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REPORTS ON IRRIGATION, DRAINAGE & HYDROLOGY IN TRINIDAD & TOBAGO, WEST INDIES

(MARCH, 1967—May, 1969)

BY JUNICHI KITAMURA

(EXPERT FROM GOVERNMENT OF JAPAN, UNDER TECHNICAL COOPERATION PLAN FOR LATIN AMERICA)

JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

APRIL, 1975

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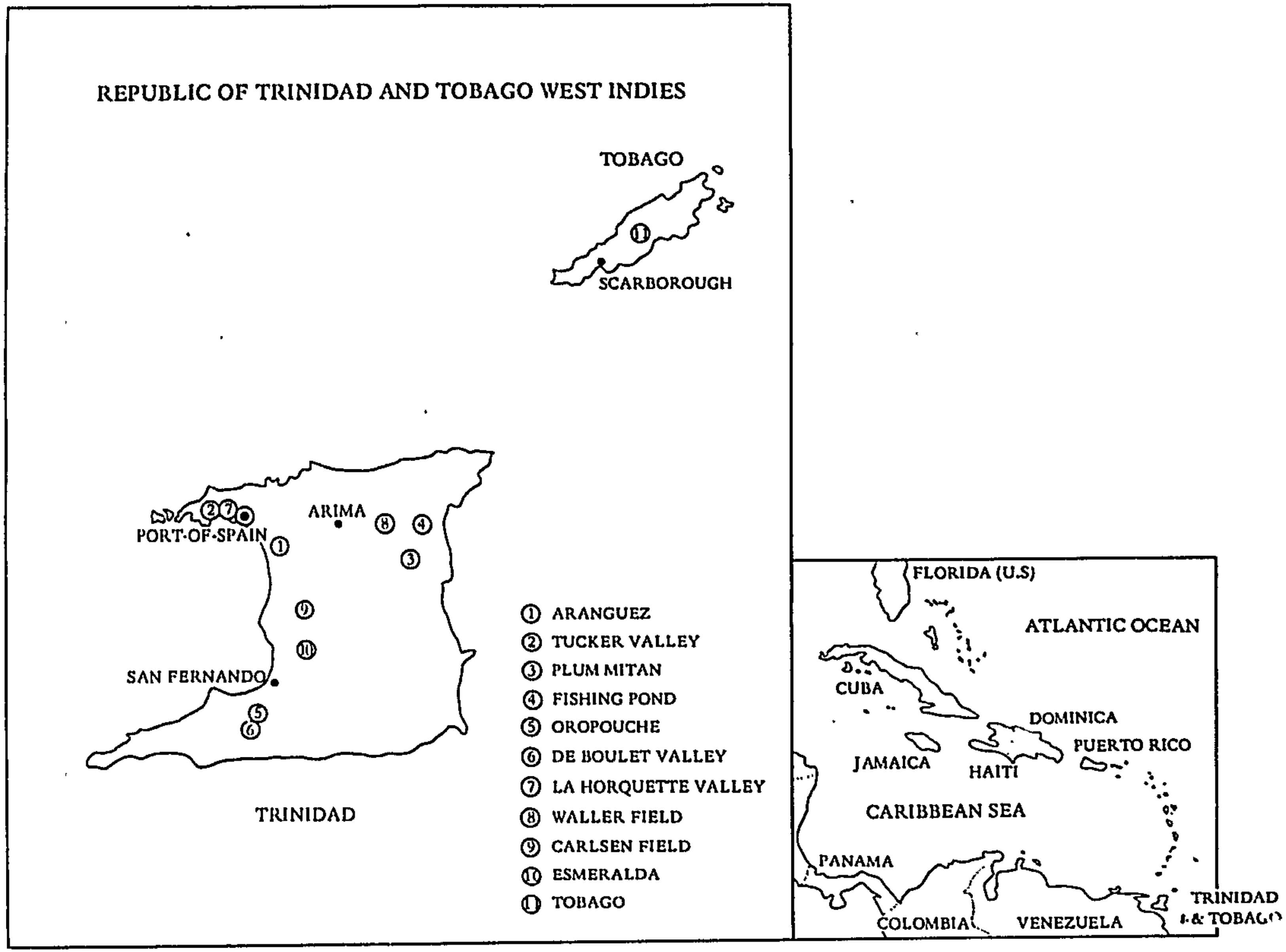
REPUBLIC OF TRINIDAD AND TOBAGO WEST INDIES
TOBAGO
SCARBOROUGH
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① ARANGUEZ
② TUCKER VALLEY
③ PLUM MITAN
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⑥ DE BOULET VALLEY
⑦ LA HORQUETTE VALLEY
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JAMAICA
HAITI
CARIBBEAN SEA
PANAMA
COLOMBIA
VENEZUELA
TRINIDAD
& TOBAGO

[I] PREFACE

[I] PREFACE

The present writer was dispatched as irrigation expert from the Ministry of Agriculture & Forestry, Japan, to the Drainage Division, the Ministry of Works, the Government of Trinidad & Tobago, West Indies, under the Technical Cooperation Plan for Latin America; and had been working from March, 1967 till May, 1969 (for two years and three months).

Trinidad & Tobago is one of the small countries in Central America. She, however, has been making best efforts to develop her land since her independence in 1962.

During the writer's stay, many kinds of reports were submitted to the Government. The office of the Drainage Division happened to be burnt in 1973, and the submitted reports were reportedly mostly lost as well as the instrument and the equipment presented by the Overseas Technical Cooperation Agency, Japan.

Fortunately, since each copy of the reports has been kept by the writer, they were compiled and printed on this occasion by the favor of the Japan International Co-operation Agency (JICA), so as to be sent to the Government.

Hoping they will be of avail not only for the technical officials in the Government of Trinidad & Tobago and also the experts sent or to be sent from the Government of Japan.

(The writer's position in Trinidad & Tobago was succeeded in order by Mr. S. Hirai, Mr. K. Itakura and at present Mr. T. Takahashi & Mr. T. Miyazato for about two years each).

April, 20, 1975

Junichi Kitamura

(Present position): Head of Development
Planning Division for Agriculture & Forestry,
Japan International Co-operation Agency, (JICA)

[II] IRRIGATION

[II] IRRIGATION

(1) ARANGUEZ ESTATE (I) 20, April, 1967

INTRODUCTION

It was told to make out Aranguez Estate plan and the inspection was carried out as follows:-

April 10th - visited there to grasp the general condition of the Estate with Mr. Mc. D. Gibbs.

April 17th - went on the shore along the Caroni River with Mr. S. Moosai Maharaj from the mouth of the San Juan River to the mouth of the William Street Drain.

April 18th - went down from the Caroni Bridge on the Princess -Margaret High-way to the sea by boat and inspected the condition of the Caroni River along the estate.

April 8th - read the report on the estate by Mr. C.B. Brown, Soil Chemist.

Description of the Estate studied:

This estate is the southern part of 334 acres, 102-96 feet in elevation, in the area surrounded by Churchill-Roosevelt Highway, the Martin Canal, the Caroni River the Boundary Road.

At the south-eastern part of the estate, San Juan River pours into the Caroni River, and at the south-western part of it William Street Drain flows into the River.

Although the estate has an embankment from the Highway to the mouth of the San Juan River, there are three spots broken by the flood, and there is no dike along the Caroni River. It can be seen the old low dike laid 300-500 feet inside from the shore broken here and there.

In the rainy season, it is said that the tidal Caroni River rises 4 feet high from the shore and inundated the estate as far as the Highway. Crops cannot be kept with brackish water. The soil mostly consists of silty clay and there is no problem of keeping the plants. On the contrary in the dry season, the irrigation water that people take from the upper reach of the San Juan, do not reach the estate, so they cannot help using the small pumps to supply the water from the William Street Drain and the near San Juan River.

It was found the crops in dry season are tomato, egg plant, water melon, cucumber, cabbage and so on. The salinity in the soil 18 inches deep in this estate reaches 0.42% and 6 inches deep 0.09% according to the other report.

FUNDAMENTAL COUNTER MEASURE

1. DRAINAGE

To embank about 8,000 feet long, with a final height of 6 feet by machinery. The height of the dike of course, must be decided by the record of the Coroni River high water level. To excavate the drain to discharge the excess water inland and use this earth as the dike material.

To repair immediately the three damaged parts of the dike along the San Juan River by manual labour. To grasp the relation between the allowable inundating time for the paddy and the possible water quantity to drain by gravity. As the result the scale of the gate will be decided and the pump establishment will be considered, when the gate cannot have the capacity enough to discharge water by gravity.

To consider the new collector canal to the San Juan River in order not to bring the excess water from the upper part, in the north of Churchill-Roosevelt Highway into the lower part of this estate.

The need for protection work with asphalt or concrete, on the place of embankment where the Caroni River acts strongly.

2. IRRIGATION

To calculate the possible water quantity to supply from the San Juan River in the dry season and in the wet season.

To calculate the water requirement for the estate in each season.

To inspect the intake water quantity for the upper part from the upper and the lower weir on the San Juan River.

To layout the 3rd weir the San Juan River for the estate- intake water quantity, water head to dam up and so on.

To consider how to remove the salt that accumulated over a long period.

To consider the requests from the people in this estate.

To block up the Blue River mouth to the Caroni River.

To excavate the drain flowing into the Caroni Swamp.

These are both favourable to the people in this estate, but.the conclusion cannot be arrived before consideration of the influence to the next areas and the Caroni Swamp.

These requirements will be studied, and more detailed reports be presented as soon as possible.

Junichi Kitamura

ARANGUEZ ESTATE (II) 17th May, 1967

The survey of the flow was made on the 10th May. Although it was near the rainy season, there were no rain for about a week before the day. Then, the river showed a figure near the basic flow discharge.

To inspect the intake water quantity for the upper part from the 1st and the 2nd head works on the San Juan River.

(1) The 1st Head Work

People are taking the water into the fields by a siphon, about 720 feet long, which the open channel follows.

The possible water intake capacity is much more than the water requirement in the dry season.

They can take the water requirement easily without raising the river surface still higher by the plank than the dam crest.

In opposition the fact shows that they need to limit the intake water quantity in the rainyseason.

In fact the siphon has the part in which they can insert the plank to stop the river water, a few feet apart from the mouth.

They said that the intake water quantity of 4.62 cusecs on that day was quite normal. (If they wanted to flow the water full of the channel, they could 8-10 cusecs.)

(2) The 2nd Head Work

They are taking the river water by a culvert about 150 feet long, which then flows in an open channel.

The mouth of the intake is in a high position and in a small section.

The possible water quantity to take in is a little, although the water surface is raised by plank 1.2 feet higher than normal. In the open channel, the water flows at a depth of 0.75 feet.

It is supposed they cannot take enough water, without improving the position and the size of the intake.

This is the reason why the overflowing is about twice as much as the intake.

The calculations of the possible water quantity to be supplied from the San Juan River in dry season and in the wet season are as below.

1) In the dry season:

As the result of the survey on the 10th of May 1967 the possible water quantity to supply to Aranguez is 3.71 cusecs, about 35% in the river flow of 10.13 cusecs before taking the water from the 1st Head Work.

Certainly it is sufficient to supply to Aranguez Estate 334 acres.

SAN JUAN RIVER:

Catchment Area	Length	Average Slope
14,915 (acres)	58,500 (feet)	1/153

The catchment area up to the 1st Head Work is nearly 14,000 acres.

$$
\begin{aligned}
&\text{10.13 cusecs} && = 0.287\ m^3/sec \\
&\text{14,000 acres} && = 60\ km^2 \\
&0.287\ m^3/sec \times \frac{100}{60}\ km^2 && = 0.48\ m^3/sec/100\ km^2 \\
& && = (0.72\ \text{cusecs}/1{,}000\ \text{acres})
\end{aligned}
$$

This figure is reasonable as the basic flow discharge.

2) In the wet season:

Place	1	2	3	4	5	6	7	8	9	10	11	12	Total inch
La Pastora, Santa Cruz(1947-65)Ave.	5.06	3.76	2.51	2.00	3.34	8.19	10.43	12.74	8.81	6.42	7.60	6.57	77.43
U.W.I.ST. Augustine (1931-65) Ave.	2.59	1.48	1.20	1.87	4.91	8.82	8.70	9.29	7.37	6.24	7.40	5.86	65.73

In case that people want to raise the rice, June will be the month when they need water the most.

The rainfall in June reaches several times as much as the one a month in the dry season.

In the same catchment area, the river flow is regarded as being in proportion to the rainfall unless the runoff coefficient changes between 0.7 and 1.0 by the rainfall.

If they do not think of the increase of the runoff coefficient by the increase of the rainfall, they will have the same times river flow as the rainfall.

No matter how they may deduct the water that overflows the weir into the sea, from the river flow, they will be able to take the twice water in June as much as the one a month in the dry season.

That means about 7.5 cusecs.

To calculate the water requirement for the Estate in each season.

1) In the dry season:

Place	Area	Intake water quantity
Aranguez North	929 (acres)	4.62 (cusecs)
Aranguez South	202	1.99
Total	1,131	6.61

$$1131 \text{ acres} \times 0.04068 = 457.5 \text{ ha} = 4{,}575{,}000 \text{ m}^2$$

$$6.61 \text{ cusecs} \times 0.0283 = 0.187 \text{ m}^3/\text{sec}$$

$$\frac{0.187 \times 30 \times 24 \times 60 \times 60}{4{,}575{,}000} = 1{,}060 \text{ mm/month} = 4.17 \text{ inch/month}$$

As the result of the survey people was using the water in the depth of about 4 inches a month.

As to Aranguez Estate :-

On the assumption that they want the water in the depth of 4 inches a month like the above mentioned areas :-

$$4 \text{ inches} = 0.1015 \text{ m}$$

$$334 \text{ acres} = 334 \times 0.40468 = 135.0 \text{ ha} = 1{,}350{,}000 \text{ m}^2$$

$$\frac{0.1015 \times 1{,}350.00}{30 \times 24 \times 60 \times 60} = 0.0529 \text{ m}^3/\text{sec} = 1.87 \text{ cusecs.}$$

they need the water of 1.87 cusecs.

2) In the wet season:

Assuming that the area for rice in the Estate is 300 acres in the gross area of 334 cusecs, the water requirement is as follows:-

$$(334 \text{ acres} \times (1 - 0.1) = 300 \text{ acres})$$

According to the report by F.A.O.

(inches)

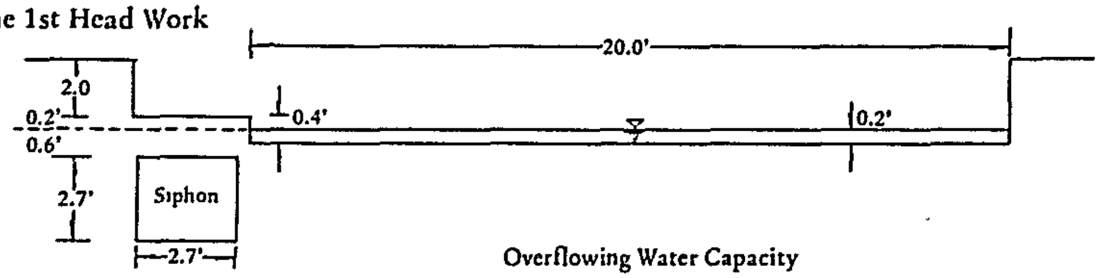
Month	5	6	7	8	9	10	Remarks
Net irrigation requirement	0.5	16.6	6.9	7.0	6.6	6.7	
Rainfall (20% wet)	2.1	6.8	6.1	7.4	5.5	4.8	
Balance	-	9.8	0.8	-	1.1	1.9	
Gross irrigation requirement	-	16.4	1.3	-	1.8	3.2	60%

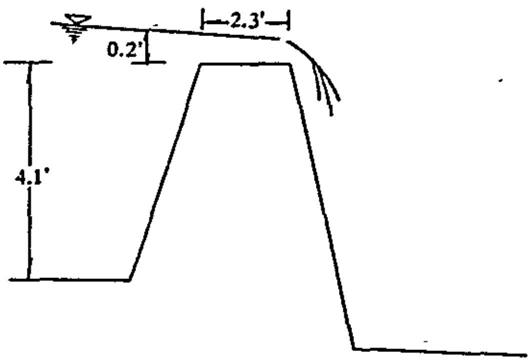
16.4 inches/month = 4,166 mm/month = 14 mm/day

$$\frac{0.014\ \text{m} \times 300\ \text{acres} \times \frac{4{,}046.8\ \text{m}^2}{\text{acres}}}{86{,}400\ \text{sec}} = 0.197\ \text{m}^3/\text{sec} = 6.95\ \text{cusec (23 cusecs/1,000 acres)}$$

Surveyed on 10th May by: Junichi Kitamura

1. On the 1st Head Work





Overflowing Water Capacity

$Q = CBH^{3/2} = 1.70\ BH^{3/2}$ (C.G.S)

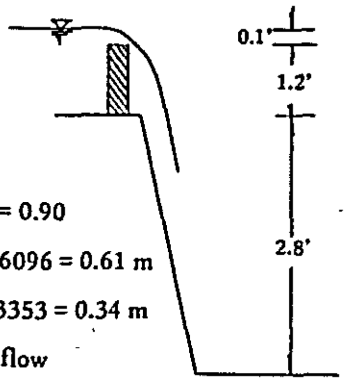
$B = 20' = 20 \times 0.3048 - 6.096 = 6.1$ m

$H = 0.20' = 0.20 \times 0.3048 = 0.061$ m

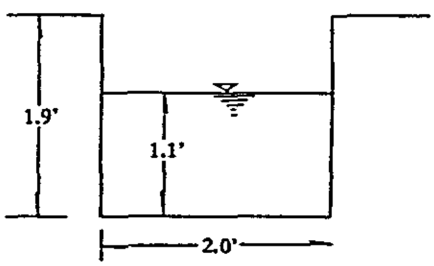
$Q = 1.70 \times 6.1 \times 0.061^{3/2}$

$= 1.70 \times 6.1 \times 0.015066 = 0.1562 = 0.156\ m^3/sec.$

$(1\ ft^3 = 0.0283\ m^3) = 5.512 = 5.51$ cusecs.



2. The Channel Section Connecting with the Outlet of Siphon



$Q = ca\ V^2 = c\ BHV^2$ C = 0.90

$B = 2.0' = 2.0 \times 0.3048 = 0.6096 = 0.61$ m

$H = 1.1 = 1.1 \times 0.3048 = 0.3353 = 0.34$ m

The Velocity of the surface flow

$V_s = 60'/26\ sec. = 60 \times 0.3048/26 = 18.29/26 = 0.70$ m/sec.

$= (2.3'/sec.)$

$Q = 0.90 \times 0.61 \times 0.34 \times 0.70 = 0.131\ m^3/sec. = 4.62$ cusec.

3. On the 2nd Head Work

Overflowing Water Capacity

$Q = CBH^{3/2} = 1.84\ BH^{3/2}$ (C.G.S)

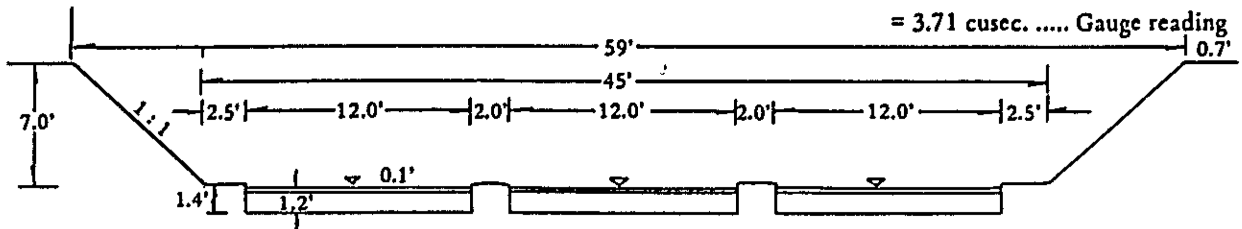
$B = 3 \times 120' = 36.0' = 36.0 \times 0.3048 = 10.97$ m

$H = 0.1' = 0.03$ m

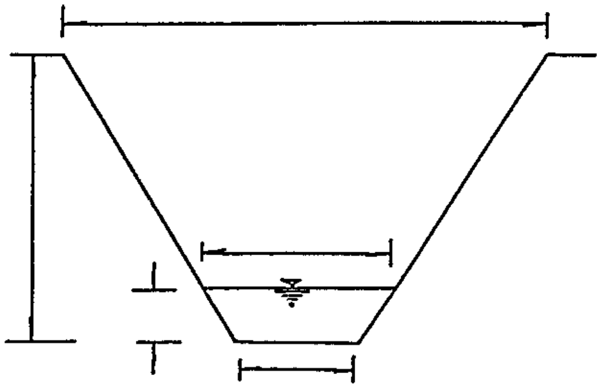
$Q = 1.84 \times 10.97 \times 0.03^{3/2}$

$= 1.84 \times 10.97 \times 0.0052 = 0.105\ m^3/sec.$

$= 3.71$ cusec. ..... Gauge reading



4. The Channel Section Connecting with the Outlet of the Culvert



$Q = cA\ V_s \quad C = 0.90$

$F = 0.75' = 0.75 \times 0.3048 = 0.23$ m

$B = 3.0' = 3.0 \times 0.3048 = 0.91$ m

$B_2 = 2.0' = 20 \times 0.3048 = 0.61$ m

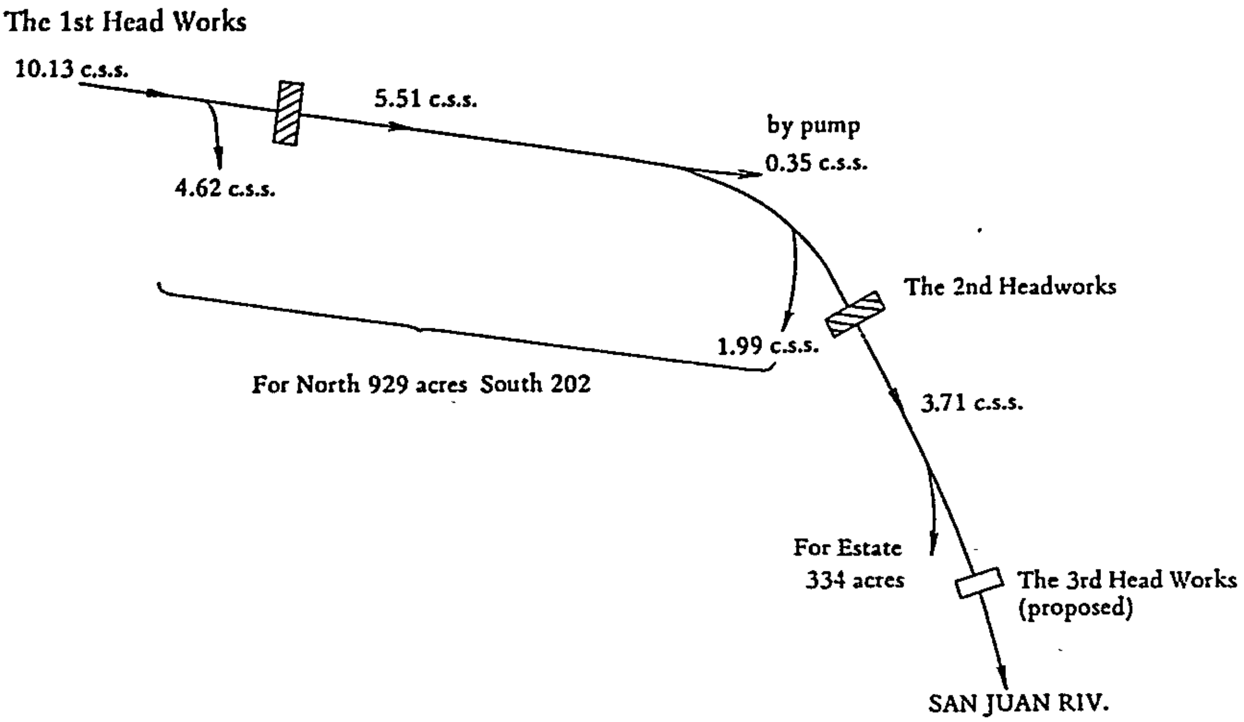
$A = 1/2(0.91 + 0.61) \times 0.23 = 0.1748 = 0.175\ m^2$

The Velocity of the surface flow

$V_s = 20'/17\ sec. = 20 \times 0.3048/17 = 6.096/17$

$= 0.359$ m/sec. (1.18'/sec.)

$Q = 0.90 \times 0.359 \times 0.175 = 0.056\ m^3/sec. = 1.99$ cusecs.



ARANGUEZ (III) 7th June 1967

The designed flood discharge is the important element for the design of Head Works. It was decided by three figures as follows.

1. The estimate of the maximum discharge of 6,000 cusecs by A.P.I Rational at Churchill-Roosevelt Highway was done on 1st December 1954, but no data could be found.

2. To calculate the estimated maximum discharge from the record of flood water level. A gauge board was set up in San Juan River at the Upper reach of 3,000 feet from the highway.

(a) Maximum H.W.L. in 1966... EL. 112.40 ft (Reading H = 10.4 ft)
(b) Past estimated Maximum H.W.L... EL 114.00 ft (Reading 12.0 ft)

In case of (a) H = 10.4 ft.

A	:	area of cross section in river flow
P	:	wetted perimeter
R	:	hydraulic radius
N	:	Kutter's roughness coefficient
V	:	velocity of river flow
Q	:	discharge
I	:	slope of water surface

$A_1 = 18.0\ ft^2 = 1.67\ m^2$
$A_2 = 258.7 = 24.02$
$A_3 = 21.3 = 1.98$

$P_1 = 18.4\ ft = 5.57\ m$
$P_2 = 38.1 = 11.53$
$P_3 = 29.0 = 8.77$

$R_1 = 0.30 \quad R_1^{2/3} = 0.448$
$R_2 = 2.08 \quad R_2^{2/3} = 1.630$
$R_3 = 0.23 \quad R_3^{2/3} = 0.375$

$N_1 = 0.080^{1/n_1} = 12.50$
$N_2 = 0.035^{1/n_2} = 28.57$
$N_3 = 0.080^{1/n_3} = 12.50$

$Q_1 = 1.67 \times 0.43 = 0.72\ m^3/s$
$Q_2 = 24.02 \times 3.58 = 86.10$
$Q_3 = 1.98 \times 0.36 = 0.71$
$\Sigma Q = 87.53\ m^3/sec = 3,090$ cusecs
$A = 27.67\ m^2 = 298\ ft^2$

About slope 1
Upstream gauge : Water stage level EL 106.7 ft.
Downstream : Water stage level 106.1
Distance between gauges 100 ft.

$$I = \frac{106.7 - 106.1}{100} = \frac{6}{1000} = \frac{1}{167}$$

$\sqrt{I} = 0.077$

Manning's formula

$V = 1/n \ R^{2/3}\ I^{1/2}$ (C.G.S)
$V_1 = 12.50 \times 0.448 \times 0.077 = 0.43$ m/s
$V_2 = 28.57 \times 1.630 \times 0.077 = 3.58$
$V_3 = 12.50 \times 0.375 \times 0.077 = 0.36$

In case of (b) $H = 12.0$ ft

A_1	=	60.2 ft^2	=	5.59 m^2
A_2	=	306.7	=	28.48
A_3	=	74.5	=	6.92
P_1	=	33.8 ft	=	10.25 m
P_2	=	38.1	=	11.53
P_3	=	37.1	=	11.22
R_1	=	0.55 m	$R_1^{2/3}$	= 0.312
R_2	=	2.47	$R_2^{2/3}$	= 1.827
R_3	=	0.62	$R_3^{2/3}$	= 0.337
N_1	=	$0.065^{1/n_1}$	=	15.38
N_2	=	$0.035^{1/n_2}$	=	28.57
N_3	=	$0.065^{1/n_3}$	=	15.38

$I = 1/167$ $\sqrt{I} = 0.077$

$V_1 = 15.38 \times 0.312 \times 0.077 = 0.36$ m/s

$V_2 = 28.57 \times 1.827 \times 0.077 = 4.02$

$V_3 = 15.38 \times 0.337 \times 0.077 = 0.40$

$Q_1 = 5.59 \times 0.36 = 2.01\ m^3/s$

$Q_2 = 28.48 \times 4.02 = 114.30$

$Q_3 = 6.92 \times 0.40 = 2.77$

$\Sigma Q' = 119.08\ m^3/sec = 4{,}220$ cusecs

$A = 40.99\ m^2 = 441\ ft^2$

3. To calculate the estimated maximum discharge from the daily rainfall precipitation. The maximum daily rainfall every year in these ten years are as follows:-

Date	La Pastora Santa Cruz	St. Joseph Govt. Stock farm.	Average maximum daily rainfall
Dec. 7th 1960	4.68"	0.42"	2.91"
Aug. 8th 1961	4.27	2.10	3.37
Aug. 9th 1962	6.35	0	3.70
Sept. 30th 1963	4.04	3.02	3.61
Influenced A Area	8,100 acres	5,800 acres 13,900	

Average maximum daily rainfall in these ten years is 3.70"

$R = 3.70" = 94$ mm

Arrival time of flood by Rziha's Formula

L	:	Distance (m)	58,500 ft	= 17,700 m
H	:	Head (m)	3,800 ft	= 1,150 m
V	:	Velocity of flood km/hr		

$(V = 20 \left(\frac{H}{L}\right) 0.6$ m/sec $= 72\left(\frac{H}{L}\right) 0.6$ km/hr

$(V = 72\left(\frac{1{,}150}{17{,}700}\right)0.6 = 72\left(\frac{1}{153}\right)0.6 = 3.52$ km/hr

$T = L/V = \frac{17.7}{3.52} = 6.2$ hr

T : Arrival time of flood

Average rainfall intensity an hour within arrival time of flood.

$$r_1 \approx \frac{R_1}{24}\frac{(24)^{2/3}}{T} \quad \frac{94}{24}\frac{(24)^{2/3}}{6.2} = 3.91 \times 2.47 = 9.7 \text{ mm/hr}$$

Estimated maximum discharge

Q	=	1/3.6 f. r_1 A.
f	:	Run off coefficient in flood 0.80
A	:	Catchment area (Km²) 13,900 acres = 5,610 ha = 56.1/km²
∴		Q = 1/3.6 x 0.8 x 9.7 x 56.1 = 121 m³/sec = 4,270 cusecs

As the result of the calculation above, the figures 4,220 cusecs and 4,270 cusecs were taken.

The figure of 6,000 cusecs is somewhat larger than the estimated maximum flood discharge in these ten years but if the safety factor is considered, it will be the reasonable figure.

To select the site for the headworks on San Juan River from the profile and cross-section downwards from the bridge on Churchill-Roosevelt Highway. Two places No. 4 and No. 20 were examined.

a) Highest elevation in profitable area : 101 ft.
b) Maximum net water requirement : 6.95 cusecs
c) Mean slope of river bed :

No. 0	No. 10	$\frac{99.11 - 98.74}{1000}$ =	$\frac{0.37}{1000}$ =	$\frac{1}{2700}$
No. 10	No. 30	$\frac{98.74 - 97.65}{1000}$	$\frac{1.09}{1000}$	$\frac{1}{920}$
No. 20	No. 30	$\frac{97.65 - 95.85}{1855}$	$\frac{1.80}{1855}$	$\frac{1}{970}$
No. 0	No. 37	$\frac{99.11 - 93.54}{5200}$	$\frac{5.57}{5200}$	$\frac{1}{930}$

No. 4

d) Distance from intake to nearest point in profitable area : 1,850 ft.
e) Existing river bed level : 98.48 ft.
f) Water level at beginning of channel:
Assuming that the surface slope in channel is $\frac{1}{1000}$ it will be 102.85 ft
$(101.00 \text{ ft} + \frac{1850}{1000} \text{ ft} = 102.85 \text{ ft})$
g) Crest level of weir:
The total loss of head to take the river water into the channel is about 0.8 to 1.0 ft according to the data.
Besides clearance of 0.5 ft. is needed.

n) So 102.85 ft + 1.0 ft. + 0.5 ft = 104.35 ft.
h) Water depth: 104.35 ft - 98.48 ft = 5.87 ft

No. 20)

(d) 50 ft.
(e) 97.65 ft.
(f) 101.05 ft.
(g) 101.05 ft. + 1.0 ft. + 0.5 ft. = 102.55 ft.
(h) 102.55 ft. - 97.65 ft. = 4.90 ft.

From a comparison of both sites No. 20 is more desirable.

To decide the flood stage right down the headworks. The maximum capable discharge in the present section at No. 20 was first checked.

Water level : 109.7 ft

A_1	=	136.0 ft^2	=	12.62 m^2	Slope 1 : upwards from No. 20 1/920
A_2	=	371.0	=	34.42	Downwards from No. 20 1/970
A_3	=	171.0	=	15.88	So the mean slope is 1/944
P_1	=	37.9 ft	=	11.5 m	$\sqrt{I}$ = 0.0325
P_2	=	47.0	=	14.2	
P_3	=	40.8	=	12.3	
					V_1 = 22.22 x 1.066 x 0.0325 = 0.77m^3
R_1	=	1.10 m $R_1{}^{2/3}$	=	1.066	V_2 = 28.57 x 1.807 x 0.0325 = 1.67
R_2	=	2.43 $R_2^{2/3}$	=	1.807	V_3 = 22.22 x 1.185 x 0.0325 = 0.86
R_3	=	1.29 $R_3{}^{2/3}$	=	1.185	
					Q_1 = 12.65 x 0.77 = 9.72 m^3/s
N_1	=	$0.045^{1/n_1}$	=	22.22	Q_2 = 34.42 x 1.67 = 57.50
N_2	=	$0.035^{1/n_2}$	=	28.57	Q_3 = 15.88 x 0.86 = 13.65
N_3	=	$0.045^{1/n_3}$	=	22.22	ΣQ = 80.87m^3/S = 2,850 cusecs

As the result of calculation above, the present section of No. 20 can only flow 2,850 cusecs. So it must be widened.

How it must be widened, was calculated. And if it is widened more 15 ft in bed width, it will be able to flow the estimated maximum flood discharge of 4,300 cusecs in these ten years.

In order to discharge 6,000 cusecs the banks must be raised more than 3' and the bed widened to a width of 50'.

Water level : 109.7 ft.

A_1	=	1,360 ft^2		=	12.62 m^2
A_2	=	551.5		=	51.20
A_3	=	114.0		=	10.58
P_1	=	37.9		=	11.5 m
P_2	=	52.3		=	15.8
P_3	=	27.8		=	8.4
R_1	=	1.10 m	$R_1{}^{2/3}$	=	1.066
R_2	=	3.24	$R_2{}^{2/3}$	=	2.190
R_3	=	1.26	$R_3{}^{2/3}$	=	1.166
N_1	=	0.045	$1/n_1$	=	22.22
N_2	=	0.035	$1/n_2$	=	28.57
N_3	=	0.045	$1/n_3$	=	22.22

Width of river bed : 31 ft.

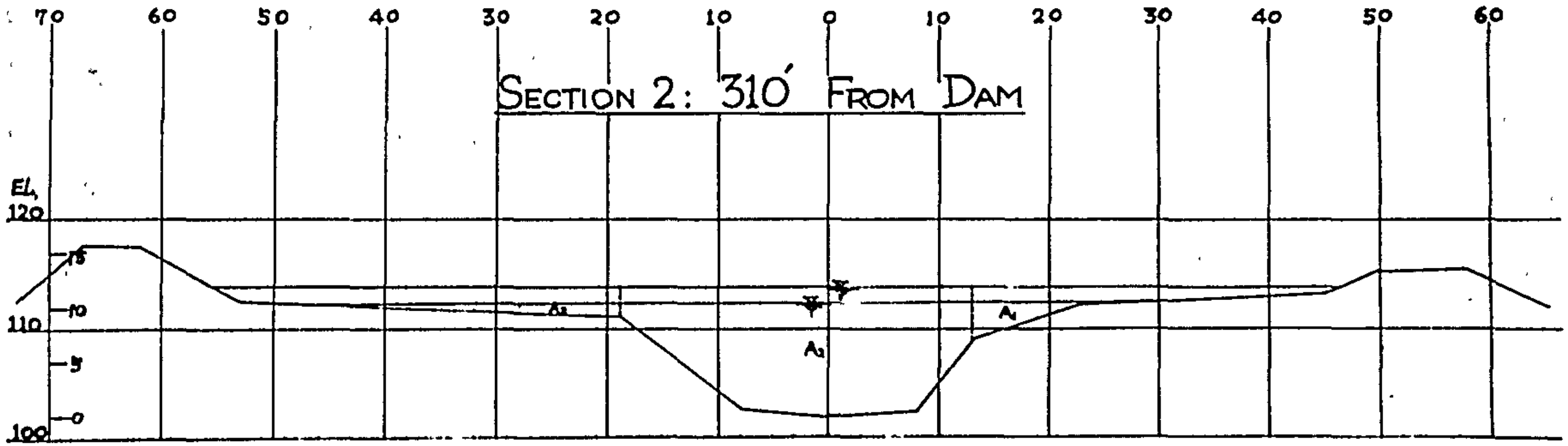
$I = \frac{1}{944} \quad \sqrt{I} = 0.0325$

V_1	=	22.22 x 1.066 x 0.325 = 0.77 m/s
V_2	=	28.57 x 21.90 x 0.0325 = 2.03
V_3	=	22.22 x 1.166 x 0.0325 = 0.84
Q_1	=	12.62 x 0.77 = 9.72 m^3/s
Q_2	=	51.20 x 2.03 = 103.85
Q_3	=	10.58 x 0.84 = 8.88
ΣQ	=	122.45 m^3/s = 4,320 cusecs

Junichi Kitamura

San Juan River (at Gauge)

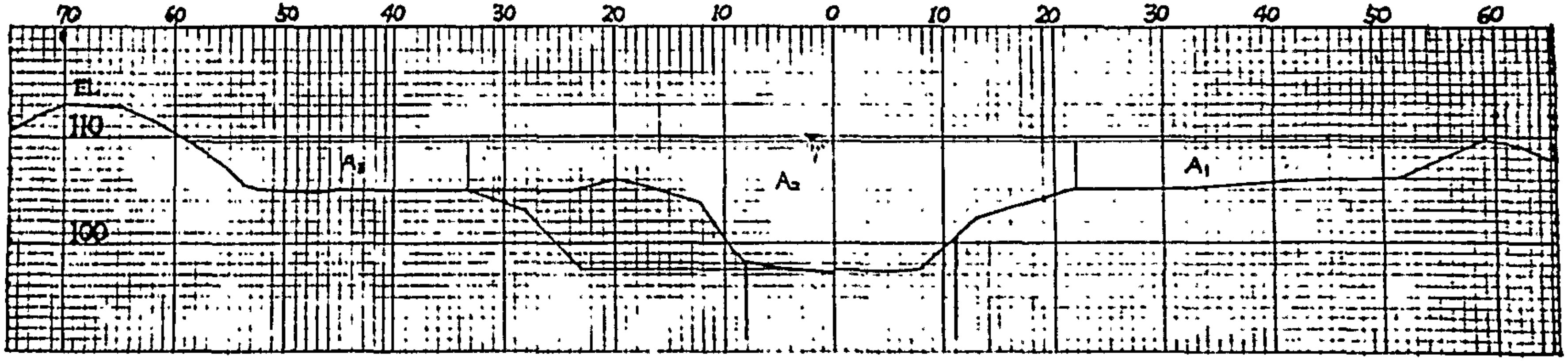
Scale 1" = 10' Vertical & Horizontal



Estimated Max. H.W.L = 114.00'
Max. H.W.L in 1966 = 112.40
Datum Line of Gauge = 102.00'

San Juan River (at Headworks expected)

Scale 1" = 10' Vertical & Horizontal



SECT 20 AT 2000' D/S OF BRIDGE

ARANGUEZ (IV) 12th June, 1967

In order to build a floating type, head works, piling and footing are needed as foundation works. An analysis of soil condition by the static sounding on the proposed site of the head works on San Juan River was carried out. Therefore the static soundings at 4 points on the river bed, 2 at the right side, the other 2 at the left side with cone penetrometer S 44, the improved type of W.E.S. were made on 5th and 10th of July.

The positions are shown on figure 4, the graphs on the figure 3, and the data on the charts 1 to 4. As can be seen from figure 3 the records at the spots No. 1 and No. 4, section 19 feet apart, resembles although the elevations of the surfaces are different. Similarly the records at the spots No. 2 and No. 3 on the lower section resemble very much.

1. UPPER SECTION

An allowable bearing stress of more than 15 lb/in^2 exists only below the elevation of 91.5 ft. The river bed elevation is about 97.7 ft, the layer between 97.7 and 91.5 ft consists of weak soil, that is, water-saturated silty clay.

2. LOWER SECTION

This section is only 15 ft down from the upper section, but it has better condition, as allowable bearing stress of more than 10 lb/in^2 found only below about 95 ft, and more than 15 lb/in^2 below elevation 93 ft.

Static Sounding with Cone Penetrometer on San Juan River

(Right side Spot 1)

Depth	Elevation	Reading on Dial guage	Cone Index (Q)	Cone bearing capacity (Q/A)	Allowable, Bearing power of ground (qa)		Remarks
cm	ft		kg	kg/cm²	kg/cm²	lb/in²	
0	101.00	0	0	0	0	0	
10		22	10	1.55	0.31	4.41	qa = α Q/A
20		25	11	1.70	0.34	4.83	
30	100.00	27	12	1.86	0.37	5.26	A = 6.45 cm²
40		34	16	2.48	0.50	7.11	Max. area of cone sec.
50		23	10	1.55	0.31	4.41	α = 0.2
60	99.00	25	11	1.70	0.34	4.83	Converting co-efficient
70		24	11	1.70	0.34	4.83	into allowable bearing
80		27	12	1.86	0.37	5.26	power of ground (in
90	98.00	34	16	2.48	0.50	7.11	case of safety factor of
100		42	19	2.94	0.59	8.39	3).
110		52	24	3.72	0.74	10.52	
120	97.00	49	22	3.41	0.68	9.67	
130		43	20	3.10	0.62	8.81	
140		41	19	2.94	0.59	8.39	
150	96.00	38	17	2.64	0.53	7.54	
160		34	16	2.48	0.50	7.11	
170		36	17	2.64	0.53	7.54	
180	95.00	39	18	2.79	0.56	7.96	
190		39	18	2.79	0.56	7.96	
200		31	14	2.17	0.43	6.11	
210	94.00	39	18	2.79	0.56	7.96	
220		41	19	2.94	0.59	8.39	
230		38	17	2.64	0.53	7.54	
240	93.00	31	14	2.17	0.43	6.11	
250		39	18	2.79	0.56	7.96	
260		29	13	2.02	0.40	5.68	
270	92.00	38	17	2.64	0.53	7.54	
280		29	13	2.02	0.40	5.68	
290	91.30	>70	>32	>4.96	>0.99	>14.07	

(Chart 1)

(Right side Spot 2)

Depth	Elevation	Reading on Cone Index Dial gauge (Q)		Cone bearing capacity (Q/A)	Allowable Bearing power of ground (qa)		Remarks
cm	ft		kg	kg/cm^2	kg/cm^2	lb/in^2	
0	101.00	0	0	0	0	0	
10		18	7	1.09	0.22	3.13	qa = α Q/A
20		20	8	1.24	0.25	3.55	A = 6.45 cm^2
30	100.00	17	7	1.09	0.22	3.13	α = 0.2
40		18	7	1.09	0.22	3.13	
50		17	7	1.09	0.22	3.13	
60	99.00	18	7	1.09	0.22	3.13	
70		16	6	0.93	0.19	2.70	
80		22	10	1.55	0.31	4.40	
90	98.00	47	21	3.26	0.65	9.23	
100		49	22	3.41	0.68	9.65	
110		31	14	2.17	0.43	6.11	
120	97.00	39	18	2.79	0.56	7.95	
130		37	17	2.64	0.53	7.52	
140		53	24	3.72	0.74	10.53	
150	96.00	75	34	5.27	1.05	14.92	
160		59	27	4.19	0.84	11.92	
170		56	26	4.03	0.81	11.52	
180	95.00	80	36	5.58	1.12	15.91	

(Chart 2)

(Left side Spot 3)

Depth	Elevation	Reading on Cone Index Dial gauge (Q)		Cone bearing capacity (Q/A)	Allowable Bearing power of ground (qa)		Remarks
cm	ft		kg	kg/cm^2	kg/cm^2	lb/in^2	
0	98.00	0	0	0	0	0	
10		9	4	0.62	0.12	1.71	qa = α Q/A
20		18	7	1.09	0.22	3.13	A = 6.45 cm^2
30	97.00	17	7	1.09	0.22	3.13	α = 0.2
40		22	10	1.55	0.31	4.41	
50		34	16	2.48	0.50	7.11	
60	96.00	55	25	3.88	0.78	11.09	
70		27	12	1.86	0.37	5.26	
80		52	23	3.57	0.71	10.10	
90	95.00	50	23	3.57	0.71	10.10	

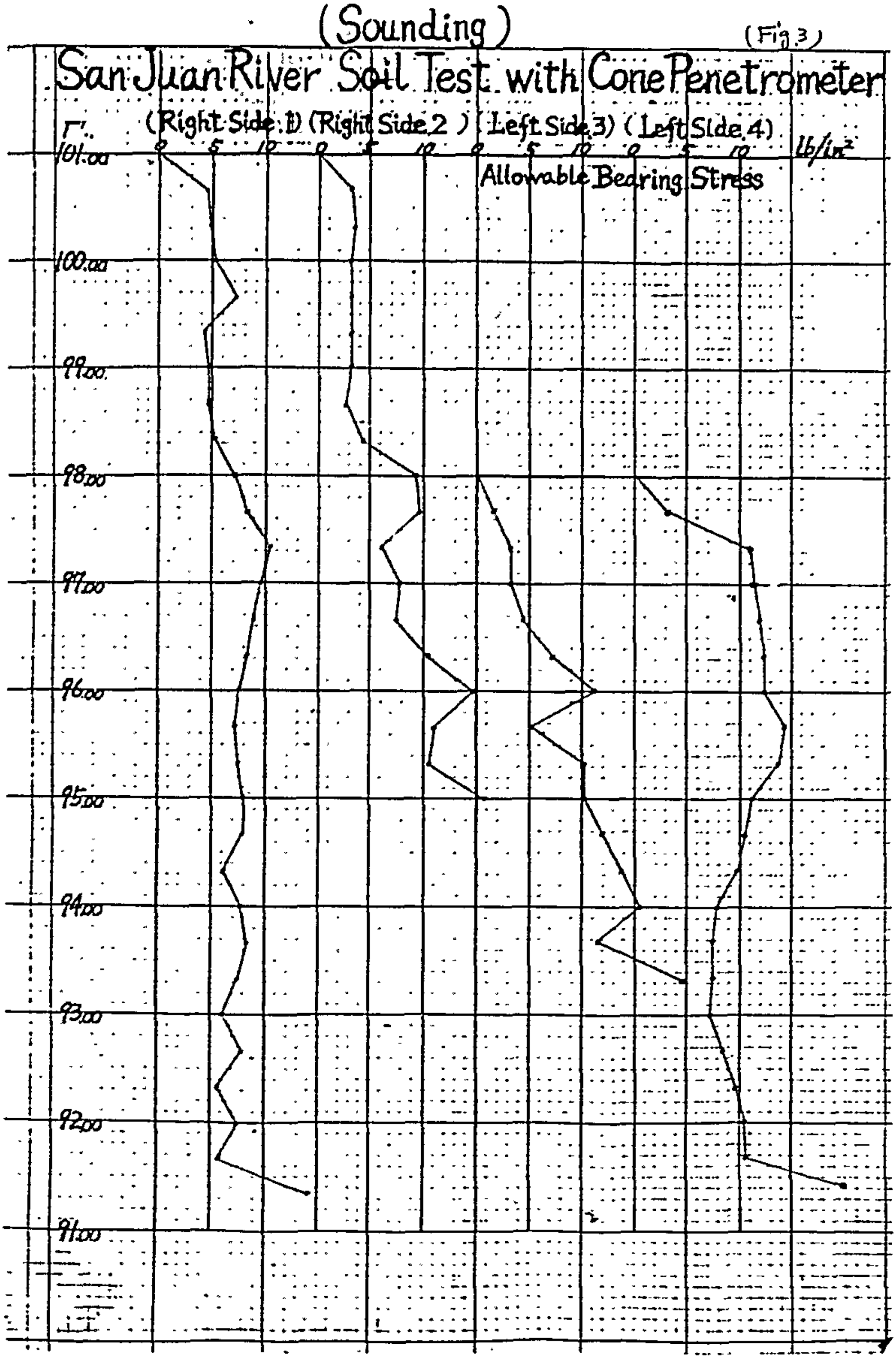
Depth	Elevation	Reading on Dial gauge	Cone Index (Q)	Cone bearing capacity (Q/A)	Allowable Bearing power of ground (qa)		Remarks
cm	ft		kg	kg/cm²	kg/cm²	lb/in²	
100		59	27	4.19	0.84	11.95	
110		68	31	4.81	0.96	13.65	
120	94.00	77	35	5.43	1.09	15.50	
130		56	26	4.03	0.81	11.52	
140	93.30	> 100	> 45	> 6.98	> 1.40	>19.91	

(Chart 3)

(Left side Spot 4)

Depth	Elevation	Reading on Dial gauge	Cone Index (Q)	Cone bearing capacity (Q/A)	Allowable Bearing power of ground (qa)		Remarks
cm	ft		kg	kg/cm²	kg/cm²	lb/in²	
0	98.00	0	0	0	0	0	
10		17	7	1.09	0.22	3.13	qa = α Q/A
20		53	25	3.88	0.78	11.09	A = 6.45 cm²
30	97.00	56	26	4.03	0.81	11.52	α = 0.2
40		59	27	4.19	0.84	11.95	
50		62	28	4.34	0.87	12.37	
60	96.00	63	28	4.34	0.87	12.37	
70		74	33	5.12	1.02	14.51	
80		68	31	4.81	0.96	13.65	
90	95.00	55	25	3.88	0.78	11.09	
100		52	24	3.72	0.74	10.53	
110		48	22	3.41	0.68	9.67	
120	94.00	39	18	2.79	0.56	7.96	
130		38	17	2.64	0.53	7.54	
140		37	17	2.64	0.53	7.54	
150	93.00	35	16	2.48	0.50	7.11	
160		42	19	2.95	0.59	8.39	
170		48	22	3.41	0.68	9.67	
180	92.00	54	24	3.72	0.74	10.53	
190		53	24	3.72	0.74	10.53	
200	91.33	>100	> 45	> 6.98	> 1.40	> 19.91	

(Chart 4)

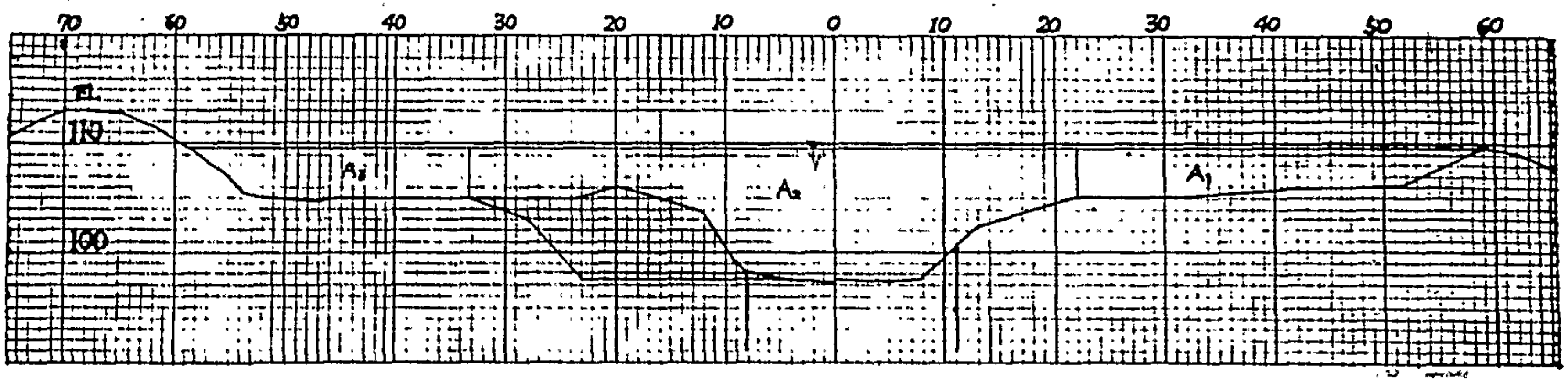

(Sounding)
(Fig.3)
San Juan River Soil Test with Cone Penetrometer
(Right Side 1) (Right Side 2) (Left Side 3) (Left Side 4)
Ft.
0 5 10 0 5 10 0 5 10 0 5 10 lb/in²
Allowable Bearing Stress
101.00
100.00
99.00
98.00
97.00
96.00
95.00
94.00
93.00
92.00
91.00

(Fig 4)

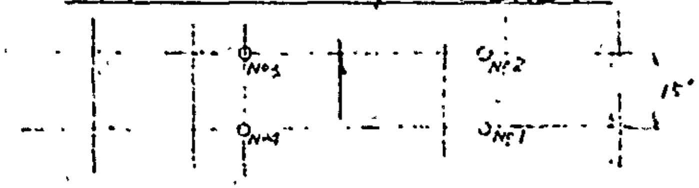
San Juan River (at Headworks expected)

Scale 1" = 10' Vertical & Horizontal

Sounding with [illegible]



Sect 20 At 2000' D/S of Bridge



ARANGUEZ (V) 20th July 1967

A meeting was held with Mr. E. J. Hamilton, Technical Officer (crop), Mr. S. Redman, Technical officer (Development) and Mr. Mc. D. Gibbs Drainage Engineer in the Ministry of Agriculture on 21st of June, to discuss the irrigation of Aranguez. My reports Aranguez (1) to (111) were explained and discussed them with the members.

The idea of irrigating the entire areas (including the upper cultivated land 929 acres and 202 acres) with sprinkler system was discussed as against furrow irrigation with the water that is raised by the headworks and led by canal. However, because of the hard labour to splash the water on the furrow it was expressed that perhaps a sprinkler system might be better. This was dis-agreed with for the following reasons.

1. In the case of this area, the general merits of sprinkler irrigation cannot be considered merits at all (ie) Saving of irrigation water - less water with sprinkler irrigation than with surface irrigation is needed to make it permeated into the required depth of soil, and uniform irrigation can be done.

Here in the Upper cultivated area, irrigation water of about 4 inches a month is available with furrow irrigation in dry season now, and for the lower land of Aranguez Estate the same amount of water is available. There is no reason why the water must be saved beyond this (cf. Report Aranguez (111), Sprinkler irrigation rather need much more water due to loss of water-application with sprinkler.

No expenses for land levelling and land smoothing is needed. Portable sprinkler irrigation is effective for slope land and undulate land.

This area is very even and hardly has any necessity of land levelling and land smoothing.

2. As the merits mentioned above are not available, the following demerit gets emphasized.

(ie) Generally more expensive than surface irrigation- The capacity, type and number of pumps will be stated below, and some sets of pipe will have to be disposed.

In Aranguez Estate, the water supply by gravity is available by raising up about 5 ft. (cf. Report Aranguez 111)

In upper cultivated area the water is now being supplied by two headworks and canals (cf. Report Anaguez 111).

3. The branch canal system in the upper cultivated land will become useless.

4. In order not to splash the water with hard labour, the improvement for furrow irrigation should be considered. Because the present way of furrow irrigation is undesirable. It is not reason enough why the furrow irrigation should be changed to sprinkler irrigation. The interval of each furrow stream, the adjustment of supply water and the height of furrow will be needed to study in relation to soil. Corruga-

tion irrigation may be necessary in some area.

To lay out the sprinkler irrigation system in Aranguez Estate. It would be necessary that the system be planned as a model in this area, however expensive it may be. Even in this case, it is uneconomical to pump up the water directly from San Juan River, as long pipelines and large capacity pumps are needed.

The sprinkler irrigation must be done after the water has been supplied by main canal from the headworks that raise the river 5 ft. higher.

Summary of Sprinkler Irrigation Plan in Aranguez Estate

(1) Area: 334 acres El. 101 ft - 96 ft.

(2) Objects of irrigation: Vegetables

(3) Plan for water supply:

(i) Unit water requirement:

Evapotranspiration (cf. Report Tucker Valley (II). (E.T).

Month	Vegetables				
	Consumptive water quantity			Rainfall in 20% wet in Caroni	Net water requirement
		inches		inches	inches
Dec.	4.4	= 112 m.m	3.7 m.m/day	2.9	1.5
Jan.	5.0	= 127	4.2 (E.T)	1.1	3.9
Feb.	4.8	= 122	4.1 (E.T)	0.5	4.3
Mar.	5.4	= 137	4.6 (E.T)	0.6	4.8
Apr.	5.3	= 135	4.5 (E.T)	0.6	4.7
May	5.4	= 137	4.6	2.1	3.3

(iii) Minimum interval for intermittent irrigation

quantity of water supply a time : T.R.A.M.

maximum consumptive water quantity a day : max E.T.

$$\text{Minimum interval} = \frac{\text{T.R.A.M.}}{\text{max. E.T.}} = \frac{35.5\ \text{m.m.}}{4.6\ \text{m.m}} = 7.7 \doteqdot 7\ \text{days}$$

(N)

Month	Dec.	Jan.	Feb.	Mar.	Apr.	May
T.R.A.M	35.5	35.5	35.5	35.5	35.5	35.5
E.T.	4.7	4.2	4.1	4.6	4.5	4.6
N	9 days	8	8	7	7	7

(4) Gross quantity of water supply a time with sprinkler

$$G = \frac{\text{T.R.A.M.}}{\text{water-application efficiency}} = \frac{35.5\ \text{m.m.}}{0.60} = \begin{matrix} 59.2\ \text{m.m.} \\ (2.34\ \text{inches}) \end{matrix}$$

(5) Gross water requirement a day

Assuming that water conveyance and diversion losses in main canal is 3% and the ones in branch canals and pipes is 5%.

$$Q = \frac{59.2\ \text{m}}{1000} \times \frac{1}{7\ \text{days}} \times 1.08 \times 334\ \text{acres} \times 4{,}047\ \text{m}^2/\text{acres} = 12{,}300\ \text{m}^3/\text{day}$$

$(= 435{,}000\ \text{ft}^3/\text{day})$.

(FC)			(W.P)
W	:	Field capacity -	Wilting point (in weight %)
Dd	:	Assumed specific gravity of soil which is considered to be 0.9 in silty clay loam	
h	:	Thickness of soil layer which is $\frac{1.5'}{4}$ = 0.375' (= 113 m.m)	
F.C	:	26% in silty clay loam	
W.P	:	12% in silty clay loam	

$$\text{A.M.} = \frac{(26 - 12) \times 0.9}{100} \times 113 = 14.2 \text{ m.m. } (= 0.56")$$

d) T. R. A. M.

(A) Layer distinc- tion	(B) Depth of Layer	(C) Ratio of mois- ture extraction	(D) A.M. in each layer inch m.m	(E) D/C inch m.m	(F) Regulation	(G) T.R.A.M inch m.m	(H) each moisture ex- traction in T.R.A.M
1st layer	0-0.375 ft	40%	0.56(14.2)	1.40(35.5)	0	1.40(35.5)	0.56inch (14.2)mm
2nd layer	0.375-0.750	30%	0.56(14.2)	1.87(47.3)			0.42(10.7)
3rd layer	0.750-1.125	20%	0.56(14.2)	2.80(71.0)			0.28(7.1)
4th layer	1.125-1.500	10%	0.56(14.2)	5.60(142.0)			0.14(3.5)
Total							1.40(35.5)

(ii) Total readily available moisture (T.R.A.M)

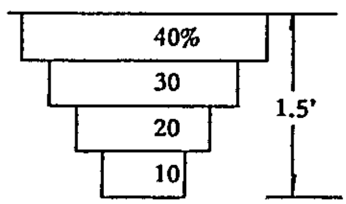
a) Effective root zone

Crops	Effective root zone
Pea	2.0 – 3.0 ft.
Bean	2.0
Cabbage	1.5 – 2.0
Carrot	1.5 – 2.0
Corn	2.5 – 3.0
Cotton	4.0
Cucumber	1.5 – 20
Lettuce	1.0
Melon	2.5 – 3.0
Onion	1.0 – 1.5
Potato	2.0
Sweet Potato	3.0
Tobacco	2.5
Tomato	3.0
Effective root zone adopted – 1.5 ft.	

b) Moisture extraction pattern in effective root zone.

Strictly speaking, each crop has a proper moisture extraction pattern.

In this case it is assumed the mean moisture extraction pattern is as follows.



c) Available moisture (A.M)

Each layer is the same soil and homogeneous, silty clay loam.

$$\text{A.M.} = \frac{\Delta W \times Dd}{100} \times h$$

(6) Irrigation intensity with sprinkler
The maximum irrigation intensity to homogeneous silty clay loam in the slope of 0% to 5% is regarded as 13 m.m. an hour.

Therefore 9 mm an hour was adopted as irrigation intensity with sprinkler considering the safety factor.

(7) Designed time required for sprinkler irrigation

$$T = 59.2 \text{ mm} \;/\; 9 \text{ mm} = 6.6 \text{ hours} \quad \text{(Net requirement)}$$
$$0.4 \quad \text{(Time for transfer)}$$
$$\text{Total} = 7.0 \text{ hour}$$

I designed the irrigation with sprinkler was done three times a day.

(ie) 6 a.m. - 1 p.m.
2 p.m. - 9 p.m.
10 p.m. - 5 a.m. (next day)

(8) Selection of sprinkler system: As mentioned above, the headworks must be built at the section 20 on San Juan River to raise the water 5 ft higher; next, the main canal of about 42,000 ft long must be built from east to west and successively four branch canals from north to south. The total length of branch canals is about 12,250 ft. The irrigation water is taken from the branch with pumps. Pumps and pipes must be moved to next zone along the branch canal after finishing water supply to the zone concerned. The pipe is disposed as the split-line arrangement to both sides from a pump.

(9) Capacity of sprinkler head

$$q = \frac{dS_1S_2}{60T} = \frac{59.2 \times 12 \times 18}{60 \times 6.6} = 32.3 \text{ l/min. } (= 7.1 \text{ Gallon/min})$$

q	:	Capacity of sprinkler head (1/min)
d	:	Depth of water supply (mm)
S_1	:	Interval of riser pipes (m) assumed as 12m (40 ft.)
S_2	:	Interval of main pipes (m) assumed as 18 m (60 ft.)
T	:	Designed time required (hr).

The following type was adopted as the sprinkler head.
Intermediate pressure sprinkler head
Pressure: 2.45 kg/cm^2 (35 lb/in^2)
Bore of nozzle : 4.8 mm x 3.2 mm (0.19" x 0.13")
Diameter of sprinkling 28.7 m (94 ft.)
Capacity of sprinkler head 32.3 l/min. (7.1 gallons/min)

(10) Scale and number of sprinkler set

Profitable area: 334 acres = 1,350,000 m^2
Area irrigated with one sprinkler : 12 m x 18 m = 216 m^2
Total number of sprinkler : 1,350,000/216 = 6,250
Capable times of transfer during minimum interval for intermittent irrigation: 7 days x 3 times/day = 21 times
Number of sprinkler required once: If one sprinkler set has 30 sprinklers, 15 each on both sides of a pump, 10 sets will be needed.

(11) Capacity of pump

Friction loss pipe : $Hf = \frac{k_1 L V^m}{D^n}$ $\quad V = Q/A = \frac{Q}{\frac{(\pi D_2)}{4}} = \frac{K_2 Q}{D_2}$

Therefore $V^m = \frac{K^m Q^m}{D_2{}^m}$ $\quad \therefore Hf = \frac{KLQ^m}{D^{2\ m+n}}$

Hf	:	Friction loss of pipe
L	:	Length of pipe
V	:	Mean velocity in pipe
D	:	Diameter of pipe
Q	:	Discharge
A	:	Area of pipe section
M	:	Exponents used in Formula for friction loss
N	:	
K	:	Friction coefficient

Weisbach's formula

$$Hf = \frac{fLV^2}{2gD} \qquad f = 0.028 - 0.018$$

There are many formulae for friction loss. In this case Weisbach's formula was used.

Friction loss of pipe with many outlets:

$$Hf = F\left(\frac{KLQ^m}{D^{2m+n}}\right) \qquad F = \frac{1}{m+1} + \frac{1}{2N} + \frac{m-1}{6N^2}$$

F	:	Friction loss coefficient of pipe with many outlets to ordinary pipe.
N	:	number of outlets which is now 15
M	:	Exponent which is now 2.

$$\therefore F = \frac{1}{2+1} + \frac{1}{2 \times 15} + \frac{2-1}{6 \times 15^2}$$
$$= \frac{1}{3} + \frac{1}{30} + \frac{1}{1{,}350} = \frac{496}{1{,}350} = 0.3674$$

By Weisbach's formula

$$K = \frac{f}{2g} = \frac{0.021}{2 \times 9.8} = 0.00107 = 1.07 \times 10^{-3}$$

$m = 2$

$n = 1$ $\quad\quad$ $L = 15 \times 12 = 180$ m

D : Diameter of pipe which is now designed as 75 mm

Q = 32.3 l/min x 15 = 484.5 l/min = 8.0751/sec = 8.075×10^{-3} m³/sec.

$$\therefore \frac{KLQ^m}{D^{2m+n}} = \frac{1.07 \times 1.80 \times (8.075)^2 \times 10^{-3} \times 10^2 \times 10^{-6}}{(75)^5 \times 10^{-15}}$$

$$= 5.292 \text{ m}$$

$$Hf = \frac{F(KLQ^m)}{D^{2m+n}} = 0.3674 \times 5.292 = 1.95 \text{ m}$$

(= 0.195 kg/cm²)

Hydrostatic head required for pump:

(1) Head for sprinkler head ..... 24.5 m

(2) Friction loss of pipe .... 1.95 m

(3) Losses of riser pipe, taps, joints valve etc ... 2.45 m

(1) x 10% ..... nearly 0 m

(4) Actual lift

$\Sigma H = 24.5 + 1.95 + 2.45 = 28.9$ m

Capacity of pump

Axial power = $QH/4{,}500\eta$ = 969 x 28.9/4,500 x 0.6 = 10.4 P.S.

Q : Discharge .... 30 x 32.3 l/min = 969 l/min

H : Total head .... 28.9 m

η : Coefficient of pump 0.6

12. A list of materials required for sprinkler irrigation

(per one set)

Name of materials	Specifications	No.	Remarks
Portable pump	Centrifugal 10.4 P.S.	1	with engine, hose strainer
Pipe	3' x 20'	60	
Riser pipe	15' - 2.5'	30	for vegetable
Joint for pipes		60	simple connecting devices
Joint for riser		30	
Sprinkler head	35 lb/in 7.1 gal/min	30	
Connecting hose	10' - 15'	1	
Manometer		1	
Elbows, Tees		a few	
End tap		1	

13. Maximum gross water requirement:

$$Q_{max} = 12{,}300\ m^3/day \times \frac{24}{3 \times 6.6} \times \frac{1}{24 \times 60 \times 60} = 0.173 m^3/sec\ (6.1\ cusecs)$$

This gross water requirement occurs with sprinkler irrigation which has water-application efficiency of 60%, water conveyance and diversion losses of 8% in March, April, and in May 20% wet.

(cf. In 'a' % of a number of years, the monthly rainfall is less than q' (inches) the month with a rainfall of q inches is called 'a' % wet).

Further the net consumptive water quantity that this gross water requirement is on the basis of is calculated as 5.4 inch a month by Blanney-Criddle formula, although maximum net water requirement is 4.8 inch a month as the rainfall of 0.6 inch a month can be expected. This gross water requirement increases more by limitation of time for pump operation, and omission of a fractional sum of minimum interval for intermittent irrigation.

If the possible water quantity to supply from San Juan River which is 3.71 cusecs (cf: Report Aranguez 11) is compared with this gross water requirement, it will be about 60%.

This rate is just equal to water-application efficiency with sprinkler.

It means that the water as much as the loss of water-application with sprinkler cannot be supplied to the fields. That is to say the water quantity available from San Juan River can be only supplied to 60% of the field of 334 acres.

It should be renewed understanding that the loss of water-application with sprinkler in the tropics is very much.

1) Water-application efficiency 0.6 x 1/0.6 = 1.67
2) Limitation of time for pump operation x 24.0/3 x 6.6 = 1.21
3) Omission of a fractional sum of minimum interval for intermittent irrigation x 7.7/7.0 = 1.10
4) Conveyance and diversion losses x 1.08

Therefore, total multiplication is 1.67 x 1.21 x 1.10 x 1.08 = 2.40

The water that is equal to the net consumptive water quantity multiplied by 2.40 is needed to the area, Aranguez Estate of 334 acres.

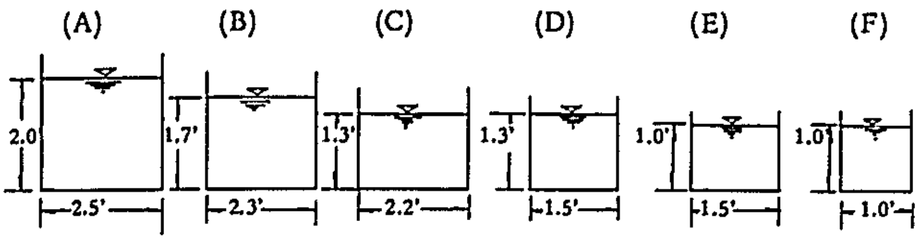
Water-application efficiency of 0.6 will be explained supplementarily.

It contains four factors as follows.

