MADRAS IRRIGATION RESEARCH PUBLICATION No. 15

PUBLIC WORKS DEPARTMENT

IRRIGATION RESEARCH STATION
POONDI MADRAS

ANNUAL REPORT FOR 1958

VOLUME-II

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N.B.—* Indicates item of priority No. 1 for discussion.

N.B.—* Indicates item of priority No. 1 for discussion.

† Indicates item of priority No. 2 for discussion.

VOLUME II.

PERIYAR MAIN CANAL—PROPOSAL TO EXCLUDE BED SEDIMENT WITH MINIMUM WASTAGE OF WATER.

(C.B.I. & P. No. B-2.)

Synopsis :

Periyar Main Canal which is the main feeder canal of erstwhile famous Periyar Project picks up its supply from River Vaigai at the site of Peranai regulator. Just like the other canals under similar conditions it had drawn heavy sediment load. A novel proposal to convert the left extreme vent of Peranai Regulator into a scouring vent was put forth by the Superintending Engineer in charge. The proposal was theoretically analysed and split up into various component parts. Optimum dimensions were arrived at for these devices with the aid of models. The efficiency of the final shape of the proposal with a wastage of water of 5, 10, 15 and 20 per cent of the full supply discharge of the canal is considered sufficient to justify the proposal. Detailed recommendations regarding each component part are also given.

1. Introduction :

1.1. Periyar Main Canal is the main feeder canal of the unique Periyar Irrigation Scheme in which the run off of Periyar basin was diverted to Vaigai basin through a mile long tunnel from the Periyar lake formed by a dam. The Periyar Main Canal picks up its supply from River Vaigai at the site of Peranai Regulator constructed across it lower down with the sole purpose of diverting the flow into the canal (vide Figure 1). Naturally supplies just sufficient for irrigation demand were let down and the canal had drawn large quantity of coarse bed load. Lately the problem of drawing sediment-free water into the canal was considered and a novel proposal to convert the left extreme vent of Peranai Regulator into a scouring vent was put forth by the Superintending Engieneer in charge.

1.2. *The Proposal.*—The sill of the Peranai Regulator is 3 feet lower than the sill of the head regulator of Periyar Main Canal. It was contended that this alone was not sufficient and two more factors will have to be realised, namely straight entry into the canal and curved entry into the scouring vent with high velocity of flow through the scouring vent. Hence it was proposed to excavate a channel axial to the head regulator and turn the left extreme vent of Peranai Regulator so as to have its alignment at about right angles to the Periyar Main Canal Regulator. It was also proposed to convert the 40′ wide vent into four vents 8 feet wide each capable of being operated individually. A few groynes were also proposed to divert the flow into the straight channel.

2. Analysis of the Proposal :

2.1. The principle on which the device works is that of an offtake from a straight channel. An interesting fact is mentioned in an article in the report on fourth meeting of I.A.H.R. that a canal offtaking at right angles from a parent canal and drawing only 1/6 of the total discharge allows ten times as much silt as the main canal. Schoklitch has given from model experiment the relationship between the division of bed load and discharge for an angle of 30°. He has also shown that most of the bed material from the parent channel entered the offtake when the angle was changed from 30°—150°, the discharge in both channels after bifurcation being the same. The relationships are reproduced in Figure 2 from which it can be seen that the offtaking channel drew as much as 50 per cent of the bed load when discharge shared is only 20%. The figure indicates in general terms that an offtake from a straight channel will draw silt in much larger proportion to its discharge.

2.2. For the abovementioned principle to be applied to the present case, the proposed scour vent should act as a distributory head, and it will eventually appear that a channelised flow into Periyar Main Canal will make the functioning of the vent much better. But a channel dug in the river bed is likely to get silted up. Whenever flood occurs and may necessitate canal excavation every year and hence a proposal which will have the flow to erode its own channel so as to give an axial approach to the head sluice, had to be sought for. Again the inclination of the scour vent in plan to the Periyar Head Regulator has to be recognised as an important factor [1A]. It was also considered that the longitudinal extension of the wall accommodating the scour vent to the desired extent would function as a divide wall during floods which is well known to exclude the bed sediment to tremendous degree.

2.3. Thus the studies were proposed to centre around the following four points :—

(*a*) To evolve a structure to induce a straight channel approach of flow into the Periyar Main Canal.

(*b*) To arrive at the best inclination (in plan) of the wall accommodating the proposed scour vent.

(*c*) To arrive at the optimum length of the wall to act as a divide wall.

(*d*) To study the sediment exclusion that would be effected by these improvements.

It is to be stated that the studies were considered worthwhile, since, at such a low stage as 1,600 cusecs in a river having a maximum flood of 70,000 cusecs, the tractive force along the bed may be little and even a wastage of 10 per cent may be able to reduce this sediment entry to a marked extent.*

3. Experimental set-up :

3.1. A mobile bed river model to a horizontal scale of 1 : 100 and vertical scale of 1 : 20 was constructed. The entire Peranai Regulator, Periyar Canal

* When Sri D. V. Joglakar raised the doubt as to what sediment exclusion could be effected with such a low wastage as 5 or 10 per cent, this was given as a reply and he was pleased to accept the point.

Head Regulator, a short reach of Periyar Main Canal and a sufficient reach of River Vaigai upstream were all simulated. Local river sand was used as mobile bed medium.

3.2. As the studies were all concerned with a channel flow and the structures were for local action, the model was proved only to an extent to find the rate of injection of sand charge which will maintain the bed configuration.

4. Experiments and Discussion :

4.1.1. *Structure to induce the flow to erode a channel for axial approach.*—The aim was to evolve a structure which will make the flow erode a channel straight to the head regulator. Hence it was obvious that it should function for the lowest possible flow in the river. As per the present regulation practice, Periyar Main Canal draws 1,450 cusecs out of which 50 cusecs is let down into the river through a sluice on the right bank to flow back into the river for Chittanai anicut. Arbitrarily a discharge of 75 cusecs was proposed to be sent through the left extreme vent of the Peranai Regulator. Hence experiments were conducted with a flow of 1,525 cusecs in the River Vaigai. The model was run for 45 hours to test each proposal installed. Sand was injected at the rate of 10 lbs./hour. After the run scour readings were taken and contours drawn. Configuration of the contours were taken as indicative of the utility of the proposal in inducing a straight approach, though the contours levels may not give quantitative scour pattern in prototype.

4.1.2. As for the structure itself, the field officers were in favour of spurs. " Spur when correctly positioned as an attracting spur causes flow to concentrate at its nose with a resultant curvature of flow which in turn caused the bed to scour and a deep channel to form. This in turn attracts the flow towards it as the river falls so the deep channel tends to persist and this continues so long as the curvature of flow remains favourable, which properly locates spur tends to ensure. A single unrelated spur or a single island will not, however, hold a river permanenty, unless it is connected with a large area of submerged rigid materials (rock, lime, kankar) " [2].

But a single spur, by creating and maintaining a curvature of flow induce it to swing back and an oscillating flow is set up. Hence, to avoid this swing, a series of spurs were proposed. But scour holes which form at the nose of these spurs is of an inverted cone shape with the apex at the nose and it should be paved. Series of such paved scour holes disturb the flow to a considerable extent. Hence, it was thought that a straight bank in rear of a spur may serve better in that it would satisfy the following three conditions :—

(1) It will prevent the swinging flow in rear of the spur.

(2) It will give a straight deep scour along its toe which will go a long way to a smooth channelised flow. This is the aim of the particular part of study.

(3) The revetment of the bank will serve as a rigid material to act in unison with the spur in holding the flow permanently.

4.1.3. With this in view tests were conducted in three series the first with a set of two spurs, the second with a straight bank in rear of a spur (the length of bank and inclination of spur being varies. This is preferably called a Guide Bank for brevity) and the third with series of three and four spurs only. Scour contours for six salient tests are given in Figures 3 to 8. Out of these the guide bank of Figure 6 is adjudged the best.

Hence, further experiments on the other parts of the study were done with this guide bank in position. Throughout the studies the efficacy of this guide bank in giving a straight approach was observed and found to be satisfactory

4.2. *Best inclination of the scour vent.*—This part of the experiment also was done with 1,525 cusecs in the river. The left extreme vent was constructed to 8 feet width and 75 cusecs was passed through that vent. A curved wall was simulated to guide the flow from the scour vent to the left vent of the Peranai Regulator. The scour vent was placed at angles varying from 90° to 120° successively. Sediment reduction over sand drawal in the basic run was calculated for each of these tests and the results are furnished in Statement I. It can be seen from the statement that the scour vent inclined at 90° to Periyar Head Regulator yielded the best results. In all the further experiments, the scour vent was kept at this inclination.

4.3. *Best length of divide wall.*—Divide walls are known to be at their best when there is flood in the river or when the full width of the river is covered by the flow. In lower stages their functioning is erratic. Hence by a few trials, a discharge of 5,000 cusecs was chosen for this part of experimentation as the discharge of 1,525 cusecs adopted so far did not cover the full width of the river. The divide wall was kept at right angles to Periyar Head Regulator as per the results obtained in the previous series of studies. For this study, the constriction of the left extreme vent was removed and all the ten shutters of Peranai Regulator were operated equally. The main canal was drawing its full supply discharge of 1,400 cusecs. Percentage reduction over the sand drawal without any device was taken as the criterion. Results are furnished in Statement II and from the sketch accompanying the statement it can be seen that the 100 feet long divide wall gives the best results.

4.4. *Sediment exclusion with the final shape of the proposal.*—4.4.1. Finally, the left extreme vent of the Peranai Regulator was divided into four equal vents 8 feet wide each and extended upto the divide wall by means of curved guide walls. Incidentally, it is to be observed that restricting the ventway is not going to give a higher velocity (as was mentioned in the original proposal) as the velocity is governed by the head causing the discharge only. But in this particular case the gate opening for a discharge of, say, 70 cusecs to be passed through a vent 40′ wide with a head of 12 feet, works out to 1.2 inches, the operation of which may be almost impossible in the field. It is presumed that this fact might have been the cause for the proposal to reduce the size of the scour vent. Anyhow, dividing the vent into smaller vents may also have a beneficial effect in that for a given gate opening, a smaller vent will function with a lower discharge. As per the present information the transported sediment passes a ventway in a more or less vertical band in the middle of a vent which suggests that a smaller vent with a larger opening may give a better sediment exclusion.

4.4.2. Hence, it was decided to divide the vent and tests carry out the further. The vent openings in the divide wall have their sill at El. 614.62 (the same as that of the Peranai Regulator) and were provided with shutters. Interveing curved walls were simulated right from the divide wall to the Peranai Regulator to maintain the high velocity of flow in rear of the vents even when they are operated individually. The outermost curved guide wall was taken above maximum water level. To improve the functioning, the frontage of this wall opposite the Periyar Head Regulator was paved so as to act as a deflecting vane of radius 200 feet. The details of the final shape of the proposal are shown in Figures 9 and 10. With the Periyar Main Canal drawing 1,400 cusecs, waste discharges of 70, 140, 210 and 220 cusecs were sent by operating one, two, three and four of the vents to the same extent successively. The reduction in percentage of sand charge over that of discharge percentage are given in Statement III. These reductions can be seen to be quite substantial.

5. Conclusions and Recommendation :

5.1. As a result of the studies conducted it can be stated that the proposal to convert the left extreme vent of the Peranai Regulator into a scour vent will improve the conditions to a marked extent. While the percentage reductions given in Statement III are considered sufficient to justify the proposal, they should not be taken to indicate directly proportional quantitative improvement. Incidentally it is to be noted that even sediment excluders require a wastage of about 20 to 30 per cent of the canal discharge. The proposed scour vent while functioning with as low a wastage as 5 per cent of canal discharge is expected not to get choked up, as sediment excluders usually do. The principle and the proposal as a whole is considered better than the excluders and may suit the prevailing conditions of the rivers in this State.

5.2.1. As regards guide bank it is to be stated that spurs as river training measures appear to have been adopted in the case of large meandering rivers. Model experiments are considered to be the only mode of solving problems of flow under non-regime condition[3]. It is considered that the present problem being one of training A channel flow in a river can easily be classified under flow in non-regime condition. The model showed that a guide bank will function efficiently. But spurs are considered much cheaper and as effective as guide banks[4]. Under these circumstances, the nature and type of construction of the guide bank (whether to construct as a long bank or as a series of armoured spurs) is left to the decision of field officers.

5.2.2. The upstream face of guide bank, acting as a spur and specially the tip should be well founded and protected (foundation being taken to an elevation lower than the scour shown in inset in Figure 9). Velocity observations made with a flow of 30,000 cusecs are given in the same figure just to indicate the magnitude of velocity that can be expected. The portions should be designed suitably to withstand the corresponding velocities.

5.3. As for the divide wall it is recommended that the foundation should be taken well below Peranai Regulator sill and after a flood the scour hole around the nose of the divide wall, could be paved to suit the profile with heavy stones each weighing not less than 500 lbs.

5.4. The outermost curved wall should be taken above the maximum water level of the pool. The other curved walls need be only to the height of flow expected in rear of individual vent.

5.5. As for the advantages on account of restricting the vent size occurr'ng the model cannot indicate any definite information becuase of the inherent defect of this type of models, namely, the suspended silt is not simulat d. Hence, it is recommended that construction work for dividing the vent can be taken up only if the field officers consider the gate opening as impractical and decide on dividing the vent straight away (vide para. 4.4.1) otherwise protctype observations can be made for one or two seasons by constricting the even temporarily before constructing the inner curved walls permanently.

6. References :

1. Report on the Fourth Meeting of the International Association for Hydraulic Research, page 237, para 8.1.

Same as item 1, page 241, para 9.1.

2. Annual Report of Central Water and Power Research Station during the year 1943, page 47.

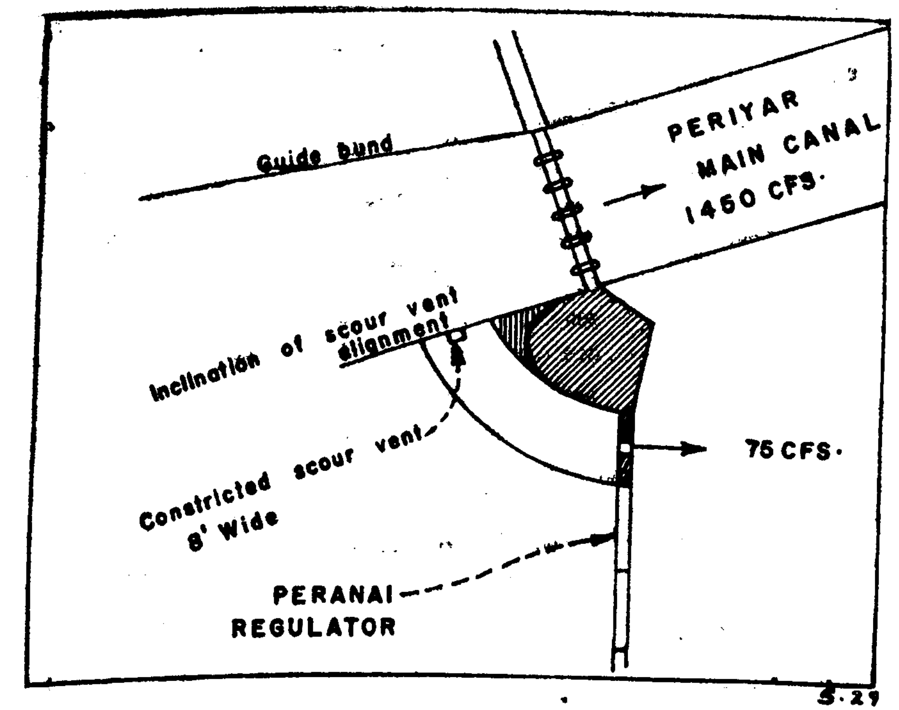
3. Maritime and Waterways Paper No. 7—"Meanders and Their Bearing on River Training", by Sir C. L. Inglis.

4. Same as item 3, page 45.

STATEMENT 1.

RESULTS OF SEDIMENT COLLECTION TESTS CONDUCTED TO FIND THE BEST INCLINATION OF SCOURS VENT ALIGNMENT.

Discharge in river	1,525 cusecs.
Rate of injection of sand charge	10 lbs./hour.
Total number of hours of run	45



No.	Details.	Total sand drawn into Periyar Main Canal.	Average hourly drawal.	Percentage reduction.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)
		LBS.	LBS.	LBS.	
1	With guide bund as in Figure 6 and no scour vent.	418	9.3	...	The left exit vent was not turned as proposed.
2	With no guide bund and with scour vent at 90°.	419	9.3	...	
3	With guide bund and scour vent at 90°.	147	3.3	64.5	These tests were conducted with guide bund as given in Figure 5.
4	With guide bund and scour vent at 100°.	718	4.9	47.3	
5	With guide bund and scour vent at 110°.	167	3.7	60.2	
6	With guide bund and scour vent at 120°.	188	4.2	54.9	Do.

NOTE.—Figures in column (5) $= \frac{9.3 - \text{Col. (4)}}{9.3} \times 100$

STATEMENT II.

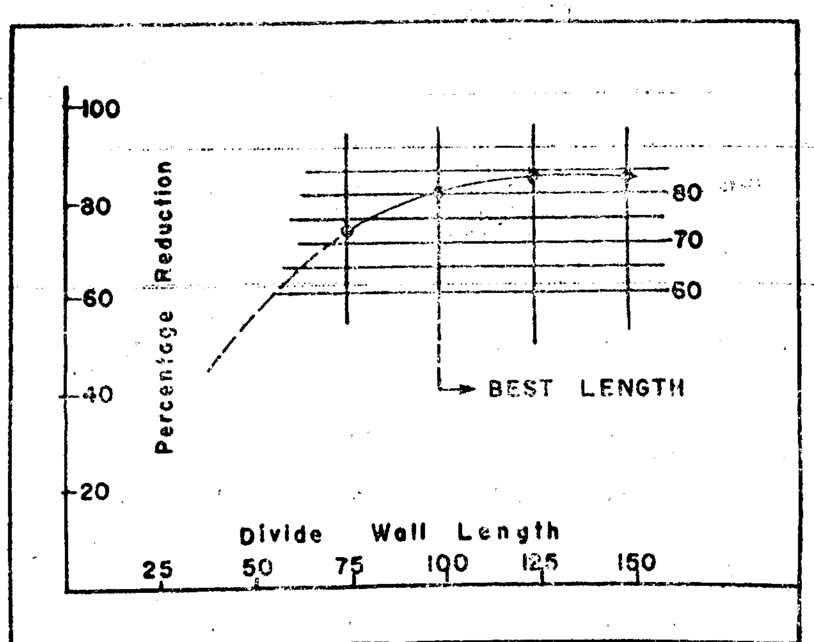
RESULTS OF SEDIMENT COLLECTION TESTS CONDUCTED TO FIND THE BEST LENGTH OF DIVIDE WALL.

Discharge in river	... ...	5,000 cusecs.
Rate of injection of sand charge	... ...	300 lbs./hour.
Total number of hours of run	... ...	45

No. (1)	Particulars. (2)	Total sand drawn into Periyar main canal. (3)	Average hourly drawal. (4)	Percentage (5)	Percentage reduction. (6)	Remarks. (7)
		LBS.	LBS.			
1	Basin run (without any divide wall).	3,080	68.4	22.8		
2	D' Wall 135' long ...	544	12.1	4.0	82.5	
3	D' Wall 125' long ...	500	11.1	3.7	83.8	
4	D' Wall 100' long ...	618	13.85	4.46	80.6	Best length (vide Figure below).
5	D' Wall 75' long ...	882	19.6	6.53	71.4	

NOTE.—(1) Col. 5 $= \frac{\text{Col. 4}}{300} \times 100$

(2) Col. 6 $= \frac{22.8 - \text{Col. 5}}{22.8} \times 100$



STATEMENT III.

RESULTS OF SEDIMENT COLLECTION TESTS TO FIND OUT SEDIMENT EXCLUSION EFFECTED BY THE PROPOSAL.

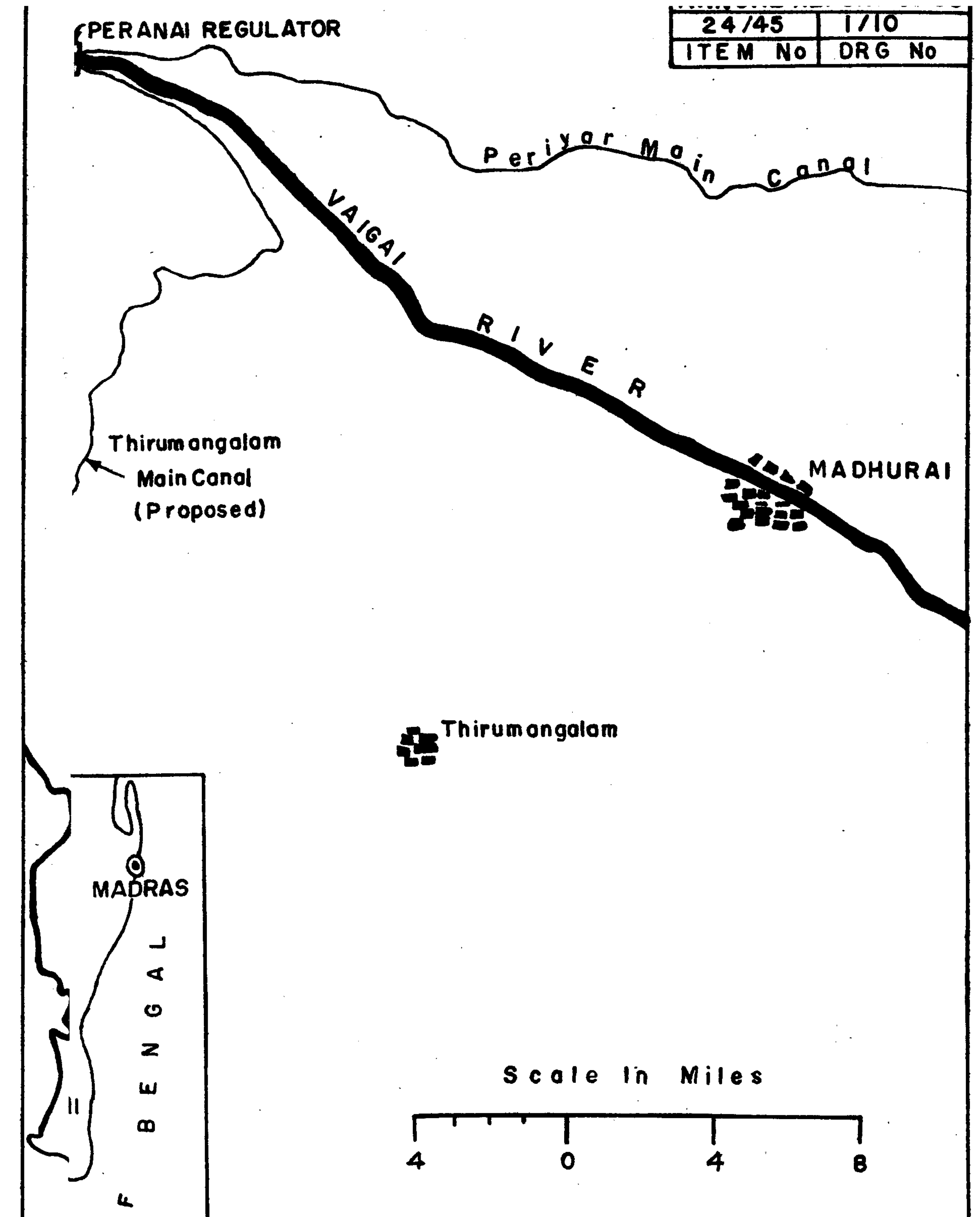
Discharge in river ...	... ...	1,400 cubic feet plus waste water
Rate of injection of sand charge	... ...	10 lbs./hour
Total number of hours of run	... ...	45 hours.

No.	Waste water.		Total sand drawn.		Average hourly drawal.		Percentage.	Percentage reduction.	Remarks.
	In C.ft.	In percentage of P.M.C. full supply discharge.	Periyar Main canal.	Peranai Regulator.	Periyar Main canal.	Peranai Regulator.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	70	5	174	384	3.87	8.53	31	67.5	
2	140	10	109	481	2.42	10.7	18.5	79.6	
3	210	15	71	510	1.58	11.33	0.12	86.0	
4	280	20	96	457	2.13	10.16	17.3	79.1	

NOTE.—(1) Col. 8 $= \frac{\text{Col. 6}}{\text{Col 6} \quad \text{Col. 7}} \times 100$

(2) Col. 9 $= \frac{\frac{1400}{1400 \times \text{Col. 2}} \times 100 = \text{Col. 8}}{} \times 100$

$$\frac{1400}{1400 \quad \text{Col. 2}} \times 100$$



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

PERIYAR MAIN CANAL
SILT EXCLUSION STUDIES
LOCATION OF
PERIYAR MAIN CANAL

Report File Drawing 58/59			
	J · E · R	A · R · O	E · E · R
	Trd J·J	Ckd	Fig ... 1

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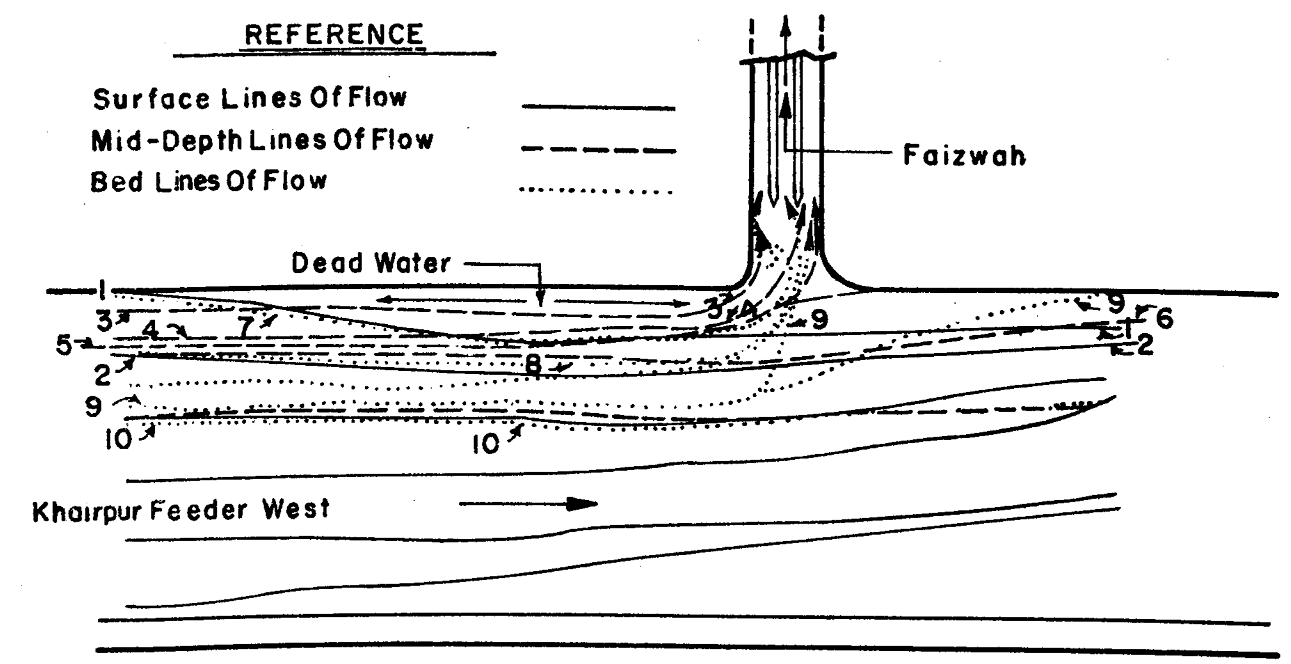

Fig. 1. Faizwah Model Experiments — Plan Showing Lines Of Flow
Q In Khairpur West Feeder = 3·8 Cusec (= 1,836 cusec.)
Q In Faizwah = 0·64 cusec (= 310 cusec.)

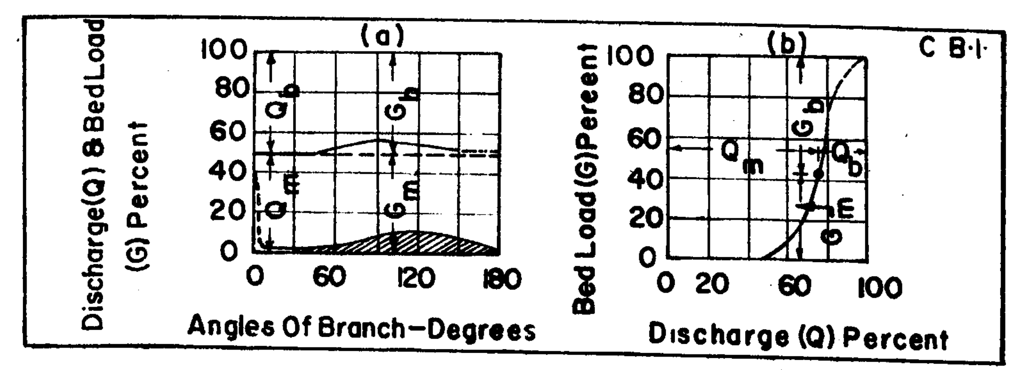

Fig 2: (a) Relation Between Division Of Bed Load And Angle Of Branch. The Subscript b And m Apply To The Branch And Main Channel Respectively (H. Bulle).

(b) Relation Between Division Of Bed Load And Division Of Discharge For An Angle Of Branch Of 30 Degrees (H. Bulle).

A-3-a File Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
	PERIYAR MAIN CANAL SEDIMENT EXCLUSION		
	Figures Reproduced From Reference — I		
59/59 Drawing	J · E · R	A · R · O	Exe·Engineer
	Trd J·J	Ckd	Fig 2

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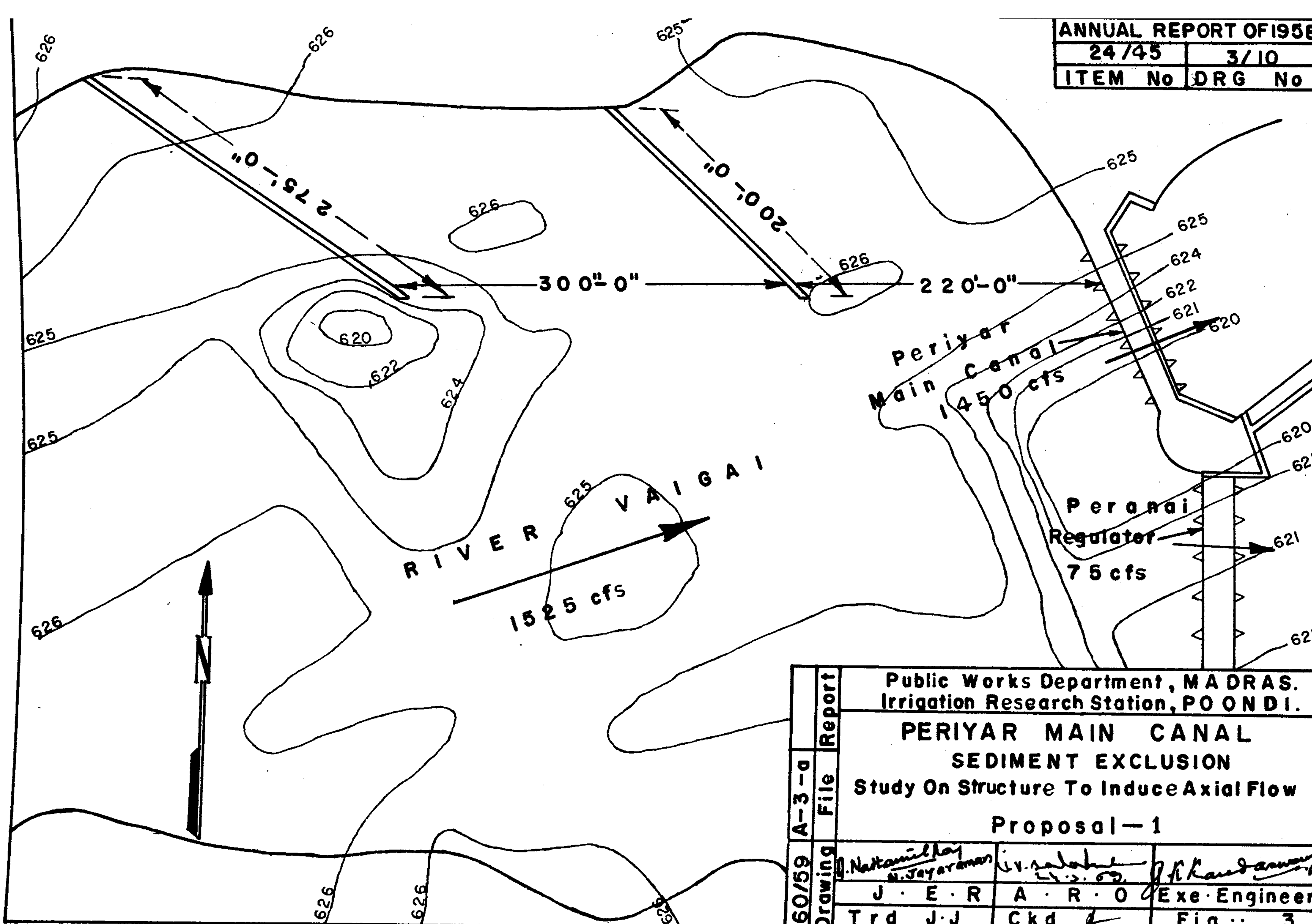

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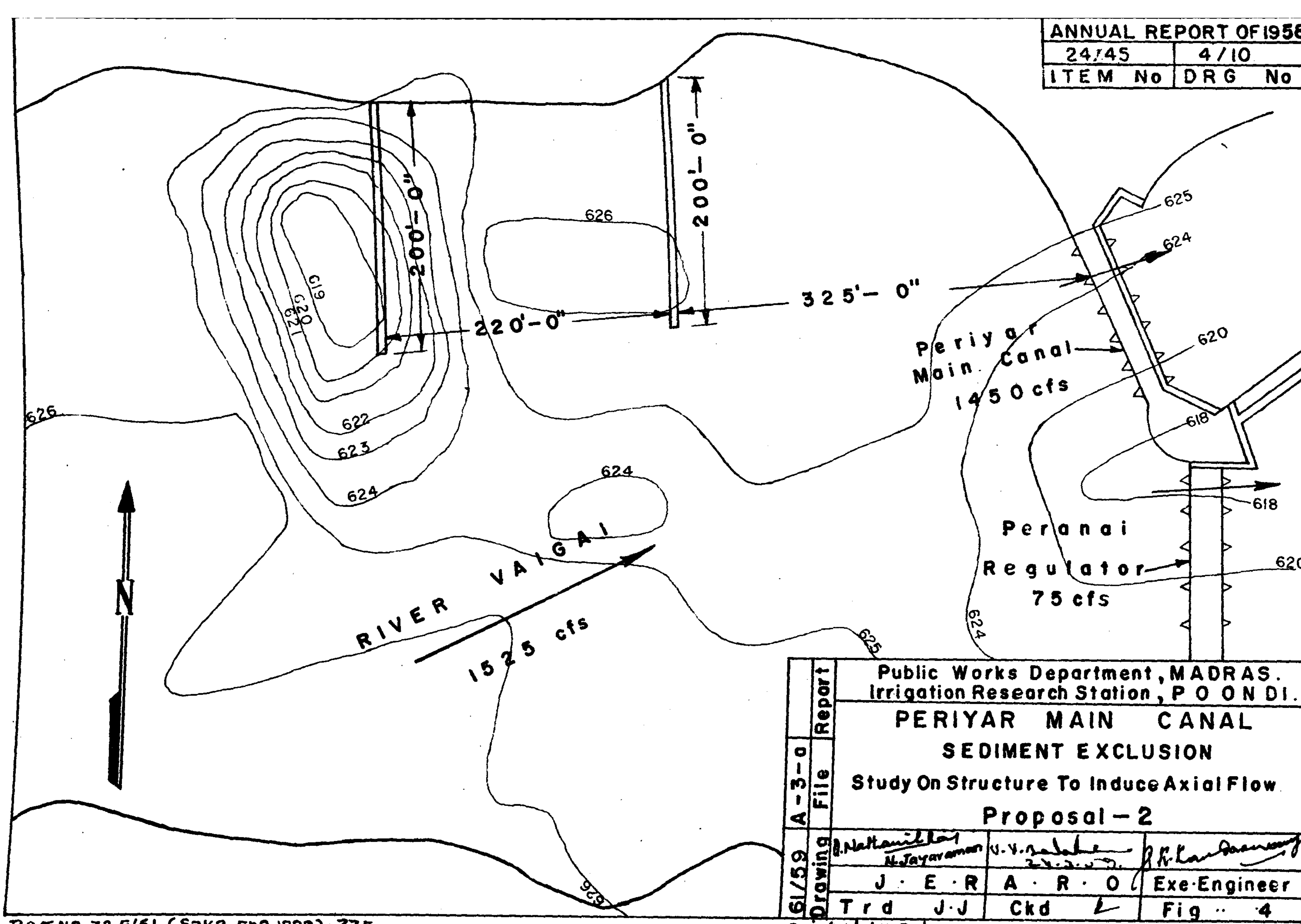

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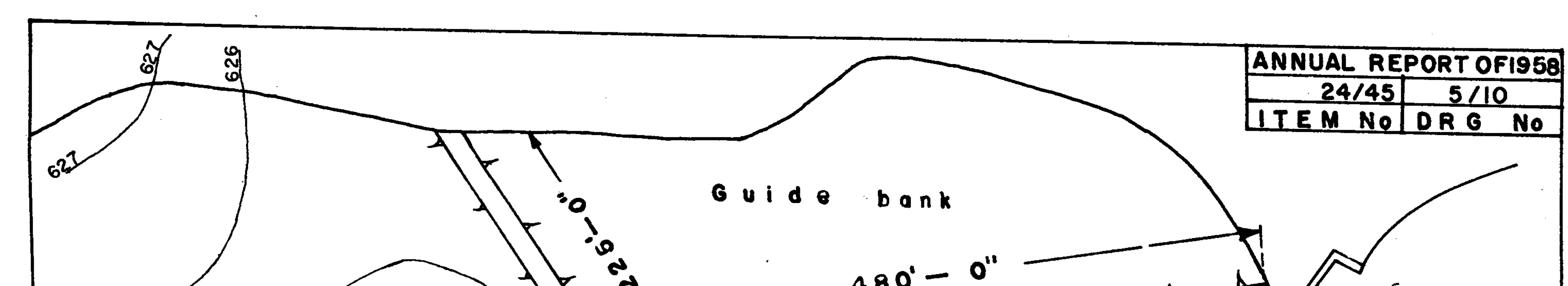

