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ANNUAL REPORT FOR 1959

VOLUME-II

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SURGE TANK

MODEL STUDIES TO FIND OUT THE MAXIMUM AND MINIMUM SURGES

1. Synopsis

This report deals with the model studies conducted at this Station to find out the maximum and minimum surges in the Surge Shaft No. 2 of the Kundah Hydro Electric Scheme. The maximum surge level has been found out for the full load throw-off and the minimum surge level for an increase of load i. e. half to full load.

The maximum surge level was found out for the different vertical scales. An arithmetical method of computation of maximum surge level involving successive trials is also given in the appendix. The results are furnished in this report in the from of a statement.

2. Introduction

The Kundah Power House No. 2 forms part of the Kundah Hydro-Electric Scheme which is under execution, in Nilgiris (Figure-1). The Power-house is situated lower down the Kundahpalam Dam which is situated about 20 miles south-west of Ooty. The Powerhouse utilises a grosshead of 2470 feet with an installed capacity of 140 M. W. and ultimate capacity of 175 M. W. Four Penstock pipe lines feed four units of 50,000 H. P. vertical, multiple, jet impulse wheels, 423 RPM of 35,000 K. W. each. Provision has been made in the Powerhouse for installing a 5th unit at a future date.

A fully lined tunnel of 11553 feet length from the Kundahpalam Dam to the surge shaft is being constructed to convey water to the head of the penstock system leading to the Powerhouse No. 2 of the Project (Vide-Figure-2). In order to reduce the water-hammer pressures in the tunnel a simple surge tank of total depth of 241.43 feet with 50 feet diameter for 110 feet from the top and with 25 feet diameter for remaining 131.43 feet has been proposed.

The crown of tunnel at the end from the surge shaft the diameter of the surge shaft and its top level have been fixed based on the calculation of the maximum and minimnm surge levels. The maximum surge level has been computed taking the instantaneous full load throw-off and the minimum surge level for the instantaneous increase of the load from half to full load. Model studies were taken up at this Station to find out the maximum and minimum surge for the above operating conditions.

3. Data

Full Reservoir level	...	El. 5330.00
Maximum drawdown level	...	El. 5280.00
Length of the tunnel from forebay to surge shaft	...	11553 feet
Gross sectional area of Water way	...	101.50 sft.
Maximum velocity of flow	...	9.86 ft/sec.
Loss of head in the tunnel due to friction, exit, entry and eddy losses for the peak flow	...	31 feet
Maximum discharge in the tunnel	...	1000 cusecs.

4. Theory of Surge Tank

If the conduit leading to the turbines is long and the head is not great, a surge tank is provided in order to prevent the excessive surges of pressure caused when the turbine gates are closed. The surge tank also serves as a source of supply when the load is suddenly increased.

If the pipe leading to turbines is closed, the water level in the surge tank will begin to rise and the water level will be oscillating until the level in the surge tank becomes steady when the flow in the conduit becomes zero.

The surge tank to be provided in the Kundha Hydro Electric Scheme is of a simple type.

Let
- L - the length of conduit between the forebay and the surge tank.
- R - ratio of the sectional area of the surge tank to the conduit.
- C - friction co-efficient.
- V_1 - velocity of water in the tunnel under steady flow before a change of load
- V_2 - velocity of water in the tunnel under steady flow after a change of load
- V - velocity in pipe line under steady flow after a change of load
- V^1 - velocity in the pipe line between the surge tank and the turbines during the period of change.
- y - height of water in the surge shaft at any instant above the level corresponding to steady flow with Velocity V.

At any instant, the head available for producing retardation in the pipe is $y - c(V_1^n - V^n)$ and the equation of flow in the pipe line is $y - c(V_1^n - V^n) = \frac{L}{g}\frac{dV}{dt}$ ... (1) The volume of water leading the turbines equals the volume leaving the forebay less that entering the surge tank.

Hence $R\frac{dy}{dt} = V - V^1$ ... (2)

(The area of the condu it and the area of the penstocks are assumed the same.)

The solution of the simultaneous equation (1) and (2) is not possible even for the condition of complete Shut-down. So it makes it desirbale to go in for model investigation for finding out the mximum and minimum surges. However, a few methods are available by which the surge and dimensions of the surge tank can be found out.

5. TECHNIQUE OF MODEL STUDIES AND SCALES

In order that the model is to be in close conformity with the prototype, the dynamic similarity in the whole set up is to be maintained i. e.

(i) that corresponding displacement in the one shall be proportional to those in the other;

(ii) that the velocities and accelerations shall be such that all forces acting on corresponding parts of the moving system are in the same ratio.

The ratio of vertical displacements (y) in the surge tank in the prototype and in its model is fixed by the ratio of the respective friction heads when there is normal flow in the conduit.

If the suffix 'm' refers to the model

$$\frac{y}{y_n} = CV^n/C_m V_m^n \quad ... (3)$$

friction is assumed to be proportional to V^n

For the equation (1) to be dimensionally homogeneous, y must have the dimensions of CV^n and of LV/Gt.

Therefore Y/Ym = LVtm / Im Vmt ... (4)

Combining the equation (3) and (4)

$$CV^n/CmVm^n = LVtm/ImV_mt$$

$$t/tm = \frac{LCm\ Vm^{n-1}}{Im\ CV^{n-1}} \quad ... (5)$$

Equation (5) fixes the time scale. For equation (2) to be dimensionally homogeneous R must have the dimension of V_t/y.

$$R/Rm = \frac{Vt}{y} \cdot \frac{Ym}{Vm\ tm} \quad ... (6)$$

Combinig equations (4), (5) and (6)

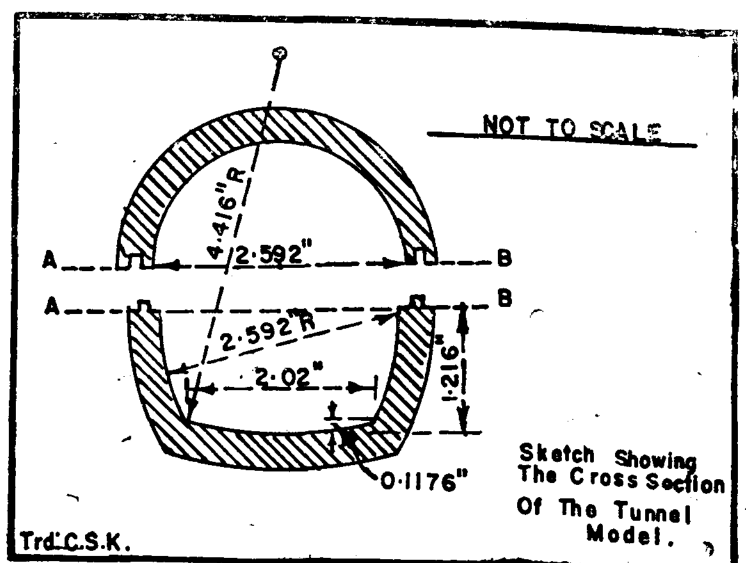
$$R/Rm = \frac{L}{Lm}\frac{(Cm)^2}{(C)}\frac{(Vm)^{2n-2}}{(V)} \quad ... (7)$$

which fixes the diameter of the surge tank in the model.

6. MODEL SET UP

A model was constructed to a scale of 1 in 500 horizontal, 1 in 200 vertical and 1 in 51 diameter for the conduit. The diameter scale of the model was fixed as 1/51 based on the diameter of penstocks. The penstocks were of diameter 4′ 9″, 4′ 3″ and 3′ 9″ and as the diameter was 4′ 3″ or 51″ for the maximum length, the diameter scale of 1/51 was used so as to have 1″ diameter pipe conveniently for this length of the penstocks. The penstocks of other diameters were made in M. S. Sheet and the tunnel was made in teak-wood.

The forebay was represented by a steel tank to which is attached a valve by means of which the water level was maintained constant. A pump was provided to pump water into the tank. The parts representing the tunnel were made of wood in two sections connected together and was made leak-proof (vide sketch below). The conduit representing the tunnel was made



Sketch Showing The Cross Section Of The Tunnel Model.

rough in order to offer sufficient friction to the flow. The tunnel was supported on mosonry pillars.

The surge tank was incorporated in the model at the end of the tunnel. Downstream of the surge tank were provided the five penstocks conveying water to the turbines. Lower down the penstocks two valves (1) and (2) marked in Figure-3) were introduced, one for adjusting the discharge and the other for instantaneous closing or opening. The quantity of water conveyed through the penstocks, was measured by a V-notch placed at the down-stream end. Necessary honeycombs were provided to damp the velocity of water passing through the V-notch.

Copper tubes were introduced into the conduit at the beginning and at the end to tap the pressures which will help to find out the losses. These copper tubes were connected by means of rubber tubes to glass tubes fixed in a manometer board, in which the pressures could be directly read. The whole model set up is shown in Figure-3.

7. EXPERIMENTS AND RESULTS.

Intially the model was run to find out the values of C and n, the two constants in the equation for loss of head $(h_f) = C\,V^n$ where V is the valocity.

A constant water level corresponding to +5,330 in the forebay of the prototype was maintained in the tank. Different discharges were sent through the conduit by adjusting the valve at the downstream end. The measurement of discharge was done by measuring the head over the V-notch. Care was taken to see that the level in the tank was maintained throughout at + 5330. The glass tubes connected to the piezometric tappings were syphoned thoroughly to ensure that there were no air bubbles.

After a steady condition of flow in the conduit and the penstocks was obtained, the piezometric readings were noted.

Taking a datum and applying the Bernoulli's equation the frictional loss in the conduit was calculated. For steady condition of flow, the water level in the surge tank was also noted. From the piezometric readings water level in surge tank and the water level in the forebay, all the losses such as entry loss, exit loss, frictional loss and total loss were calculated.

Curves in Figures (4) show the variation of loss of head with respect to Velocity^n. From the curve (total loss of head Vs velocity) the values of c and n were 0·079 and 1·799 respectively.

To find out the size of the surge tank in the model for a vertical scale of 1 in 200.

As given earlier $Y/Ym = CV^n/CmVm^n$

In the model $Y/Ym = 200$ ... (1).

Therefore $Y/Ym = \dfrac{CV^n}{CmVm^n} = 200$

Loss of head in the prototype = 31 feet (as furnished by the Electricity Board).

$$\frac{31}{Cm\,Vm^n} = 200$$

Therefore $Cm\,Vm^n = \dfrac{31}{200} = 0.0155$

But $Cm = 0{\cdot}079$ and $n = 1.799$

$0{\cdot}079\ Vm^{1.799} = 0.0155$

$Vm = 1.454$ ft/sec.

The model velocity is 1.454 ft/sec. corresponding to a velocity of 9.86 ft/sec. for a peak discharge of 1,000 cusecs during the full load condition.

We have $\frac{Y}{Ym} = \frac{LVtm}{lmVmt}$

i.e. $\frac{31}{0.0155} = \frac{L}{L/500} \times \frac{9.86}{1.454} \times \frac{tm}{t}$

$200 = \frac{500 \times 9.86}{1.454} \times \frac{tm}{t}$

$t/tm = \frac{500 \times 9.86}{1.454 \times 200} = 16.95$

which is the time scale in the model.

Also we have $R/Rm = \frac{VtYm}{VmtmY}$ where

$R = \frac{\text{Area of the surge tank}}{\text{Area of the section of tunnel}}$

$R/Rm = \frac{9.86}{1.454} \times 16.95 \times \frac{1}{200} = 0.5748$

Top diameter

$R = \pi/4 \times \frac{50 \times 50}{101.5} = \frac{625\,\pi}{101.50} = 19.35$

$R_m = \frac{R}{0.5748} = \frac{19.35}{0.5748} = 33.67$

Area of the model surge tank for the proto type diameter of 50′ = R_m x area of the section of the tunnel

$= \frac{33.67 \times 101.5}{51^2} =$ sq. ft.

Diameter $= \sqrt{\frac{33.67 \times 101.5 \times 4}{51 \times 51 \times \pi}} = 1.292$ feet.

Bottom diameter

$R = \pi/4 \times \frac{25^2}{101 \cdot 5} = 4.8375$

$R = \frac{R}{0.5748} = \frac{4.8375}{0.5748} = 8.4175$

Area of the surge tank at the bottom = R_m x area of section of tunnel in the model

$= 8.4175 \times \frac{101.5}{51 \times 51}$ sq. feet

Diameter of the surge tank at the bottom

$= \frac{8.4176 \times 101.5}{51 \times 51} \times 4/_{\pi} = \sqrt{0.418}$

$= 0.646$ feet.

The surge tank of the above mentioned dimensions for the top and bottom portion was incorporated in the model. Discharge in the model for a model velocity of 1.454 ft/sec. (corresponding to 9.86 ft/sec. under peak discharge of 1000 cfs. in the prototype).

= Area of the tunnel x velocity.

$= \frac{101.5}{51 \times 51} \times 1.454 = 0.05674$ cfs.

This discharge of 0.05674 cusecs was allowed to flow through the tunnel keeping the water level at the forebay corresponding to + 5330.00. The discharge was adjusted by the valve at the downstream end. After getting a steady condition in the model, the water level in the surge tank was noted down by means of the gauge. The valve at the downstream end was closed instantaneously. The surge levels along with the time between the maximum and minimum surge levels and between the minimum and maximum surge levels were noted down (Vide Figure-5).

A series of such observations were also made and the average was taken as the maximum surge level. The results are given in the statement attached.

The minimum surge level could not be studied in this model because the vertical scale was much smaller when compared to the diameter scale and hence under the minimum surge conditions the tunnel was not flowing full. The only consideration on which the vertical scale 1/200 was fixed was the availability of sufficient drop from the surge tank to the power house which is 2253.57 in the prototype. The vertical scale of the model was modified to 1/50, the other scales being the same for studying the minimum surge levels and also for confirming the results of the maximum surge levels already obtained. The penstocks were not represented in this model as it was found that there was not any effect on the surge levels due to the removal of penstocks. The two valves were incorporated just downstream of the surge tank. The diameter of the surge tank was arrived at as already scribed.

velocity in the model corresponding to a peak harge of 1000 cfs in the prototype } = 3.143 ft/sec.

a = 31.37 and R/Rm = 1.968

ameter corresponding to 50 feet dia. the prototype = 8.3856 inches

Diameter corresponding to 25 feet dia. in the paototype = 4.1928 inches

Discharge in the model = area of tunnel x velocity

$= \frac{101.5}{51 \times 51} \times 3.143$

$= 0.1227$ cfs.

with the above discharge the maximum surge was found out in a similar manner as already obtained.

In the first series of experiments with a vertical scale of 1 in 200, the maximum surge level was observed to be = 5353.95

In the second series of experiments with a vertical scale of 1 in 50, the maximum surge level was observed to be = 5353.65

A curve has been drawn between time and surge levels and is given in Figure (5).

To fin dout the minimum surge lsvels

The minimum surge level was to be found out for the condition when the load was increased instantaneously from half to full. The discharge through tunnel was to be increased from 500 cfs to 1000 cfs while the water level in the forebay Dam was at the minimum drawdown level (i.e.)+ 5280.00.

The water level in the steel tank was maintained to correspond to the minimum draw down level of +5280. The two valves were so abjusted that when the valve No.2 was opened the discharge was increased from half to full. Initially a discharge of 0.06135 cfs (corresponding to a 1/2 load discharge of 500 cfs in the prototype) was allowed to flow in the tunnel. After attaining the steady condition, the valve No.2 was opened instantaneously increasing the load from half to full. The minimum surge level and the time elapsed to attain the minimum surge level were noted down. The statement gives the minimum surge level. Minimum surge level as found in the model is+5225.30.

8. DISCUSSION OF RESULTS AND CONCLUSIONS

Maximum Surge Level

As obtained in the first model = 5353.95
As obtained in the second model = 5353.65

In the appendix, computations of maximum surge level by arithemet mothod involving successive trials are given. The maximum surge level calculated and found to be 5354.98.

The surge level as given by the Electricity Board is 5352.00. But in the model the surge level is found to be 5353.95.

The results are tabulated in the statement.

Maximum Surge Level

The minimum surge level as obtained in the model is 5225.30. As per the calculation of minimum surge level by the Electricity Board is 5223.47 The results are given in the Statment.

The time elapsed to attain the minimum surge level from steady reading is found to be 153,08 secs.

9. REFERENCES

(1) ' Hydro Electric Hand Book " by Creager and Justin.

(2) " Scale models in hydraulic Engineering " by Allen.

(3) " Minutes of the Proceedings of the Institution of Civil Engineers"- Volume 219-Paper No. 4523.
" The Investigation of the Surge Tank problem by model experiments " by Professor Arnold Hartly Gibson D.Sc.M.Inst.C.E.

(4) Journal of the Hydraulic Division-Proceedings of the American Society of Civil Engineers. Paper No. 2225 Hy. 10, October 1959.
" An Analysis of a simple surge tank " by Frank Dram.

STATEMENT

	COMPUTATIONS			MODEL
	Electricity Board	Arithmetical method	Vertical Scale 1/200	Vertical Scale 1/50
Maximum Surge level	5352.00	5354.98	5353.85	5353.65
Minimum Surge level	5323.47	—	—	5225.30

APPENDIX

Reference.

Journal of the hydraulic Division. Proceeding of the American Society of Civil Engineers October 1959 Paper No. 2825 Ay. 10 " AN ANALYSIS OF SIMPLE SURGE TANK " by " FRANK DRUM "

Nomenclature.

t - time of period in seconds.
Q - initial discharge in the conduit or into tank (cfs)
Q_s- Conduit discharge at the beginning or end of period (cfs)
q_2- Rate of filling of surge tank begenning or end of period (cfs)
q_a- change in conduit discharge during the period (cfs)
A - Area of the surge tank (Sft)
a - area of the conduit (sft)
L - Length of conduit in feet
g - acceleration due to gravity (32.2'/sec)
h_z- Rise in surge tank during the period
h_f- hydraulic head losses in conduit at the beginning or end of period
h_a- retardation head at beginning or end of period.
c - coefficient of discharge.

When the turbine gates are closed, immediately the water level in the surge tank will begin to rise until a steady level is obtained the change in the surge tank levels (h_L) during the period of time (t) will depend on the area of the tank (A) and the average rate at which the tank is filled.

It is expressed as $h_z = \frac{q_{z1}+q_z2}{2} \times \frac{t}{A}$ ... (1)

The change in the discharge in the conduit

$= Q_a = q_{s1} - q_{s2}$ ... (2)

For complete closure of gates

$Q_{s1} = Q_{z1}$ and $Q_{s2} = Q_{z2}$ ... (3)

According to the lows of motion, the average head required to retard the flow during a period of time is

$\frac{h_{a1} + h_{a2}}{2} \times \frac{Q_a L}{t_{ga}}$ ... (4)

The total acceleration head necessary to reduce the conduit discharge during a period i.e.

$h_{a2} - h_{a1}$ ... (5)

As a result of this retardation, the conduit head losses will be decreased

$= h_{f1} - h_{f2}$ ... (6)

Under the maximum load off conditions the head necessary to decelerate the flow in the conduit, plus the decrease in hydraulic head losses, should equal the change in levels in the surge tank during each period of the first quarter cycle.

$H_z = (h_{a2} - h_{a1}) + (h_{f1} - h_{f2})$ ... (7)

and $h_{s2} - (h_{f1} - h_{f2}) = (h_{a2} - h_{a1})$ ... (8)

The table shows the solution of a simple surge tank problem by the successive arthemetic step method of integration in which successive traill were made for each period until all factors are in balance.

In the analysis instead of assuming equal intervals of time, equas intervals of change in discharge are assumed. This method obviates computing a partial period at the end of the quarter cycle. Also, if the hydraulic head losses are considered, it fixes the amount of losses for each period, thus precluding the necessity of computing these losses with each successive trial. Considering the complete closure of the gates, it has been found convenient to devide into ten equal increments and the hydraulic head losses vary as the square of the velocity.

Considering the table, it will be noted that the figure shown in column 4 of this table are the average conduit discharges and that they are also the average discharges into the tank. Therefore, for each period 't' in equation 1 is the only unknown. Equation '4 can be resolved into the from

$$h_{a1} + h_{a2} = \frac{2q_{a1}}{tga} \quad \ldots \quad (9)$$

By combining equations 1 and 7 and substituting the equivalent of ha of equation 9, the following is resolved,

$$\frac{q_{z1} + q_{z2}}{2} + \frac{t}{D} = \frac{2q_aL}{tga}\, 2L_{a1} + (L_{f1} - L_{f2}) \quad \ldots \quad (10)$$

$$\frac{2q_aL}{tga} - \frac{q_{z1} + q_{z2}}{2}\, \frac{t}{A} = 2L_{a1} - (L_{f1} - L_{f2})$$

$$h_{a1} + h_{a2} - h_2 = 2h_{f1} - (L_{f1} - h_{f2})$$

It will be noted in the table that this difference in head is known progressively for each period before any trial is made. Although not immediately apparent from the footnotes of the table, successive value of t are tried until the t is determined which results in the difference between h2 and $h_{a1} + h_{a2}$ equal to the value in column 14 of this table.

Footnotes for filling the column :

1. Sometime of (2)'s.
2. Determinied by successive trials until (2) $= \frac{(9)}{(10)} + \frac{L}{ga}$
3. Assume conduit discharge in reduced in ten equal increments.
4. $\frac{(3)_1 + (3)_2}{2}$
5. $\frac{(2) \times (4)}{A}$ from equations 1 & 3.
6. Water surface elevation at begining of period plus (5)
7. Losses for corresponding discharges in (3)
8. (7) - $(7)_2$ from equation (6)

9. $(3)_1 - (3)_2$ from equation 2.
10. $\frac{(11)_1 + (11)_2}{2}$ from equation 4.
11. (11) + (12) from equation 5.
12. (5) - (8) from equation 8.
13. $(11)_1 + (11)_2$
14. $2(11)_1$ - (8)

Sequence :

Complete (3), (4), (7), (8), and (9) for the entire quarter cycle. Then ditermine, (2), (5), (12), (11), (10), (1) and (6) successively for each period. (13) and (14) can be used for a check so as to bring the table to balance.

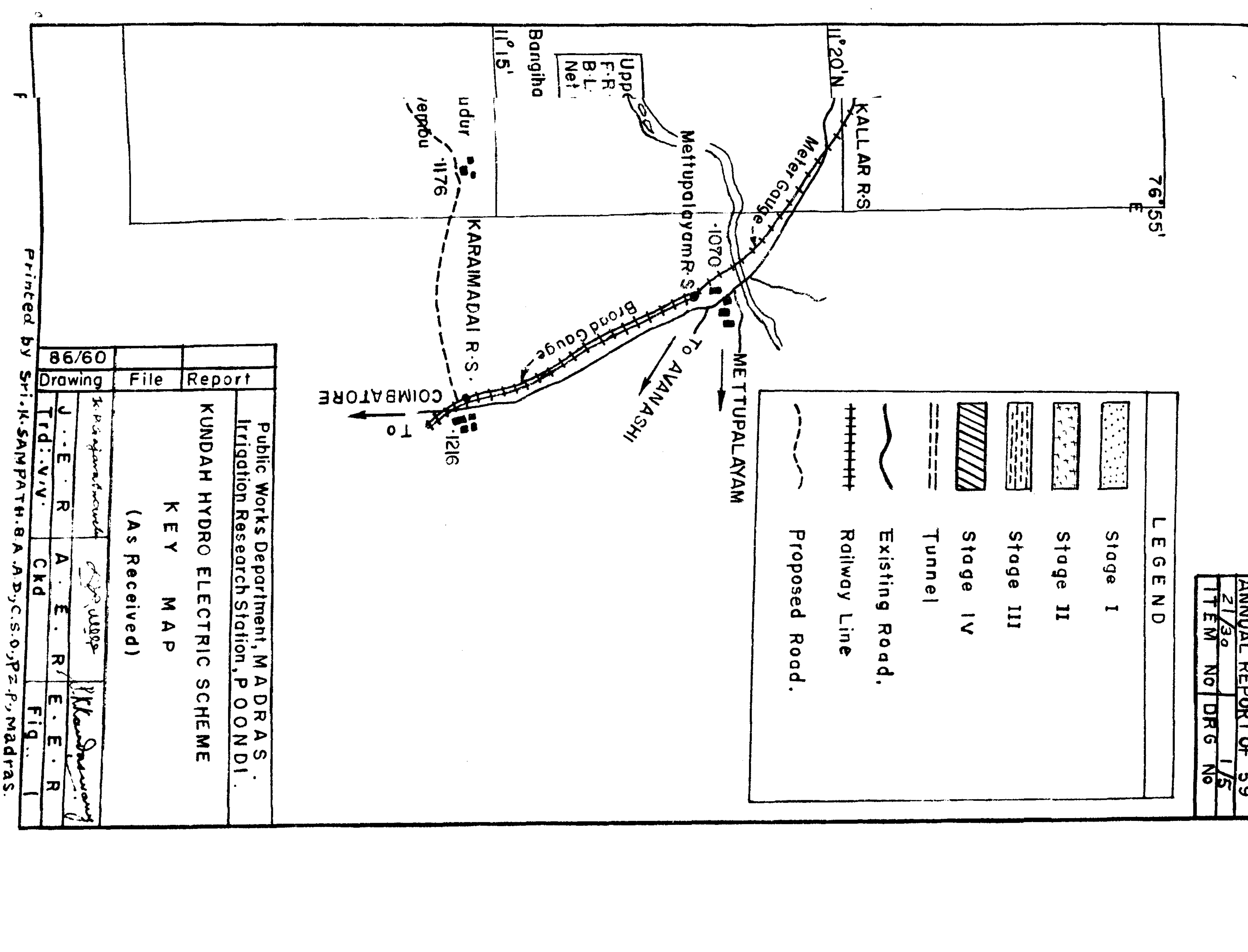
COMPUTATIONS OF SURGE BY ARITHEMETICAL METHOD

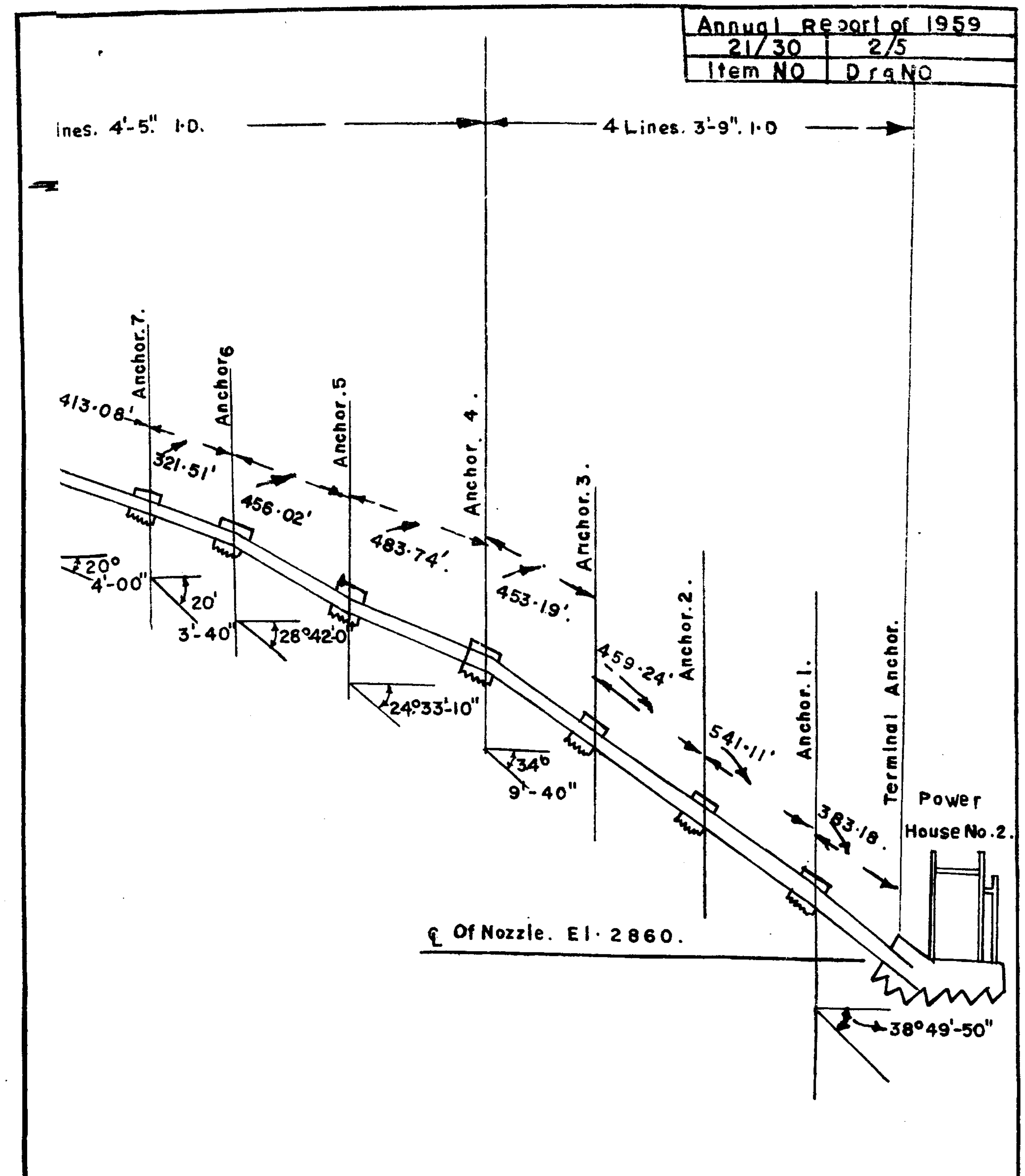
(involving successive trials)

Surge Tank Area ... 1965 Sft.
Conduit length ... 11583 feet.
Initial discharge ... 1000 cfs.

Area of conduit ... 101.5 feet.
Hydraulic Head loss. 31 feet.

Time.		Conduit discharge.			Hydraulic Head Losses.			Change in discharge.	Retardation Head.		
Accumu-lated. (1)	Incre-ment. (2)	Instant. (3)	Average. (4)	Rise. (5)	Water Elevation. (6)	Instant. (7)	Incre-ment. (8)	(9)	Average. (10)	Instant. (11)	Increment. (12)
0.000		1000			5299.0	31				00.00	
	44.8		950	21.66			5.9	100	7.88		15.76
44.8		900			5220.66	28.1				15.76	
	20.2		850	8.732			5.27	100	17.494		3.468
65.0		800			5329.398	19.83				19.228	
	17.47		750	6.667			4.63	100	20.247		2.037
82.47		700			5336.065	15.2				21.265	
	16.15		650	5.34			4.05	100	21.910		1.290
98.62		600			5341.405	11.15				22.565	
	15.36		550	4.299			3.40	100	23.00		0.899
113.98		500			5345·704	7.75				23.454	
	14.89		450	3.408			2.80	100	23.758		0.608
128·87		400			5349.112	4.95				24.062	
	14.55		350	•2.592			2.16	100	24.178		0.632
143.42		300			5351.704	2.79				24.494	
	14.36		250	1.827			1.55	100	27.6325		0.277
157.78		200			5353.531	1.24				24.771	
	14.25		150	1.087			0.93	100	24.851		0.157
172.03		100			5354.618	0.31				24.928	
	14.18		50	0.3608	5354.9788		0.31	100	24.9534		0.508
186.21		00				0.0				24.9788	





REPORT.	Public Works Department—Madras. Irrigation Research Station—Poondi.	
FILE	KUNDAH HYDRO ELECTRIC PROJECT. PLANT. 2. L·P PIPELINE AND PENSTOCKS PLAN AND PROFILE.	
12.5/60. DRAWING.	M·Poovayya	
	J·E·R	A·R·O	E·E·R.
	Trd: C·S·K.	Ckd:	Fig No. 2.

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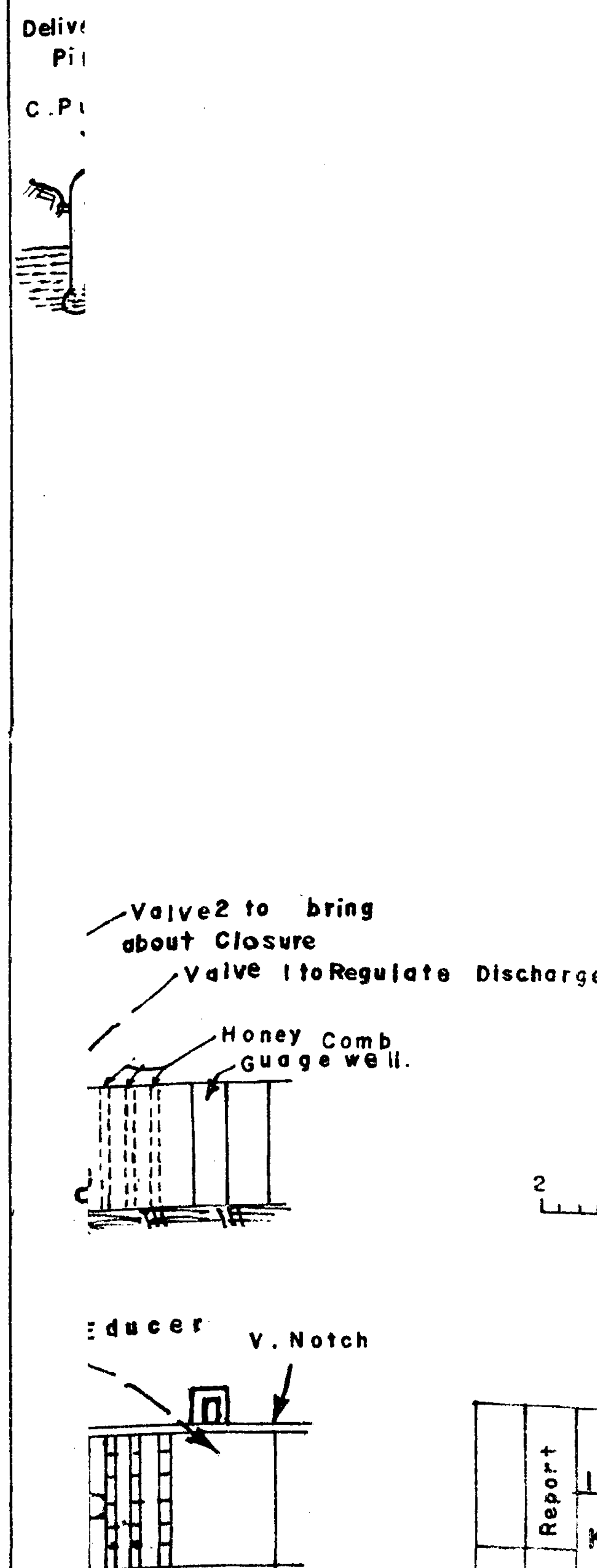

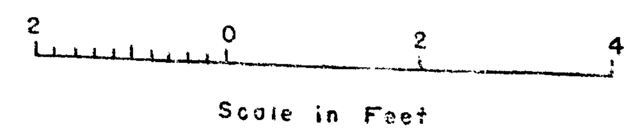

Report	Public Works Department — Madras. Irrigation Research Station — Poondi.			
File	KUNDAH HYDRO ELECTRIC PROJECT. PLANT. 2. MODEL LAYOUT.			
192/60 Drawing	M. Poovayya	U.V.[illegible]		
	J. E. R	A. R. O	E. E. R.	
	Trd: C. S. K.	Ckd:	Fig. 3.	

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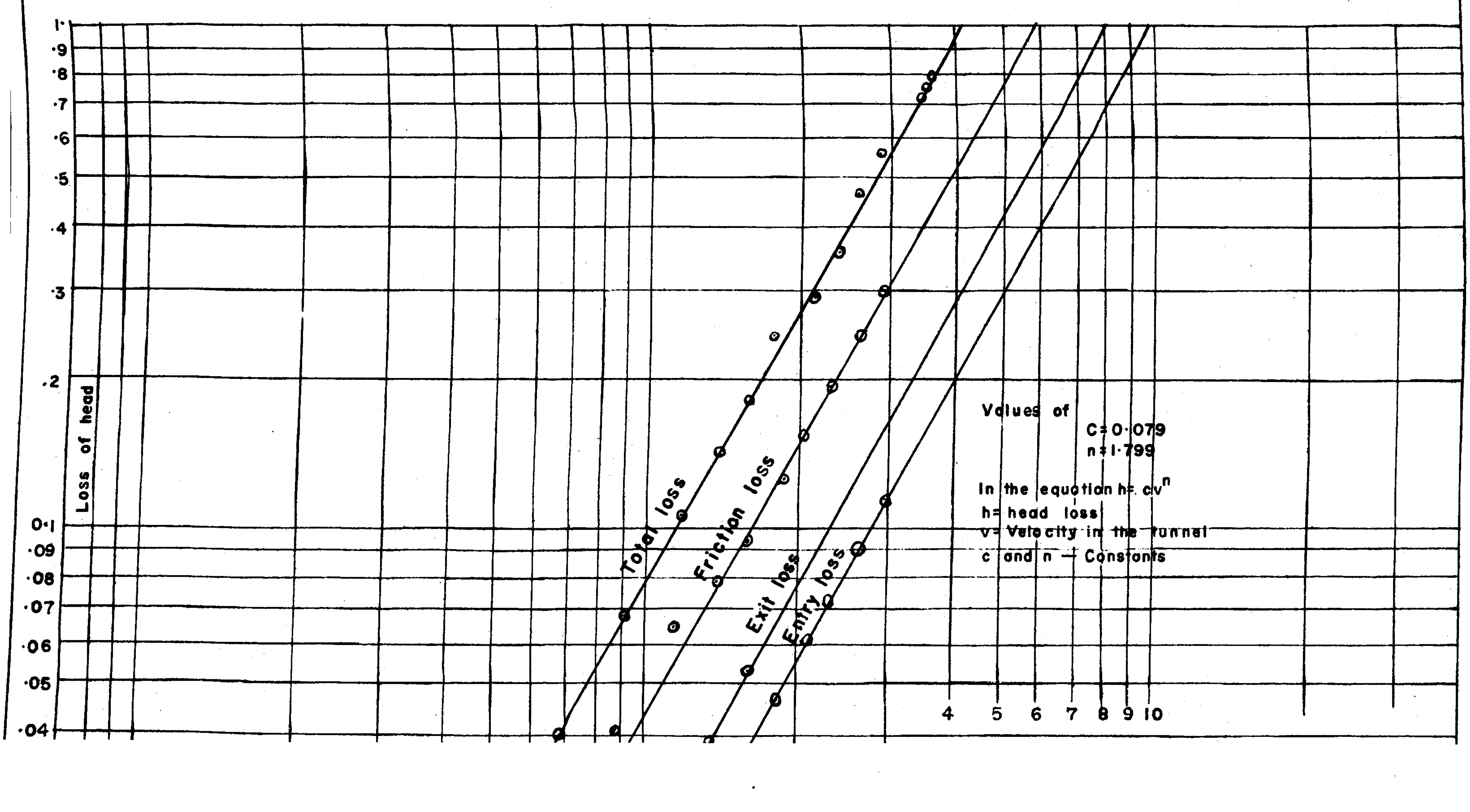

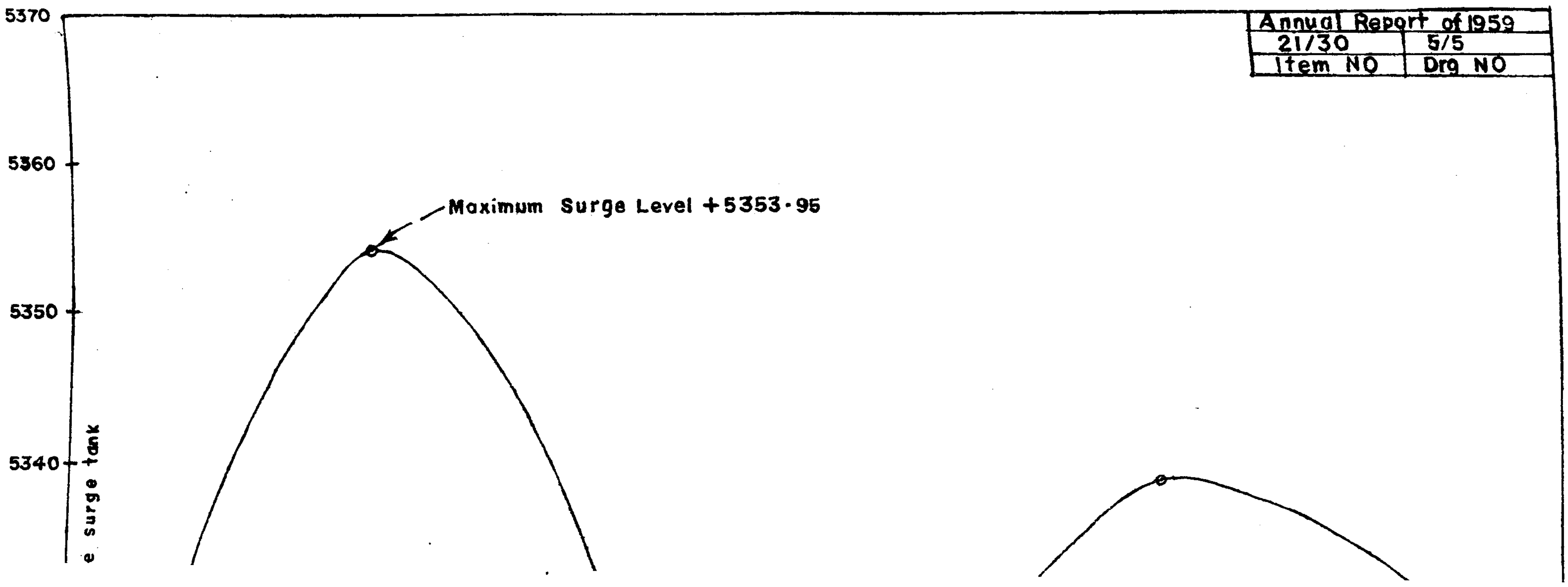
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e surge tank

Sandynallah Spillway

ENERGY DISSIPATION STUDIES

1. Introduction

1.1. The Sandynallah Dam forms part of the Sandynallah Reservoir Scheme which is under execution in the Nilgris District. The dam is to be built across the Sandynallah Stream and the aim is to divert the water to the Pykara Reservoir and increase the generating capacities of the existing Singara and Moyar Power Stations (Vide Figure-1).

1.2. The total length of the dam is 790 feet of which 130 feet is spillway section (Vide Figure-2). The crest of the spillway is 90 feet above the deepest foundation level and the maximum overflow depth is 9 feet. The spillway consists of three spans of 38′8″ each and two piers of 7 feet thickness each. The spillway is to discharge 11,000 cusecs with 9 feet head over the crest. This report deals with the studies conducted to device a suitable energy dissipation arrangement below the spillway.

2. Data

1.	Length of spillway including pier thickness	... 130 feet
2.	Mex No. and size of spans	... 3 Nos. (each 38′8″)
3.	Thickness of pier	... 7 feet
4.	Maximum head over crest	... 9 feet
5.	Height of spillway above the deepest foundation level	... 90 feet
6.	Crest level of spillway	... El. 7030 0
7.	F.R.L. & M.W.L.	... El. 7039.0
8.	Deepest foundation level	... El. 6940.0
9.	Average bed level existing at site	... El. 6957.00
10.	Maximum estimated flood discharge	... 11,000 cusecs
12.	Shape of cutwater in the pier in front	... Semi-circular

3. Problem

The problem is to find out a suitable energy dissipation arrangement in rear of the spillway.

4. Model

4.1. A sectional model to a scale of 1/50 was constructed first in a masonry flume of sufficient width to accommodate the full length of the spillway. The profiles of the spillway tried in this model were four in number (including the tentative profile) (Vide Figure 3 (a) to 3 (d) as the Designs office of the Electricity Board revised the tentative profile three times successively. The studies conducted on these profiles were described separately below.

4.2. A three dimensional model to a scale of 1/100 in a separate tray was then constructed for finalising the energy dissipation arrangements. The ground contours in this model were moulded as per drawing received from the Snperintending Engineer, Technical Civil. The alignment of the river course both upstream and downstream for (650') and (700') respectively, and the cross-sections were incorporated as per the drawing received from the field. The section of the spillway was finished according to the finalised spillway profile (Figure 3d, a Revised profile No. 3). The layout plan of the comprehensive model is shown in the drawing No. 4.

4.3. Model Scales

A. SECTIONAL MODEL

1.	Linear scale	L_r	1/50
2.	Area scale	$(L_r)^2$	1/2,500
3.	Volume scale	$(L_r)^3$	1/1,25,000
4.	Velocity scale	$(L_r)^{\frac{1}{2}}$	1/7.07
5.	Discharge scale	$(L_r)^{5/2}$	1/17,675

B. THREE-DIMENSIONAL MODEL

1.	Linear scale	L_r	1/100
2.	Area scale	$(L_r)^2$	1/10000
3.	Volume scale	$(L_r)^3$	1/10,00,000
4.	Velocity scale	$(L_r)^{\frac{1}{2}}$	1/10
5.	Discharge scale	$(L_r)^{5/2}$	1/1,00,000

5. Experiments

5.1. Sectional model

5.1.1. As the bed of the river at dam site was reported to de rocky, the necessity for scour observations was not felt and hence experiments for scour effect were not considered in the regular studies for various energy dissipation arrangements.

5.1.2. Tentative profile and Revised Profile Nos. 1 (Vide Figure 3(a) and 3(d):

Experiments were conducted by erecting a stilling pool with a baffle wall 8 feet high located in the rear of the toe of the dam, with the floor of thr stilling pool kept at El. 6952 (Vide statement 1). The downstream bed afte baffle was moulded to El. 6956 (as against El. 6955, provided in the design taking into account 1 foot over burden. Proction blocks were tried with the length of the stilling pool successively changed as given in the Statement-I. Experiments were conducted with the same 8 feet high baffle wall but with the level of the stilling floor kept at El. 6950 and the downstream bed of the river moulded to El. 6956 as explained in the same Statement-1.

5.1.3. Similar experiments were repeated with 9 feet high baffle wall and the stilling floor kept at El. 6948 the other details being the same as above.

The details of the maximum hump height and the resulting velocities beyond the stilling pool for the experiments described in paras 5.1.2, and 5.1.3 and 5 are also tabulated in Statement-1.

5.1.4. Revised Profile No. 2 (vide Figure 3(c))

Then the spillway profile was revised for the second time (vide Figure 3 (c) by the Designs Office of the Electricity Board. However, the experiment were continued with this revised profile incorporated in the sectional model. As the main factors, which govern the energy dissipation arrangemen are height of the spillway and the overflow head on the crest, and in-as-much as these two factors were not changed the experiments conducted for the previous profile were not repeated for this profile.

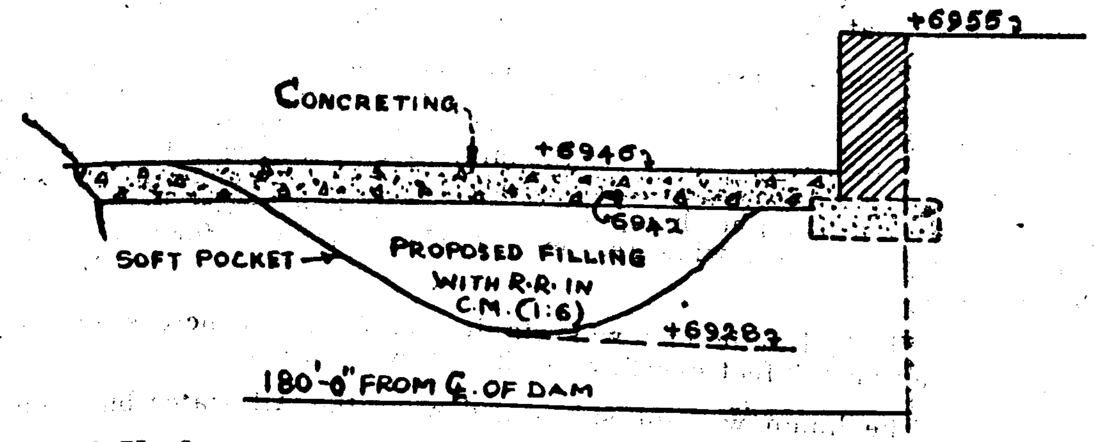
Further experiments were conducted with the stilling basin floor kept constant at El. 6950 and the bed level downstream of the baffle wall kept at El. 6955 (vide Statement II). Baffle walls of different heights say 7feet, 8feet and 9feet were tried, for various lengths of the stilling basin. Friction blocks in single as well as two rows were tried to suit the particular arrangement studied. The maximum hump elevation and resulting velocities beyond the stilling pool for the experiments described in this para are given in Statement-II.

5.1.5. Revised Profile No. 3 (Vide Figure 3 (d))

The spillway profile was again revised by the Electricity Board and the experiments were continued, with this new profile incorporated in the sectional model. Experiments on similar lines as explained in para 5.1.4. were conducted and the details of the maximum hump elevation and the resulting velocities are furnished in Statement-III.

FILED OFFICERS SUGGESTIONS

5.1.6. The site of the Sandynallah dam was visited by the Research staff and relevant particulars governing the energy dissipation arrangement were studied at site. Excavation for foundation was in progress in the left portion of the dam and in the rear of the spillway. Rock was being trimmed down to the level of 6942 probably to avoid some loose joints and fissures. There was also a soft pocket of about 30 feet width in the direction of flow downstream of the toe of spillway. The centre of the soft pocket was said to be 137'6" downstream of the centre line of the dam and the rock outcrop was stated to be at the level 6928 (deepest) in the pocket (vide sketch below.)



5.1.7. Based on the above field particulars, the field officers suggested the following arrangement:

(1) The baffle-wall may be moved farther downstream so that the soft pocket would be contained within the stilling pool.

(2) The soft pocket would be plugged up suitably with core to the level 6942 and a unifrom concrete slab of 4' thick laid to make the stilling basin floor level as 6946. The downstream bed would be cut to El. 6956. These details are shown in the sketch above.

5.1.8. Experiments were conducted incorporating the above suggestions in the sectional model. The depth of the stilling pool basin being 9' in this case and length stilling being fairly long, it was decided that the use of friction blocks would be superfluous and hence the same were not tried. The position of the baffle wall was successively changed 180'0" to 150'0" downstream of the centre line of the dam at intervals of 5'0". The details of the maximum hump elevation and the resulting velocities for the experiments described in para 5.2.3. are furnished in the Statement IV.

5.2. Three Dimensional Model

5.2.1. After a study of the results obtained in the sectional model it was decided to have the floor of the stilling basin at El. 6946 for the reasons explained in the previous paras.

5.2.2. The same experiments done with the sectional model based on field officers suggestions were repeated in the three dimensional model. The details of the maximum hump obtained in the three dimensional model for the experiments mentioned above were incorporated in the Statement-IV.

DISCUSSION OF RESULTS

6.1. Sectional Model.

6.1.1. From the results of the experiments carried out in the sectional model (of the tentative and revised profile No.1). vide Statement-1, it can be seen that a stilling basin of 45 feet length and 9 feet depth with the stilling basin floor kept at El. 6948 gives the minimum hump (El. 6969.80).

6.1.2. From the results of the experiment done on the Revised profile No. 2 (vide Statement II it can be seen that a stilling pool of length 35 feet and depth 7 feet, with stilling pool floor at El. 6950.0. and two rows of friction blocks (serial No. 15) gives the minimum hump 6965.6. However, the other similar arrangement (Serial No. 11) with 45 feet long stilling basin which gives the next higher hump i.e. El. 6966.5. is considered to be the best of this series for the following reasons: (Vide Figure 5(a) for flow profiles and details).

(1) The hydraulic jump was stable even for 2 feet increase over and above the maximum 9 feet overflow on the spillway. crest.

(2) The hump was not submerged when the tail water built up to 6967.0.

As it was thought that the stilling basin floor if kept at El. 6950.0, would economise rock cutting the same floor level was kept throughout this series of the experiments. However as the profile was again revised (Revision No.3.) the experiments were continued with the revised profile No. 3.

6.1 3. From the results of the experiments conducted on the revised profile No. 3 (vide Statement-III) it can be seen that stilling basin of length 45 feet and 7 feet with the stilling basin floor kept at El. 6950.0 and two rows friction blocks (for details vide Figure 5(b)) gives the minimum hump height (El. 6966.55') This is more or l ss the same as the arrangement as was explained in para 6.1.2. Hence this would have been recommended as the efficient energy dissipation arrangement for adoption but for the difficulties obtaining in the field with regard to the location of the soft pocket etc., explained above in para 5.2.1.

6.2. Filed Officers Suggestions.

6.2.1. From the results of the experiment conducted on the lines of the field officers suggestions (vide statement IV), it can be seen that all the arrangements gave very low humps i.e. below the tail water elegation 6967.0 unlike the previous series. This is mainly due to excessive length of the stilling basin. However, the hump elevations were slightly built up on keeping the tailwater El. 6967.0 in all the cases. The flow pattern in all the cases were almost similar and the hump almost got flattened to the tail elevation of 6967.0. Though the relative merits of the arrangements (effecting only in length of pool) could not be distinguished by marked differences, a selection of the best arrangement could be made in as follows:

The stilling basin of length 110 feet downstream or of 105 feet would be the best means of enclosing the soft pocket within the stilling basin and avoiding any settlement failure in future. Hence they are considered to be the best of the arrangements.

6.3. Three Dimensional Model:

The experiments as per field officers' suggestions only were conducted on the three-dimensional model. The flow pattern as observed in the sectional model was also realised to in all the cases to the three-dimensional model.

7. CONCLUSION

A 9 feet deep stilling basin with up stream face of the baffle wall at 110 feet downstream of the toe of the spillway or at 105 feet downstream of the spillway and with its floor at El. 6946. is recommended for adoption (vide Figure-6).

——:o.——

SANDYNALLAH RESERVOIR SCHEME
SANDYNALLAH SPILLWAY

Photo to accompany the Energy Dissipation Studies

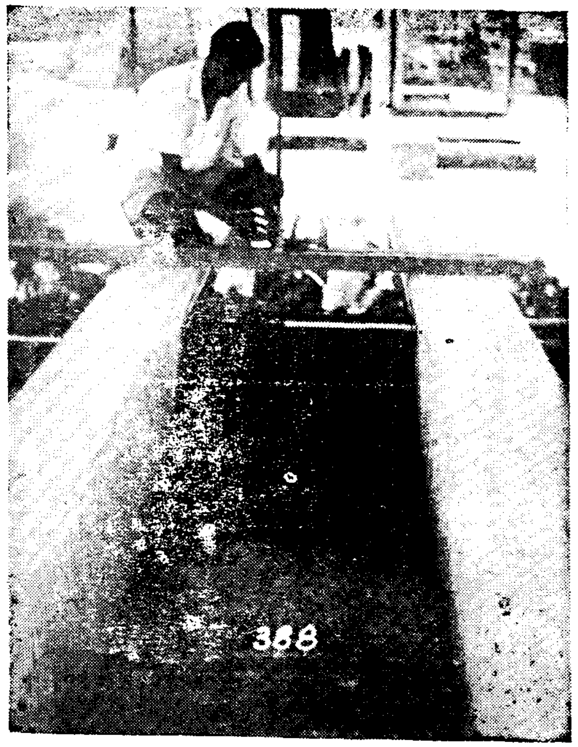


PHOTO No. .
SECTIONAL MODEL FOR ENERGY DISSIPATION STUDIES

Flow pattern for the stilling pool 9 feet deep 110 feet long with depressed floor elevation at 6946 This is for no tail water control.

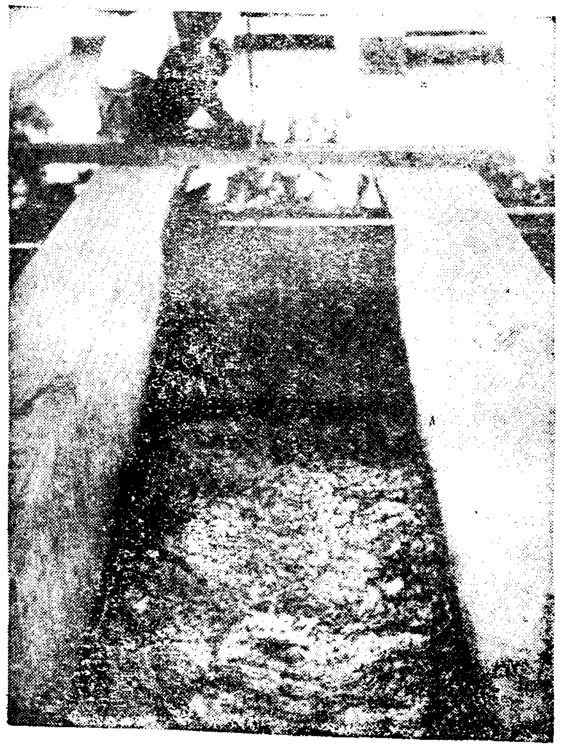


PHOTO No. .
SECTIONAL MODEL FOR ENERGY DISSIPATION STUDIES

Flow pattern for the same arrangement as above when tail water was controlled at El. 6967.

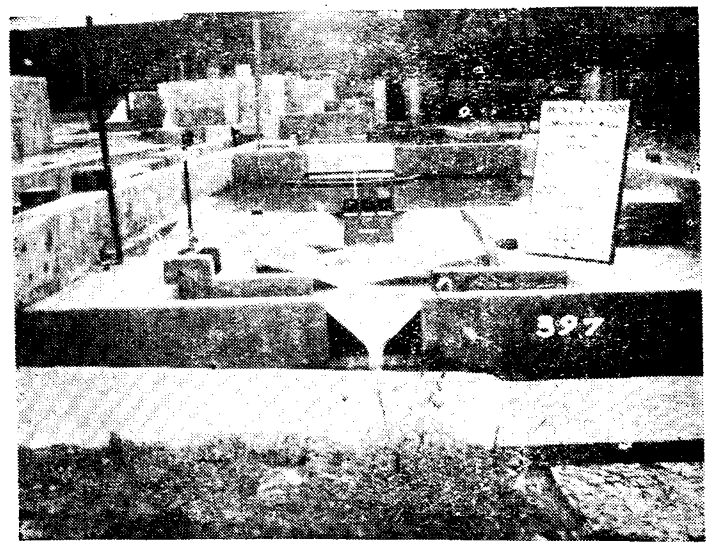


PHOTO No.
Front view of the three dimensional model when maximum free head of 9 feet was maintained over crest.

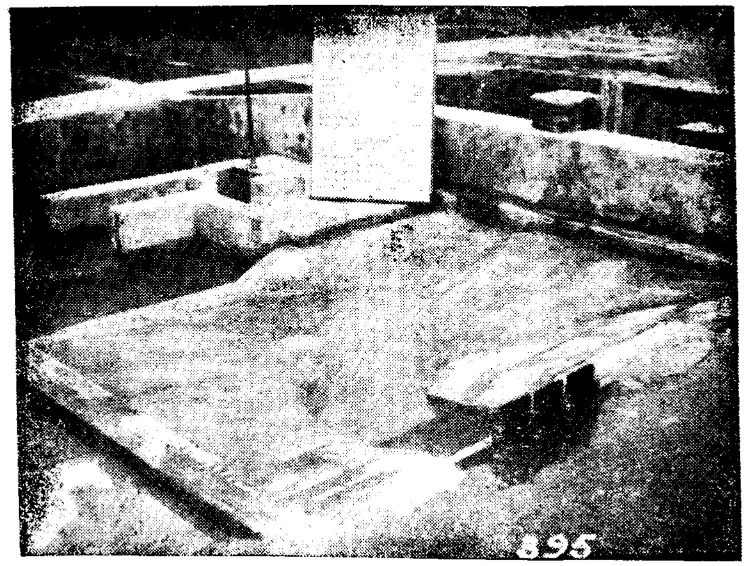
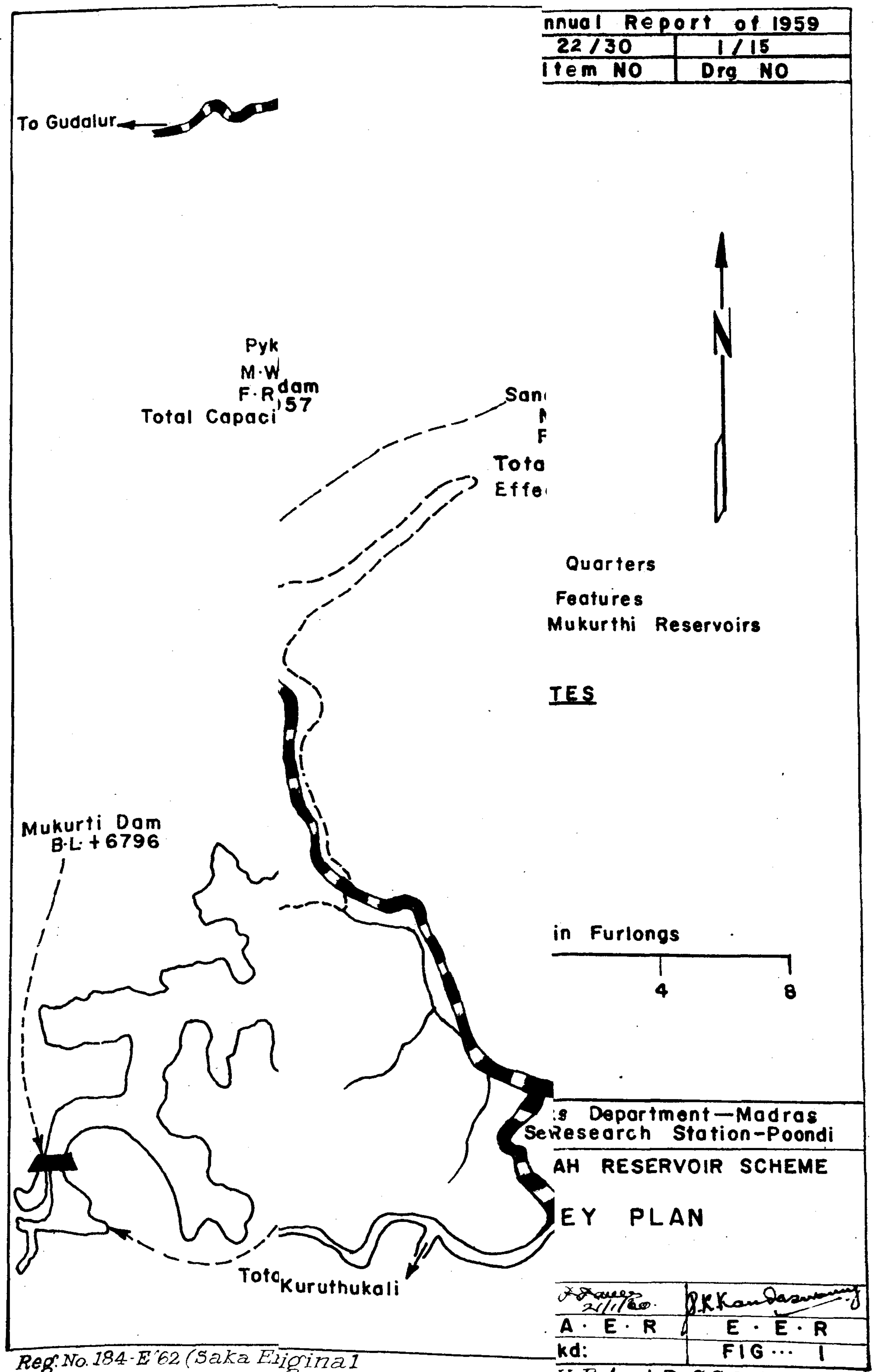


PHOTO No.
View of the three-dimensional model from left flank of the model, when maximum free head of 9 feet was maintained over crest.



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H.B.A., A.D.,C.S.O., Madras-5.

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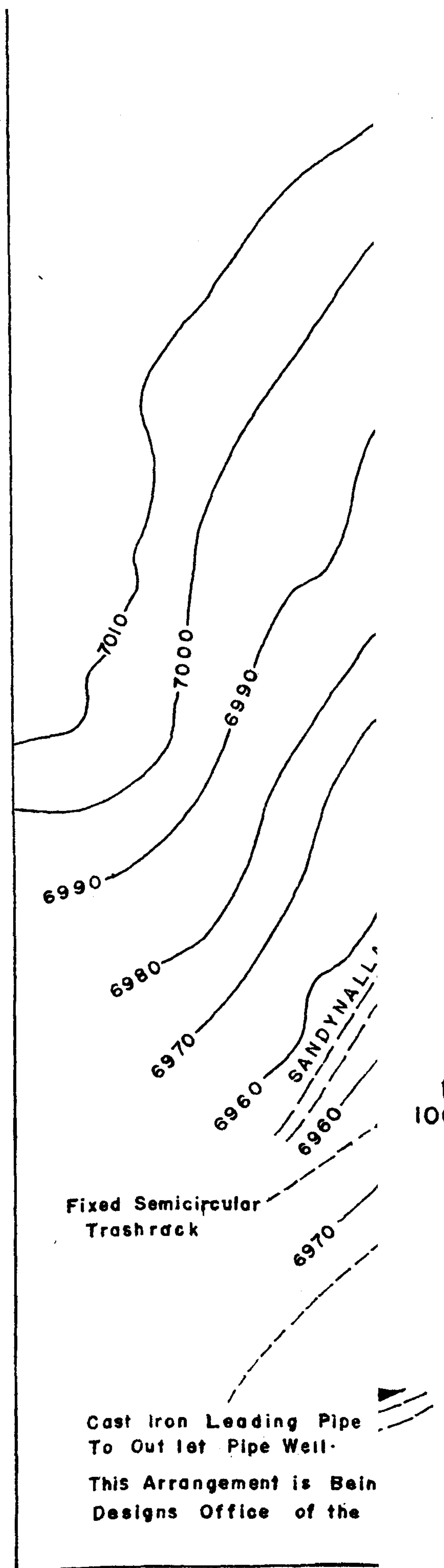

HYDRAULIC PARTICULARS

Length of dam at top	790 Ft.
Length of Earthen dam	560 Ft.
Length of Masonry Section (Bet. CH. 385 & CH. 615)	230 Ft.
Length of Spillway (Bet. CH. 435 & 565)	130 Ft.
Spans of the Spillway	3 Nos. 38 Ft. 8 in. Each
Piers	7 Ft. Thick.
Crest	El. 7030
M.W.L & F.R.L.	El. 7039.
Over flow Head	9 Ft.
Maximum Discharge	11,000 CFS.

NOTE:- This drawing is Mainly Based on the Plan San. Hes-I and the Revisions made by the Designs office of the E.B. With Respect to the Following Particulars.

1. Length of Spillway and Provision of Gate and piers.
2. Position of Spillway and Bulk Head Section in the entire Length of the Composite dam.

Scale in Feet

100 0 100 200

File Report

Public Works Department—Madras.
Irrigation Research Station—Poondi.

Sandynallah Reservoir Scheme

LAY-OUT PLAN OF THE SANDYNALLAH DAM

Drawing 287/59

K.P. Gajaravadivelu 21-1-60	[signature] 21/1/60.	P.K. Ramdasan[signature]
J.E.R	A.E.R	E.E.R
Trd: V.V.	ckd:	FIG. 2

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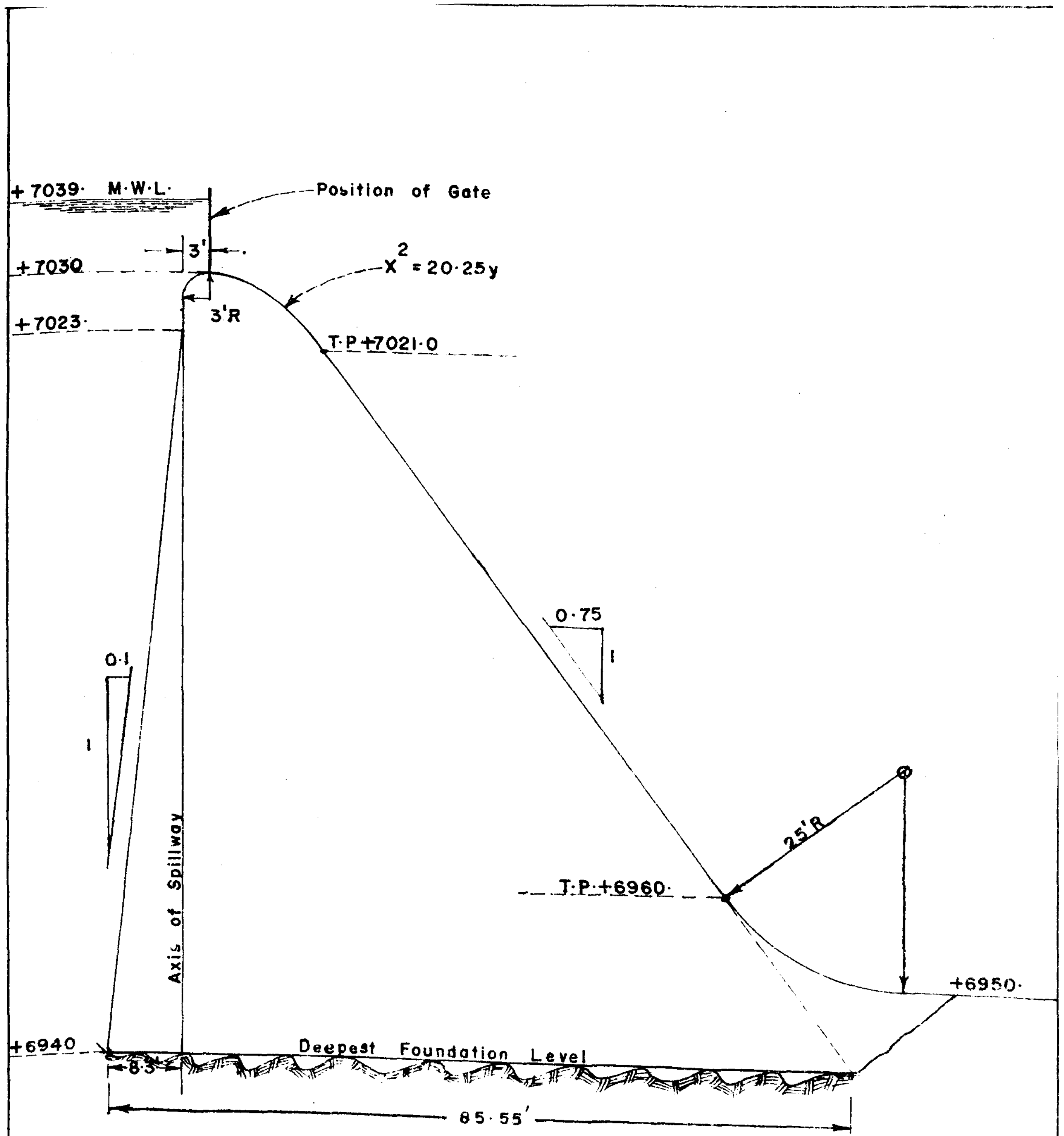

Public Works Department—Madras
Irrigation Research Station—Poondi·

Sandynallah Reservoir Scheme

SANDINALLAH SPILLWAY

TENTATIVE PROFILE.

Report

File

Drawing 9/60

J·E·R·	A·E·R·	E·E·R·
Trd: V·V·	ckd:	FIG··· 3a

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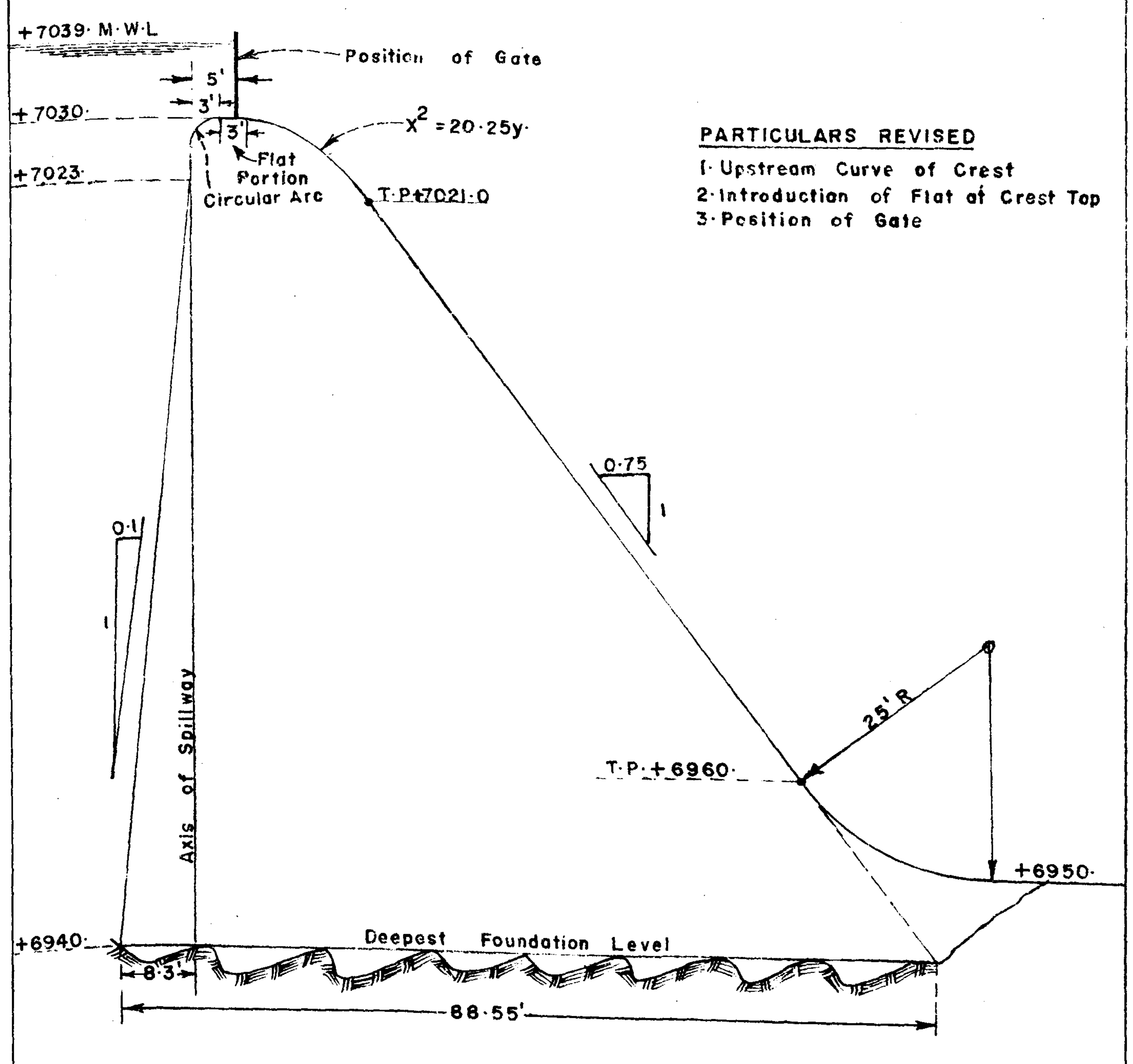

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Irrigation Research Station — Poondi·
Sandynallah Reservoir Scheme

SANDINALLAH SPILLWAY
REVISED PROFILE No·1.

K.P. Gajendramurthy 21-1-60	[signature] 21/1/60	V.K. Kandaswamy
J·E·R	A·E·R	E·E·R
Trd: V·V·	chd:	FIG··· 3.D.