1) Evaporation loss from sprinkler
2) Interception of vegetation from reaching soil surface } 10%

3) Evaporation loss from, wet surface of soil - 15%
4) Uniformity of sprinkling with sprinkler itself and influence of wind; Uniformity coefficient 0.80

Therefore (1 - 0.1 - 0.15) x 0.80 = 0.60
The efficiency was determined as 0.60.



The canal must be lined with concrete and have vertical side walls to keep the depth as deeply as possible for the suction pipe of pump.

The water level was calculated considering the head loss due to diversion and section contraction at the end of each canal was 0.1 ft.

Junichi Kitamura

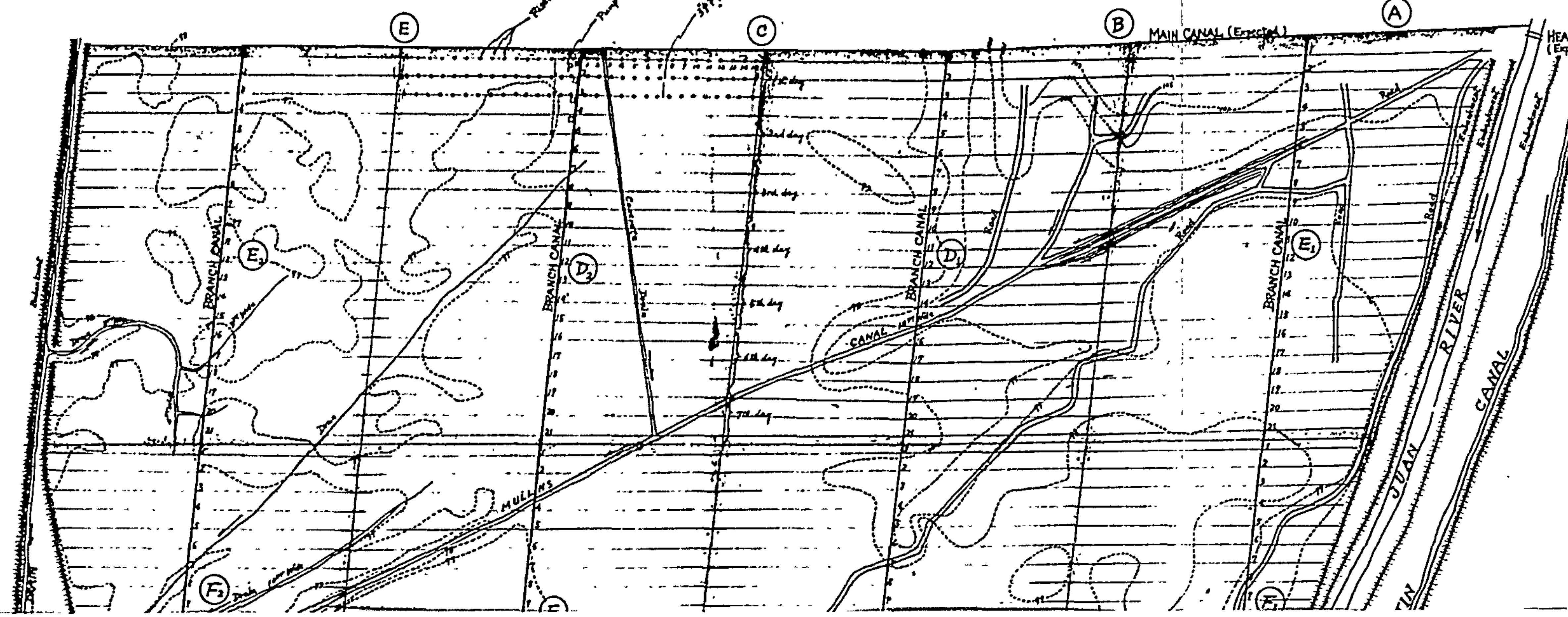
(14) Design for Canal

Design of the canal section which can flow the gross water requirement sufficiently is needed, even if there are times when San Juan River is not able to supply the water requirement. It is showed in the following tables and the appended map.

	No.	W.L. of beginning	W.L. of End	Length	Slope	Planned discharge	n	N	D	H	B	A	P	R	$\sqrt{R}$	NR	$D\sqrt{R}$	V	Q
		ft.	ft.	m		m^3/s				m	m	m^2	m	m				m/s	m^3/s
Main	A	101.05	100.64	220	1/1,800	0.20	0.017	1.994	0.438	0.60	0.75	0.45	1.95	0.231	0.480	0.461	0.918	0.502	0.226
	B	100.54	99.87	360	"	0.16	"	"	"	0.50	0.70	0.35	1.70	0.206	0.454	0.411	0.892	0.460	0.161
	C	99.77	99.10	360	"	0.10	"	"	"	0.40	0.65	0.27	1.45	0.186	0.431	0.371	0.869	0.427	0.115
	E	99.00	98.37	338	"	0.04	"	"	"	0.30	0.45	0.135	1.05	0.129	0.357	0.256	0.795	0.322	0.044
Branch	D_1	99.77	99.14	378	1/2,000	0.06	0.017	1.899	0.444	0.40	0.45	0.18	1.25	0.144	0.380	0.273	0.824	0.332	0.060
	D_2	99.00	98.37	378	"														
	E_1	100.54	99.91	378	"														
	E_2	98.27	97.64	378	"	0.04	"	"	"	0.30	0.45	0.135	1.05	0.129	0.357	0.244	0.801	0.305	0.041
	E_3	99.04	98.41	378	"														
	E_4	98.27	97.64	378	"														
	F_1	99.81	99.13	457	"														
	F_2	97.54	96.90	386	"	0.02	"	"	"	0.30	0.30	0.09	0.90	0.100	0.316	0.19	0.760	0.250	0.023
	F_3	98.31	97.76	331	"														
	F_4	97.54	97.05	294	"														

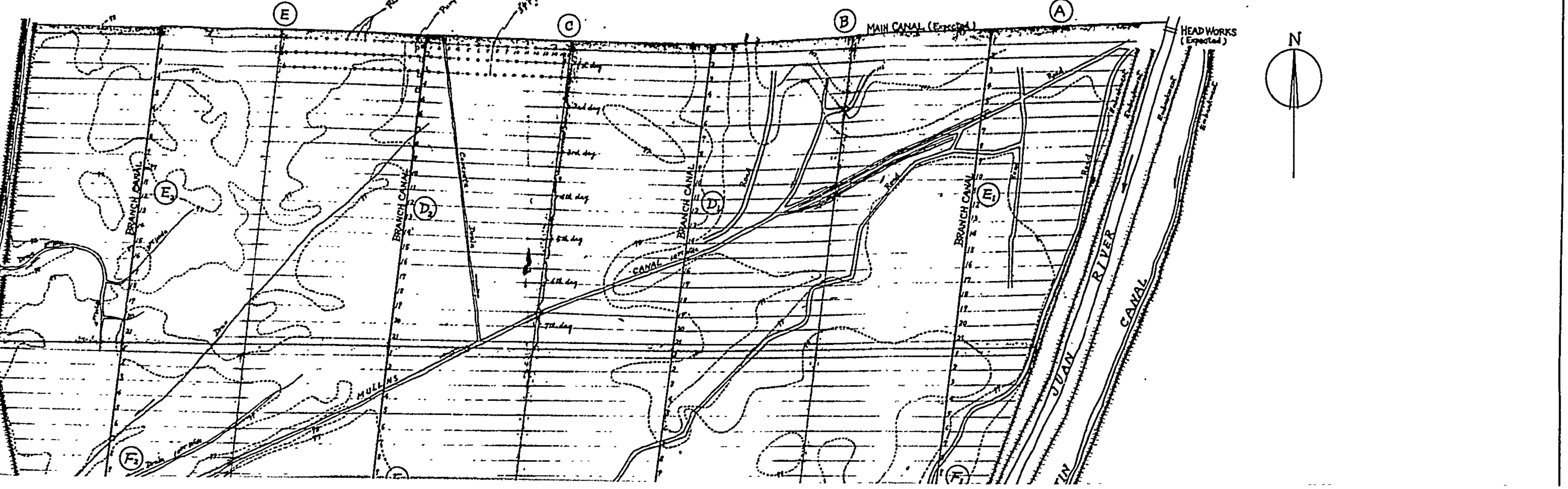
AREA 334

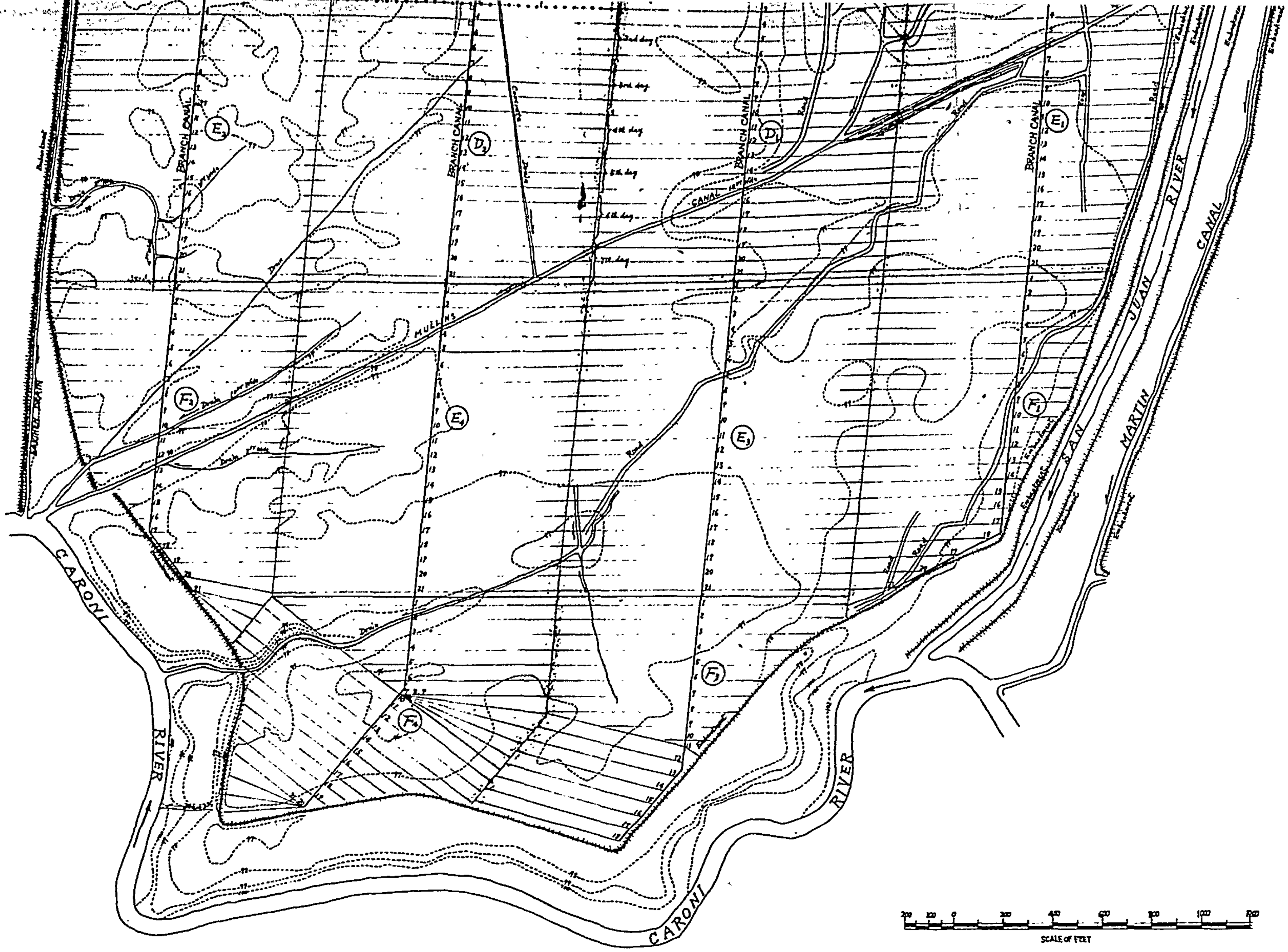
SPRINKLER IRRIGATION PLAN FOR LOWER ARANGUEZ



AREA 334 acres

LER IRRIGATION PLAN FOR LOWER ARANGUEZ




BRANCH CANAL
BRANCH CANAL
BRANCH CANAL
BRANCH CANAL
CANAL
MULLINS
Road
E2
D2
D1
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F2
E4
E3
F1
F3
F4
SAN JUAN RIVER
MARTIN RIVER
CANAL
CARONI
RIVER
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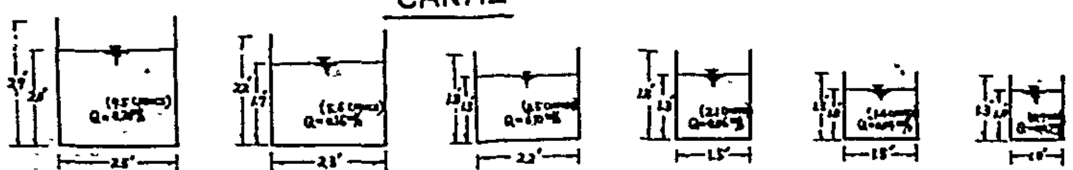
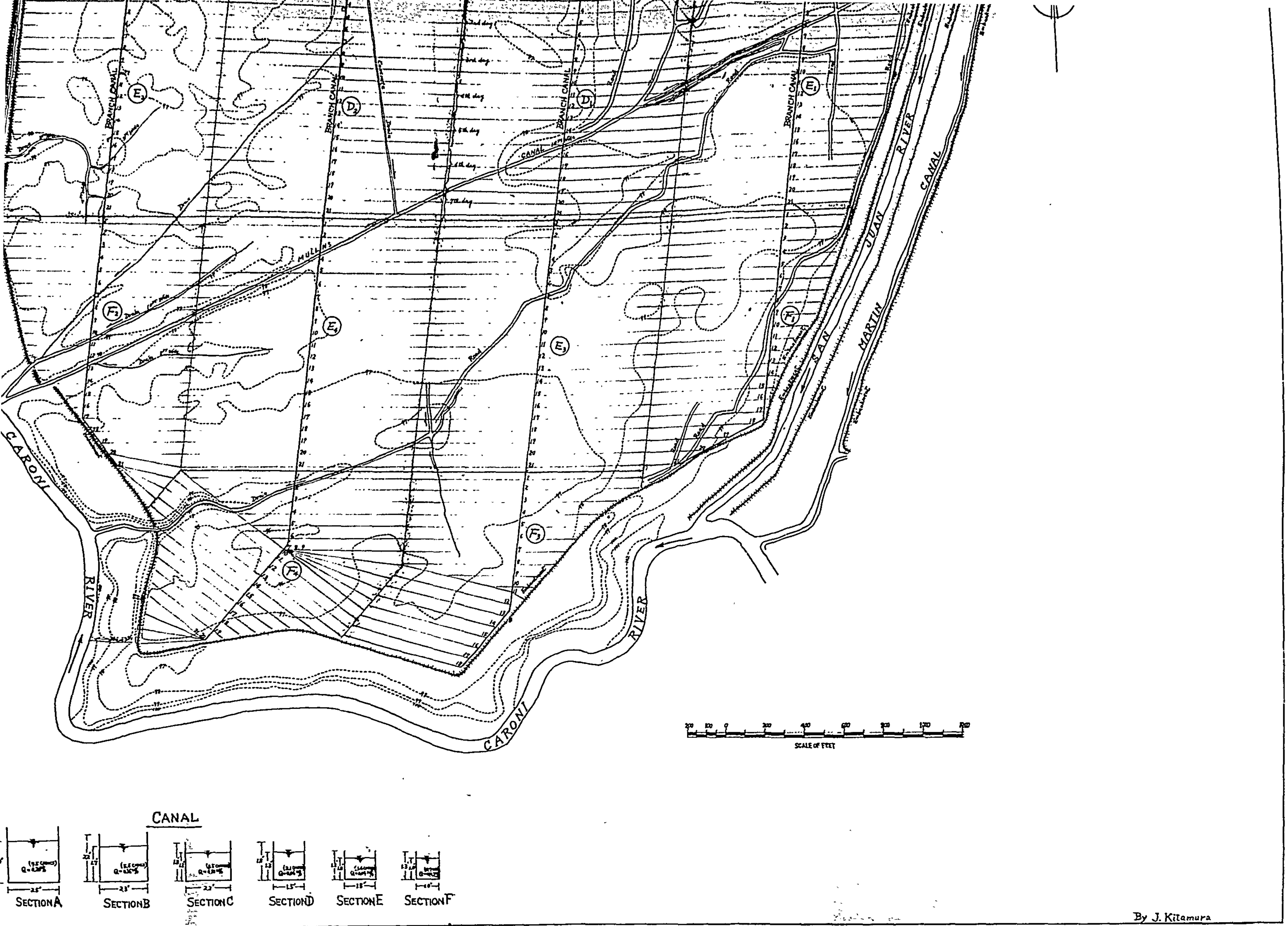
SCALE OF FEET

CANAL


SECTION A
SECTION B
SECTION C
SECTION D
SECTION E
SECTION F


BRANCH CANAL
MULLINS CANAL
SAN JUAN RIVER
SAN MARTIN CANAL
CARONI RIVER
E1
E2
E3
E4
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D2
F1
F2
F3
F4
SCALE OF FEET
CANAL
SECTION A
SECTION B
SECTION C
SECTION D
SECTION E
SECTION F
By J. Kitamura

ARANGUEZ (VI) August, 1967

DRAINAGE PLAN FOR LOWER ARANGUEZ

Drainage must be proceeded irrigation for the area.

As lower Aranguez is now receiving water from upper cultivated Aranguez the whole of Aranguez is regarded as one drainage area, that is about 1200 acres of which 664 acres are north of Churchill Roosevelt Highway and 536 acres are South of it. However, the drainage area of 536 acres south of the Highway, if a new collector canal to the San Juan River is built along the highway, will become a separate drainage area.

1) Determination of height of embankment for the Caroni River. The data picked up as shown in the following chart is from the Old Pumping Station along the Caroni River.

(Chart 1) HIGH WATER LEVEL RECORD AT OLD PUMPING STATION, CARONI RIVER
DATUM 93.17 ft.

Year	1960															
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
Nov. 26	/	102.4	102.5	102.5	102.7	102.9	103.0	103.0	103.0	103.1	103.1	103.1	103.0	/	/	/	/
27	/	> 103.2	102.5	102.5	102.7	102.9	103.0	103.0	103.0	103.1	103.1	103.1	103.1	/	/	/	/
28	/	102.9	102.8	102.6	102.3	102.1	101.7	101.4	101.1	100.9	100.7	100.4	100.1	/	/	/	/
Year	1961																
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
Oct. 21	99.0	98.7	98.5	98.3	98.4	98.7	98.7	99.5	99.9	100.1	100.3	100.6	100.8	/	/	/	/
22	103.4	103.6	103.7	103.8	103.8	103.8	103.8	103.8	103.8	/	/	/	/	/	/	/	/
23	102.6	102.4	102.2	102.0	101.7	101.5	101.2	101.2	101.1	101.0	100.9	100.7	100.5	/	/	/	/
Year	1962																
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
July 28	98.7	98.7	98.8	98.9	99.0	99.0	99.1	99.2	99.3	99.4	99.7	100.3	100.7	/	/	/	/
29	102.3	102.3	102.4	102.5	102.6	102.6	102.7	102.8	102.8	/	/	/	/	/	/	/	/
30	101.6	101.5	101.4	101.2	101.0	100.8	100.6	100.4	100.2	100.2	100.1	100.0	100.0	99.9	99.8	99.7	/
Year	1963																
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
Sep. 30	/	95.3	95.9	96.1	96.3	96.5	96.6	96.4	96.3	98.1	98.5	98.5	99.8	99.4	99.7	99.8	100.1
Oct. 1	/	100.9	101.2	101.3	101.4	101.4	101.5	101.5	101.5	101.5	101.5	101.5	101.5	101.6	101.6	101.6	101.7
2	/	101.5	101.6	101.6	101.6	101.6	101.7	101.6	101.5	101.4	101.4	101.3	101.3	/	/	/	/
Year	1964																
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
July 21	/	98.5	98.6	98.7	98.7	98.5	98.9	99.1	99.5	100.0	100.4	100.7	100.9	101.0	101.0	101.1	101.1
22	/	101.1	101.2	101.3	101.3	101.4	101.4	101.4	101.4	101.4	101.5	101.7	101.7	/	/	/	/
23	/	101.9	101.9	101.8	101.5	101.5	101.7	101.7	101.7	/	/	/	/	/	/	/	/
24	/	99.4	99.3	99.3	99.2	99.2	99.2	99.1	99.1	/	/	/	/	/	/	/	/
Year	1965																
Time	6 AM	7	8	9	10	11	12	1 PM	2	3	4	5	6	7	8	9	10
Day																	
Nov. 24	/	/	/	/	/	/	/	/	100.6	100.6	100.5	100.5	100.5	/	/	/	/
26	/	/	/	/	/	/	/	/	100.7	101.7	101.8	101.8	101.9	101.9	101.9	101.9	101.9
27	/	/	/	/	> 102.0	102.0	102.0	102.0	102.0	102.0	102.0	102.0	102.0	102.0	102.0		
28 >	102.0	102.0	102.0	102.0	102.0	102.0	102.0	101.9	102.8								

In 1960 and 1965 the highest water levels were above the highest graduation on the gauge boards therefore no readings are available.

However the water did not overflow the embankment which is at 104'.

A free board is calculated by the formula as follows.

$$Fb = Q^{1/3} / 10\,(m)$$

Q : high water discharge (m^3)

Fb : free board (m)

There are only a few data of high water discharge at Piarco by F.A.O. report.

Year	Discharge
1947	5,700 cusecs
1950	7,300
1953	5,600

Drainage area at Piarco is 96,233 acres.

The high water discharge at Old Pumping Station was found to be as follows.

Drainage area at Old Pumping Station is 158,751 acres, by the map showing catchment area plan of the Caroni.

Therefore

$$Q\ max. = 7{,}300 \text{ cusecs} \times \frac{158{,}751}{96{,}233} \text{ acres} = 12{,}000 \text{ cusecs} = 340\ m^3$$

$$\therefore \quad Fb = (340)^{1/3} / 10 = 0.70\ m = 2.3\ ft = 3\ ft.$$

As the result of the calculation, height of the embankment can be decided as 107 ft.

2) Decision of Capacity and number of pumps for drainage in case of no collector canal.

i) Arrival at the curve of relation between flooding level and area, flooding volume and area.

The flooding area was measured with a planimeter and the flooding volume was calculated as the following Table.

(Chart 2) CAPACITY OF ARANGUEZ AREA TO KEEP FLOOD

Elevation	Area		Mean Area	Volume	Total Volume		
ft.	acres	ha	acres	acre. ft.	acre ft.	ft^3	m^3
96 >	3	1.21	1.5	1.5	1.5	65,340	1,850
97 >	108	43.71	55.5	55.5	57.0	2,482,920	70,316
98 >	312	126.26	210.0	210.0	267.0	11,630,520	329,376
99 >	390	157.83	351.0	351.0	618.0	26,920,080	762,377
100 >	430	174.01	410.0	410.0	1,028.0	44,779,680	1,268,161

ii) Calculation of flood discharge at proposed pumping station.

(Chart 3) PROBABILITY FROM DATA OF RAINFALL IN CARONI

Probability	24 hrs Rainfall
1/1	2.9 inch
1/2	3.4 (50% wet)
1/5	4.0 (80% wet)
1/10	4.5 (90% wet)
1/20	5.0
1/30	5.3
1/50	5.7
1/100	6.3

Period of time	Dec. rainfall in ins.	Evapo-transpiration in ins.	Storage capacity in ins.	Net rainfall to be drained off in ins.
1 day	4.00	0.15	0.60	3.25
2 days	5.25	0.30	0.60	4.35
3 days	6.00	0.45	0.60	4.95
5 days	6.65	0.75	0.60	5.30

Distance from the farthest point to upper reach of main drain

l_1 = 4,170 ft = 1,250 m

Distance from upper reach of main drain to proposed pumping station

l_2 = 10,420 ft = 3,130 m

Time required to flow into drain on field

$$t_1 = \frac{l_1}{V_1} \qquad V_1 \quad : \text{ velocity of flow on field 0.3 m/sec.}$$

$$= \frac{1,250}{0.30 \times 60} = 69.5 \text{ min}$$

Time required to flow down in drain

$$t_2 = \frac{l_2}{V_2} \qquad V_2 \quad : \text{ velocity of flow in drain}$$

$V_2 = \frac{(H)^{0.6}}{l_2}$ (m/s) H : Difference of elevation between upper reach and proposed pumping station.

$H = 109.0\text{ ft} - 94.0\text{ ft} = 15.0\text{ ft}$

$= 4.57\text{ m}$

$V_2 = 20\frac{(4.57)^{0.6}}{3{,}130} = 20(0.00146)^{06} = 0.398 \fallingdotseq 0.40\text{ m/s}$

$t_2 = \frac{3{,}130}{0.40 \times 60} = 130.4\text{ min}$

Total time for arrival

$T = t_1 + t_2 = 69.5 + 130.4 = 199.9\text{ min.}$

$\fallingdotseq 3.33\text{ hr.}$

Maximum flood discharge

$Q_{max} = 0.2778\,frA$: f : run-off coefficient at peak of flood wich is 0.30 on plain field

$r = \frac{r_{24}(24)^{2/3}}{24\ T}$ r : mean hourly rainfall intensity within period till arrival of flood

A : drainage area which is 1,200 acres (= 4.856 km^2)

r_{24} = daily rainfall which is 4.0" (101.6 mm) in 80% wet.

$r = \frac{101.6\ (24)^{2/3}}{24\ \ 3.33} = 4.233\,(7.207)^{2/3} = 4.233 \times 3.731 = 15.79\text{ mm.}$

$Q_{max.} = 0.2778 \times 0.30 \times 15.79 \times 4.856 = 6.39\text{ m}^3/\text{s}$

Total run-off for a day

$\Sigma Q = r.f'.A$ f' : total runoff coefficient in one flood which is 0.5 on plain field.

$\Sigma Q = \frac{101.6}{1{,}000} \times 0.50 \times 4.856 \times 10^6 = 246{,}685\text{ m}^3$

Time for run-off

$\frac{(1+\lambda)}{2}\ T\ 3{,}600\ Q\,max = r.f'.A$

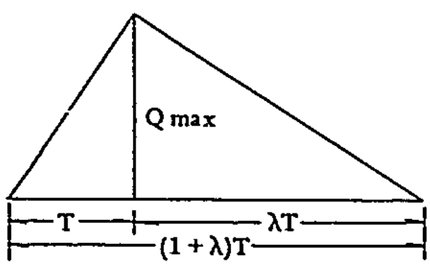
$\frac{(1+\lambda)}{2} \times 3.33 \times 3{,}600 \times 6.39 = 246{,}685$

$76{,}603\,(1+\lambda) = 493{,}370$

$(1+\lambda) = 6.44$

$\lambda = 5.44$

$\therefore\ \lambda T = 5.44 \times 3.33 = 18.12\text{ hr.}$



iii) External water level

As shown chart 1. All the hourly water level cannot be obtained since only observations between 12 hours and 15 hours was done.

However it can be supposed that high water level above 96 ft, sometimes 97 ft will continue for three days or more. In the circumstances regulating gates are useless for discharge of internal water.

Therefore, during these occasions only discharge by pump must be considered.

Regulating gates will have to be designed in the condition that the external water level is below 96 ft, and will decrease pump operation.

iv) Capacity and number of pumps

In order to drain the flood in 80% wet in a day, drainage discharge required is calculated as follows.

$$qa = Q/24 \times 60 \times 60 = 246{,}685/86{,}400 = 2.86\ m^3/sec\ (171.6\ m^3/min)$$
$$(= 101\ \text{cusecs})$$

Therefore if two of the same type propeller pumps are used, the capacity will be 85 m^3/min, in diameter of 800 mm (31.5 inches) and the axial horse power of 85 HP when the total head is about 3.5 m (= 12 ft).

(Chart 4)

Time hr.	Flood Discharge $Q\ m^3/s$	Mean flood discharge $Qm\ m^3/s$	Mean flood discharge $Qm\ m^3$	Total flood discharge $\Sigma Qm\ m^3$	Drainage discharge by pump $qa\ m^3$	Flooding volume $Q_f\ m^3$	Flood stage El. ft.	with no pump: Total flood discharge $\Sigma Q\ m^3$	with no pump: Flood stage El. ft.
0	0	0	0	0	0	0	< 96.00	0	< 96.00
1	1.90	0.95	3,420	3,420	3,420	0	< 96.00	3,420	< 96.00
2	3.85	2.88	10,368	10,368	10,200	168	< 96.00	13,788	96.30
3	5.80	4.83	17,388	17,556	10,200	7,356	96.05	31,176	96.55
(3.3)	(6.39)								
4	6.14	6.24	22,464	29,820	10,200	19,620	96.36	53,640	96.81
5	5.79	5.97	21,492	41,112	10,200	30,912	96.55	75,132	97.04
6	5.43	5.61	20,196	51,108	10,200	40,908	96.66	95,328	97.17
7	5.08	5.26	18,936	59,844	10,200	49,644	96.78	114,264	97.23
8	4.73	4.91	17,676	67,320	10,200	57,120	96.87	131,940	97.31
9	4.38	4.56	16,416	73,536	10,200	63,336	96.97	148,356	97.37
10	4.02	4.20	15,120	78,456	10,200	68,256	97.00	163,476	97.43
11	3.67	3.85	13,860	82,116	10,200	71,916	97.02	177,336	97.49
12	3.32	3.50	12,600	84,516	10,200	74,316	97.05	189,936	97.54
13	2.96	3.14	11,304	85,620	10,200	75,420	97.06	201,240	97.59
14	2.61	2.79	10,044	85,464	10,200	75,264	97.06	211,284	97.61
15	2.26	2.44	8,784	84,048	10,200	73,848	97.03	220,068	97.63
16	1.90	2.08	7,488	81,336	10,200	71,136	97.01	227,556	97.66
17	1.55	1.73	66,228	77,364	10,200	67,164	96.97	233,784	97.67
18	1.20	1.38	4,968	72,132	10,200	61,932	96.94	238,752	99.69
19	0.85	1.03	3,708	65,640	10,200	55,440	96.84	242,460	97.71
20	0.49	0.67	2,412	57,852	10,200	47,652	96.73	244,872	97.72
21	0.14	0.32	1,152	48,804	10,200	38,604	96.64	246,024	97.73
(21.4)	(0)								
22	0	0.04	144	38,746	10,200	28,548	96.47	246,168	97.73
23	0	0	0	28,548	10,200	18,348	96.34	246,168	97.73
24	0	0	0	18,348	10,200	8,148	96.07	246,168	97.73
25	0	0	0	8,148	8,148	0	< 96.00	246,168	97.73

The seepage flow from dykes seems to be very little on account of long path of percolation in designed section of the embankment along the Caroni River. Therefore seepage volume was not calculated, although the capacity of pump must have clearance to discharge seepage water.

3) Capability of discharge with collector canal.

The area of Upper Aranguez is about, 55% of the whole Arangues Area.
Therefore, it is expected to contribute approximately 55% of the flood discharge.

$Q = 6.39\ m^3/s \times 0.55 = 3.5\ m^3/s$ (= 124 cusecs)

i) Decision of section at mouth of collector canal

$I = \frac{1}{1{,}000}$ Assuming the following section

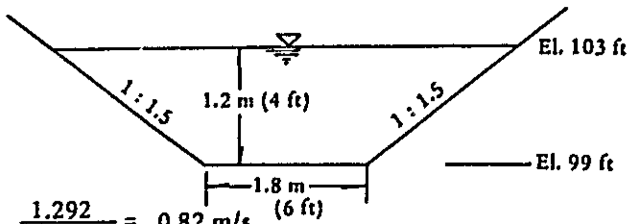
$n = 0.03$ $N = 1.831$

$P = 6.12$ (m) $D = 0.7365$

$A = 4.32$ (m)

$R = 0.706$ (m)

$\sqrt{R} = 0.8402$



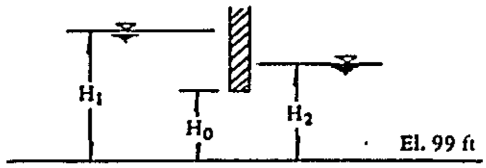
$$V = \frac{NR}{D + \sqrt{R}} = \frac{0.706 \times 1.831}{0.7365 + 0.8402} = \frac{1.292}{1.5767} = 0.82\ m/s$$

$\therefore$ $Q = AV = 4.32 \times 0.82 = 3.54\ m^3\ s > 3.5\ m^3/s$

ii) Scale of sluice gates

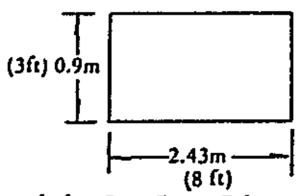
$H_1 = 1.2$ m

$H_2 = 0.9$ m Assuming that the head of 0.3 m (= 1 ft) is given the gate.



$$Q = mb\,H_o\,\sqrt{2g\,(H_1 - H_2)}$$
$$= 0.66 \times b \times 0.9\ \sqrt{2 \times 9.8 \times 0.3}$$
$$= 1.44b = 3.5$$

$b = 2.43$ m



iii) Relation of water level between collector canal and the San Juan River

The existing invert level of the San Juan River at the bridge on Churchill Roosevelt Highway is nearly 99 ft and the top of the embankment is about 112 ft to 114 ft.

On the other hand the designed invert level of the collector canal is 99 ft. which equals to the one on the San Juan and the water level is only 103 ft. It means that the water level of the collector canal is much lower than the flooding level of the San River and the collector canal cannot discharge the internal water to the San Juan by gravity for most of the time.

This upper Aranguez further needs pumps to discharge the internal water even if it has collector canal and sluice gate. This therefore eliminates the use of sluice gates.

4) DESIGN OF REGULATING GATE IN LOWER ARANGUEZ

As mentioned above the internal water cannot be discharged by gravity during large flood due to the high water level of the external water. It appears that only 60% of the maximum flood discharge can be accomodated by gravity.

$$Q = 6.39 \times 0.6 = 3.83 = 4.0\ m^3/s\ (= 141\ cusecs)$$

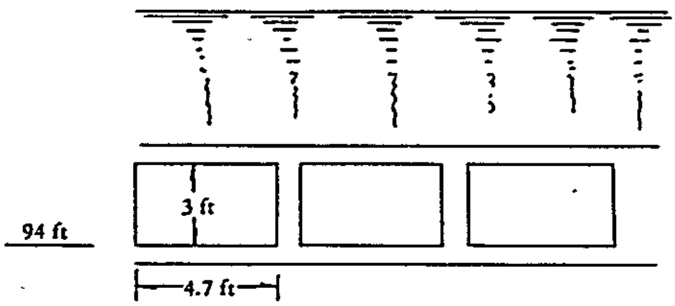
Elevation of the sill is assumed as 94.00 ft.

H_1 = 3.0 ft (= 0.90 m)
H_2 = 2.0 ft (= 0.60 m)
H_0 = 2.0 ft (= 0.60 m)

Assuming that the head of 1 ft (= 0.3 m) is given the gate.

$$\begin{aligned} Q &= mbHo\sqrt{2g(H_1 - H_2)} \\ &= 0.66 \times b \times 0.6\sqrt{2 \times 9.8 \times 0.3} \\ &= 0.96\,b = 4.0 \\ \therefore b &= 4.17\ m \quad 3\ gates \times 1.44\ m\ (= 4.7\ ft). \quad \fallingdotseq \quad 4.2\ m \end{aligned}$$

As the result three box culverts attached sluice which are 4.7 ft wide each, 3 ft high each are needed.



5) Drainage Canal

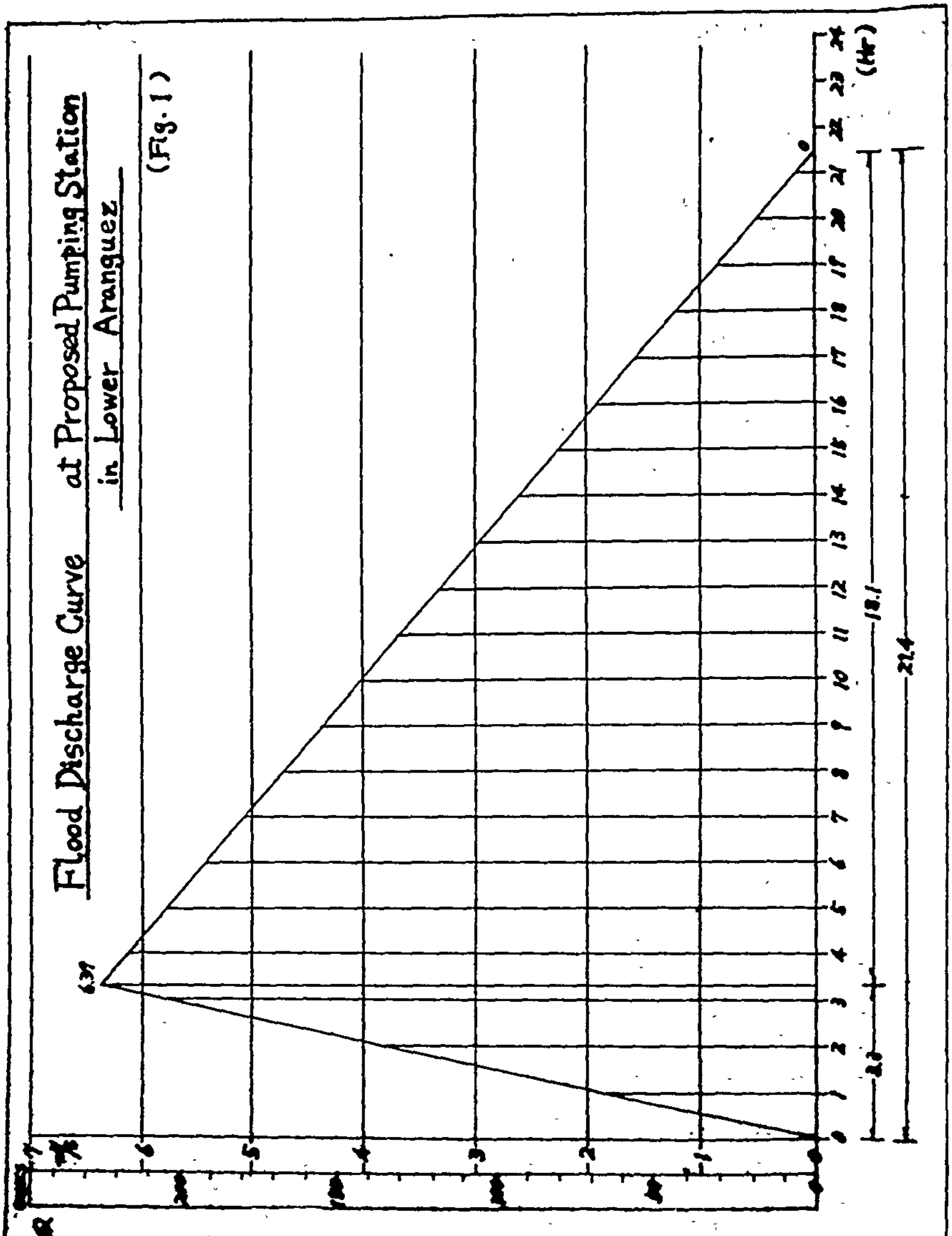
In the area drained by pumps like this lower Aranguez, a canal-bed can hardly be sloped, so the water can only flow with surface slope brought about by pump operation.

Consequently while the pumps are not operated the canal got stagnated and water plants are apt to grow thick. On the contrary the canal may be eroded with the flow during a flood. Therefore maintenance work must be carried out at all times.

Along the inside of the embankment to the Caroni River a drainage canal must be excavated concurrently as a retarding basin. The excavated soil can be used for the embankment.

The present drainage canal, Mullins canal can also be used as the main one.

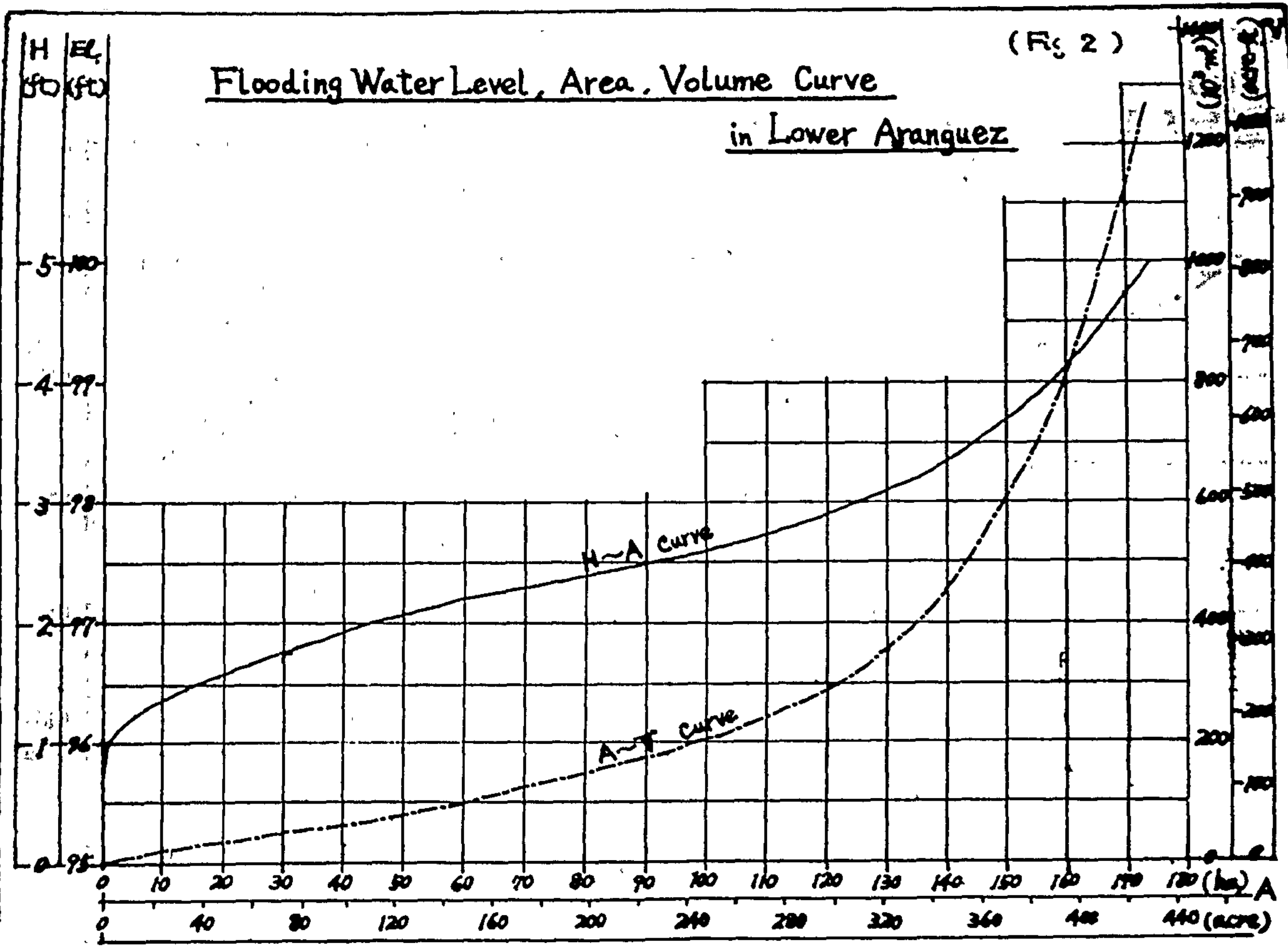
Junichi Kitamura



Flood Discharge Curve at Proposed Pumping Station
in Lower Aranguez
(Fig. 1)
6.39
(Hr)
18.1
21.4

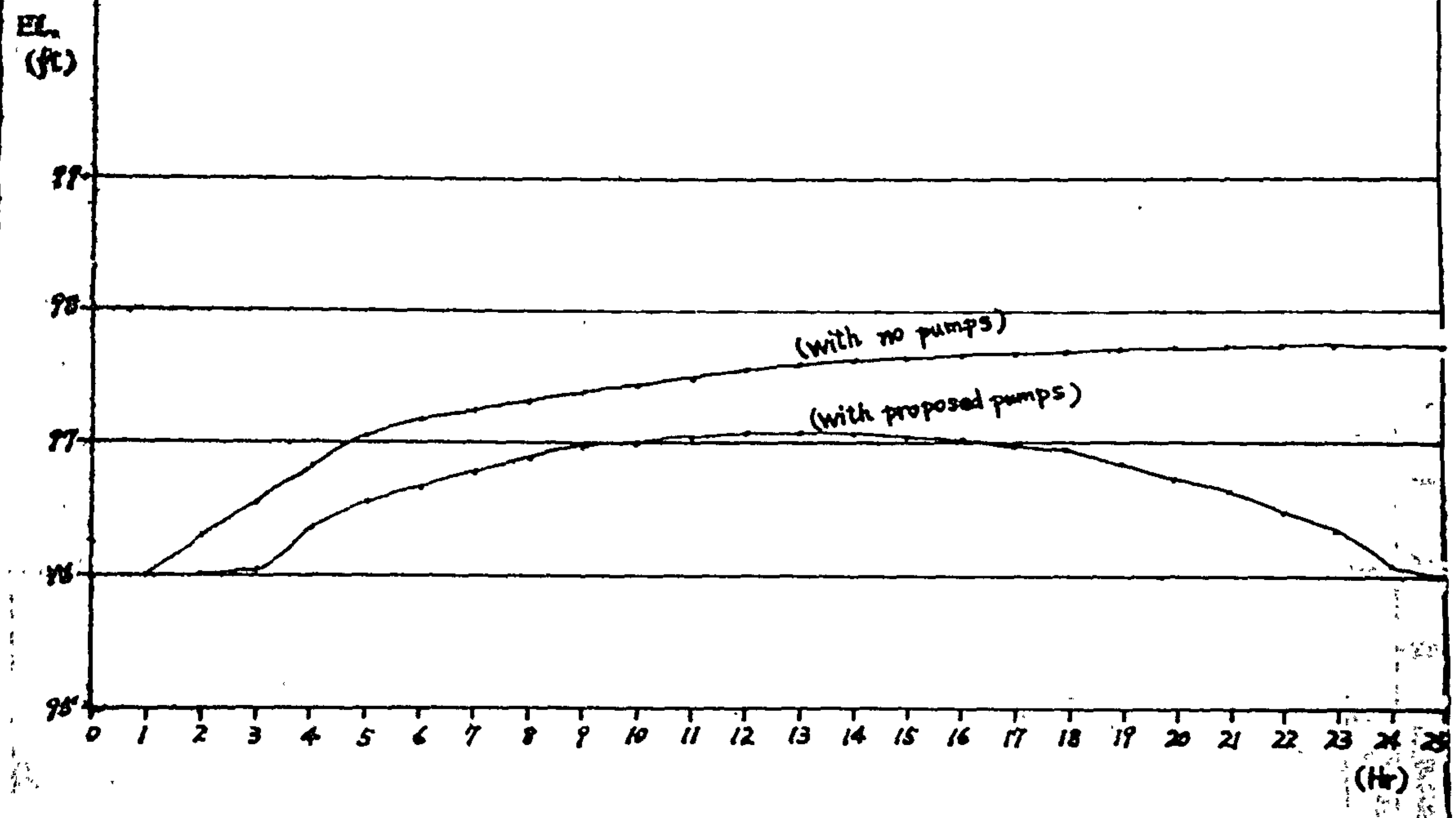
(Fig. 2)

Flooding Water Level, Area, Volume Curve in Lower Aranguez



(Fig. 3)

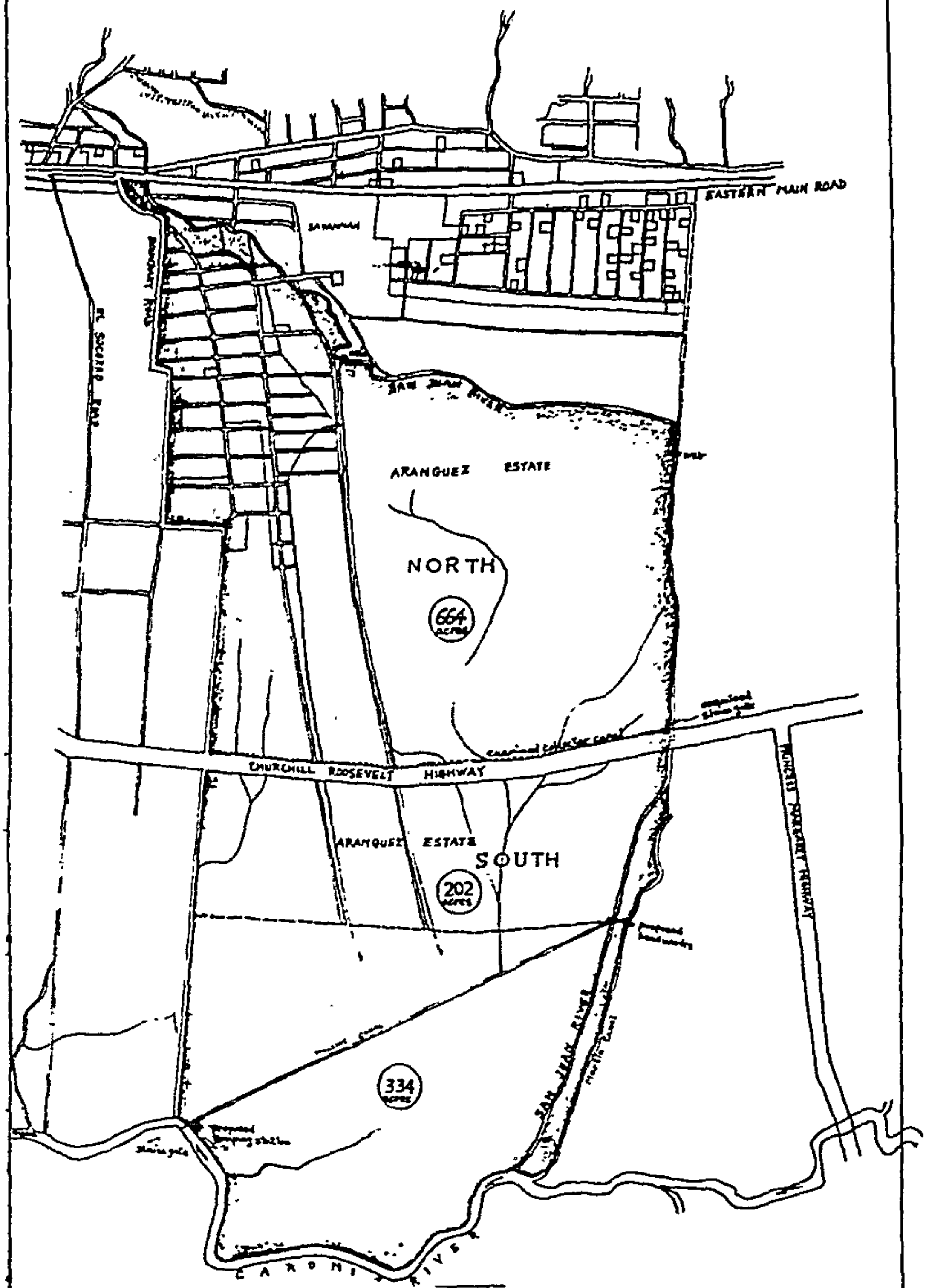
Internal Flooding Water Level
in Lower Aranguez



(Fig.4)

DRAINAGE AREA IN ARANGUEZ

12 Chain = 1 in.



ARANGUEZ (VII) October, 1967

DESIGN OF HEADWORKS

1. INTRODUCTION

In the case of building a weir, a movable part must usually be arranged to repress my rise of water level which may have a bad influence on the upper reaches of a river.

Generally, the movable part which can control the designed flood discharge on the designed high water level must be made in the low-water channel.

In this case, the required water level to take the water into the fields is only about 5 feet high from the lowest river bed and there is no necessity for raising the water above the low-water channel.

The present river section, however, is very narrow and can carry only 2,850 cusecs. Therefore, widening the river section and repressing the rise of water level by weir as low as possible are necessary. The discharge of 4,300 cusecs is adopted as the designed flood discharge. (cf. Report Aranguez III)

The dimensions of the lower reach immediately next to the headworks are as follows:

Designed flood discharge	Qmax	=	4,300 cusecs (= 121.8 m^3/s)
Area of river-section	A_2	=	74.4 m^2
Mean velocity	V_2	=	$Qmax/A_2 = 121.8/74.4 = 1.65$ m/s
Designed highwater level	109.7 ft (cf. Aranguez III)		
Lowest riverbed level	97.65 ft		

2. DETERMINATION OF THE WATER LEVEL OF THE UPPER REACH IMMEDIATELY NEXT TO THE HEADWORKS

The following two types are now considered:

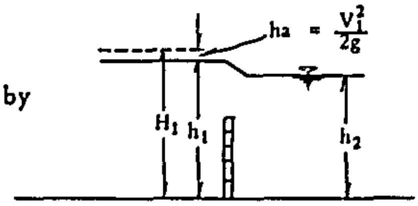
(A) Flash board weir
(B) Sluice gate weir

1) Flash board weir

Crest level .... El 102.55 ft (cf. Aranguez III)

Assumptions
- Gate sill level .... El 98.0 ft
- High water level of the upper reach immediately next to the weir .... El 110.0 ft
- Width and number 7 ft wide .... 6 gates

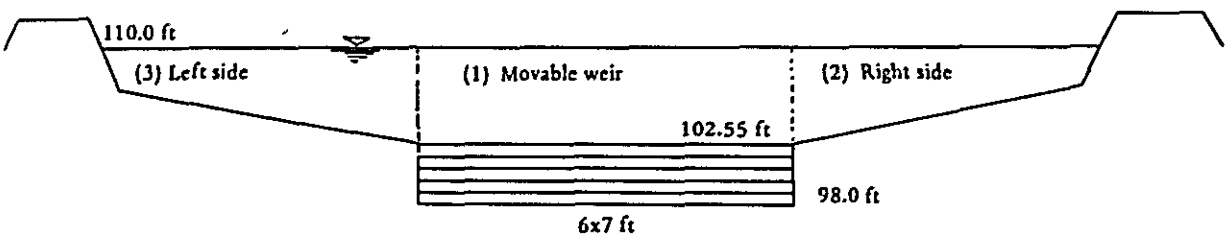
(1) Discharge from the movable weir



Q_M	=	$0.95\, b_o h_2 \sqrt{2g\,(H_1 - h_2)}$
bo	=	b - knh_1
k	:	coefficient of construction by pier which is 0.02 now.
n	:	number of construction
g	:	Acceleration of gravity
h_1	=	110.0 - 98.0 = 12.0 ft = 3.60
h_2	=	109.7 - 98.0 = 11.7 ft = 3.57 m

Assuming that ha $= \frac{V_1^2}{2g} =$ 0.15 m (velocity head of approach)

	H_1	=	$h_1 + ha$ = 3.66 + 0.15 = 3.81 m
	b	=	6 x 7 ft = 42 ft = 12.80 m
	be	=	12.80 = (0.02 x 12 x 3.66) = 11.92 m
∴	Q_M	=	$0.95 \times 11.92 \times 3.57 \sqrt{2 \times 9.8\,(3.81 - 3.57)}$ = 87.69 m^3/s
	A_M	=	bh_1 = 12.80 x 3.66 = 46.85 m^2



(2) Discharge from the right side

A_R	=	14.31 m^2
P_R	=	37.1 ft = 11.31 m I = 1/944 $I^{1/2}$ = 0.0325
P_R	=	1.265 m $R_R^{2/3}$ = 1.170 1/n = 40 (lined with stone and mortar)
V_R	=	$\frac{1}{n} R^{2/3} I^{1/2}$ = 40 x 1.170 x 0.0325 = 1.45 m/s
Q_R	=	AV = 14.31 x 1.45 = 20.75 m^3/s

(3) Discharge from the left side

A_L	=	10.16 m^2
P_L	=	29.2 ft = 8.90 m
R_L	=	1.142 m $R_L^{2/3}$ = 1.093
V_L	=	40 x 1.093 x 0.0325 = 1.42 m/s
Q_L	=	10.16 x 1.42 = 14.43 m^3/s

ΣQ = 122.87 m^3/s ≒ 121.8 m^3/s (Designed flood discharge)

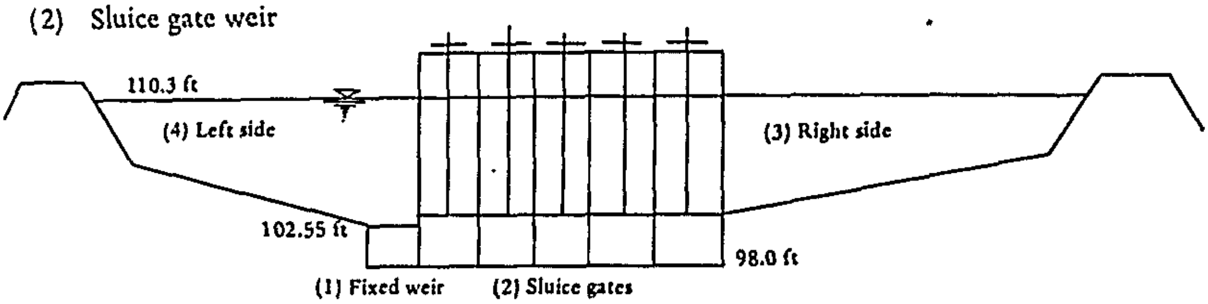
ΣA = 71.32 m^3 =

∴ mean velocity V_1 = 1.72 m/s

$$h_a = \frac{V_1}{2g} = \frac{(1.72)^2}{2 \times 9.8} = 0.15$$

Therefore, the water level at the upper reach next to the headworks is found to be 110.0 ft.

(2) Sluice gate weir



Crest level of fixed weir = 102.55 ft

Assumptions
- (gate sill level = 98.0 ft
- (high water level of the upper reach immediately next to the weir = 110.3 ft
- (width and number of gates = 7 ft wide; 5 gates
- (width of fixed weir = 7 ft

(1) Discharge from the fixed weir

$$Q_{1F} = 0.95\, b_o h_2 \sqrt{2g(H_1 - h_2)}$$
$$b_o = b - knh$$
$$h_1 = 110.3 - 102.55 = 7.75 \text{ ft} = 2.36 \text{ m}$$
$$h_2 = 109.7 - 102.55 = 7.15 \text{ ft} = 2.18 \text{ m}$$

Assuming that $h_a = \frac{V_1^2}{2g} = 0.14$ m (velocity head of approach)

$$H_1 = h_1 + h_a = 2.36 + 0.14 = 2.50 \text{ m}$$
$$b_o = 2.13 - 0.04 \times 1 \times 2.36 = 2.04 \text{ m} \quad (k = 0.04,\ n = 1)$$
$$\frac{h_2}{H_1} = \frac{2.18}{2.50} > 2/3 \quad \therefore \text{ This is in the condition of sub-merged weir}$$

$$\therefore \quad Q_F = 0.95 \times 2.04 \times 2.18 \sqrt{2 \times 9.8\,(3.50 - 2.18)} = 10.58 \text{ m}^3/\text{s}$$
$$A_F = bh_1 = 2.13 \times 2.36 = 5.03 \text{m}^2$$

(2) Discharge from the sluice gates

$$h_1 = 110.3 - 98.0 = 12.3 \text{ ft} = 3.75 \text{ m}$$
$$h_2 = 109.7 - 98.0 = 11.7 \text{ ft} = 3.57 \text{ m}$$

Assuming that $ha = \frac{V_1^2}{2g} = 0.14$ m (velocity head of approach)

$$H_1 = hi + ha = 3.75 + 0.14 = 3.89 \text{ m}$$
$$b = 5 \times 7 \text{ ft} = 35 \text{ ft} = 10.67 \text{ m}$$
$$bo = 10.67 - 0.04 \times 10 \times 3.75 = 9.17 \text{ m} \quad (k = 0.04, n = 10)$$
$$\therefore \quad Q_M = 0.95 \times 9.17 \times 3.57 \sqrt{2 \times 9.8\,(3.89 - 3.57)} = 77.87 \text{ m}^3/\text{s}$$
$$AM = 10.67 \times 3.75 = 40.01 \text{ m}^2$$

(3) Discharge from the right side

$$A_R = 14.77 \text{ m}^2$$
$$P_R = 44.6 \text{ ft} = 13.59 \text{ m} \quad I = 1/944 \quad I^{1/2} = 0.0325$$
$$P_R = 1.087 \text{ m} \qquad R_R^{2/3} = 1.057 \quad 1/n = 40 \text{ (lined with stone and mortar)}$$
$$V_R = 40 \times 1.057 \times 0.0325 = 1.37 \text{ m/s}$$
$$Q_R = 14.77 \times 1.37 = 20.23 \text{ m}^3/\text{s}$$

(4) Discharge from the left side

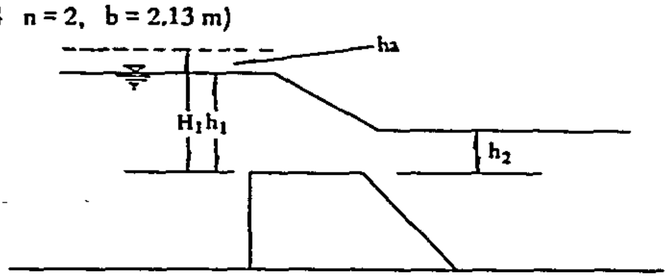
$$A_L = 10.51 \text{ m}^2$$
$$P_L = 34.4 \text{ ft} = 10.49 \text{ m}$$
$$R_L = 1.002\text{m} \quad R_L^{2/3} = 1.001$$
$$V_L = 40 \times 1.001 \times 0.0325 = 1.30 \text{ m/s}$$
$$Q_L = 10.51 \times 1.30 = 13.66 \text{ m}^3/\text{s}$$
$$Q = 122.34 \text{ m}^3/\text{s} \risingdotseq 121.8 \text{ m}^3/\text{s (Designed flood discharge)}$$
$$A = 70.32 \text{ m}^2$$
$$\therefore \quad \text{mean velocity } V_1 = 1.73 \text{ m/s}$$
$$ha = \frac{V_1^2}{2g} = \frac{(1.73)^2}{2 \times 9.8} = 0.15 \text{ m} \risingdotseq 0.14 \text{ m}$$

Therefore, the water level of the upper reach immediately next to the headworks will be 110.3 ft.

3. DETERMINATION OF THE HEIGHT OF SLUICE GATE

When the sluice gates are shut the fixed weir will function as a spillway. The relationship between the overflow depth on the fixed weir and the discharge is shown as follows.

$$Q = cbo\,H_1^{3/2}$$
$$bo = b\text{-}knh_1 \quad (k = 0.04 \quad n = 2, \quad b = 2.13 \text{ m})$$
$$H_1 = h_1 + \frac{V_1^2}{2g}$$



h_1	assumed ha	bo	H_1	$H_1^{3/2}$	Q	$A(=b_1h_1)$	V	ha
m	m	m	m	-	m^3/s	m^2	m/s	m
0.10	0.08	2.12	0.18	0.076	0.27	0.2130	1.27	0.082
0.15	0.09	2.12	0.24	0.117	0.42	0.3195	1.31	0.088
0.20	0.09	2.11	0.29	0.156	0.56	0.4260	1.31	0.088
0.25	0.10	2.11	0.35	0.207	0.74	0.5325	1.39	0.099
0.30	0.11	2.11	0.41	0.263	0.94	0.6390	1.47	0.110
0.40	0.12	2.10	0.52	0.375	1.34	0.8520	1.57	0.126

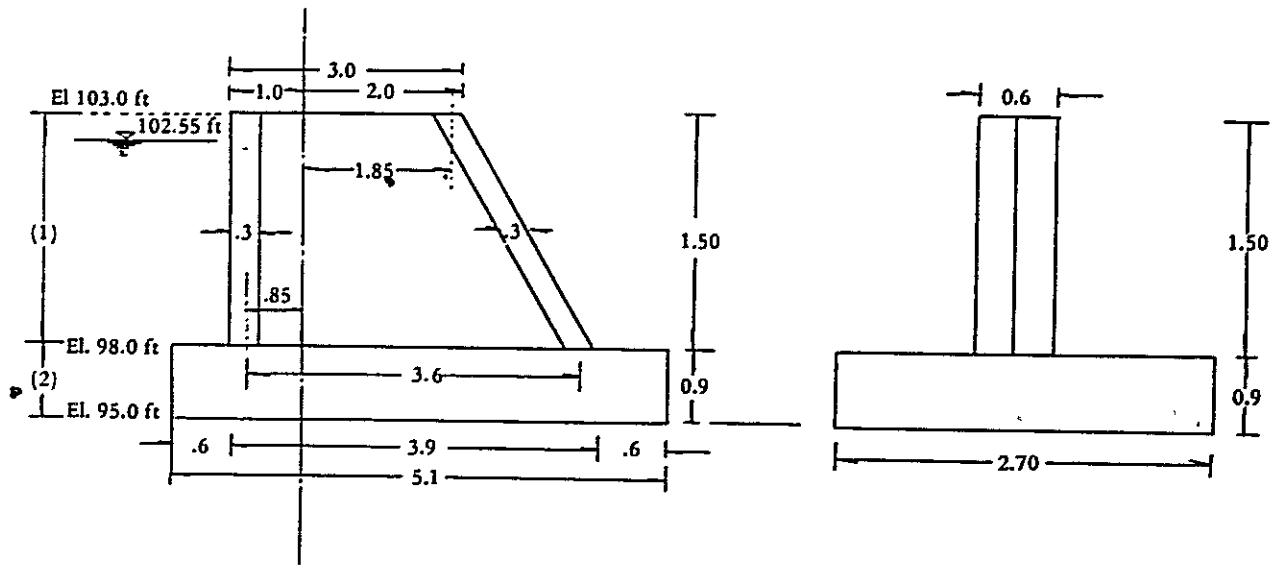
If the gates are kept close, until the discharge reaches 0.5 m^3/sec (= 17.7 cusecs) H_1, which is the upper water depth from the crest and velocity head of approach will be 0.27 m (0.9 ft).

When this H_1 is adopted as clearance of the gates, the height of them will be

$$(102.55 + 0.9) - 98.0 = 5.45 \text{ ft} = 1.66 \text{ m}$$

4. STABILITY COMPUTATION ANALYSIS OF PIERS

1) Flash board weir (m)



No.	W (ton)	x (m)	W x (t-m)	y (m)	W y(t-m)
1	½(3.0 + 3.9 - 0.3)1.5 x 0.6 x 2.4 = 6.84	0.725	4.96	1.62	11.08
2	0.9 x 5.1 x 1.7 x 2.4 = 29.74	0.95	28.25	0.45	13.38
Σ	36.58		33.21		24.46
Mean		0.91		0.74	

Conditions for design

Bulk density of concrete	W_1	=	$2.4\ t/m^3$
Coefficient of uplift	u	=	0.4
Unit weight of sedimental sand and silt	W_s	=	$1.0\ t/m^3$

Because, density on land is $1.6\ t/m^3$ and assuming that air space ratio is 0.36, unit weight in water will be $1.6 - (1.0 - 0.36) \times 1.0 = 1.0\ t/m^3$

Case

(1) Taking off the flash board inf lood

(a) Uplift acting upward against the base

No. 1 Block:

$h_1 = 110.0 - 98.0 = 12.0\ ft = 3.66\ m$

$H_1 = 3.66 + 0.15 = 3.81\ m \quad (ha = 0.15\ m)$

$h_2 = 109.7 - 98.0 = 11.7\ ft = 3.57\ m$

$U_1 = ½(H_1 + h_2)\ u.W_W.A \quad W_W$: Density of water

$= ½(3.81 + 3.57) \times 0.4 \times 1.0 \times (3.9 - 0.3) \times 0.6 = 3.19\ t$

Position of center of gravity: $X_{u1} = \frac{3.6}{3} \cdot \frac{3.81 + 2 \times 3.57}{3.81 + 3.57} - 0.85 = 0.93\ m$

from y axis.

No. 2 Block:

$h_1 = 110.0 - 95.0 = 15.0\ ft = 4.57\ m$

$H_1 = 4.57 + 0.15 = 4.72\ m$

$h_2 = 109.7 - 95.0 = 14.7\ ft = 4.48\ m$

$U_2 = ½(4.72 + 4.48) \times 0.4 \times 1.0 \times (5.1 \times 2.7) = 25.34\ ton$

$$X_{U_2} = \frac{5.1}{3} \cdot \frac{4.72 + 2 \times 4.48 - 1.60}{4.72 + 4.48} = 0.93\ m$$

(b) Hydrostatic pressure acting against the upstream side of the pier

No. 1 Block:

Head at the highest point $(h_U = 110.0 - 103.0 = 7.0\ ft = 2.13\ m$

$H_U = 2.13 + 0.15 = 2.28\ m$

Head at the lowest point $h_L = 110.0 - 98.0 = 12.0\ ft = 3.66\ m$

$H_L = 3.66 + 0.15 = 3.81\ m$

$P_{S1} = ½(H_U + H_L).\ W_W.\ (H_U - H_L).\ B \quad$ B: Breadth of the pier

$= ½(2.28 + 3.81) \times 1.0 \times 1.53 \times 0.6 = 2.80\ t$

Position of center of gravity from the base of No. 1 block

$$Y_{PS} = \frac{1.53}{3} \cdot \frac{2 \times 2.28 \times 3.81}{2.28 + 3.81} = 0.70\ m$$

(c) Hydrostatic pressure acting against the downstream side of the pier

No. 1 Block: Head at the highest h_U = 109.7 - 103.0 = 6.7 ft = 2.04 m

Head at the lowest point h_1 = 109.7 - 98.0 = 11.7 = 3.57 m

$$P_{S_1} = ½(2.04 + 3.57) \times 1.0 \times 1.53 \times 0.6 = 2.58\ t$$

$$Y_{P_1} = \frac{1.53.}{3}\ \frac{2 \times 2.04 + 3.57}{2.04 + 3.57} = 0.51 \times \frac{7.65}{5.61} = 0.70\ m$$

(d) Weight of water acting on the up stream side of the pier.