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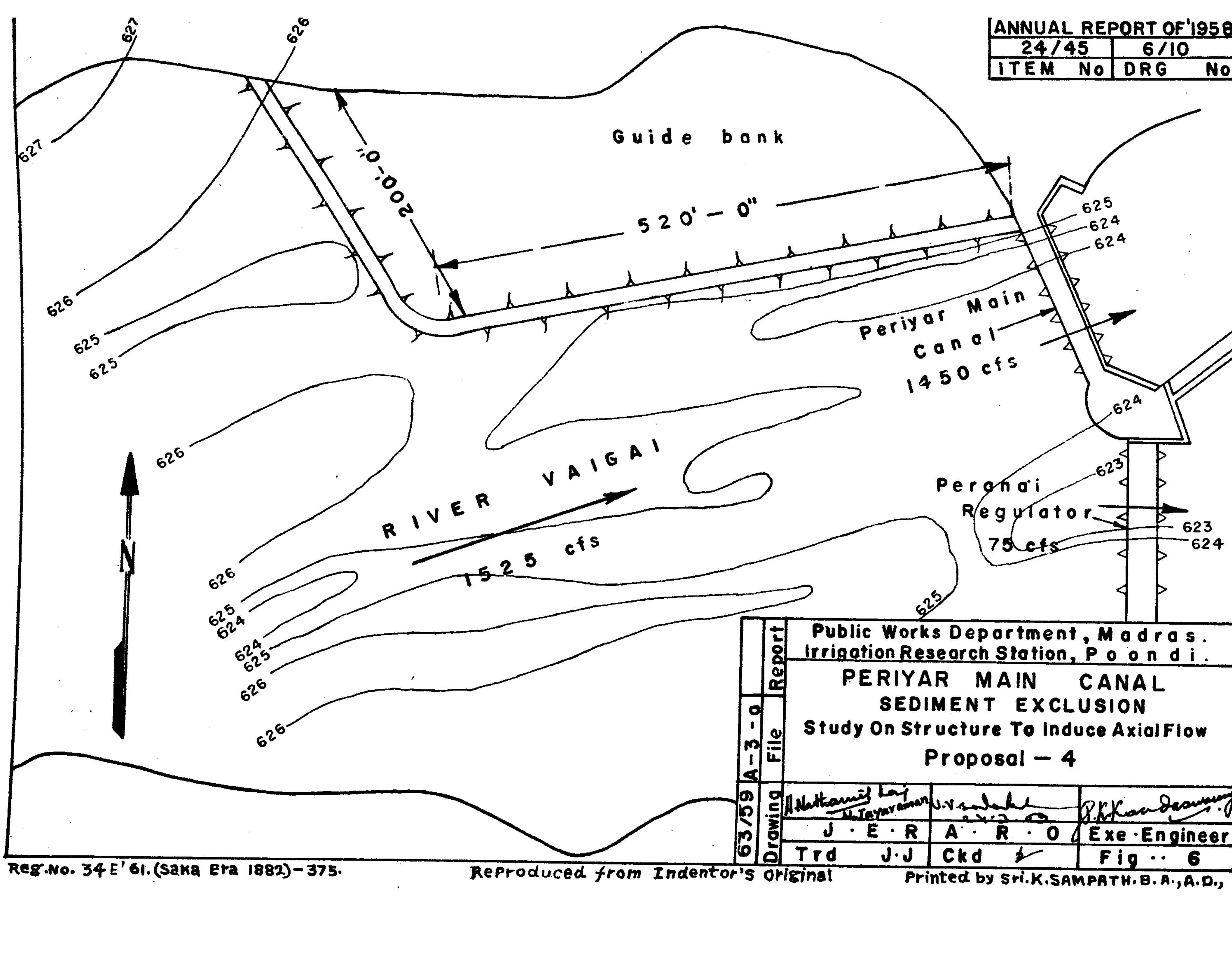

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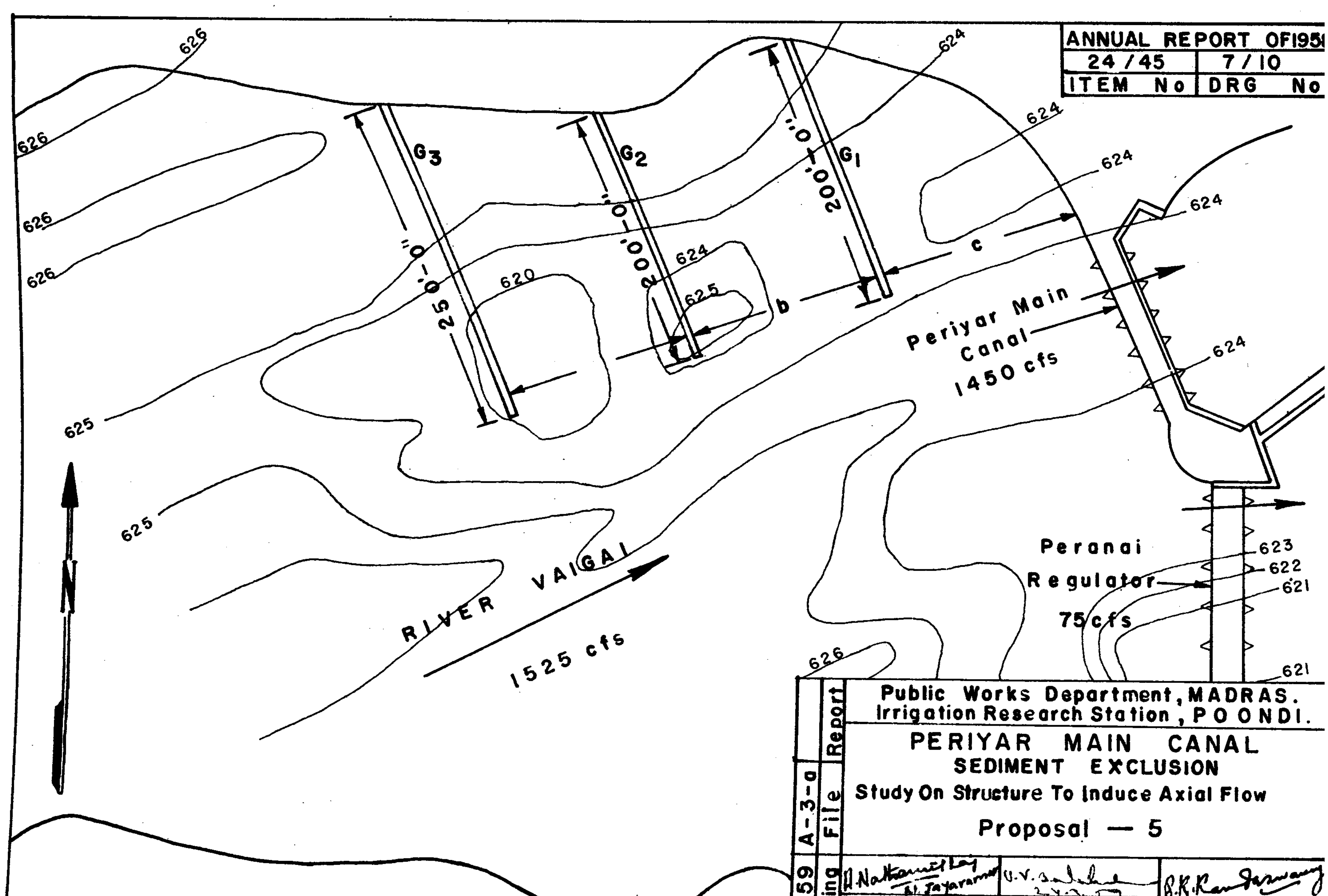

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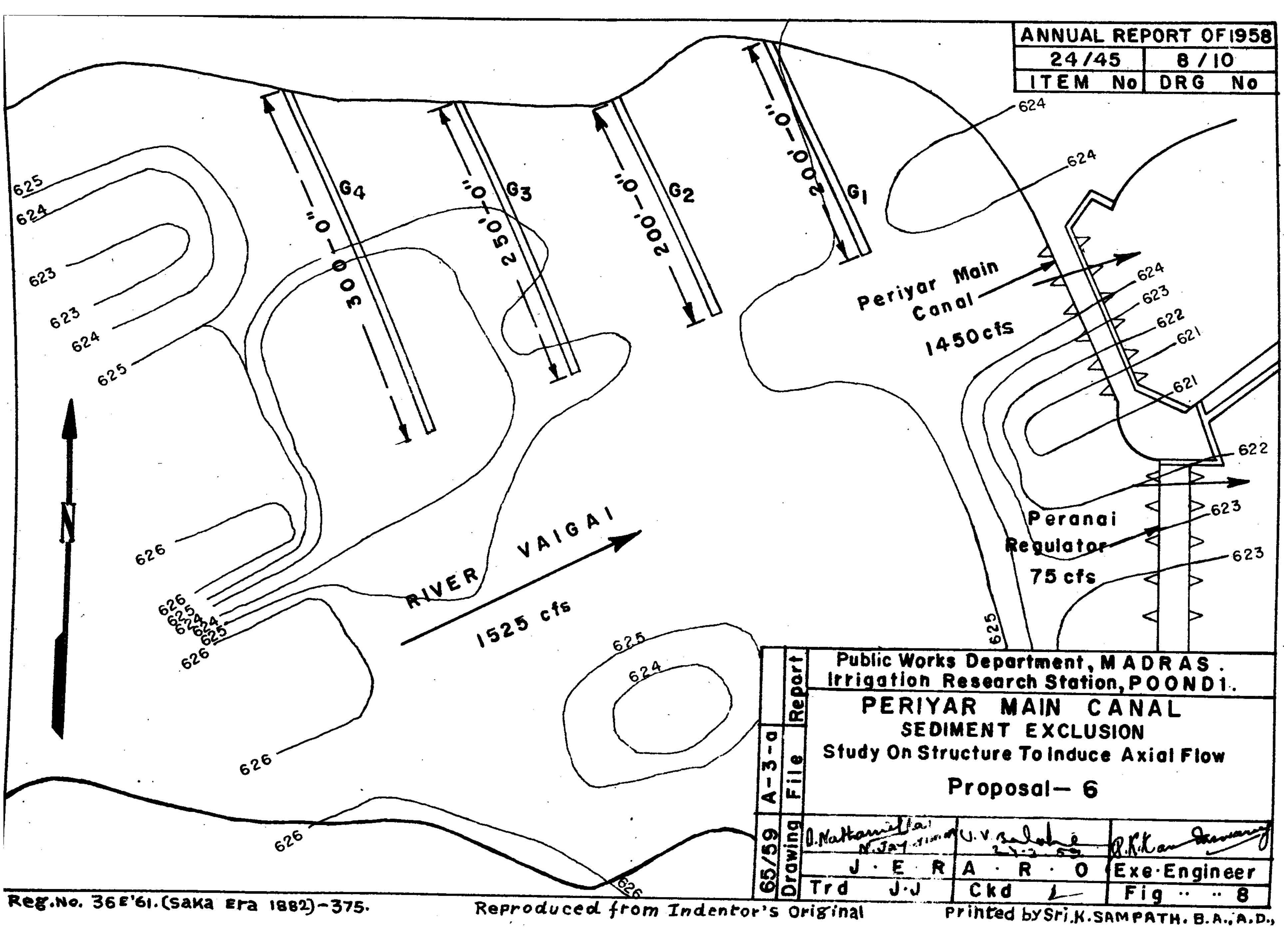

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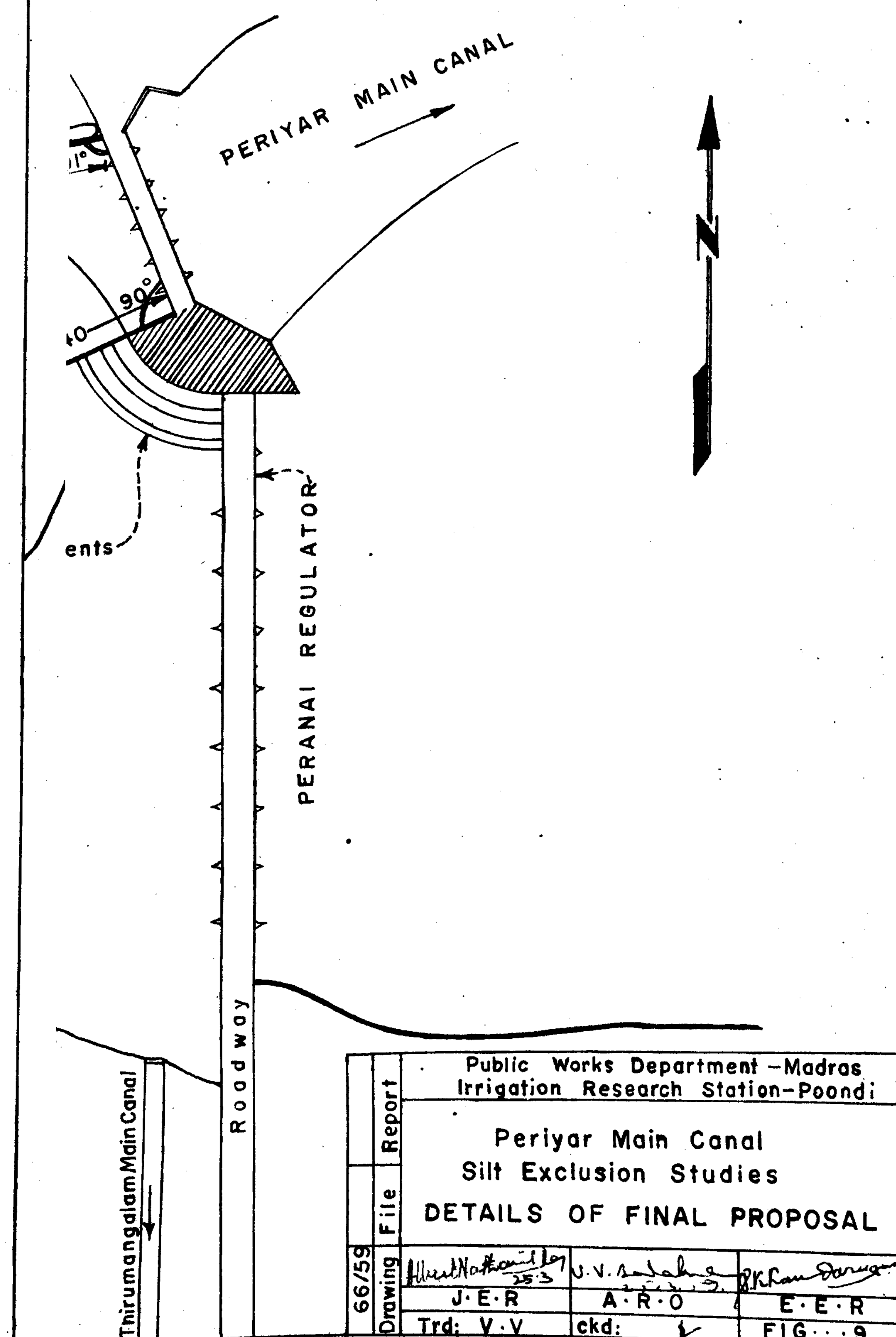

Public Works Department – Madras
Irrigation Research Station–Poondi

Periyar Main Canal
Silt Exclusion Studies
DETAILS OF FINAL PROPOSAL

Drawing 66/59	File	Report
J.E.R	A.R.O	E.E.R
Trd: V.V	ckd:	FIG...9

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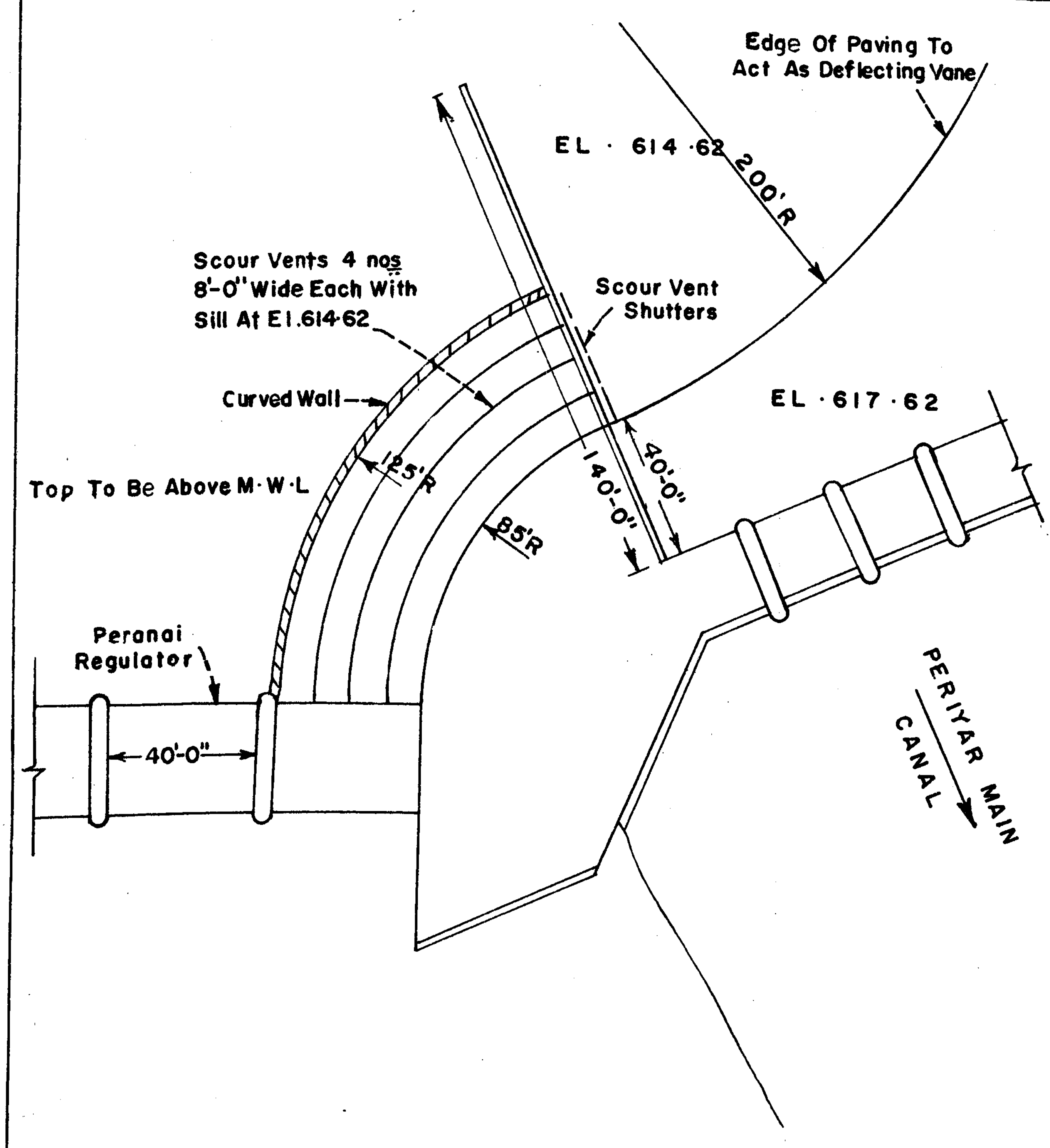

Scale in Feet

40 0 40 80

ANNUAL REPORT OF '58	
24/45	10/10
ITEM No	DRG No

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

PERIYAR MAIN CANAL
SILT EXCLUSION STUDIES
Details Of Scour Vents

Drawing 67/59	File	Report
J E R	A : R · O	E · E · R
Trd V·V	Ckd	Fig.. 10

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SREEVAIKUNTAM ANICUT—EQUITABLE DISCHARGE.

1. Synopsis :

The report deals with the studies conducted to establish equitable distribution of discharge through the two canals taking off from either side of the anicut, by shifting the deep course of the river from the left margin to the centre. Three groynes of length 435 feet, 495 feet and 342 feet respectively were found to be the best to achieve the object. Two curved leading channels bifurcating from the shifted deep course, a little higher up the anicut were finalised. Circular curve has been found to be the ideal one for the shape of the nose at the bifurcation point. To protect the groynes and the nose during floods, launching aprons have been proposed.

2. Introduction :

2.1. Srivaikuntam anicut is the eight and the last anicut across the river Thambraparani which originates from the Western Ghats and flows into the gulf of Mannar through the Tirunelveli district for about 75 miles, draining an area of 1,750 square miles. Two main canals, namely the North Main Canal and the South Main Canal take off from either end of the anicut, their sill elevations being kept at the same level.

2.2. *Salient features of the anicut and the canals anicut—*

Crest level lower portion	39.40 (1,026 ft. long).
Crest level upper portion	41.10 (226 ft. long).
Upstream apron level	31.40.
Sand vent right side	9 Nos. 4′ × 4′.
Sand vents left side	9 Nos. 4′ × 4′.
Total No. of spans (29 spans each 38′ 0″ long, 4 spans each 37′ 6″ long).	33.
Piers	32 Nos. 4′ 0″ thick.

North Main Canal—

Sill level of the head sluice	31-40.
No. of vents	6 Nos. 4′ × 8′.
Full supply depth in rear	6′ 0″.
Full supply discharge	1,330 cusecs.
Bed width	36′ 0″.
Ayacut	12,093 acres.

South Main Canal—

Sill level of the head sluice	31.40.
No. of vents, 5 Nos.	2 Nos. 6′ × 6′.
	3 Nos. 4′ × 8′.
Full supply depth in rear	7′ 0″.
Full supply discharge	1,525 cusecs.
Bed width	50′ 0″.
Ayacut	12,185 acres.

3. Problem :

This anicut, being the tail end one, does not receive adequate supply and no water flows over the anicut except at times of flood. As the low discharge runs for a partial width of the river, deep course usually gets formed in the river along left margin due to a sharp bend of the river, at this site. The North Main Canal, by virtue of its closer proximity to the deep course of the river, draws more water than its due share, thereby depriving the South Main Canal of its full share of supply. The problem, therefore, consists in shifting the deep course of the river towards the centre of the river for establishing conditions for equitable distribution of water in the canals at times of low discharge in the river.

4. Model :

A model has been constructed to a scale of 1/1?0 Horizontal and 1/36 vertical representing the entire anicut and the canals on either side and a reach of about a mile upstream and about half a mile downstream of the anicut. Preliminary studies have been conducted and the model proved.

5. Studies :

5.1. Experiments were conducted with a discharge of 27,216 cusecs in the river—the discharge just sufficient to make the bed silt move - the two canals drawing their designed discharges. The rate of sand injection was found out to be 11.25 lbs. per hour to keep the river in the regime condition. This condition was maintained for all the proposals studied.

5.2. To shift the deep course of the river towards the centre, the following five proposals consisting of groynes placed on the left side of the river were tried. After 54 hours of run for each proposal, scour profiles were taken across the river along the five cross-sections and the levels were plotted :—

(*a*) A single groyne 320 feet long was tried (vide Figure 1).

(*b*) Three groynes 360 feet, 457 feet and 375 feet long were tied (vide Figure 2).

(*c*) Three groynes 435 feet, 495 feet and 342 feet long were tried (vide Figure 3).

(*d*) With a view to get reduced lengths of groynes three groynes 283 feet, 334 feet and 342 feet long were tried (vide Figure 4).

(*e*) Three groynes 300 feet, 334 feet and 345 feet long were tried (vide Figure 5).

5.3. A statement regarding the performance of the groynes in all the five proposals is given in Statement I. A comparison of the scour profiles plotted for all the five proposals shows that the third proposal is found to be the best (vide Figures 6 and 7). This proposal consists of three groynes each 435 feet, 495 feet and 342 feet long and has been found to be very effective in shifting the deep course of the river towards the centre.

5.4. *Studies on the equitable distribution of the discharges.*—5.4.1. The groynes if constructed as mentioned above will be useful only for diverting the deep course towards the centre of the river. The problem is to establish an equitable discharge through the two canal head sluices. In view of the fact that the ayacut for the two canals is almost the same being nearly 12,000 acres, it was decided to send in the model almost equal discharges through the two canals.

5.4.2. As a first trial, a main deep channel 250 feet wide was moulded at the centre of the river along the shifted deep course, bifurcating closer to the anicut, into two leading branch channels 125 feet wide each with straight entries to the canal head sluices. In the second trial the deep course was bifurcated at a point, a little higher up the anicut into two curved leading channels of equal width as before. In the third proposal, keeping the bifurcation point and the shape of the curved leading channels as the same, the widths of the leading branch channels to the South Main Canal and the North Main Canal were made as 143 feet and 107 feet respectively. In the fourth trial the corresponding widths of the branch channels were altered to 160 feet and 90 feet having all other things the same as before. Of all these four proposals, the last one was found to function most satisfactorily in sending almost equal discharges through the two canals (vide Figure 8).

5.4.3. It is considered that the excavation of the main deep channel 250 feet wide is unnecessary as it may involve lot of desilting work. Instead of that the shifted deep course itself can be used as the main supply channel and the curved leading channels can take-off from the deep course. Also, the shape of the nose at the bifurcation point has been studied.

5.4.4. In the model, after the deep course of the river was shifted to the centre, the two Branch channels of width 160 feet and 90 feet to the South Main Canal and the North Main Canal respectively were incorporated. The nose was made in teakwood with a circular curve at the bifurcation point, and fixed in position. Low discharges ranging from 1,000 cusecs to 4,000 cusecs were allowed in the river, the idea being that if the discharges through the canals are distributed equitably for these low discharges then it will automatically hold good for higher discharges also. The discharges through the two canals were measured and it was observed that almost equal discharges were drawn through the two canal head sluices. A graph is plotted representing the variation of the ratio of the discharges through the two canals with the discharges in the river. It is seen therefrom that the ratios of the two discharges in the canals lie within the limits of 1.00 and 1.15, thereby indicating the corrected nature of the equitable distribution (vide Figure 9).

5.4.5. For the low discharges the velocity of water in the river and the branch channels was very low and little or no scour was observed. When a discharge of 4,000 cusecs was running in the river a negligible scour of 2′ 0″

prototype) was observed at the nose. So much so, any rigid structure without sharp edges can be had for the nose. From the point of view of ease of construction, a circular arc about 10 feet radius may preferably be used for the nose at the bifurcation point.

5.5. *Studies for the Protection of the Groynes and the Nose from Floods.*—5.5.1. When high discharges flow in the river, considerable scour is produced around the groynes and the nose and so they have to be protected from being undermined. Launching aprons around the groynes and the nose have been proposed. In the model, small pebbles were spread around the groynes and the nose of bifurcation. A discharge of 27,216 cusecs was allowed in the river with the usual conditions as before. After eight hours of run the launching apron was found to have sunk to a depth of 1′ 6″ (prototype value). Some more pebbles were put in so that the apron level is maintained constant and the model was continued to run as before. Afterwards no sinking of the apron was observed. So this proposal was found to be satisfactory. In the prototype large size boulders can be dumped around the groynes and the nose. If there is any sinking of the dumped stones due to heavy floods, some more boulders may be dumped so that the apron is kept at constant level.

5.5.2. For studying the function of this launching apron the following photographic views were taken in the model and compared :—

View I—Bed moulded to the original level, with the groynes and nose in position.

View II.—Conditions of bed around the groynes and nose after 40 hours of run for a discharge of 27,216 cusecs.

View III—Bed with the proposed launching apron around the groynes and nose before running the model.

View IV—Bed with the launching apron around the groynes and nose after running the model for 40 hours for a discharge of 27,216 cusecs.

5.5.3. Practically no difference in level can be noticed in the bed around the groynes in the Views I, III and IV. Thus, it is seen that the provision of launching apron protects the groynes and the nose during high floods to a great extent.

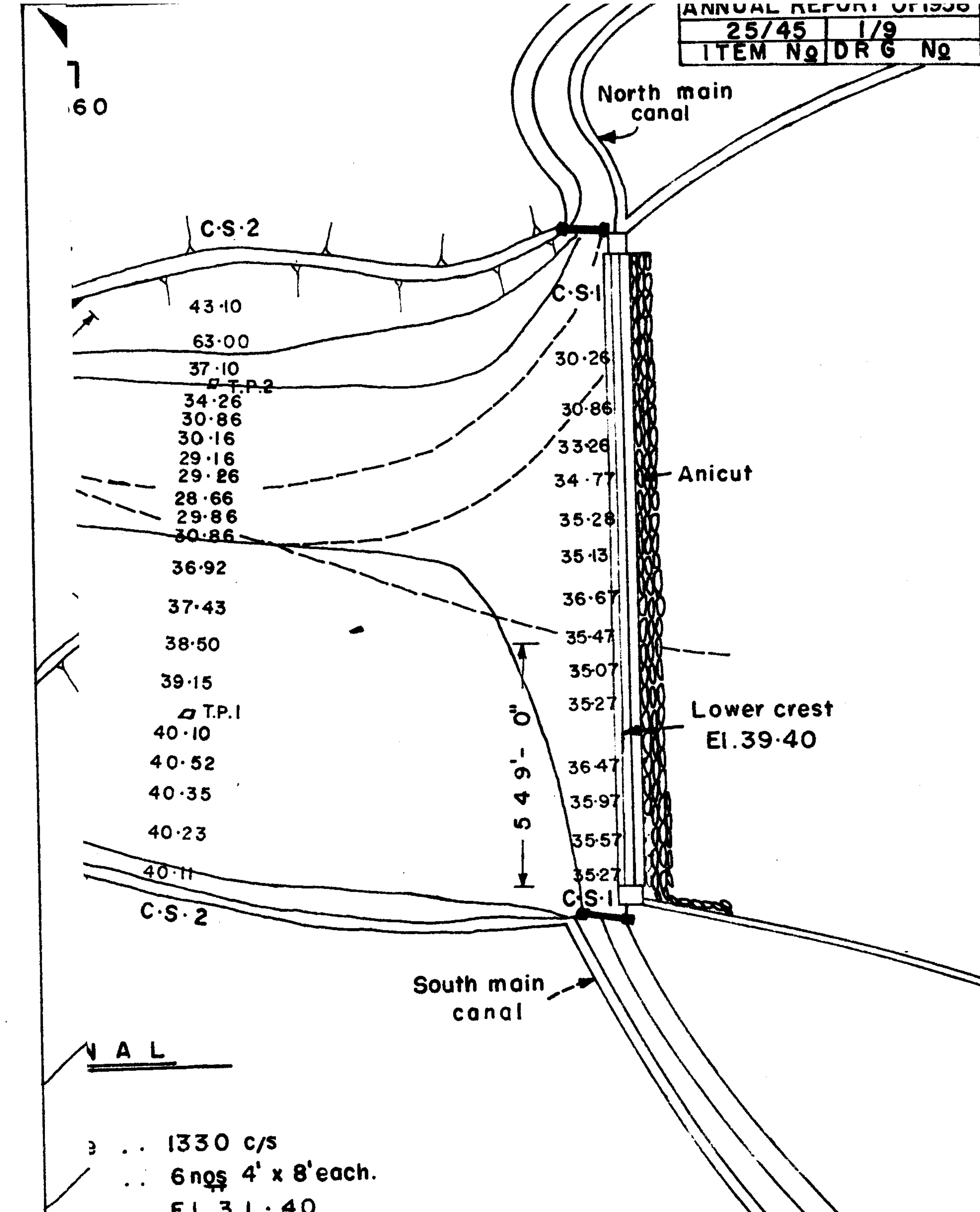
6. Conclusions :

The deep course of the river which was originally very close to the left margin has been shifted towards the centre by means of three groynes 435 feet, 495 feet and 342 feet long, constructed on the left margin of the river. The shifted deep course of the river can be used as the supply channel which bifurcates into two curved leading channels to the two canal head sluices, with a nose at the bifurcation point, the width of the leading channel to the South Main Canal being 160 feet and the other being 90 feet. The nose may be a rigid structure and it may be circular in shape at the bifurcation. point. Launching aprons may be provided around the groynes and the nose for protecting these structures during high floods. The above proposals have been finalised and are recommended for adoption at site.

STATEMENT I.

SRIVAIKUNTAM ANICUT DIVERSION OF DEEP COURSE TOWARDS RIGHT.

Sl. No. (1)	*Details of groynes.* (2)	*Performance.* (3)	*Remarks.* (4)
1	A single groyne of length 330 feet with position and size as marked in Figure 1.	The bottom filaments were passing 549 feet away from the right abutment (Total length of the anicut 1,380 feet.)	It was desided to have some more groynes.
2	Three groynes of length 360 feet, 457 feet and 376 feet as marked in Figure 2.	The bottom filaments were passing 549 feet away from right abutment.	The first groyne was not effective. The other two more having only local influence.
3	Three roynes of length 435 feet, 495 feet and 342 feet as marked in Figure 3.	The bottom filaments were passing only 309 feet away from the right abutment.	All the three were effective by way of giving a better performance—FINALISED.
4	Three groynes of length 283 feet, 334 feet and 342 feet as marked in Figure 4.	The bottom filaments were passing 435 feet away from right abutment.	The first and the second groynes were not effective.
5	Three groynes of length 300 feet, 334 feet, 342 feet as marked in Figure 5.	The bottom filaments were passing 492 feet away from the right abutment.	The first two groynes were not effective.



NAL

.. 1330 c/s
.. 6 nos 4' x 8' each.
.. El. 31·40
.. 6'

.. 1525 c/s
.. 2 nos 6'x 6' each
.. El. 31·40
.. 7'

rs. of run cross sections
laces marked C·S·1, C·S·2,
C·S·5 .

	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
Report	SRIVAIKUNTAM ANICUT EQUITABLE DISTRIBUTION IN THE CHANNELS. (Design no I)		
File B-2	S. Lakshmanan	[illegible]	[illegible]
Drawing 125/59	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd. [illegible] 61	Fig. I

Reg.

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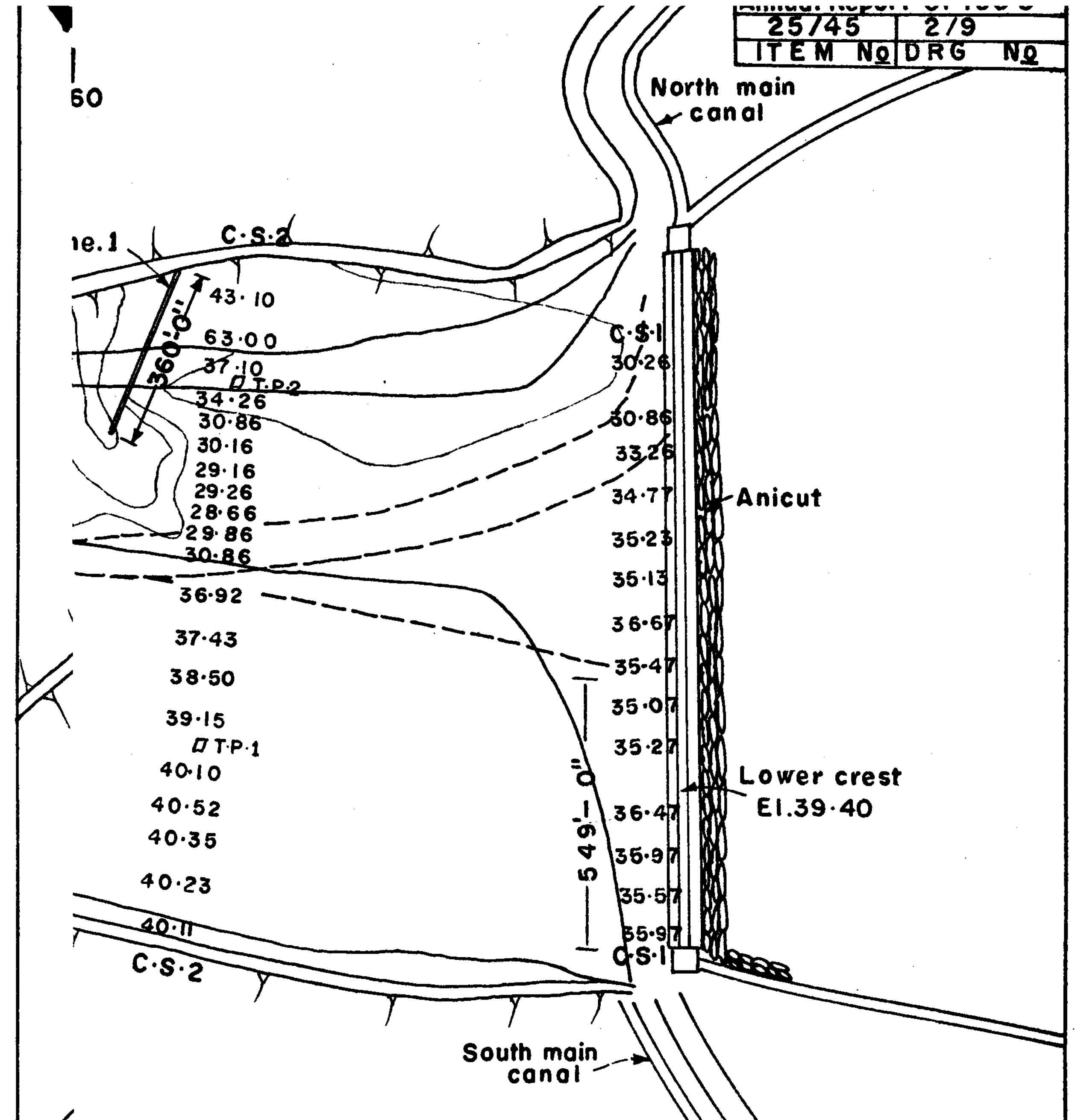
25/45	2/9
ITEM No	DRG No



NAL

e .. 1330 c/s
.. 6 nos 4'x 8'each
.. El.31 ·40
.. 6'

.. 1525 c/s
.. 2 nos 6'x 6'each.
.. El.31·40
.. 7'

's. of run cross sections

aces marked C·S·1, C·S·2,

·S·5

Drawing 126/59 B-2 File Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
	SRIVAIKUNTAM ANICUT EQUITABLE DISTRIBUTION OF DISCHARGES (Design no 2)		
	S. Chinnamma	U.V. [illegible]	S.K.Kandaswamy
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd. [illegible]	Fig. 2