Report · File · Drawing 10/60

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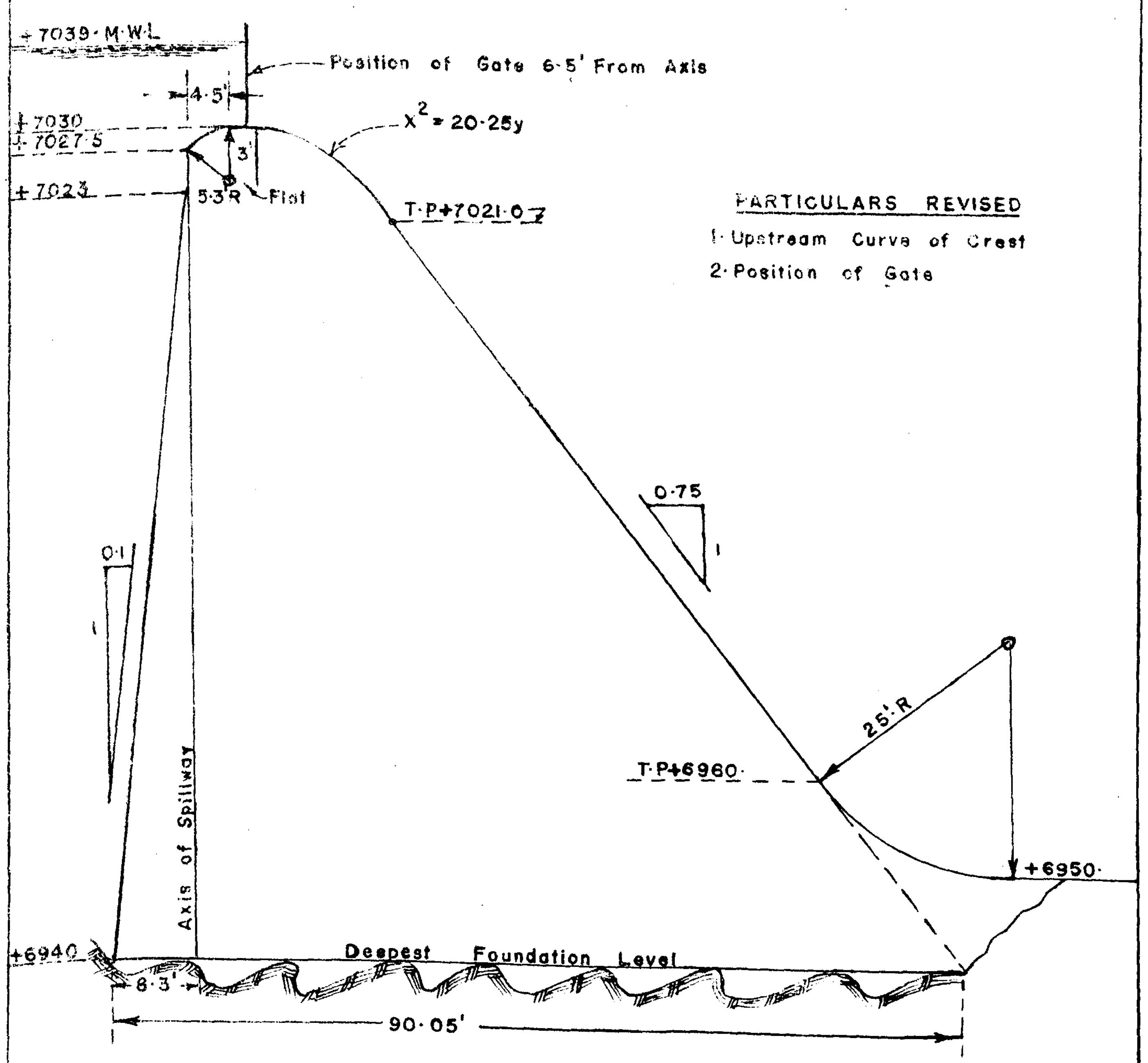

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Sandynallah Reservoir Scheme
SANDINALLAH SPILLWAY
REVISED PROFILE No·2.

Drawing 11/60

19-1-60	21/1/60·	
J·E·R	A·E·R	E·E·R·
Trd: C·S·K·	ckd:	FIG· · ·3·C·

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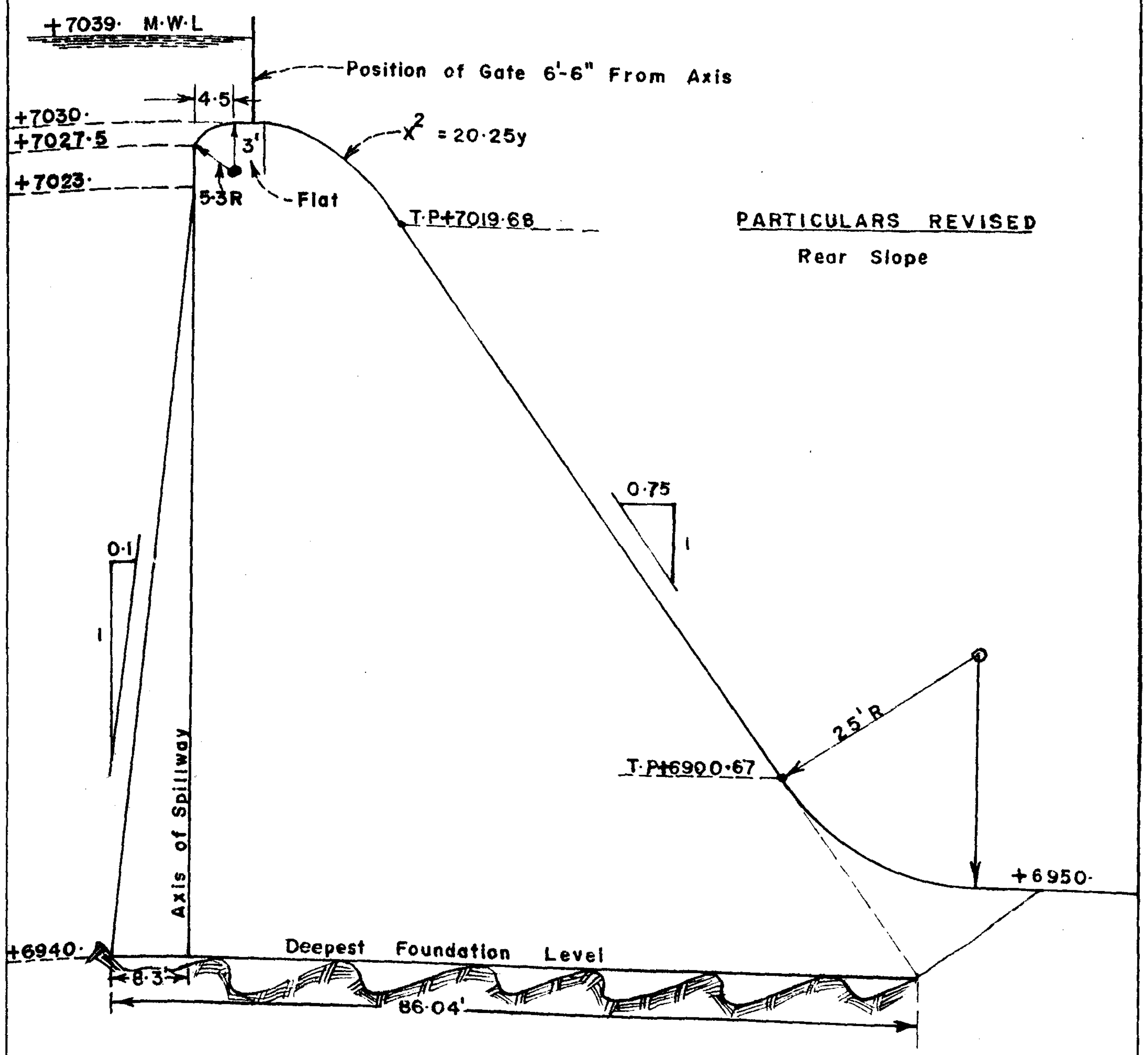

Public Works Department— Madras·
Irrigation Research Station—Poondi·
Sandynallah Reservoir Scheme
SANDINALLAH SPILLWAY
REVISED PROFIE No·3

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		(signed) 19-1-60	(signed)	(signed)
		J· E· R·	A· E· R·	E· E· R
		Trd: V·V·	Ckd:	FIG· · 3d·

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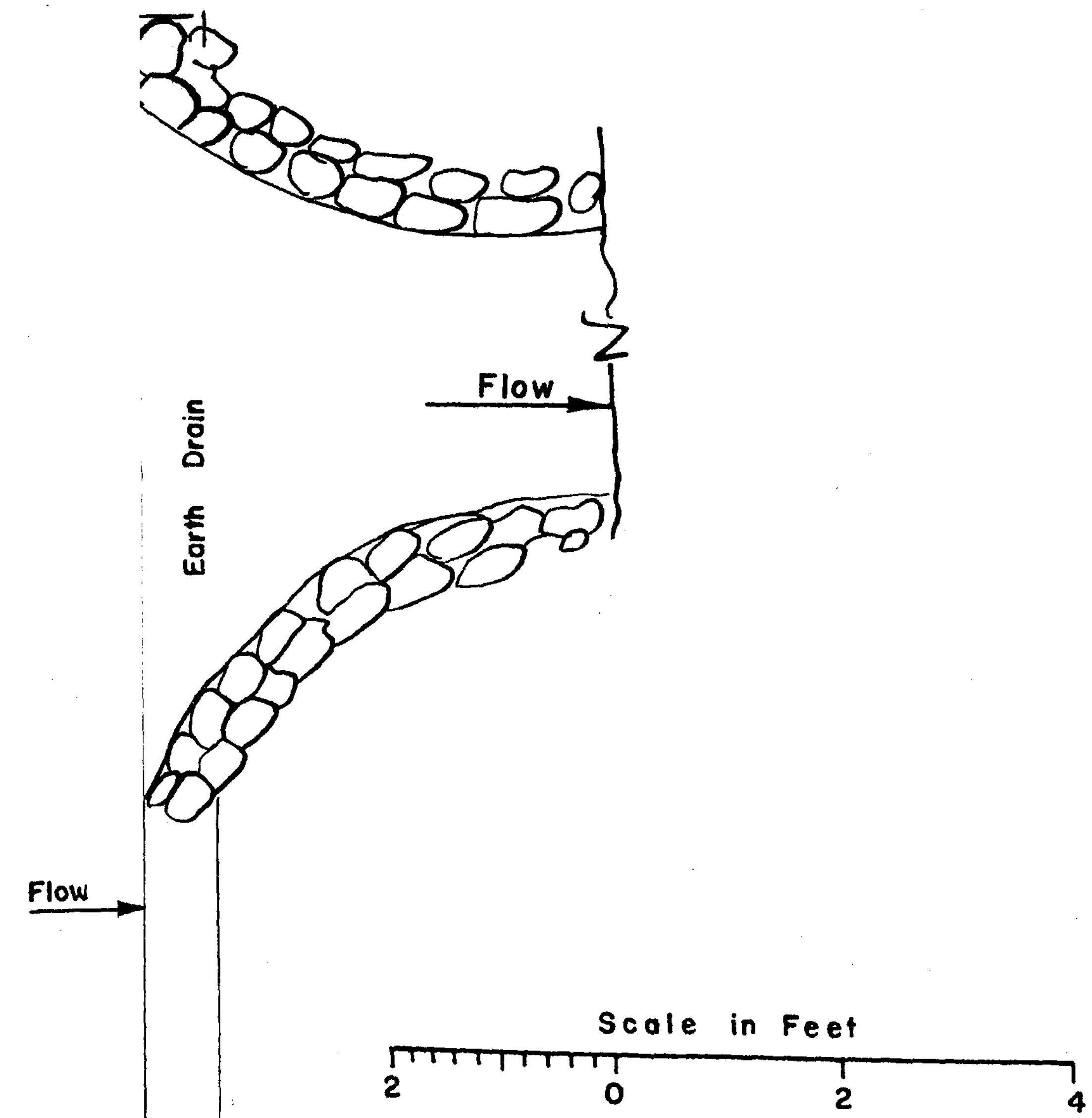

Public Works Department — Madras.
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Sandynalla Reservoir Scheme
Sandynalla Spillway
Layout Plan of the Comprehensive Model

Report File Drawing 288/59

K. P. Gajendran 21-1-60	21/1/60	
J·E·R	A·E·R	E·E·R
Trd: T.KS	Ckd:	FIG····4

by Sri K. SAMPATH, B.A., A.D., C.S.O., Madras-5

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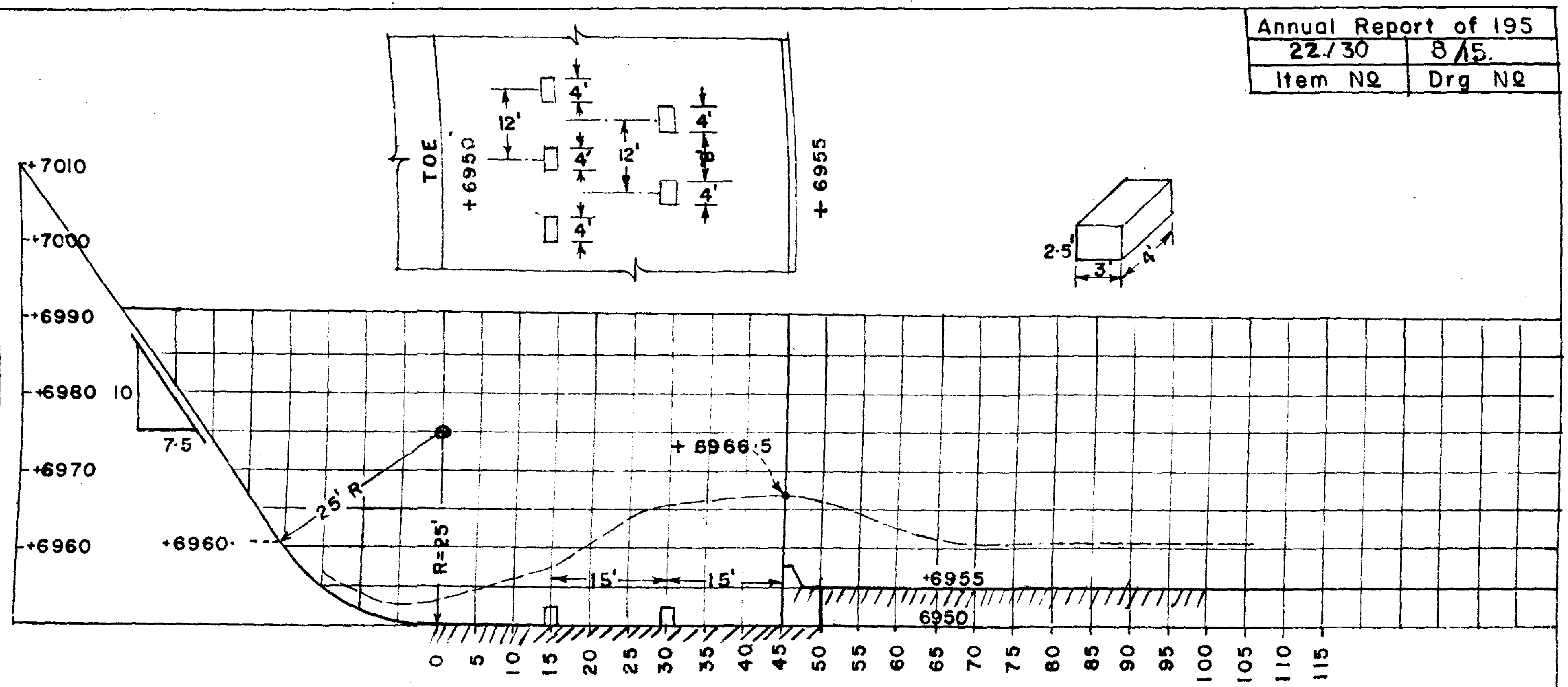

Legend

Length of pool	= 45'
Depth of pool	= 7'
Length of spillway	= 130'
M · F · D	= 11,000 cusecs
M · W · L	= +7039
Crest	= +7030
Over flow head	= 9'
Length of each span	= 38'- 8"
Width of pier	= 7'
Stilling floor elevation	= +6950
Apron elevation	= +6955

Public Works Department — Madras.
Irrigation Research Station — Poondi.

Sandynallah Reservoir Scheme
SANDYNALLAH SPILLWAY

ENERGY DISSIPATION STUDIES
FLOW PROFILE

Report			
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Drawing 1/60	K.P. Gajarathnavelu 19-1-60	[signature] 2/1/60.	[signature]
	J · E · R	A · E · R	E · E · R
	Trd: C·S·K	Ckd:	FIG ... 5·a

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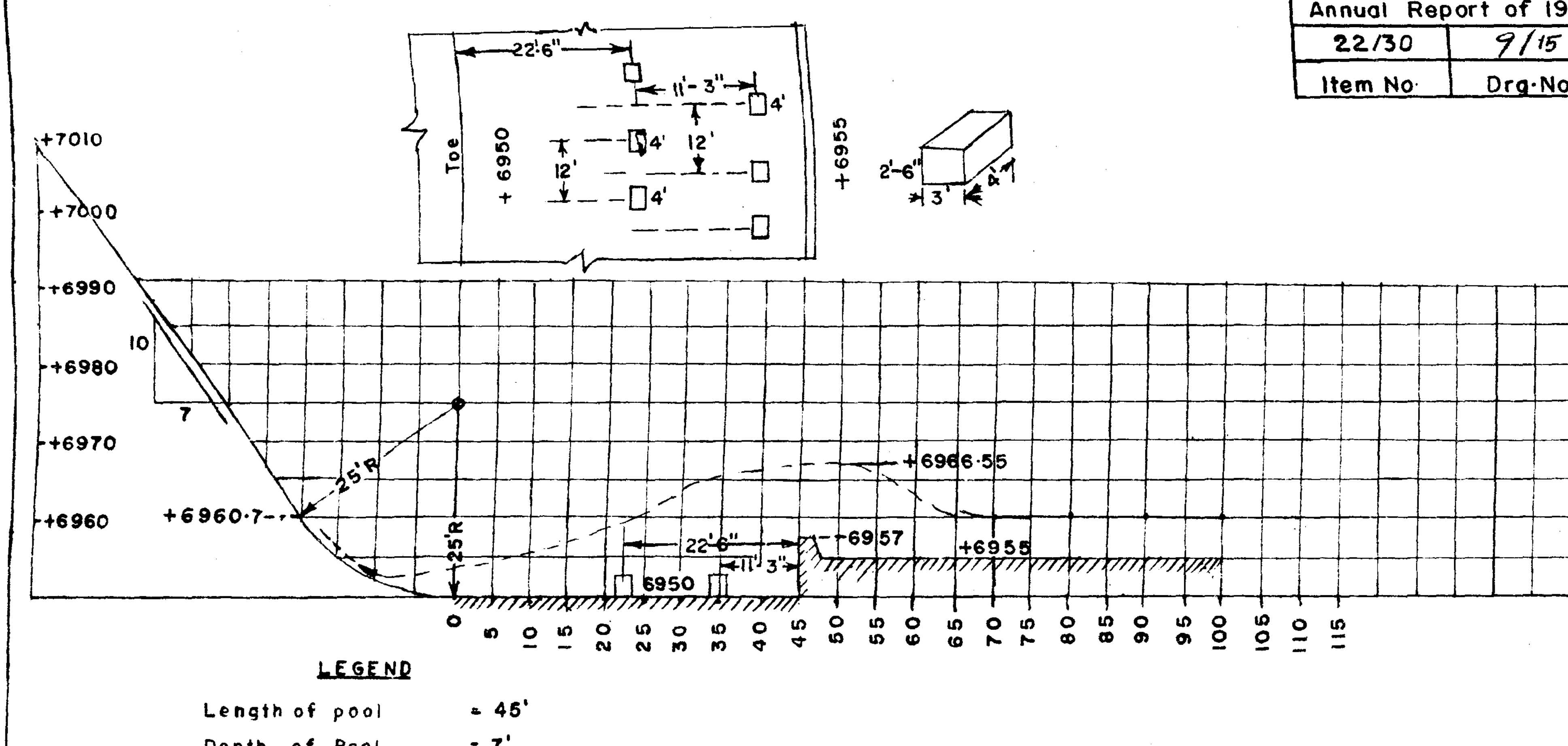

LEGEND

Length of pool	= 45'
Depth of Pool	= 7'
Length of Spillway	= 130'
M·F·D	= 11000 Cusecs
M·W·L	= +7039
Crest	= +7030
Over flow head	= 9'
Length of each Span	= 38'-8"
Width of Pier	= 7'
Stilling floor elevation	= +6950
Apron elevation	= +6355

Public Works Department— Madras
Irrigation Research Station— Poondi

Sandynallah Reservoir Scheme
SANDYNALLAH SPILLWAY
ENERGY DISSIPATION STUDIES
FLOW PROFILE

Report			
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	Trd: C·S·K	ckd:	FIG· · 5·b·

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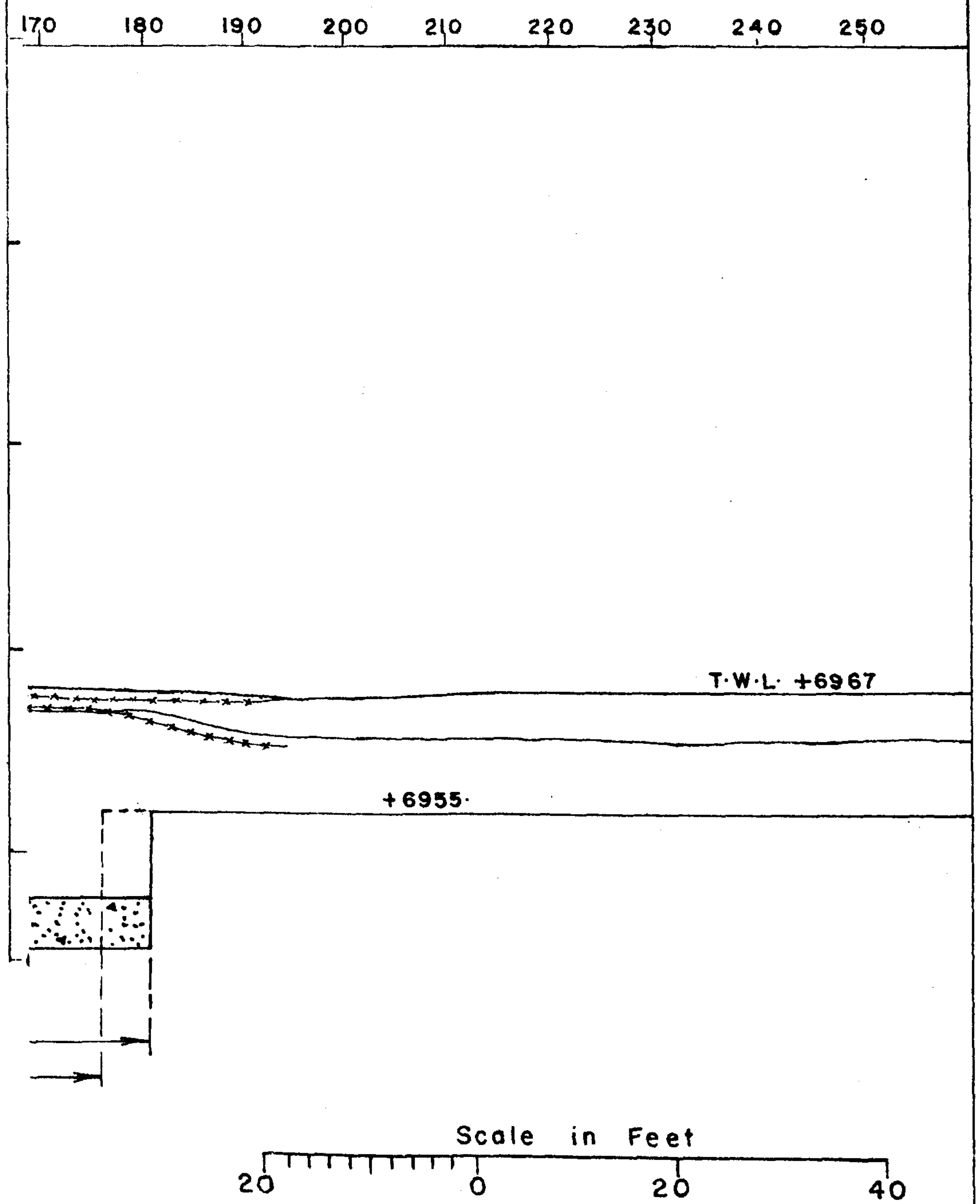
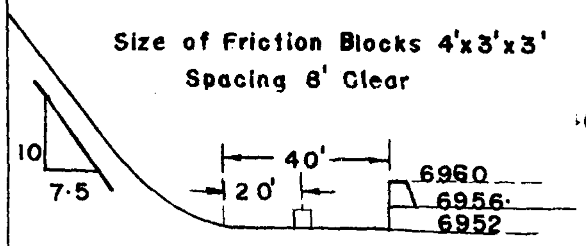
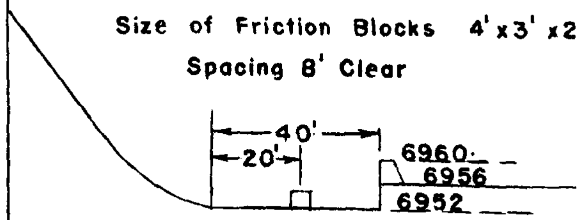
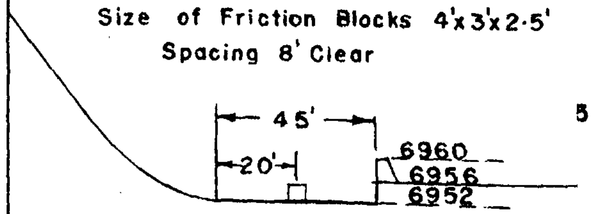
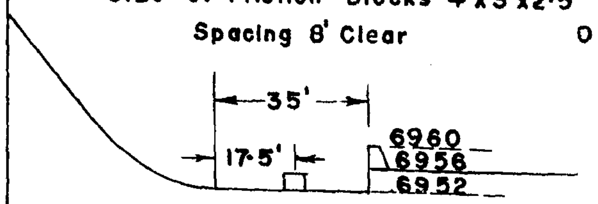
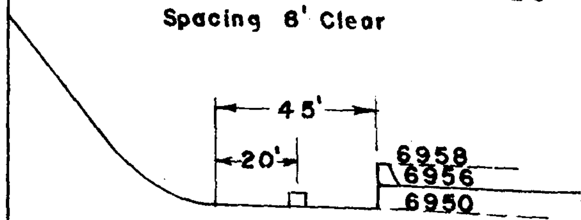


Report	Public Works Department — Madras Irrigation Research Station — Poondi·
	SANDINALLAH SPILLWAY FLOW PROFILES AS OBSERVED IN THE SECTIONAL MODEL
File	
Drawing 2/60	

K. R. Guruswami [?] 19-1-60	[signature] 21/1/60	[signature]
J E R	A E R	E · E · R
Trd: V·V·	ckd:	FIG · · · 6·

No.	DESCRIPTION OF ARRANGEMENTS	P ION [·	VELOCITY IN FEET PER SECOND AT 200Ft. D/S OF AXIS OF SPILLWAY	VELOCITY IN FEET PER SECOND AT 250Ft. D/S OF AXIS OF SPILLWAY	REMARKS
1	Size of Friction Blocks 4'x3'x3' Spacing 8' Clear 	·0	23·17'/Sec	20·35'/Sec	
2	Size of Friction Blocks 4'x3'x2 Spacing 8' Clear 		21·30'/Sec	20·15'/Sec	
3	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear 	5	21·30'/Sec	17·90'/Sec	
4	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear 	0	20·15'/Sec	14·7'/Sec	BEST
5	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear 				

Public Works Department—Madras
Irrigation Research Station—Poondi
Sandynallah Reservoir Scheme

STATEMENT-I

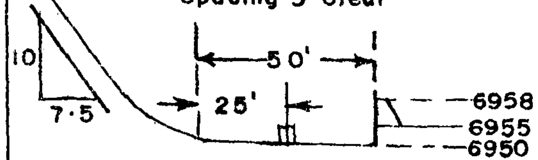
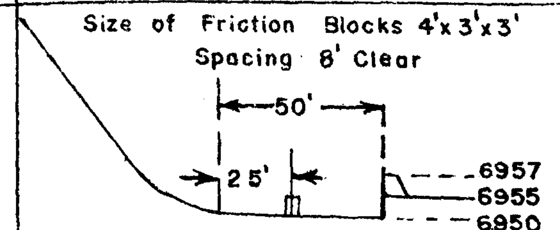
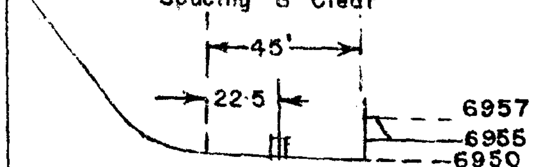
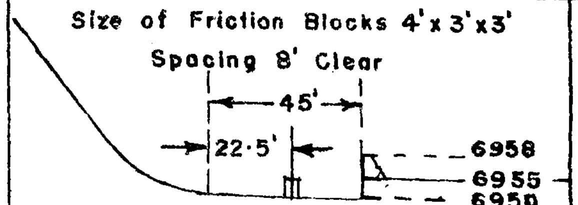
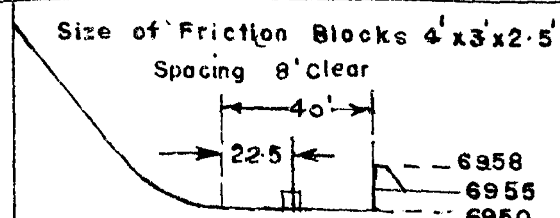
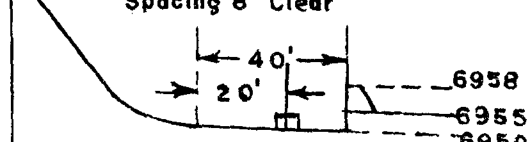
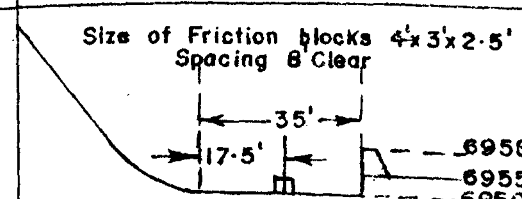
TENTATIVE PROFILE AND REVISED PROFILE No·1

(Vide Fig·3a & 3b)

P. Gajendran 19-1-60	21/1/60	J. K. Kandaswamy
J E R	A E R	E E R
Trd: V·V·	ckd:	

Drg·No·15/60

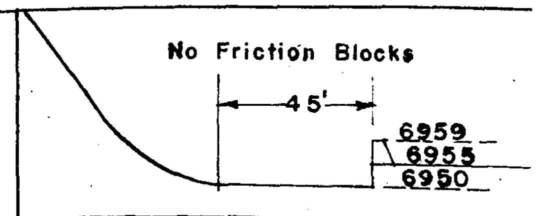
STATEMENT–II

No	DESCRIPTION OF ARRANGEMENT	MAXIMUM HUMP ELEVATION	VELOCITY IN FEET PER SECOND AT 200 Ft D/S OF AXIS OF SPILLWAY	VELOCITY IN FEET PER SECOND AT 250 Ft D/S OF AXIS OF SPILLWAY	REMARKS
1	Size of Friction Blocks 4'x3'x3' high Spacing 5' Clear  	6969·0	22·0'/Sec	20·4'/Sec	
2	Size of Friction Blocks 4'x3'x3' Spacing 8' Clear  	6967·7	20·35'/Sec	19·35'/Sec	
3	Size of Friction Blocks 4'x3'x3' Spacing 8' Clear  	6967·6	20·35'/Sec	17·9'/Sec	
4	Size of Friction Blocks 4'x3'x3' Spacing 8' Clear  	6968·3	18·62'/Sec	12·7'/Sec	
5	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear  	6969·7	23·17'/Sec	20·15'/Sec	
6	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear  	6968·7	22·7'/Sec	19·36'/Sec	
7	Size of Friction blocks 4'x3'x2·5' Spacing 8' Clear  	6968·25	20·35'/Sec	19·35'/Sec	

Reg.No. M0·E'62 (Saka Era 1884)-375. Reproduced from Indentor's Original Printed by Sri K. SAMPATH, B. A., A.D. C.S.O. Madras-5.

(Continued)

8



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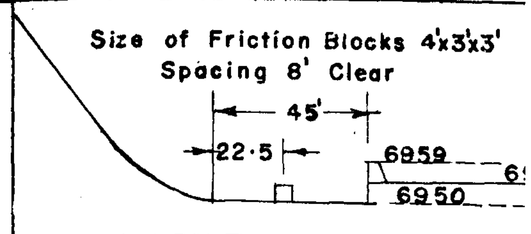

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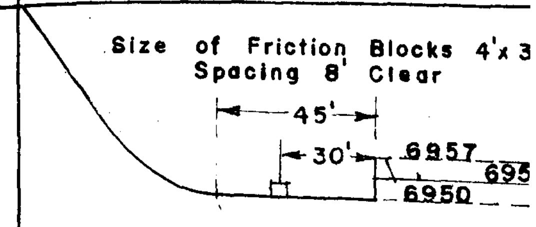

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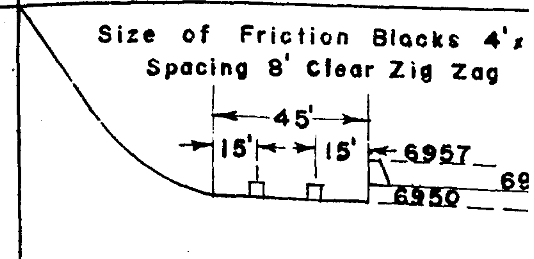

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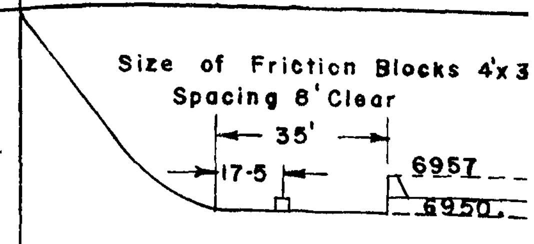

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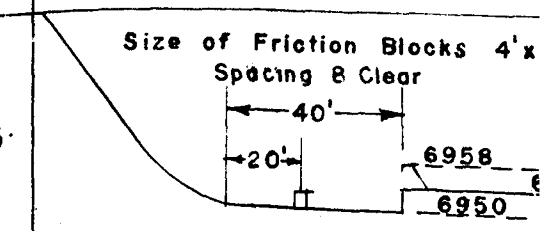

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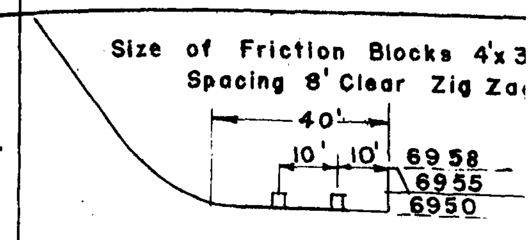
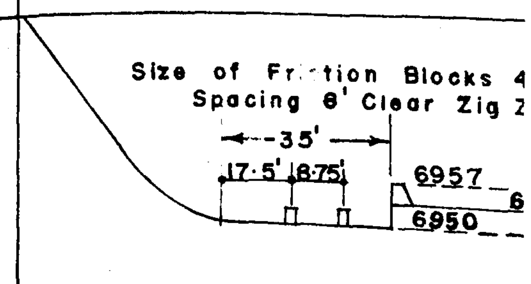
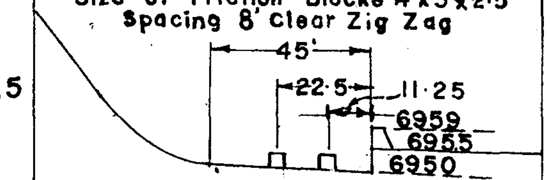
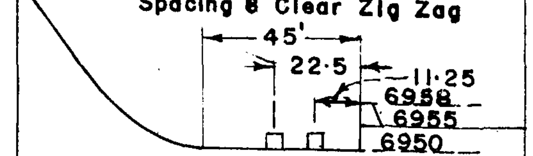
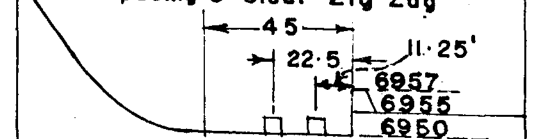
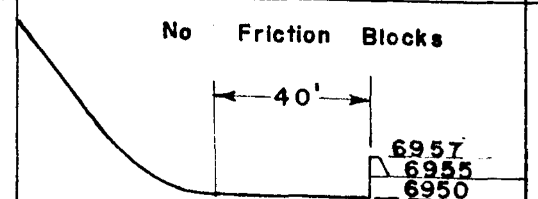


15·



Reg. No. 185-E'62 (Saka Era 1884) 37

No	DESCRIPTION OF ARRANGEMENT	HUMP ELEVATION MAX·	VELOCITY IN FEET PER SECOND AT		REMARKS
			150Ft· D/S OF TOE OF SPILLWAY	200Ft· D/S OF TOE OF SPILLWAY	
15	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6968·5	17·32'/Sec·	11·65'/Sec·	
16	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6967·0	14·7'/Sec·	13·7'/Sec·	
17	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6966·55	14·7'/Sec·	12·70'/Sec·	BEST Vide Fig·5b· For The Flow Profile
18	No Friction Blocks 	6970·95	20·35'/Sec	19·35'/Sec·	

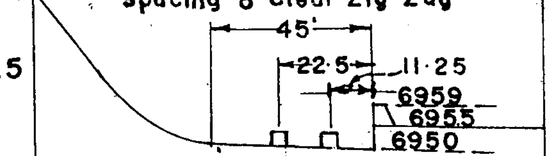
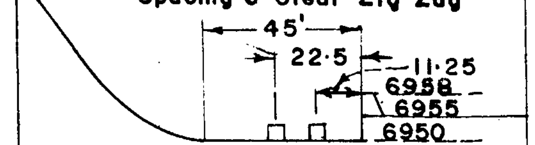
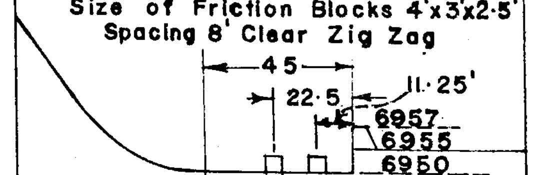
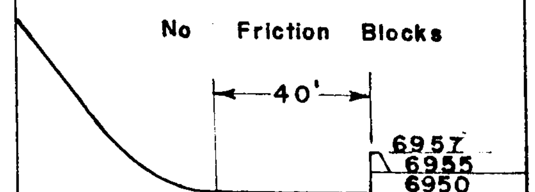
Public Works Department–Madras
Irrigation Research Station–Poondi
Sandynallah Reservoir Scheme
STATEMENT–III
FOR SECTIONAL MODEL OF REVISED PROFILE No·3

ANNUAL REPORT OF 1959	
22/30	14/15
ITEM NO	DRAWING NO

Report. File Drawing 17/60

K.P. Gajavadhan 21-1-60	21/1/60	
J E R	A E R	E · E · R
Trd: V·V·	ckd:	

Printed by Sri K.SAMPATH.B.A., A.D,C.S.O.,Madras-5.

No	DESCRIPTION OF ARRANGEMENT	HUMP ELEVATION MAX·	VELOCITY IN FEET PER SECOND AT 150Ft· D/S OF TOE OF SPILLWAY	VELOCITY IN FEET PER SECOND AT 200Ft· D/S OF TOE OF SPILLWAY	REMARKS
15	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6968·5	17·32'/Sec·	11·65'/Sec·	
16	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6967·0	14·7'/Sec·	13·7'/Sec·	
17	Size of Friction Blocks 4'x3'x2·5' Spacing 8' Clear Zig Zag 	6966·55	14·7'/Sec·	12·70'/Sec·	BEST Vide Fig·5b· For The Flow Profile
1.8	No Friction Blocks 	6970·95	20·35'/Sec	19·35'/Sec·	

Report. File	Public Works Department–Madras Irrigation Research Station–Poondi Sandynallah Reservoir Scheme STATEMENT–III FOR SECTIONAL MODEL OF REVISED PROFILE No·3	
17/60 Drawing	K.P. Gajavadhanh 21-1-60 J E R Trd: V·V·	21/1/60 A E R ckd:
	E · E · R	

ANNUAL REPORT OF 1959	
22/30	14/15
ITEM NO	DRAWING NO

Printed by Sri K.SAMPATH.B.A., A.D, C.S.O., Madras-5.

Sandynallah Reservoir Scheme

SANDYNALAH SPILLWAY-PRESSURE STUDIES

1. Introduction

The Sandynallah spillway forms part of the composite dam to be built across the Sandynallah stream in the Nilgris District at a place 5 miles North-West Ootacamund (vide Figure-1). The water stored in this reservoir would be diverted to the Para Reservoir and utilised to increase the generation of power in the existing Singara and Moyar Power Stations. The spillway crest is 90 feet above the deepest foundations (vide Figure-2). The spillway portion of the dam consists of 3 spans of 38′8″ each with two piers of 7′0″ thick each. It is designed to surplus a maximum flood discharge of all 11,000 cusecs with an overflow head of 9 feet. This report deals with the studies conducted to determine whether the designed profile is devoid of subatmospheric pressures and if not, whether it will develop the negative pressures only within reasonable limits for various conditions of flow.

2. Data

1.	Length of spillway including pier thickness	...	130 feet.
2.	No. and size of spans of spillway	...	3 Nos. (each 38′8″ long)
3.	Pier thickness	...	7 feet
4.	Maximum head over the crest	...	9 feet
5.	Height of spillway above deepest foundation level	...	90 feet
6.	Crest level of spillway	...	El.7030
7.	F. R. L. or M. W. L.	...	El. 7039
8.	Deepest foundation level	...	El. 6,940
9.	Maximum estimated flood discharge (vide Figure-3)	...	11,000 cusecs.

3. Problem

3.1. The problem is to find out the distribution of pressures along the spillway profile for the provisional profile (Figure 3 (a)) received from the Superintending Engineer, Technical (Civil).

3.2. To find out the distribution of pressure along the spillway surface for the revised profila No. 1 (Figure 3 (b)) sent by the Superintending Engineer, Technical (Civil) subsequently.

3.3. To find out the distribution of pressures along the spillway surface for the revised profile No. 2 (Figure 3 (c)) received/from the Superintending Engineer Technical Civil.

3.4. To find out the distribution of pressures along the spillway surface for the revised profile No. 3 (Figure 3 (d)) received from the Superintending Engineer Technical (Civil).

3.5. As the location of the crest gate was finalised by the Designs Office of the Electricity Board (on the consideration of the convenience of erection, accessibility for the inspection, maihtenance ect.,) the necessity to find out the best location for the crest gate such that is gives only positive pressures for all conditions of gate opening did not arise.

4. Model

4.1. A sectional model to a scale of 1/50 was constructed in a separate flume of sufficient with to accommodate the entire length of spillway with 3 spans. Whenever a revised profile was sent from the Board, the previous model was demolished partly or completely as the case may be, and a new one constructed in its place to suit the revision. Piezometers of 1/8" bore copper tubes were embedded in the body of the spillway at salient points along the surface of the profile such that one end of each tube was normal to the surface of the profile at each point. The other ends of the copper tubes were connected by means of suitablo rubber tubes to a row of 1/8" bore glass tubes fitted to a monometric board.

4.2. The manometric board was made of teakwood. The particular spillway profile, which was then under study, was painted on the board to the same scale of 1/50. The glass tubes referred to obove were fitted on to this board vertically such that they indicate in the profile on the board, the points at which the copper tubes were embedded in the sectional model of the same profile. The board was graduated and fixed in such a way that the crest of the model and the crest of the painted profile on the board were at the same elevation. By this arrangement, it was possible to read directly the magnitude of lhe pressure occurring at each of the salient points for the various conditions of flow.

4.3. Tentative Profile (Scale 1/50)

Upstream bed level	... El. 6950
Downstream bed level	... El. 6955
Radius of bucket	... 25 feet
No. of spans represented	... 3
Shape of piers	... Semi-cirular (7' dia).
Upstream curve at crest	... Quadrant of a circle with 3 feet radius
Position of gate	... 3' from axis of spillway (Downstream)
Tail water control (Vide Figure-3 (a))	... Not maintained

4.4. Revised Profile No. 1 (Scale 1/50)

Upstream bed level maintained	... El. 6950
Downstream bed level maintained	... El. 6955
Radius of bucket	... 25 feet
No. of spans represented	... 3
Shape of piers	... Semi-circular (7' diameter)
Upstream curve at the crest	... As per Creager's Profile for lower happe.

A flat portoin of 3'0" width was introduced at the crest thus increasing the width of spillway at bottom.

Position of gate	... 5'0" downstream from Axis of spillway.
Tail water control (Vide Figure 3 (b))	... Not maintained.

4.5. Revised Profile No. 2 (Scale 1/50)

Upstream bed level maintained	... El. 6950
Downstream bed level maintained	... El. 6955
Radius of bucket	... 25 feet
No. of spans represented	... 3
Shape of piers	... Semi-circular (7 feet dia)
Upstream curve at the crest	... Circular arc of radius 5.3. feet.

The flate portion of 3'0" width as provided in the previous profile was increased to 4'6" consequently increasing the width of spillway still further.

Position of gate (Vide Figure 3 (c)	... 6'6" from Axis of spillway (downstream)

4.6. Revised Profile No. 3 (Scale 1/50)

The same particulars as in the previous profile (i.e. Revised Profile No. 2) were adopted except for the following change :—

The rear batter of the spillway was kept as 7H to 10V for this profile (as against 7.5H to 10V in the previous cases) thus decreasing the base width of the spillway by 4 feet (Vide Figure-3 (d)).

5. Experiments

The experiments were conducted on all the four profile sent by the Designs Office of the Board for the following conditions of flow:

(a) Free overflow of 9 feet and 3 feet over the crest.

(b) Controlled flow of gate openings of 1 foot, 2 feet, 3 feet and 6 feet with M. W. L. at El. 7039.

(c) The same experiments were repeated for 25% increase of head over the M. W. L. for the Revised Profile No. 3 alone.

6. Discussion of Results

6.1. Tentative Profile (Figure-3 (a))

From the statement No-1, it is seen that no negative pressure occurs on the surface of the profile for different free overflows over the crest. However, in the case of controlled flows by gate opening, very small negative pressure occurs in the parabolic portion of the Elevation 7029.5. It is seen that a maximum negative pressure of 1.00 foot occurs for 3 feet opening with M. W. L. maintained in the reservoir.

6.2. Revised Profile No. 1 (Figure (3))

From the statement No. 2 it is seen again that no negative pressure occurs on the surface of the profile for different heads of flow for uncontrolled flow. As in the case of the provisional profile, very small negative pressure occurs in the parabolic portion of the crest at the El. 7029.0 for gate controlled flow. The maximum negative pressure recorded is 0.75 foot of water for 3 feet gate opening with maximum water level in the reservoir. The negative pressures are found to increase for increased gate opening from 1 foot to 3 feet.

6.3. Revised Profile No. 2 (Figure 3 (c))

From statement No. 3 it is seen that negativepressures occur near the crest for free overflow conditions at the elevation 7029.60, 7029.0, 7028.0 and 7025.0 on the parabolic of the crest at El. 7012.0 on the downstream batter. The maximum negative pressure i.e. 1.5 feet of water occurs at El. 7028.0 for 9 feet free head over crest.

From statement No. 3 it is is seen that negative pressurese occur for different gate opening with M. W. L. in the reservoir at the following elevations.

+ 7029.50	On the parabolic portion of the crest	+ 7012.00 on the D/S profile	
+ 7029.00		+ 6967.00 in the bucket portion	
+ 7028.00			
+ 7025.00			

+ 7012.00 on + 6967.00 In the bucket portion.

The maximum negative pressure is 1.8 feet of water and the same occurred for 6 feet gate opening with maximum water level maintained in the reservoir.

The negative pressures increased with the increase of the free head over the crest. The same did not decrease for the increase of gate openings as is the usual case. This is perhaps due to the flat portion of 3′0″ introduced at the crest of the spillway.

6.4. Revised Profile No. 3 (Figure 3 (d))

From statement No. 4, it is seen that no negative pressure occurs on the surface of the profile for any head over the crest for Uncontrolled flow. From the same statement, it is also seen that very small negative pressures occur at the following elevations, for gate opening with M. W. L. maintained in the reservoir :

(1) El. 7029.00 near the crest.

(2) El. 6967.00 in the bucket portion.

The maximum negative pressure occurred is 0.5 foot of water at the elevation 6967.0 for 1 foot gate opening.

The profile has the maximum negative pressure of 0.5 foot of water and that too was fluctuating as positive and negative. Hence this profile can be taken as to be free from any negative pressure under vatious conditions operations.

6,5. Revised Profile No. 3 (Figure 3 (d))

(Special Test for 25% increase of design head)

From statement No. 5 it is seen that for the 25% increase of the design head depth of 9 feet, the negative pressurs are found to occur at the following elevations, for free flow condition.

El. 7029·0 El. 7028·0 El. 7027·0 El. 7025·0 El. 7023·0	On the parabolic region near the crest.

The maximum negative pressure of —1.0 foot of water occurs uniformly at elevations 7027.0, 7025.0 and 7023.0.

For the different gate openings with increased maximum water level maintained in the reservoir, the negative pressures are found to occur at the following elevations for controlled flow conditions:

El. 7029·0 El. 7028·0 El. 7027·0 El. 7025·0 El. 7023·0	Parabolic region near the crest.
El. 6991·0 El. 6988·0 El. 6982·0 El. 6967·0	Rear batter region.

El. 6955.0 Rear bucket region

The maximum negative pressure of —1.25 feet of water occurs at the elevation 7029.0 for the gate openings of 2 feet and 3 feet.

The negative pressure increase in magnitude as the free head of flow over the crest increases. But the negative pressures show the tendency to increase with increase in the gate opening, contrary to the usual trend. This may be due to a 3 feet wide flat portion introduced in the narrow crest of the spillway and the position of gate on the flat portion (i.e. 3′0″ downstream from the starting of the flat portion).

The relative performance of all the above mentioned profiles can be seen from the summary of the results furnished below :

SUMMARY OF RESULTS SHOWING MAXIMUM NEGATIVE PRESSURES OBTAINED FOR FREE OVERFLOWS OVER THE CREST

Sl. No	Description	Head over crest			Remarks
		9 feet	6 feet	3 feet	
1.	Provisional Profile	...	...	...	The pressures furnished are in feet of water in proto type.
2.	Revised Profile No. 1	...	...	...	
3.	Revised Profile No. 2	—1.5	—1.0	—0.2	
4.	Revised Profile No. 3	...	...	...	

SUMMARY OF RESULTS SHOWING MAXIMUM NEGATIVE PRESSURE OBTAINED FOR PARTIAL GATE OPENINGS WITH MAXIMUM WATER LEVEL MAINTAINED UPSTREAM

Sl. No.	Description	Gate Opening				Remarks.
		1 foot	2 feet	3 feet	6 feet	
1.	Provisional Profile	—0.75	—0.75	—1.00	—0.25	The pressures furnished are in feet of water in prototype.
2.	Revised Profile No. 1	—0.25	—0·5	—0.75	...	
3.	Revised Profile No. 2	—1.0	—1.5	—1.5	—1.8	
4.	Revised Profile No. 3	—0.5	—0.2	—0.2	...	

The pressures obtained over the Revised Profile No. 3 for different free overflow conditions are shown plotted as ordinates normal to the surface of the profile, in Figures 4 (a) 4 (b) and 4 (c).

Similarly, the pressures over the same profile for different gate openings with maximum water level maintained upstream are shown plotted as ordinates normal to the surface of the profile in Figures 5 (a), 5 (b), 5 (c) and 5 (d).

7. CONCLUSION

The profile of the Sandynallah spillway was revised thrice by the Designs Office of the Electricity Board, Experiments were conducted for all the four profiles sent by the Electricity Board. In view of the fact that the Revised profile No. 3 developed the least negative pressures, the same may be adopted for construction.

8. SUMMARY

A tentative profile for the Sandynallah spillway was first tested for distribution of pressures on the spillway surface for 9 feet, 6 feet and 3 feet free overflows and 1 foot, 2 feet, 3 feet and 6 feet gate openings with maximum water level maintained in the reservoir. This revealed a maximum negative pressure of —1.0 foot of water. This profile was successively revised three times by the Superintending Engineer Technical (Civil) of the Electricity Board. The revised profiles No. 1, 2 and 3 developed the maximum negative pressures of —0.75 foot of water, —1.8 feet of water and —0.5 foot of water respectively. The revised profile No. 3 developed the least negative pressures and hence the same has been finalised for adoption in the execution.

SANDYNALLAH RESERVOIR SCHEME

SANDYNALLAH SPILLWAY—PRESSURE STUDIES

PHOTOS

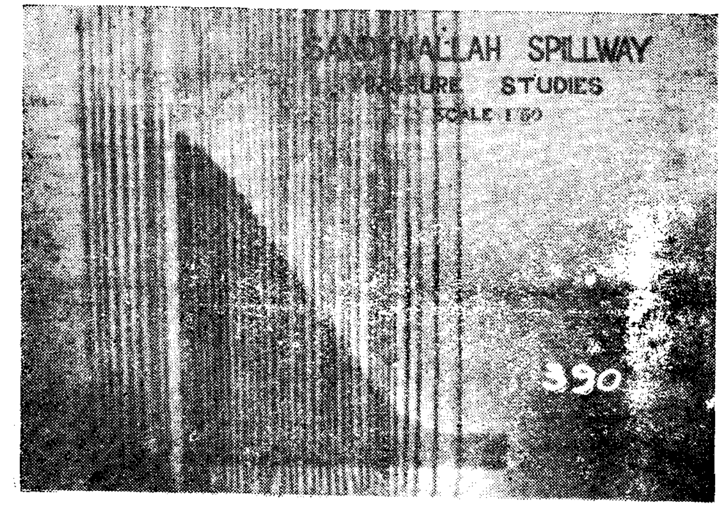


Photo No. 1

Manometric board showing the liquid columns in the piezometric tubes when the maximum uncontrolled flow of 9 feet head over crest was maintained.

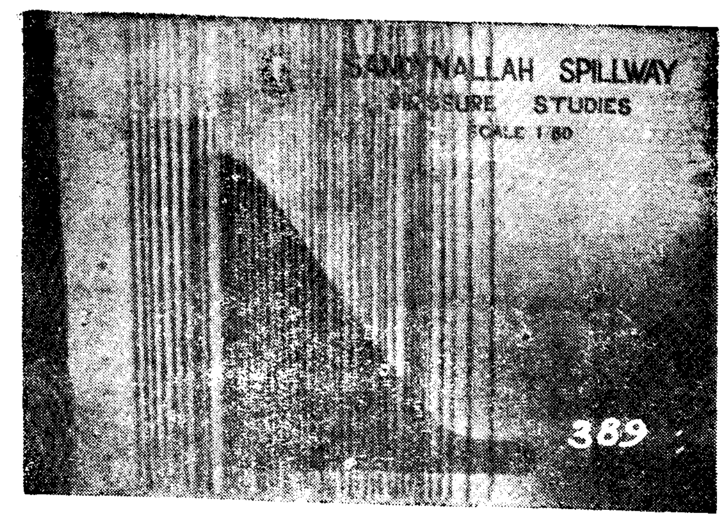


Photo No. 2

Manometric board showing the liquid columns in the piezometric tubes when the controlledflow of 1 ft. gate opening was maintained with water level upstream at Maximum water level.

Sandynallah Spillway

STATEMENT OF PRESSURES FOR VARIOUS PROFILES

STATEMENT I.

Figure 3 (a).

Provincial Profile Scale 1/50

Piezo meter.	Piezometer elevation.	Free flow over crest			Gate opening with M. W. L. upstream			
		3 feet.	6 feet.	9 feet.	1 foot.	2 feet.	3 feet.	6 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	7012.00	21.0	24.0	27.0	27.0	27.0	27.0	27.0
2	7016.00	17.0	20.0	23.0	23.0	23.0	23.0	23.0
3	7020.00	13.0	16.0	18.9	19.0	19.0	19.0	18.87
4	7023.00	10.0	13.0	15.8	16.0	15.87	16.0	15.87
5	7025.00	8.0	10.87	13.5	14.0	13.87	14.0	13.75
6	7027.00	6.0	8.75	11.0	11.50	11.75	11.75	11.25
7	7028.50	4.25	5.25	5.25	9.0	8.50	7.50	5.75
8	7029.25	3.50	5.50	6.62	9.25	9.25	8.50	7.00
9	7030.00	2.00	2.50	2.00	0.12	1.00	1.50	2.12
10	7029.75	1.75	2.00	1.50	0.25	0.12	0.25	1.36
11	7029.50	1.00	0.75	0.25	—0.75	—0.75	—1.00	—0.25
12	7029.00	1.36	1.50	1.00	0.75	0.50	0.25	0.75
13	7028.00	1.50	1.00	0.75	0.75	0.50	0.50	0.50
14	7027.00	1.00	1.00	0.75	0.87	0.75	0.50	0.50
15	7025.00	1.12	1.00	1.00	1.00	1.00	1.00	1.00
16	7023.00	1.12	1.25	1.75	1.25	1.25	1.25	1.50
17	7021.00	1.75	2.00	2.50	1.75	1.87	2.00	2.50
18	7018.00	1.25	1.50	2.50	1.50	1.50	1.75	2.25
19	7015.00	1.25	1.75	2.75	1.00	1.50	1.75	2.50
20	7012.00	2.00	2.75	3.50	2.12	2.50	3.00	3.50
21	7009.00	1.50	1.50	2.00	1.25	1.25	1.25	1.87
22	7006.00	2.00	2.50	3.25	2.00	2.25	2.50	3.00

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
23	7003.00	2.25	2.75	3.50	2.25	2.75	2.75	3.00
24	7000.00	1.75	2.00	2.75	1.75	2.00	2.00	2.50
25	6997.00	3.00	4.25	5.00	3.25	4.00	4.25	5.00
26	6994.00	2.50	2.75	3.25	2.50	2.75	2.75	3.00
27	6991.00	2.75	4.00	4.75	3.00	3.75	4.00	4.75
28	6988.00	1.75	2.50	3.00	1.75	2.12	2.50	2.75
29	6985.00	1.75	2.50	3.00	1.75	2.12	2.50	2.75
30	6982.00	1.50	2.00	2.50	1.50	1.87	2.00	2.50
31	6979.00	2.50	3.75	4.50	2.75	3.50	3.75	4.25
32	6976.00	2.50	3.25	3.75	2.50	3.00	3.25	3.75
33	6973.00	1.00	2.00	2.50	1.12	1.75	1.75	2.50
34	6970.00	2.50	3.75	4.50	2.50	3.50	3.75	4.25
35	6967.00	2.50	5.25	6.00	3.12	5.00	5.25	5.76
36	6964.00	0.75	2.50	3.25	0.75	2.00	2.25	2.87
37	6961.00	1.87	3.50	5.00	2.12	3.12	3.50	3.75
38	6960.00	2.50	6.25	7.50	3.00	5.12	6.00	7.12
39	6957.00	5.00	14.00	17.50	6.50	11.75	13.75	16.62
40	6955.00	4.00	10.25	14.75	4.50	8.12	10.25	14.00
41	6953.00	3.50	10.25	15.50	4.00	8.00	10.12	14.75
42	6951.00	4.25	13.50	12.25	5.00	10.12	13.50	18.75
43	6950.00	2.00	6.25	9.75	2.00	4.75	6.75	9.25

Sandynallah Spillway

STATEMENT OF PRESSURES FOR VARIOUS PROFILES

STATEMENT II

Figure 3(b).

Revised Profile No. 1 Scale 1/50.

Piezo meter.	Piezometer elevation.	Free head over crest			Gate opening with M.W.L. upstream			
		3 feet.	6 feet.	9 feet.	1 foot.	2 feet.	3 feet.	6 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	7012.00	21.0	24.0	27.0	27.0	27.0	27.0	27.0
2	7016.00	17.0	20.0	23.0	23.0	23.0	23.0	23.0
3	7020.00	13.0	16.0	19.0	19.0	19.0	19.0	19.0
4	7023.00	9.75	12.75	15.25	16.0	16.0	16.0	15.87
5	7025.00	4.75	6.50	7.75	14.00	13.75	13.50	12.00
6	7027.00	5.75	7.75	8.00	12.00	11.75	11.20	9.00
7	7028.50	2.75	3.75	3.75	7.00	6.25	6.25	5.25
8	7029.25	2.75	3.75	3.75	7.00	6.50	6.25	5.25
9	7029.25	2.75	3.75	3.75	7.00	6.50	6.25	5.25
10	7029.75	0.50	0.75	0.50	0.25	0.50	0.25	1.25
11	7029.00	1.00	1.50	1.25	—0.25	—0.50	—0.75	—0.12
12	7029.00	1.00	1.50	1.25	1.00	1.00	0.75	1.00
13	7028.00	1.25	1.25	0.87	1.00	1.00	0.75	1.00
14	7027.00	1.00	1.00	0.50	0.75	0.75	0.50	0.75
15	7025.00	0.75	0.75	0.50	0.75	0.75	0.50	0.75
16	7023.00	1.00	1.00	1.50	0.75	0.75	0.50	0.75
17	7021.00	1.50	2.00	2.75	2.00	2.00	2.12	2.50
18	7018.00	1.25	1.75	2.25	1.25	1.25	1.50	2.12
19	7015.00	1.00	1.50	2.25	1.25	1.50	1.75	2.50
20	7012.00	1.50	2.25	3.00	1.25	1.7[illegible]	2.00	2.75
21	7009.00	1.00	2.25	2.00	2.50	2.75	3.00	3.50
22	7006.00	1.50	2.00	2.75	1.75	2.00	2.00	2.50

23	7003.00	1.75	2.25	2.87	2.25	2.50	2.75	3.00
24	7000.00	1.50	1.75	2.12	1.00	1.00	1.75	1.75
25	6997.00	2.50	4.00	5.00	3.00	3.50	3.87	4.25
26	6994.00	2.00	2.75	3.10	2.75	3.00	3.00	3.50
27	6991.00	2.00	3.75	5.25	3.87	4.50	4.87	5.25
28	6988.00	2.75	4.75	5.25	3.87	4.50	4.87	5.25
29	6985.00	1.50	2.75	3.50	2.00	2.50	2.75	3.00
30	6985.00	1.00	1.25	1.75	1.75	2.00	2.50	2.87
31	6979.00	1.75	3.25	4.00	2.25	3.25	3.50	4.00
32	6976.00	2.00	3.50	4.00	2.25	3.00	3.00	3.50
33	6973.00	0.75	1.87	2.50	1.00	1.25	1.50	2.00
34	6970.00	2.00	3.50	4.00	2.50	3.00	3.50	4.00
35	6967.00	2 00	5.00	5.75	3.00	4.00	4.87	5.50
36	6964.00	0.50	2.50	3.25	0.75	1.50	1.50	2.25
37	6961.00	1.25	2.00	3.00	1.00	1.25	1.50	2.25-
38	6960.00	2.25	7.00	9.00	3.00	5.00	5.87	8.75
39	6957.50	3.50	12.75	16.50	5.75	9.50	13.25	16.25
40	6955.00	3.25	10.00	15.00	5.50	8.75	12.25	15.50
41	6953.00	2.87	9.50	15.00	4.25	7.00	10.50	14.75
42	6951.00	2.75	16.75	16.75	4.25	7.00	11.00	16.00
43	6950.00	1.25	9.25	9.25	2.00	3.25	5.25	8.50

SANDYNALLAH SPILLWAY

STATEMENT OF PRESSURES FOR VARIOUS PROFILES

STATEMENT - 3.

Revised Profile No. 2 Scale 1/50 Figure 3(c).

Piezo meter.	Piezometer elecation.	Free head over crest			Gate opening with M.W.L. upstream			
		3 feet.	6 feet.	9 feet.	1 foot.	2 feet.	3 feet.	6 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	7012.00	21.00	24.0	27.0	27.0	27.0	27.0	27.0
2	7016.00	17.0	20.0	23.0	23.0	23.0	23.0	23.0
3	7020.00	13.0	16.0	19.0	19.0	19.0	19.0	19.0
4	7023.00	10.0	13.0	15.8	16.0	16.0	16.0	16.0
5	7025.00	8.0	11.0	13.0	14.0	14.0	14.0	14.0
6	7027.00	6.0	8.5	10.2	12.0	12.0	12.0	10.0
7	7028.50	4.5	7.3	7.5	10.50	10.50	9.0	8.0
8	7029.25	3.55	5.25	6 75	9.75	8.95	7.75	6.0
9	7030.00	1.2	2.0	2.0	0.0	1.20	2.00	2.80
10	7029.75	1.5	1.25	1·25	0.25	0.25	0.45	1.45
11	7029.50	—0.5	0.5	—0.3	—0.50	—0.50	—0.50	0.3
12	7029.00	0.2	0.0	—0.2	—0.50	—1.00	—1.00	—0.5
13	7028.00	0.0	—0.2	—1.5	—1.0	—1.5	—1.5	—1.8
14	7027.00	1.0	1.0	1.0	1.5	1.00	1.0	1.0
15	7025.00	0.0	0.0	—1.0	0.0	—0.2	—0.2	—0.2
16	7023.00	0.0	1.0	1.0	1.0	1.0	1.0	1.2
17	7021.00	0.2	0.0	1.0	1.0	1.0	1.0	1.2
18	7018.00	0.8	1.0	1.8	1.0	1.0	1.0	2.0
19	7015.00	0.0	0.0	1.0	0.5	0.5	1.0	1.8
20	7012.00	—0.2	—1.0	0.0	—0.8	—1.0	—1.0	0.0
21	7009.00	0.0	1.0	1.2	0.2	0.5	0.8	1.0
22	7006.00	0.0	0.5	1.0	0.0	0.2	0.5	1.0

23	7003.00	0.5	1.0	1.5	0.8	1.0	1.0	1.0
24	7000.00	1.0	1.5	2.0	1.0	1.2	1.5	2.0
25	6997.00	0.5	1.0	2.0	0.8	1.0	1.0	1.5
26	6994.00	0.5	1.0	2.0	1.0	1.0	1.0	1.5
27	6991.00	0.0	0.2	1.0	0.0	0.0	0.0	0.5
28	6988.00	0.0	0.5	1.0	0.0	0.0	0.2	1.0
29	6985.00	0.0	0.5	1.0	0.0	0.2	0.2	1.0
30	6985.00	0.0	0.2	1.0	0.0	0.0	0.2	1.0
31	6979.00	0.5	1.2	2.0	0.5	1.0	1.0	1.5
32	6979.00	0.5	1.2	2.0	0.5	1.0	1.0	1.5
33	6976.00	1.0	1.2	1.5	1.0	1.0	1.2	1.5
34	6970.00	1.0	2.0	2.2	0.5	1.2	1.8	2.0
35	6967.00	0.0	0.0	0.2	-0.5	0.5	0.0	0.0
36	6964.00	0.0	1.0	2.0	0.0	1.0	1.0	2.0
37	6961.00	4.0	9.0	12.0	4.0	2.0	9.0	11.2
38	6957.00	1.0	3.5	7.5	1.0	2.0	4.0	8.0
39	6957.50	1.0	3.5	7.5	1.0	2.0	3.0	5.7
40	6955.00	0.0	5.0	10.0	10.2	2.0	4.0	8.0
41	6953.00	4.2	13.0	21.2	5.0	10.0	14.0	20.0
42	6951.00	1.5	7.2	13.0	9.0	4.0	6.0	11.0
43	6950·00	2.0	9.0	14.0	2.2	5.0	7.0	13.0

SANDYNALLAH SPILLWAY

STATEMENT OF PRESSURES FOR VARIOUS PROFILES

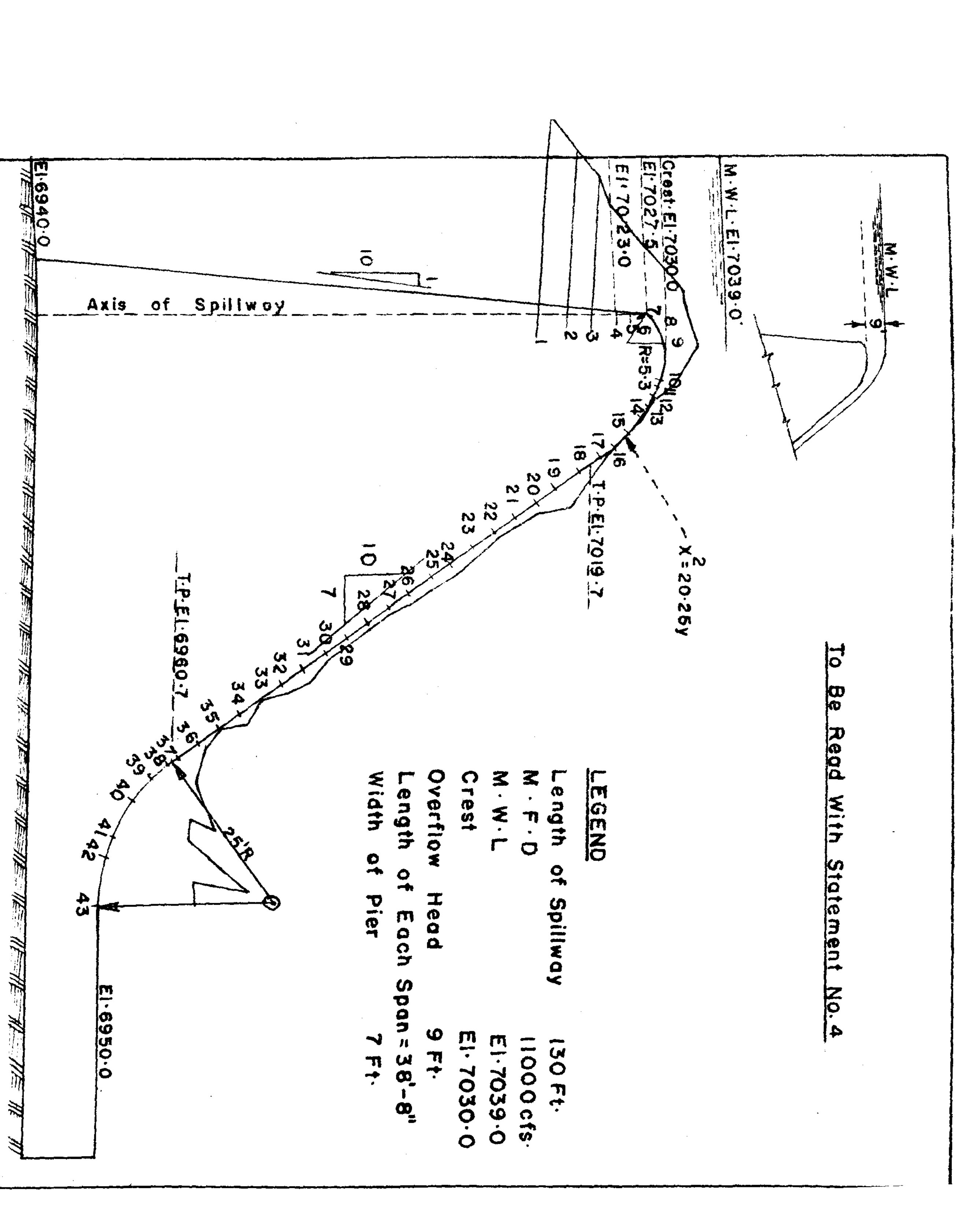
STATEMENT 5.

Figure 3 (d)

Revised Profile No. 3 Scale 1/50.

Piezo meter.	Piezometer elevation.	Free head over crest.	Gate open with water level at 7041·25			
		11·25 feet.	1 foot.	2 feet.	3 feet.	6 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	7012.00	29.25	29.25	29.25	29.25	29.25
2	7016.00	25.00	25.25	25.25	25.25	25.25
3	7020.00	20.75	21.00	21.00	21.00	21.25
4	7023.00	17.75	18.00	18.00	18.25	18.00
5	7025.00	15.50	16.00	16.00	16.00	16.00
6	7027.00	12.00	14.00	13.75	13.50	13.00
7	7028.50	7.00	12.5	12.25	12.00	10.25
8	7029.25	5.75	11.5	11.25	11.00	9.00
9	7030.00	5.00	10.75	10.5	10.00	8.00
10	7029.75	2.00	0.25	1.00	2.00	2.5
11	7029.75	1.25	0.0	0.0	0.5	1.25
12	7029.00	—0.50	—0.75	—1.25	—1.25	—0.75
13	7028.00	—0.75	0.0	—0.50	—1.00	—0.75
14	7027.00	—1.00	—0.25	—0.5	—1.00	—1.00
15	7025.00	—1.00	0.0	0.0	—0.25	—0.75
16	7023.00	—1.00	0.0	—0.25	—0.50	—0.75
17	7021.00	0.50	0.5	0.5	0.5	0.5
18	7018.00	2.50	0.5	1.0	1.25	2.00
19	7015.00	1.75	0.0	0.0	0.50	1.00
20	7012.00	2.00	0.25	0.75	0.75	1.25
21	7009.00	2.00	0.25	0.75	1.00	1.50
22	7006.00	2.00	0.25	0.75	0.75	1.25

23	7003.00	2.25	0.75	1.00	1.25	1.75
24	7000.00	2.75	1.25	1.75	2.00	2.50
25	6997.00	2.25	0.50	0.75	1.25	1.50
26	6994.00	2.25	0.75	1.00	1.25	1.75
27	6991.00	1.25	—0.25	0.0	0.25	0.75
28	6998.00	1.25	—0.25	0.25	0.25	0.75
29	6985.00	1.25	0.0	0.25	0.50	1.00
30	6982.00	1.75	—0.25	0.0	0.25	1.75
31	6970.00	2.50	0.5	1.25	1.75	2.00
32	6976.00	1.75	0.75	1.00	1.25	1.50
33	6973.00	2.00	0.5	1.00	1.00	1.5
34	6970.00	2.00	0.75	1.00	1.25	1.75
35	6967.00	2 00	—0.25	0.25	0.75	1.25
36	6964.00	3.50	0.75	1.5	2.00	2·75
37	6961.00	6.50	0.0	1.5	2.50	4.75
38	6960.00	13.75	4.00	8.00	9.75	1.75
39	6957.00	10.75	1.50	3.00	4.25	7.50
40	6958.00	10.75	—0.25	0.75	2.00	6.2[illegible]
41	6953.00	18.50	2.5	6.75	·9.25	13.75
42	6951.00	25.00	1.75	4.75	7.00	11.00
43	6950.00	26.00	2.50	6.25	9.50	13.25



Note:—For Figures 1 to 6, Refer Previous Item.