No. 1 Block:

$$W_1 = H_U W_w.A$$
$$= 2.28 \times 1.0 \times 0.85 \times 0.6) = 1.16\ t$$
$$X_{W_1} = -0.43\ m_m$$

No. 2 Block:

$$W_2 = W_1 + H_L.\ W_w.\ A$$
$$= 1.16 + 3.81 \times 1.0 \times (1.6 \times 2.7 - 0.85 \times 0.6) = 1.16 + 14.52 = 15.68t$$
$$X_{w_2} = \frac{1.16 \times (-0.43) + 14.52 \times (-0.80)}{15.68} = -0.77\ m$$

(e) Weight of water acting on the down stream side of the pier.

No. 1 Block:

$$W'_1 = \frac{(h_o + h_1)}{2}.W_w.\ A_1 + h_v.\ W_w.\ A_2$$
$$= \frac{2.04 + 3.57}{2} \times 1.0 \times (0.9 \times 0.6) + 2.04 \times 1.0 \times (1.85 \times 0.6) = 3.78t$$
$$XW'_1 = \frac{½(2.04 \times \frac{1.85 + 2.04}{2} + 3.57 \times 0.9)}{2.04} = 1.54\ m$$

No. 2 Block:

$$W'_2 = W'_1 + h_L.W_w.A = 3.78 + 3.57 \times 1.0 \times (3.5 \times 2.7 - 2.75 \times 0.6) =$$
$$3.78 + 27.85 = 31.63\ t$$
$$X'_{w_2} = \frac{3.78 \times 1.54 \times 27.85 \times 1.75}{31.63} = 1.72\ m$$

Case

(2) Setting up the flash board in low water

(a) Sedimental earth pressure acting against the upstream side of the pier.

No. 1 Block:

$$P_e = ½\ Wekah^2.B$$
$$= ½ \times 1.0 \times 0.33 \times 1.39^2 \times 0.60 = 0.19\ t$$

(Assuming that the internal friction angle of soil is 30°, the earth pressure coefficient, will be

$$(ka = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - ½}{1 + ½} = 1/3 = 0.33$$

$$yP_e = 1/3 \times 1.39 = 0.46m$$

(b) Hydrostatic pressure acting against the board and the pier Interval of each center of the pier B = 9 ft = 2.73 m

No. 1 Block $P_{s_1} = ½ \times 1.0 \times 1.39^2 \times 2.73 = 2.65$ t

$Y_{PA} = Y_{Pe} = 0.46$ m

(c) Uplift acting upward against the base

No. 1 Block: $U_1 = ½ h.u W_w.A$

$= ½ \times 1.39 \times 0.4 \times 1.0 \times (3.6 \times 0.6) = 0.60$ t

$XU_1 = \frac{36}{3} - 0.85 = 0.35$ m

No. 2 Block: $U_2 = ½ \times 2.29 \times 0.4 \times 1.0 \times (5.1 \times 2.7) = 6.31$ t

$X_{U_2} = \frac{5 - 1}{3} = 0.1$ m

(d) Water weight acting on the upstream side of the pier

No. 1 Block: $W_1 = hW_w.A = 1.39 \times 1.0 \times (1.6 \times 2.7 - 85 \times 0.6) = 5.30$ t

$$X_{W_1} = \frac{(0.75 \times 2.7) \times (-1.23) + (2.1 \times 0.85) \times (-0.43)}{(0.75 \times 2.7) + (2.1 \times 0.85)} = -0.85 \text{ m}$$

When all the forces calculated above are as shown on figures 1 & 2 it is found that the resultants of these forces are within the middle third of the base of each section.

2) Sluice Gate Weir

No.	W	X	WX	y	W.y
1-1	1.50 x 0.3 x 2.73 x 2.4 = 2.95	0.35	1.03	6.69	19.74
1-2	$\frac{0.9+0.6}{2}$ x 0.15x1.5x2.4 = 0.41	0.35	0.14	6.47	2.65
1-3	1.5x1.53x0.6x2.4 = 3.30	0.35	1.16	5.63	18.58
Total	6.60		2.33		40.97
Mean		0.35		6.15	
2	$\frac{3.3+4.5}{2}$ x3.96x0.6x2.4=22.24	0.80	17.79	2.79	62.05
3	0.9 x 6.0 x 2.73 x 2.4 = 35.38	1.10	38.92	0.45	15.92
1	6.66	0.35	2.33	6.15	40.97
1+2	6.66+22.24 = 28.90	$\frac{20.12}{28.90} = 0.70$	2.33+17.79=20.12	$\frac{103.02}{28.90} = 3.57$	40.97+62.05=103.02
1+2+3	28.90+35.38 = 64.28	$\frac{59.04}{64.28} = 0.92$	20.12+38.92=59.04	$\frac{118.94}{64.28} = 1.85$	103.02+15.92=118.94

Case (1)

Winding up the gate in flood

(a) Uplift acting upward against the base

No. 2 Block:

$$h_1 = 110.3 - 98.0 = 12.3\ \text{ft} = 3.75\ \text{m}$$
$$H_1 = 3.75 + 0.14 = 3.89\ \text{m}$$
$$h_2 = 109.7 - 98.0 = 11.7\ \text{ft} = 3.57\ \text{m}$$
$$U_2 = \tfrac{1}{2}(3.89 + 3.57) \times 0.4 \times 1.0 \times (4.5 \times 0.6) = 4.03\ \text{t}$$
$$X_{U_2} = \frac{4.5}{3}\ \frac{3.89 + 2 \times 3.57}{3.89 + 3.57} - 1.15 = 1.07\ \text{m}$$

No. 3 Block:

$$h_1 = 110.3 - 95.0 = 15.3\ \text{ft} = 4.66\ \text{m}$$
$$H_1 = 4.66 + 0.14 = 4.80\ \text{m}$$
$$h_2 = 109.7 - 95.0 = 14.7\ \text{ft} = 4.48\ \text{m}$$
$$U_3 - \tfrac{1}{2}(4.80 + 4.48) \times 0.4 \times 1.0 \times (6.0 \times 2.73) = 30.40\ \text{t}$$
$$X_{U_3} = \frac{6.0}{3}\ \frac{4.80 + 2 \times 4.48}{4.80 + 4.48} - 1.9 = 1.07\ \text{m}$$

(b) Hydrostatic pressure acting against the upstream side of the pier.

No. 2 Block:

$$Ps_2 = \tfrac{1}{2} \times 1.0 \times 3.89^2 \times 0.6 = 4.54\ \text{t}$$
$$Y_{PS} = \frac{3.89}{3} = 1.30\ \text{m}$$

(c) Hydrostatic pressure acting against the down stream side of the pier.

No. 2 Block:

$$P'_{S_2} = \tfrac{1}{2} \times 1.0 \times 3.57^2 \times 0.6 = 3.82\ \text{t}$$
$$Y'_{ps} = \frac{3.57}{3} = 1.19\ \text{m}$$

(d) Water weight acting on the upstream side of the pier

No. 3 Block:

$$W_3 = 3.89 \times 1.0 \times (1.9 \times 2.73 - 1.15 \times 0.6) = 17.51\ \text{t}$$
$$X_{W_3} = \frac{(0.75 \times 2.73) \times (-1.53) + (1.15 \times 2.13) \times (-0.58)}{(0.75 \times 2.73) + (1.15 \times 2.13)} = -1.01\text{m}$$

(e) Water weight acting on the down stream side of the pier

No. 2 Block:

loaded breadth $b = 1.2 \times \frac{3.57}{3.96} = 1.08\ \text{m}$

$$\therefore\ W'_2 = \tfrac{1}{2} \times 3.57 \times 1.0 \times (1.08 \times 0.6) = 1.16\text{t}$$
$$X'_{W_2} = 2/3 \times 1.08 + (2.3 + 0.12) = 3.14\ \text{m}$$

No. 3 Block:

$$W'_3 = W'_2 + 3.57 \times 1.0 \times (4.1 \times 2.7 - 3.5 \times 0.6) = 1.16 + 32.02 = 33.18\text{t}$$
$$X_{W_3} = \frac{(1.16 \times 3.14) + (32.02 \times 2.05)}{33.18} = 2.09\ \text{m}$$

(f) Weight of gate and attachment

gate	$2.73 \times 1.66 = 4.53\ m^2$	$4.53\ m^2 \times 40\ gk/m^2$	=	0.2 t)	
Spindle......			=	0.1 t)	$G_g = 0.65$ t
Winch				0.2 t)	
Operators		2 x 75 kg	=	0.15t)	

Case (2)

Closing the gate in low water

(a) Sedimental earth pressure acting against the upstream side of the pier

No. 2 Block: $P_{e_2} = 0.19$ t (c.f flashboard weir)

$Y_{Pe} = 0.46$ m

(b) Hydrostatic pressure acting against the gate and the pier

No. 2 Block: $P_{S_2} = 2.65$ t (c.f flashboard weir)

$Y_{Pe} = 0.46$ m

(c) Uplift acting upward against the base

No. 2 Block: $U_2 = ½ \times 1.39 \times 0.4 \times 1.0 \times (4.5 \times 0.6) = 0.75$ t

$X_{U_2} = \frac{4.5}{3} - 1.15 = 0.35$ m

No. 3 Block: $U_3 = ½ \times 2.29 \times 0.4 \times 1.0 \times (6.0 \times 2.7) = 7.42$ t

$X_{U_3} = \frac{6.0}{2} - 1.9 = 0.1$ m

(d) Water weight acting on the upstream side of the pier

No. 3 Block: $W_3 = 1.39 \times 1.0 \times (1.9 \times 2.7 - 1.15 \times 0.6) = 6.17$ t

$X_{W_3} = \frac{(0.75 \times 2.73) \times (-1.53) + (2.1 \times 1.15)}{(0.75 \times 2.73) + (2.1 \times 1.15)} \times (-0.58) = -1.01$ m

When all the forces calculated above are as shown on figures 3 & 4, it is found that resultants of these forces are within the middle third of the base of each section.

(3) Stability against sliding

Case (a) Taking off the flash board during design flood sum of vertical forces

$\Sigma V = G_2 - U_2 + W_2 + W'_2$

$= 36.58 - 25.34 + 15.68 + 31.63 = 58.55$ t

Sum of horizontal forces

$$\Sigma H = P_{S_1} - P'_{S_1}$$
$$= 2.80 - 2.58 = 0.22 \text{ t}$$

Coefficient of sliding

$$S = \frac{\Sigma H}{\Sigma V} = \frac{0.22}{58.55} = 0.0038$$

Coefficient of friction $f = 0.7$

$$\frac{f}{S} = \frac{0.7}{0.0038} = 184 > 1.5$$

Therefore, this case is safe from sliding

Case (b) Setting up the flash board in low water

$$\Sigma V = G_2 - U_2 + W_2$$
$$= 36.58 - 6.31 + 5.30 = 35.57 \text{ t}$$
$$\Sigma H = P_{e_1} + P_{S_1}$$
$$= 0.19 + 2.65 = 2.84 \text{ t}$$
$$S = \frac{\Sigma H}{\Sigma V} = \frac{2.84}{35.57} = 0.08$$
$$\frac{f}{S} = \frac{0.7}{0.08} = 8.75 > 1.5$$

Therefore this case is also safe from sliding.

(4) Bearing capacity of foundation

(a) In case of winding up the sluice gate in flood

No. 3 Block: $\Sigma W: = 85.22$ t

eccentric distance $e = 0.10$ m (c.f. figure 3)

compressive stress

$$\max = \frac{\Sigma W}{B.L}(1 + \frac{6e}{L})$$
$$\frac{85.22}{2.73x6.0} \times \frac{(1 + 6 \times 0.1)}{6.0} = 5.72 \text{ t/m}^2$$

(b) In case of closing the sluice gate in low water

No. 3 Block: $\Sigma W = 63.68$ t

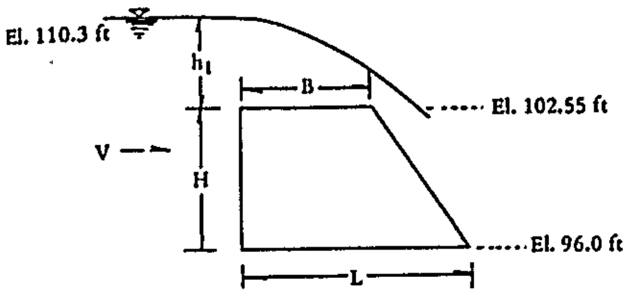
$e = 0.25$ m (cf. figure 4)

$$\max = \frac{63.68}{2.73 \times 6.0} \times \frac{(1 + 6 \times 0.25)}{6.0} = 4.86 \text{ t/m}^2$$

According to the report, Aranguez (1V), allowable compressive stresses were 5.6, 11.2, 7.1, 7.8 t/m^2 each at the elevation of 95 ft as the result of 4 soundings capacity for foundation even without piling. But piling is recommended to increase the factor of Safety.

(5) Design of the fixed weir in the latter case

1) Determination of the fundamental section



Bligh's formula : $L = \frac{H + h_1}{\sqrt{\gamma}}$ $B = \frac{h_1}{\sqrt{\gamma}}$

H: Height of weir

h_1: Maximum water depth above dam crest

γ: specific gravity of concrete which is 2.3

$$H = 102.55 - 96.0 = 6.55 \text{ ft} = 2.00 \text{ m}$$

$$h_1 = 110.3 - 102.55 = 7.75 = 2.36 \text{ m}$$

$$L = \frac{2.00 + 2.36}{\sqrt{2.3}} = 2.87 \text{ m}$$

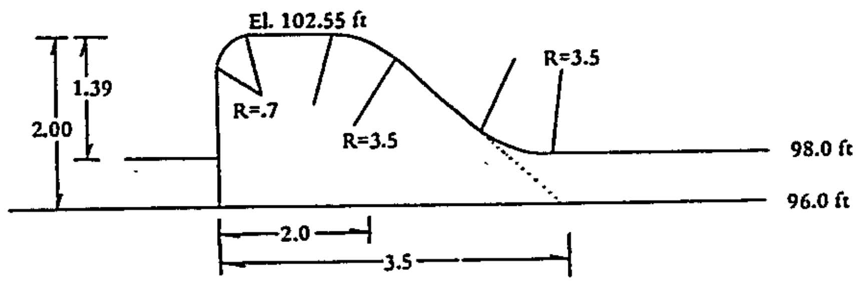
$$B = \frac{2.36}{\sqrt{2.3}} = 1.56 \text{ m}$$

Taking clearance of 20% $L = 2.87 \times 1.2 = 3.44 \fallingdotseq 3.5$ m

$B = 1.56 \times 1.2 = 1.87 \fallingdotseq 2.0$ m

2) Amendment of the section

The section is decided as the following figure (m)



3) Design of apron floor

a) Path of percolation

Bligh's formula $L \geqq CH$

$L = 18 \times 1.39 = 25.02$ m

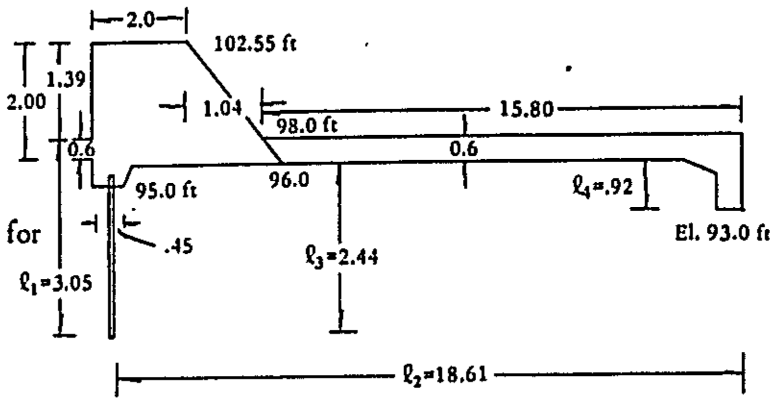
maximum difference of water level between upstream and down stream can be regarded height of weir. Therefore,

H: height of weir which is 1.39 m

C: coefficient which is 18 in case of very fine sand and silt.

Assuming that iron sheet piles are driven into soil down to an elevation of 88 ft, horizontal distance, l_2 necessary will be as follows. (m)

$l_2 = L - (l_1 + l_3 + l_4)$
$= 25.02 - (3.05 + 2.44 + 0.92)$
$= 18.61$ m
(length of apron floor is 15.80 m)



b) Length of apron necessary for prevention of piping

Bligh's formula $l = 1.2C\sqrt{\frac{Ha}{4}} = 0.6\,c\,\sqrt{Ha}$
$= 0.6 \times 18 \times \sqrt{1.39} = 12.74 \text{ m} < la = 15.80 \text{ m}$

l: needful length of apron
C: Bligh's coefficient
Ha: height from apron floor to crest of weir.

Therefore la which is 15.80 m is adopted as needful length of apron.

c) Thickness of apron floor in down stream

$$t \geqq \frac{4}{3}\frac{Ha - h}{\gamma - 1}$$

t: Thickness of apron floor
4/3: Safety factor
Ha: maximum difference of water level between the upstream and the down stream.
h: loss head from the upstream side of weir to an optional point which is as follows.

$$h: = \frac{(Ha)\ L'}{L}$$

L: = whole path of percolation which is 25.02 m
L': path of percolation to an optional point
γ: specific gravity of concrete which is 2.3

Thickness at the beginning of the apron in down stream can be calculated as follows:

$$L' = 3.05 + 2.44 + 3.50 = 8.99 \text{ m}$$

$$h = \frac{(1.39)}{25.02} \times 8.99 = 0.50 \text{ m}$$

$$\therefore\ t \geqq \frac{4}{3} \times \frac{1.39 - 0.50}{2.3 - 1} = 0.91 \text{ m} \quad (= 3 \text{ ft})$$

d) Weephole

As the cut-off wall to prevent piping is placed at the end of apron floor, the weep-holes are necessary to reduce the uplift acting against whole apron.

ϕ 1" x 1 m each

e) Apron floor in the upper reach

There is no formula to decide it. Therefore it is decided optionally as follows.

$l_s = 4.5$ (15 ft)
$t = 0.45$ m (1.5 ft)

Iron sheet piles must be driven for safety.

4) Design of riprap

Bligh's formula: $l = 3c\sqrt{\frac{Ha}{2}}\sqrt{\frac{q}{7}} = 0.66\,c\,\sqrt{Ha.\,q}$

l : Whole length of protecting work in lower stream, that is, length of apron and that of riprap.

c : *Bligh's coefficient which is now 18*

Ha : Difference between crest of weir and surface at the end of apron which is 1.39 m

q : Unit overflow discharge on fixed weir in designed flood which will be

$$q = \frac{Q}{b} = \frac{10.58}{2.13} = 4.96\ m^3/s$$

as $Q = 10.58\ m^3/s$

$b = 2.13$ m

$l = 0.66 \times 18\sqrt{1.39 \times 4.96} = 31.20$ m

length of apron la = 15.80 m

Therefore, length of riprap $l_r = 31.20 - 15.80 = 15.40$ m

It is suggested that the bolster or basket made of iron wire coated plastics be used.

(6) Design of silt ejector

1. Calculation of discharge for silt ejecting on the San Juan River.

Now discharge on the San Juan River is being observed at two points right at the lower reach from the second headworks (in this connection the third headworks is now proposed).

Readings at the upper gauge at 10 a.m., taken at 10.00 a.m. each day from June until August (1967) are as follows:

Reading	0.6 ft	0.7	0.8	0.9	1.0	1.1	1.2	1.3	Total
Days	2days	16	37	16	8	1	3	9	92 days

Here is one discharge measured on the second headworks on 10th of May 1967.

$Q = 0.105\ m^3/s$ (= 3.71 cusecs) (cf. Aranguez (11)

at that time, reading of the upper gauge was 0.7 ft.

a) Therefore, coefficient of roughness can be calculated from this data and the measured cross section at the upper gauge.

Upper stream gauge: El. 102.87 ft (As datum is 102.17 ft $H = 0.7$)
Down stream gauge: El. 102.50 ft
Distance 200 ft. $\therefore\ I = \frac{0.37}{200} = \frac{1}{541} = 0.00185$

$I^{1/2} = 0.0430$

$\therefore$ Hence $1/n = 30$

$A = 3.55\ ft^2 = 0.330\ m^2$
$P = 9.0\ ft = 2.74\ m$
$R = A/P = 0.330/2.74 = 0.120\ m$
$R^{2/3} = 0.244$ Assuming that $1/n = 30$
$V = \frac{1}{n} R^{2/3} I^{1/2} = 30 \times 0.244 \times 0.0430 = 0.315\ m/s$
$\therefore\ Q = AV = 0.330 \times 0.315 = 0.104\ m^3/s$

b) Discharge in H = 0.8 ft

Upstream gauge: El 102.97 ft
Down stream gauge: −) El 102.70
0.27 ft

$\therefore\ I = \frac{0.27}{2.00} = 0.00135$
$I^{1/2} = 0.0367$
$A = 4.25\ ft^2 = 0.395\ m^2$
$P = 9.3\ ft = 2.83\ m$
$R = A/P = 0.395/283 = 0.1395\ m$
$R^{2/3} = 0.269$
$\frac{1}{n} = 30 \quad \therefore\ V = \frac{1}{n} R^{2/3}\ I^{1/2} = 30 \times 0.269 \times 0.00367 = 0.296\ m/s \quad Q = AV$
$= 0.395 \times 0.296 = 0.117\ m^3/s$

c) Discharge in H = 0.9 ft.

Upstream gauge: El 103.07 ft
Down stream gauge: −) El 102.80
0.27

$\therefore\ I = \frac{0.27}{200} = 0.00135$
$I^{1/2} = 0.0367$
$A = 5.17\ ft^2 = 0.480\ m^2$
$P = 9.6\ ft = 2.93\ m$
$R = A/P = 0.48/2.93 = 0.1638\ m \qquad R^{2/3} = 0.299$
$\frac{1}{n} = 30 \quad V = \frac{1}{n} R^{2/3} I^{1/2} = 30 \times 0.299 \times 0.0367 = 0.329\ m/s$
$Q = AV = 0.480 \times 0.329 = 0.158\ m^3/s$
$\fallingdotseq 0.16\ m^3/s.$

This discharge is planned as the minimum discharge for silt ejecting.

2. Computation of maximum ejectable soil-diameter.

Assuming that two gates on the right side in the movable five gates (six gates in case of the flash board weir) are used both for silt ejecting and for discharging design flood, total width of gates as silt-ejector will be 2 x 2.13 = 4.26 m

$$h_c = \sqrt[3]{\frac{Q^2}{gb^2}}$$

$$= \sqrt[3]{\frac{(0.16)^2}{9.8 \times (4.26)^2}} = 0.0524 \text{ m}$$

$$V_c = \frac{Q}{bhc} = \frac{0.16}{4.26 \times 0.0524} = 0.717 \text{ m/s}$$

$$\frac{V_c}{1.5} \geqq c\sqrt{d}$$

$$\frac{0.717}{1.5} \geqq 5.0\sqrt{d} \quad \therefore \quad d \leqq 0.0091 \text{ m } (= 0.36 \text{ inch})$$

Q: discharge on silt ejecting which is 0.16 m^3/s
g: acceleration due to gravity
b: total width of gates which is 4.26 m
h_c: critical depth
V_c: critical velocity
c: coefficient which is 4.5 - 5.5
d: maximum diameter of soil

This diameter may be used as the maximum ejectable soil diameter, because the river bed consists of fine sand and silt.

3. Determination of silt-ejector sill level.

At present, the lowest river bed level is 97.65 ft. Judging from the river slope and the form of cross section, there will be no change in river-bed level.

The silt-ejector will level is therefore accepted as 98 ft that is the same as the sill level of the other gates for discharging flood.

Design of movable parts for discharging flood

This movable part consists of the same five gates 7 ft wide containing two gates for ejecting silt (six gates in case of flash board weir).

This design must be considered in order to reduce the back-water effect and not to hinder river flow. However, there is a limitation to facilitate manual gate operation and so width of gates may be taken as 7 ft. This sill level determination is the same as that of the silt-ejector.

1) Length of apron floor necessary for movable parts.
(In case of sluice gate weir)

$$l_a = 1.8c\sqrt{\frac{Ha}{4}} = 0.9\sqrt{Ha}$$

$$Ha = 103.45 - 98.0 = 5.45 \text{ ft} = 1.66 \text{ m}$$

$$l_a = 0.9 \times 18 \times \sqrt{1.66} = 20.90 \text{ m}$$

2) Thickness of apron floor downstream

Path of percolation: $L = CH = 18 \times 1.66 = 29.90$ m

Thickness of apron right below the gate can be calculated as follows:

$$L' = 3.05 + 2.44 + 1.90 = 7.39 \text{ m}$$

$$h = \frac{(1.66)}{29.90} \times 7.39 = 0.41 \text{ m}$$

$$\therefore \quad t \geqq \frac{4}{3} \times \frac{1.66 - 0.41}{2.3 - 1} = 1.28 \text{ m}$$

Although stability computation analysis of piers was done for an apron thickness of 0.9 m, it found that a thickness of more than 1.28 m is necessary - (Please see calculation above).

3) Length of riprap

$$l = 4.5 \sqrt{\frac{Ha}{3}} \cdot \sqrt{\frac{q}{7}} = 1.0\,c \sqrt{Ha.\,q}$$

$$Ha = 1.66 \text{ m}$$

$$q = \frac{Q}{b} = \frac{72.95}{10.67} = 6.84 \text{ m}^3/\text{s}$$

$$\therefore \quad l = 1.0 \times 18 \sqrt{1.66 \times 6.84} = 62.62 \text{ m}$$

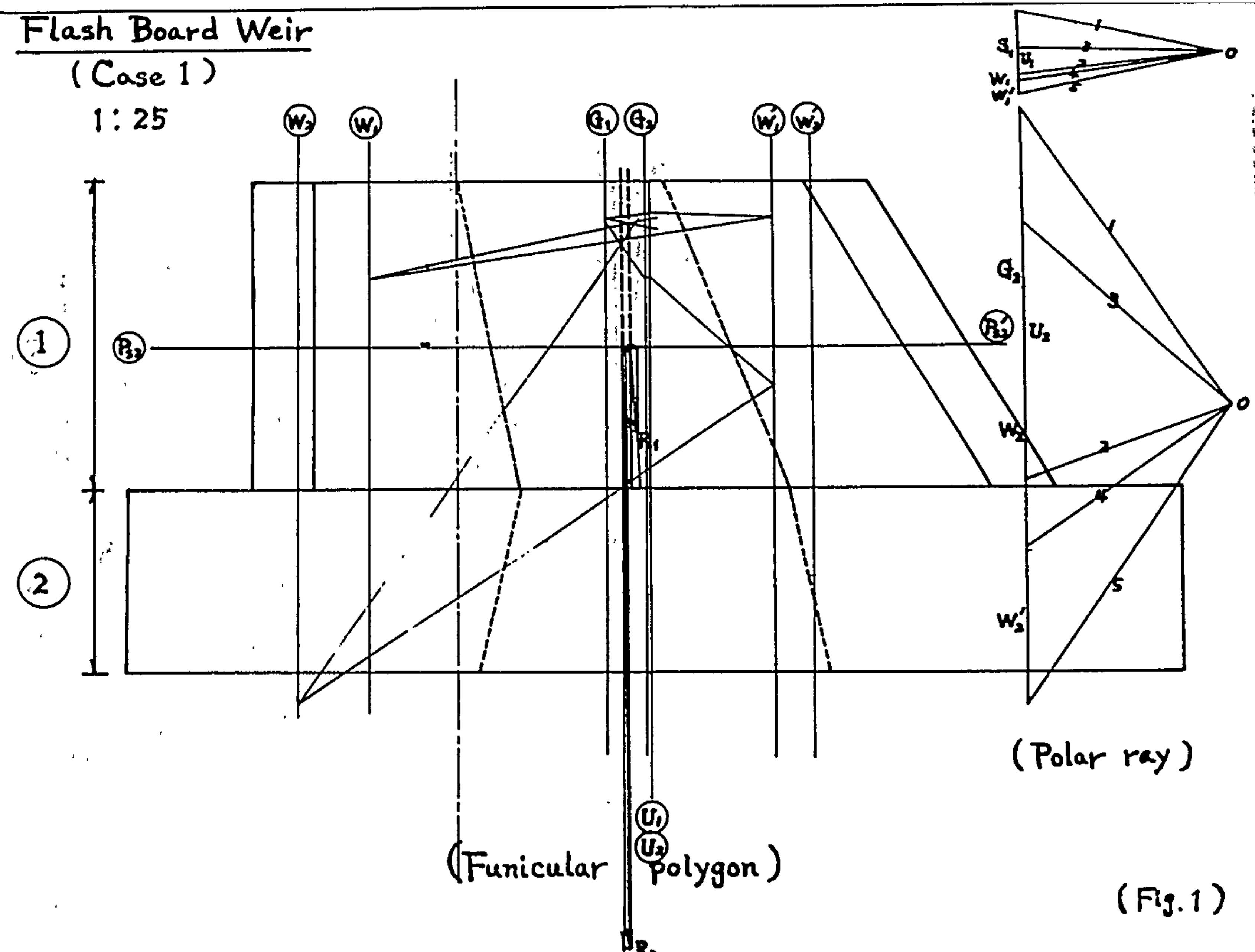
length of apron: $l_a = 20.90$ m

$\therefore$ length of riprap: $l_r = 62.62 - 20.92 = 41.70$ m

Design of intake and main canal will be carried out on next report.

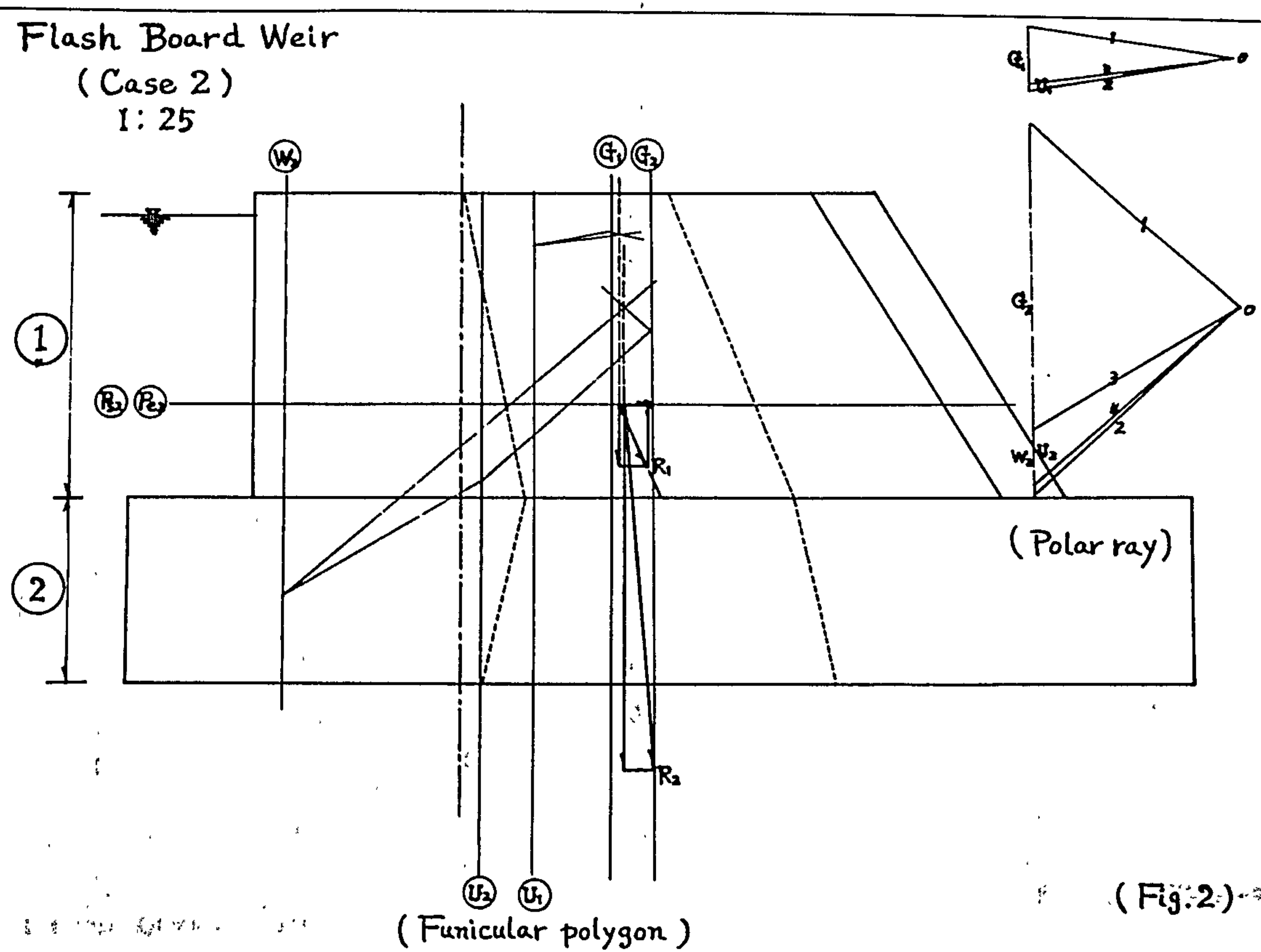
Junichi Kitamura

Flash Board Weir
(Case 1)
1 : 25



(Fig.1)

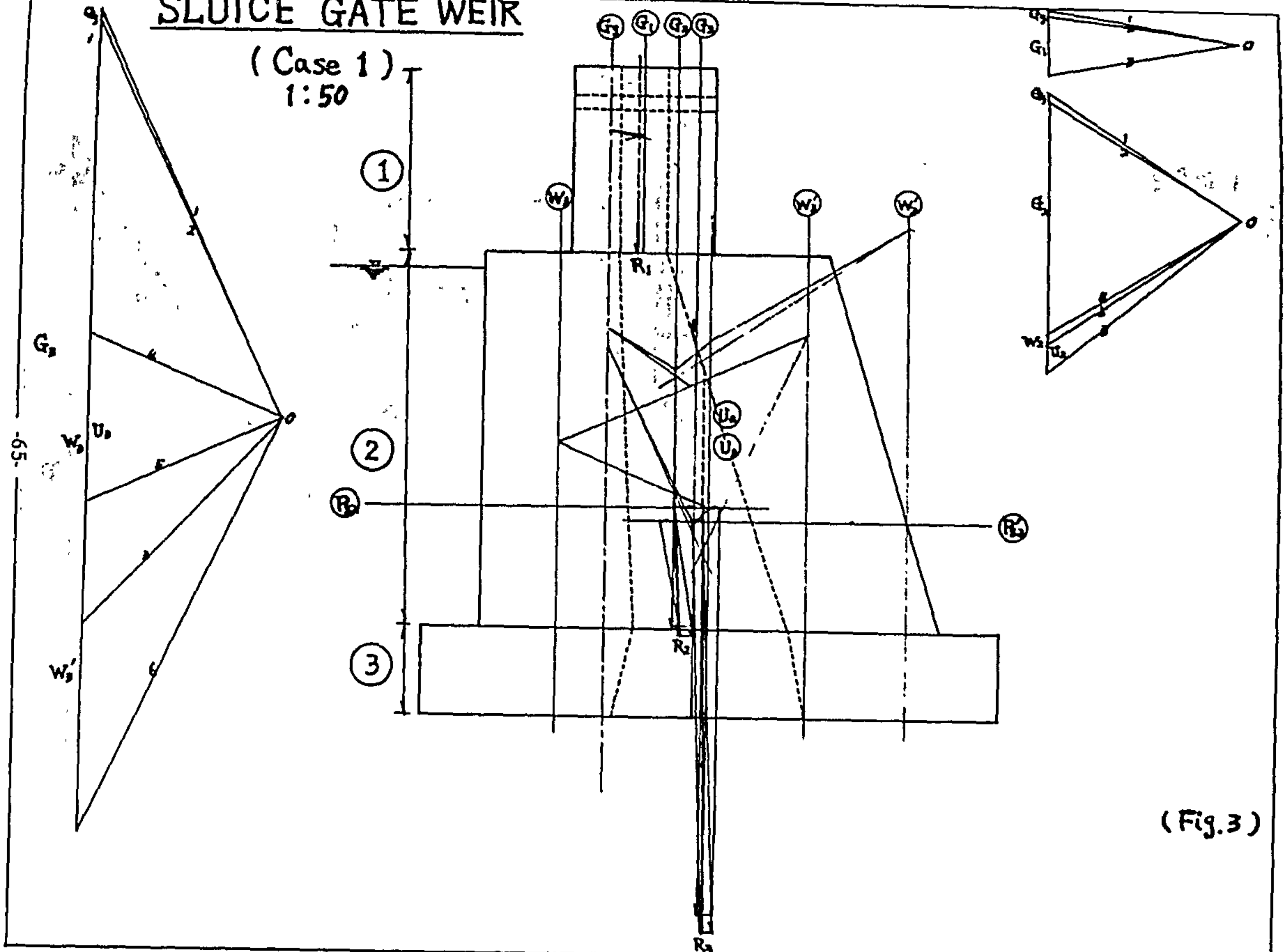
Flash Board Weir
(Case 2)
1 : 25



(Fig. 2)

SLUICE GATE WEIR

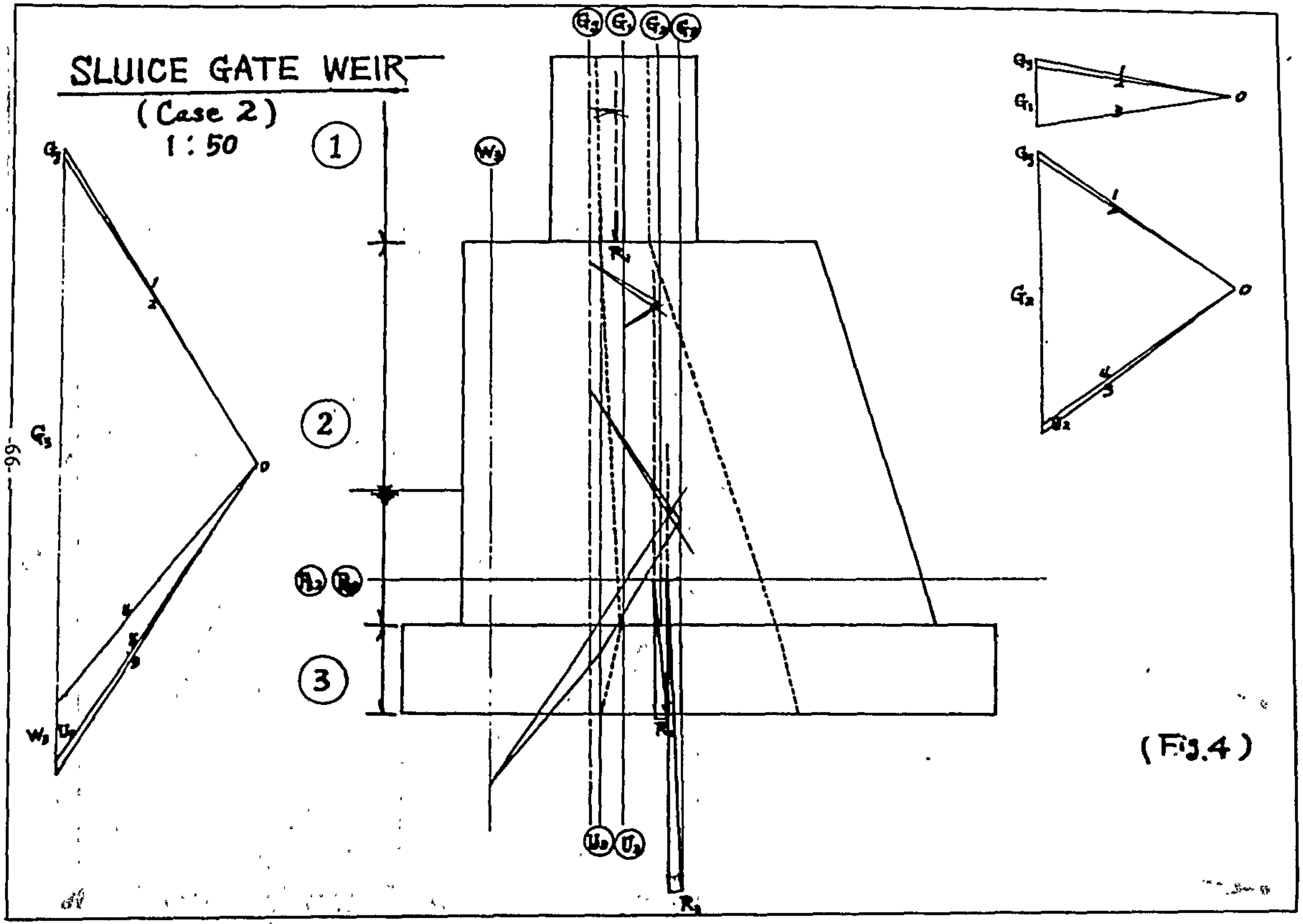
(Case 1)
1:50



(Fig.3)

SLUICE GATE WEIR

(Case 2)
1 : 50



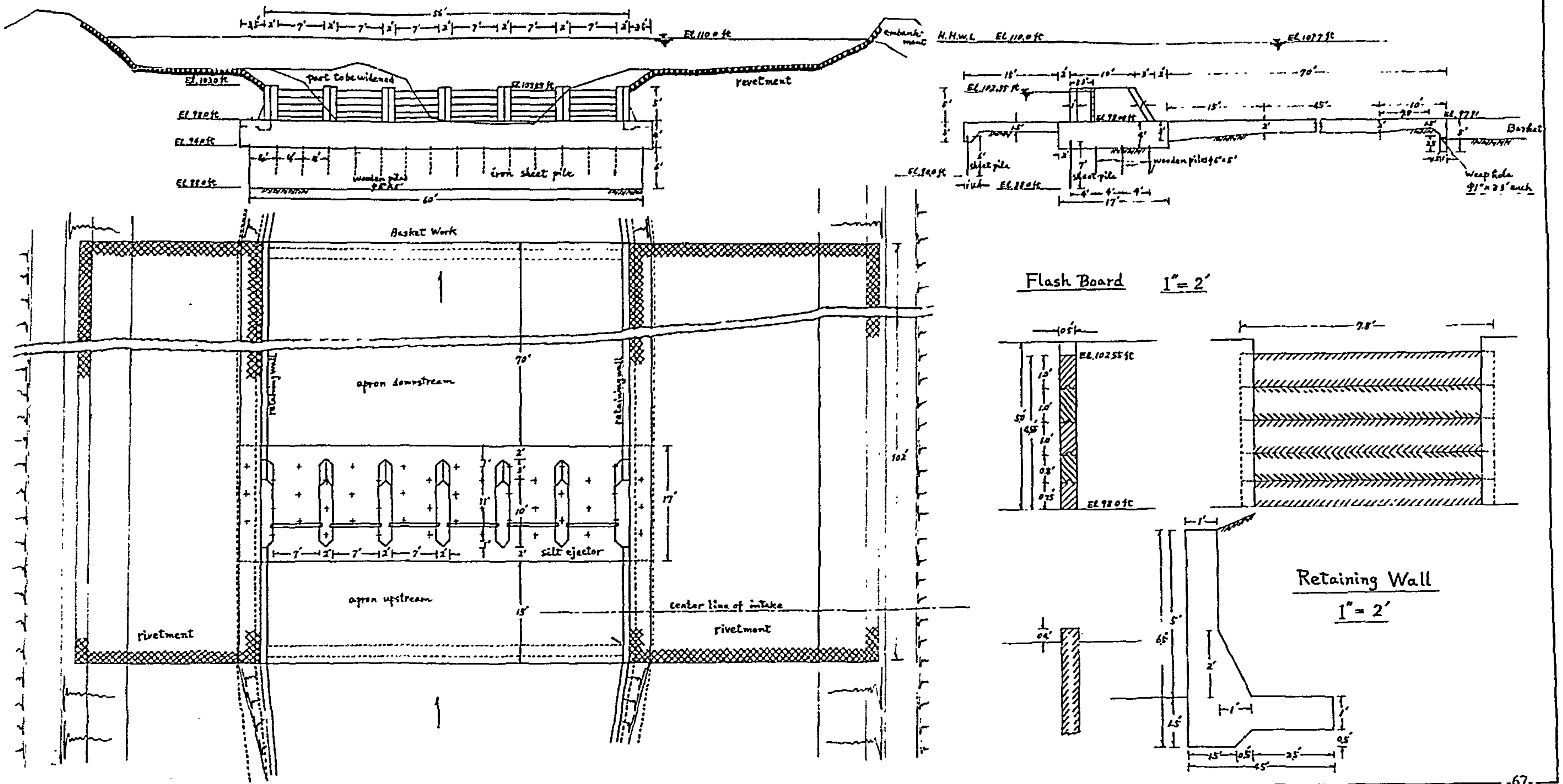
(Fig.4)

(Fig.5)

Design of Head Works on San Juan River

Flash Board Weir Type 1" = 10'

(at section 20)

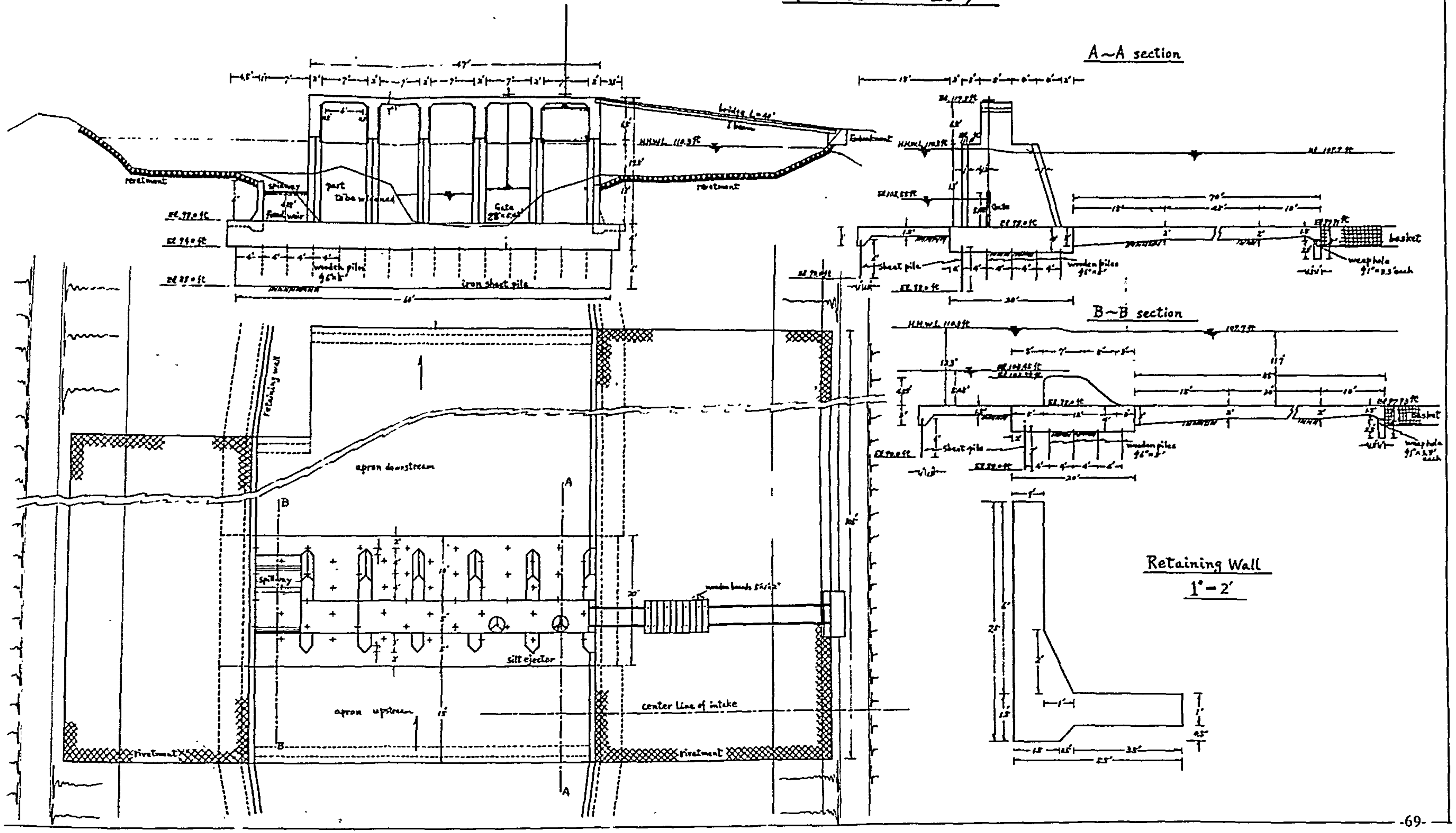


(Fig.6)

Design of Head Works on San Juan River

Sluice Gate Weir Type 1"=10'

(at section 20)



ARANGUEZ (VIII) December, 1967

I. INTRODUCTION:

Calculation of backwater on the San Juan River was requested under conditions listed below.

1) Position concerned .... The lower reach immediately next to the bridge on Churchill-Roosevelt Highway, 5265 feet up from the confluence with the Caroni River.

2) Designed flood discharge .... 1300 cusecs (=36.82m³/sec)

3) Water level of the Caroni River at the mouth of the San Juan River .... 104.5 feet

The longitudinal and cross sections were prepared for this calculation.

The San Juan River from the mouth up to the position concerned were divided into four.

(i.e.)	i)	the mouth	No. 30
	ii)	No. 30	No. 20
	iii)	No. 20	No. 10
	iv)	No. 10	the position concerned

The section No. 37 was substituted for the month, and the section No. 1 for the position concerned.

II. DETERMINATION OF THE DESIGNED FLOOD STAGE AT EACH SECTION CHOSEN WITHOUT CONSIDERATION OF THE INFLUENCE OF BACK-WATER FROM THE CARONI RIVER

Three or four water levels at each section were assumed and these discharges were calculated and are shown on table below, they were then plotted on section paper and the water levels required were finally determined as shown on Figure 1.

		(Water level)
at	No. 1	110.10 ft.
	No. 10	108.90
	No. 20	106.90
	No. 30	105.80
	No. 37	103.20

Cross Section No.	Elevation	Part No.	Section Area A	Wetted Perimeter P	Hydraulic Radius R	$R^{2/3}$	Coeff. of Roughness N	Slope I	$I^{1/2}$	Velocity V	Discharge Q	ΣQ
1	109.0	1	15.1□' 1.40m²	27.0' 8.23m	0.17m	0.307	0.080	1/2,170= 0.000461	0.0215	m/s 0.08	m³/s 0.11	m³/s
		2	46.5 4.32	21.5 6.55	0.66	0.758	0.065	•	•	0.25	1.80	
		3	259.0 24.06	37.0 11.28	2.13	1.656	0.035	•	•	1.02	24.50	
		4	79.5 7.39	46.0 14.02	0.53	0.655	0.065	•	•	0.22	1.63	
		5	9.5 0.88	14.0 4.27	0.21	0.353	0.080	•	•	0.09	0.08	27.40
	110.0	1	114.0 10.59	41.8 12.74	0.83	0.883	0.065	•	•	0.29	3.07	
		2	291.0 27.03	37.0 11.28	2.40	1.793	0.035	•	•	1.10	29.73	
		3	151.0 14.03	64.5 19.66	0.71	0.796	0.065	•	•	0.26	3.65	36.45
	111.0	1	168.0 15.61	45.3 13.81	1.13	1.084	0.065	•	•	0.36	5.62	
		2	323.0 30.01	37.0 11.28	2.66	1.920	0.035	•	•	1.18	35.41	
		3	215.0 19.97	67.0 20.42	0.98	0.987	0.065	•	•	0.33	6.59	47.62
10	108.0	1	19.8 1.84	20.2 6.16	0.30	0.448	0.080	1/1,390= 0.000720	0.0268	0.15	0.28	
		2	64.5 5.99	27.0 8.23	0.73	0.811	0.065	•	•	0.33	1.98	
		3	229.0 21.27	35.7 10.88	1.95	1.560	0.035	•	•	1.19	25.31	
		4	13.8 1.28	21.0 6.40	0.20	0.340	0.080	•	•	0.11	0.14	27.71
	108.5	1	29.8 2.77	21.8 6.64	0.42	0.561	0.080	•	•	0.18	0.50	
		2	78.0 7.25	27.0 8.23	0.88	0.918	0.065	•	•	0.38	2.76	
		3	244.0 22.67	35.7 10.88	2.08	1.629	0.035	•	•	1.25	28.34	
		4	24.4 2.27	22.3 6.80	0.33	0.478	0.080	•	•	0.16	0.36	31.96
	109.0	1	40.8 3.79	23.4 7.13	0.53	0.655	0.065	•	•	0.27	1.02	
		2	91.5 8.50	27.0 8.23	1.03	1.020	0.045	•	•	0.61	5.19	
		3	259.0 24.06	35.7 10.88	2.21	1.696	0.035	•	•	1.30	31.28	
		4	35.8 3.33	23.7 7.22	0.46	0.596	0.080	•	•	0.20	0.67	38.16
20	106.0	1	41.0 3.81	42.0 12.80	0.30	0.448	0.080	1/870= 0.001150	0.0339	0.19	0.72	
		2	198.8 18.47	34.2 10.42	1.77	1.462	0.035	•	•	1.42	26.23	
		3	19.0 1.77	23.8 7.25	0.24	0.386	0.080	•	•	0.16	0.28	27.23
	107.0	1	84.0 7.80	43.5 13.26	0.59	0.703	0.065	•	•	0.37	2.89	
		2	228.8 21.26	34.2 10.42	2.04	1.608	0.035	•	•	1.56	33.17	

Cross Section No.	Elevation	Part No.	Section Area A	Wetted Perimeter P	Hydraulic Radius R	Hydraulic Radius $R^{2/3}$	Coeff. of Roughness N	Slope I	$I^{1/2}$	Velocity V	Discharge Q	Discharge ΣQ
		3	52.5 4.88	25.8 7.86	0.62	0.727	0.065	1/870= 0.001150	0.0339	0.38	1.85	37.91
	107.5	1	106.0 9.85	44.4 13.53	0.73	0.811	0.065	"	"	0.42	4.14	
		2	243.8 22.65	34.2 10.42	2.17	1.676	0.035	"	"	1.62	36.69	
		3	44.0 4.09	26.5 8.08	0.51	0.638	0.065	"	"	0.33	1.35	42.18
	104.5	1	98.0m^2 9.10m^2	56.6' 17.25	0.53	0.655	0.080	1/1680= 0.000595	0.0244	0.20	1.82	
		2	14.1 1.31	5.9 1.80	0.73	0.811	0.065	"	"	0.30	0.39	
		3	197.7 18.37	25.0 7.62	2.41	1.797	0.035	"	"	1.25	22.96	
		4	21.0 1.95	11.7 3.57	0.55	0.671	0.065	"	"	0.25	0.49	
		5	39.2 3.64	27.7 8.44	0.43	0.570	0.080	"	"	0.17	0.62	26.28
30	105.5	1	145.0 13.47	57.0 17.37	0.78	0.847	0.065	"	"	0.32	4.31	
		2	245.0 22.76	33.5 10.21	2.23	1.707	0.035	"	"	1.19	27.08	
		3	88.5 8.22	35.5 10.82	0.76	0.833	0.065	"	"	0.31	2.55	33.94
	106.0	1	173.5 16.12	58.7 17.89	0.90	0.932	0.065	"	"	0.35	5.64	
		2	259.0 24.06	33.5 10.21	2.36	1.772	0.035	"	"	1.24	29.83	
		3	106.0 9.85	36.5 11.13	0.88	0.918	0.065	"	"	0.34	3.35	38.82
	100.0	1	5.1 0.47	4.2 1.28	0.37	0.514	0.065	1/590= 0.001704	0.0413	0.33	0.15	
		2	107.4 9.98	24.3 7.41	1.35	1.221	0.035	"	"	1.44	14.37	
		3	5.0 0.46	7.5 2.29	0.20	0.342	0.080	"	"	0.18	0.08	14.60
	102.0	1	12.8 1.19	7.0 2.13	0.56	0.679	0.065	"	"	0.43	0.51	
37		2	151.4 14.07	24.3 7.41	1.90	1.534	0.035	"	"	1.81	25.47	
		3	28.0 2.60	14.5 4.42	0.59	0.703	0.065	"	"	0.45	1.17	27.15
	103.5	1	7.5 0.70	12.0 3.66	0.19	0.330	0.080	"	"	0.17	0.12	
		2	255.0 23.69	46.5 14.17	1.67	1.408	0.035	"	"	1.66	39.33	39.45

Remarks Assumption of surface slope

For the surface slope at each section, the mean value of the river-bed slope between ten cross sections is adopted. (viz)

For No. 1 No. 1 - No. 10 $I = \dfrac{99.11 - 98.67}{900} = \dfrac{0.44}{900} = 0.000461 = \dfrac{1}{2{,}170}$

No. 10 No. 5 - No. 15 $I = \dfrac{98.48 - 97.76}{1{,}000} = \dfrac{0.72}{1{,}000} = 0.000720 = \dfrac{1}{1{,}390}$

No. 20	No. 15 - No. 25	$I = \frac{97.76 - 96.15}{1,400} = \frac{1.61}{1,400}$	$= 0.001150$	$= \frac{1}{870}$
No. 30	No. 25 - No. 35	$I = \frac{96.15 - 95.02}{1,900} = \frac{1.13}{1,900}$	$= 0.000595$	$= \frac{1}{1,680}$
No. 37	No. 30 - No. 37	$I = \frac{95.85 - 93.54}{1,355} = \frac{2.31}{1,355}$	$= 0.001704$	$= \frac{1}{590}$

III. DETERMINATION OF THE WATER LEVEL AT EACH SECTION CHOSEN, DURING THE DESIGNED FLOOD DISCHARGE INCLUSIVE OF THE INFLUENCE OF THE BACKWATER EFFECT OF THE CARONI RIVER

1) Computation of the mean coefficient of roughness at each section.

Cross Sec-tion	Elevation	Part No.	P	n	$n^{3/2}$	$Pn^{3/2}$	$\Sigma Pn^{3/2} / \Sigma p$	$\bar{n} = (\Sigma Pn^{3/2} / \Sigma p)^{2/3}$
1	110.10 ft	1	42.2' 12.86 m	0.065	0.01657	0.2131		
		2	37.0 11.28	0.035	0.00654	0.0738		
		3	64.8 19.75	0.065	0.01657	0.3273		
		Total	43.89			0.6142	0.013994	0.058
10	108.90	1	22.8 6.95	0.065	0.01657	0.1152		
		2	27.0 8.23	0.045	0.00955	0.0786		
		3	35.7 10.88	0.035	0.00654	0.0712		
		4	23.1 7.04	0.080	0.02263	0.1593		
		Total	33.10			0.4243	0.012819	0.055
20	106.90	1	43.4 13.23	0.065	0.01657	0.2192		
		2	34.2 10.42	0.035	0.00654	0.0681		
		3	25.6 7.80	0.065	0.01657	0.1292		
		Total	31.45			0.4165	0.013243	0.054
30	105.80	1	57.8 17.62	0.065	0.01657	0.2920		
		2	33.5 10.21	0.035	0.00654	0.0668		
		3	36.0 10.97	0.065	0.01657	0.1818		
		Total	38.80			0.5406	0.13933	0.058
37	103.20	1	11.2 3.41	0.080	0.02263	0.0772		
		2	46.1 14.05	0.035	0.00654	0.0919		
		Total	17.46			0.1691	0.009685	0.045

2) Determination of backwater level at each section

Basic formula:

$$h = \frac{nm^2 \cdot Q^2 \cdot \ell}{Am^2 \cdot Rm \cdot Rm^{1/3}}$$

h	:	Difference of water level between any two successive cross sections
n_m	:	mean coefficient of roughness between any two successive cross sections
Q	:	Designed flood discharge which is 36.82 m^3/s
ℓ	:	Distance between any two successive cross sections
Am	:	Mean area of any two successive cross sections
Rm	:	Mean hydraulic radius between any two successive cross sections

No.		No. 37	No. 30	Mean	Remarks
1	Elevation	104.50 ft	106.45 ft		Q = 1,300 cusecs
2	Assumed h	1.95 ft = 0.59 m			= 36.82 m^3/s Q^2 = 1355.71
3	A	350.0□' 32.52 m^2	602.3□' 55.95 m^2	44.24 m^2	nm = 0.0515
4	P	99.3' 30.27 m	132.0' 40.23 m	35.25 m	nm^2 = 0.00265
5	R			1.255 m	ℓ = 1,464 ft = 446.5 m
6	$R^{1/3}$			1.079	
7	A^2			1,957.2	
8	$n^2 \cdot Q^2 \cdot \ell$			1,604.1	
9	$A^2 \cdot R \cdot R^{1/3}$			2,650.3	
10	h			0.60 m	

No.		No. 30	No. 20	Mean	Remarks
1	Elevation	106.45 ft	108.35 ft		Q = 1,300 cusecs
2	Assumed h	1.90 ft = 0.58 m			= 36.82 m^3/s Q^2 = 1,355.71
3	A	602.3□' 55.95 m^2	521.2□' 48.42 m	52.19 m^2	nm = 0.0560
4	P	132.0' 40.23 m	120.1' 36.61 m	38.42 m	nm^2 = 0.00314
5	R			1.358 m	ℓ = 1,845 ft = 562.4 m
6	$R^{1/3}$			1.107 m	
7	A^2			2,723.8	
8	$n^2 \cdot Q^2 \cdot \ell$			2,394.1	
9	$A^2 \cdot R \cdot R^{1/3}$			4,094.7	
10	h			0.58 m	

No.		No. 20	No. 10	Mean	Remarks
1	Elevation	108.35 ft	109.75 ft		Q = 1,300 cusecs = 36.82 m^3/s Q^2 = 1,355.71 .
2	Assumed h	1.40 ft 0.41 m			
3	A	521.1□' 48.42 m^2	470.3□' 43.69 m^2	46.06 m^2	n_m = 0.0545
4	P	120.1' 36.61 m	112.5' 34.29 m	35.45 m	nm^2 = 0.00297
5	R			1.299 m	ℓ = 1,000 ft = 304.8 m
6	$R^{1/3}$			1.090	
7	A^2			2,119.7	
8	$n^2 \cdot Q^2 \cdot \ell$			1,227.3	
9	$A^2 \cdot R \cdot R^{1/3}$			3,001.3	
10	h			0.41	

No.		No. 10	No. 1	Mean	Remarks
1	Elevation	109.75 ft	110.75 ft		Q = 1,300 cusecs = 36.82 m^3/s
2	Assumed h	1.00 ft 0.31 m			
3	A	470.3□' 43.69m^2	680.8□' 63.25m^2	53.47m^2	Q^2 = 1,355.71
4	P	112.5' 34.29m	155.9' 47.52m	40.91m	nm = 0.0565
5	R			1.307 m	nm^2 = 0.00319
6	$R^{1/3}$			1.093	ℓ = 955 ft = 291.1 m
7	A^2			2,859.0	
8	$n^2 \cdot Q^2 \cdot \ell$			1,258.9	
9	$A^2 \cdot R \cdot R^{1/3}$			4,084.3	
10	h			0.31	

This calculation was carried out step by step.

The water level at the cross section No. 37 was first taken as 104.50 ft just above the confluence with the Caroni River.

The difference between the water level at No. 37 and at No. 30, h was assumed.

Then the water level at No. 30 was determined and the cross section area, A, and the wetted perimeter P, were measured. The hydraulic radius R was also calculated using these values.

On the other hand, the mean coefficient of roughness n, the designed flood discharge Q and distance between No. 37 and No. 30, ℓ had already been determined.

Thus h by the basic formula was calculated with the data mentioned above.

And the 'trial and error' method was continued until 'h' by the basic formula coincided with the assumed h.

Subsequently the calculation between No. 30 and No. 20, No. 20 and No. 10, No. 10 and No. 1 were carried out in the same manner. The result were as follows.

No. 37	....	104.50 ft	
No. 30	....	106.45	
No. 20	....	108.35	
No. 10	....	109.75	
No. 1	....	110.75	(cf. Fig. 2)

IV. CONCLUSION

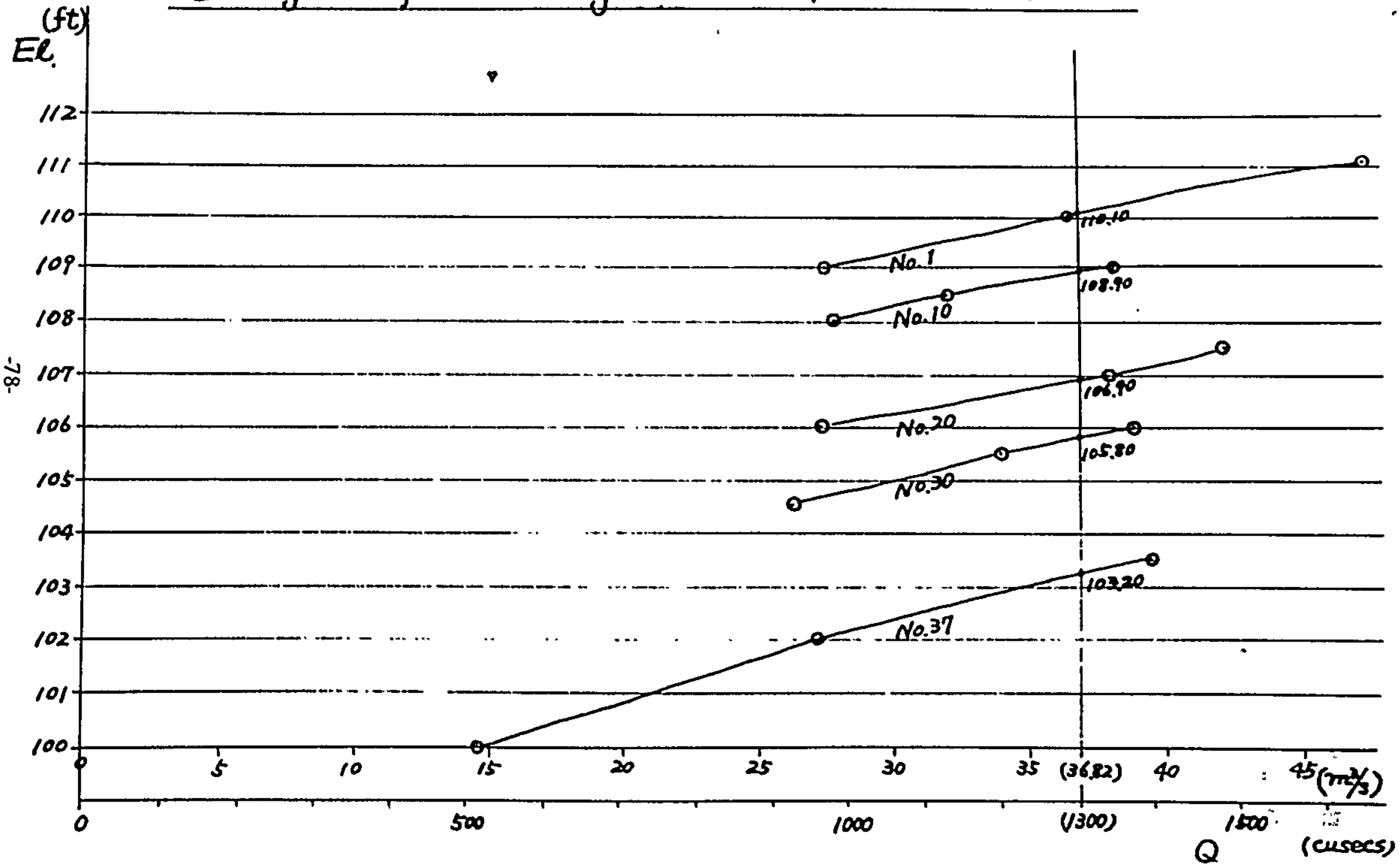
The backwater level at No. 1 is still 0.65 ft higher than the designed flood stage there.

It will, however, coincide with the designed flood stage at some point not far away upstream of No. 1.