Reg

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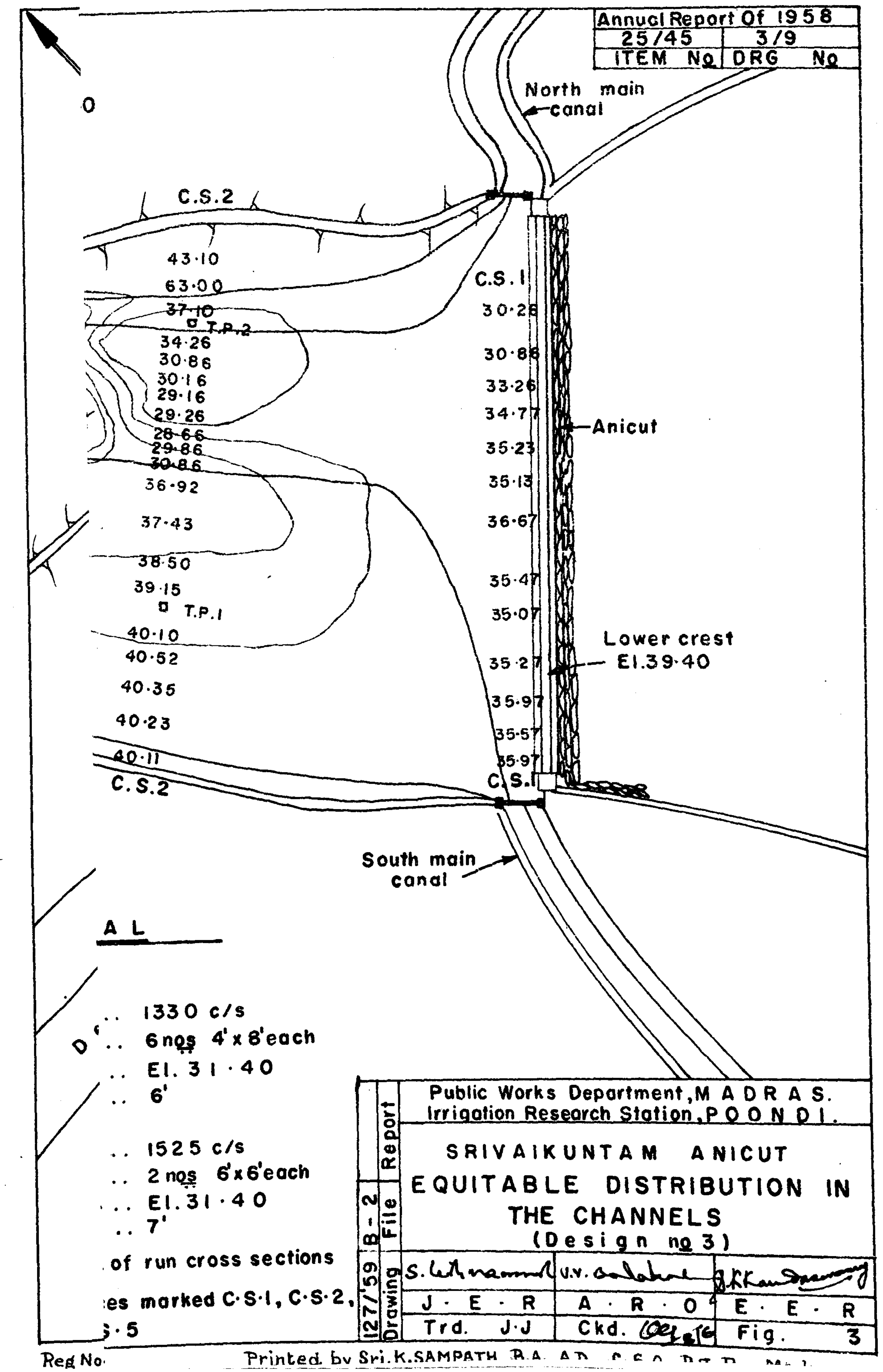
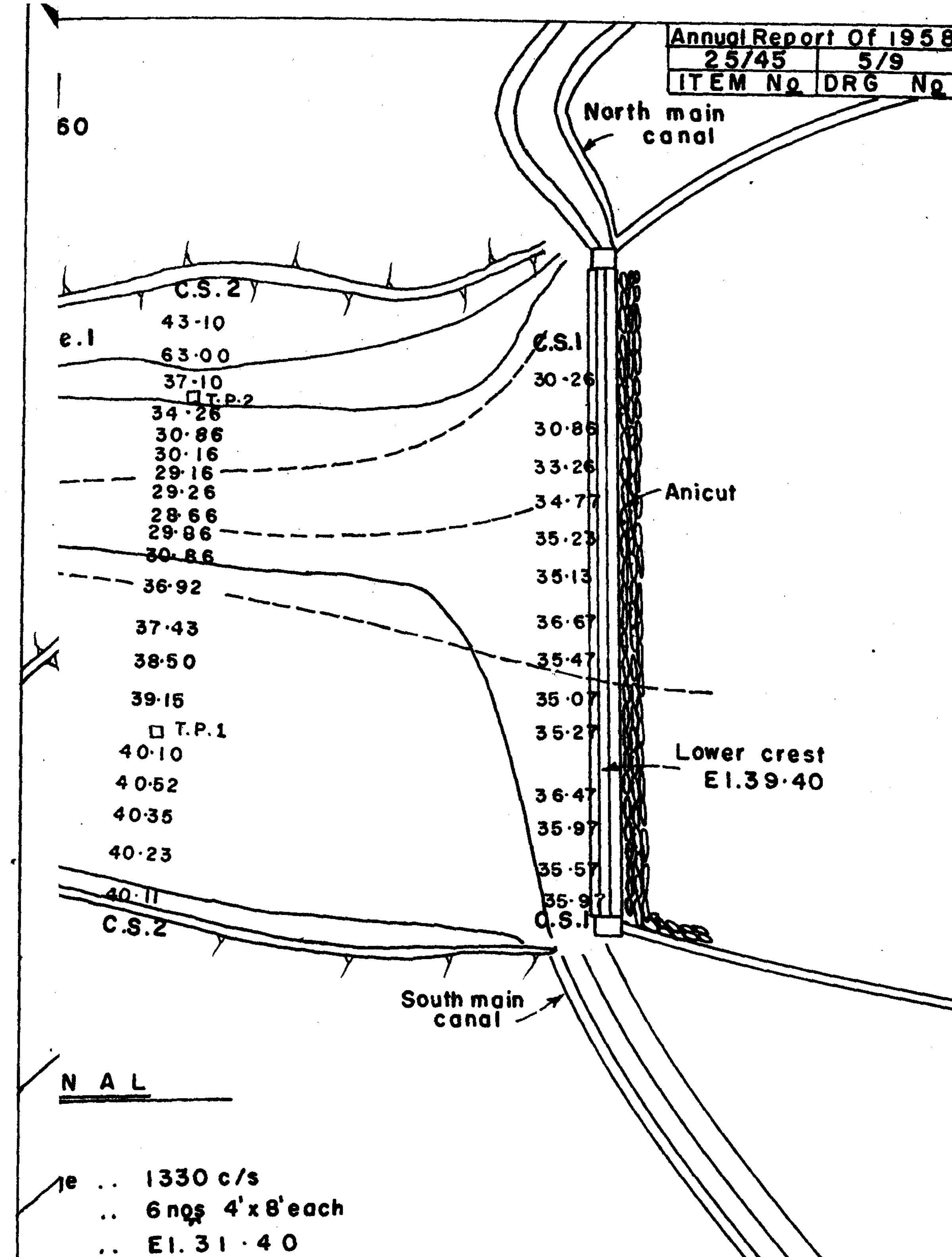


Annual Report Of 1958
25/45
3/9
ITEM No
DRG No
North main canal
C.S.2
C.S.1
43·10
63·00
37·10
T.P.2
34·26
30·86
30·16
29·16
29·26
28·66
29·86
30·86
36·92
37·43
38·50
39·15
T.P.1
40·10
40·52
40·35
40·23
40·11
30·26
30·86
33·26
34·77
35·23
35·13
36·67
35·47
35·07
35·27
35·97
35·57
35·97
Anicut
Lower crest El.39·40
South main canal
A L
.. 1330 c/s
.. 6 nos 4'x 8' each
.. El. 31 · 40
.. 6'
.. 1525 c/s
.. 2 nos 6'x 6' each
.. El. 31 · 40
.. 7'
. of run cross sections
es marked C·S·1, C·S·2,
·5
Public Works Department, MADRAS.
Irrigation Research Station, POONDI.
SRIVAIKUNTAM ANICUT
EQUITABLE DISTRIBUTION IN THE CHANNELS
(Design no 3)
127/59 B-2
Drawing File Report
J · E · R
A · R · O
E · E · R
Trd. J·J
Ckd.
Fig. 3
Reg No.
Printed by Sri.K.SAMPATH B.A.

Annual Report Of 1958	
25/45	5/9
ITEM No	DRG No



N A L

e .. 1330 c/s
.. 6 nos 4'x8'each
.. El. 31·40
.. 6'

e .. 1525 c/s
.. 2 nos 6'x6'each
.. El. 31·40
.. 7'

rs. of run cross sections
laces marked C·S·1, C·S·2,
C·S·5.

File B-2 Report 129/'59 Drawing	Public Works Department, MADRAS. Irrigation Research Station, POONDI.
	SRIVAIKUNTAM ANICUT EQUITABLE DISTRIBUTION OF DISCHARGES (Design no 5)

S. [signature]	V.V. [signature]	[signature]
J·E·R	A·R·O	E·E·R
Trd. J·J	Ckd. [signature]	Fig. 5

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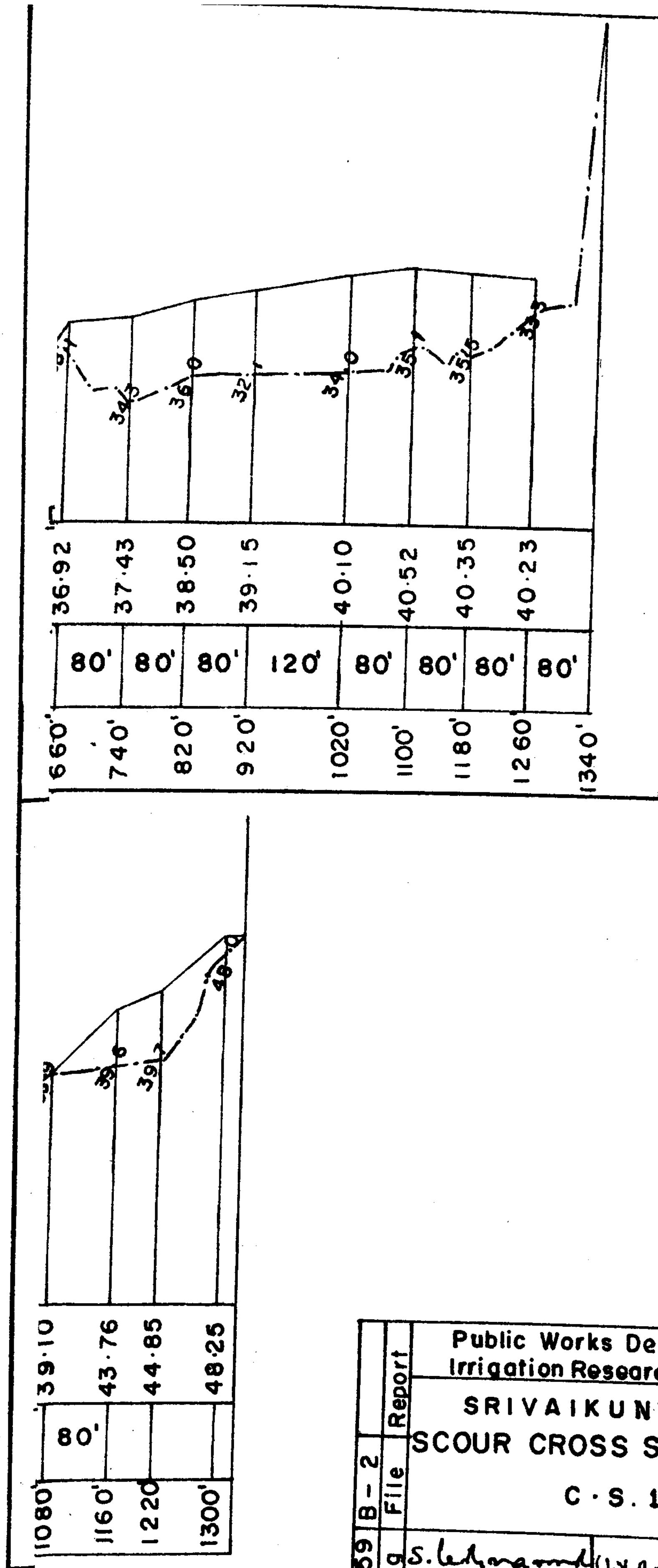
Annual Report of 1958	
25 / 45	6/9
ITEM No	DRG No

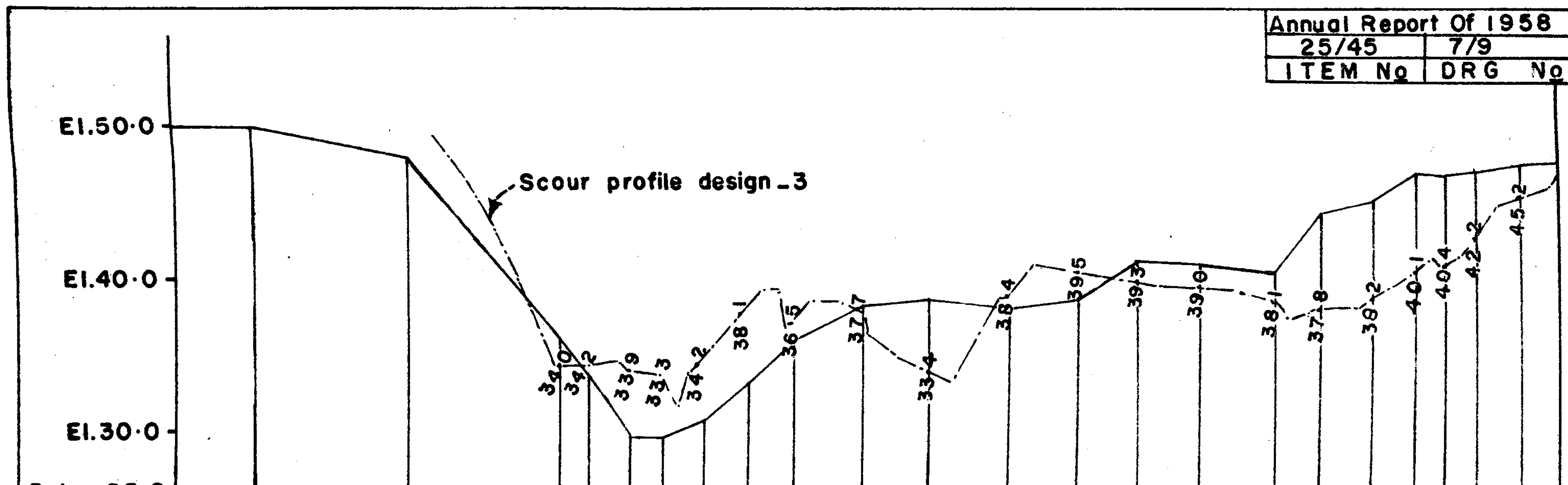


Public Works Department, MADRAS.
Irrigation Research Station, POONDI.

SRIVAIKUNTAM ANICUT
SCOUR CROSS SECTIONS FOR DESIGN_3
C·S.1 TO C·S.4

Report File Drawing 130/59 B-2			
	S. Lakshmanan	U.V. Adakshi	J.C. Rangaswamy
	J·E·R	A·R·O	E·E·R
	Trd. J·J	Ckd.	Fig. 6

Annual Report Of 1958	
25/45	7/9
ITEM No	DRG No



R · L	49·87	48·19	35·99	32·89	29·19	29·29	30·39	32·89	35·61	37·92	38·47	37·97	38·27	40·85	40·47
Distance	100'	200'	200'	40	50'	50'	50'	60'	60'	90'	90'	100'	90'	80'	
Total Distance	100'	300'	500'	540'	590'	640'	690'	750'	810'	900'	990'	1090'	1180'	1260'	1340'

CROSS SECTION _ 5

NOTE :— Location of cross section marked in fig. no 3

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File B - 2	SRIVAIKUNTAM ANICUT SCOUR CROSS SECTION FOR DESIGN_3 C . S . 5		
Drawing 131/59	S. Lakshmanan	U.V. Bandahari	J.K. Kandaswamy
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd.	Fig. 7

Reg.No.10.E-61-(Saka Era-1882)-375.
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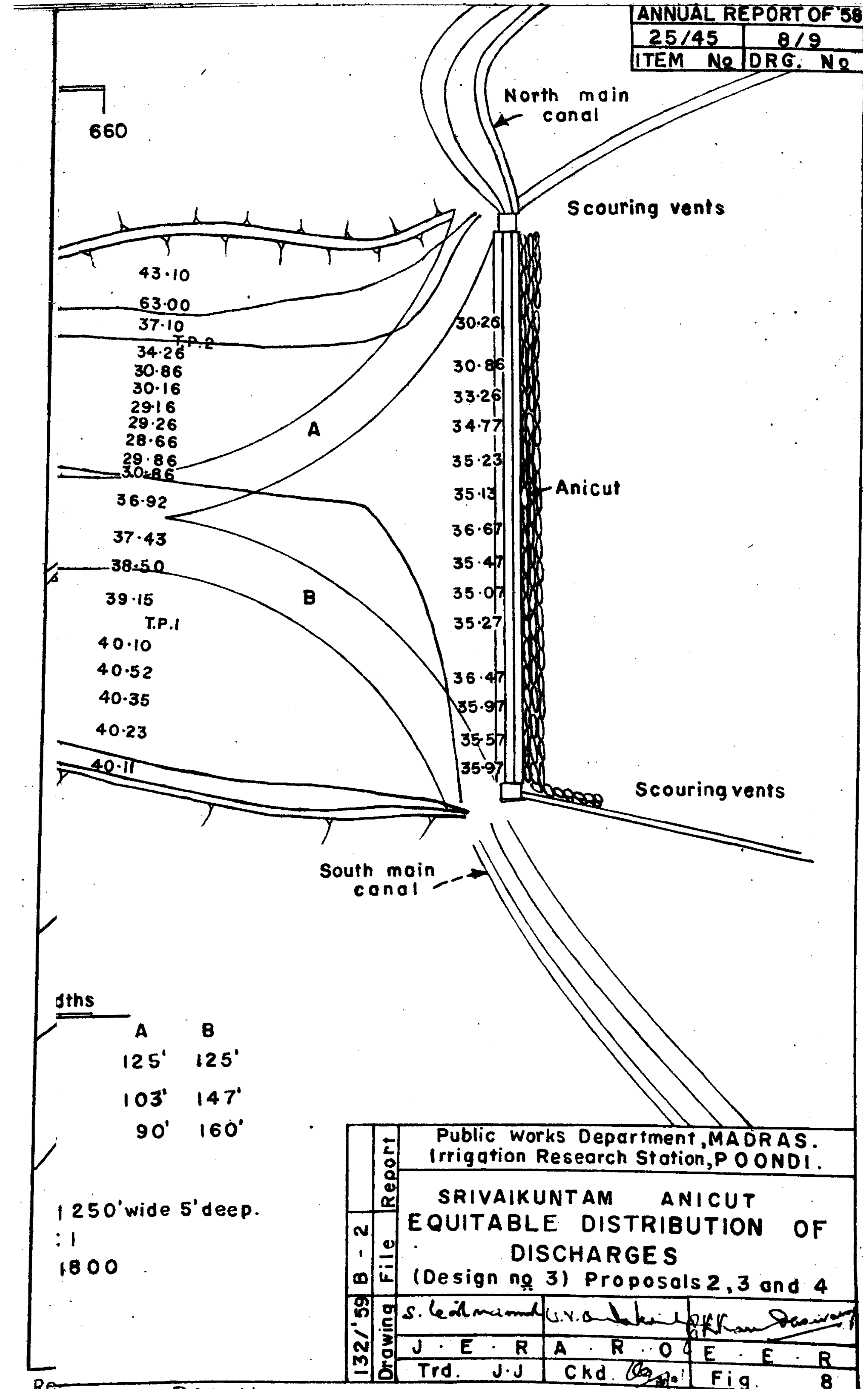
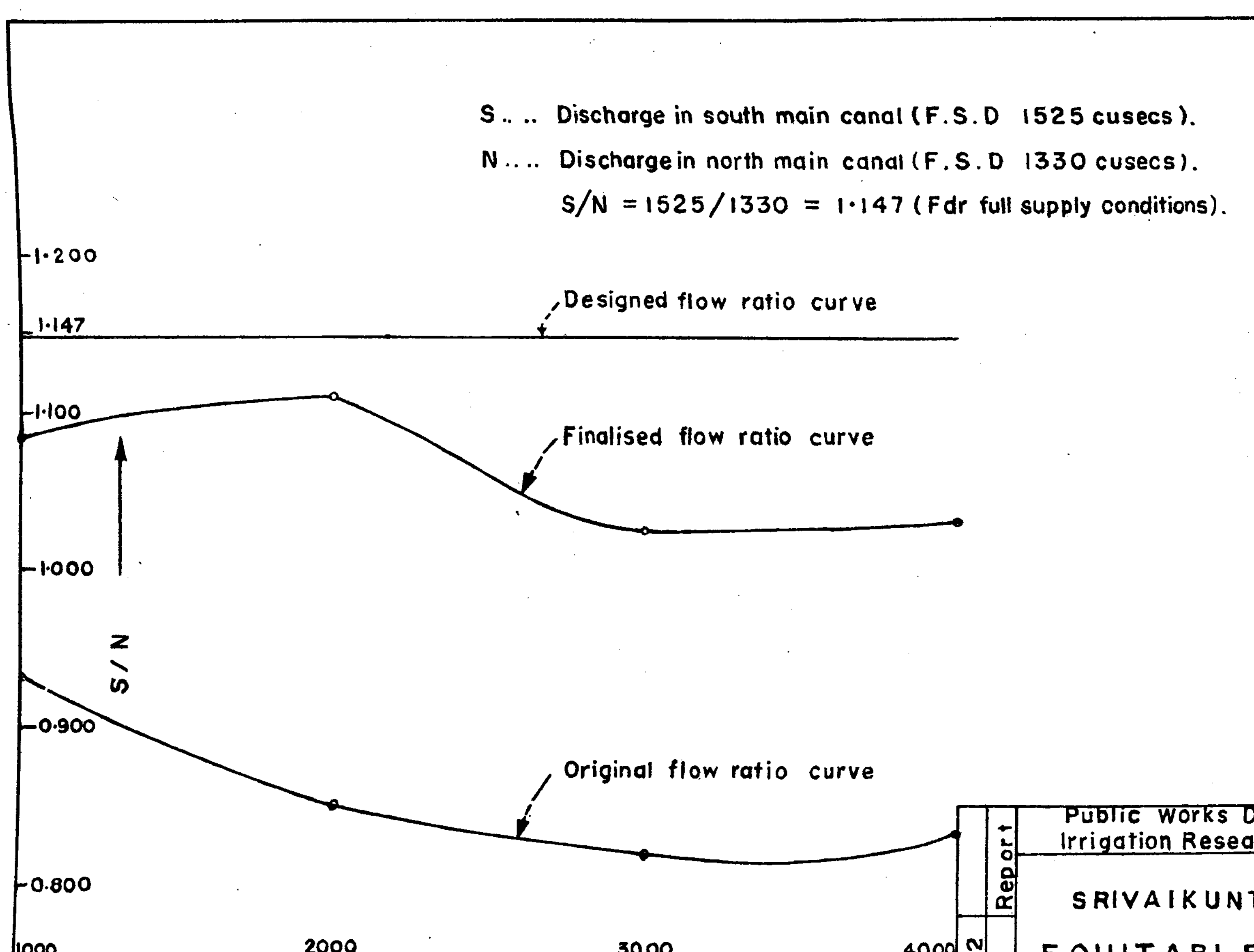

ANNUAL REPORT OF '58
25/45
8/9
ITEM No
DRG. No
660
North main canal
Scouring vents
43·10
63·00
37·10
T.P.2
34·26
30·86
30·16
29·16
29·26
28·66
29·86
30·86
36·92
37·43
38·50
39·15
T.P.I
40·10
40·52
40·35
40·23
40·11
A
B
30·26
30·86
33·26
34·77
35·23
35·13
36·67
35·47
35·07
35·27
36·47
35·97
35·57
35·97
Anicut
Scouring vents
South main canal
dths
A
B
125'
125'
103'
147'
90'
160'
I 250'wide 5'deep.
: I
I800
Public Works Department, MADRAS.
Irrigation Research Station, POONDI.
SRIVAIKUNTAM ANICUT
EQUITABLE DISTRIBUTION OF DISCHARGES
(Design no 3) Proposals 2,3 and 4
132/'59
B - 2
Drawing
File
Report
J · E · R
A · R · O
E · E · R
Trd. J·J
Ckd.
Fig. 8

Annual Report Of 1958	
25/45	9/9
ITEM No	DRG No



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

SRIVAIKUNTAM ANICUT

EQUITABLE DISCHARGE

Report			
File B – 2			
Drawing 133/59	S.	U.V	
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd.	Fig. 9

PALAR BARRAGE SILT EXCLUSION STUDIES

(C.B.I. & P. No.)

Introduction :

The existing anicut across River Palar was proposed to be converted into a barrage. Sediment exclusion studies were conducted to exclude coarse bed load from the four canals taking off above the proposed barrage. As a result of studies conducted in the model, a device consisting of a divide wall, divert wall and a breast wall was finalised and this was reported in the previous annual report of this Station. This report deals with the further experiments conducted for arriving at the optimum dimensions of the divide and divert walls.

Experimental Set-up :

A river model of Palar constructed to a scale of 1 : 216 horizontal and 1 : 36 vertical was utilised for the studies. Experiments were conducted with 17,040 cusecs in the model equivalent to 1,70,040 cusecs in the prototype with a rate of sand injection of 190 lbs./hour. Sediment collection tests alone were conducted.

Best Length of Divide Wall :

Keeping the divert walls on the left and right sides 500 feet and 400 feet long respectively the length of the divide wall was progressively decreased from 250 feet to 100 feet (vide Figure 1). Each trial was run for a total number of 72 hours and in each case the percentage reduction of silt entering in all canals based on the silt entry for the basic run was found out. The results are that the divide walls of 200 feet length each are best suited for the purpose (vide Sketch 1).

Best Length of Divert Walls :

The top of the divert wall would act as silt excluding head during maximum floods. Experiments were conducted for low discharges in the river such as 50% and 30% of the maximum flood discharge. Various lengths of divert walls were tried both on left and right sides of the river, near the proposed barrage. Results of the model studies conducted to that effect are furnished in Statement II. It was concluded that 400 feet long divert wall one on either side will give the best performance.

Conclusion :

The optimum length of the divert walls and divide walls on the both sides of the proposed barrage for the maximum exclusion of silt entering into the canals are suggested.

PALAR BARRAGE SILT EXCLUSION STUDIES.

STATEMENT I.

Results of sediment collection tests conducted to arrive at the best length of divide wall.

Discharge ... ... 17,040 c.fs. (Prototype).
Rate of injection of sand charge ... ... 190 lbs./hour.

No.	Length of divide wall left/right.	Particulars.		Name of canal.			Total.
			Mahendra-vadi.	Kaver-pauk.	Sakkara-mallur.	Dusi.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	No device	Average drawal	4.64	19.48	1.24	7.09	32.45
2	250'/230'	Average drawal	1.70	3.77	0.05	0.50	6.02
		Percent reduction	0.74	81	96	93	81.4
3	200'/200'	Average drawal	2.30	3.22	0.06	0.65	6.23
		Per cent reduction	50	84	95	91	80.8
4	150'/150'	Average drawal	1.98	7.22	0.19	1.48	10.87
		Per cent reduction	57	63	85	79	66
5	100'/100'	Average drawal	3.40	9.48	0.28	1.57	14.73
		Percent reduction	27	51	77	78	55

Note.—(1) Average Drawal—Percentage of sand drawn per hour averaged over a run of 72 hours. Percentage being over total injected sand.

(2) Per cent reduction is over the average drawal of study 1 wiht no device.

STATEMENT II.

Results of sediment collection tests conducted to arrive at the best length of divert walls.

Discharge ... ... 51,120 c.ft. (Prototype).
Rate of injection of sand charge ... ... 255 lbs./hour.

No.	Length of divert wall left side.	Particulars.		Name of canal.			Total.
			Mahendra-vadi.	Kaveri-pauk.	Sakkara-mallur.	Dusi.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	No device	Average drawal	1.19	3.25	0.06	1.09	5.59
2	500'/400'	* Average drawal	0.47	0.88	0.04	0.39	1.78
		† Per cent reduction	60	73	33	64	68
3	400'/400'	Average drawal	0.79	0.69	0.01	0.42	1.91
		Per cent reduction	34	79	84	62	66
4	300'/300'	Average drawal	0.55	1.24	0.14	0.76	269
		Per cent reduction	54	62	—133	30	52

* Average drawal—Vide Note under Statement I.
† Per cent reduction—Vide Note under Statement I.

PULLAMBADI CANAL SCHEME—OFFTAKE PROPOSALS TO AVOID ENTRY OF COARSE BED LOAD

(C.B.I. No. 27.)

1. Introduction :

1.1. River Aiyar, tributary to River Cauvery rises in Killi Hills and joins the parent river just above Upper Anicut after traversing a distance of 33 miles. There are 14 anicuts across this river for irrigation purposes, the last one, Neiveli Anicut being located at M. 26 F.5 from the source.

1.2. The offtake of the leading channel of Pullambadi Main Canal is about 4 miles below the Neiveli Anicut on the left bank of River Aiyar just before its confluence with River Cauvery. There is no other structure nearby across this river which is likely to affect the functioning of the present scheme. Just below this offtake point there is an existing channel, Perumalai channel with a maximum discharge of 500 cusecs taking off from the same river.

1.3. The average bed level of the River Aiyar opposite the offtake point is El. 236.25. As the canal bed level was at El. 230.75 at start which was 800 feet from the offtake point it was considered that a lot of course bed load will enter the channel due to the steep fall. Hence, to control sand entry three proposals were formulated. They are given below :—

Proposal I :

(*a*) Constructing a drop at the offtake point with the top at El. 237.00,

(*b*) Excavating a leading channel of 52' × 9.5' with pucca flood banks on either side, and

(*c*) Constructing a head sluice of 4 vents of 10' × 9.5' with sill at El. 230.75.

Proposal II :

(*a*) Excavating a leading channel of 120' × 4.25'.

(*b*) Constructing a combined head sluice and drop consisting of 10 vents of 10' × 4.32' with sill at El. 236.18 and bed in rear at El. 230.75.

Proposal III :

(*a*) Excavating a leading channel of 100' × 4.25'.

(*b*) Constructing a head sluice of 9 vents of 10' × 4.32' with sill at El. 236.18.

(*c*) Widening the rear channel to 118' × 4.25' from 33' × 9.5'.

(*d*) Constructing a drop 5.43' high at C/4 of Main Canal.

For all the three proposals the head sluice was proposed to be located 800 feet from the offtake point.

2. Problem :

This Station was requested to conduct hydraulic model studies to suggest the best proposal from the silting point of view.

3. Model :

A river model to horizontal scale of 1 : 100 and vertical scale of 1 : 40 was constructed. The flood bank on the left, the leading canal and the Pullambadi Canal were all constructed in masonry and finished smooth. Cross section templates were prepared with 1/4" rods fixed on to teakwood reepers. The head sluice for all the three proposals and the drop were prepared in teakwood. Measurement of discharge was made over right angled V notches at head and in rear of canal. Other stilling and sand collecting arrangements were made as usual. Sand completely passing through 40 mesh sieve was used as a river bed material (vide Figure 1).

4. Experiments and Discussion :

4.1. *Proving the model.*— First, experiments were conducted to find out the regime rate of sand injection and to find whether the river bed contours would be maintained for this rate of sand injection. A suitable discharge of 1,620 cusecs in the River Aliyar was chosen for purposes of experimentation. Without any control of rear water level the water level in the river opposite the outake was read to be El. 232.15. As this was higher than the minimum water level for which the head sluice had been designed, viz., El. 240.50 it was considered suitable. Next sand injection rate was arrived at. After a few trials and some notifications to fix the flow along left margin the rate was arrived at, to be 30 lbs./hour. But it was observed that the deep bed of the river got silted up and the berms scoured out though about the same quantity of injected sand was ejected out by the model. This was found to be due to the following reasons. The deep channel was with steep sides at site itself. In the model vertical exaggeration is given as a necessity to simulate curvature and allied phenomena so much so the sides became still steeper. The sand used in the model having an angle of repose less than the steepness, adjusted itself to give a slightly flatter section by silting in the centre and scouring at the berms. As the studies were to be only a comparative nature the matter was left at that end the experiments continued with the regime rate of sand injection of 30 lbs./hour. During this phase of proving the model the canal was kept closed.

4.2. *Experiments.*—As the problem was to suggest the best proposal from the silting point of view only sediment collection tests were conducted. The three proposals were constructed one after the other and after maintaining the water level in the river at El. 243.15, by operating the tail gate, the canal was made to draw the full supply discharge of 1,066 cusecs. The full supply depth was maintained in the canal by operating the tail gate in rear. The sand collected at hourly intervals were tabulated and the average hourly drawals over a continuous run of 16 hours was taken as the criterion for comparing the performance. The results for the three proposals are given in the tabular statement appended. It will clearly be seen that the hourly sand drawals are almost identical in all the three cases and that the average is about 90% of the injected sand, through the discharge in the canal is only 67% of the total discharge in Aliyar River.

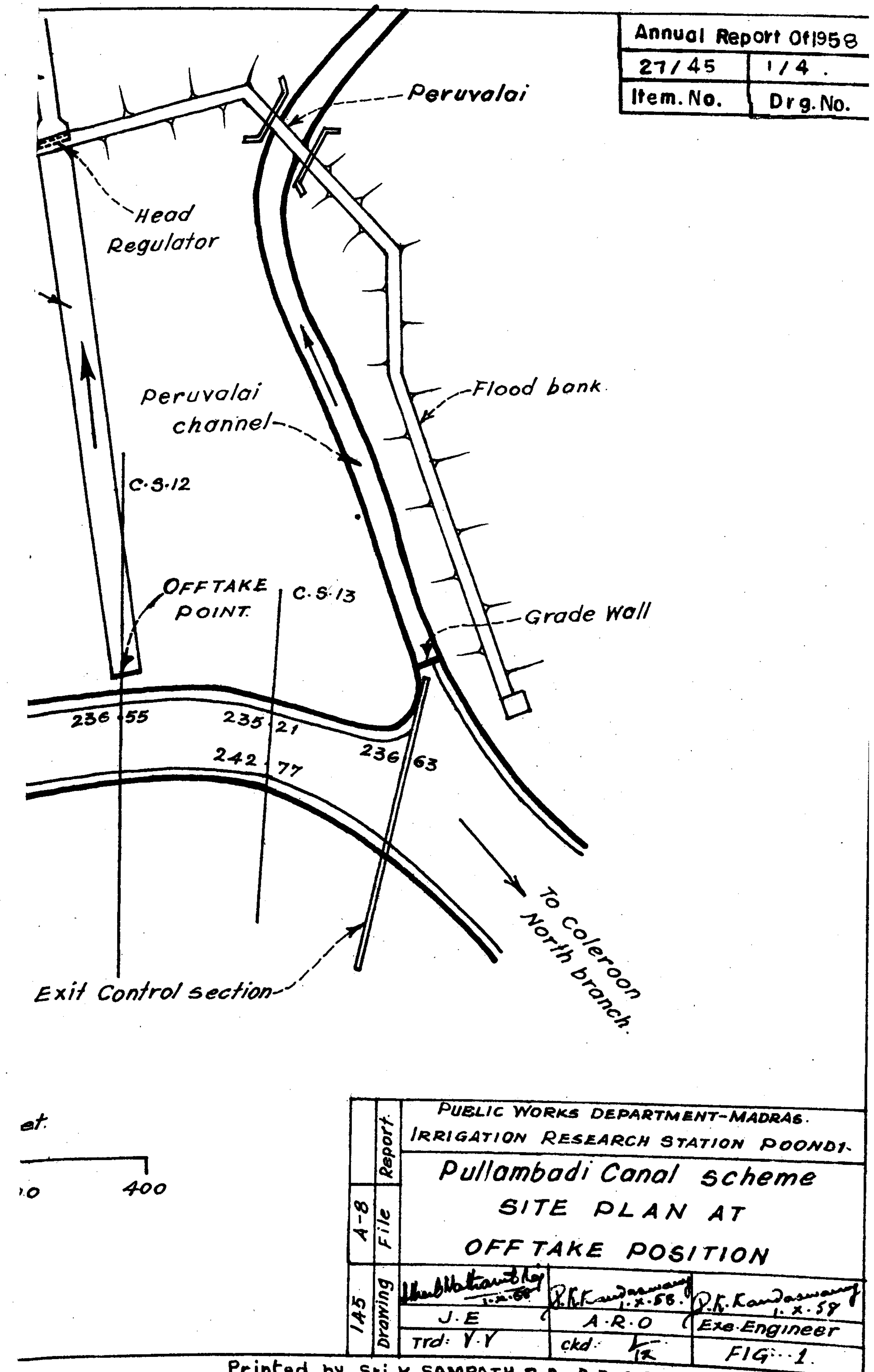
4.3. Some minor details at the offtake were modified to suit the conditions obtaining and they are given below :

4.3.1. For proposal 1 the leading channel of 52′ × 9.5′ was taken straight and 1 c.ft. with a straight entry, with a drop of length 64 feet at top covering the whole width at entry. The canal being deflected through 100° from river axis, was not able to draw the full discharge for this entry width, By experiments it was found that a flared entry with the drop of length 205 feet will just provide means to draw the full supply discharge. Experiments were done with this flared entry only for this proposal (vide Figure 2).

4.3.2. For the second and third proposals the leading channel flared was of section 180′ × 4.25′ yet, the entry was made slightly flared, vide Figures 3 and 4, the outer flared portion being almost the same dimension as that for proposal I. Even with this flared entry it was observed that only about two-thirds the right side width of the leading channel was utilised, and in order to utilise the full width, the flare should have been much more. It was possible to draw the full supply discharge in the canal without any difficulty.

5. Conclusion :

5.1. The silt exclusion studies conducted to select the best of the three proposals mentioned in the preceding paragraphs have revealed that all the three proposals are more or less identical in so far as their silt drawing nature is concerned. This proves that the geometrical characteristics of the leading channel, drop and head sluice as well as their relative position do not play as much an effective part as the entry condition of the leading channel. So much so, the best proposal is that which is the most economical in the cost of construction. Based on this consideration a suitable selection may be made.