Annual Report of 1959	
23/30	7/13
Item No·	Drg·No·

Drawing 21/60	FREE OVER FLOW		
	J·P·Gajaraj[illegible] 27-1-60	[illegible] 29/1/60	P.K.[illegible]
	J E R	A·E·R	E·E·R
	Trd: V·V·	ckd:	FIG·7·

Reg. No.103E '62 Saka Era 1884) 375

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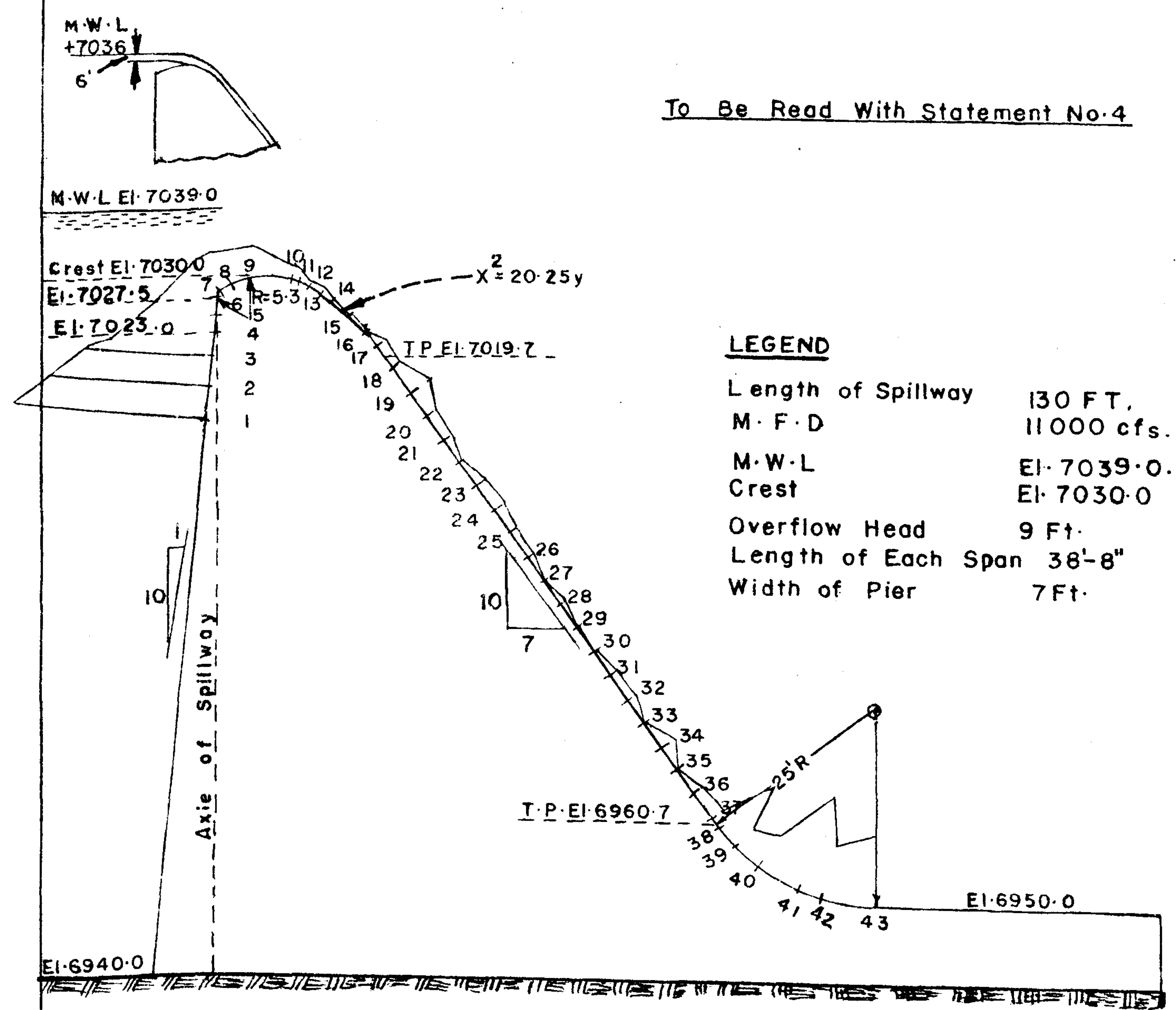

LEGEND

Length of Spillway	130 FT.
M·F·D	11000 cfs.
M·W·L	El·7039·0.
Crest	El·7030·0
Overflow Head	9 Ft·
Length of Each Span	38'-8"
Width of Pier	7 Ft·

Scale in Feet

20 0 20 40

Public Works Department Madras
Irrigation Research Station – Poondi

Sandynallah Reservoir Scheme
SANDYNALLAH SPILLWAY
PRESSURE STUDIES
PRESSURE ORDINATES FOR 6 Ft·
FREE OVER FLOW

Report / File / Drawing 22/60

K.P. Gajwatwade 27-1-60	9/1/60	B. L. Kandaswamy
J·E·R.	A·E·R.	E·E·R.
Trd. C.S.K.	ckd.	Fig. 8

Annual Report of 1959	
23/30	8/13
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Reg No104 E'62 (Saka Era 1884) 375

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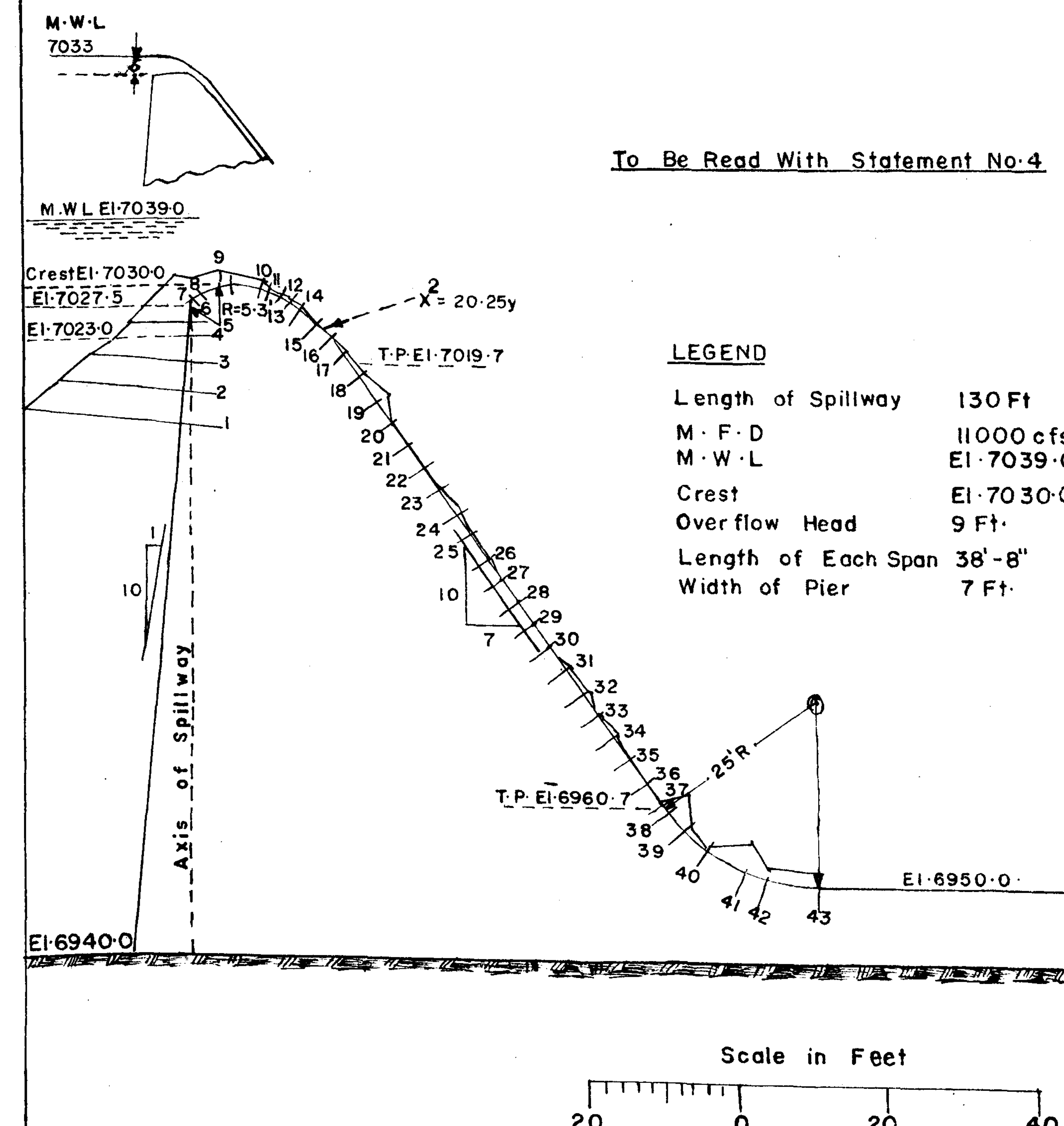

LEGEND

Length of Spillway	130 Ft
M·F·D	11000 cfs
M·W·L	El·7039·0
Crest	El·7030·0
Over flow Head	9 Ft·
Length of Each Span	38'-8"
Width of Pier	7 Ft·

Public Works Department Madras
Irrigation Research Station—Poondi

Sandynallah Reservoir Scheme
SANDYNALLAH SPILLWAY
PRESSURE STUDIES
PRESSURE ORDINATES FOR 3 Ft· FREE OVERFLOW

Annual Report of 1959	
23/30	9/13
Item No·	Drg·No·

Drawing File Report 23/40

K·P·Gajarathnam 27-1-61	[signature] 29/11/60	[signature]
J·E·R	A·R·O	E·E·R
Trd: c·s·k	ckd:	FIG· 9·

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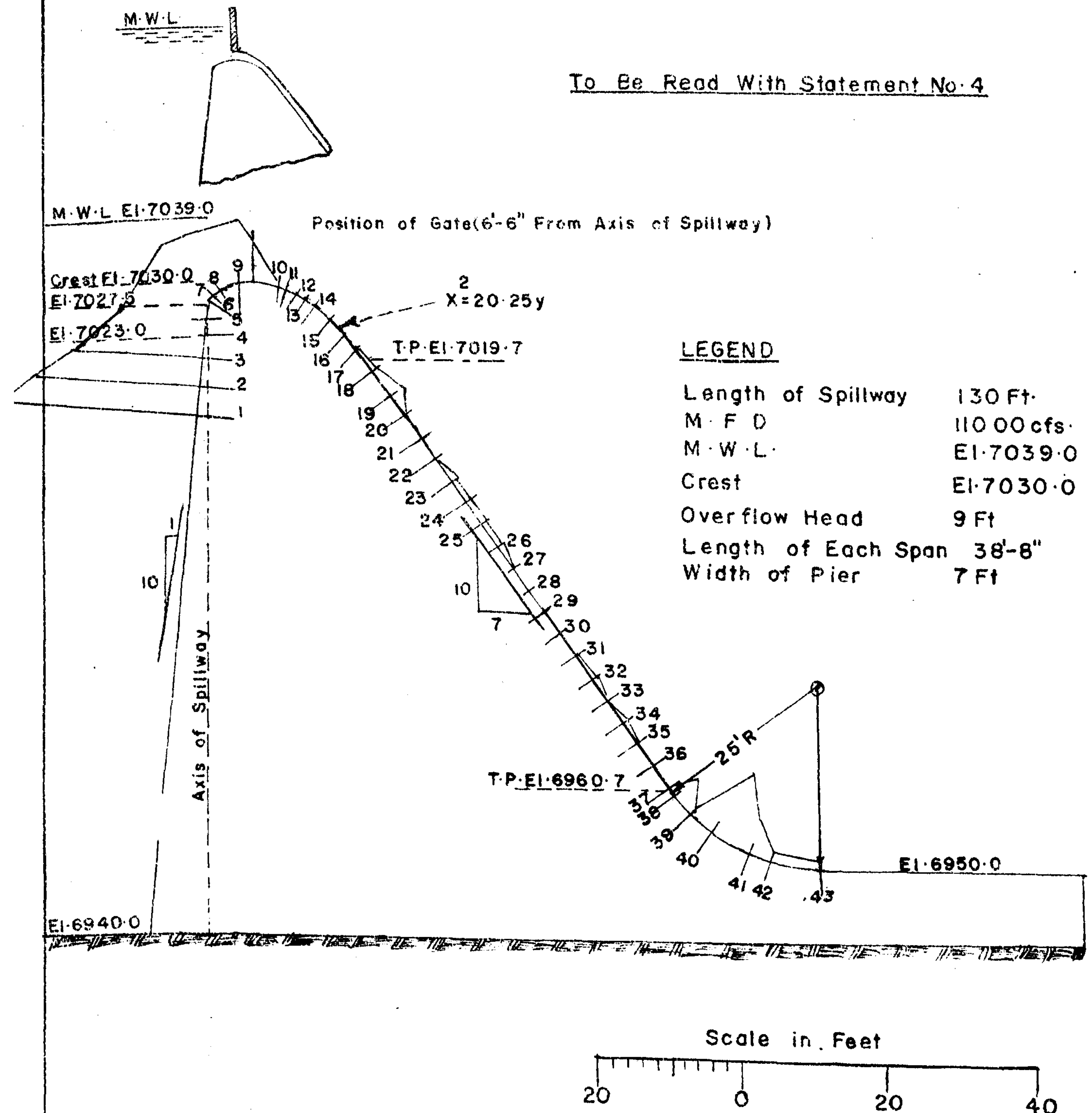

Report	Pulic Works Department — Madras Irrigation Research Station - Poondi
File	Sandynallah Reservoir Scheme SANDYNALLAH SPILLWAY PRESSURE STUDIES PRESSURE ORDINATES FOR 1Ft· GATE OPENING
24/60 Drawing	K.P. Gajavathan 27-1-60 / 29/1/60 / J.K. Kandaswamy
	J·E·R / A·E·R / E·E·R
	Trd: C·S·K / ckd: / FIG·10·

Annual Report of 1959	
23/30	10/13
Item No·	Drg·No·

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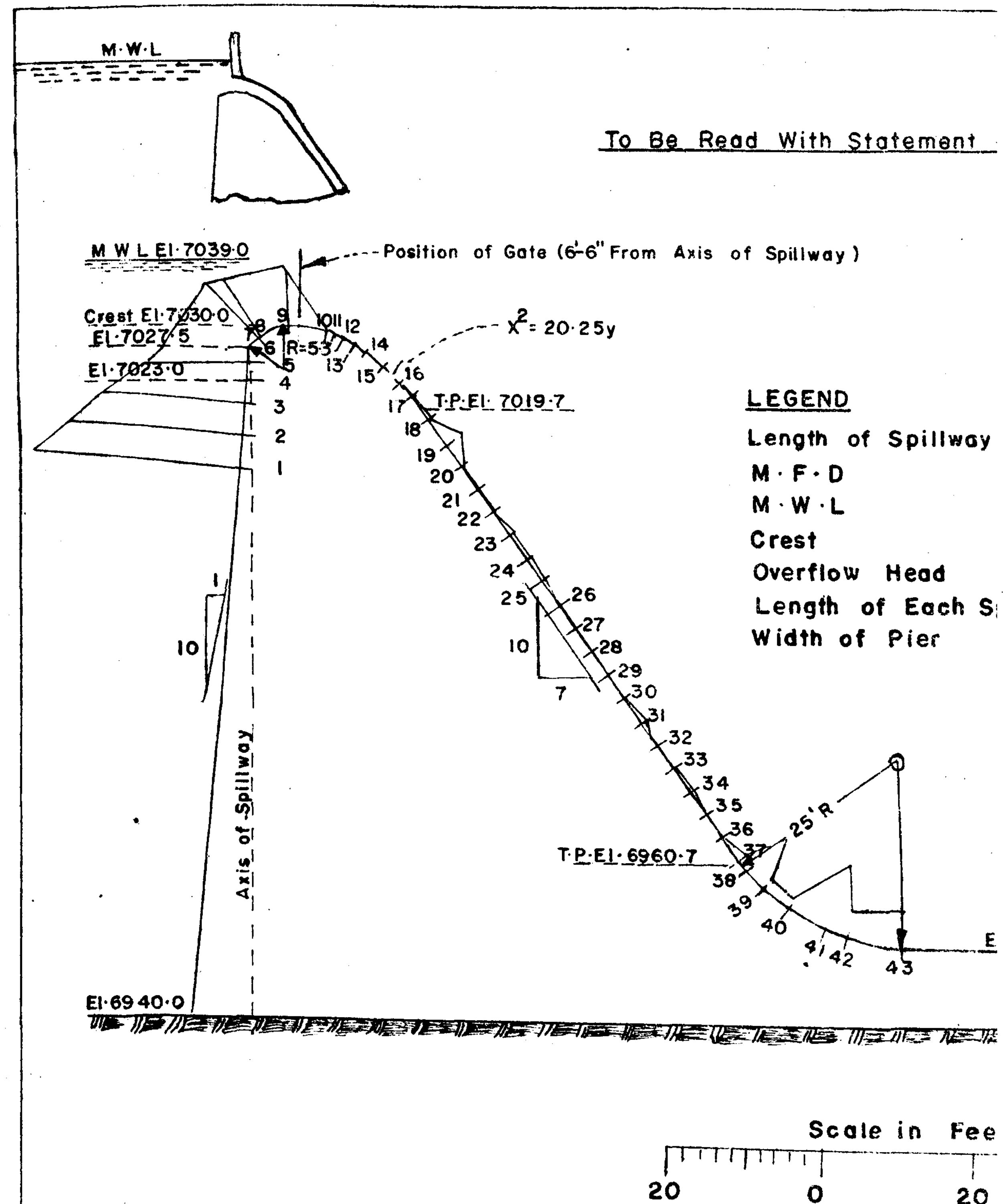

Public Works Departme
Irrigation Research St
Sandynallah Reservoir
SANDYNALLAH S
PRESSURE ST
PRESSURE ORDINAT
GATE OPEN

Annual Report of 1959	
23/30	11/13
Item No·	Drg· No·

Report | File | Drawing 25/60
J E R | A·E·R
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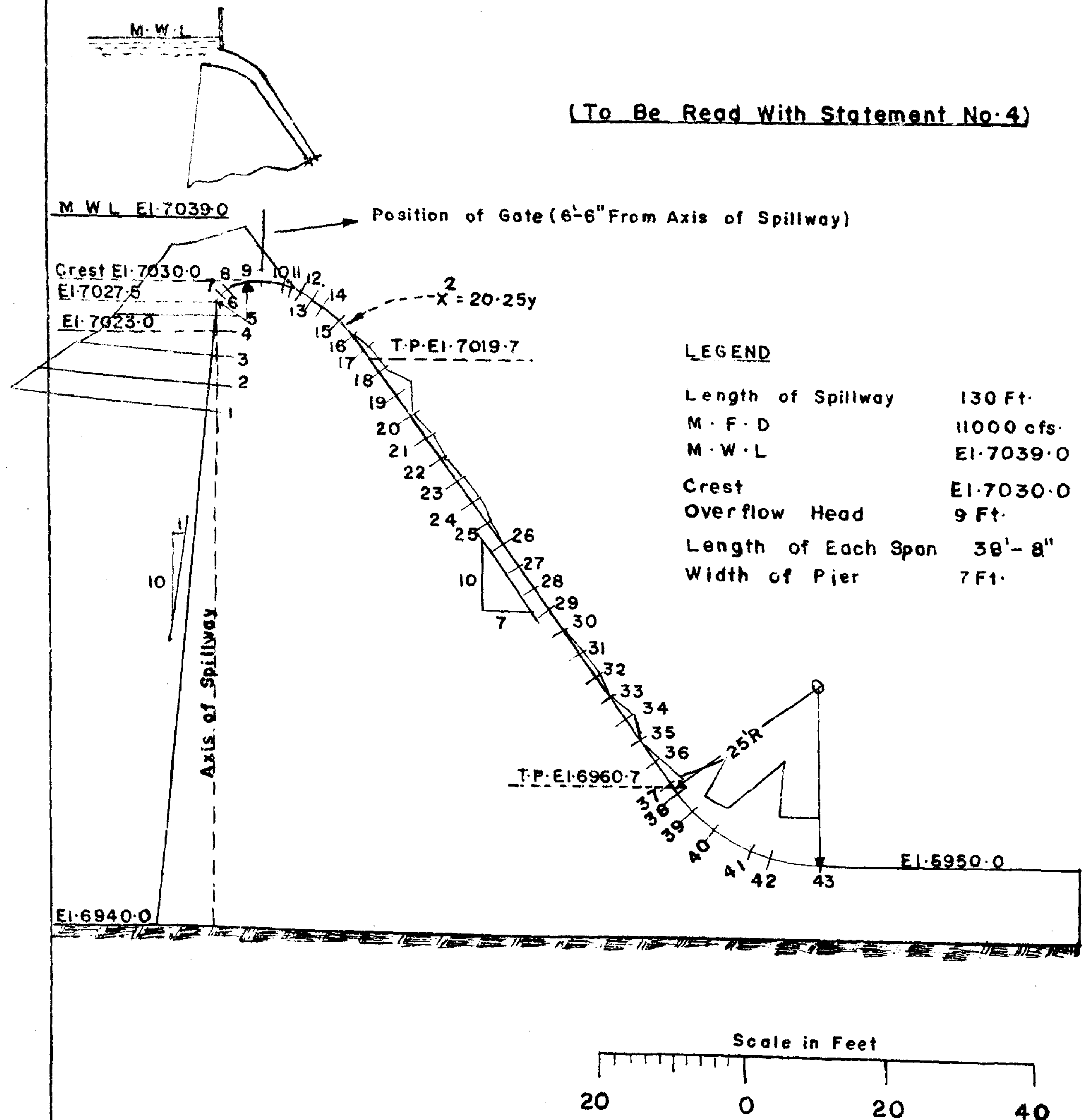

Public Works Department – Madras
Irrigation Research Station – Poondi

Sandynallah Reservoir Scheme
SANDYNALLAH SPILLWAY
PRESSURE STUDIES
PRESSURE ORDINATES FOR 3Ft· GATE OPENING

File Report
Drawing 26/60

J·E·R	A·E·R	E·E·R
27-1-60	29/1/60	
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Annual Report Of 1959	
23/30	12/13
Item No	Drg· No

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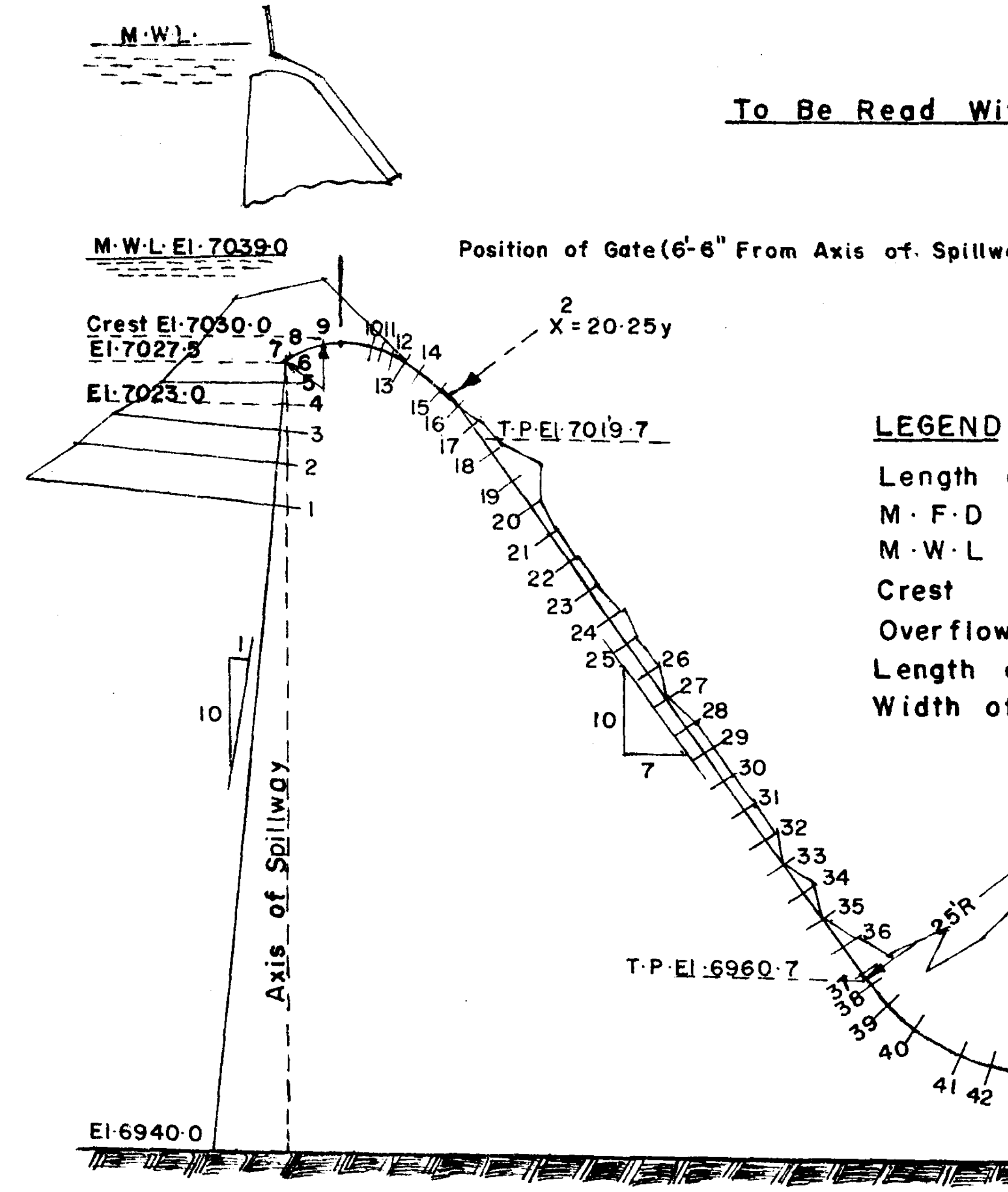

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Report | File | Drawing 27/60

K.P.Gajarathnaidu 27-1-60

J · E · R

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Annual Report. of 1959	
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Upper Bhavani Spillway

ENERGY DISSIPATION ARRANGEMENTS

The Upper Bhavani dam is an integral part of the Kundah Hydro-Eelctric Scheme which is now under execution in the Nilgris District. The dam will be 1,400 feet long at its top, 266 feet high and 21 feet wide at the top. The storage capacity of reservoir is 3,047 M.Cft with a catchment area of 12.8 Sq.miles. The spillway crest is 240 feet above the deepest foundation. The spillway portion of the dam consists of 2 spans of 25′0″ each with full pier 15′0″ thick at centre and two half and pires at the end. It is designed to surplus a maximum flood discharge of 16,500 cusecs with an overflow head of 20 feet (Figure-1) Studies were initiated in a scale model to evolve a suitable energy dissipating device at the toe of the spillway.

A sectional model has been constructed to a scale of 1/100 inside a parallel-side masonry flume 3 feet wide. The floor of the flume downstream of the model was sufficiently depressed so as to accommodate the mobile bed moulded with pergravel of size 3/8″ to 1/4″ for scour observations.

A three dimensional model to a scale of 1/100 has been constructed in a separate tray. The various appurtenances of the dam except the scour vent were represented in the model. The contours at the site for 600 feet downstream of the dam and 200 feet upstream of the dam were simulated in the model.

A suitable energy dissipator for a dam of 240 feet height with rocky bed and with comparatively low tail water level had to be evolved in the model study.

The conventional type of spillway energy dissipator worked out to be a costly affair involving enormous amount of rock cutting and concreting. Friction blocks for such a high spillway cannot be adopted for fear of abnormal negative pressures and consequent cavitation damages. The best type of spillway was therefore thought to be "trajectory Bucket-type energy dissipator". So a flip bucket of 60 feet radius was proposed to be studied in the model. Experiments for finalising the flip angle have been proposed to be conducted in the model for the observation of pressure along the bucket surface with particular reference to the flip and and for measuring the range and height of the projectile and the consequent scour action.

***Note*:—For Figures,**

Refer Next Item.

KUNDAH HYDRO ELECTRIC SCHEME

UPPER BHAVANI SPILLWAY - PRESSURE STUDIES

1. Synopsis

The spillway profile of the Upper Bhavani was tested for the distribution of pressures, under the conditions of controlled and uncontrolled heads of flow on the crest. The pressures that occurred on the surface of the profile for various operations are furnished in a statement and plotted on the profile for two representative cases.

2. Introduction

The Upper Bhavani dam is an integral part of the Kundah Hydro Electric Project which is now under execution in the Nilgris District. (The key plan and profile of the Kundah Hydro Electric Scheme is given in Figure-1). This dam will be built across the Bhavani river at its upper reaches. The dam will be 1400 feet long at its top, 266 feet high and 21 feet wide at the top (vide Figure-2 for the dam). The storage of the reservoir is 3,047 M. Cft. with a catchment area of 12·8 sq. miles. This reservoir is connected to the Avalanche reservoir through a tunnel of 9,740 feet long. The spillway crest is 240 feet above the deepest foundations, The spillway portion of the dam consists of 2 spans of 25′0″ each with one full pier 15′0″ thick at centre and two half piers at ends. It is designed to surplus a maximum flood discharge of 16,500 cusecs, with an overflow head of 20 feet. The spillway is to be operated by a vertical lift gate positioned at 5′6″ downstream of the axis of the spillway.

3. Data

Length of spillway including pier thickness	... 65 feet
No. and size of spans of spillway	... 2 Nos. each 25′ long
Pier thickness	... 15 feet
Maximum head over the crest	... 20 feet
Height of spillway above deepest foundation level	... 240 feet
Crest level of spillway	... El. 7,450.0
F. R. L. or M. W. L.	... El. 7,470.0
Deepest foundation level	... El. 7,210.0
Maximum estimated flood discharge	... 16,500 cusecs.

4. Problem

4.1. The problem is to find out the distribution of pressures along the spillway profile (vide Figure-3) received from the Superintending Engineer, Technical (Civil) the State Electricity Board.

4.2. As the location of the crest gate was predetermined by the Board perhaps for other consideration such as the convenience of operation etc., the necessity to find out the best location for the crest gate such that it gives only positive pressure for all conditions of gate opening did not arise.

5 Model

5.1. A sectional model to a scale of 1/100 was constructed in a separate flume of 3 feet width. Piezometers of 1/8″ bore copper tubes were embedded in the body of the spillway at salient points along the surface of profile such that one end of each tube was normal to the surface of the profile at that point. The other ends of the copper tubes were connected by means of suitable rubber tubes to a row of 1/8″ bore glass tubes fitted no a manometric board.

5.2. The manometric board was made of teakwood. The particular spillway profile, which was then under study, was painted on the board to the same scale of 1/100. The glass tubes referred to above were fitted on to this board vertically such that they indicate in the profile on the board, the points at which the copper tubes were embedded in the sectional model of the same profile on the board, the points at which the copper tubes were embedded in the sectional model of the same profile. The board was graduated and fixed in such a way that the crest of the model and the crest of the painted profie on the board were at the same elevation. By this arrangement, it was possible to read directly the magnitude, of the pressure occuring at each of the salient points for the various conditions of flow.

5.3. The 3 feet long crest of the model was divided in to three spans withtwo full piers in the middle and to half piers at the ends. This arrangement provided for three spans each of 0.85′ length (i. e. 85 feet prototype dimension) as against only two spans each of 25 feet length in the prototye. This dimensional disparity in the arrangement of the span does not affect the results of study as it is more convenient for conducting the studies and avoiding the 'scale- effect'.

6. Experiments

The experiments were conducted on the profile as sent by the Designs Office of the Board for the following conditions of flow:

(a) Uncontrolled heads of flow on crest such as 20′, 15′, 10′ 7′, 5′ and 3′.

(b) Controlled flow of gate openings of 15′, 10′, 7′, 5′, 3′, 2′ and 1′ with M. W. L. at El. 7470.0.

(c) The same experiments were repeated for 25% increase of the design head over the maximum water level.

7. Discussion of Results

7.1. From the statement of pressures it is seen that negative pressures occur on the surface of the profile for different uncontrolled Head of flow on crest at the following elevations.

El. 7440.0 El. 7436 0 El. 7484.8	Parabolic crest region.
El. 7420.0 El. 7380.0 El. 7370.0 El. 7360.0 El. 7350.0	Upper Region of rear Batter.
El. 7220.0	Bucket region.

The maximum negative pressure of—4.0 feet of water occurs at the elevation 7360.0for 20 feet, 15 feet, and 10 feet uncontrolled head of flows on crest.

7.2. From the same statement, it is seen that negative pressures occur for different gate openings with maximum water level in the reservoir at the following elevation :

El. 7450.0	Crest.
El. 7440.0 El. 7424.8	Parabolic Region.
El. 7420.0 El. 7380.0 El. 7360.0 El. 7350.0	Rear batter.

The maximum negative pressure of 4.0. feet of water occurs at the elevation 7360.0 for gate openings of 15 feet, 10 feet, 7 feet and 5 feet.

7.3. The variation of pressure on the profile of the spillway is on an average similar for different operations of controlled and uncontrolled heads of flow on crest. However, the magnitude of negative pressure occurring on the profile is maximum in the case of the maximum uncontrolled head of flow on crest i. e. 20 feet head over the crest with maximum water level in the pool. In the case of the controlled flows by gate opening, the lesser the gate opening, the larger are the average valus of the negative pressure and the mumber of points of their occurrence. The variation of the pressures are plotted diagramatically on the surface profile of the spillway in Figure 3 for the conditions of the maximum controlled head of flow of 20 feet on crest and the controlled head of flow of 1 foot gate opening with maximum water level in the reservoir.

7.4. SPECIAL TEST 25% INCREASE OF THE DESIGN HEAD OVER THE CREST.

From the statement of pressures it can be seen that the magnitude of the negative pressure, for the uncontrolled head of flow on crest increased by 25% has lessened instead of being the otherway. The maximum negative pressure of—3.0 feet of water occurs at the elevation 7440.0.

In the cose of controlled heads of flow on crest with the increased design head by 25% the magnitude of pressure was larger for the gate openings varying between 15 feet to 5 feet. The maximum negative pressure of —4.5 feet of water offurred at elevations 7440.0 and 7360.0 for 15 feet gate opening and 5 feet gate opening respectively with the pool elevation 7475.0.

8. Conclusion

The surface profile on the Upper Bhavani spillway was tested for various critical operations. The maximum of the negative pressures which were developed on the profile was only—4.0 feet of water, and it is within the allowable limits The spillway profile as sent by Board is found to be safe against negative pressure and hence the same is recommended for adoption.

PHOTOS

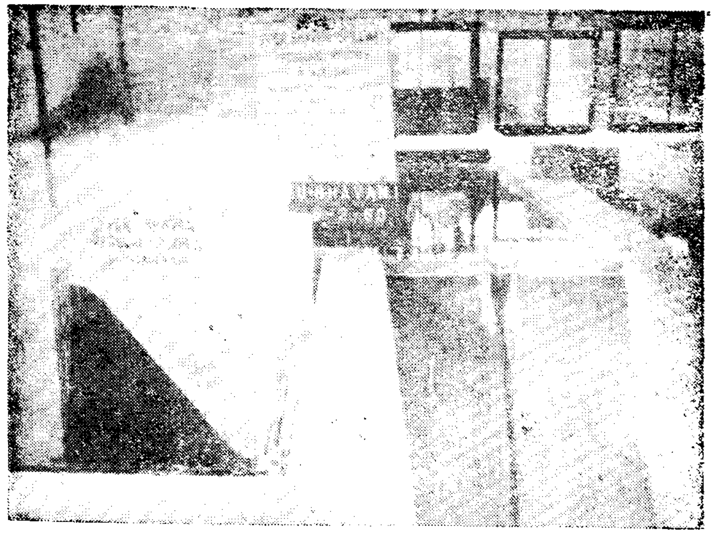


Photo No. 1
Central Model Set up.

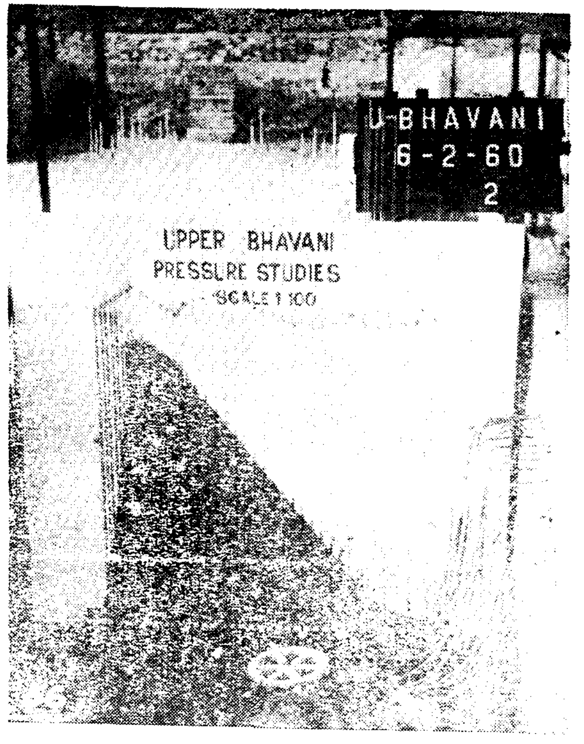


Photo No. 2
Manometric Board Reading when uncontrolled head of flow of 20 feet is maintained on crest of spillway.

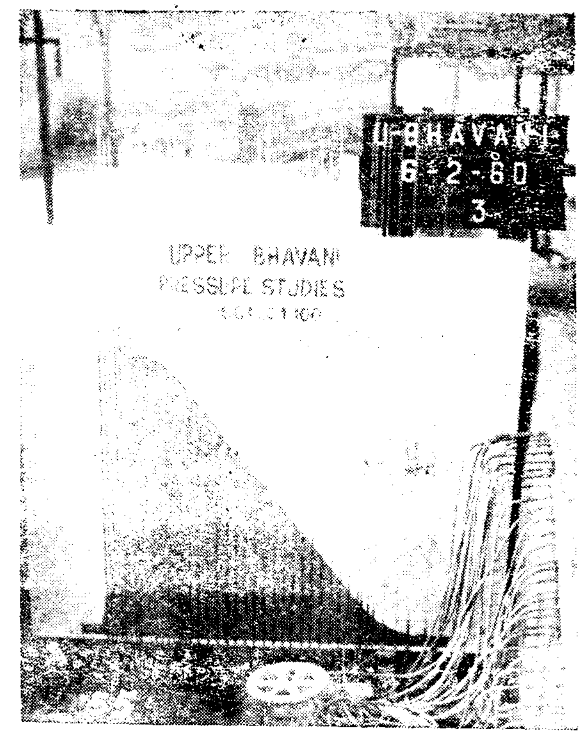


Photo No. 3

Manometric Board Reading when controlled head of 1 foot on crest is maintained by Gate operation.

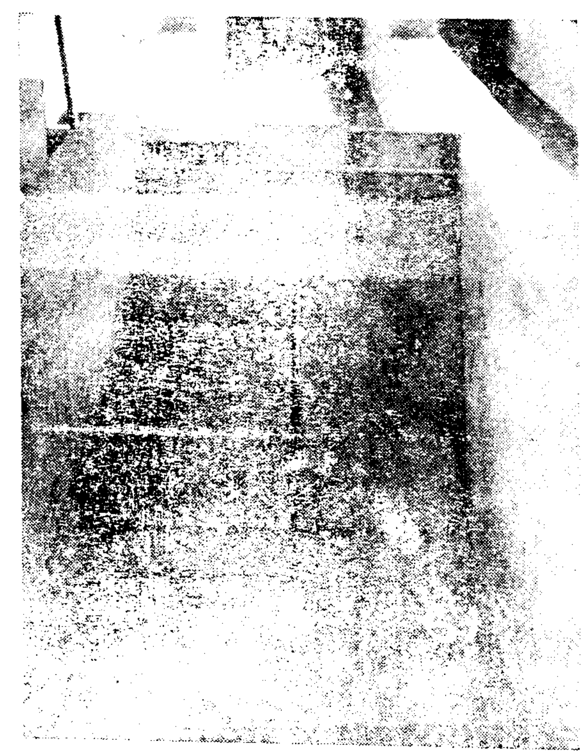


Photo No. 4

Sectional Model for pressure studies.

ATEMENT 1(a).

Piezo meter. No.	Piezometer elevation.	Uncontrolled head over crest					
		20 feet.	15 feet.	10 feet.	7 feet.	5 feet.	3 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	7410.0	60.0	55.0	50.0	47.0	45.0	43.0
2	7420.0	50.0	45.0	40.0	37.0	35.0	33.0
3	7430.0	40.0	35.0	30.0	27.0	25.0	23.0
4	7440.0	29.0	25.0	20.0	17.0	15.0	13.0
5	7447.0	19.0	17.0	12.0	10.0	8.0	6.0
6	7448.5	11.5	10.5	8.5	7.5	5.5	4.5
7	7450.0	6.0	8.0	7.0	5.5	3.5	2.0
8	7450.0	8.5	8.0	5.0	3.5	2.0	1.0
9	7449.0	4.5	4.0	3.0	2.0	1.5	1.0
10	7447.0	2.5	2.5	2.0	1.5	1.0	1.0
11	7444.0	1.0	1.5	1.5	1.0	0.5	0.5
12	7440.0	—3.0	—2.0	—1.0	—1.5	—1.0	—1.0
13	7436.0	0.0	0.0	0.0	—0.5	—0.5	—1.0
14	7432.0	4.5	3.0	2.0	2.0	1.0	0.5
15	7428.0	2.0	2.0	1.0	0.5	0.0	0.0
16	7424.8	2.2	1.2	0.2	—0.8	—0.8	—0.8
17	7420.0	2.0	1.0	0.0	—2.0	—2.0	—2.0
18	7410.0	3.0	2.0	1.0	1.0	1.0	1.0
19	7400.0	4.5	3.0	2.0	2.0	1.0	0.5
20	7390.0	3.0	2.0	1.0	1.0	1.0	1.0
21	7380.0	—3.0	—3.0	—3.5	—3.5	—3.0	—3.0
22	7370.0	—2.0	1.0	0.0	—1.0	—1.0	—1.0

STATEMENT 1(b).

Piezo meter. No.	Piezometer elevation.	Controlled head over crest with M.W.L. upstream (by Gate opening)						
		15 feet.	10 feet.	7 feet.	5 feet.	3 feet.	2 feet.	1 foot.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	7410.0	60.0	60.0	60.0	60.0	60.0	60.0	60.0
2	7420.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
3	7430.0	40.0	40.0	40.0	40.0	40.0	40.0	40.0
4	7440.0	29.0	29.5	30.0	30.0	30.0	30.0	30.0
5	7447.0	19.5	20.5	22.0	22.0	23.0	23.0	23.0
6	7448.5	12.0	15.5	17.5	18.5	20.0	21.0	21.5
7	7450.0	10.0	12.5	13.0	12.0	13.0	13.0	12.5
8	7450.0	9.0	6.0	5.0	3.5	1.5	0.5	—1.0
9	7449.0	4.0	2.0	1.0	0.0	0.0	0.0	0.0
10	7447.0	1.5	0.0	—1.0	—1.0	—1.0	0.0	0.0
11	7444.0	0.5	0.0	0.0	0.0	0.0	0.5	0.5
12	7440.0	—2.5	—3.0	—2.5	—2.5	—2.5	—2.5	—2.0
13	7436.0	0.0	0.0	0.0	0.5	0.0	—0.5	0.0
14	7432.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5
15	7428.0	2.5	2.5	2.0	2.0	2.0	1.5	1.0
16	7424.8	1.2	0.2	0.2	—0.3	—0.3	—0.3	—0.8
17	7420.0	0.5	0.0	—0.5	—1.0	—1.0	—2.0	—2.0
18	7410.0	4.0	3.0	2.5	2.0	2.0	2.0	2.0
19	7400.0	4.5	3.5	3.0	2.5	2.5	2.0	2.0
20	7390.0	2.5	2.0	1.5	1.0	1.0	1.0	1.0
21	7380.0	—3.0	—3.5	—3.5	—3.5	—3.5	—3.5	—3.0
22	7370.0	0.5	0.0	—0.5	—0.5	—0.5	—0.5	—0.5

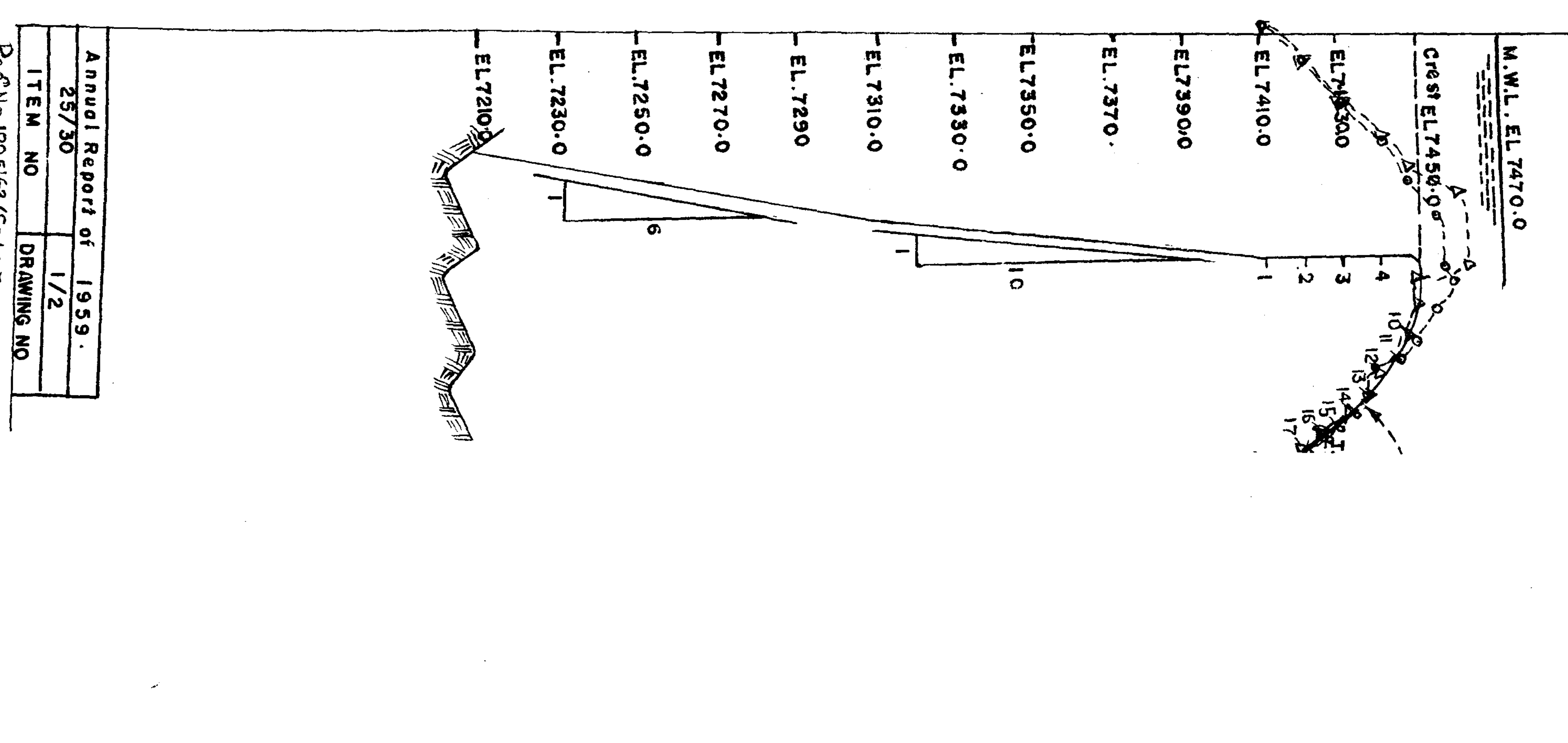
23	7360.0	—4.0	—4.0	—4.0	—3.5	—2.0	—1.0
24	7350.0	—2.0	—2.0	—2.0	—1.0	0.0	0.0
25	7340.0	5.0	4.0	3.0	2.5	2.0	1.0
26	7330.0	6.0	6.0	5.0	5.0	4.0	1.5
27	7320.0	1.0	0.5	0.5	1.0	1.0	0.5
28	7310.0	6.0	6.0	5.0	4.5	3.0	1.5
29	7300.0	6.0	4.0	4.0	3.5	2.0	1.0
30	7220.0	7.0	7.0	6.0	5.5	3.5	1.5
31	7280.0	6.0	6.0	4.5	5.5	4.0	1.5
32	7270.0	7.0	6.0	4.0	3.0	2.0	—0.5
33	7260.0	8.0	7.0	6.0	5.5	3.0	1.0
34	7250.0	8.0	6.5	5.0	6.0	3.0	1.0
35	7240.0	11.0	6.5	5.0	4.5	2.5	0.5
36	7237.15	13.35	9.35	6.85	5.25	3.35	1.85
37	7230.0	14.0	6.00	2.0	2.5	1.5	0.5
38	7226.0	12.0	6.5	1.0	3.0	1.0	0.0
39	7220.0	8.0	2.0	2.0	0.0	0.0	0.0
40	7214.5	14.0	8.0	2.5	0.5	0.0	—0.5
41	7211.5	17.5	10.5	3.5	2.5	1.0	0.0
42	7210.0	11.5	8.0	4.0	2.5	1.5	1.0

23	7360.0	—4.0	—4.0	—4.0	—4.0	—3.5	—3.0	—3.0
24	7350.0	—1.0	—1.5	—1.5	—1.5	—1.0	—0.5	0.0
25	7340.0	5.0	4.5	4.0	3.5	3.0	3.0	3.0
26	7330.0	6.0	6.0	6.5	6.0	6.0	5.5	3.5
27	7320.0	2.0	1.0	1.0	1.0	1.0	1.0	1.0
28	7310.0	6.0	6.0	6.0	6.0	5.0	4.0	3.0
29	7300.0	5.0	5.0	5.0	4.5	4.0	3.0	2.0
30	7290.0	7.0	7.0	7.0	7.0	6.0	5.0	3.0
31	7280.0	8.0	8.0	8.0	7.0	6.5	5.0	3.0
32	7270.0	6.0	6.5	6.0	7.0	4.5	3.0	1.5
33	7260.0	8.5	9.0	8.5	8.0	6.5	5.0	3.0
34	7250.0	10.5	10.0	9.0	8.0	6.5	5.0	3.0
35	7240.0	11.5	10.5	9.5	7.5	6.0	4.5	2.0
36	7237.15	15.35	13.85	11.85	9.85	7.35	5.85	3.35
37	7230.0	17.0	14.0	10.0	6.5	4.0	2.5	1.5
38	7226.0	16.0	12.0	8.5	6.0	4.0	3.0	1.0
39	7220.0	12.0	7.0	4.0	1.0	0.5	0.0	0.5
40	7244.5	14.5	9.5	6.5	2.5	1.5	0.5	0.0
41	7211.5	18.5	13.0	10.5	6.5	4.0	2.5	1.0
42	7210.0	10.5	10.0	7.0	5.0	4.0	3.0	1.5

STATEMENT 1(c).

Piezo meter. No.	Piezometer elevation.	Uncontrol head with 25% increase over designed.	Controlled head over crest with designed head increased by 25% (by gate opening)							
			20 feet.	15 feet.	10 feet.	7 feet.	5 feet.	3 feet.	2 feet.	1 foot.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	7410.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0	65.0
2	7420.0	55.0	55.0	55.0	55.0	55.0	55.0	55.0	55.0	55.0
3	7430.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0	45.0
4	7440.0	34.0	34.0	34.0	34.5	35.0	35.0	35.0	35.0	35.0
5	7447.0	23.0	23.0	24.0	25.0	27.0	27.5	28.0	28.0	28.0
6	7448.5	10.5	11.5	15.0	18.5	21.5	23.5	26.5	26.0	27.0
7	7450.0	6.0	5.0	13.0	15.5	16.0	16.0	16.5	16.5	16.5
8	7450.0	7.0	10.0	8.0	6.0	5.0	4.0	2.0	0.0	—1.0
9	7449.0	5.5	7.0	2.5	1.0	0.0	—0.5	—0.5	—0.0	0.0
10	7447.0	2.0	1.0	—1.0	—2.5	—2.0	—2.0	—1.0	—1.0	—0.5
11	7444.0	0.5	0.0	—1.5	—2.0	—1.0	—0.5	0.0	0.0	0.5
12	7440.0	—3.0	—4.0	—4.5	—4.0	—4.0	—3.5	—3.5	—3.0	—2.5
13	7436.0	—1.0	—0.5	—1.0	—1.0	—1.0	—1.0	—0.5	—0.5	0.0
14	7432.0	2.0	2.0	1.5	1.0	1.0	—1.5	1.5	1.5	1.5
15	7428.0	3.0	3.0	3.0	2.5	2.0	2.0	2.0	1.0	1.0
16	7424.8	3.2	2.2	1.2	0.7	0.2	—0.3	—0.8	—0.8	—0.8
17	7420.0	3.0	2.0	1.5	0.5	0.0	0.0	—1.0	—2.0	—2.0
18	7410.0	5.0	5.0	4.0	3.0	2.5	2.0	2.0	2.0	1.5
19	7400.0	6.5	6.0	5.0	4.0	3.5	3.0	2.5	2.5	2.0
20	7390.0	4.5	4.0	3.5	2.0	2.0	1.5	1.0	1.5	1.5
21	7380.0	—2.0	—1.0	—2.0	—3.0	—4.0	—4.0	—4.0	—3.5	—3.5
22	7370.0	3.0	2.0	1.0	0.0	0.0	0.0	—0.5	0.0	—0.5

23	7360.0	—2.5	—3.0	—3.5	—4.0	—4.0	—4.5	—4.0	—3.5	—2.0
24	7350.0	—1.0	0.0	—0.5	—1.0	—1.5	—1.5	—1.0	—0.5	0.0
25	7340.0	6.5	7.0	6.0	5.0	4.5	3.5	3.0	2.5	2.0
26	7330.0	6.0	8.0	7.0	7.0	6.0	6.5	6.0	5.5	4.0
27	7320.0	1.5	3.0	2.0	1.0	1.5	1.5	1.0	1.5	1.0
28	7310.0	7.0	7.0	7.0	6.0	6.0	6.0	5.0	4.5	3.0
29	7300.0	6.5	7.0	6.0	5.0	4.0	4.5	4.0	3.5	2.0
30	7290.0	8.0	8.0	8.0	7.0	7.0	7.5	6.0	5.0	3.0
31	7280.0	7.0	9.0	9.0	8.0	7.0	8.0	7.0	5.5	3.0
32	7270.0	8.0	7.0	7.0	7.0	5.0	5.0	4.5	3.0	1.0
33	7260.0	9.0	9.0	10.0	10.0	8.0	8.0	7.0	5.0	1.0
34	7250.0	8.0	12.0	10.5	11.0	9.0	7.0	7.0	5.5	3.0
35	7240.0	13.0	14.5	14.0	11.5	9.0	9.0	6.5	4.5	2.0
36	7237.15	16.85	18.85	17.35	14.85	10.85	8.85	...	6.35	3.85
37	7230.0	24.0	25.0	22.0	16.0	10.0	8.0	4.5	3.0	1.0
38	7226.0	21.0	23.5	20.0	14.0	8.0	7.0	4.5	3.0	1.0
39	7220.0	16.0	20.0	16.5	10.0	4.5	2.0	0.5	0.0	0.0
40	7214.5	18.5	21.5	18.5	12.5	7.5	4.5	1.5	0.5	0.0
41	7211.5	23.5	26.7	22.5	16.5	12.5	8.5	4.5	3.0	1.0
42	7210.0	14.0	14.5	12.0	11.0	8.0	6.0	4.0	3.0	2.0



Kundah Hydro Electric Project
Power House No. 2. - Tail Race Proposals

Report for the Design of Weir Profile and Calibration of Weir

1. Synopsis

This report contains the preliminary studies made on the tail race weir with an attempt to calibrate the same. A design for the Tail race for the Power House No. 2 of the Kundah Hydro Electric Scheme has been proposed by the Electricity Board.

The model of the tail race proposals revealed certain peculiarities unforeseen in the design out by the Electricity Board. The flow upstream of the weir was uneven and formed eddies. The different arrangements tried for improving the flow condition could not lead to a perfect solution. However, the profile of the weir has been revised and studied in the model ; the results of these studies have been presented in this report.

Studies would be continued for devising suitable energy dissipation arrangements at the toe of the channel togéther with a design for the chute and the same will form the subject matter for the final report on the problem.

2. Introduction

2.1. The Kundah Power House No. 2 forms part of the Kundah Hydro Electric Scheme which is under execution in Nilgris, vide Figure 1. The Power House No. 2 is situated lower down the Kundapalam Diversion Dam which is situated about 20 miles South West of Ooty. The Power House utilises a gross head of 2,470 ft. With an installed capacity of 140 M. W. and ultimate capacity of 175 M. W. Four penstock pipe lines feed unit 50,000 H. P. vertical, multiple jet-impulse wheels, 428 R. P. M., of capa 35,000 K. W. each. Provision has been made in the Power House for installing a 5th unit, at a later date. The Power House is built to accommodate five units ultimately and is 302 feet long 78 feet wide and 50 feet high above generator floor.

2.2. The outlet discharges from the units are proposed to be collected in a broad (247′9″) but very short (i. e. 25′0″ in the direction of flow) tail race pool and surplussed over a measuring weir situated 55 feet downstream of the end sills of outlet conduits of the units. The weir 40′0″ wide is situated across an approach channel (rectangular) of 40′0″ width constricted from the tail race pool (vide Figure 2(a) and 2(b). The surplused water is led along a fairly steep chute and allowed to join the Pegumbuhallah stream which is flowing at right angles to the flow direction of the chute. The difference in elevation between the crest of weir and the bed of the stream is about 87.5 feet (vide Figure 3 and 4).

The problem embaraces the following three aspects:

(1) To study the hydraulic performanee of the weir and improve the flow conditions for accurate calibration of the same.

(2) To design the steep channel as a chute spillway as regards its hydraulic performance.

(3) To evolve a suitable energy dissipating arrangement at the foot of the chute spillway.

This report deals with the first aspect alone.

3. Data

Maximum discharge over weir and channel 5 x 200 c/s from each unit	... 1000 c/s
Length of weir	... 40 feet
Crest level of weir	... El. 2847.5
Sill of discharge conduit of unit	... El. 2848.0
Maximum water level in Tail race pool	... El. 2852.0
Slope of Tail race pool from unit 1 to 5 in 200 Maximum water level when forebay is formed across the Pegumbuhalla stream for future Power Development lower down	... El. 2852.0
Total fall from crest of weir to the bed of Pegumbuhalla stream	... 87.5 feet.

4. Model

4.1. In the construction of this model the simulation of the discharge from the outlet conduits alone was taken as the criterion for judging the flow pattern over the weir. This was done by regulating the required model discharge through the model section of the conduit tube. In Figure 5, showing the plan of the model, from a common pond water is supplied to the 5 chambers, B_1 B_2 B_3 B_4 and B_5 which are separate chambers (with interconnection between themselves at bottom) connected to the outlet conduit chambers I, 2, 3, 4, and 5 separately. From the chambers B_1, B_2, B_3, B_4 and B_5 measured quantities of water pass over V notches C_1, C_2, C_3, C_4 to another and C_5 to another set of chambers D_1, D_2, D_3, D_4 and D_5 respectively. The head developed in these chambers produce the necessary velocity and discharge through the outlet conduits. The water from these conduits flow into the pool and finally over the weir. The surplussed water flows down the chute channel and takes a right-angled bend at the toe of the chute, through a side channel at the end of which a V notch was fixed for measuring the discharge passing over the weir. In the outlet conduits, the inlet and exit conditions have been simulated as in the prototype. Since the discharge passing through the conduits is measured and kept constant and the shape and size of the conduits are reproduced to scale, it is to be taken that the flow petterns in the rear of the conduits will be as in the prototype.

4.2. As the weir profile as sent by the designs office of the Electricity Board is more or less like a non-overflow section of a dam, it is felt imperative to study the suitability of the profile and suggest smooth parabolic profile based on pressure studies also. Hence piezometeric points were tapped by

means of 1/8″ bore copper tubes along the centre line of the weir profile, the circular curve and some portion of the chute slope. The copper tubes were connected by means of rubber tubes to 1/8″ bore glass tubes attached over a manometric board over which the profile of the weir was painted to the same scale as the model (vide Photo in Figure-5). The board has been so set in position as to have any point of the profile at the same elevation as the corresponding point on the model. Care has been taken to see that the piezometric tappings are normal to the surface of the profile. The manometric board has been graduated to scale so that pressure readings can be directly read off from the board.

4.3. MODEL SCALES

Linear	... 1/20
Velocity	... $1/20^{1/2}$ = 1/4.472
Discharge	... $1/20^{5/2}$ = 1/1788 8

4.4. In the construction of the model other accessories like pipes, valves etc., for regulation and gauge pot for measurement of discharge over V-notches were provided.

5. Experiments

5.1. When either the 1st unit or the 5th unit alone was functioning there was deep swirl on one side of the approach channel upstream of the weir and the head over the weir was not unifrom but undulating. However, for other conbination of operations of units also, there was some slight swirls and undulating head over the crest.

The undulating head over the crest for various operations of units were gauged and digramatically shown in Figure-6 (vide photo in Albumn-A).

5.2. It was first thought that the flow pattern in the approach channel would be improved by lengthening the same upstream by adoption of arrangement as shown in Figure 7. The flow pattern was studied by incorporating the above arrangement in the model.

5.3. As the above arrangement did not improve the flow pattern, honey-comb like gratings 40′ wide and covering the weir length were tried by installing across the mouth of the approach channel (vide Figure-8).

5.4. As the flow over the original profile of the weir was not moothly-guided downstream and the consequent turbulence on the chute was undesirable, it was felt imperative at this stage of study to evolve a profile over which the sheet of water may pass smoothly. Hance a profile based on Creager's co-ordinates of lower nappe was incorporated in the model and the experiments to improve the flow pattern upstream of weir, were continued.

5.5. The circular curve in the retaining wall at the mouth of the approach channel was changed to an elliptical transition as shown in Figure-9. To improve the flow further gratings as shown in Figure-8 C) were installed at the exit of the discharge conduits in an alighment circular in plan (vide Fignre-9). Again a small regulating weir was fixed upstream of the main weir (i.e. at Ch. 80) throughout the length of the common tail race pool (vide photo in Fig. 10). The flow over the weir for this arrangements were gauged. (vide Figure-10 for flow profile on weir and Photo in Albumn-B).

5.6. In addition to the arrangement described in para 5.5., another circular grating as shown in Figure 8-a) was fixed at the approach channel (vide Figure 11 and photos in albumn C) and experiments repeated.

5.7. During the course of the studies, the water levels in the common tail race pool were recorded for a few modifications tried. They are given in Statement 1. Also the maximum water levels in the outlet conduits of the units were recorded for the arrangements described in paras 5.5. and 5.6. They are furnished in Statement II.

5.8. PRESSURE STUDIES ON THE ORIGINAL PROFILE (SENT BY THE ELECTRICITY BOARD - VIDE FIGURE (12))

Experiments were conducted for different depth of free overflows on the weir crest by different operation of the units (vide Figure-12) for piezometric arrangements and the statement of pressures).

5.9. PRESSURE STUDIES ON THE IMPROVED PROFILE OF WEIR
Experiments were repeated as described in para 5.8. (vide Figure-13) for the piezometric arrangements and the statement of pressures).

6. Discussion

6.1. From Figure-6, it is seen that depths of flow over the crest for operation 1(a) and 2(a) are depressed at the left side of the weir and right side of the weir respectively. The upstream condition of the above depression can be seen in photos 1(a) and 2(a) of Albumn A. This is due to the unsymmetry of the positions of the discharging units with reference to the position of the weir for the above conditions of operation. For other operations of the units described above, the non-unifrom overflow on crest, is due to the reason enunciated above and also due to sudden turn of flow along the sidewalls of the tailrace pool at the mouth of the approach channel to the weir.

6.2. The experiment described in para 5.2. further constricted the width of the already narrow tail race pool at the mouth of the approach channel (vide Figure-7) and hence the non-uniformity in the flow was observed to a greater degree.

6.3. Experiments with straight honeycombs (vide Figure-8) fixed at the mouth of the approach channel made the flow over the weir still worse. The level in the common tail race pool rose upto El. 2853.0 for the grating shown in Figure 8(b) and just above El. 2853.0 for the grating shown in Figure 8(a). This induced supercritical flow and secondary waves in the approach channel and over the weir.

6.4. By the incorporation of the Creager's profile for the weir, the flow on the chute was smoothly guided. This is, achieved, because the air pocket between the surface of the original profile and the lower surface of the sheet of flow on weir has been obviated by the incorporation of the revised profile (Figure 13).

6.5. From Figure-10, it is observed that the unevenness of flow on the weir crest in spite of improved profile exists. But the relative depression of the flow on crest for the same conditions of flow is slightly minimised. The function of the long weir proposed to be located in the tail race pool is to correct the unsymmetrical flow and make it approach the calibration weir as nearly straight as possible. However, the sharp turnings immediately upstream of the calibration weir are inevitable and so the flow conditions are yet to be corrected satiscactorily.

STATEMENT I.

Water level in Tail Race Pool.

	Place where gauged.	All five units functioning. (1)	Unit 1 alone not functioning. (2)	Unit 1 and 2 not functioning. (3)	Unit 1, 2 and 3 not functioning. (4)	Unit 1, 2, 3 & 4 not functioning. (5)	Unit 5 alone not functioning. (6)	Units 5 and 4 not functioning. (7)	5, 4 & 3 not functioning. (8)	Units 5, 4, 3 and 2 not functioning. (9)
Tail race pool as per E.B's. design and without any modification to improve flow of weir.	Right corner opposite Unit 5.	2,851.0	2,850.76	2,850.22	2,850.02	2,849.3	2,850.52	2,849.8	2,849.42	2,948.72
	Left corner opposite Unit 1.	2,851.78	2,851.1	2,850.30	2,849.56	2,849.1	2,851.5	2,851.36	2,850.84	2,850.0
ail race pool as per Fig. 9 For the grating at the approach channel.	Right corner opposite Unit 5.	2,851.6	2,850.86	2,850.62	2,850.40	2,849.38	2,850.8	2,850.18	2,849.66	2,848.96
	Left corner opposite Unit 1.	2,852.30	2,851.2	2,850.3	2,849.7	2,848.94	2,852.16	2,852.06	2,851.44	2,849.76
Tail race pool as per Figure-9.	Right corner opposite Unit 5.	2,851.70	2,851.70	2,851.2	2,850.92	2,849.56	2,851.54	2,850·70	2,850.0	2,849.08
	Left corner opposite Unit 1.	2,852.64	2,851.50	2,850.70	2,850.08	2,849.04	2,852.38	2,852.08	2,851.32	2,849.8

Maximum allowable water level in the tail race pool—El. 2,852.00

STATEMENT II.

Water level in outlet conduits.

	Outside conduits of units				
	Unit. 1	Unit. 2	Unit. 3	Unit. 4	Unit. 5
A. The Tail Race pool as per Figure-9 ...	2,853.28	2,853.04	2,852.84	2,852.70	2,852.88
B. Tail Race pool as per Figure-9, except for the grating at the approach channel ...	2,852.94	2,852.72	2,852.74	2,852.5	2,852.6
C. Tail Race pool as per Figure-9, except for gratings at approach channel and at conduit sills and regulating weir ...	2,851.82	2,851.46	2,851.36	2,851.08	2,851.20

Maximum allowable water level in the Tail Race Pool — El. 2,852.00

Pie zometer number.	Pressures in feet of water in prototype					Piezometer Elevation.
	All units functioning.	Units 1, 2, 3 and 4 functioning.	Units 1, 2 and 3 functioning.	Units 1 and 2 functioning.	Unit 1 alone functioning.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	2.2	2.2	2.1	2.0	1.4	2847.3
2	0.1	0.5	0.8	...	0.9	2847.5
3	...	0.3	0.6	0.6	0.5	2847.0
4	—0.4	—0.1	0.1	0.2	0.2	2846.5
5	0.3	0.4	0.5	0.5	0.4	2845.5
6	0.1	0.2	0.2	0.2	0.2	2844.5
7	0.3	0.3	0.3	0.3	0.2	2844.0
8	0.4	0.4	0.4	0.3	0.3	2842.5
9	0.3	0.3	0.3	0.3	0.2	2842.5
10	0.2	0.2	0.2	0.2	0.2	2841.5
11	0.2	0.2	0.2	0.2	0.2	2840.5
12	0.1	0.1	0.1	0.1	0.2	2939.5
13	0.3	0.2	0.2	0.2	0.2	2837.5
14	0.6	0.5	0.5	0.3	0.3	2837.5
15	...	—0.1	...	—0.1	—0.1	2836.5
16	0.4	0.3	0.4	0.3	0.2	2836.0
17	0.7	0.6	0.4	0.3	0.2	2835.0
18	0.8	0.8	0.5	0.4	0.1	2834.0
19	0.19	1.9	1.4	0.7	0.4	2833.0
20	—0.1	...	—0.4	—0.6	—0.6	2832.0
21	2.3	2.3	1.8	1.5	1.0	2831.0
22	2.6	2.5	2.2	1.7	1.3	2830.0
23	4.0	3.8	3.4	3.1	2.6	2829.0
24	2.9	2.7	2.2	1.8	1.4	2828.0
25	4.8	4.6	4.1	3.6	3.0	2827.0
26	—0.6	—0.9	—1.3	—1.5	—1.5	2826.0
27	0.8	0.4	—0.1	—0.6	—0.6	2825.0
28	3.0	2.7	2.3	1.7	1.3	2894.0
29	3.7	3.3	2.6	2.1	1.3	2823.10
30	0.3	...	—0.3	—0.5	—0.5	2822.5
31	—1.2	—1.7	—1.6	—1.5	—1.2	2821.5
32	—0.1	—0.1	—0.3	—0.4	—0.3	2820.5
33	1.1	1.0	0.8	0.6	0.4	2819.5
34	1.6	1.5	1.2	1.1	0.8	2818.5
35	0.5	0.2	0.1	...	—0.1	2817.5

Number.	Co-ordinates of Profile (Calculated for 4.5 ft. head) x'	y'
1	0	0.194
2	0.45	0.045
3	0.90	0
4	1.35	0.023
5	1.80	0.104
6	2.7	0.405
7	3.6	0.869
8	4.5	1.5
9	5.4	2.25
10	6.3	3.15
11	7.65	4.725
12	9.0	6.615
13	11.25	10.530
14	11.5	11.8

ALBUM-1.

For the Tail race pool and Weir profile as seen by Electricity Board

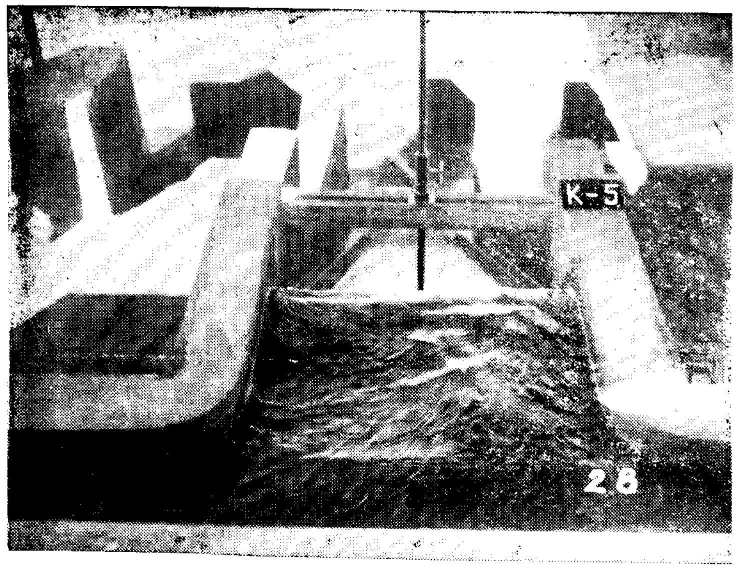


Photo No. 1(a)
Flow pattern upstream of weir when unit 1 a lone is functioning.

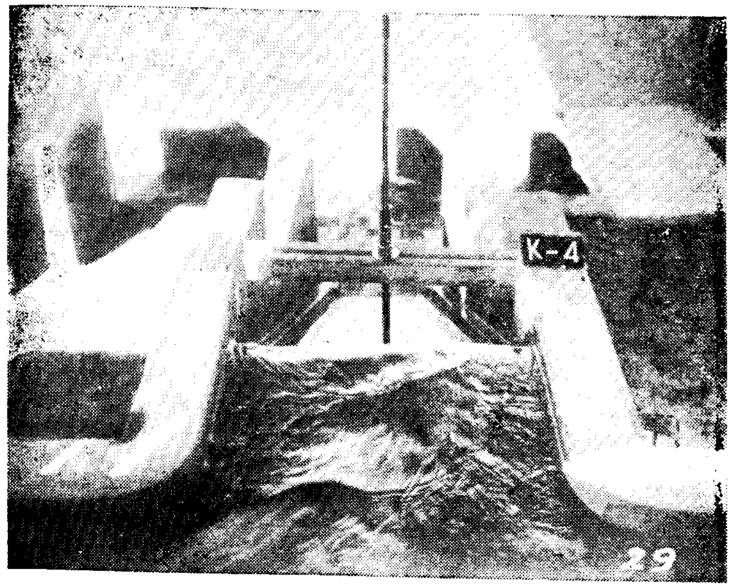


Photo No. 1(b)
Flow pattern upstream of weir when units 1 and 2 are functioning.

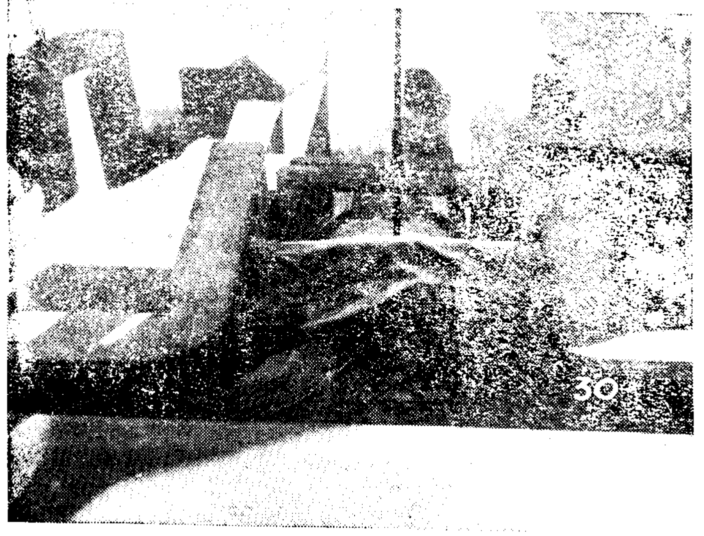


Photo No. 1(c)
Flow pattern upstream of weir when units 1, 2 and 3 are functioning.

Photo No. 1(d)
Flow pattern upstream of weir when units 1, 2, 3 and 4 are functioning.

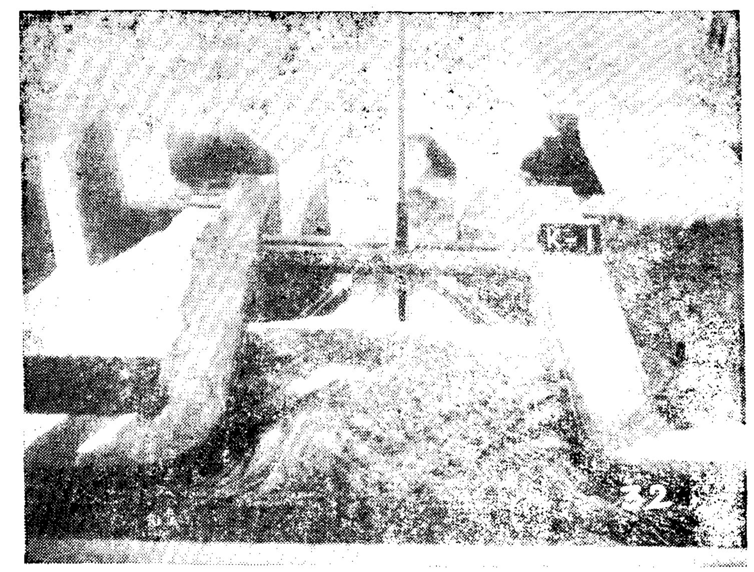


Photo No. 1 (a)
Flow pattern upstream of weir when all the five units are functioning.

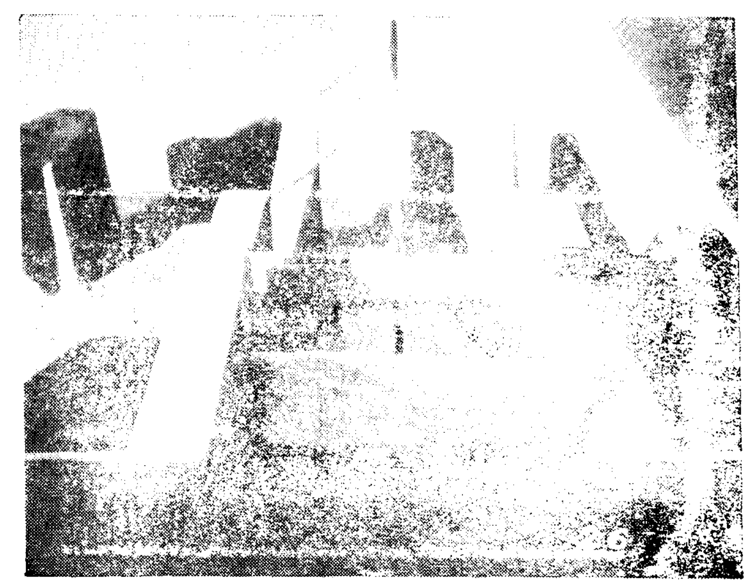


Photo No. 2(a)
Flow pattern of weir when unit 5 alone is functioning.

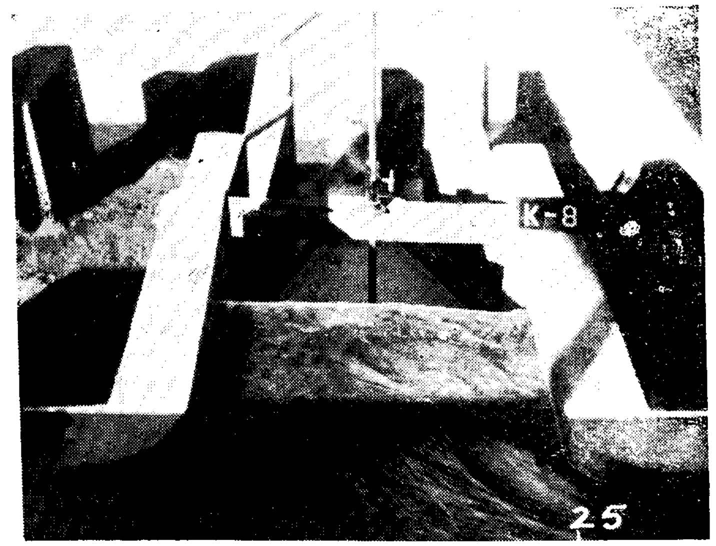


Photo No. 2 (b)
Flow pattern upstream of weir when units 5 and 4 are functioning.

ALBUM-B

For the Improved weir Profile and Modified Exit conditions.

