292 \text{ sq. in.}$$

Use 1/2" round Mild Steel bars at 5 1/2" c. to c. As = 0.43 sq.in. As the wall is subject to forces due to vibration, and to other indeterminate stresses, the reinforcement will be used on both faces. The beam, at the lower edge of the slab may have to support the pressure exerted by the gate.

Depth of center of gravity of the gate 3.00 m. or 9.8 ft.

Height of gate 1.00 m. or 3.28 ft.

Intensity of pressure

$P = 3.28 \times 9.8 \times 62.5 = 2000$

Allow 50 % impact 1000

Total 3000 l.b.

$$M = \frac{wl^2}{10} = \frac{3000 \times (1.5 \times 3.28)^2}{10} \times 12 = 86000 \text{ in.lb.}$$

$$d = \sqrt{\frac{86000}{106.7 \times 8}} = 10 \text{ in.}$$

Total depth $10 + 1\ 1/2 + 1/2 = 12$ in.

$$As = \frac{M}{fsjd} = \frac{86000}{16000 \times 0.885 \times 10} = 0.60 \text{ sq. in.}$$

Use 3 - 1/2" round Mild Steel bars As = 0.60 sq. in.

Top reinforcement 2 - 1/2" round bars

Bending up 1 bar at $1/5^{th}$ the span As = 0.60 sq. in.

Maximum shear

$$\frac{3000 \times 1.5 \times 3.28}{2} = 7400 \text{ lbs.}$$

Use 1/4" round Stirrups As = 0.10 sq. in.

The spacing of stirrups is given by formula

$$S = \frac{fsjd\ As}{V - v'bjd} = \frac{16000 \times 0.885 \times 10 \times 0.10}{7400 - 40 \times 8 \times 0.885 \times 10} = 3 \text{ in.}$$

2. Side and Wing Walls

The side and wing walls to be of 1:8 cement gravel concrete. The side wall to be vertical on the side facing the water and to have a batter 1/4 on the other side. The wing walls to have a batter 1/11 on both sides.

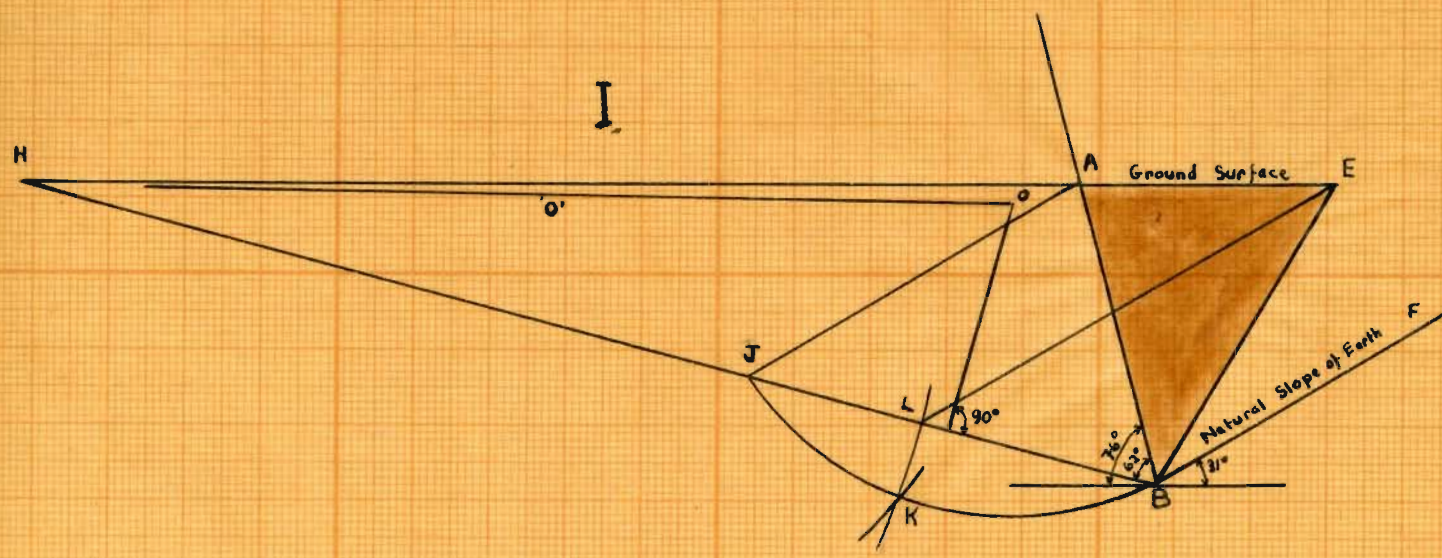
The stabilizing effect of the water pressure has not been considered. The walls will rest over the rock at elevation 26.00 m. and the top will be at elevation 31.50 m.

The dangerous section is at the elevation 27.00 m. for the wing walls and at 27.50 m. for the side walls. These are respectively the elevations of the bed of the sluiceway and the bottom of the canal, below which the counteracting effect of the river material will come into action.

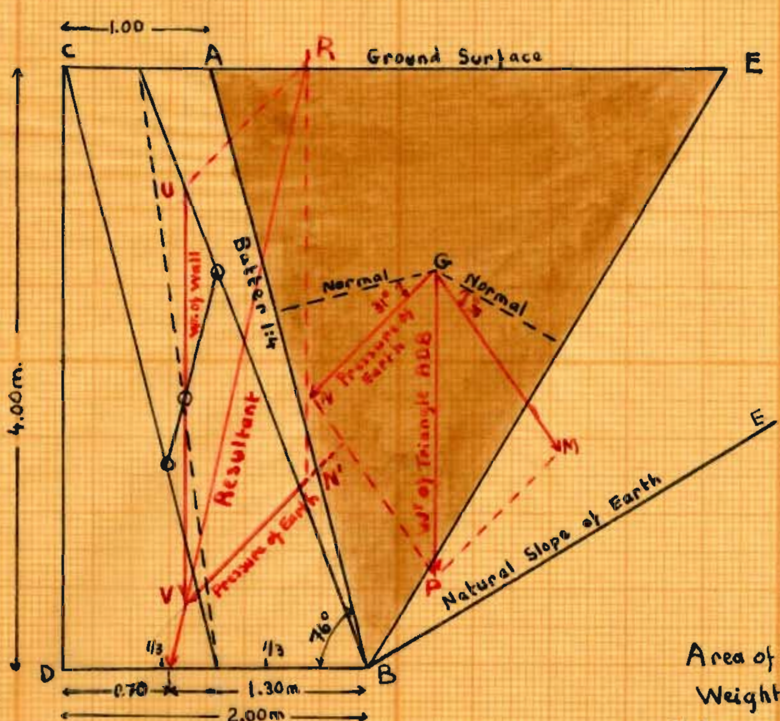
Fig. 3

DESIGN OF SIDE WALLS

I



II Scale 1/50



Area of Triangle AEB $\frac{6.7 \times 8}{2} = 26.8\ cm^2$

Weight " " $26.8 \times 1.50 = 40.2$ "