Junichi Kitamura

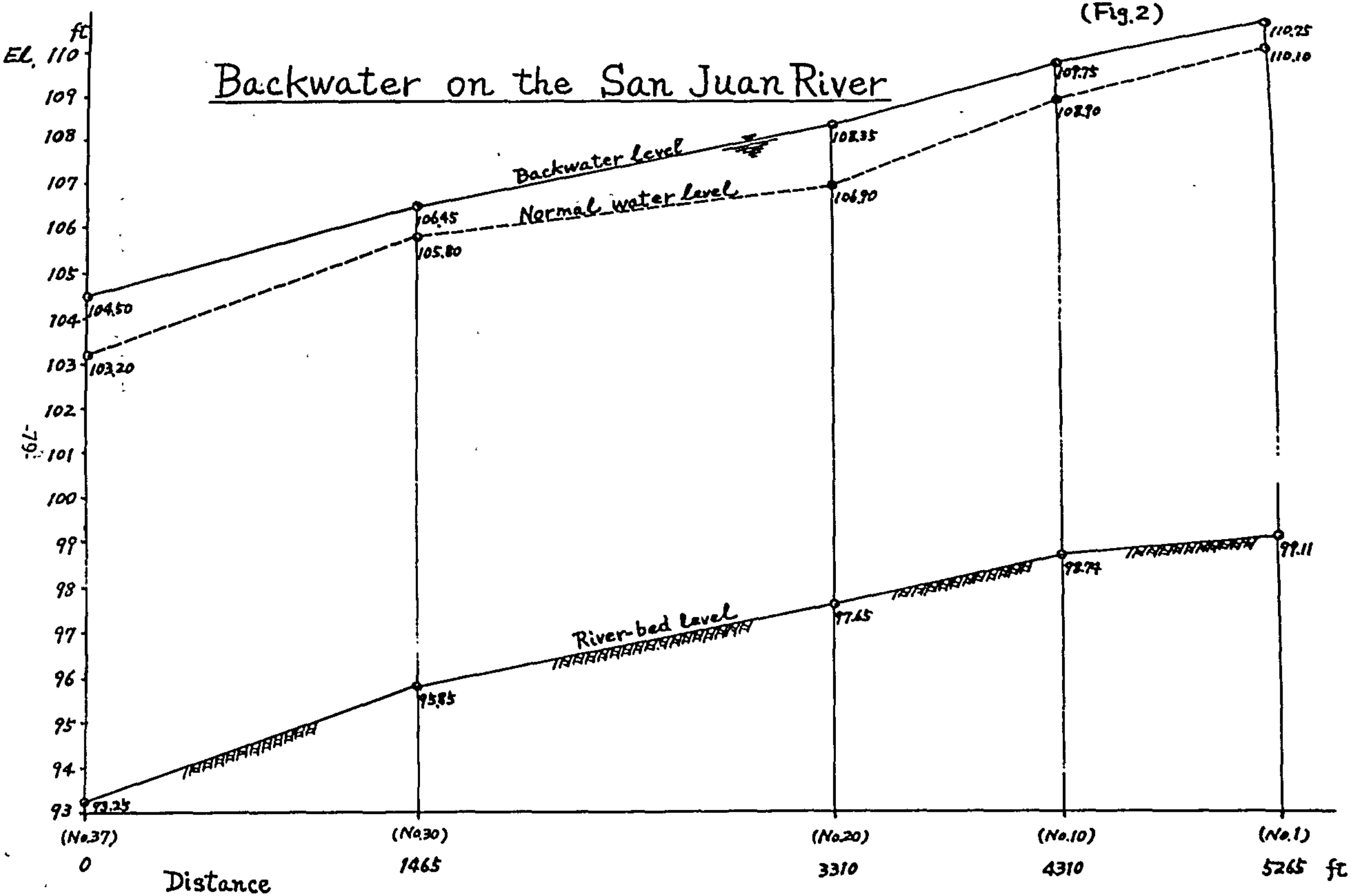
(Fig. 1)

Designed flood stage on the San Juan River



(Fig.2)

Backwater on the San Juan River



ARANGUEZ IX February, 1968

MOVEMENT

9 and 15, January

Visited the proposed headworks site at Section No. 20, 2,000 ft away from the Bridge on Churchill Roosevelt Highway.

17, January

Requested that stakes be put every 500 ft from the bridge to the site, that a benchmark be set up in the area and that a detailed cross section at the site of the headworks be made.

24, January

The works mentioned above were carried out. Visited there to check the staked out area.

1, February

Had the area resurveyed because of a discrepancy of about 2 feet in elevation.

INTRODUCTION

The design of the intake will now be undertaken. The intake must be set in the effective range of the silt-ejector which stabilizes the course of the river flow and maintains its depth.

The lower edge of the intake will have to be kept away 3 - 5 m upstream from the pier of the silt-ejector so as to preclude the entry of eddies containing silt in high density into the main, irrigation canal. An overhanging sill is also recommended to further prevent an inflow of silt.

BASIS OF THE DESIGN

1. On the report Aranguez (111), the highest elevation in the irrigated (or improved) areas as determined as 101.00 ft, and the water level at the beginning of the main canal was regarded as 101.05 ft the crest level of the weir was estimated as 102.55 ft.

2. On the report Aranguez (VII), the sill-level of silt-ejector was determined as 98.0 ft.

3. On the report Aranguez (111), the maximum possible water quantity available from the San Juan River in dry season was calculated as 3.71 cusecs. (= 0.105 m^3/sec).

4. On the other hand, on the report Aranguez (V) the maximum gross water requirement for

sprinkler irrigation was calculated as 6.10 cusecs (= 0.173 m^3/s) and the section at the beginning of the main irrigation canal had a width of 2.5 ft, a depth of 2.0 ft and a slope of 1/1,800. The designed intake water quantity (that is the maximum gross water requirement for sprinkler irrigation in some cases and the maximum possible water quantity available from the river in dry season in other cases), the water level at the beginning of the main canal and the sill-level of the silt-ejector have already been finalised.

They must be determined by the method of 'trial and error' because of their inter-relationship.

POSITION OF THE INTAKE

The distance between the lower edge of the intake and the pier of the silt-ejector is adopted as 10 ft for the reason aforementioned.

TYPE OF INTAKE

If the intake is designed as an open channel, heavy expenditure for preventing invasion of the flood into the field will be needed, for building tall regulating gates, embanking dikes and the like:

Effective use of the existing diles are desirable.

Therefore, culvert-type will be the best. (Cylindrical pipe will be not always appropriate on account of its permitting an easy inflow of silt).

Culverts for 'intakes' which go through dikes are very important structures on river works.

Close attention for safety against flood should be paid.

1. The culvert must be set up at a right angle to the dike.

2. Foundation works to prevent differential settlement are necessary.

3. Cut-off walls and clayey core are needed to keep the structure water-tight.

4. Revetment of the dike-slope connecting with the intake is necessary.

5. The work must be carried out under dry condition to the exclusion of water.

SILL-LEVEL OF THE INTAKE

The sill-level must be determined in the position in which as little silt as possible is taken from the river.

Generally a perpendicular distribution of silt content in a river shows maximum silt content is at the position 0.8 H down from the water surface. The higher the sill level is set up, the larger width of intake is demanded. The lower the sill is set up, the more silt flows into the culvert. It is suggested that sill-level must be determined in a position 1-3 ft higher than the silt-ejector level. In this case, 3 ft being adopted, sill-level will be 101.00 ft.

INFLOW VELOCITY AT THE INTAKE

The determination of the most suitable velocity for the intake is somewhat difficult. However it must be determined, so as to get the most appropriate condition hydraulically and economically.

In case of a high velocity, the width of the intake may be small, the cost of construction may also be small.

On the other hand, silt-inflow-quantity and loss of head will be large, a small sedimentation basin will therefore be needed, and vice versa. Generally speaking, the velocity of 0.6 to 1.0 m/s is demanded. In this case 2 ft/sec (= 0.6 m/sec) is adopted.

WIDTH OF THE INTAKE

Under the following conditions the width of the intake, B can be calculated.

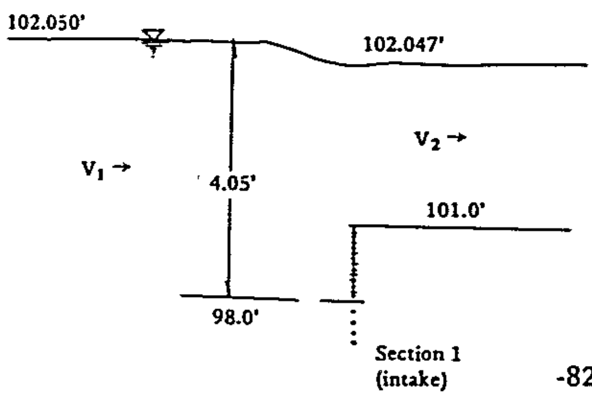
the maximum gross water requirement:	$Q = 0.173\ m^3/s$ (6.1 cusecs)
the sill-level of the intake:	101.00 ft
the velocity of inflow:	$V = 0.6\ m/s$ (20 ft/s)

Assuming that the water level at the intake is 102.05 ft
the water depth will be: $H = 102.05 - 101.0 = 1.05$ ft (= 0.32 m)

$$A = Q/V = 0.173/0.6 = 0.29\ m^2$$
$$B = A/H = 0.29/0.32 = 0.91\ m$$

LOSS OF HEAD

1. Loss of head at inlet



$$hi = \frac{fi\ V_2^2}{2g} + \frac{V_2^2 - V_1^2}{2g}$$

hi	:	Loss of head by inflow
fi	:	Coefficient of loss
V_1	:	Velocity before inflow
V_2	:	Velocity after inflow
g	:	Acceleration due to gravity which is $9.8\ m^2/sec^2$

Depth H must be determined by assuming a loss of head due to velocity of flow $\frac{(v^2)}{2g}$. In assuming a loss of head, the velocity after inflow must also be assumed.

The velocity after inflow is estimated by dividing the discharge by the product of the initial depth (H_1) by the width (B) of the culvert.

V_1 = 0 m/s (in case of taking a whole water)

$V^2 = Q/A = Q/BH_1$

$= \frac{0.173}{09.1 \times 1.234} = 0.154$ m/s

$\frac{V_2^2}{2g} = \frac{(0.154)^2}{2 \times 9.8} = 0.001$ m

H_1 : Depth before inflow which is 4.05' (= 1.234 m)

B : Width of the intake which is 0.91 m

H_2 : Depth after inflow

Therefore, assuming that hi = 0.001 m

$$V_2 = \frac{Q}{BH^2} = \frac{0.173}{0.91 \times (1.234 - 0.001)} = \frac{0.173}{1.122} = 1.543 \text{ m}$$

fi = 0.5 (in case of sharp corner)

$$hi = 0.5 \times \frac{(1.543)^2}{2 \times 9.8} + \frac{(1.543)^2}{2 \times 9.8} = 0.001 \text{ m } (= 0.003')$$

This result coincides with the assumed value.

The water level will be 102.050 - 0.003' = 102.047'

2. Loss of head due to charge in Section at inlet:

$$hc = fc \frac{V_2^2}{2g} + \frac{V_2^2 - V_1^2}{2g}$$

fc : Coefficient of loss

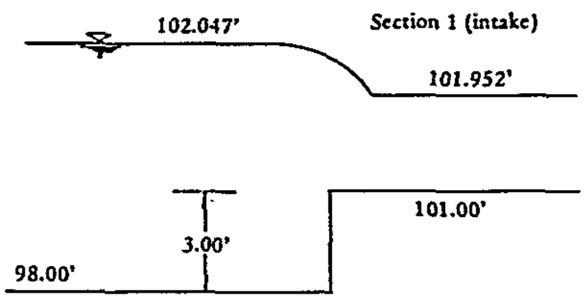
A_2/A_1	0.01	0.1	0.2	0.4	0.6	0.8	1.0
fc	0.44	0.41	0.38	0.29	0.18	0.09	0.00

This table shows the experimental values by Weisbach.

H_1 = 102.047' - 98.00' = 4.047' = 1.234 m

H_2 = 102.047' - 101.00' = 1.047' = 0.319 m

$$A_2/A_1 = BH_2/BH_1 = \frac{0.91 \times 0.319}{0.91 \times 1.234} = 0.25 \quad \therefore fc = 0.36$$



V_1 = 0.154 m/s (cf. 1)

$V_2 = Q/A = \dfrac{0.173}{0.91 \times 0.319} = \dfrac{0.173}{0.290} = 0.597$ m/s

$\dfrac{V_2^2}{2g} = \dfrac{(0.597)^2}{2 \times 9.8} = \dfrac{0.358}{19.6} = 0.018$ m

Therefore, assuming that hc = 0.030 m

$V_2 = \dfrac{Q}{BH} = \dfrac{0.173}{0.91 \times (0.319 - 0.030)} = \dfrac{0.173}{0.263} = 0.658$ m/s

$hc = \dfrac{0.36 \times (0.658)^2}{2 \times 9.8} + \dfrac{(0.658)^2 - (0.154)^2}{2 \times 9.8} = 0.029$ m (= 0.095')

This result coincides very nearly with the assumed value. The water level will be

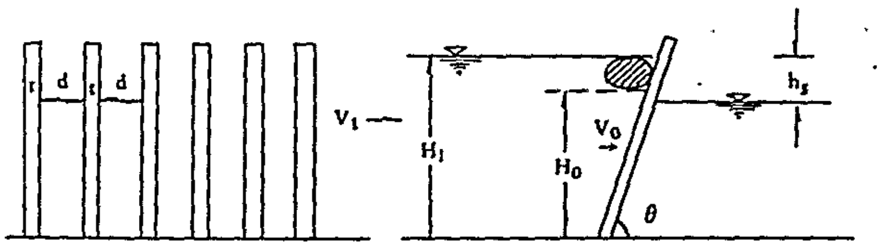
102.047' - 0.095' = 101.952'

3. Loss of head due to pier.

In this case there is no loss of head because piers are not needed.

4. Loss of head due to screen.

Section 11 (screen)



$h_s = B \sin\theta \left(\dfrac{t}{d}\right)^{4/3} \dfrac{V_0^2}{2g}$ (where $V_0 = \dfrac{V_1 H_1}{H_0}$)

B : Coefficient due to the shape of screen cross section which is 2.42 in case of a rectangle

θ : Inclination angle of screen = 70°

t : Thickness of screen grating = ½" (= 12.7 mm)

d : Interval of screen grating = 2" (= 50.8 mm)

V_1 : Velocity upstream of the screen when there is no floating matter = 0.597 m/s

Vo : Velocity upstream of the screen when there is floating matter = $V_0 = \dfrac{V_1 H_1}{H_0}$

H_1 : Depth in front of the screen which is 0.952 = 0.29 m

H_0 : Depth in front of screen when there is floating matter = (0.952' - 0.500') = 0.452' (= 0.14m)

(The depth occupied by floating matter is assumed to be 0.500 ft)

Hence $V_0 = \dfrac{0.597 \times 0.29}{0.14} = 1.24$ m/s

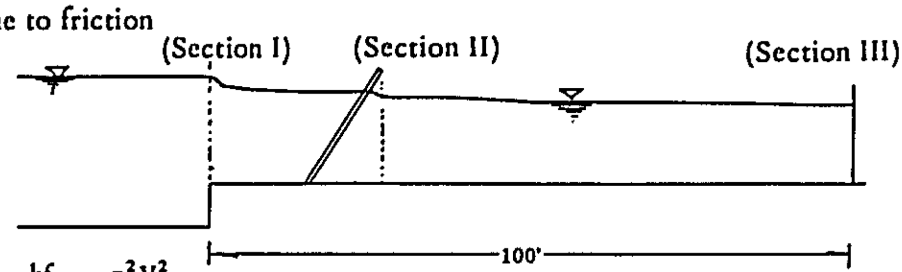
$\sin \theta \; = \; \sin 70^\circ \; = \; 0.93969$

$h_s \; = \; 2.42 \times 0.03969 \dfrac{(12.7)}{5.08} \dfrac{4}{3} \times \dfrac{(1.24)^2}{2 \times 9.8} \; = \; 0.028 \text{ m} \; = \; 0.092'$

The water level will be

101.952' - 0.092' = 101.860'

5) Loss of head due to friction



$I \; = \; \dfrac{hf}{\ell} = \dfrac{n^2 V^2}{R^{4/3}}$

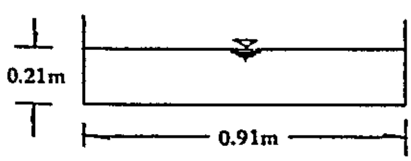
I	:	Slope of water surface
hf	:	Loss of head due to friction
ℓ	:	length of the culvert which is adopted as 100 ft = (30.48 m)
n	:	Coefficient of roughness which is 0.017
V	:	Velocity of flow
R	:	Hydraulic radius
Q	=	$0.173 \text{ m}^3/\text{s}$
H	=	101.860' - 101.000' = 0.860' = 0.26 m
B	=	0.91 m
A	=	$0.91 \times 0.26 = 0.237 \text{ m}^2$
V	=	Q/A = 0.173/0.237 = 0.730 m/s
R	=	$\dfrac{A}{2H + B} = \dfrac{0.237}{1.43} = 0.166$ m
I	=	$\dfrac{(0.017)^2 \times (0.730)^2}{(0.166)^{4/3}} = \dfrac{0.000289 \times 0.5329}{0.091231} = 0.00169 \; \left(= \dfrac{1}{592}\right)$
hf	=	$30.48 \times 0.00169 = 0.052 \text{ m} = 0.171'$

The water level will be

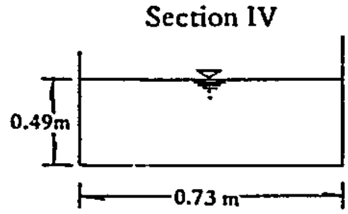
101.860' - 0.171' = 101.689'

6) Loss of head due to gradual change of the section

Section III



Section IV



The section at the beginning of the main irrigation canal is already calculated to be :
Width = 2.5' and slope of 1/1,800. Wherefore, if the discharge is 0.173 m^3/s (= 6.10 cusecs) the velocity and the depth will be as follows:

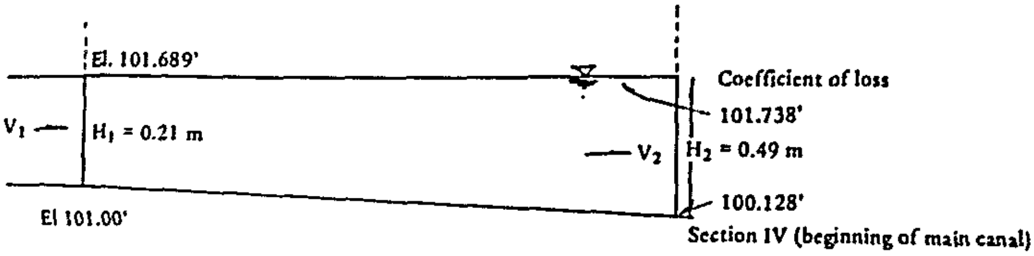
Assumed depth H	Width B	Sectional Area A	Wetted perimeter P	Hydraulic radius R	$\sqrt{R}$	Roughness coeff(n)	Kutter's N	Kutter's D	NR	$D+\sqrt{R}$	Velocity V	Calculated discharge Q
0.49m	0.75m	0.368m^2	1.73m	0.213m	0.462	0.017	1.994	0.438	0.425	0.900	0.472m/s	0.1737m^3/s

The calculated discharge by assumed depth of 0.49 coincides with the required discharge of 0.173m^3
At which time, the velocity will be 0.472 m/s.

$$h_t = f_t \frac{|V_2^2 - V_1^2|}{2g} + \frac{V_2^2 - V_1^2}{2g} + hf$$

$$hf = \frac{n^2 Vm^2\, l}{R^{4/3}}$$

ht : Loss of head due to gradual change of the section.



Section III

H_1 = 101.689' - 101,000' = 0.689' = 0.21 m

A = 0.91 x 0.21 = 0.191 m^2

$V_1 = Q/A = \frac{0.173}{0.191} = 0.905$ m/s

P_1 = 2 x 0.21 + 0.91 = 1.330 m

$R_1 = A/P_1 = \frac{0.191}{1.330} = 0.144$ m

V_2 = 0.472 m/s (cf. above table)

R_2 = 0.213 m (cf. above table)

$Vm = \frac{V_1 + V_2}{2} = \frac{0.905 + 0.472}{2} = 0.689$ m/s

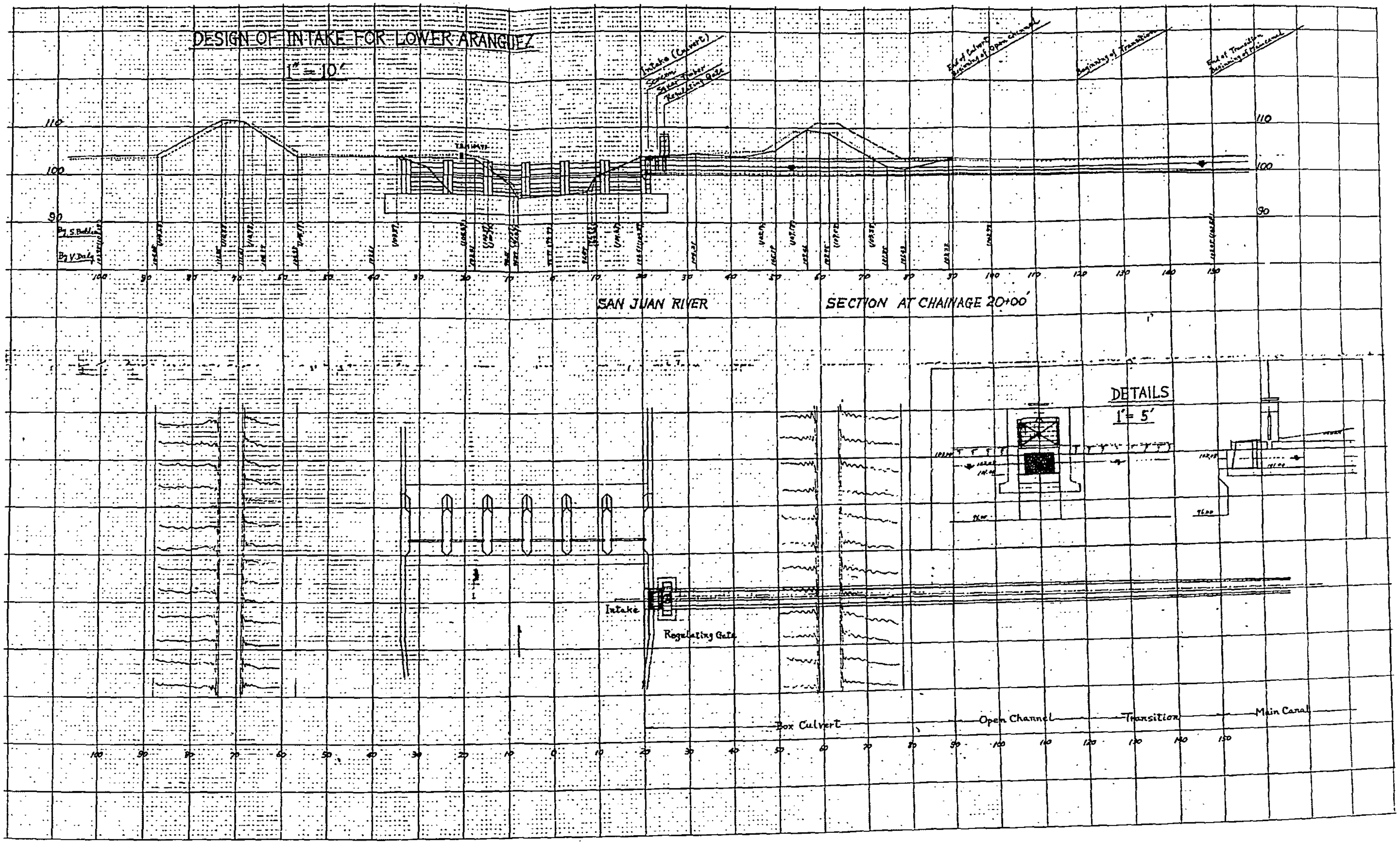
$Rm = \frac{R_1 + R_2}{2} = \frac{0.144 + 0.213}{2} = 0.179$ m

l = 30' = 9.14 m n = 0.017

$\therefore$ $hf = \frac{(0.017)^2 \times (0.689)^2 \times 9.14}{(0.179)^{4/3}} = 0.0124$ m

ft = 0.1 (in case of bell-mouth)

$h_t = 0.1 \times \frac{[(0.472)^2 - (0.905)^2]}{2 \times 9.8} + \frac{(0.472)^2 - (0.905)^2}{2 \times 9.8} + 0.0124$


DESIGN OF INTAKE FOR LOWER ARANGUEZ
1" = 10'
Intake (Culvert)
Screen
Scour Timber
Regulating Gate
End of Culvert
Beginning of Open Channel
Beginning of Transition
End of Transition
Beginning of Main canal
110
100
90
By S. Baddoo
By V. Daly
SAN JUAN RIVER
SECTION AT CHAINAGE 20+00'
DETAILS
1" = 5'
Intake
Regulating Gate
Box Culvert
Open Channel
Transition
Main Canal

$$= \frac{0.0596}{19.6} - \frac{0.5962}{19.6} + 0.0124$$

$$= 0.030 - 0.0304 + 0.0124 = -0.015\,m = -0.049'$$

The water level will be 101.689' + 0.049' = 101.738'

As the depth H is 0.49m (=1.61'), the sill-level of the canal will be

101.738' - 1.61' = 100.128'

As mentioned above, the highest elevation in area to be irrigated is 101.00 ft.

The water level of 101.738' is therefore sufficient to lead the water into the areas.

Examination of the water level at section IV in case the discharge is 0.105 m^3/s (= 3.71 cusecs which is the maximum possible water quantity available from the San Juan River in dry season.

Assumed depth H	Width B	Sectional area A	Wetted perimeter P	Hydraulic radius R	R	Roughness coeff. n
0.34 m	0.75 m	0.255m²	1.43 m	0.178 m	0.422	0.017

Kutter's N	Kutter's D	NR	$D+\sqrt{R}$	Velocity V	Calculated discharge Q
1.994	0.438	0.355	0.860	0.413 m/s	0.105 m^3/s

The calculated discharge by assumed depth of 0.34 m coin-cides with the required discharge of 0.105 m^3/s. As the depth is therefore determined as 0.34 m (= 1.115') the water level will be 100.128' + 1.115' = 101.243'. This is also high enough to lead the water into the fields.

Sedimentation basin

Although a sedimentation basin is inevitably necessary for intake on a large scale, this case it is not necessary because the intake on a small scale.

REMARKS

The longitudinal and cross sections on the San Juan River were made by the survey in 1967.

Moreover the survey for the cross section at No. 20 was suggested to be carried out in January, 1968 at which time a difference of about 2 feet in elevation relatively was found.

The third survey for the cross section was undertaken and it was found that the survey in January this year was correct. As a result, the sill level of the headworks has to be lowered from 98 ft to 96 ft.

Strictly speaking the dynamic calculation for the headworks should be re-done. As the dynamic calculation carried out on the report Aranguez (VII), however has enough clearance for safety, it is suggested that the calculation for the headworks 2 feet higher than before can be accepted without any change.

Junichi Kitamura

ARANGUES (X) Final Report October 1968

INTRODUCTION

The installation costs for irrigation and drainage scheme which were planned and designed in the report (I) - (IX) so far, will be estimated in this report.

Before the estimate the calculation for quantity of materials required for the structures must be carried out.

Contents in this report are shown as follows:-

A. Irrigation Scheme

A-1) In case of Furrow Irrigation

1) Headworks (Flashboard weir
(Sluice gate weir
2) Intake
3) Main and Branch canals
4) Intake Pipes into Furrows
5) Roads in the area

A-2) In case of Sprinkler Irrigation

1) Headworks (Flash board weir
(Sluice gate weir
2) Intake
3) Main and Branch canals
4) Facilities for Sprinkling
5) Roads in the area

B. Drainage Scheme

1) Widening of the San Juan River
2) Embankment against the Caroni River
3) Installation of two pumps (85 HP dia = 800 mm)
4) Installation of control gates
5) Drains in the area

Contents of the reports 'Aranguez'

Aranguez	(I)	General study for countermeasures
	(II)	Measurement of discharge on the San Juan River in dry season. Calculation for the maximum volume of water supplied from the San Juan River in dry season and for water requirement in case of furrow irrigation.
	(III)	Calculation for flood discharge on the San Juan River
	(IV)	Test for bearing capacity of the ground at headwork site
	(V)	Sprinkler irrigation scheme
	(VI)	Calculation for flood discharge on the Caroni River and drainage scheme
	(VII)	Design of headworks - Flushboard type and Sluice gate type
	(VIII)	Calculation for backwater on the San Juan River
	(IX)	Design of intake
	(X)	Calculation for quantity of materials required for the structures and estimate of the cost for irrigation and drainage scheme

Calculations for quantity of materials:

A. Irrigation Scheme

A-1) Furrow Irrigation

1) Headworks

1-1) Flash Board Weir

i) Volume of cut: (Exclusive of the one for widening the San Juan River)

for	Both Sides	(2 x 15' x 8' x 110') + 4 x ½(15' x 8' x 20') = 31,200 ft^3
	Fore apron	70' x (12.5' x 1.5' + 2.5' x 3.0') = 1,838 ft^3
	Footing of piers	70' x 17' x 4' = 4,760 ft^3
	Rear apron	70' x (70' x 2' + 2.5' x 3.0') = 10,325 ft^3
	Rip-raps (gabion baskets)	½(70' + 50') x 20' x 3' + 50' x 40' x 3' = 9,600 ft^3
	Total = 57,723 ft^3	

Although by using a formula the necessary length of Rip-Rap work was calculated to be 41.7 m (138 ft) only 60 ft is adopted in this design.

ii) Volume of embankment. 2 x ½ (6' x 9') x 3' x 110' = 4,950 ft^3
(It is assumed that an embankment of 3 ft in height is carried out from the beginning of the foreapron to the end of the rear apron).

iii) Sheet piles: (6 ft in height x 60'
(7 ft in height x 60'

iv) Wooden piles: dia. = 6" 1 = 5' 16 x 3 = 48 piles

v) Concrete (1 : 2 : 4):

Piers . . . $7 \times ½(10' + 13') \times 5' \times 2' = 805\ ft^3$

Sidewalls . . . $2 \times [(1' \times 5') + (4.5' \times 1') + ½(1' \times 2') + ½(1.5' + 2') \times 0.5'] \times 102'$
$+ 4 \times ½[\qquad\qquad] \times 20' = 2,777\ ft^3$

Total 3,582 ft^3

vi) Concrete (1 : 3 : 6)

Fore apron . . . $[(15' \times 1.5') + ½(1' + 2.5') \times 1.5'] \times 54' = 1,357\ ft^3$

Footing of piers . . . $4' \times 17' \times 63' = 4,284\ ft^3$

Rear apron . . . $[15' \times ½(3' + 2') + (45' \times 2') + 7.5' \times ½(2' + 1.5')$
$+ (1.5' \times 2.5') + ½(2.5' + 1.0') \times 1.0' + (2.5' \times 1.0')] \times 54' =$
$8,026\ ft^3$

Total 13,667 ft^3

vii) Forms

Fore apron . . . $(3' \times 54') + 2 \times [(1.5' \times 15') + ½(2.5' + 1.0') \times 1.5'] = 212\ ft^2$

Footing of piers . . . $2 \times (4' \times 63') + 2 \times (4' \times 17') = 640\ ft^2$

Rear apron . . . $(5' \times 54') + (2.5' \times 54') + 2 \times [15' \times ½(3' + 2') + (45' \times 2') + 7.5' \times$
$½(2' + 1.5') + (2.5' \times 1.5') + 1' \times ½(1' + 2.5') + (2.5' \times 1.0')] =$
$702\ ft^2$

Piers . . . $7 \times ½[(2 \times 8' + 4 \times 1.41') + (2 \times 11' + 4 \times 1.41')] \times 5.0' = 862\ ft^2$

Sidewalls . . . $2 \times (6.5' + 5.24') \times 102' + 4 \times ½(6.5' + 5.24') \times 20' + 4 \times [(1' \times 5')$
$+ (4.5' \times 1') + ½(1' \times 2') + ½(1.5' + 27') \times 0.5'] = 291\ ft^2$

Total 5,327 ft^2

viii) Flash boards

7.8' x 1.0' x 0.5' = 46.8 board ft . . . 3 x 6 = 18 planks . . . 842.4 board ft.

7.8' x 0.8' x 0.5' = 37.4 board ft . . . 1 x 6 = 6 planks . . . 224.4 board ft.

7.8' x 0.75' x 0.5' = 35.1 board ft . . . 1 x 6 = 6 planks. . . 210.6 board ft.

Total 1,278 board ft.

ix) Revetment (Rubble Masonry)

$(31' + 41.5') \times 102' = 7,395\ ft^2$

x) Reinforcement bars

Piers . . . $805\ ft^3 \times 0.002 \times 489\ lbs/ft^3 = 787$ lbs.

Sidewalls . . . $2,777\ ft^3 \times 0.002 \times 489\ lbs/ft^3 = 2,716$ lbs

Total 3,503 lbs.

xi) Rip-raps (Gabion Baskets)

$½(54' + 30') \times 20' + 30' \times 40' = 2,040\ ft^2$

$\fallingdotseq$ 114 baskets (6' x 3' x 3')

1-2) Sluice Gate Weir

i) Volume of cut: (Exclusive of the one for widening the San Juan River)

for Both Sides . . . $2 \times 15' \times 8' \times 110' + 4 \times ½(15' \times 8' \times 20') = 31,200\ ft^3$

Fore apron . . . $70' \times (12.5' \times 1.5' + 2.5' \times 3.0') = 1,838\ ft^3$

Footing of piers . . . $70' \times 20' \times 4' = 5,600\ ft^3$

Rear apron . . . $70' \times (70' \times 2' + 2.5' \times 3.0') - 8' \times 15' \times 2' = 10,085\ ft^3$

Rip-raps (Gabion Baskets)

$½(70' + 50') \times 20' \times 3' + 50' \times 40' \times 3' + 8' \times 15' \times 3' = 9,960\ ft^3$

Total 58,683 ft^3

(Although the rip-raps of 41.7 m (= 138 ft) was calculated from a formula as a necessary length for the movable part, that of 60 ft is adopted in this design).

ii) Volume of backfill: $2 \times 5' \times 8' \times 110' + 4 \times ½(5' \times 8' \times 20') = 10{,}400\ ft^3$

iii) Volume of embankment: $2 \times ½(6' + 9') \times 3' \times 110' = 4{,}950\ ft^3$

(It is assumed that an embankment of 3 ft in height is carried out from the beginning of the fore apron to the end of the rear apron).

iv) Sheet piles: (6 ft in height x <u>60'</u>
(7 ft in height x <u>60'</u>

v) Wooden piles: dia. = 6" 1 = 5' 14 x 4 = <u>56 piles</u>

vi) Concrete (1 : 2 : 4)

Piers . . . $6 \times ½(12' + 16') \times 13' \times 2' = 2{,}184\ ft^3$

Upper frames . . . $[(47' \times 1') + 6 \times (2' \times 5.5') + 10 \times (½ \times 0.5' \times 0.5')] \times 5' = 571\ ft^3$

Sidewalls . . . $[(1' \times 6') + (5.5' \times 1') + ½(1' \times 2') + ½(1.5' \times 2.0') \times 0.5'] \times 2 \times 105'$
$+ 4 \times ½ [\qquad] \times 20' = 3{,}345\ ft^3$

Total <u>6,100 ft^3</u>

vii) Concrete (1 : 3 : 6)

Fore apron . . . $[(15' \times 1.5') + ½(1' + 2.5') \times 1.5'] \times 54' = 1{,}357\ ft^3$

Footing of piers . . . $4' \times 20' \times 63' = 5{,}040\ ft^3$

Rear apron . . . $[15' \times ½(3' + 2') + (45' \times 2') + 7.5' \times ½(2' + 1.5') + (1.5' \times 2.5')$
$+ ½(2.5' + 1.0') \times 1.0' + (2.5' \times 1.0')] \times 54' - [10' \times ½(2' + 1.5') +$
$(5' \times 2')] \times 8' = 7{,}806\ ft^3$

Fixed weir . . . $½(7' + 12') \times 4.55' \times 7' = 303\ ft^3$

Abutment . . . $2.5' \times 4' \times 7' = 70\ ft^3$

Total 14,576 ft^3

viii) Forms

Fore apron . . . $(3' \times 54') + 2[(1.5' \times 15') + ½(2.5' + 1.0') \times 1.5'] = 212\ ft^2$

Footing of piers . . . $2 \times 4' (63' + 20') = 664\ ft^2$

Rear apron . . . $(5' + 2.5') \times 54' + 2[15' \times ½(3' + 2') + (45' \times 2') + 7.5' \times ½(2' + 1.5')$
$+ (2.5' \times 1.5') + 1.0' \times ½(1' + 2.5') + (2.5 \times 1.0)] - [10' \times ½(2' + 1.5')$
$+ (5' \times 2')] = 675\ ft^2$

Piers . . . $6 \times ½[(2 \times 10' + 4 \times 1.41') + (2 \times 14' + 4 \times 1.41')] \times 13.0' = 2{,}312\ ft^2$

Upper frames . . . $[(2 \times 6.5') + 5 \times (5' + 6' + 5' + 2 \times 0.7')] \times 5' + 2 [(47' \times 1') +$
$(6 \times 2' \times 5.5') + (10 \times ½ \times 0.5' \times 0.5')] = 729\ ft^2$

Sidewalls . . . $2 \times (7.5' + 6.24') \times 105' + 4 \times ½(7.5' + 6.24') \times 20' + 4 \times$
$[(1' \times 6') + (5.5' \times 1') + ½(1' \times 2') + ½(1.5' + 2.0') \times 0.5'] = 3{,}490 ft^2$

Fixed weir . . . $4.55' \times 7' = 32\ ft^2$

Abutment . . . $2 \times (2.5' \times 7') + 2 \times (2.5' \times 4') = 55\ ft^3$

Total <u>8,169 ft^2</u>

ix) Gates and accessories:

$5\ \text{sets} \times (7.8' \times 5.45') = 5\ \text{sets} \times 42.5\ ft^2$

x) Revetment (Rubble masonry):
$(31' + 41.5') \times 105' = 7{,}613\ ft^2$

xi) Reinforcement bars:

Piers	$2{,}184\ ft^3 \times 0.004 \times 489\ lbs/ft^3 = 4{,}272\ lbs$
Upper frames	$571 \times 0.004 \times 489 = 1{,}117 lbs$
Sidewalls	$3{,}345 \times 0.002 \times 489 = 3{,}271\ lbs$
Total <u>8,660 lbs</u>	

xii) Rip-raps (Gabion baskets): $\frac{1}{2}(54' + 30') \times 20' + (30' \times 40') + (8' \times 15') = 2{,}160\ ft^2$
= 120 baskets (6' x 3' x 3')

xiii) I-beams:
(D) (B) (Weight)
Size 6" x 3½" x 11.5 lbs
2 x 40' = 80'
80' x 11.5 lbs/ft = 920 lbs

xiv) Planks:
Size 5' x 1' x 2" = 10 board ft.
ro planks x 10 board ft = 400 board ft.

(Remarks)

The above calculations for quantity of materials required for headworks were carried out on the basis of the layout plans attached to the Report (VII).

As mentioned in the Report (IX), however, the sill level of headworks must be lowered from 98 ft to 96 ft as a result of the third survey.

Consequently, the calculation for quantity of materials must also be carried out on the basis of the new improved layout plans. In this report, the old layout plan however, was used for estimating purpose because of no large difference between them.

2) Intake

It consists of Box culvert of 70 ft

Open channel (Transition) of 60 ft Intake gate and screen

i) Volume of cut:

for Box culvert and open channel (exclusive of volume of cut of the bank)
$\frac{1}{2}(13' + 5') \times 4' \times (70' + 60') = 4{,}680\ ft^3$

Box culvert under the bank
$\frac{1}{2}(5' + 30') \times 6' \times 11' = 1{,}155\ ft^3$

Intake gate $\frac{1}{2}(20' + 10') \times 5' \times \frac{1}{2}(4' + 14') = 675\ ft^3$ Total $6{,}510\ ft^3$

ii) Volume of backfill: $6{,}510 - [(4.5' \times 3.5' \times 70') + \frac{1}{2}(4' + 3.5') \times 3.5' \times 60' + (4' \times 9' \times 1') + (4' \times 1.5' \times 2')] = 4{,}571\ ft^3$

iii) Concrete (1 : 2 : 4)

Box culvert $(4.5' \times 3.4' - 3' \times 2') \times 70' = 675\ ft^3$

Open channel $\frac{1}{2}[(4.2' \times 3.3' - 3 \times 2.58) + (3.7 \times 3.3' - 2.5' \times 2.5')] \times 60' = 370\ ft^3$

Structure for gate $(2' \times 7' \times 1') + 2(1.2' \times 2' \times 7') + (9' \times 4' \times 1') = 84\ ft^3$
Overhanging sill $\frac{1}{2}(1' + 2') \times 1' \times 46' = 7\ ft^3$
Total $\underline{1,136\ ft^3}$

iv) Blinding concrete (1 : 4 : 8):
$[(4' \times 9') + (5' \times 70') + \frac{1}{2}(5' + 4') \times 60'] \times 0.3' = 197\ ft^3$

v) Forms:
Box culvert $[2(3.4' + 2') + 3'] \times 70' = 2,268\ ft^2$
Open channel $2(3.3' + 2.5') \times 60' = 696\ ft^2$
Structure for gate $2(4' + 9') \times 1' + 4(1.2' + 2') \times 7' + 2(2' + 7') \times 1' = 144\ ft^2$
Overhanging sill $2 \times \frac{1}{2}(1' + 2') \times 1' + (2.4' \times 4.6') = 14\ ft^2$
Total $\underline{3,122\ ft^2}$

vi) Reinforcement bars:
Box culvert and structure for gate
$(675\ ft^3 + 84\ ft^3) \times 0.004 \times 489\ lbs/ft^3 = 1,485\ lbs$
Open channel and overhanging sill
$(370\ ft^3 + 7\ ft^3) \times 0.002 \times 489\ lbs/ft^3 = 369\ lbs$
Total $\underline{1,854\ lbs}$

vii) Gate and accessories: 1 set x $(4.2' \times 2.5')$ = $\underline{1\ set \times 10.5\ ft^2}$
viii) Screen: 1 set x $(3' \times 2')$ = $\underline{1\ set \times 6\ ft^2}$

3) Main and Branch Canals (cf. Report V)
3-1 Main canals

			(l) (b) (h)
A	220m	=	722' x 2.5' x 2.0'
B	360m	=	1,181' x 2.3' x 1.7'
C	360m	=	1,181' x 2.2' x 1.3'
E	338m	=	1,109' x 1.5' x 1.0'

l: Length of canal
b: Breadth of canal
h: Depth of canal

i) Volume of cut:
A . . . $722' \times \frac{1}{2}(4' + 9') \times 2.5' = 11,733\ ft^3$
B . . . $1,060' \times \frac{1}{2}(4' + 11') \times 3.5' = 27,825\ ft^3$
Total $\underline{39,558\ ft^3}$

ii) Volume of fill
B . . . $121' \times \frac{1}{2}(6' + 10') \times 2' = 1,936\ ft^3$
C . . . $1,181' \times \frac{1}{2}(6' + 9') \times 1.5' = 13,286\ ft^3$
E . . . $1,109' \times \frac{1}{2}(6' + 9') \times 1.5' = 12,476\ ft^3$
Total $\underline{27,698\ ft^3}$

iii) Volume of backfill:
A . . . $11,733\ ft^3 - 722' \times (2.5' + 2 \times 0.6') \times 2.5' = 5,054\ ft^3$
B . . . $27,825\ ft^3 - 1,060' \times (2.3' + 2 \times 0.6') \times 2.5' = 18,550\ ft^3$
Total $\underline{23,604\ ft^3}$

iv) Concrete (1 : 2 : 4):
A . . . $722' \times [(2 \times 0.6' \times 2.7') + (3.7' \times 0.6')] = 3,942\ ft^3$
B . . . $1,181' \times [(2 \times 0.6' \times 2.2') + (3.5' \times 0.6')] = 5,598\ ft^3$
C . . . $1,181' \times [(2 \times 0.6' \times 1.8') + (3.4' \times 0.6')] = 4,960\ ft^3$
E . . . $1,109' \times [(2 \times 0.5' \times 1.3') + (2.5' \times 0.5')] = 2,828\ ft^3$
Total $\underline{17,328\ ft^3}$

v) Blinding concrete (1 : 4 : 8):

A . . . 3.7' x 0.3' x 722' = 801 ft^3
B . . . 3.5' x 0.3' x 1,181' = 1,240 ft^3
C . . . 3.4' x 0.3' x 1,181' = 1,205 ft^3
E . . . 2.5' x 0.3' x 1,109' = 832 ft^3
Total 4,078 ft^3

vi) Forms:

A . . . 2 x (2.7' + 3.3') x 722' = 8,664 ft^2
B . . . 2 x (2.2' + 2.9') x 1,181' = 12,046 ft^2
C . . . 2 x (1.8' + 2.4') x 1,181' = 9,920 ft^2
E . . . 2 x (1.3' + 2.0') x 1,109' = 7,319 ft^2
Total 37,949 ft^2

vii) Gates and accessories:

	(b)	(h)
A → B	1 set x 2.8'	x 2.2'
B → C	1 set x 2.7'	x 1.8'
C → E	1 set x 2.0'	x 1.3'

Free boards of concrete canals were determined on the basis of the following assumptions:-

1) One third of the maximum depth as a rule
2) 0.3' in depth is adopted as the minimum value.

The earth requirement for fill for main canals as well as the branch canals can be obtained from a higher portion of land near the main canal B.

3-2) Branch Canals

	(ℓ)		(b)	(h)
D_1	378m = 1,240 ft	x	1.5'	x 1.3'
D_2	378m = 1,240 ft			
E_1	378m = 1,240 ft	x	1.5'	x 1.0'
E_2	378m = 1,240 ft			
E_3	378m = 1,240ft			
E_4	378m = 1,240ft			
F_1	457m = 1,499 ft	x	1.0'	x 1.0'
F_2	386m = 1,266 ft			
F_3	331m = 1,086 ft			
F_4	294m = 965 ft			

i) Volume of cut:

E_1 . . . 300' x ½(3' + 6') x 1.5' = 2,025 ft^3
F_1 . . . 900' x ½(2.5' + 4.5') x 1.0' = 3,150 ft^3
Total 5,175 ft^3

ii) Volume of fill:

D_1 ... $1,240' \times ½(3' + 5') \times 1.0' = 4,960 \text{ ft}^3$
D_2 ... $1,240' \times ½(3' + 6') \times 1.5' = 8,370 \text{ ft}^3$
E_1 ... $940' \times ½(3' + 6') \times 1.5' = 6,345 \text{ ft}^3$
E_2 ... $1,240' \times ½(3' + 5') \times 1.0' = 4,960 \text{ ft}^3$
E_3 ... $1,240' \times ½(3' + 6') \times 1.5' = 8,370 \text{ ft}^3$
E_4 ... $1,240' \times ½(3' + 5') \times 1.0' = 4,960 \text{ ft}^3$
F_1 ... $599' \times ½(2.5' + 4.5') \times 1.0' = 2,126 \text{ ft}^3$
F_2 ... $1,266' \times ½(2.5' + 3.5') \times 0.5' = 1,899 \text{ ft}^3$
F_3 ... $480' \times ½(2.5' + 4.5') \times 1.0' = 1,680 \text{ ft}^3$
F_4 ... $965' \times ½(2.5' + 5.5') \times 1.5' = 5,790 \text{ ft}^3$
Total $49,460 \text{ ft}^3$

Note:- Cutting or filling is not necessary for 606 ft in F_3

iii) Volume of backfill:
E_1 ... $2,025 \text{ ft}^3 - 300' \times (1.5' + 2 \times 0.5') \times 1.5' = 900 \text{ ft}^3$
F_2 ... $3,150 \text{ ft}^3 - 900' \times (1.0' + 2 \times 0.5') \times 1.0' = 1,350 \text{ ft}^3$
Total $2,250 \text{ ft}^3$

iv) Concrete (1 : 2 : 4):
D_1 ... $1,240' \times (2 \times 0.5' \times 1.8') + (2.5' \times 0.5') = 3,782 \text{ ft}^3$
D_2 ... $1,240' \times (2 \times 0.5' \times 1.8') + (2.5' \times 0.5') = 3,782 \text{ ft}^3$
E_1 ... $1,240' \times (2 \times 0.5' \times 1.3') + (2.5' \times 0.5') = 3,162 \text{ ft}^3$
E_2 ... $1,240' \times (2 \times 0.5' \times 1.3') + (2.5' \times 0.5') = 3,162 \text{ft}^3$
E_3 ... $1,240' \times (2 \times 0.5' \times 1.3') + (2.5' \times 0.5') = 3,162 \text{ ft}^3$
E_4 ... $1,240' \times (2 \times 0.5' \times 1.3') + (2.5' \times 0.5') = 3,162 \text{ ft}^3$
F_1 ... $1,499' \times (2 \times 0.5' \times 1.3') + (2.0' \times 0.5') = 3,448 \text{ ft}^3$
F_2 ... $1,266' \times (2 \times 0.5' \times 1.3') + (2.0' \times 0.5') = 2,912 \text{ ft}^3$
F_3 ... $1,086' \times (2 \times 0.5' \times 1.3') + (2.0' \times 0.5') = 2,498 \text{ ft}^3$
F_4 ... $965' \times (2 \times 0.5' \times 1.3') + (2.0' \times 0.5') = 2,220 \text{ ft}^3$
Total $31,290 \text{ ft}^3$

v) Blinding concrete (1 : 4 : 8):
D_1, D_2, E_1, E_2, E_3 and E_4 $6 \times 1,240' \times 2.5' \times 0.3' = 5,580 \text{ ft}^3$
F_1, F_2, F_3 and F_4 $(1,499' + 1,266' + 1,086' + 965') \times 2.0' \times 0.3' = 2,890 \text{ ft}^3$
Total $8,470 \text{ ft}^3$

vi) Forms:
D_1, D_2 ... $2 \times 1,240' \times 2 \times (1.8' + 2.3') = 20,336 \text{ ft}^2$
E_1, E_2, E_3 and E_4 ... $4 \times 1,240' \times 2 (1.3' + 1.8') = 30,752 \text{ ft}^2$
F_1, F_2, F_3 and F_4 ... $(1,499' + 1,266' + 1,086' + 965') \times 2 \times (1.3' + 1.8') = 29.859 \text{ ft}^2$
Total $80,947 \text{ ft}^2$

vii) Gate and accessories:
$\left.\begin{array}{l} B \rightarrow D_1 \\ C \rightarrow D_2 \end{array}\right\}$ 2 sets x 20' x 1.8'
$\left.\begin{array}{l} A \rightarrow E_1 \\ E \rightarrow E_2 \\ D_1 \rightarrow E_3 \\ D_2 \rightarrow E_4 \end{array}\right\}$ 4 sets x 20' x 1.3'

$$\left.\begin{array}{l} E_1 \rightarrow F_1 \\ E_2 \rightarrow F_2 \\ E_3 \rightarrow F_3 \\ E_4 \rightarrow F_4 \end{array}\right\} \text{4 sets x 1.5' x 1.3'}$$

4) Intake Pipes into Furrows

Rigid plastic siphon pipes

q = 1.02 l/s (= 0.036 cusecs)

d = 5 cm (= 2") (Assuming that head of 0.5 ft can be given)

10 pipes/block x 10 block = 100 pipes

(Reason)

Although an appropriate run (length of ridge) must be determined after a test for furrow irrigation at the area concerned, a maximum run can be assumed to be 180 m (= 590 ft) which is half as long as the maximum interval of the branch canals, because the longitudinal slope of the ridge is nearly zero and the ridge consists of silty clay loam.

Subsequently, the head ditch will be unnecessary for this area, if each furrow can be supplied with water directly by the branch canal.

In Caroni near the Aranguez area, furrow irrigation has already been implemented and the interval of ridge or furrow is 4' to 6'.

6 ft (= 1.8 m) is therefore adopted as the interval of furrow (b) in this area.

∴ Maximum length of block: L_{max} = 378 m = 1,240 ft

Minimum time interval of furrow irrigation: 7 days (cf Report V)

Length required to be irrigated in one day (l):

$l = L_{max}/7 \text{ days} = 378 \text{ m}/7 \text{ days} = 54 \text{ m/day} \fallingdotseq 180 \text{ ft/day}$

Number of ridges possible to be irrigated in one day (N):

$$N = l/b = \frac{54\text{m} (= 180\text{ft})}{1.8\text{m} (= 6 \text{ ft})} = 30$$

Assuming that time required to irrigate a furrow is 8 hours, (cf. Report V). number of furrows to be irrigated at the same time in a block will be.

$n = 30 \div (24 \text{ hours} \div 8 \text{ hours}) = 10$

(cf. Report V)

Total readily available moisture: T.R.A.M. = 35.5 mm

Assuming that loss of seepage into deeper layer is 15%, gross water requirement will be,

$$Q = \frac{35.5 \text{ m}}{1{,}000} \times \frac{1}{(1 - 0.15)} \times \frac{1}{7 \text{ days}} \times 1.08 \times 334 \text{ acres} \times 4{,}047 \text{ m}^2/\text{acres}$$

$= 8{,}700 \text{ m}^3/\text{day} = 0.101 \text{ m}^3/\text{sec} = 3.57 \text{ cusecs}$

(* losses of water conveyance and diversion)

As the water quantity possible to be supplied from the San Juan River is 0.105 m^3/sec (=3.71 cusecs), it is enough for the gross water requirement.

Gross water requirement for one standard block will be,

$$Qs = 0.101\ m^3/sec \times \frac{(378\ m \times 360\ m)}{334\ acres} \times 4,047\ m^2/acre = 0.0102\ m^3/sec$$
$$= 10.2\ l/sec$$

Therefore gross water requirement for one standard furrow will be.

$$Qs/n = 10.2\ l/sec \times \frac{1}{10} = 1.02\ l/sec\ (= 0.036\ cusec)$$

5) Roads in the Area

Building new roads in this area is being considered in this report, excluding the access road to the proposed headworks which is already available.

(Length)

i) Main road along main canal A.B.C and E – 722' + 1,181' + 1,181' + 1,109' = 4,193'
ii) Branch road along branch canal E, F – 1,240' + 1,499' = 2,739'
iii) Branch road along branch canal D_1, E_3, F_3 – 1,240' + 1,240' + 1,086' = 3,566'
iv) Branch road along branch canal D_2, E_4, F_4 – 1,240' + 1,240' + 965' = 3,445'
v) Branch road along branch canal E_2, F_2 – 1,240' + 1,266' = 2,506'
vi) Branch road on the boundary between E_1, D_1, D_2, E_2 and F_1, E_3, E_4, F_2 – 4,400'
vii) Branch road on the boundary between E_3, E_4 and F_3, F_4 – 2,400'

(Breadth)

i) *Overall breadth B = 15'*
ii) - vii) Overall breadth B = 10'

(Elevation)

It is assumed that the surface levels of the roads along the irrigation canals are 1 ft higher than the sill levels of the canals, and the surface levels of the roads not along the irrigation canals are 2 ft higher than field level.

(cf. A-1, 3-1 and 3-2)

Volume of cut:

i) B . . . ½(15' + 18') x 1.5' x 722' = 17,870 ft^3
C . . . ½(15' + 20') x 2.5' x 1,060' = 46,375 ft^3
ii) E_1 . . ½(10' + 11') x 0.5'x 300' = 1,575 ft^3
Total 165,820 ft^3

Volume of fill with transported earth:

i)	B . . .	½(15' x 21') x 3.0' x 121' = 6,534 ft^3
	C . . .	½(15' + 20') x 2.5' x 1,181' = 51,669
	E . . .	½(15' + 20') x 2.5'x 1,109' = 48,519
ii)	E_1 . . .	½(10' + 15') x 2.5' x 940' = 29,375
	F_1 . . .	½(10' + 14') x 2.0' x 599' = 14,376
iii)	D_1 . . .	½(10' + 14') x 2.0' x 1,240' = 29,760
	E_3 . . .	½(10' + 15') x 2.5' x 1,240' = 38,750
	F_3 . . .	½(10' + 13') x 1.5' x 1,086' = 18,734
iv)	D_2 . . .	½(10' + 15') x 2.5' x 1,240' = 38,750
	E_4 . . .	½(10' + 14') x 2.0' x 1,240' = 29,760
	F_4 . . .	½(10' + 15') x 2.5' x 965' = 30,156
v)	E_2 . . .	½(10' + 14') x 2.0' x 1,240' = 29,760
	F_2 . . .	½(10' + 13') x 1.5'x 1,266' = 21,839
vi)		½(10' + 14') x 2.0' x 4,400' = 105,600
		½(10' + 14') x 2.0' x 2,400' = 57,600
		Total 551,182 ft^3

(Note: Cutting or filling is not necessary for 900 ft in F.)

A-2) Sprinkler Irrigation
1) Headworks
1-1) Flash Board Weir cf. A-1) 1-1)
1-2) Sluice Gate Weir cf. A-1) 1-2)
2) Intake cf. A-1) 2)
3) Main and Branch Canal cf. A-1) 3-1) 3-2)
4) Facilities for Sprinkling cf. Report (V)

10 sets of equipment and instruments on the following list.

Items	Specifications	No.	Remarks
Portable Pump	Centrifugal 10.4 P.S.	1	with engine hose and strainer
Pipes	3' x 20'	60	
Riser pipes	1.5' - 2.5'	30	for vegetable
Joints for pipes		60	with simple connecting devices
Joints for riser pipes		30	
Sprinkler heads	35 lb/m^2 7.1 gal/min.	30	
Connecting hose	10' - 15'	1	
Manometer		1	
Elbows, tees		a few	
End tap		1	

B. Drainage Scheme

1) Widening of the San Juan River

i) Volume of cut:

Assuming that a river cross sectional area of No. 20 is required as an average one,

$$\underset{(L)}{5{,}284\ ft} \times \underset{(A)}{127.5\ ft^2} = 673{,}710\ ft^3 = 24{,}950\ yd^3\ (= 19{,}080\ m^3)$$

L: Distance of the San Juan River from the bridge on Churchill Roosevelt highway to its mouth

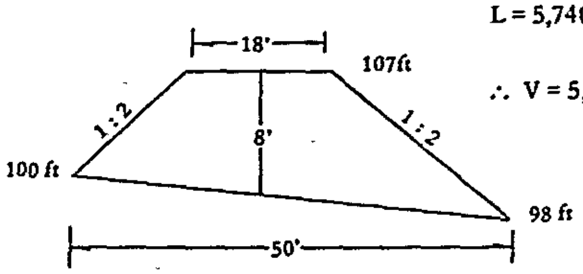
A: The cross sectional area (No. 20) to be cut

2) Embankment against the Caroni River

This work must be implemented not only for Aranguez but also for other lower lands along the river.

In this report, volume of embankment for only Aranguez Estate, however, will be calculated that is, volume of bank from the mouth of the San Juan River to the mouth of Mullins Canal along the Caroni River.

i) Volume of bank:



$A = ½(18' + 50') \times 8' = 272\ ft^2$

$L = 5{,}740'$

$$\therefore\ V = \underset{(L)}{5{,}740\ ft} \times \underset{(A)}{272 ft^2} = 1{,}561{,}280\ ft^3 = 57{,}825\ yd^3\ (=44{,}210 m^3)$$

3) Installation of pumps

i) Number, and scale of pumps:
2 sets of axial flow pumps 800 mm (=2.6') in diameter and 85 HP in axial horse power

ii) Foundation works:
intakes, screens and sumps (2) two each
(dimension of sump . . . 40' by 10' and 12' high)

iii) Pump house: 30' by 35' and 23' high

iv) Transmission line:

electric power required for pumps . . . 2 x 85kW = 170kW
length of transmission line . . . . about 0.8 mile

4) Installation of control gates

i) Volume of cut: ½(20' + 32') x 6' x 60' = 9,360 ft^3

ii) Volume of backfill:

9,360 ft^3 - (19.1' x 5.5') x 60'= 3,057 ft^3

iii) Concrete (1 : 2 : 4): Cutoff walls — 2[(23.1' x 9.5') - (19.1' x 5.5')] x 1' = 229 ft^3
(assuming that two cut-off walls of 2 feet wide and 1 foot thick are attached around box culverts)

Culverts . . (19.1' x 5.5' - 3 x 4.7' x 3') x 60' = 3,768 ft^3

Total 3,997 ft^3

iv) Forms:

Cut-off walls . . . 4[(23.1' x 9.5') - (19.1' x 5.5')] + 4 x (9.5' x 1') = 496 ft^2

Culverts . . . [3(2 x 3' + 4.7') + (2 x 5.5') x 60' + 2 (19.1' x 5.5') - (3 x 4.7' x 3.0')] = 2,709 ft^2

Total <u>3,205 ft^2</u>

v) Reinforcement bars:

Cut-off walls and culverts . . . 3,997 ft^3 x 0.004 x 489 lbs/ft^3 = 7,818 lbs

vi) Wooden piles: d = 6' l = 10' 11 x 4 = 44 piles (spaced 6' by 6')

vii) Gates and accessories:

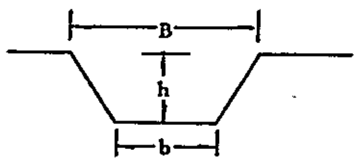
3 sets x 3.5' x 5'

5) Drains in the area (cf Report V and Plan)

It is assumed that drains are excavated on the boundary line of each block.

i)	Perimeter drain inside and along the embankment	9,900 ft
ii)	North to South drain between E_1, F_1 and D_1, E_3, F_3	2,600
iii)	North to South drain between D_1, E_3, F_3 and D_2, E_4, F_4	3,500
iv)	North to South drain between D_2, E_4, F_4 and E_2, F_2	2,800
v)	East to West drain between E_1, D_1, D_2, E_2 and E_3, E_4	4,400
vi)	East to West drain between E_3, E_4 and F_3, F_4	2,400

No.	Total length	Type A	Type B	Type C
i)	9,900 ft	2,650 ft	2,800 ft	4,450 ft
ii)	2,600	1,240	1,360	-
iii)	3,500	1,240	2,260	-
iv)	2,800	1,240	1,560	-
v)	4,400	4,400	-	-
vi)	2,400	2,400	-	-
Total	25,600 ft	13,170 ft	7,980 ft	4,450 ft



	b	B	h	A
Type A	2 ft	8 ft	3 ft	15 ft^2
Type B	4	10	3	21
Type C	5	20	5	62.5

Volume of Cut:

Type A 13,170ft x 15ft^2 = 197,550ft^3
Type B 7,980 x 21 = 167,580
Type C 4,450 x 62.5 = 278,125

Although it was stated in the report (VI) that the existing Mullin Canal would be able to be used as a main drainage canal, after the drainage scheme would have been carried out, new drainage canals are proposed in this report, because the old canal impedes furrow irrigation and systematic drainage scheme. Subsequently the old canal must be abolished and be filled.

Estimate of the Cost for the works:

A. Irrigation scheme

A-1) Furrow Irrigation

1) Headworks

1-1) Flash Board Weir

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
i	Cut	57,723ft³ = 2,138	yd³	8¢	$ 171	by machinery
ii	Backfill	10,400ft³ = 385	yd³	5¢	$ 19	by machinery
iii	Embankment	4,950ft³ = 183	yd³	5¢	9	by machinery
iv	Sheet Piles	6' x 60	ft	$ 16.00	960	
		7' x 60	ft	$ 20.00	1,200	
v	Wooden piles	(6"d x 5') 48	piles	$ 6.00	288	
vi	Concrete (1:2:4)	3,582ft³ = 133	yd³	$ 32.00	4,256	
vii	Concrete (1:3:6)	13,667ft³ = 506	yd³	$ 29.50	14,927	
viii	Forms	5,327	ft²	36¢	1,918	
ix	Flash boards	1,278	board ft	30¢	384	
x	Rubble masonry	7,395ft² = 822	yd²	$ 7.67	6,305	9" - 12" thick
xi	Reinforcement bars	3,503	lbs	35¢	1,226	
xii	Gabion baskets	(6' x 3' x 3') 114	baskets	$ 51.00	5,814	
	Total				$37,477	

1-2) Sluice Gate Weir

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
i	Cut	58,683ft³ = 2,173	yd³	8¢	$ 173	by machinery
ii	Backfill	10,400ft³ = 385	yd³	5¢	19	by machinery

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
iii	Embankment	4,950ft^3 = 183	yd^3	5¢	9	by machinery
iv	Sheet piles	6' x 60	ft	$ 16.00	960	
		7' x 60	ft	$ 20.00	1,200	
v	Wooden piles	(6'd x 5')				
		= 56	piles	$ 6.00	336	
vi	Concrete (1 : 2 : 4)	6,100ft^3				
		=226	yd^3	$ 32.00	7,232	
vii	Concrete (1 : 3 : 6)	14,576ft^3				
		= 540	yd^3	$ 29.50	15,930	
viii	Forms	8,169	ft^2	36¢	2,941	
ix	Gates and accessoires	(7.8' x 5.45')				
		= 5	sets	$ 2,500.00	12,500	
x	Rubble masonry	7,613ft^2				
		= 846	yd^2	$ 7.67	6,489	9" - 12" thick
xi	Reinforcement bars	8,660	lbs	35¢	3,031	
xii	Gabion baskets	(6' x 3' x 3')				
		= 120	baskets	$ 51.00	6,120	
xiii	I-beams	920	lbs	60¢	552	
xiv	Planks	400	board ft	30¢	120	
	Total				$57,612	

2) Intake

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
i	Cut	6,510ft^3				
		= 241	yd^3	$ 1.65	398	by manual labour
ii	Backfill	4,571ft^3				
		= 169	yd^3	$ 2.20	372	by manual labour
iii	Concrete (1 : 2 : 4)	1,136ft^3 = 42	yd^3	$ 32.00	1,344	
iv	Concrete (1 : 4 : 8)	197ft^3 = 7	yd^3	$ 25.00	175	
v	Forms	3,122	ft^2	36¢	1,124	
vi	Reinforcement bars	1,854	lbs	35¢	649	
vii	Gate and accessories	(4.2' x 2.5')				
		= 1	set	$ 450.00	450	
viii	Screen	(3' x 2')				
		= 1	set	$ 15.00	15	
	Total				$ 4,527	

3) Main and Branch canals

3-1) Main canals

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
i	Cut	$39,558ft^3$=1,465	yd^3	10¢	$ 147	by backhoe
ii	Fill with transported earth	$27,698ft^3$=1,026	yd^3	35¢	359	mean distance 700 yds
iii	Backfill	$23,604ft^3$=874	yd^3	$ 2.20	1,923	by manual labour
iv	Concrete (1 : 2 : 4)	$17,328ft^3$=642	yd^3	$ 32.00	20,544	
v	Concrete (1 : 4 : 8)	$4,078ft^3$=151	yd^3	$ 25.00	3,775	
vi	Forms	37,949	ft^2	36¢	13,662	
vii	gates and accessories	(2.8' x 2.2') 1	set	$ 300.00	300	
		(2.7' x 1.8') 1	set	$ 250.00	250	
		(2.0' x 1.3') 1	set	$ 150.00	150	
	Total			$	$41,110	

3-2) Branch canals

No.	Items	Quantities	Units	Unit costs	Costs	Remarks
i	Cut	$5,175ft^3$=192	yd^3	10¢	$ 19	by backhoe
ii	* Fill with transported earth	$49,460ft^3$=1,832	yd^3	40¢	733	mean distance 1,500 yds
iii	Backfill	$2,250ft^3$=83	yd^3	$ 2.20	183	by manual labour
iv	Concrete (1 : 2 : 4)	$31,290ft^3$=1,159	yd^3	$ 32.00	37,088	
v	Concrete (1 : 4 : 8)	$8,470ft^3$=314	yd^3	$ 25.00	7,850	
vi	Forms	80,947	ft^2	36¢	29,141	
vii	Gates and accessories	(2.0' x 1.8') 2	sets	$ 200.00	400	
		(2.0' x 1.3') 4	sets	$ 150.00	600	
		(1.5' x 1.3') 4	sets	$ 125.00	500	
	Total				$76,514	

* A bulldozer, a tractor - shovel and dump-trucks must be used for transportng earth.

4) Intake-pipes into Furrows.

100 pipes x $5.00 = $500

5) Roads in the Area:

Cut: $65,820ft^3$ = $2,438yd^3$ $2,438yd^3$ x 8¢ = $195.
Fill with transported earth:
$551,182ft^3$ = 20,416 yd^3 $20,416yd^3$ x 40¢ = $8,166
Total $8,361

6) Summary of the Estimate for
A-1) Furrow Irrigation

i) Case I:

Headworks ... Flash weir ...	$ 37,477
Intake	4,527
Main canals	41,110
Branch canals	76,514
Intake-pipes into furrows	500
Roads	8,361
Total	$ 168,489 T.T

ii) Case II:

Headworks ... Sluice gate weir ...	$ 57,612
Intake	4,527
Main canals	41,110
Branch canals	76,514
Intake-pipes into furrows	500
Roads	8,361
Total	$ 188,624 T.T

(Exclusive of the costs for depreciation and adjustment of machineries to be used for works)

A-2) Sprinkler Irrigation

1) Headworks
1-1) Flash Board Weir cf. A-1) 1-1)
1-2) Sluice Gate Weir cf. A-1) 1-2)
2) Intake cf. A-1) 2)
3) Main and Branch canals cf. A-1) 3-1) 3-2)
4) Facilities for sprinkling

For one set of equipment and instruments

No.	Items	Quantities	Units	Unit Costs	Costs	Remarks
i	Centrifugal Pump and Engine	1	sets	$ 3,300	$ 3,300	with suction hose and trolley
ii	Pipes (ϕ 3" x 20')	60		$ 33.31	1,999	with simple, connecting devices
iii	Riser pipes (ϕ 1" x 2')	30		$ 5.00	150	
iv	Sprinkler heads	30		$ 12.15	365	
v	Connecting hose	1		$ 30.00	30	
vi	Manometer	1		$ 14.00	14	
vii	Elbows Tees	a few			20	
viii	End Tap	1		$ 10.00	10	
	Total				For one set $ 5,888	∴ for 10 sets $58,880

5) Roads in the Area cf. A-1) 5)

6) Summary of the Estimate for Sprinkler Irrigation

i)*	Case I:	Headworks . . . Flash Board weir . . .	$	37,477
		Intake		4,527
		Main canals		41,110
		Branch canals		76,514
		Facilities for sprinkling		58,880
		Roads		8,361
		Total	$	226,869 T.T.
ii)	Case II:	Headworks . . . Sluice gate weir	$	57,612
		Intake		4,527
		Main canals		41,110
		Branch canals		76,514
		Facilities for sprinkling		58,880
		Roads		8,361
		Total	$	247,204 T.T.