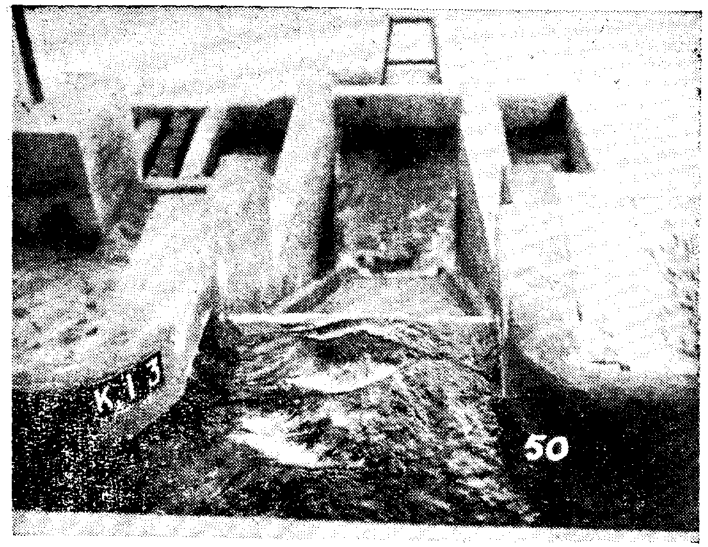


Photo No. 1(a)
Flow pattern upstream of weir when unit 1 alone is functioning.

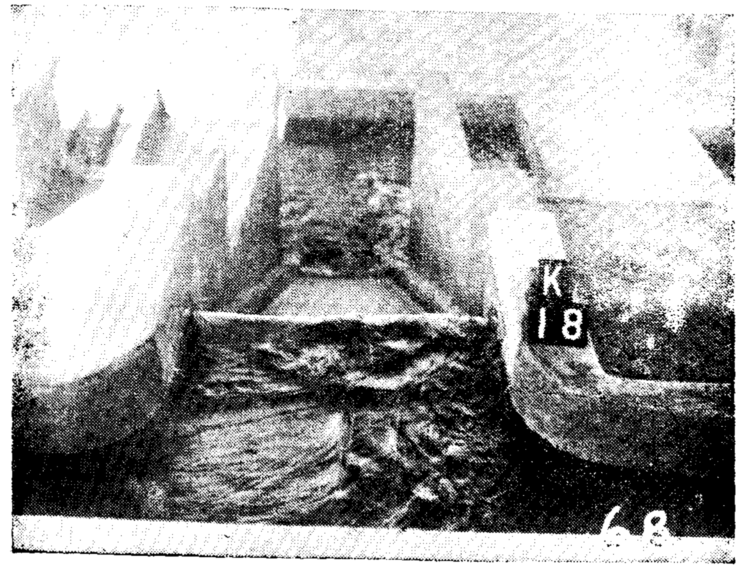


Photo No. 1(b)
Flow pattern upstream of weir when units 1 and 2 are functioning.

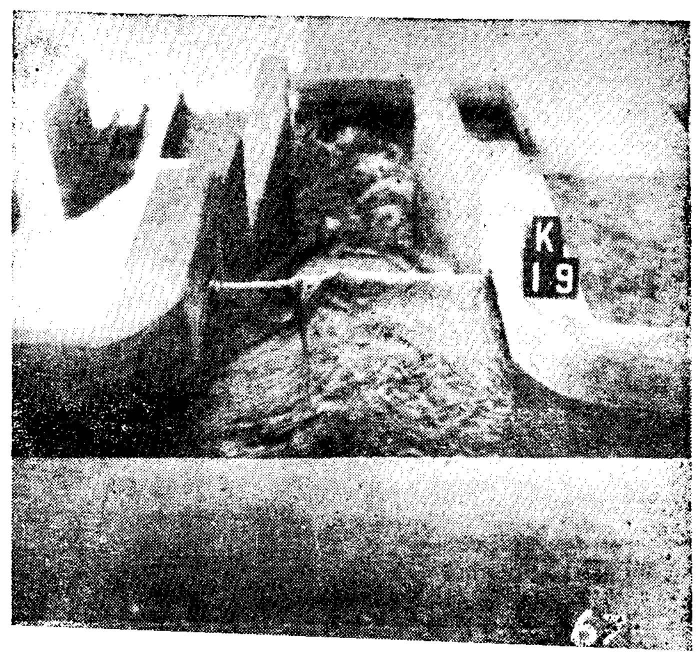


Photo No. 1(c)
Flow pattern upstream of weir when units 1, 2 and 3 are functioning.

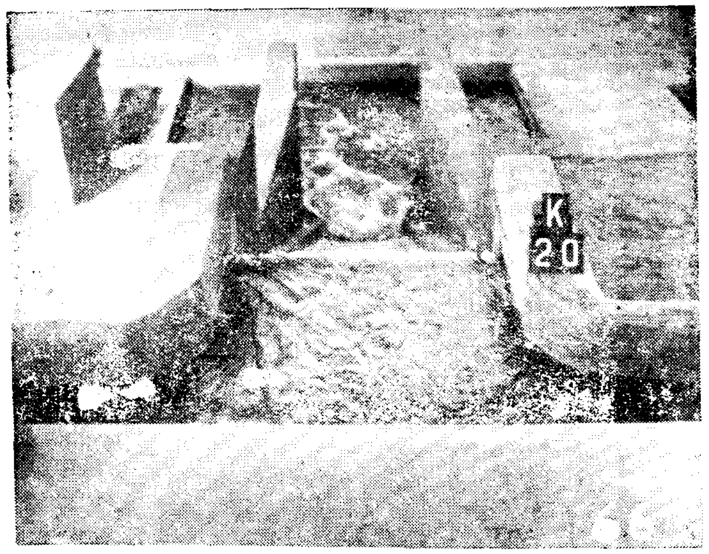
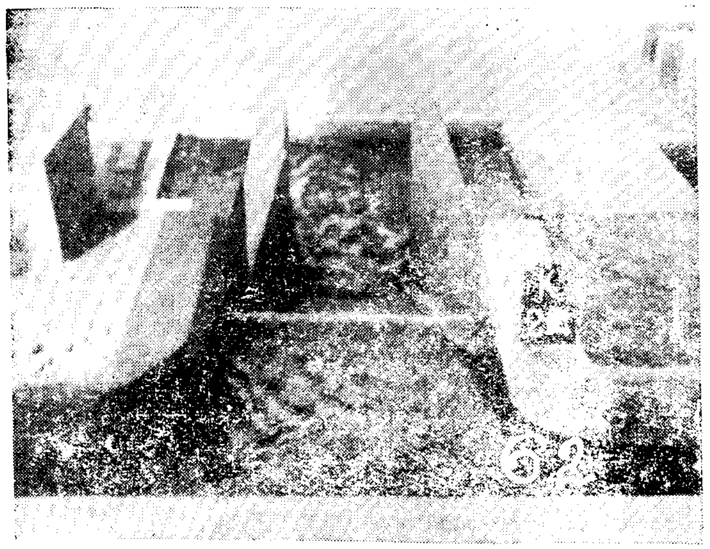


Photo No. 1(d)
Flow pattern upstream of weir when units 1, 2, 3 and 4 are functioning.



Pnoto No. 1(e)
Flow pattern upstream of the weir when all the five units are functioning.

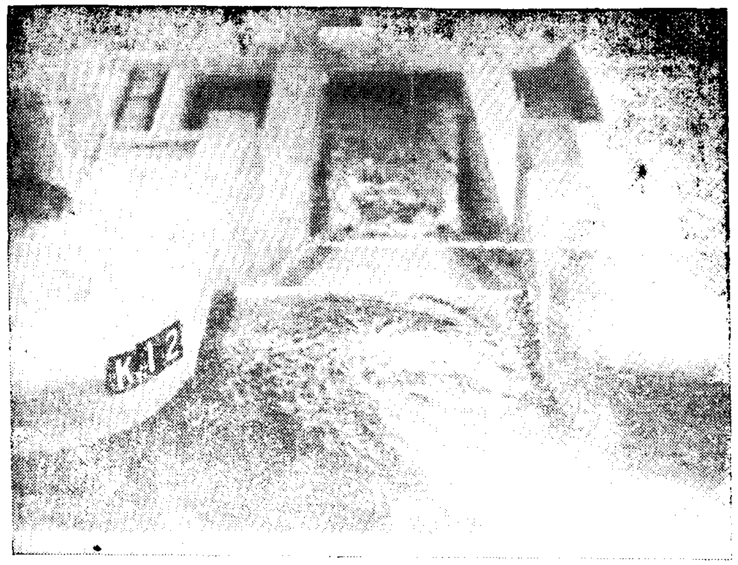


Photo No. 2(a)
Flow pattern upstream of weir when unit 5 alone is functioning.

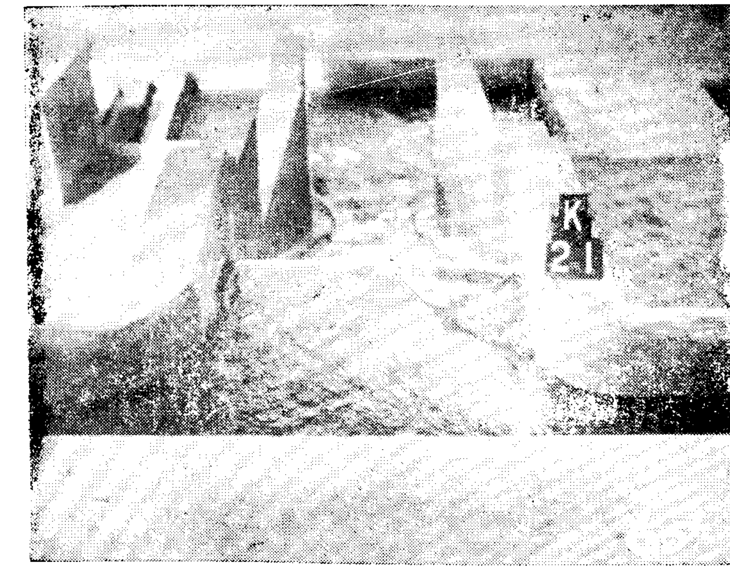


Photo No. 2(b)
Flow pattern upstream of weir when units 5 and 4 are functioning.

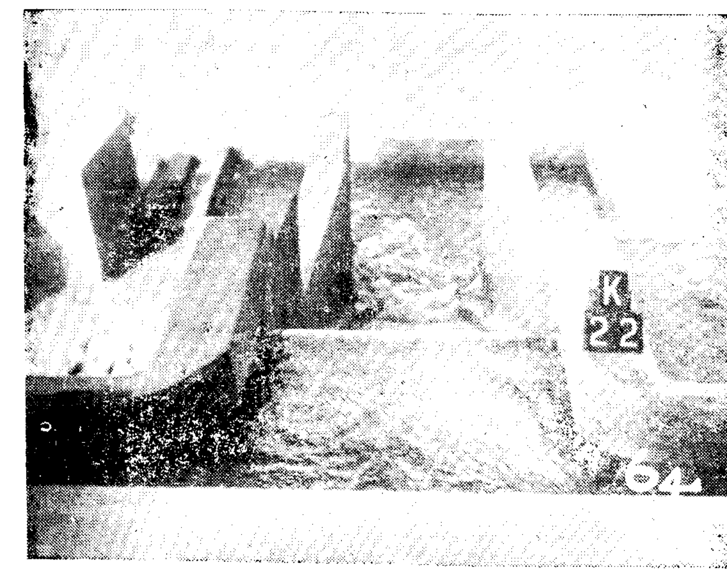


Photo No. 2(c)
Flow pattern upstream of weir when units 5, 4 and 3 are functioning.

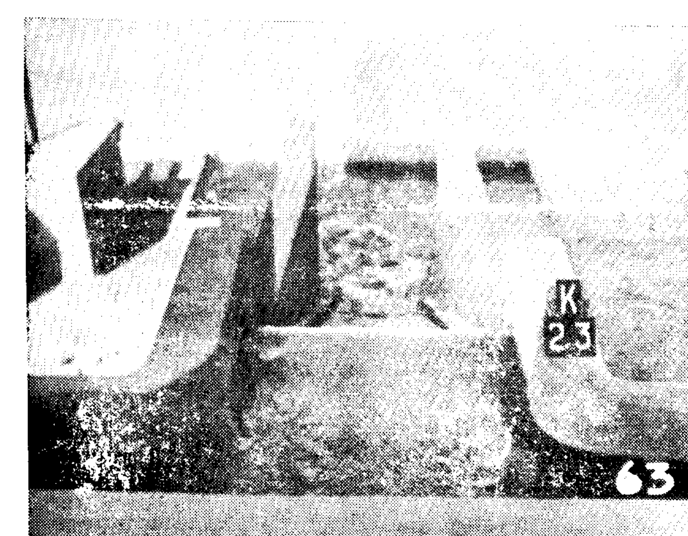


Photo No. 2(d)
Flow pattern upstream of weir when units 5, 4, 3 and 2 are functioning.

ALBUM-C

For the Improved Weir Profile and Modified Exit and Approach Conditions

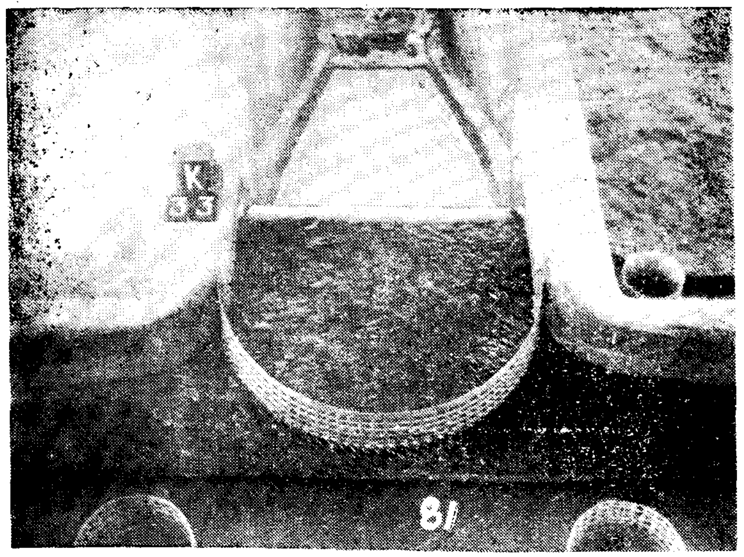


Photo No. 1(a)

Flow pattern upstream of weir when unit 1 alone is functioning.

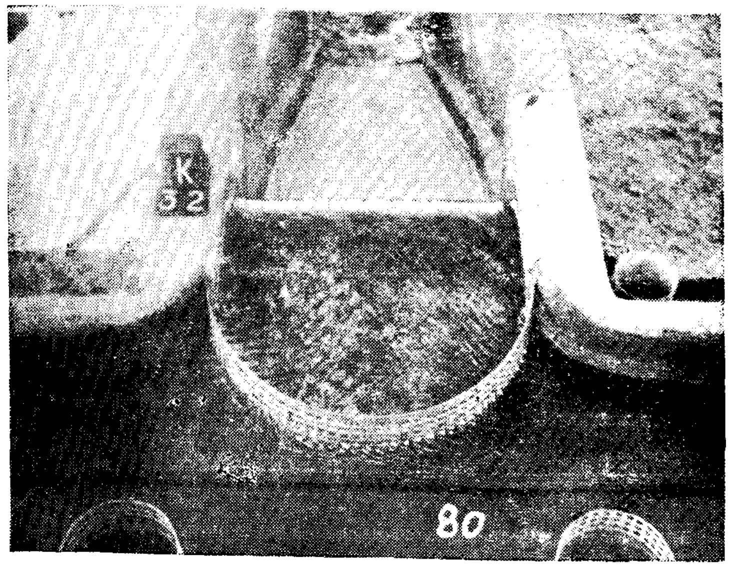


Photo No. 1(b)

Flow pattern upstream of weir when units 1 and 2 are functioning.

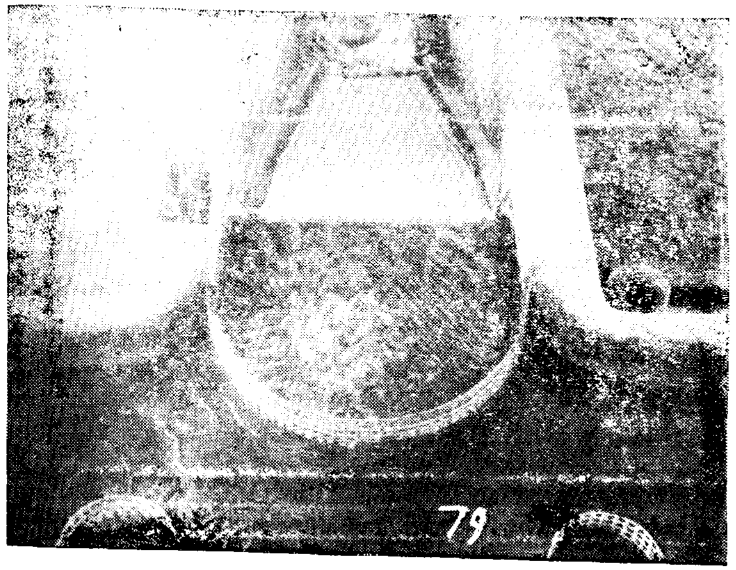


Photo No. 1(c)

Flow pattern upstream of weir when units 1, 2 and 3 are functioning.

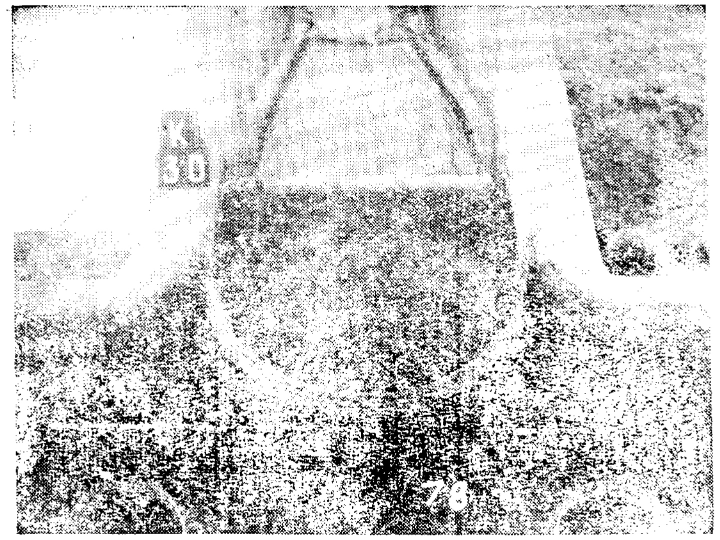


Photo No. 1(d)
Flow pattern upstream of weir when units 1, 2, 3 and 4 are functioning.

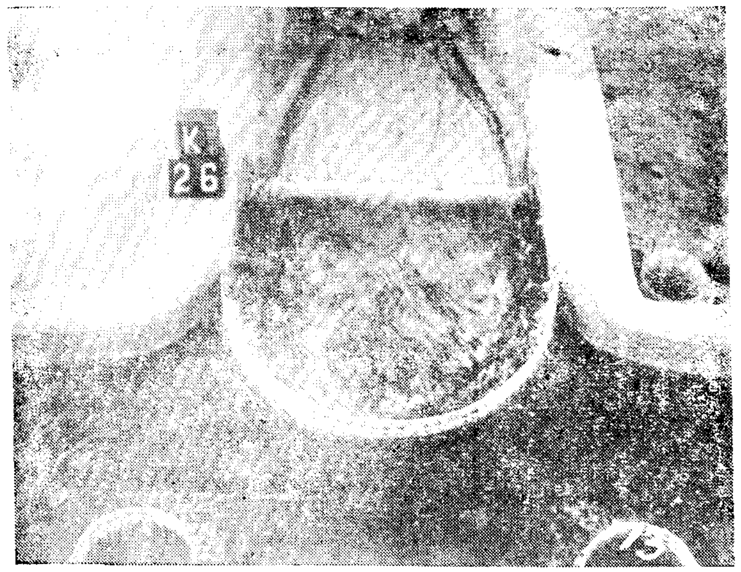


Photo No. 1(e)
Flow pattern upstream of weir when all the five vents are functioning.

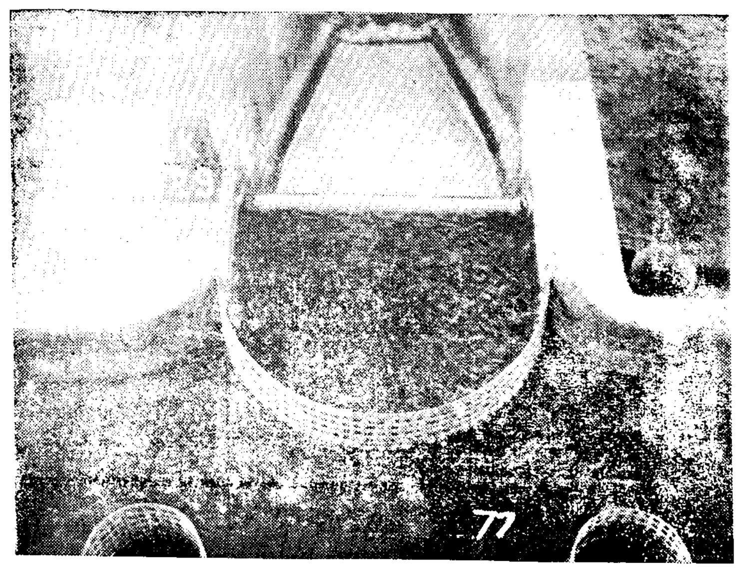


Photo No. 2(a)
Flow pattern upstream of weir when unit 5 alone is function ing.

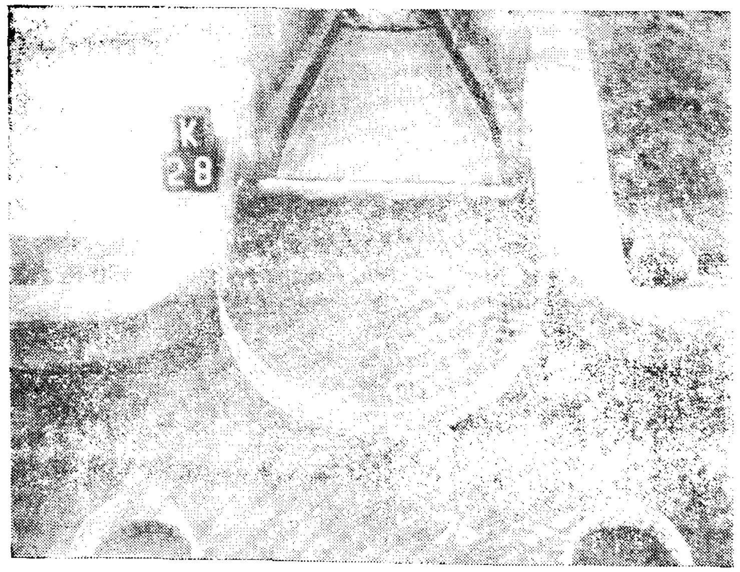


Photo No. 2(b)
Flow pattern upstream of weir when units 5 and 4 are functioning.

Photo No. 2(c)
Flow pattern upstream of weir when units 5, 4 and 3 are functioning.

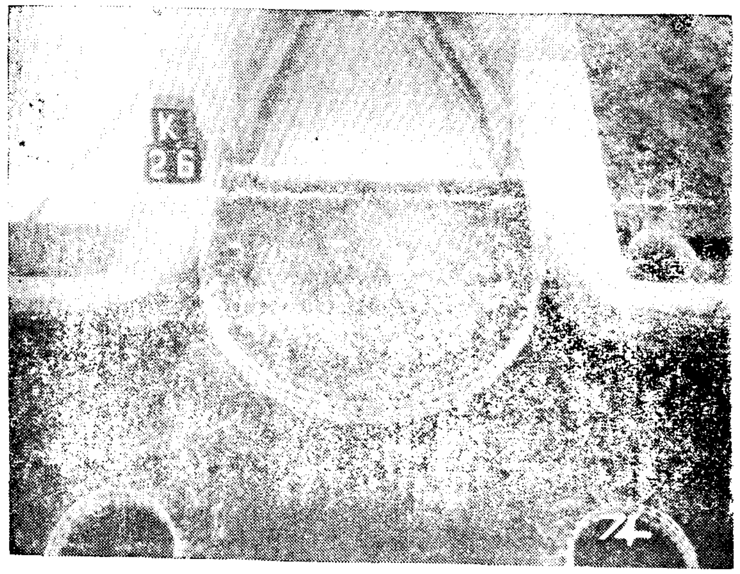


Photo No. 2(d)
Flow pattern upstream of weir when units 5, 4, 3 and 2 are functioning.

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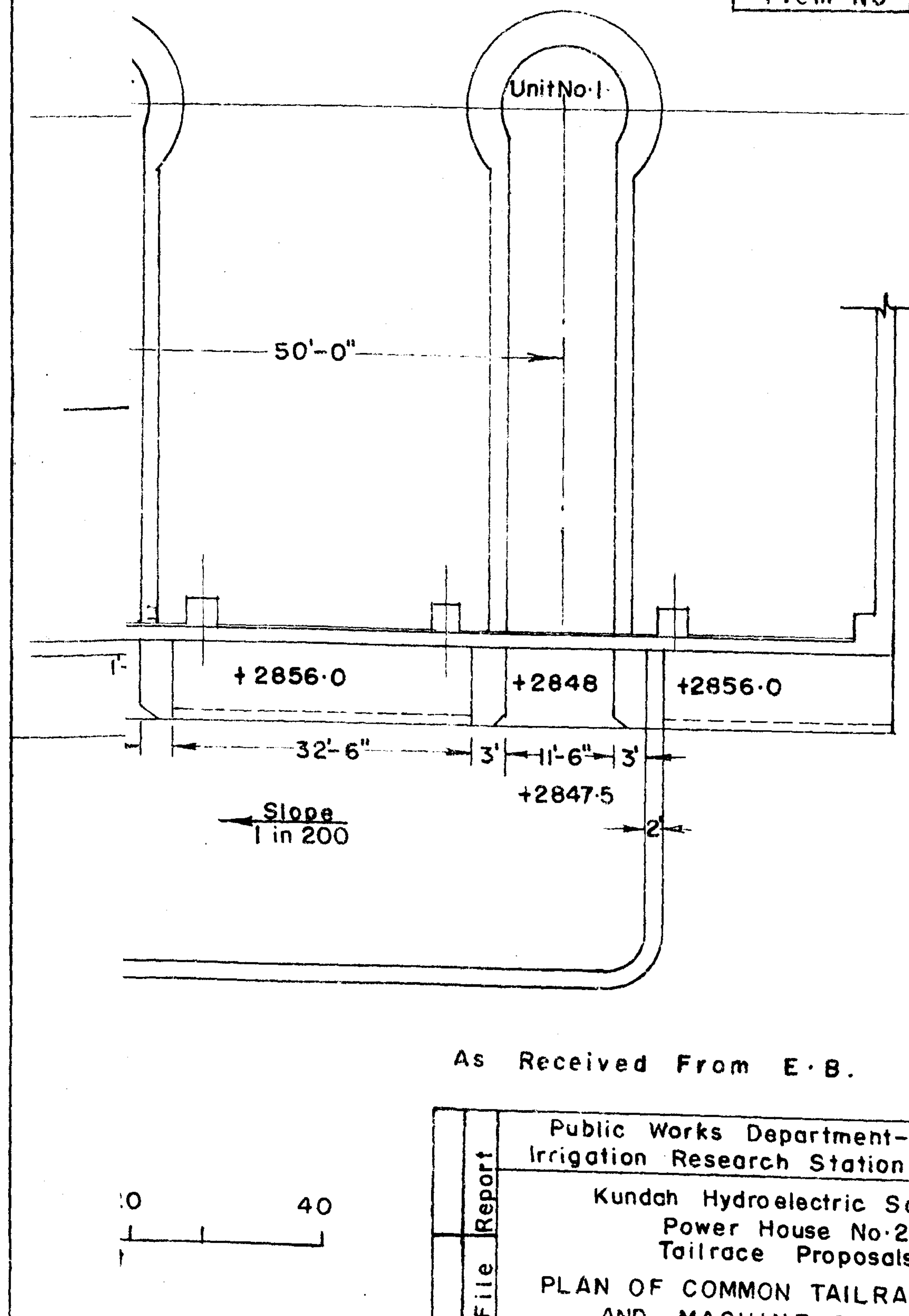

As Received From E·B.

Public Works Department—Madras
Irrigation Research Station—Poondi

Kundah Hydroelectric Scheme
Power House No·2
Tailrace Proposals

PLAN OF COMMON TAILRACE POOL AND MACHINE CHAMBERS

Report		
File		
Drawing 87/60		
J·E·R	A·E·R	E·E·R
Trd: v·v·	ckd:	FIG··· 2(a)

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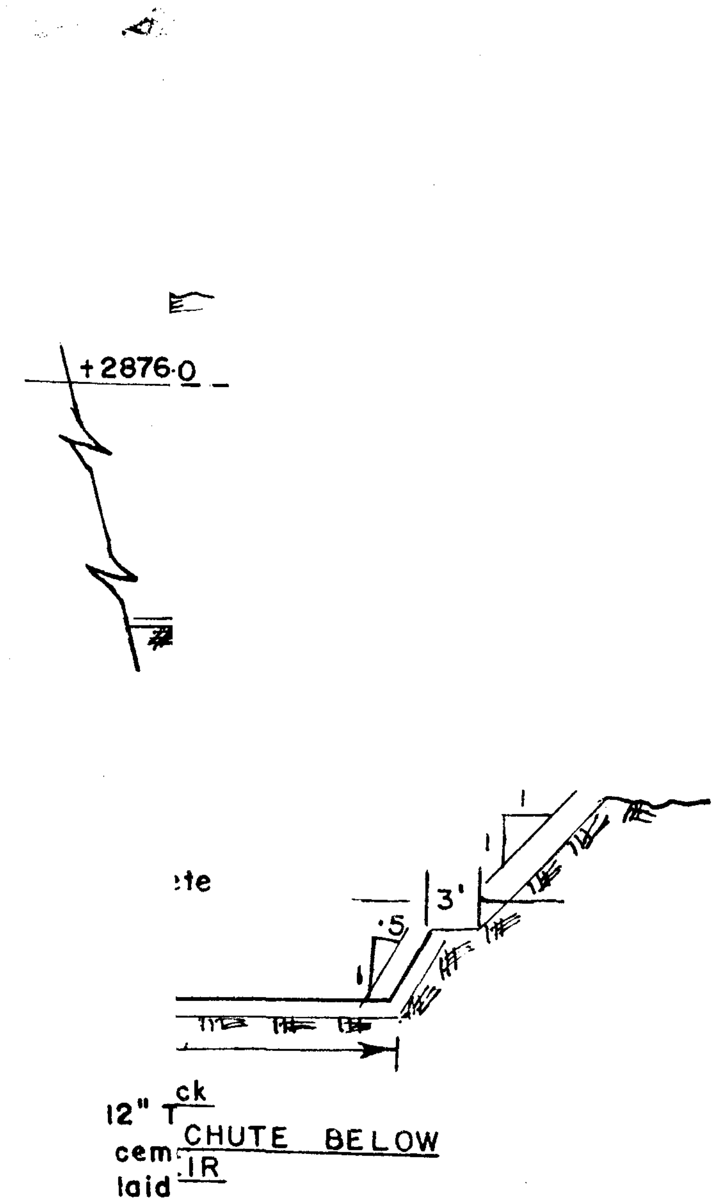
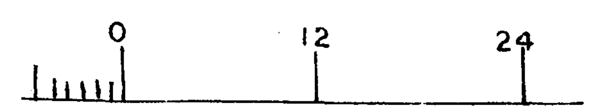


CHUTE BELOW
:IR

ıay be removed up to jointed rock
lope the bottom & sides may be lined

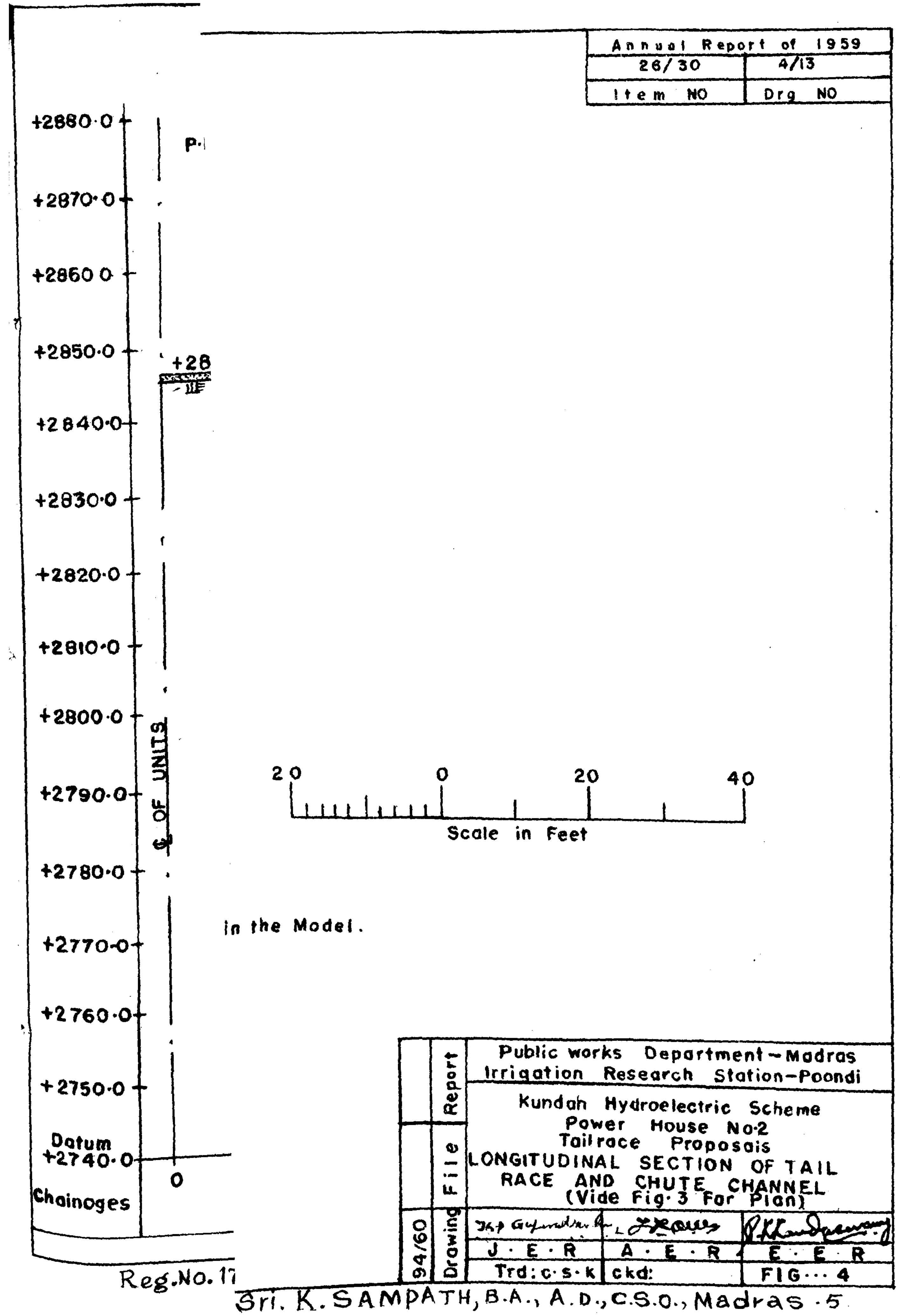
reduced lower down to about 20 feet



Scale in Feet

Report	Public Works Department-Madras Irrigation Research Station-Poondi·
File	Kundah Hydro electric Scheme Power House No·2 Tail race Proposals SECTIONAL DETAILS OF THE PLAN IN FIG 2(a)

89/60 Drawing	K.P. Gajendran	[signature]	[signature]
	J·E·R	A·E·R	E·E·R·
	Trd: c·s·k	ckd·	FIG···2(b)



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℄ OF UNITS
20
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20
40
Scale in Feet
in the Model.
Report
File
Drawing
94/60
Public works Department – Madras
Irrigation Research Station – Poondi
Kundah Hydroelectric Scheme
Power House No·2
Tailrace Proposals
LONGITUDINAL SECTION OF TAIL RACE AND CHUTE CHANNEL
(Vide Fig·3 For Plan)
J·E·R
A·E·R
E·E·R
Trd: c·s·k
ckd:
FIG··· 4
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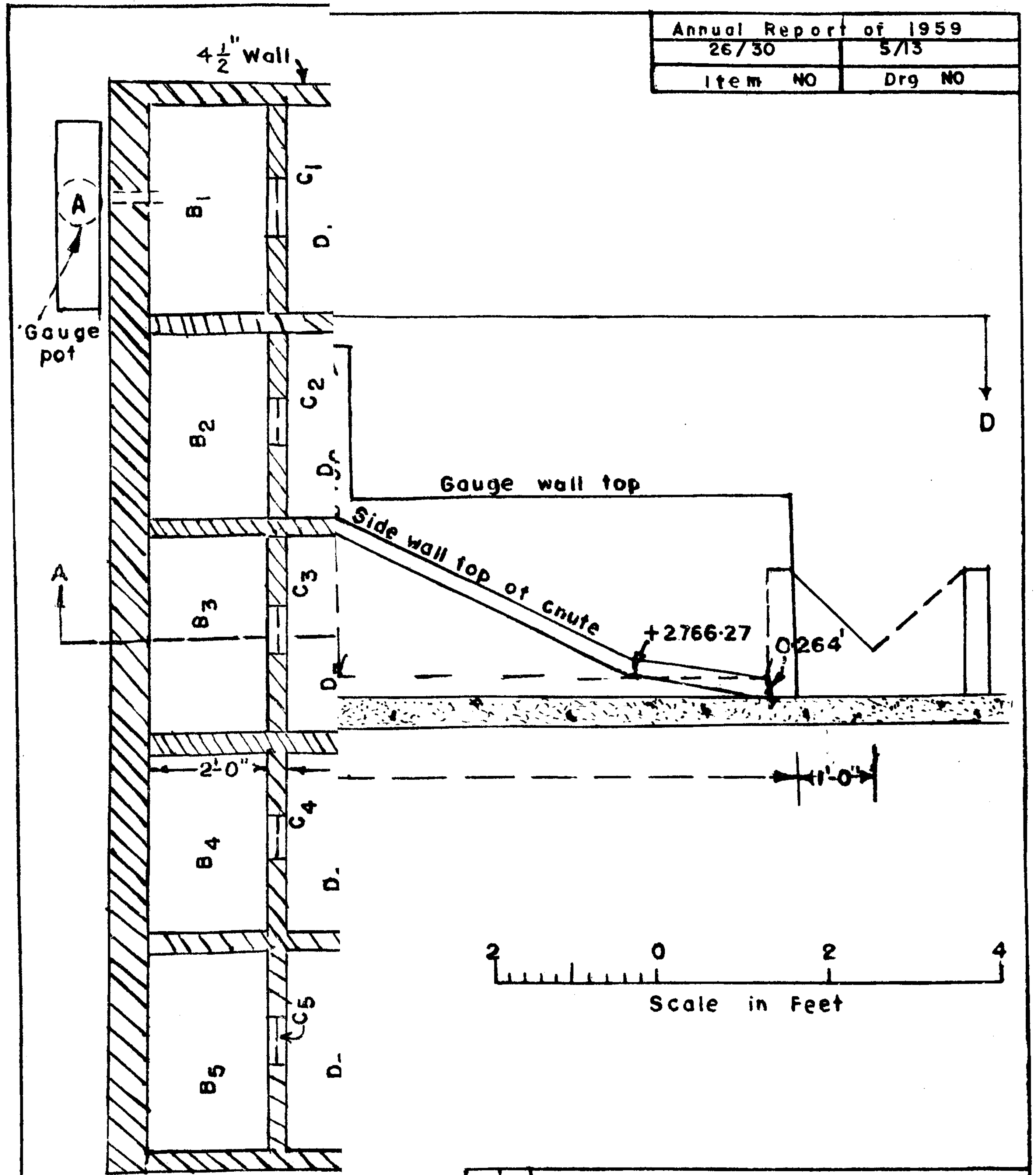
Sri. K. SAMPATH, B.A., A.D., C.S.O., Madras·5



Public Works Department-Madras
Irrigation Research Station-Poondi

Kundah Hydroelectric Scheme
Power House No·2·
Tailrace Proposals
MODEL SET-UP

K.P. Gajendran		
J·E·R	A·E·R	E·E·R
Trd: c·2·k	ckd:	FIG·· 5

File Report Drawing 98/60

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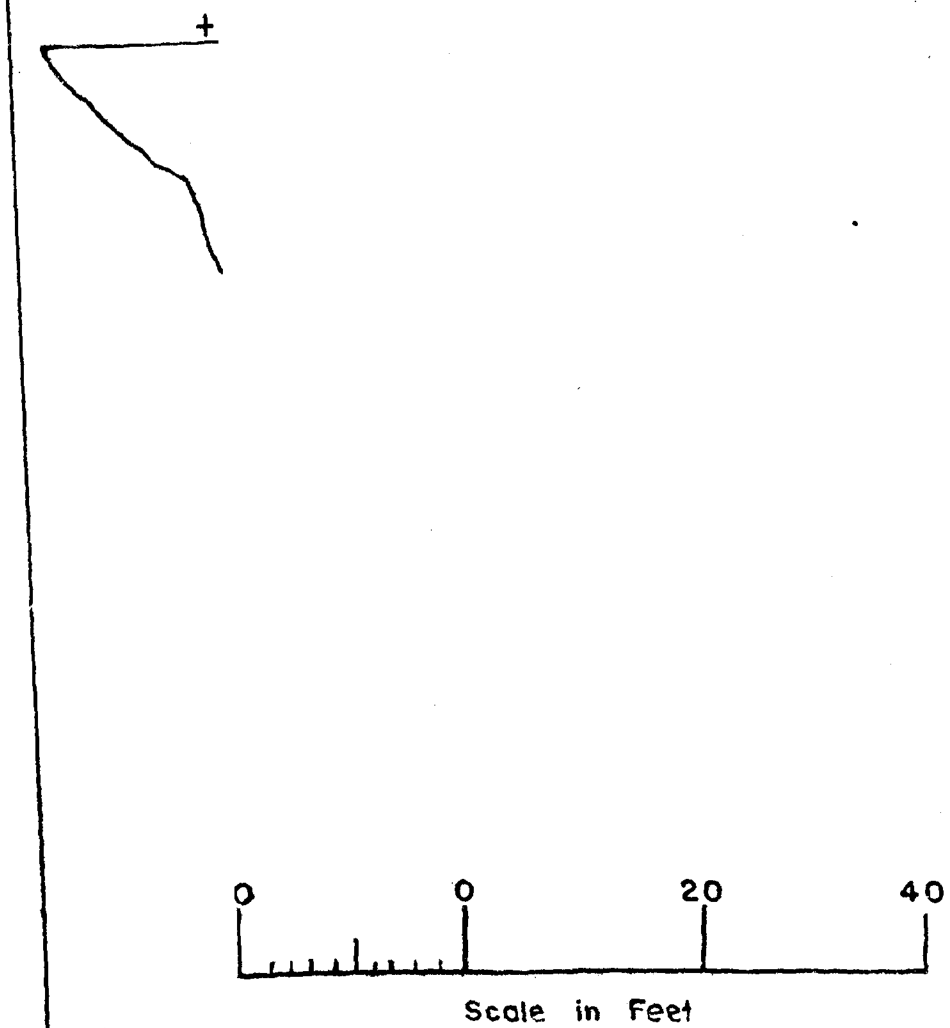

Report
File
Drawing
91/60

Public Works Department-Madras
Irrigation Research Station-Poondi

Kundah Hydro-electric Scheme
Power House No. 2
Tailrace Proposals
FLOW PROFILE ON CREST OF WEIR
(ORIGINAL PROFILE)

K. P. Gajendran		P. K. Kandaswamy
J. E. R	A. E. R	E. E. R
Trd: C.S.K	Ckd:	FIG. 6.

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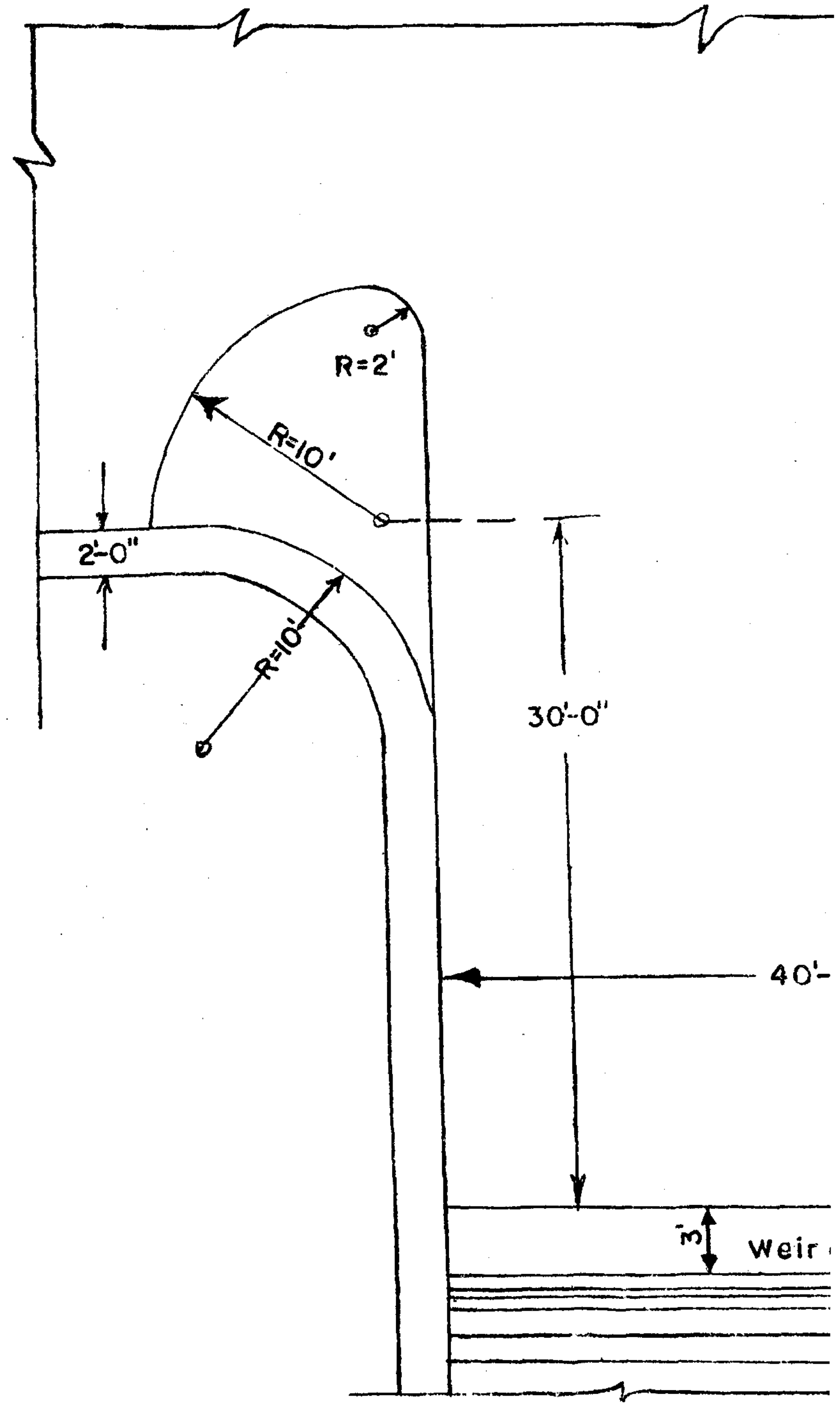

PLAN SHOWING ARRANG
FLOW PATTERN U/S

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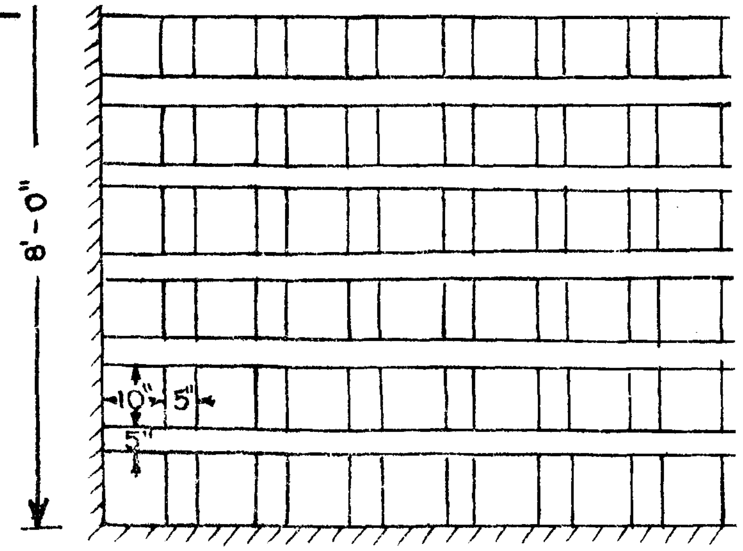
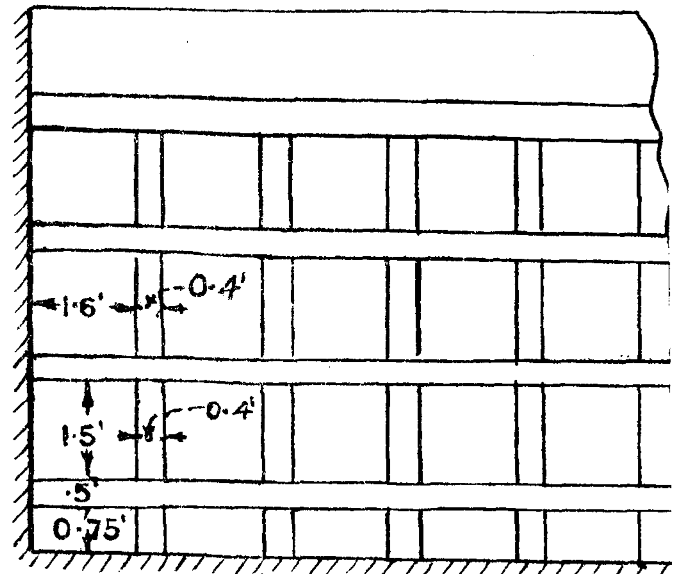
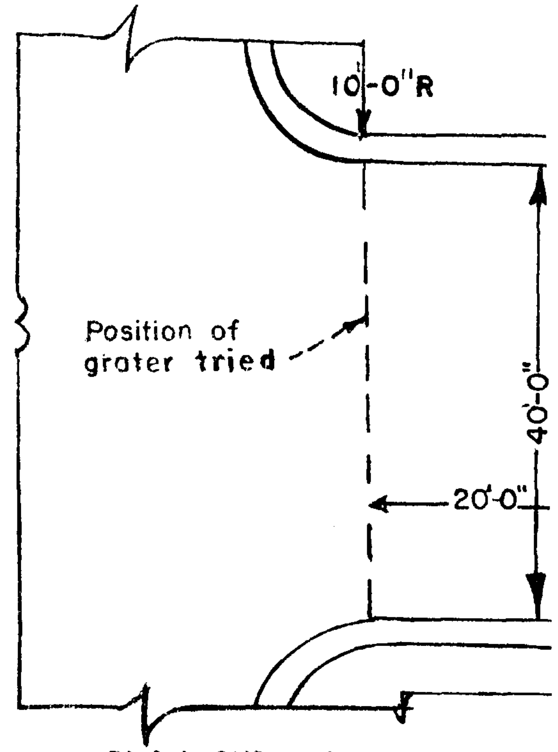


(a) GRATE No-1 (Not to scale)



(b) GRATE No 2 (Not to sc



PLAN SHOWING GRATE POSI

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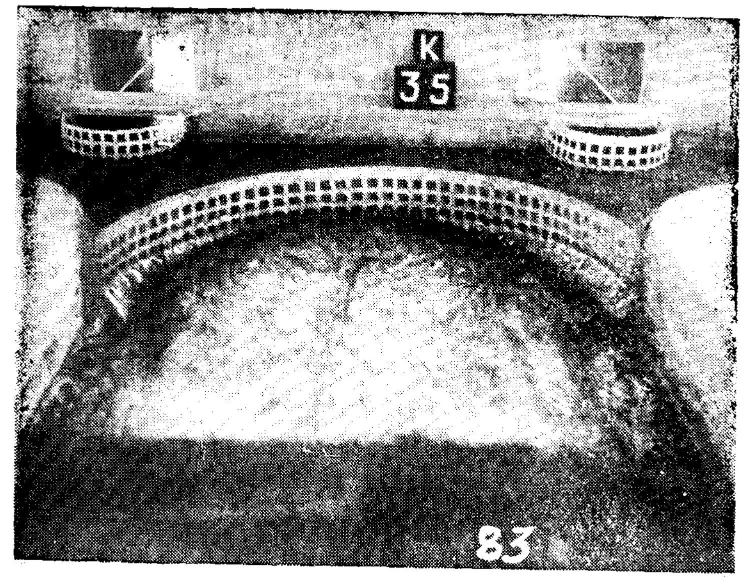
Item NO	Drg NO



10 0 10 10

Scale in Feet

Report	Public Works Department - Madras Irrigation Research Station - Poondi
File	Kundah Hydroelectric Scheme Power House No. 2. Tailrace Proposals FINAL ARRANGEMENT TO IMPROVE FLOW CONDITIONS UPSTREAM OF WEIR
Drawing 92/60	J.K.P. Gajendran — J·E·R — Trd: c·s·k / A·E·R — ckd: / E·E·R — FIG...9

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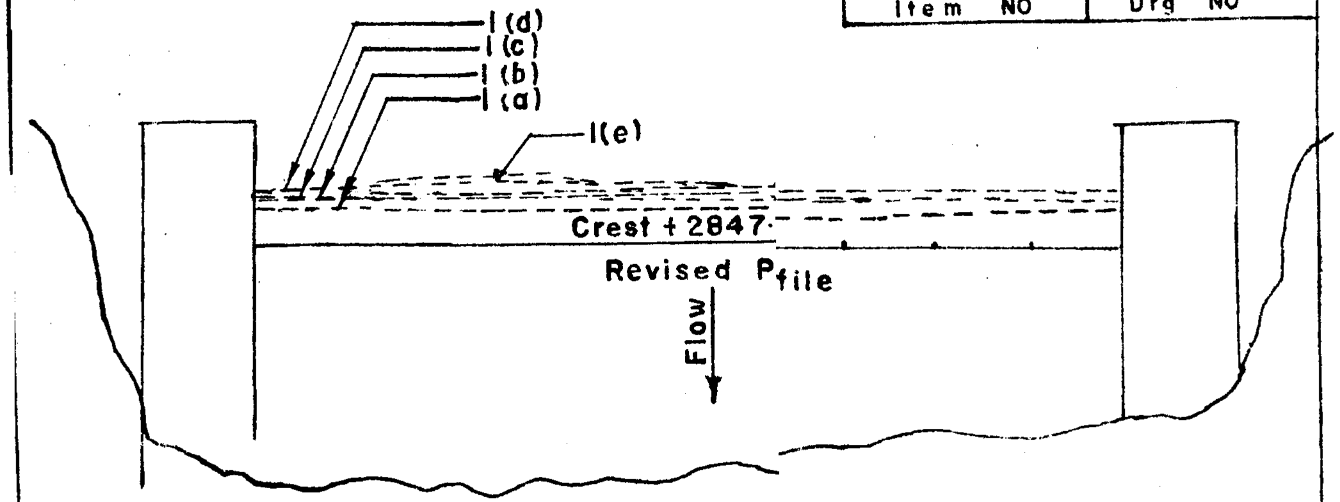
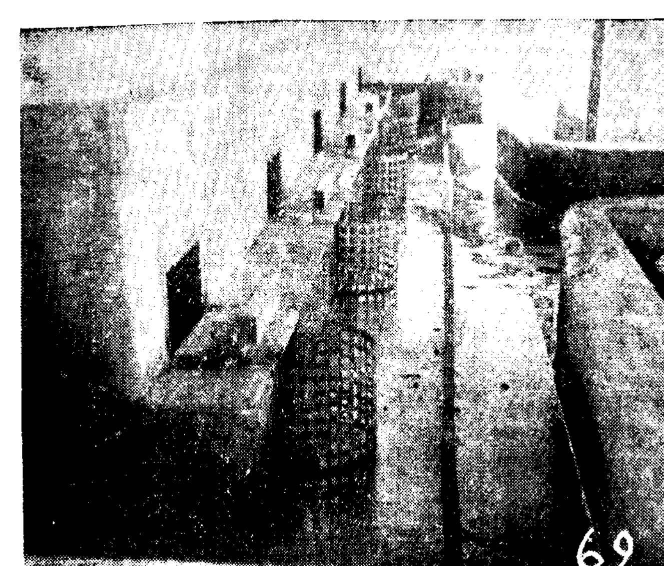
Item NO	Drg NO



No·1

1·(a) Flow profile on crest wh ne is Functioning
1(b) " " " " "4 are "
1(c) " " " " "x3 " "
1(d) " " " " "x2 " "
1(e) " " " " "



Works Department – Madras
ion Research Station-Poondi

ndah Hydroelectric Scheme
Power House No·2
Tailrace Proposals
PROFILE ON CREST OF WEIR
RRANGEMENTS AS IN PHOTO
HERE

[signature]		P.K. [signature]
·R	A·E·R	E·E·R
s·k	ckd:	FIG ... 10

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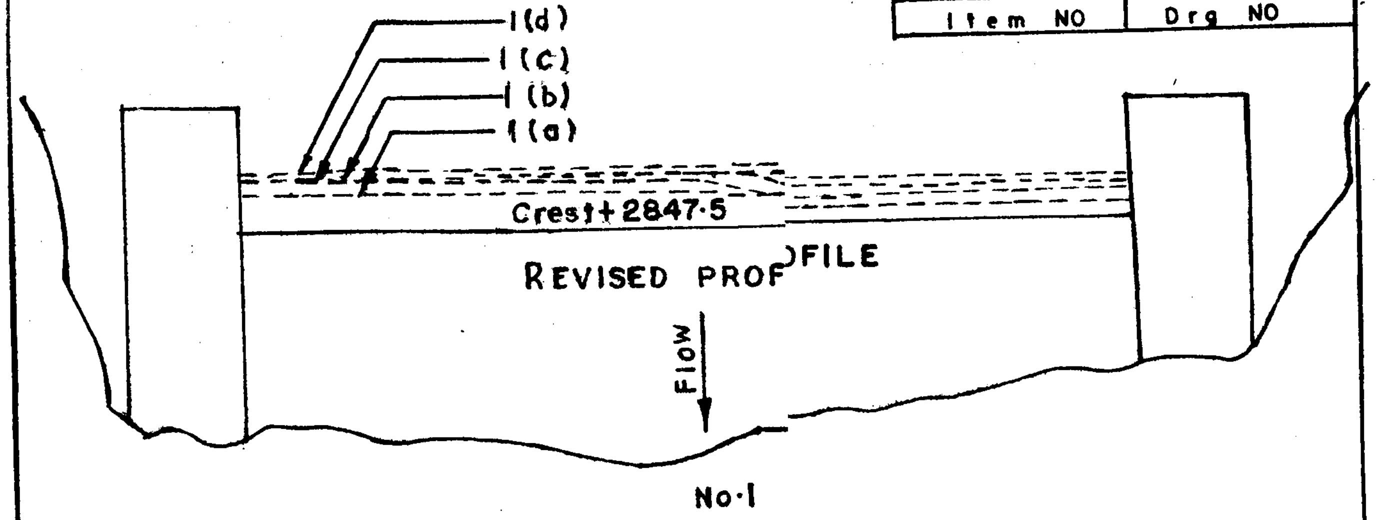

1·(a)	Flow	profile	on	crest	when	is	Fuctioning	
1·(b)	"	"	"	"	"	are	"	
1.(c)	"	"	"	"	"	"	"	
1.(d)	"	"	"	"	"	2 "	"	
1.(c)	"	"	"	"	"			

...lic Works Department — Madras
...tion Research Station-Poondi

...ndah Hydroelectric Scheme
Power House No·2
Tail race Proposals
...PROFILE ON CREST OF WEIR
...ARRANGEMENTS AS SHOWN IN
FIG·9· or PHOTO HERE

...		
...E·R	A·E·R	E·E·R
...c·s·k	ckd:	FIG·11

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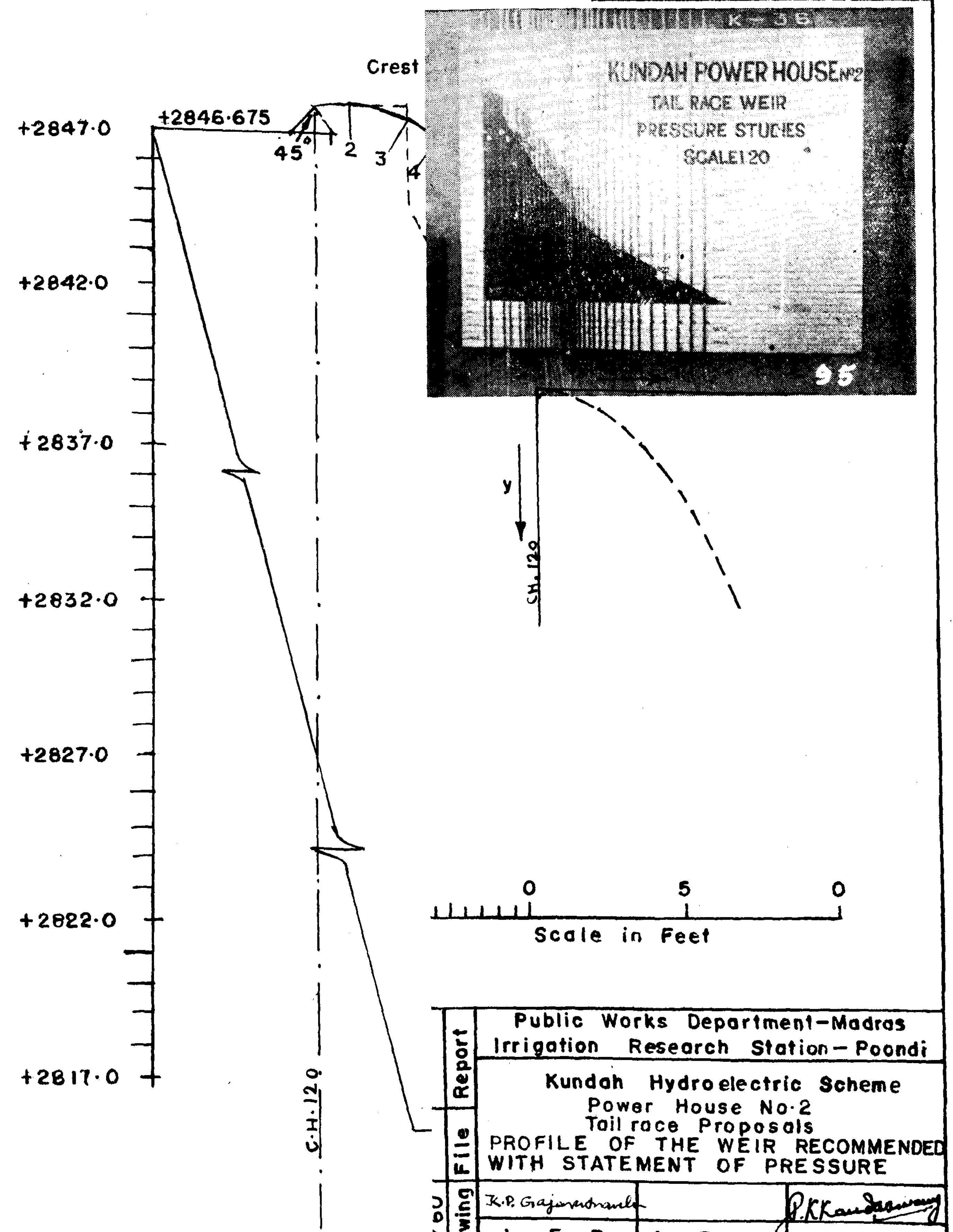


Public Works Department—Madras
Irrigation Research Station—Poondi

Kundah Hydroelectric Scheme
Power House No·2
Tail race Proposals
PROFILE OF THE WEIR RECOMMENDED WITH STATEMENT OF PRESSURE

File Report

Drawing 961/60

K.P. Gajendran		R.K. Kandaswamy
J · E · R	A · E · R	E · E · R
Trd: c·s·w	ckd:	FIG···13

Reg. No. 206 E '62 (Saka Era 1884)-375 Sri, K. SAMPATH, B.A., A.D., C.S.O., Madras-5.

CONTROL OF EVAPORATION LOSSES FROM OPEN STORAGES SYNOPSIS

For assessing the utility of hexadecanol for reducing loss due to evaportion from large open storages, experiments were carried out in a medium sized tank called the Buderi tank situated about two miles south west of Irrigation Research Station, Poondi.

Various quantities of the solution of cetyl alcohol desolved in Mineral turpentine were sprayed daily on the tank water surface for a certain period in each case with a view to find the maximum amount of savings in eavporation that can be achieved by the use of this chemical and also to find the amount of chemical required to give optimum results. The effect of wind rain etc., as the chemical film was also studied.

1. Introduction

1.1. Evaporation is a process by which a liquid or solid is changed into vapour. The exact nature of this phenomenon is not yet clearly understood. But a logical explanation based on the transfer of momentum by the continuous movement of the modecules of water is that the faster moving molecules near the surface escape into the atmosphere. There will also be a return of these molecules back into the parent medium i.e. water body. If on the whole, the number of molecules espacing into the air is greater than the number of molecules coming back to the water body there is said to be evaporation from the water surface.

Since evaporation causes a considerable loss of useful sotroge, finding a method for reducing this loss will result in saving considerable quantity of valuable water especially in tropical countries.

1.2. The work of Mansfield of the Commonwealth Scientific and Industrial Research Organisation in Australia in regard to the conservation of water by a monomolecular film of Cetyl Alcohol over water has opened out immense possibilities in the field of reduction in Evaporation losses. Hence studies were conducted for finding the feasibility of the economic usage of this chemical for conserving water from large open storages. This study was sponsered by the Central Water and Power Commission.

1.3. Before starting the field experiments in Poondi preliminary experiments were conducted in the concrete and soil Research laboratory for arriving at a suitable solvent for the chemical cetyl alcohol. After truing out various solvent such as SGP spirit 40/60 55/115 C etc., it was decived that mineral turpentine will be a suitable solvent for the chemical and was used in the experiments that were conducted in the field.

1.4. Reports on the studies conducted peior to August (i.e. at Sengarai Uthukottai and Buderi tanks) have already been sent all the details. This report deals with the experiments conducted in the period between February and May, 1959.

2. Experimental set up:

2.1. The experiments were conducted in a tank called Buderi Tank which is situated just two miles south west of Poondi Research Station and adjacent to the Poondi Reservoir on the western side (vide Figure-1). This tank has a water spread area of about 28.2 acres and a maximum depth of about 10.5 feet at F.R.L. For purposes of these studies a field laboratory has been set up on the northern fore shore of the tank containing (*a*) a Maximum and Minimum thermometer for recording temperatures (*b*) a wet and dry bulb thermometers, for humidity observations (*c*) a rain gauge (*d*) one Anemometer for knowing the wind velocity (*e*) a wind vane for knowing the direction of wind (*f*) 2 land pans of the standard U. S. Weather Bureau type. In addition to this there, 4 feet diameter pans were installed to float on the water surface of Buderi tank. A gauge well was fixed in the tank to take observations of water levels from which loss due to evaporation could be computed. A seepage meter to measure the seepage through the tank bed was also installed in the tank. Two land pan and one gauge well were installed in the adjacent Poondi Reservoir for recording similar observations. Later one 8 feet dia. land pan and one 2 feet dia land pans were added to the field observatory at Buderi for comparing the evaporations loss obtained from these pans with that of the standard 4 feet dia U. S. weather Bureau pan. Also one 2 3½ feet square wooden pan and an iron pan of similar size were installed to find the difference in evaporation loss when the pans are made of these two different materials. Floating rafts were used for the prupose of sprayings taking readings of water levels from the floating pans, replenishing the water on the floating pans and for taking the film strength at various places on the tank surface. The location of all these experimental apprutenances in and around the tank are shown in Fig. 2. A detailed survey of the tank was carried out and the capacity and waterspread area of the tank for various elevations were placed and is shown in Fig. 3.

3. Experiments:

3.1. In the set of experiments that were carried out the object was to collect data regarding:

( ) The amount of chemical to be used per acre and the concentration of the solution required to give optimum results.

(2) The extent up to which loss due to evaporation can be reduced by spreading the chemical cetyl alcohol over water surface.

(3) The best and the most economical method of applying the chemical for storage reservoirs.

Before starting the regular experiments with a view to clear the water surface of dirt and seam a large quantity of solution i.e. 9 lbs of cetyl alcohol in 20 gallons of mineral turpentine was sprayed on the Buderi water surfaces. The chemical while spreading looks in its film all the floating debris and thus in swept along with the film by the wind towards the leeward shore The area covered by the solution and the actuals preading of the chemical solution was easily visible to the naket eye in that the maximum waves and ripples get quelled over the treated surface. Various quantities of the solution were sprayed for the following 4 days in order to get the water surface practically clean of dirt and other flashing debris. Regular experiments were then commenced.

3.2. SURFACE PRESSURE OF FILM:

It is gathered from available literature that the ideal condition for securing maximum resistance to evaporation is the one at which the surface pressure of the cetyl alcohol film is maintained at 40 dynes / sq.c.m. Hence for assessing the actual surface pressure of the film spread on the tank a far indicator oil such as coconut oil, caster oil and alic acid were tentatively used for indicating surface pressure below 17 dynes / sq.c.m. 17 to 30 dynes / sq.c.m. and 30 to 40 dynes / sq.c.m. respectively. Later shell high speed diesel oil, shell vitrea oil 13 shell vitrea oil 21 and shell Ensis fluid 256 were used for indicate surface pressure of 13 dynes, 16 dynes, 24 dynes and 40 dynes respectively.

Various methods were tried for testing the film strength on the water surface by the use of these oils when these oils were directly applied on the tank water surface they behaved irratically. Hence, it was decided to test the film strength by taking the treated water of the tank in a sauce pan and conducting the tests with oils. This Method was also found unsatisfactory as the film got raptured or dislodged of the sample was not taken carefully. Next an enclosure 3′0″ dia open at top and bottom was suspended in the Surface and tests were carried out within this enclosure. This proved to be more effective then the other methods and it was adopted for taking of film strength. The Oily patches of surface were carefully removed after every test.

3.3. QUANTITY AND CONCENTRATION OF THE CHEMICAL SOLUTION:

3.3.1. To arrive at the optimum concentration of the solution various preparations of the chemical and the solvent applied and observations were recorded for a certan period in each case. Treatment with the chemical solution were given to the tank surface, to one of the floating pans provided in Buderi tank and to one each of the land pans provided on the shores of Buderi tank and Poondi reservoir. The other pans were left untreated in order to collect the corresponding data for untreated surface. The experiments were done in four series the proportions used being 3/4 lb. 1 lb. 1¼ lb. 1½ lb and 2 lbs. in 10 gallons of mineral turpentine. All the relevent meteorological data and the loss of water from the pans as well as the tank were recorded during the period of experimentation. The result of these experiments have already been given in the previous reports. It was found that in all these cases the film spread easily over the water surface. As the mineral turpentine is used only as a carrier to facilitate the spreading application and spreading of the chemical film over the water surface and by itself has no effect on the reduction of evaporation loss it was decided to use as low or quantity of the mineral turpentine as possible to reduce the cost of this operation. It was found that in 10 gallons of mineral turpentine upto 4½ lbs. of cetyl alcohol is easily soluble at atmospheric temperatures and that when this solution was applied to the tank surface it spread with equal facility. Hence it was decided to adopt this concentration for the experiments conducted later.

3.3.2. In the next series of experiments that were conducted, various quantities of the chemical solution were applied to the tank water surface keeping the concentration of the solution as constant. 4½ lbs. of cetyl alcohol to 10 gallons of mineral turpentine. Various quantities of the solution ranging from 1/2 a gallon to 5 gallons in increments of 1/2 a gallon were applied every day to the tank water surface and the solvent observations were recorded. Also 6 gallons of the solution were sprayed for some days on the Buderi tank water surface. These quantities were tried with a view to find the quantity of the chemical required for getting the maximum savings in evaporation loss. The results of these experiments are given in Statement 3. Observations without any treatment to the tank were also recorded for a certain period in between these series of treatment so that this result can be used as a basis of comparison with the results obtained after treatment with various quantities of the chemical.

3.4. METHOD OF APPLICATION OF THE CHEMICAL:

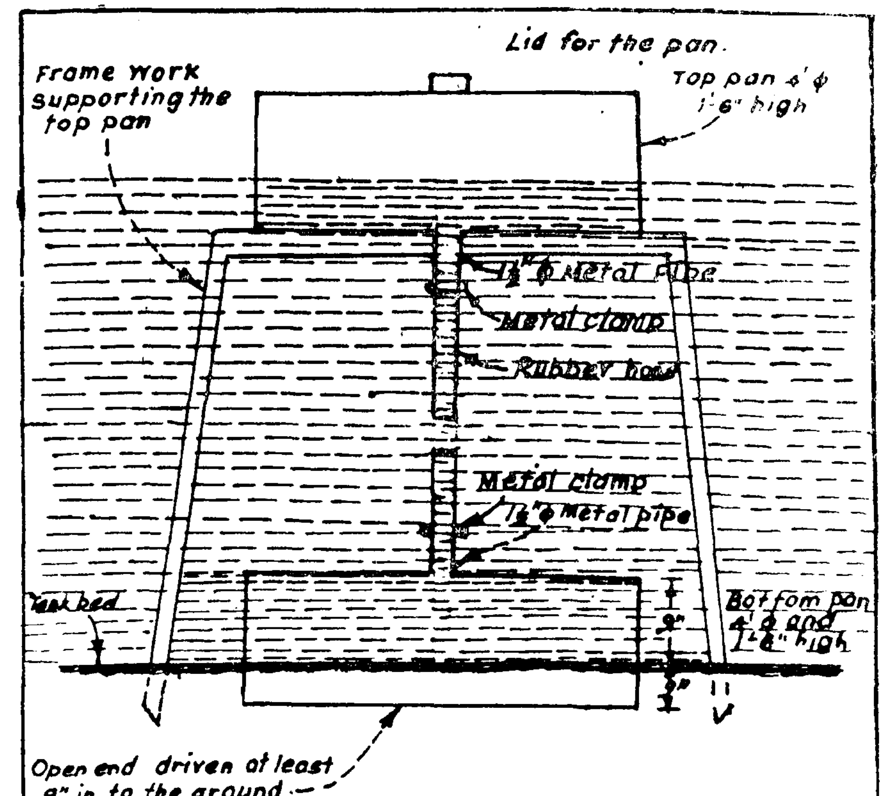
In the beginning spraying has done from a floating raft moving at right angles to the direction of wind. Later spraying was done along the wind ward shore by means of spraying carried by a man walking along the shore. It was observed that both these methods were defective, when the velocity of wind was high in that the film was swept towards the leeward shore in a very short time after it was applied to the surface. Hence it was decided to have the chemical drinking from dispensing units made out of tins with a fill made at the bottom. These were moulded on stands along wind ward shore. It was observed that this method had also certain draw-backs. The lateral dispersion of the film was not sufficient to cover the entire tank. The number of such unit and also number of dispensing points per unit were then increased to overcome this drawback- Even then it was found that when the wind velocity was low it took a long time for the chemical from these units to spread and cover the entire surface of the tank. Hence, finally it was decided to spread the film over the water surface by spraying along the windward shore if the wind velocity was sufficiently high or in two or more lines perpendicular to the wind directive by carrying the spraymax in the raft. This was supplemented by dispensing units from which the solution was allowed to drip slowly. By this means it was attempted to maintain the film over the entire area of the tank. The spraying by spraymax was so done as to cover the entire area of the tank initially. But when the wind velocity was high, there was no sufficient strength of film near the windward shore. Hence, installation of the dispensing units from which the solution was dripping helped in maintaining the film near the windward shore. Figure 4 and 5 show the method of spraying adopted for certain days during these experiments.

4. Seepage Loss:

4.1. For calculating loss of water due to evaporation alone from a tank reservoir etc., it is necessary that the loss due to seepage should be known as the latter forms an integral part of the total loss. In order to assess the quantity of water lost through the tank bed a device called the seepage meter was evolved in this Research Station and installed in Buderi Tank.

This device consists of two cylindrical pans of equal dia and size (i. e., 4′ dia and 4½′ high vide figure below). In both these pans a hole of 1½″ dia. is drilled in the middle of the pan and short metal pipes about 4″ long and of the same dia. so that of the holes are welded to these holes to project outside

from the bottom. These pipes are for the purpose of connecting these pans to each other by means of shipping the ends of a rubber hose on them.