Using a scale 1 linear cm. to 10 square centimeter GP = 4 cm

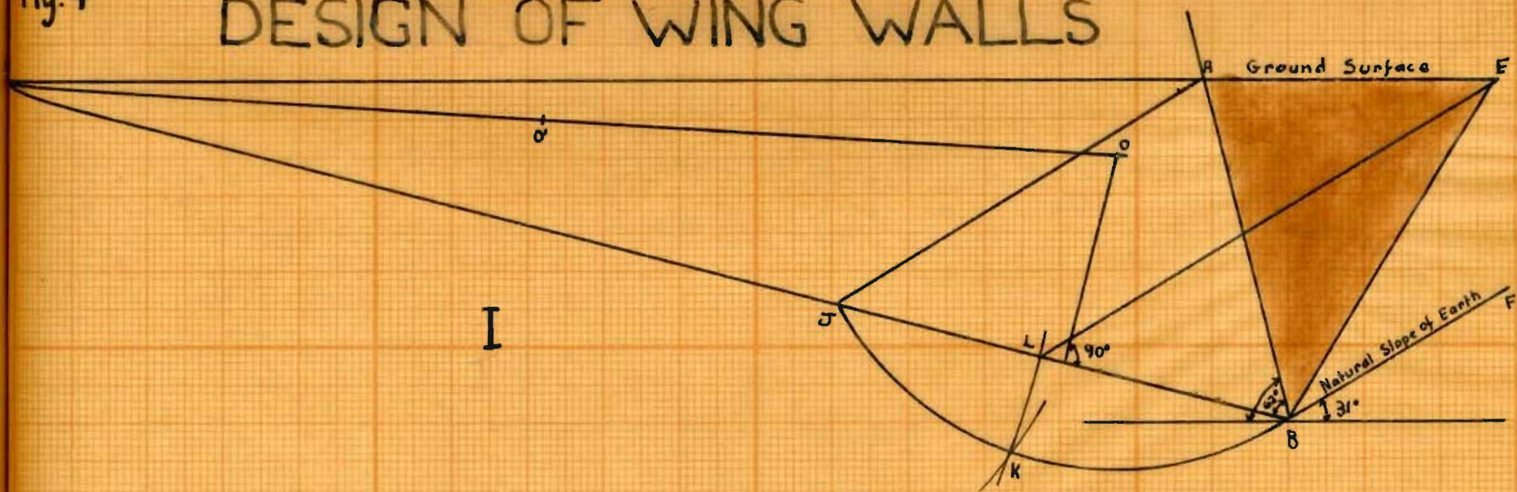
Area of Trap. CADB $\frac{2+4}{2} \times 8 = 24\ cm^2$

Weight " " " $24 \times 2.25 = 56\ cm^2$

UV = 5.6 cm

Fig. 4

DESIGN OF WING WALLS



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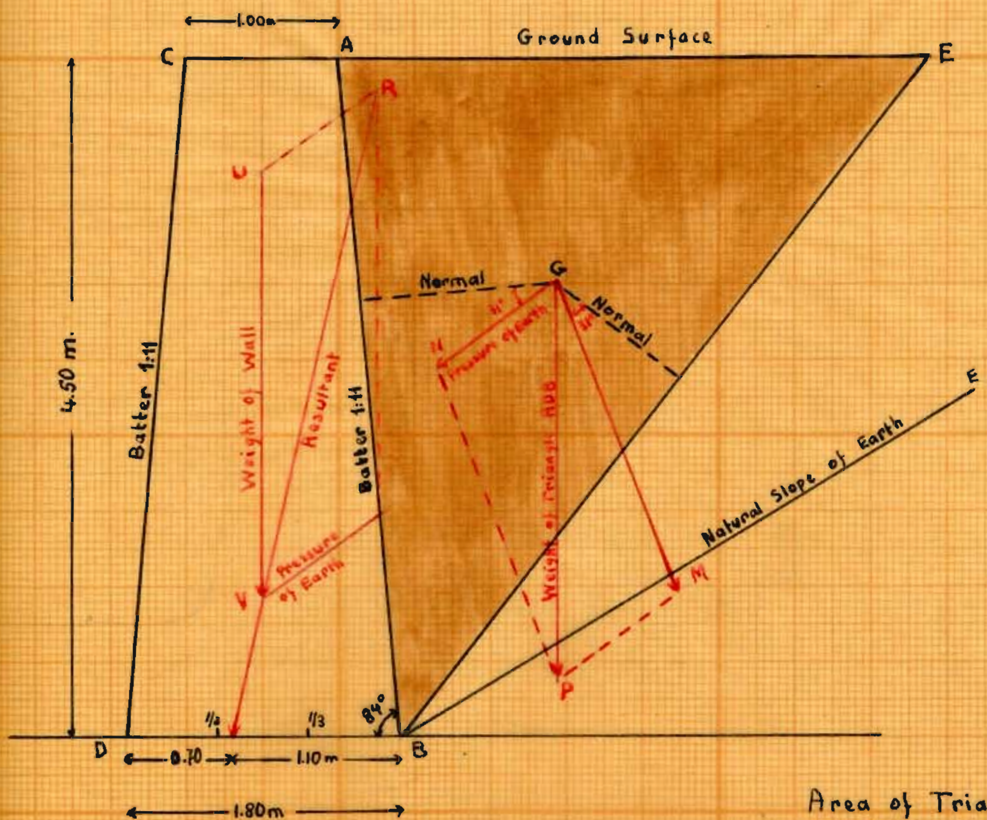


II Scale 1/50

Area of Triangle AEB $\frac{7.8 \times 9}{2} = 35.1\ cm^2$

Weight " " " $35.1 \times 1.5 = 52.73$ "

Using a scale of Forces equal to 1 linear cm = 10 Sq. cms. GP = 5.27 cm.

Area of Trapezoid CADB $\frac{2+3.6}{2} \times 9 = 25.2\ cm^2$

Weight " " " $25.2 \times 2.25 = 56.7$ "

UV = 5.67 cm.

A graphical analysis of the side and wing walls at this section is presented in figs. 3 and 4. The angle of natural slope of the material is taken at 31^0, and the surface of the ground is horizontal. It would be natural to suppose that the part of the earth exerting pressure on the wall would be determined by the angle of natural slope, all material from a natural horizontal grade up to this angle being able to take care of itself, and all the material above the angle needing the wall to hold it in place. Experiment has shown that this is not strictly true, for as the earth settles into place, certain forces of friction and tendencies toward a state of equilibrium come into play, creating internal stresses which produce conjugate pressures. The exact determination of these internal stresses demand very complicated computations which would, alone, become a subject of a thesis. I will refer to a graphical method described in "Kidder's and Parker's" handbook, and which has been found to be sufficiently accurate for all practical purposes.

Consider a slice 1 ft. thick. For determining the prism of earth draw a line parallel to the face of the wall. (Fig 3 and 4,I) Draw BH making an angle ABH, equal to 2 ϕ, with the back of the wall; continue this line until it meets at H the slope of the surface of the earth back of the wall, prolonged. From A, the top of the wall, draw AJ parallel to BF the natural slope of the fill. This has been taken at 31^0. Erect a perpendicular from the middle of JB, and with any point, O, as a center, on this perpendicular, describe an arc passing through J and B. Draw HO and bisect it, and with O' as a center and OO' as a radius describe the arc cutting the arc JKB at K. Again with a radius HK and with H as center, describe the arc KL, and finally, from L, draw LE parallel to JA. The intersection of this line with the surface of the ground locates the point E. The line EB is the line of the cleavage-plane which separates the part of the backing which bears against the wall from the part which exerts no lateral pressure.

The next step is to determine the dimensions of the wall to resist the thrust of the earth. Draw to a larger scale the triangle representing the base of the prism of earth, and find its center of gravity G. (Figs. 3 and 4, II) From this point draw two normals, one to the back of the wall and the other to the line of the cleavage-plane. Draw the two lines, GM and GN, making angles ϕ with these normal. Lay off vertically, from G, at a scale of 1 linear centimeter to 10 square centimeters, the line GP. Resolve this weight-line along GN and GM. This will give the magnitude and direction of the pressure of the earth against the wall. Apply this pressure at a point on the back of the wall, one third the distance from the bottom.

To resist this overturning tendency, the weight of the wall, combined with the pressure of the earth behind it should produce a resultant which falls within the middle third. The foundation laying over a tooth shaped rock, crushing and sliding effects are not considered.

As is seen in figures 3 and 4, II, the resultants fall within the middle third and the walls are safe.

d) Checking Devices

The main object of the headgates is to keep a constant flow in the canal under different heads. This is done by regulating the opening of the gates. We will compute the opening of the gates for a constant flow in the canal under different heads.