(* Exclusive of the costs for depreciation and adjustment of machineries to be used for the works)

B) Drainage Scheme

1) Widening of the San Juan River

Cut: 24,950 yd^3 x 8¢ = $1,996 (by dragline)

2) Embankment against the Caroni River

This embankment will be built up with the earth which is to be cut for drainage canal inside and along the embankment.

Embankment: 57,825 yd^3 x 5¢ = $2,891

3) Installation of pumps

No.	Items	Quantities	Units	Unit Costs	Costs	Remarks
i	Axial flow pumps and accessories	2	sets	$30,000	$60,000	Ø 800 mm 85HP
ii	Foundation works	1		$50,000	50,000	cf. Report 'Oropouche IV'
iii	Pump house	30' x 35' x 23' = 117	yd^2	$ 200	23,400	
iv	Transmission lines and accessories	0.8	mile	$13,000	10,400	170kW
	Total				$143,800	

4) Installation of Control Gates

No.	Items	Quantities	Units	Unit Costs	Costs	Remarks
i	Cut	9,360ft^3=347	yd^3	8¢	$ 28	
ii	Backfill	3,057ft^3=113	yd^3	5¢	6	
iii	Concrete (1 : 2 : 4)	3,997ft^3=148	yd^3	$ 32.00	4,736	
iv	Forms	3,205	ft^2	36¢	1,154	
v	Reinforcement bars	7,818	lbs	35¢	2,736	
vi	Wooden piles	d=6" l=10' 44	piles	$ 11.00	484	
vii	Gates and accessories	3.5' x 5.0'–3	sets	$ 1,200	3,600	
	Total				$12,744	

5) Drains in the Area

Cut:	Type A	197,550ft^3 =	7,317yd^3
	Type B	167,580ft^3 =	6,207
	Type C	278,125ft^3 =	10,302
	Total		23,826yd^3

Cost (by backhoe) 23,826yd^3 x 10¢ = $2,383

6) *Summary of the Estimate for Drainage Scheme

Widening of the San Juan River	$	1,996
Embankment against the Caroni River		2,891
Installation of Pumps		143,800
Installation of Control Gates		12,744
Drains in the Area		2,383
Total	$	163,814

* (Exclusive of the costs for depreciation and adjustment of machineries to be used for the works)

Conclusion

If 10% of total cost for irrigation and drainage scheme is added as contingency.

A-1) the cost for furrow irrigation will be

(Case I) $168,489 x 1.1 ≒ $185,000 T.T
(Case II) $188,624 x 1.1 ≒ $207,000 T.T

A-2) the cost for sprinkler irrigation will be

(Case I) $226,869 x 1.1 ≒ $249,000 T.T
(Case II) $247,204 x 1.1 ≒ $271,000 T.T

B) The cost for drainage scheme will be

$163,814 x 1.1 ≒ $180,000 T.T.

The costs for preparing the land for furrow or sprinkler irrigation are not included in above total.

Supplement

Running (operating and maintaining) and depreciating cost for sprinkler irrigation will be stated here.

1) Operating costs:

Labour: 3 pump-operators for 3 shifts in one day

3 x $7.57 = $22.71 ... (1)

4 labourers for movement of facilities

$4 \text{ labourers} \times \frac{2\ \text{hr}}{8\ \text{hr}} \times 3 \text{ times} = 3.0 \text{ labourers}$

3 x $6.61 = $19.83 ... (2)

Materials: $20 \text{ hours a day } \frac{0.3\ \ell}{\text{H.P.} \times \text{hr}} \times 11\ \text{H.P} \times 20\ \text{hr} = 66\ \ell = 14.5 \text{ gals}$

fuel (light oil) 14.5 gals x 47 ¢ /gal = $6.82

engine oil $(\frac{1}{30})$ 0.44 gals x $1.14¢ /gal = 0.50

gasoline $(\frac{1}{180})$ 0.26 gals x 54¢ /gal = 0.14

grease 0.4 lb x 45¢ /lb = 0.18

others (3% of above total) = 0.23

$ 7.87 / ... (3)

Total (1) + (2) + (3) = $50.41 / 1 set

2) Maintaining costs:

		(A)	(B)	(C)	(D)	(E)	$(F) = \frac{(A) \times (E)}{(B)}$
No.	Items	New Costs	Economic durable periods of time	Total rates for daily adjustment	Total rates for periodical adjustment	(C) + (D)	Costs for adjustment a day
i	Pump	$ 1,100	$\frac{8{,}000\text{hr}}{20\ \text{hr}}$ = 400 days	0.80	-	0.80	$ 2.20
ii	Engine	$ 1,700	$\frac{8{,}000\text{hr}}{20\ \text{hr}}$ = 400 days	0.40	1.0	1.40	$ 5.95
iii	Trolley	$ 500	3,000 days	0.50	-	0.50	$ 0.08
iv	Pipes	$ 2,600	3,000 days	0.30	-	0.30	$ 0.26
	Total						$ 8.49

3) Running cost for sprinkler irrigation

Running cost for sprinkler irrigation will be,

\$50.41 + \$8.49 = \$58.90/1 set 1 day
(operating (maintaining
cost) cost)

For 10 sets in the whole area,

10 x \$58.90 = \$589.00/day

4) Depreciating cost for sprinkler irrigation

		(A)	(B)	(C)	(D) = (A)/(B) x (C)	
No.	Items	New Costs	Economic durable periods of time	Total rate for depreciation	Cost for deprecia-tion	Remarks
i	Pump	\$ 1,100	$\frac{8,000hr}{20 hr}$ = 400 days	0.90	\$ 2.48	
ii	Engine	\$ 1,700	$\frac{8,000hr}{20 hr}$ = 400 days	0.90	\$ 3.83	
iii	Trolley	\$ 500	3,000 days	0.90	\$ 0.15	
iv	Pipes	\$ 2,600	3,000 days	0.90	\$ 0.78	
	Total				\$ 7.24	

As calculated above, depreciating cost for sprinkler irrigation is

\$7.24 / 1 set 1 day

For 10 sets in the whole area

10 x \$7.24 = \$72.40 / day

In conclusion it must be stated that in order to determine unit costs, information were given by Mr. C. Marshall, Mr. I. Ramsawack, Engineering Assistant II, in Drainage Division, Mr. Massiah, Electrical Engineer, T & TEC, and Mr. R. Abreu, in Tractors & Machinery (Trinidad) Ltd.

Junichi Kitamura

(2) TUCKER VALLEY (I) 10th May 1967

INTRODUCTION

My movement as to this Tucker Valley plan is as follows:-

April 24th - Visited there to grasp the general condition with Mr. Mc.D.Gibbs Drainage Engineer, and Mr. J.C. Douglas, Technical Officer, was explained on a sketch at the branch office of the Ministry of Agriculture.

Visited the Design Section in the U.S. Naval Base after looking at the area and was shown the detailed map.

April 27th - Attended the Conference on Tucker Valley at the Ministry of Agriculture and discussed the outline plans for the area. Attendants were:-

Mr. J. C. Douglas:	Technical Officer, Town and Country Planning Division, Ministry of Finance, Planning & Development
Dr. C. Brown:	Land Capabilities, U.W.I.
Mr. F. Sankeralli:	Executive Director, Water and Sewerage Authority
Mr. R. Amoroso:	Sanitary Engineer, Water and Sewerage Authority
Mr. Mc.D. Gibbs:	Drainage Engineer, Ministry of Works
Mr. J. Kitamura:	Irrigation Engineer

DESCRIPTION OF THE AREA STUDIED:

The road to this area goes through the U.S. Naval Base, so people should have permits for the area. There are mostly forests above 500 ft., the tree crops (avocado, mango and the like) 500 ft. to 100 ft and the wild fields due to the shortage of water in dry season. There are citrus plants in some parts below 100 ft, contour. This would result in considerable difficulties for a survey crew.

In the east of the area two valleys lie north and south and two other shall ones east and west. There are a lot of small private impounded reservoirs here and there, and quite a few wells in the lower parts. (I did not have any chance to check the bored depth and the water quantity of the wells), but these wells were bored not for agriculture, but for domestic supply.

At the conference the members regarded the places below 100 ft. as the agricultural land. The area is approximately 1,000 acres 30% of which is under citrus as mentioned above. Agriculture suggested 40-65 vegetable farms each 3-5 acres and 35 dairy farms each 20 acres besides the citrus. In this area available water capacity is about 3,000,000 gallon/day and the present water quantity in use is about one third of it, 1,000,000 gallon/day of which a half, 500,000 gallon/day is for agriculture and another half 500,000 gallon/day is for agriculture and another half 500,000 gallon/day is for domestic use. The crops in this area need 2 to 4 inches/month as irrigation water. It means 45,000 gallon/acre month. The dry season is about six months, from November to April. So net water requirement will be calculated approximately as follows:-

45,000 gallon/acre month x 1,000 acre x 6 months = 270,000,000 gallons.

There is a proposed dam site, but as yet no survey information is available. However, the proposed area is very peameable. At the upper reaches of the dam site there are several good springs which discharges water all year round. There is an adequate supply during the wet season which led members to the conclusion that there would be sufficient water for the dam.

FUNDAMENTAL REQUIREMENTS

1. To decide the profitable area in Tucker Valley
2. To calculate the water requirement for this area
3. To confirm the available water capacity
4. To inspect the dam site whether it can store water enough to supply or not
5. To design the (Height of Dam
   (Length of Dam
   (Volume of Dam
   (Dam type and materials
6. To consider how to prevent the dam from water permeating
7. To consider use of wells as the subsidiary source of water
8. To inspect the conducting canal
   (the position of the route
   (the scale and extension
9. To consider how to irrigate
   (the furrow irrigation
   (the sprinkling irrigation

The above resulted from one visit, and I may yet change my requirement after new information becomes available, all except, of course, the water requirements.

Hereafter I would like to go there very often and collect fundamental data as soon as possible and I would like to have the opinion from the authorities in all the fields.

Junichi Kitamura

TUCKER VALLEY (II) 30th May 1967

The detailed maps on Tucker Valley was received from the Town and Country Planning Division on 2nd May, and the measurement of the profitable area was made with them. It was tried to be measured by the contour lines and land capability.

To calculate the water requirement, the area was classified into three parts. They are the vegetable farm, the citrus plants area and the dairy farm.

The net water requirement was calculated by using Blanney-Criddle Formula. As the crop coefficient, the same figures were used as these in the Western U.S.A. The result of the calculations show the net water requirement in dry season to be 4.3 inches a month.

The results of measurements on the map show below the profitable area in Tucker Valley.

Contour lines	Acres
Below 100 ft	960
100 ft - 500 ft	1,410
TOTAL	2,370
Agriculture Capability 1 & 2	1,019
Agriculture Capability 3 & 4	892
Forest	459
TOTAL	2,370

Based on the agriculture capability 1 & 2, 1,019 acres were decided as the profitable area.

The water requirement for this area was calculated on the assumption that people keep the fields as follows:-

Vegetable farm	200)	Total
Citrus fruit	250)	1,000 acres
Dairy farm	550)	

BY	:	BLANEY - CRIDDLE FORMULA
		$U = K \cdot F = KZtp$
U	:	Consuming water quantity (inches)
F	:	Total of the monthly consuming water elements
t	:	Average air temperature a month ($^{\circ}F$) (Chart 3)
p	:	Ratio of the sun-shining hours a month to the ones a year (Chart 4 & 5)
k	:	Crop co-efficient

(CHART 1)

Month	Temperature (t) °F	Ratio of the sunshining hrs (p) %	(t) x (p) (F)	Consuming water quantity		
				Vegetable k = 0.70	Citrus k = 0.65	Pasture k = 0.85
January	79.0	9.1	7.2	5.0 inches	4.7	6.1
February	79.7	8.5	6.8	4.8	4.4	5.8
March	81.4	9.4	7.7	5.4	5.0	6.5
April	83.4	9.1	7.6	5.3	4.9	6.5
May	83.5	9.2	7.7	5.4	5.0	6.5
June	81.8	7.4	6.1	4.3	4.0	5.2
July	81.4	8.2	6.7	4.7	4.4	5.7
August	81.7	8.2	6.7	4.7	4.4	5.7
Sept.	82.1	7.5	6.2	4.3	4.0	5.3
Oct.	82.3	7.9	6.5	4.6	4.2	5.5
Nov.	81.1	7.6	6.2	4.3	4.0	5.3
Dec.	79.7	7.9	6.3	4.4	4.1	5.4
Total				57.2	53.1	69.5

(CHART 2)

Month	Rainfall in 20% wet (Chart 7)	Consuming water quantity			Net water requirement		
		Vegetable	Citrus	Pasture	Vegetable	Citrus	Pasture
January	1.32	5.0	4.7	6.1	3.7	3.4	4.8
February	0.59	4.8	4.4	5.8	4.2	3.8	5.2
March	0.43	5.4	5.0	6.5	5.0	4.6	6.1
April	0.46	5.3	4.9	6.5	4.8	4.4	6.0
May	1.37	5.4	5.0	6.5	4.0	3.6	5.1
June	6.25	4.3	4.0	5.2	-	-	-
July	7.65	4.7	4.4	5.7	-	-	-
August	6.54	4.7	4.4	5.7	-	-	-
September	4.94	4.3	4.0	5.3	-	-	(0.4)
October	4.75	4.6	4.2	5.5	-	-	(0.7)
November	4.92	4.3	4.0	5.3	-	-	(0.4)
December	2.92	4.4	4.1	5.4	1.5	1.2	2.5
Total	42.14	57.2	53.1	69.5	23.2	21.0	29.7 (31.2)

Total net water requirement

23.2 x 1/12 x 200 x 43,560 = 16,820,000 ft^3		Vegetable
21.0 x 1/12 x 250 x 43,560 = 18,210,000		Citrus
29.7 x 1/12 x 550 x 43,560 = 59,230,000		Pasture
Total 94,260,000 ft^3	2,665,000 m^3	= 585,000,000 gallons

This amounts to about twice as much as the figure that was calculated approximately in the report of Tucker Valley (1) and means 4.3" a month.

(94,260,000 ft^3 x 12 inch 1,000 acres x 43,560 ft^2 x 6 months = 4.3 inches/acre. month).

(CHART 3) Mean air temperature (Based on readings at Piarco).

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1951	77.8	77.9	80.2	82.5	83.0	81.0	81.2	81.3	81.7	82.5	80.2	80.0
1952	79.9	80.7	82.1	84.3	84.9	81.8	81.1	81.6	82.1	82.8	81.9	80.0
1953	79.2	79.3	80.3	84.7	82.5	82.0	81.6	82.1	82.2	82.0	81.7	79.4
1954	79.0	79.7	80.8	82.4	84.0	82.2	81.2	81.3	81.8	81.1	79.4	78.6
1955	79.0	79.5	80.2	82.6	84.4	80.9	80.9	81.5	80.7	81.3	81.0	79.9
1956	79.5	79.1	80.4	81.7	81.5	81.3	81.5	80.7	81.5	81.3	81.0	79.1
1957	78.2	79.0	81.0	82.3	82.6	81.8	81.9	82.5	83.7	83.9	81.5	80.1
1958	79.3	80.7	84.1	85.0	82.4	81.5	81.6	82.8	84.2	82.5	81.2	80.0
1959	80.0	80.9	82.5	84.5	84.9	82.3	82.2	81.9	82.2	81.2	80.8	80.5
1960	79.2	80.0	81.6	82.9	84.4	82.4	81.5	81.5	82.0	82.0	81.2	79.4
1961	77.8	80.1	82.1	84.2	85.3	81.8	81.7	82.5	81.7	82.6	81.5	81.0
1962	79.5	78.9	81.3	85.2	84.8	83.9	81.3	80.8	83.0	84.1	82.7	80.6
1963	79.8	80.7	80.9	83.3	81.4	81.6	81.4	82.2	81.1	83.6	80.9	79.8
1964	77.9	79.9	81.5	82.2	83.3	81.0	80.6	81.7	81.3	80.8	80.3	77.4
14 yrs Aver.	79.0	79.7	81.4	83.4	83.5	81.8	81.4	81.7	82.1	82.3	81.1	79.7

Mean: /½(reading at 8 a.m. + reading at 2 p.m.)

(CHART 4) Hours of sunshine (Piarco field University of the West Indies).

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1948	8.1	8.2	7.4	6.7	8.3	5.4	6.6	7.0	7.1	6.5	6.9	7.0
1949	8.2	7.6	7.6	8.4	8.4	6.6	7.8	6.2	6.5	6.9	7.1	5.2
1950	5.4	6.0	6.5	7.6	6.5	5.2	7.5	6.4	6.8	7.0	7.2	6.5
1951	7.0	4.6	7.3	8.4	8.3	5.9	6.1	7.0	5.9	7.2	6.2	7.0
1952	8.4	9.2	8.7	10.0	8.3	6.6	7.1	6.8	7.3	6.2	7.6	6.5
1953	6.8	7.4	7.1	9.4	6.0	7.0	6.4	6.8	5.3	6.7	6.3	6.4
1954	6.8	8.1	8.7	6.5	8.5	7.1	6.5	6.3	5.0	6.3	5.6	5.5
1955	8.5	8.2	6.2	6.8	9.1	6.4	7.0	7.0	5.8	5.3	6.1	7.2
1956	6.7	7.4	7.9	7.3	7.3	7.0	7.8	6.6	5.9	6.4	6.6	5.8
1957	7.3	8.1	9.1	8.0	6.9	5.8	7.4	7.5	8.1	7.3	6.5	7.1
1958	7.2	8.9	9.7	8.1	6.7	5.6	5.8	7.6	8.4	6.3	6.8	5.6
1959	8.3	9.0	9.0	9.1	8.4	6.8	7.8	7.6	7.6	6.1	7.4	7.5
1960	9.2	9.6	8.5	7.4	9.9	7.8	7.8	6.2	7.2	6.4	7.5	7.5
1961	8.7	9.6	8.2	10.1	9.6	7.2	7.2	7.6	6.1	7.1	6.4	9.5
1962	8.4	7.5	9.0	9.5	8.3	6.9	6.4	6.4	6.8	8.9	6.5	6.4
1963	8.2	8.8	8.3	7.7	5.3	7.2	7.4	7.9	6.1	9.1	6.5	8.3
1964	8.9	9.6	8.8	6.9	9.2	6.4	6.0	8.0	6.6	6.3	7.2	6.5
17 yrs Aver.	7.8	8.1	8.1	8.1	7.9	6.5	7.0	7.0	6.6	6.8	6.7	6.8

(CHART 5) Ratio of the sun-shining hours a month to the ones a year

Month	hours of sunshine	No. of days	(A) x (B)	Ratio (%)
	(A)	(B)	(C)	
January	7.8	31	241.8	9.1
February	8.1	28	226.8	8.5
March	8.1	31	251.1	9.4
April	8.1	30	243.0	9.1
May	7.9	31	244.9	9.2
June	6.5	30	195.0	7.4
July	7.0	31	217.0	8.2
August	7.0	31	217.0	8.2
September	6.6	30	198.0	7.5
October	6.8	31	210.8	7.9
November	6.7	30	201.0	7.6
December	6.8	31	210.8	7.9
Total		365	2,657.2	100.0

(CHART 6) MONTHLY RAINFALL PRECIPITATION (inches)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug	Sept.	Oct.	Nov.	Dec.	Total
1957	3.45	1.79	0.15	4.12	2.37	4.61	7.41	8.65	7.17	7.63	8.70	9.02	65.07
1958	2.16	1.19	0.63	4.49	5.43	18.26	15.68	8.75	5.58	7.61	4.78	4.06	78.62
1959	0.73	1.23	0.59	0.54	3.49	4.42	4.55	7.08	10.47	7.92	4.45	5.09	50.56
1960	1.58	0.61	0.44	1.65	1.51	8.60	7.89	9.66	7.34	8.00	5.33	9.31	61.92
1961	2.47	1.19	1.53	0.34	0.32	5.34	17.72	12.18	6.89	4.31	19.86	2.98	75.13
1962	1.22	1.36	1.72	0.62	2.28	7.97	9.30	14.76	7.65	6.33	2.82	3.81	59.84
1963	3.38	0.87	2.13	1.64	7.64	7.73	8.79	7.19	10.42	6.02	6.99	2.55	65.35
1964	2.03	0.30	2.57	0.82	2.52	7.39	13.22	7.99	8.13	4.69	6.58	5.45	61.69
1965	6.57	1.92	1.00	1.54	2.49	5.85	10.03	6.24	6.97	6.47	8.26	6.89	64.23
1966	1.70	4.89	0.90	3.33	5.51	15.56	11.18	9.80	9.20	8.92	10.81	6.17	87.92
Aver.	2.53	1.54	1.17	1.91	3.35	8.57	10.57	9.23	7.98	6.79	7.86	5.53	67.03

(CHART 7) RAINFALL PRECIPITATION IN 20% WET AT DIEGO MARTIN (inches)

Month	St. Clair		Ratio (B)/(A)	Diego Martin		Remarks
	Mean Monthly Rainfall	20% Wet		Mean Monthly Rainfall	20% Wet	
	(A)	(B)	(C) = (A)%	(D)	(E) = (D) x (C)	
January	2.49	1.30	52.2	2.53	1.32	
February	1.92	0.54	38.1	1.54	0.59	The Rainfall in
March	1.80	0.66	36.7	1.17	0.43	20% wet is deriv-
April	2.71	0.65	24.0	1.91	0.46	ed from F.A.O.
May	4.42	1.81	41.0	3.35	1.37	report
June	7.19	5.25	73.0	8.57	6.25	
July	8.03	5.82	72.4	10.57	7.65	
August	9.86	7.00	70.9	9.23	6.54	
September	8.49	5.26	62.0	7.98	4.94	
October	6.55	4.58	70.0	6.79	4.75	
November	7.48	4.69	62.7	7.86	4.92	
December	5.21	2.76	52.9	5.53	2.92	
TOTAL	66.15	40.32		67.03	42.14	

Junichi Kitamura

TUCKER VALLEY (III) In June 1967

To survey the damsite, my visit to Tucker Valley with Mr. J.C. Douglas was made on the 22nd May, 1967. As the warning board on which it was written that U.S. Military Corps set the traps in the vicinity was found on the way to Benjamin Creek, our return was needed and the circumstances were related to the Executive officer Mr. R. L. Kays. Our survey was then guided by a seaman.

The stream in the creek seemed to have increased a little, since it showered that morning, but it disappeared underground in some places. Our visit was made up to the spot 500 yards far from the car parking place through the creek. To select the suitable dam site, after surveying, I selected three spots and investigated them.

(Dam site 1).

About 300 yards up the creek from the car park, a good dam site was found as indicated on the map. The water flowed on the ground there and both sides of the creek are favourably steep. The basic rocks are Schist, Quartzite and the like, formed by recrystallization of dynamic metamorphism. The rocks of both sides also consist of Crystalline Schist abundant in Schistosity.

The top soil on the river bed is a coarse weathered sand. It should be thought that the weathering action goes through to the foundation. The topography is ideal as a dam site, but it is most important, to survey the condition of the foundation with boring and permeability test.

Even if it has no problem on the basic bearing capacity the proper countermeasure against danger of sliding and leak of water from the weathered crack will be needed.

(Dam Site 2)

About 100 yards up the creek from the car park there is also a favourable dam site. About all the condition on it are the same as the 1st one except for a little wider section, thicker sandy top soil and widen catchment area.

(Dam Site 3)

It is the place where people are planning as the Tucker Valley underground dam site 1,600 yards down from the car park as showed on the map. It has the catchment about three times as large as on the 1st & 2nd dam site. As the river bed gradient is smaller than the ones on the 1st and 2nd dam site, the dam site concerned has larger water storage capacity.

The depth to the basic rocks or even the condition on the top soil could not be surveyed because of a heavy vegetal cover. But it can be guessed that the basic rocks are situated pretty deep and the thich top soil is very permeable, because a lot of entirely dried creeks could be seen here and there. Hereafter the situation of the basic rocks must be made certain of with boring test.

The above is most important considering the construction cost of a dam. It is the most important key to influence the dam construction expense Under certain circumstances, a high dam must be built under the ground, despite the height of the embankment.

To calculate the water quantity collected from the catchment area.

The rainfall precipitation at River Estate in Diego Martin near Tucker Valley was used.

The mean rainfall precipitation a year in these ten years in 67.03 inches as showed on chart 8. The mean run-off coefficient is computed as 41.5% as shown on chart 9.

The ratio of the rainfall in 20% wet to the mean rainfall at St. Clair Rainfall Station is 61.0% as shown on chart 10.

The catchment areas are as follows:

Dam site 1.	377.0 acres = 152.3 ha
Dam site 2.	402.0 acres = 162.2 ha
Dam site 3.	1,725.3 acres = 697.0 ha

Thus the possible water quantity collected from the catchment areas are as follows:

Dam Site 1

$$1{,}523{,}000\text{m}^2 \times 67.03 \text{ inches} \times 0.0254 \text{ m/inch} \times 0.61 \times 0.415 = 655{,}000 \text{ m}^3$$

Dam Site 2

$$1{,}622.00\text{m}^2 \times 67.03 \text{ inches} \times 0.0254\text{m/inch} \times 0.61 \times 0.415 = 698{,}000\text{m}^3$$

Dam Site 3

$$6{,}970{,}000\text{m}^2 \times 67.03 \text{ inches} \times 0.0254\text{m/inch} \times 0.61 \times 0.415 = 3{,}000{,}000\text{m}^3$$

Total net water requirement in 20% wet is 2,665,000m^3. (585,000,000 gallons) as mentioned in the report Tucker Valley (II).

It means that the catchment area at Dam site 3 can only supply the water requirement in 20% wet in Tucker Valley.

To calculate the water storage capacity at Dam site 3 and decide height and length of the dam.

Each contour in the area cannot be measured precisely, as only having the map to a scale of 1 inch to 1,000 ft. The result made a rough estimate is showed on Fig. 3. and Chart 11 and 12.

At the section A - A

(Height of dam · 75 ft (Elevation 245 ft)

(Length of dam 1,030 ft

At the section B - B

(Height of dam 80 ft (Elevation 253 ft)

(Length of dam 1,000 ft

Which is better A - A or B - B concerning topography and geology will have to be examined hereinafter.

(CHART 8) THE SUMMARY OF THE CLASSIFICATION

	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	Total	Average
0-0.2"	105	112	115	121	103	118	129	121	127	114	1,165	116.5
0.2 - 0.4	34	38	35	36	34	32	38	46	38	47	378	37.8
0.4 - 0.6	27	28	26	14	20	26	16	24	22	20	218	21.8
0.6 - 0.8	13	16	8	17	16	10	12	14	9	19	134	13.4
0.8 - 1.0	5	7	4	2	8	9	7	6	14	12	74	7.4
1.0 - 1.2	4	5	4	4	6	5	7	6	6	7	54	5.4
1.2 - 1.6	4	3	3	2	7	2	3	5	2	7	38	3.8
1.6 - 2.0	4	2	1	1	1	3	2	0	1	4	19	1.9
2.0 - 2.4	1	2	0	2	1	1	1	0	1	3	12	1.2
2.4 - 3.2	1	2	0	3	0	0	0	0	1	0	7	0.7
3.2 - 4.0	0	1	0	0	0	0	1	0	0	0	2	0.2
4.0 - 6.0	0	0	0	0	0	0	0	0	0	0	1	0.1
Total Days	198	216	196	202	197	201	216	222	221	233	2,102	210.2
Annual(inches) Rainfall	65.07	78.62	50.56	61.92	75.15	59.84	65.35	61.69	64.23	87.92	670.33	67.03

(CHART 9)

	Average Raining Days	Average Rainfall	Correction	Run off Co. eff	Run-off	Remarks
			x0.94675			
0 - 0.2'	116.5	x 0.1 = 11.65'	11.03'	20%	2.206'	
0.2 - 0.4	37.8	x 0.3 = 11.34	10.74	30	3.222	
0.4 - 0.6	21.8	x 0.5 = 10.90	10.32	40	4.128	
06 - 0.8	13.4	x 0.7 = 9.38	8.88	42	3.730	
0.8 - 1.0	7.4	x 0.9 = 6.66	6.31	45	2.840	
1.0 - 1.2	5.4	x 1.1 = 5.94	5.62	50	2.810	
1.2 - 1.6	3.8	x 1.4 - 5.32	5.04	55	2.772	
1.6 - 2.0	1.9	x 1.8 - 3.42	3.24	60	1.944	
2.0 - 2.4	1.2	x 2.2 = 2.64	2.50	65	1.625	
2.4 - 3.2	0.7	x 2.8 = 1.96	1.85	70	1.295	
3.2 - 4.0	0.2	x 3.6 = 0.72	0.68	75	0.510	
4 0 - 6.0	0.0	x 5.0 = 0.00	0.00	80	0.000	Average of
6.0 -	0.1	x 8.7 = 0.87	0.82	90	0.738	run-off Co. eff.
ANNUAL RAINFALL		70.80 67.03/70.80=0.94675	67.03		27.82	27.82/ 67.03=41.5%

(CHART 10) Ratio of 20% Wet to Mean Rainfall at St. Clair Rainfall Station

	Mean inch	20% Wet inches	Remarks
January	2.49	1.30	
February	1.92	0.54	
March	1.80	0.66	
April	2.71	0.65	
May	4.42	1.81	
June	7.19	5.25	
July	8.03	5.82	
August	9.86	7.00	
September	8.49	5.26	
October	6.55	4.58	
November	7.48	4.69	
December	5.21	2.76	Ratio
Total	66.15	40.32	40.32/66.15=61.0%

derived from F.A.O. Report

(CHART 8-1) THE CLASSIFICATION OF DAILY RAINFALL PRECIPITATION
River Estate Diego Martin

1957	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0 - 0.2"	20	12	4	6	8	10	8	11	9	6	4	7	105
0.2 - 0.4	2	1	-	2	2	6	4	2	2	3	5	5	34
0.4 - 0.6	2	2	-	1	3	4	3	3	4	-	4	1	27
0.6 - 0.8	-	-	-	-	-	1	4	2	2	1	2	1	13
0.8 - 1.0	-	-	-	-	-	-	-	-	2	1	1	1	5
1.0 - 1.2	-	-	-	-	-	-	1	1	1	-	1	-	4
1.2 - 1.6	-	-	-	-	-	-	-	1	-	1	1	1	4
1.6 - 2.0	-	-	-	-	-	-	-	1	-	2	-	1	4
2.0 - 2.4	-	-	-	-	-	-	-	-	-	-	-	1	1
2.4 - 3.2	-	-	-	1	-	-	-	-	-	-	-	-	1
													198days

1958	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0 - 0.2"	11	4	2	7	7	9	11	12	9	13	14	13	112
0.2 - 0.4	-	1	2	1	6	3	5	6	6	3	3	2	38
0.4 - 0.6	-	-	-	3	-	5	6	4	3	2	2	3	28
0.6 - 0.8	2	1	-	-	2	3	2	1	2	1	1	1	16
0 8 - 1.0	-	-	-	-	1	2	1	-	-	2	1	-	7
1.0 - 1.2	-	-	-	-	1	-	1	1	-	2	-	-	5
1.2 - 1.6	-	-	-	-	-	3	-	-	-	-	-	-	3
1.6 - 2.0	-	-	-	1	-	1	-	-	-	-	-	-	2
2.0 - 2.4	-	-	-	-	-	-	1	1	-	-	-	-	2
2.4 - 3.2	-	-	-	-	-	-	2	-	-	-	-	-	2
3.2 - 4.0	-	-	-	-	-	1	-	-	-	-	-	-	1
													216days

1959	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0 - 0.2	12	11	3	6	6	15	19	8	7	12	11	5	115
0.2 - 0.4	-	-	2	1	-	4	3	5	7	5	5	3	35
0.4 - 0.6	-	-	-	-	-	1	4	5	3	6	3	4	26
0.6 - 0.8	-	1	-	-	1	1	-	1	-	2	1	1	8
0.8 - 1.0	-	-	-	-	1	-	-	1	2	-	-	-	4
1.0 - 1.2	-	-	-	-	-	1	-	1	-	1	-	1	4
1.2 - 1.6	-	-	-	-	1	-	-	-	2	-	-	-	3
1.6 - 2.0	-	-	-	-	-	-	-	-	1	-	-	-	1
													196days

1960	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0 - 0.2"	9	8	13	10	7	10	14	13	7	14	6	10	121
0.2 - 0.4	1	-	-	2	3	4	4	7	5	3	2	5	36
0.4 - 0.6	-	-	-	-	1	2	-	2	1	3	3	2	14
0.6 - 0.8	1	-	-	1	-	3	2	1	2	2	4	1	17
0.8 - 1.0	-	-	-	-	-	-	-	-	-	1	-	1	2
1.0 - 1.2	-	-	-	-	-	-	2	-	-	2	-	-	4
1.2 - 1.6	-	-	-	-	-	1	-	4	-	-	-	-	2
1.6 - 2.0	-	-	-	-	-	-	1	-	-	-	-	-	1
2.0 - 2.4	-	-	-	-	-	1	-	-	-	-	-	1	2
2.4 - 3.2	-	-	-	-	-	-	-	1	1	-	-	1	3
													202days

1961	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0 - 0.2"	12	5	6	4	6	17	4	5	11	10	10	13	103
0.2 - 0.4	3	-	-	-	-	6	3	6	2	6	7	1	34
0.4 - 0.6	1	2	-	-	-	-	4	5	3	3	2	-	20
0.6 - 0.8	1	-	-	-	-	2	1	2	5	1	3	1	16
0.8 - 1.0	-	-	-	-	-	-	1	5	1	-	1	-	8
1.0 - 1.2	-	-	-	-	-	1	3	-	-	-	1	1	6
1.2 - 1.6	-	-	1	-	-	-	4	1	-	-	1	-	7
1.6 - 2.0	-	-	-	-	-	-	-	-	-	-	1	-	1
2.0 - 2.4	-	-	-	-	-	-	1	-	-	-	-	-	1
2.4 - 3.2	-	-	-	-	-	-	-	-	-	-	-	-	-
3.2 - 4.0	-	-	-	-	-	-	-	-	-	-	-	-	-
4.0 - 6.0	-	-	-	-	-	-	-	-	-	-	-	-	-
6.0 -	-	-	-	-	-	-	-	-	-	-	1	-	1
													197 days

(CHART 8-2)

1962	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0-0.2'	19	12	4	10	5	9	12	8	9	8	8	14	118
0.2-0.4	-	1	1	-	1	5	7	4	3	4	5	1	32
0.4-0.6	-	-	1	-	2	3	1	5	2	2	2	3	21
06-0.8	-	-	1	-	1	2	-	2	2	2	2	-	10
0.8-1.0	-	-	-	-	-	2	1	4	1	-	-	1	9
1.0-1.2	-	-	-	-	-	1	1	3	-	-	-	-	5
1.2-1.6	-	-	-	-	-	-	-	-	2	-	-	-	2
1.6-2.0	-	-	-	-	-	-	1	1	-	1	-	-	3
2.0-2.4	-	-	-	-	-	-	1	-	-	-	-	-	1
2.4-3 2	-	-	-	-	-	-	-	-	-	-	-	-	-
													201days
1963	**Jan.**	**Feb.**	**Mar.**	**Apr.**	**May**	**June**	**July**	**Aug.**	**Sept.**	**Oct.**	**Nov.**	**Dec.**	**Total**
0-0.2'	12	11	12	7	12	9	11	14	11	3	13	14	129
02-0.4	5	1	3	1	4	4	5	1	5	-	4	5	38
04-0.6	-	-	-	2	1	-	2	4	-	3	3	1	16
06-0.8	-	-	1	-	3	1	1	2	1	1	2	-	12
08 1.0	-	-	-	-	-	1	2	-	-	3	1	-	7
1.0-1.2	1	-	-	-	1	1	-	-	2	1	1	-	7
1.2-1.6	-	-	-	-	-	1	1	-	1	-	-	-	3
16 2.0	-	-	-	-	-	-	1	1	-	-	-	-	2
2.0-2.4	-	-	-	-	-	1	-	-	-	-	-	-	1
2.4-3.2	-	-	-	-	-	-	-	-	-	-	-	-	0
32-4.0	-	-	-	-	-	-	-	-	-	-	-	-	1
													216days
1964	**Jan.**	**Feb.**	**Mar.**	**Apr.**	**May**	**June**	**July**	**Aug.**	**Sept.**	**Oct.**	**Nov.**	**Dec.**	**Total**
0 0.2'	9	9	8	9	9	10	10	9	14	11	16	7	121
02-0.4	1	-	-	4	-	6	7	5	3	8	4	8	46
04-06	-	-	1	1	1	3	3	6	3	1	3	-	24
06-0.8	-	-	-	1	1	3	1	1	4	-	1	2	14
08-1.0	1	-	-	-	1	-	2	-	1	-	1	-	6
1.0-1.2	-	-	-	-	-	1	3	1	-	1	-	-	6
1.2-1.6	-	-	-	-	-	-	2	1	-	-	1	1	5
16-2.0	-	-	-	-	-	-	-	-	-	-	-	-	-
													222days
1965	**Jan.**	**Feb.**	**Mar**	**Apr.**	**May**	**June**	**July**	**Aug.**	**Sept.**	**Oct.**	**Nov.**	**Dec.**	**Total**
0-0.2'	12	14	7	7	8	13	11	16	9	7	10	13	127
0.2 0.4	5	1	2	3	-	3	5	1	6	3	4	5	38
0.4-0.6	3	-	-	1	-	2	5	4	2	2	2	1	22
06-0.8	-	-	-	-	-	3	-	2	1	1	1	1	9
0.8-1 0	2	1	-	-	-	-	3	-	2	2	3	1	14
10-1.2	1	-	-	-	2	-	1	-	1	1	-	-	6
1.2-1.6	-	-	-	-	-	1	1	-	-	-	-	-	2
16-2.0	-	-	-	-	-	-	-	1	-	-	-	-	1
2.0 2.4	-	-	-	-	-	-	-	-	-	-	1	-	1
2.4-3.2	-	-	-	-	-	-	-	-	-	-	-	1	1
													221 days
1966	**Jan.**	**Feb.**	**Mar.**	**Apr.**	**May**	**June**	**July**	**Aug.**	**Sept.**	**Oct.**	**Nov.**	**Dec.**	**Total**
0-0.2'	10	6	3	9	11	5	9	8	13	10	9	21	114
0.2-0.4	1	1	-	-	5	9	10	4	4	5	5	3	47
04-06	2	1	-	1	1	3	2	2	4	1	3	-	20
06-08	-	-	1	-	-	6	3	4	-	1	2	2	19
08-1.0	-	-	-	-	-	-	2	3	2	1	2	2	12
1.0-1 2	-	-	-	1	-	2	1	-	1	1	-	1	7
1.2-1 6	-	1	-	1	1	1	1	1	-	1	-	-	7
16-2.0	-	-	-	-	1	2	-	-	1	-	-	-	4
2.0-2.4	-	1	-	-	-	-	-	-	-	1	1	-	3
2.4-3.2	-	-	-	-	-	-	-	-	-	-	-	-	-
													233 days

(CHART 11) WATER STORAGE CAPACITY

At Dam site 3 A - A Section

Contour	Area	Average	Area	Volume .	Total Volume
ft	(acres)	(acres)	(m^2)	(m^3)	(m^3)
170 >	0	-	-	-	-
180 >	5.6	· 2.8	11,320	34,000	34,000
200 >	27.9	16.8	67,950	412,000	446,000
220 >	42.2	35.1	142,000	862,000	1,308,000
240 >	64.0	53.1	214,500	1,302,000	2,610,000
260 >	94.5	79.3	320,500	1,945,000	4,555,000
280 >	120.8	107.7	435,500	2,641,000	7,196,000
300 >	158.6	139.7	565,000	3,430,000	10,626,000

(CHART 12)

At Dam site 3 B - B Section

Contour	Area	Average	Area	Volume	Total Volume
ft	(acres)	(acres)	(m^2)	(m^3)	(m^3)
173	0	-	-	-	-
180	3.4	1.7	6,900	15,000	15,000
200	22.1	12.8	51,800	314,000	329,000
220	33.2	27.7	111,900	678,000	1,007,000
240	54.0	43.6	176,200	1,069,000	2,076,000
260	83.5	68.8	278,000	1,686,000	3,762,000
280	109.8	96.7	391,000	2,370,000	6,132,000
300	147.1	128.5	519,500	3,148,000	9,280,000

TUCKER VALLEY (VI) January, 1968

MOVEMENT

21 Oct. 1967	Asked to survey the proposed dam site and to make a profile and cross sections.
18 Nov.	Surveying was finished and drawing was begun.
5 Dec.	Was requested to calculate the rough estimate of the whole irrigation work for this area by Mr. W. Dookeran, Ministry of Planning and Development.
13 Dec.	Completed the calculation, explained the details and presented it as a confidential paper.
8 Jan. 1968	Asked to survey the proposed dam site supplementarily again. (This survey will begin on 15th of January).

INTRODUCTION

This time a water distribution system for the Tucker Valley area will be described, as the detailed calculation of the dam cannot be carried out until the profile and the cross sections have been completed.

Furrow irrigation is desirable as it is the most economical and easiest method in this case.

The canals must be lined with concrete to prevent leakage of water.

As stated on the confidential report, the irrigation is limited to the area classified as agricultural capability 1 and 2. The following statement is quoted from the report.

... 'It should be stated, however, that there is no possibility of supplying irrigation water for the area classified as agricultural capability 3 and 4 at the present stage for the two following reasons:

(1) The elevation of the water taken from the dam cannot be raised higher than 200 feet so as not to increase the dead water. In other words the irrigation water from the dam cannot be supplied by gravity for the area.

(2) The capable water quantity collected from the catchment area in the droughty year once in five years is nearly equal to the water requirement for the area classified as agricultural capability 1 and 2.

It is therefore regretted that the irrigation plan for the areas 3 and 4 cannot be set up immediately.

On the other hand the cost of water supply for the tree-crops will be rather expensive.' ...

DESCRIPTION OF THE AREA STUDIED

The Cuese River which originates from a point near the coast overlooking Macquèripe Bay flows down in a southenly direction towards Carenage Bay through a narrow area of flat land surrounded by hills.

From the middle of this flat area of land Tucker Valley stretches up east. The streams from Tucker Valley and Benjamin Creek go underground on the way to the Cuese River. The soil of the riverbed consists of weathered coarse sand and gravel from crystalline chist and is very permeable.

From the north to the south, the following areas are situated: Macqueripe area, Golf Course area, Plantation area, Rifle range and Naval Air station.

The surrounding hills are not so steep, the maximum height is 1,600 feet from the sea level.

The area classified as agricultural capability 1 and 2 are mostly below 200 feet from the level of the sea. (assumed to be 0.00)

WATER DISTRIBUTION SYSTEM (cf. figure)

The lowest river-bed level at the proposed dam site is nearly 170 feet, above the sea level. Consequently 200 feet is adopted as an appropriate intake water level.

The crops concerned, that is, vegetables, citrus and pasture were changed for only vegetables.

The calculation for the water requirement must therefore be done over again, strickly speaking.

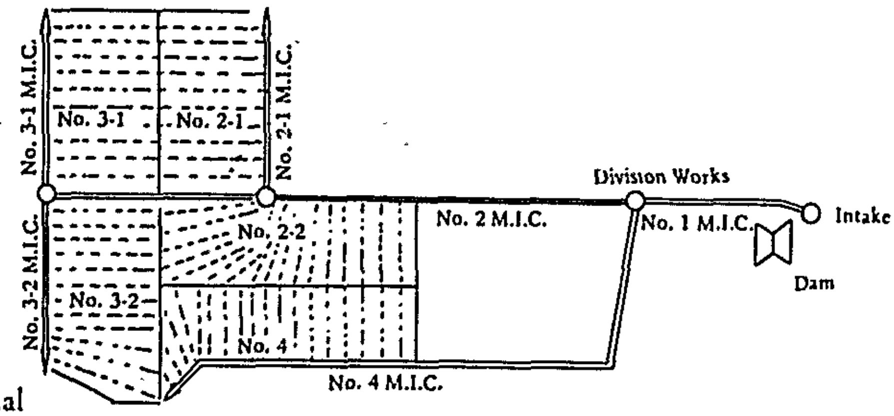
The determination of the canal section, however was carried out with the water requirement as it stands because of no large difference between the two water requirements.

The area is divided into five parts.

(viz)	The upper right side	No. 3-1
	The lower right side	No. 3-2
	The upper left side	No. 2-1
	The middle left side	No. 2-2
	The lower left side	No. 4

They are supplied water from the main irrigation canal, No. 3-1, No. 3-2, No. 2-1, No. 2 and No. 4 respectively.

The water is conveyed from the main canal to feeders by feeder controls. Moreover it is also distributed from the feeder to each furrow. Three diversion works are needed so as to divide the main canal into two. (cf. the following sketch)



(1) Main irrigation canal

Benefited area 1,019 acres = 412.4 ha

Maximum water requirement 5.0 inches a month (cf. report Tucker Valley II)

5.0 inches/month = 127mm/30 days = 4.23 mm/day.

Assuming that the water-conveyance losses are as follows,

on main canal	5%
on feeder	20%
Total	25%

The total water requirement will be

$$Q = 412.4 \text{ ha} \times 10^4 \times 4.23 \text{ mm} \times 1.25 \times 10^{-3}/86{,}400 \text{ sec} = 0.253 \text{ m}^3/\text{sec} \quad (= 8.93 \text{ cusecs})$$

Therefore the dimensions are in the followng tables.

(Table 1)

No. of route	Length		Items: Open channel	Items: Siphon	Items: Tunnel	Discharge		Accumulative area	
1	300 ft	90m	30m	-m	60m	8.93cusecs	0.253m^3/s	1,019acres	412.4ha
2	5,200	1,590	1,360	230	-	6.36	0.180	726	293.8
3	2,200	670	-	670	-	3 00	0.085	341	138.0
4	9,800	2,990	2,780	210	-	2.58	0.073	293	118.6
2-1	5,300	1,620	1,440	-	180	2.01	0.057	231	93.5
2-2	0	0	-	-	-	1.34	0.038	154	62.3
3-1	4,900	1,490	1,490	-	-	1.06	0.030	121	49.0
3-2	8,800	2,680	2,680	-	-	1.94	0.055	220	89.0
Total	36,500	11,130	9,780	1,110	240				

(Table 2)

No. of Route	Grade of Canal-bed	Items		
		Open Channel	Siphon	Tunnel
1	1 / 1,000	B H 3.0' x 2.0'	-	B H 3.0' x 3.0'
2	"	2.5 x 1.8	Ø 2.5'	-
3	-	-	Ø 2.0	-
4	1 / 1,000	1.8 x 1.2	Ø 2.0	-
2-1	"	1.5 x 1.2	-	3.0 x 3.0
2-2	-	-	-	-
3-1	1 / 1,000	1.2 x 1.0	-	-
3-2	"	1.5 x 1.2	-	-

(Table 2-1) Calculation of Open Channel

No. of route	Planned discharge	Slope	Rouchness coeff. n	N	D	Depth H	Breath B	Cross section Area A	Wetted perimeter P	Hydraulic Radius R	R⅔	NR	D + R⅔	Velocity V	Discharge Q
	m^3/s													m/s	m^3/s
1	0.253	1/1,000	0.017	1.994	0.438	2.0' 0.61m	3.0' 0.91m	$0.56m^2$	2.13m	0.263m	0.513	0.524	0.951	0.55	0.308
2	0.180	•	•	•	•	1.8 0.55	2.5 0.76	0.42	1.86	0.226	0.475	0.451	0.913	0.49	0.206
4	0.073	•	•	•	•	1.2 0.37	1.8 0.55	0.20	1.29	0.155	0.394	0.309	0.832	0.37	0.074
2-1	0.057	•	•	•	•	1.2 0.37	1.5 0.46	0.17	1.20	0.142	0.377	0.283	0.815	0.35	0.595
3-1	0.030	•	•	•	•	1.0 0.30	1.2 0.37	0.11	0.97	0.113	0.336	0.225	0.774	0.29	0.319
3-2	0.055	•	•	•	•	1.2 0.37	1.5 0.46	0.17	1.20	0.142	0.377	0.283	0.815	0.35	0.595

Concerning calculation of the inverted siphon, a route survey must first be carried out in order to determine the length and the required pressure of pipes.

In case of the siphon on the route of No. 3 enough head can be obtained to lead the water, because all the areas of No. 3-1 and No. 3-2 are below the 100 ft contour. The most economical diameter of pipes must therefore be selected for the route of NO. 3.

On the other hand, in case of the siphons on the route of No. 2 and No. 4, the loss of head must be diminished as much as possible, because the maximum elevation in the areas, No. 2-1, No. 2-2 and No. 4 is approximately at the 200 ft contour.

The diameter of pipe must be enlarged sufficiently to reduce the loss of head.

The diameters of the pipes on Table 2 were simply assumed. A further study of that will therefore be necessary.

Concerning the water tunnel the minimum workable cross section is large enough to let the water flow down.

The completed cross section with concrete-lining was assumed as 3 feet by 3 feet with an arch.

Incidentally the figures on Table 2 are somewhat different from the ones on the confidential report. The former figures are correct as a result of detailed calculation.

(2) Diversion works

Three diversion works are necessary for the main irrigation canals viz.

from No. 1 to No. 2 and No. 4
from No. 2 to No. 3 and No. 2-1
from No. 3 to No. 3-1 and No. 3-2

In order to divide water accurately at all times, it is necessary to study the type of diversion works. However, Jet flow diversion is recommended because it necessitates a lower head than the other types.

(3) Feeder control

Feeder control are necessary, each about 660 feet (= 200 m) on the main canal. It is therefore supposed that 40 feeder controls are necessary for the area.

As the feeder control, two sluice gates must be set at each distribution junction.

(1) at the lower reach just next to an inlet of a feeder on a main canal
(2) at an inlet of a feeder

These sluice gates can be the same as those in the Caroni area, the Fishing Pond area and the Plum Mitan area.

(4) Feeder

U shaped precast concrete channels are desirable as feeders. It will be very economical, easy to convey and install.

A feeder of 50m (=165 ft.) in length per hectar (=247 acres) is necessary for leading the irrigation water into the area. Therefore total length will be.

412.4 ha x 50 m/ha = 20,260 m

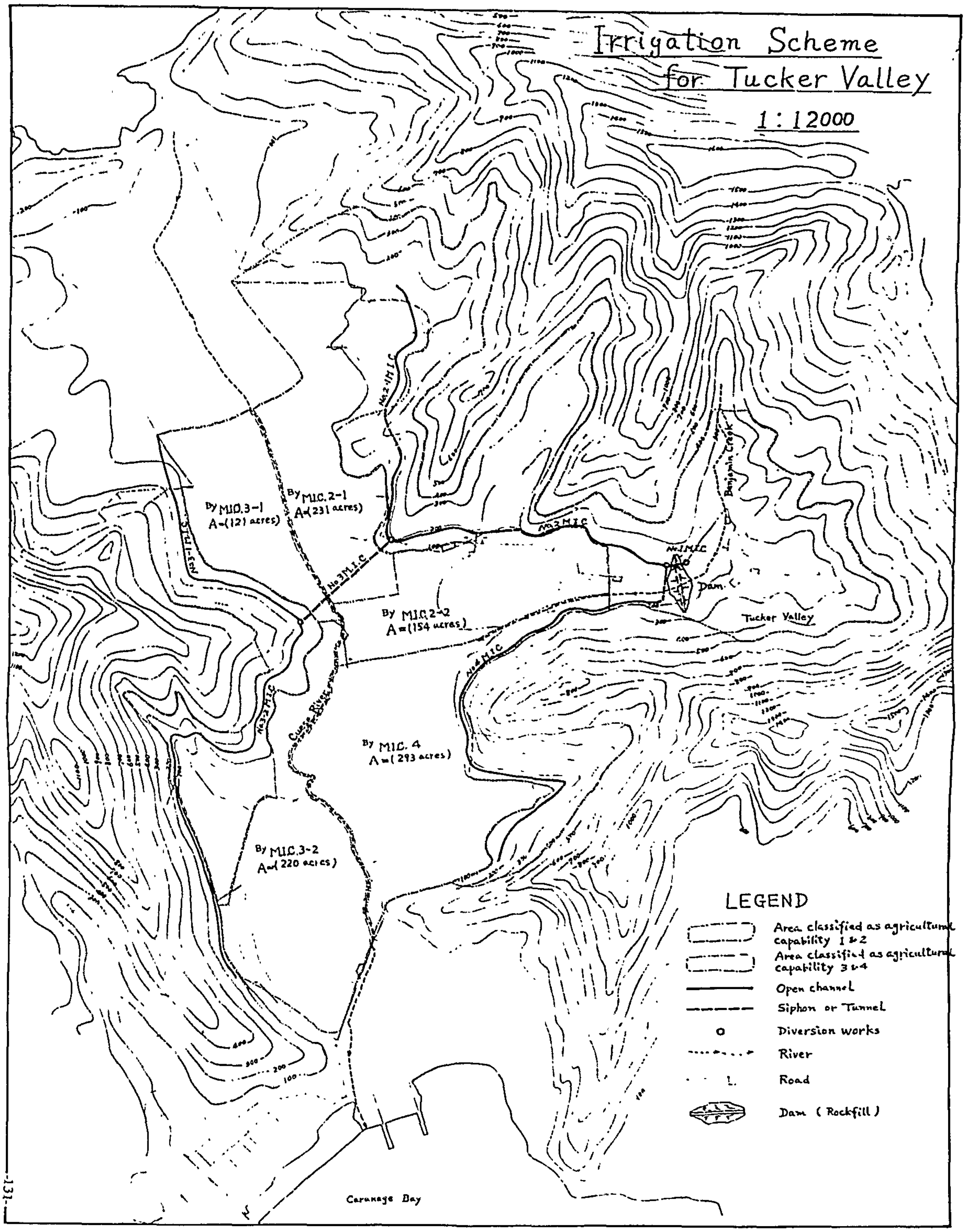
Taking clearance of 20%

20,620 m x 1.2 = 25,000 m

As the implements to supply the water from the feeder to the furrow, siphon pipes made of synthetic resin and shuttering boards are needed.

A shuttering board could be used for keeping the water level of the feeder constant.

Junichi Kitamura


Irrigation Scheme
for Tucker Valley
1 : 12000
By MIC. 3-1
A=(121 acres)
By MIC. 2-1
A=(231 acres)
By MIC. 2-2
A=(154 acres)
By MIC. 4
A=(293 acres)
By MIC. 3-2
A=(220 acres)
No.3 M.I.C.
No.2-1 M.I.C.
No.4 M.I.C.
No.3-2 M.I.C.
Benjamin Creek
Dam
Tucker Valley
Cuesa River
Carenage Bay
LEGEND
Area classified as agricultural capability 1 & 2
Area classified as agricultural capability 3 & 4
Open channel
Siphon or Tunnel
Diversion works
River
Road
Dam (Rockfill)

TUCKER VALLEY (V) February, 1968

MOVEMENT

13, Feb. Discussed with Mr. W. Dookeran the irrigation scheme for Tucker Valley area. Promised him to design a different scheme for irrigating 200 (or 400) acres with underground water by 20th instant.

15, Feb. Visited there to investigate existing active wells with Mr. E. Wyke. Got Mr. Joseph to guide us to wells No. 3, No. 4, No. 16 and No. 17.

INTRODUCTION

It was suggested by the Ministry of Planning and Development that an irrigation scheme for this area should be designed on a small scale so as to make use of underground water at this stage, as an irrigation scheme requiring the building of a dam seems to be untenable at this time due to the high expenditure involved. According to the suggestion the aim is to irrigate an area of 200 acres as a first stage, and then 200 more acres subsequently.

For this puprose the following must be carried out;

1. Selection of area to be irrigated.
2. Determination of the water requirement.
3. Determination of details of the wells. - number, depth, diameter, and position.
4. Determination of details of pipes to convey water - length, diameter, type and position.
5. Determination of details of pumps - capacity, type, house power and so on.
6. Estimation of the cost - initial cost, recurrent cost.

DESCRIPTION OF THE AREA STUDIED

The Cuese River flows down from the north to the south, and pours into Carenage Bay. In the middle of the route, a tributary which flows down from the East to the West joins it.

On both sides of the flat land along the Cuese River several shallow valleys, are located. There is a record of existing active wells on the map of Utility lines - Chaguaramas, by Town and Country Planning Division.

It is said that these data were collected during dry season in 1956. The locations of these wells are shown on the map attached. Water is being pumped up to the top of the hills mainly for domestic supply.

Well No. 13 is the only one situated near the upstream section of the Cuese River. It is therefore not shown on the map). The others group on an alluvial fan at the mouth of a shallow valley on the lower right side of the Cuese River.

Well No.	Gal/min	Gal/day	m^3/day	Dia of wall	Depth well	Length of suction pipe	Horsepower of pump	Remarks
TV No. 3	250	360,000	1,638	8" - 6"	120'	ø=4" 6-'	25 HP	vertical centrifugal
TV No. 4	450	648,000	2,948	12"-8"-6"	140'	ø=4" 50'	40	turbine pump 220/
TV No. 5	115	165,600	754					440V 3 phase
TV No. 7	150	216,000	983					
TV No. 13	90	129,600	590					
TV No. 16	160	230,400	1,048	8"-6"	160'	ø=4" 70'	25	
TV No. 17	210	302,400	1,376	8"-6"	120'	ø=4" 70'	25	
Total	(150)	2,052,000	9,337					
Grand total		2,268,000	10,319					

DETAILS

1) Selection of the area to be irrigated

The area was selected due to its topographical condition and the locations of existing active wells as shown on the map attached.

(proposed area of No. 1 200 acres
(proposed area of NO. 2 200 acres

2) Determination of the water requirement maximum net water requirement for vegetables in a droughty year, once in five years, is 5.0 inches a month. (cf. Report: Tucker Valley (II).

5.0" = 127.0 mm
127.0/30 = 4.23 mm/day
4.23m/1,000 x 200 acres x 4,046.86 m^2/acre = 3,423.6 m^3/day

Assuming losses of conveyance as follows;

in pipes	5%
feeders	20%
total	25%

3,423.6 m^3/day x 1.25 = 4,280 m^3/day
= 942,000 gals/day

3) Determination of details of the wells

Assuming that a well like No. 7 (=216,000 gals/day) is capable of being bored,
942,000/216,000 ≒ 4.36 ≒ 5
five wells are necessary

In the proposed area No. 1, total water quantity which would well up from these five wells would be enough for the area. On the other hand in the proposed area No. 2, more than five wells may be necessary for the area, because the water quantity from the well may be less than the well No. 7.

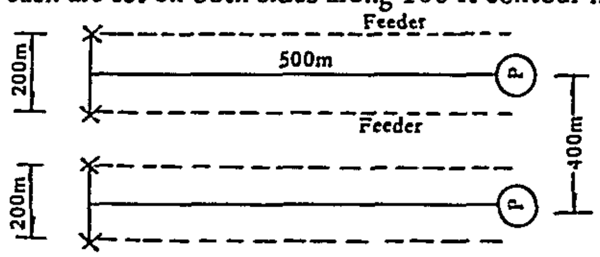
A diameter of 8"-6" and a depth of 150' will be necessary, based on data obtained from wells, No. 3 No. 16 and No. 17.

It is desirable that the locations in Area No. 1 be selected along the right side of the Cuese River and those in Area No. 2 be selected in the locality where the tributary goes underground and meets the Cuese River and those in Area No. 2 be selected in the locality where the tributary goes underground and meets the Cuese River.

4) Determination of details of the pipes

A plastic pipe is recommended for water conveyance, because it is very light and flexible and it can be connected very easily. Of course, it can be used for a long time as it is. The cost is not so expensive, because it is being made in Trinidad.

The pipe is to be arranged as shown in the following sketch. The pipe of 500m in length is set from the well up to the contour of 100ft, at that point it branches into two, and the branches of 100ft each are set on both sides along 100 ft-contour line.



The water is eventually discharged into feeders at the end of the branches.

Diameter of pipes.

$Q = 4{,}280 m^3/day = 0.0495 \approx 0.05\ m^3/sec.$

$\therefore\ q = 0.01 m^3/sec/pump$

If velocity in pipes are assumed to be less than 1.5 m/s,

the diameter will be (as follows;)

main pipe = 4 inches

branch pipe = 3 inches

$q = 0.01\ m^3/sec$	$q = 0.005\ m^3/sec$
d = 4.0 inches	(d = 3.0 inches
v = 1.20 m/s	(v = 1.12 m/sec
loss head h = 20 m/1,000 m	(h = 17m/1,000m

5) Determination of details of pumps actual head -30m (100ft)

Friction loss d = 40 m

$H = h_L = 20/1{,}000 \times 500 = 10.0m.$

d = 30"

$H = hl = \frac{17}{1{,}000} \times 100 = 1.7\ m$

Total head required

$\Sigma H = 30.0 + 10.0 + 1.7 = 41.7m$

taking clearance of 10%

$41.7 + 0.1 \times 41.7 \fallingdotseq 46m$

Type; Centrifugal turbine - 3 stage

(60 cycle 220V 11kW (= 15HP)

($q = 0.01m^3/sec$

H = 46m 5 sets are needed

diameter of suction end 80mm

diameter of delivery and 70mm

diameter of suction pipe 4"

6) Estimation of the cost

a) Initial cost for the proposed Area No. 1 (200 acres)

i) Boring

diameter 8" - 6"

depth 150' 5 spots (inclusive of bsetting casing)

5 x 150' = 750' 750' x $30/1' = $22,500

ii) Price of pumps and motors

pumps 5 sets x $1,500/set = $7,500

motor 5 sets x $750/set = $3,800

iii) Cost for installation of pumps and motors

5 sets x $200/set = $1,000

iv) Cost for pump house

12' by 18' x 5 houses = 1,080ft²

1,080ft² x $5/ft² = $5,400

v) Price of pipes and accessories

d = 40 inches 500m x 5 = 2,500m

2,500m x $48/6m = $20,000

d = 3.0 inches 100m x 2 x 5 = 1,000m

1,000m x $33/6m = $5,500

accessories 2% of above total $500

total $26,000.

vi) Cost for pipe disposition

d = 4.0 inches 2,500m/6m = 417

d = 3.0 inches 1,000m/6m = 167 total 584 pipes

584 pipes÷ 40 pipes/day = 14.6 ≒ 15 days

15 days x 4 persons x $8 ≒ $500

vii) Cost for transmission line of electricity

Assuming that a line of 300 yards on an average is extended from the present line to a new pump house,

5 spots x $200/100yds x 300yds = $3,000

viii) Cost for feeders (inclusive of setting them)

Assuming that U-shaped precast concrete channel is used,

∴ 50m/ha

200 acres x 0.405 ha/acre x 50m/ha x 1.2 = 4,860m

4,860m x $8/m = $38,900

ix) Contingency and total cost

above total:		$108,600
Contingency above 20%		$ 21,700
Total		$130,300
	≒	$131,000
	∴	$655/acre

b) Recurrent cost for Area No. 1 (200 acres)

i) Electricity charge per month

55kW x 24 hr x 30 days = 39,600 KWH

total capacity of transformer

55kW/0.9 x 0.7 ≒ 90 KVA

39,600 KHW x 1.60 ¢ /KWH = $634

90KVA x $7.15/KVA = $644

total $1,278/month

ii) Cost for maintenance

1 person x $250 = $250

Oil, grease and the like $50

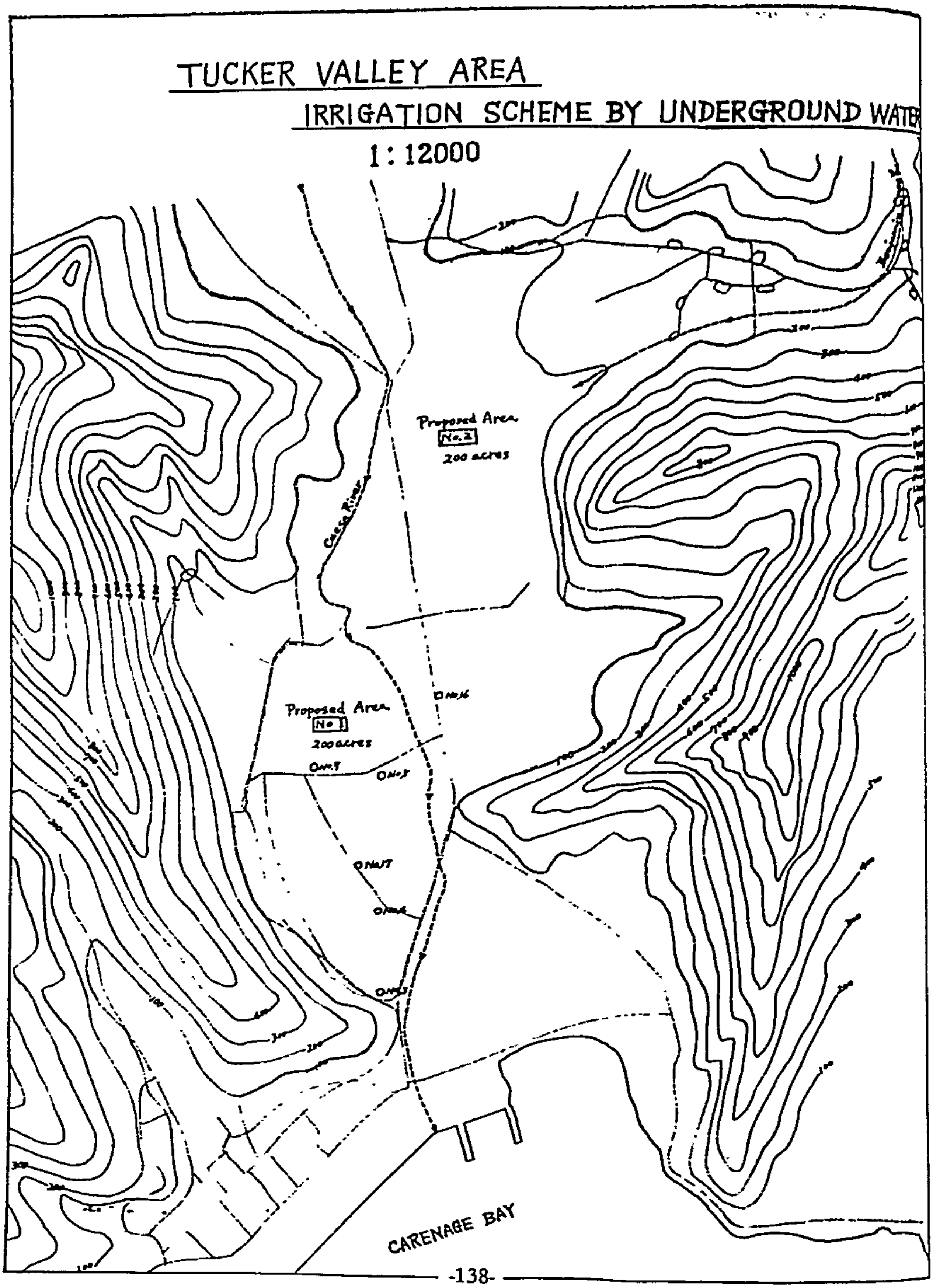
$300

Total cost $1,600/month

Supplement Acreage of agricultural land in Tucker Valley.

Contour	Acreage
500ft-200ft	1,032
200ft-100ft	378
100ft	960
Total	2,370

Junichi Kitamura


TUCKER VALLEY AREA
IRRIGATION SCHEME BY UNDERGROUND WATER
1 : 12000
Proposed Area
No.2
200 acres
Proposed Area
No.1
200 acres
CARENAGE BAY

TUCKER VALLEY (VI) May, 1968

MOVEMENT

7, Feb.	Requested additional survey of proposed dam site
5, Mar.	Undertook preparation of contour map.
12, Mar.	Determined the center line of the dam.
13, Mar.	Started preparing standard detailed cross section, plan, longitudinal section and typical cross sections of dam.
22, Apr.	Calculated earth volume for cutting and filling.

INTRODUCTION

In reports (I) to (III), the scale and the approximate location of the dam were determined.

In report (IV), the dam volume was estimated by a simple formula and an approximate cost of the dam was determined.

In report (V), an interim irrigation scheme using sub-surface water was established for 200 acres.

In this report (VI) the dam center will be determined and the detailed earth volume required will be calculated as the survey for the proposed dam site has been carried out. Moreover Sediment volume will be estimated.

(1) Determination of the location of the dam

For the Survey, an assumed center line was set up so as to make a right angle with the existing main Tucker Valley Road, by referring to A-A and B-B, lines which were assumed in the calculation of the storage capacity in report (III).

However, the actual center of the dam was determined as that axis having a point of rotation which lies on the center line assumed for surveys, 500 ft south of the road, and being rotated 120-10' east about its point of rotation. This line was based on the contour map prepared from the Survey information and is shown on plan.

(2) Drawing up the profile

The profile was drawn up by measuring distances and hiehgts on the 1 foot contour map. Bench mark used was based on T.G.R. datum for which the mean sea level is 96.92 ft.

All the levels in the reports so far, however, have not been related to the T.G.R. datum but to a datum for which the mean sea level was adopted as 0.00 ft. Attention to the levels this time must therefore be paid.

The crest level is determined as follows:

Crest level = Full water level + Clearance = 340' + 10' = 350'

Full water level: = cf. Report (III)

Clearance = Height of wave due to wind + Overflow depth + Height for safety

= 3.0' + 3.3' + 3.3' = 9.6' ≑ 10'

From consideration of the surface configuration the bedrock level was assumed to be, as shown on profile.

The minimum level is 250 ft which results in a maximum cutting depth of 22 ft.

(3) Drawing up the standard cross section

Rock-fill dam-type with a central core wall is recommended. Depth of topsoil to be scraped may be assumed to be 3 ft. Clay for the core wall, sand for the filter zone, river bed deposits for the random zone and limestone for rock zone must be used. These materials should be obtained as near as possible to the site so as to minimize the cost of transportation. As there is, according to the geological map, a random distribution of limestone about the site, the use of this material is recommended.