SEEPAGE METER

One of these pans is inverted and rammed with the bed such that at least 9″ its dies penetrate into the soil. The other pan with its open end facing upwards is supported on a framework above the first pan with its bottom and least 9″ below the water surface of the tank. The top pan is covered by a lid for prevention of loss of water due to evaporation. Water is poured into the top pan to the same level as outside water in the tank. As the pans are connected to each other the portion of the bottom pan above the tank bed will also be full of water. When the water from the bottom pan seepage through the bed of the tank the water level in the top pan will get lowered by a corresponding amount and as the pans are of the same diameter the fall in level of water in the top pan will directly indicate the loss of water due to seepage from the tank bed.

4.2. This meter was installed in the deep portion near the bund in Buderi Tank. The readings obtained from this seepage meter are given in Statement 2. This meter gives the only seepage loss through the bed. For calculating the total loss of water due to seepage the amount loss due to seepage through the bund also has to be taken into account. Further, the seepage obtained from this meter is only representative of the seepage through the surrounding area of the tank bed where it is installed. There may be variations in the bed material in the different places in the tank and consequently the rate of seepage may vary from place to place in the tank bed. This aspect is being studied by installing a similar device at a different place in the tank.

4.3. The seepage through the bund was computed. This bund is located between Poondi Reservoir and buderi tank and hence has water on both sides. The difference in water levels between the two reservoirs varied from 0·2′ at the beginning to 1·1. feet at the end of the period of experimentation (i.e. from January upto end of May). Hence it was found that the daily loss due to seepage through the bund was very little when computed in the daily loss through the bed i.e., when the difference in water level was about 0·75′ loss through the tank bed during that period was of the order of about 8,000 cft. per day.

For finding out the total loss of water due to seepages from the tank the observations recorded during the period when the tank and the pans were left untreated were made use of. The seepage loss was computed from these observations.

4.4. For computing the seepage loss and also for calculations the savings in evaporation loss from the tank when treated, it was assumed that the loss from pan No. 6 (untreated floating pan) multiplied by a pan co-efficient of 0·8 will indicate the loss due to natural evaporation from the tank surface if untreated. This pan was chosen because it is situated almost in the middle of the tank and will be subject to the average of the meterological factor that affect the various portions of the tank. Land pans are generally considered superior to the floating pans. In this particular case it was observed that floating pan No. 5 which is situated near the southern end of the tank indicated evaporation loss to a lessor extent therein pan No. 6 which is situated in the middle of the tank. This indicates that the portion of the tank around pan No. 5 is not subject to the same meterological conditions as the portion around pan No 6. The same may be true in the case of land pans also. Hence it is considered that the results obtained from pan No. 6 will be a close approximation to the loss from that tank than that obtained from the other pans. The co-efficient choson is the one recommended by the Sub-Committee on Evaporation of the Special Committee on Irrigation and Hydraulics of the American Society of Civil Engineers. They have recommended a co-efficient of 0.80 with a range from 0.70 to 0.80 for the U.S. Geological M floating pans. The same co-efficient has been adopted for this pan also though the size and shape is different.

The total see-page was computed as shown below:—

(i) Period of observation 23—2—59 to 1—3—59 i.e. 7 days—(untreated period).

Difference in water level between Buderi and Boondi lakes during the period =0.14′ to 0.19′.

Evaporation loss from pan No. 6 as observed for these 7 days—0.14 feet.

Natural evaporation loss from the tank =(0.8 x loss from pan No. 6) 0.144 x 0.8 = 0.1152 feet.

Total loss from the tank as observed from Gauge well readings—0.156.

Hence loss due to see page for the period of 7 days = 0.156 – 0.1152 = 0.0408

Average loss due to seepage per day during the period = $\frac{0.0408}{7}$ = 0.0059′.

(2) Period of observation—19—3—58 to 29—3—59 i.e. 11 days.

Difference between Buderi and Poondi water levels during the period—about 0.4 feet.

Natural evaporation loss observed from pan No. 6 for these 11 days = 0.245 feet.

Natural evaporation loss from the tank = (0.8 x loss from pan —) = 0.196′.

Total loss from the tank = 0.256.

Hence loss due to seepage for the period of 11 days = 0.256—0.196 = 0.060.

Average loss due to seepage per day during the period = 0.0055′.

In case 1 x 2 as the difference is very little the seepage is taken as 0.006′ per day.

(3) Period of observation 10—5—59 to 16—5—59 i.e. 7 days.

Difference in water between Buderi and Poondi during this period = 0.9′ to 1.0′.

Evaporation loss from pan No. 6 for these 7 days = 0.191 feet.

Natural evaporation loss from the tank—(0.8 x loss from pan 6)—0.1528 feet.

Total loss from the tank = 0.202 feet.

Hence loss due to seepage for that period of 7 days = 0.202 —0.1528 = 0.0492 feet.

Average loss due to seepage per day during the period = 0.007 feet.

For the purpose of calculation of the sagings in evaporation loss obtained for the different quantities of solution spread on the tank surface the seepage losses per day during these series are taken as final.

Series 1 to series 7 (5—2—59 to 23—4—59) ... 0.006 feet per day.

Series 8 to Series 11 (24—4—59 to 31—5—59) ... 0.007 feet per day.

5. Effect of Wind:

The wind was observed to have beneficial as well as detrimental effect on the process as a whole. To clean the water surface as preliminary to the regular the wind was beneficial whereas it was detrimental in that it carried away the film towards the leewards the leward shore thus vitiating the effects of the completely. Moreover, the violent surface waves caused by the wind ruptured the film and made it almost useless for prevention of loss due to evaporation.

6 Effect of Rainfall:

The effect of rainfall on the chemical film was also studied. It was observed that after a heavy downpour there was almost no film left on the surface of the tank water even through that of the falling raindrops break up the film completely and after the rains when the indicator oils were used for testing the film strength they showed absence of any film strength of the film was completely broken up.

Regular studies were not conducted during the rainy days as rainfall usually brought with it such complications as inflow and outflow which were not easily measurable with any degree of accuracy.

7. Collection of Data :

7.1. Maximum and Minimum Temperatures, Humidity at 8 a.m., 12 noon and 4 p.m. Rainfall measurements if any intensity and direction of wind, collected daily during the period of experimentation are given in Statement 1 along with the evaporation obtained from the pan No. 3 which is a land pan as that of Buderi tank. Statement No. 2 gives the loss of water from the pans seepage meter and the reservoir recorded daily. Statement No. 3 shows the loss of water daily due to evaporation retained from 8 feet dia pan, 2 feet dia pan, 4 feet dia pan, the 3½′ square wooden pan and the 3½′ pan.

During these series of experiments it was observed that after the continuous spraying of the chemical was stopped the film strength reduced gradually and by the evening of the second day there was almost no trace of film left on the water surface of the tank. It was also observed that on windy days there were thread-like formation of the chemical film near the leewad shore. These formations also disappeared by the evening of the second day after spraying was stopped.

The Buderi tank is mainly used for raising fingerlings and stocking fishes and it was seen that where some chemical particles were thrown on the water surface the fishes (though they swam around them) did not swallow these particles. The treatment of the tank with this chemical does not seem to have any hard effect on the fishes reared in the tank.

A certain weighed amount of chemical was left in the sun for some days and it was found that there was no loss in chemical due to exposure in the sun though the chemical melted in the hottest part of the day solidified in the evenings.

7.2. A possible reason for the gradual appearance of the sprayed film may perhaps be that apart from the possibility of destruction of the some of the chemical by the bacteria present in the water the molecules of the film itself gets thrown into the air along with the escaping molecules of water and is swept off by the prevailing wind. As evaporation is only partly reduced by the formation of this film and some of the water molecules still escape into the air it is probable that the molecules of cetyd alcohol also gets away by these escaping molecules into the air.

8. Results and Conclusions :

8.1 For calculating the percentage of savings in evaporation loss due to the spreading of these various amount of chemical on the tank water surface it is necessary that the evaporation loss from the tank surface if the water is left untreated should be known. The loss in water level obtained from the Buderi gauge will during these periods of spraying indicated the loss of water due to evaporation from the treated surface and the loss due to seepage. The seepage from the tank has already been computed.

8.2. The percentage of savings was worked out as shown below :

The daily loss from pan No. 6 multiplied by a co-efficient 0.8 was taken as a fair indication of the natural evaporation loss from the tank surface it left untreated.

The percentage of savings in evaporation loss due to the treatment by the Chemical ... $\dfrac{100\,(0.8 \text{ x loss from pan } 6 - (\text{loss from Buderi tank seepage loss})}{0.8 \text{ x Pan No. } 6}$

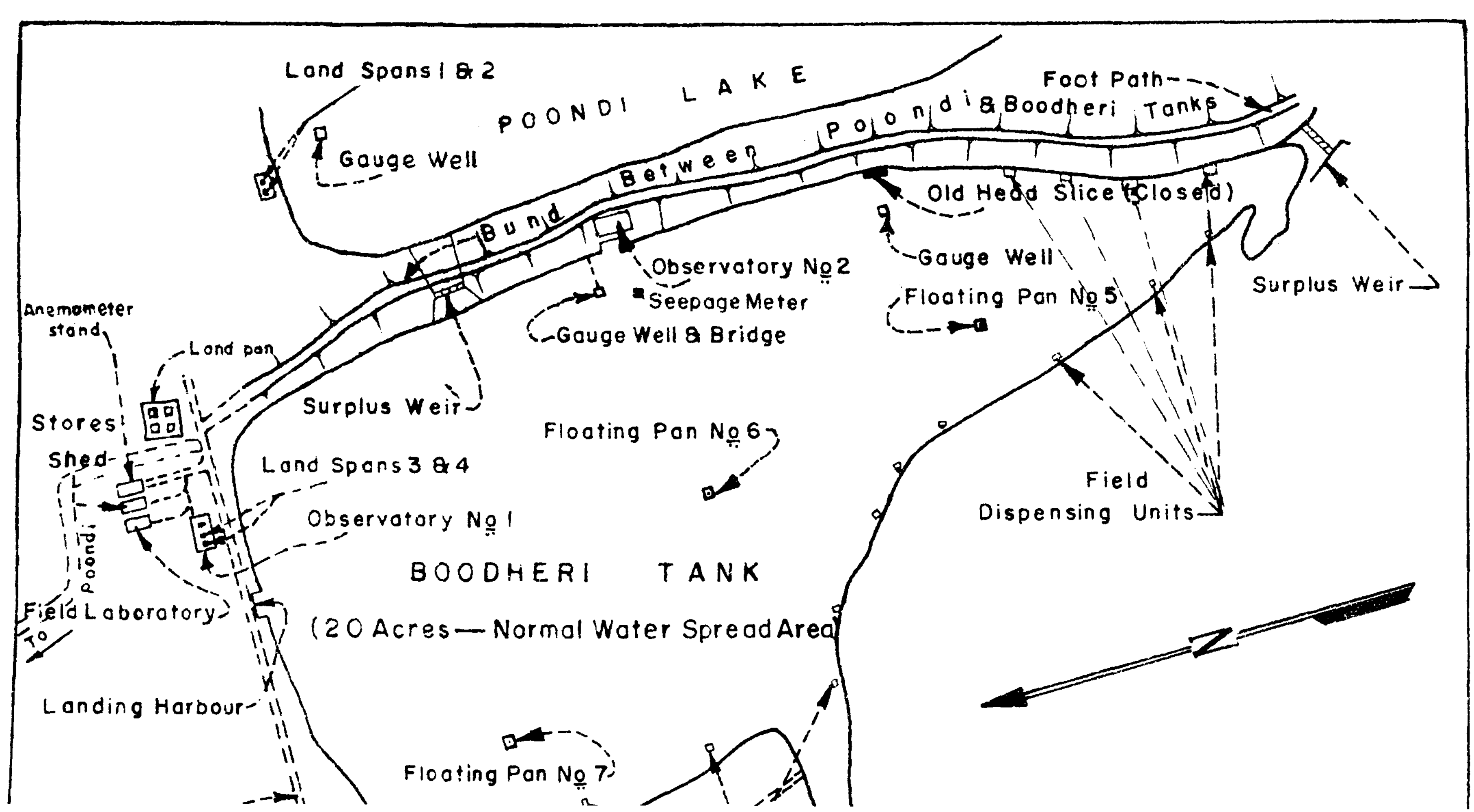
The percentage of savings worked out for the various quantities of the chemical are given in Statement-4. This statement also contain the percentage of savings obtained from the various treated pans. These pans have been compared with similar pan left untreated. The percentage of savings retained from Buderi tank for the various amount of chemical sprayed worked out in the basis of 16 per acre per day is plotted and is shown in Figure-9.

8.3. Statement No. 5 show the waterspread area of the tank during these periods of spraying and also the quantity of water lost tgriyg evaporation alone of the tank has been left untreated. The amount of water saved due to treatment with various quantities of the chemical and the details of the cost of operation are also given in this statement. The water levels and capacity of Buderi tank for the period between February and May for both conditions i.e., with the treatment given and without any treatment are shown in Figure 10.

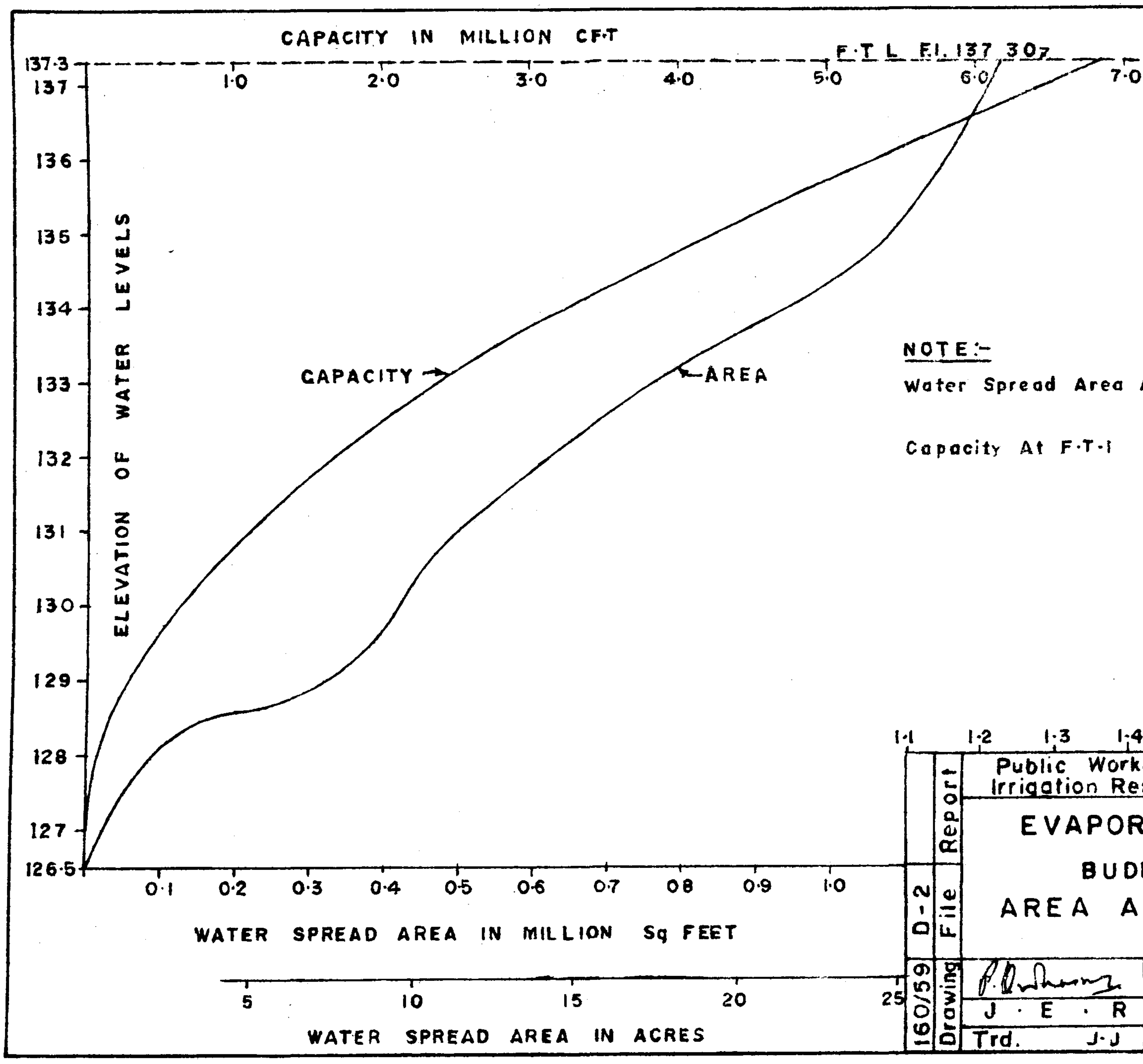
It can be seen from these results that a certain quantity of valuable water can be saved by the use of this chemical. It is felt that some more detailed studies should be conducted before this process is recommended for large scale use for conserving water from open storages.

8.4. Scope for further Research :

As seen in Figure 9, the percentage of savings in evaporation loss that can be achieved by using the chemical is proportional to its quantity used upto an optimum value which has yet to be determined. The solvent, mineral turpentine, is a carrier of the chemical for its effective spreading over the water surface, and it forms a considerable slice of the recuring expenditure in the entire operation. This solvent gets evaporated after serving its intended purpose, and so it is worth while considering the feasibility of either replacing this solvent by a cheaper material or restricting its use to the minimum so as to make the scheme economically feasible. An emulsion with water is a possible substitute and this aspect will have to be studied in the course of carrying out further experiments.



Land Spans 1 & 2
POONDI LAKE
Gauge Well
Bund Between Poondi & Boodheri Tanks
Foot Path
Old Head Slice (Closed)
Surplus Weir
Observatory No 2
Seepage Meter
Gauge Well & Bridge
Gauge Well
Floating Pan No 5
Anemometer stand
Land pan
Stores Shed
Surplus Weir
Floating Pan No 6
Field Dispensing Units
Land Spans 3 & 4
Observatory No 1
BOODHERI TANK
(20 Acres — Normal Water Spread Area)
To Poondi
Field Laboratory
N
Landing Harbour
Floating Pan No 7



NOTE:-

Water Spread Area At F·T·L – 12,29,400 sq.ft. Or 28·2 Acres.

Capacity At F·T·L 6·86 Million cft.

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Public Works Department, MADRAS.
Irrigation Research Station POONDI

Report File D-2

EVAPORATION STUDIES

BUDERI TANK

AREA AND CAPACITY CURVES

Drawing 160/59	J·E·R	A·R·O	E·E·R
	Trd. J·J	Ckd.	Fig.2

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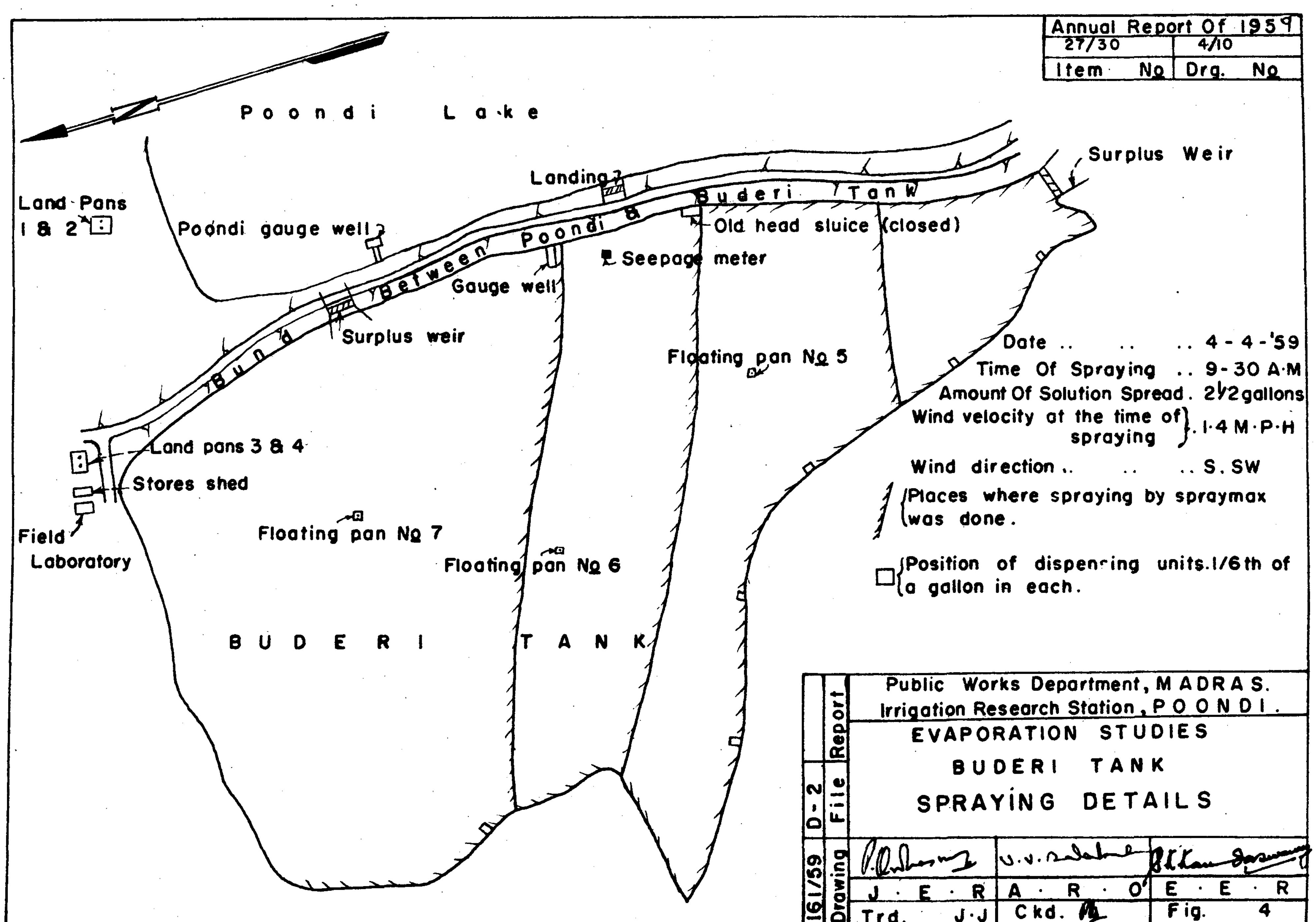

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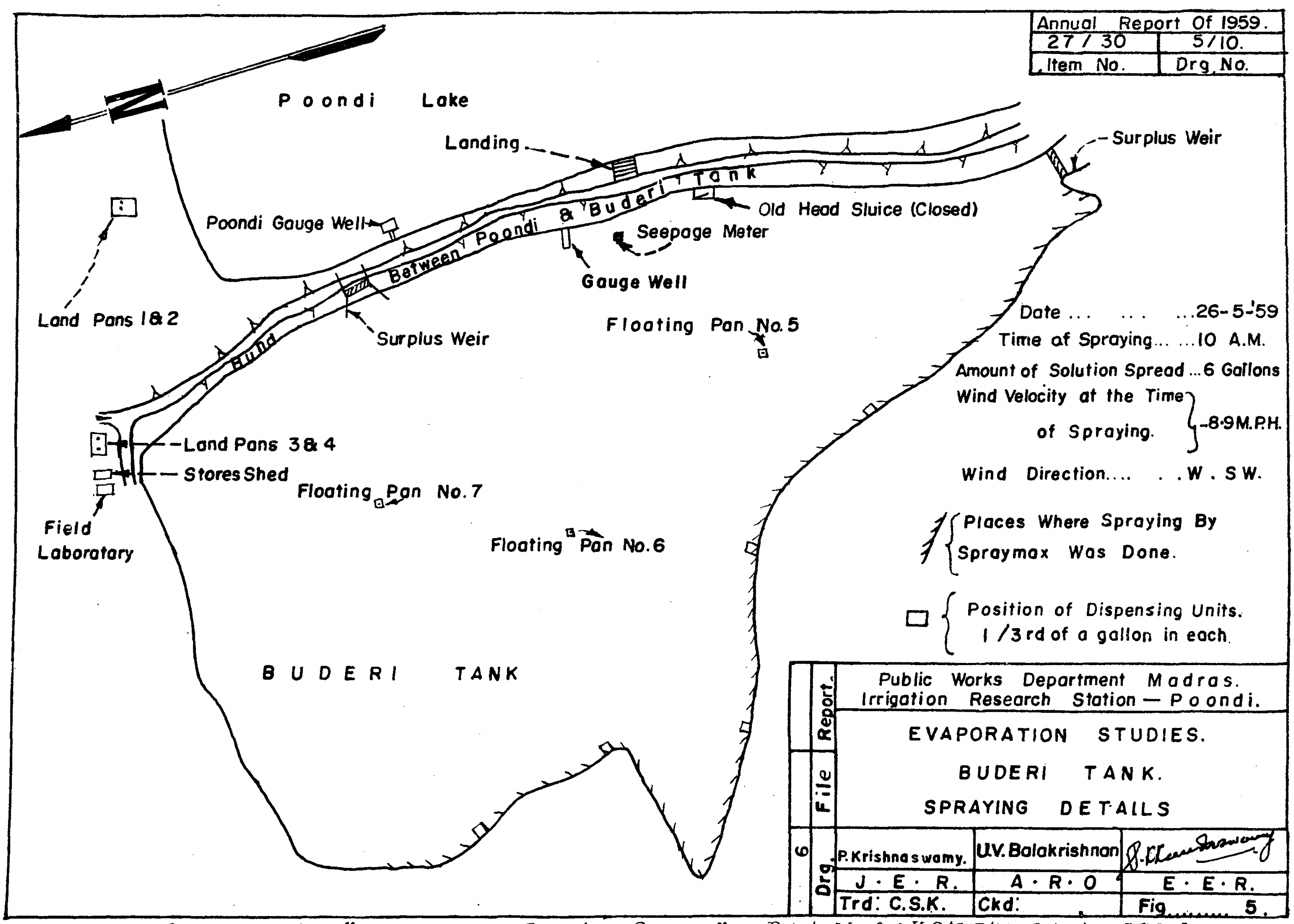

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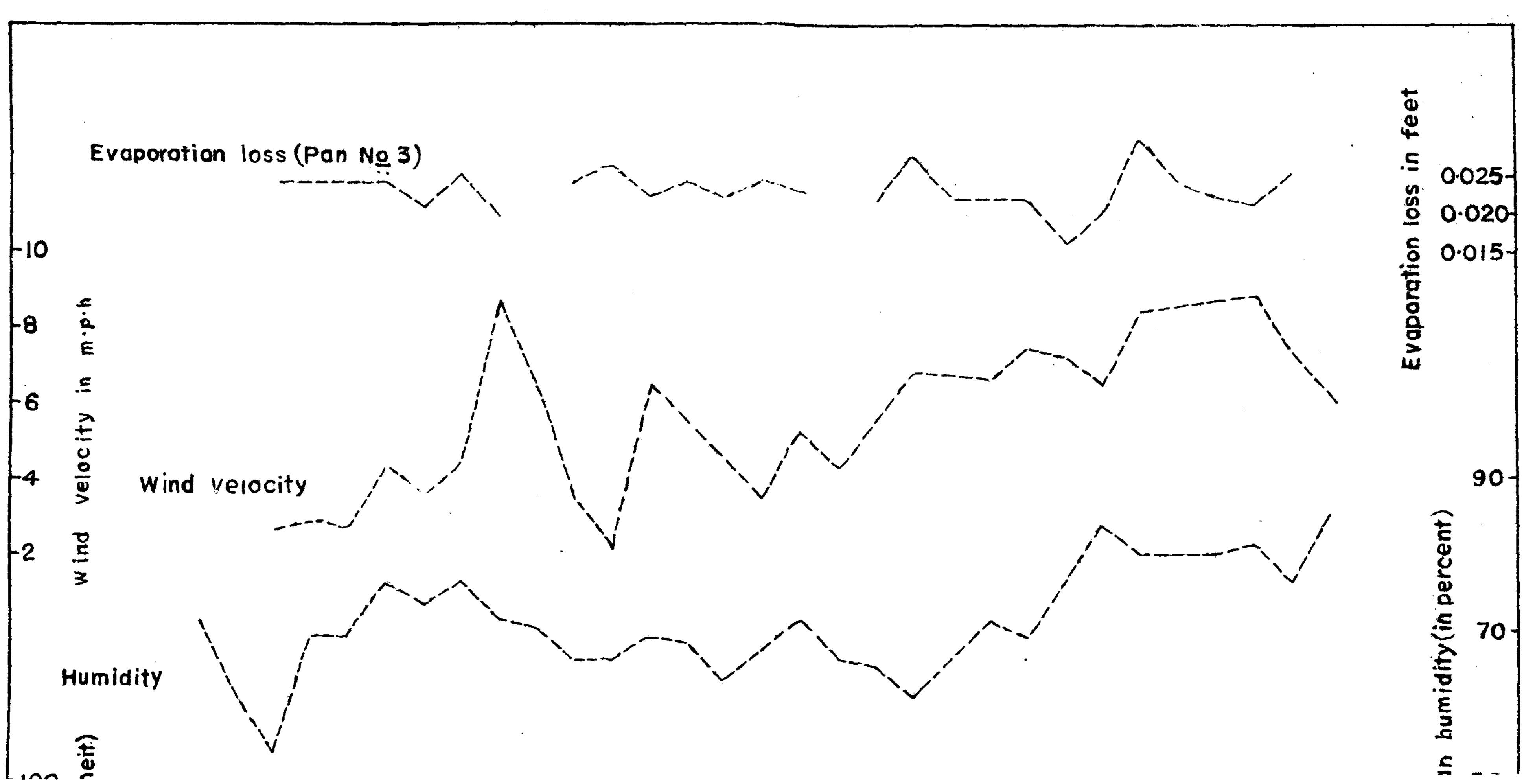
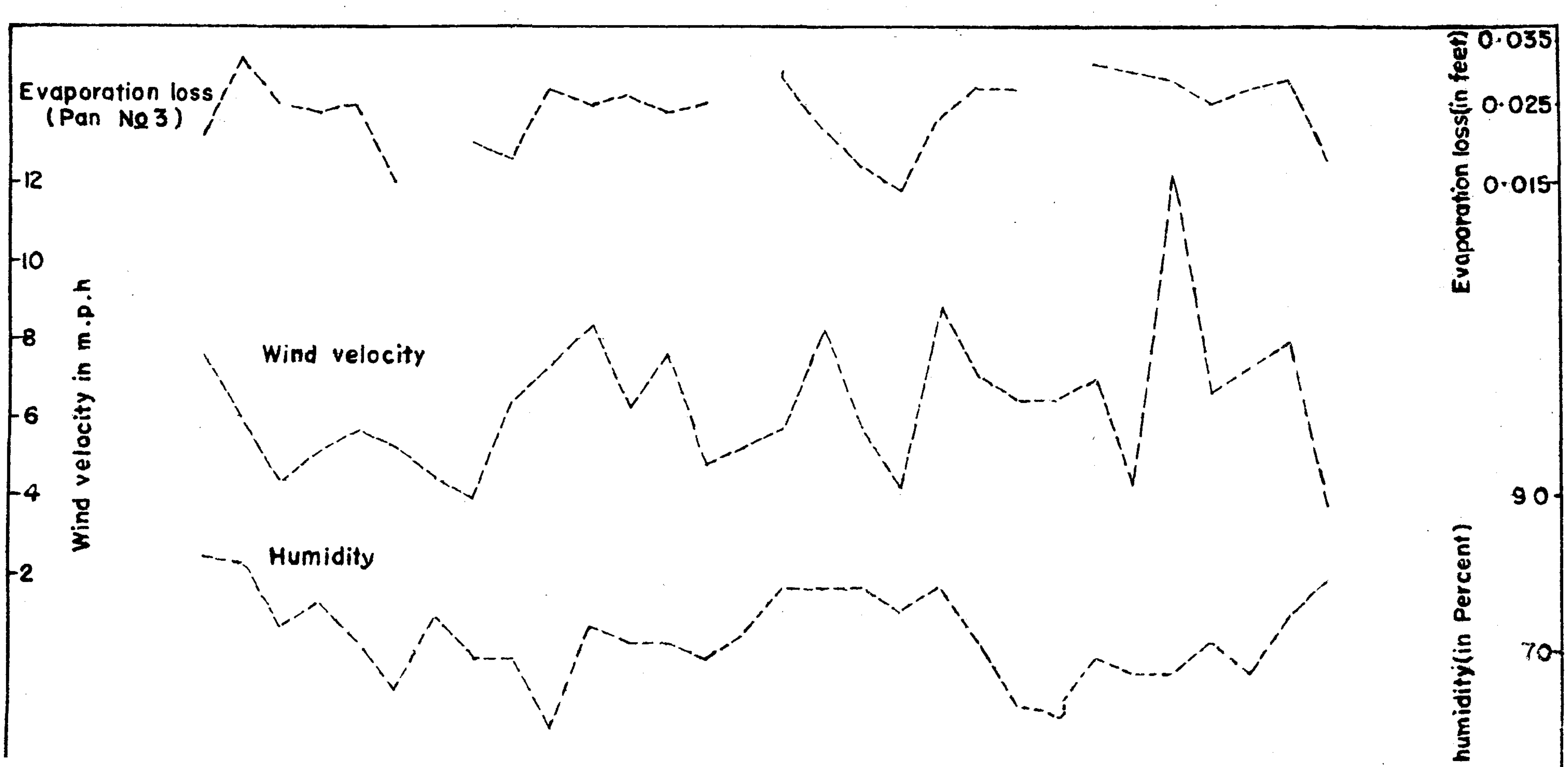
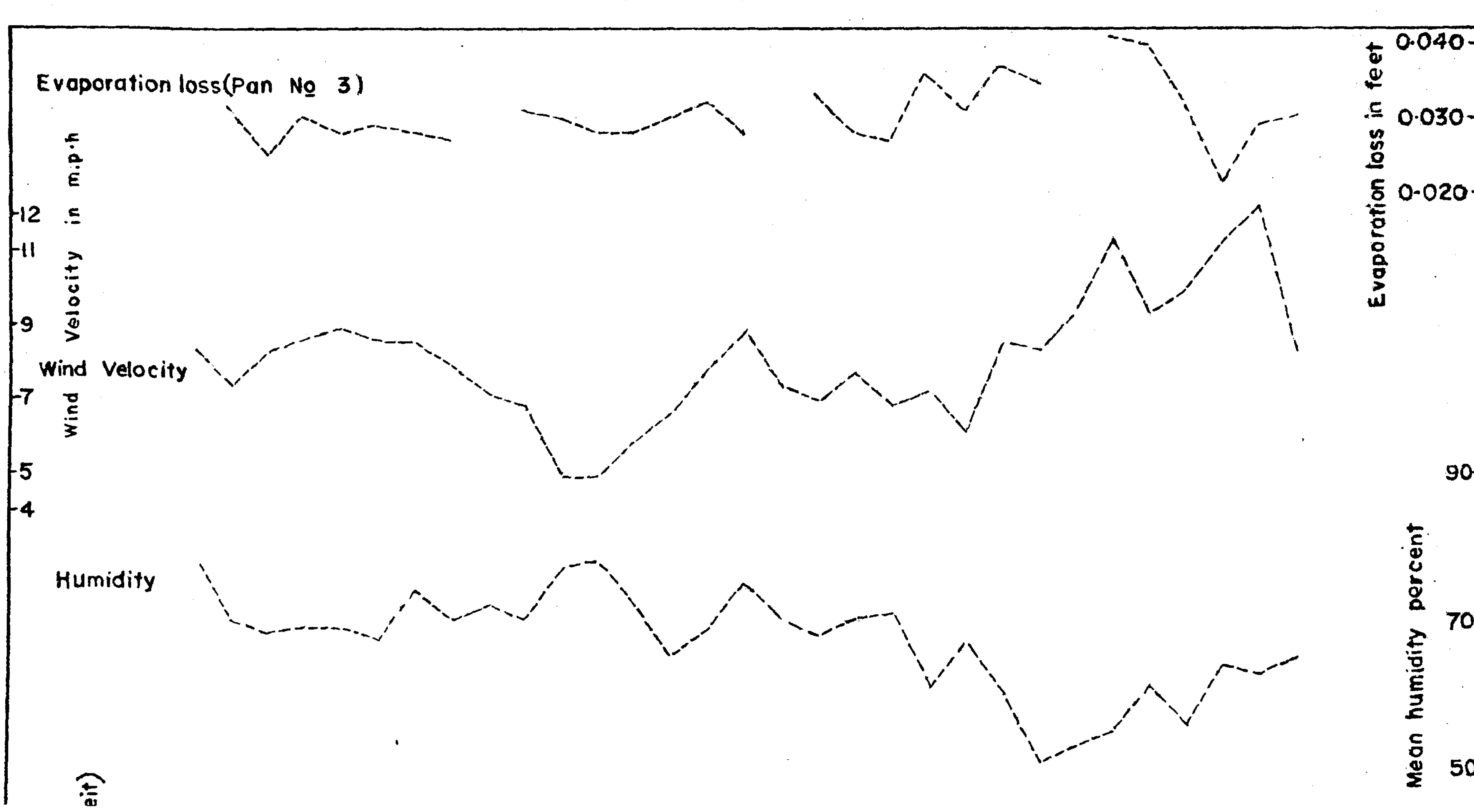
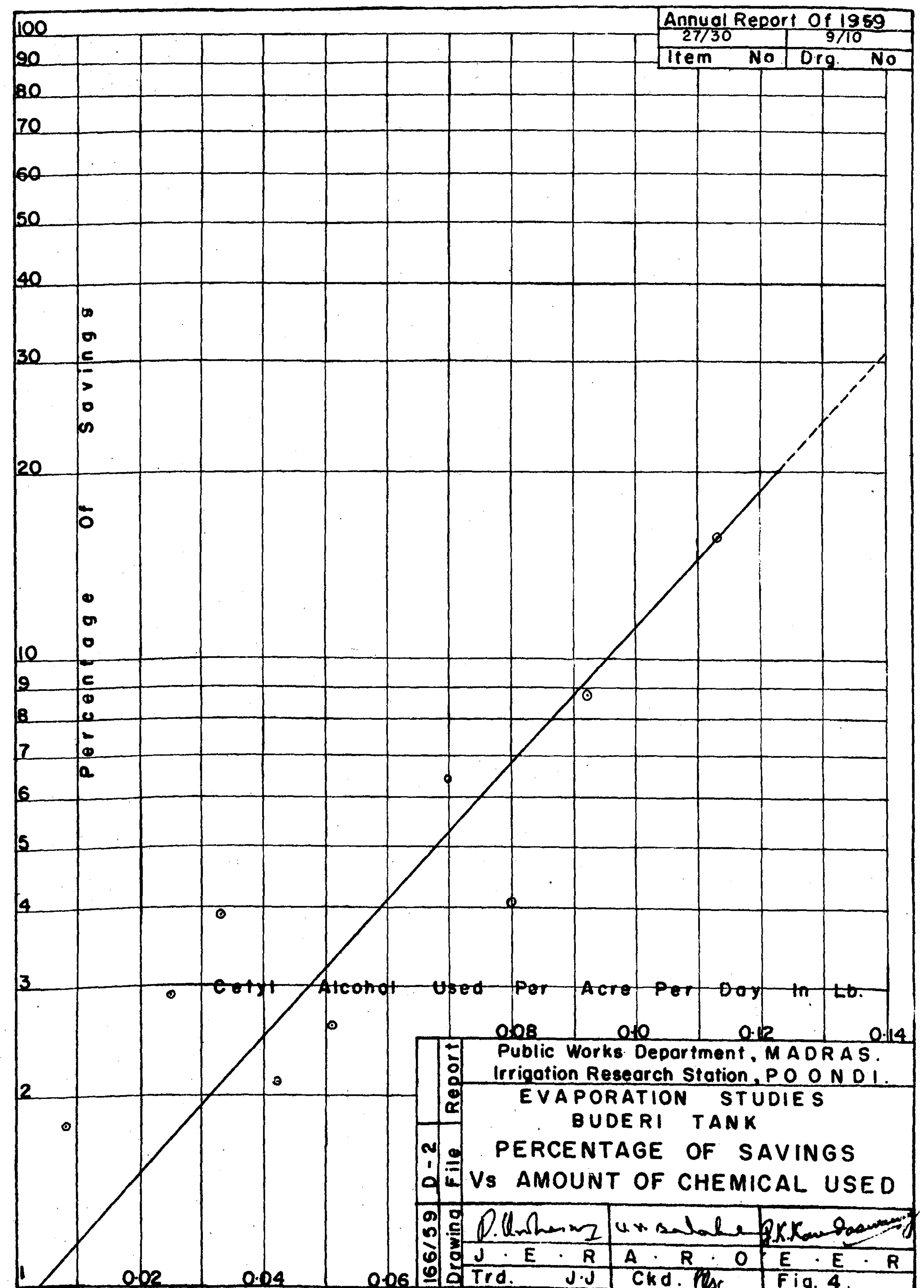


Evaporation loss (Pan No 3)
Wind velocity
Humidity
Wind velocity in m.p.h
10
8
6
4
2
Evaporation loss in feet
0·025
0·020
0·015
90
70
in humidity (in percent)


Evaporation loss
(Pan No 3)
Wind velocity
Humidity
Wind velocity in m.p.h
12
10
8
6
4
2
Evaporation loss(in feet)
0·035
0·025
0·015
humidity(in Percent)
90
70


Evaporation loss(Pan No 3)
Wind Velocity
Humidity
Wind Velocity in m.p.h
12
11
9
7
5
4
Evaporation loss in feet
0·040
0·030
0·020
Mean humidity percent
90
70
50
eit)



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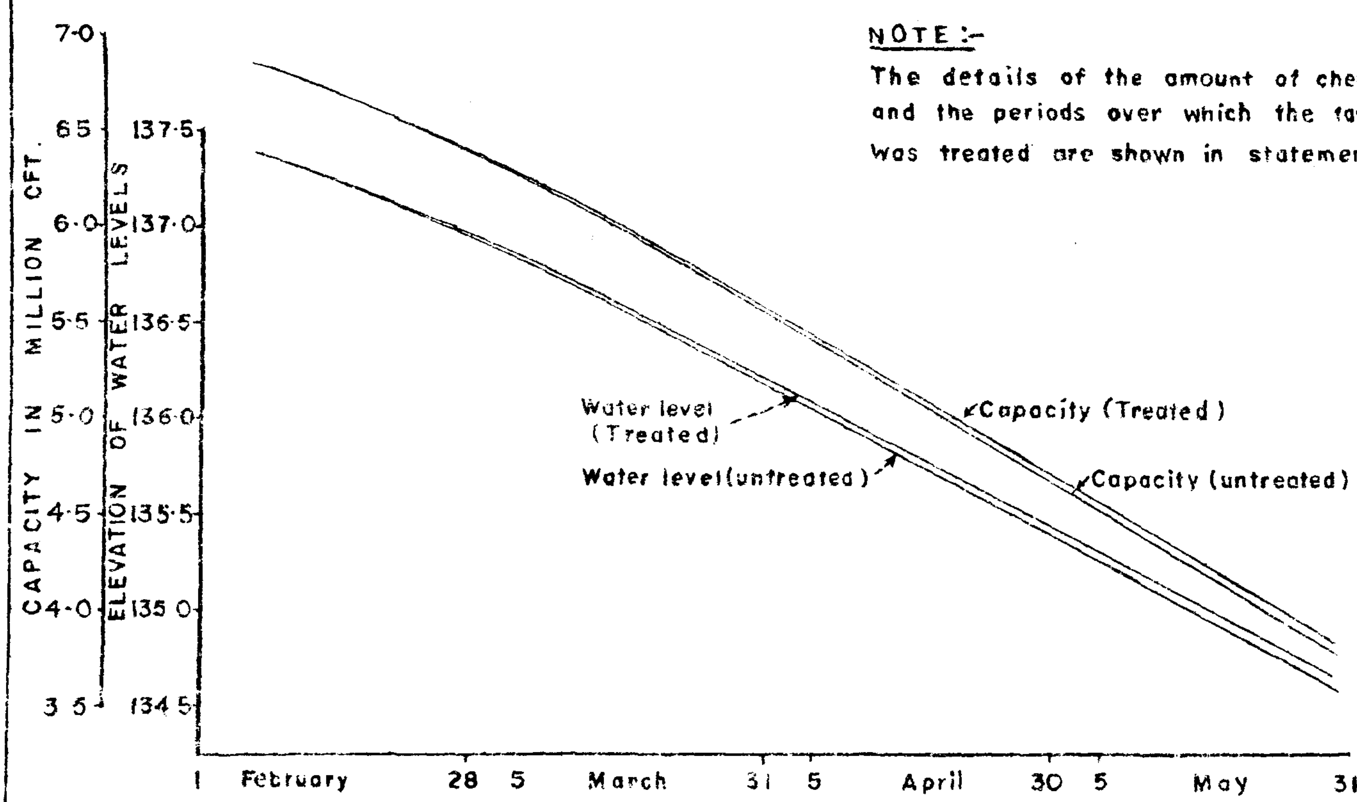
NOTE :-
The details of the amount of chemical used and the periods over which the tank surface was treated are shown in statement 5



Report	Public Works Department, MADRAS Irrigation Research Station, POONDI EVAPORATION STUDIES BUDERI TANK			
File D-2	WATER LEVEL AND CAPACITY WITHOUT TREATMENT & WITH TREATMENT			
Drawing 167/59	J·E	R	A·R·O	2·E·R
	Trd	J·J	Ckd.	Fig 5

LIST OF SYMBOLS

q — quantity of seepage per foot width
q_O — quantity of seepage per foot width through the dam
q_F — quantity of seepage per foot width through the foundation
V — Velocity of flow
K — permeability of the soil
K_O — permeability of the material of the dam
K_F — permeability of the material of the foundation
K_C — permeability of the material of the core-wall
K — permeability of the material of the shell
μ — K_C / K_S
H — upstream head
h — tail water depth
m_1 — upstream slope
m_2 — downstream slope
m — slope of the core-wall
$\propto$ — downstream toe angle of arth dam
b — top width of the dam
B — bottom width
d — depth of the foundation
D — depth of the cut-off wall
β — distance of cut-off from the heal of the dam
l — length of the path of percolation
P_Q — percentage reduction of seepage quantity due to core-wall slope
ϕ — function denoting equipotential lines
ψ — function denoting flow lines
$n\psi$ — number of flow tubes in a flow net
$n\phi$ — number of potential steps
S — shape factor $\frac{n\psi}{n\phi}$
R — Resistance of the electrolyte
ρ — specific resistance of the electrolyte
l' — length of the conductor
A' — cross sectional area of the conductor
g — acceleration due to gravity
σ — surface tension of water
γ — specific weight of water
R — Reynold's number
W — Weber's number
F — Froude's number

SUB-SOIL FLOW-EARTH DAMS

Synopsis :

The seepage through earth dams depends on many factors which include the type of material used and the method of construction of the dam, the type of foundation available, the depth and location of cut-off walls, the location and slope of the internal core walls, and the drainage measures provided etc. Experimental studies have been conducted to find out the effect of these various factors on the seepage quantity. These results are compared with the results obtained by the electrical analogy method and with the computed values using a few of the well-known formulae.

1. Introduction:

1. 1. The advance of the modern soil mechanics has opened a new field of research for the irrigation engineers mostly useful for the design and construction of earth dams. However, to compute the quantity of seepage through earth dams, at present there are only a few empirical formulae based upon many assumptions and approximations. If a number of experimental techniques of precise nature could be conducted on a rather wide range and thorough basis, a reasonably sound theory could be proposed for this phenomenon of seepage. Furthermore, rational charts and simplified formulae could be evolved. With this objective in view, experiments were conducted both by hydraulic model studies and electrical analogy techniques and the following pages present the outcome of such an attempt.

2. Seepage Flow:

The filtering of water through the granular soils of the earth bund is called the seepage flow. The seepage phenomenon has got the following important significance.

2. 1. A certain quantity of useful storage is lost by way of seepage. But unless the foundation is very porous and thepermeability of the material of construction of the bund is very high, this loss is not bound to be of any great importance.

2. 2. A considerable portion of the dam gets saturated and this reduces appreciably the safety of the side slope.

2. 3. If the seepage water issues from a relatively higher point on the downstream side in the form of a spring the finer particles are likely to be washed away and this may lead to the failure of the dam.

2.4. If the material of the dam is very fine and clayey and if the seepage is appreciable, this will cause "Sloughing" on the downstream side and certain remedial measures may have to be adopted.

2. 5. If the quantity of seepage is more and if suitable drainage measures have not been provided the lands adjacent to the bunds may become water logged.

3. Dimensional Analysis of seepage Flow:

3. 1. The various factors affecting the seepage quantity q per foot length of dam are as follows:—

(1) Permeability of the material of the dam Ko (the horizontal and vertical permeabilities are assumed to be the same).

(2) the top width of the dam b

(3) the bottom width of the dam B

(4) the upstream slope m_1 (if there are berms, the average slope is to be taken into consideration)

(5) the downstream slope m_2 (if there are berms and different slopes, the average slope should be taken)

(6) the depth of water on the upstream side H

(7) the depth of tail water h

(8) the depth of foundation d

(9) the depth of the cut-off provided D

(10) the permeability of the foundation material K_F

(11) the factor to represent foundation stratification $\propto$

(12) the distance of the cut off from the heel of the dam β

(13) the specific weight of water

(14) the dynamic viscosity of water μ

(15) the surface tension of water σ

(16) the acceleration due to gravity g

(17) the velocity of flow V

Writing down these different quantities in the form of a function

$$f\,(q,\ K_1,\ b,\ B,\ H,\ h,\ m_1,\ m_2,\ d,\ D,\ K_F,\ \propto,\ \beta,\ \gamma,\ \mu,\ \sigma,\ g,\ V) = 0$$

Combining these eighteen physical quantities. by the principle. of dimensional analysis,

$$q = \phi\left(F, R, W, K_1, K_F, \frac{b}{H}, \frac{B}{H}, \frac{h}{H}, m_1, m_2, \frac{d}{H}, \propto, \frac{D}{H}, \frac{\beta}{H}\right) \text{ where}$$

R = Reynold's number
W = Weber's number
F = Froude's number.

Froude's number may not have much influence but still it is included in the functional relationship to make the study complete in all respects.

The effect of each variable on q was studied by putting up a number of hydraulic models.

4. Permeability and Permeameters:

The seepage equantity through the earth dam depends on the coefficient of permeability which is determined by the voids present in the soil. Two permeameters were constructed in the local workshop for the measurement of the permeability coefficients.

4. 1. Variable head permeameter (Photo 1)

4. 1. 1. This is used for finding out permeability of soils of very low permeability such as silty clay or clay. This permeameter consists of a steel cylinder 6 inches internal diameter and 8 inches high (Figure-1). It is surmounted by a cone at the top. A tube is fixed at the top of the cone, provided with a stop cock. The wooden scale graduated in inches gives the head of water acting on the sample from the beginning of the graduations. To get the total head over the sample a value of 16·5 inches should be added to every reading

4. 1. 2. At the bottom the water is collected in a circular tank 12 inches in diameter ond 4 inches in height. It is also provided with an overflow pipe to maintain a constant water level. Steel plates with $\frac{1}{8}$ inch diameter holes, closely spaced are provided at the top and bottom of the sample instead of conventional porous plates.

4.1.3. The permeameter cylinder is filled up with the sample and given the same degree of consolidation as in the model. It is saturated fully by keeping it immersed in water for 24 hours. Then it is fitted in position and the head observations are made at an interval of half an hour for 24 hours.

4. 1. 4. The permeability is computed by using the formula:

$$K = \frac{a}{A} \times \frac{l}{T} \times 2{\cdot}3 \ \text{long}_{10} \frac{h_1}{h_2}$$

where K = permeability coeffieient

a = the area of cross section of tube

A = the area of cross section of the sample

l = thickness of the sample

h_1 h_2 = heads at the beginning and at end of time interval of T.

The average of the several calculated values is taken as the permeablility value for the material of the dam. In the same way, the permeability in the vertical direction also can be found with certain modifications. A sample sheet of computation is given in the S tatement No. 1.

4. 2. Constant head permeameter (Photo-2)

4. 2. 1. This is used for finding the permeability coefficient of sandy soils. It consists of a steel cylinder six inches internal diameter and eight inches high (Figure-2). Two copper tubes each 1/4 inch diameter are provided with a central spacing of six inches. At the top and bottom suitable covering lids are provided with porous discs covered with fine mesh. Suitable manometric set up has been constructed to find out the head lost between the two tappings.

4. 2. 2. The sample whose permeability is to be found out is made saturated with water and is taken in the cylinder. The connections are made and water is allowed to flow up through the sample till steady conditions are obtained. The discharge through the sample is found out by collecting it in a measuring jar for a known interval of time. The loss of head between the two tappings is recorded.

4. 2. 3. From the Darcy's law it can be obtained that

where K = coefficient of permeability

V = Volume of water collected for the time 't' secs

A = Cross-sectional area of the sample

l_1 = distance between the two pressure outlets

h = difference in head between the two manometric tappings

Permeability experiment have been conducted for various sizes of sand. The sample computation and the results are given in the Statement No. 2.

5. HYDRAULIC MODEL — FOR A HOMOGENEOUS SECTION

5. 0. 1. Variation of q with h/H

A hydraulic model of the standard homogeneous section shown in Figure-3 was constructed with sand inside a glass flume (Photo-3). The seepage was measured by collecting the seepage quantity in a measuring jar for a known interval of time. The depth of the tail water was maintained at any desired level by means of a tail gate and the variation of q with $\frac{h}{H}$ was obtained (Figure-4).

5. 0. 2. The variation of q with $\frac{h}{H}$ was also analysed theoretically using Dupuit's equation,

$$q = \frac{K}{2l}(H^2 - h^2)$$

where K = permeability coefficient

l = Length of the path of percolation

H = upstream head

h = tailwater depth.

It was seen that Dupuit's equation does not give a true picture of the seepage flow since in a certain range q increases with any increase in h instead of decreasing as shown in the figure. The same can also be analysed by slightly modifying Kozeny's basic equation which is given below :

$$q = 2 \propto a_o$$

$$a_o = \sqrt{d_1^2 + H^2 - d_1}$$

where d_1 = horizontal distance in the Casagrande's equation

H = upstream head.

5. 1. *Hydraulic model technique*

5. 1. A sandy model of a standard section was constructed with 14-18 sand in a masonry flume with the sides made rough to simulate the continuous boundary (Photo-4).

5. 1. 2. Due to excessive seepage velocities there was tremendous amount of piping at the toe of the model dam. Various devices were tried to arrest this piping without interferring with the seepage phenomonon. Finally red gravel retained on sieve No. 4 was spread on the downstream slope of the sandy models to the desired height and this successfully avoided piping (Photo 5). Further, the single layer of gravel offers a friction which is negligibly small and hence it does not affect the seepage quantity.

5. 1. 3. To suppress the soil evaporation; a wet cloth was spread over the dam to cover the surface exposed to atmosphere. This cloth was kept moist so that the capillary water was not allowed to get evaporated and hence the continuous vertical movement of water was not allowed to get evaporated and hence the continuous vertical movement of water was prevented (Photo 6.) This device was adopted on the advice of Sri D. V. Jogleker on the occasion of his visit to the Station.

5. 2. *Variation of q with m_1*

5. 2. 1. The upstream slope m_1 of the model was varied from 1 on 2.5 to 1 on 5 and in each case ths seepage quantity was measured. The same was also analysed theoretically using Casagrande's equation (Figure-5).

$$q = K a \sin^2 \propto$$

where $a = \sqrt{H^2 + d_1^2} - \sqrt{d_1^2 - H^2 \cot^2 \propto}$

d_1 = the horizontal diatance in the Casagrande's equation.

$\propto$ = downstream toe angle.

The correlation between the experimental and theoretical results is fairly satisfactory.

Variation of q with m_2 can be also studied in the similar way.

5. 3. *Variatiou of q with $\frac{b}{H}$*

5. 3. 1. In the same model the variation of q with $\frac{b}{H}$ was studied for values of $\frac{b}{H}$ ranging from 0.10 to 0.40 and the results are plotted and shown in Figure-6. As the value of $\frac{b}{H}$ increases the seepage gets reduced and the trend of variation seems to be more or less in a straight line with negative slope

5. 4. *Cut-off walls in pervious foundations* :

It was attempted to find out the effect of the depth and location of a cut off in a pervious foundation with a homogenious dam sections of the same permeability (Figure-7). If 'd' is the depth of the pervious stratum and 'D' be the depth of the cut-off wall, then the ratio $\frac{D}{d}$ is plotted against the corresponding seepage quantities (Figure 8). As a result of the experiments it was concluded that when a homogeneous dam is constructed over a pervious foundation, the cut off walls does not have any appreciable effect on the seepage, unless there is an impervious core wall also. However further experiments will be conducted to find out the effect of cut-off walls in deep pervious stratum,

5. 5. *Dams curved in plan* :

This set of experiments was initiated to find out the effect of curvature on the quantity of seepage in the case of dams which are curved in plan. If R represents the radius of curvature of the dam and H the upstream head, then the nature of variation of q with $\frac{R}{H}$ was studied by varying $\frac{R}{H}$ from 1. 0. to 0. 1. The radius of the dam was varied from 10 feet to 100 feet, and the variation of q with $\frac{R}{H}$ is shown in Figure-9. The results were not satisfactory and were not conducive to further analysis.

5. 6. *Variation of q with $\frac{d}{H}$*

5. 6. 1. When an earth dam is constructed over a previous foundation, there is seepage through the dam and the foundation as well. The seepage through the foundation is calculated assuming that the

foundation acts like s pipe through which seepage occurs. The foundation is imagined as a tube discharging under a head of H feet.

Seepage quantity through the foundation per foot length $q_F = k_F \, d \frac{H}{l}$

where k_F = permeability coefficient of the foundation material

d = depth of foundation

$l = b + (H + F.B)\, m_1 + (H + F.B) m_2$.

for explanation of the symbols the difinition sketch may be referred (Figure-10). NOTE. F. B. = Free Board

The seepage through the dam q_0 is calculated by the equation

$$q_0 = k_0 a \sin^2 \alpha$$

where $a = \sqrt{H_1 + d_1^2} - \sqrt{d_1^2 - H^2 \cot^2 \alpha}$

k_0 = permeability of the material of the dam

d = Horizontal distance in the Casagrande's equation.

α = downstream toe angle.

The ratio $\frac{q_F}{q_0}$ is computed theoretically and this is plotted against $\frac{d}{H}$ where d in this case refers to the depth of the foundation and H the upstream head (Figure-10).

5. 6. 2. Hydraulic experiments were conducted with different depths of foundation so that $\frac{d}{H}$ varies from 1.5 to 0. In all cases the ratio of the permeabilities of the dam and foundation was kept as 1.0. The experimental results are shown along with the theoretical values in the Figure-11. The graph also presents results of the electrical analogy flow net method and specific resistance method, the details of which are described elsewhere in this report.

6. Zoned Sections

In the experiments zoned sections the shell portion was moulded with 24-30 sand or 40-60 sand and the core wall portion was built with Poondi red soil. It has been found that with the model head of 1 foot there is practically no seepage through the core wall and so far all studies in the model "Poondi red soil" can safely be assumed to be totally impervious.

6. 1. *Impervious core wall on pervious foundation*

A model of the standard section was moulded with 24—30 sand on a pervious foundation of 30 feet depth. The foundation was also moulded with 24—30 sand (Figure-12). The seepage through the model was found out. Then, a vertical core wall was constructed with Poondi red soil and the quantity of seepage q_0 was determined. The slope of core wall was gradually increased till it became 1 on 1, and the seepage quantity q_x was found out. The percentage reduction in seepage is given as a ratio $\frac{q_x - q_0}{q_0}$. The percentage reduction of seepage was plotted against the

slope of the core wall in a log-log sheet and the equation of the curve was found to be $PQ = 43m^{0.16}$ (Figure 13).

where P_Q = percentage reduction of seepage

m = slope of the core wall.

It was thought that the depth of the previous foundation may have some effect on the equation. So the experiment was repeated with a foundation depth of 90 feet. The results are shown in the same figure and the correlation equation was found to be $P_Q = 30^{\,0.16}$. From these cases it follows that only the multiplying factor varies and it is more for lesser foundation depth.

6. 2. *Core wall of finite permeability on impervious foundation*

6. 2. 1. It was proposed to study the effect of core wall slope on the seepage reduction for dams resting on level impervious foundations with core wall of finite permeability. To investigate this the shell of the standard dam section was made of gravel passing through No. 4 sieve and retained on No. 10 sieve. The core wall was made of sand of different sizes to have different permeability ratios. The results of such experiments are shown in Figure 14. The equation of the family of curves is $PQ = c_1 m^{C_2}$. The value of c_2 is found to be 0.42 in all these cases. The value of the constant c_1 varies with permeability ratios.

6. 2. 2. Experiments were conducted using different materials for the core wall for different slopes and in each case the seepage quantity was found out. The percentage reduction could be evaluated as mentioned in the earlier paragraph, and in each case a graph was plotted showing the relationship of the core wall slope with percentage reduction of seepage (Figure 14). The equations correlating percentage reduction of seepage and core wall slope for different permeability ratios were found out. The variation of the constant c_1 with the ratio of permeability between the shell and the corewell is shown plotted in Figure 15. From the graph it is inferred that C^1 varies in proportions with μ, (the ratio of permeability) and the equation is found to be $c^1 = 1{\cdot}2\mu + 13{\cdot}6$.

So, the general equation for the family of curves correlating seepage reduction and the slope of the corewall for different permeability ratios is got as

$$P_Q = (1{\cdot}2\mu + 13{\cdot}6)\, m^{0{\cdot}42}$$

An alternative method of plotting these results has also been presented in (Figure 16).

7. Electrical Analogy Method :

7. 0. 1. Two dimensional Electrical analogy studies were conducted to obtain equipotentials and hence the flow nets for different sections tested. The basic principle that electric and hydraulic potentials are similar in nature is obvious. It follows consequently that for similar of boundary conditions, the equipotentials must be the same in both cases.

7. 0. 2. Electrical analogy studies were conducted for homogeneous earth dames, having a permeable foundation of 90 feet depth. The phreatic line has to be fixed approximately in the electrical model. This was accomplished by Casagrande's method. The electrical analogy method of fixing the phreatic line is not very reliable and sufficiently accurate unless a very

high resistance with uniformity is used as the electrolyte. The fixing of the phreatic line in the model as per Casagrande's method is detailed in Figure-17.

7. 0. 3. The model was constructed in teak wood and plasticine was used to make it water proof. Water slightly acidified is used as the electrolyte. The upstream slope and downstream toe were taken as 100 percent and 0 percent equipotentials respectively and these were simulated in the model by thick copper plates. An A. C. 220 volts stepped down to 6·3 volts by means of a transformer was fed across the two electrodes. The 6·3 volts were distributed across a potentiometer which was used as a potential divider. The potentiometer was divided into twenty parts each representing 5% of the total drop across the terminals. A probe carried by a pentograph for exploring the equipotentials was connected to the input terminals of the vertical amplifier of the Cathode ray Oscillograph which was used as a null indicator. A connection was taken from the other terminal of the vertical amplifier to the adjustable part of the potentiometer. A marking stylus was attached to the other end of the pentograph. As the probe was moved in the electrolytic tray the equipotential points wers explored which were indicated by the least variation in the beam of the oscillograph. All such points were marked in a graph sheet by pressing the stylus. Equipotential lines were obtained by joining these points by smooth lines. Flow net patterns were accomplished by drawing flow lines orthogonal to the equipotential lines (Figure-19).

7. 1. Flow nets and calculation of seepage quantities :

7. 1. 1. Flow nets are applicable to simple cases of irrotational flow which is satisfied by the Laplace's equation

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0$$

This equation represents two family of curves intersecting at right angles. Applying this to this particular use of electrical analogy.

$$\frac{\delta^2 H}{\delta \phi^2} + \frac{\delta^2 H}{\delta \Psi^2} = 0$$

where ϕ and Ψ are functions relating to the stream lines and equipotential lines of the flownet.

7. 1. 2. The flow pattern in the case of percolation through earth dams depends on the difference in head 'H' between the upstream and downstream water levels. Since the potential drop is the same between the two equipoential lines, the drop in H is divided along any flow line into equal increments $\triangle H$; which is equal to $\frac{H}{n\phi}$ where $n\phi$ is the total number of potential steps.

The rate of flow through a unit of one channel per foot length of the dam is $q = K A \frac{dh}{ds}$ (Darcy's law)

where A = area of cross section

$\frac{dh}{ds}$ = piezometric gradient along the flow path

$$q = K\delta n \frac{H}{n\phi/ds}$$

where δn is the width, *ds* is the length of the square in the channel. Since δn = *ds*,

$$q = K \frac{H}{n\phi}$$

Therefore for $n\Psi$ channels each carrying a flow q the total flow

$$Q = K H \frac{n\Psi}{n\phi}$$

Using this formula for the seepage quantity, the seepage flow was computed from the flow net obtained for the homogeneous earth dam section by electrical analogy method. In the section experimented upon, the foundation depth was reduced from 90 feet to 0 feet by steps of 6 feet and in each case the flow net as drawn.

7. 1. 3. If 'q' is the seepage flow through the dam and foundation and if 'q_0' is the seepage through the dam alone, then q^F is equal to $q - q_0$.

$$\text{Therefore } \frac{q_F}{q_F} = \frac{q - q_0}{q_0} = \frac{\$ - \$_0}{\$_0}$$

where \$ and $\$_0$ are called shape factors ($n\Psi/n\phi$) representing the ratio of the flow tubes to the potential steps in each case respectively. The value of $\frac{q_F}{q_0}$ thus found out is plotted against $\frac{d}{H}$ in the same graph along with the the hydraulic experimental values for the same model (Figure-11).

7. 2. Specific resistance method :

However, another interesting and simple electrical method is available for finding out the seepage quantity alone, even though the nature of flow may not be explored in detail. The shape factor $\frac{n\Psi}{n\phi}$ may be directly deduced without constructing the flow net, by measuring the resistence of the electrolyte between the 100 per cent and 0 per cent equipotential boundaries and also determining the specific resistance by standard methods for a sample of a electrolyte used in the tray. It can be shown that $\frac{n\Psi}{n\phi} = \frac{\rho}{R}$, as given below, where ρ is the specific resistance and R is the resistance of the electrolyte between the 100 per cent and 0 per cent equipotentials:

The seepage through a porous medium of uniform irrotational flow is given by $q = K A \frac{H}{l}$

The specific resistance in electricity in given by the relation $\rho = \frac{RA'}{l'}$

R = Resistance of conductor.

A' = Area of cross section of the conductor

l' = length of the path taken for the flow of electricity.

$$\frac{\rho}{R} = \frac{A'}{l'}$$

Since the electrical flow and hydraulic flow are analogous under irrotational flow condition $\frac{A'}{l'}$ may be subsituted for $\frac{A}{l}$

$$\because q = K H \frac{\rho}{R}.$$

From the flow net q is found to be

$$q = K H \frac{n\Psi}{n\phi}$$

$$\therefore \frac{n\Psi}{n\phi} = \frac{\rho}{R} = \$$$

Thus factor \$ is called as the shape factor. Experiments for finding out the relationship between $\frac{d}{H}$ and $\frac{q_r}{q_o}$ were conducted employing this method and the results are shown plotted in the same graph (Figure 11).

8. Conclusions :

8. 1. The various factors affecting the quantity of seepage through the earth dam has been discussed in detail with subsequent reference to the relevant theories. The experiments conducted both in hydraulic sandy models and electrical analogy models prove the satisfactory reliance of the existing theories with slight disparity in some cases. Some of the important conclusions drawn from the series of experiments are summarised below.

8 2. The Dupuits' equation for the various of seepage quantity with the tail water head is not satisfactory when compared with the experimental values while Kozeny's basic equation with slight modifications may yield better agreement. Casagrande's equation for seepage flow, is fairly satisfactory with experimental values obtained for various slopes of the dam. The experiments conducted for finding out the effect of the depth and location of the cut off wall did not give results conducive of any analysis. Further experiments may have to be conducted for cut-off walls in deep previous strata.

8. 3. The variation of the seepage quantity with foundation depth i.e. $\frac{q_r}{q_o}$ with $\frac{d}{h}$ was studied in all the three methods mentioned in the report and was also computed theoretically. Hydraulic experimental values fairly agree with the theoretical results, while electrical methods give slightly higher values. This may be due to the assumption of the same phreatic line, adopted from Casagrande's formula for all foundation depth.

8. 4. Experiments conducted for zoned sections with pervious foundation and impervious core wall indicate that the seepage flow increases with the increase in the foundation depth. The relation between the reduction in seepage due to a core wall of impervious nature as well as core walls having a finite permeability with its previous foundation will be ascertained by putting up some more sandy models, and finding out the seepage quantity for various foundation depths.

8. 5. From the experiments conducted for zoned sectional models on impervious foundation it was found that the seepage quantity varies exponentially with the increase of the core wall slope. The seepage is maximum for a vertical core, decrease rapidly for increase in slopes at lower range, but rather slowly for increase in slopes at higher range. It is learnt that the percentage reduction of seepage due to a core wall slope is greater for higher slope values and for higher values of permeability ratios of the core wall and the shell.

8. 6. On the whole it can be concluded that this paper constitutes the bulk of many hydraulic experiments which hinge their reliance on some of the existing theories. The experiments may have their own limitations and other defects in the mode of construction etc., but the attempt made here offers clue for further research in this field.

STATEMENT I.

Permeability computations for Silty Clay Material

Apparatus used — Variable head permeameter

$$\text{Formula} - K = \frac{a}{A} \times \frac{l_1}{T} \times 2{\cdot}3 \log_{10} \frac{h_1}{h_2}$$

A = cross sectional area of the sample

a = area of cross section of the tube in which the water head stands.

l_1 = distance between the two pressure outlets.

No. (1)	h_1 inch (2)	h_2 inch (3)	T hours (4)	K inch/hour (5)
1	33·625	33·00	4·50	·000852
2	,,	32·875	6·00	·000666
3	,,	32·813	6·50	·000504
4	,,	32·781	7·00	·000467
5	,,	32·688	7·50	·000437
6	,,	32·688	8·00	·000410
7	,,	32·625	8·50	·000386
8	,,	32·563	9·00	·000365
9	41·500	40·688	5·00	·000706
10	,,	40·625	6·00	·000657
11	,,	40.625	6·50	·000606
12	,,	40·500	7·00	·000466
13	,,	40·433	7·50	·000436
14	,,	40·375	8·00	·000409
15	,,	40·313	8·50	·000386
16	37·5	36·313	8·00	·000411
17	,,	36·281	8·50	·000389
18	,,	36·188	9·00	·000367

Average value of K = 0·000494 inch/hour.

STATEMENT II.