We have $q = AV$

$A = 6d$ d being the opening of the gates, in meter

$V = 0.7\sqrt{2gh} = 0.7 \times 4.43\sqrt{h}$

$q = 6d \times 0.7 \times 4.43\sqrt{h}$

but $q = 6.00$ M.C./sec.

therefore $d = \frac{6}{6 \times 0.7 \times 4.43\sqrt{h}} = \frac{0.322}{\sqrt{h}}$

1. Q = 500 M.C./sec.

$h = 1.57 + 0.15 = 1.72$ meters

$d = \frac{0.322}{\sqrt{1.72}} = \underline{0.246 \text{ meters}}$

2. Q = 400 M.C./sec.

$h = 1.39 + 0.15 = 1.54$ m.

$d = \frac{0.322}{\sqrt{1.54}} = \underline{0.26 \text{ m.}}$

3. Q = 300 M.C./sec.

$h = 1.18 + 0.15 = 1.33$ m.

$d = \frac{0.322}{\sqrt{1.33}} = \underline{0.28 \text{ m.}}$

4. Q = 200 M.C./sec.

$h = 0.92 + 0.15 = 1.07$ m.

$d = \frac{0.322}{\sqrt{1.07}} = \underline{0.31 \text{ m.}}$

5. Q = 100 M.C./sec.

$h = 0.6' + 0.15 = 0.76$ m.

$d = \frac{0.322}{\sqrt{0.76}} = \underline{0.37 \text{ m.}}$

For a flow of 60 M.C./sec and less the height of 0.15m. will be omitted, because at this value the losses due to percolation etc. will become sensible.

6. Q = 60 M.C./sec.

$h = 0.47$ m.

$d = \frac{0.322}{\sqrt{0.47}} = \underline{0.47 \text{ m.}}$

Fig. 5

Diagram Showing the Head and the Corresponding Opening of the Regulators For a Constant Flow of 6.00 M.C. in the Canal

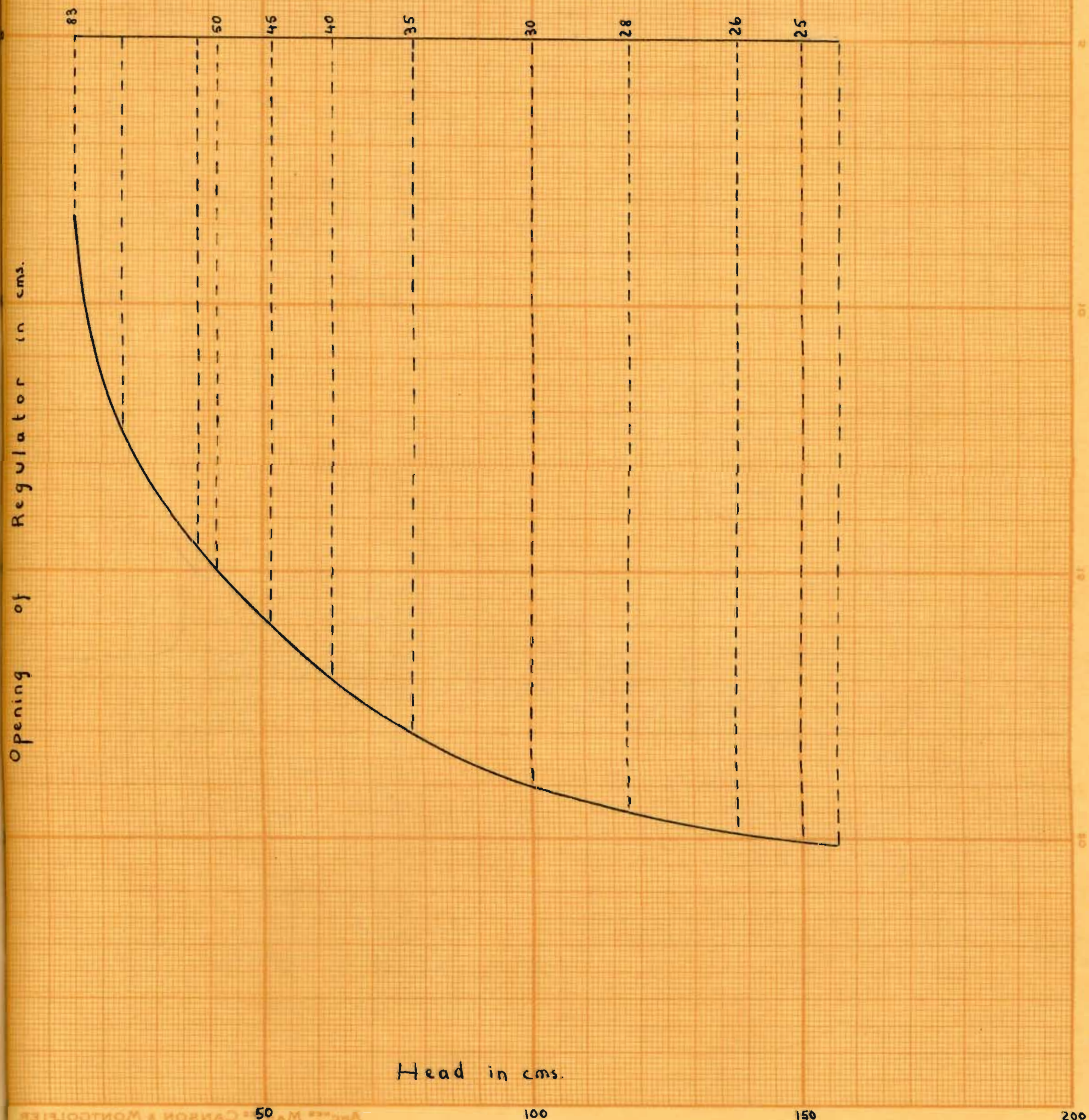
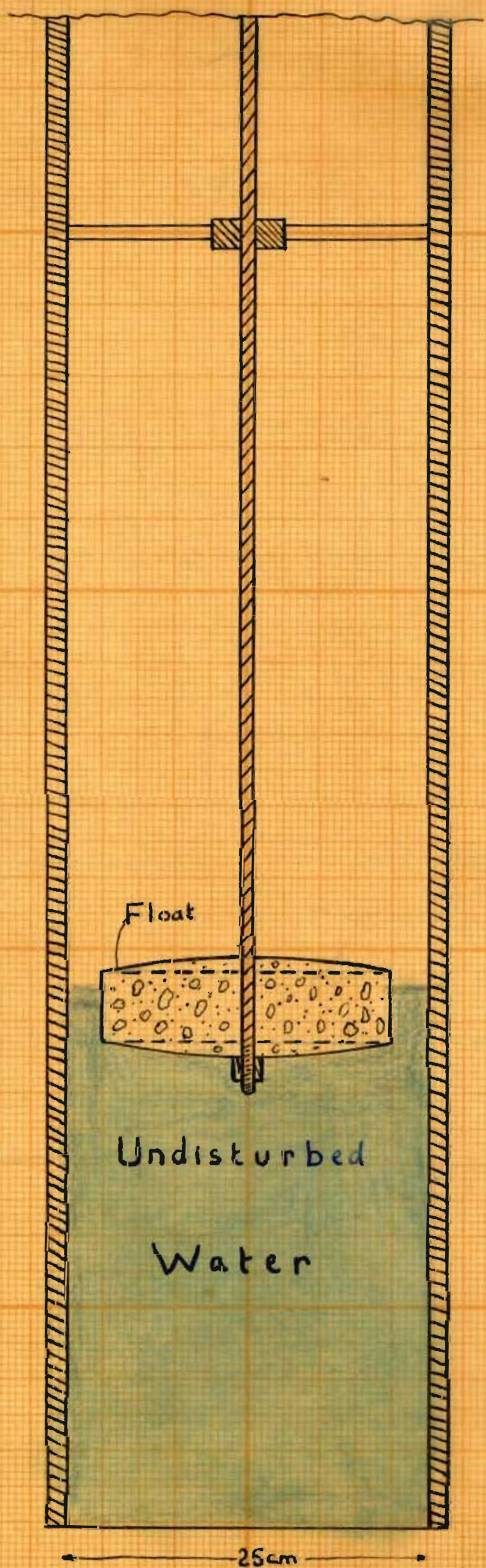
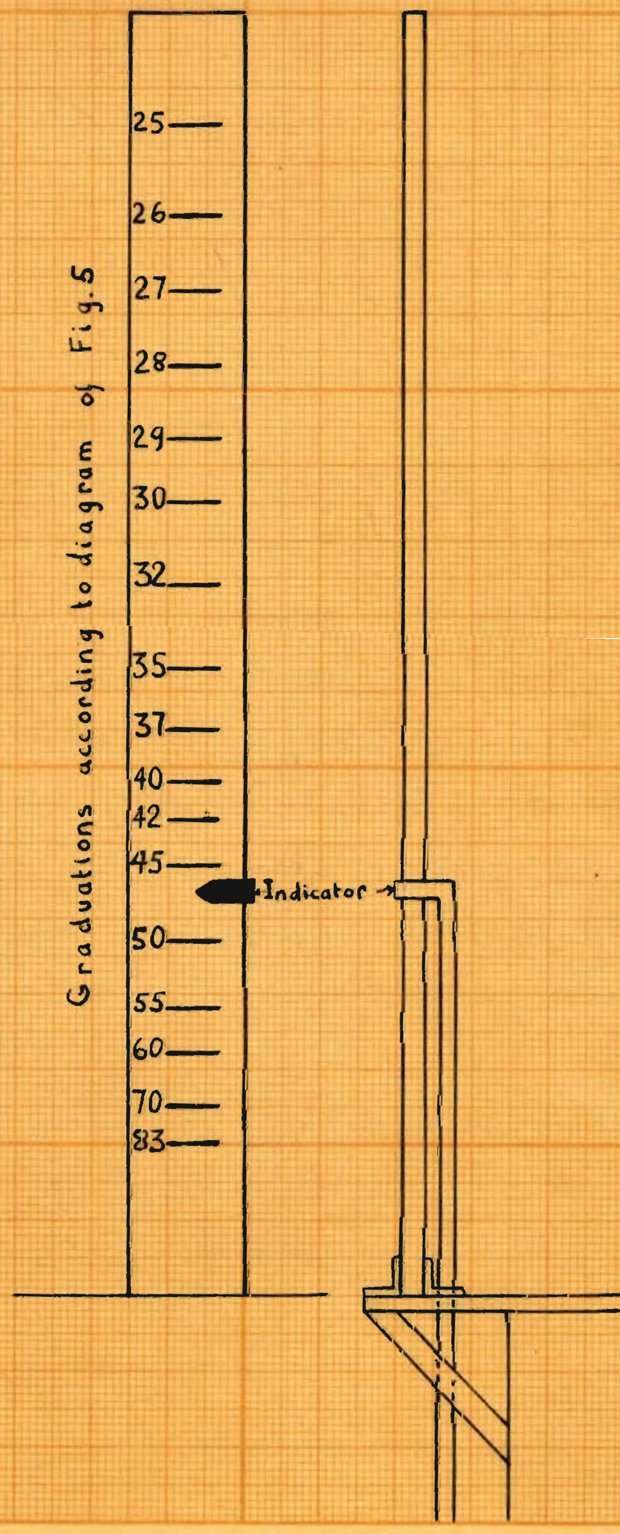