If the entire requirement cannot be supplied near the site. supplemental use of schist will have to be considered. In this case, physical and dynamic test for the schist will be necessary. The slope of the upper side of the dam is assumed to be 1:2.2 and that of the lower side to be 1:20.

The slope must be finally determined by stability analysis based on the results of Tests on the materials.

Berms 10ft wide at an elevation fo 290ft are set on each slope.

(4) Making cross sections

Volume of excavation may be classified into:

(i) volume of top soil to be scraped

(ii) volume of trench for corewall

Volume of fill may also be classified into

(i) volume for core wall

(ii) the other zones

(5) Results of summing up volume

The result is shown on table 1 and 2, that is,

volume of 'cut' - 29,000m^3 (38,000 yd^3)

volume, of 'fill - 235,000m^3 (307,000 yd^3)

Thus, efficiency of storage = Storage capacity/'Fill' volume = $3,000,000m^3/235,000m^3$ = 12.8

(The efficiency is mostly between 10 and 5 in Japan) and its minimum is below 5.

This volume of 'fill' $235,000m^3$ is 25% more than the volume of $187,500m^3$ which was estimated by a simple formula in Report (IV).

The quality of materials and the location of the bed rock-surface, however, have not been confirmed yet. Slope of dam and depth to be excavated are therefore still changeable. Thus, the most that can be said is that the actual volume of 'fill' lies within the range of $200,000m^3 \pm 50,000m^3$.

(6) Estimation of sediment volume expected

Dimension:

Catchment area - 6.97 km^2 (= 2.69 $miles^2$)
Geology - Schist and limestone
Maximum difference in elevation - 450 m (= 1,500ft)
Slope of river bed at the site - about 1/50
Mean yearly rainfall - 60 inches
Temperature - 90° - 65°F

The unit sediment volume in such an area as has the above dimensions can be regarded as 100-300m^3/Km^2/year judging from the data observed in many existing dams in Japan.

The maximum value which is $300m^3/km^2$/year being adopted the estimated sediment volume will be

$$6.97\ km^2 \times 300\ m^3/km/year = 2,100m^3/year$$

As the storage capacity of the dam is about $3,000,000m^3$ The yearly sediment volume is only 0.07% of this capacity. This will present no problem in the near future.

(7) Items of investigation needed hereafter:-

(a) Investigation for materials
- o Determination of location of borrow-pit and quarry by test boring and pitting.
- o Estimation of the volume capable of being supplied by test boring and pitting.
- o Evaluation of the quality of materials by physical and dynamic test.

(b) Investigation for base of dam
Confirmation of bedrock by boring

(8) Items of study needed hereafter

(a) Stability analysis based on results of soil and rock tests.
- o Determination of dam slope.

(b) Computation of volume of 'Cut' and 'Fill' based on determination of dam slope and bedrock level.

   o Determination of dam volume.

(c) Design of by-pass channel, spill-way and intake equipment.

Junichi Kitamura

(Table 1) Calculation of Earth Volume

Section Nos.	Interval	Total				Items				Trench for core
		(A) Cutting				(B) Topsoil to be scraped				
	ft/(m)	Reading of Planimeter	Actual area	Mean Area	Volume	Reading of planimeter	Actual area	Mean Area	Volume	(A) - (B)
		(c.m^2)	(m^2)	(m^2)	(m^3)	(cm^2)	(m^2)	(m^2)	(m^3)	(m^3)
1 + 28	0/0.00	0.0	0	0	0.0	0	0	0	0	0
1 + 31	3/0.91	2.3	13.2	6.6	6	2.3	13.2	6.6	6	0
2	19/5.79	3.4	19.6	16.4	95	3.4	19.6	16.4	95	0
2 + 36	36/10.97	6.3	36.3	28.0	307	6.3	36.3	28.0	307	0
3	14/4.27	7.8	44.9	40.6	173	7.8	44.9	40.6	173	0
4	50/15.24	9.9	57.0	51.0	777	9.9	57.0	51.0	777	0
5	50/15.24	12.6	72.6	64.8	988	12.6	72.6	64.8	988	0
5 + 23	23/7.01	13.2	76.0	74.3	521	13.2	76.0	74.3	521	0
6	27/8.23	13.7	78.9	77.5	638	13.7	78.9	77.5	638	0
6 + 37	37/11.28	15.3	88.1	83.5	942	15.3	88.1	83.5	942	0
7	13/3.96	14.9	85.8	87.0	345	14.9	85.8	87.0	345	0
8	50/15.24	15.9	91.6	88.7	1,352	15.9	91.6	88.7	1,352	0
8 + 31	31/9.45	15.4	88.7	90.2	852	15.4	88.7	90.2	852	0
9	19/5.79	23.0	132.5	110.6	640	18.1	104.3	96.5	559	81
9 + 35	35/10.67	28.6	164.7	148.6	1,586	15.7	90.4	97.4	1,039	547
10	15/4.57	31.6	182.0	173.4	792	15.5	89.3	89.9	411	381
11	50/15.24	38.9	224.1	203.1	3,095	21.6	124.4	106.9	1,629	1,466
12	50/15.24	28.4	163.6	193.9	2,955	15.4	88.7	106.6	1,625	1,330
13	50/15.24	28.1	161.9	162.8	2,481.	17.5	100.8	94.8	1,445	1,036
14	50/15.24	22.8	131.3	146.6	2,234	15.8	91.0	95.9	1,462	772
15	50/15.24	18.8	108.3	119.8	1,826	15.3	88.1	89.6	1,366	460
15 + 32	32/9.75	16.8	96.8	102.6	1,000	14.8	85.2	86.7	845	155
16	18/5.49	12.7	73.2	85.0	467	11.3	65.1	75.2	413	54
17	50/15.24	13.2	76.0	74.6	1,1[illegible]7	11.2	64.5	64.8	988	149

Section Nos.	Interval	Total				Items				Trench for core
		(A) Cutting				(B) Topsoil to be scraped				
	ft/(m)	Reading of Planimeter	Actual Area	Mean Area	Volume	Reading of Planimeter	Actual Area	Mean Area	Volume	(A) - (B)
		(c.m^2)	(m^2)	(m^2)	(m^3)	(cm^2)	(m^2)	(m^2)	(m^3)	(m^3)
17 + 19	19/5.79	12.7	73.2	74.6	432	11.9	68.5	66.5	385	47
18	31/9.45	12.7	73.2	73.2	692	11.8	68.0	68.3	645	47
19	50/15.24	10.0	57.6	65.4	997	9.5	54.7	61.4	936	61
20	50/15.24	8.4	48.4	53.0	808	8.4	48.4	51.6	786	22
21	50/15.24	5.7	32.8	40.6	619	5.7	32.8	40.6	619	0
21 + 23	23/7.01	3.1	17.9	25.4	178	3.1	17.9	25.4	178	0
21 + 33	10/3.05	2.0	11.5	14.7	45	2.0	11.5	14.7	45	0
21 + 35	2/0.61	0.0	0	5.8	4	0.0	0	5.8	4	0
Grand total	1007/306.93				28,984				22,376	6,608

(Table 2)

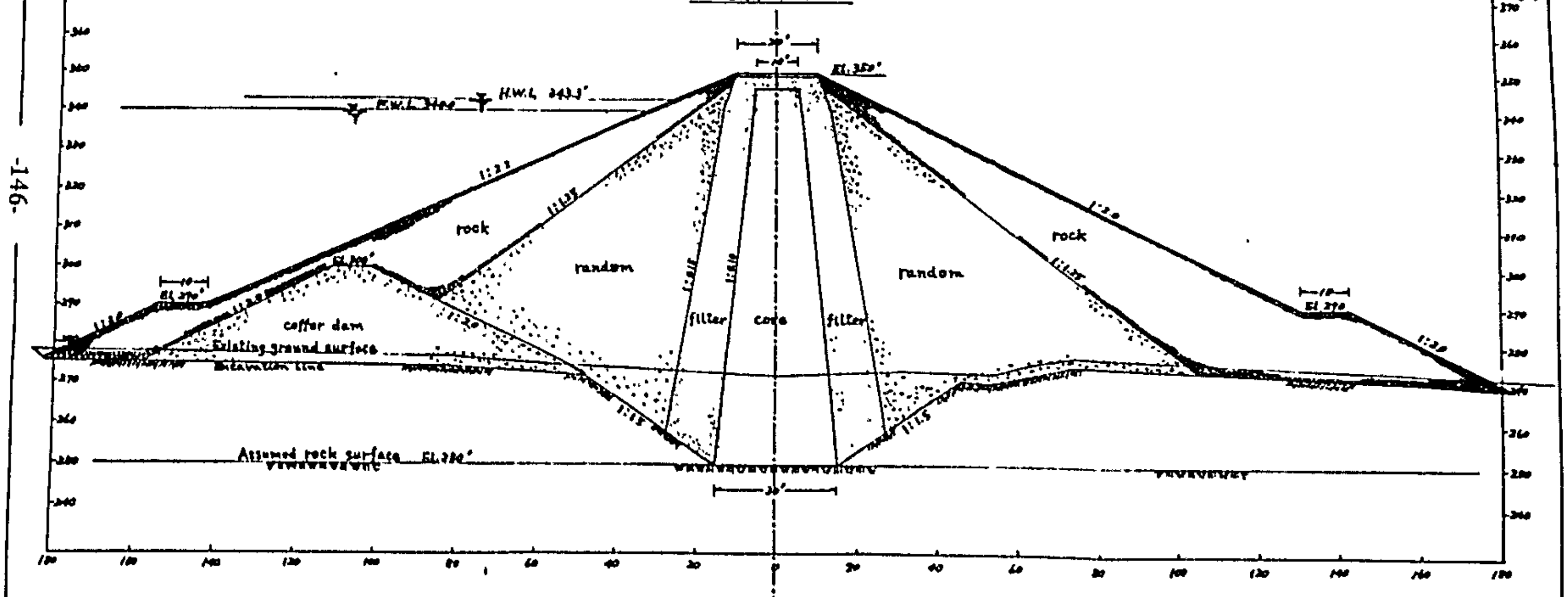
Section Nos.	Interval	Total				Items				
		(D) Filling				(E) For Corewall				(F) For the others
	(ft)/(m)	Reading of Planimeter	Actual Area	Mean Area	Volume	Reading of Planimeter	Actual Area	Mean Area	Volume	(D) - (E)
		(cm^2)	(m^2)	(m^2)	(m^3)	(cm^2)	(m^2)	(m^2)	(m^3)	(m^3)
1 + 28	0/0.00	0.0	0	0	0	0.0	0	0	0	0
1 + 31	3/0.91	2.0	11.5	5.8	5	0.0	0	0	0	5
2	19/5.79	8.1	46.7	29.1	168	1.3	7.5	3.8	23	145
2 + 36	36/10.97	28.9	166.5	106.6	1,169	3.6	20.7	14.1	155	1,014
3	14/4.27	33.4	192.4	179.5	766	3.9	22.5	21.6	92	674
4	50/15.24	54.2	312.2	252.3	3,845	5.3	30.5	26.5	404	3,441
5	50/15.24	82.5	475.2	393.6	5,998	7.1	40.9	35.7	544	5,454
5 + 23	23/7.01	81.6	470.0	472.6	3,313	6.4	36.9	36.9	273	3,040
6	27/8.23	87.2	562.3	486.2	4,001	6.3	36.3	36.6	301	3,700
6 + 37	37/11.28	102.2	588.7	545.5	6,153	6.4	36.9	36.6	413	5,740
7	13/3.96	111.3	641.1	614.9	2,435	6.9	39.7	38.4	152	2,283
8	50/15.24	186.3	1,073.1	857.1	13,062	12.3	70.8	55.3	843	12,219
8 + 31	31/9.45	226.6	1,305.2	1,189.2	11,238	20.4	117.5	94.2	890	10,348
9	19/5.79	244.3	1,407.2	1,356.2	7,852	24.9	143.4	130.5	756	7,096
9 + 35	35/10.67	251.0	1,445.8	1,426.5	15,221	28.0	161.3	152.4	1,626	13,595
10	15/4.57	248.1	1,429.1	1,437.5	6,569	28.9	166.5	163.9	749	5,820
11	50/15.24	244.0	1,405.4	1,417.3	21,600	28.0	161.3	163.9	2,498	19,102
12	50/15.24	234.5	1,350.7	1,378.1	21,002	25.6	147.5	154.4	2,353	18,649
13	50/15.24	228.4	1,315.6	1,333.2	20,318	24.7	142.3	144.9	2,208	18,110
14	50/15.24	212.7	1,225.2	1,270.4	19,361	23.6	135.9	139.1	2,120	17,241
15	50/15.24	188.5	1,085.8	1,155.5	17,610	19.6	112.9	124.4	1,896	15,714
15 + 32	32/9.75	157.0	904.3	995.1	9,702	16.2	93.3	103.1	1,005	8,697
16	18/5.49	140.4	808.7	856.5	4,702	14.6	84.1	88.7	487	4,215
17	50/15.24	114.2	657.8	733.3	11,175	12.1	69.7	76.9	1,172	10,003
17 + 19	19/5.79	112.9	650.3	654.1	3,787	10.8	62.2	66.0	382	3,405
18	31/9.45	100.8	580.6	615.5	5,816	10.8	62.2	62.2	588	5,228
19	50/15.24	76.4	440.1	510.4	7,778	8.9	51.3	56.8	866	6,912
20	50/15.24	55.8	321.4	380.8	5,803	6.8	39.2	45.3	690	5,113
21	50/15.24	28.5	164.2	242.8	3,700	3.9	22.5	30.9	471	3,229
21 + 23	23/7.01	10.3	59.3	111.8	784	1.6	9.2	15.9	111	673
21 + 23	23/7.01	10.3	59.3	111.8	784	1.6	9.2	15.9	111	673
21 + 33	10/3.05	1.7	9.8	34.6	106	0.0	0	4.6	14	92
21 + 35	2/0.61	0.0	0	4.9	3	0.0	0	0	0	3
Grand Total	1007/306.93				235,042				24,082	210,960

(Fig.1)

STANDARD CROSS SECTION
OF TUCKER VALLEY DAM

1" = 20'

SECTION No.10



(Fig.2)

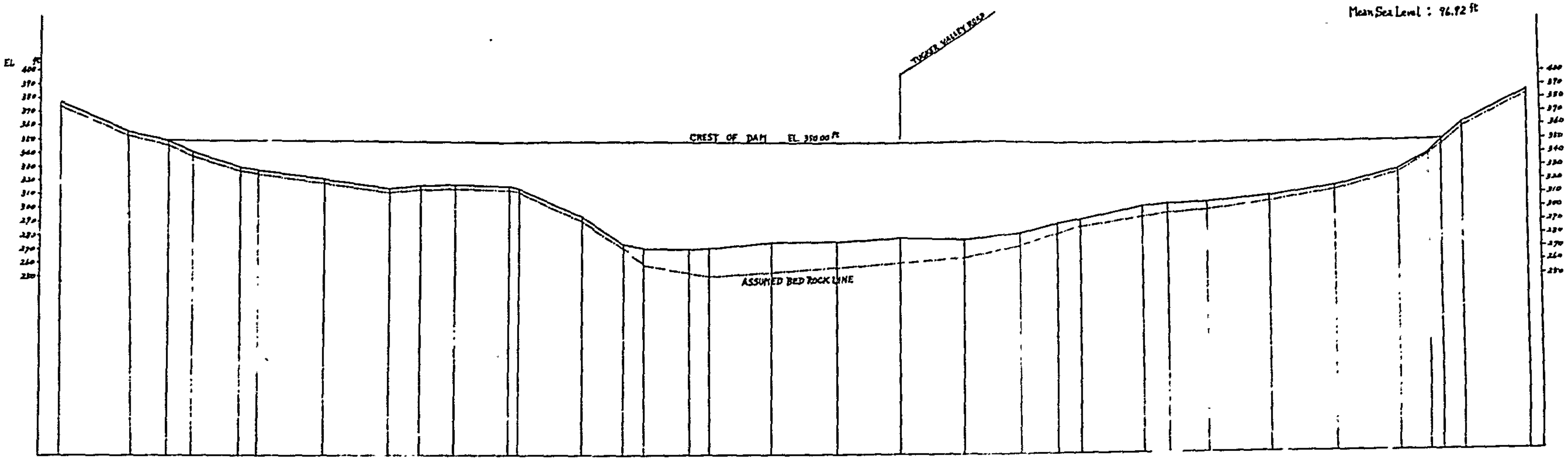
LONGITUDINAL SECTION OF PROPOSED DAM SITE IN TUCKER VALLEY

SCALE: VERTICAL 1" = 40'
HORIZONTAL 1" = 40'

Levels shown on this drawing are based on the following B.M. □ mark with arrow in center of bridge B 1/8 over Industry River on Western Main Road.

Level Book : 866/1

Mean Sea Level : 96.92 ft

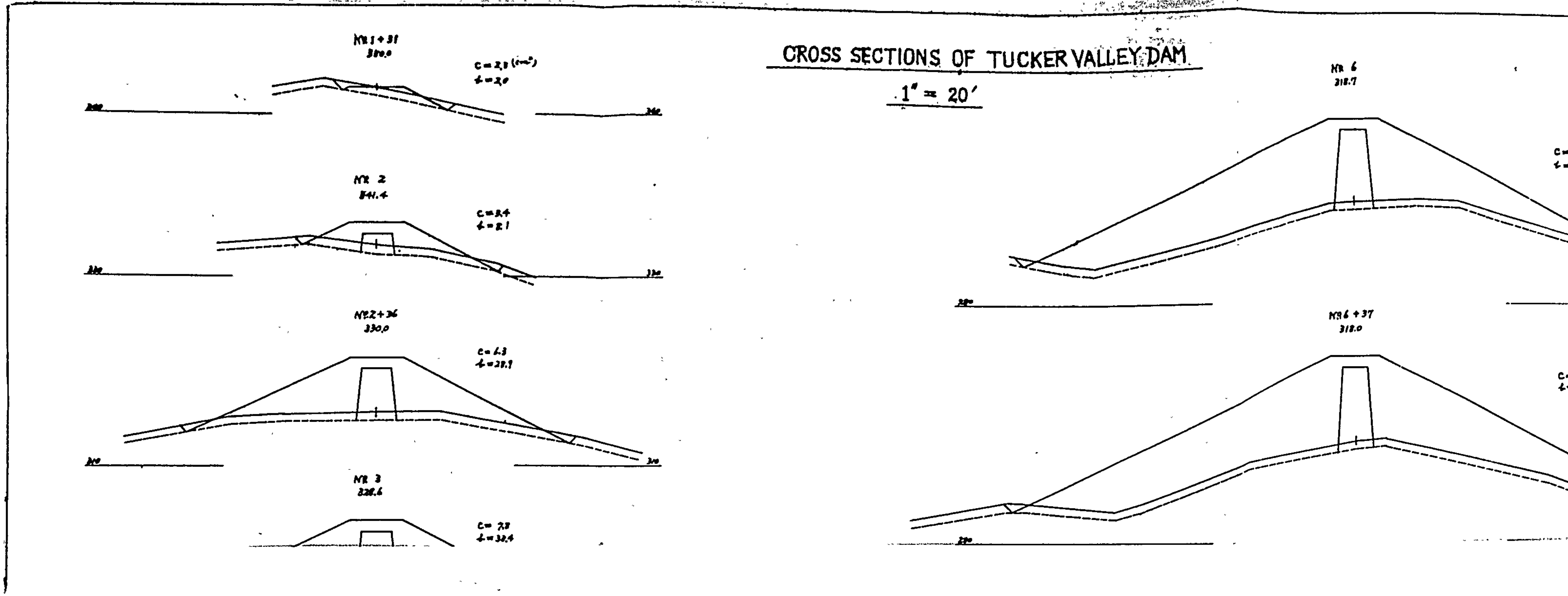


SECTION NO.S	INTERVAL IN FEET	TOTAL DISTANCE IN FEET	EXISTING GROUND LEVEL	ASSUMED BED ROCK LEVEL	CUTTING HEIGHT
No. 0	0	0	377.0	374.0	3.0
No. 1	50	50	356.0	353.0	3.0
+ 31	31	81	350.0	347.0	3.0
No. 2	19	100	341.4	338.0	3.4
+ 36	36	136	330.0	327.0	3.0
No. 3	14	150	328.6	326.0	2.6
No. 4	50	200	322.0	319.0	3.0
No. 5	50	250	315.5	312.0	3.5
+ 23	23	273	317.0	314.0	3.0
No. 6	27	300	318.7	315.0	3.7
+ 37	37	337	318.0	315.0	3.0
No. 7	13	350	312.8	313.0	2.8
No. 8	50	400	294.9	292.0	2.9
+ 31	31	431	295.0	292.0	3.0
No. 9	19	450	272.3	261.0	11.3
+ 35	35	485	274.6	255.0	19.6
No. 10	15	500	277.0	255.0	22.0
No. 11	50	550	276.2	254.0	22.2
No. 12	50	600	277.4	259.0	18.4
No. 13	50	650	279.7	262.0	17.7
No. 14	50	700	279.8	266.0	13.8
No. 15	50	750	282.7	274.0	8.7
+ 32	32	782	290.0	283.0	7.0
No. 16	18	800	293.3	288.0	5.3
No. 17	50	850	303.3	295.0	8.3
+ 19	19	869	305.2	299.0	6.2
No. 18	31	900	305.5	300.0	5.5
No. 19	50	950	310.1	306.0	4.1
No. 20	50	1000	318.8	314.0	4.8
No. 21	50	1050	328.2	326.0	2.2
+ 23	23	1073	340.0	338.0	2.0
+ 33	10	1083	350.0	347.0	3.0
No. 22	17	1100	362.1	357.0	3.1
No. 23	50	1150	376.5	374.0	2.5

By J. Kitamura

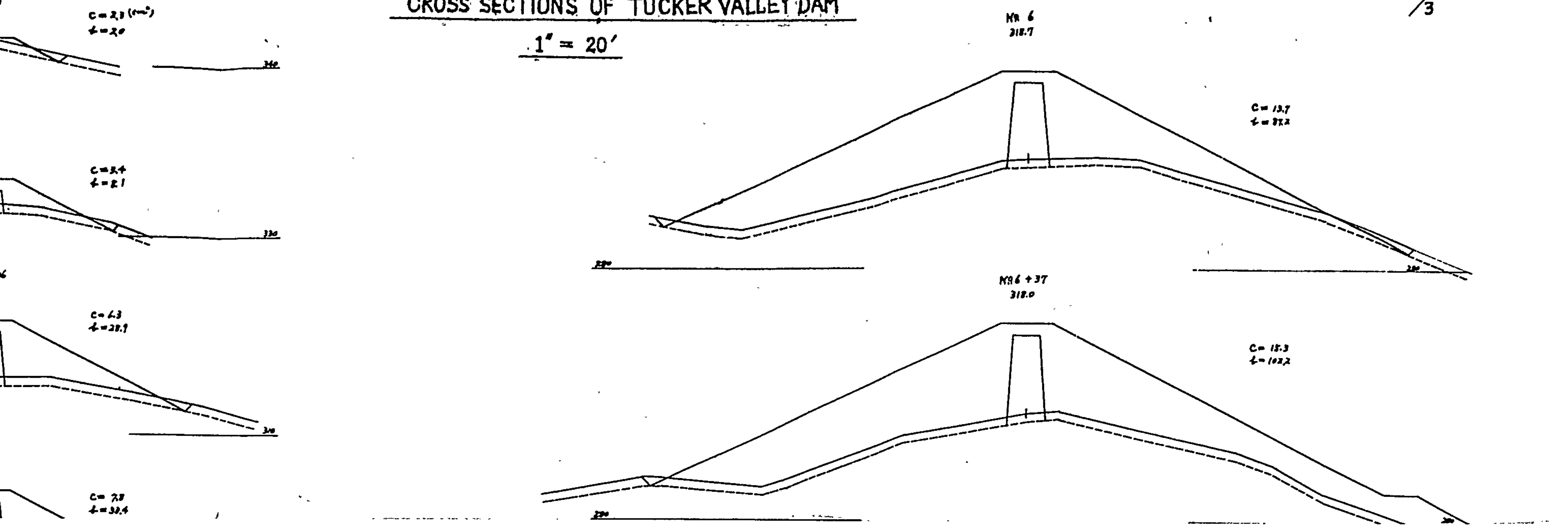
CROSS SECTIONS OF TUCKER VALLEY DAM

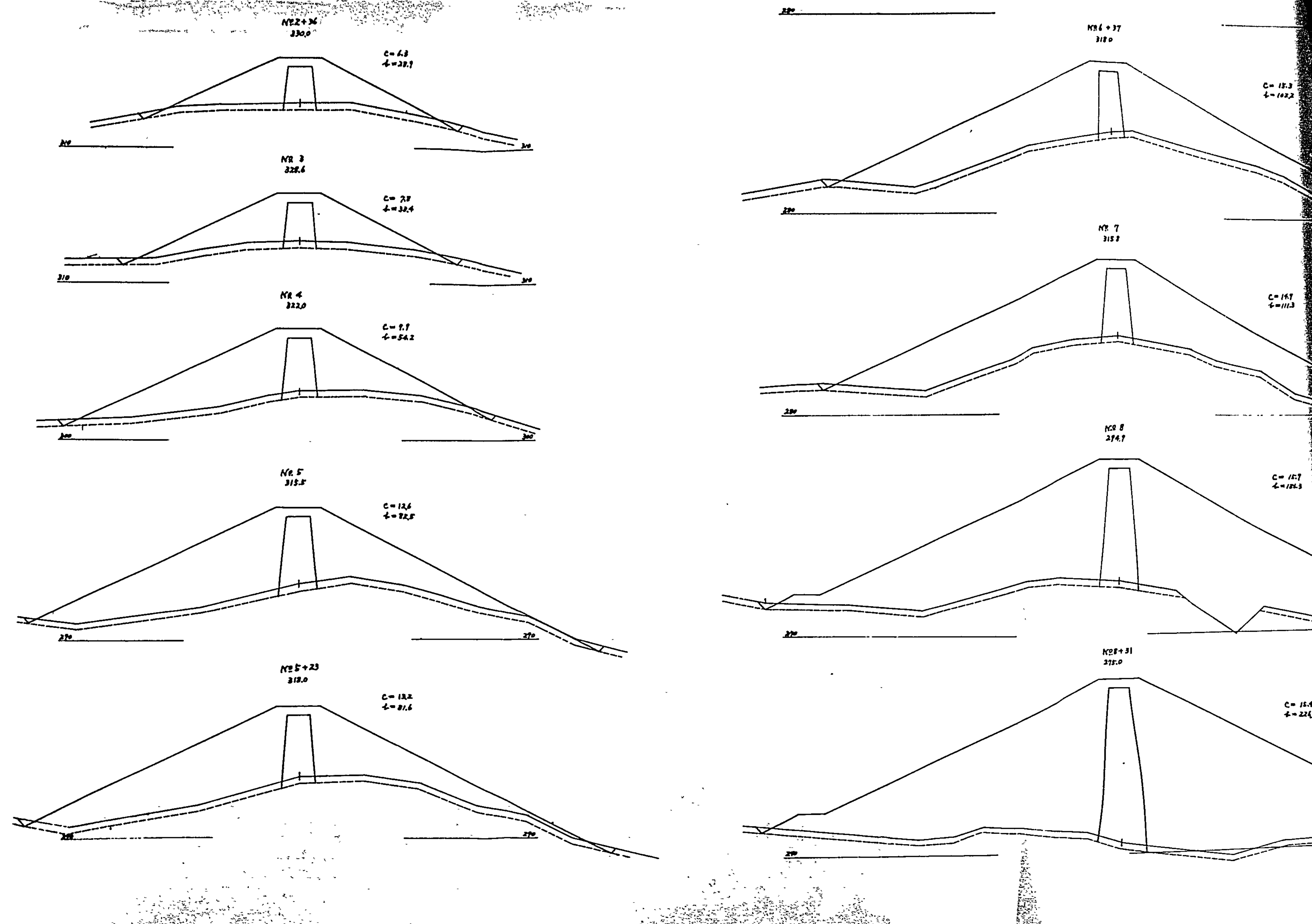
1" = 20'



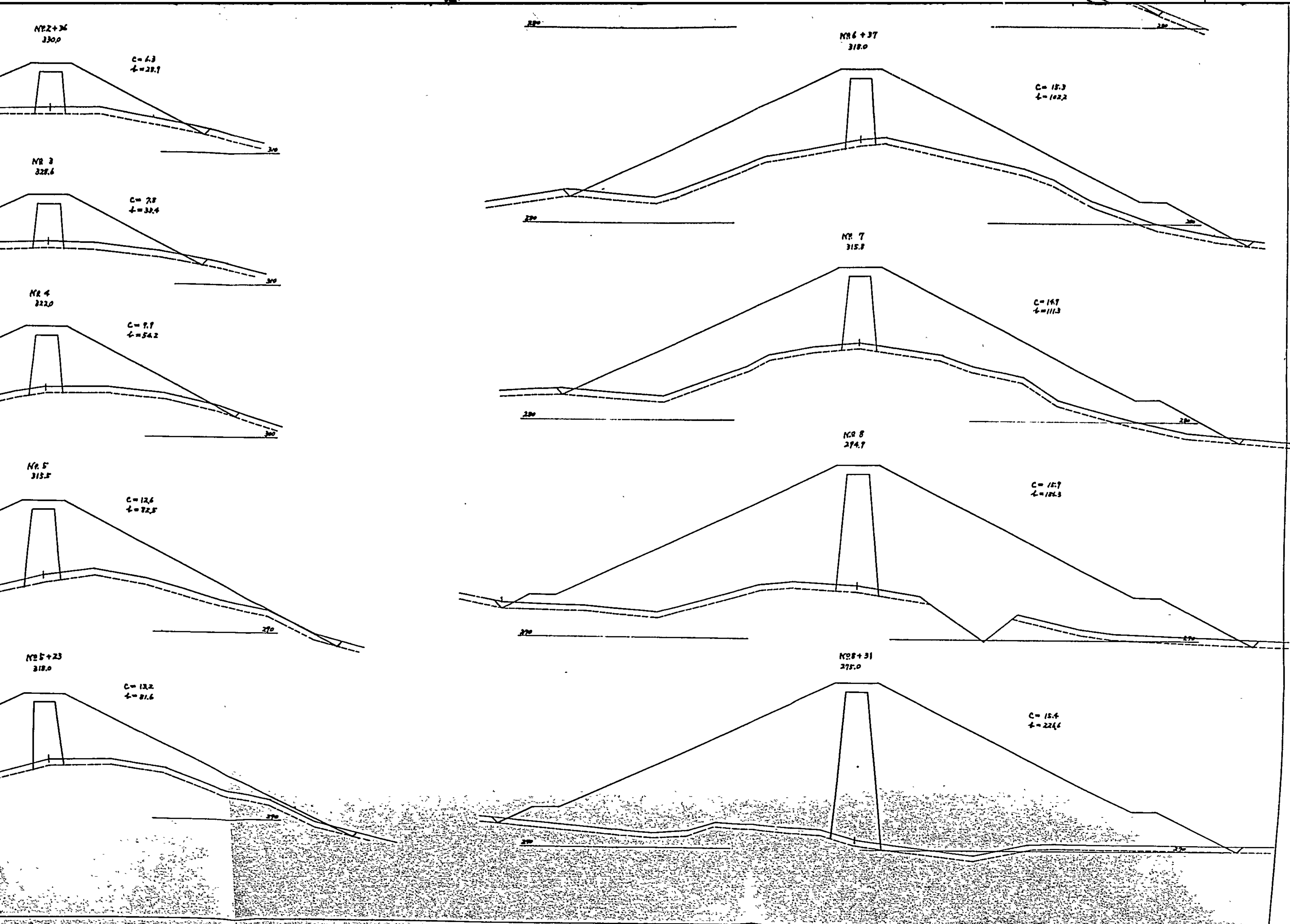
CROSS SECTIONS OF TUCKER VALLEY DAM

1" = 20'





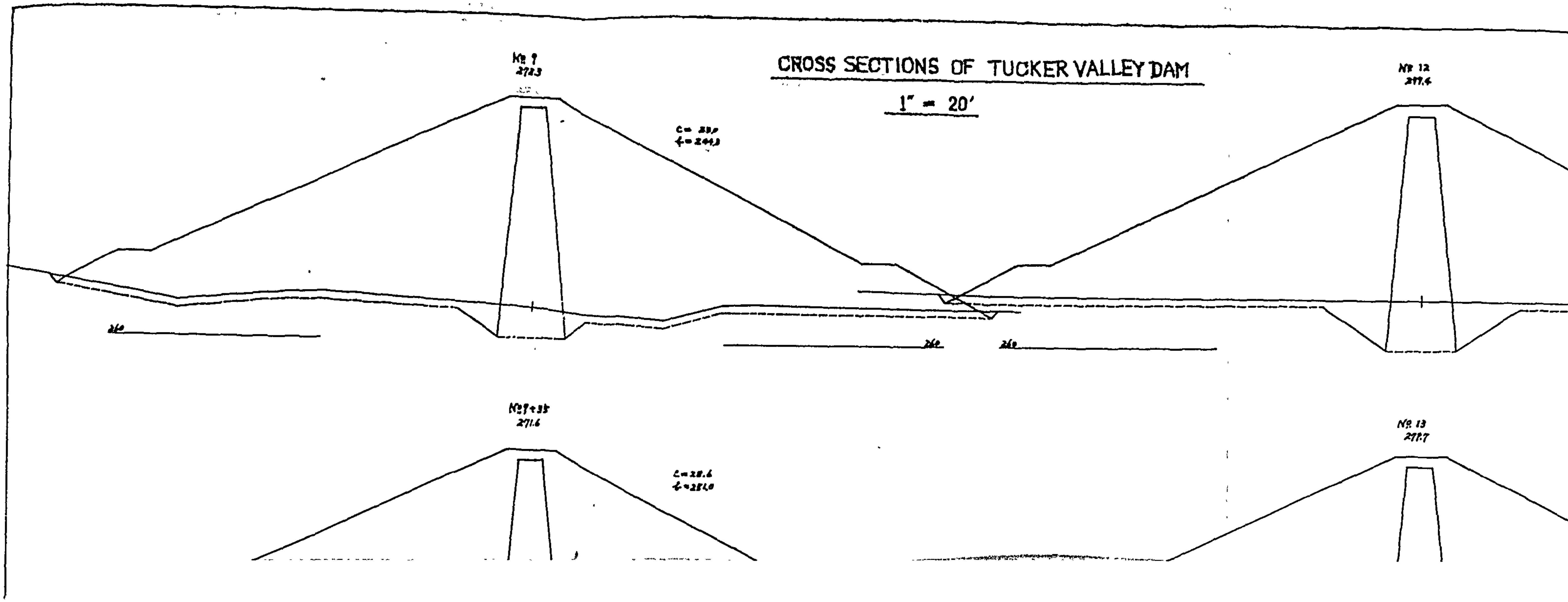
NR 2+36
330,0
c= 6.8
NR 3
328.6
c= 7.8
NR 4
322,0
c= 9.9
NR 5
315.5
c= 12,6
NR 5+23
313.0
c= 13.2
NR 6+37
318.0
c= 15.3
NR 7
NR 8
294.9
NR 8+31
275.0



№2+36
330,0
c= 6.3
f=28.9
№ 3
328.6
c= 7.8
f=33.4
№ 4
322,0
c= 9.9
f=54.2
№ 5
315.5
c= 12.6
f=82.5
№5+23
318.0
c= 13.2
f=81.6
№6+37
318.0
c= 15.3
f=102,2
№ 7
315.8
c= 14.9
f=111.3
№ 8
294.9
c= 15.9
f=186.3
№8+31
295.0
c= 15.4
f=226,6

CROSS SECTIONS OF TUCKER VALLEY DAM

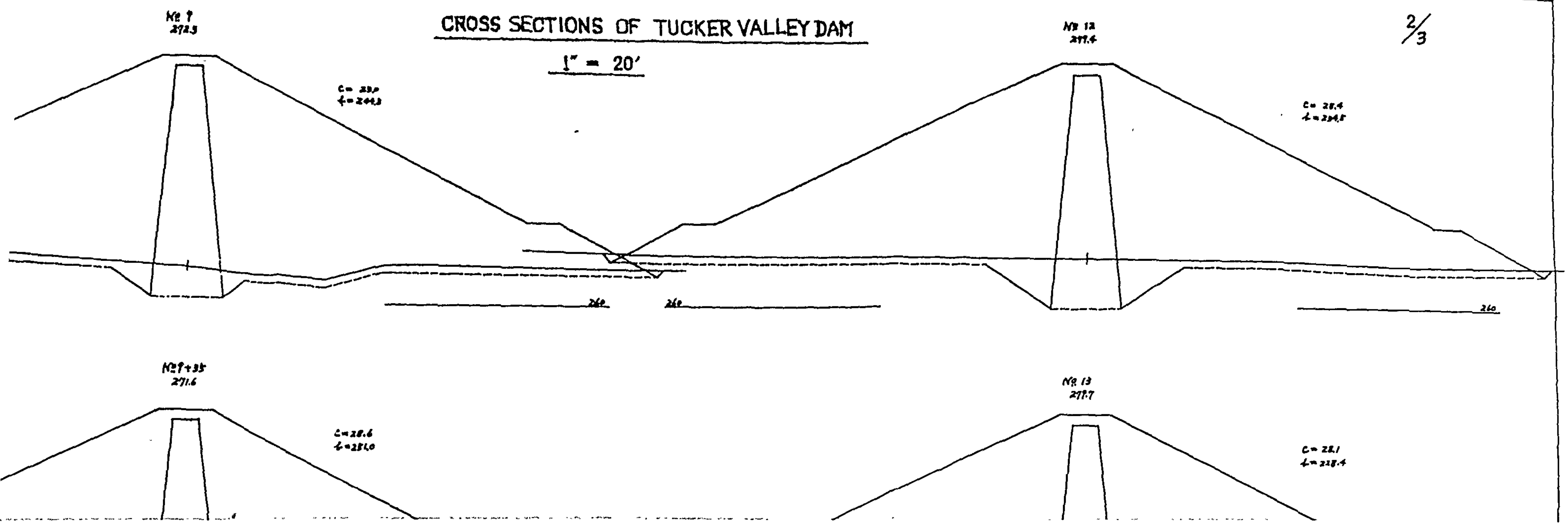
1" = 20'

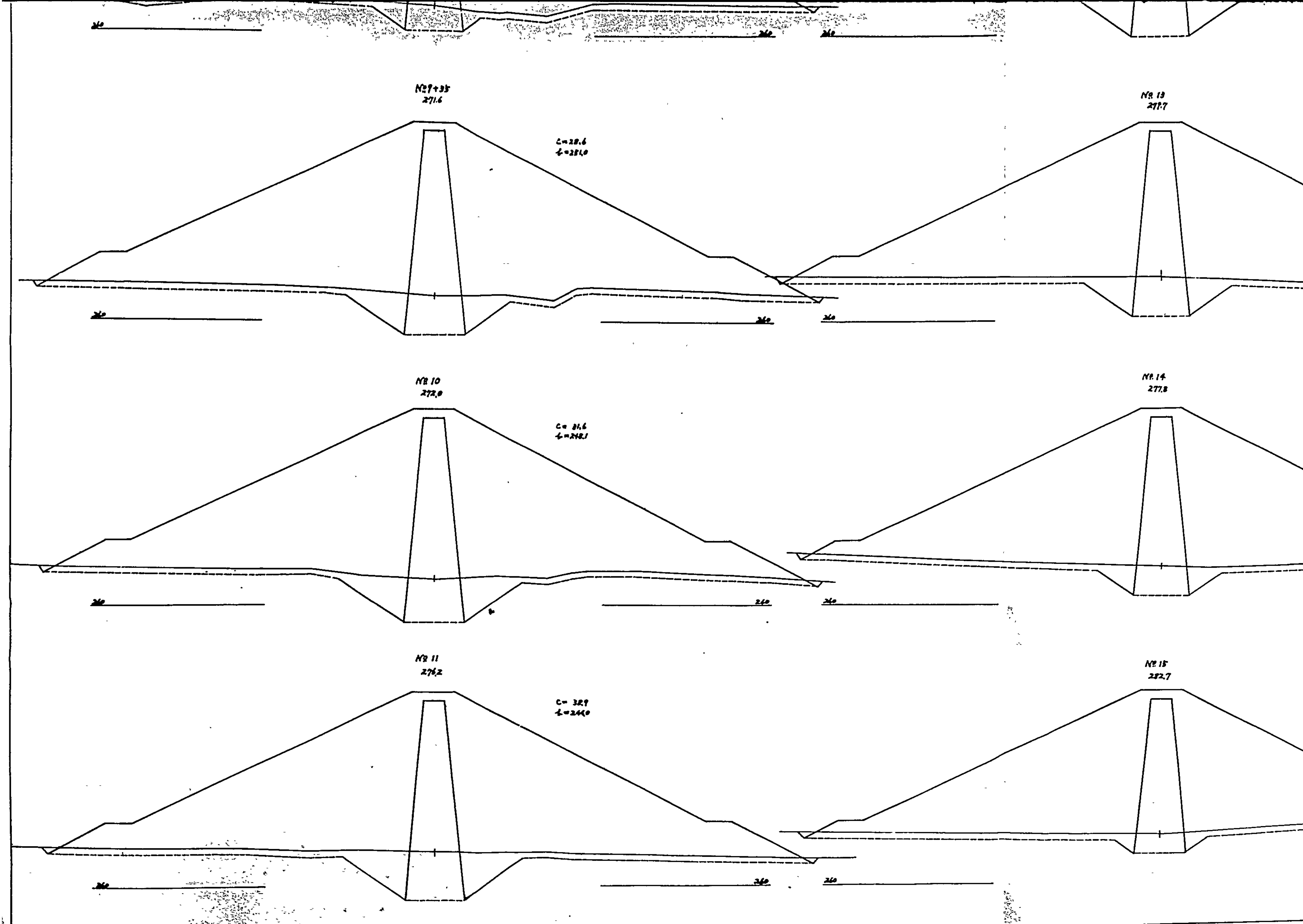


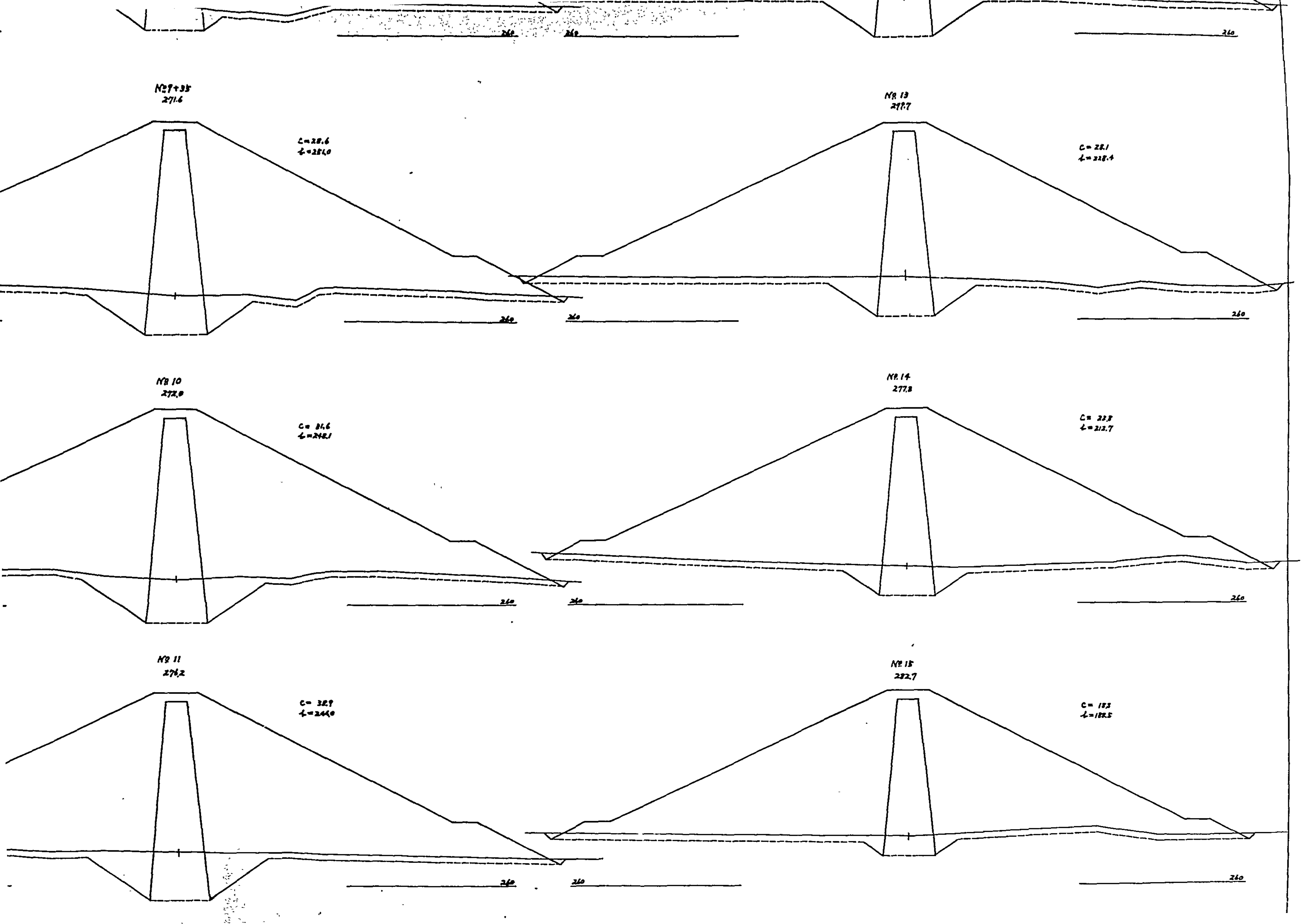
CROSS SECTIONS OF TUCKER VALLEY DAM

1" = 20'

2/3



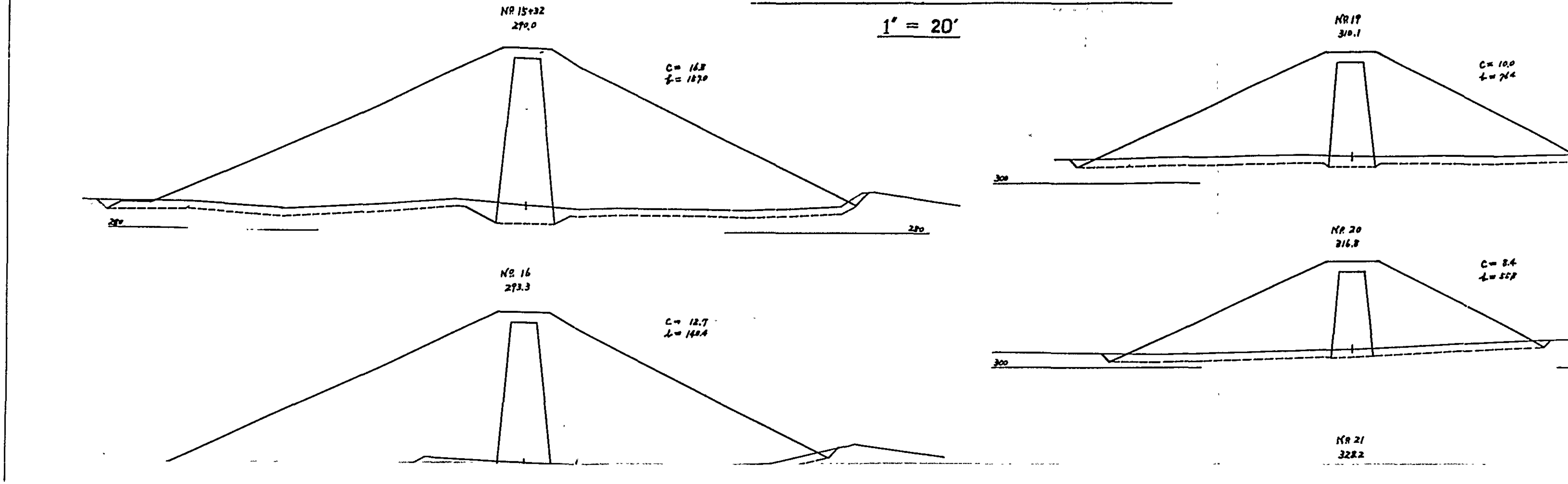

№ 9+35
271.6
c=28.6
№ 10
272.0
c= 31.6
№ 11
276.2
c= 32.9
№ 13
№ 14
277.8
№ 15
282.7

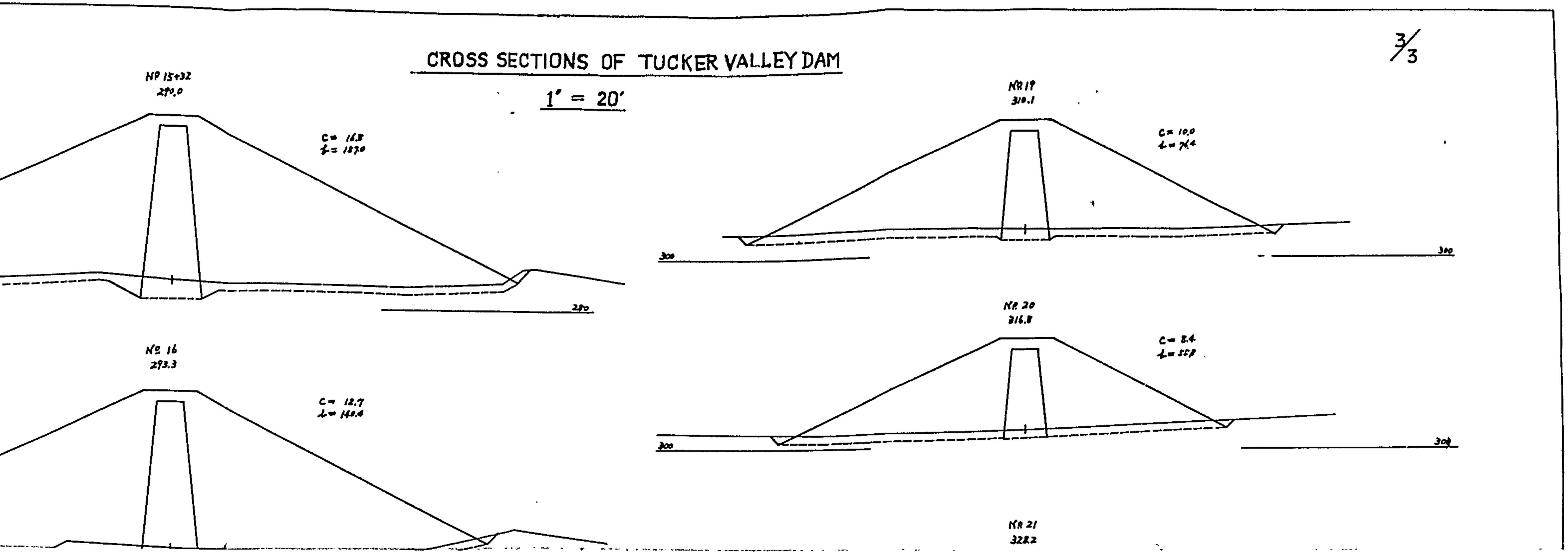


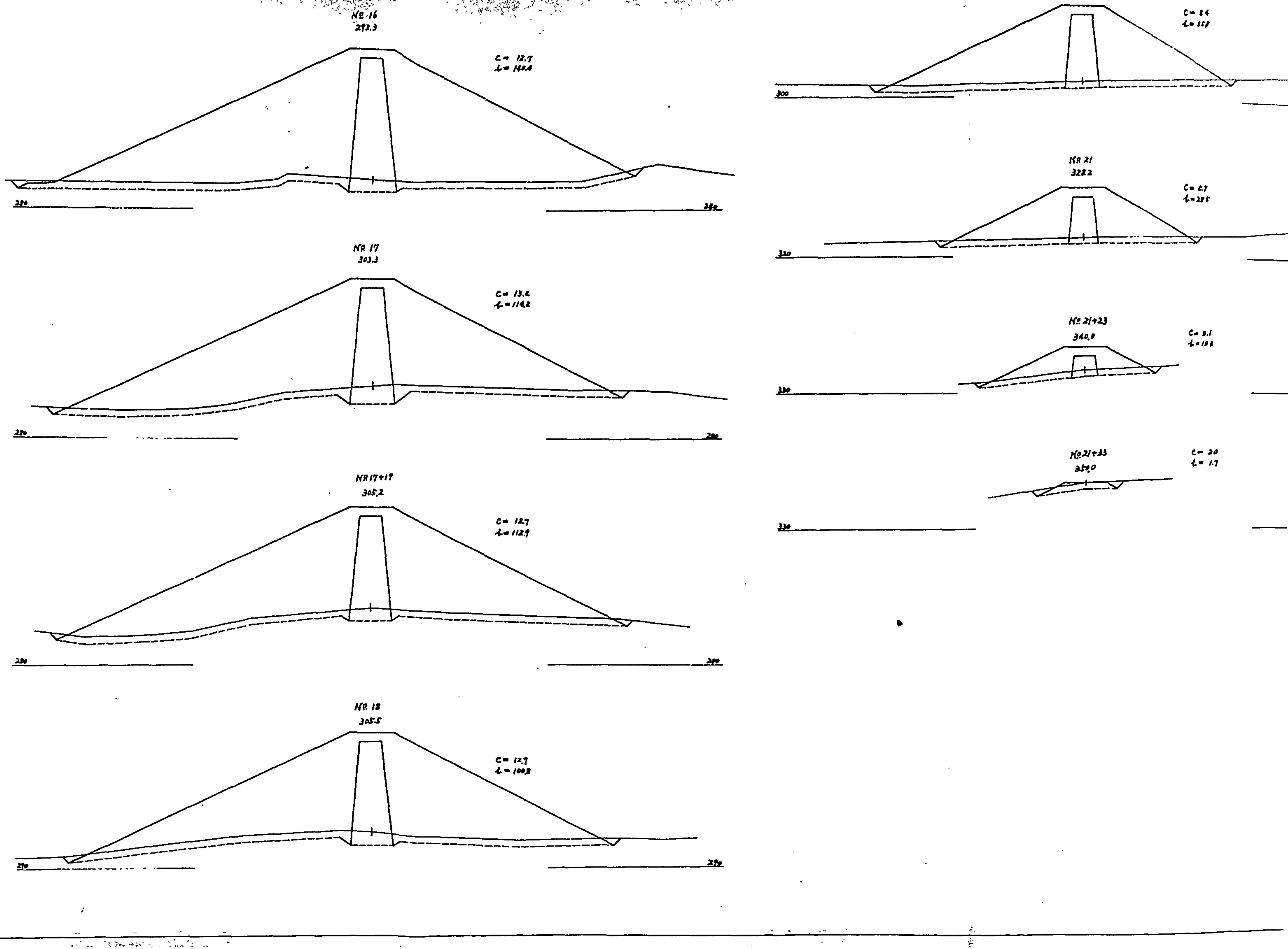
№ 9+35
271.6
C=28.6
ℓ=251.0
№ 13
279.7
C=28.1
ℓ=228.4
№ 10
272.0
C= 31.6
ℓ=248.1
№ 14
277.8
C= 22.8
ℓ=212.7
№ 11
276.2
C= 32.9
ℓ=244.0
№ 15
282.7
C= 18.3
ℓ=182.5
260
260
260
260
260
260
260
260
260

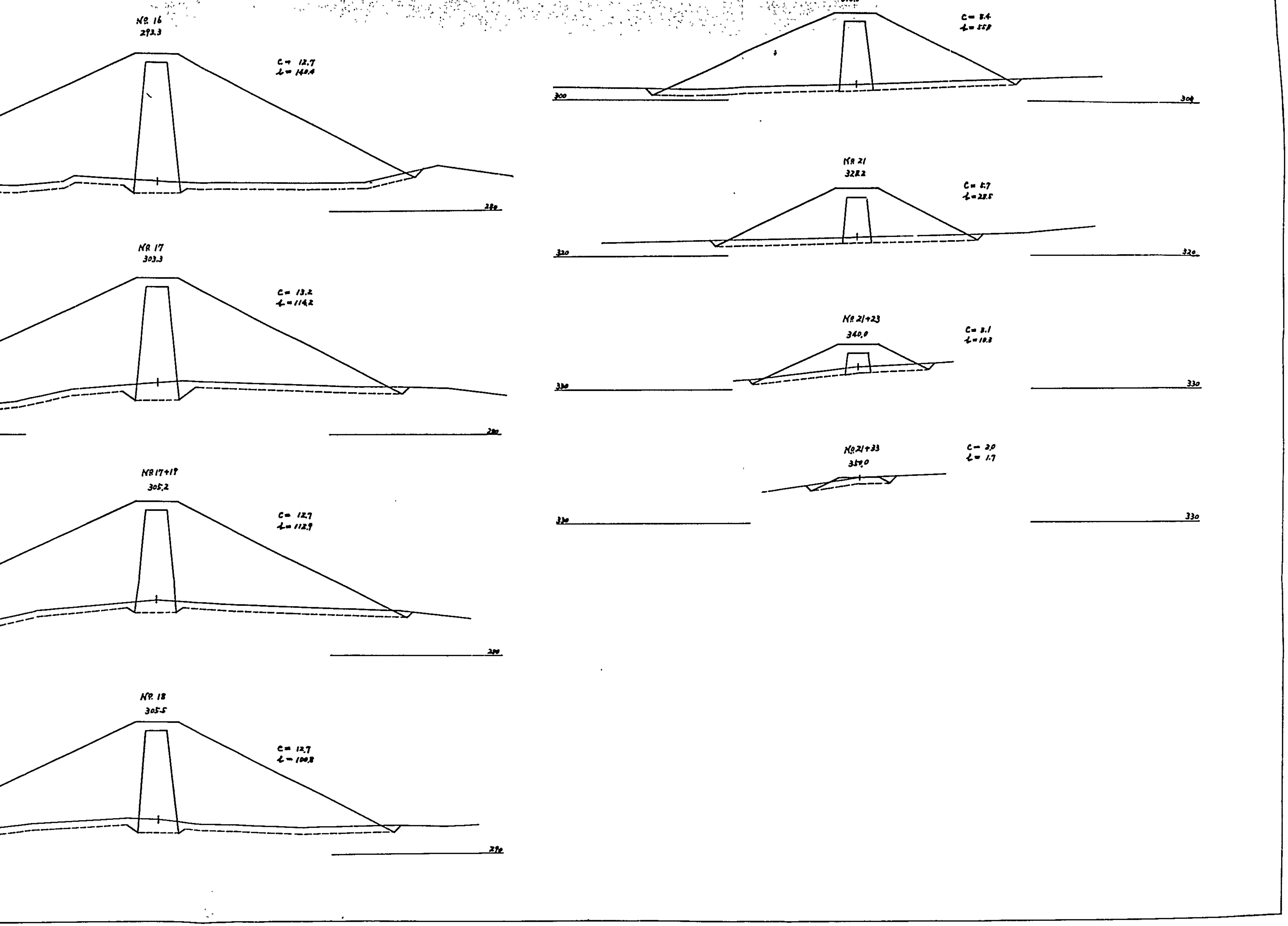
CROSS SECTIONS OF TUCKER VALLEY DAM

1" = 20'




CROSS SECTIONS OF TUCKER VALLEY DAM
1" = 20'
3/3
Nº 15+32
290.0
C= 16.8
Nº 16
293.3
C= 12.7
280
Nº 19
310.1
C= 10.0
300
Nº 20
316.8
C= 8.4
300
Nº 21
328.2

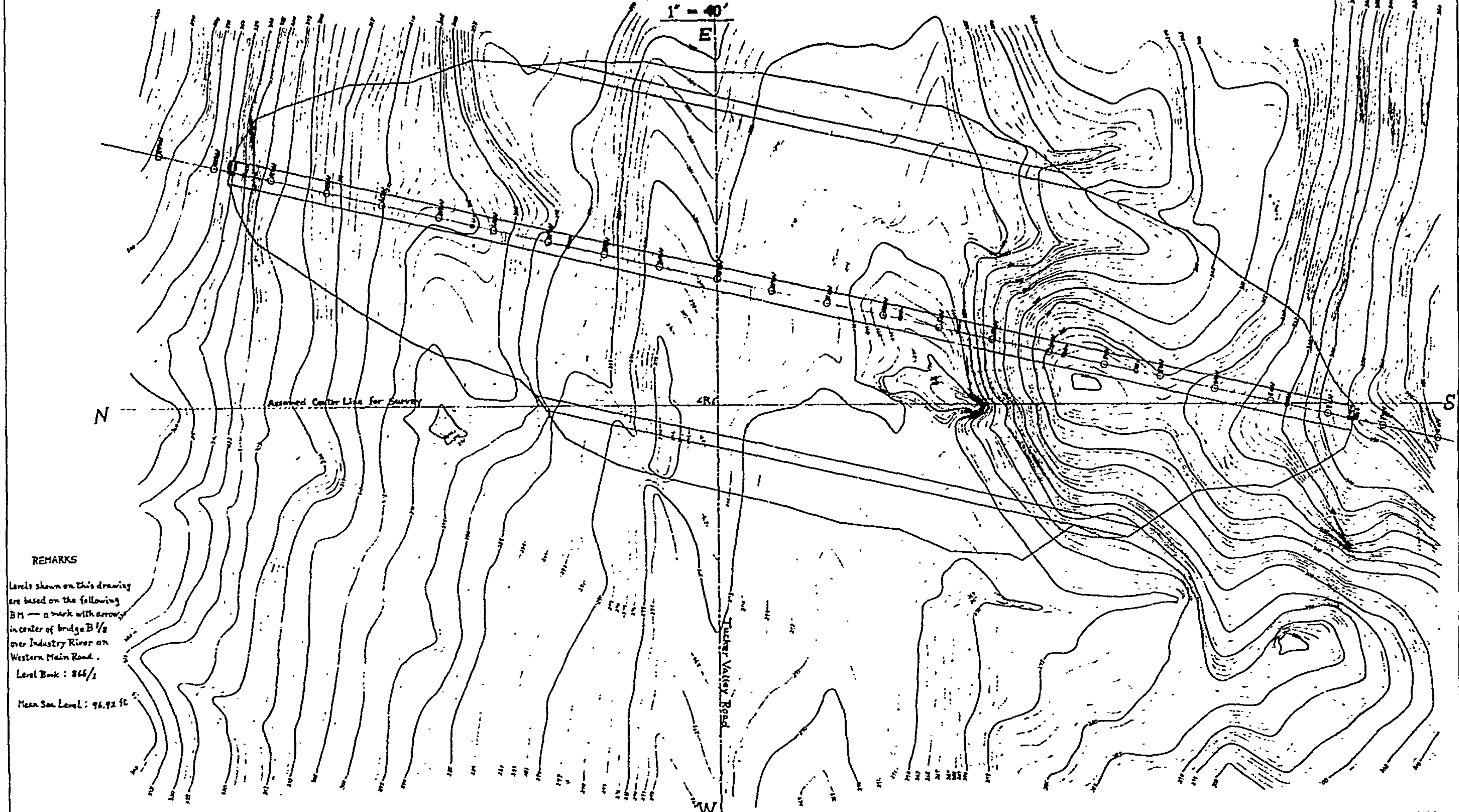

NR 16
293.3
c= 12.7
L= 140.4
NR 17
303.3
c= 13.2
L= 114.2
NR 17+19
305.2
c= 12.7
L= 112.9
NR 18
305.5
c= 12.7
L= 100.8
300
NR 21
328.2
320
NR 21+23
340.0
NR 21+33
359.0
c= 20
L= 1.7


Nº 16
293.3
C= 12.7
L= 140.4
290
Nº 17
303.3
C= 13.2
L= 114.2
290
Nº 17+19
305.2
C= 12.7
L= 112.9
290
Nº 18
305.5
C= 12.7
L= 100.8
290
C= 8.4
L= 55.8
300
Nº 21
328.2
C= 5.7
L= 28.5
320
320
Nº 21+23
340.0
C= 3.1
L= 10.3
330
330
Nº 21+33
357.0
C= 2.0
L= 1.7
330
330

(Fig.4)

PLAN OF PROPOSED DAM SITE IN TUCKER VALLEY

1" = 40'



TUCKER VALLEY (VII) - (Final Report) - January, 1969

MOVEMENT

22, Oct.	Supplemental survey for plan of Tucker Valley was started.
13, Nov.	The survey was finished. As a result, contour lines and roads were added on the plan. Designs of diversion channel, cofferdam, spillway and intake structure were undertaken.
18, Nov.	Modification of the plan was completed.
16, Dec.	The designs of the structures stated above were completed.

INTRODUCTION

In this report, calculations to determine dimensions and locations of diversion channel, cofferdam, spillway and intake structure for Tucker Valley Dam will be carried out.

The detailed designs and the estimate of costs will not be done in this report, because there is need for a detailed topographical map and geological survey.

The rough estimate of dam cost, in the extraordinary confidential report which was submitted to the Ministry of Planning and Development in December 1967, include cost of each of the structures.

The location of each structure was shown on the plan Fig. 1.

DESIGN OF DIVERSION CHANNEL AND COFFERDAM

The stream which flows through the dam site must be diverted by some appropriate means for carrying out foundation works and banking in building a dam.

The design of a diversion channel is very important for a fill-dam, and the type of a diversion channel must be selected by the following factors.

a) Characteristics of run-off from the catchment area
b) Topographical condition at dam site.
c) Geological condition at the foundation of the dam.
d) Type of the dam, its height, degree of its importance
e) Relation with other structures.

1) Determination of the type of diversion channel

A Tunnel which is built thorugh the existing right side slope at the damsite is adopted as the diversion channel. Because it can be used as a part of the intake structure after completion of the dam.

A diversion channel which is built under the dam, is quite dangerous for the dam itself, because the flood discharge is not small; as mentioned below, and the channel will become a weak point against leakage after completion of the dam.

2) Calculation of design flood

There are, no records of river discharge and daily rainfall in Tucker Valley. Therefore, the records of daily rainfall in Diego Martin, adjacent to Tucker Valley, must be used for this purpose.

	River Estate - Diego Martin				
Year	Highest daily rainfall for year	Year	Highest daily rainfall for year	Year	Highest daily rainfall for year
1929	3.25 inches	1942	(No record) inches	1955	2.38 inches
30	3.60	43	4.45	56	1.78
31	2.70	44	(No record)	57	2.68
32	3.00	45	(No record)	58	3.31
33	2.95	46	2.70	59	1.80
34	1.90	47	2.25	60	3.13
35	2.70	48	2.35	61	8.65
36	1.97	49	4.20	62	2.24
37	2.81	50	3.94	63	3.42
38	2.16	51	3.14	64	1.40
39	2.70	52	2.53	65	2.74
40	3.20	53	3.46	66	2.39
41	2.50	54	1.99	67	2.62

Calculation of probable rainfall by Hazen's method is shown on the following table.

i	i/n x 100	Rainfall	i	i/n x 100	Rainfall	i	i/n x 100	Rainfall
1	2.78	8.65 inches	13	36.11	3.00 inches	25	69.44	2.39 inches
2	5.56	4.45	14	38.89	2.95	26	72.22	2.38
3	8.33	4.20	15	41.67	2.81	27	75.00	2.35
4	11.11	3.94	16	44.44	2.74	28	77.78	2.25
5	13.89	3.60	17	47.22	2.70	29	80.56	2.24
6	16.67	3.46	18	50.00	2.70	30	83.33	2.16
7	19.44	3.42	19	52.78	2.70	31	86.11	1.99
8	22.22	3.31	20	55.56	2.70	32	88.89	1.97
9	25.00	3.25	21	58.33	2.68	33	91.67	1.90
10	27.78	3.20	22	61.11	2.62	34	94.44	1.80
11	30.56	3.14	23	63.89	2.53	35	97.22	1.78
12	33.33	3.13	24	66.67	2.50	36	100.00	1.40

n = 36

Probability	Diego Martin (A)	Tucker Valley (B)	Remarks
1/2	2.95 inches	2.66 inches	(B) = 0.9 (A) (Maximum (Rainfall
1/5	3.75	3.38	
1/10	4.20	3.78	
1/20	4.70	4.23	
1/50	5.30	4.70	
1/100	5.75	5.18	
1/200	6.20	6.03	
1/500	6.70	6.03	
1/1,000	7.00	6.30	

According to the isohyetal map of monthly rainfall in June, 1950 (in wet season)

Tucker Valley has the monthly rainfall of 11.3' and

River Estate - Diego Martin has that of 13.7'

It means that the monthly rainfall in Tucker Valley is about 85% in Diego Martin in wet season. Similarly in March, 1950 (in dry season)

Tucker Valley has the monthly rainfall of 1.4' and

River Estate - Diego Martin has 2.9'

It means that the monthly rainfall in Tucker Valley is about 48% in Diego Martin in dry season.

In total, in 1950, Tucker Valley has the yearly rainfall of 63' and River Estate - Diego Martin has 71'.

It means that the yearly rainfall in Tucker Valley is about 89% in Diego Martin.

As a result mentioned above, the coefficient for daily rainfall in Tucker Valley is adopted to be 0.90.

1) Determination of the designed basic rainfall precipitation (R)

The maximum daily rainfall in probability of 10 years in Tucker Valley must be used for the calculation of design flood for the diversion channel. (i.e) R = 3.78' = 96.0 mm

2) Determination of the arrival time of flood from the upmost to the dam site by Rziha's formula (T).