Permeability computation for 30-40 sand

Apparatus used — Constant head permeameter

$$\text{FORMULA}: K = \frac{v}{At} \times \frac{l}{h} = \frac{{\cdot}322v}{th}$$

v = volume of water collected in 't' seconds

A = cross sectional area of the sample

h = head loss

No. (1)	h_1 (2)	h_2 (3)	head loss h_2 feet. (4)	V volume of water collected in Cft. (5)	't' sec. (6)	K ft./sec. (7)	K Average ft./sec. (8)
1	242	204	0·38	·0448	60	·000635	
2	236	198	0·38	·0455	,,	·000644	
3	230	172	0·38	·0458	,,	·000658	
4	224	186	0·38	·0459	,,	·000660	000656
5	219	180	0·39	·0452	,,	·000665	
6	214	175	0·39	·0490	,,	·000675	

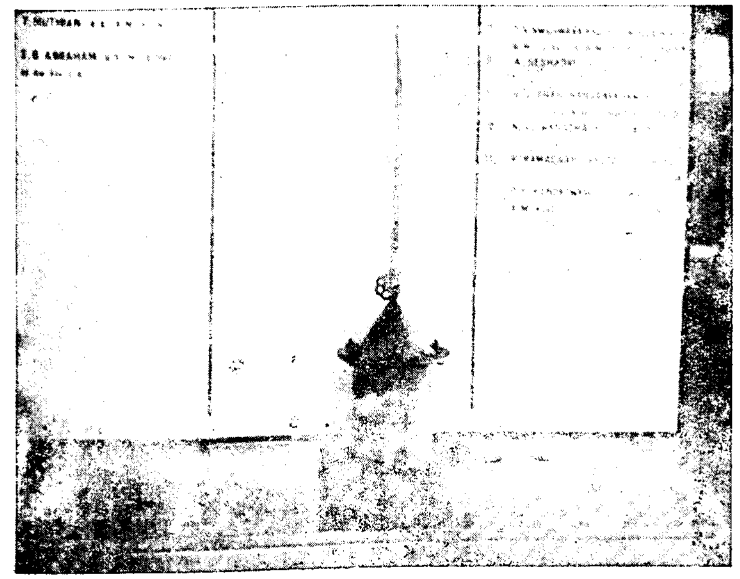


PHOTO No. 1
Variable head Permeameter

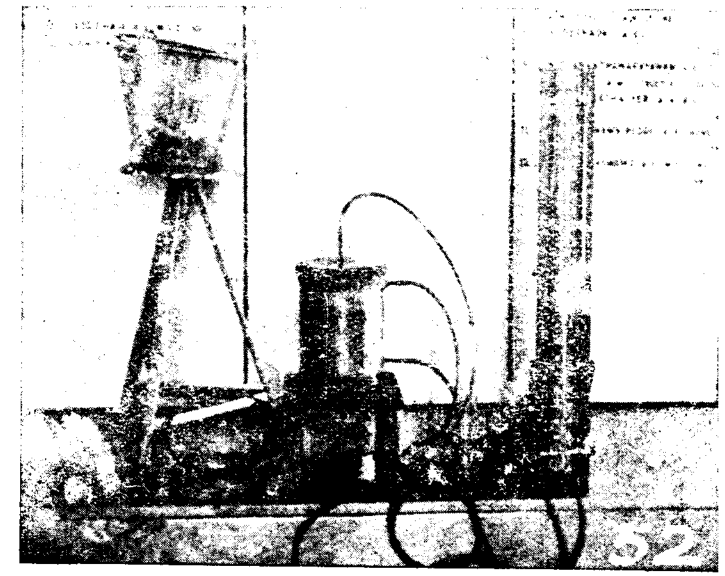


PHOTO No. 2
Constant head Permeameter

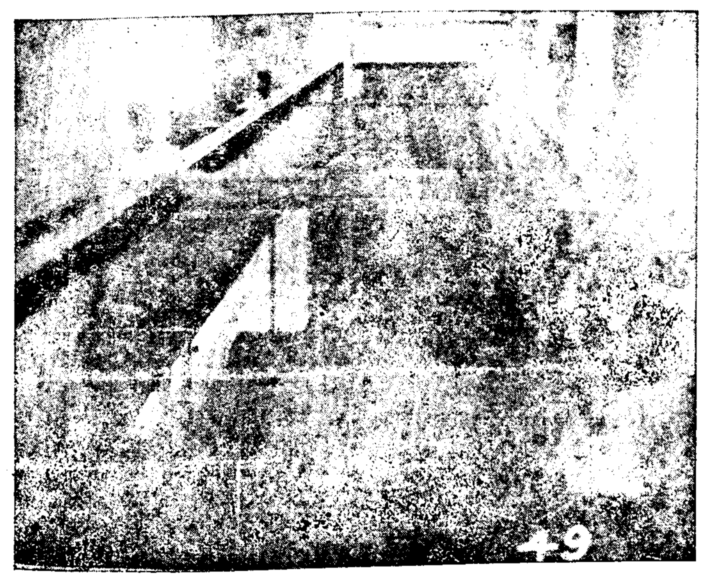


PHOTO No. 3
View of the Hydraulic Model

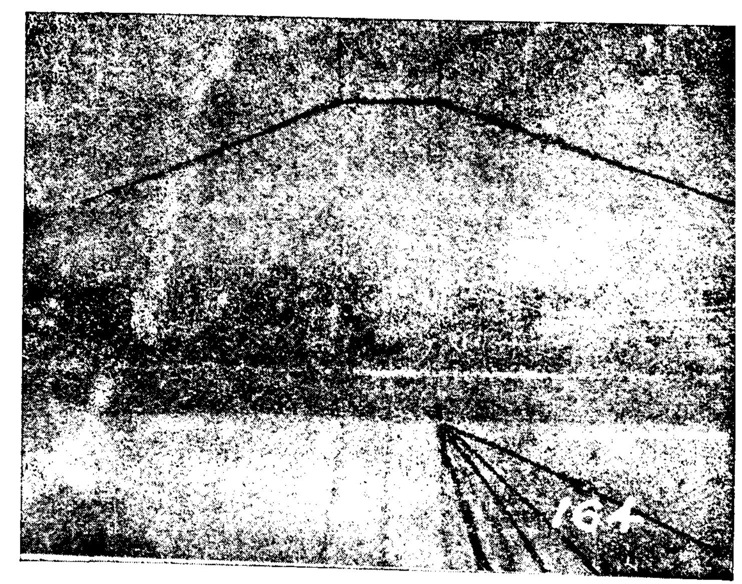


PHOTO No. 4
Boundary Roughness of the model flume

PHOTO No. 5
Prevention of Pipin

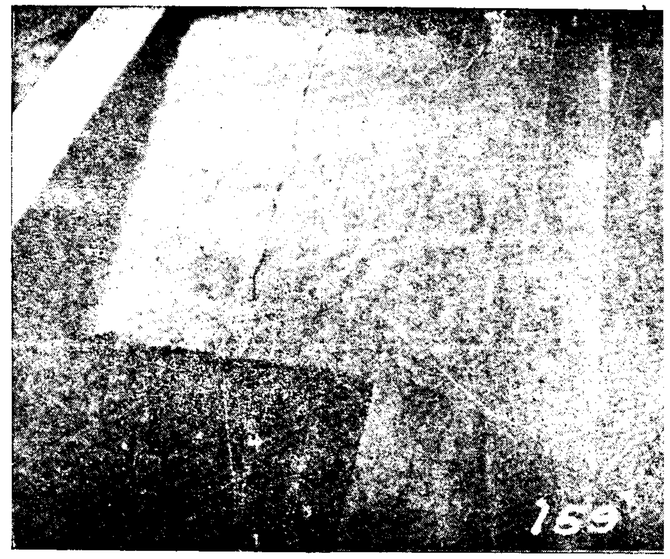


PHOTO No. 6
Prevention of Soil Evaporation

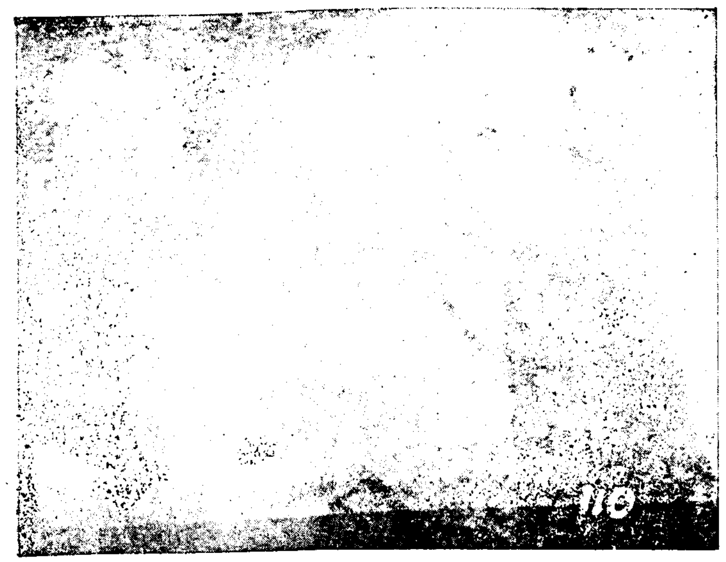


PHOTO No. 7
Earthdam Curved in Plan

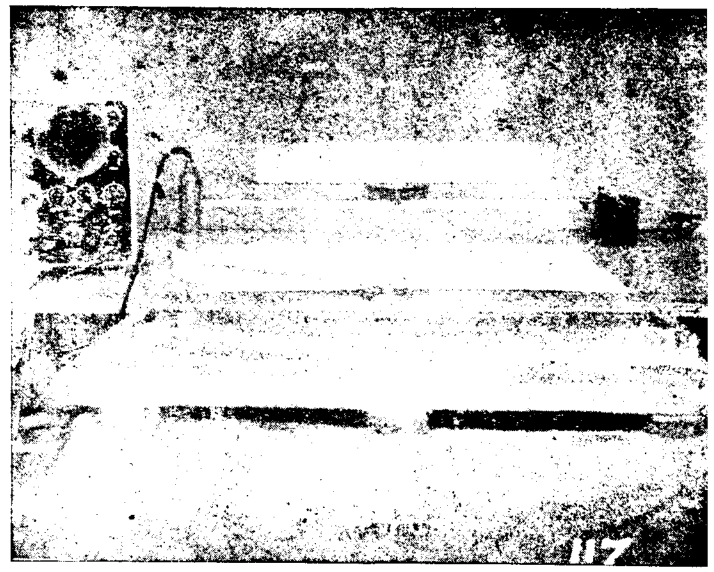


PHOTO No. 8
Electrical analogy set up

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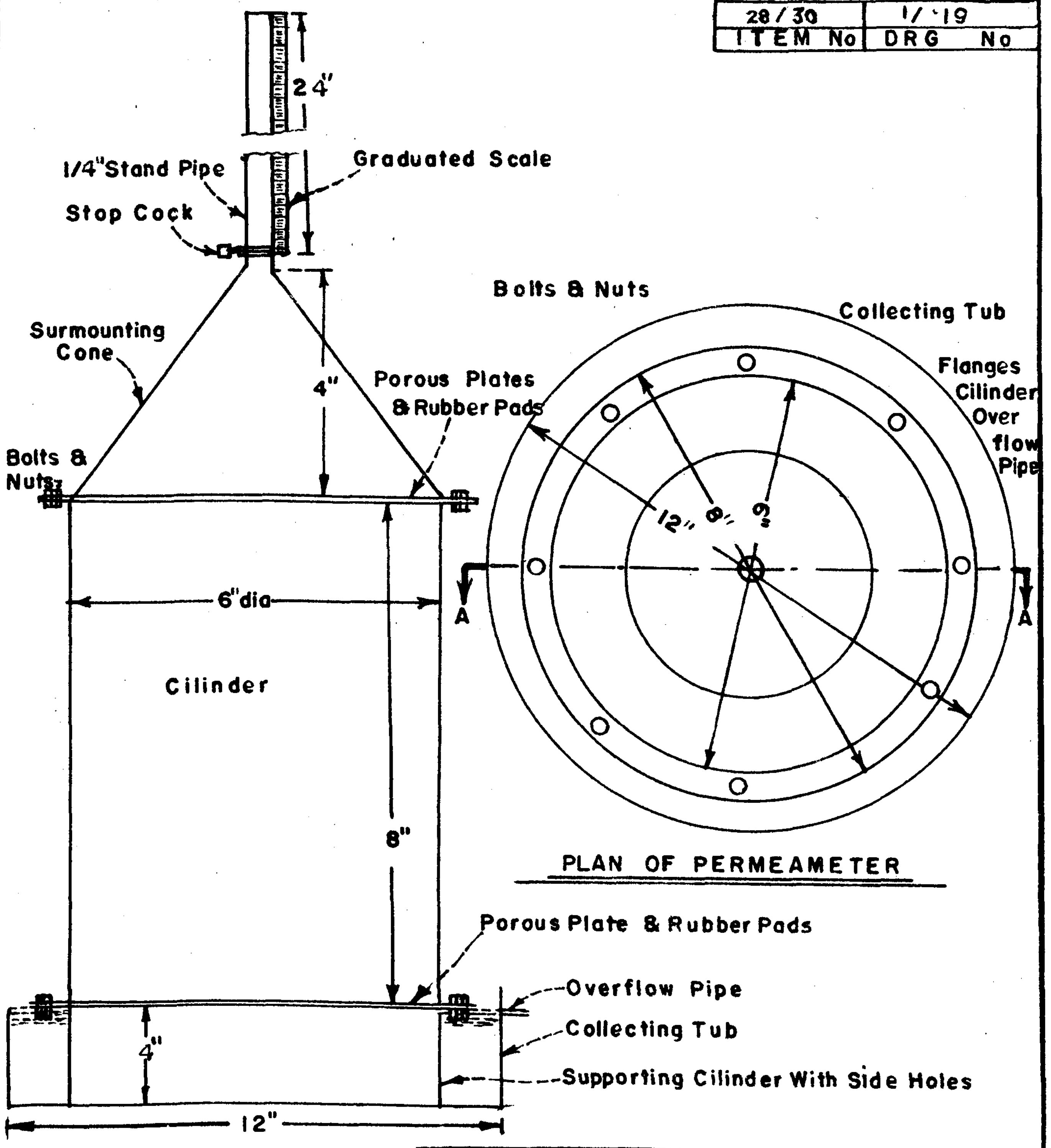

SECTION ON AA

PLAN OF PERMEAMETER

Public Works Department, MADRAS.
Irrigation Research Station, POONDI.

VARIABLE HEAD PERMEAMETER

(Not To Scale)

SUB-SOIL FLOW — EARTH DAMS

Report	File	Drawing	II

J · E	A · R · O	Exe · Engineer
Trd V·V	Ckd	Fig ·· 1·5

Reg. No. 197 E'62 (Saka Era 1884)-375. Printed by Sri. K. SAMPATH. B.A., A.D., C.S.O., P.Z.P., Madras.
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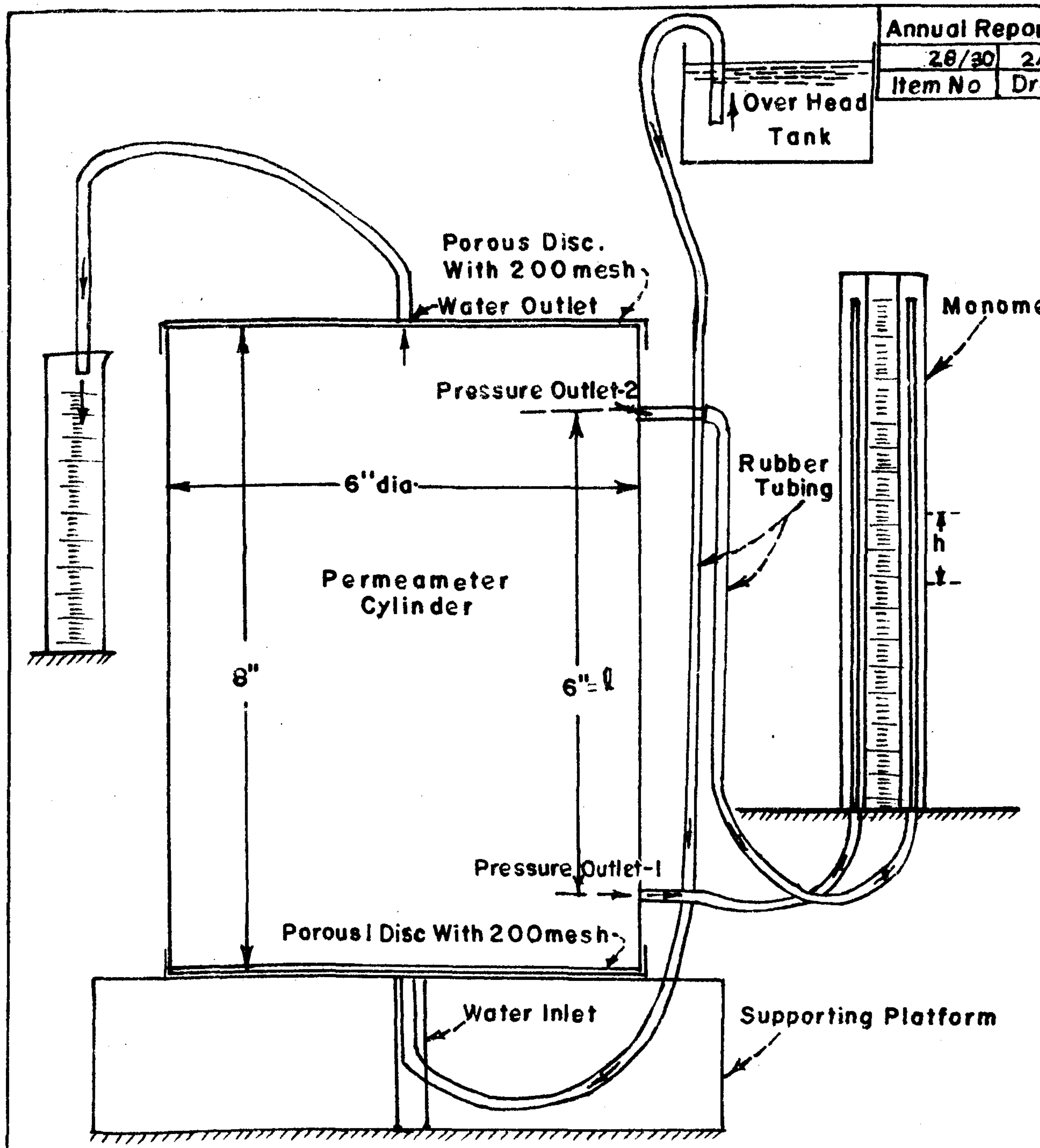
28/30	2/19
Item No	Drg No.



NOTE The Arrows Inside The Tubing And The Cylinder Indicate The Direction Of Flow Of Water.

Public Works Department, MADRAS.
Irrigation Research Station, POONDI.

CONSTANT HEAD PERMEAMETER

(Not To Scale)

SUB-SOIL FLOW—EARTH DAMS

Report	File	Drawing 12
J. E	A. R. O	E. E. R
Trd V.V	Ckd	Fig. 2/19

Reg. No. 198 E'62 (Saka Era 1884)-375. Printed by Sri. K. SAMPATH. B.A., A.D., C.S.O., P.Z.P. Madras.
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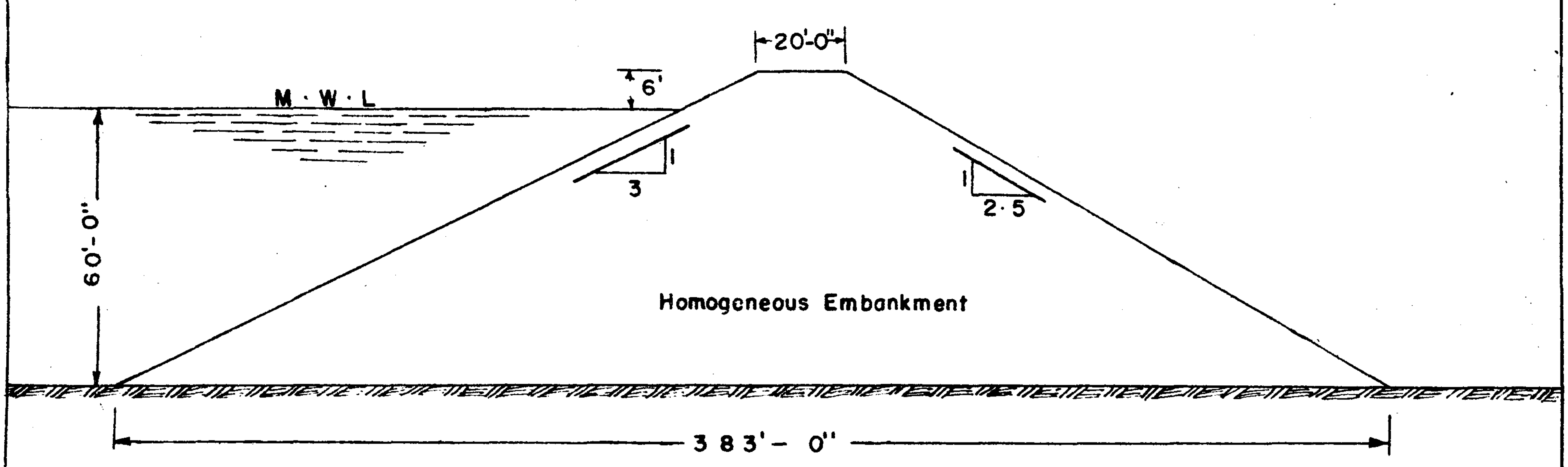
28/30	3/19
ITEM No	DRG No



Note :- Scale Of Model 1/60

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.	
C-15 File	SUBSOIL FLOW — EARTH DAMS STANDARD SECTION OF HOMOGENE-OUS DAM ON IMPERVIOUS FOUNDATION (Not To Scale)	
99/59 Drawing	J · E · R / Trd. J·J	A · R · O / Ckd.	E · E · R / Fig. 3

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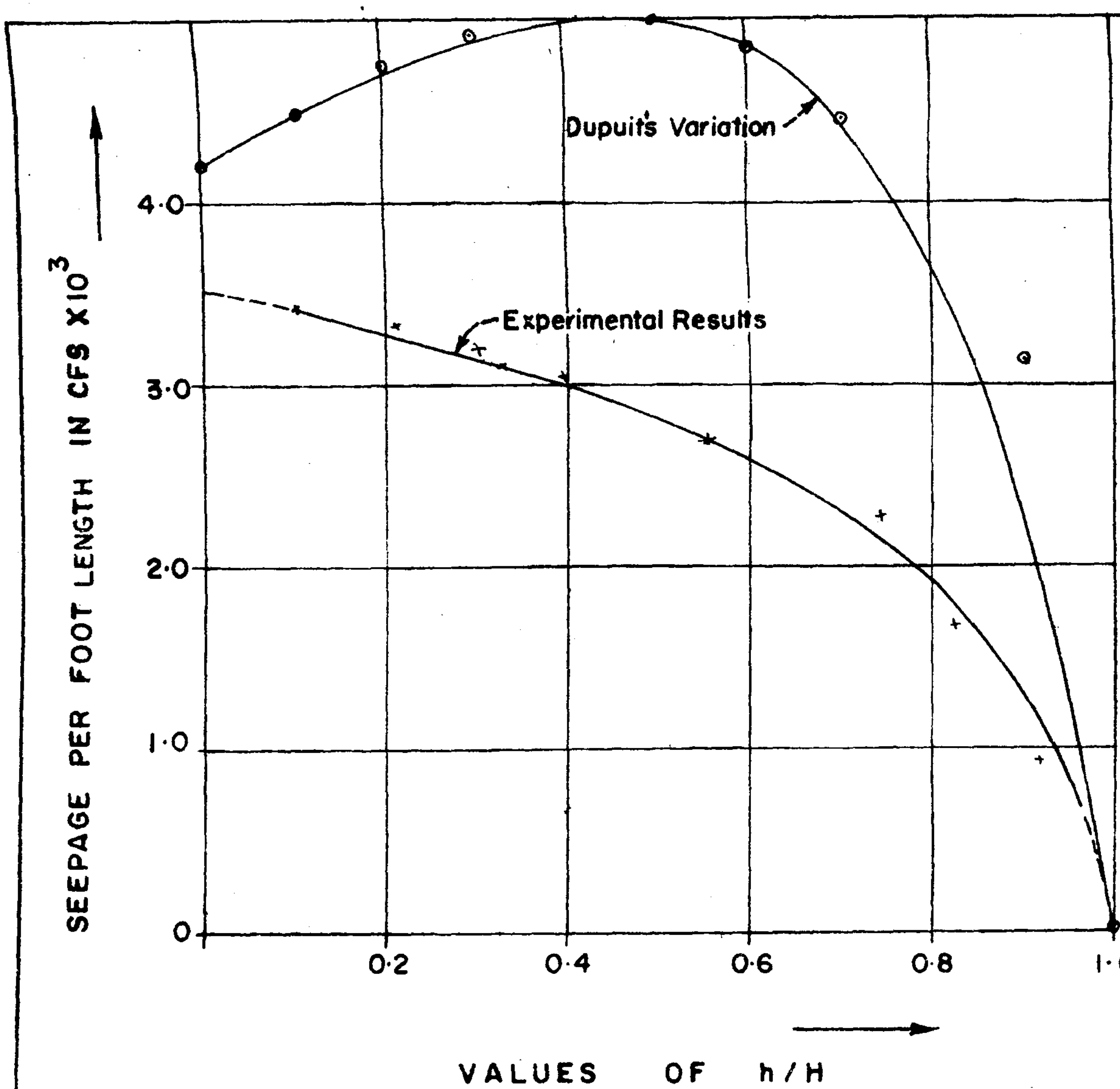
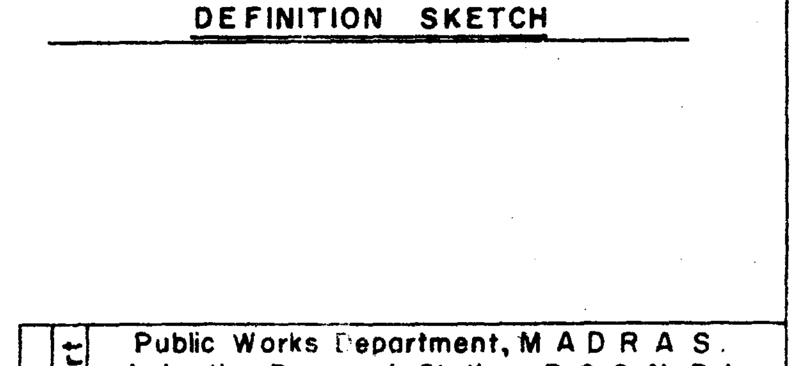
ITEM No	DRG No





DEFINITION SKETCH

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File C–15	SUBSOIL FLOW – EARTH DAMS VARIATION OF SEEPAGE WITH TAILWATER		
Drawing 100/59	NRajasekharan 4/5	U.V. [illegible]	[illegible]
	J. E. R	A. R. O.	E. E. R
	Trd. J.J	Ckd. [illegible] 4/5	Fig. 4

Reg. No. 200-E'62 (Saka Era 1884) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH B.A., A.D., C.S.O., Madras-5.

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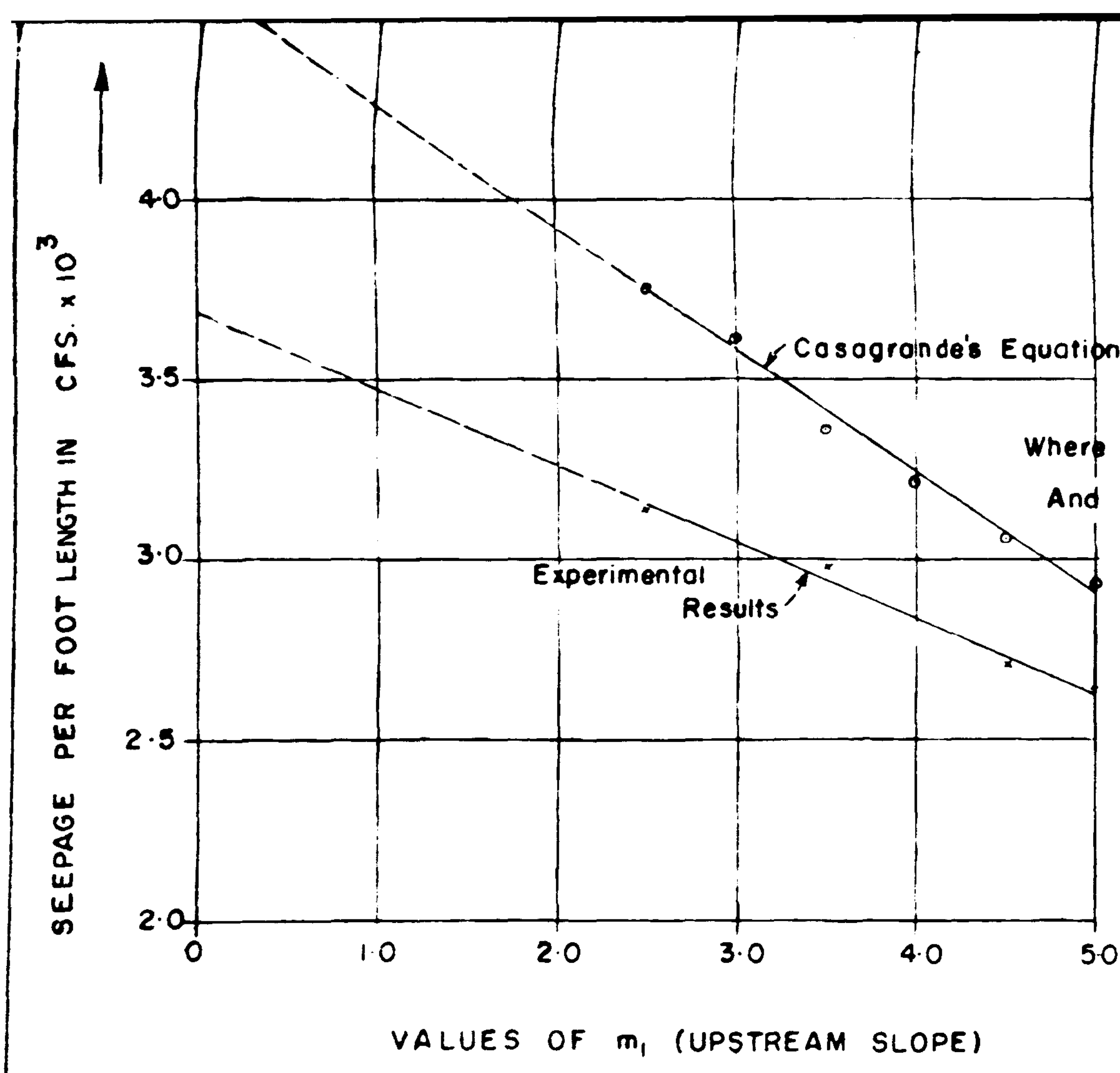
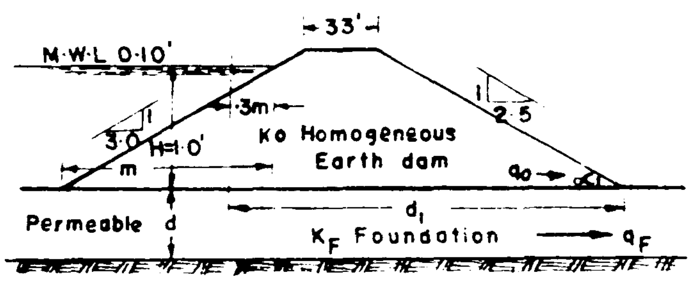
ITEM No	DRG No



$q_Q = K\,a\,\sin^2\alpha$

Where $a = \sqrt{H^2 + d_1^2} - \sqrt{d_1^2 - H^2 \cot^2\alpha}$

And d = the horizontal distance in the casagrande's equation



Report	Public Works Department, M A D R A S. Irrigation Research Station, P O O N D I.		
File C - 15	SUBSOIL FLOW — EARTH DAMS		
	VARIATION OF SEEPAGE QUANTITY WITH UPSTREAM SLOPE		
Drawing 101/59	Majanaman 4/5	J. V. Sondobrina 4.5	K. Kandaswamy
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd	Fig. 5

Reg. No. 201-E'62 (Saka Era 1884) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH B.A., A.D., C.S.O, Madras-5.

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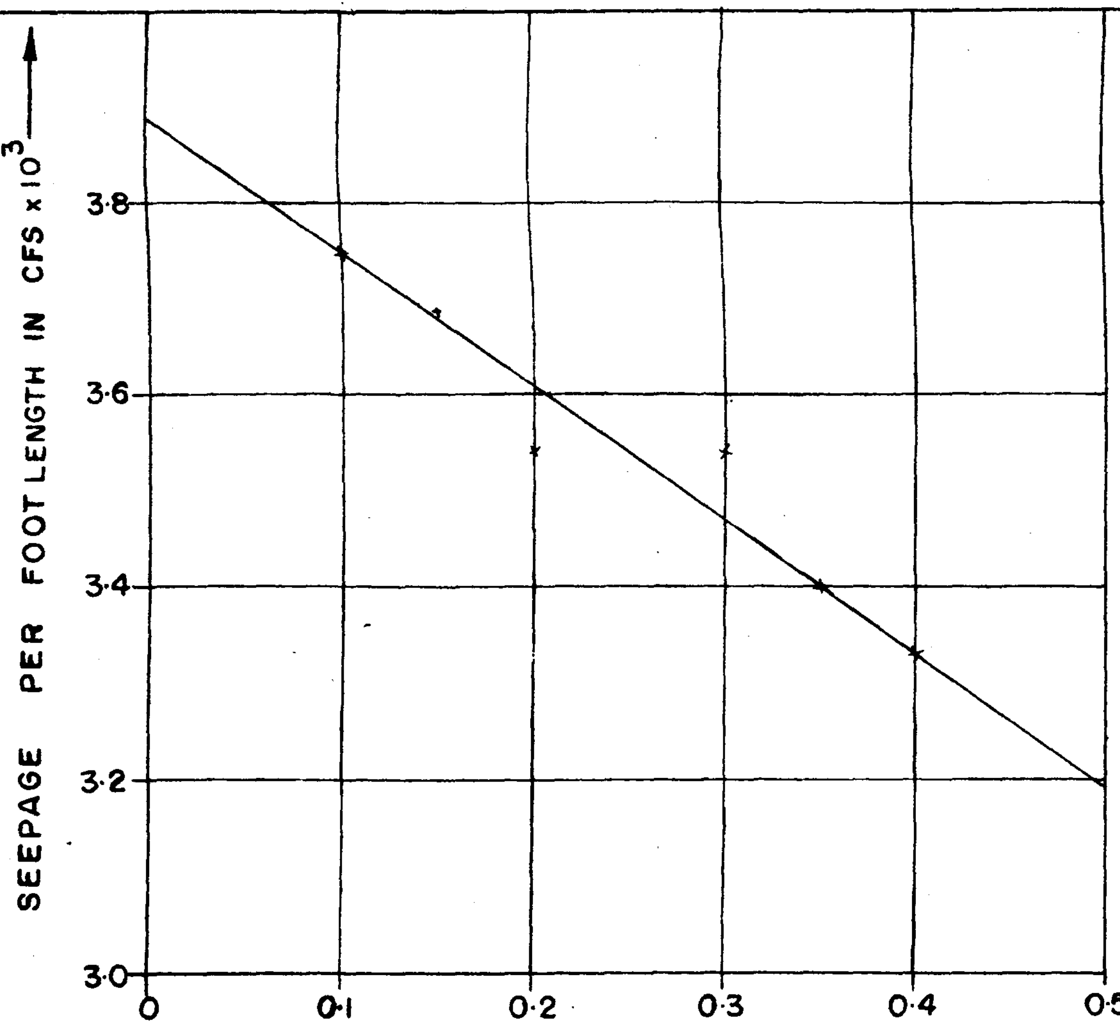
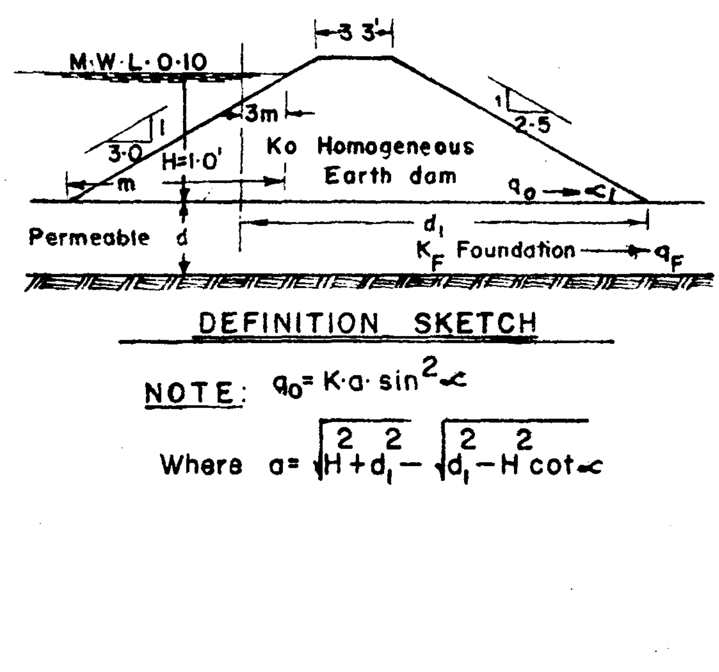
ITEM No	DPG No





DEFINITION SKETCH

NOTE: $q_0 = K \cdot a \cdot \sin^2 \alpha$

Where $a = \sqrt{H^2 + d_1^2} - \sqrt{d_1^2 - H^2 \cot \alpha}$

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	SUBSOIL FLOW — EARTH DAMS VARIATION OF SEEPAGE QUANTITY WITH TOP WIDTH		
Drawing 102/59	J.E.R	A.R.O	E.E.R
	Trd. J.J	Ckd.	Fig. — 6

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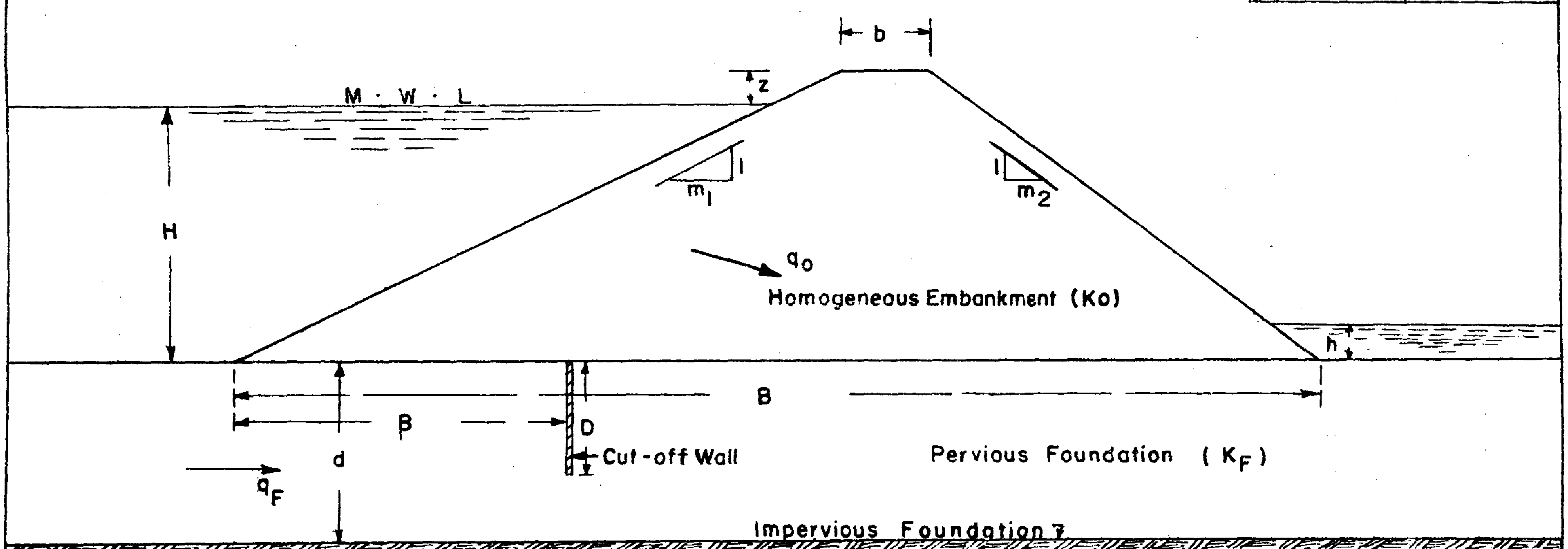
ITEM No	DRG No



Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI		
File C-15	SUBSOIL FLOW — EARTH DAMS		
	DEFINITION SKETCH		
	(Not To Scale)		
Drawing 98/59	N Rajarathnam 4/5	S.V. [illegible]	[illegible]
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd. [illegible] 4/5	Fig···7

ANNUAL REPORT OF 195	
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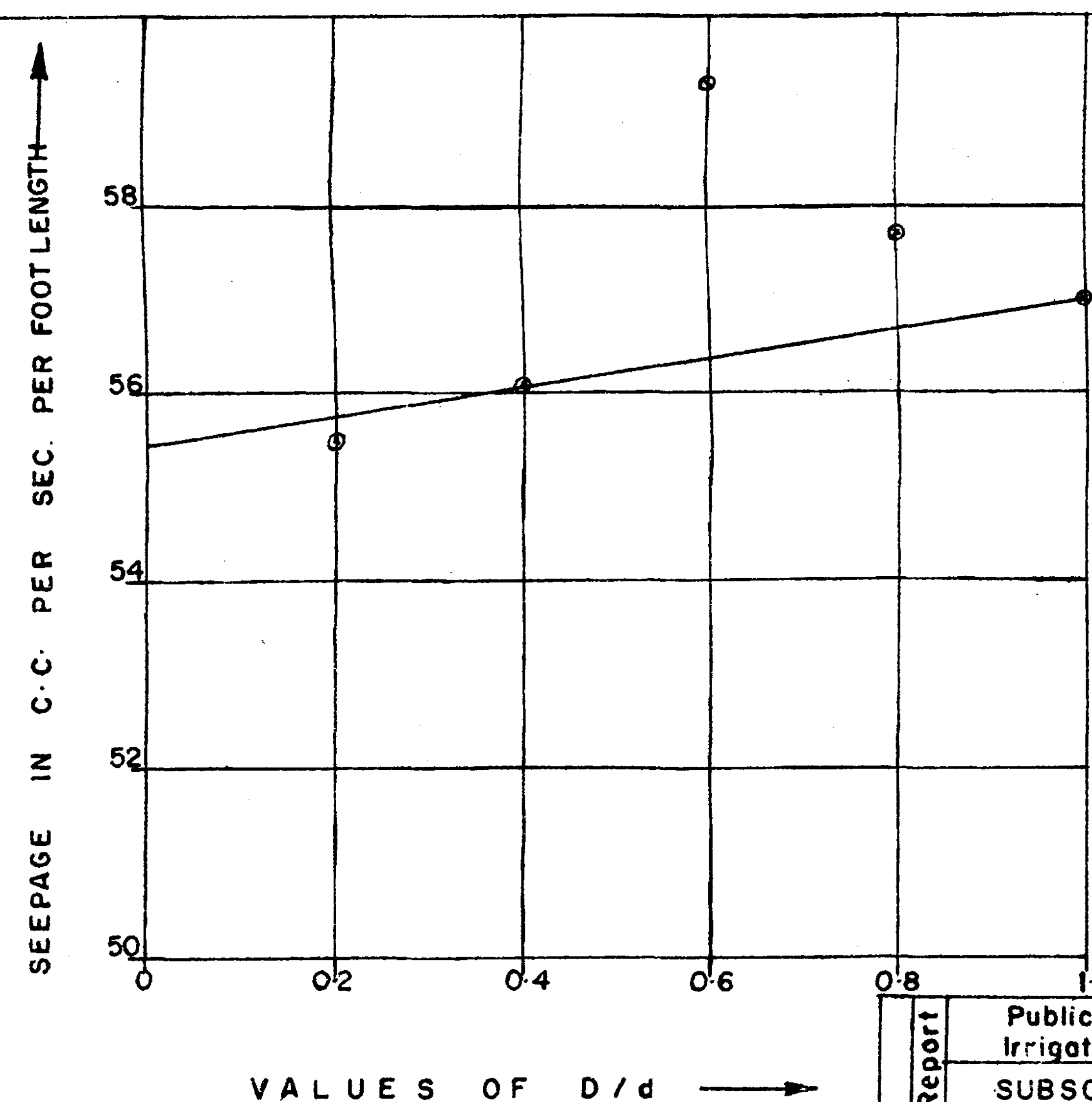
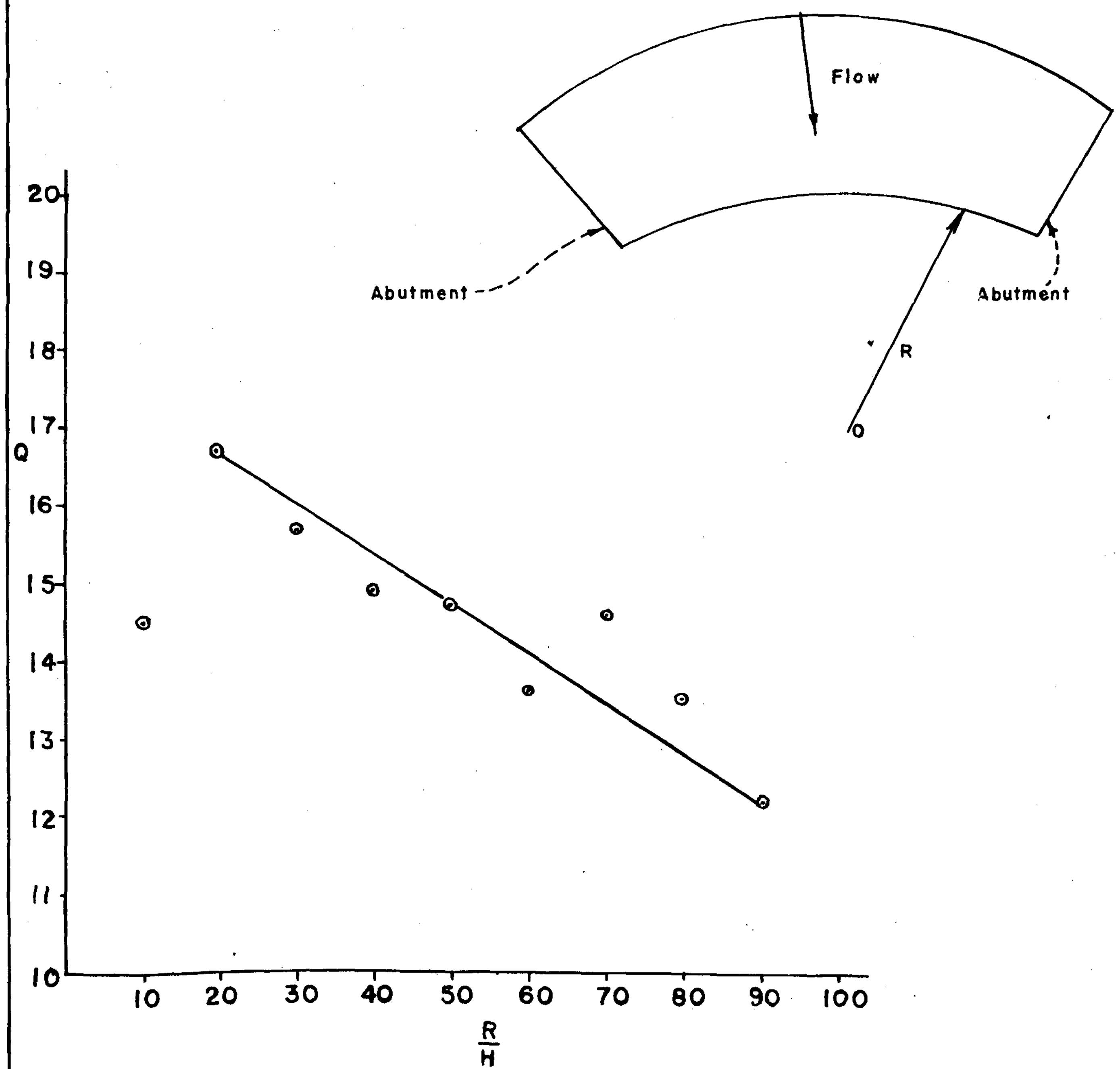
ITEM No	DRG No



Report	Public Works Department, M A D R A S. Irrigation Research Station, P O O N D I.
	SUBSOIL FLOW —EARTH DAMS
C-15 File	REDUCTION OF SEEPAGE WITH CUT-OFF WALLS
105/59 Drawing	J · E · R / Trd. v·v · A · R · O / Ckd. · E · E · R / Fig. 8

Reg No. 204-E'62 (Saka Era 1884) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH B.A., A.D.C.S.O, Madras-5



Q = Seepage quanty in C·C·/Sec·

R = Radius in Feet

Public Works Department—Madras.
Irrigation Research Station—Poondi.

SUBSOIL FLOW-EARTH DAMS
VARIATION OF Q WITH $\frac{R}{H}$ IN
CURVED DAMS

Report	File	Drawing 187/59

B. Thomas	V. V. Subrahmanyam	A. K. Kandaswamy
R·A·	A·R·O	E·E·R
Trd. T.K.S	Ckd. B.	FIG· 9·

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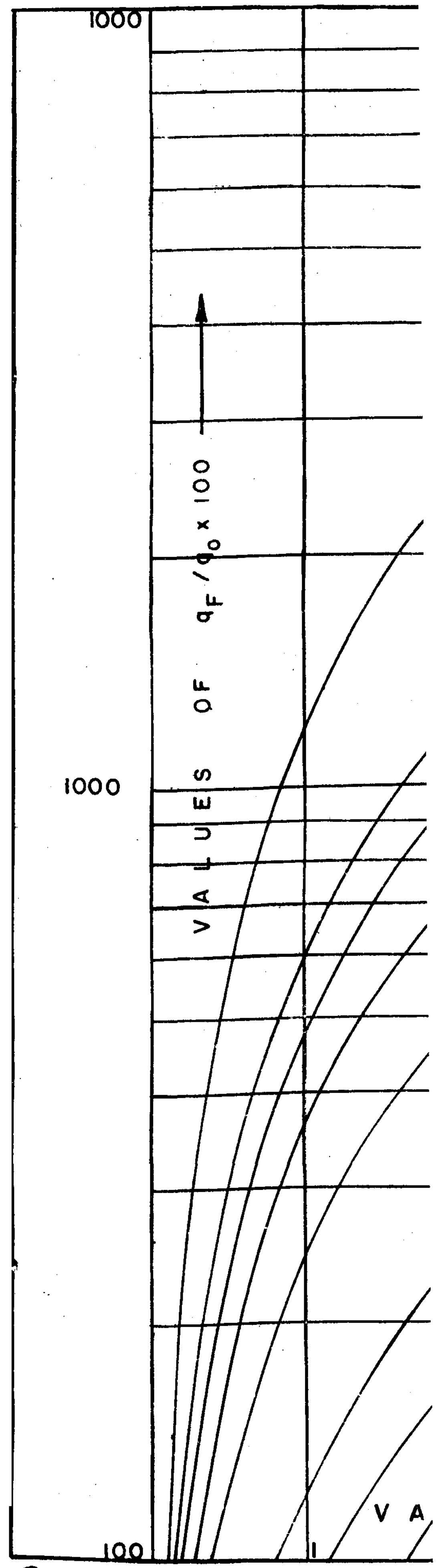
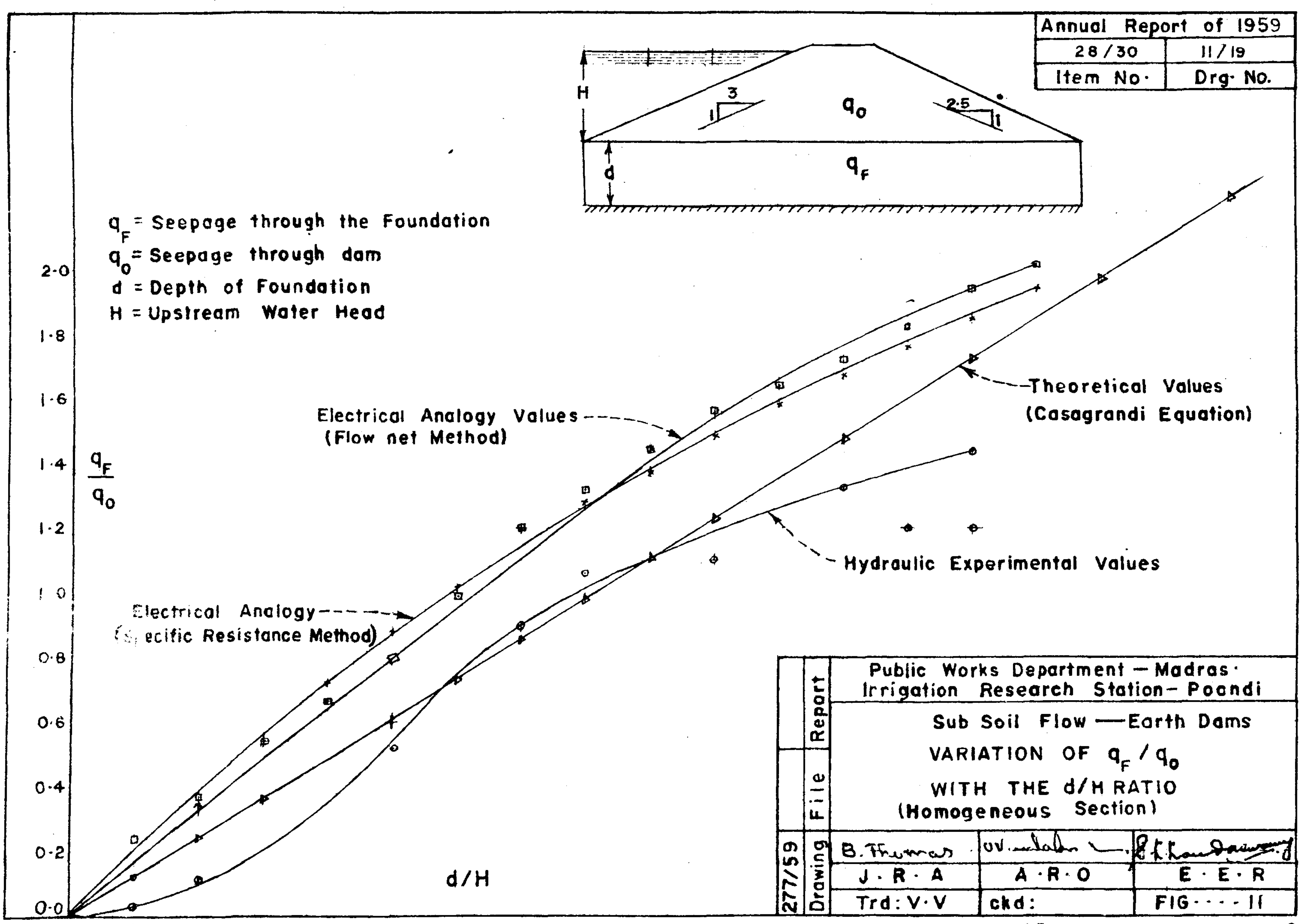


Reg No 233 E'62 (Saka Era 1884)



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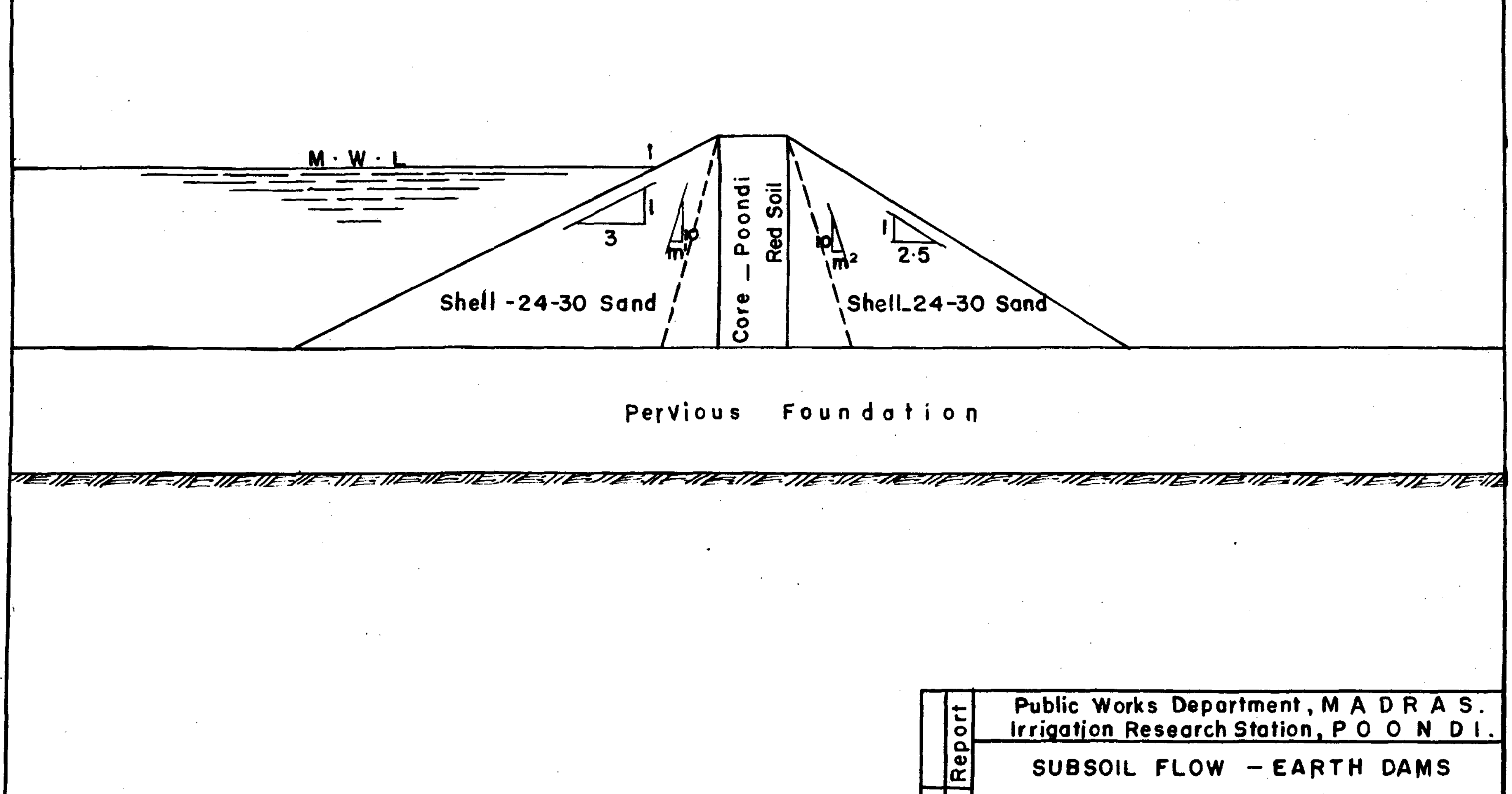

Drawing	File	Report			
106/59	C-15		Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
			SUBSOIL FLOW – EARTH DAMS		
			EFFECT OF COREWALL SLOPE ON QUANTITY OF SEEPAGE		
			J·E·R	A·R·O	E·E·R
			Trd. J·J	Ckd.	Fig. ··· 12·

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Annual Report of 1958	
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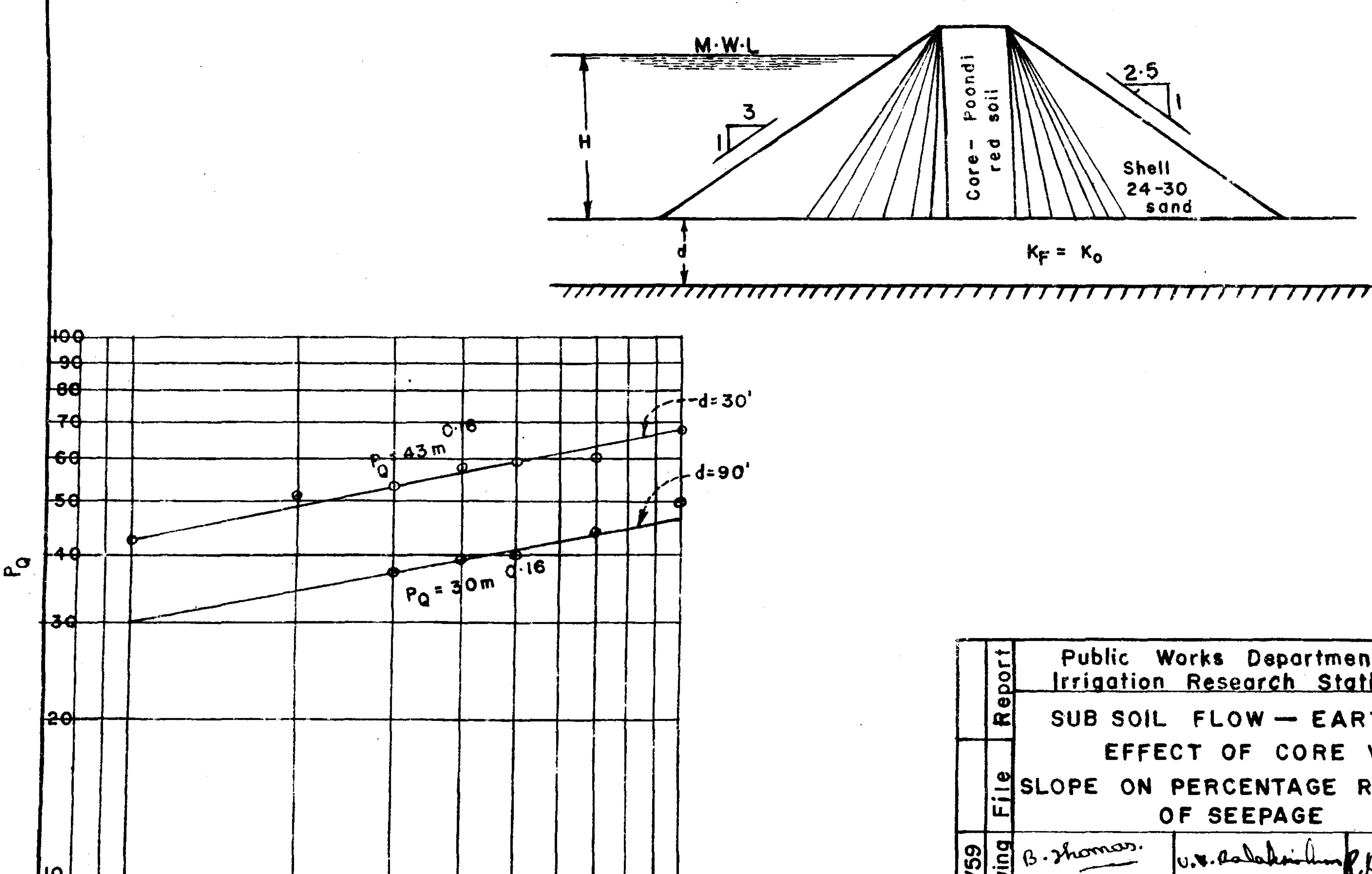
Item Nº	Drg·Nº



Report	Public Works Department - Madras. Irrigation Research Station - Poondi.		
File	SUB SOIL FLOW — EARTH DAMS EFFECT OF CORE WALL SLOPE ON PERCENTAGE REDUCTION OF SEEPAGE		
184/59 Drawing	B. Thomas.	V. V. Balakrishnan	[illegible]
	R·A·	A·R·O·	E·E·R
	Trd: V·V·	ckd: B	FIG····13·

Reg. No 235 E'62 (Saka Era 1884) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH B.A., A.D., C.S.O., Madras.

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LEGEND

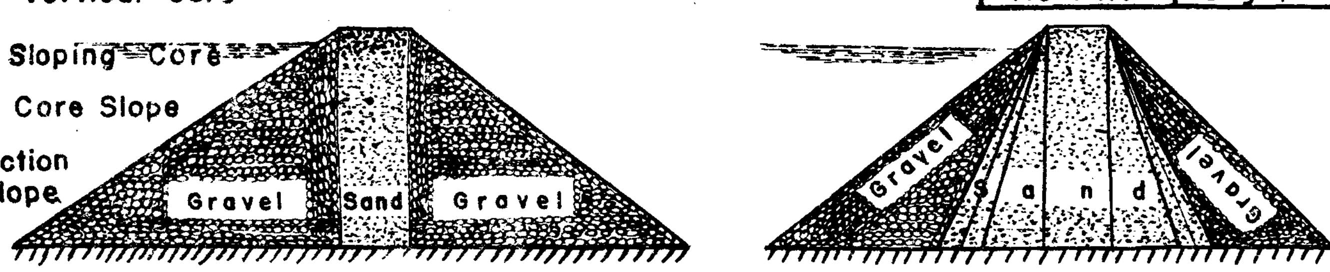
q_o = Seepage through a dam With Vertical Core

q_x = Seepage through the dam With Sloping Core

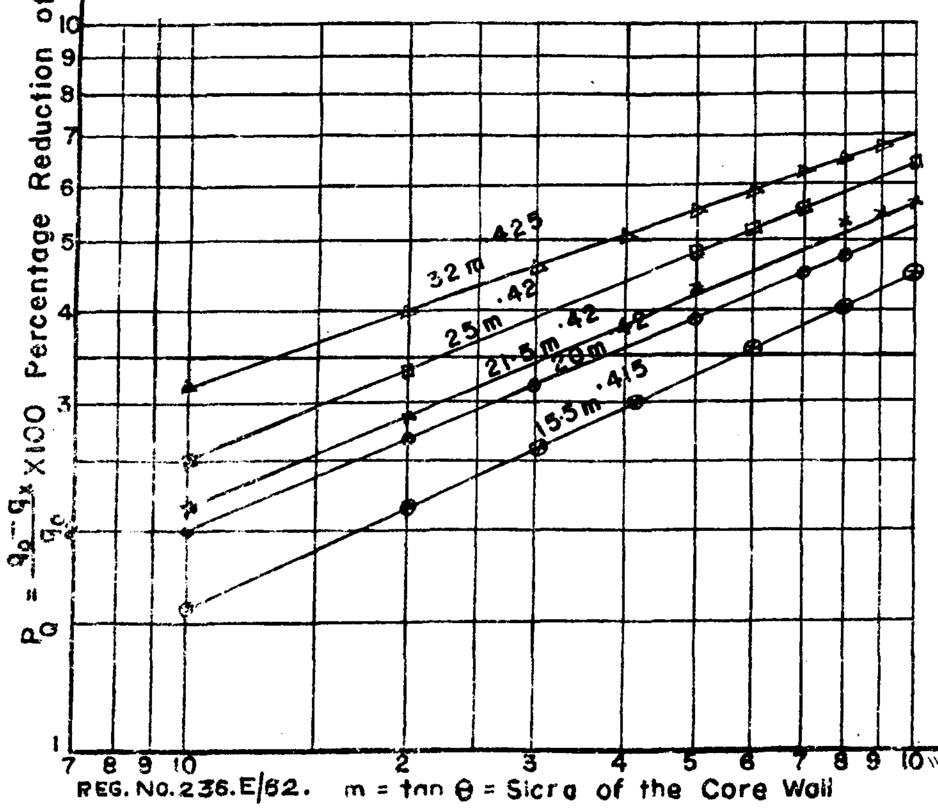
$q_o - q_x$ = Reduction in Seepage due to Core Slope

$P_Q = \frac{q_o - q_x}{q_o} \times 100$ = Percentage Reduction in Seepage due to Core Wall Slope.

m = tan θ = Slope of the Core Wall



Permeability Ratio 1·75
" " 4·80
" " 6·70
" " 11·20
" " 17·40



REG. No. 236.E/52.

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Report	Public Works Department–Madras. Irrigation Research Station–Poondi.
File	Sub Soil Flow – Earth dams VARIATION OF PERCETAGE REDUCTION OF SEEPAGE QUANTITY WITH SLOPE OF THE CORE

Drawing 278/59		
B. Thomas.	[signature]	[signature]
J·R·A	A·R·O	E·E·R
Trd: V·V·	ckd:	FIG... 14.

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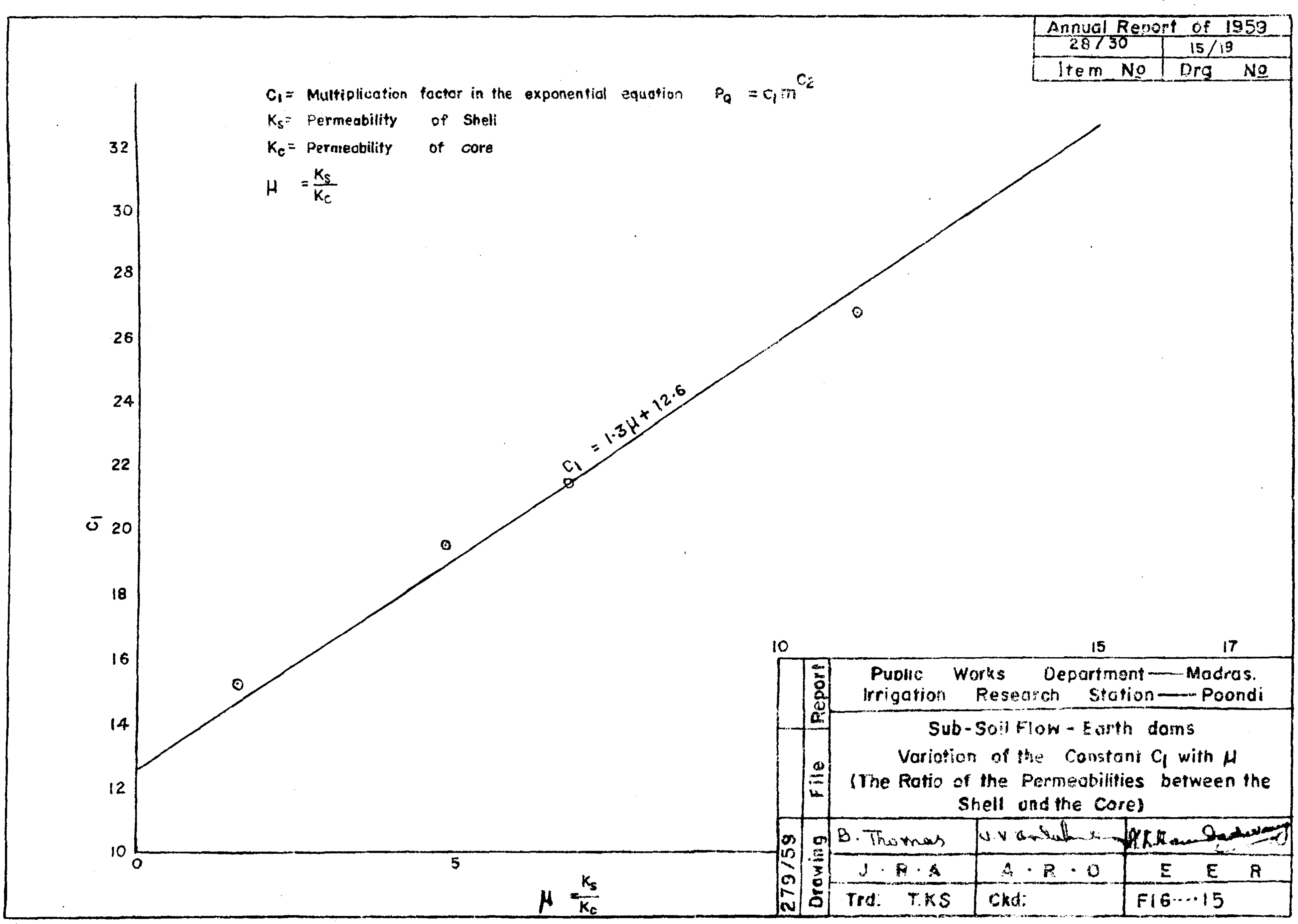

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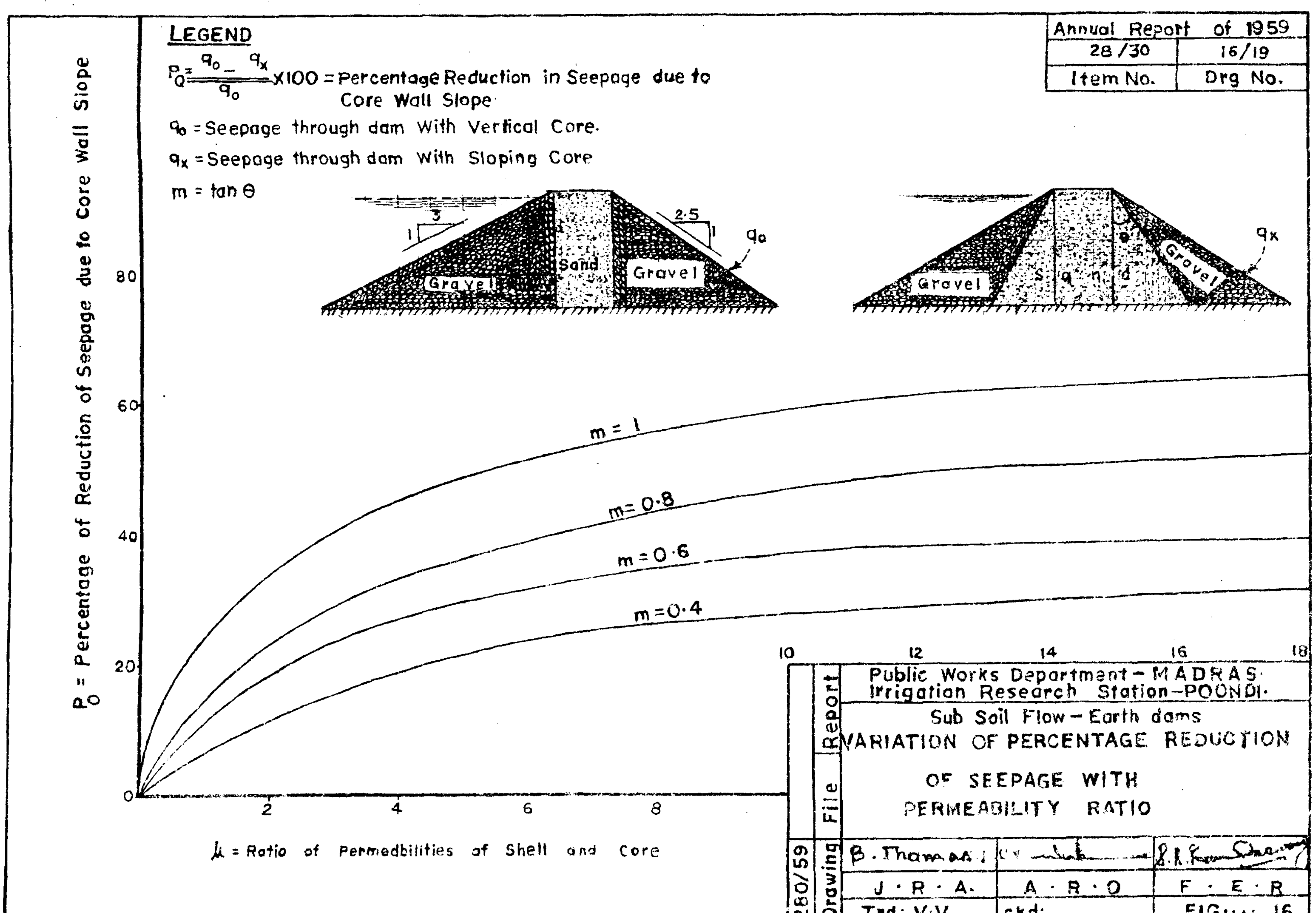
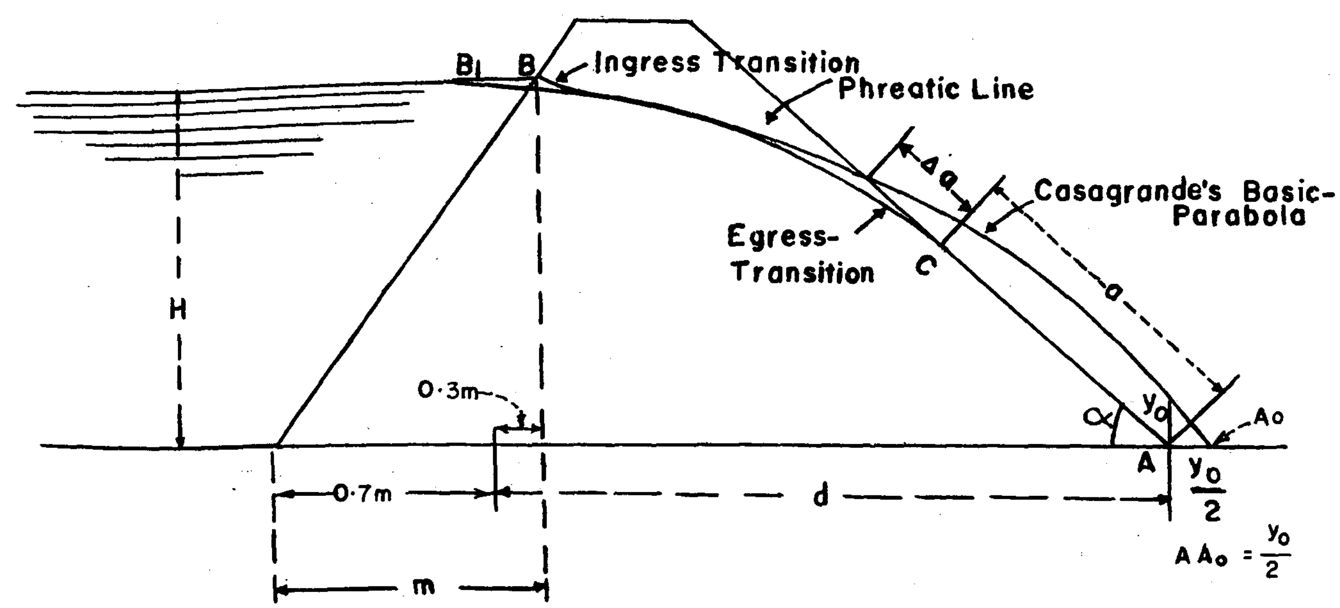


Reg. No. 273-E'62 (Saka Era 1884)-375 Reproduced from Indentor's Original Printed by Sri K. SAMPATH B.A., A.D.C.S.O., Madras.



Note: $a = \sqrt{H^2 + d_1^2} - \sqrt{d_1^2 - H^2 \cot^2 \alpha}$

$$x = \frac{y^2 - y_0^2}{2y_0}$$

$$y_0 = \sqrt{H^2 + d_1^2} - d_1$$

See page 665 of vol· III Greager and Justin Engineering For Dams·

Public Works Department—Madras.
Irrigation Research Station—Poondi.

Report

SUB SOIL FLOW—EARTH DAMS.
FIXING THE PHREATIC LINE.

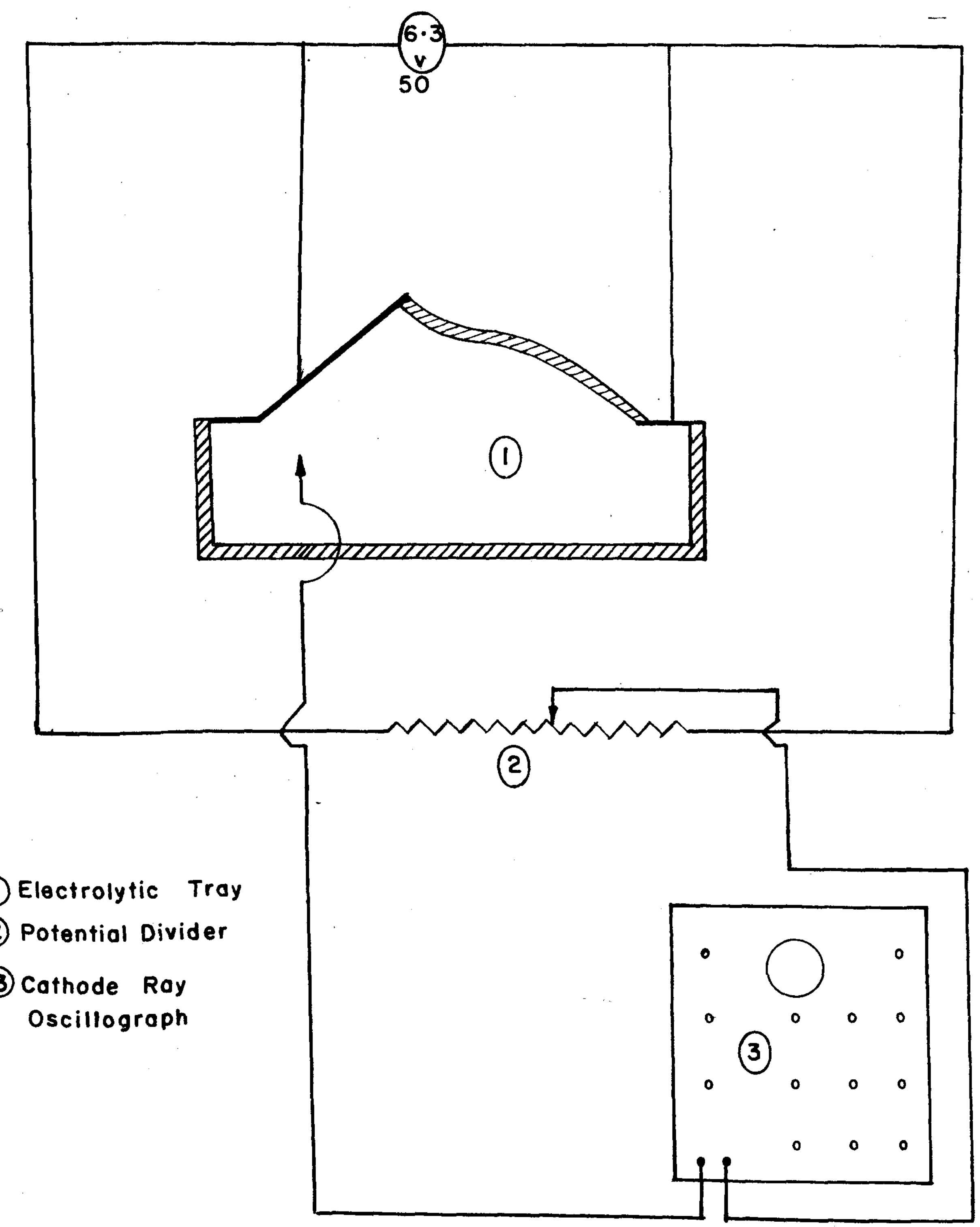
File

Drawing 188/59

Annual Report Of 1958	
28/30	17/19
Item No	Drg No

B. Thomas	U.V. Sakil	A.K. Kandaswamy
R·A·	A·R·O	E·E·R
Trd: T.K.S	Ckd:	FIG... 17

Reg No237E'62(Saka Era1884)-375 Reproduced from Indentor's original. Printed by Sri K.SAMPATH B.A., A.D., C.S.O., Madras-5.