Fig. 6



CHECKING DEVICE

Scale 1/5



7. Q = 40 M.C./sec.

$$h = 0.38 \text{ m.}$$

$$d = \frac{0.322}{\sqrt{0.38}} = 0.52 \text{ m.}$$

8. Q = 20 M.C./sec.

$$h = 0.24 \text{ m.}$$

$$d = \frac{0.322}{\sqrt{0.24}} = 0.61 \text{ m.}$$

9. Q = 10 M.C./sec.

$$h = 0.15 \text{ m.}$$

$$d = \frac{0.322}{\sqrt{0.15}} = 0.83 \text{ m.}$$

Fig. 5 gives a graphical representation of this variation.

By projecting this curve over the abcissa, which represents the head, we will obtain a scale whose graduations correspond to the opening to give to the regulator to keep a constant discharge in the canal.

This scale is then inverted and printed over a board.

A perforated cast iron pipe is fixed in the river bed. The water is allowed to penetrate the pipe only from the holes, and therefore it is undisturbed. A cylindrical float is placed in the pipe. The float is bolted to a 20 m/m round M.S. bar which has at the upper end an indicator sliding along the graduated board.(Fig. 6)

When the water level in the river varies, so does the indicator and shows the corresponding opening that the gates should have.

Other boards having a metric scale graduation are placed over each gate. The indicator is connected to the lifting attachment. These boards indicate the opening of the gates.

If electric power was available, it would have been possible to install an electric regulator. As this is not the case, the regulation has to be done by hand. The operator will read the figure indicated by the first scale and then will manipulate the lifting devices of the gates so as to obtain the same reading on the other scales.

This simple device will permit to keep a fairly constant flow in the canal.

e) Operating Platform

The operating platform to be of reinforced concrete and to have a width of 1.5 meters or 5 feet. Every slab will be freely supported over the piers in order to prevent cracks in case any one of the piers settles.

The live load is taken at 200 l.bs. per sq. ft.

The own weight is assumed to be 75 " " " "

Total 275 " " " "

$$M = \frac{wl^2}{8} = \frac{275 \times 5 \times 5}{8} \times 12 = 10600 \text{ in. l.bs.}$$

$$d = \sqrt{\frac{10600}{106.7 \times 12}} = 2.9 \text{ in.}$$

Depth 2.9 + 2 + 0.25 = 5.15 in. say 6. in.

$$As = \frac{10600}{16000 \times 0.885 \times 2.9} = 0.26 \text{ sq. in.}$$

Use 1/2" round M.S. bars at 8 in. C. to C. As = 0.29 sq. in.

f) Gates

As the gates to be used for the scouring sluices have the same dimensions and are under nearly similar conditions, for simplicity in construction, a single design will apply for both of them. The bottom of the scouring sluice gate is at elevation 27.00. The maximum water level is 30.72 m. The pressure exerted by the water over the gate is equal to the average pressure times the area of the gate. But as the depth of the gate is small compared to the maximum height of water and also because the error is on the safe side, the pressure over the gate will be considered as equal all throughout to that existing at the bottom.

The gate is to be of a steel sheet reinforced with standard steel channels.

Total pressure over a meter width of gate

W = 3.72 x 1000 = 3720 kg/sq. meter

Pressure acting over 1 channel $\frac{3720}{4}$ = 925 kg. per meter run

Allow 50 % impact 463

1388 kg.

$$M = \frac{wl^2}{8} = \frac{1388 \times 1.5 \times 1.5}{8} = 390 \text{ kg.m.}$$

or 390 x 2.2 x 3.28 x 12 = 33800 in. lbs.

At a unit stress of 18000 lb./sq.in $\frac{I}{C} = \frac{33800}{18000} = 1.88$

Use channels 3 x 148 in. at 9.00 lb. per ft $\frac{I}{C} = 2.10$

Use a 3/8 in. Steel plate. The bearing surfaces of the sides are lined with bronze plates to give a smoother and better wearing bearing surface.

g) Gate Lifting Devices

The operation of the gate requires the application of a force which will overcome the weight of the gate and frictional resistance produced by the pressure on the gates.

According to Rankine the coefficient of static friction of metals over metals varies from 0.15 to .25. According to A.L. Hardis this figure seems low, and has to be increased in order to take care of the additional friction that comes from the rust. Therefore chose a coefficient of 0.40.

Pressure over the gate 3.22 x 1.5 x 1000 = 4830 kg.

Friction 40 %		1932 Kg.
Gate 4 channels	100 kg.	
Steel plate	118 kg.	
Variations	100 kg.	
	318 kg.	318 kg.
		2250 kg.