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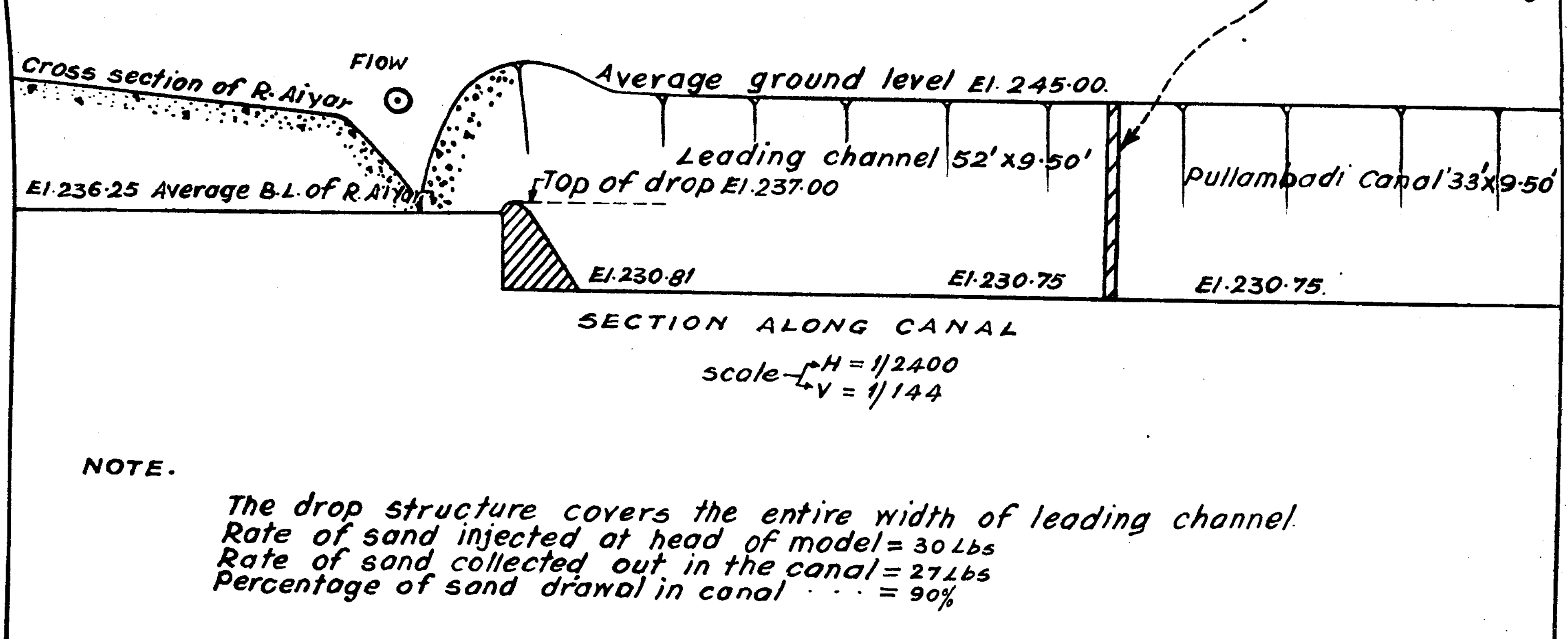
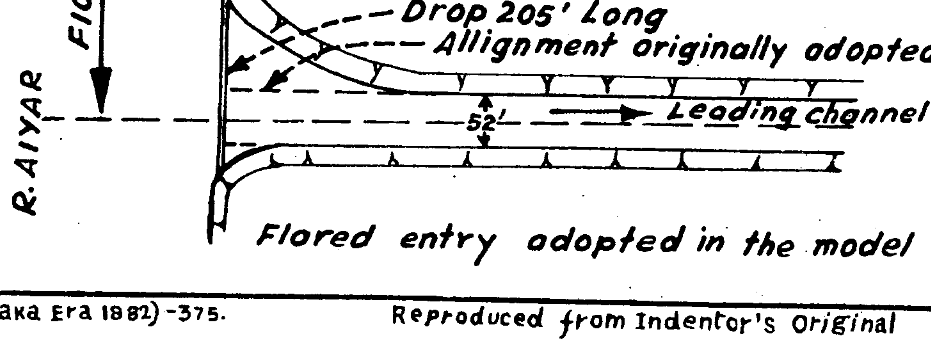


SECTION ALONG CANAL

scale H = 1/2400, V = 1/144

NOTE.

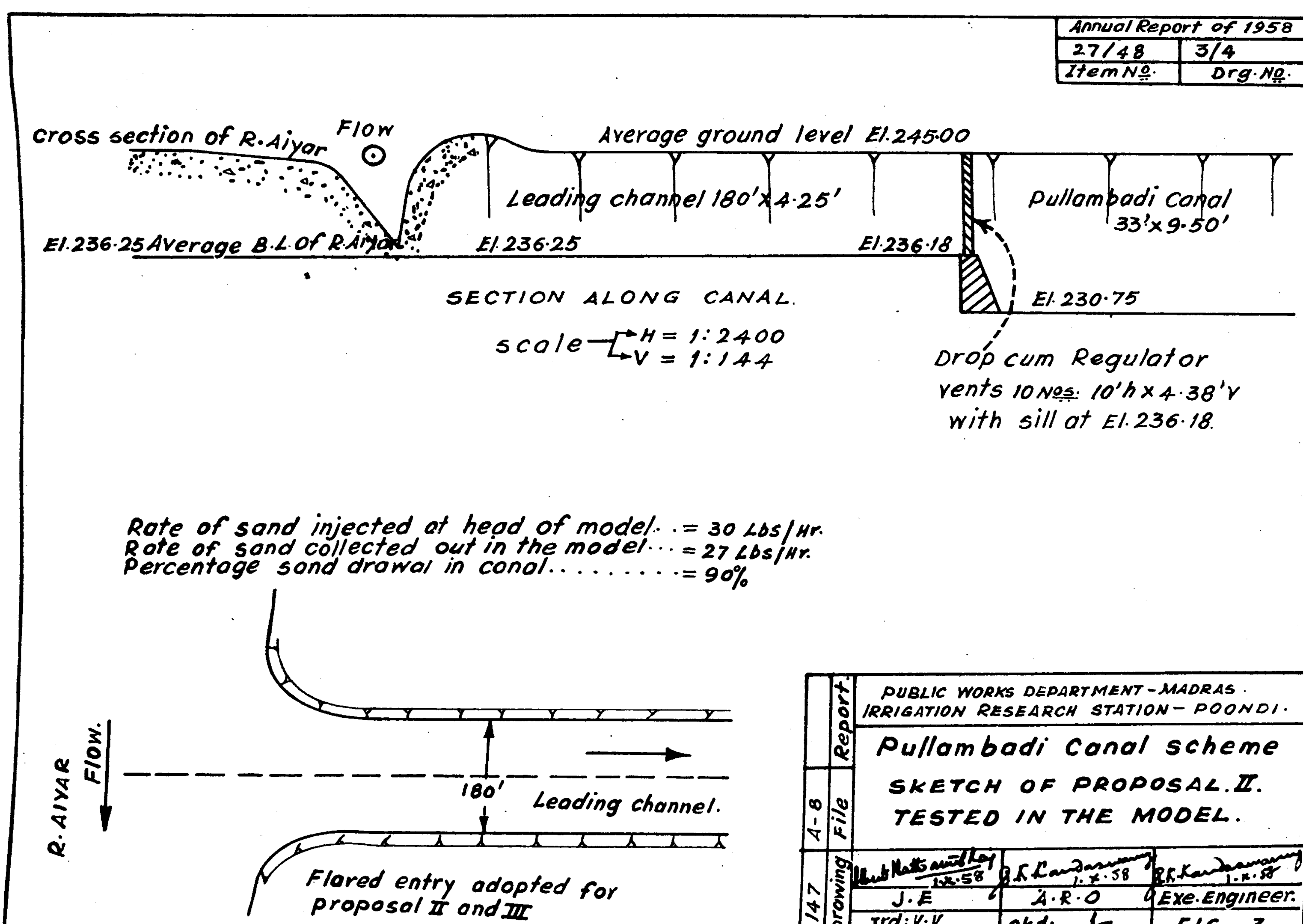
The drop structure covers the entire width of leading channel.
Rate of sand injected at head of model = 30 Lbs
Rate of sand collected out in the canal = 27 Lbs
Percentage of sand drawal in canal . . . = 90%



Flared entry adopted in the model

Report		PUBLIC WORKS DEPARTMENT-MADRAS. IRRIGATION RESEARCH STATION-POONDI.
File	A-8	Pullambadi Canal scheme SKETCH OF PROPOSAL. I. TESTED IN THE MODEL.
Drawing	146	J.E / A.R.O / Exe. Engineer Trd: V.V / Ckd: / FIG... 2.

Reg. No. 45 E'61. (Saka Era 1882) - 375. Reproduced from Indentor's Original. Printed by Sri K. SAMPATH. B.A., A.D., C.S.O., P.Z.P., Madras.



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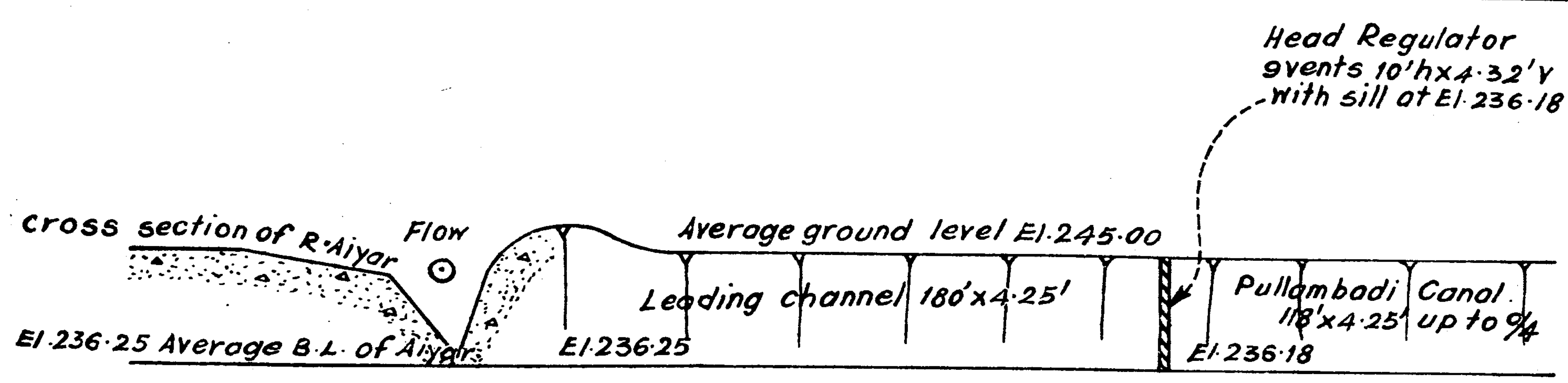
Annual Report of 1958	
27/48	4/4.
Item No.	Drg. No.



SECTION ALONG CANAL.

scale — H = 1:2400, V = 1:144

NOTE 1. Pullambadi Main canal will be of section 118' x 4·25' for the first four furlongs after which the section will be 33' x 9·5'

2. The drop of height 5·43' will be located at 0/4.

Rate of sand injected at the head of model = 30 Lbs
Rate of sand collected out in the canal = 25 Lbs
Percentage sand drawal in canal = 84%

Report.	PUBLIC WORKS DEPARTMENT - MADRAS. IRRIGATION RESEARCH STATION - POONDI.	
File 4-8	Pullambadi Canal scheme. SKETCH OF PROPOSAL III. TESTED IN THE MODEL	
Drawing 148	J.E. 1.x.58	A.R.O 1.x.58	Exe. Engineer 1.x.58
	Trd: V.V.	Ckd:	FIG...4.

Reg. No. 47 E'61. (Saka Era 1882) 375. Reproduced from Indentor's Original Printed by Sri. K. SAMPATH, B.A., R.D., C.S.O., P.Z.P., Madras.

NEW KATTALAI HIGH LEVEL SCHEME—OFFTAKE POSITION

1. Introduction :

1.1. *Necessity for the Project.*—The River Cauvery is the largest perennial river of Madras State, and had been utilised for development of irrigation to an almost complete extent through various schemes, of which the Mettur, the Grand Anicut and Kattalai High Level canal schemes deserve special mention. Even though the Cauvery flows through Tiruchchirappalli and Thanjavur Districts, the area under river Irrigation in Thanjavur District is very much larger than in Tiruchi District. At present irrigation in Tiruchi District is confined to the Vayyakonda channel and existing Kattalai High Level Canal executed in the year 1935. Observing the benefiits derived from these schemes, the ryots of Tiruchi District, were clamouring for the extension of the Kattalai High Level Canal to bring additional areas under irrigation. Thus the New Kattalai High Level Canal scheme, which is for utilising the surplus waters of the Cauvery after meeting the existing demands of irrigation below the Mettur Dam, came up for consideration.

1.2. *Kattalai High Level Canal.*—There is an existing bed regulator across Cauvery near Mayanoor in Tiruchi District which goes by the name Kattalai High Bed Regulator. A combined South Bank and Kattalai High Level Canal take off from the right side and water from the river is led through a leading channel 850 feet long. Kattalai High Level Canal branches off from the combined channel at mile 0/5 (vide Figure 1). To reduce silt entry into the canal a silt trap had also been constructed in front of the leading channel to the combined canal. The silt trap as existing is at an inclination of 65° to the alignment of the leading channel and ends in a scour vent of 2 Nos. 20′ × 8.5′ in line with the regulator. There is also a small irrigation channel named Krishnarajapuram channel with a maximum discharge of 40 cusecs. The site plan is shown in Figure 2 and typical sections of the Kattalai Bed Regulator are shown in Figure 3. The details of silt trap are shown in Figure 4.

1.3. *Proposals for the Scheme.*—Various proposals ranging from an entirely new canal taking off a few miles upstream of the Kattalai Bed Regulator to remodelling of the existing High Level Canal were considered. But all the alternatives were essentially direct irrigation schemes and as much supplies could be counted upon only in favourable years. Therefore the scheme was modified as a tank irrigation scheme, with the New Kattalai High Level Canal functioning as the main supply channel for filling the existing tanks during the periods when Mettur is surplusing, and with some direct irrigation in years of favourable storage at Mettur.

1.4. *Main features of the Scheme.*—The scheme comprises of a new high level canal which is divided into three reaches. In the first reach of 40 miles the new canal will run parallel and close to the existing Kattalai High Level Canal and there is no irrigation in this reach (vide Figure 1). The second reach will be 24 miles long and areas of 5,658 acres under tanks and 8,622 acres directly will be irrigated. The third reach will irrigate 6,342 acres under tanks. This canal will have a head regulator just by the side of the head regulator for the existing combined South Bank and Kattalai High Level Canal, and water from the river Cauvery will be led through a leading channel 920 feet long with its offtake at the upstream portion of the silt trap. The Krishnarajapuram channel will be provided with a new leading channel parallel to that of the South Bank Canal and new regulator beside the latter.

2. **Problem** :

The problem referred to the Station was to examine the proposed offtake arrangements for the new Kattalai High Level Canal.

3. **Data** :

Hydraulic data relevant to the problem are given below :

Kattalai Bed Regulator—

Average river bed level	El. 318.50.
Length of regulator	4,225 feet.
Crest level of regulator (ch. 0—865) ...	El. 322.95.
(Ch. 0 is left end of regulator) (ch. 865-4,225).	El. 320.95.
Height of falling shutters	2 feet.
Length for which falling shutter is fixed.	3,360 feet (from ch. 865-4,225).

Combined South Bank and Kattalai High Level Canal—

Bed level of leading channel	El. 312.00.
Sill level of Head regulator	El. 315.00.
No. and size of vents	4 Nos. ; 19′ H × 4′ 6″ V.
Bed width of the canal	78 feet.
Full supply depth	5 feet.
Full supply discharge	1,380 cusecs.
Bed fall	1 in 2,640.

Krishnarajapuram Channel—

Sill level of regulator	El. 320.30.
No. and size of vent	2 Nos. ; 4′ H × 3.35′ V.
Full supply depth	7 feet.
Full supply discharge	40 cusecs.

New Kattalai High Level Canal—

Sill level of Head regulator	El. 313.70.
No. and size of vents	4 Nos. ; 9′ H × 6′ V.
Bed width of canal	36′ 6″.
Full supply depth	9′ 0″.

Full supply discharge	1,064 cusecs.
Bed fall	1 in 6,600.

Silt trap and scour vent—

Bed level of trap	El. 312.00.
Top level of trap	El. 318.00 Upstream. El. 317.25 Downstream.
Width of trap	81 feet.
Length of trap	358 feet.
Sill level of Scour vent	El. 312.0.
No. and size of vents	3 Nos. ; 20′ H × 8.5′ V.

Note.—All elevations are local Thanjavur levels.

4. **Models** :

4.1. Studies were conducted in two models ; a full width model to a scale of 1 : 300 H and 1 : 50 V with rigid bed and the other a partwidth model to a scale of 1 : 120 H and 1 : 24 V with mobile bed.

4.2. *Full width model.*—The river Cauvery was simulated to a distance of about 9,000 feet upstream and 2,000 feet downstream. The flood banks were constructed in masonry and finished smooth. River cross sections and bed were also plastered and finished smooth. The Kattalai Bed Regulator silt trap, scour vent and the canal head regulators were prepared in wood and fixed. G. I. shutters were provided for the scour vent and canal regulators. The leading channels and the canal head regulators were prepared in wood and fixed. G. I. shutters were provided for the scour vent and canal regulators. The leading channels and the canals were simulated in masonry. Measuring V notches and automatic tail gate arrangements were made in rear of the canals. V notch was utilised to measure model discharge. One deviation was that the Krishnarajapuram channel was not reproduced but its discharge of 40 cusecs was added on to the South Bank Canal discharge and experiments conducted.

4.3. *Partwidth model.*—Cauvery river was simulated to a distance of 3,000 feet upstream and bed regulator for a distance of 600 feet from the scour vent. Flood bank on the right, flow line bund on the left, leading channels and canals were all constructed in masonry and finished smooth. The bed regulator, silt trap, scour vent and canal head regulators were all prepared in teak wood. Still galvanized iron shutters were provided for the scour vent and canal head regulators. Cross section templates constructed in masonry and 1 foot length at inlet was made up of medium sized pebbles. The bed material was sand passing through 40 mesh sieve. In this model also the Krishnarajapuram channel was not simulated but its discharge was added on to the South Bank Canal discharge as the discharge is not measurable in the model. The canals were simulated for a length of about 6 feet in the model. Measuring V notches and automatic tail gates were incorporated in rear of the canals. The model discharge was measured at V notch at the head. Special inlet weir arrangements had to be made due to the difference in level between leading flume and model. Platform made up of M. S. angles was also prepared for housing workers to inject sand charge.

5. Experiments and Discussion :

5.1. *Experiments in full width model* :—5.1.1. A discharge equivalent to a head of overflow of 4.6 feet over Kattalai Bed Regulator, which was 65,796 cusecs was chosen for purpose of experimantation. Discharge scale ratio of 1 : H V 1.5, i.e., 1 : 10,670 was adopted as it was a rigid bed model. Discharge equivalent to 65,796 cusecs was allowed in the river. The falling shutters were removed. The canals were made to draw their respective full supply discharges. It was observed that laminar flow persisted throughout the model due to the smoothness of the surface First the inlet section and then relevant areas of the model were roughened locally. For roughening the smooth top plaster was removed and was replaced with mortar of very coarse sand finished rough. This roughening was done progressively (vide Figure 2). Finally, when turbulant flow was observed throughout the model experiments were started. D square grid was painted at 100′ (prototype) intervals. Flow lines were observed with a weighted wax ball and the flow lines could be defined only beyond 600 feet from the scour vent and within that distance, i.e., in front of silt traps a still pool was observed with a low return flow (vide Figure 5). With the help of these flow lines the width of the partwidth model and its horizontal scale were fixed. In the partwidth model Kattalai Bed Regulator was simulated to a distance of 900 feet from the scour vent.

5.1.2. After fixing the relevant length of the Kattalai Bed Regulator in the partwidth model the flow line for the boundary alone was traced downstream of the regulator also and the flow on the two sides were physically separated by a galvanized iron sheet. The discharge passing in the partwidth was actually measured in this model with a V notch set in rear and the corresponding prototype discharge to be adopted for the partwidth model was arrived as to be 30,484 cusecs.

5.2. *Experiments in the partwidth model* :—5.2.1. Experiments were done to examine whether the position of the head regulator along the leading channel would have any effect on sediment drawal. For the first experiment the regulator for the New Kattalai High Level Canal was placed at the proposed section, i.e., by the side of the existing regulator of the South Bank Canal. Trial run was made to arrive at the correct rate of injection of sand. Discharge scale of 120^2 was adopted. Model discharge equivalent to 30,484 cusecs was sent in the model. In this case also a discharge of 1,420 cusecs was allowed in the South Bank Canal as a sum of its discharge and that of Krishnarajapuram channel. Thus the two canals simulated in the model were made to draw their respective full supply discharges of 1,420 cusecs and 1,054 cusecs. The rest was allowed to flow over the Kattalai Bed Regulator. The head of overflow was checked and as it fluctuated only by a small amount of 0.1 foot, experiments were started.

5.2.2. Various rate of sand injection was tried. Shoaling and scouring in the model was observed and a rate of injection of 3/4 c.ft. or 63 lbs./hour was arrived at. After arriving at the rate the sand drawal per hour was very little and hence drawal per day was taken as basis. The model was run for six days and drawal of sand per day averaged over the run of six days was taken as basis for comparison.

5.2.3. The head regulator of New Kattalai High Level Canal alone was shifted 525 feet upstream and the experiment repeated. For the third experiment the regulator was shifted 475 feet further upstream and the sand drawal was found out as before (vide Figure 6).

5.2.4. The results of the sediment collection tests are given in the statement accompanying. From that it can readily be seen that the position of the regulator along the leading channel does not have much effect on the sediment drawal of the canal.

5.3.1. As for the other particulars of offtake it is considered desirable to point to standard information available and a note is given below accordingly. The additional informations are from Buckley's Irrigation Pocket Book.

5.3.2. *Sill level of offtake.*—" Sill of the Head Regulator should be kept as high as possible ". The idea is based on the fact that higher the sill lesser will be the intake of sediment. But in the case under consideration the left side wall of silt trap is at El. 312.00 and that of New Kattalai High Level Canal Regulator at El. 313.70. As the silt trap covers the mouth completely there is obviously no advantage of raising the sill upto El. 318.00. If at all the sill is to be raised it should be raised above El. 318.00 to derive benefits by it.

5.3.3. *Waterway of regulator.*—" Ample waterway should be allowed in the regulator so that only top layer is taken into the canal and the velocity of intake is reduced ". The ventway provided for the combined South Bank and Kattalai High Level Canal is 343 sq. feet for 1,380 cusecs whereas, the ventway provided for the new Kattalai High Level Canal is only 216 sq. feet for 1,064 cusecs which falls short of the proportional area by 50 sq. feet. In view of the fact that the former had silted the short fall of ventway can be considered to indicate that the ventway is not at least ample. This point is worthy of further consideration by field officers.

5.3.4. (*a*) Head regulator should be kept closed during heavy floods in the river which make it necessary to open all the scour vents.

(*b*) To keep the scour vents closed as much as possible so as to allow of still water in front whereby deposition of silt outside canal head may be encouraged.

6. Conclusion :

6.1. As a result of studies and from standard information the following conclusions are arrived at while examining the offtake position for the New Kattalai High Level Canal :—

(1) The position of the regulator along the leading channel has no appreciable effect on its sediment drawal.

(2) The sill level should be as high as possible. But there is no possibility of any advantage in raising the sill if it is not done to an elevation above El. 318.00.

(3) The waterway allowed appears to be insufficient and this is pointed out for consideration by field officers.

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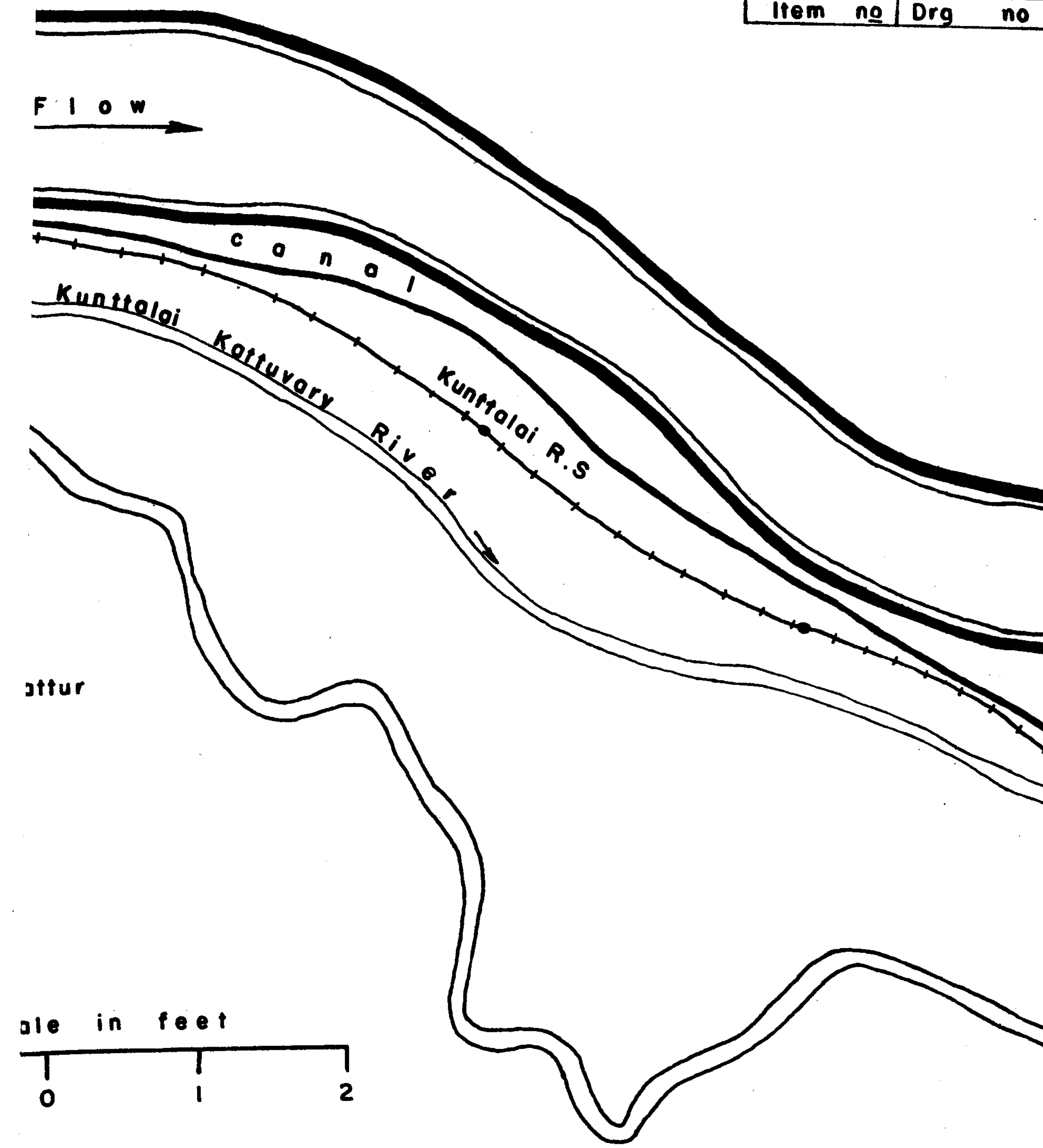

A-7 19/58 File Report

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

NEW KATTALAI HIGH LEVEL CANAL
OFF_TAKE POSITION
LOCATION MAP

93/58 Drawing

Albert N. Raj	M. Balasubrmaniam	
J·E·R	A·R·O	E·E·R
Trd. J·J	Ckd.	Fig. 1

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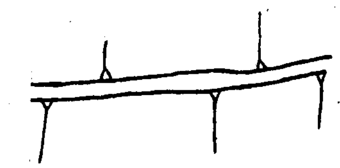


Shows surface of model roughened for creating turbulent flow.

HYDRAULIC PARTICULARS.

Kattalai Bed Regulator

Crest	.. El. 320·95
Height of falling shutter	.. 2 feet.
Length	.. 4225 feet.

Silt Trap

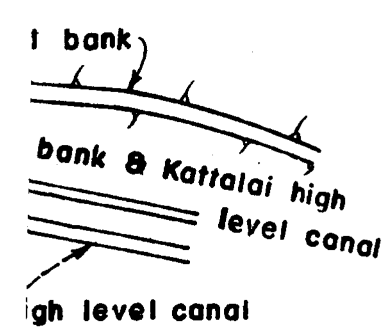
Bed level	.. El. 312·00
Width	.. 81 feet
Length	.. 356 feet

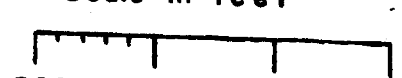
Combined South Bank And Kattalai High Level Canal

Sill level..	.. El. 315·00
Bed width	.. 78 feet.
F.S. Depth	.. 5'– 0"
F.S. Discharge ..	.. 1380 cusecs.

New High Level Canal

Sill level	.. El. 313·70
Bed width	.. 36 feet.
F.S. Depth	.. 9'– 0"
F.S. Discharge.. ..	.. 1064 cusecs.





A – 7 19/58 File Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
	NEW KATTALAI HIGH LEVEL CANAL OFF-TAKE POSITION GENERAL PLAN AND DETAILS OF FULL WIDTH RIGID MODEL		
Drawing	Albert N. Raj	M. Balasubramaniam	
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd.	Fig. 2

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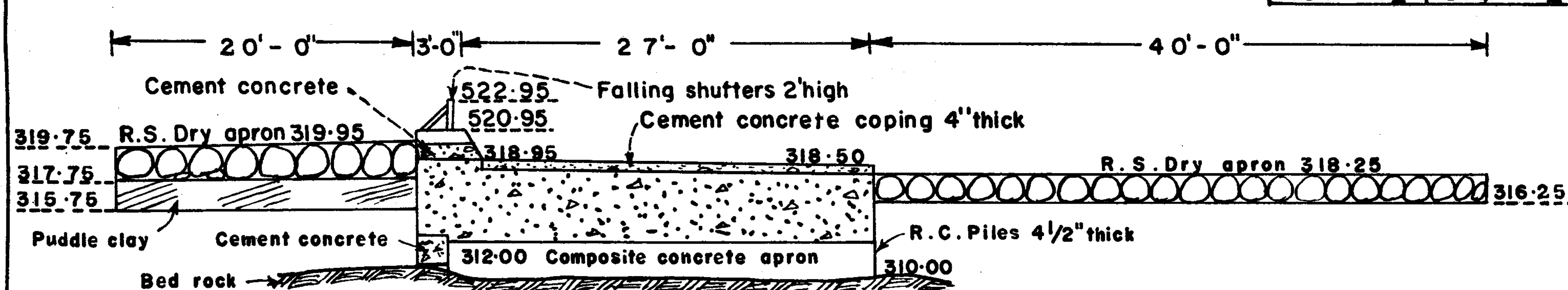
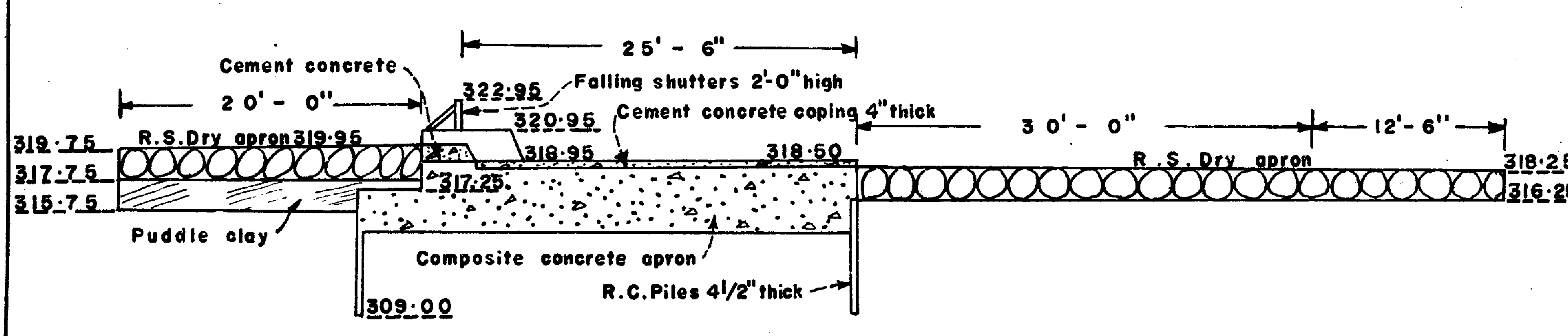


C. S. AT 230



C. S. AT 500

NOTE :-

The levels noted are Tanjore levels.

To get G.T.S. add 3·75

Scale in feet

Report 19/58	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File A - 7	NEW KATTALAI HIGH LEVEL CANAL OFF-TAKE POSITION TYPICAL SECTION OF KATTALAI BED REGULATOR		
Drawing	Albert N·Raj	M Balasubramaniyam	[signature]
	J. E. R	A. R. O	E. E. R
	Trd. J·J	Ckd.	Fig. 3

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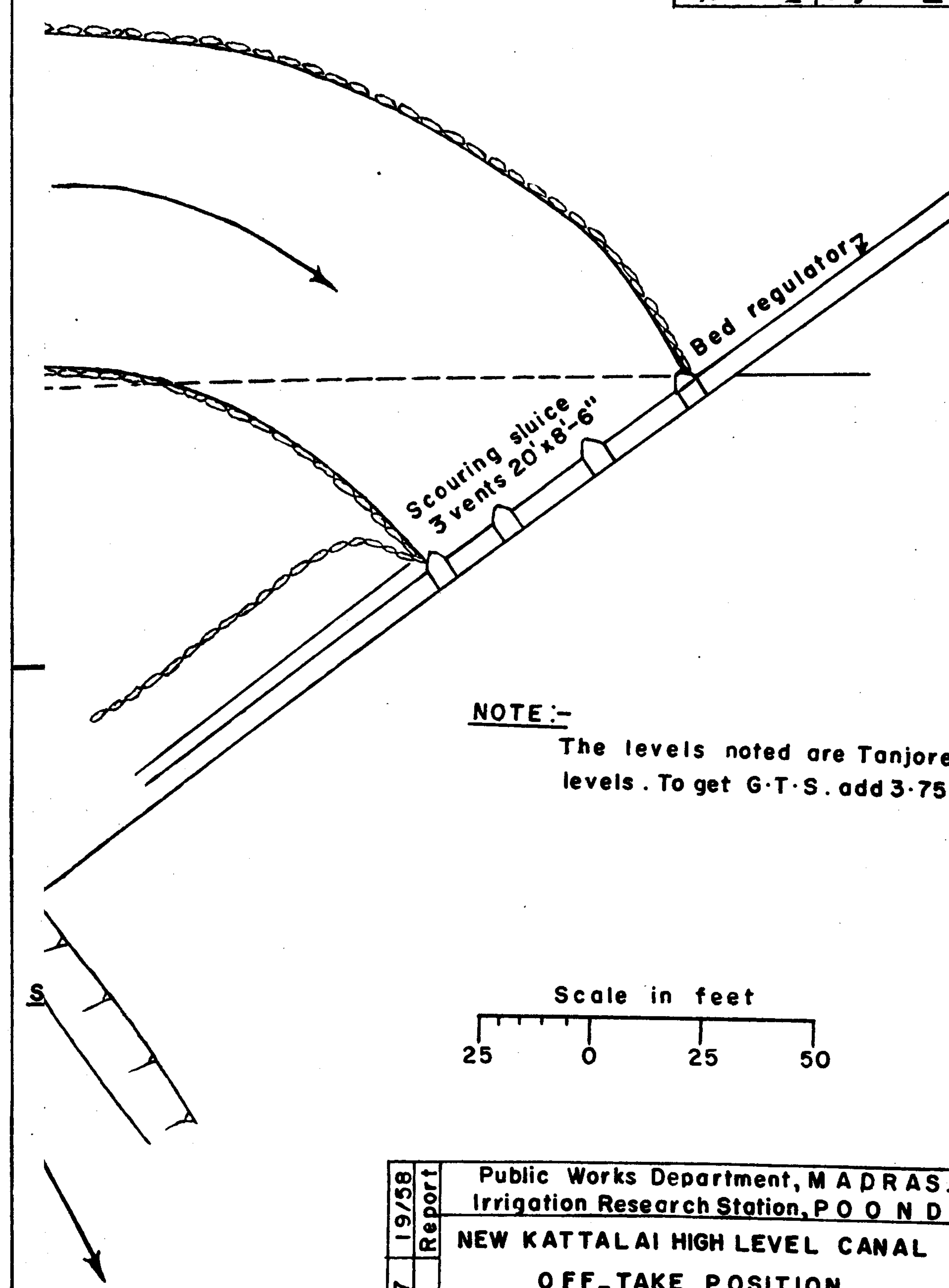

NOTE:-
The levels noted are Tanjore levels. To get G·T·S. add 3·75

Scale in feet

25 0 25 50

A-7 File 19/58 Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.
	NEW KATTALAI HIGH LEVEL CANAL OFF_TAKE POSITION DETAILS OF SILT TRAP

Drawing	Albert N·Raj	M·Balasubramaniam	[signature]
	J·E·R	A·R·O	E·E·R
	Trd. J·J	Ckd.	Fig. 4

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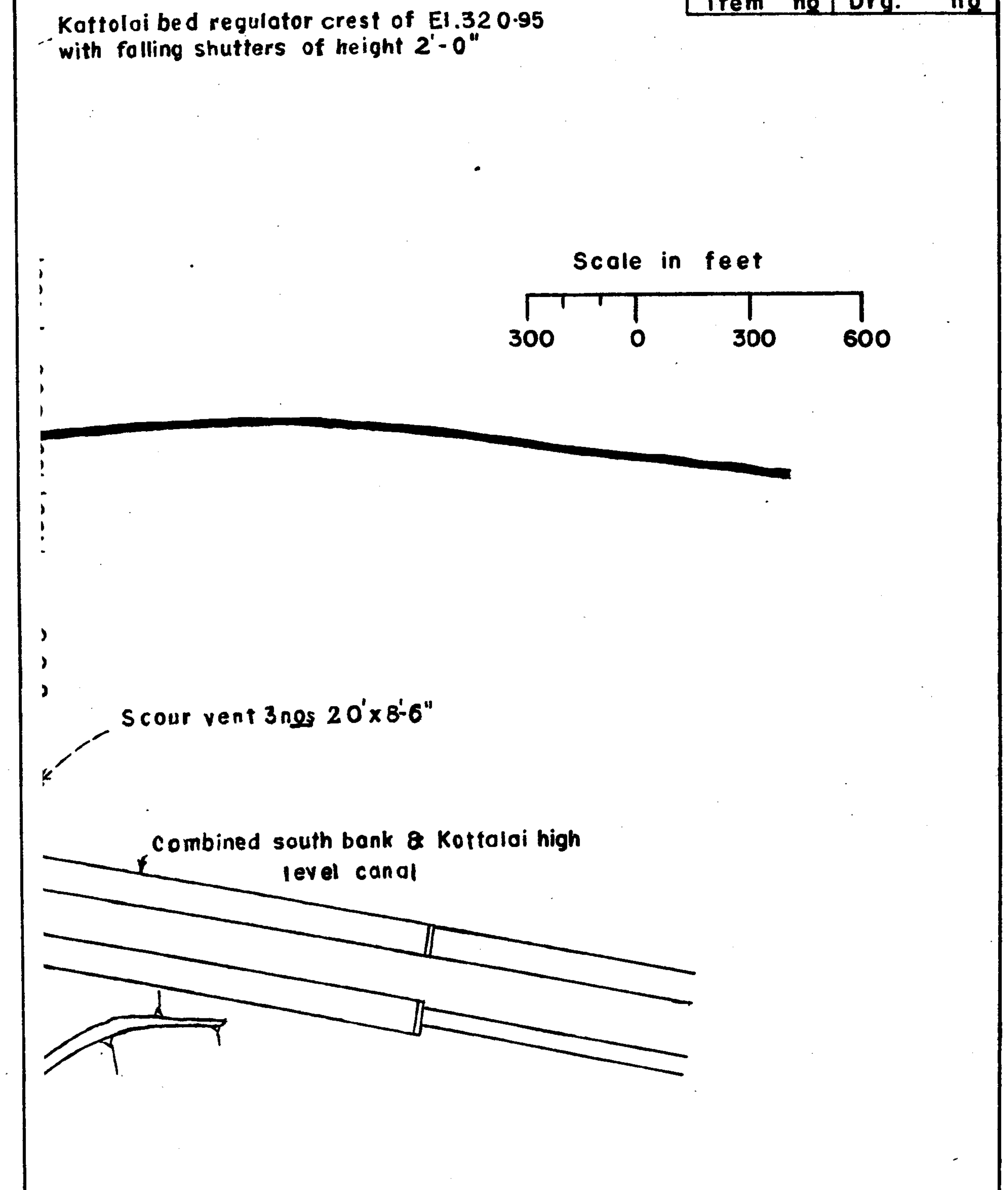
Item no	Drg. no



A-7 19/58 File Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.
	NEW KATTALAI HIGH LEVEL CANAL OFF-TAKE POSITION FLOW LINES OBSERVED IN THE FULL WIDTH RIGID BED MODEL

Drawing			
	Albert N Raj	N Balasubramaniam	[signature]
	J · E · R	A · R · O	E · E · R
	Trd J·J	Ckd.	Fig. 5

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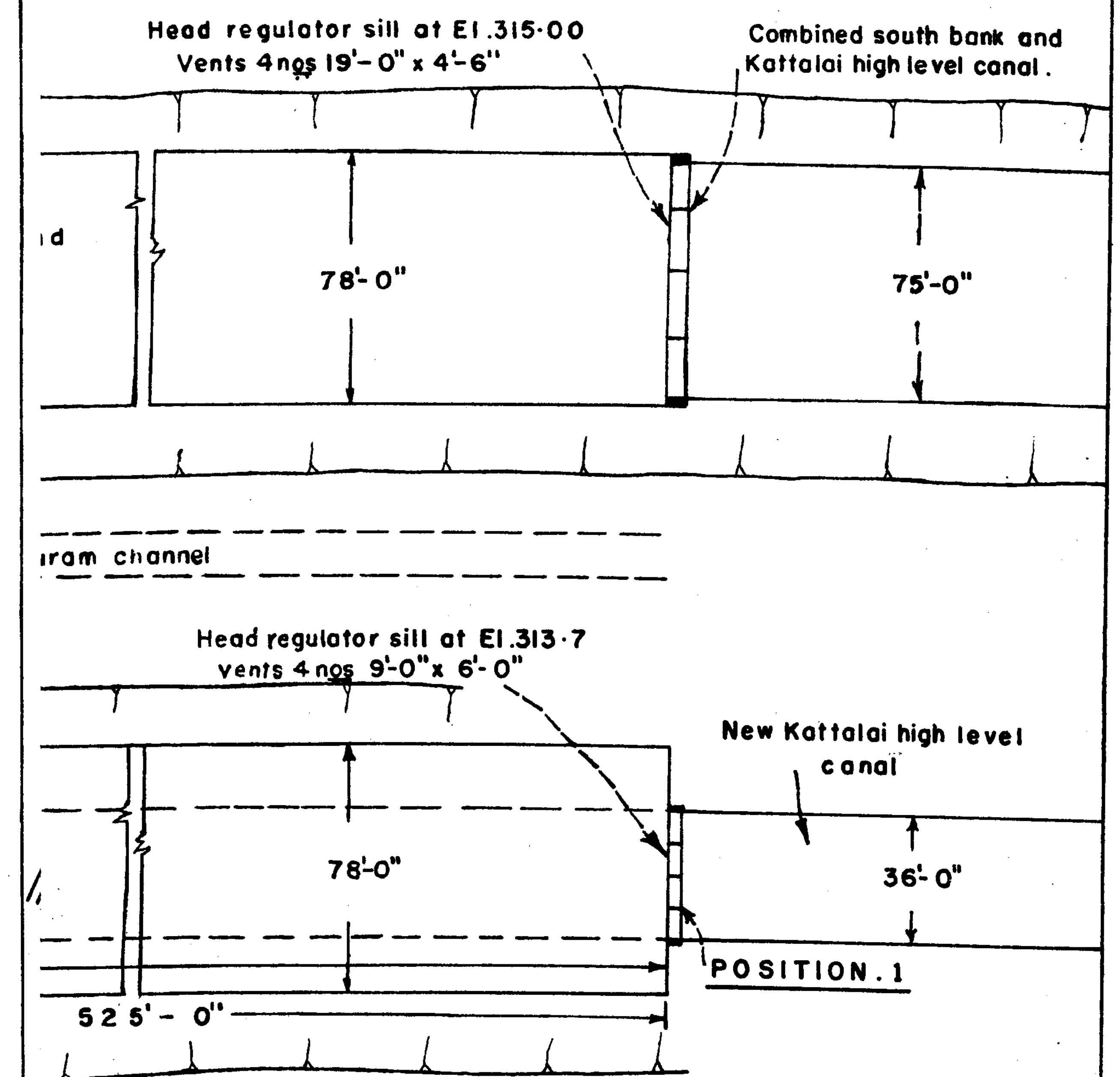

ale in feet

0 50 100

A-7 19/58
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Public Works Department, MADRAS.
Irrigation Research Station, POONDI.