① Electrolytic Tray
② Potential Divider
③ Cathode Ray Oscillograph

Annual Report of 1958		
28/30	16/19	
Item No	Drg. No	

Public Works Department – Madras
Irrigation Research Station – Poondi

SUB SOIL FLOW — EARTH DAMS
ELECTRICAL ANALOGY SET UP

Report File Drawing 189/59	B. Thomas.	V.V.Balakrishnan	Khandaswamy
	R · A ·	A · R · O	E · E · R
	Trd: V·V·	ckd: B	FIG · · · 18 ·

Note.

Necessity for a New Type of Design of Channel

1. Whatever type of silt exclusion device is adopted upstream of the headsluice of a channel taking off from a river having only flashy floods, it is almost impossible to completely exclude the entry of silt into the channel. The reason for this is that in times of low discharges in the river, all the water has to be taken into the channel without any wastage through the sand vents, thereby nullifying the action of the silt exclusion devices Taking into account the above, fact, some means of pushing down the entering silt further into the channel, to the first tank, or the first drop and of keeping the rear of the headsluices clear of silt, has to be devised so that full supply discharge can be drawn into the channel without being obstructed by the silted-up bed.

2. With a trapezoidal section for the channel, a non-silting velocity can be obtained when high discharges are flowing in the channel and very little of silting-up is likely to take place for this condition of flow. Further, in times of floods in the river, scour vents are usually operated to minimise the entry of silt into the channel. A major portion of the silting up may occur only during the period of recession of the flood, when the discharge may drop down to very low magnitude. Because of low discharge small velocities prevail and all the silt that finds its way into the channel gets deposited in the channel bed. It should be noted that during low flows in the river the scouring action of the sand vents will not be felt by the headsluices with the result that a large quantity of silt may enter the channel during this period.

4. 3. Hence,it seems desirable to induce greater velocities in the canal during low discharges, by providing a narrow inner section at the bottom of the usual trapezium to carry the low discharges so that the bed material entering the channel is carried right through without and deposition in rear of headsulice. This highvelocity flow during low discharges may be maintained up to the first tank in the channel system and the portion of the channel beyond this may be of the usual type. For this purpose an attempt has been made to present a new type of design for such channels.

4. THEORETICAL INVESTIGATION OF THE PROBLEM

4. 1. Two formulae are commonly used for computation of velocity of flow in an open channel[1].

(i) MANNING'S FORMULA

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \text{ where}$$

V = Velocity of flow in feet/second
n = Coefficient of rugosity
R = Hydraulic radius in feet
S = Slope of the channel bed

(ii) GANGUILLET AND KUTTER'S FORMULA

This gives the value of C in the chezy formula,

V = $G\sqrt{RS}$ where
V = Velocity of flow in feet/second
C = a constant given by the above formula
R = hydraulic radius in feet
S = Slope of the channel bed

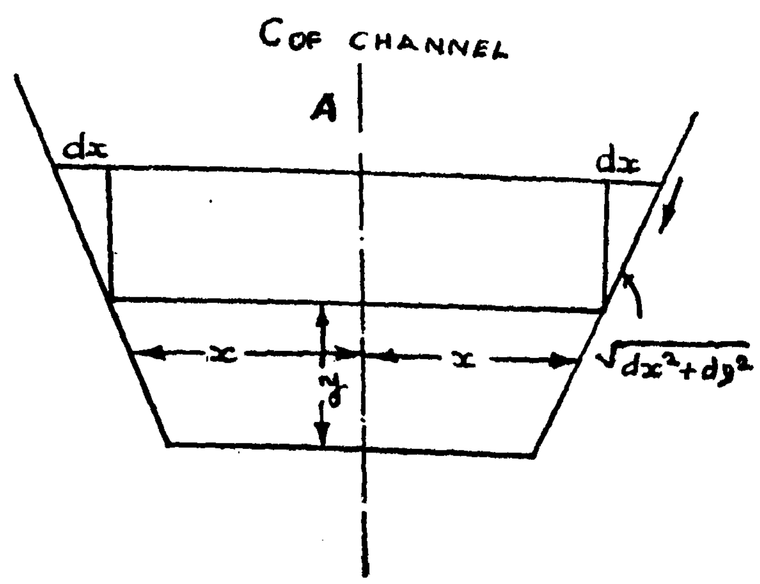
$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{\;}}{1 + \frac{n}{\sqrt{R}}\left(41.65 + \frac{0.00281}{S}\right)}$$

4. 2. For a channel with a given slope and in a given material, the velocity of flow (V) is a function only of the hydraulic radius (R) as can be seen from the above formulae. Therefore, the only condition to be satisfied for velocity to remain constant over different depth of that the hydraulic radius (R) must be constant for any depth of flow. Such a type of channel has been dealt with in the book "Hydraulics of steady flow in open channels" (2) by C. J. Posey and S. M. Woodward and also in the book "Hydraulics and its applications" by A.H. Gibson (3).

4. 3. Let A be the area of the channel section and P be the wetted perimeter at any assumed depth of flow. Let dA and dP the small increments in the wetted area and wetted perimeter for a small increase in depth.

$$\text{Then, } R = \frac{A}{P} = \frac{A + dA}{P + dP} = \frac{dA}{dP} = \frac{x\,dy}{\sqrt{dx^2 + dy^2}}$$

Please see the figure given below :



Sketch 1.

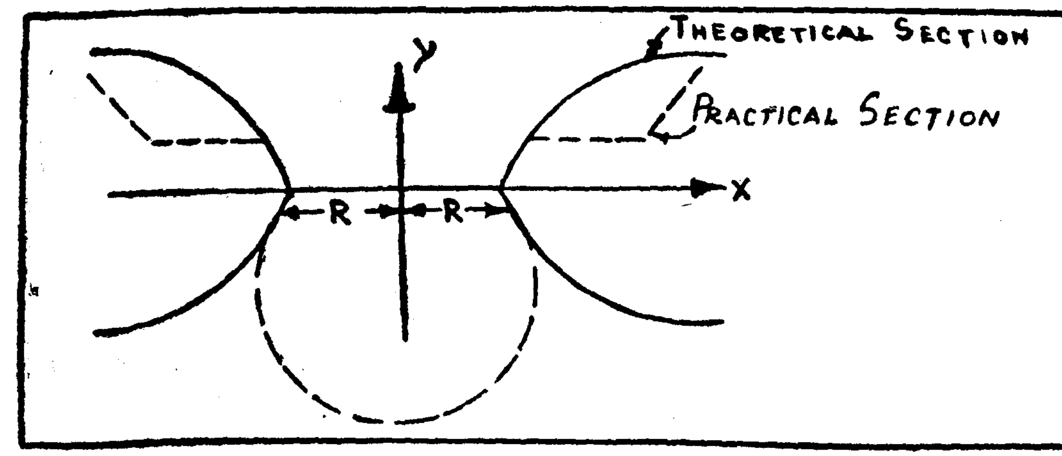
The general solution of the above differential equation is

$$y = R\,\text{Cosh}^{-1}\frac{x}{R} + K$$

Axes may be chosen so as to eliminate the Constant K

$$\because\ Y = R\,\text{Cosh}^{-1}\frac{x}{R} \quad \& \quad X = R\,\text{Cosh}^{-1}\frac{y}{R}$$

The curves represented by the above equation are given in the adjoining sketch. The curve is not closed even at the bottom and hence it may be concluded that such a channel deos not exist. But if we allow the restriction that the depth of flow will always be above a certain level, a bottom may be fitted to the curve as shown by the dotted circle at the bottom in the Figure.



Sketch 2.

It is necessary for the bottom to have a hydraulic Radius R. Figure-20 shows circular and rectangular elements of the correct dimensionsfitted to the curve to form channels which have constant hydraulic radius as long as the water level is above the top of the elementary section.

As the theoretical curves are very difficult to form at the site, a type of two-zoned section shown dotted in the Sketch No. 2 above the X-axis may be suitable from a practial point of view. Hence the design of such a trapezoidal section has been attempted for the Kaveripauk Channel.

5. DESIGN PROCEDURE FOR THE KAVERIPAUK CHANNEL

5. 1. Hydrulic particulars

1. Full supply discharge	...	3443 cusecs
2. Sill of head sluice	...	El. 488.18
3. Rear F. S. L.	...	El. 492.18
4. Full supply depth	...	4 feet
5. Slope in the first reach from R. S. to the first drop at 1/0	...	4. 8 feet per mile (as per regrading proposal).

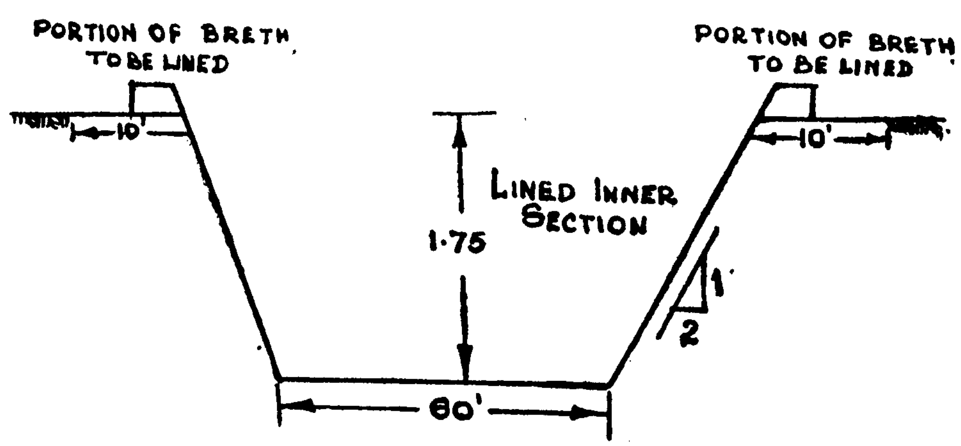
5. 2. Design for inner section

5. 2. 1. Analysis of flow data by flow - duration curve

As the Kaveripauk channel is carrying widely varying discharges, the flow data were proposed to be analysed by means of flow duration curve. The flow duration curve consists of a plot of values of channel flow in order of magnitude as ordinates and percent of time as abscisses, the curve thus showing the flow equalled or exceeded for any desired percentage of time covered by the record" (4) The discharge data of the Kaveripauk channel available for the years 1914-45, 1946-47 and 1954-55 have been taken for analysis. The computations have been made by the procedure given by H. K. Barrows in his book "Water-Power Engineeing" and is presented in (Statement No. 6). Vide foot note at the bottom of the Table-1. Each dot in the columns of years represents a day and the number of dots or days in each assumed flow range is shown by the figures at the right of each yearly block of dots. A flow duration curve has also been drawn and is shown in Figure (21). In plotting the curve, the lower figures of the flow range are plotted against the corresponding percent of time. For illustration, 50 cfs in the flow range 50-99 has been plotted against the corresponding percentage of 93.6. The discharge for the design of the inner section of the channel has been taken from the curve as the one corresponding to the flow equalled or exceeded for 30% of the time which has been taken arbitrarily for the design purpose. The discharge thus taken for the curve is 620 cusecs.

5. 2. 2. Design of the inner section

Assume an inner section as shown in the adjoining sketch. This inner section is to be lined to withstand the high velocities induced.



For lined channels, $n = 0.015$

Adopting Manning's Formula,

$$V = \frac{1.486}{0.011} \times R^{2/3} \times (\cdot 0009091)^{1/2}$$

$$= 4.076\, R^{2/3}.$$

$Q = 4.076\, A\, R^{2/3}$ where Q is discharge in cusecs. When the section is flowing full, i.e., depth of water = 2.25 ft.

Area A = 108.06 sq.ft.

Wetted Perimeter P = 64.94 feet.

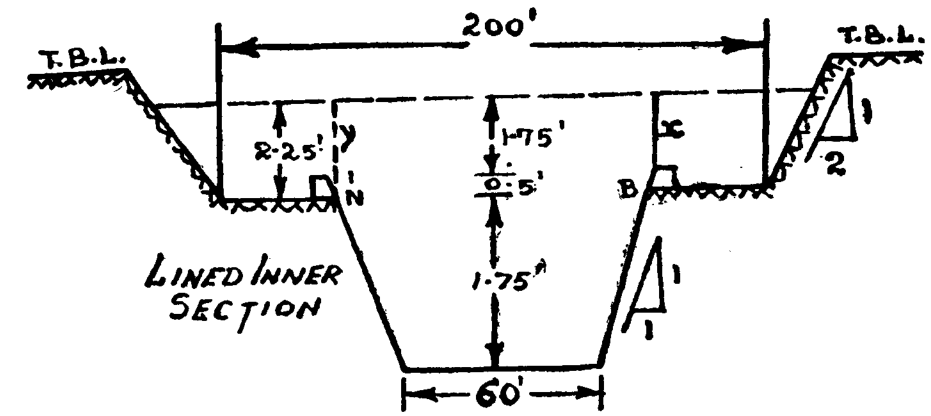
Hydraulic radius $R = \frac{A}{P} = \frac{108.06}{64.94} = 1.664$

$\therefore V = 4.076\,(1.664)^{2/3} = 5.723$ feet/second.

Discharge $Q = 4.076 \times 102.06 \times (1.664)^{2/3} = 620$ cusecs.

Hence, the assumed section will be able to carry 620 causecs for which it has to be designed.

5.3. Design of the entire section to carry the full supply discharge—



The inner section alone is to be lined and the rest of the section will be of earth. The calculation of discharge for the overbank flow(2) has been made by assuming imaginary vertical sections AA & BB above the inner channel and computing the discharge in the side portions Y and Z and the central portion x seperately, considering the areas and wetted perimeters of each to end at the imaginary vertical sections AA & BB.

For the sides, (Portion Y and Z), $n = 0.025$ $S = \cdot 0009091$.

$$V = \frac{1.486}{.025} \times R^{2/3} \times (.0009091)^{1/2}$$

$$= 1.794\, R^{2/3}$$

$$Q = 1.794\, A\, R^{2/3}$$

Assume a total depth of flow of 4 feet.

Q in Central portion X—

$Q = 4.078\, A\, R^{2/3}$

Wetted area A = 250.56 sq.ft.

Wetted perimeter P = 64.94 feet.

$\therefore$ Hydrulic Radius R = 3.865 feet.

$\therefore V = 4.076 \times (3.865)^{2/3} = 6.734$ feet.

$\therefore Q = 4.076 \times 250.98 \times (3.865)^{2/3} = 2520$ cusecs.

Q in side (Y)—

Wetted area A = 158.0 sq.ft.
Wetted perimeter P = 73.87 feet.
∵ Hydraulic Radius = R = 2.165 feet.
∵ $V = 1.794 \times (2.165)^{2/3} = 3.001$ feet/second.
$Q = 1.794 \times 158.6 \times (2.164)^{2/3} = 476$ cusecs.

Similarly, in side (Z), Q = 476 cusecs.

∴ Total discharge in canal = 2520 + 476 + 476 cusecs.
= 3472 cusecs.

as against the full supply discharge of 3,443 causecs.

5. 3. 2. The conventional channel section designed to carry the full supply discharge of 3443 cusecs under the given conditions is shown in sketch No. 5 (see below). The velocities obtaining in the new type of channel and the conventional one have been shown plotted against the corresponding discharges in Figure 10[22]. At a discharge of 100 cusecs, velocity in the conventional section is 1 foot/second and the velocity in the in the new type of section is 2.75 feet/second, i.e., 2.75% more than in the case of the conventional section. From the Figure 10[22], it can be seen that non-silting velocities are induced in the new type of channel section even at low discharges and it may be reasonably expected that silting of the bed may not occur in this section. The maximum velocity induced in the lined section when the full supply discharge is flowing, is about 10 feet/second and this can be withstood by this lined canal.

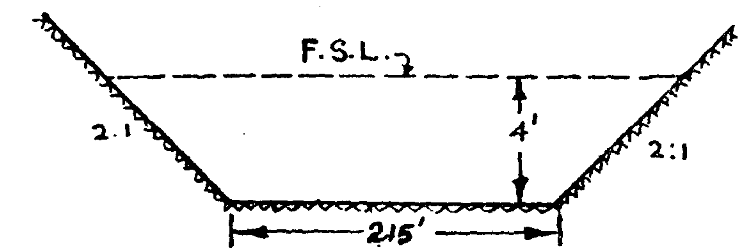
5. 3. 3. The final designed section is shown in Figure-4. This section has been superimposed on the latest cross sections of the channel received from the field and is presented in Figure 5 through 8. The second drop in the channel system occurs at the second mile from the headsluice and the new type of section will be carried up to the second crop only. Below the second drop, the channel design may be in accordance with the regarding proposals already envisaged. The velocities realisised in the reach below the second drop are presented in Figure-11 both before and after the improvements. Since increased velocities are induced in the regraded design of this reach, it is reasonable to expect non-silting condition in this reach also.

5. 4. Design of transition :

The transition from the headsluice structure to the two-zoned section below it will be suitably designed and a tentative design of the transition is shown in Figure-9. As the transition has to withstand high velocities, it may be constructed in masonry or lined suitably.

6. ECONOMICS OF THE DESIGN.

The conventional channel section to carry the full supply discharge of 3443 cusecs under the given conditions has been designed using the Manning's formula in the usual manner and is shown in the figure below. This is also the section adopted in the regarding proposal.



From an economical point of view, the new type of section involves lesser volume of earthwork than the conventional one but entails the cost of lining. The volume of earthwork saved by adopting the new type of section has been calculated and found to be 2,930,000 c.ft. (Please see calculation sheet attached).

For 1 mile length of canal, volume of earth work saved = 2930 units.
∵ Amount saved by adopting the new type of section = 2930 x 20 = Rs. 58,600/-

Rate for earthwork is taken as Rs. 20/- including lead and lift.

Extra cost of lining the canal for the present design :

Taking one running foot of canal section and the lining thickness as 4" volume of lining work
= 64.94 x 1/3 x 1 = 21.65 Cft.

For 1 mile length of canal, volume of lining work = 21.65 x 5,280 = 1,14,500 Cft.
= 1,445 units.

Assume o cost of Rs. 100/- per unit of lining work
Total cost = 1,145 x 100 = Rs. 1,14,500/-

∴ Net extra cost involved by adopting the new type of design = 1,14,500 - 58,600 = Rs. 55,900/-
or say = Rs. 56,000/-

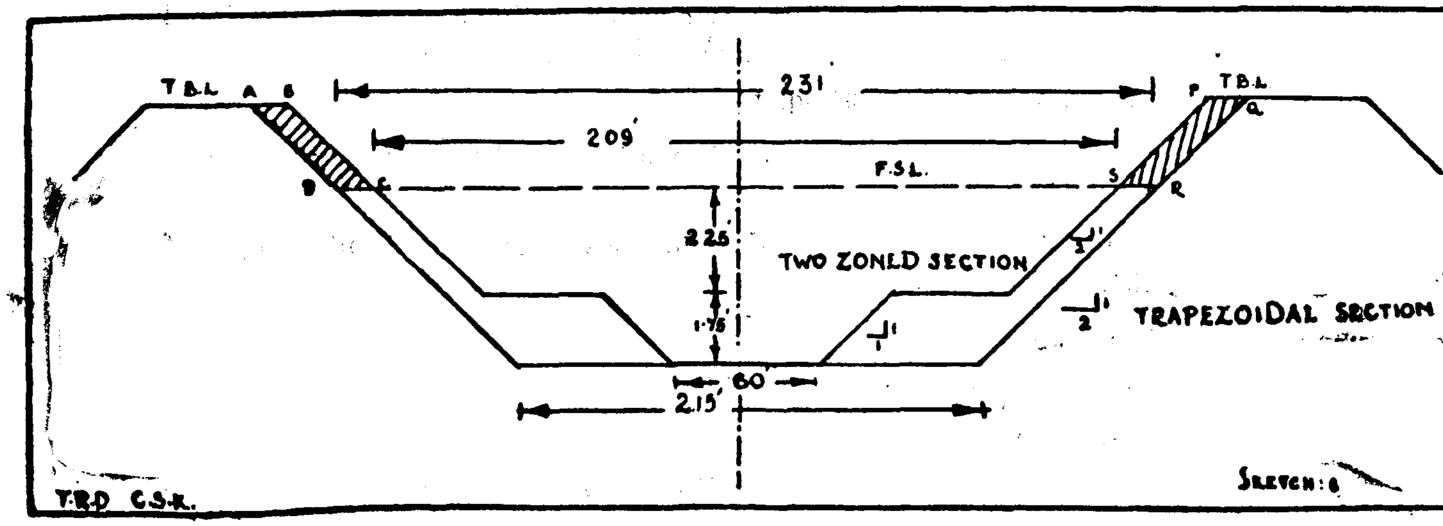
This extra cost is capital outlay will more than offset the cost of maintenance of the old type of channels by desilting annually.

7. CONCLUSION

The new type of design that has been presented in this paper may be tried as an experimental measure in one of the Palar anicut channels, namely, the Kaveripauk Channel. It will be of immense interest to the hydraulic engineer in charge of design of channels to see how far this new type of design is effective in solving the silt problem successfully. Judging by the hydraulic computations and figures furnished, the present design is expected to be effective in flushing the silt away from the entrance of the channel, thereby sending down the designed discharge into the channel without undue rise of rear full supply level in the channel.

8. REFERENCES

1. "Hand book of Hydraulics" by H. W. King
2. "Hydraulics of steady flow in open channels" by C. J. Posey and S. M. Woodward.
3. "Hydraulics and its applications" by A. H. Gibson.
4. "Water-power Engineering" by H. K. Barrows



CALCULATIONS FOR QUANTITY OF EARTHWORK SAVED BY ADOPTING THAT TWO-ZONED SECTION.

Take one cross-section of the channel in the figure above, the two types of sections are shown superimposed up to the top of banks with the proper dimensions marked on them.

I. Upto F.S.L.

Area of cross section of two zoned section = 568.16 sq.ft.

Area of cross section of the trapezoidal section $= \frac{(215+231)}{2} \times 4$

$= 892$ sq.ft.

$\therefore$ Difference in area $= 892 - 568.16 = 323.84$ sq.ft.

II. From F.S.L. upto T.B.L.

The vertical distances between F.S.L. and T.B.L. are taken from figures 5 to 8 at all the cross sections shown in the figures.

At C.S. 1,	the vertical distance	— 19 feet.
C.S. 2,	do	— 10 ,,
C.S. 3,	do	— 10 ,,
C.S. 4,	do	— 14 ,,
C.S. 5,	do	— 9 ,,
C.S. 6,	do	— 9 ,,
C.S. 7,	do	— 7 ,,
C.S. 8,	do	— 6 ,,
	Total ...	84 feet

Therefore, average vertical distance in the first reach, up to first mile $= \frac{84}{8} = 10.5$ feet.

Difference in area between the two zoned section and the conventional section = Areas A B C D + Area P Q R S (shown shaded in Figure).

= 2 x 11 x 10.5 = 231 sq.ft.

∵ Total area of excavation saved by adopting the two zoned section = 323.8 + 231 = 554.8 sq.ft.

∵ Volume of earthwork excavation saved for the 1 mile length of canal = 554.8 x 5280 c.ft.

= 2929344 c.ft.

= 2929.34 units (1 Unit — 1000 c.ft.).

or say 2930 units.

This figure has been taken for discussion of the economics of the two designs.

STATEMENT No 6.

(TABLE 1.)

COMPUTATION TABLE FOR FLOW DURATION CURVE OF KAVERIPAUK CHANNEL.

(Taken from the book "Water Power Engineering" by H. K. Barrows).

S. No.	Flow in c.fs.	1944-45	1946-47	1954-55	Total number of days.		Percent-age of time.	emarks.
(1)	(2)	(3)	(4)	(5)	(6)		(7)	(8)
1.	0.24		1 .	1		140	100	
2.	25.49	......6 .			7	138	98.6	
3.	50.99	........8	6 ..	2	16	131	93.6	
4.	100.124 ..	2 ..	2		4	115	82.1	
5.	125.149 ..	2	5		7	111	79.4	
6.	150.199	5	7		12	104	74.3	
7.	200.224 ...	3	1		4	92	65.7	
8.	225.249 .	....	4		5	88	62.9	
9.	250.299	4	4		8	83	59·3	
10.	300.349	......7 ..	2		9	75	53.6	
11.	350.399 ..	2 ..	2 ..	2	6	66	47.1	
12.	400.449 .	1 ..	2 ..	2	5	60	42.9	
13.	450.499 ..	2 .	1 ..	2	5	55	39.3	
14.	500.599	..	2	4	6	50	35.7	
15.	600.699 ...	3 .	1 ..	2	6	44	31.4	
16.	700.799 ..	2 ...	3 .	1	6	38	27.1	
17.	800.899 ..	2 ...	3		5	32	22.9	
18.	900.999 ..	2	2		4	27	19.3	
19.	1000.1249	.....	11		11	23	16.4	
20.	1250.1499 .	1 ..	2 ..	2	5	12	8.6	
21.	1500.1749	..	2 .	1	3	7	5.0	
22.	1750.1999				—	4	2.9	
23.	2000.2499	•	..	2		4	2.9	
24.	2500.2999		..	2	2	2	1.4	
25.	3000.3499				—	—	0	
	Total ...	53	64	23	140			

Column 1.—Serial No.

Column 2.—The flow in the canal has been divided arbitratly into different flow ranges and Col. 2 shows the assumed flow ranges.

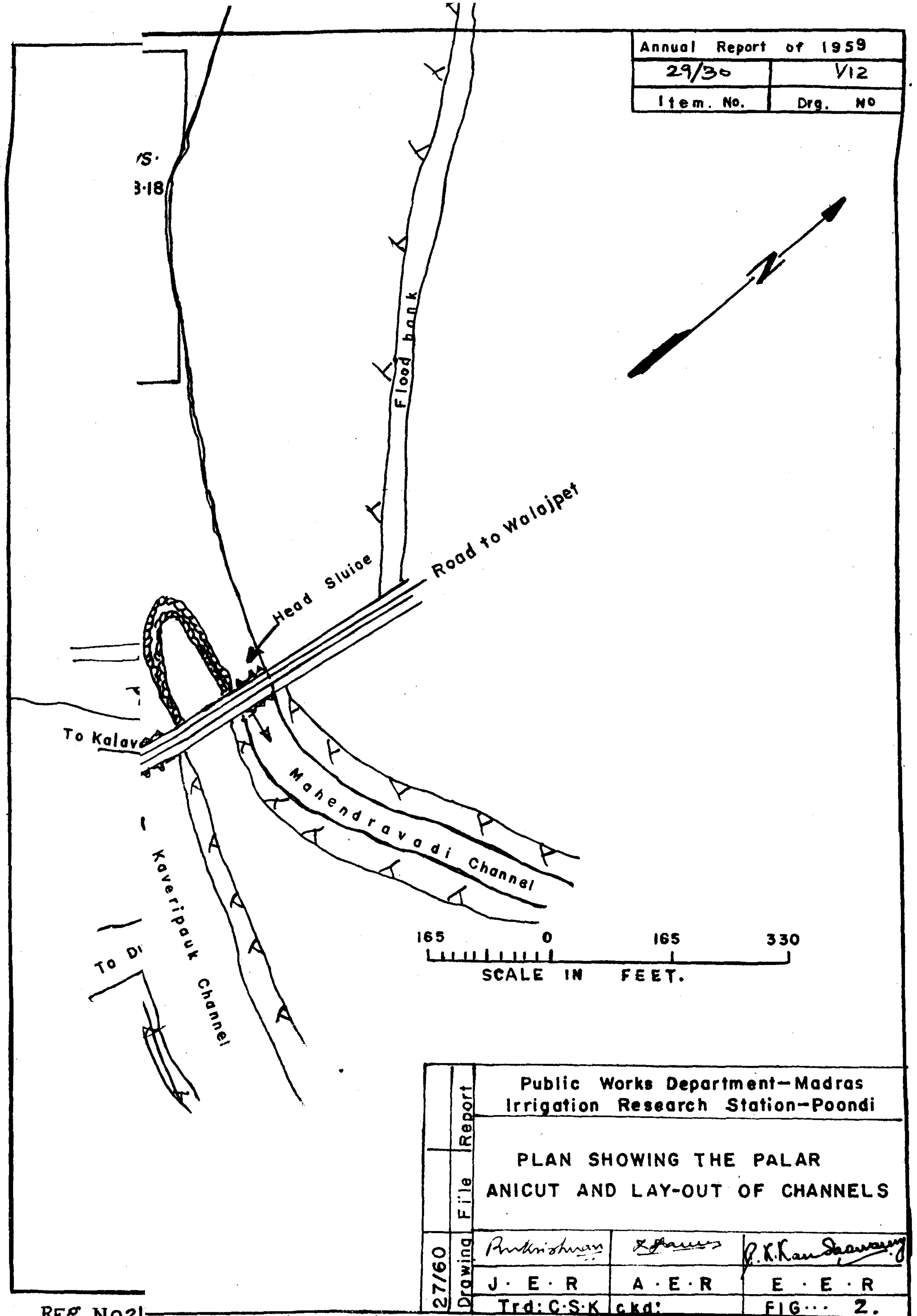
Column 3, 4 and 5.—The years taken for analysis.

Column 6.—The total number of days the flow in the canal was within the corresponding flow range during the period considered.

Column 7.—The progressive total of the number of days starting from the bottom.

Column 8.—Percent of time calculated assuming the flow has been equal to or more than zero cusecs for 100 days.

Column 9.—Remarks.



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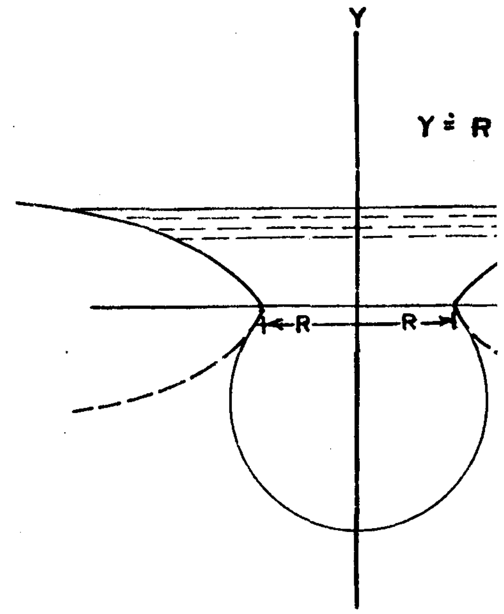
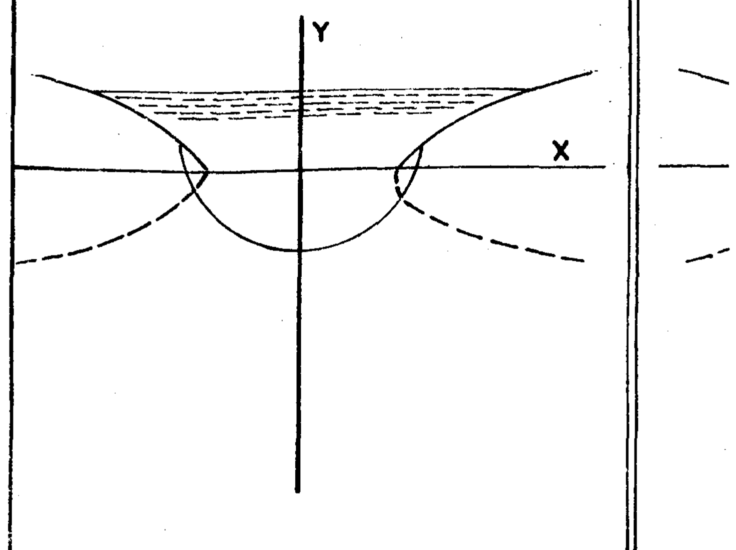
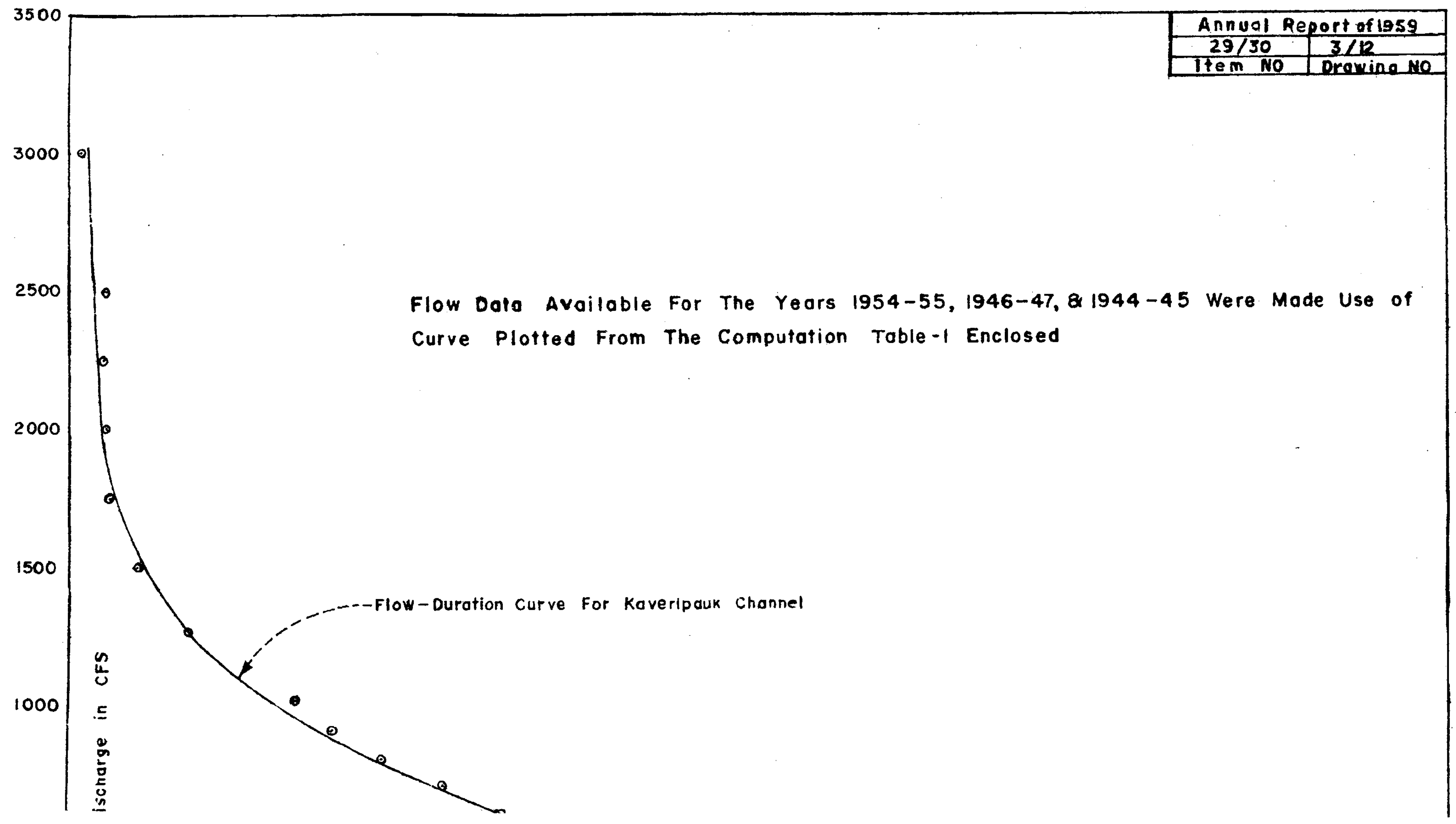




LEGEND:

R = Hydraulic Radius

XY = Co ordinates Along X and Y Axis


Annual Report of 1959
29/30
3/12
Item NO
Drawing NO
Flow Data Available For The Years 1954-55, 1946-47, & 1944-45 Were Made Use of
Curve Plotted From The Computation Table-I Enclosed
Flow-Duration Curve For Kaveripauk Channel
ischarge in CFS
3500
3000
2500
2000
1500
1000

LEGEND:

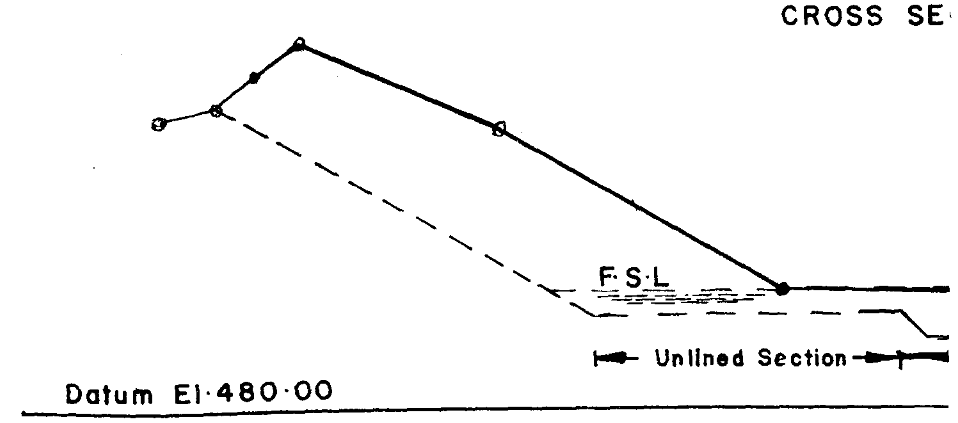
F·S·L :- Full Supply Level

LEGEND:-

F·S·L - Full Supply Level

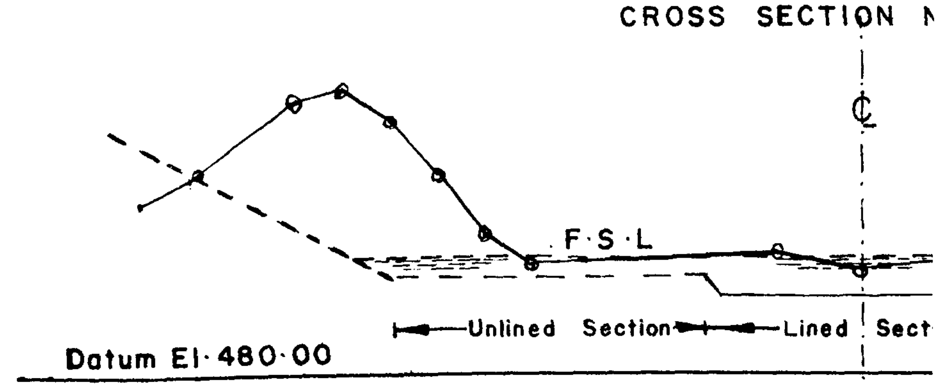
Area of Excavation 27 500 Sq·Ft·

CROSS SE



Area of Excavation 27800 Sq·Ft·

CROSS SECTION N



Scale- Horizontal 1" = 40 Ft·
Vertical 1" = 20 Ft·

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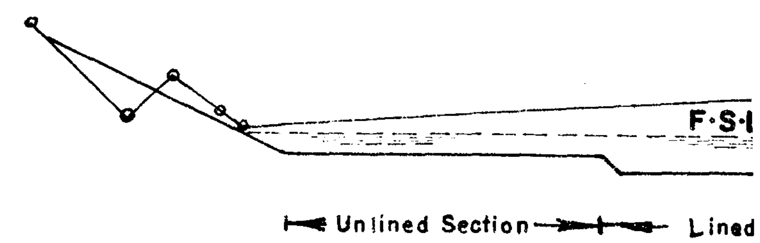
LEGEND:

F·S·L :- Full Supply Level

CROSS SECTION N

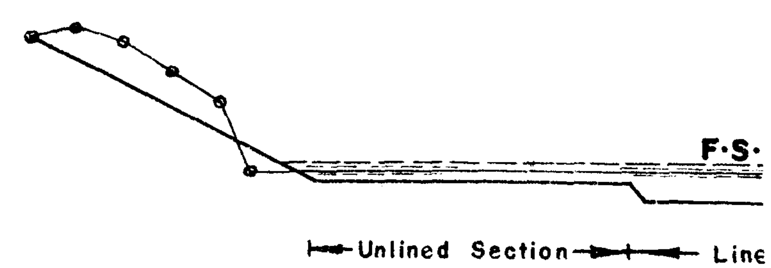
Area of excavation 10,200 Sq·Ft·



Datum El·480·00

CROSS SECTION

Area of excavation 7040 Sq·Ft·



Datum El·480·00

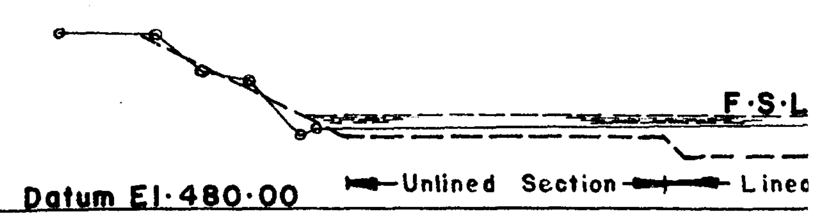
Scale – Horizontal 1" = 40Ft·
Vertical 1" = 20Ft

LEGEND

F·S·L:– Full Supply Level

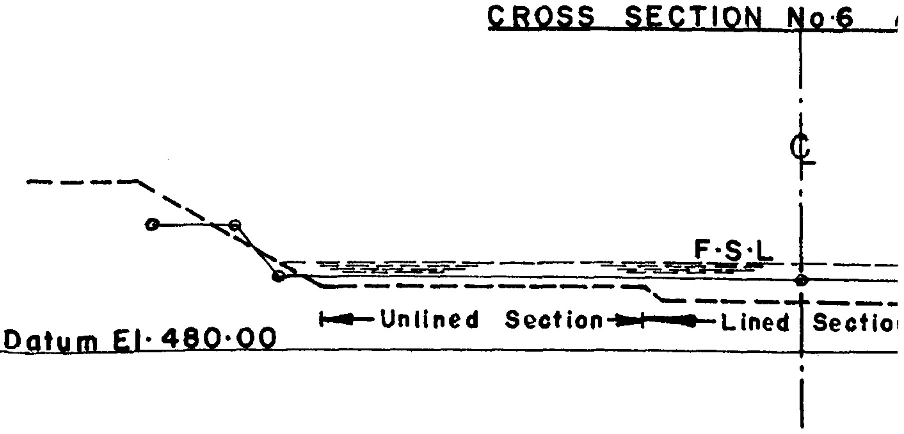
CROSS SECTION No

Area of excavation 5120 Sq·Ft·



Area of excavation 4480 Sq·Ft·

CROSS SECTION No·6



Scale– Horizontal 1" = 40Ft·
Vertical 1" = 20Ft

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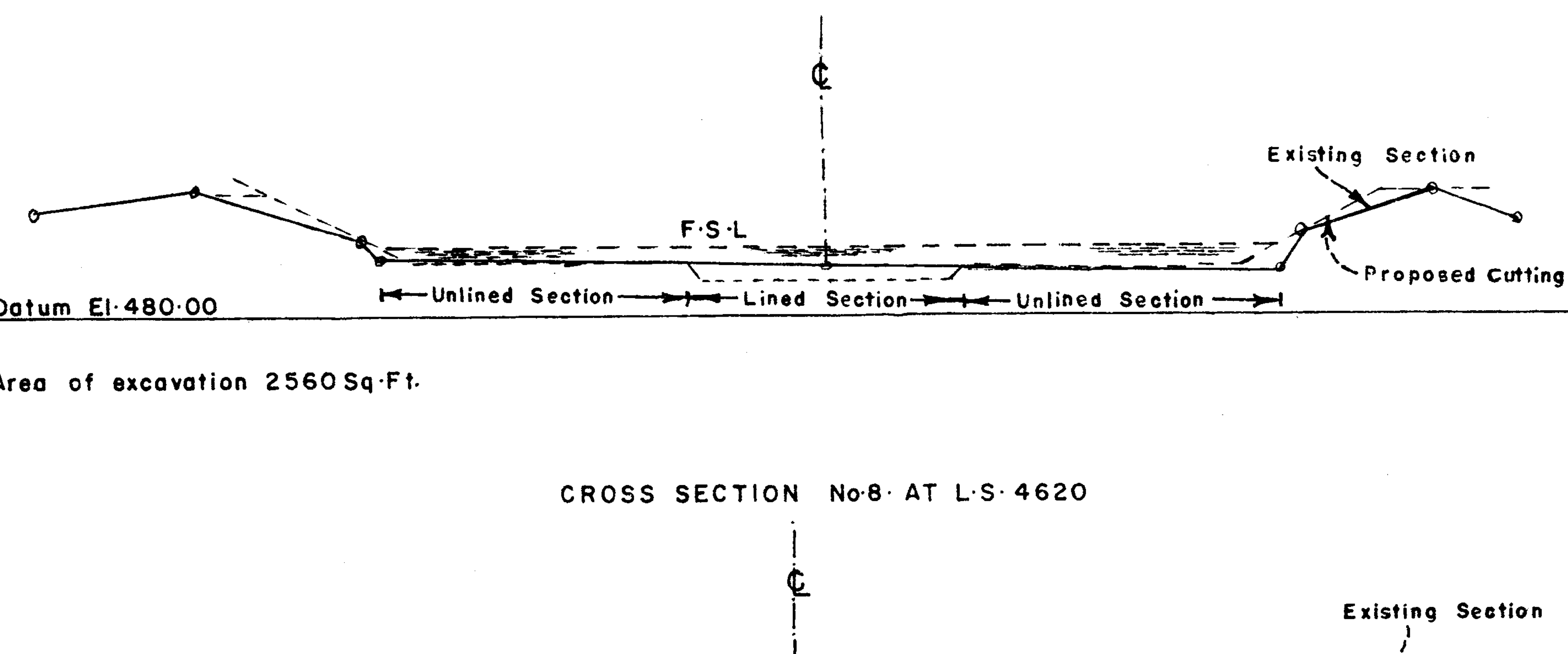
Item NO	Drawing NO

LEGEND

F·S·L. Full Supply Level

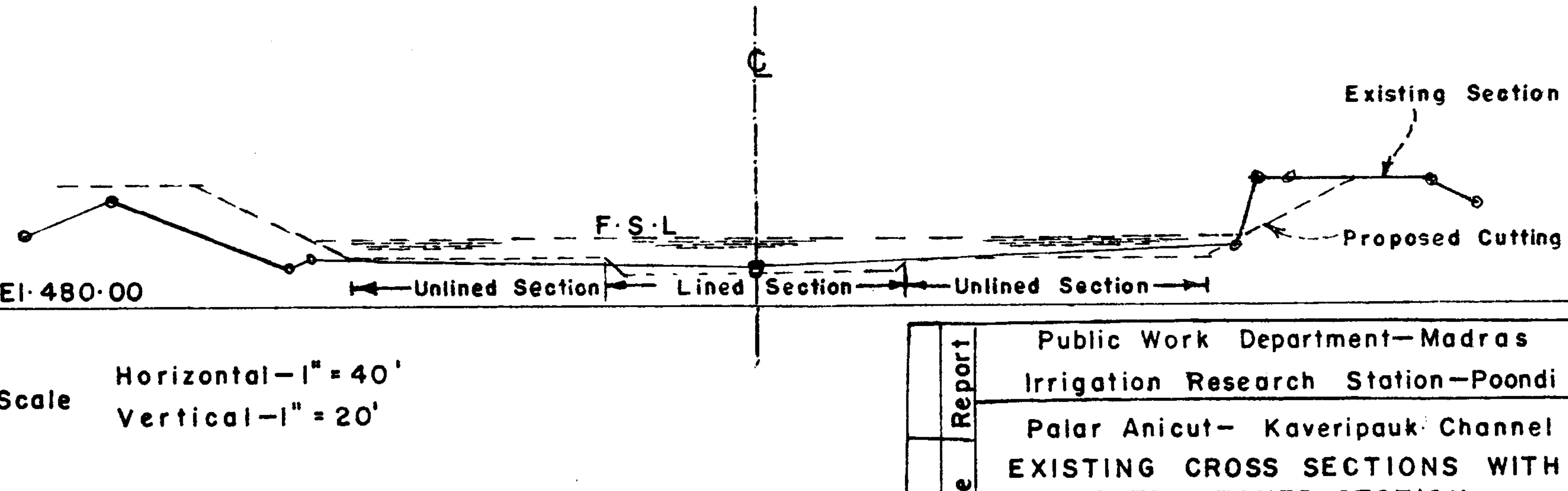
Area of excavation 1920 Sq·Ft·

CROSS SECTION No·7 AT L·S·3960



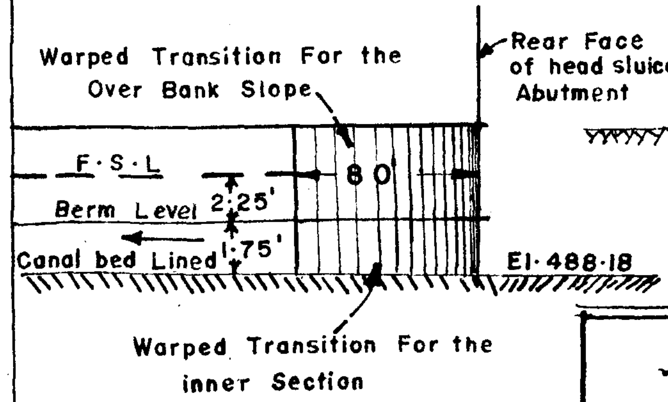
Area of excavation 2560 Sq·Ft·

CROSS SECTION No·8· AT L·S·4620

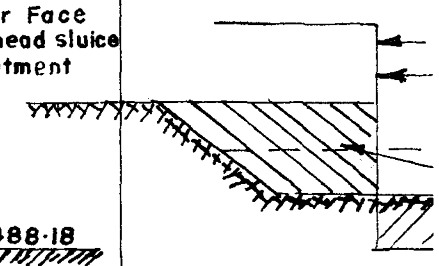


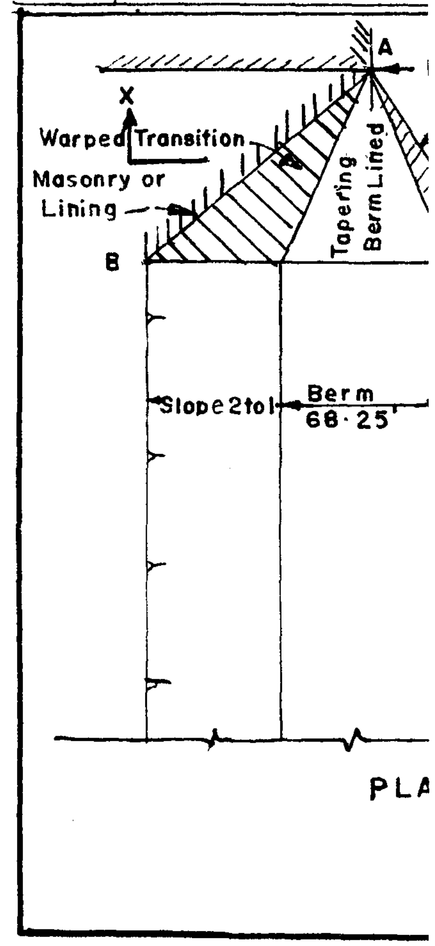
Scale Horizontal—1" = 40'
Vertical—1" = 20'

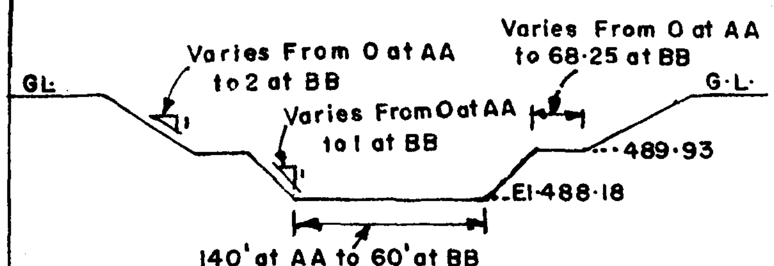
Report	Public Work Department—Madras Irrigation Research Station—Poondi		
File	Palar Anicut— Kaveripauk Channel EXISTING CROSS SECTIONS WITH THE TWO ZONED SECTION SUPERIMPOSED		
Drawing 34/60	J·E·R	A·E·R	E·E·R
	Trd: C·S·K·	ckd:	FIG···8



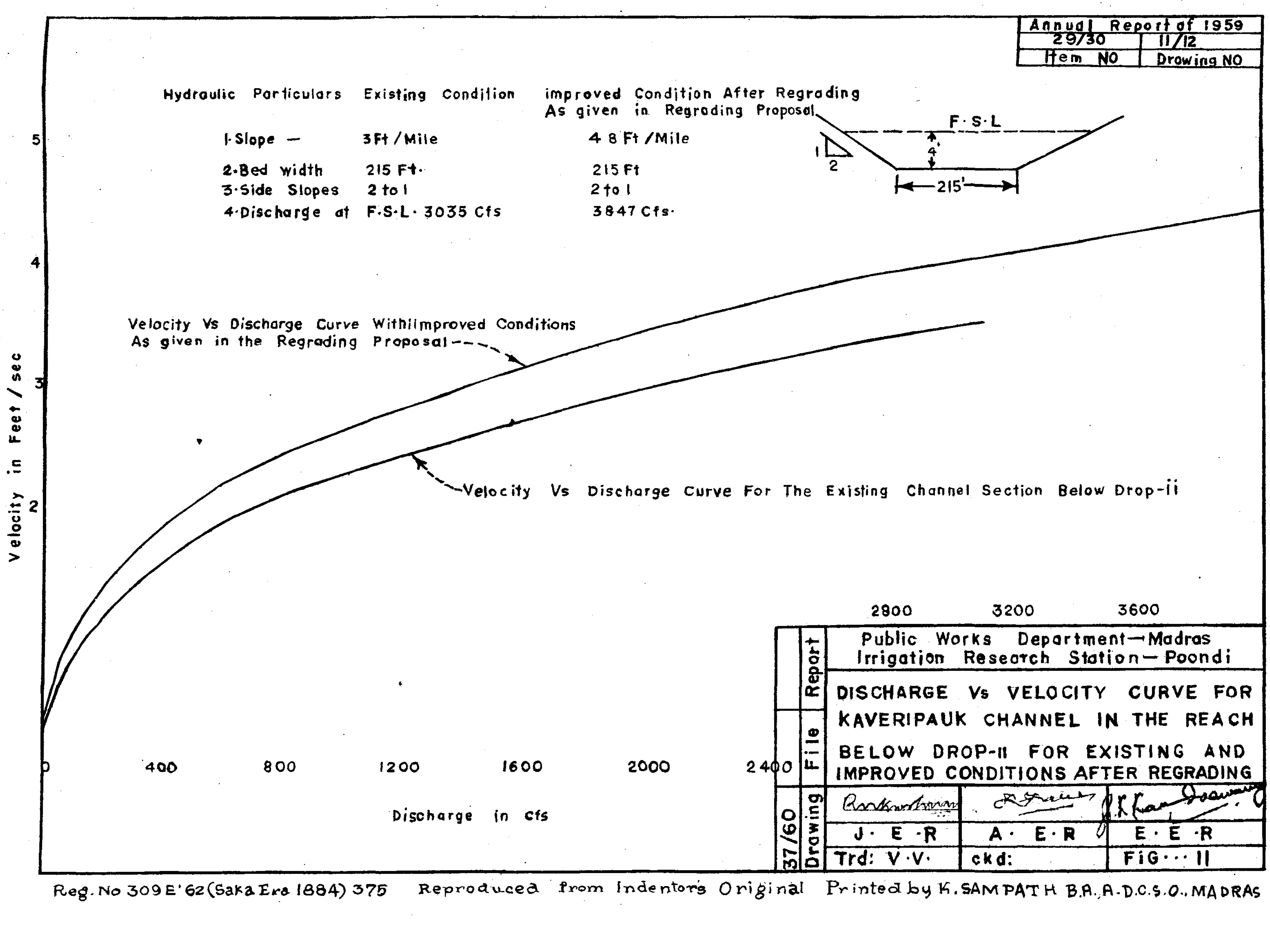
SIDE SECTIONAL ELEVATION







SECTION X-X



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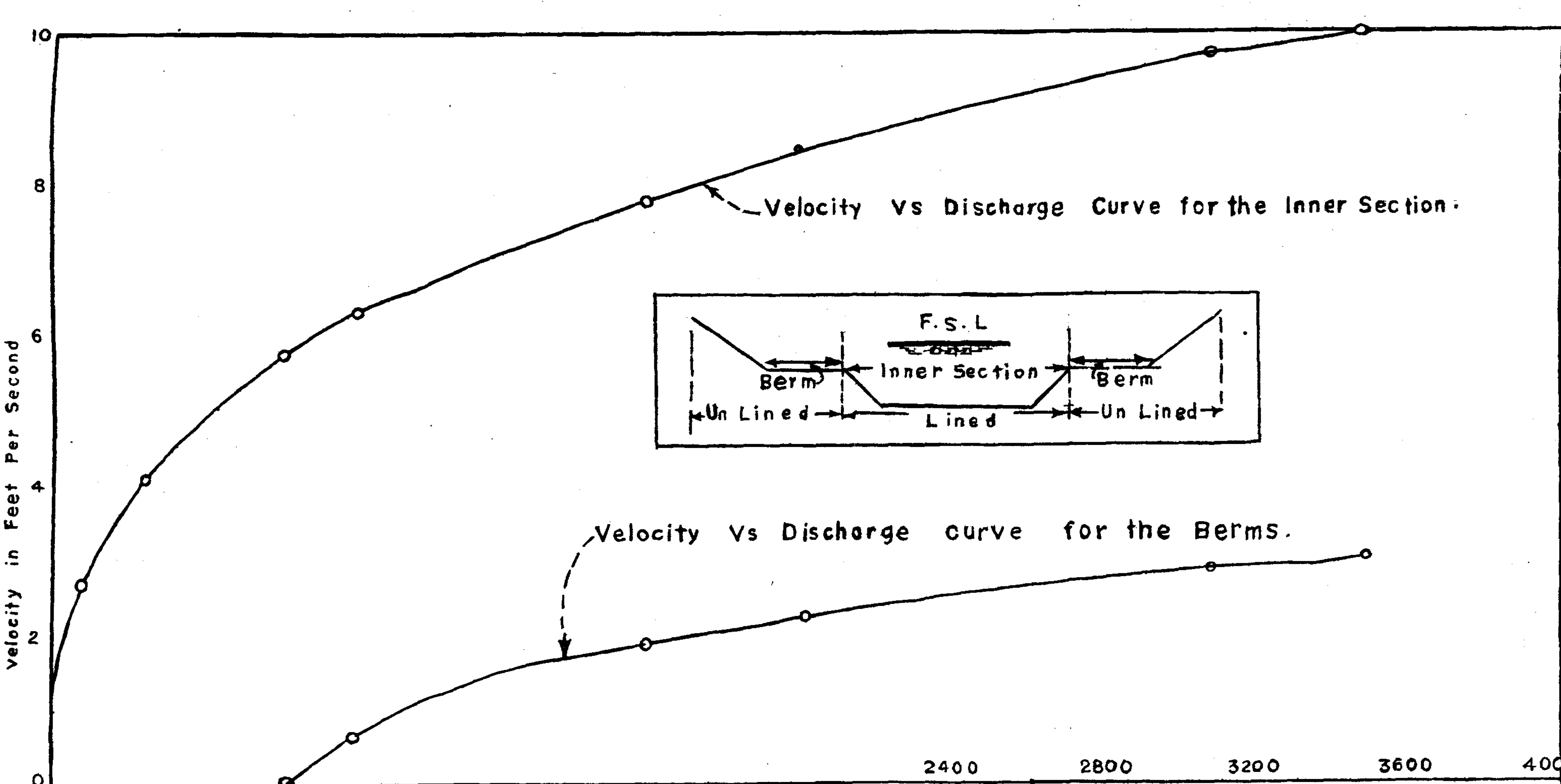


Public Works Department – Madras
Irrigation Research Station – Poondi
Palar Anicut – Kaveripaur Channel

VELOCITY vs DISCHARGE CURVE FOR THE TWO ZONED SECTION

Report	File	Drawing 33/60
J.E.R	A.E.R	E.E.R
Trd. C.S.K	Ckd.	FIG...12

Annual Report of 1959	
29/30	12/12
Item NO	Drawing NO

Reg.No.310'E.62(Saka Era 1884) 375. Reproduced from Indentor's Original Printed by Sri K.SAMPATH B.A.,A.D., C.S.O., MADRAS

Flushing the River Cooum by Tidal action of the Sea

1. SYNOPSIS.

1. 1. This report mainly deals with the studies conducted to arrive at an optimum section for channelising the river Cooum to increase the tidal influx into the river for effective flushing. The nature of problem has been given and a brief description of the models constructed and the tidal mechanism to produce tides at the sea has also been given. The computation method for calculating the tidal discharge and tidal head at various places along the river has been briefly explained. The studies conducted for various proposals have been discussed in detail and the results analysed. The limitations of the model studies conducted have been indicated. Based on the results of the model experiments conducted, recommendations on the following aspects have been made with the primary objective of flushing the Cooum River by the tidal action to the extent possible, consistent with economy and feasibility of the proposal recommended (Vide Figure-14).

(1) Channelization of the river course.

(2) Trimming the north arm of the Cooum river around the Island grounds.

(3) Control works and the training works at the mouth of the Cooum River.

2. INTRODUCTON.

2. 1. The River Cooum has its origin about 40 miles to the west of Madras above an anicut across Kortaliyar at Kesavaram which is 13 miles south-west of Thiruvallur. Within the City of Madras the river splits itself into two branches near its mouth resulting in the formation of the Island ground and the two arms join once again just west of the Napiers Bridge. The Buckingham Canal also enters into the north arm and the South Buckingham Canal starts from the south arm of the river just west of the University Buildings. The course of the river Cooum within the city limit is shown in Figure-1. The maximum flood discharge of the river is about 20,000 cusecs. The Cooum is cut off from the sea by a sand bar which is opened only during the monsoon floods. Generally, the bar is open only for about three to four months in the year, i.e , from October to January and during this period the river becomes tidal and receives the benefit of the flushing action of the sea water. During the remaining months when there is no flow in the river this bar is closed by the action of the waves and littoral drift along the coast and the Cooum has no connection with the sea and becomes a stagnant pool. Also some quantity of sewage from the city sewarage system is let into the river due to the insufficient capacity of the sewage pumping stations.

3. PROBLEM.

3. 1. For most part of the year, the river Cooum remains a stagnant pool of water since its direct communication with the sea is intercepted by the sand bar at its mouth. As the water level goes down, due to the exposure of mud banks strewn with decaying organic matter and due to the concentration of sewage resulting from the reduction in dilution a noxious stink is produced which is an impending danger to the hygienic condition of the Madras City.

3. 2. If the river is made tidal throughout the year by keeping the bar cleared constantly and further inflow of sewage into the river is prevented, the river will most probably become a clean, tidal river. But, unfortunately the maximum tidal range obtained in the sea of Madras is only about 4 feet and this small range has to be utilised to produce flushing the river to the best possible extent. For achieving this, a narrow and well maintained section has to be cut and maintained in the bed of the river so that the tidal action of the sea is fully utilised, for the best possible tidal propogation in the river.

3. 3. Various aspects have to be considered, such as the optimum width of the section, depth of the section, narrowing the width towards the upper reaches, reducing the width of the north arm of the river, the width of cut required to be maintained in the bar site in keeping with economy, and efficiency, the control work at the river mouth such as a weir etc. Since scale model experiments are now commonly accepted as an important aid to hydraulic Engineering design, detailed studies have been conducted in this station and also these aspects have been alayised theoretically.

4. MODEL.

4. 1. A model of the Cooum river within the Madras City limits, i. e., upto 30,000 feet from the sea mouth was constructed to a scale of 1/400 Horizontal and 1/20 Vertical. The horizontal scale was fixed as 1/400 for accommodating the river upto Aminjikarai within the space available for constructing the model. As regards the choice of the vertical scale, the governing factors are :

(1) that the motion of water should be turbulant so that the possibility of scale effect due to visocosity is reduced to a minimum ;
(2) velocities obtained should be sufficiently high and measurable ;
(3) vertical exaggeration should not be too large as it will impose a severe strain on the sand at a slope or too small as to make the measurements of small tidal ranges trouble-some (1).

4. 2. As the study mainly pertains to the tidal propagation in the river and no sediment movement or bank erosion comes into the picture, a rigid bed model was considered suitable and criterion (1) mentioned above was applid to fix up the vertical scale. The time scale was adopted as follows :

Time scale $T = \frac{X}{\sqrt{Y}}$

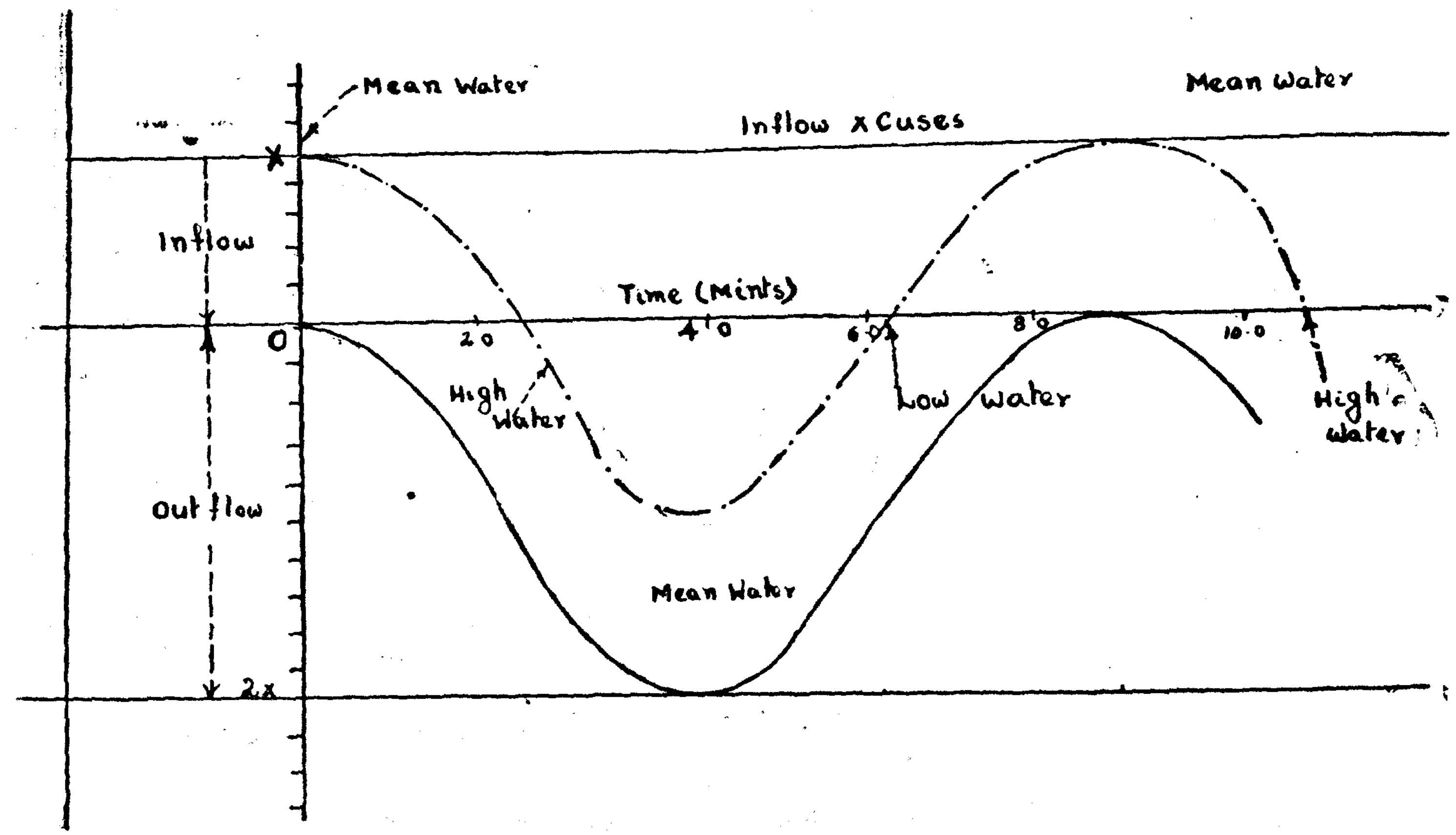
Where 1 : X is the horizontal scale
and 1 : Y is the vertical scale.

According the this time scale the period of one tidal cycle in the model works out to 12.41/400 x $\sqrt{20}$ = 8 units 20 sec. The model was moulded with the existing cross-sections in the river portion and the studies conducted.

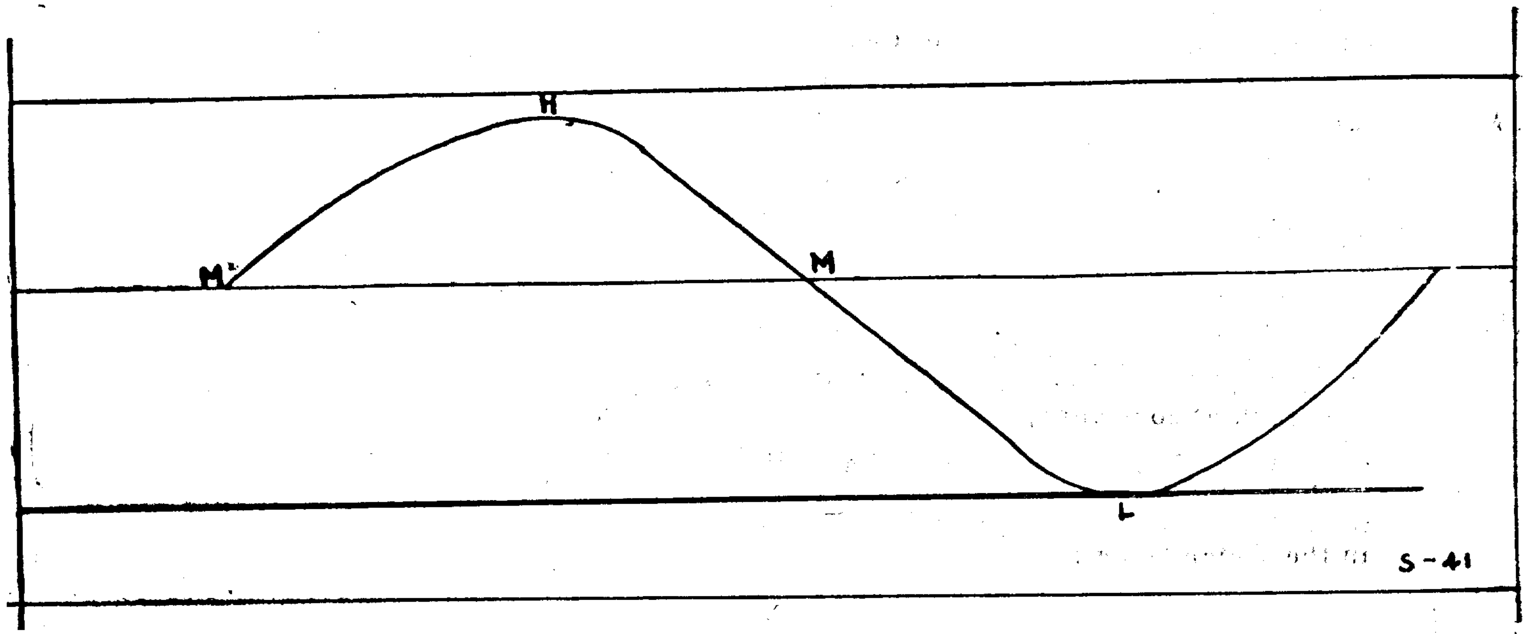
4. 3. For conducting studies to arrive at proposals for flushing the river by utilising the tidal flow, tides have to be generated at the sea end. A simple mechanism was enolved to produce the tides of the required pattern onthe principl es ennunciated below :

4. 4. It is well known that when a circulatory motion is converted into a translatory motion the latter will be simple harmonic, provided the connecting rod is along in comparison to the Crank radius. Considering the discharge characteristics over a spillway, the discharge varies linearly with the gate openings in major part of the range. That is, if the gate opening varies in the certain pattern within the linear range, then the discharge also will vary in the same pattern. Combining this and the principle of simple harmonic motion, it can clearly be concluded that if a gate is connected on to a crank shaft by a fairly long connecting rod so that the variation in the gate opening lies within the linear range of the discharge characteristic, a simple harmonic variation of discharge will be obtained by rotating the crank at constant speed. (2)

4. 5. Now, let us suppose that a discharge of X cusecs is let into the model and an overflow gate is so operated on the principles stated above that the outflow varies from 0 to 2X cusecs. The net inflow will vary as shown in the sketch below :



The horizontal line above X'O shows the constant inflow of X cusecs. The harmonic curve below X'O shows the varying outflow. Superimposing the latter curve over the former the resultant inflow into the tray is obtained. Now examining a simple harmonic tidal curve shown below, it can be seen



that the maximum rate of variation of water levels occurs at Mean sea level there being two slacks going by the same of high water and low water slacks. The area of the water surface being constant in the tray of the model, the variation in water level will be proportional to the net inflow. Hence, it can be clearly seen that if mean water level is maintained at the starting point of sketeh-1, the first zero net inflow point will give the high water and the lowest point will show another mean water and the second were net inflow will show low water. Based on these principles, the tidal mechanism was designed. The details of the mechanism are given in Figure-2 and the mechanism was perfected after a number of trials.

4. 6. Studies were conducted in this model constructed to a scale of 1/400 horizontal and 1/20 vertical with the existing prototype cross-sections simulated in the model. The tidal propogation into the river due to the maximum tidal range of four feet when the mouth of the river was kept fully open was observed at various sections of the river. The cross section at a distance of 700 feet from the Napier Bridge was continued up to the sea, and the width of the cross-section at the top was 600 feet. The rise and fall of the water levels at various sections at regular time intervals were taken simultaneously. It was found that the tidal range goes on reducing as it travels upstream of the river and the water level due to tidal action does not go below mean sea level in the upper reaches of the river.

4. 7. Reynolds criterion for ensuging turbulant flow requires that

$h^3 \frac{x}{y} \sqrt{\frac{H}{30}} >$ should be greater than 0.09 where

1 : x is the horizontal scale
1 : y is the vertical scale
h is the spring tidal range at the mouth of the model.
and H is the spring tidal range at the mouth of the prototype.

In the present model, where the horizontal reach is 1 : 400 and vertical scale 1 : 20,

$h^3 x \sqrt{\frac{H}{30}}$ works out to .058

Though this is less than criterion given by Reynolds, the functioning wa quite satisfactory. This is substautial by the statement in the first referenc cited at the end of the report that Reynold himself experimented som models where the value of h^3 or $h^3 \frac{x}{y} \sqrt{\frac{H}{30}}$ worked out to be less than ·09 but still the models gave satisfactory results. Joglakar in his article on "Hydraulic model studies-Limitations[3] has mentioned that in the case of the Hoogly model this value worked out to be only 0.045, but still the model gave satisfactory results. He further emphasises that it must be clearly understood that though Reynold's criterion may be suitable for estuaries which are very wide relative to their depth, yet where the depth is considerable relative to the width as is the case in river channels adopting correct vertical exaggeration is very important.

Though the model in question gave satisfactory performance it was felt that the vertical exaggeration of 20 was too much. Further, it was proposed to retain this model in which the original cross sections had been incorporated intack and the proposals of channelisation tested in another model so that any comparative study can be conducted, if required. Hence, another model with a vertical exaggeration of 10 and with a horizontal scale 1:200 and vertical scale 1 : 20 was constructed for studying the different channelised sections of the river. The value of $h^3 \frac{x}{y} \sqrt{\frac{H}{30}}$ in this case works out to only ·029 but due to the above reasons pointed out, the model could be safely relied upon, for giving satisfactory performance. The river Cooum was simulated from the sea mouth upto the Chetput Railway Bridge in the new model. The same tidal mechanism with some slight changes was utilised to produce a tidal range corresponding to four feet. In this model, the period for one tidal cycle works out to be $12.41/200 \times \sqrt{20} = 16$ minutes 40 seconds. Moreover, in order that the motion in a tidal model shall be similar to that in the prototype, it is necessary that the velocity shall be sufficiently great to ensure "turbulent flow" and from experiments on models of symmetrical estuaries Reynold came to the conclusion that in order to ensure this, the product "h^3e" should not be less than 0.09 where 'h'. is the rise of tide in feet at the seeward end of the model and 'e' the vertical exaggeration of scale.[3] In the case of the existing model,

$h - 4/20$ (the tidal range being 4 feet).
$- 0.2$ feet.