The opening force is 2250 kg. and the closing force is 1614 kg. The gate lifting device is to be able to exert an opening as well as a closing force. It is attached to the piers, so that the operating platform will not be subject to this uplift due to the closing force. The span being in excess of the depth, twin lifting attachments will be used. The device consists of a rack fastened to the gate, a geared pinion to transmit the force to the rack, additional gears to give greater mechanical efficiency, and an operating wheel with an arm rigidly connected to the same axle as the pinion.

This device can be obtained from manufacturers who make a speciality of the construction of gate lifts for irrigation structures. The relation between the operating force and the total pull exerted on the gate is expressed by the equation

$$F = \frac{Prr_1}{e^2RR_1}$$

for a triple spur geared device.

r being the radius of pinion fixed on the same axle as the operating wheel.

r_1 being the radius of pinion acting on rack.

R " " " " operating wheel.

R_1 " " " " main geared wheel.

e " " efficiency of gearing and bearing, 90% for cut teeth.

$$F = \frac{2250}{0.81} \times \frac{rr_1}{RR_1} = 2780 \frac{rr_1}{RR_1}$$

The value of F has to be less than 30 kg.

By taking $\frac{rr_1}{RR_1}$ = 100 from a catalogue issued by the manufacturers,

we get F = 27.80 kg.

A man can apply this force at the rate of 30 meters per minute

The time necessary to lift the gate $\frac{2250}{27.8 \times 30}$ = 2 minutes 35 seconds.

h) Spillway

Although the regulator is designed to keep a constant flow in the canal, practically it never fulfills its function perfectly, and the water level in the canal can exceed the allowable limit. In addition, an obstruction in the canal, formed either by some material or body falling in the canal or by closing checkgates farther down in the canal can raise the water level of the canal.

This might cause damage to the structure and therefore is undesirable. A spillway is to be provided. This consists of a simple opening in the canal so that the unnecessary amount of water discharges at right angle to the direction of the flow in the canal. Its position is located a short distance beyond the headworks. The maximum amount of water to be discharged is assumed to be 1/4 of the flow in the canal or 6/4 = 1.5 M.C./sec. Due to the increase of the flow the water level of the canal will increase by 0.20 m.

The length of spillway is given by the formula

$$Q_1 - Q_0 = 2/_3\, u\, b h_1 \sqrt{2 g h_1}$$

Q_1 being the discharge before the spillway

Q_0 " " " after " "

u " coefficient

h_1 " the drop

$$1.5 = 2/3 \times 0.80 \times b \times 0.20 \sqrt{2 \times 9.8 \times 0.20} \quad \therefore b = 7.20 \text{ m.}$$

Engels, after many experiments, has found that the above given formula is not accurate. He gives the formula

$$Q_1 - Q_0 = 2/3\, u \sqrt{2g} \sqrt[3]{b^{2.5} \times h^5}$$

b = 15.2 say 16 meters

C - DIVERSION WEIR

a) Choice of Weir Length

The main object of a diversion weir is to raise the water level in the river to divert the desired flow in the canal through the headgates. The location of the weir is imposed by the existence of the canal.

The length of the weir affects the height to which the water will be backed up on the upstream side. From consideration of economy, after several trials, a length of 90 meters has been adopted. This figure could have been reduced to 35 or 40 meters. But in this case the disadvantages would be

(a) the use of a dividing wall about 15 meters long to form a sluiceway.

(b) the maximum water level would be

Vo assumed 4 M./sec.

μ " 1.15

$$ho = \frac{Vo^2}{2g} = \frac{16}{19.6} = 0.84$$

$$500 = 1.875 \times 40 \times H^{3/2} \left(1 + \frac{1.5 \times 1.15 \times 0.84}{H}\right)$$

$$6.66 = H^{3/2} + 1.5\,H^{1/2}$$

$$H = 2.60 \text{ meters.}$$

This means an increase of over one meter in the head, and would necessitate higher and stronger piers, wing panel and side walls, gates and gate lifting devices, and also a stronger weir.

All advantages and disadvantages being considered, the 90 meter weir has been found more economical and has been adopted.

Investigations on the site have shown the existence of a formation of uncracked limestone at an elevation of 26.00 - 26.25 meters, which covers all the area of the headworks. The surface of the rock should be chipped off to remove the impurities and to give a rough surface to the rock.

The weir is to be of 1:8 cement ballast plain concrete, having a specific gravity of ρ = 2.25. In some instances the weir is going to be wholly submerged. It is well to assume under such conditions that the specific gravity of the weir should be reduced to $\rho - 1 = 2.25 - 1 = 1.25$. Actually, however, the resistance of the weir wall to overturning relative to its base at floor level is not impaired by floatation, but as weight in this case is a desideratum, the weir wall should be designed as if this were the case.

Bligh gives an approximate rule for the base and crest widths of a trapezoidal weir

$$\text{Base} = \frac{Z + H}{\sqrt{\rho}} \quad \text{or} \quad \frac{Z + 0.6H}{\sqrt{\rho}}$$

$$\text{Crest} = \sqrt{H} + \sqrt{Z}$$

Z = height of weir, in feet.

H = height of water level above the crest, in feet

ρ = specific gravity.

In our case H = 1.57 m. or 5.15 ft.

Z = 29.15 - 26.00 = 3.15 m. or 10.3 ft.

$$\text{Base} = \frac{10.3 + 5.15}{\sqrt{1.25}} = 13.80 \text{ ft. or } \underline{4.20 \text{ meters}}$$

$$\text{Crest} = \sqrt{5.15} + \sqrt{10.3} = 5.48 \text{ ft. or } \underline{1.70 \text{ meters}}$$

b) Shape of the Weir

The "Ogee" weir has been found to suit the best to our conditions. The downstream face is formed to fit the curve of the underface of the falling water. Its advantages are :

1. The water will hit the floor horizontally, so that it produces no impact, and a thick floor will not be required.

2. The water will always be in contact with the weir, and no vacuum is produced on the downstream face

3. The friction losses will become smaller than in the case of a trapezoidal weir.

The "Ogee" is formed by the combination of several curves. The "National Resources Committee, Washington, D.C."; recommends the following shape:

The top curve to be a parabola whose equation is

$$x^2 = 1.97\ Hy$$

x and y being the coordinates

H = the maximum water height above the crest

$$x^2 = 1.97 \times 1.57y = 3.09y$$

This curve will stop at the point x = 0.75y or $\frac{y}{x} = \frac{1}{0.75}$

By differentiating the equation $x^2 = 3.09y$ and equating it to $\frac{1}{0.75}$

$$\frac{dy}{dx} = \frac{2\ x}{3.09} = \frac{1}{0.75}$$

$$x = \frac{3.09}{2 \times 0.75} = 2.06$$

Taking different values of y and solving for x

y = 0.05	x = 0.39
y = 0.10	x = 0.55
y = 0.20	x = 0.78
y = 0.40	x = 1.13
y = 0.60	x = 1.36
y = 0.80	x = 1.56
y = 1.00	x = 1.75