Velocity of flood : $V = 72\,(H/L)^{0.6}$ (Km/hr)

H: Difference of river-bed level between the upmost and the damsite which is 280m (=920')

L: Length of river from the upmost to the dam site which is 2,590m (=8,500')

T: Arrival time of flood from the upmost : T = L/V (hr)

$$V = 72(H/L)^{0.6} = 72\left(\frac{280}{2,590}\right)^{0.6} = 18.95\ \text{Km/hr}$$

$$T = L/V = \frac{2.59}{18.95} = 0.14\ \text{hr} < 1.0\ \text{hr}$$

3) Determination of the mean hourly rainfall intensity during T. (r)

The mean hourly rainfall intensity during T. (r) is calculated by the maximum daily rainfall (R)

(i.e) R = 3.78" = 96.0mm

$$r = \frac{R}{24}\left(\frac{24}{T}\right)^{2/3} = \frac{96.0}{24}\left(\frac{24}{1.0}\right)^{2/3} = 33.3 \text{ mm/hr}$$

Since T is less than 1.0 hr, 1.0 hr must be used for this calculation instead of T itself.

4) Determination of the maximum flood discharge (Qmax)

$$Q_{max} = \frac{1}{3.6} f.r.A$$

f: *run-off coefficient at flood peak*

A: *catchment area (km^2)*

Run-off coefficient at flood peak (f) must be adopted to be more than 0.8 for design of fill dam.

$$\therefore\ Q_{max} = \frac{1}{3.6} \times 0.8 \times 33.3 \times 6.97 = 51.6 \text{ m}^3\text{/sec}$$

5) Hydraulic calculation of water tunnel

Conditions
i) Designed flood discharge is 51.6m^3/sec (= 1,822 cusecs)
ii) Length of tunnel is 171.6m (= 563')
in which straight part is 112.2m (= 368')
curved part is 59.4m (=195')
iii) Type of tunnel is a standard horse-shoe shaped (2r)
iv) Invert level of the inlet of tunnel is 283'
Invert level of the outlet will be $283' - \frac{563'}{150} = 279.25'$

Assuming that the slope of tunnel is 1/150.

Although the lowest level of river-bed just before the dam site is 277.5' the invert level of the inlet cannot be determined lower than 283' because of the topographical condition.

v) Area of the tunnel must be determined to be a pipe line with full water when the desinged flood is discharged.
For the purpose, waterdepth just before the inlet of the tunnel must be 1.5 times as deep as the height of the tunnel.
vi) Velocity of approach at inlet is negligible because the front of the inlet is ponded by cofferdam.
vii) Transition to the inlet is set.
viii) There is no influence from the lower reaches of the channel at the outlet.

Hydraulic calculation

r: radius of tunnel

$A = ar^2$ A: cross sectional area of tunnel

$= a\left(\frac{D}{2}\right)^2$ a: coefficient which is 3.317 in case of standard horse-shoe shaped section.

$H =$	$hr + ho + hf + hb$	H:	total loss of head
$hv =$	$1.0 \times \frac{V^2}{2g}$	hv:	loss of head due to changing velocity
		ho:	loss of head at inlet
$ho \div fo \times \frac{V^2}{2g} = 0.5 \times \frac{V^2}{2g}$		hf:	loss of head due to friction
		hb:	loss of head due to bending
$hf = f \times {}^{L}/_{D} \times \frac{V^2}{2g}$		g:	accerelation due to gravity which is $9.8m/sec^2$
		L:	length of tunnel which is 171.5m
$f = \frac{122 \times n^2}{D^{1}/_{3}}$		D:	diameter of tunnel = 2r
		n:	coefficient of roughness which is 0.015
$hb = fb \times \frac{(I^o)}{90^o}^{1/2} \times \frac{V^2}{2g}$			

$$\therefore\ H = \left[1.0 + 0.5 + \frac{122 \times (0.015)^2}{D^{1}/_{3}} \times \frac{171.5}{D} + 0.07 \frac{(111^o.33')^{1/2}}{90^o} \right] \frac{V^2}{2 \times 9.8}$$

$$= (1.0 + 0.5 + \frac{4.708}{D^{4}/_{3}} + 0.08) \times \frac{Q^2}{19.6\ A^2}$$

$$= (1.58 + \frac{4.708}{D^{4}/_{3}}) \quad \frac{(51.6)^2}{19.6 \times [3.317 \times (\frac{D}{2})^2]^2} \quad = \frac{1}{D^4} \ (312.16 + \frac{930.08}{D^{4}/_{3}})$$

fb: coefficient of loss of head due to a ratio of radius of bending to diameter of tunnel $\frac{R}{D}$ which is about 0.07 (R = 100')

I: angle subtended by curvewhich is 111°33'

Q: designed flood discharge which is $51.6\ m^3/sec$.

r	D		D^4	$1/D^4$	$D^4/3$	$\frac{930.08}{D^4/3}$	$312.16 + \frac{930.08}{D^4/3}$	H		Water level at inlet 279.25' + D + H	Necessary water level for tunnel being filled with water 283' + 1.5D
1.5m	3.0m	9.84ft	81.00	0.01234	4.326	215.00	527.16	6.51m	21.35ft	279.25'+9.84'+21.35'=310.44'	283'+1.5x9.84'=297.76'
1.6	3.2	10.50	104.86	0.00954	4.717	197.18	509.34	4.86	15.94	279.25'+10.50'+15.94'=305.69'	283'+1.5x10.50=298.75'
1.7	3.4	11.15	133.63	0.00748	5.114	181.87	494.03	3.70	12.14	279.25'+11.15'+12.14'=302.54'	283'+1.5x11.15'=299.73
1.8	3.6	11.81	167.96	0.00595	5.519	168.52	480.68	2.86	9.38	279.25'+11.81'+9.38'=300.44'	283'+1.5x11.81'=300.72'

As a result of the calculation, the radius of 1.7m is adopted

(Note
E_l, 279.25' is the invert level of the outlet.
E_1, 283.00' is the invert level of the inlet.

The crest level of the cofferdam must be determined to be 306.00' as the water level at inlet of the tunnel is 302.54'. (The crest level of 300' which has been assumed in the report IV, must be therefore modified. (cf. Fig. 6)

6) Calculation of earth requirement for cofferdam (cf. Fig. 7)

$$V = \tfrac{1}{2}(10' + 118.5') \times 31' \times \tfrac{1}{2} \times 550' = 547{,}730 ft^3 = 20{,}286 yd^3 = 15{,}510 m^3$$

7) Hydraulic calculation for the upper open channel which is followed by water tunnel.

Conditions

i) The minimum bed width required for machinery operation: $b = 4.00m$ (= 13.12ft)

ii) The maximum allowable velocity: $V = 2.00$ m/s (= 6.56ft/s)

iii) coefficient of roughness: $n = 0.030$ (earthen section)

iv) side slope of canal : $Z = 1.5$

v) designed flood discharge : $Q = 51.6\ m^3/s$ (= 1,822 cusecs)

Determination of the critical slope

$$\frac{Q}{b^{2.5}} = \sqrt{g}\left(\frac{dc}{b}\right)^{3/2}\left[1 + Z\left(\frac{dc}{b}\right)^{3/2}\right] / \left[1 + 2Z\left(\frac{dc}{b}\right)^{1/2}\right] = k'$$

g: acceleration due to gravity which is $9.8m/sec^2$

dc: critical depth

Therefore, $k' = \frac{Q}{b^{2.5}} = \frac{51.6}{32.00} = 1.6125$ from the table $\left(\frac{dc}{b}\right) = 0.498$

$\therefore dc = 0.498 \times 4.00 = 1.992m$

Cross sectional area: $A = dc\,(b + Zdc) = 1.992\,(4.00 + 1.5 \times 1.992) = 13.920m^2$

Wetted perimeter: $P = b + 2\sqrt{1^2 + 1.5^2}\,dc = 4.00 + 2 \times 1.803 \times 1.992 = 11.183m$

Critical velocity: $Vc = Q/A = 51.6/13.920 = 3.707m/s$

Hydraulic radius: $R = A/P = 13{,}920/11{,}183 = 1{,}245$ $\quad R^{2/3} = 1.160$

Critical slope: $Ic = \frac{(nV)^2}{R^{2/3}} = \frac{(0.030 \times 3.707)^2}{1.160} = (0.09587)^2 = 0.009191 \doteqdot \frac{1}{109}$

Determination of the depth in sub-critical flow (d)

Assuming that slope is $\frac{1}{600}$

$$\frac{(nQ)}{I^{1/2}\cdot b^{8/3}} = \left(\frac{d}{b}\right)^{\frac{5}{3}}\left[\left(1 + Z\frac{d}{b}\right)^{\frac{5}{3}} / \left(1 + 2\frac{d}{b}\sqrt{1 - Z^2}\right)^{\frac{2}{3}}\right] = k'$$

$$k' = \frac{nQ}{(I)^{1/2} b^{\frac{8}{3}}} = \frac{0.030 \times 51.6}{(0.00167)^{1/2} \times (4.00)^{8/3}} = \frac{1.548}{0.04087 \times 40.314} = 0.9395$$

from the table, $d/b = 0.762$ $\quad \therefore d = 0.762 \times 4.00 = 3.048m$

$A = d\,(b + Zd) = 3.048\,(4.00 + 1.5 \times 3.048) = 26.13m^2$

$V = Q/A = 51.6/26.13 = 1.97m/s < 2.00m/s$

Therefore, the depth in sub-critical flow is determined as 3.048m (= 10.00') when the slope is 1/600.

In reality, this depth in sub-critical flow will be held back by the necessary head of 7.10m (23.29') for the inlet of the water tunnel which is raised by the cofferdam.

Design of Spillway

The type of a spillway must be determined in consideration of the topography and the geology at a damsite and the structures connected with the spillway.

In this case, it is determined that a sideweir type will be established at the left side of the damsite.

Because, in topography, the left side is better than the right side of the damsite to build the spilway, although in geology, the superiority of the left side is not known so far.

As to the structures connected with the spillway the main driving channel and (as mentioned above) the water tunnel as the diversion channel are planned to be established along the right side of the damsite.

Therefore there will be some restriction if the spillway is built at the right side.

1) Determination of the overlfow depth.

$Q = CLH^{\frac{3}{2}}$ $\quad Q = 127.5\ m^3/sec$

Assuming that C = 2.1

$$L = \frac{Q}{CH_2} = \frac{127.5}{2.1\ H\ 3/2} = \frac{6.07}{H\ 3/2}$$

H	1.0m (= 3.3')	1.1m (=3.6')	1.2m (=3.9')
L	60.7m (=199.1')	52.6m (=172.5')	46.2m (151.5')

As the overflow depth can be adopted as 1.2m at the maximum without changing the crest level of dam 350'; determined in the report (VI), the length of the sideweir will therefore be 46.2m (=151.5')

2) Check in the abnormal flood which is 1.2 times as much as the designed flood.

the designed flood discharge $Q = 127.5m^3/sec.$

the abnormal flood discharge $Q' = 127.5 \times 1.2 = 153.0m^3/sec$

$$\therefore \quad H' = (\frac{Q'}{CL})2/3 = (\frac{153.0}{2.1 \times 46.2})^{2/3} = 1.355m\ (= 4.44')$$

(F.W.L)	(Height of wave due to wind)		(Overflow depth in abnormal flow)	(Crost level of dam)
340.0'	3.0'	+	4.44'	= 347.5' < 350'

Therefore, teh dam will also be safe in the abnormal flood which is 1.2 times of the designed flood.

3) Determination of the cross section and the profile of the sidewire type spillway.

Conditions:- Overflow depth: $H = 1.2m$

Length of sideweir : $L = 46.2m$

Designed flood: $Q = 127.5m^3/sec$

Bottom width of spillway at the beginning of the sideweir: $b_1 = 3.00m$

Bottom width of spillway at the end of the sideweir $b_2 = 6.00m$

Side-slope of the cross section:

Inflow side $m_1 = 0.7$

Opposite side $m_2 = 0.5$ ($m = 0.6$ in average)

Formula required for the design:

Hinds's formula

$$\begin{cases} V = ax^n \\ Qx = qx \\ h = \frac{n+1}{n} hv \end{cases}$$

V: mean velocity of the spillway at an optional point in sideweir (m/s)

x: distance between the beginning of the sideweir and the optional point in sideweir (m)

a: coefficient for velocity

n: exponent for velocity (0.4 – 0.8)

Qx: discharge at x (m^3/s)

q: inflow per unit length of side weir (m^3/s)

h: vertical distance between the sideweir crest and the water surface of the spillway at the optional point

hv: velocity head which is $\frac{V^2}{2g}$

Determination of (n) in the formula

(n) must be determined between 0.4 and 0.8

The various (n)s must be assumed and the corresponding (a)s which make the minimum volume of cut at the end of the side weir, must be calculated.

After that these (n) s and corresponding (a)s must be compared in order that one couple which has the least volume cut will be selected.

As it will, however, take much time for this work, (n) is determined to be 0.55 empirically.

To find the corresponding (a) which makes the minimum volulme of cut at the end of the side weir, the following equation may be applied.

$$\frac{A^3}{T} = \frac{n+1}{n} x \frac{Q^2}{g}$$

T: width of water surface in cross section which is $b_2 + 2md$

$A = d(b_2 + md)$ $V = Q/A$ $a = V/L^n$

$$\therefore \quad \frac{(d/b_2)^3 \times [1 + m(d/b_2)]^3}{1 + 2m\,(d/b_2)} = \frac{n+1}{n} \times \frac{Q^2}{g\,b_2^5}$$

$b_2 = 6.0m$ $L = 46.2m$
$m = 0.6$
$n = 0.55$
$Q = 127.5m^3/s$ $g = 9.8$ m/sec^2

$$\frac{n+1}{n} \times \frac{Q^2}{gb_2^5} = \frac{1.55}{0.55} \times \frac{(127.5)^2}{9.8 \times (6.0)^5} = 2.818 \times \frac{16,256}{76,205} = 0.3590$$

Assuming that $(d/b_2) = 0.623$

$$\frac{(d/b_2)^3 \times [1 + m(d/b_2)]^3}{1 + 2m\,(d/b_2)} = \frac{(0.623)^3 \times (1 + 0.6 \times 0.623)^3}{1 + 2 \times 0.6 \times 0.623} = \frac{0.2418 \times (1.3738)^3}{1.7476} = \frac{0.62694}{1.7476} = 0.359$$

Therefore $d = 0.623b_2 = 0.623 \times 6.0 = 3.738m$

$A = d(b_2 + md) = 3.738\,(6.00 + 0.6 \times 3.738) = 30.8/m^2$

$V = Q/A = 127.5/30.81 = 4,138$m/sec

$a = V/L^n = 4.138/(46.2)^{0.55} = \frac{4.138}{8.233} = 0.5026$

$\therefore \quad V = 0.5026\, X^{0.55}$

Table of calculation for velocities, discharges, and cross sectional areas of the spillway between the beginning and the end of the sideweir.

x	log x	0.55 log x	x 0.55	V	Q	A
1.0	0.000 0000	0.000 0000	1.0000	0.503	2.760	5.487
3.0	0.477 1213	0.262 4167	1.8299	0.920	8.279	8.999
5.0	0.698 9700	0.384 4335	2.4234	1.218	13.799	11.329
8.0	0.903 0900	0.496 6995	3.1383	1.577	22.078	14.000
12.0	1.079 1812	0.593 5497	3.9224	1.971	33.116	16.802
16.0	1.204 1200	0.662 2660	4.5948	2.309	44.155	19.123
20.0	1.301 0300	0.715 5665	5.1948	2.611	55.194	21.139
25.0	1.397 9400	0.768 8670	5.8731	2.952	68.992	23.371
30.0	1.477 1213	0.812 4167	6.4926	3.263	82.791	25.373
35.0	1.544 0680	0.849 2374	7.0670	3.552	96.590	27,193
40.0	1.602 0600	0.881 1330	7.6056	3.823	110.388	28.875
46.2	1.664 6420	0.915 5531	8.2329	4.138	127.500	30.810

$V = 0.5026\, X^{0.55}$ $Q = qx = \frac{127.5}{46.2} \times X = 2.7597X$

Table of calculation for depths of spillway between the beginning and the end of the side weir.

x	b	b^2	2.4A	$b^2 + 2.4A$	$\sqrt{b^2 + 2.4A}$	$-b+\sqrt{b^2 + 2.4A}$	$d = \frac{-b+\sqrt{b^2+2.4A}}{1.20}$
1.0	3,065	9,394	13.169	22.563	4.750	1.685	1,404
3.0	3.195	10.208	21.598	31.806	5.640	2.445	2.037
5.0	3.327	11.069	27.190	38.259	6.185	2.858	2.382
8.0	3.519	12.383	33.600	45.983	6.781	3.262	2.718
12.0	3.779	14.281	40.325	54.606	7.390	3.611	3.009
16.0	4.039	16.314	45.895	62.209	7.887	3.848	3.207
20.0	4.299	18.481	50.734	69.215	8.320	4,021	3.351
25.0	4.623	21.372	56.090	77.462	8.801	4.178	3.482
30.0	4.948	24.483	60.895	85.378	9.240	4.292	3.577
35.0	5.273	27.805	65.263	93.068	9.647	4.374	3.645
40.0	5.597	31.326	69.300	100.626	10.031	4.434	3.695
46.2	6.000	36.000	73.944	109.944	10.485	4.485	3.737

$A = (b + md)d = bd + 0.6d^2 \quad \therefore\ d = \frac{-b \pm \sqrt{b^2 + 1.4A}}{1.20} \quad b = 3.00 + \frac{3.00}{46.2}\ x = 3.00 + 0.064935X$

Table of calculation for friction loss of spillway between the beginning and the end of the side weir.

x	d	b	A	P	R	$R^{4/3}$	C^2R	V^2	Δx	$V^2 x \Delta x$	hf
1.00	1.404	3.065	5.487	6.349	0.864	0.823	3,658	0.253	1.00	0.253	0.000
3.00	2.037	3.195	8.999	7.959	1.131	1.178	5,236	0.846	2.00	1.692	0.000
5.00	2.382	3.327	11.329	8.898	1.273	1.380	6.133	1.484	2.00	2.968	0.000
8.00	2.718	3.519	14.000	9.876	1.418	1.546	6,871	2.487	3.00	7.461	0.001
12.00	3.009	3.779	16.802	10.816	1.553	1.798	7,991	3.885	4.00	15,540	0.002
16.00	3.207	4.039	19.123	11.539	1.657	1.961	8,716	5.331	4.00	21.324	0.002
20.00	3.351	4.299	21.139	12.136	1.742	2.096	9,316	6.817	4.00	27.268	0.003
25.00	3.482	4.623	23.371	12.766	1.831	2.240	9,956	8.714	5.00	43.570	0.004
30.00	3.577	4.948	25.373	13.314	1.906	2.363	10,502	10.647	5.00	53.235	0.005
35.00	3.645	5.273	27.193	13.798	1.971	2.471	10 982	12.617	5.00	63.085	0.006
40.00	3.695	5.597	28.875	14.238	2.028	2.567	11,409	14.620	5.00	73.100	0.006
46.20	3.737	6.000	30.810	14.740	2.090	2.672	11,876	17.123	6.20	106.163	0.006

$P = b + [\sqrt{1^2 + (0.5)^2} + \sqrt{1^2 + (0.7)^2}]\ d = b + (1.1180 + 1.2207)\ d = b + 2.3387d$

$R = A/P$

$C^2R = (\frac{1}{n} R^{1/6})^2 \times R = \frac{1}{n^2}\ R^{4/3} = 4{,}444.44\ 4\ R^{4/3} \quad hf = \frac{V^2\Delta_x}{C^2R} \quad n = 0.015$

Table of calculation for depth from the crest of the side weir to the invert of spillway between the beginning and the end of the side weir.

x	d	v_2	h	hf	Σhf	$Y=d+h+\Sigma hf$
1.00	1.404	0.253	0.036	0.000	0.000	1.440
3.00	2.037	0.846	0.122	0.000	0.000	2.159
5.00	2.382	1.484	0.213	0.000	0.000	2.595
8.00	2.718	2.487	0.358	0.001	0.001	3.007
12.00	3.009	3.885	0.559	0.002	0.003	3.571
16.00	3.207	5.331	0.766	0.002	0.005	3.978
20.00	3.351	6.817	0.980	0.003	0.008	4.339
25.00	3.482	8.714	1.253	0.004	0.012	4.747
30.00	3.577	10.647	1.531	0.005	0.017	5.125
35.00	3.645	12.617	1.814	0.006	0.023	5.482
40.C0	3.695	14.620	2.102	0.006	0.029	5.826
46.20	3.737	17.123	2.462	0.006	0.035	6.234

$$h = \frac{n+1}{n} \times \frac{V^2}{2g} = \frac{0.55+1}{0.55} \times \frac{V^2}{2 \times 9.8} = 0.14378\ V^2$$

A profile of the spillway is shown on Fig. 4. So as to modify the curved invert slope which had been calculated above, by a straight one, a new straight invert slope must be lined between the calculated spillway invert at the end of the sideweir and the calculated one at any point one third to one tenth of the length of the side weir down from the beginning of it.

In this case, this new invert slope is determined to be 1/12 (cf. Fig. 4)

Modification of water surface level can be done by tracing the water surface up from the point at which the critical depth occurs. However, this trace of the water surface cannot be done, because the design of spillway down stream of the side weir is not carried out in this report.

Design for Intake Facilities

Intake facilities must be established in consideration of the topography the geology and the connection with the main driving channel.

In this case, water from the dam is taken by a sloped outlet pipe and is connected to the water tunnel which has been used as a diversion channel. This water tunnel is used as an inverted siphon at the end of which a shaft is set up.

The water from the shaft is led to the main driving channel.

Hand-operated gates are also set at the reduced inlet and outlet of the water tunnel as a silt ejector and a blow-off to discharge sediments to a floodway.

(For setting up an outlet tower, a connecting bridge of 150 - 200 ft and more foundation works are necessary.

Accordingly, the cost for the outlet tower will be more expensive than that of the sloped outlet pipe).

1) Determination of diameters of outlet holes and the sloped outlet pipe.

The cross sectional area of the sloped outlet pipe must be more than twice that of an outlet hole as a standard. Therefore, diameter and number of the holes must first be determined.

The number of the holes are determined to be four and their positions are as follows.

	Elevation of its center	Depth from F.W.L.
No. 1 hole	333'	7'
No. 2 hole	325'	15'
No. 3 hole	315'	25'
No. 4 hole	300'	40'
Silt ejector gate	283'	57'

Although the irrigable area in the designed dry year is 1,019 acres (= 412.4 ha) as mentioned in the report (I) - (IV), the total area required for irrigation in Tucker Valley is about 2,000 acres (= 809.4ha).

This total area must be considered as the maximum to determine the diameter of the holes.

$$Q = 809.4 \text{ ha} \times 10^4 \times 4.23\text{mm} \times 1.25 \times 10^{-3} / 86{,}400 = 0.495 \text{ m}^3/\text{s} \ (= 17.5 \text{ cusecs})$$

In case of orifice,

$Q = CA(2gH)^{1/2}$

C: coefficient of discharge which is 0.62

A: cross sectional area of orifice

H: head from the center of orifice

$\therefore\ Q = 0.62 \times \pi(\frac{D}{2})^2 \times (19.6H)^{1/2} = 2.1558D^2H^{1/2}$

D	200mm	300	400	450	500
D^2	$0.040m^2$	0.090	0.1600	0.2025	0.2500
Q	$0.0862H^{1/2}$	$0.1940H^{1/2}$	$0.3449H^{1/2}$	$0.4365H^{1/2}$	$0.5390H^{1/2}$

D: Diameter of orifice

	H	0.30m (1.0')	0.61 (2.0')	0.92 (3.0')	1.22 (4.0')	1.52 (5.0')	1.83 (6.0')	2.13 (7.0')	2.44 (8.0')	2.74 (9.0')	3.05 (10.0')
	$H^{1/2}$	0.548	0.781	0.959	1.105	1.233	1.353	1.459	1.562	1.655	1.746
	D=200mm (=0.65')	0.047	0.067	0.083	0.095	0.106	0.117	0.126	0.135	0.143	0.151
	300 (=0.98')	0.106	0.152	0.186	0.214	0.239	0.262	0.283	0.393	0.321	0.339
Q	400 (=1.31')	0.189	0.269	0.331	0.381	0.425	0.467	0.503	0.539	0.571	0.602
m^3/s	450 (=1.48')	0.239	0.341	0.419	0.482	0.538	0.591	0.637	0.682	0.722	0.762
	500 (=1.65')	0.295	0.421	0.517	0.596	0.665	0.729	0.786	0.842	0.892	0.941

The diameter of the hole can be determined to be 450mm (≑ 1.5') because, it can take the maximum water requirement by the head of more than 4.2 ft. Cross sectional area of the hole

$$A_h = \frac{\pi}{4}D^2 = \frac{3.14}{4} \times (0.45)^2 = 0.159m^2$$

Assuming that the cross sectional area of the sloped outlet pipe is twice that of an outlet hole.

$$Ap = aAh = 2 \times 0.159 = 0.318m^2$$

$$\therefore dp = \frac{(4An)^{1/2}}{\pi} = \frac{(4 \times 0.318)^{1/2}}{3.14} = 0.636m$$

If diameter of 2.5 ft is taken
d = 2.5 ft = 0.762m > 0.636m

Therefore the diameter of the pipe is determined as 2.5ft. When the water tunnel is used as an outlet conduit pipe, loss of friction is negligible, because the cross sectional area is very large in comparison with the discharging capacity and so the velocity is very little.

At the commencement of discharge from the dam, the volume of water (Q) to be stored in the water tunnel which functions as an inverted siphon, will be,

cross sectional area	$A = 9.568m^2$
length of tunnel	$L = 170m$

$$\therefore Q = A.L = 9.586 \times 170 = 1{,}630m^3$$

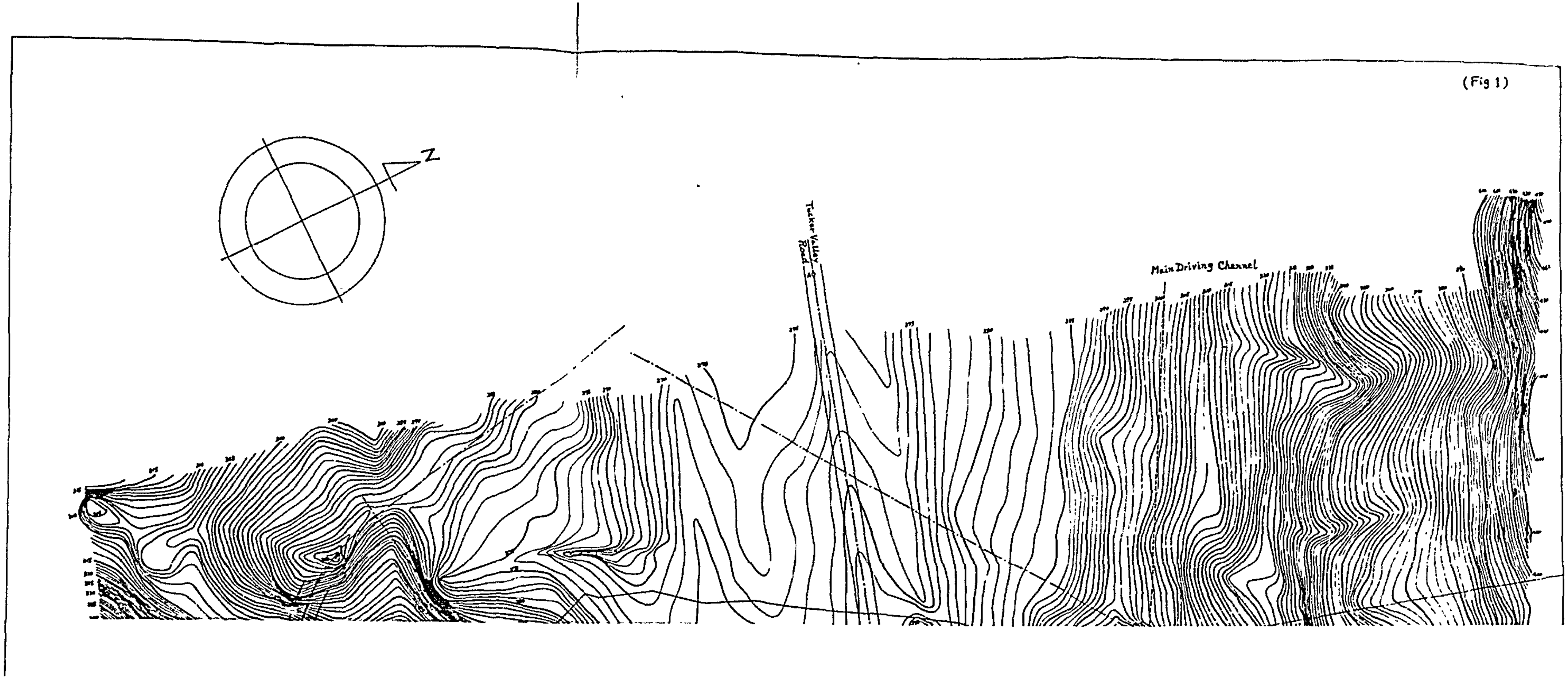
(Storage capacity of the dam is about $3{,}000{,}000m^3$)

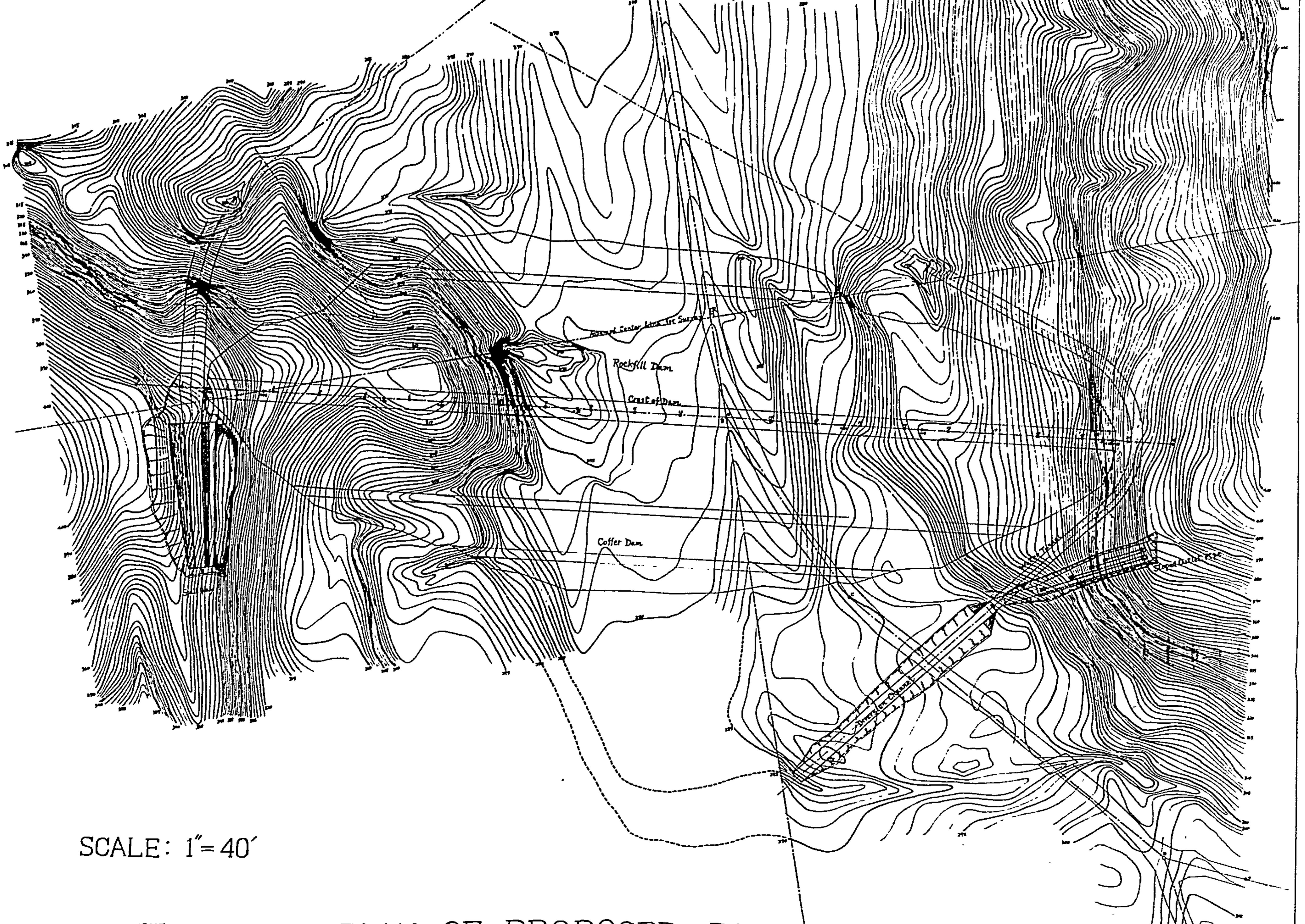
Assuming that the discharge (q) is $0.5\ m^3$ /sec, the time (T) required for filling the water tunnel will be,

$$T = Q/q = 1{,}630/0.5 = 3{,}260\ sec = 545\ min \fallingdotseq 0.9\ hr.$$

Junichi Kitamura

(Fig 1)





SCALE: 1″= 40′

NOTE

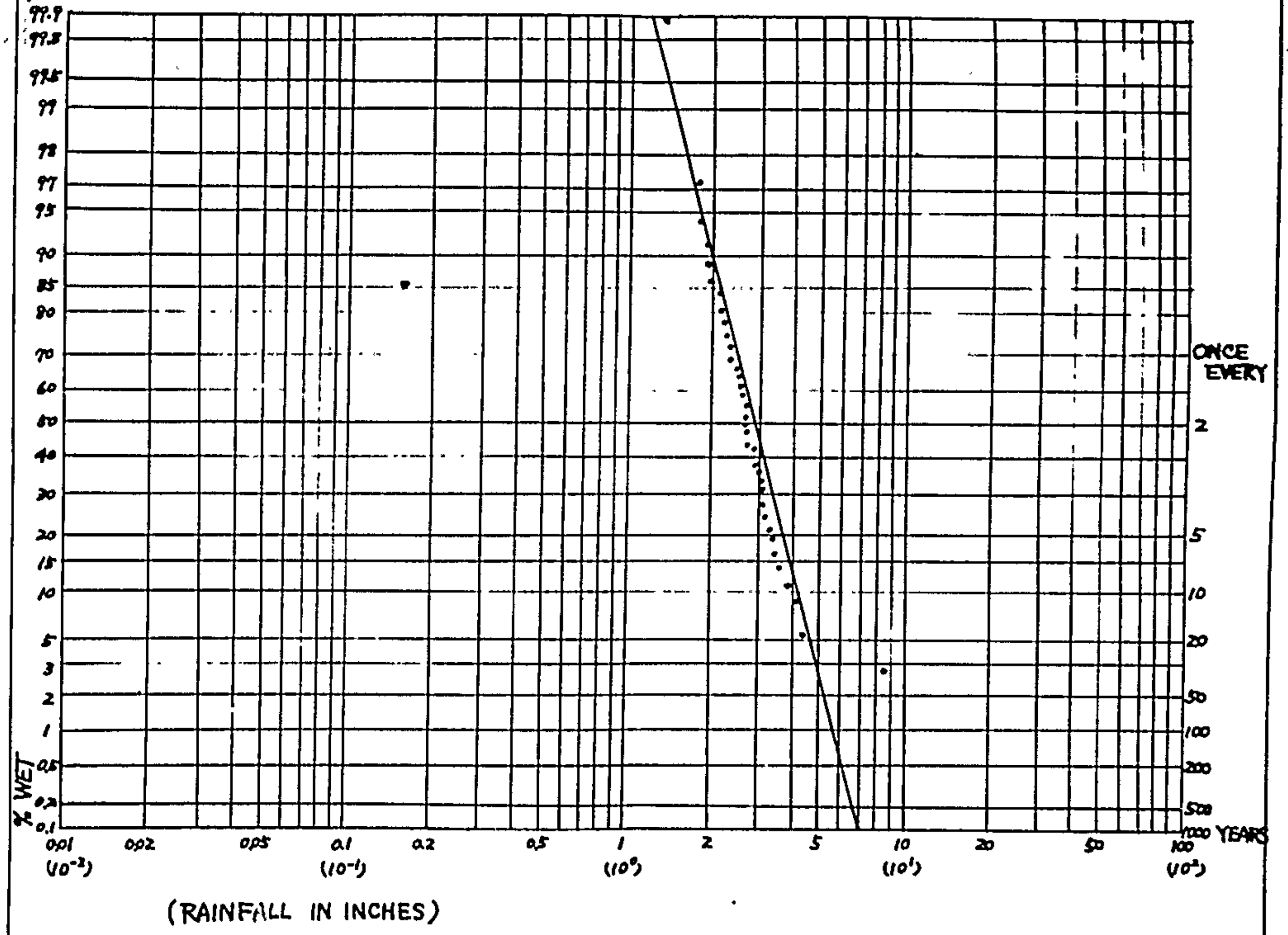
Levels shown on this drawing are based on the following B.M. —— □ mark with arrow in center of bridge B 1/8 over Industry on Western Main Road. Level Book : 866/1
Mean Sea Level: 96.92 ft

PLAN OF PROPOSED DAM AND ATTACHED STRUCTURES IN TUCKER VALLEY

BY HAZEN'S METHOD

(Fig.2)

PROBABILITY CURVE OF MAXIMUM DAILY RAINFALL IN A YEAR
AT TUCKER VALLEY

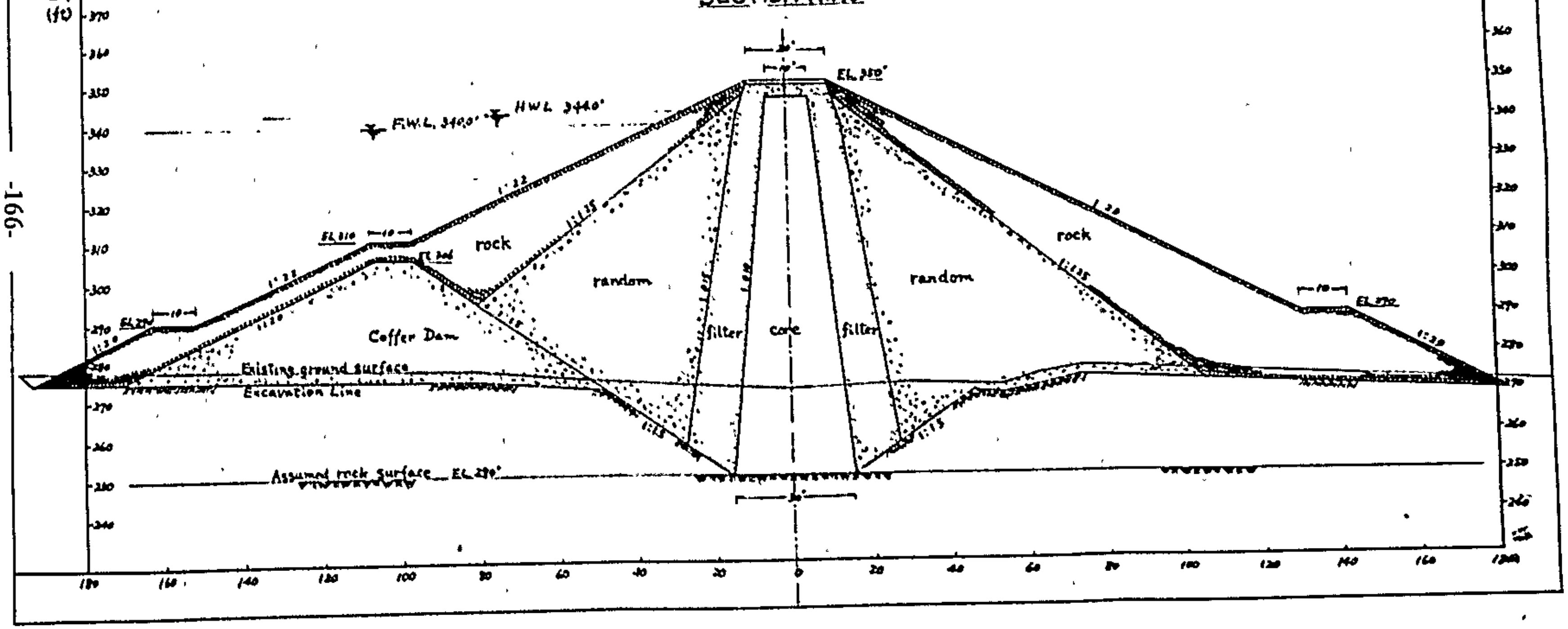


(Fig.3)

MODIFIED STANDARD CROSS SECTION OF TUCKER VALLEY DAM

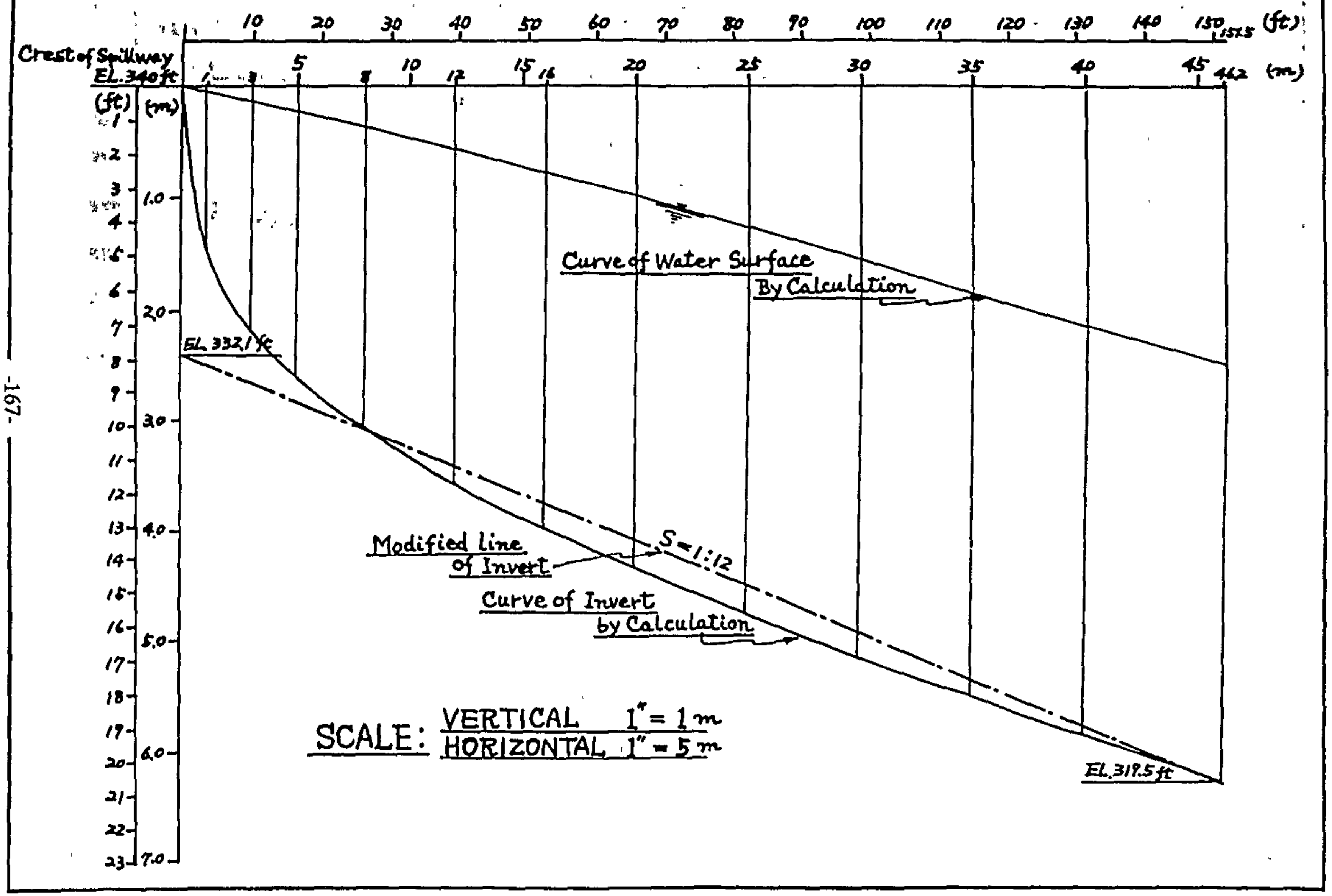
1" = 20'

SECTION No.10



(Fig. 4)

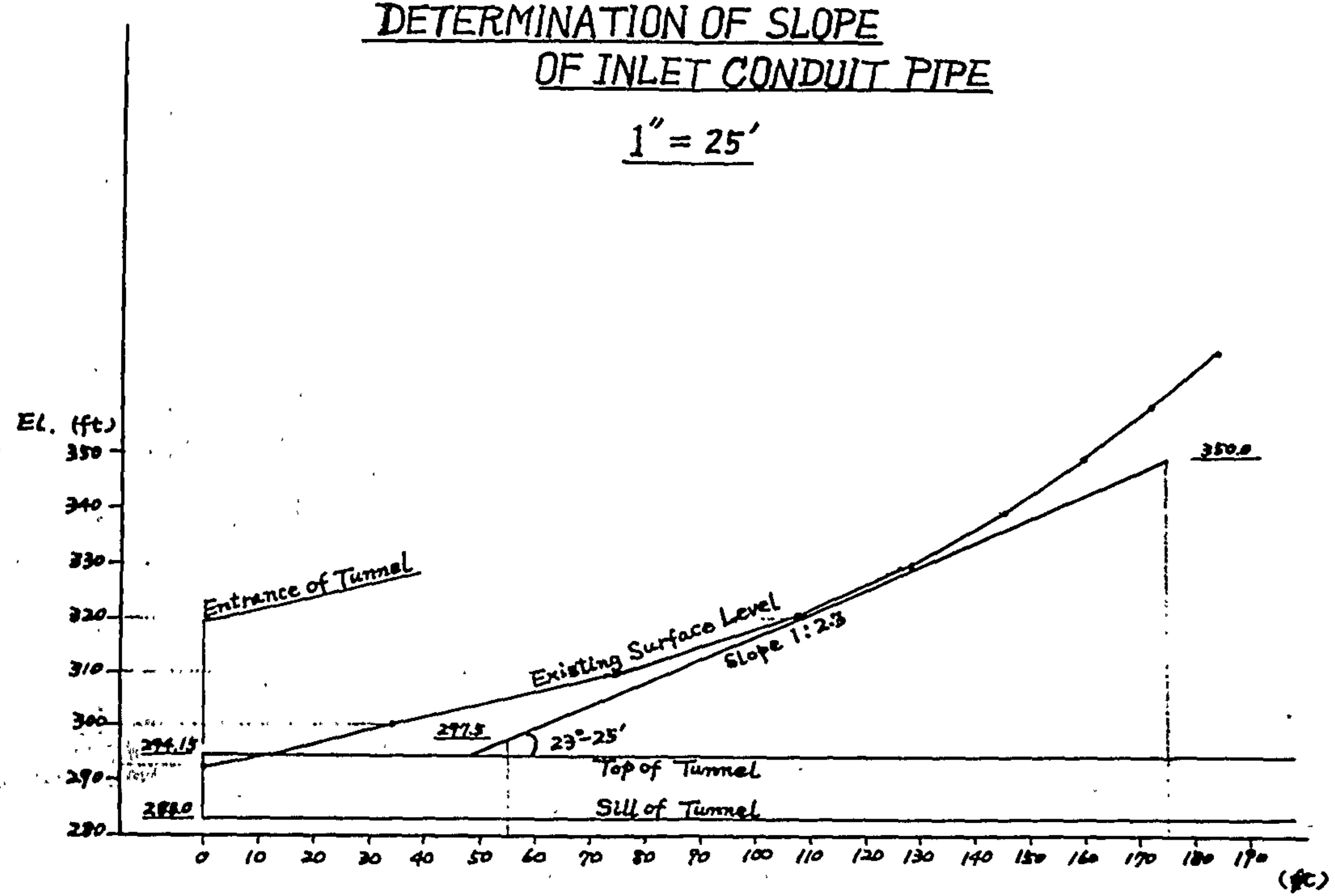
PROFILE OF SIDE WEIR TYPE — SPILLWAY



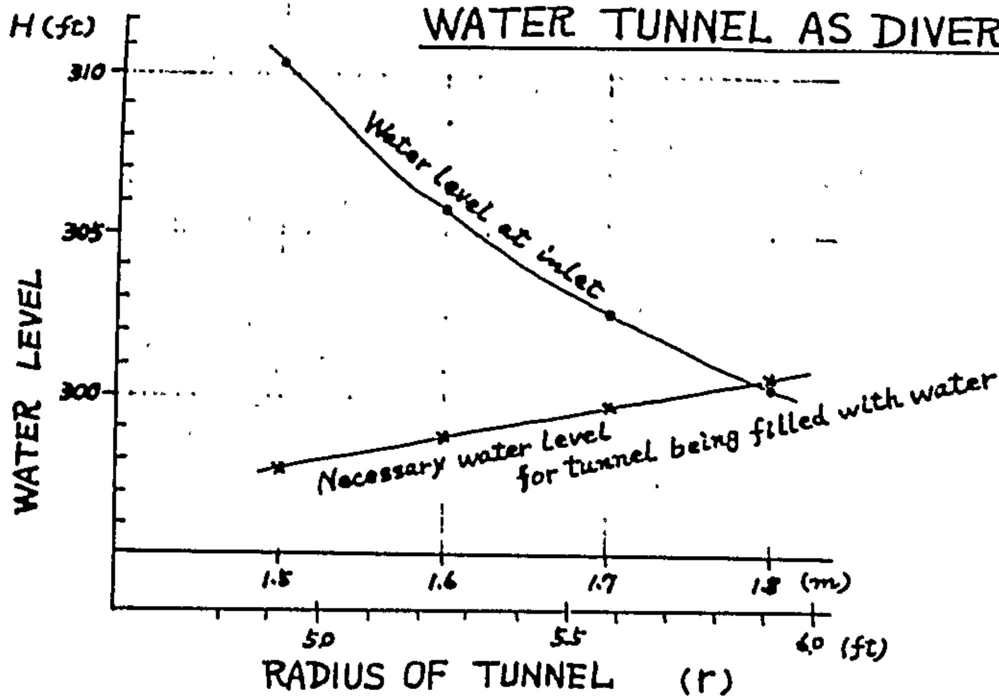
(Fig.5)

DETERMINATION OF SLOPE OF INLET CONDUIT PIPE

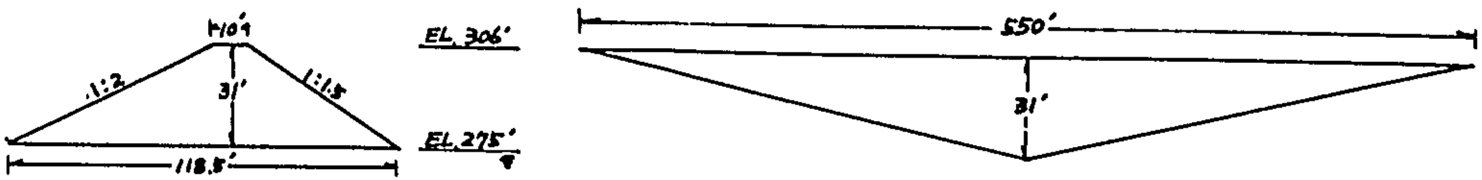
$1'' = 25'$



WATER TUNNEL AS DIVERSION CHANNEL (Fig.6)



COFFER DAM (Fig.7.)



CROSS SECTION

LONGITUDINAL SECTION

(3) PLUM MITAN (I) September, 1967

MOVEMENT

6, Apr. Visited Plum Mitan to see the conditions in dry season with Mr. J. V. Smith (Superintendent North)

On that occasion we visited

1. Main irrigation control
2. Block 11 along the Cuche River
3. Block 11 and 111, Main Drain along the Road 2
4. Confluence of Cuche and intake to Block 111
5. Sand Hill and Jagroma Cut from Road 3

It was in the hargest time of water melon, and discussión were held about the profit of water melon against invested capital (mainly wages) except family labour

$300 T.T./acre

Total income

$2,300 T.T./acre

15, Aug. I went there again to look at rice transplanting and the conditions in wet season with Mr. Smith, and saw the irrigation control that is:

M.I.C. 1	I.C.1/1	I.C. ½	I.C. 1/3	along the Cuche
M.I.C. 2	I.C. 2/1	I.C. 2/2	I.C. 2/3	along the Road 3

and saw Block 11 being transplanted along Caltoo Road.

From the end of the road, we went down to a dragline that was working to excavate and embank the soil. Then on Perimeter Drain by boat and went up to Sand Hill, and saw Jagroma Cut in which greases were growing thick, and two ponds near the cut in which a lot of cascadeaux were being kept.

On the way to Sand Hill, was shown several timbers on the shore that were brought from the forest by a bull and were kept as the sleeper for the dragline.

Description studied

Making observation of geographical position, all the rivers that flow southeast from Central Range have short lengths, small catchment areas, and the mouths which open to Nariva Swamp. The highest mountain near this area is Mt. Harris which has the elevation of 884 ft. The range continues mostly in the height of 400 to 500 ft. The sources of the rivers come out from the range and the catchment areas up to the mouth to Nariva Swamp are as follows:

(Chart 1)

	Catchment Area	
Huila River	1.40 sq. miles	3.63 km^2
Petit Poole River	2.43	6.30
Mitan River	1.37	3.56
Cuche River	4.98	12.89
Canque River	4.49	11.62
Biche River Caratal River)	5.00	12.94

The elevation of Plum Mitan rice area ranges from about 8 ft to 0 ft. The area is situated slightly above the high tide and has sand Hill southeast. According to the observation at the station of Plum Mitan Rice Scheme, the monthly rainfall precipitation is like the table below and mean rainfall a year shows nearly 100 inches.

(Chart 2)

inches

Month / Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1956	4.48	5.47	5.12	6.32	6.57	9.87	6.23	15.71	11.50	13.58	13.56	15.18	113.59
1957	6.42	2.57	0.53	10.15	7.23	9.68	13.67	7.21	6.86	4.45	15.28	10.48	94.54
1958	6.68	3.72	1.21	4.17	11.53	17.67	-	7.47	5.65	14.90	8.68	7.17	-
1959	2.05	0.97	0.61	-	-	9.15	6.57	10.38	5.24	13.91	16.49	7.26	-
1960	-	-	-	-	-	-	-	-	-	-	-	-	-
1961	-	-	-	-	-	6.37	10.52	8.39	13.59	16.01	15.04	4.69	-
1962	4.10	5.66	0.42	2.02	1.88	9.45	9.54	10.81	1.89	1.30	8.71	9.31	65.09
1963	2.54	5.77	2.07	1.53	7.72	16.52	13.52	3.73	17.38	2.21	11.31	12.05	96.35
1964	3.55	1.69	2.08	4.91	7.36	23.29	16.16	7.38	11.10	16.18	13.54	14.52	121.76
1965	9.19	3.62	1.15	1.60	6.34	11.81	12.41	10.88	3.29	10.85	17.63	15.29	104.07
Total	39.01	29.47	13.19	30.70	48.63	113.81	88.63	81.96	76.50	93.39	120.24	95.96	595.40
Mean	4.88	3.68	1.65	4.39	6.95	12.65	11.08	9.11	8.50	10.38	13.36	10.66	97.29

In this area, rice expansion scheme was carried out in May 1952 to August, 1957. 4 Blocks of land, sections roads, canals and gates were designed and constructed and rice-cultivation were gradually begun. But the rice cultivations are now only being done in parts of Block 11 and 111, not in Block 1 and IV yet. The area where rice is not being raised in Block 11 and 111 is too high to take the water from the gates . . . . . . It seems to be the main rason. It is said that there are enough water to irrigate the enough labour to keep rice fields at present.

The estimated areas of each block are as follows.

(Chart 3)

Block I	100 acres
Block II	150
Block III	240
Block III	100
Block IV	355
Total	945

It is more than about ten years since the facilities for rice cultivation have been constructed, and they seemed to be pretty poor condition due to shortage of maintenance work. For example main access road along the Cuche Cut is very uneven and muddy, and can barely pass a car. The distributary No. 3 in Block II is filled up with mud and grass as much so that it does not look like a canal. A few irrigation control gates are bent and cut their spindle, and all of them are very rusted.

Concerning the drainage a dragline is operating on the perimeter drain for expansion of drain and embankment to prevent the high tide intrusion from the Nariva Swamp good results therefore can be expected. The rice transplanting was carried out too late. The seedlings were mature but under nourished, although the rainy season had come later this year than usual. The interval of transplanting was too far and irregular, and the field was not smoothed due to too wide boundary to level with no machinery.

RECOMMENDATION

1. Improvement of Rice-Cultivation Technique

This matter will be left to another expert. But it can be said that the efficient rice-cultivation in Plum Mitan must be carried out with the help of machinery and livestock.

2. Arrangement and Expansion of Maintenance Work

It is feared that all that has been excellently constructed about ten years ago will be ruined through unless a radical maintenance work is done at present.

a. Road and embankment:-
   levelling uneven surface
   bank revetment in some parts
   spreading gravel on the surface

b. Canal especially irrigation canal:-
   lining all the irrigation canal if possible so as not to lose the water and to save the expense to maintain
   excavating mud from the bed of both irrigation canal and drain
   taking out the grasses from the canal

c. Control gate
repainting all the gates
changing the spindles of some of them
replacing the frames of some of them or bolting
repairing the concrete structures

3. Changing all the area for paddy fields

There is still no paddy dield in Block I and IV. Rice cultivation is being done only in parts of Block II & III. Now that rice expansion scheme is being carried, it is imperative that rice is raised in the entire area. The reason for paddy field not being cultivated must be studied and remedial measures taken. As mentioned above, the water cannot be taken from the canal to the land concerned due to the high elevation. From the contour map, however, this does not appear to be the case.

The intake water levels are as follows:-

Main Irrigation Control 1.	105 on the map (nearly El. 5 ft)
Main Irrigation Control 2.	108 on the map (El. 8 ft)
Main Irrigation Control 3.	108 on the map (El. 8 ft)

Of course these intake water levels are higher than each controlled area. If the water level were not raised by sluice gates as expected, or the contour lines were in error. The water could not be led smoothly to the areas. However, a proper investigation is necessary. In the former case the means to raise the water as expected must be considered, for example, new main control gates for intakes of Block III and IV. In the latter case, conveying the soil from high parts to lower the base must be examined and the raising of the water to proposed level must also be considered.

In block I and IV, legal procedures have not finished yet. They should be solved in the early stages.

4. Water in Dry Season

The main crops are water melons, tomatoes, peas, beans, cabbages, and other vegetables. All the rivers in this area that have small catchment areas and cannot supply enough water for the crops. The water requirements have to be calculated first. On the other hand, proposed dam site to store the water in rainy season has to be decided after studying its possible water quantity collected from the catchment area and available storage capacity.

a. Calculation of water requirement in dry season

Net water requirement was calculated as 19.2 inches in dry season (cf. chart 5)
Total net water requirement

$$\underset{\text{(inches)}}{19.2} \times \frac{1}{12} \times \underset{\text{(acres)}}{945} \times 43{,}560 = 65{,}862{,}720\ \text{ft}^3 = 1{,}865{,}232\ \text{m}^3$$
$$= (410{,}247{,}686\ \text{gallons}).$$

(Chart 4) Rainfall Precipitation in 20% Wet at Plum Mitan (inches)

Month	NARIVA			PLUM MITAN		Remarks
	Mean monthly rainfall (A) (A)	20% wet (B)	Ratio (C) = (B)/(A) %	Mean monthly rainfall (D)	20% wet (E) = (D) x (C)	
Jan.	6.86	2.73	39.8	4.88	1.94	
Feb.	4.26	1.13	26.5	3.68	0.98	The rainfall in 20% wet is
Mar.	3.54	1.39	39.3	1.65	0.65	derived from F.A.O. report.
Apr.	4.17	1.37	32.9	4.39	1.44	
May	9.01	3.53	39.2	6.95	2.72	
June	13.50	10.24	75.9	12.65	9.60	
July	11.81	8.73	73.9	11.08	8.19	
Aug.	11.96	10.50	87.8	9.11	8.00	
Sept.	8.80	7.89	89.7	8.50	7.62	
Oct.	9.12	6.87	75.3	10.38	7.82	
Nov.	11.05	7.04	63.7	13.36	8.51	
Dec.	12.56	4.14	33.0	10.66	3.52	
Total	106.64	65.56		97.29	60.99	Ratio 60.99/97.29=62.3%

(Chart 5) Net Water Requirement for Vegetables

Month	Rainfall in 20% wet (above table)	Consumptive water quantity (vegetable)	Net water requirement	Remarks
Jan.	1.94	5.0	3.1	Consumptive water
Feb.	0.98	4.8	3.8	quantity for vegetable
Mar.	0.65	5.4	4.8	is shown in detail on
Apr.	1.44	5.3	3.9	the report. Tucker
May	2.72	5.4	2.7	Valley (II)
June	9.60	4.3	-	
July	8.19	4.7	-	
Aug.	8.00	4.7	-	
Sept.	7.62	4.3	-	
Oct.	7.82	4.6	-	
Nov.	3.51	4.3	-	
Dec.	3.52	4.4	0.9	
Total	60.99	57.2	19.2	

(Charr 6) Mean Raining Days Classified with Rainfall Precipitation at Plum Mitan (1956-1965)

(Days)

Month / Rainfall	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
0-0.2"	12.2	10.0	10.5	9.9	10.9	10.8	11.3	12.1	10.2	7.3	7.7	11.4	124.3
0.2-0.4	3.7	2.9	0.9	2.9	3.3	4.7	4.5	4.0	2.9	3.7	4.8	5.6	43.9
0.4 0.6	1.1	1.0	0.1	0.9	1.9	2.1	1.9	2.0	2.1	1.6	1.7	18.5	
0 6-0.8	1.0	0.2	0.1	0.9	0.7	2.4	2.1	1.9	1.4	1.8	2.1	2.2	16.8
0.8-1.0	0.1	0.1	0.1	0.1	0.4	1.4	0.6	0.9	0.6	1.3	1.3	1.1	8.0
1.0 1.2	0.2	0.1	0.2	0.4	0.9	0.8	1.1	0.8	0.4	0.9	1.0	0.8	7.6
1 2-1.6	0.1	0.1	0	0	0.4	1.4	1.4	0.6	0.7	1.1	1.1	0.8	7.7
1.6-2.0	0.2	0.2	0	0	0.4	0.7	0.4	0.2	0.2	0.4	1.1	0.2	4.0
2.0-2.4	0.2	0.1	0	0.1	0.3	0.4	0.1	0.2	0.2	0.1	0.2	0 2	2.1
2.4-3.2	0	0.1	0	0.3	0.1	0.3	0.4	0.2	0.3	0.3	0.4	0.3	2.7
3.2 4.0	0	0	0	0	0	0.1	0.1	0.0	0.1	0.2	0.1	0.1	0.7
4.0-6.0	0	0	0	0	0	0	0	0.1	0.1	0.1	0.2	0.1	0.6
Total	18.8	14.8	11.9	15.5	19.3	25.1	24.1	22.9	19.1	19.3	21.6	24.5	236.9

(Chart 7) Calculation of Mean Run-off Coefficient

	Mean Raining Days	Mean Rainfall	Correction	Run-off Coeff.	Run-off	Remarks
0" - 0.2"	124.3	x 0.1 = 12.43"	x 0.99428 12.36	20%	2.472"	
0.2 - 0.4	43.9	x 0.3 = 13.17	13.09	30	3.927	Mean Run-off Coeff.
0.4 - 0.6	18.5	x 0.5 = 9.25	9.20	40	3.680	45.07/97.29=46.3%
0 6 - 0.8	16.8	x 0.7 = 11.76	11.69	42	4.910	
0.8 - 1.0	8.0	x 0.9 = 7.20	7.16	45	3.222	
1.0 - 1.2	7.6	x 1.1 = 8.36	8.31	50	4.155	
1.2 - 1.6	7.7	x 1.4 = 10.78	10.72	55	5.896	
1.6 - 2.0	4.0	x 1.8 = 7.20	7.16	60	4.296	
2.0 - 2.4	2.1	x 2.2 = 4.62	4.59	65	2.984	
2.4 - 3.2	2.7	x 2.8 = 7.56	7.52	70	5.264	
3.2 - 4.0	0.7	x 3.6 = 2.52	2.51	75	1.883	
4.0 - 6.0	0.6	x 5.0 = 3.00	2.98	80	2.384	
Total	97.29/97.85= 0.99428	97.85	97.29		45.073	

b. Calculation of the possible water quantity collected from catchment areas of proposed dam site. (cf. Fig. 2). After studying appropriate dam site on a map drawn to a scale of one to fifty thousand, the following conclusion was reached:-

On the Huila River the Petit Poole River and the Mitan River in the north of Plum Mitan, appropriate dam sites cannot be found due to the bad lay of land and small catchment area. However, one can be found in the upper reach of the Cuche River where two main tributaries meet. Of course its catchment area is wide enough to collect water for the whole of Plum Mitan in dry season. Similarly it can be found on the Canque River close by Biche Village and it catchment area is wide enough to collect the water required.

The village has been damaged by the flood from the Canque River lately. Therefore, if this proposed dam is used for the purpose of both irrigation and flood control, it will have more effect. The Biche River and the Caratal River in the south of Plum Mitan both have good lay of land, but very small catchment area. The Charuma River is too far from there. As a result of studying it is desirable to build a dam on the Cuche or/and the Canque.

The new irrigation canal will supposedly not be necessary and the water will flow directly from the dam into each river and be irrigated by main irrigation control, even if some high field may be requisite for pumps to raise the water.

Catchment area of the Cuche River is 1.93 $mile^2$ (= 5.01 km^2)
Catchment area of the Canque River is 2.56 $mile^2$ (=6.36 km^2)
Yearly rainfall in 20% wet is 60.99 inches (cf. Chart 4)
Mean run-off coefficient is 46.3% (cf. Chart 7).

Thus the possible water quantity collected from the catchment areas are as follows:

The Cuche River

$$5{,}010{,}000\ m^2 \times 60.99\ \text{inches} \times 0.0254\ m/\text{inch} \times 0.463 = 3{,}590{,}000 m^3$$

The Canque River

$$6{,}360{,}000\ m^2 \times 60.99\ \text{inches} \times 0.0254\ m/\text{inch} \times 0.463 = 4{,}560{,}000\ m^3$$

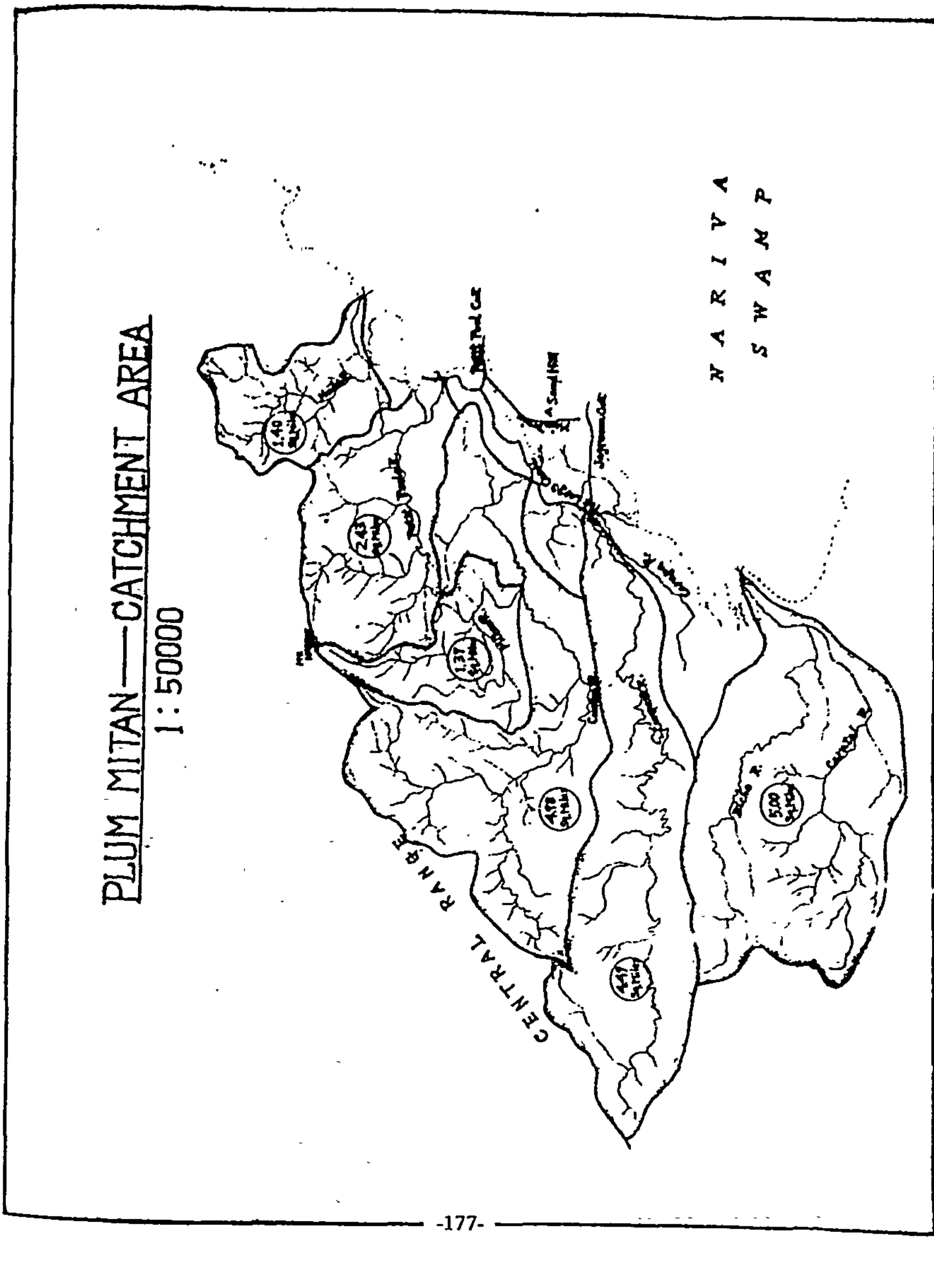
Total net water requirement in 20% wet is 1,865,000 m^3 = (410,248,000 gallons).