NEW KATTALAI HIGH LEVEL CANAL
OFF-TAKE POSITION
DETAILS OF POSITION TESTED

Drawing	Albert N·Raj	M·Balasubramaniam	
	J·E·R	A·R·O	E·E·R
	Trd. J·J	Ckd.	Fig. 6

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NEW KATTALAI HIGH LEVEL CANAL SCHEME—SEDIMENT EXCLUSION STUDIES

Synopsis.

The New Kattalai High Level Canal takes off from the River Cauvery at the site of Kattalai Bed Regulator beside the existing combined South Bank and Kattalai High Level Canal. Sediment exclusion studies were conducted to evolve a device to prevent entry of coarse bed load into the canal as the existing silt trap was found to be ineffective. Studies were conducted in two stages, one being devoted to devise ways and means to improve the functioning of the existing silt trap and the other to evolve an entirely new device. A divide wall 200 feet long perpendicular to the Kattalai Bed Regulator is recommended. A slight modification in the alignment of the leading channel for the New Kattalai High Level Canal has been proposed to give an equitable distribution of sediment load between the two canals. A regulation procedure which aids the functioning of the divide wall is also recommended.

1. Introduction :

1.1. *Necessity for the Project.*—The River Cauvery is the largest perennial river of Madras State, and had been utilised for development of Irrigation to an almost complete extent through various schemes, of which the Mettur, the Grand Anicut and Kattalai High Level Canal schemes deserve special mention. Eve ı though the Cauvery flows through Tiruchirappalli and Thanjavnr Districts, the area under river irrigation in Thanjavur District is very much larger than in Tiruchirappalli District. At present irrigation in Tiruchirappalli District is confined to the Uyyakonda channel and existing Kattalai High Level Canal executed in the year 1935. Observing the benefits derived from these schemes, the ryots of Tiruchirappalli District were clamouring for the extension of the Kattalai High Level Canal to bring additional areas under irrigation. Thus, the New Kattalai High Level Canal Scheme, which is for utilising the surplus waters of the Cauvery after meeting the existing demands of irrigation below the Mettur Dam, came up for consideration.

1.2. *Kattalai High Level Canal.*—There is an existing bed regulator across Cauvery near Mayanoor in Tiruchirappalli District which goes by the name Kattalai Bed Regulator. A combined South Bank and Kattalai High Level Canal take off from the right side and water from the river is led through a leading channel 850 feet long. Kattalai High Level Canal branches off from the combined channel at mile 0/5 (vide Figure 1). To reduce silt entry into the canal a silt trap had also been constructed in front of the leading channel to the combined canal. The silt entry trap as existing is at an inclination of 65° to the alignment of the leading channel and ends in a scour vent of 3 Nos. 20′ × 8.5′ in line with the regulator. There is also a small irrigation channel named Krishnarajapuram channel with a maximum discharge of 40 cusecs. The site plan is shown in Figure 2 and typical sections of the Kattalai Bed Regulator are shown in Figure 3. The details of silt trap are shown in Figure 4.

1.3. *Proposals for the Scheme.*—Various proposals ranging from an entirely new canal taking off a few miles upstream of the Kattalai Bed Regulator to remodelling of the existing High Level Canal were considered. But all the alternatives were essentially direct irrigation schemes and as such supplies could be counted upon only in favourable years. Therefore the scheme was modified as a tank irrigation scheme, with the new Kattalai High Level Canal functioning as the main supply channel for filling the existing tanks during the periods when Mettur is surplussing, and with some direct irrigation in years of favourable storage at Mettur.

1.4. *Main features of the Scheme.*—The scheme comprises of a new high level canal which is divided into three reaches. In the first reach of 40 miles the new canal will run parallel and close to the existing Kattalai High Level Canal and there is no irrigation in this reach (vide Figure 1). The second reach will be 24 miles long and areas of 5,658 acres under tanks and 8,622 acres directly will be irrigated. The third reach will irrigate 6,342 acres under tanks. This canal will have a head regulator just by the side of the head regulator for the existing combined South Bank and Kattalai High Level Canal, and water from the river Cauvery will be led through a leading channel 920 feet long with its offtake at the upstream portion of the silt trap. The Krishnarajapuram channel will be provided with a new leading channel parallel to that of the South Bank Canal and a new regulator beside the latter.

2. Problem :

As a first stage of the study the position of the head regulator for the new canal along the leading channel was examined and it was concluded that the position, as such, did not have appreciable effect on sediment drawal. The field officers had then decided to have the head regulator by the side of the existing head regulator at the end of the leading channel of 1,000 feet length.

With this finalised position of head regulator, studies were required to be conducted to exclude coarse bed load.

3. Data :

Hydraulic data relevant to the problem are given below :

Kattalai Bed Regulator—

Average river bed level	El. 318.50.
Length of regulator	4,225 feet.
Crest level of regulator (ch. 0-865) ...	El. 322.95.
(Ch. 0 is left end of regulator). ch. 865-4225	El. 320.95.
Height of falling shutters	2 feet.
Length for which falling shutters is fixed. (From ch. 865-4225).	3,360 feet.

Combined South Bank and Kattalai High Level Canal—

Bed level of leading channel	El. 312.00.
Sill level of Head regulator	El. 315.00.

No. and size of vents	4 Nos. ; 19′ × 4.5′ each.
Bed width of the canal	78 feet.
Full supply depth	5 feet.
Full supply discharge	1,380 cusecs.
Bed fall	1 in 2,640.

Krishnarajapuram Channel—

Sill Level of regulator	El. 320.30.
No. and size of vent	2 Nos. ; 4′ × 3.35′ each.
Full supply depth	7 feet.
Full supply discharge	40 cusecs.

New Kattalai High Level Canal—

Sill level of Head regulator	El. 313.70.
No. and size of vents	4 Nos. ; 9′ × 8′ 6″ each.
Bed width of canal	36′ 6″.
Full supply depth	9′ 0″.
Bed fall	1 in 6,600.

Silt trap and scour vent —

Bed level of trap	El. 312.00.
Top level of trap	El. 318.00 upstream. El. 317.25 downstream.
Width of trap	81 feet.
Length of trap	358 feet.
Sill level of scour vent	El. 312.00.
No. and size of vents	3 Nos. ; 20′ × 8.5′ each.

Note.—All elevations are local Thanjavur levels.

4. Model :

The mobile bed model constructed already to a horizontal scale of 1 : 120 and vertical scale of 1 : 24 was utilised for these studies also. But with the large vertical scale the sand movement was observed to be negligible. To improve the sand movement the vertical scale was proposed to be reduced to 1 : 48 as it is an accepted practice to lower the vertical exaggeration of part-width model[1]. The model was modified to suit this vertical scale.

5. Experiments and Discussion :

5.1. *Proving the model.*—With the lowered vertical exaggeration, the discharge scale of H^2 could not be realised where H = horizontal scale. The flow in rear of the Kattalai Bed Regulator submerged it and hence the rear bed levels had to be lowered. With the bed regulator functioning free, the required gauge reading of 4.6 feet was maintained and even then the sand movement was found to be negligible. Hence the model was given a slight tilt to give the required sand movement. With this tilt the movement was observed to be good and the experiments were switched over to arrive at the discharge scale. Model discharge for the gauge reading of 4.6 feet was observed and by correlating it with the corresponding prototype discharge, the discharge scale was arrived at to be 1 : 21,940. Comparison of this with the discharge scale according to Froude's criterion which works out to 1 : 39,950 will reveal that a model discharge required is about two times that arrived at by Froude's criterion.

5.2. Experiments were conducted in two series, one being devoted to devise ways and means to improve the functioning of the existing silt trap and the other to evolve an entirely new device. Just as in other similar model studies the basic run with sand injection of 240 lbs./hour was conducted, the percentage sand drawal in the canal was computed and was taken as the basis for purposes of comparison. The results of basic run along with those for the entire study are given in Figure 5 in the tabulation form.

5.3.1. *I Series.*—At the very outset, it was clear that the river being a perennial one and the location of the bed regulator being such that quite a large quantity of water will have to be let down for irrigation of ayacut lower down, a divide wall suggests itself as a good solution [2] [3]. A divide wall 265 feet long was tested and found to be satisfactory by model experiments also (vide Study 2). Yet, with a view to explore the possibility of adopting other devices, a series of study was conducted to improve the functioning of the existing silt trap. The principle of creating a zone of suction in operation of silt excluders was utilised. The principle of straight entry into canal and curved entry into the scour vent which was found to be successful in the case of Periyar Main Canal [4] was also used in conjunction with the means of improving the silt trap.

5.3.2. As a first study the outer curved wall of the silt trap was raised above the water level. A breast wall across the silt trap with an extension upstream of the silt trap, was incorporated. The scour vents were kept open fully. Though the silt trap got choked up at intervals the overall reduction was encouraging ; but it was observed that the new canal got little or no benefit of silt reduction, the bulk of the benefit going only to the existing canal (vide Proposal 3).

The extension of the breast wall was omitted for the next proposal which gave more or less a similar performance (vide Study 4). Here also the new canal derived only a minor portion of the benefit. This is attributed to the fact that the zone of suction of the vent created by the breast wall across the silt trap did not extend beyond the centre line of the existing canal. Hence, the zone of suction was extended by incorporating barrels as in excluders. These barrels got choked up completely after two hours (vide adjacent photo) and the proposal was hence abandoned.

As the zone of suction could not be extended, the next alternative of having the total discharge of the two canals to pass through the zone of suction created by the vent under breast wall was tested. The mouth of the leading channel of the new canal was closed and the discharge for both the canals was drawn through the leading channel of the existing canal which was bifurcated about 500 feet downstream of the silt trap. This arrangement gave better

sediment exclusion than the previous one but the choking was found to be more severe (vide Proposal 5).

5.3.3. *Analysis of the studies of Series I.*—The proposals (one to five) gave encouraging results as far as sediment exclusion is concerned but the following two drawbacks were observed. The portion of the silt trap between the breast wall and the scour vent got choked up at intervals (vide Statement I) and in the prototype, where suspended load will also be present, the choking may be more frequent and at the worst may be even permanent. Due to severe curvature induced on the scouring water entering the vent there was severe disturbance of the entry with vigorous vortex formations. These disturbances may assume devastating proportion at the site throwing all the bed materials into suspension and eroding the banks. The outer curved bund of the silt trap will have to be raised which will mean that the existing bund will have to be demolished and a new bund formed. This is a structural consideration affecting the economy of the proposal. Hence, studies were switched over to improve on the proposal of divide wall.

5.4. *Series II.*—Another divide wall 400 feet long was tested and was found to be more efficient than the 265 feet long divide wall (vide Proposal 2), but even in this case bulk of benefit went to the existing canal and the new canal derived only a small measure of the benefit (vide Proposal 6).

An attempt was made to obtain equitable distribution of sediment load between the two canals as follows : The two leading channels were combined at the silt trap end and the leading channel for the new canal was so aligned as to be on the concave side of the combined leading channel. Incidentally it was found possible by this arrangement to reduce the length of the divide wall. By visual observation the divide wall of 200 feet length was fixed. Experiments were conducted with still-pond regulation. Sediment reduction to a tune of 95% overall, was obtained (vide Proposal 7). The alignment of the leading channel of the new canal gave redistribution of sediment load so much that the new Kattalai High Level Canal did not draw any sand at the end of the experiment and the existing canal drew all the sand (vide Study 7).

The new device had given almost complete exclusion with none of the disadvantages observed in the case of the proposals of the first series. It is also observed that the construction of the divide wall will not be costly as the rock outcrop is observed at the site at a depth of about 10 feet from river bed level. Hence, further studies were centred on improving the functioning of the divide wall.

5.5. It was proposed to improve the efficiency of the finalised device by adopting intermittent operation of the scour vents. With the scour vents kept closed the canals were open for a duration of a fixed number of hours and then the canal regulators were closed and the scour vent kept for a fraction of the duration of canals opening. Three studies were conducted on this aspect. In the first study canals were run for four hours and scour vents half-an-hour, in the second the cansls for eight hours and scour vents for one hour and in the third the canals were run for three hours and scour vents half-an-hour. Comparing the first two runs it can be observed that the ratio of the scour vent run to canal run is the same, i.e., one in eight but the first had given a better sediment exclusion indicating that intermittant operation of scour vents at shorter intervals will be better than that for longer intervals. No conclusive results could be obtained as to the best ratio of scour vent run to canal run.

5.6. *Verification and final alignment.*—For verification, the final device with the divide wall and the new alignment of the leading channels were incorporated in the full width rigid bed model to study the approach condition. It was observed that the curvature of the right bank of the leading channel of the New Kattalai High Level Canal corresponding to a radius of 450′ was acute and a return flow zone along the curved bank was clearly observed. Hence the curve was smoothened out by adopting a larger radius (1,000 feet). Sediment collection test with intermittent operation of 4 hours canal run to half-an-hour scour vent was run gave 95 per cent sediment reduction against cent per cent reduction as in the case of the curved bank of radius 450 feet. Though the reduction is less than in the case of the curve with 450 feet radius the larger radius is acceptable in view of the fact that the flow into the canal is much smoother than in the former case.

6. Conclusions and Recommendations :

6.1. As a result of sediment collection tests conducted in connection with the functioning of the New Kattalai High Level Canal and the combined Kattalai High Level and South Bank Canal it was concluded that a short divide wall 200 feet long and an alignment of the leading channel as given in Figure 6 will give sediment exclusion to an almost complete extent.

6.2. To improve the functioning of the divide wall an attempt was made by adopting intermittent operation. The conclusion on this aspect is that intermittent operation of the scour vent should be at as short intervals as possible and practical. No definite conclusion could be arrived at as to the best ratio of scour vent run to canal run. The ratio of one to eight with canal run of 4 hours was found to yield good results in the model. The time scale for distorted models is given by $H/\sqrt{V}$ where H is horizontal scale and V is vertical scale. In the present case the time works out to 17.3. But no definite recommendation for intermittent operation in terms of prototype time can be given for the following reason. Because of the inherent defect of this type of model that suspended silt is not reproduced, the rate of silting is slower and the rate of scouring greater in the model by proportion than in prototype (5). Roughly it may be stated that the scour vents may have to be kept open for about 12 hours for every $3\frac{1}{2}$ days period. After constructing the divide wall it is recommended that sounding may be taken along the silt trap and on the canal side of the divide wall before and after operation of the scour vents according to the rough timing given above and the data may be furnished to this Station so that a practical regulation procedure can be formulated.

6.3. *Recommendations regarding the component parts of the device.*—(*a*) If a better is proposed for the divide wall which, of course, will have to be founded on rock strata, it is better to have it on one side. The divide wall should then be placed with the vertical face on the river side. The top of the divide wall should be above maximum flood level with adequate free board.

(*b*) The entire outer curved bund of the silt trap (shown as X in Figure 4) should be removed upto El. 312.00 (i.e., sill of the scour vent). Only then a deep channel will be formed axial to the scour vent when it is operated.

(*c*) It is known that a scour hole gets formed at the tip of the divide wall in the shape of an inverted cone with the axis along the tip of the divide wall, It is essential that the scour hole should be there as it aids the sediment exclusion to a great extent. Hence it is recommended that at the end of the first season after construction of the divide wall the resulting scour hole may be pitched to the profile with boulders.

(*d*) Noses for the junction of New Kattalai High Level Canal and Krishnarajapuram Channel, and the junction of the latter and the combined South Bank and Kattalai High Level Canal may be of a suitable shape. A scour hole will get formed at these noses also and hence suitable protection at the end of the noses for scour considering the extra turbulance that is likely to occur at the noses will have to be made.

It is to be emphasized that all the sediment exclusion is effected by the divide wall and the alignment of the leading channels is only a refinement observed to give re-distribution of bed load between the two canals and the latter plays only a very minor part in sediment exclusion.

7. References :

(1) Annual Report (Technical) of the Central Board of Irrigation, 1942, Page 92.

(2) Report on the fourth meeting of the International Association of Hydraulic Research, Page 182 (Part C).

(3) Report on Palar Barrage presented to the 28th Research Committee Meeting of Central Board of Irrigation and Power, Para 6 (Conclusions and Deliberations).

(4) " Periyar Main Canal—Sediment Exclusion Studies." I. R. S. Report No. 2/59 of this Station.

(5) Annual Report of Central Water and Power Research Station, Poona for the year 1945, Page 113.

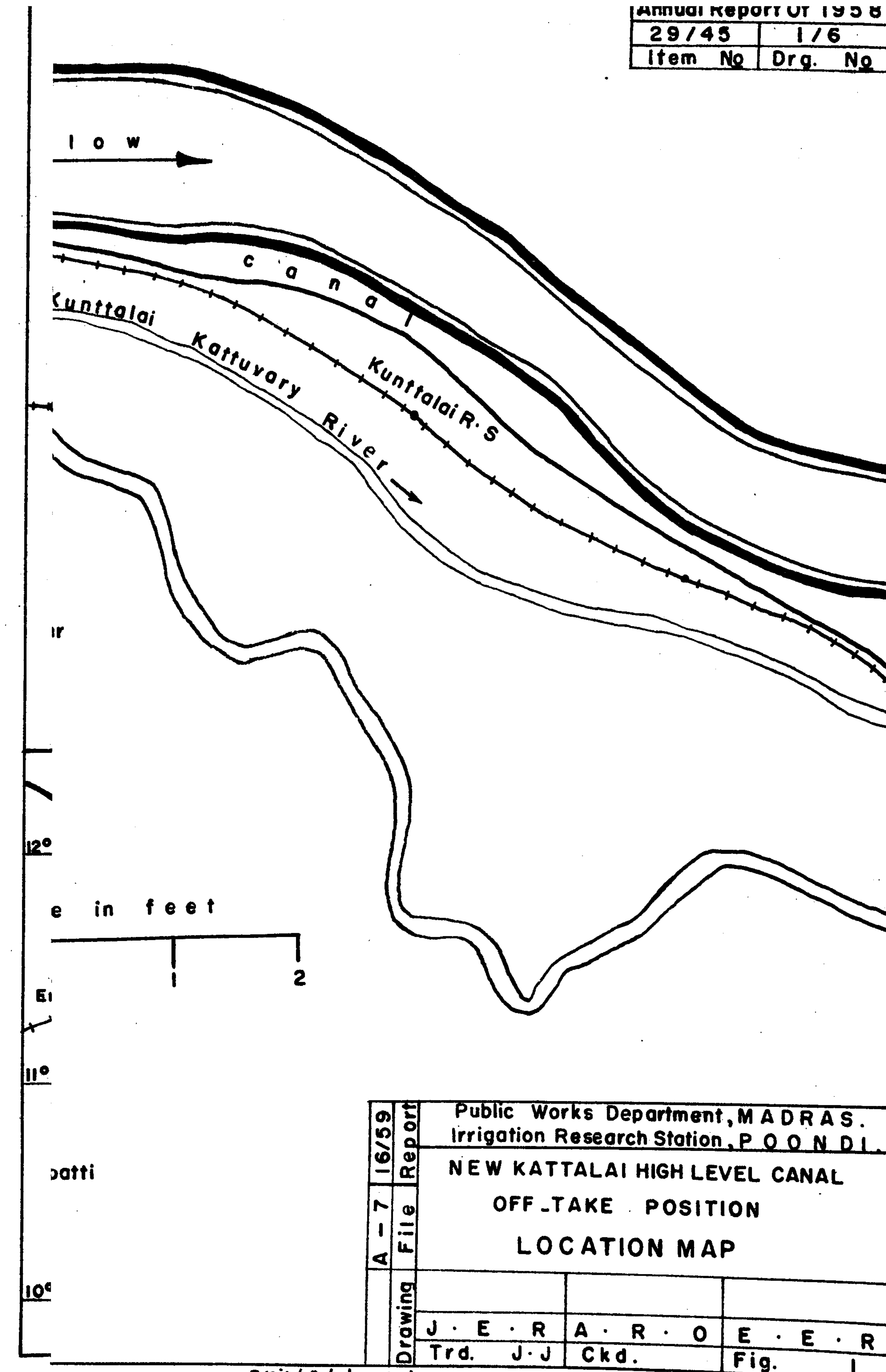
NEW KATTALAI HIGH LEVEL CANAL SCHEME—SEDIMENT EXCLUSION STUDIES.

STATEMENT 1.

SHOWIUG HOURLY SAND DRAWALS WITH SAND INJECTION RATE OF 240 LBS. PER HOUR FOR PROPOSAL 5.

Hour.	*N.K.H.L. Canal (lb.)*	*S.B. Canal (lb.)*	*Remarks.*	*Hour.*	*N.K.H.L. Canal (lb.)*	*S.B. Canal (lb.)*	*Remarks*
1	...	...		21	...	...	
2	...	...		22	...	...	
3	...	...		23	...	...	
4*	5	...		24	2	...	
5*	22	...		25	8	2	Choked.
6	4	...		26	15	...	Do.
7	5	...		27	4	2	
8	8	1		28	2	...	
9	11	...	Choked.	29	4	...	
10	20	...	Do.	30	...	...	
11	7	...		31	10	3	
12	5	...		32	20	2	Choked.
13	2	2		33	4	...	
14	...	...		34	3	2	
15	1	...		35	3	...	
16	4	1		36	3	...	
17	3	...		37	...	...	
18	4	...	Choked.	38	...	...	
19	4	...	Do.	39	...	...	
20	3	...		40	1	2	

* Silt trap between breast wall and scour vent got choked.



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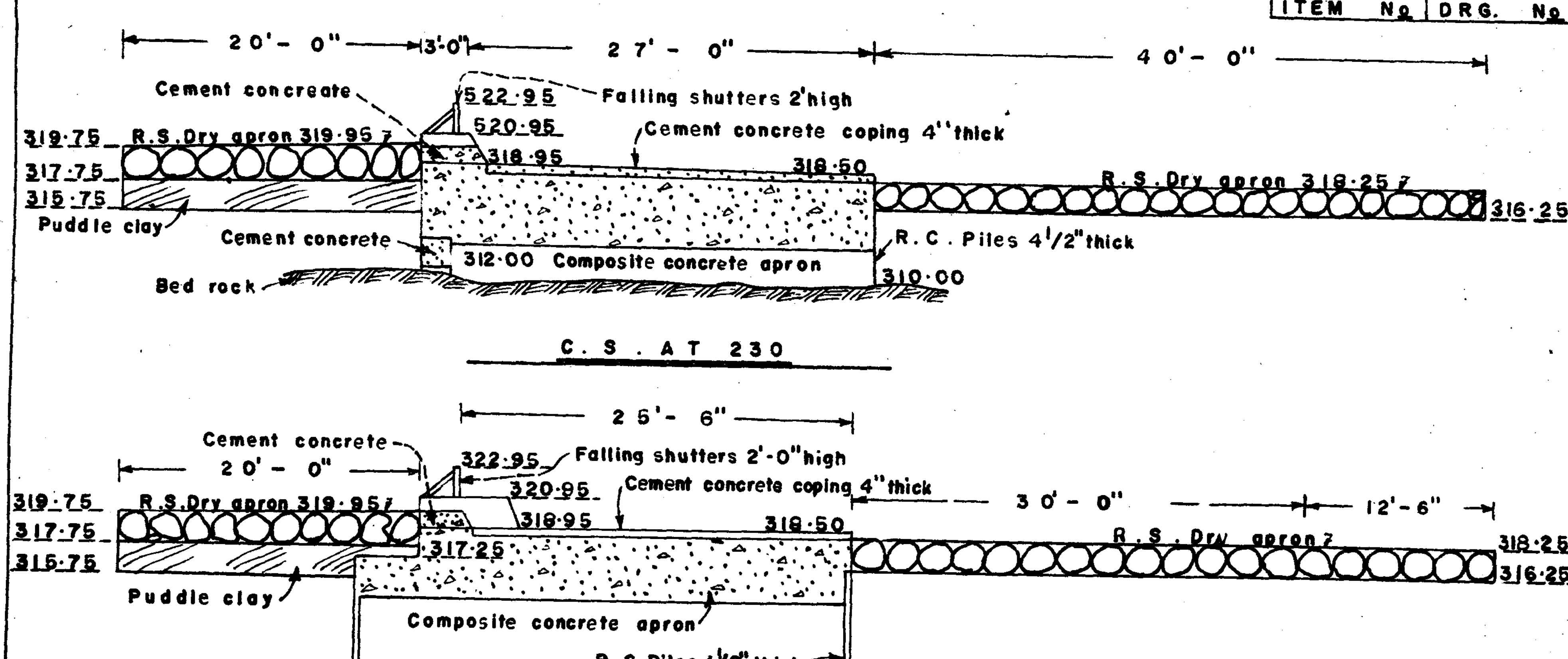
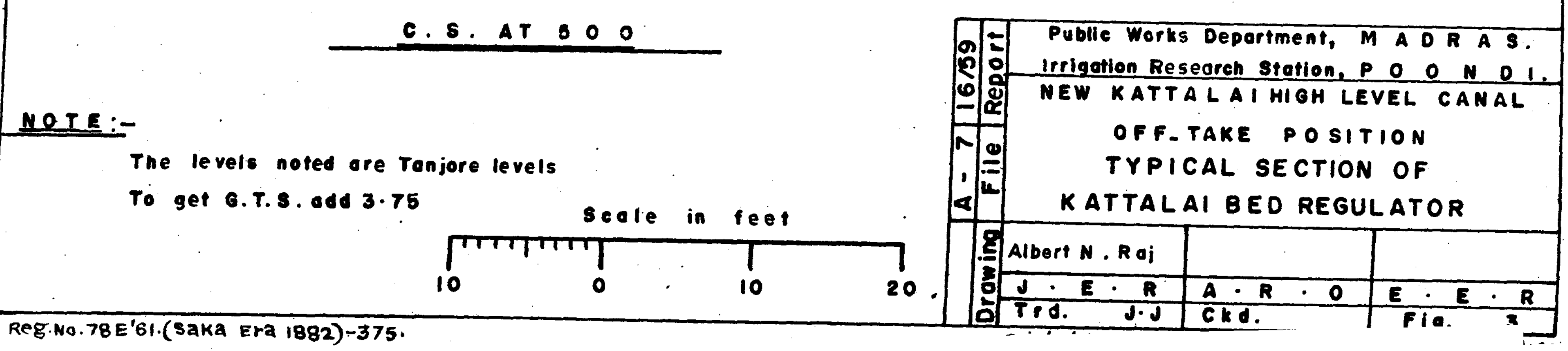


C. S. AT 230



C. S. AT 500

NOTE:-

The levels noted are Tanjore levels

To get G.T.S. add 3·75

Scale in feet

10 0 10 20

Drawing	File	Report
A - 7		16/59

Public Works Department, MADRAS.

Irrigation Research Station, POONDI.

NEW KATTALAI HIGH LEVEL CANAL

OFF-TAKE POSITION

TYPICAL SECTION OF

KATTALAI BED REGULATOR

Albert N. Raj		
J · E · R	A · R · O	E · E · R
Trd. J·J	Ckd.	Fig.

Reg.No.78E'61.(Saka Era 1882)-375.

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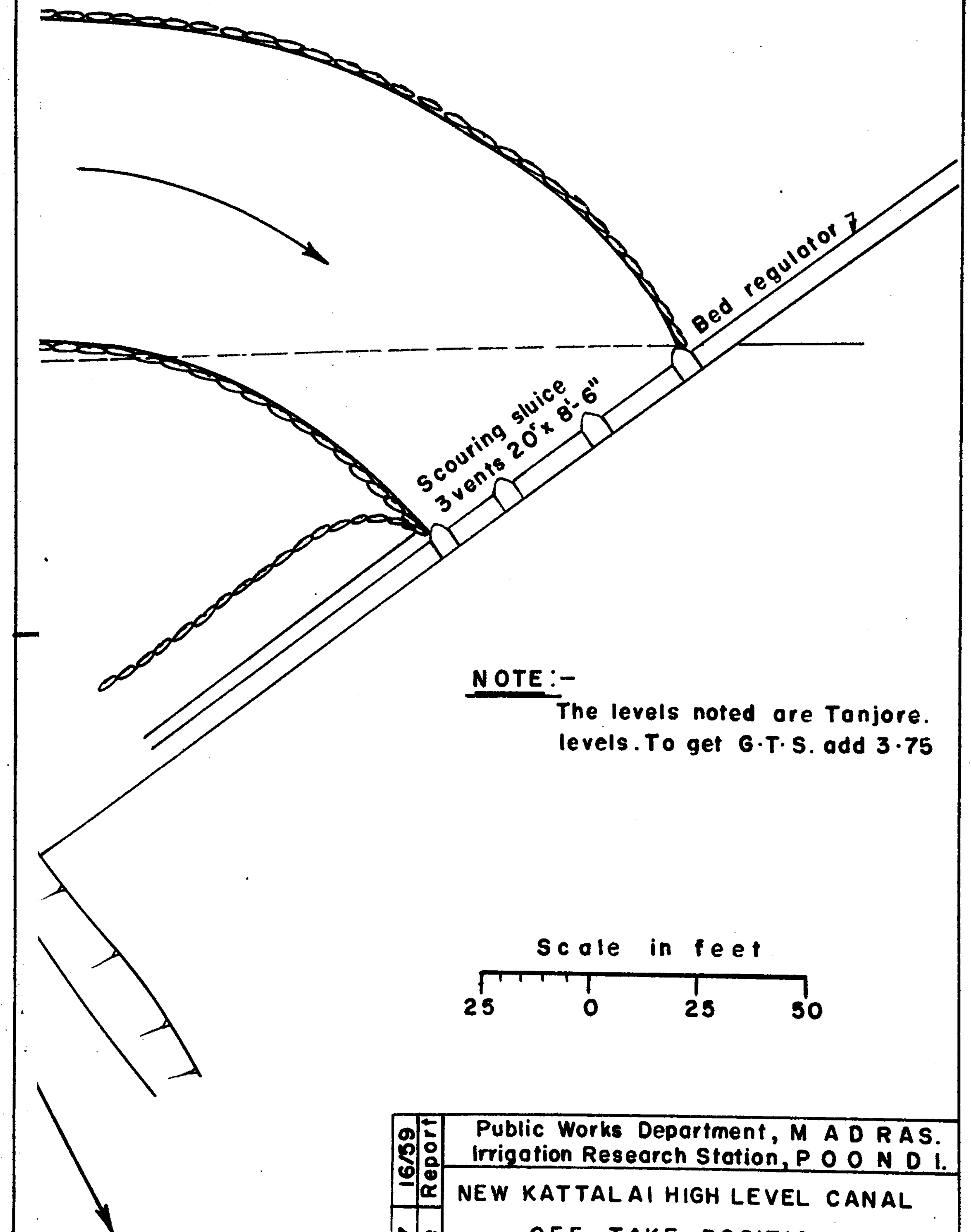

NOTE:-
The levels noted are Tanjore. levels. To get G·T·S. add 3·75

Report 16/59	Public Works Department, M A D R A S. Irrigation Research Station, P O O N D I.		
File A-7	NEW KATTALAI HIGH LEVEL CANAL OFF-TAKE POSITION DETAILS OF SILT TRAP		
Drawing			
	J · E · R	A · R · O	E · E · R
	Trd. J·J	Ckd.	Fig. 4

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No	Particulcals
1.	CAUVERY :d regulator '3nos. 20'x8·5' nnel for combine :anal and kattalai Silt t hannel for new jh level canal
2.	x Divide wall ed regulator 3nos 20'x 8·5' CAUVERY R. hannel for combin k canal and kattal canal Silt t eading channel f ai high level anal (x) Vide details in i
3.	Curved above w d regulator 3 nos·20'x8·5' xx CAUVERY R. annel for combine canal and kattal canal Silt tr ading channel lai high l canal xx vide details o
4.	Cu ra wi ed regulator 3 nos. 20'x8·5 CAUVERY R. xx annel for combin k canal and katt canal Silt t ading channel f ai high level anal xxx vide details
5.	cur ra wa ed regulator 3nos. 20'x8·5' CAUVERY R. Bre annel for combin canal and katta canal Silt Clo annel for new gh level canal

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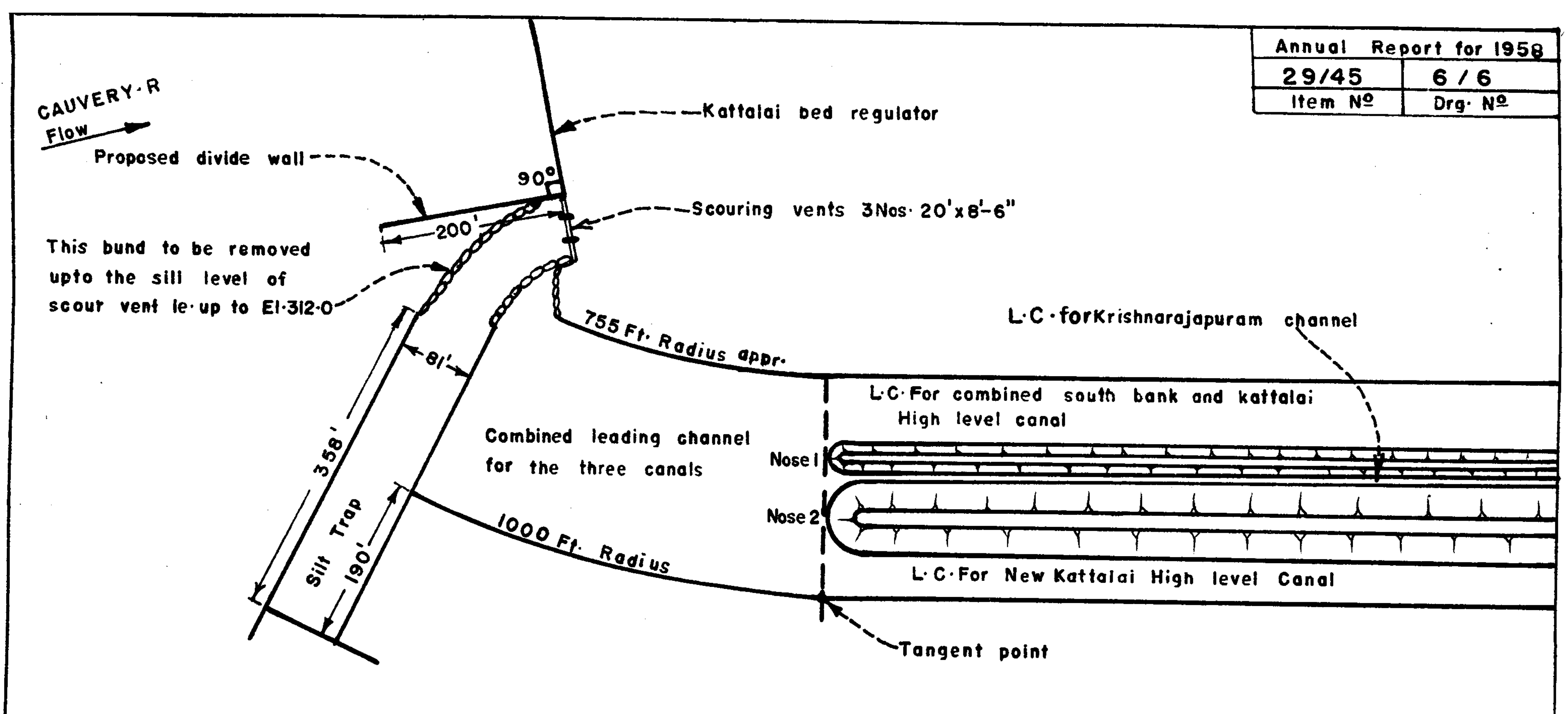

NOTE:
1· Only essential dimensions are given·
2· Dimensions For The Inner Curve noses etc· Will have to be fixed at field

Report 16/59	Public Works Department Madras Irrigation Research Station Poondi New Kattalai High level Canal Scheme	
File A-7	SEDIMENT EXCLUSION STUDIES FINAL ALLIGNMENT OF THE LEADING CHANNEL	
Drawing 115/59	Albert Nattanielray 2/1-6 J·E·R	U·V·Balakrishnan A·R·O	P.K. Ramaswamy E·E·R
	Trd: V·V	ckd· 2/1-6	FIG····6

KALARI TANK SUPPLY CHANNEL—FINAL REPORT

(C.B.I. & P. No. C. 8.)