Fig. 7

TRAPEZOIDAL WEIR

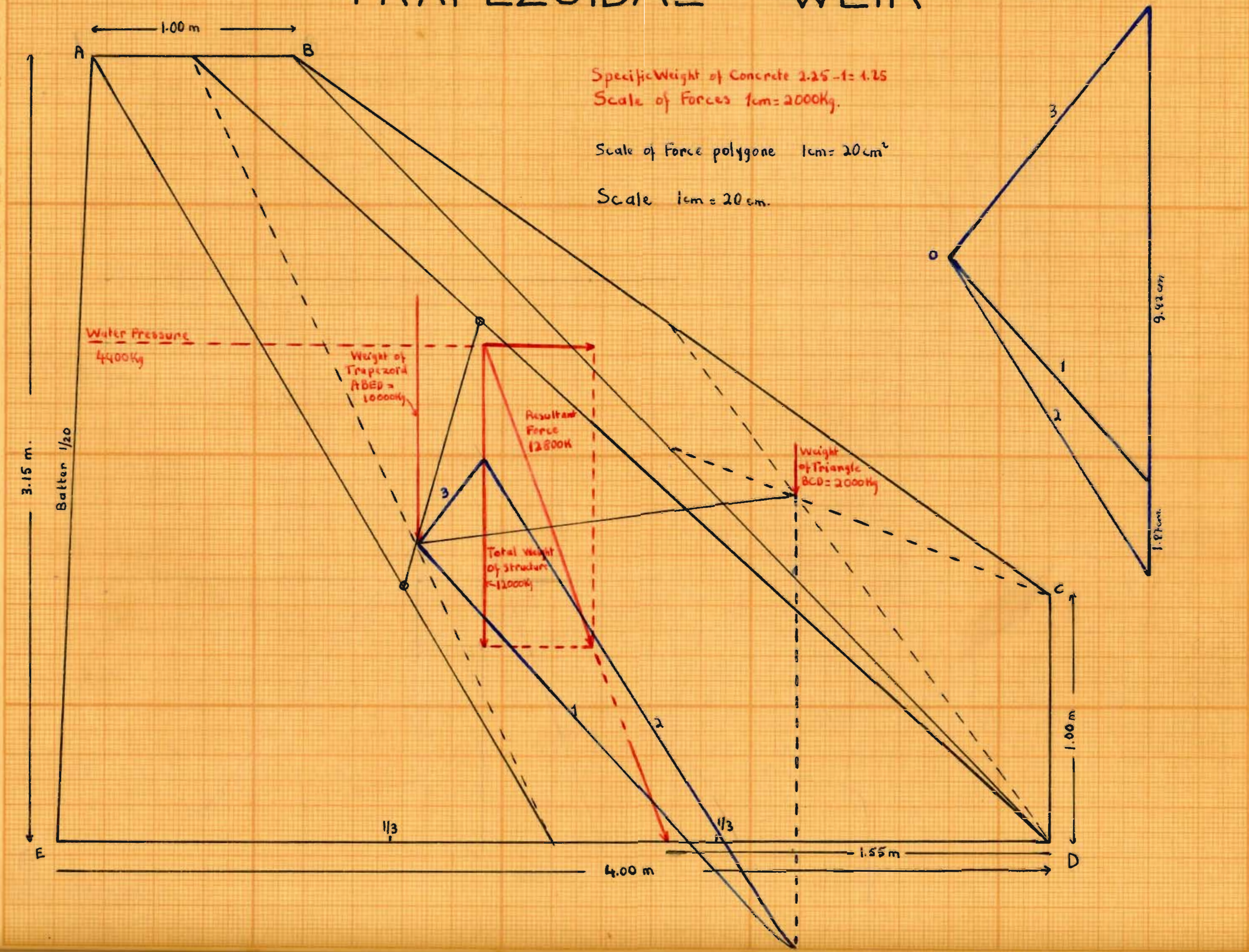
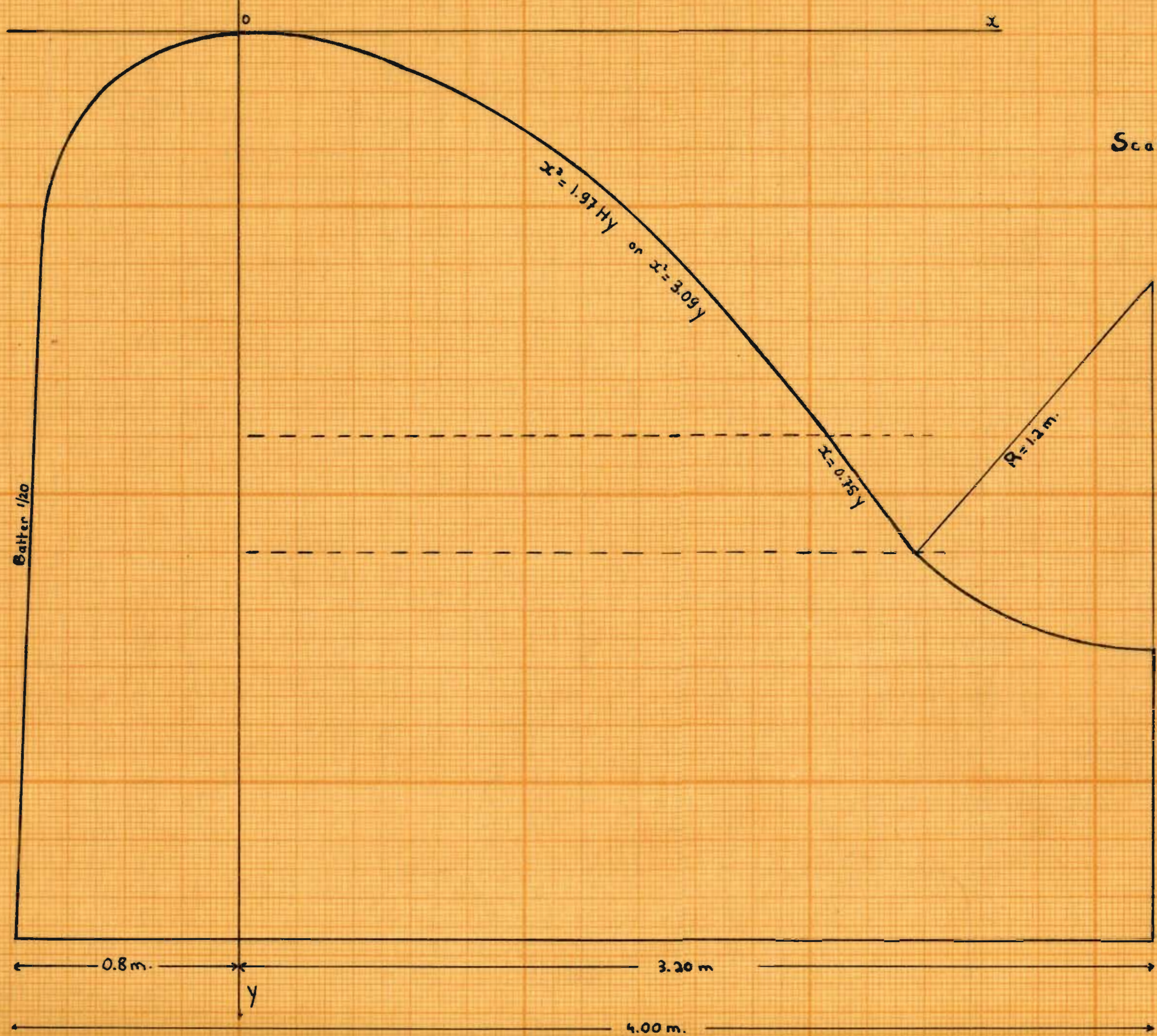


Fig. 8

OGEE WEIR

Scale 1cm = 20cm.



$y = 1.20$ .... $x = 1.91$

$y = 1.40$ .... $x = 2.06$

The parabola, at the point $x = 2.06$, is to be followed by a straight line having an inclination $\frac{x}{y} = 0.75$. This line is tangent to the parabola. After the straight line an arc of a circle should come, having a radius $R = 1.20$ m. This reverse curve will deflect the water horizontally.

The "ogee" weir is shown in figure 8.

The result of an "Ogee" shape is the elimination of impact, but the water is discharged at the foot of the "Ogee" with a high velocity, which requires the protection of the streambed for some distance downstream. According to a number of authorities on engineering practice in India, the use of "Ogee" falls in India have been abandoned and replaced by direct falls on a strong apron for the reason that the action of the high scouring velocity resulting from the "Ogee" was more difficult to protect against than the force of impact. This difficulty I believe, is largely due to the fact that most of the Indian weirs are built on soft foundations of silt or sand, and the above objections do not hold true for the case of our weir.

c) Stability of the Weir

The conditions for stability of the weir against hydrostatic pressure are

1. There must be no tension in the plane between the weir wall and the foundation.
2. There must be no overturning.
3. There must be no tendency to slide on the joint with the foundation or on any horizontal plane above the base.