It means that the possible water quantity collected from the catchment area of the Cuche River is about twice as much as total net water requirement, and the one of the Canque River is about two and a half.

Both can supply the water requirement in 20% wet in Plum Mitan.

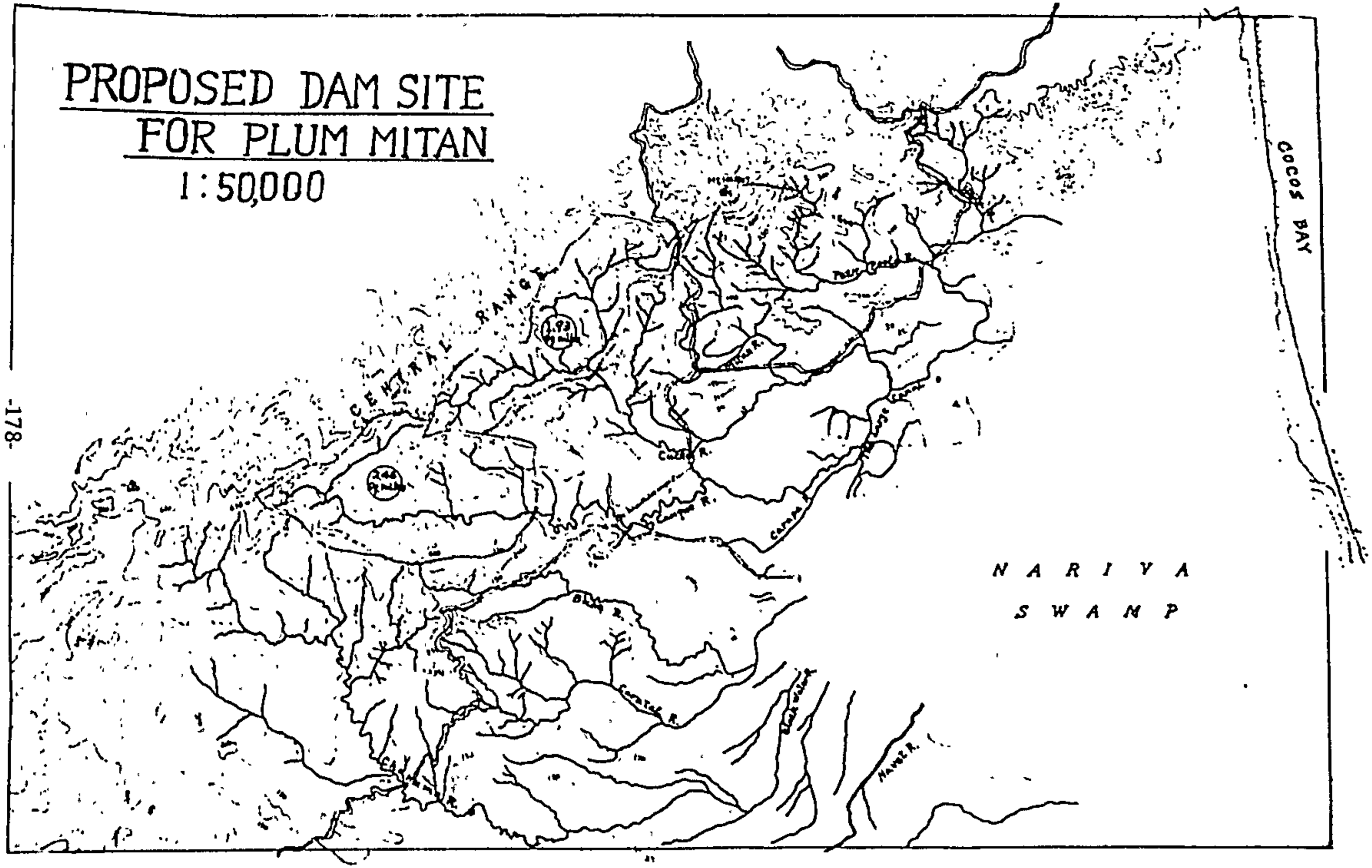
The following studies will be necessary in case that one out of two is chosen. Topographical and geological studies; the relationship between storage capacity and dam height, length, and volume; and feasibility of material supply for dam.

Junichi Kitamura


PLUM MITAN —— CATCHMENT AREA
1 : 50000
CENTRAL RANGE
NARIVA SWAMP

PROPOSED DAM SITE FOR PLUM MITAN

1:50,000



PLUM MITAN (II) September 1967

MOVEMENT

30, Aug.	Investigated the proposed dam site on the Cuche River with Mr. C. Taylor. Mr. E. Wyke and Mr. J.V. Smith. It is situated at the confluence of two tributaries about one mile west from Poole Village along Cunapo Southern Road.
7, Sept.	Obtained maps drawn to the scale of one by ten thousand and one by twenty-five thousand from mapping & control, Lands & Survey Division.
12, Sept.	Investigated the proposed dam site on the Canque River with Mr. J. V. Smith with help of cutlassmen. It is situated on the river about 0.3 mile north from the pump house and the small dam for water supply to Biche Village. Explored about a half a mile upward from the dam site.

Present aspect at proposed dam site

1. The Cuche River

The topographical condition on both sides of the site can hardly be surveyed because of this vegetation. It can be supposed however, from the contoured map that the site has very good condition topographically and a high storage efficiency. Fragments of rocks on the river bed mainly consist of hard sandy or silty shale, partially limestone. Althougoh the detailed base-surveying by boring is needed hereafter, cement grouting must be carried out enough as the curtain to prevent leakage from dam.

As to the bearing capacity of the base, the site has no problem because of the low dam of 75 ft in height as mentioned below. The road for the works will be available by expansion and improvement of the present trace. Both fill-dam type and concrete dam type can be adopted because of good topographical condition.

In case of fill-dam type a survey of borrow-pits near the site must be carried out, Rock-fill dam type is desirable because sufficient clay as core hard sandy or silty shale as transition and rock are readily available.

In case of concrete dam type study of the method to supply aggregates to the site is needed, as efficient use of sandy or silty shale for the dam is recommended. (cf. figure 1 & 2).

2. Canque River

This dam site is not inconvenient topographically as it is 0.3 mile away from the small dam for water supply to Biche Village. The dam site recommended in the Plum Mitan report (1) on the map of one in fifty thousand, was found impossible to build when checking the detailed map of one in ten thousand. The hills on the left side of a tributory are too low to store the water up to the necessary level.

The new dam site is therefore decided at the place where the water shed of that tributory is as suggested in this report, although a sub-dam will still be needed. (cf. figure 1 & 3).

The length of the dam will become very long, as the left side is not steep. The fill-dam type therefore must be adopted.

Fragments of rock on the river bed are mostly hard sandy or silty shale reddish on the surface, partially sand stone. The base is the same geological condition as the one on the Cuche River. The top soil is supposedly thick especially on the left side. In the hills near the dam site a borrow-pit survey must be carried out.

Calculation of available water quantity collected from catchment area (Continued from Plum Mitan (I).)

The available water-quantity collected from the Canque River was already calculated on the report Plum Mitan (I), but it is calculated here again, because the dam site must be changed and omitted the catchment area of one tributary.

Old catchment area 2.46 sq. miles = 6.36 km^2

New catchment area 2.00 sq. miles = 5.18 km^2

Therefore available water quantity will be

$$5{,}180{,}000m^2 \times 60.99 \text{ inches} \times 0.0254 \text{ m/inch} \times 0.463 = 3{,}715{,}000m^3$$
$$= 817{,}207{,}000 \text{ gals} = 131{,}194{,}000 \text{ ft}^3$$

It is nearly the same as the Cuche River and about twice as much as the net water requirement in 20% wet of 1,865,000m^3 (=410,248,000 gals) so it will be sufficient, even if the losses of evaporation, seepage and water-conveyance is included in the total water requirement.

(Chart 1) Calculation of Storage Capacity

1. On the Cuche River

Contour		Height		Area	Mean Area	Volume	Total Volume
75 ft	>	5	1.5m	5,000 m^2	2,500m^2	3,750m^3	3,750m^3
100	>	25	7.5	73,000	3,900	292,500	296,250
125	>	25	7.5	283,000	178,000	1,355,000	1,631,250
150	>	25	7.5	758,000	520,500	3,903,750	5,535,000

2. On the Canque River (Chart 2) Calculation of Storage Capacity

Contour		Height		Area	Mean Area	Volume	Total Volume
100 ft	>	10 ft	3.0m	30,000m^2	15,000m^2	45,000m^3	45,000m^3
125	>	25	7.5	173,000	101,500	761,250	806,250
150	>	25	7.5	392,000	282,500	2,118,750	2,925,000
175	>	25	7.5	721,000	556,500	4,173,750	7,098,750

Decision of scale of dam

Assuming that 1.5 time of the net water requirement is stored, the storage water capacity will be

$$1,865,000 \times 1.5 = 2,797,725\ m^3 \doteqdot 2,800,000 m^3$$

1. Dam on the Cuche River

Dam height	75 ft (from the lowest river bed)
Dam length	430 ft
Crest elevation	El 145 ft (Full water level El 135 ft)

2. Dam on the Canque River

Dam height	70 ft (from the lowest river bed)
Dam length	900 ft
Crest elevation	160 ft (full water level El 150 ft)
Sub-dam	1 spot (height 0 - 15 ft length 220ft)

Comparison and examination between two dams

As stated above, if the profitable area is limited to only Plum Mitan Rice Expansion Scheme Area, either dam can supply the water requirement. On the other hand if both dams are built, they can supply the still more irrigation water for about 1000 acres in dry season.

Taking the former case here, comparison and examination between the two dams can be done. That is to say the dam site on the Cuche River has much advantages of topography in comparison with the one on the Canque River. Although the dam height, the distance from the dam to the profitable area and the condition of the road for conveying the materials are nearly equal in either case, the dam length ont the Cuche River will be shorter than a half as long as on the Canque River. Consequently the sam volume on the Cuche River will be also less than a half as much as on the Canque River. Besides in the case of the Canque River, the condition will become worse as the sub-dam is necessary to store the water up to the proposed elevation due to low hills in the left side of the river.

If the flood control for Biche Village, however, is considered the dam on the Canque River will be needed by all means, and the scale of it will have to be examined by the calculation from this point of view.

Examination of flood control capability

On condition that the flood in the probability of once every ten years occurs while the dam on the Canque River is in full water level, the flood control capability is examine as below.

1. Decision of scale of spillway

As designed flood discharge for spillway, the discharge of 1.2 time as much as the one in the probability of once every one hundred years must be adopted.

The daily rainfall in the probability of once every one hundred years will be 6.8" according to chart 6

The designed daily rainfall, therefore, will be 6.8" x 1.2 = 8.16" (= 207m)

Length from the most upper reach of the river to the proposed dam site 1 = 4,500m

Difference of elevation between the upper reach and the proposed dam site,

$H = 350ft - 150ft = 200ft = 60m$

Velocity of flow in this river,

$$W = 72 \frac{(H)^{0.6}}{1} = 72 \frac{(60)^{0.6}}{4,500} = 72\,(0.0133)^{0.6} = 72 \times 0.0749 = 5.39 \text{ (km/hr)}$$

Time required to reach the proposed dam site from the most upper reach,

$$T = 1/W = \frac{4.5}{5.3} = 0.835 \text{ hr} \fallingdotseq 1 \text{ hr.}$$

Mean hourly rainfall intensity within the period till arrival of flood,

$$r_t = \frac{R_{24}}{24} \times \frac{(24)^{2/3}}{T} = \frac{207}{24} \times \frac{(24)^{2/3}}{1} = 8.63 \times 8.32 = 71.7 \text{ (mm/hr)}$$

Catchment area, $A = 2.0$ sq. miles $= 5.18$ km^2

in which, hilly land area $A_1 = 4.79$)

full water area $A_2 = 0.39$)

Run-off coefficient on hilly land at the peak of the flood.

f_1 0.8 (in case of the calculation for spillway)

Run-off coefficient on water surface

$f_2 = 1.0$

Therefore, maximum flood discharge,

$Q_1 = 0.2778 f_1 rtA_1 = 0.2778 \times 0.8 \times 71.7 \times 4.79 = 76.33$ m^3/s

$Q_2 = 0.2778 ft_2 rtA_2 = 0.2778 \times 1.0 \times 71.7 \times 0.39 = 7.77$

Total will be = 84.10 m^3/s

Length of the spillway

$$Q = cLH^{2/3} \quad \therefore\ L = \frac{Q}{cH^{2/3}} = 2.158 \text{ (cf. below)}$$

Assuming that the overflow depth H to maximum flood discharge is 1.2m (=4ft), length of the spillway will be

$$L = \frac{84.10}{2.158 \times (1.2)^{2/3}} = \frac{84.10}{2.158 \times 1.3145} = \frac{84.10}{2.8919} = 29.70 \text{ m } (\fallingdotseq 100 \text{ ft})$$

Concerning the discharge coefficient C, there is the following formula,

$$Cd = 2,200 - 0.0416 \frac{(Hd)}{W}$$

Assuming that W is 1.2m, Cd will be

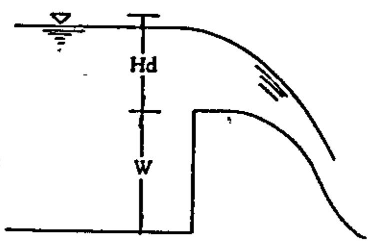
$Cd = 2,200 - 0.0416 = 2.158$ as Hd is 1.2m

Cd: discharge coefficient in H = Hd

C: discharge coefficient to optional hydrostatic head

Hd: designed overflow depth to maximum flood discharge

H: optional overflow depth



On the other hand, there is the following general formula.

$$C = \frac{1.60\,\left(1 + 2a \frac{(H)}{Hd}\right)}{1 + a \frac{(H)}{Hd}} \qquad \text{In-case of } H = Hd \quad C = Cd = 2.158$$

Therefore, $a = 0.5361$

$$C = 1.60 \frac{1 + 1.0722 \frac{(H)}{(Hd)}}{1 + 0.0536 \frac{(H)}{(Hd)}}$$

2. Calculation of discharge from spillway

½ Qdt corresponding with each overflow depth H is calculated from formula

$Q = cLH^{3/2}$ and as dt = 15 minutes as the following table.

(Chart 3)

H	H/Hd	$1+1.0722\frac{H}{Hd}$	$1+0.536\frac{H}{Hd}$	C	L	$H^{3/2}$	Q	½Qdt
m					m		m^3/s	m^3
0.01	0.0083	1.0089	1.0045	1.605	30.0	0.0010	0.05	23
0.05	0.0416	1.0446	1.0223	1.637	30.0	0.0112	0.55	248
0.10	0.0833	1.0893	1.0447	1.668	30.0	0.0316	1.58	711
0.20	0.1667	1.1787	1.0894	1.731	30.0	0.0894	4.64	2,088
0.30	0.2500	1.2680	1.1340	1.788	30.0	0.1643	8.81	3,965
0.40	0.3333	1.3577	1.1789	1.844	30.0	0.2530	13.98	6,291
0.50	0.4167	1.4465	1.2233	1.890	30.0	0.3536	20.50	9,225
(0.51)	(0.4250)	(1.4557)	(1.2278)	(1.897)	(30.0)	0.3642	(20.73)	(9,329)
0.60	0.5000	1.5361	1.2681	1.937	30.0	0.4648	27.00	12,150
0.70	0.5830	1.6250	1.3125	1.980	30.0	0.5857	34.80	15,660
1.00	0.8333	1.8930	1.4465	2.097	30.0	1.0000	62.90	28,305
1.20	1.0000	2.0722	1.5361	2.158	30.0	1.3145	85.00	38,250

3. Calculation of flood discharge from catchment area maximum daily rainfall in the probability of once every ten years, $R_{24} = 5.5" = 139.7mm$ (cf. Chart 6)

Mean hourly rainfall intensity within the period till arrival of the flood

$$R_t = \frac{R24(24)^{2/3}}{24\,T} = \frac{139.7}{24} \times \frac{(24)^{2/3}}{1} = 5.82 \times 8.32 = 48.4 \text{ mm/hr}$$

Run-off coefficient on hilly land at the peak of the flood

$f_1 = 0.45$

Run-off coefficient on water surface

$f_2 = 1.00$

Maximum flood discharge

$Q_1 = 0.2778 f_1 \; RtA_1 = 0.2778 \times 0.45 \times 48.4 \times 4.79 = 28.98$

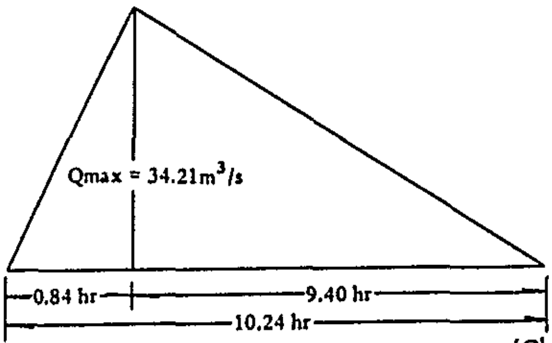
$Q_2 = 0.2778 f_2 rtA_2 = 0.2778 \times 1.00 \times \underline{48.4 \times 0.39 = 5.23}$

Total $34.21 m^3/s$

Time from the peak of the flood till its termination.

½ Qmax. $(1 + \lambda)$ 3,600T = 1,000f' R_{24}A Total run-off coefficient f' is 0.8

½ x 34.21 $(1 + \lambda)$ 3,600 x 0.835 = 1,000 x 0.8 x 139.7 x 5.18



$51{,}417.63\,(1+\lambda) = 578{,}916.8$

$(1+\lambda) = \dfrac{578{,}916.8}{51{,}417.63}$

$\lambda = 11.26$

$\lambda T = 11.26 \times 0.835 = 9.40$ hr

(Chart 4)

Time	q	qdt	½(q₁+q₂)dt	Time	q	qdt	½(q₁+q₂)dt
min	m^3/s	m^3	m^3	min			
0	0	0	0				
15	10.18	9,162	4,581	360	15.43	13,887	14,297
30	20.36	18,324	13,743	375	14.52	13,068	13,478
45	30.54	27,486	22,905	390	13.61	12,249	12,659
(50.4)	(34.21)	(30,789)	-	405	12.70	11,430	11,840
60	33.63	30,267	30,276	420	11.79	10,611	11,021
75	32.72	29,448	29,858	435	10.88	9,792	10,202
90	31.81	28,629	29,039	450	9.97	8,973	9,883
105	30.90	27,810	28,110	465	9.06	8,154	8,564
120	29.99	26,991	27,401	480	8.15	7,335	7,745
135	29.08	26,172	26,582	495	7.24	6,516	6,926
150	28.17	25,353	25,763	510	6.33	5,697	6,107
165	27.26	24,534	24,944	525	5.42	4,878	5,288
180	26.35	23,715	24,125	540	4.51	4,059	4,469
195	25.44	22,896	23,306	555	3.60	3,240	3,650
210	24.53	22,077	22,487	570	2.69	2,421	2,831
225	23.62	21,258	21,668	585	1.78	1,602	2,012
240	22.71	20,439	20,849	600	0.87	783	1,193
255	21.80	19,620	20,030	(6,144)	(0)	(0)	-
270	20.89	18,801	19,211	615	0	0	573
285	19.98	17,982	18,392	630	0	0	0
300	19.07	17,163	17,573				
315	18.16	16,344	16,754				
330	17.25	15,525	15,935				

4. Relation between the storage capacity and the water depth above the sill of spillway which is the elevation of 150 ft.

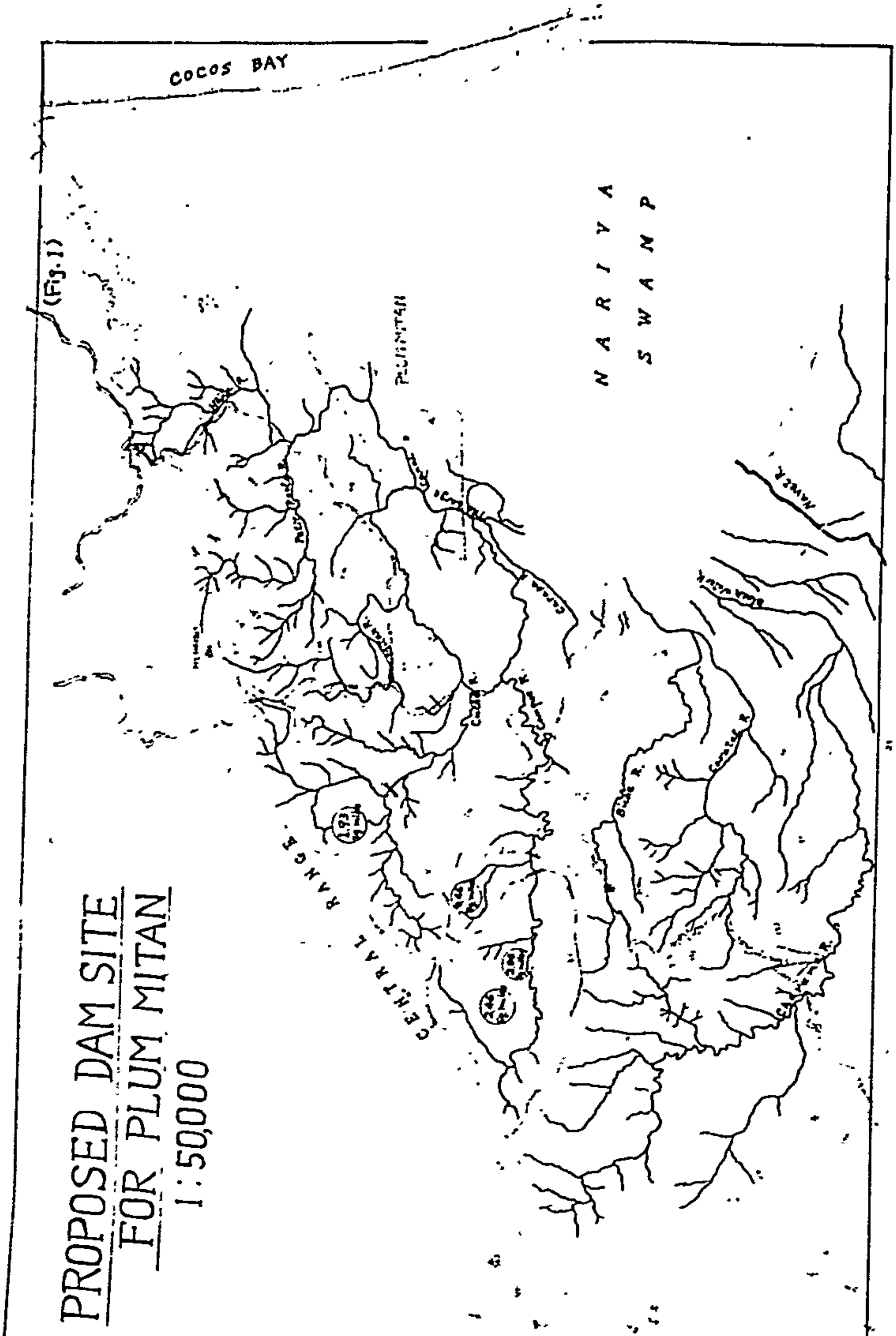
(Chart 5)

Elevation	Each depth	Area	Mean area	Volume	Total volume	Remarks
ft	m	m^2	m^2	m^3	m^3	
150 >	-	392,000	-	-	-	
151 >	0.3	405,000	398,500	119,550	119,550	
152 >	0.3	417,000	411,000	123,300	242,850	
153 >	0.3	428,000	422,500	126,750	369,600	
154 >	0.3	439,000	433,500	130,050	499,650	
155 >	0.3	450,000	444,500	133,350	633,000	

5. Result of the calculation for the flood control capability on diagram

In case that the flood in the probability of once every ten years occurs, in the condition that the dam on the Canque River is in full water level, maximum water level will reach up to 0.51m (=1.7ft) above the sill of the spillway which is built 100 ft. wide and the maximum flood discharge of 20.73 m^3/s, that is, about 40% of the maximum flood discharge can be cut by the dam.

Junichi Kitamura


COCOS BAY
(Fig.1)
PLUM MITAN
NARIVA SWAMP
CENTRAL RANGE
PROPOSED DAM SITE
FOR PLUM MITAN
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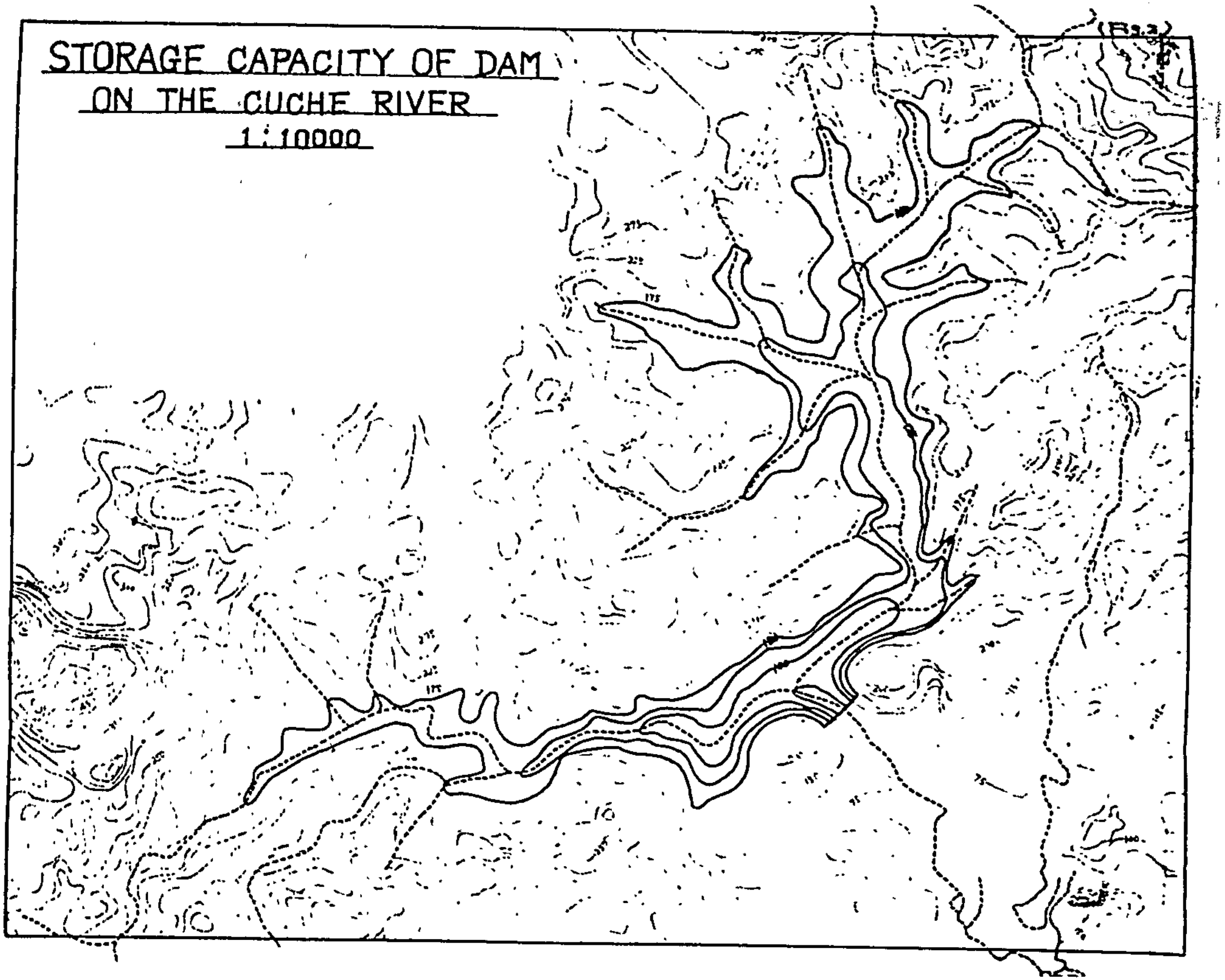
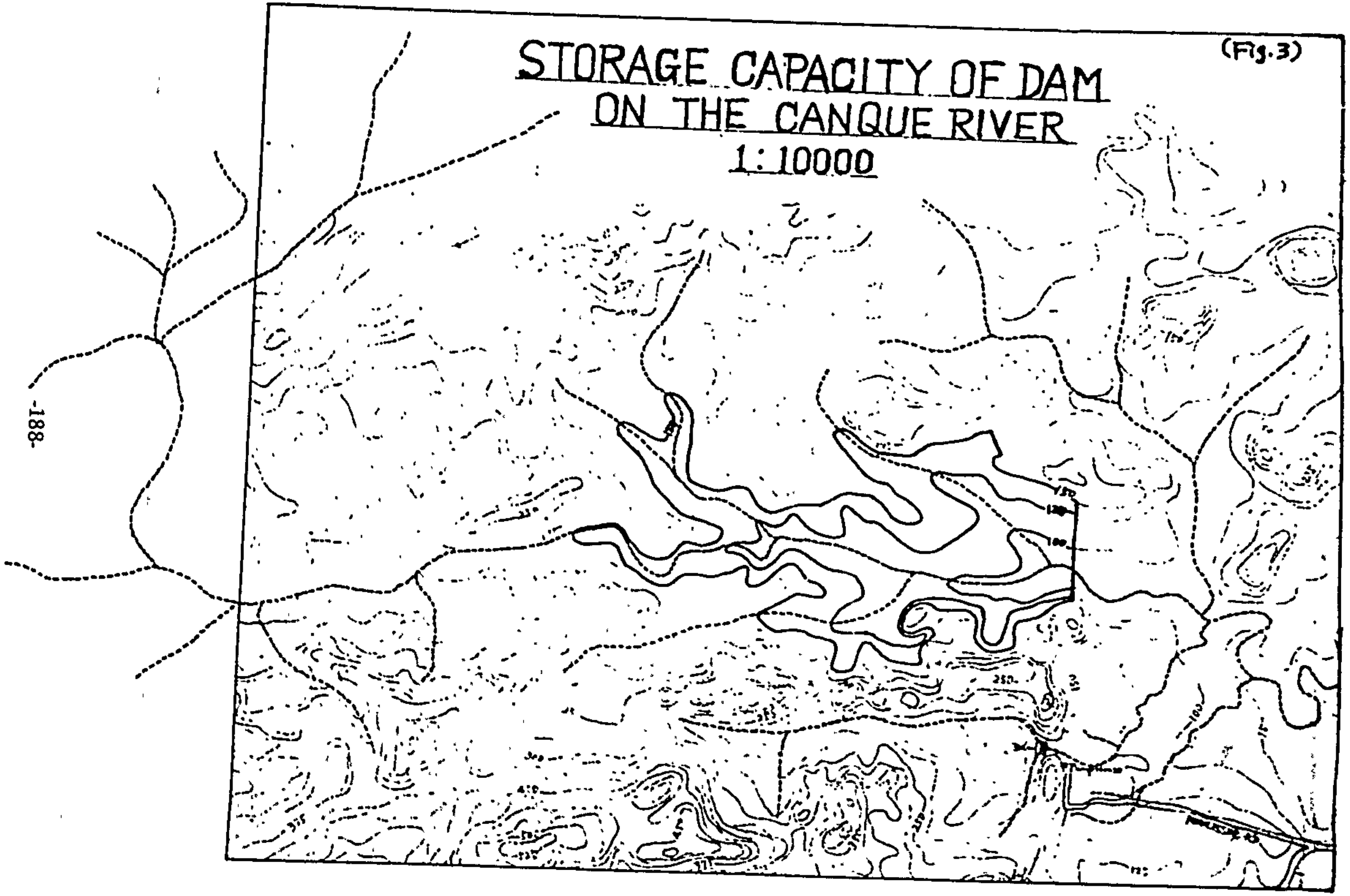


STORAGE CAPACITY OF DAM
ON THE CUCHE RIVER
1:10000

(Fig.3)



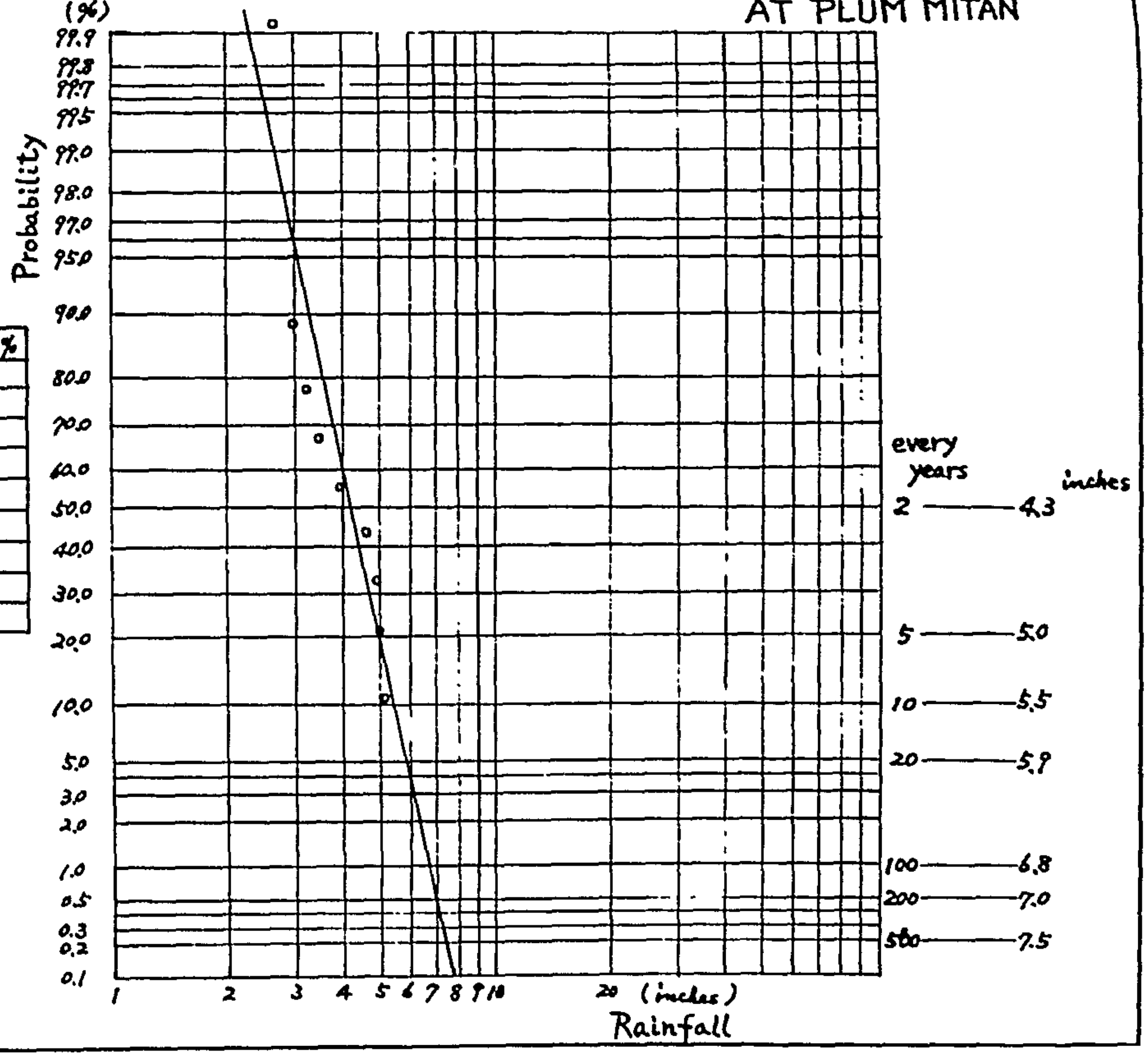
STORAGE CAPACITY OF DAM ON THE CANQUE RIVER

1:10000

PROBABILITY CURVE OF DAILY RAINFALL AT PLUM MITAN

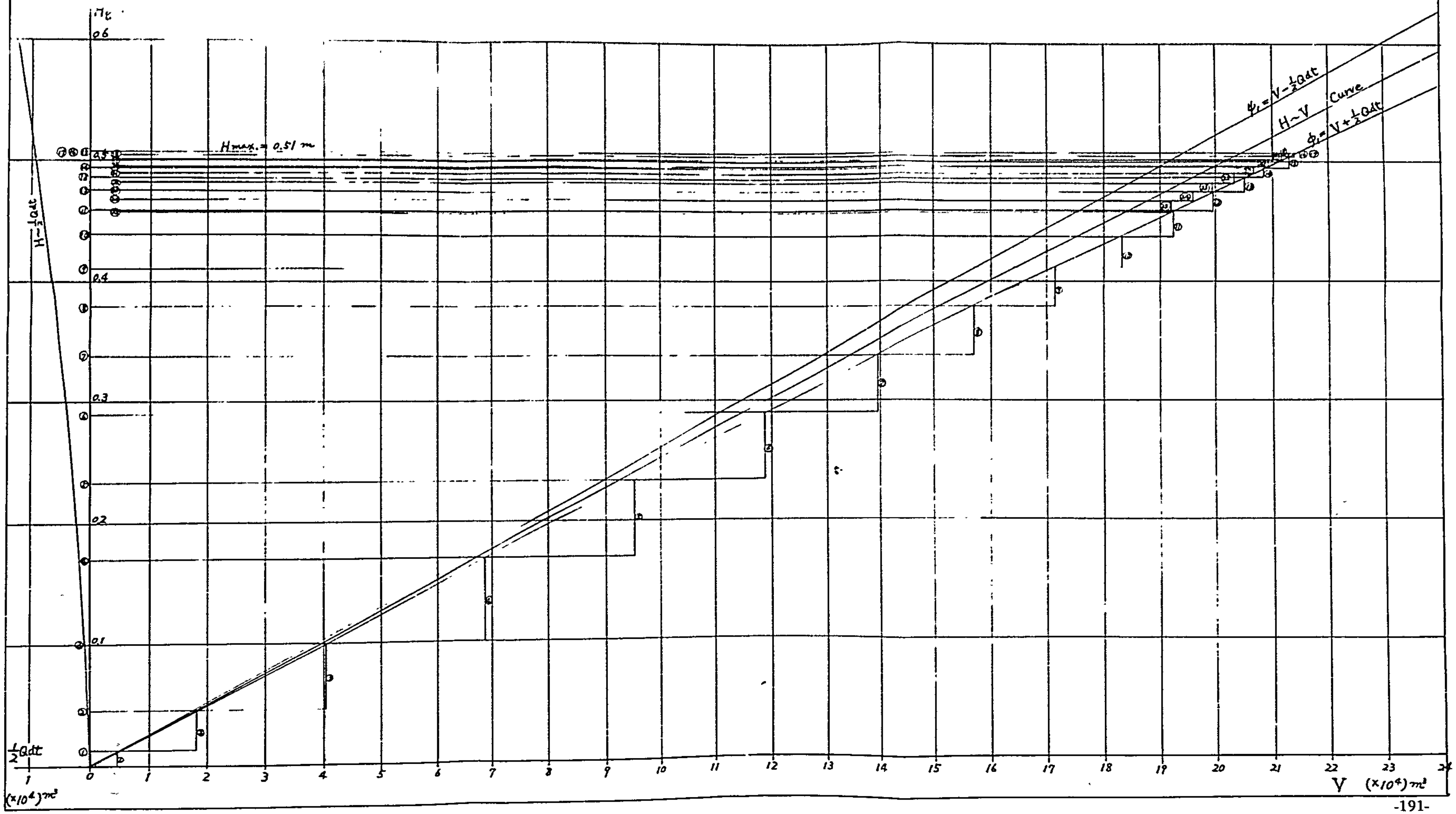
(Fig. 4)

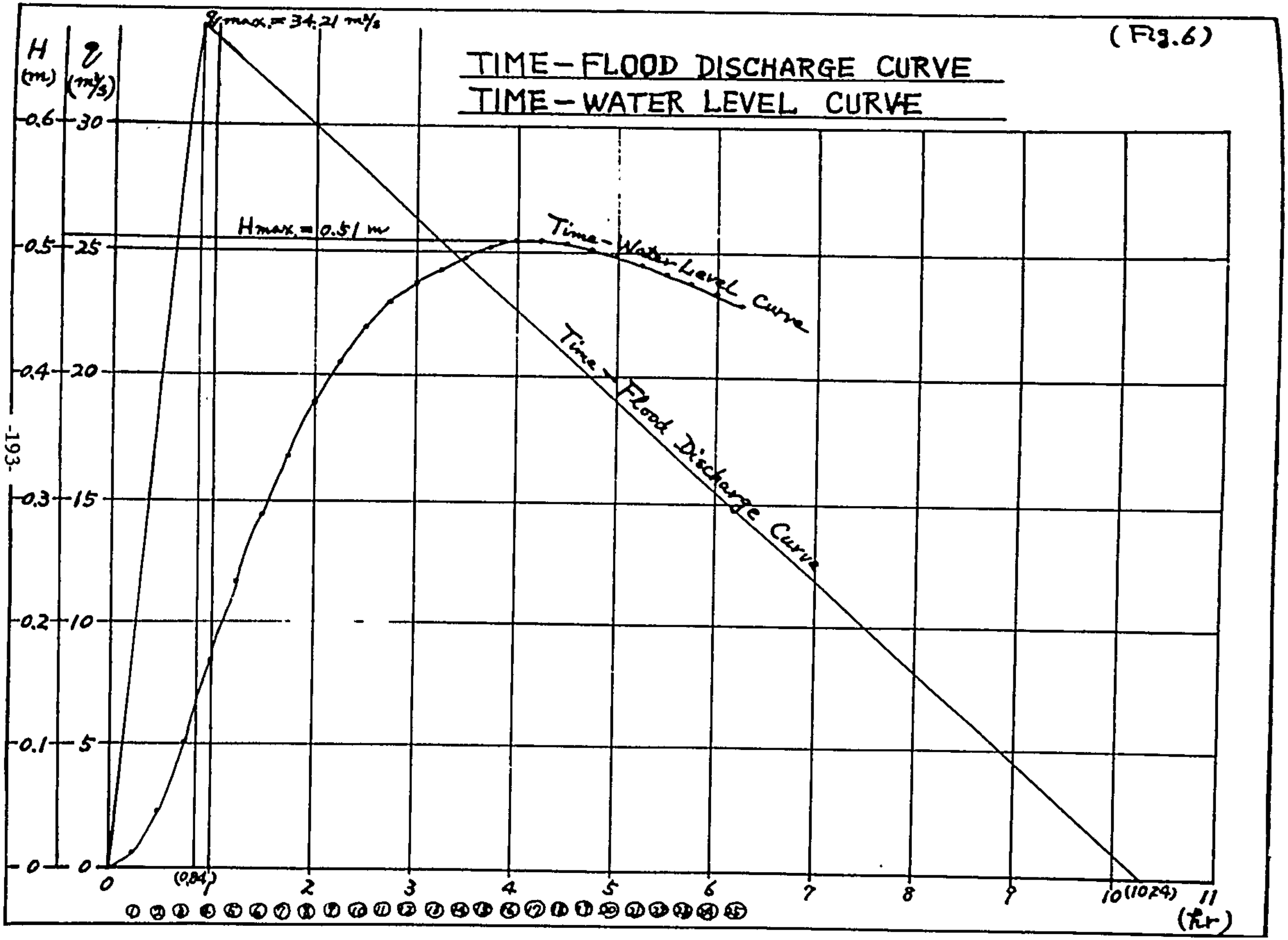
Order (i)	max. daily rainfall a year	$\frac{i}{N} \times 100$ %
1	6.04"	11.1
2	5.00	22.2
3	4.89	33.3
4	4.60	44.4
5	3.92	55.6
6	3.45	66.7
7	3.23	77.8
8	2.96	88.9
9	2.64	100.0



(Fig. 5)

CALCULATION OF FLOOD CONTROL CAPABILITY ON DIAGRAM




(Fig.6)
TIME-FLOOD DISCHARGE CURVE
TIME-WATER LEVEL CURVE
$q_{max.} = 34.21\ m^3/s$
$H_{max.} = 0.51\ m$
Time-Water Level Curve
Time-Flood Discharge Curve
H (m)
q (m³/s)
0.6
0.5
0.4
0.3
0.2
0.1
0
30
25
20
15
10
5
0
0
(0.84)
1
2
3
4
5
6
7
8
9
10
(10.24)
11
(hr)

(4) FISHING POND September, 1967

MOVEMENT

12, Apr.	Visited Fishing Pond to study the outline of irrigation scheme with Mr. S. M. Maharaj.
2, Sept.	Read the report by F.A.O. to grasp the scheme and examined the geological condition on the map concerned.
6, Sept.	Went there again and saw Windbelt area in wet season with Mr. E. Wyke and Mr. J. V. Smith. On the way, locked at the proposed headworks site on the Oropouche River near Felmina Tobal Road and looked around the watershed of the Caigual River.
12, Sept.	On the way to Plum Mitan, dropped in Le Mont Section with Mr. Reddock (Agricultural Officer, Ministry of Agriculture) and Mr. J. V. Smith and saw the proposed dam site on the Caigual River.

DESCRIPTION OF THE AREA STUDIED

This area is a small swamp which lies south of the lower Oropouche River, inside the hill along Matura Bay. The Caigual River runs there are divides the area into two parts, the Windbelt Area and Le Mont Section. The watershed of the Caigual River consists of low hilly land below 150 ft, and has a total area of about 9 square miles. The river pours into the Oropouche River near her mouth at Matura Bay.

The Oropouche River which has large watershed more than 100 square miles and has a meandering course north of Fishing Pond with abundant water even during the dry season. A geological study shows, sandstone and sandy shale in the hilly area and silty or clayey alluvium in lower lands.

The mean yearly rainfall precipitation in Fishing Pond for the last ten years is about 80 inches as shown below. The rice expansion scheme in this area was commenced in 1957 and ended in 1966 as the first 10 year period. In the Windbelt Area 130 acres had their paddy fields arranged and rice transplanted this year. This is out of a total of 140 acres. On the other hand, the Le Mont Section of about 500 acres has not been finished yet, and only 50 acres were cultivated. This area was ploughed by government initially but maintained privately by about 50 farmers only seepage water and rainfall, however, can be used for the fields at present. In dry season, the farmers keep water melon mainly and tomatoes partially due to shortage of irrigation water.

Main and secondary feeders, drains, and control gates in the Windbelt Area are all kept under much better condition than Plum Mitan area.

(Chart 1) Monthly Rainfall Precipitation at Fishing Pond

(inches)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1957	5.75	1.97	0.77	5.94	5.63	5.76	9.84	5.82	6.11	8.53	12.92	12.55	81.59
1958	7.43	1.58	0.40	6.93	10.03	12.33	14.87	6.30	4.61	14.09	9.24	2.68	90.49
1959	1.87	1.69	1.32	1.28	3.64	7.54	4.71	6.92	5.32	17.43	15.12	5.81	72.65
1960	2.91	1.42	0.77	9.04	3.03	9.29	6.84	10.23	3.25	11.04	8.46	6.89	73.17
1961	3.82	1.28	2.13	1.35	1.83	9.60	7.26	9.48	17.70	11.90	12.84	3.82	83.01
1962	2.39	3.90	0.39	1.51	3.75	6.45	9.05	17.16	6.44	3.73	5.60	3.57	63.94
1963	5.43	2.68	2.45	2.14	7.42	8.87	7.55	3.69	15.37	2.94	13.88	5.60	78.02
1964	3.52	1.15	1.72	8.66	4.08	11.37	16.32	5.71	9.90	15.31	10.35	7.70	95.79
1965	9.06	2.72	2.38	1.43	5.10	8.88	10.30	9.26	7.33	10.31	18.55	16.54	101.86
1966	2.20	1.98	0.33	2.59	4.78	15.19	7.83	8.62	9.56	10.78	9.63	12.23	85.72
Total	44.38	20.37	12.66	40.87	49.29	95.28	94.57	83.19	85.59	106.06	116.59	77.39	826.24
Mean	4.44	2.04	1.26	4.09	4.93	9.53	9.45	8.32	8.56	10.60	11.66	7.74	82.62

RECOMMENDATION

1. Urgent completion of the facility for irrigation and drainage in the Le Mont Section as in F.A.O. report.

According to the report, the water that is taken from the Caigual River and Chateau Ravine is led to the Windbelt Area and from the end of the area is to be sent to the Le Mont Section by the siphon that crosses under the Caigual River and main feeder. The facilities in the Windbelt has been completed already, but the water has never been sent to the Le Mont Section because there are little cultivated lands yet due to intrusion of sea water with no strong and improved perimeter embankment along the Oropouche River. Strengthening and improvement of the perimeter embankment are of primary importance.

2. Water supply in dry season

As mentioned above, water melon is now kept because it is not so susceptible to damage by spells of drought like the other crops. However, the other fruit vegetables will be kept if enough irrigation water is supplied.

Three means can be considered.

(a) Use of artesian ground water by bore hole and pump in some cases.
(b) Use of the water from the Oropouche River by headworks and canals.
(c) Use of the water from the Caigual River by dam.

The total water requirement in dry season was calculated as follows.

Irrigated area	The Windbelt Area	140 acres
	The Le Mont Section	500
	Total	640 acres

Net water requirement amounts to 21.6 inches a year as chart 3.

Therefore, total net water requirement will be

$$21.6 \times \frac{1}{12} \times 640 \times 43{,}560 = 50{,}181{,}000 \text{ ft}^3 = 312{,}569{,}000 \text{ gallons} = 1{,}421{,}000 \text{ m}^3$$

The maximum net requirement per second will be calculated as follows:
Maximum net water requirement per month is 4.9 inches in March as chart 3.

$$\text{Therefore } 4.9 \times \frac{1}{12} \times 640 \times 43{,}560 \times \frac{1}{30} \times \frac{1}{86{,}400} = 4.39 \text{ cusecs} = 0.125\text{m}^3/\text{sec.}$$

Comparison and examination of three methods as mentioned above.

(a) Use of artesian ground water by bore hole and pump in some cases

If underground water can be used satisfactorily by boring in Chateau Ravine or just above the upper reach of the intake on the Caigual River, the expense for water supply to this area in dry season will be satisfied and if the bore hole become an artesian flowing well, no pumps will be necessary. Conveyance losses will be very little, because of short distance between wells and the intake.

(Chart 2) Rainfall Precipitation in 20% wet at Fishing Pond

(inches)

Month	NARIVA		Ratio	FISHING POND		Remarks
	Mean Monthly Rainfall	20% wet		Mean Monthly Rainfall	20% wet	
	(A)	(B)	(C) = (B) / (A) %	(D)	(E) = (D) x (C)	
Jan.	6.86	2.73	39.8	4.44	1.77	The rainfall in 20% derived from F.A.O. report
Feb.	4.26	1.13	26.5	2.04	0.54	
Mar.	3.54	1.39	39.3	1.26	0.50	
Apr.	4.17	1.37	32.9	4.09	1.35	
May	9.01	3.53	39.2	4.93	1.93	
June	13.50	10.24	75.9	9.53	7.23	
July	11.81	8.73	73.9	9.45	6.98	
Aug.	11.96	10.50	87.8	8.32	7.30	
Sept.	8.80	7.89	89.7	8.56	7.68	
Oct.	9.12	6.87	75.3	10.60	7.98	
Nov.	11.05	7.04	63.7	11.66	7.43	
Dec.	12.56	4.14	33.0	7.74	2.55	
Total	106.64	65.56		82.62	53.24	

(Chart 3) Net Water Requirement for Vegetables

(inches)

Month	Rainfall in 20% wet (above table)	Consumptive water quant. (Vegetable)	Net water requirement	Remarks
Jan.	1.77	5.0	3.2	Consumptive water quantity for vegetable is shown in detail on the report Tucker Valley (II)
Feb.	0.54	4.8	4.3	
Mar.	0.50	5.4	4.9	
Apr.	1.35	5.3	3.9	
May	1.93	5.4	3.5	
June	7.23	4.3	-	
July	6.98	4.7	-	
Aug.	7.30	4.7	-	
Sept.	7.68	4.3	-	
Oct.	7.98	4.6	-	
Nov.	7.43	4.3	-	
Dec.	2.55	4.4	1.8	
Total	53.24	57.2	21.6	

The problem, however, is that there are no means to compute the water quantity of wells except by actual test borings. It is possible that the wells might not be able to deliver the water requirement duirng the dry season. Computation of storage capacity under ground will be the most important element. As things stand now it is not sure that the gross water requirement (4.39 cusecs x 1.5 = 6.6 cusecs) can be guaranteed. Nevertheless, from future survey, the following matters must be decided.

Namely:-

- How many bore holes are necessary.
- Which points they must be bored at.
- How deeply they must be bored.
- How wide the intervals of boring must be.

Even if it is unfortunately proved that the underground water cannot supply the water requirement it will be surely valuable as supplementary water.

(b) Use of the Water from the Oropouche River by headworks and canals

The proposed headworks site is situated on lower reach of the Oropouche River near the point at which the Gunapo River and the Sangre Grande River meet each other. The size of the headworks could be found by surveying this site in detail.

The water level supposedly will have to be raised up to elevation of 25 ft as mentioned below. It means that the headworks will have to have a height of 10 to 15 ft, and a length of 70 to 100 ft.

The river is very narrow at the proposed site, and the flood discharge is comparatively great. Therefore, the use of movable type weir is advisable so that the embankment along the Oropouche River may be protected. The total length of the driving channel amounts to about 5 miles including a few short water tunnels and aqueducts across the tributaries. On the assumption that the slope of the channel is 1/2,000, the required head will be 5 miles x 5,280 x 1/2,000 = 13.2 ft. As the maximum elevation of intake to Fishing Pond Area is nearly 8 ft, the elevation of water surface at the weir will be needed higher than 13.2 + 8.0 = 21.2 ft, that is about 25 ft.

Riverflow measurement at the upper reach (the catchment area is 21.8 sq. miles) of the Oropouche River is being carried out by the Consultant, M.M. Dillon, since 1965. Checking the river flow at the proposed headworks site by converting the data with the ratio of the catchment area, more than 100 cusecs can be available. Now there is a dam site considered as the first priority development at the upper reach of the Oropouche River. Should the dam be constructed, much more irrigation water will be available during the dry season.

(Chart 4) Droughty Water Quantity on the Oropouche River

Month/year	1965	1966	Remarks
	cusecs	cusecs	These figures are minimum daily mean discharge at the station.
Jan.	-	70.0	W.A.S.A. which has the catchment area of 21.8 sq. miles.
Feb.	-	46.0	22.9 cusecs/21.8 sq. miles = 1.05 cusc./sq./mile = 1.64 cusec/1,000 acres
Mar.	-	29.0	
Apr.	-	22.9	
May	-	26.6	
June	-	83.8	
July	41.8	84.4	
Aug.	52.0	61.3	
Sept.	48.0	76.5	
Oct.	37.0	60.8	
Nov.	61.5	84.0	
Dec.	99.9	99.9	

The minimum daily mean discharge per 100 acres in April last year was 1.64 cusec. It is about twice as much as the discharge of 0.72 cusec on the San Juan River which was surveyed at the beginning of this May. It means that the Oropouche River has more abundant droughty water than the San Juan River. Consequently at the proposed headworks site in the lower reach of the Oropouche River which has a catchment area of more than 100 sq. miles, more than 100 cusecs will be discharged even in dry season.

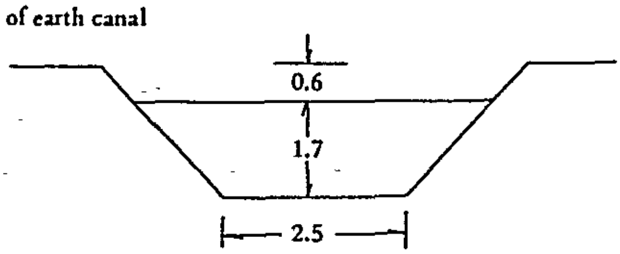
Incidentally, the measurement of salinity at the site will be necessary,although it is supposed that the brackish water will not reach there as the elevation of the river bed is about 10 to 15 ft and the site is about 8.3 miles away from the mouth of the river.

As mentioned above, the droughty water quantity on the Oropouche River will be more than 100 cusecs. On the other hand the water requirement for Fishing Pond in dry season is calculated as only 6.6 cusecs. It is therefore, very uneconomical to build the weir and the canal for only this area. Similarly a water-tunnel cannot be made in smaller section than the one which is necessary for workability even if the water led through it is little. It is desirable therefore that the water requirement for the area along the proposed canal to Fishing Pond will be taken together.

Design of canal

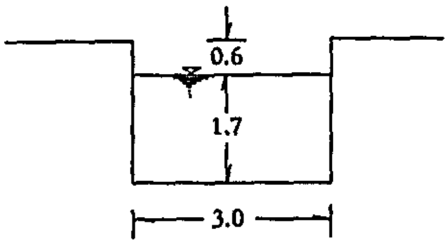
If the water quantity of 6.6 cusecs that is 1.5 time of the net water requirement for only Fishing Pond is led by the canal, the section will be calculated as follows.

In the case of earth canal



I = 1/2,000
n = 0.030
b = 2.5 ft (0.75m)
H = 1.7 ft (0.50m)
V = 1.0 ft/sec. (0.30m/s)
Q = 6.6 cusecs (0.187m^3/s)

In case of concrete-lining canal



I'= 1/2,000
n = 0.015
b = 3.0 ft (0.75m)
H = 1.7 ft (0.50m)
V = 1.7 ft/sec (0.52m/s)
Q = 6.9 cusecs (0.195m^3/s)

(c) Use of the water from the Caigual River by dam

There is an appropriate dam site justbefore the place where the Caigual River goes across Caigual Road. Just at the upper reach of the proposed dam site, two main tributaries meet. As mentioned above, however, this area consists of low hilly land and cannot accomodate the building of a high dam. As a result the reservoir will become very shallow, on the other hand the storate area will be very wide.

The available water quantity collected from the catchment area is calculated as follows, and suffice the water requirement.

Catchment area at proposed dam site	5.33 sq. miles (= 1,380.5 ha)
Yearly rainfall precipitation in 0% wet in Fishing Pond	53.24 inches (cf. chart 2)
Run-off coefficient	46.3% (cf. Report Plum Mitan I)

Therefore, the available water quantity collected from the catchment area will be

1,380.5 ha x 10^4 x 53.24 inches x 0.0254 m/in x 0.463 = 8,643,000m^3 (=305,242,000ft^3)
(=1,901,352,000 gals)
On the other hand, the net water requirement is 1,421,000,000m^3) (= 50,181,000 ft^3)
(cf: this report page 3) (=312,569,000 gals)

The storate capacity is calculated as the following table. But it is estimated by the map drawn to a scale of one by fifty thousand.

(Chart 5)

Elevation	Storage area		Mean storage area	Height between each section		Storage capacity	Total storage capacity
ft	sq miles	m^2	m^2	ft	m	m^3	m^3
10 >	0	0	0	0	0	0	0
25 >	0.23	596,000	298,000	15	4.5	1,341,000	1,341,000
30 >		1,000,000	798,000	5	1.5	1,197,000	2,538,000
50 >	2.20	5,696,000	3,147,000	20	6.0	18,882,000	21,420,000

As seen in the above table, the water is stored very shallow. It can be therefore supposed that there will be large loss of evaporation. The calculation of evaporation-loss from the reservoir surface is carried out as follows.

On the assumption that the full water level is 30 feet, the full water area will be 1,000,000m².

According to the literature on this matter, the evaporation from wide water surface is on an average 0.7 time as much as that from evaporation pan.

Evaporation record with evaporation pan. at Navet Dam (chart 6)

Month/Year	1966	1967
	inches	inches
Jan.	4,915	4,655
Feb.	5,270	5,430
Mar.	7,865	6,695
Apr.	7,145	6,005
May	6,050	7,240
June	3,140	4,770
July	4,210	4,455
Aug.	4,705	5,165
Sept.	4,365	
Oct.	5,420	
Nov.	3,590	
Dec.	3,610	
Total	60,285	

If the evaporation loss in the proposed reservoir on the Caigual River is calculated by using the evaporation record with evaporation pan at Navet Dam in 1966, it will be as follows.

(Chart 7)

Month	Evaporation Record	Total	Mean water surface area	Co-efficient	Calculated Evaporation loss
	inches	inches			
Dec.	3,610				
Jan.	4,915				
Feb.	5,270	34,855	$1,000,000m^2 \times ½ = 500,000$	x 0.7	
Mar.	7,865	(0.855m)			$309,750m^3$
Apr.	7,145				
May	6,050				
June	3,140	7,350	$1,000,000m^2 \times 2/3 = 670,000m^2$	x 0.7	$87,700m^3$
July	4,210	(0.187m)			
Aug.	4,705				
Sept.	4,365	18,080	$1,000,000m^2 \times 1 = 1,000,000m^2$	x 0.7	$321,300m^3$
Oct.	5,420	(0.459m)			
Nov.	3,590				
					$718,750m^3$

As a result of the calculation the yearly evaporation loss from water surface in the reservoir comes up to about 30% of storage capacity. Only the evaporation loss in dry season, however, can be considered because the one in wet season can be supplied by rainfall.

It amounts to about 12% of the storage capacity.

The net available storage capacity will be

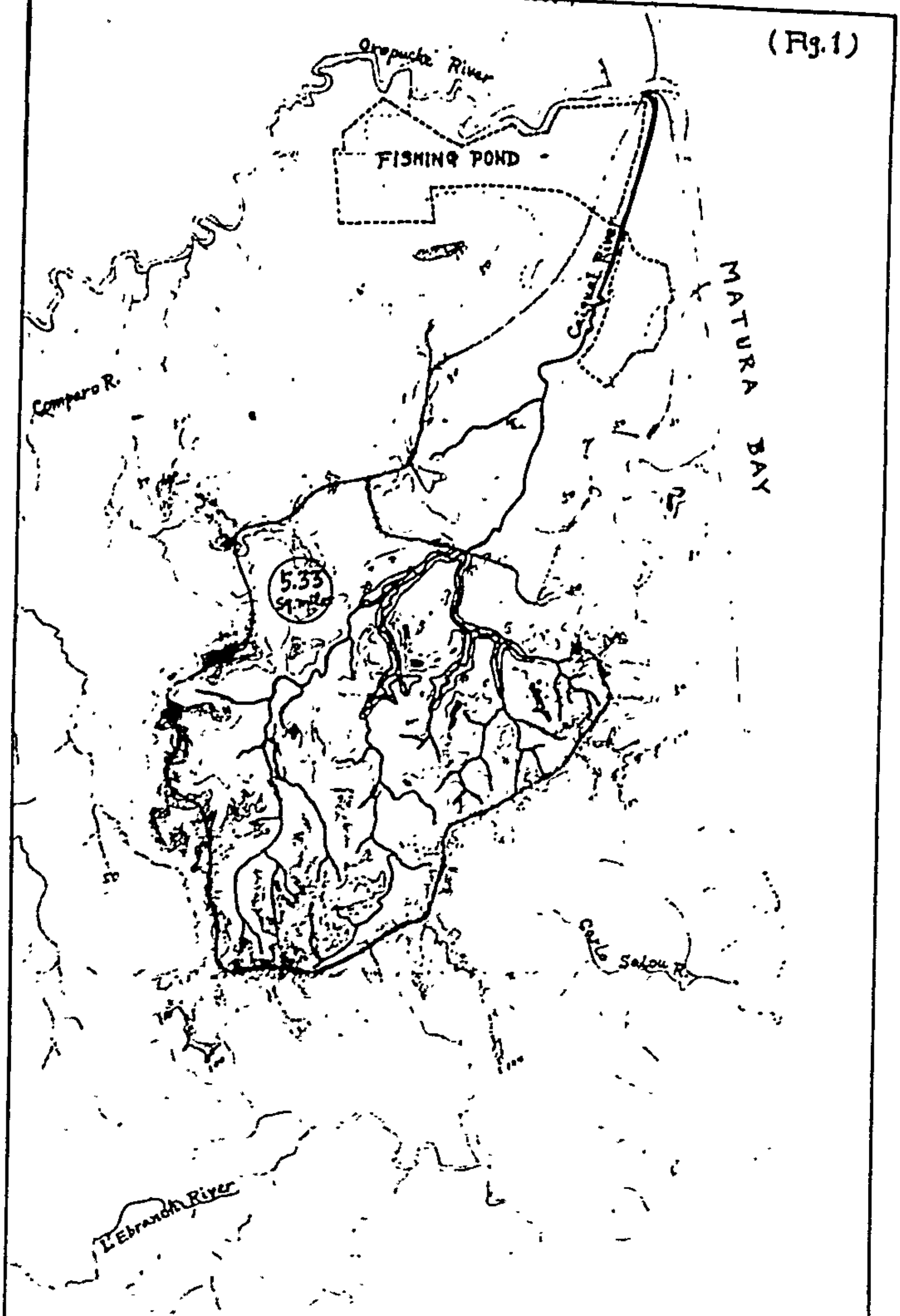
$$2,538,000 - 309,750 = 2,228,000 m^3$$

Granted that there are also seepage loss at the reservoir and water-conveyance losses by canal, the net water requirement of $1,421,000 m^3$ can be supplied. Then the height of the dam will be about 25 ft and the length about 750 ft.

These three proposals must be examined carefully, and offering the greatest technical and economical advantages.

In this December or next January, the detailed map of one to ten thousand in this area will be issued. The continuation will be therefore resumed after completion of the map.

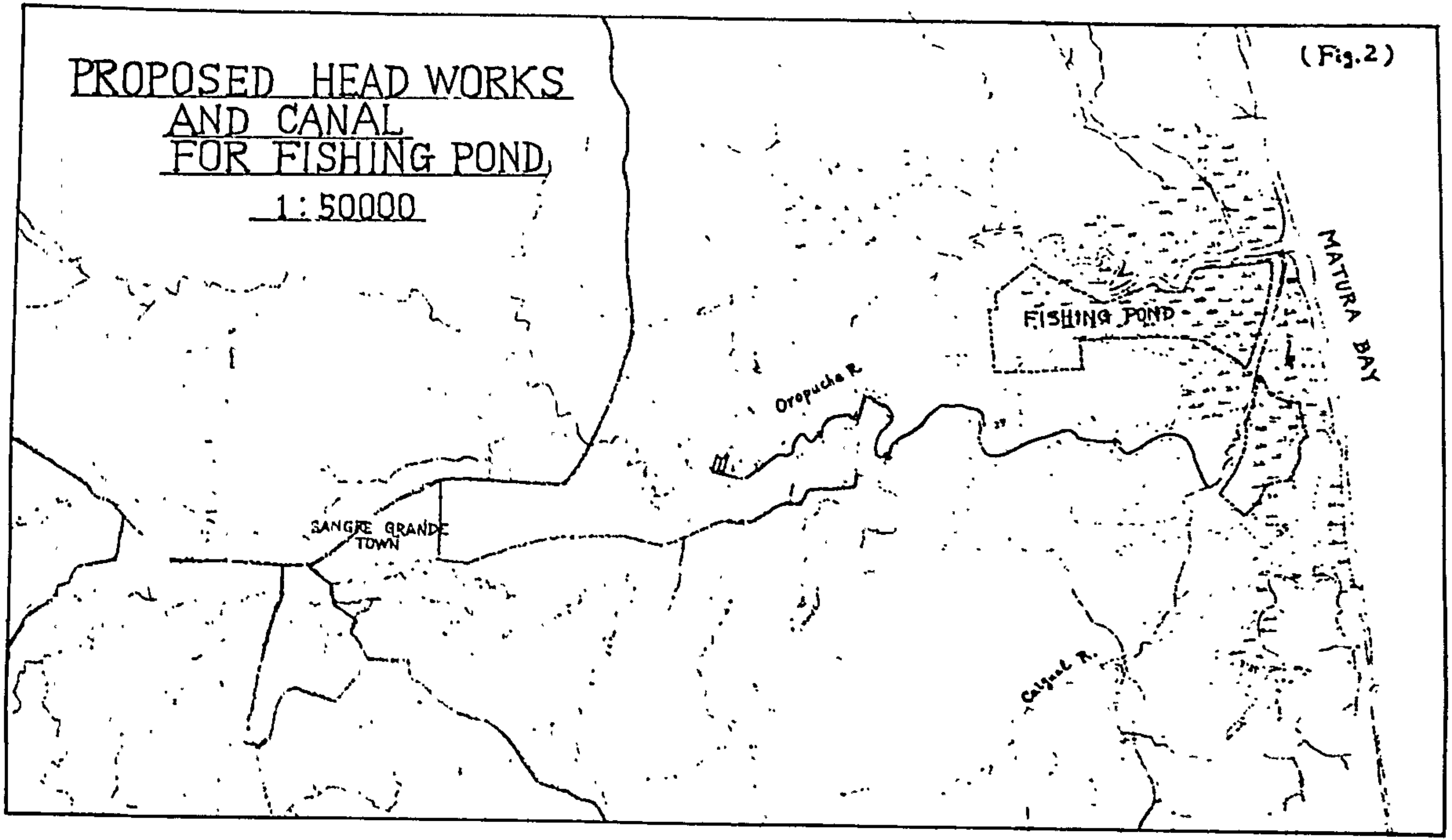
Junichi Kitamura



FISHING POND

CATCHMENT AREA OF CAIGUAL RIVER

1 : 50000


PROPOSED HEAD WORKS
AND CANAL
FOR FISHING POND
1 : 50000
(Fig.2)
MATURA BAY
FISHING POND
Oropuche R.
Cajual R.
SANGRE GRANDE
TOWN