1. Introduction :

1.1. Supply to Kalari tank is drawn from river Vaigai (right bank) at Averanendal Village limits in Paramakudi Taluk, Ramanathapuram District.

1.2. When the supply channel had an open head it was reported that during floods water rushed into the channel carrying with it large quantities of silt which in turn, silted up the entire channel. Consequently the channel was unable to draw the supply during low stages of flow in river Vaigai. This problem was referred to this Station for model studies.

1.3.1. A partwidth model of the river Vaigai reproducing only about 1,000 feet length of the river upstream of the head-sluice was constructed and studies conducted to find out the best location of head sluice.

1.3.2. After model experiments. a head sluice located 230 feet from the head of the leading channel was recommended.

1.4.1. In continuation of the above, it was felt that some silt exclusion device may also be provided at the offtake point to improve the performance.

1.4.2. This report deals with the experiments conducted to study the performance of the head sluice on a full width model reproducing about 5,000 feet length of the river upstream of the head sluice.

2. Data :

Head Sluice—

Sill level	... El. 89.0.
No. and size of vents 5 Nos. ...	... 8′ 0″ × 3′ 6″.
Pier thickness	... 2′ 6″.
Discharge	... 490 cusecs.

Supply Channel—

Bed width	... 47′ 0″.
Full supply depth	... 4′.
B. L. at off-take	... El. 89.0.

River Vaigai—

Maximum flood discharge ...	... 42,900 cusecs.
M. F. L. at site of offtake ...	... El. 94.0.

3. Model :

A model with a mobile bed of—14 sand to a distorted scale of 1/120 horizontal and 1/24 vertical was constructed. In this model the entire width of the river was reproduced to a distance of about 5,000 feet upstream of the head sluice and about 400 feet downstream. All the arrangements for measuring discharges in the river and canal, maintaining water levels both upstream and downstream, etc., have been made. The area reproduced in the model is shown in Figure 1.

4. Experiments :

The head sluice was made of teakwood and fixed in position at 230 feet from the head of the leading channel. A maximum flood discharge of 42,900 cusecs was allowed in the river and full supply discharge of 490 cusecs was drawn in the canal maintaining the full supply depth. For these conditions of run a sand injection rate of 10 c.ft. per hour was found to give regime conditions. Hence this rate was adopted and the silt distribution between river and canal was observed.

Similarly 50%, 25%, 10% and 5% of maximum flood discharge were allowed in the river and observations of silt distribution were made in each case.

5. Results :

The results of observations are tabulated in the Statement I enclosed herewith. The percentage of silt entering the canal for M.F.D. conditions as observed in the full width model compares favourably with the result obtained previously on the part width model as seen from Statement II.

In all the conditions of runs, the flow pattern obtaining near the head sluice was closely observed. It was found that because of the curvature effect, supply to the canal was drawn from the concave side of the head. Consequently, the bed load was diverted away from the canal into the river and only the silt-free water entered the canal. As seen from Statement I very little quantity of silt moved into the canal for all discharges. A graph connecting the discharge in the river and the silt distribution between river and canal has been drawn to illustrate the above point.

6. Conclusion :

As such it may be safely concluded that the location of the head sluice is very favourable for drawal of silt-free water into the canal. The performance and behaviour of the head sluice and the supply channel may be watched in the field for about two or three seasons after the construction of the head sluice to see how far the recommended location of the head sluice has been effective in drawing into the canal as much silt-free discharge as is practically possible. Therefore, if any further model studies are found necessary on silt-exclusion devices, the problem will be taken up again, if a reference is made to this Station.

STATEMENT I.

Serial No.	*Conditions of run.*	*Total number of hours of run.*	*Silt distribution.* Percentage of sand in river.	*Percentage of sand in canal.*
1	M.F.D. in river F.S.D. in canal	48	99.50	0.50
2	50% of M.F.D. in river F.S.D. in canal ...	48	99.38	0.62
3	25% of M.F.D. in river F.S.D. in canal ...	48	99.29	0.71
4	10% of M.F.D. in river F.S.D. in canal ...	53	98.10	0.9
5	5% of M.F.D. in river F.S.D. in canal ...	36	99.53	0.47

STATEMENT II

COMPARISON OF SILT DISTRIBUTION RESULTS BETWEEN PARTWIDTH MODEL AND FULL WIDTH MODEL.

Serial No.	*Partwidth model.*	*Full width model.*	*Remarks.*
1	% of silt in canal: (P_1) $P_1 = \frac{0.27}{52} \times 100 = 0.502\%$	% of silt in canal : (P_1) $P_1 = 0.5$ (from Statement I.)	
2	% of silt in river = 99.498% *N.B.*—Conditions of run : (1) The maximum flood discharge of 42,900 cusecs was allowed in river. (2) The M.F.L. was maintained in river. (3) The designed discharge of 490 cusecs was passed through canal. (4) Full supply depth of 4 feet was maintained in the channel. (5) The rate of injection of silt was 13 c.ft. per hour.	% of silt in river = 99.5% *N.B.*—Conditions of run : (1) The maximum flood discharge of 42,900 cusecs was allowed in river. (2) The M.F.L. was maintained in river. (3) The designed discharge of 490 cusecs was passed through canal. (4) Full supply depth of 4 feet was maintained in the channel. (5) The rate of injection of silt was 10 c. ft. per hour.	

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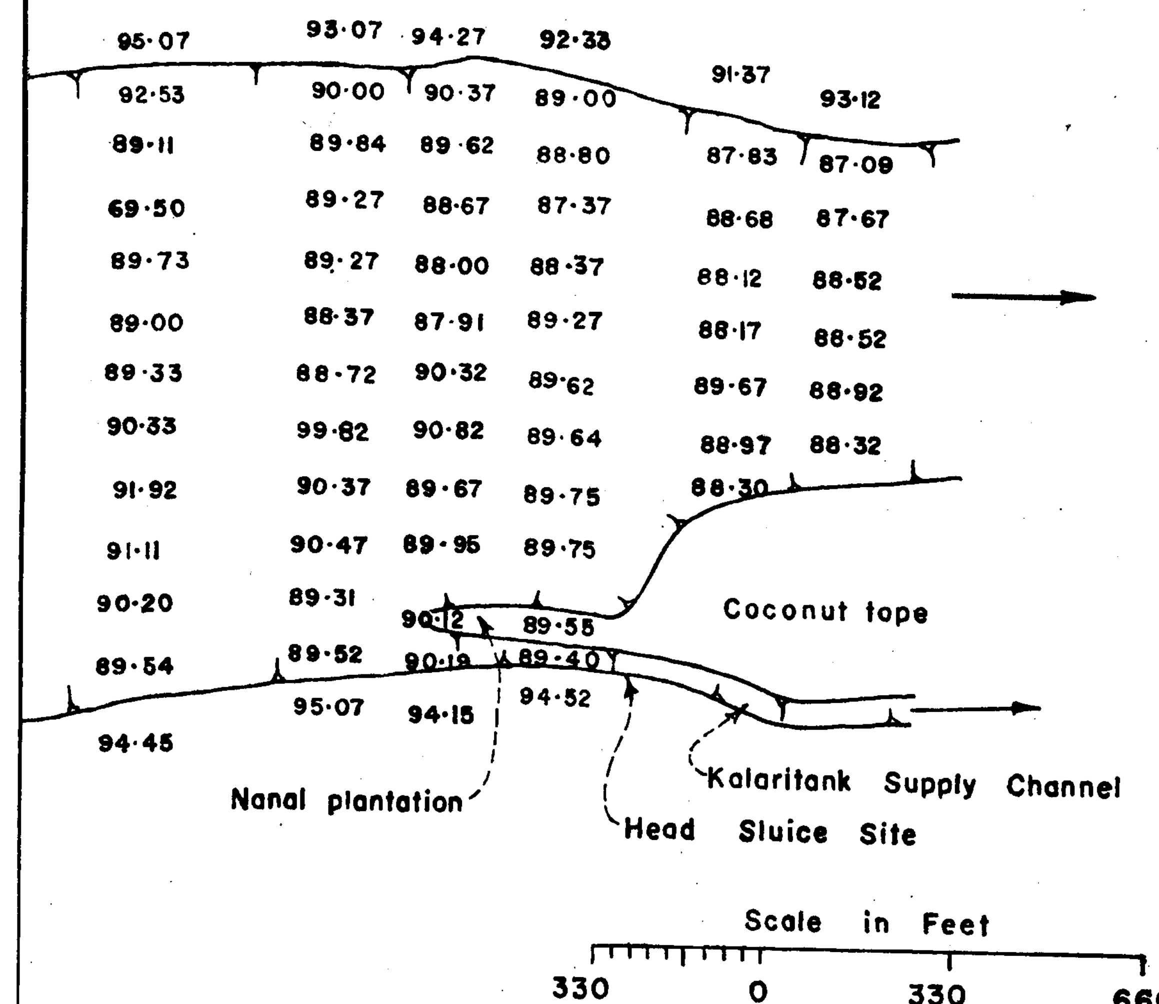

Report	Public Works Department — Madras Irrigation Research Station - Poondi
File	Kalari Tank Supply Channel FIGURE SHOWING AREA REPRODUCED IN THE MODEL
Drawing 94/59	J·E·R	A·E·R	E·E·R
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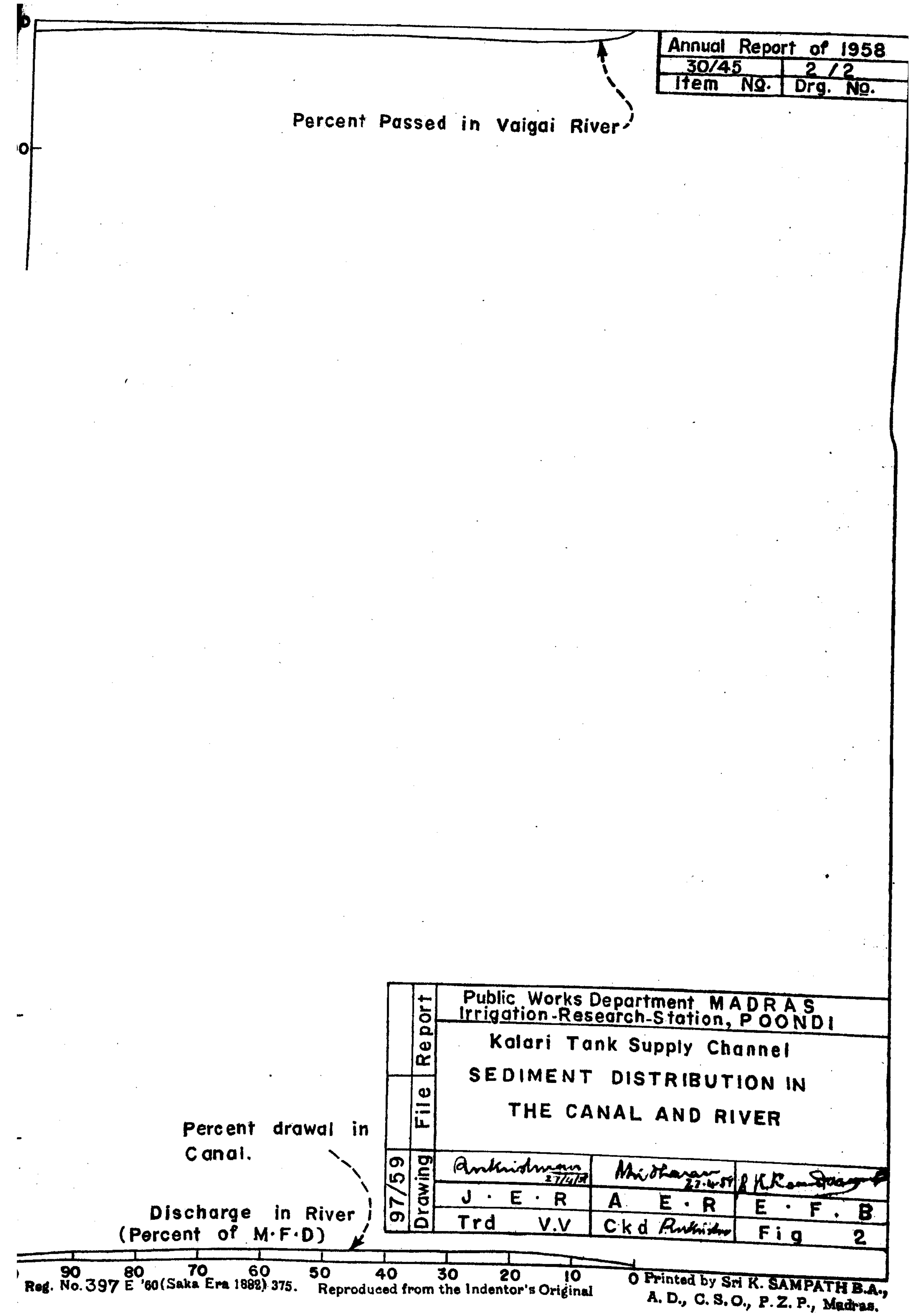
Annual Report of 1958
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Item No.
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Percent Passed in Vaigai River
Public Works Department MADRAS
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SEDIMENT DISTRIBUTION IN
THE CANAL AND RIVER
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J · E · R
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Fig 2
Percent drawal in
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Discharge in River
(Percent of M·F·D)
90
80
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NATHAMPATTI SUPPLY CHANNEL—SILT EXCLUSION STUDIES.

(C.B.I. & P. No.)

Nathampatti channel takes off from Arjuna Nadhi in Srivilliputtur taluk of Ramnad district. The channel takes off with an open head at a point where the river takes a sharp bend. It is stated that during floods in the river, major portion of the water finds its way into the channel without passing down the river as the result of which the river course is completely silted up. This silt uitimately enters the supply channel thereby silting up the bed of the channel.

Studies were conducted to exclude the silt and control the flow into the supply channel. It has been proposed to put up a head sluice at the off-take to control the flow in the channel during times of the maximum flood in the river. This will enable the diversion of the supply water into the river course thereby scouring the bed. In spite of this it was observed in the model a considerable quantity of silt entering the channel.

So a groyne of 100 feet length placed perpendicular to the head sluice was tried, which was found to be effective for maximum discharge in the river. Experiments are in progress for testing the effectiveness of the groyne for low discharges.

CHITRACHAVADI ANICUT—SILT EXCLUSION STUDIES

(C.B.I. & P. No. C-8.)

1. Introduction :

Across river Noyyil, there is an anicut near Chitrachavadi. The water from the anicut is drawn into the Chitrachavadi Channel through a head sluice situated in the left margin of the river. The salient features of the anicut and the channel are given below :

2. Data :

(*a*) *Anicut*—

Crest level	El. 1,405.26.
Length of anicut	174.75 feet.
Maximum flood discharge	12,050 cusecs.

(*b*) *Head sluice*—

No. and size of vents	4 Nos. of 6′ 0″ × 5′ 6′ each.
Sill level	El. 1,400.55.
F.S. discharge	146 cusecs.
F.S. depth	4.6 feet.

(*c*) *Sand vent*—

No. and size of vent	1 No. ; 4′ 0″ × 5′ 8″.
Sill level	El. 1,399.63.

3. Problem :

It is reported that in times of floods, large quantities of silt are deposited in front of the head sluice and are also drawn throigh the canal. Consequently the canal is getting silted up. Hence a method will have to be devised by conducting model studies to prevent deposition of silt in front of the head sluice and silt drawal into the canal.

4. Model and Experiments :

A distorted model to a scale of 1/25 horizontal and 1/10 vertical has been constructed. The bed of the river is moulded with sieved sand of 40 size. Various proposals of divide walls and groynes have been tried for exclusion of silt taking the silt distribution between canal and sand vent as the governing factor.

5. Further Programme of Work :

Different types of groynes for creating an effective still pond from which the canal will draw the supply will be tried and the most effective device recommended.

GRAND ANICUT CANAL—SILT EXCLUSION STUDIES

1. Introduction :

Grand Anicut situated about 12 miles from Tiruchchirappalli is an old irrigation structure across Ullar, which is a prehistoric breach on the left margin of Cauvery river joining Coleroon some distance downstream from the anicut. At the site of the anicut the river Cauvery divides into Cauvery, Vennar and Ullar (already described). There are regulators across Cauvery and Vennar. After the construction of Mettur Dam, a canal was taken off from the right bank of Cauvery river near the Grand Anicut called the Grand Anicut Canal (Figure 1).

2. Hydraulic Particulars :

(*a*) *Cauvery—*

(1) Average bed level of Cauvery at Grand Anicut.	El. 185.00.
(2) Normal water level (Upstrram of the regulators)	El. 196.31.

(*b*) *Grand Anicut Canal—*

	Existing Head Sluice	*Proposed Head Sluice.*
(1) Sill level	El. 185.91.	El. 187.91.
(2) Full supply level	El. 192.91.	El. 192.91.
(3) Bed width	180 feet.	110 feet.
(4) Bed fall	0.15/1,000.	0.15/1,000.
(5) Existing discharge ...	4,100 cusecs.	...
(6) Existing size of vents ...	30′ × 5′ 4″ (6 Nos.).	...
(7) Additional discharge ...	...	1,273 cusecs.
(8) Size of additional vents ...	...	3 vents of 20′ × 5′ or 2 vents of 30′ × 5′.

3. Problem :

The bed of the Grand Anicut Canal in rear of the head sluice is silted up to an average depth of 3 to 4 feet with the result that the present head sluice is not able to draw the required full supply of 4,100 cusecs. It was suggested that an additional head sluice may be constructed on the right bank of the upland drain so that the backed-up waters in the upland drain can be supplied in the Grand Anicut Canal through the proposed head sluice (Figure 2). It was felt that the supply thus drawn from the upland drain may not contian much of silt and it was decided to study by model experiments the approximate silt reduction that can be attained by drawing a discharge of 1,273 cusecs through the additional regulator and the rest of the supply through the existing regulator.

4. Model :

A partwidth geometrically similar model to scale 1/24 was put up representing the right bank and a portion of the Cauvery river above the anicut, some length of the upland drain, the Grand Anicut Canal and the regulator, and the Vennar regulator. The river bed and the canal bed were moulded in sand passing through No. 14 sieve.

A rectangular weir was provided at the upstream end of the model for measuring the discharge let into the river. Another weir was provided downstream of the Grand Anicut Canal to measure the quantity of water passing through the canal. The additional regulator provided was of 3 vents of 20 feet span. Silt traps were provided downstream of Vennar regulator, Grand Anicut Canal and the proposed additional regulator, so that the quantity of sand passing through each of these can be separately collected for the analysis of the results.

5. Experiments and Results :

Two comparative experiments were conducted to study the effect of drawing partial discharge through the additional regulator. The water level upstream of the regulators, i.e., in the Cauvery river was maintained at El. 196.31 which is the normal water level upstream of the regulators as furnished from the field. As per the details received from the field, the maximum discharge passed through the Vennar regulator during the last season was 12,000 cusecs and in the Grand Anicut Canal the maximum discharge passed was 4,300 cusecs. The corresponding discharges were maintained in the model through the respective off-takes. The water level in the Grand Anicut Canal was maintained at El. 192.91 which is the full supply level of the canal. The water level in the Vennar was maintained at El. 188.28. Of the total discharge corresponding to 4,300 cusecs passed into the Grand Anicut Canal, a discharge corresponding to 1,273 cusecs was passed through the proposed additional regulator and the rest of the discharge through the existing regulator. Sand was injected at the rate of 1 c.ft. per hour. The model was run under these conditions for 372 hours at the rate of 12 hours per day. The quantities of sand that passed through Vennar and Grand Anicut Canal were collected at the end of each day and measured. The quantity collected thus was steady only after the model was run for 216 hours. Only after this the model attained a regime condition. Hence, the quantity of sand collected was considered only for the rest of the 156 hours of run. The following table gives the quantities and percentages of sand collected in the above case :—

	Quantity of sand collected.	*Percentage of sand collected.*
Vennar	85 C. ft.	63%
Grand Anicut Canal including the additional regulator ...	50 C. ft.	37%
		100%

The quantity of sand that passed through the proposed additional regulator into Grand Anicut Canal was separately collected and measured. It was only 3% of the total sand injected.

The next experiment was carried out after closing the proposed additional regulator completely and drawing the full discharge corresponding to 4,300 cusecs through the existing regulator alone. The rest of the conditions such as upstream and downstream water levels, discharge through Vennar, etc., being the same the model was run for 372 hours. Here again for 216 hours the sand collection was not steady. Especially, in the Grand Anicut Canal, the bed that formed when the additional regulator was also functioning got changed and a state of regime was attained only after 216 hours. The quantity of sand collected was considered only for the rest of the 156 hours of run.

The following table gives the quantities and percentages of sand collected :—

	Quantity of sand collected.	*Percentage of sand collected.*
Vennar	77 C. ft.	58%
G. A. Canal	56 C. ft.	42%
		100%

6. Discussion on Results :

The first experiment has been conducted with the proposed additional regulator drawing a discharge equivalent to 1,273 cusecs. Only the balance of discharge, i.e., a discharge corresponding to 4,300 — 1,273 or 3,027 cusecs was drawn through the existing Grand Anicut Canal head sluice. A stagnant pool was formed in the upland drain upstream of the new regulator. because of the backed up river water and the discharge drawn through the upland drain was found to be almost free of sand. Scouring of bed was observed downstream of the additional regulator. But the existing head sluice of Grand Anicut Canal began to draw sand almost immediately after the experiment was started. After some hours of run sand was seen entering the upland drain slowly. Though the pool of water appeared to be stagnant in the beginning, because of the constant drawal of water through the additional regulator, movement was seen in the upland drain also, though very slowly. It took considerable time for the sand to move and reach the additional regulator. Even after this, very little quantity of sand passed through the regulator as seen from the results obtained. (Only 3% of the total sand injected.) But the upland drain was silted up to a considerable level (corresponding to 6 feet depth in the prototype). Of course, these were conditions attained in the model with all the model limitations. To cite a few of them, there is the condition that during heavy rains the upland drain may be discharging into the river and the frequency of such floods, the discharge and the quantity of silt change in the flood waters are factors that are not known and are of a varying nature. The effect of such flood draining into the river on the performance of the additional regulator cannot be predicted precisely. However, it reflects the performance on a qualitative basis.

Therefore, the experiment was conducted, limiting to the condition that a steady flow was maintained in the river, i.e., or certain water level corresponding to normal water level was maintained upstream of the regulators and the flow through the regulators were maintained constant for the experiments. Under these conditions, the upland drain had some still pond effect in the beginning of the experiment which went on decreasing due to drawal of sand into the drain and consequent raising of the bed level of the drain when a discharge of 1,273 cusecs was drawn through the additional regulator.

It was observed that the proposed additional regulator also started contributing to the sand drawal into the Grand Anicut Canal during the later stage of the experiment.

To obtain a comparative result for assessing the efficacy of the proposal in the matter of silt reduction, another experiment was conducted in the same model under the same conditions of operation, with the additional regulator, kept closed, and with the full discharge, corresponding to 4,300 cusecs, being drawn through the existing Grand Anicut Canal Regulator itself. The model was run for the same number of hours as in the previous experiment and a state of regime was attained almost after the same number of run as in the previous experiments. The number of run for which the sand collection was taken into account for comparison in both the cases was the same.

The following table gives the results obtained in both the experiments :—

Name of river or channel.	*With the additional regulator functioning (Drawing 1,273 cusecs).*		*Without the additional regulator functioning.*	
	Quantity of sand collected.	*Percentage of sand collected.*	*Quantity of sand collected.*	*Percentage of sand collected.*
Vennar	85 C.ft.	63%	77 C.ft.	58%
Grand Anicut Canal ...	50 C.ft.	37%	56 C.ft.	42%

A comparison of the above results show that there has been no appreciable reduction of silt due to the functioning of the additional regulator while drawing 1,273 cusecs through it. The percentage of overall silt reduction works out to only 5%. Though the quantities of sand distribution as obtained in the model, are not strictly comparable with those of the prototype distribution, due to the model limitations, especially in a partwidth model (where only two regulators are represented), yet, this will serve to furnish a qualitative analysis as to the effectiveness of the functioning of the additional regulator with regard to silt reduction. Even though a discharge of 1,273 cusecs (about 30% of the total discharge) was drawn through the additional regulator, the consequent silt reduction has been only 5%. As such the proposed additional regulator cannot be said to be efficient in so far as silt reduction is concerned.

On the other hand, the quantity of sand that was collected in the rear of the additional regulator was only 3% of the total sand injected, or 7.5% of the total sand that was collected in the Grand Anicut Canal. Or in other words, when the additional regulator is drawing about 30% of the discharge of the Grand Anicut Canal, it draws only about 7.5% of the sand that comes into the Grand Anicut Canal. Though this seems to be a redeeming feature in favour of the proposal, other observations reveal (as already seen) that the

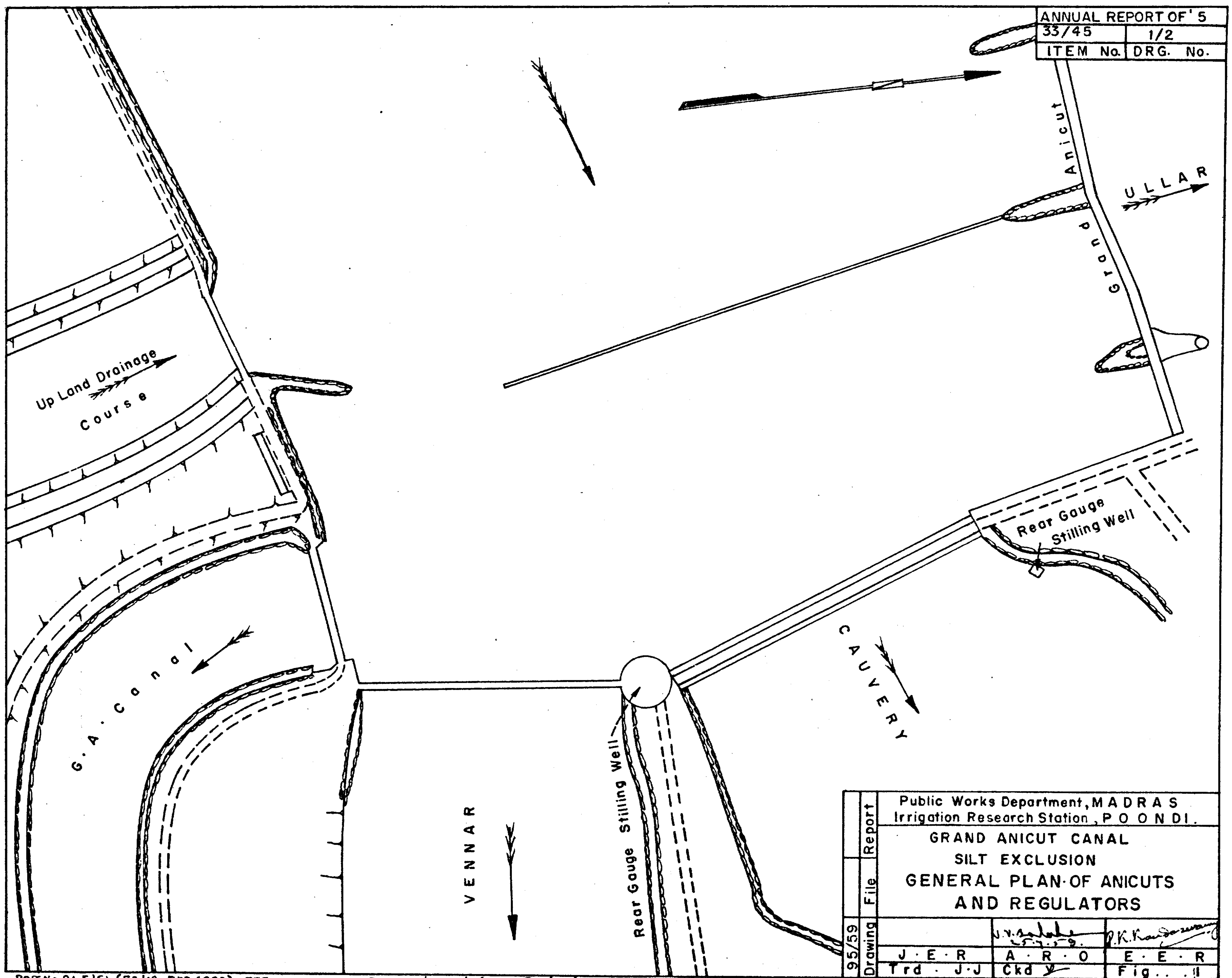
total silt reduction that could be effected in the Grand Anicut Canal is not appreciable. The lesser quantity of sand observed to be contributed by the additional regulator is probably due to the fact that sand collections in the rear of the additional regulator could not be done as perfectly as desirable. The water passing through the additional regulator had an appreciable velocity and immediately after it left the regulator it came into contact with the already flowing water in the canal. The consequent turbulance created kept the moving sand in suspension and only the coarser sand got deposited in the silt trap provided for this purpose, downstream of the additional regulator. The rest of the sand passed through the canal and was accounted for in the total collection made at the end of the Grand Anicut Canal. But it is only the small quantity of sand remaining in suspension because of the turbulance that might have escaped the accounting for the additional regulator, but had been included in the total sand collected for the Grand Anicut Canal. At any rate, it is a significant point to note that this silt charge of the 30% supply taken through the proposed regulator is less concentrated than that of the 70% supply let down through the Grand Anicut Canal regulator.

Conclusion :

From the results obtained, it is seen that the additional regulator drawing 1,273 cusecs has not got much effect in reducing the sand that is moving into the Grand Anicut Canal. Perhaps, this may serve to supply more discharge into the Grand Anicut Canal in case the existing regulator is not able to draw the full discharge required. But its performance in reducing the incoming sand into the Grand Anicut Canal seems to be doubtful.

Anyhow, some more experiments will be conducted in the vertically exaggerated comprehensive model and the results and conclusions will be furnished as a supplement to this report. Also, the suggestions and directions given by the Chief Engineer (Irrigation) as per the latest inspection notes will be studied in detail in the comprehensive model.

Consequent on the proposal of drawing part supply through the additional regulator, the mouth of the upland drain is likely to get silted up, destroying thereby the effectiveness of the upland drain in giving the much-needed drainage facilities to the adjoining irrigated lands. Due considerations to this aspect of the problem will also be given in the pursuit of further studies.



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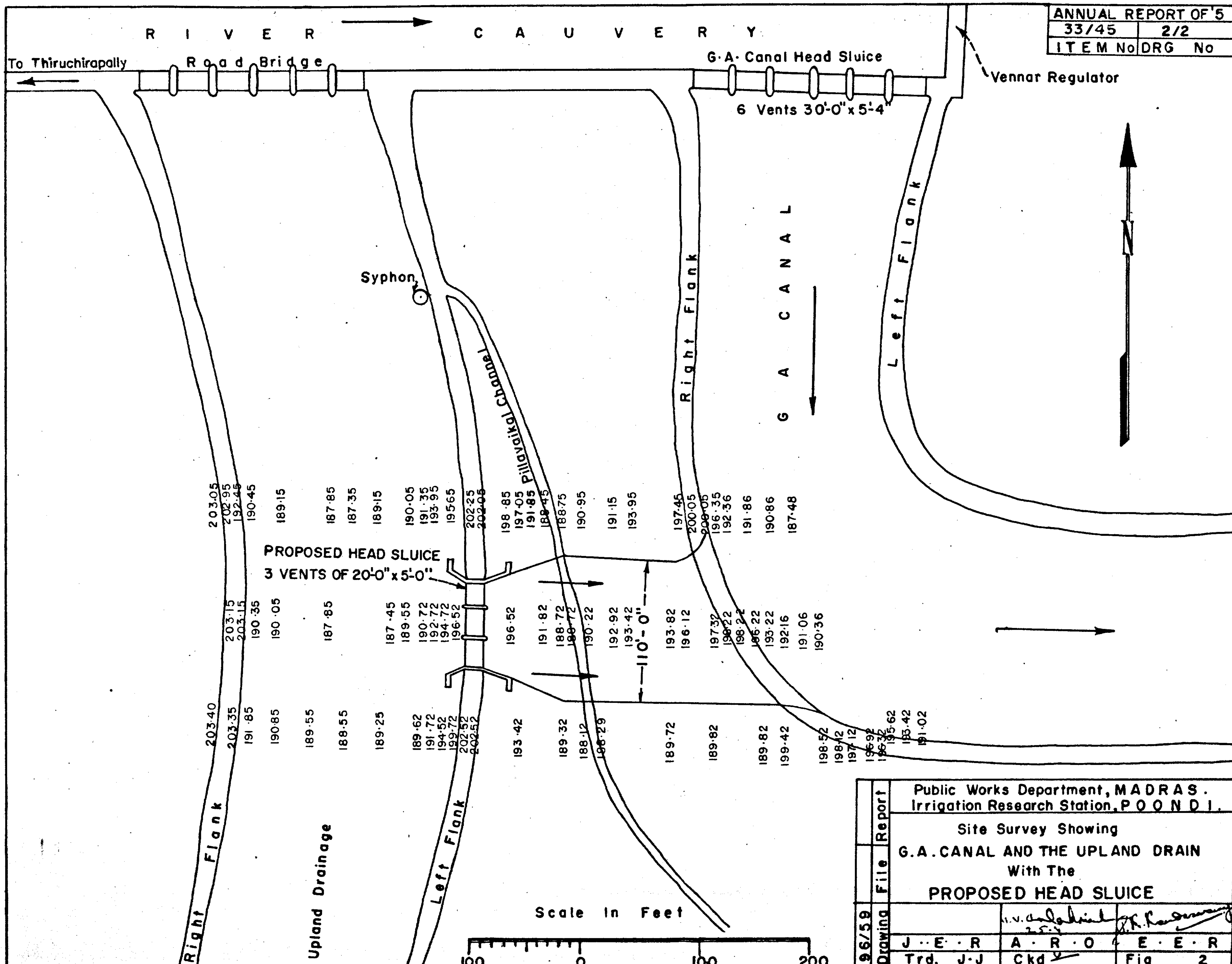

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RIVER MODEL EXPERIMENTATION WITH SEMI-RIGID MEDIUM—KEERANALLUR BRIDGE—STUDIES ON BANK EROSION.

(C.B.I. & P. No. B-12.)

Synopsis.

A model technique, for conducting studies on rivers and channels using semi-rigid material as the medium, has been developed in this Station, for the study of erosion of protected bunds by direct impact of the flow, the curvature of flow playing no significant part. The development of the technique, its limitations and probable pitfalls are given in the first part. A typical study on the erosion of the protected bank upstream of a bridge has been solved successfully with the aid of this technique and is presented in the second part.

PART I.

1. Introduction :

The equations for model scales derived from the regime equations of Sir Gerald Lacey are discussed below :

" No river or channel model is in regime but it is assumed that the equations apply equally to models which exhibit the same departure from regime either of scour or of fill, as their prototype. Regime equations are accepted as embodying the laws of similarity for all alluvial channels." [1]

The four basic equations derived for models from the original regime are as follows :—

$$W \propto Q^{\frac{1}{2}} \quad \dots \quad 1$$

$$V \propto f^{\frac{1}{2}} \; D^{\frac{1}{2}} \quad \dots \quad 2$$

$$V \propto \frac{(D)^{\frac{1}{2}}}{V} (RS)^{\frac{1}{2}} \quad \dots \quad 3$$

$$f \propto m^{\frac{1}{2}} = k m^{\frac{1}{2}} \quad \dots \quad 4$$

where W is the effective width,

Q is the discharge,

V is the mean velocity,

f is the silt factor $= K\,V^2/D$,

D is the effective depth,

R is the hydraulic mean depth,

S is the slope, and

m is the mean diameter of bed silt.

Employing the above four basic equations the following model equations can be derived :—

$$W \propto Q^{1/2} \qquad \ldots \quad 5$$

$$D \propto (Q/m^{1/2})^{1/3} \qquad \ldots \quad 6$$

$$D/W \propto \frac{1}{(mQ)^{1/6}} \qquad \ldots \quad 7$$

The last equation stated in a different form, i.e., $W/D \propto (mQ)^{1/6}$ indicates that as the discharge goes down the W/D ratio goes down ; in other words, a smaller channel is relatively narrower than a large channel. This conclusion demonstrates that vertical exaggeration is inevitable for all models of alluvial channels. It also indicates that in cross sections which are geometrically similar, the mean diameter of the bed silt would vary inversely as the discharge. Hence, a geometrically similar river model would require material of enormous grain size to simulate bed movements[1].

It is well known that curvature can be reproduced effectively in a model by giving vertical exaggeration. The amount of exaggeration is given by $V\,E = H^{1/3}$ or $V = H^{2/3}\,2$ where V E = Vertical Exaggeration V = Vertical scale.

2. Development :

In mobile bed models dealing with bank erosion, the vertical exaggeration entails a complication in that the material in the model bank is called upon to stand at a slope very nearly equal to or greater than the angle of repose, and so may slip at the slightest disturbance. When the curvature of flow is the dominant factor causing erosion, the approximation of tilting the bank back around the axis of water line is adopted. In these cases, models of lower vertical exaggeration with suitable vanes to reproduce curvature can also be adopted. But there are certain other channels where the formed and pitched banks are damaged not so much by the curvature of flow but by direct impact of the flow. In such cases, the necessity for vertical exaggeration being negligible, geometrically similar models are considered not only desirable but also acceptable. The other drawback of geometrically similar model is still there, namely, that these models will have to be moulded with large sized incoherent material like gravel. The red gravel is fragile and so loses its size gradually by impact. Hence the possibility of using a coherent or semi-rigid material has been examined.