4. The maximum pressure on any plane or on the foundation must not exceed the prescribed limit.

Fig. 7 shows a trapezoidal weir. Fig. 8 has been drawn according to the recommendations of the "National Resources Committee" on "Ogee" weirs. The two weirs have nearly the same cross section and the same moment of Inertia relative to the base. In order to simplify the work, the check on the stability of the weir has been determined from a graphical analysis of the trapezoidal weir.

The forces acting over the weir are :

1. The normal water pressure on the upstream face; the maximum head is (29.15-27.00) + 1.57 = 3.72 meters.
2. The normal water pressure on the downstream fall due to tail water. This has been found to be 1.95 m., but 0.75m. only will be considered as being effective.
3. The weight of the water supported by the crest. This force is small and adds to the stability of the weir. By neglecting it we will be on the safe side.

Fig. 7 gives the following results :

1. The resultant force falls well within the middle third. So there is no tension in the plane between the weir wall and the foundation.
2. The factor of safety against overturning is $\frac{12}{4.4} = 2.72$
3. The sliding force is 4.4 tons. The resisting force is 12 x 0.6 = 7.2 tons, without considering the additional resistance which is due to the toothed shape. .
4. The maximum pressure on the foundation is given by the formula $S = \frac{P}{A} \pm \frac{Mc}{I}$

$$P = 12000 \text{ kg.} \qquad M = Pe$$

$$e = \text{eccentricity} = \frac{4}{2} - 1.55 = 0.45 \text{ m. or } 45 \text{ cm.}$$

$$S = \frac{12000}{400 \times 100} \pm \frac{12000 \times 45 \times 200 \times 12}{100 \times 400 \times 400 \times 400} = 0.3 \pm 0.20$$

$$S \text{ Max} = 0.50 \text{ kg/cm}^2$$

$$S \text{ Min} = 0.10 \text{ kg/cm}^2$$

The weir is safe.

The weir being built over rock, percolation and uplift are not considered.

d) Emergency gates.

In periods of flood, when the water level increases above the highest expected, the emergency gates will prevent the destruction of the headworks. Actually such a flood might never happen. But as we often hear about destructions of headworks due to floods, and the enormous losses resulting from them, it is wise to invest some money to protect the structures and the neighbouring lands. The construction of these emergency gates is quite simple and will not cost much. In normal periods, the openings are closed with stoplogs groved in the piers. In periods of flood these stoplogs are removed and the water flows from these openings, which have a total width of 6.00 meters.

The amount of water relieved will depend upon the flow. The calculations for a flow of 500 M.C. will be presented.

The water flowing through the emergency gates will be considered of being formed of 2 layers, flowing one over the other. the top layer is assumed to obey the law of discharges through weirs

$$q_1 = 1.875 \, BH^{3/2} \left(1 + 1.5\alpha \frac{ho}{H}\right)$$

$B = 6.00$ meters

$H = 1.57$ meters

$$q_1 = 1.875 \times 6 \times 1.57^{3/2} (1 + 1.5 \times 1.15 \times \frac{0.46}{1.57})$$

$$q_1 = 33 \text{ M.C./sec.}$$

The bottom layer obeys the law $Q = AV$. The bottom of the gates is at elevation 27.00 meters. V is equal to 3.00 M./sec.

$$q_2 = 6.00 \times 2.15 \times 3 = 39.00 \text{ M.C./sec.}$$

Total discharge $q_1 + q_2 = 33 + 39 = 72$ M.C./sec.

This means the emergency gates are able to relieve the weir by discharging about 14-15 % of the flow.

e) Foot Bridge

The foot bridge is needed:

1. To pass from one bank of the river to the other one.
2. To inspect the weir and locate deficiencies.
3. To operate the emergency gates.

The bridge will be supported at 6.00 meter intervals by concrete walls resting over the weir.

The width of the bridge will be 1.50 meters to allow two persons to cross each other.

The bridge is supposed to carry a uniform load of 100 lb./sq.ft. plus a concentrated load of 3000 lb.

1. Design of Slab

Assuming the width of a beam to be 10 in. the clear span of the slab will be 5'-0" - 2 x 10" = 3'4". The concentrated load will be assumed as distributed over a width of slab of 2 ft.

$$M = \frac{wL^2}{10} + \frac{3000}{2} \times \frac{L}{4} = \frac{100x(3.33)^2}{10} \times 12 + \frac{3000}{2} \times \frac{3.33}{4} \times 12$$

$$M = 1340 + 15000 = 16340 \text{ in. lbs.}$$

$$d = \frac{16340}{106.7 \times 12} = 3.58 \text{ in.}$$

According to practice the spacing of stirrups should not exceed 3/4d. 21 x 3/4 = 15.75 sqy 15 inches.

The same spacing will be used throughout the beam, except at the place where the bars are bent up, because these bars will take care of the shear.

The unit bond stress is given by formula

$$u = \frac{V}{\Sigma o j d} = \frac{9000}{4 \times 2.36 \times 0.885 \times 21} = 51 \text{ lb./sq.in.}$$

in which 2.36 is the perimeter of the 3/4 in. rods.

3. Design of Supports

The load transmitted by 1 beam is equal to

$$2 \times 6000 + 3000 = 15000 \text{ l.bs.}$$

Using a 600 l.b. concrete, the required area of the supportwill be

$$\frac{15000}{600} = 25 \text{ sq. in.}$$

Two 5 x 5 in. square reinforced concrete columns could be used to support the bridge. But considering also the effect of impact and the scouring effect of moving water, a 30 cm. thick plain concrete wall will be used in lieu. This wall will extend through the width of the bridge and will be monolithically cast with the weir.

D- SCOURING SLUICES

a) Purpose

The main objects of a scouring sluice are :

1. To maintain a well defined channel in front of the head-gates to the canal by scouring the silt or sand deposited in front of the gates.
2. To prevent the entrance into the canal of the coarse material carried by the river water.

The river bed being of sand and gravel, the presence of the weir will oblige the particles (silt, sand, fine gravel) carried in suspension in the water, to be deposited on the upstream side of the weir. The erosive effect of the flood flows will not be sufficient to wash out the deposits.

In the Litani river the silt deposition is not excessive as in the case of rivers flowing through sand. The scouring sluices will only be located near the headgates. On the other end of the weir the emergency gates will also help in scouring the deposits.

b) Design of Piers

Maximum upstream water level	30.72 m.	
Tail water assumed as effective	27.75 "	
Bottom level of gates	27.00 "	
Opening of gate	1.50 "	
Width of pier	1.00 "	
Width of water supported by one pier $1.00 + \frac{2 \times 1.5}{2} =$	2.50 "	
Length of piers	4.00 "	
Height of water 30.72-27.75 =	3.00 "	(approx)
Height of weir 31.50-27.00 =	4.50 "	

1. Stabilizing forces

Weight of pier 4.00 x 0.90 x 4.50 x 2.25 = 36.4 tons

2. Overturning forces

Water pressure $\frac{3.00 \times 3.00}{2} \times 2.5 = 11.25$ tons.

3. Factor of safety $\frac{36.4}{11.25} = 3.23$

4. Check on overturning

Moments about right edge

$$M = 36.4 \times 2 - 11.25 \frac{(3.75)}{3} = 58.75 \text{ Ton-meters.}$$

Distance of the point of application of the resultant force from the right edge

$$D = \frac{58.75}{36.4} = 1.61 \text{ meters.}$$

Eccentricity $\frac{4.00}{2} - 1.61 = 0.39$ meters.