$e - \frac{H}{Y} - \frac{400}{200} - 20$

$h^3e - (0.2)^3 \times 20 - 0.008 \times 20$
$- 0.16$

which is much higher than 0.09.
Suppose, a vertical exaggeration of 10 is adopted in the model with the same vertical scale, Then,

$$h^3e - (0.2)_3 \times 10 - 0.008 \times 10$$
$$- 0.08$$

which is very close to the Teynolds criterian of 0.08. Therefore, a bigger model to a scale of 1/200 Horizontal and 1/20 vertical with the vertical exaggeration of 10 was constructed by the side of the existing smallmodel, simulating the Cooum River upto Chetput Railway Bridge as in the previous model. The same tidal mechanism with slight modifications was used to produce a tidal range of four feet for conducting further studies.

5. MATHEMATICAL COMPUTATION.(4)

5. 1. The discharge due to tidal action and the tidal head at various places were computed mathematically. The two equations representing the tidal motion are the "Continuity Equation" and "Eluration Dynamic Equation." The "Continuity Equation" is the law of conservation of mass stating that in a short length of channel the volume of water stored up is equal to the difference between the volume of water entering the channel and the volume leaving the reach in a short interval time. The "Danamic Equation" of tidal motion is based on Newton's Second Law. The law of conservation and variation of momentum per unit length in the longitudinal direction of the estuary is used. The two equations are given below:

The "Continuity Equation" is

$$\frac{dq}{dx} + B \frac{dh}{dt} = O$$

The "Dynamic Equations" is

$$\frac{dh}{dx} + \frac{v}{g} \times \frac{dv}{dx} + \frac{1}{g} \frac{dv}{dt} \frac{v/v/}{c^2z} - O$$

5. 2. For solving these equations there are ever so many methods like harmonic method. finite difference method, characteristic computation method, method of iteration etc. Of all these methods the iterative method was got valid reasons for its advantage over other mathematical methods in that it involves comparatively less labour in providing fairly accurate results.

5. 3. For obtaining the solution for these equations mentioned above, by the method of iteration, these equations have to be integrated over x from O to x after successive approximation (in this case after the second approximation). Finally we get the equations giving the values of tidal discharge and tidal head with respect to distance and time as follows:—

$$Q_{..} = Q_o + B h'_o x + \frac{B\, Q''_o\, x^2}{2gbz} + \frac{B/Q_o/Q'_o\, x^2}{c^2 b^2 z^3} - 3/2\, \frac{B\, Q_o/Q_o/h'_o\, x^2}{c^2 b^2 z^4}$$

and

$$h_{..} = h_o + \frac{Q'_o\, x}{gbz} + \frac{Bh''_o\, x^2}{2gbz} - \frac{(B+b)\, Q_o\, h'_o\, x}{gb^2z^2} + \frac{Q_o/Q_o/\, x}{c^2b^2z^3} + \frac{B/Q_o/h'_o\, x^2}{c^2b^2z^3}$$

were

$Q_{..}$ — the discharge at a place at a distance of x from the sea mouth
Q_o — the discharge at the sea mouth
$h_{..}$ — Tidal head at a place at a distance of x from the sea mouth
h_o — Tidal head at the sea mouth
B — Top width of the channel
b — bottom width of the channel
z — Depth of water in the channel
C — Chezy's coefficient
g — acceleration due to gravity.

Note : The suffix in $Q_{..}$ and $h_{..}$ denotes the second order of trial made.

5. 4. To begin with the computation, an approximate estimation of the discharge and tidal curves at the downstream end of the river are necessary. The tidal curve is roughly estimated from the tide tables.

The discharge is estimated from the relation

$Q = Qu + Sh'$

were

Q — discharge at downstream and
Qu — fresh water upland discharge
S — Tidal surface area at an instant 't'
h′ — gradient of tidal head at the iime 't'.

5. 5. Here diffierent channelised sections were assumed and the discharge and tidal head at diffierent sections along the river were computed. For different instants the first and the second derivatives of 'Q' and 'h' were computed. These values were substituted in the final equations to determine the tidal and discharge variation at a distance x upstream from the downstream end. The process was repeated and the discharge and tidal variations for other successive segments were calculated.

6. 0. EXPERIMENTS

6. 1. Studies were conduced in the bigger model (Horizontal scale 1/200 : Vertical scale 1/20) to arrive at the optimum section for channelisation of the river so that the tidal effect is felt in the river to the greatest extent possible For arriving at the optimum section the following proposals were tried (Vide Figure-3).

Proposal	At sea mouth		At Napiers Bridge		At Chetput Rly. Bridge	
	Width	Bed level	Width	Bed level	Width	Bed level
I	200 ft	+ 10.00	150 ft	+ 10.00	100 ft	+ 15.00
II	200 ft	+ 10.00	150 ft	+ 12.00	50 ft	+ 16.00
III	200 ft	+ 12.00	150 ft	+ 12.00	75 ft	+ 18.00

6. 2. After arriving at the optimum channelised section, studies were conducted to find out whether any reduction in the width of the north arm of the river will increase the tidal effect. The width of the channelised section of the north arm was kept originally on 150 ft. in the model, this being the same width as that of the channelised section near the Napier Bridge. It was uniformly reduced as that of the other arm.

Hence different widths of the north arm were tried. First the width of the north arm was reduced to 2/3 then to 1/3 and finally it was completely cl osed.

6. 3. With the idea of reducing the accretion of sand carried by the littoral drift near the Cooum mouth, a wide and shallow cut at the mouth was proposed. But to find the effect of the wide-shallow cut in the tidal propogation inside the river and to arrive at the optimum cut to be provided at the detailed model experiments were conducted. Different widths of cuts at the mouth, namely, 200 feet, 300 feet and 400 feet with the bed level at + 14.00 and with a boat basin from the sea mouth upto the Napiers Bridge were tried (Vide Figure-8).

6. 4. Then studies were conducted wi h some training works at the mouth to increase the ebb velocity. It was proposed to construct a tidal weir at the rivir mouth with a central deep leading channel taking off from the weir at an angle. A boat basin of width equal to the length of the Napeiers Bridge was proposed to be formed in between the tidal weir and the Napiers bridge. First, a weir of 320 feet length with the crest level at + 20.50 with a leading channel of 60 feet width with the bed level at + 12.00 was studied in the model. But the flood discharging capacity of the weir with the Central deep channel was found to be slighty lower then the estimated flood of 20,000 cusecs. On the other hand, if a tidal weir of 300 feet length with the crest level at + 20.00 and a central leading channel of 70 feet width, with the bed level at + 12.00 were provided (Vide Figure 11) the estimated flood discharge can be safely passed (vide appendix-1). The idea of providing the deep channel is two-fold, the first one being to minimise the cost of maintaining the deep channel by dredging and the other one to induce high velocity flow during the ebb tide period so that the sand accretion may be scoured as much as posible.

6. 5. All these studies were conducted for a tidal range of four feet produced by the same tidal mechanism,. The water levels at different points along the river during a tidal cycle were observed at regular intervals of time to assess the change in the tidal range.

6. 6. While conducting the experiments in the first two trial sections, velocity or discharge could not be directly measured because of the following reasons :—

(1) The model velocity was very low and could not be measured with the instruments available.

(2) The velocity and discharge were changing instantaneously, varying according to the tidal cycle.

6. 7. But an attempt was made to measure these instantaneous velocities by means of surface floats allowed to float for a very short interval of time. It can easily be seen from the tidal curve that the maximum velocity and discharge during a tidal cycle will occur when the sea level is in between the flood tide and ebb tide levels i. e. at mean sea level. This is due to the fact that the slope of the tidal curve is maximum only at this instant, the velocity and discharge being proportional to this slope dh/dt (time rate of variation of tidal head). Hence, this instant was shosen in the model for the observation of velocity. Surface floats were allowed to flot at this instant at various points along the river and the distance travelled in a period of 20 second was noted. This will approximately give the maximum velocity since the period for which the float was allowed to travel i. e. 20 second is very small compared to the total tidal period of 16 minutes 40 seconds in the model. For obtaining the average velocity in the section, a correction factor of 0.9 was applied to the surface velocities. Knowing the area of cross-

section at a particular place where the velocity was observed, the discharge was calculated.

7. 0. RESULTS AND DISCUSSIONS

7. 1. Practically there is no change in the tidal range observed at different points along the river for all the three proposed sections as seen from Figure-3. Figure-4 shows the maximum flood tide and ebb tide levels as computed by the method of iteration described in paras 5.1 to 5.5 of this report. The variation in the tidal range is very small and varies only in the second decimal of the prototype value. This small change could not be determined accurately in the model. The tidal range was observed to be the same throughout in the model. Therefore, the aspect could not be made use of for compuring and arriving at the optimum channelised section.

7. 2. The discharges at different points along the river due to the tidal action were computed by the iterative method and furnished in Figure-5. For the proposed section I the initial discharge at the sea mouth is found to be more while it is the less in the case of proposed section III and still less in the case of proposed section II. It is interesting to note that the discharge curve for the proposed section I is steep while that of the section III is less steep and that of section II is still less steep. It can be inforred that in the case of proposed section I the discharge decreases rapidly as the tide travels upstream and in the case of proposed section III the decrease is less rapid and in the case of section II the decreased still less rapid. As soon from the figure the difference between the discharges for the proposals II and III is not much. If proposal III is adopted it involbes minimum earthwork in channelising the river as the bed level goes up to + 18.00 in the upper reaches of the river. Hence the proposal III, namely, 200 feet width at the sea mouth, 150 feet width at Napiers Bridge at a distance of 1450 feet from the sea mouth and 75 feet width at the Chetput Railway Bridge at a distance of nearly 30,000 feet from the sea mouth, with the bed levels at + 12.00 from the sea mouth upto the Napiers Bridge and with uniform rise upto 18.00 at Chetput Railway Bridge, was selected as the optimum channelised section for the river Cooum. The channelised sections with the berms have been checked for their adequacy to carry the normal flood discharge were found to be ufficient.

7. 3. For the proposal III, the instataneous velocities at different places along the river were observed by surface float method as already explained and discharges along the river were calculated. There is clese conformity but not exact agreement between the discharge curve obtained by the model studies and tne computed discharge curve-vide Figure-6. This disparity is perhaps due to the fact that in the theoretical computations the presence of North arm is ignored.

7 4. The results of the experiments conducted in the model for different widths of the north arm show that as the width of the north arm is reduced the velocity and hence the discharge due to tidal action are increased along the river but the increase is very small. For getting the maximum benefit of the tidal action in the river Cooum, the optimum condition of the north arm would be its complete closure as seen from Figure-7, Although this may be desirable from the point of view of tidal action in the river, yet it is better not to close or unduly minimise the width of the north arm from the point of view of flood in the river. This north arm has been found to play a vital role as a flood carrier and it is this consideration that should weigh most while taking decision on either complete closure or reduction of the width of the north arm.

So, a bed width of 80 feet at Napiers Bridge reducing gradually to 60 feet at the point of bifurcation above the Island grounds with uniform bed slope was proposed for the section of the north arm to satisfy the twin consideration of tidal action and flood.

7. 5. The velocities obtained at different places along the river for the different proposals of wide-shallow cut tested with the boat basin in between the sea mouth and the Napiers Bridge (vide Figure-8) are furnished in Figure-9. In the same figure is shown the velocity curve for a width of 200 feet keeping the bed level at + 12.00. The maximum discharges at a different place were calculated from the velocities observed. Figure-10 gives the maximum discharges as obtained for different proposals tried. The discharge curve for a narrow cut of 200 feet and for a deeper level of + 12.00 is also shown in the same figure.

7. 6. It is seen from the results that as the width of the cut at the mouth is increased the velocity due to tidal action is also increased. There is considerable increase in the velocity for a width of 300 feet over that for a width of 200 feet. Any further wider cut, say 400 feet, has not much increasing effect on velocity. The same is the case with the discharge considering only the three different widths at the mouth with the bed level at + 14.00. Taking into consideration the variation of discharge for the narrow-deep cut (i.e. 200 feet at + 12.00) eventhough the discharge is higher at the initial reaches then that for the wide-shallow cut it goes on reducing and is less at the upper reaches of the river. So, as seen from the figures No. 9 and 10 for the wide-shallow cut the velocity is greater throughout the river, but the discharge is less at the reaches and more at the upper reaches of the river as compared with the velocity and discharge for the narrow deep cut. As regards the choice between the narrow deep cut and wide-shallow cut at the mouth, though there is some decrease in the discharge at the initial reaches, since there is considerable increase in the velocity throughout the river in the case of the wide-shallow cut, the flushing action will be more effective for the wide-shallow cut at the mouth and hence it is preferable. From economy point of view maintaining a cut of 300 feet width will be cheaper then that of 400 feet width. Therefore a cut of 300 feet width at the sea mouth with the bed level at + 14.00 was proposed to increase the effect of tidal action.

7. 7. Then studies were conducted with the training work at the mouth as explained already in para 6. 4. A tidal weir of 320 feet length was constructed with the crest level at + 20.50 at a distance of 500 feet downstream of the Napiers Bridge. A central deep channel of 60 feet width with the bed level at 12.00 was provided at an inclination of 30° with the direction of normal to the beach.[5] The tidal were was located at a distance of 500 feet from the Napiers Bridge. This is only a tentative location and detailed studies will be conducted in a comprehensive model to be constructed, to arrive at the exact location of the tidal weir. Then the second proposal namely a tidal weir of 300 feet length with the crest level at 20.00 and the central leading channel of 70 feet width with the bed level at + 12.00 at the same inclination of 30° was incorporated in the model. The details of the training works at the mouth are given schematically in Figure-11. Studies were conducted for both these proposals. The velocities at different places were observed and furnished in Figure-12. In the same figure is shown the velocity curve for a width of 300 feet at the mouth, keeping the bed level at + 41.00. From the figure it can be seen that the velocity in the initial reach i. e. in the narrow deep channel is relatively high, but in the river portion it falls down owing to the increased water way provided. There is not much of difference between the velocities obtained for the weir proposal is slightly higher than that obtained for 300 feet plain width at the mouth with the bed level at + 14.00

7. 8. The maximum discharges into the river at different places were calculated. Figure 13 gives the discharge curves obtained for the two proposals with the tidal weir and the leading channel and for the proposal with a cut of 300 feet at the mouth with the bed level at + 14.00. As seen from the figure the discharge for the first proposal with the tidal weir 320 feet long and the leading channel of 60 feet side is somewhat less in the initial reaches and afterwards slightly higher than that for the shallow-wide cut of 300 feet at + 14.00. But in the case of the tidal weir 300 feet long and leading channel 70 feet wide, the discharge is slightly higher everywhere along the river eventhough the increase is small. So far as the discharge are concerned one is not very considerably different from the other.

7. 9. For both the proposals with the tidal weir and the inclined deep leading channel the discharge is slightly higher than that for the one with 300 feet cut at the mouth keeping bed level at + 14.00. Since the velocity for the proposal with the tidal weir is relatively high especially in the initial reaches i. e. in the leading channel portion, the sand accretion may be scoured to some extent during the ebb tide and due to increased velocity and discharge throughout the river the flusing action will be better. But considering the two proposals with the tidal weir and the leading channel, the second proposal (a tidal weir of 300 feet long and a deep channel of 70 feet wide) seems to be better then the first one (a tidal weir of 320 feet long and a deep channel of 60 feet wide) from the point of view of flood discharging capacity. Hence, for the training and control works at the mouth, the second proposal with the tidal weir of 300 feet length and the leading channel of 70 feet width has been recommended.

7. 10. Regarding the distance upto which the sea water travels into the river, a float left in the model at the Napiers Bridge at beginning of the flood tide travelled upto a distance corresponding to about 9000 feet up the river for a tidal range corresponding to four feet. Taking in to account the model limitations it may be said that the replacement by sea water may take place for about two miles from the sea mouth for each tidal cycle.

8. 1. CONCLUSIONS

Studies were conducted for a number of proposals to arrive at an optimum section for the channelised section for the river Cooum, with suitable training work at the mouth, to increase the tidal influx into the river and the ebb velocity so that the river gets flushed to the greatest extent possible. Finally a sectiou for the river Cooum, as detailed below; was finalised and recommended for adoption at the site giving due consideration to the feasibility of the proposals recommended.

(1) A tidal weir 300 feet long with the crest level at + 20.00 constructed at a distance of 500 feet downstream of the Napiers Bridge and a central deep channel 70 feet wide with the bed level at + 12.00 and inclined at an angle of 30° to the normal to the weir. The exact location of the weir and the angle of inclination of the leading channel will be decided after further model studies as mentioned separately.

(2) A bed width of 150 feet at Napiers Bridge and 75 feet at Chetput Railway Bridge with the bed level's at + 14.00 at Napiers Bridge and + 18.00 at Chetput Railway Bridge with unifrom rise.

(3) A bed width of 80 feet for the north arm near the Napiers Bridge and 60 feet at the other end with uniform rise in bed level.

9. 0. LIMITATIONS OF THE MODEL STUDIES

The results arrived at by conducting studies in the model are only qualitative and not quantitative for the following reasons:

(1) The waves and the littoral drift were not reproduced in the present model but only tides were reproduced.

(2) The South Buckingham Canal and the North Buckingham Canal were not incorporated in the model. This could not be done for the reason that space will not be available for incorporating these canals; further, the incorporation of these canals are not essential as the studies conducted are only comparative studies for chosing the optimum channelised sections.

(3) The velocities observed for the tidal flow were not measured by precise instruments but only observed approximately by the surface float method and again the observed velocity was multiplied by an arbitrary correction factor of 0.9 to get the average velocity.

10. 0. SCOPE FOR FURTHER STUDIES

A comprehensive model will be put up simulating the waves and the littoral drift to study the exact location of the tidal weir and the best alignment of the leading channel so as to increase the velocity of flow during the ebb tide period.

The experiments in this model will mostly be conducted to study the various aspects of the sand bar formation at the mouth.

11. 0. REFERENCES

(1) J. Allen. Scale models in Hydraulics Engineering-Longmans, Green & Co., London, New York, Taronto.

(2) Flushing of the River Cooum by Tidal Action of the Sea-Miscellaneous Report No. 9/59. I. R. S.

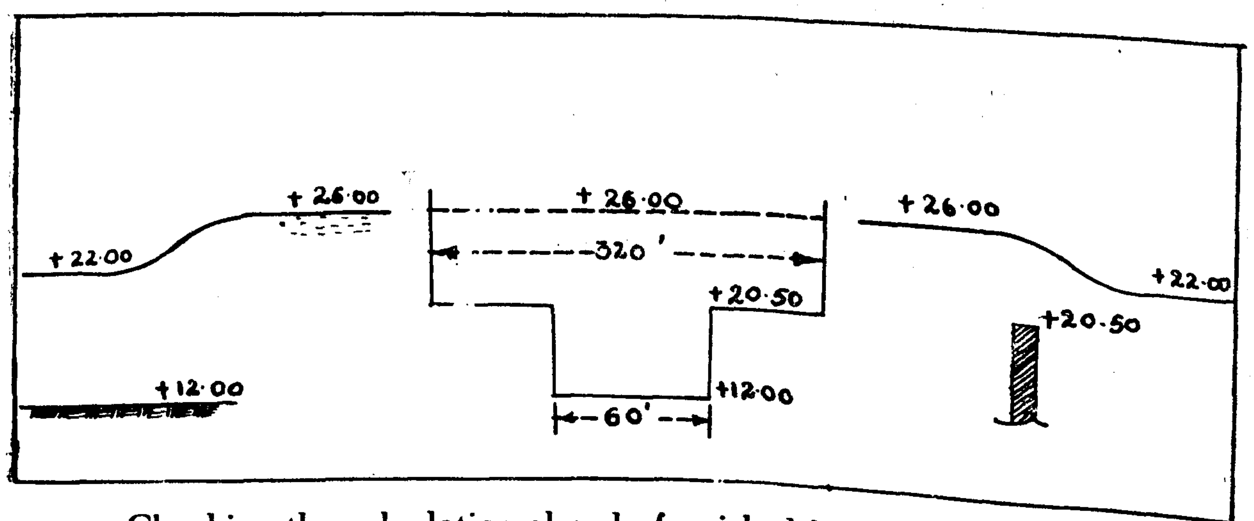
(3) D. V. Joglekar, Principle of Hydraulic model studies and their limitations C. B. I. & P. Journal Vol. 16, No. 1.

(4) G. Sinha, Mathematical computation of tidal propogation in Estuarine Rivers C. B. I. & P. Journal-Volume 15-No. 1.

(5) Flushing the River Cooum with tidal water-Field and Laboratory experiments - Annual Report for 1955 - Irriga-Research Station, Poondi.

APPENDIX

CALCULATION OF AFFLUX FOR THE MAXIMUM FLOOD DISCHARGE 20,000 CUSECS



Checking the calculation already furnished for the above section

Calculation for the deep open portion, assuming, an afflux of 4 feet. Treating this as a combined weir flow and orifice flow and using the formula $Q = C\,1\,H^{3/2}$ and $Q = 6\,A\sqrt{H}$ respectively.

$Q_1 = (3.09 \times 60 \times 4^{3/2}) + 6 \times 10 \times 60 \sqrt{4}$

$= 1480 + 7200$

$= \underline{8680 \text{ cusecs.}}$

Over the weir portion

$Q_2 = 3.09 \times 260 \times 4^{3/2} + 6 \times 260 \times 1.5 \times \sqrt{4}$

$= 6420 + 4680$

$= 11100$ cusecs.

Total discharge $= Q_1 + Q_2$

$= 8680 + 11100$

$= 19780$ cusecs

= say 20,000 cusecs nearly.

But it will be more accurate if we treat this as a submerged flow over a sharp crested weir.

Then for the submerged flow, assuming this a sharp creat**e**d weir

$Q = C\,L\,H^{3/2} \times f$

where f is the reduction factor which will vary with the submergence ratio

and c = the co-efficient of discharge

$= 2/3 \quad Cd \quad \sqrt{2g}$

= 3.24 assuming a value of 0.605 for Cd

Thus calculating the discharge passing over the weir above the crest level for the entire length of 320 feet.

$Q_1 = 3.24 \times 320 \times 5.5^{3/2} \times f.$

submergence ratio $= \frac{1.5}{5.5} = 0.273$

Reduction factor for a submergence ratio of 0.273 is 0.95*

$Q_1 = 3.24 \times 320 \times 5.5^{3/2} \times 0.95$

$= \underline{12700 \text{ cusecs}}$

For the central deep portion, treating this as an orifice flow

$Q_2 = 6\,A\,\sqrt{H}$

$= 6 \times 8.5 \times 60 \times \sqrt{4}$

$= 6120$ cusecs.

Total discharge $= 12700 + 6120$

$= \underline{18820 \text{ cusecs}}$

But the estimated discharge is 20,000 cusecs.

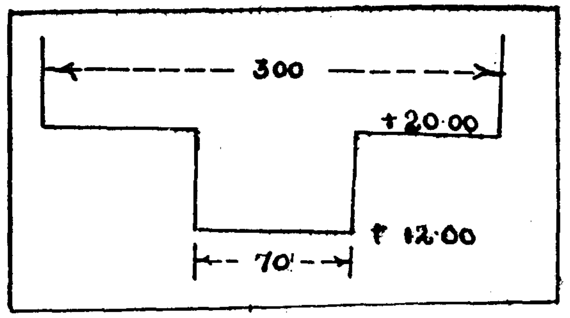
If some alterations are made in the weir section as given below the estimated discharge of 20,000 cusecs can be passed over the weir.

Keep the weir length as 300 feet instead of 320 feet. Increase the width of the deep channel at 70 feet as against 60 feet.

and keep the weir crest at 20,00 instead of 20.5 as proposed already.

Now calculating the discharge for this modified weir section in the same method as before

* International Commission on Irrigation and Drainage Annual Bulletin 1958—Page 50 Open Channel Water Measurement with the broad-crested weir by C. D. Smith.



Submergence ratio = $\frac{2}{6}$ = 0.33

Reduction factor for a submergence ratio of 0.33 is 0.93

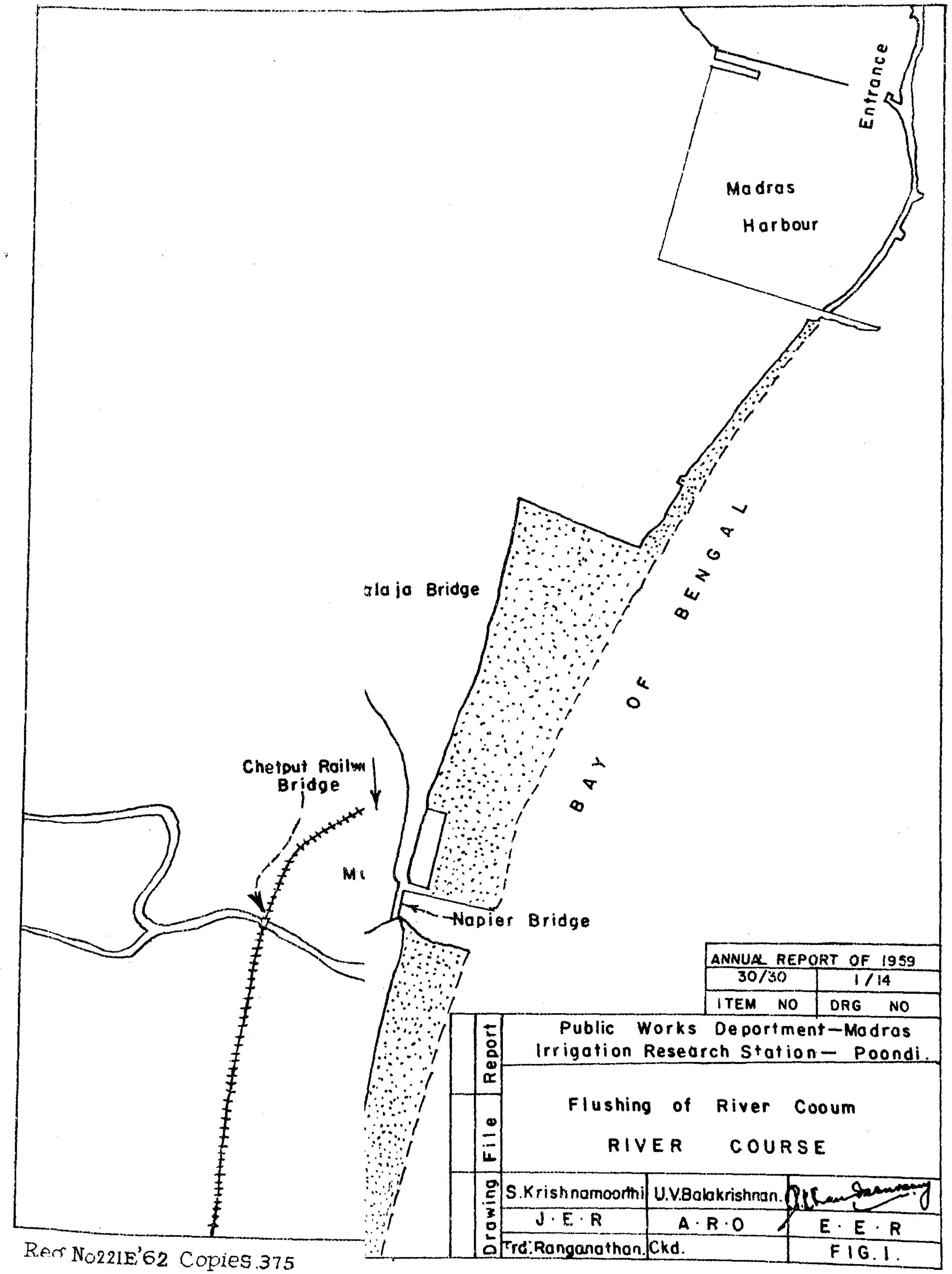
Q_1 = 3.24 x 300 x $6^{2/3}$ x 0.93
= 13250 cusecs.

Q_2 = 6 x 8 x 70 x $\sqrt{4}$
= 6720 cusecs.

Total discharge = 13250 + 6720
= 19,970 cusecs
say 20,000 cusecs.

and hence this section can be adopted.

.... -:O:- ....



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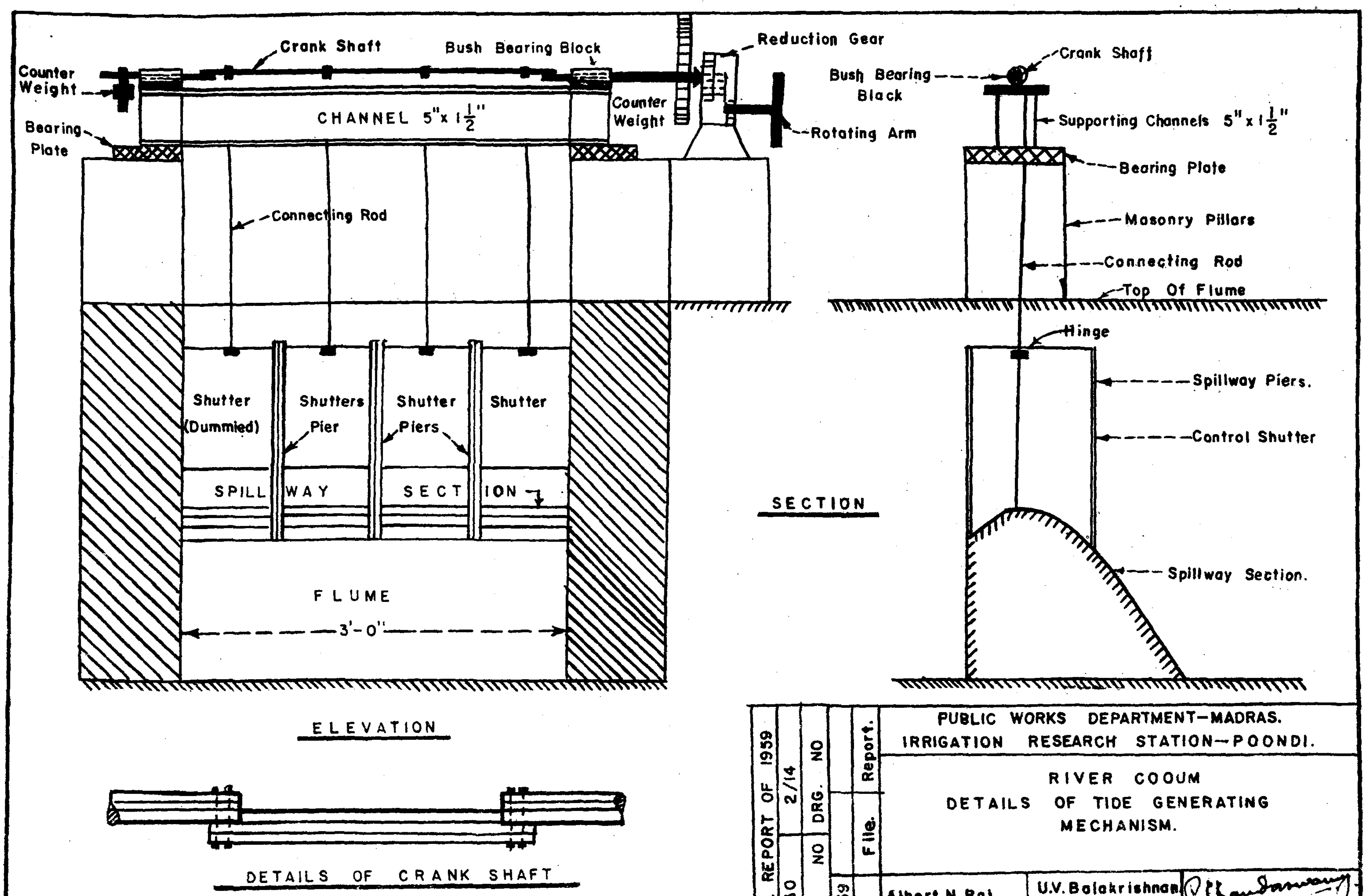

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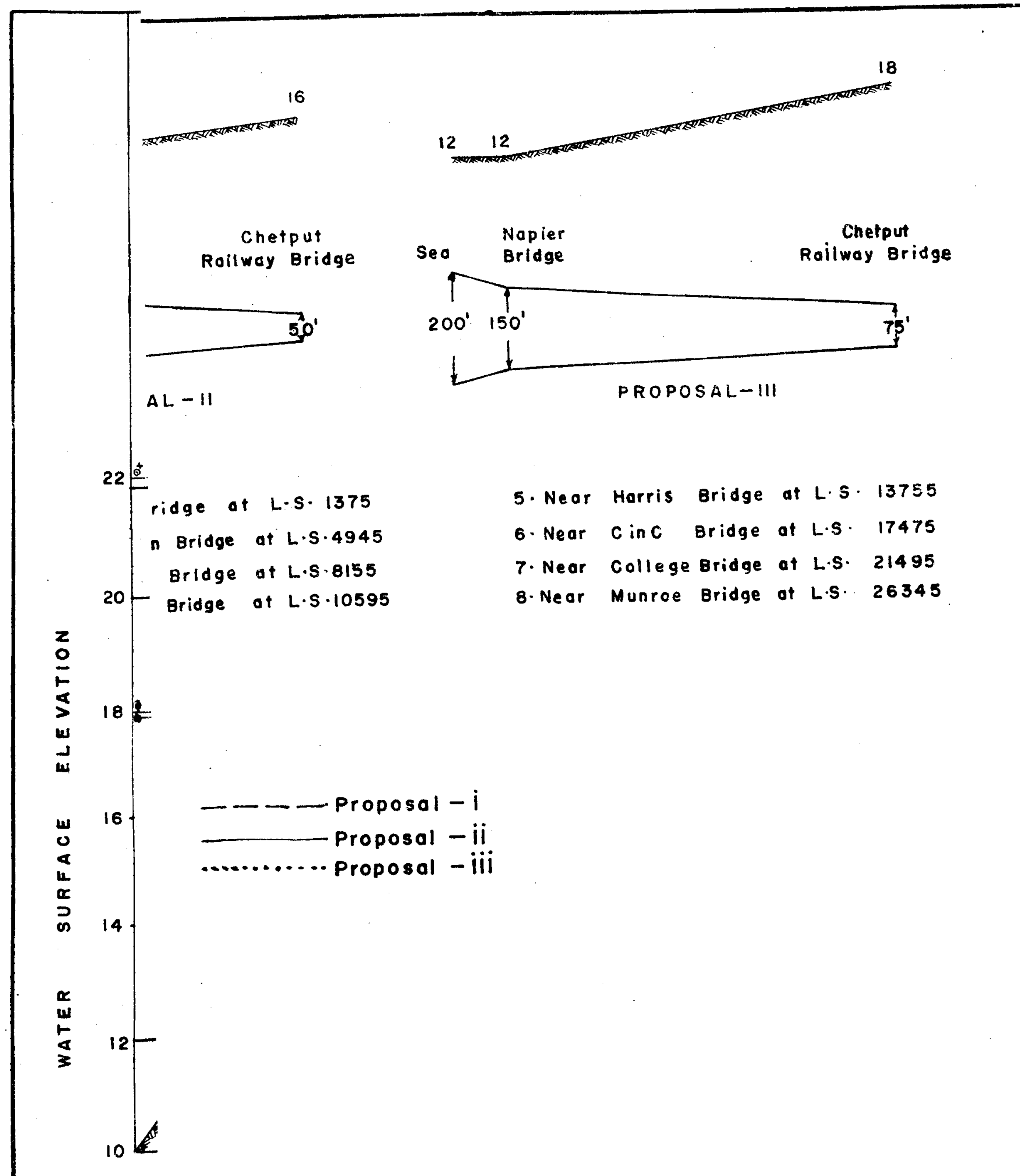

ANNUAL REPORT 30/30

ITEM NO

Public Works Department — Madras
Irrigation Research Station — Poondi

Report

File

Max Ebb and Flood Tide levels in River Cooum for Proposed Section I II & III

Drawing 259/59	S. Krishnan	V. B. Joshi	K. Kaveswamy
	J·E·R	A·R·O	E·E·R
	Trd: V.V.	Ckd:	FIG ... 3

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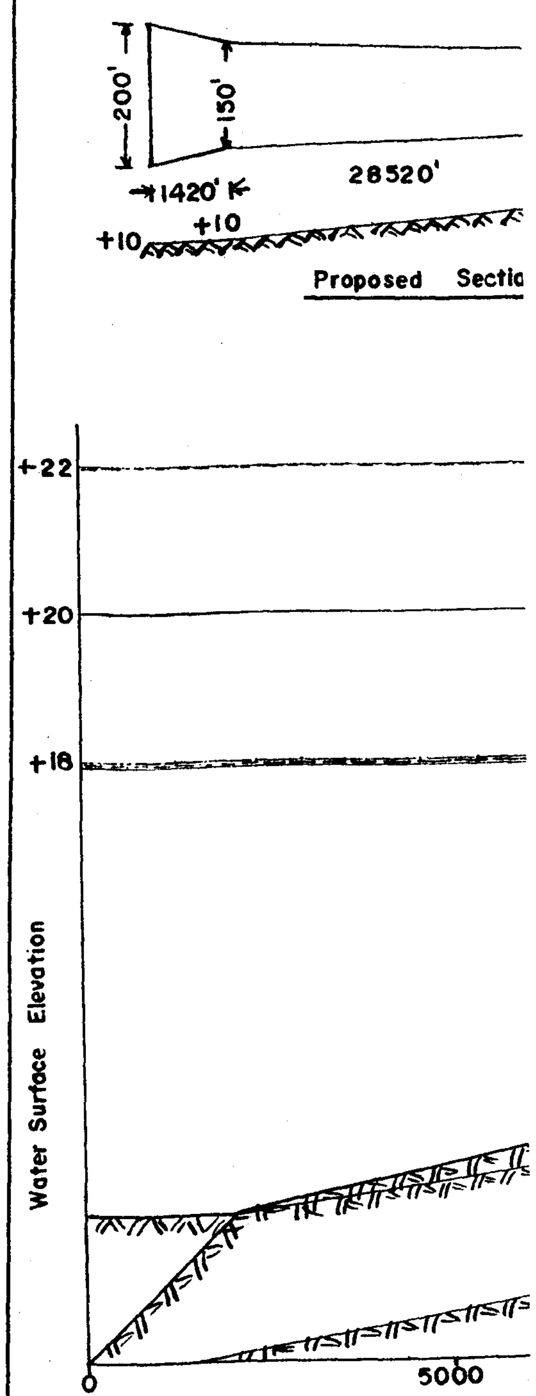

Public Works Department — Madras.
Irrigation Research Station — Poondi.

Max. Ebb and Flood Tide levels
River Cooum for
Proposed Section I II & III
(Computation)

Report / File / Drawing 260/59			
	B. Thomas.	U.V. [illegible]	[illegible]
	J·R·A	A·R·O	E·E·R
	Trd T.K.S	Ckd.	FIG ···· 4

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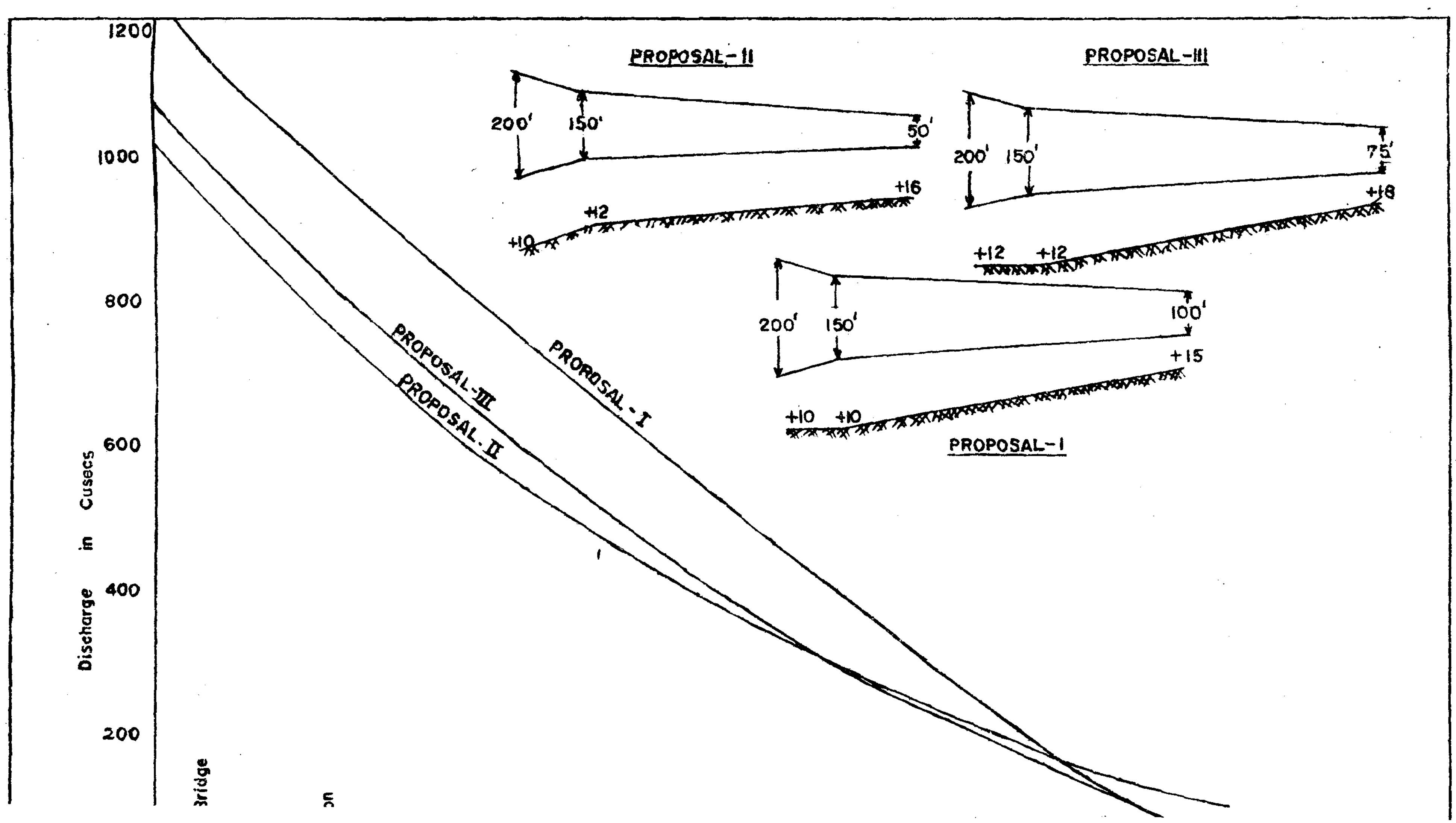

PROPOSAL-II
200'
150'
50'
+16
+12
+10
PROPOSAL-III
200'
150'
75'
+18
+12
+12
200'
150'
100'
+15
+10
+10
PROPOSAL-I
PROROSAL-I
PROPOSAL-III
PROPOSAL. II
1200
1000
800
600
400
200
Discharge in Cusecs

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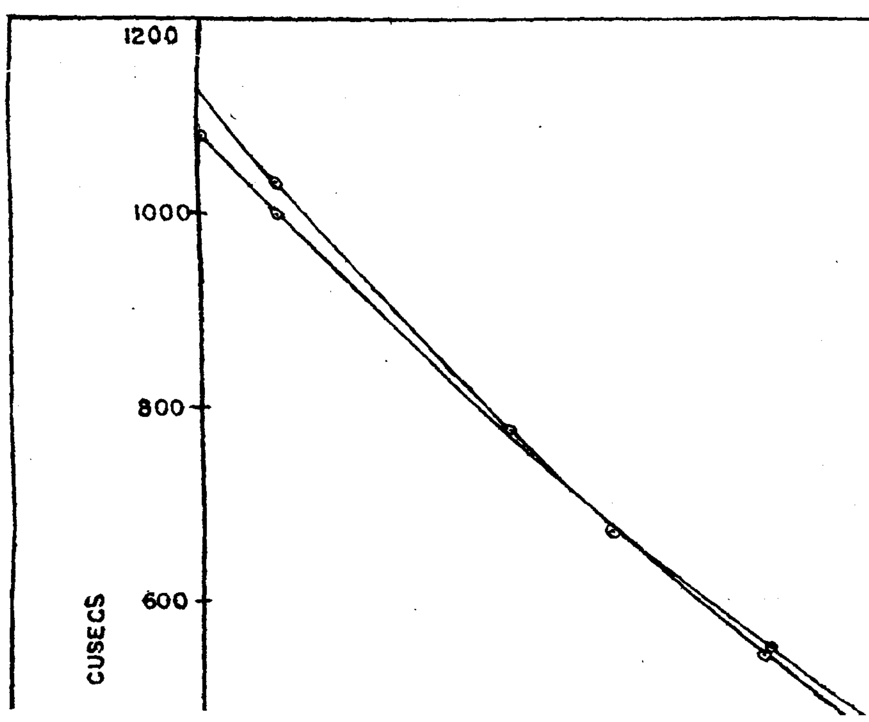
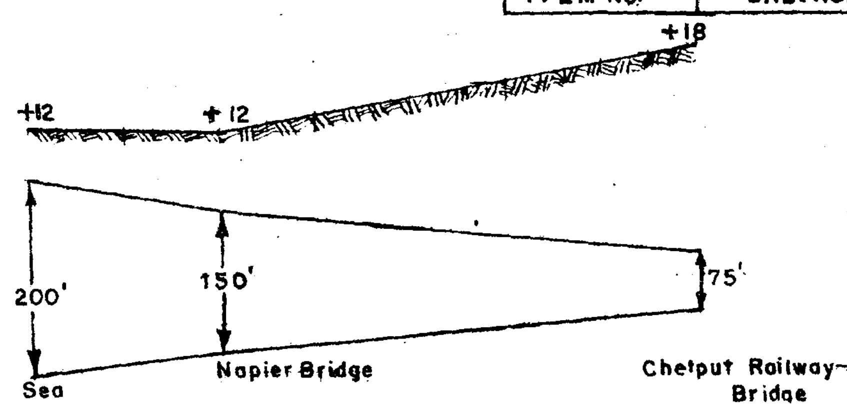
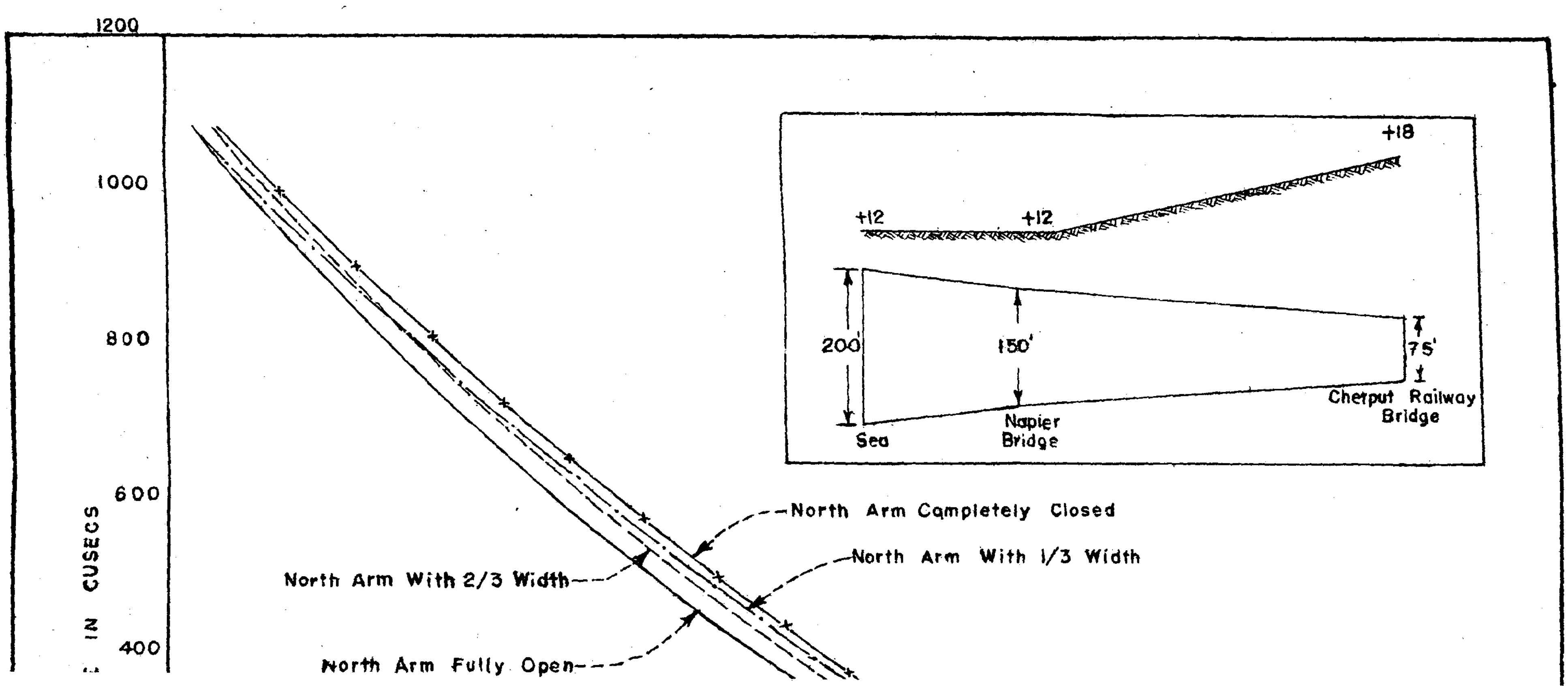
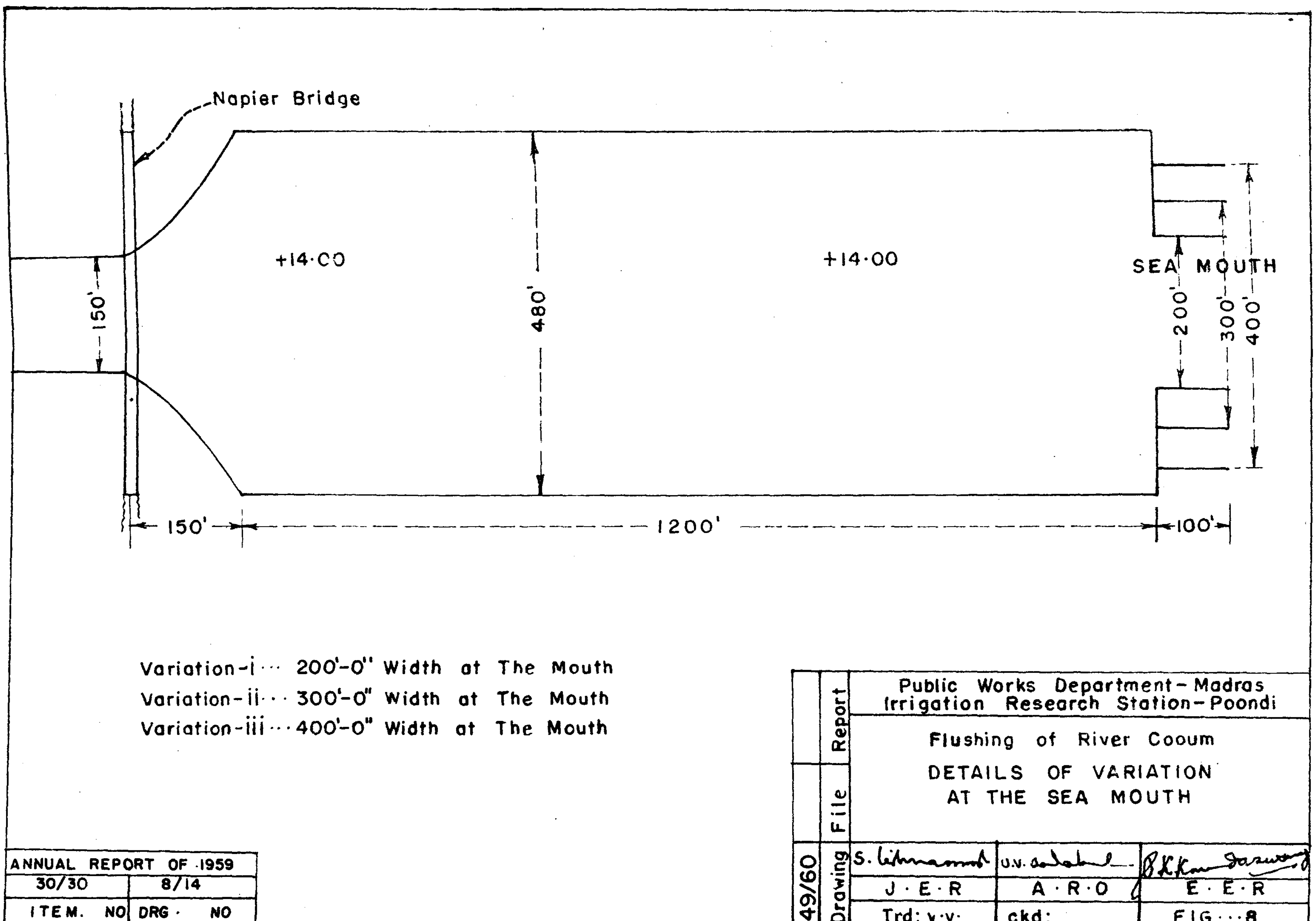





1200
1000
800
600
400
IN CUSECS
+12
+12
+18
200'
150'
75'
Sea
Napier Bridge
Chetput Railway Bridge
North Arm Completely Closed
North Arm With 1/3 Width
North Arm With 2/3 Width
North Arm Fully Open



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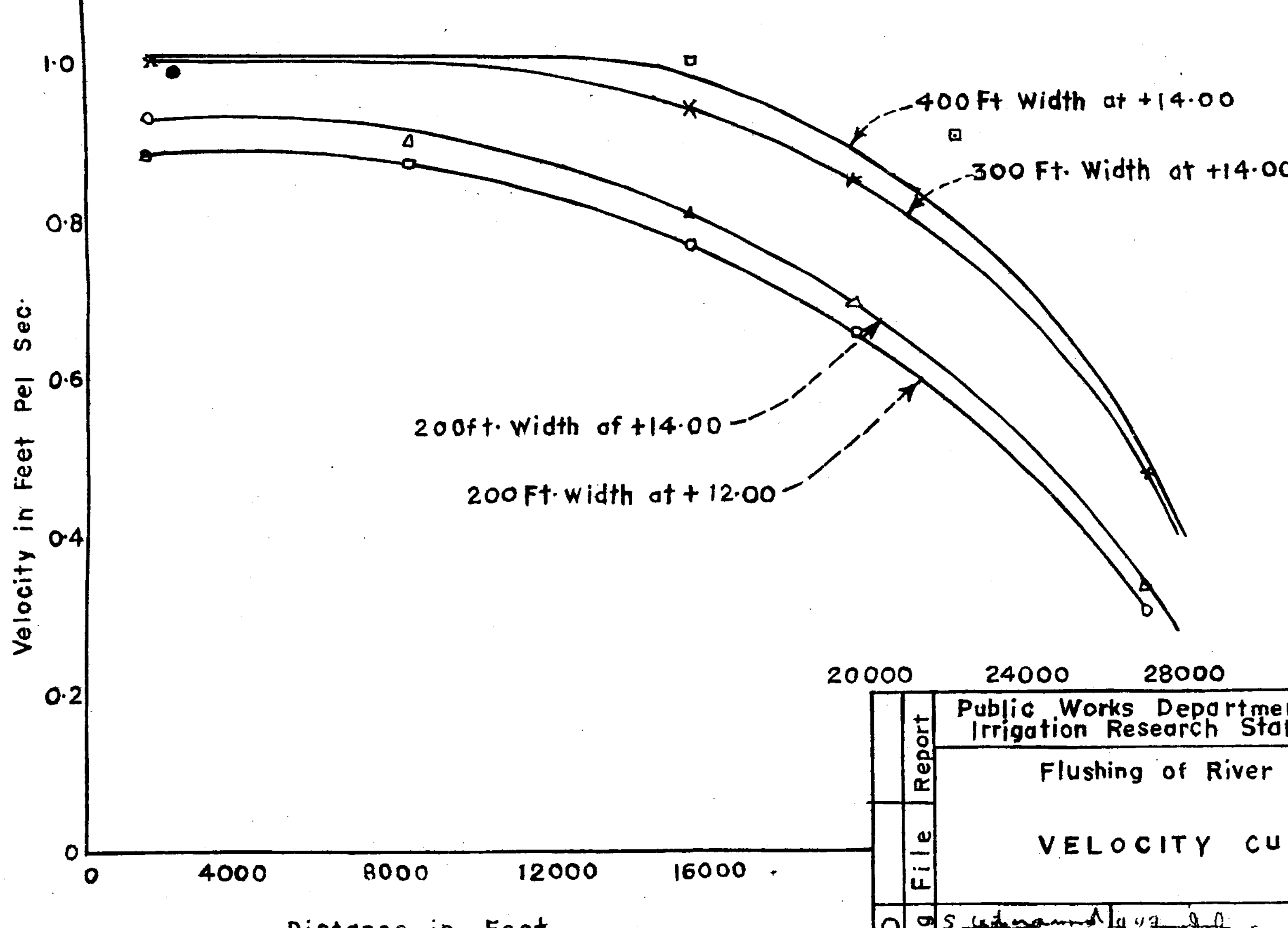

Public Works Department — Madras
Irrigation Research Station — Poondi

Flushing of River Cooum

VELOCITY CURVES

Report			
File			
Drawing 50/60	J·E·R	A·R·O	E·E·R
	Trd. C·S·K·	ckd:	FIG.·:9

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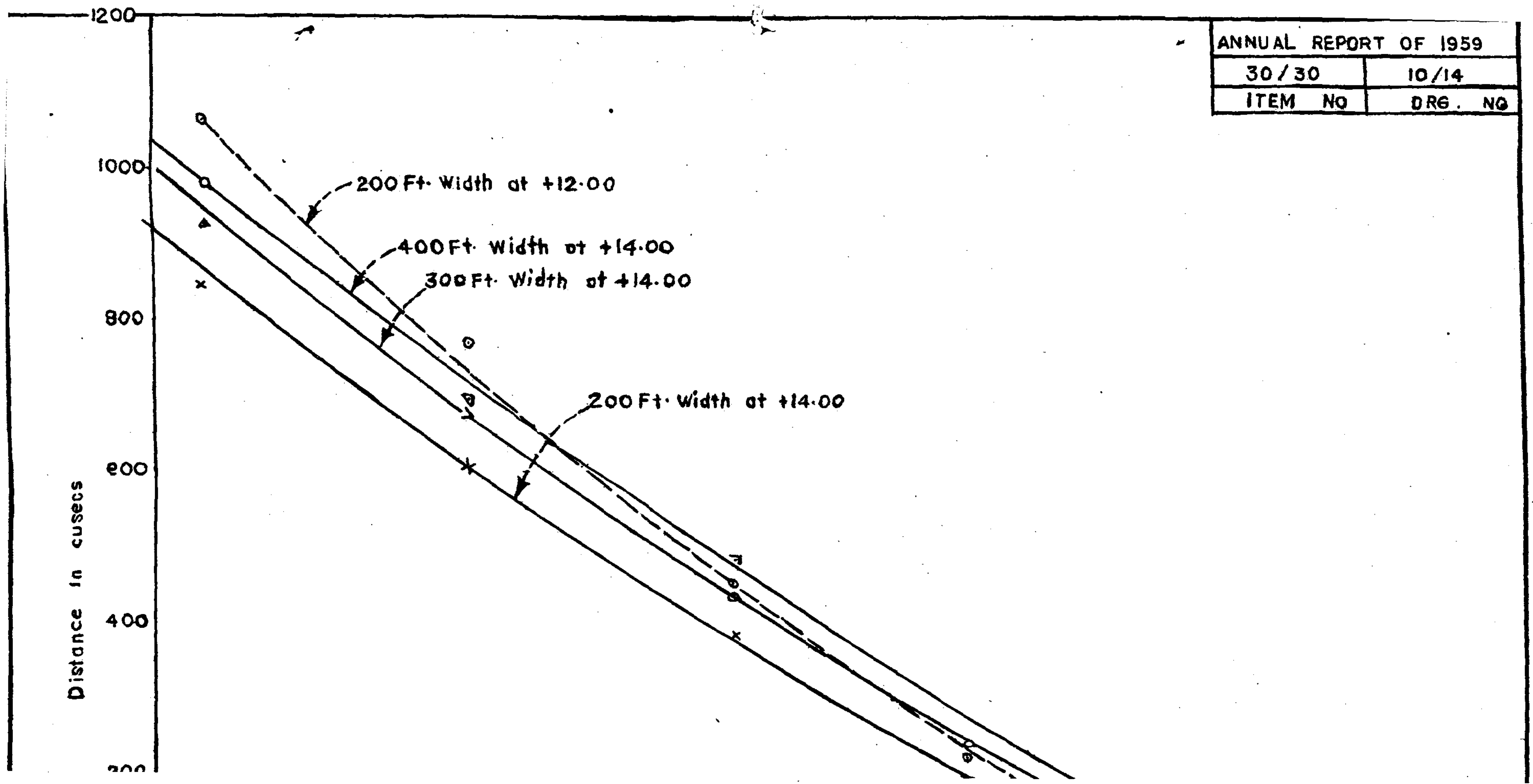

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roposal-ii· With the Tidal Weir And
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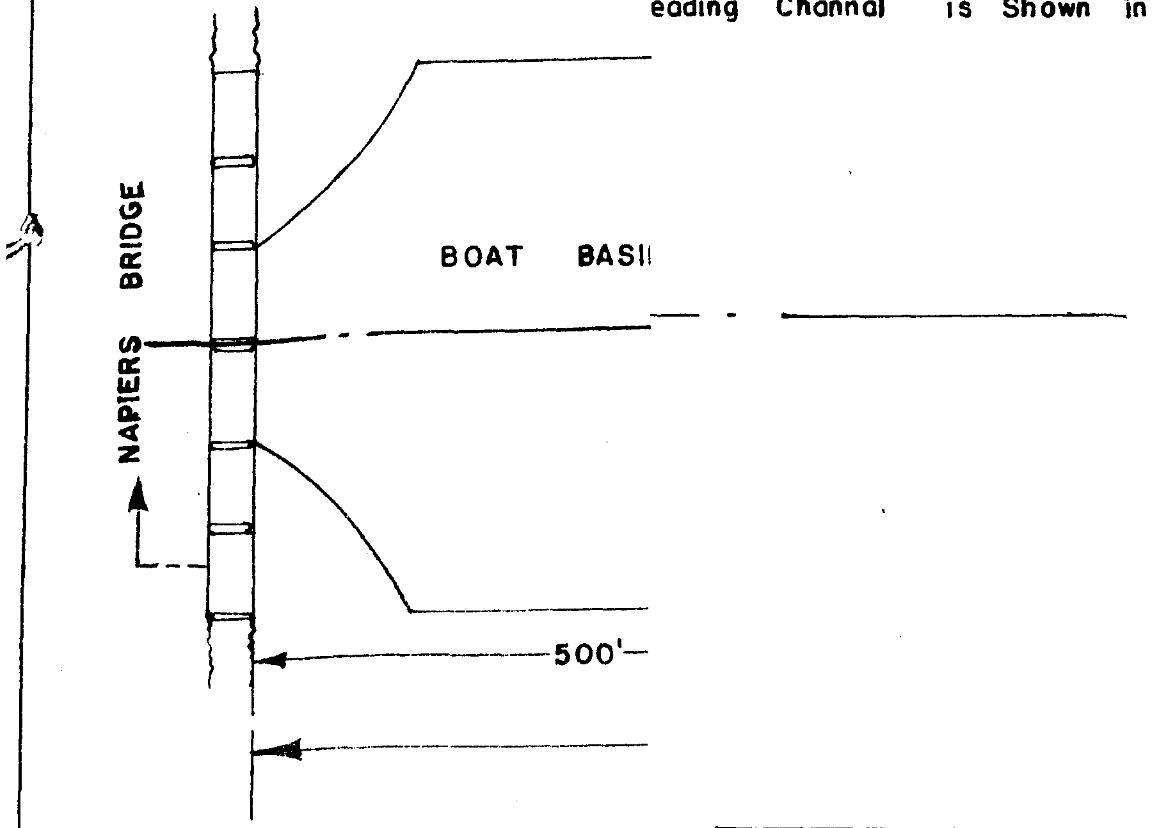

Report.	Public Woks Department——Madras. Irrigation Research Station——Poondi.		
File	Flushing of River Cooum LOCATION OF TIDAL WEIR AND LEADING CHANNEL		
Drawing 99/60	S. [signature]	[signature]	[signature]
	J · E · R	A · R · O	E · E · R ·
	Trd: C · S · K	Ckd:	Fig. No.... 11.

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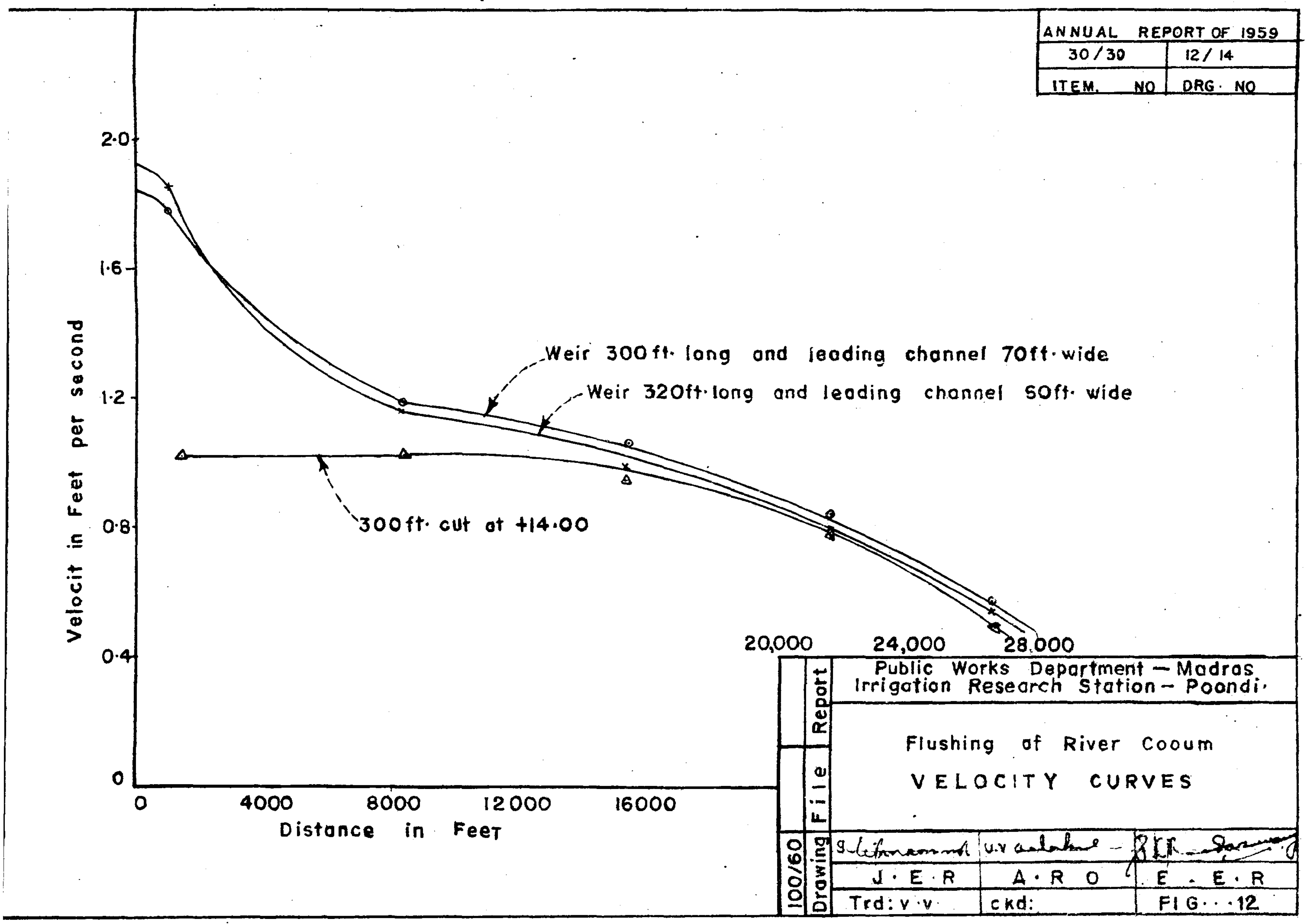

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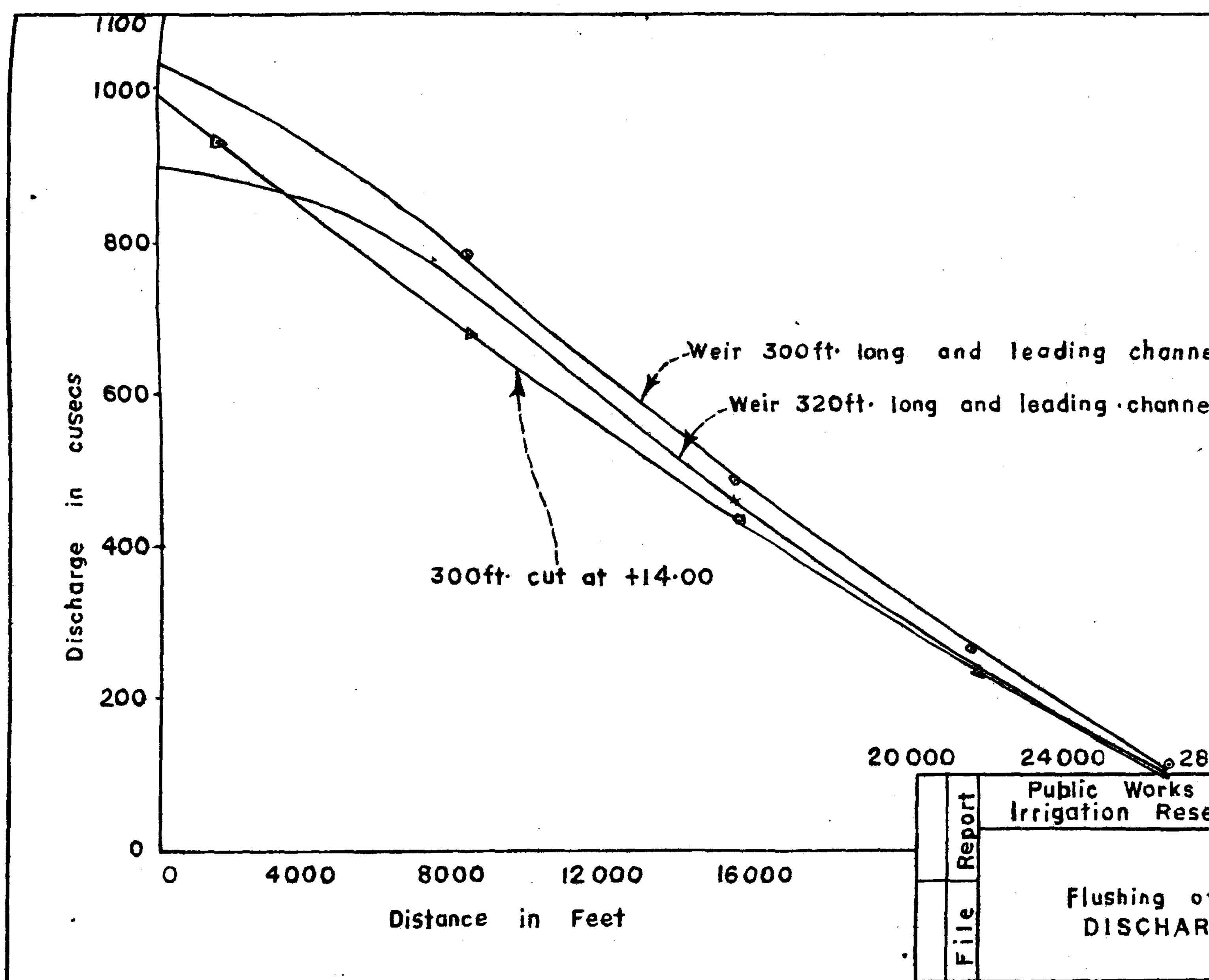

Report	Public Works Department — Madras Irrigation Research Station — Poondi		
File	Flushing of River Cooum DISCHARGE CURVES		
Drawing 101/60	J. E. R	A. R. O	E. E. R
	Trd: v. v.	ckd:	FIG...13

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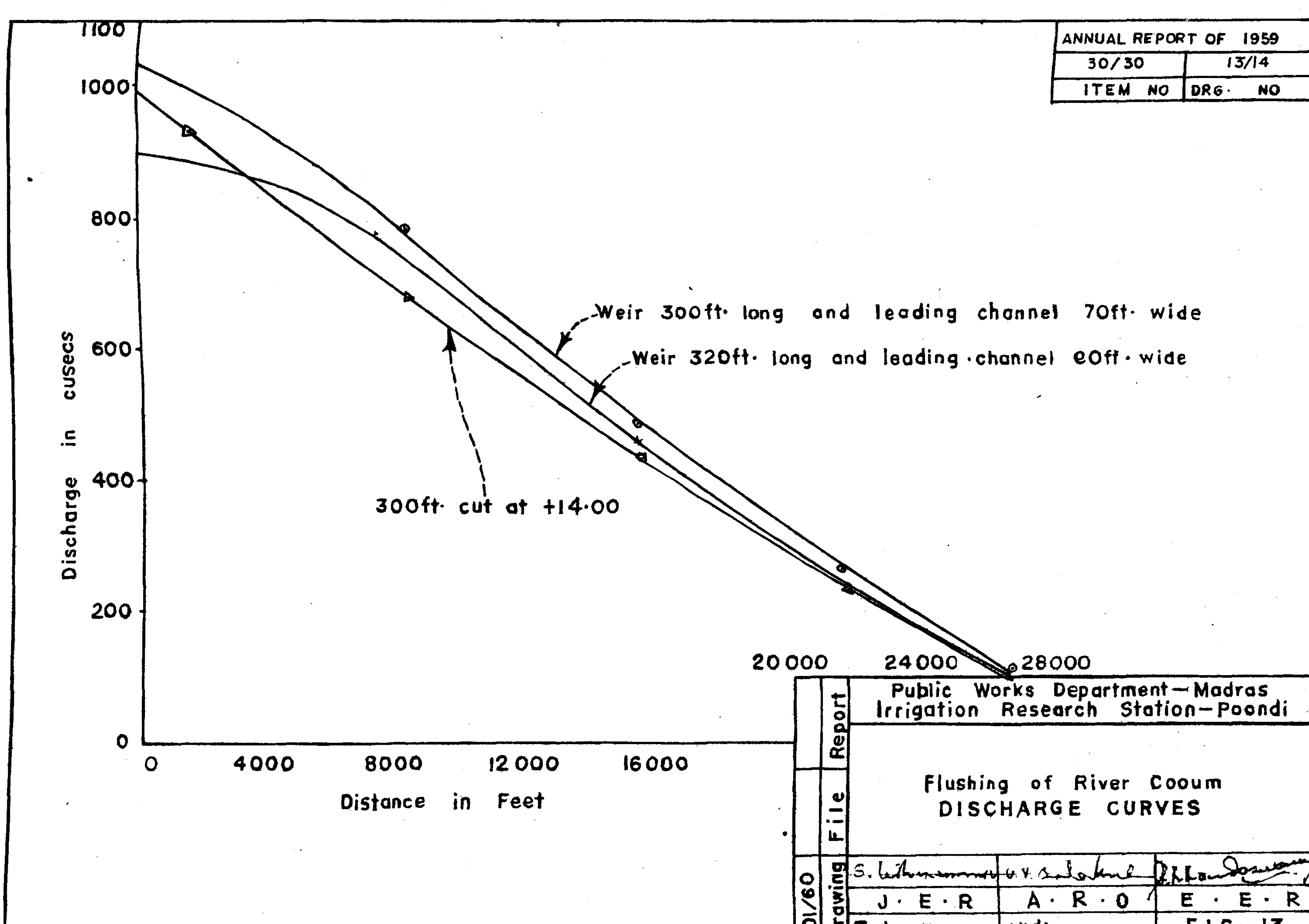

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