Preliminary observations were made during the studies made on erosion of the bank of a stream named " Malattar Odai " and has been presented in the Annual Report of this Station for 1957. Mixtures of clay and sand in the proportions varying from 1 : 1 to 1 : 8 were prepared into rolls and dried in order to find the possibility of development of cracks. The model was moulded with these mixes and was operated to find their eroding properties. The 1 : 1 mix was found to develop cracks whereas the 1 : 8 mix showed considerable eroding tendency. A mix of clay to sand of 1 : 3 was found to be free from cracks and its eroding qualities were also found to be suitable and adequate. Thus, a model technique named as semi-rigid bed model came into being. As

per this technique the entire model should be moulded with this mixture and the discharge which would simulate the prototype scour after a reasonable number of hours of run has to be arrived at by trial.

Experiments should be conducted for this discharge-scale ratio.

3. Difficulties and Pitfalls :

Though the technique appears to be feasible, the following difficulties are likely to be met with while actually operating the models. River models being large, the quantity of material required is also considerable and consequently unskilled manual labour is utilised for preparation of the mix. As such, uniformity in the mix is difficult, if not impossible to ensure. Thus, an imperceptible human source of error is involved and this can be overcome only by strict technical supervision of the entire process of mixing. The presence of clay as one of the ingredients makes the lower portion of the model material saturated with water and at times vitiate all attempts at laying by providing a mobile oscillating medium. The saturated material beneath the surface prevents complete and uniform drying and induce heavy scour by retaining the moisture in some areas of the model. The entire area of the model under study requires removal to a sufficient depth and replacement with altogether new material for each study, and this increases the cost of running the model. Probable pitfalls are that improper mix may indicate too much or too little scour in comparison. Difference in drying of various areas under study may give misleading results. In addition the technique has its own limitation of being adoptable only for studies of localised action of direct impact of the flow.

4. Conclusion :

The technique is nevertheless. acceptable as a river model technique since river models are mainly used for visualising a problem, for verifying the data and for conducting comparative, qualitative studies. This technique permits the use of a geometrically similar model and provides the best means for visualising a problem in doing away with the inherent psychological effect that a vertically exaggerated model may produce in the mind of an experimenter. It is equally good ss a conventional mobile model constructed with incoherent material for verifying the data. It has been proved beyond doubt, that by overcoming the difficulties and avoiding the pitfalls enumerated in the paper, fairly accurate qualitative studies could be conducted.

PART II.

1. Introduction :

Keeranallur Bridge is situated at M. 15/7 of Mallapuram-Parappanangadi Road in Kerala State. This is resting on screw piles and had been constructed in the year 1906. The bridge and its immediate precincts were strengthened in the year 1940 as the foundation of the bridge were found to have been weakened by excessive floods.

Keeranallur River is a small stream with a catchment of 12 square miles at the bridge site. The bridge is situared in the river about 350 feet away from its confluence with River Kadalundi. In times of low flood in both the rivers or high flood in the Keeranallur alone the Keeranallur flows into Kadalundi but in times of high floods in the Kadalundi only or in both the rivers the Kadalundi flows into the Keeranallur, spills over its comparatively low banks and the flood joins the sea by a shorter route. The Keeranallur acts as an auxiliary flood carrier for the Kadalundi (vide Figure 1).

But the bridge in question being fairly old with inadequate waterway could not allow the high floods. With a view to restrict the magnitude of these floods, a cross bund was constructed across the entrance to the Keeranallur with a navigation vent 30 feet wide at base (vide Figure 2). This construction was completed in 1942. During the floods of that year the toe of the right of Keeranallur was scoured out and the revetment slipped in several places. The problem as referred to this Station is to analyse the causes of this erosion and suggest the necessary remedial measures to correct the flow and avoid the scour.

2. Causes for the Scour :

Field officials have concluded that the navigation vent or notch in the cross bund is the main source causing the damage. In view of the more rapid fall to the sea obtaining by the shorter route provided by the Keeranallur, there appears to have been a tremendous rush of water through the notch which has issued the discharge against the revetment and caused the scour.

In addition to this rapid fall it is also observed that by having the notch on the downstream (reckoned along the direction of flow in Kadalundi river) end of the cross head, the rapid flow has been forced to take a severe turn before entering the bridge and the approach is not axial. It is this oblique approach that appears to have damaged the abutment bank of the bridge. For the maximum flood discharge of 21,540 cusecs the linear waterway provided is meagre. This must have been the cause for the weakening of the foundations of the bridge prior to construction of the cross bund.

Studies have been conducted to arrive at the best solution for avoiding the erosion and for protecting the bridge abutment. Bridge-cum-regulator is one of the suggestions given by field officials.

3. Data :

Data pertinent to the studies are given below :

(1) *Kadalundi River*—

(*a*) Maximum flood discharge ... ... 81,540 cusecs.
Surface fall ... ... 1 in 2,640.
Water surface elevation near the notch ... 56.0.

(*b*) Dominant discharge ... ... 30,090 cusecs.
Surface fall ... ... 1 in 2,640.
Water surface elevation near the notch ... 47.2.

(2) *Keeranallur River*—

(*a*) Maximum flood discharge ... ... 21,540 cusecs.
Surface fall ... ... 1 in 2,640.
Water surface elevation at bridge site ... 55.65.

(*b*) Dominant discharge ... ... 4,450 cusecs.
Surface fall ... ... 1 in 2,640.
Water surface elevation at bridge site ... 47.0.

4. Model :

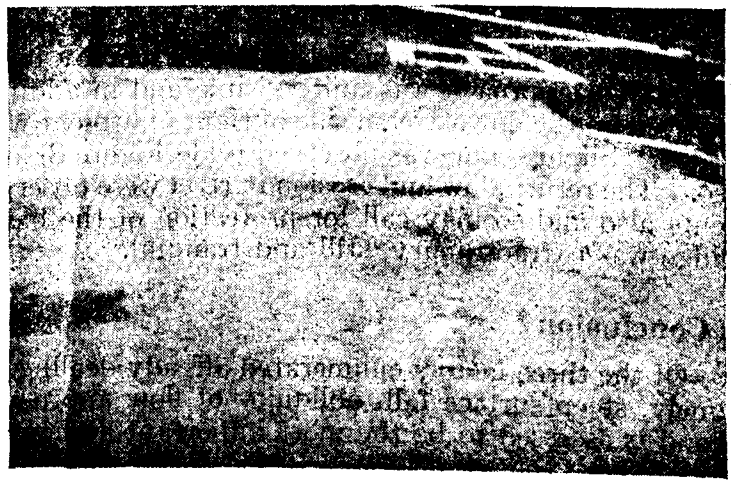
A geometrically similar model to a scale of 1 : 48 was constructed. The Kadalundi River was laid in sand and the Keeranallur river was moulded with the mix of clay and sand (1 : 3). The bridge and the cross bund with the notch were made of teakwood and fixed in position.

After a few trials a discharge of 2.5 cusecs was fixed as model dischrge to give the scour pattern in conformity with the prototype. The flood levels observed in the field at the notch and at the bridge sites were maintained in the model. Heavy scour was observed right under the bridge, which never showed any signs of ceasing. This confirmed the view that the linear waterway of the bridge is inadequate. To simulate the site condition, the floor of the bridge and its apron were made rigid. With these modifications, the model was run fixing the time of each experimentation as 12 hours. When the model was finally run through 12 hours, scour was observed in the corresponding places of the model and prototype, i.e., upstream of right abutment and downstream of cross bund AB (vide Figure 3).

5. Experiments and Discussions :

Experiment 1.—As suggested by many field officers the cross bund AB and CD shown in Figure 3 was removed. The actual removal of the cross bund was made after the flow became steady in order to observe the increase in flow into Keeranallur. The flow did not increase but had a smooth entry into Keeranallur and consequently there was no scour anywhere downstream of cross bund AB except at the bridge abutment (vide Figure 4).

However this experiment indicated that the inadequate linear waterway of the bridge controlled the flow rather than the cross bund and this is not a desirable feature as the bridge may be seriously damaged if sudden and excessive floods occur. Therefore, it has been concluded that flow control at the entry would be desirable.



Smooth entry of flow into Keeranallur with the cross bund removed.

Experiment 2.—To control the flow at the site of the cross bund, the notch itself was proposed to be used. But it was shifted upstream (reckoned in the direction of flow in Kadalundi river) by lengthening the cross-bund AB. Another cross bund AB (vide Figure 5) was placed as a submerged repelling groyne upstream of the bridge. Severe scour was observed on the right bank of Keeranallur downstream (reckoned in the direction of flow in Keeranallur river) of the submerged groyne and also at the junction of the left bank of the Keeranallur river and the left bank of Kadalundi river (Point C). The flow overtopping the bund A[1] B[1] was observed to continue to have oblique approach and the failure of the submerged groyne indicated the futility of correcting the flow approach by any such device.

Experiment 3.—The top of bund AB was raised above water level and the submerged groyne was removed. The sharp point C was smoothened out with a curve of radius 80 feet. The flow smoothly entered the river and there formed a still pool which acted as a water barrier in front of the right bank of Keeranallur. But obliquity of flow to the bridge was still present and scour was observed at the right abutment though not to the extent observed previously (vide Figure 6).

Experiment 4.—The only solution appeared to be appropriate is to re-orient the bridge alignment so as to suit the flow approach in a straight direction and so it was shifted by 50 feet towards right bank of Keeranallur. The right abutment was shifted towards the notch to give a horizontal tilt of 7°. This proposal indicated no scour (vide Figure 7).

Experiment 5.—The next studies were conducted on the proposal of a bridge-cum-regulator. The bridge and cross bund AB and CD were all restored to their original position as shown in Figure 8. Shutters were installed for all the four spans of bridge. The total discharge let into the model was maintained constant. The discharge into the Keeranallur river was reduced successively to a quarter, half and three-fourths of that which was allowed originally by operating all the four shutters. While quarter and half the discharge did not give any scour three quarter discharge indicated conspicuous scour at the abutment (vide Figure 8).

Experiment 6.—The cross bund AB and CD were removed. Three quarter discharge was allowed into Keeranallur river by operating the shutters. The obliquity of approach was quite visible and in addition severe return flow was observed on the downstream side of right abutment with the bank being severely eroded. Slight scour was observed (vide Figure 8) an upstream right abutment also. The return flow and erosion in river were observed for the previous experiments also and so may call for protection of the bank. The flow upstream of bridge was extraordinary, still and tranquil.

6. Conclusion :

Of the three factors enumerated already dealing with the causes of damage, namely, steep surface fall, obliquity of flow approach and control of flow, the second is assessed to be the most important and predominating over the other two. When the model was tested with regulator-cum-bridge and oblique approach the scour produced indicates that the flow will have to be corrected, even if the surface fall is reduced by way of fixing up the shutter to the bridge. Scour observed in experiment 1 with the cross bund AB and CD removed indicates that removal of cross bund alone does not correct the flow to the desired extent and this is also borns out by the last experiment. It is obvious that the flow control at the site of the bridge is not desirable in that the excess flow will have to be allowed to spill over the right bank if some more serious damage to the bridge is to be avoided. The cross bund AB (vide Figure 5) does fulfil the two-fold objects of controlling the flow and also training it to some extent. Since a new structure is proposed to be built advantage of the opportunity has been taken to re-orient the structure to suit the flow which has been partly trained by cross bund AB. Since the mere shifting of the existing bridge alone has proved so effective in correcting the flow, a new bridge with adequate linear water way is expected to function better.

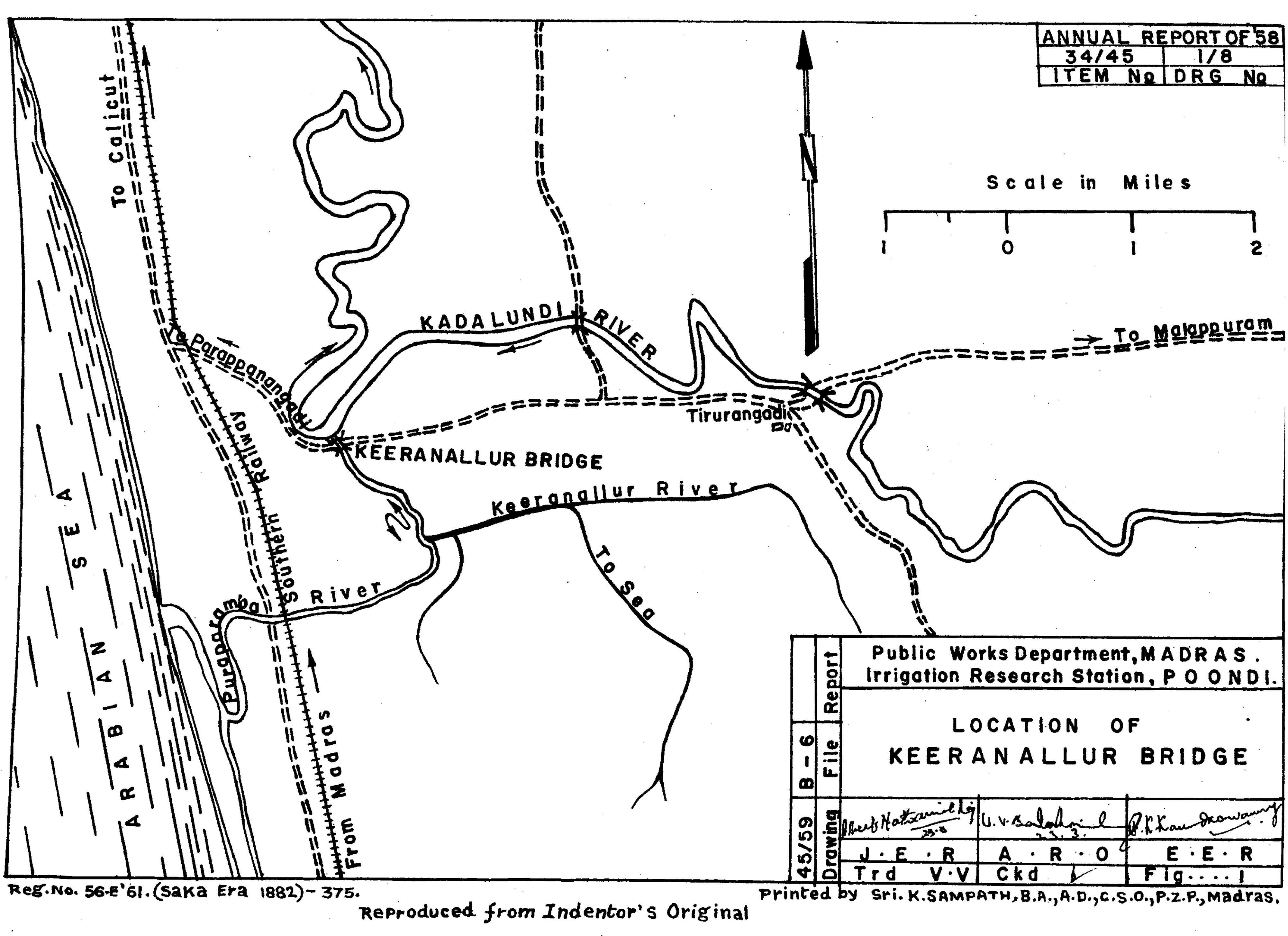
Shifting of the navigation vent and training the left bank at the offtake of Keeranallur as given in Figure 7 are recommended. The new bridge should have sufficient waterway and the right abutment should be farther away from Malappuram than the right abutment of the bridge in the shifted position. The navigation vent should be designed as a controlling section to allow the design flood discharge to be allowed into the Keeranallur. The silted up bed should be excavated to form a channel for the corrected flow.

The bridge-cum-regulator with strong bank protection works considered necessary (vide experiment 6 appears to be too elaborate a proposal, which will no doubt result in some additional advantages ; but it may not be commensurate with the extra cost involved.

7. References :

(1) Article on " The selection of Alluvial model scales " by Gerald Lacey, page 91 of Annual Report (Technical) of C.B.I. India—1942.

(2) Article on " Reproduction of curvature in a model," page 9—Annual Report on Indian Waterways Experiment Station, Poona for the year **1943.**



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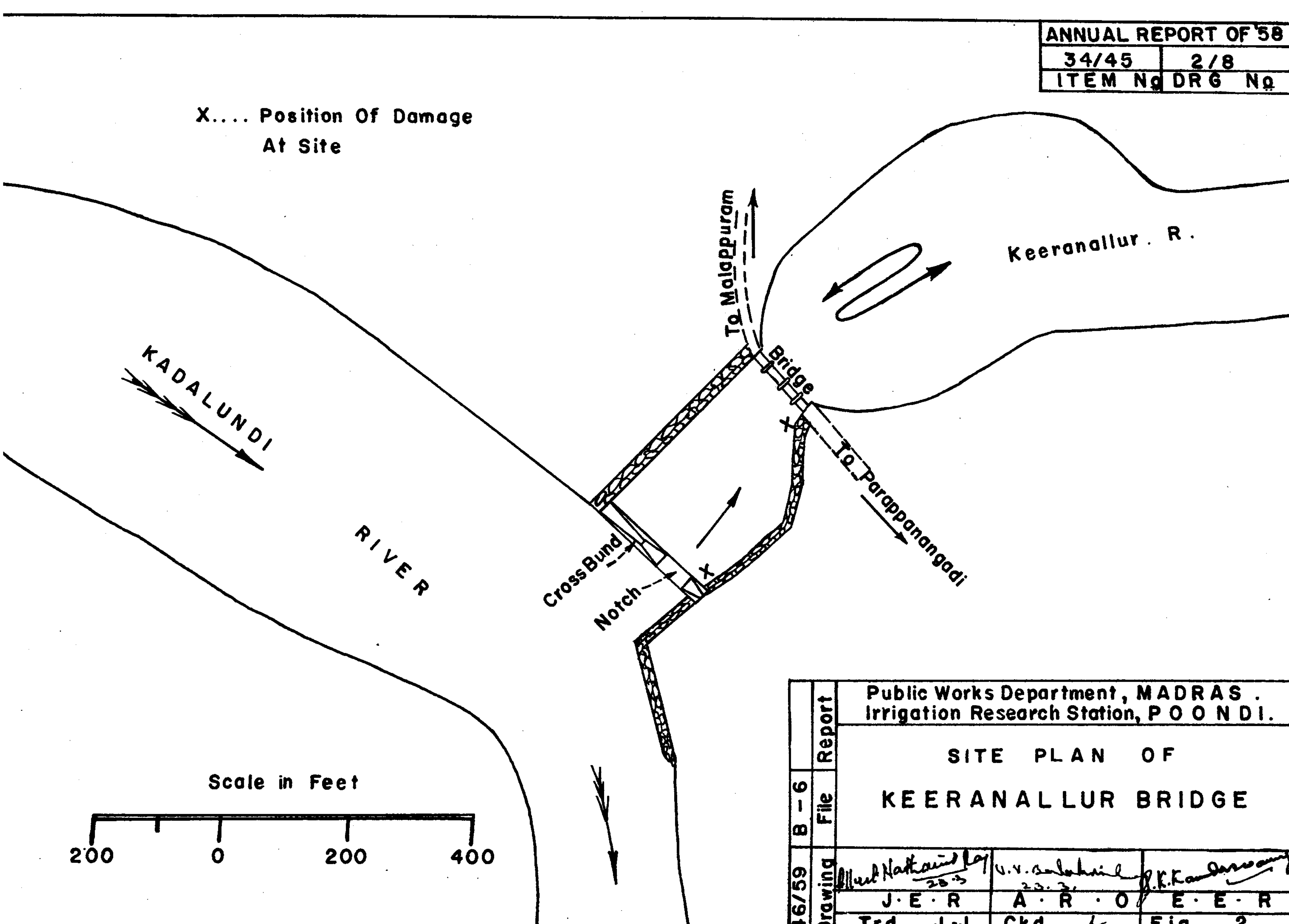

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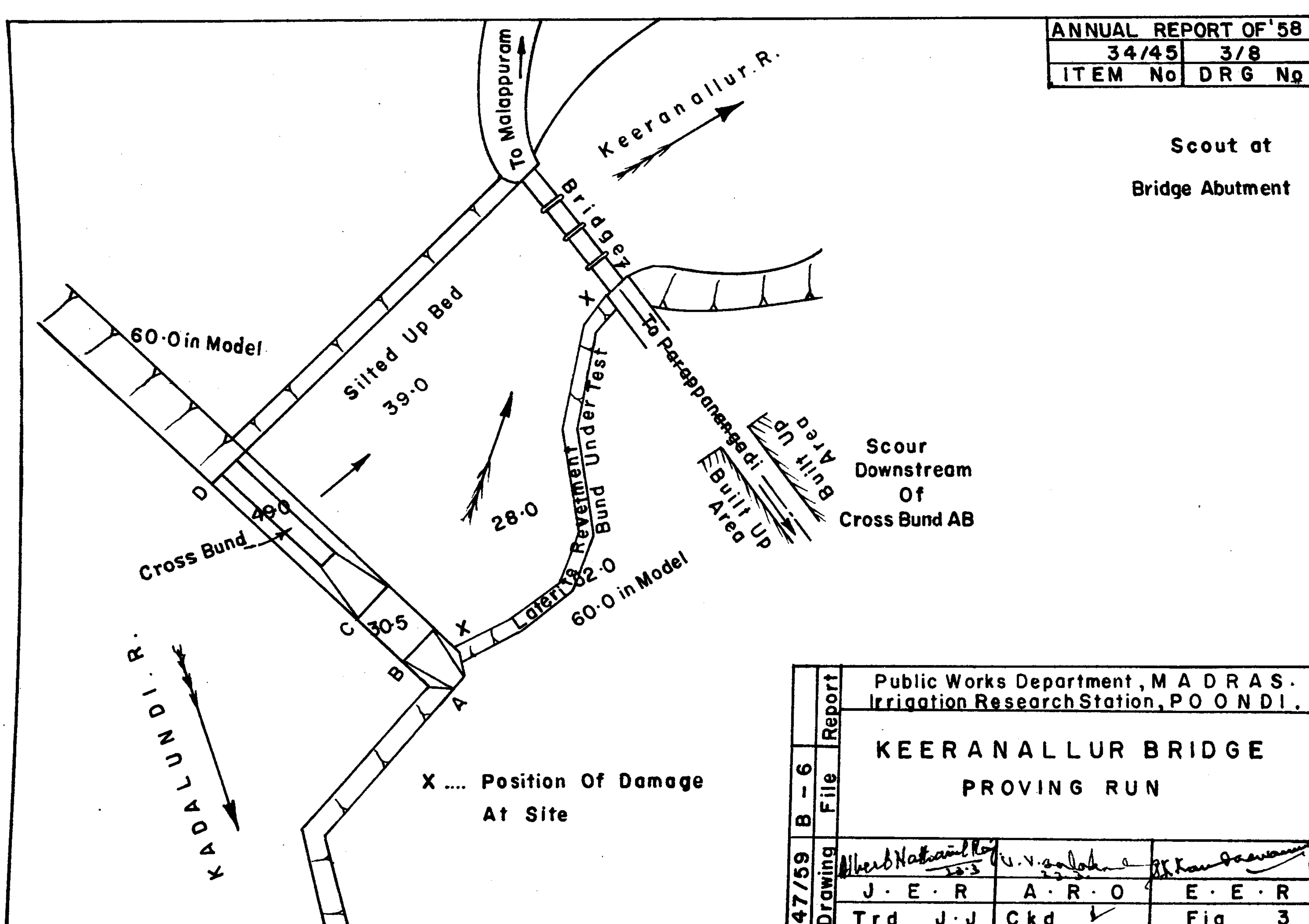

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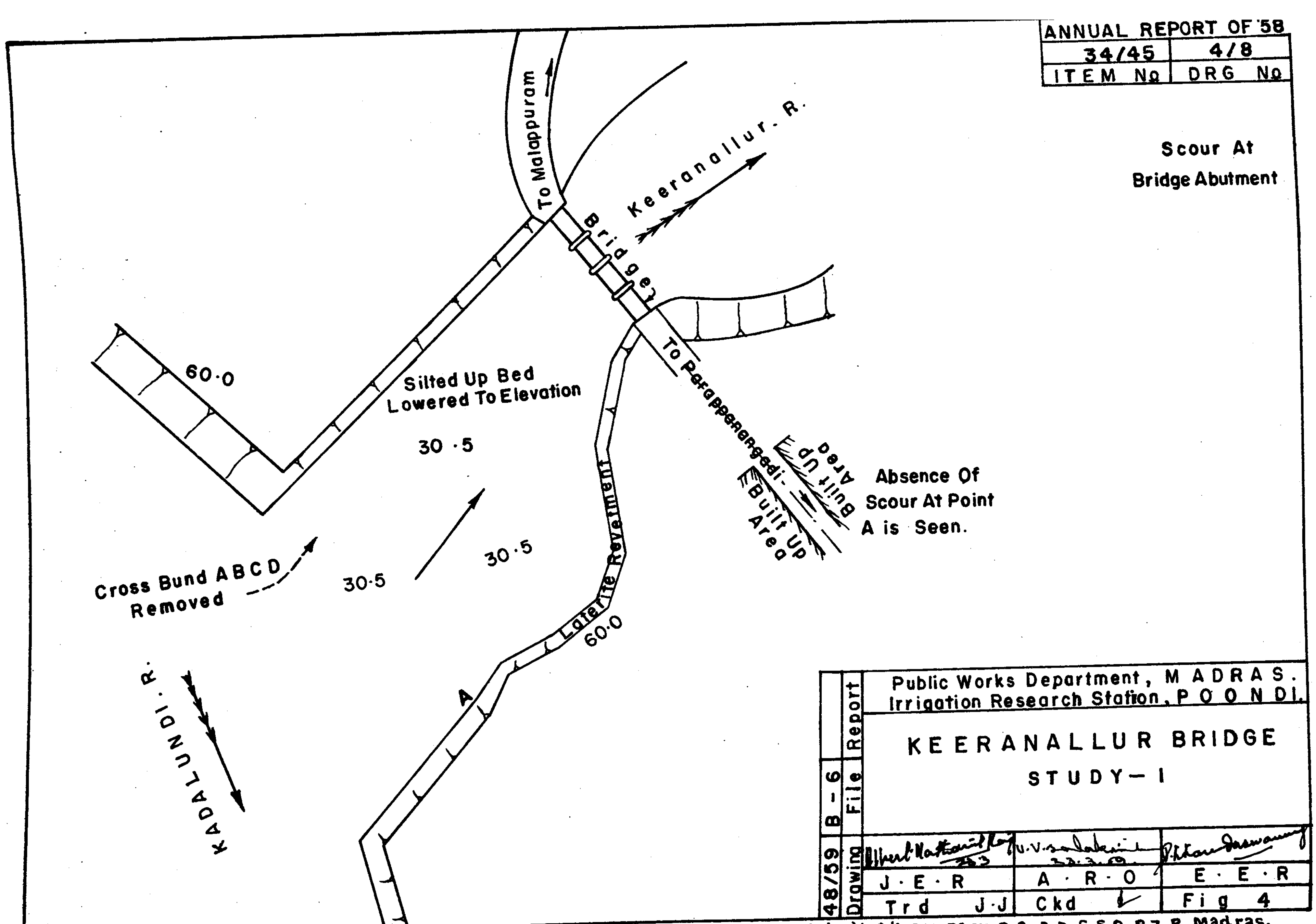

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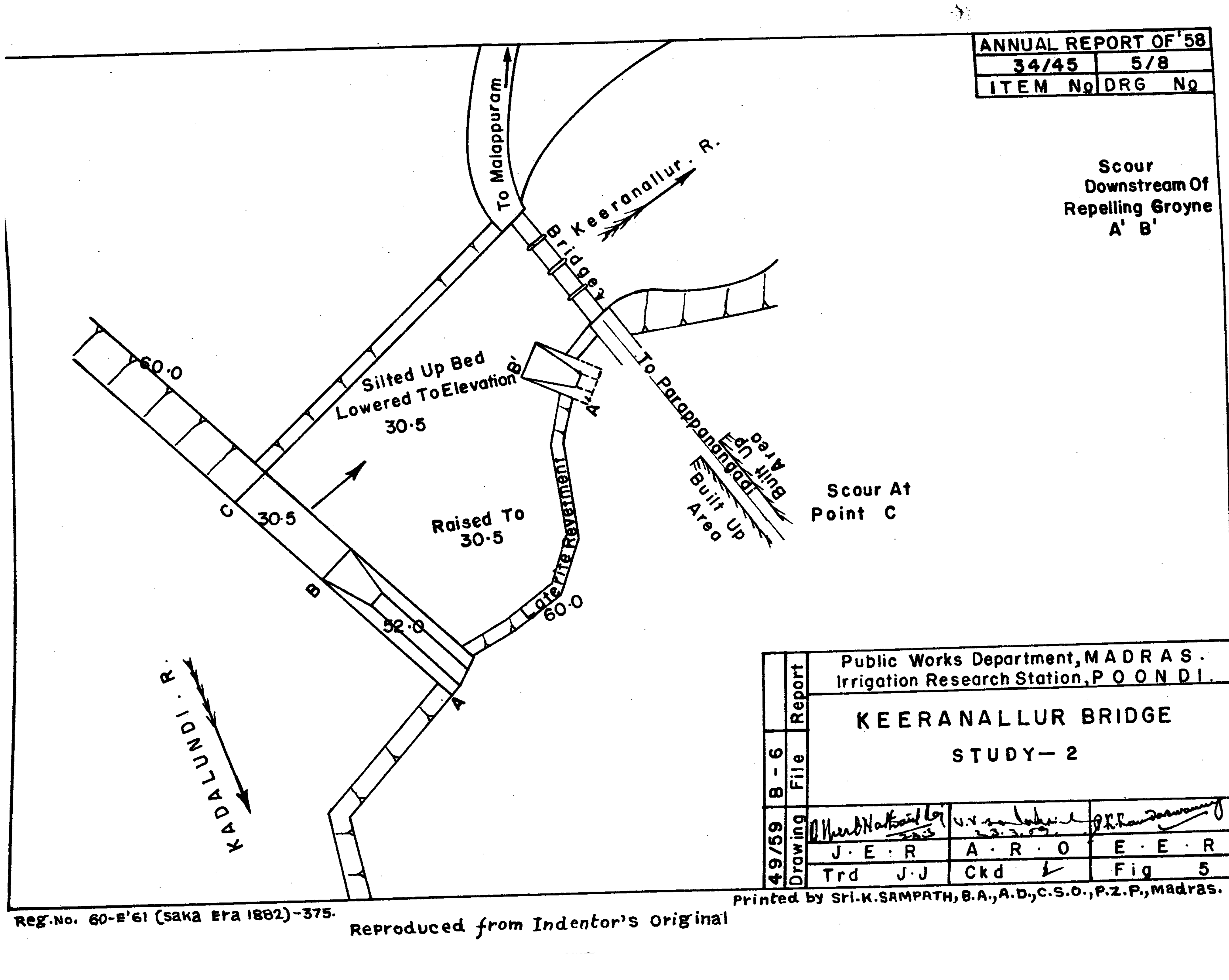

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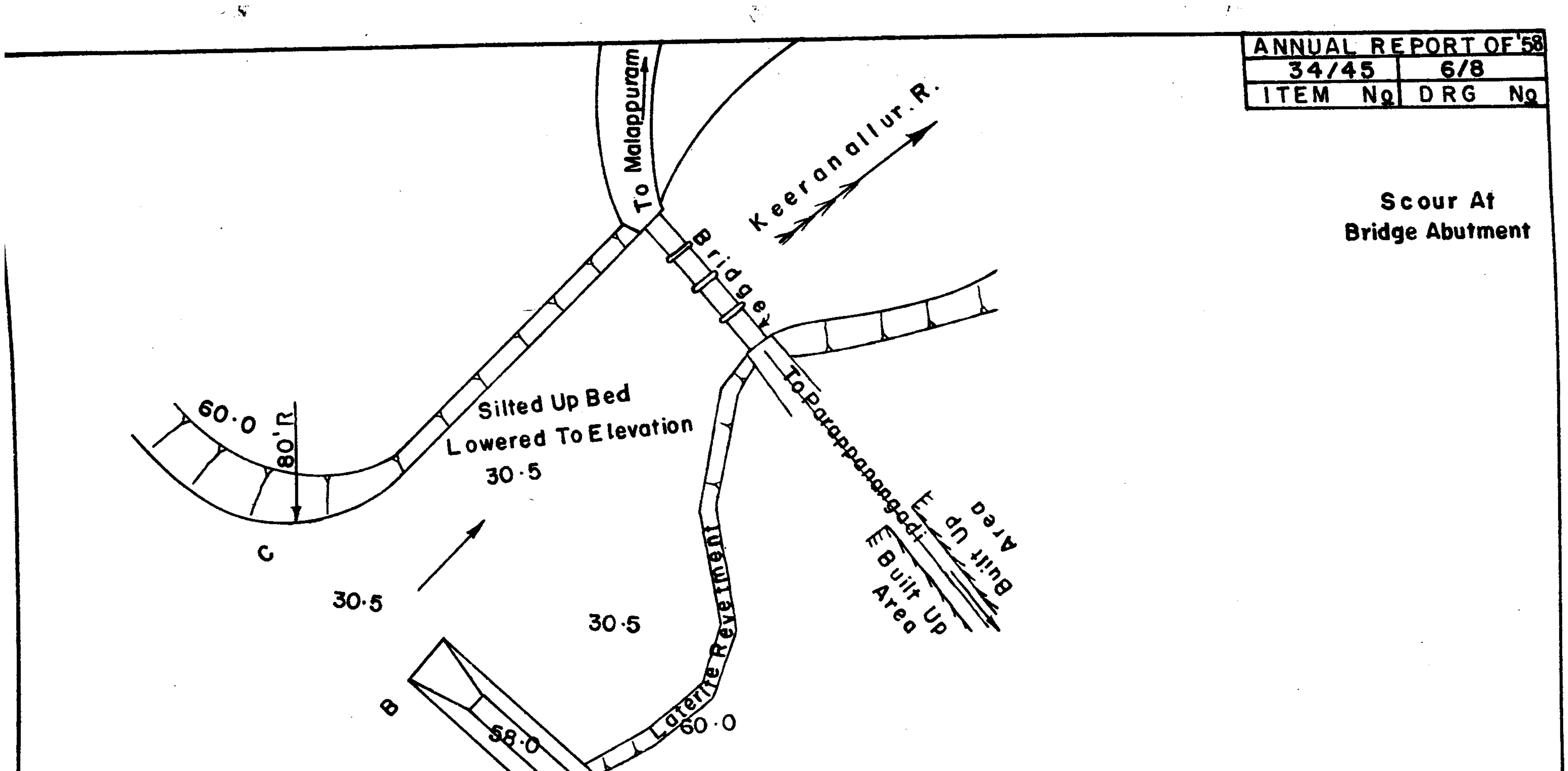

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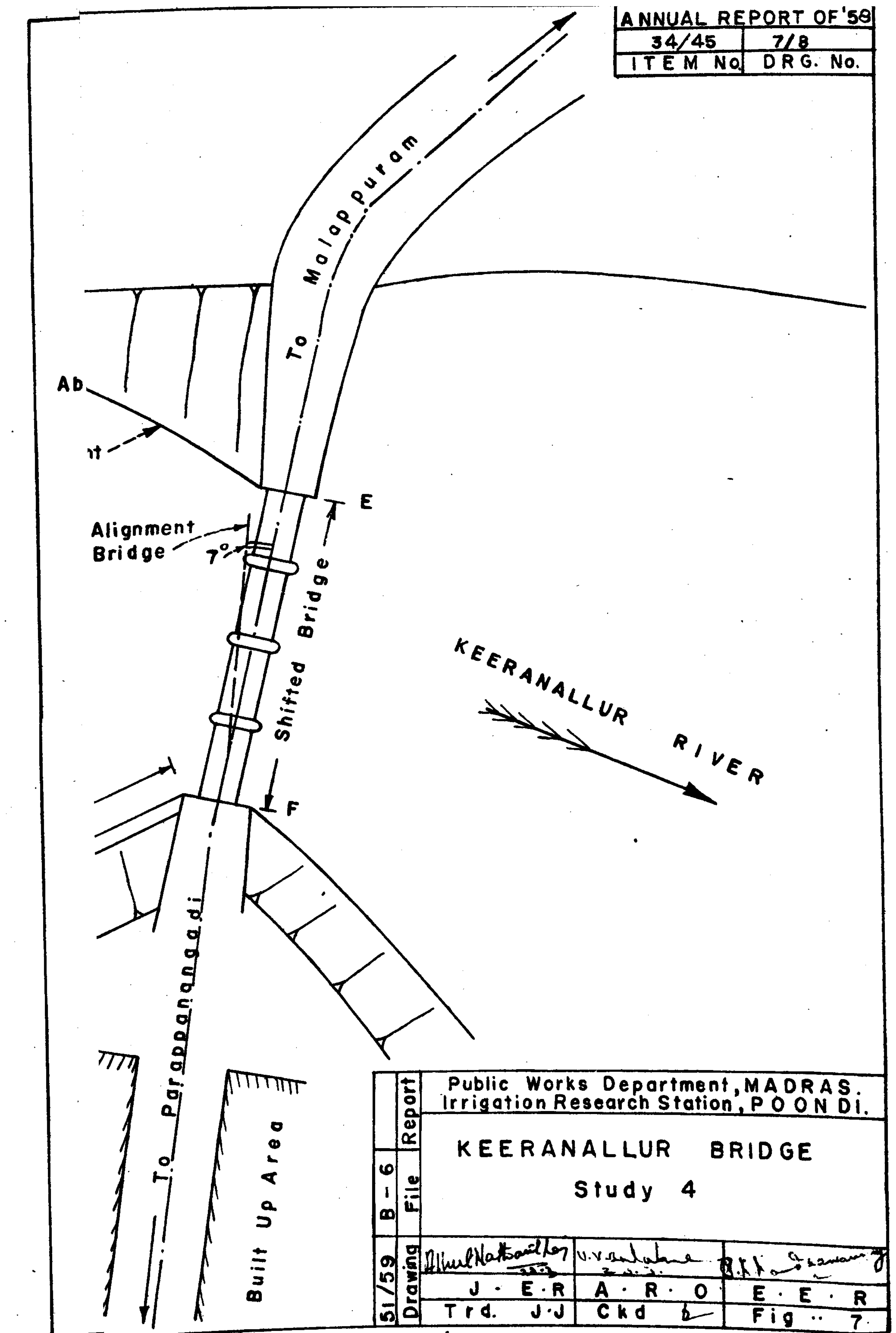

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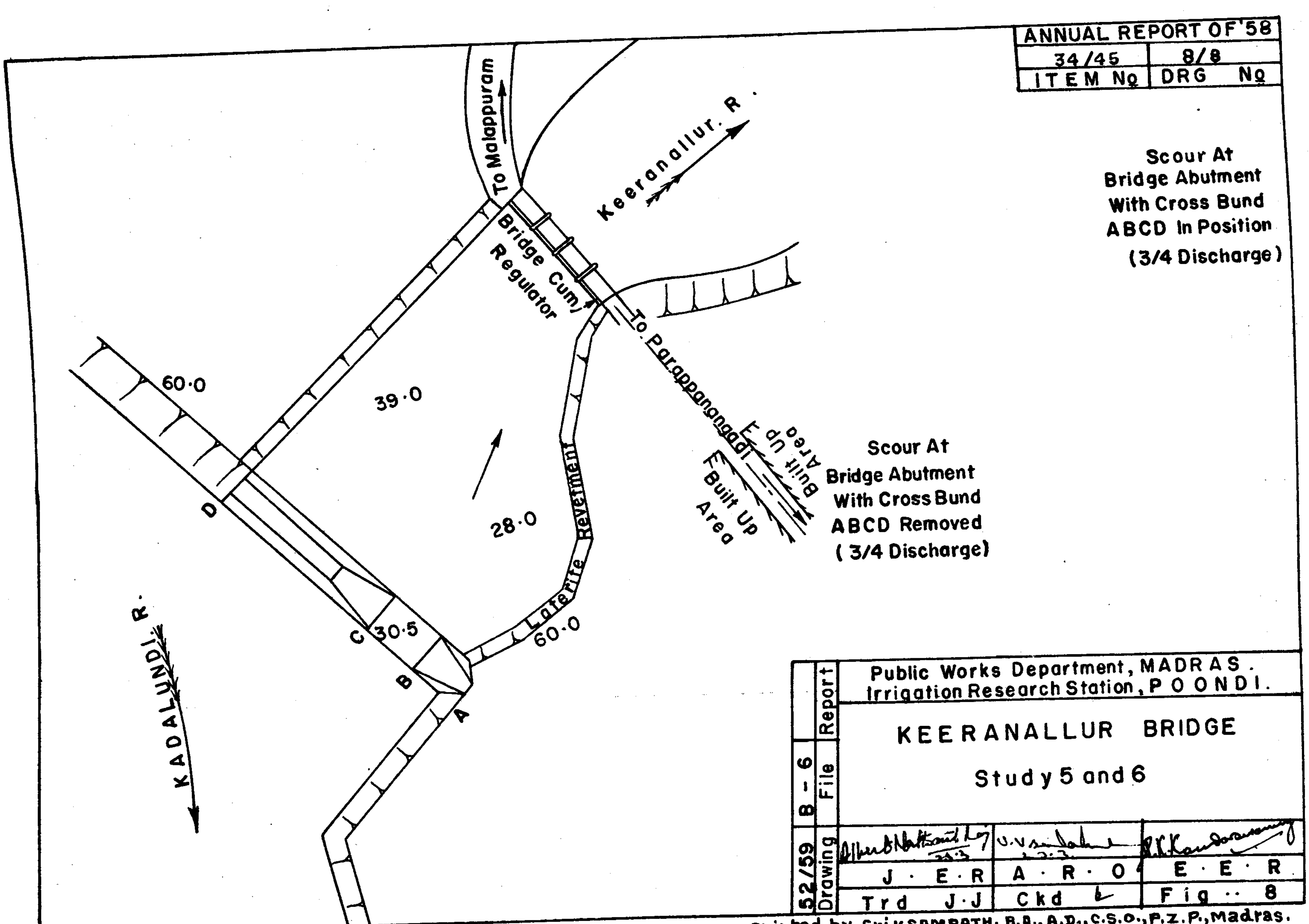

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AVALANCHE SCOUR VENT ENERGY DISSIPATION STUDIES.