5. Maximum stress $S = \frac{P}{A} \pm \frac{MC}{I}$

$$= \frac{36400}{400 \times 90} \pm \frac{36400 \times 39 \times 200 \times 12}{90 \times 400 \times 400 \times 400}$$

$$= 1.01 \pm 0.59$$

$$S \max = 1.60 \text{ kg./cm}^2$$

$$S \text{ Min} = 0.42 \text{ kg./cm}^2$$

6. The preceding analysis shows that by using for the scouring sluices the same piers as for the main headgates, the factor of safety becomes higher. But this not being excessive, and for the sake of simplicity in construction, the same design will apply for both of them.

C) <u>Gates and gate lifting devices.</u>

The same as those designed for the main headgates.

d) Operation

The scouring sluices will not be operated at maximum flood period. We will assume that the maximum flow at which they will be operated is 200 M.C./sec.

Elevation of upstream water at 200 M.C./sec	29.15 + 0.92	= 30.07 m.
" " tail " " " "	27.00 + 1.13	= 28.13 m.
Head		1.94 meters.

Average head up to the center of gravity of opening $1.94 - \frac{1.00}{2} = 1.44$ meters.

$$V = 0.7\sqrt{2gh} = 0.7\sqrt{2 \times 9.8 \times 1.44} = 3.72 \text{ M./sec.}$$

Elevation of upstream water at 60 M.C./sec	29.15 + 0.47	= 29.62
" " tail " " " "A	27.00 + 0.58	= 27.58
Head		2.04 m.

Average head up to the center of gravity of opening $2.04 - \frac{1.00}{2} = 1.54$ m.

$$V = 0.7\sqrt{2 \times 9.8 \times 1.54} = 3.84 \text{ M./sec.}$$

The velocities found are effective and are able to transport even small gravels. The actual velocities are higher than those computed, because the velocity of approach has not been considered.

According to some authorities in irrigation practice, the scouring is effective up to a distance upstream equal to 10 times the opening of the scouring gate. As our gate is 1.50 meters wide, this distance will be equal to 15 meters. Thus the scouring velocity wall extend up to a distance beyond the end of the regulator openings, and the operation of the scouring sluices will clear the entrance to the canal from deposits.

But as in our case the scouring velocity is high, probably in certain cases above 4.00 m/sec., we will consider the scouring as effective up to a distance equal to 20 meters upstream of the scouring sluices. All this area will be reinforced by a layer of plain concrete 40 cm. thick.

According to Bligh the downstream floor has to be 25 to 50% longer than the floor of the weir. The latter being 6.00 meter, the downstream floor, by adding 50%, will be 9.00 meters. Again, a 40 in. thick plain concrete reinforcement will be used.

After extensive experiments on the Sutlej River in India, it has been found that the application of the following points when operating the sluices helps the removal of the deposits.

1. Keep the canal headgates closed during the period of flood flows when the maximum amount of silt is carried. As the maximum flood of the Litani occurs in February or March, when the irrigation water is not needed, there is no trouble in doing so.
2. Keep the scouring sluice gates closed as long as possible when the headgates are open, and when necessary, to open only part of the sluiceway gates, and those farthest away from the headgates in order to concentrate the scouring channel a short distance away from the headgates.
3. When necessary to scour out the material deposited in the sluiceway, close completely the canal headgates.

Much attention has been given to the scouring sluices, because they are an important element of the Headworks, and on the other hand their study is still on the experimental stage. Many weirs that had been designed without considering the scouring of the deposits have either been abondoned or require prohibitive expenses to remove the deposits by mechanical means.

IV ECONOMICAL CONSIDERATIONS

A - ESTIMATE OF COST OF HEADWORKS

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
a) Excavator						
1. Surface excavation average 25 cms.		100.00				
deep, commencing at surface level		10.00	1000.00			
			1000.00	M.S.	0.40	400.00
2. Trench and bulk excavation under weir, piers and walls to receive concrete						
Weir		90.00				
		3.00				
		1.00	270.00			
Piers	8	4.00				
		1.00				
		1.00	32.00			
Walls		30.00				
		2.00				
		1.00	60.00			
			362.00	M.C.	2.00	724.00
3. Extra over for excavation of any description for breaking up and getting out rock.		50.00	50			
			50	M.C.	3.00	150.00
4. Extra over for excavation under water		200.00	200			
			200	M.C.	2.00	400.00
b) Concretor						
1. Plain concrete 1:8 ballast in foundations, floors, weir, piers and walls. The ballast existing in the river bed can be used, after being screened.						
Unit price of M.C. of plain concrete		100.00				
Floors		6.00				
		0.40	240			
Collecting & screening the ballast 6.00		100.00				
Use of shuttering 4.00 — Weir		3.00				
Cement (4 bags) 20.00		3.00	900			
Casting & fixing of forms 10.00		4.00				
Tools, overhead, etc. 10.00 — Piers	8	1.00				
Total 50.00 £.Syr.		4.50	128			
			1268			1674.00

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
Brought over			1268			1674.00
Walls		30.00 1.50 4.50	202			
Founds		130	130			
Variable			1600	M.C.	50.00	80000.00

2. Reinforced Concrete

Crushed stone 0.8MC @ 12 = 9.60
Sand 0.4 " @ 6 = 2.40
Shuttering timber = 10.00
Cement (6 bags + transport) = 30.00
Labour to fix formwork & steel, & to cast = 25.00
Steel 80 kg. @ 0.50 = 40.00
Overhead = 13.00
Total 130.00 £.Syr./M.C.

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
Beams	2	100.00 0.25 0.60	30.00			
Slabs		125.00 1.50 0.15	26.80			
Panel Walls	6	1.50 0.20 3.50	6.20			
			63.00	M.C.	130.00	8200.00

c) Plasterer

3. Plaster all 1:8 ballast concrete with 1:3 cement sand mortar in 2 coats; total thickness 2 cms.

Screened sand 1 M.C. 10.00
Cement 8 bags 40.00
50.00/M.C.

or 50.00 x 0.02 = 1.00 £.Syr.per MS
Labour 0.75 " "
Total 1.75 " "

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
Weir		100.00 10.00	1000.00			
Floors		100.00 8.00	800.00			
Piers	8	10.00 4.00	320.00			
Walls		30.00 4.00	120.00			
			2240.00	M.S.	1.75	3820.00

d) Steelwork

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
Steel gates	7	500.00	3500.00			
Rails	4	125.00	500.00			
			4000.00	Kg.	1.00	4000.00

e) Woodwork

	Times	Dimensions	Quantities	Unit of Measure	Unit Price £.Syr.	Totals £.Syr.
	10	1.80 0.25 3.50	16.00			
			16.00	M.S.	300.00	4800.00
						102494.00

Summary

Excavator	1674.00		
Concretor	88200.00		
Plasterer	3820.00		
Steelwork	4000.00		
Woodwork	4800.00		
Lifting & Checking Devices	6000.00		
	--------	108494.00	
Preparatory work (deviating water etc.) 10%		10849.40	
Unforeseen, overhead, design supervizing, contractor's gain 25 %		29835.85	
Total		149,179.25	L. Syrian

B - CONCLUSION

It is seen that with to-day's very high current prices the cost of the headworks comes out to be 150,000 L.Syrian. If we consider that the Kazmieh Irrigation Scheme, when completed, will cost over 10,000.000, the headworks will represent

$$\frac{150,000}{10,000,000} = 1.5\ \%\ \text{of the total}$$

It will also be interesting to note that the cost of headworks distributed over a period of 20 years, will mean

$$\frac{150,000}{4000 \times 20} = 1.875\ \text{L.Syr./hectare/annum.}$$

All these figures lead us to conclude that the cost of the Headworks, compared to the services they can render is negligible. Up to now, due to the fact that the project is partly completed, and therefore only a fraction of the lands is irrigated, the lack of adequate Headworks has not been felt much. But once the project will be completed, and the 4000 hectares will have to be irrigated methodically and with a maximum of efficiency, then the services rendered by well designed Headworks will be appreciated.

Georges S. Khachadurian

Beirut the Twenty Seventh of June 1946.