(C.B.I. & P. No. C-2 (ii).)

1. Introduction :

The Avalanche dam forms part of the Kundah Hydro-electric scheme. A design for energy dissipation arrangements in rear of the scour vent of the dam was recommended. Subsequently it was found that the top elevation of the basin and the apron as constructed were different from those recommended from this Station and as such certain modifications in the design were found to be necessary. This report is about the revised design evolved in order to suit the field conditions resulted from constructions.

2. Data :

(1) M.W.L.	El. 6515.00.
(2) Sill of scour vent	El. 6350.00.
(3) Size of vent	5′ × 7′.
(4) Top elevation of basin	El. 6,327.0 as adopted in the design recommended.
	El. 6,330.00 as found suitable in the field.
(5) Top elevation of apron	El. 6,336.0 as adopted in the design recommended.
	El. 6,328.0 as constructed in the field.
(6) Width of basin	30 feet as adopted in the design recommended.
	24 feet as constructed in the field.

3. Problem :

In view of the great disparity in the levels of the basin and apron between as adopted in the design recommended from this Station and as constructed in the field, the whole design had to be revised and the revised one should consist in refixing the height of the baffle wall and in finding out suitable friction blocks to be placed inside the stilling basin.

4. Experiments :

4.1. Conforming to the construction in the prototype, changes were incorporated in the model by reducing the basin from 30 feet to 24 feet in width and raising its floor level from 6,327 to + 6330 and lowering the apron from

6,333 to + 6,328 with the result that the basin got higher than the apron by 2 feet as against the fact that it was lower by 6 feet in the previous design. However with the baffle wall with its top at El. + 6,335 as in the previous design, a depth of 5 feet was created for the basin and experiments conducted for M.W.L. in the reservoir. Energy dissipation was found to be satisfactory only for partial gate operation up to half vent, beyond which the jet issuing out from the sluice was found dashing against the baffle wall with violent noise and escaping over it in an unstable and undesirable manner. With a view to obtain a stable hydraulic jump, the baffle wall was raised by 1 foot, 2 feet and 3 feet and for each case studies were made.

Raising of the baffle wall to + 6,338 by 3 feet above the level 6,335 recommended previously was found absolutely necessary to produce hydraulic jump when full vent was in operation. It is to be noted that depth of the stilling basin required is 8 feet and it is same as that of the stilling basin recommended previously.

4.2. After the top elevation of the baffle wall was finalised, experiments were conducted to improve the performance of the basin. Various types of blocks and different rows of blocks were tried and as a result 2 blocks of size 4′ × 3′ × 4′ each placed symmetrically at a distance of 25 feet from the toe of the glacis were finalised as sufficient to effect satisfactory energy dissipation.

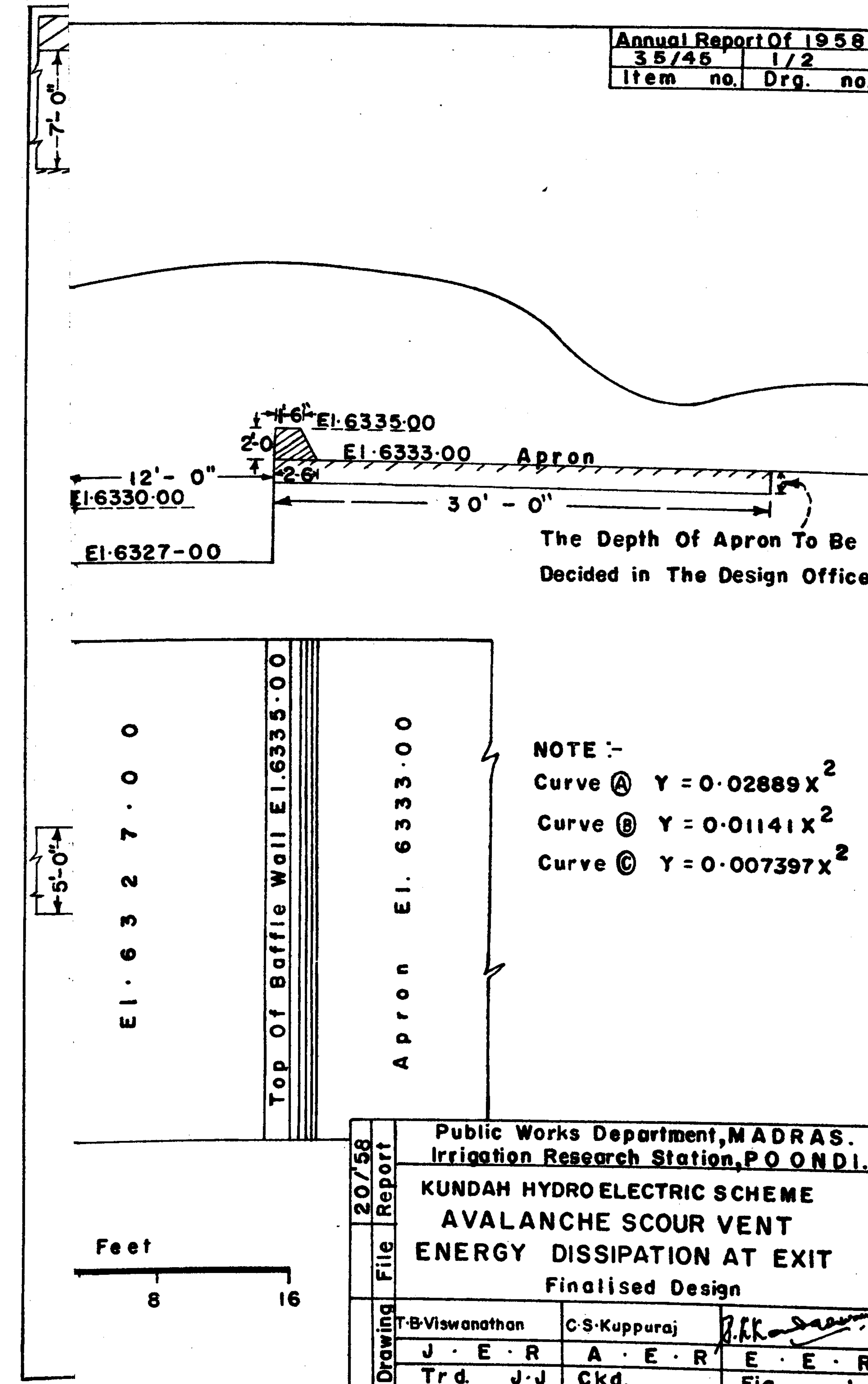
4.3. The length of basin was 44 feet in the design recommended previously. But due to raising of the level of the basin its length got increased to 53 feet though encroaching on the glacis. With a view to economise the length of the basin, studies were made for various lengths below 53 feet and the length 45 feet was found sufficient.

4.4. Though the site of the basin and apron is of rocky nature, studies were made also on the velocities of the flow immediately in rear of the baffle wall. The velocities observed were in the order of 20 to 22 feet per second as against 17 feet per second in the previous design. This high velocitiy was expected because of the baffle wall being 10 feet high above the apron and as such it could not be avoided. However, it is to be noted that as the place is rocky it can withstand very well the high velocity of 22 feet per second and therefore doubts need not be entertained as to the safety of the structure.

4.5. For the design of the divide wall, the one between the scour vent and the spillway, top elevations of the water surface profile were taken and plotted. The top elevation of the standing wave in the stilling basin was observed to be + 6,349 and therefore it is necessary to have the top of the divide wall at + 6,350.

5. Conclusion :

The design which has been recommended for adoption in Irrigation Research Division Report No. 7 L/58 had to be revised in the light of the changes made by the field authorities in the actual construction of the structures. The revised design consists of a stilling basin of length 45 feet formed by a baffle wall at the end with the top at + 6,338 and provided with 2 blocks of size 4′ × 3′ × 4′ and placed symmetrically inside at a distance of 25 feet from the toe of the glacis. The top of the divide wall should be at + 6,350. The whole design is shown in Figure 2 and it is recommended for adoption.



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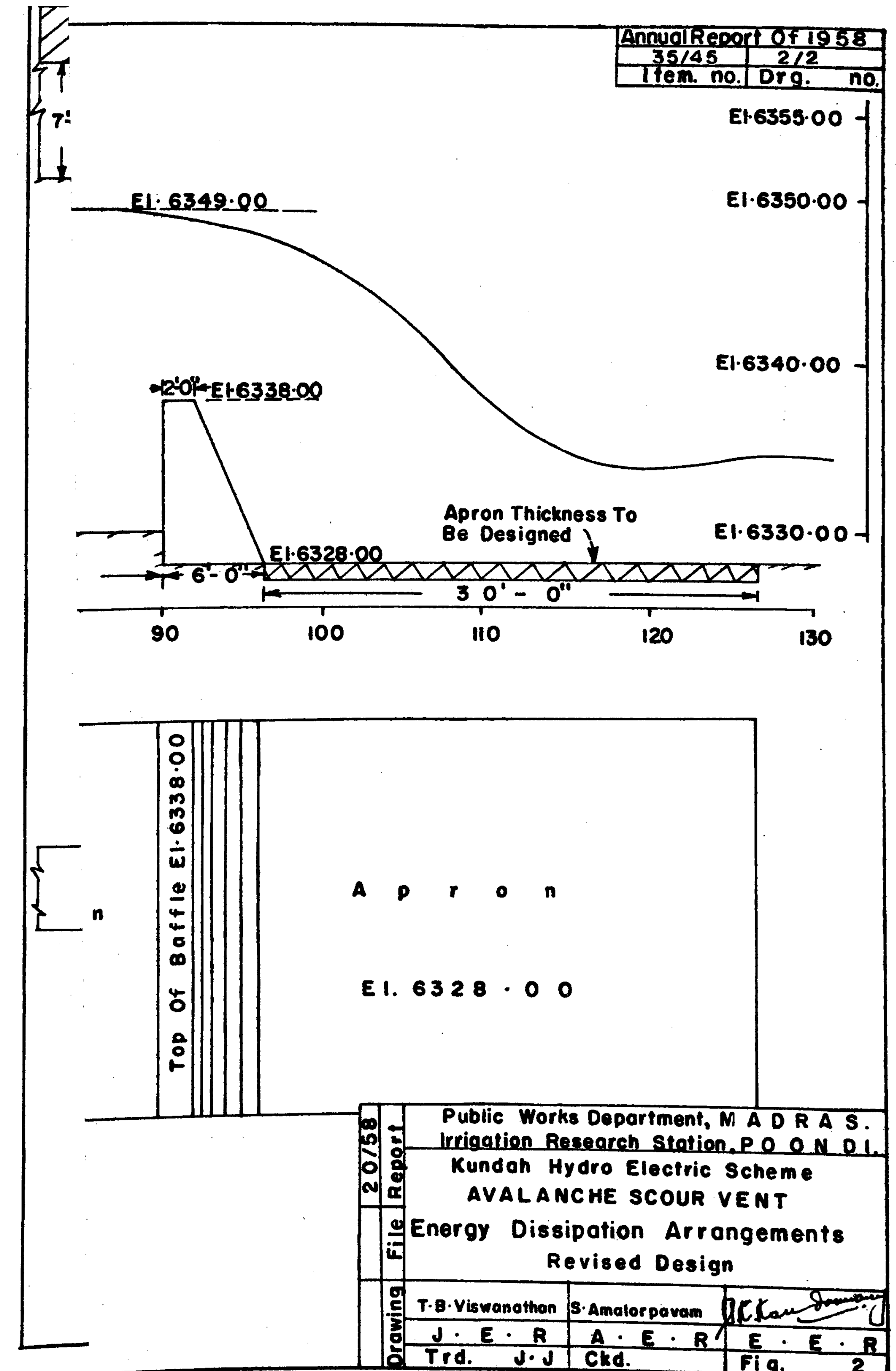

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AVALANCHE DAM COEFFICIENT OF DISCHARGE IN SCOUR VENT.

(C.B.I. & P. No. C-2 (ii).)

1. Introduction :

1.1. In the first stage of the Kundah Hydro-Electric Scheme, three dams are under execution of which Avalanche Dam is one. This is being constructed across the Avalanche river which is a tributary of river Kundah.

1.2. In this dam, one scouring vent is provided to prevent silt accumulation in the vicinity of the offtake of the low pressure pipe line.

2. Data :

(i)	Average bed level (downstream) ...	El. 6328.0
(ii)	Top of dam	El. 6520.0
(iii)	F.R.L.	El. 6515.0
(iv)	Sill of scouring vent	El. 6350.0
(v)	Size of vent	5′ × 7′
(vi)	Basin Elevation	El. 6327.0
(vii)	Apron Elevation	El. 6333.0
(viii)	Top of baffle	El. 6335.0
(ix)	Width of basin	44′ 0″
(x)	Size of vent at Exit	10′ × 3′ 1″

3. Problem :

Experiments for the design of the exit portal and the energy dissipation arrangements have been conducted in this station previously and suitable recommendations sent separately. Now the discharges for various openings and for heads ranging from maximum water level to minimum draw down level have to be determined and suitable calibration charts drawn.

4. Model :

4.1. A model to a scale of 1/20 was made in teakwood and was installed in one of the bays of high head tank having a maximum head of about 14′ 0″. For this model a head of 8.25 feet was required above sill of vent. The finalised designs of both energy dissipation and exit portal were incorporated in the model.

4.2. For measurement of discharges, a ' V ' notch was provided downstream of the vent. Suitable honeycombs were provided above the ' V ' notch to make the water flow uniformly over the 'V' notch. A gauge well was provided between 'V' notch and the honeycomb to measure with accuracy the head over the 'V' notch.

4.3. The gate opening was regulated by a shutter provided with screw gearing arrangements. The measurement of the opening was done with the help of a pointer attached to the top of the shutter rod.

4.4. The water level inside the tank was indicated by a water level indicator which functioned by a simple 'U' tube arrangement.

4.5. *Model Scales* :

Linear scale	1/20
Discharge scale	1/1789
Velocity scale	1/4.48
Time scale	1/4.48

5. Experiments and Discussion of Results :

5.1. Experiments were done for the following gate openings :—

(1) Full gate.

(2) 3/4 opening.

(3) 1/2 opening.

(4) 1/4 opening.

For each opening, observations were made for various heads at intervals of 10′ 0″ ranging from the maximum water level to the minimum draw-down level. For each reading, after adjustment of head, about 15 minutes were alloweed to make the flow over the 'V' notch and in the gauge well steady. Also before the start of the experiment the pointer was checked for true indication of the opening.

5.2. For this study, the finalised energy dissipating arrangements were incorporated in the model (vide Figure 1). The discharges obtained are plotted and discharge characteristics are furnished in Figure 2. The value of observed discharges for various heads are given in Statement I.

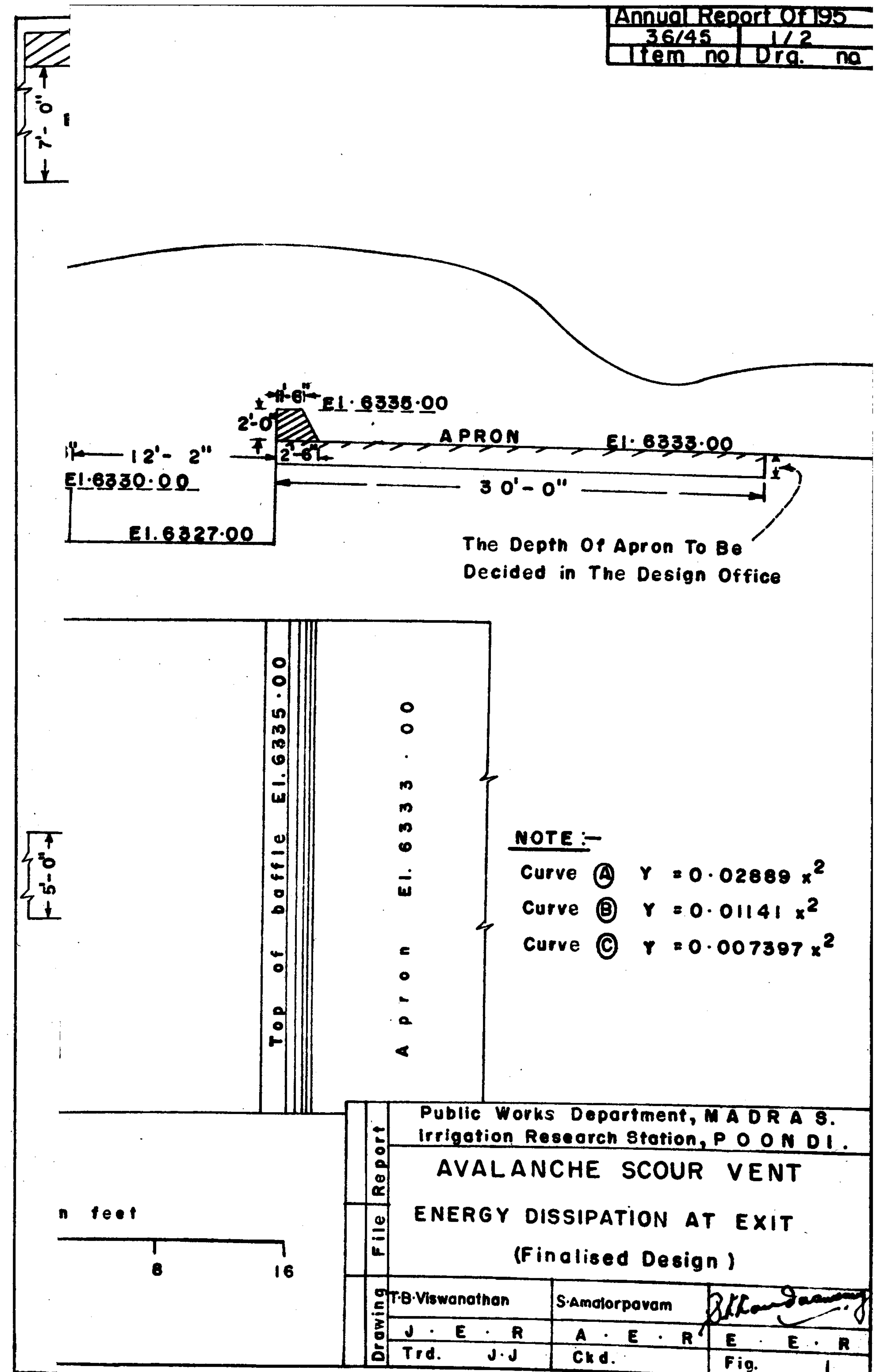
6. Summary :

Avalanche dam forms part of the Kundah Hydro-Electric Scheme for which the scour vent was designed at this Station. Experiments for the determination of coefficient of discharge for this sluice were conducted and results are given in Figure 2 and Statement I.

STATEMENT I.

STATEMENT OF DISCHARGES FOR VARIOUS OPENINGS AND DIFFERENT UPSTREAM WATER LEVELS

Water surface elevation.	*Discharge in cusecs for the opening.*			
	Full.	3/4	1/2	1/4
6,515	1,783	1,510	1,240	795
6,495	1,680	1,420	1,155	715
6,475	1,550	1,340	1,070	655
6,455	1,420	1,250	985	585
6,435	1,280	1,155	875	515
6,415	1,120	1,035	760	440
6,395	930	868	650	354
6,375	690	615	475	230



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M·V

HEAD OVER SILL IN FEET

NOTE :—

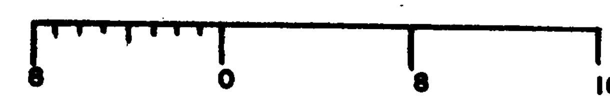
Coefficient Of Discharge = 'C'

In the formula $Q = Cd \ A\sqrt{2gh}$

Where $'C' = Cd\sqrt{2g}$

$A = 7 \times 5 = 35$ sq.ft.

Scale in feet



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Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.
File	AVALANCHE SCOUR VENT Cd EXPERIMENTS DISCHARGE CHARACTERISTICS

Drawing	T.B. Viswanathan	S. Amalorpavam	[signature]
	J · E · R	A · E · R	E · E · R
	Trd. J· J	Ckd.	Fig. 2

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KUNDAH HYDRO-ELECTRIC SCHEME—KUNDAHPALAM SCOUR VENT—ENERGY DISSIPATION STUDIES.

(C.B.I. & P. No. C-2-(ii))

1. Introduction :

The final recommendations for the energy dissipating arrangements below the Kundahpalam scour vent were furnished in the Annual Report for the year 1957. As the recommended design was altered during construction to avoid heavy rock cutting it was necessary that the changes were incorporated in the model and the flow observations made. Blue prints of the changed design were also received for conducting model studies after incorporating the changes. This report deals with the observation made in the model with the alterations proposed at the site, and with the suggestions of remedial measures for correcting the flow.

2. Problem :

To observe the flow conditions for the revised arrangement of the stilling basin (skew in plan with respect to the Central line of the Scour Vent) and

Offer comments on the performance of the energy dissipation arrangement (vide Figure 1).

3. Experiment ;

3.1 The experiments were conducted on the same models section and 3 dismensiona' constructed previously to a scale of 1/24 and 1/50 respectively. The salient details of the model after incorporating the changes were as below :—

Maximum water level in the reservoir ...	El. 5330
Stilling basin floor	El. 5240
Width of the basin	22 feet.
Apron elevation	El. 5249.
Depth of basin	12.5 feet. (9 feet sunk basin and 3.5 feet baffle).

The left side flare of 1 in 4 at the exit end of the scour vent was continued in the model upto the end of divide wall between the spillway stilling basin and the scour vent stilling basin. The right side wall was turned to be parallel to the left side wall at a distance of 22 feet. The baffle wall was kept at right angles to the left side wall at the same location as was recommended previously (at 55 feet) (vide Figure 1).

3.2. Flow observations were recorded for the following 3 (three) conditions:

(*a*) With the altered design as furnished by the Superintending Engineer, Technical (Civil) in his letter, dated 15th December 1958 with a pool of length 55 feet, width 22 feet, and depth 12.5 feet, the pool being skew in plan as shown in Figure 1 and the baffle wall kept at right angles to the flow in the basin.

(*b*) With the right side guide wall kept straight from the exit end of the scour vent (from where the exit width is 16 feet) to a point where the baffle wall is located (i.e., with a width of pool varying from 16 feet to 22 feet) and with the baffle wall at right angles to the flow in the basin (vide Figure 1).

(*c*) With the same arrangements as given in (*b*) but with the baffle wall kept parallel to the exit end of the scour vent.

3.3. The flow observations for each of the 3 cases mentioned above are given below :

(*a*) Due to sharp turning of the right side wall and due to the curvature given to the right side flare to joint with the turned position of the right side wall there were cross flows from the right side towards the centre of the basin. The depth of flow on the right side was also more than that on the left side. This induced heavy whirling action just in front of the exit of the scour vent.

(*b*) With right side wall kept straight as shown in dotted line in Figure 1 the cross flows were reduced to a great extent. The whirling action also ceased. However the standing wave that formed was not parallel to the exit end of the scour vent but was skew, with the right side edge of the wave advancing more than the left side edge of the wave.

(*c*) With the right side wall kept as in case (*b*) and the baffle wall kept parallel to the exit end as shown in dotted line in Figure 1 it was seen that the flow conditions improved. The standing wave that formed now was nearly parallel to the exist end of the scour vent.

4. Conclusion :

4.1. The flow observations for the altered design is furnished in paragraph 3.3 (*a*). The flow profiles and the resultant velocity observed are shown plotted in Figure 1. It can be seen that the maximum height of hump now is El. 5,271.3 against hump elevation of 5,263.5 in the recommended design.

4.2. A few changes that were introduced to improve the flow conditions are described in paragraphs 3.2 (*b*) and 3.2 (*c*) and 3.3 (*b*) and 3.3 (*c*).

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Nº	LEGEND	DESCRIPTION	DETAILS
1	------	Stilling pool 67'long 12'deep & 40' Width Without blocks:	El·5252·0 — El·5249·0 — El·5240·0 — 67'
2	——	Stilling pool 67'long 12·5' depth & 40' Width Without blocks	El·5252·5 — El·5249·0 — El·5240·0 — 67'
3	— ·— ·—	Stilling pool 67'long 12·5' deep & 40'Width with one row of 3 blocks 4'x 4x5' at 20'From end of basin	El·5252·5 — Blocks 4'x4'x5' — El·5242·5 — El·5240·0 — 67'
4	········	Stilling pool 67' long 12·5'deep & 40' Width With two rows of blocks 4'x4'x5' at 20' &32'From end of basin	El·5252·5 — Blocks 4x4x5' — El·5249·0 — El·5240·0 — 12'—20' — 67'
	Final design to be adopted now —x—x—	Stilling pool 67'long 12·5' depth without blocks and skew in plan	Vide Plan

A–12-a 3/59	File Report·	PUBLIC WORKS DEPARTMENT—MADRAS IRRIGATION RESEARCH STATION—POONDI· Kundah Hydro-electric Scheme Kundahpalam Scour vent ENERGY DISSIPATION SURFACE PROFILES·		
13	Drg·Nº.	P.K. Jayaganapathy / J·E·E / Trd: V.V	Nithanan / A·E·R / ckd:	R.K. Kandaswamy / Ex·e·Engineer / FIG··· 1

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KUNDAH HYDRO-ELECTRIC SCHEME—KUNDAH PALAM DIVERSION DAM—RIVER TRAINING WORKS.

(C.B.I. & P. No. C-2-(ii))

1. Introduction :

The Kundah Scheme is a major Hydro-Electric Scheme for the Madras State. It is under execution in Nilgiris District. This scheme aims at utilising, for hydel power development, the water resources of the River Kundah and its tributaries which are located at altitudes varying from El. 7,000.0 to El. 5240.0 above M.S.L. Three dams, namely Avalanche and Emerald (across the main tributaries of river Kundah) in the upper reaches and Kundahpalam Diversion Dam in the lowest reach of the river are now being constructed. These dams are located at a distance of about 20 miles from Ootacamund (vide Figure 1). There is an existing road bridge for crossing the river Kundah near the Kundahpalm village just upstream of the dam site. When the Diversion Dam gets completed, this bridge will get submerged in the forebay reservoir. Hence for communication across the river Kundah, a new bridge has been proposed at about 900 feet downstream from the centre line of the dam. This report deals with the river training works that should be carried out to accommodate the discharge from the spillway, scour vent, the necessary protection arrangements near the toe of the dam and to facilitate smooth and equitable distribution of flow through the four vents of the proposed bridge downstream.

2. Data :

Deepest bed level of the river at the dame site.	El. 5240.0
Deepest bed level of the river near the proposed bridge site	El. 5226.0
Average slope of the river	1/40
Maximum computed flood discharge for the spillway	50,000 cusecs.
Maximum computed tail water near the spillway	El. 5255.00
Maximum computed flood level near bridge site	El. 5236.50
No. and size of spans for the bridge ...	4 Nos; 5′ 6″ each.
No. and thickness of piers	3 Nos; 5′ 6″ each.
Deck level of bridge	El. 5248.33
Top of roadway	El. 5253.0
Width of roadway	22 feet.
Shape of cut-waters and ease-waters ...	Semi-circular.

3. Problem :

The problem is to evolve suitable river training works for (1) accommodating the protective arrangements for energy dissipation in rear of the spillway and the scour vent and (2) facilitating smooth and equitable distribution of flow in all the 4 spans of the proposed bridge.

4. Model :

4.1. A comprehensive model representing the ground contours upto 200 feet upstream and 1,200 feet downstream of the dam site and the finalised dam sections and the protection arrangements below the dam for energy dissipation was constructed to a scale of 1/50. The model was constructed in masonry to facilitate necessary river training works to be carried out with ease. The bridge with its piers was constructed in masonry to facilitate necessary river training works to be carried out with ease. The bridge with its piers was constructed in teakwood and fixed at the given alignment. In the pier at the left extreme end 1/8″ copper tubes were embedded at salient points along vertical sections (vide Figure 3) both at the right and left face of it to register the pressures obtaining at the sides of the pier when there is flow upto M.F.L. under the bridge. The pressures were noted from a manometer battery board on which a row of 1/8″ glass tubes connected with the above copper tubes were fitted.

5. Model Scale :

Linear scale	... ...	1/50	
Velocity scale	... ...	$\frac{1}{\sqrt{50}}$	$= \frac{1}{7.1}$
Discharge scale	... ...	$\frac{1}{50^{5/2}}$	$= \frac{1}{17680}$

6. Experiments :

To determine the pressure distribution along the left and the right faces of the extreme left pier, four separate experiments with full, 3/4th, 1/2 and 1/4th the maximum flood discharge over the spillway were conducted.

7. Discussion of Results :

7.1. *River training works for the spillway and the scour vent upto 500 feet downstream of the dam.*—The ground on the right side of the river rises steeply and also turns towards the left within a few feet of the toe of the dam site (vide Figure 2). Due to this turn, there was no proper drainage of the maximum discharge from the spillway. The ground on the right side acted as a barrier and interfered with the independent functioning of the spillway energy dissipation arrangement. Further as the scour vent is also located to the right of the spillway to accommodate the stilling basin for the scour vent and to ensure proper drainage arrangement for the scour vent discharges also, the ground on the right side had to be trimmed to a great extent. This was done in several stages to involve minimum of earth cutting since the ground on the right was of rocky nature with only a few feet of over-burden. A warped transition from almost a vertical section near the end of the scour vent stilling basin to a maximum flat slope of 1 to 1 at about 500 feet downstream from the dam

on the right side was given. This arrangement afforded proper draining with a few feet of free board over the maximum tail water at each place along the right bank. Some minor training works have also been carried out on the left side of the river. Details of cross sections (Figure 5) of the finalised arrangement are shown on the field plans received.

7.2. *River Training Works for the proposed bridge site at 900 feet downstream of the dam.*—About 200 feet upstream of the proposed bridge location there is curve where the river takes a turn towards the right. To study the effects of this bend in the river on the flow at the bridge site detailed spot levels and cross sections were called for. Accordingly the river from ch. 600 feet to ch. 1,200 feet was moulded as per the latest plans received from the field. It was first observed that there was full flow in the extreme left vent of the bridge and negligible flow in the right side vents. The water surface also shot up along the semi-circular cut waters in front and spilled over the roadway. The river portion on the right side was gradually trimmed involving cutting of the right side ground to suitable slopes to ensure normal flow in the extreme right side vent under the bridge. The bed level of the river above and below the bridge was also trained suitably. To overcome the effects of the bend in the river on the left side a straight embankment from the left abutment pier upto ch. 700 feet upstream and thence joined by a smooth curve to the slope of the river at the left bank at about ch. 650 feet was found to be most effective (vide Figure 5). The maximum water level observed in the model near the bridge site after river training was El. 5,236.25 against a calculated maximum water level of El. 5236.50 at the same site.

7.2.1. *Psessure studies for the bridge pier at the extreme left.*—It was observed that with the maximum discharge over the spillway, the right face of the pier registered a maximum negative pressure of — 17.08 feet of water for the prototype at the junction of the right side of the pier with the semi-circular cut water portion. At the junction of the left face with the cut water portion a maximum of — 12.58 feet of water for the prototype was observed for the same discharge (vide Figure 3 and Statement I). Negative pressures of varying magnitude for even a discharge of 1/4th the maximum was observed through the left and right faces of the pier. The distribution of pressures along the pier faces is furnished in Statement I.

8. Recommendation :

8.1. The river training works as detailed in the plan showing spot levels (Figure 5) and the Cross Section (Figure 4) and showing the proposed levels superimposed on the cross sections received from the field are recommended for being carried out at site.

8.2. The heavy negative pressures noticed on the faces of the piers are injurious to the masonry and may endanger the safety of the structure. However, the authorities of the Electricity Board were of the view that the occurrence of a flood of the magnitude of 50,000 cusecs was a very remote possibility and that no serious consideration need be given to this aspect. Hence further studies were not taken up for reducing the velocities in this reach. However the fact that heavy negative pressures are developed on the pier faces is placed on record for such action as they may deem fit.

STATEMENT I.

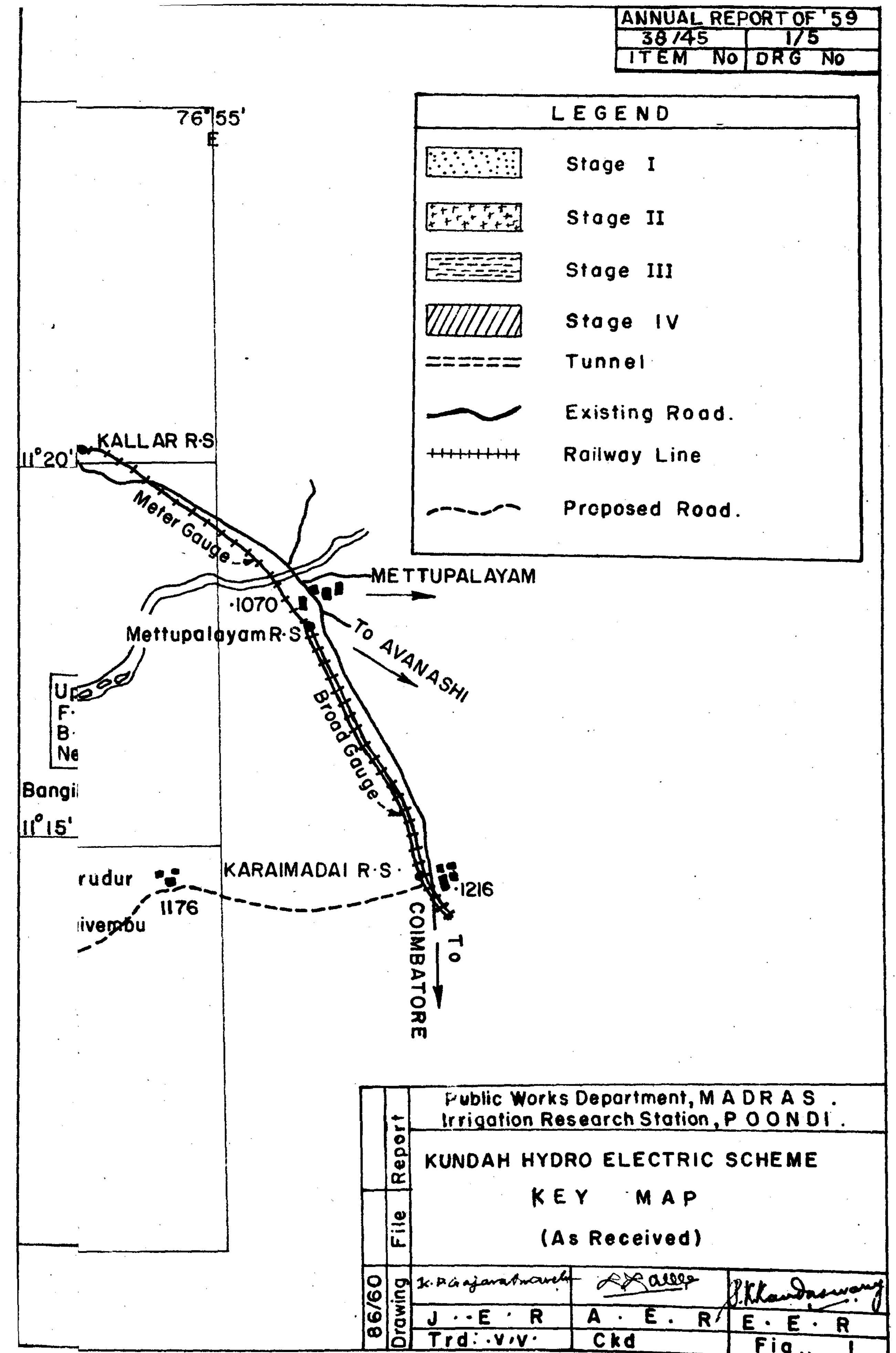
KUNDAH HYDRO-ELECTRIC SCHEME—PRESSURE STUDIES FOR THE LEFT EXTREME PIER UNDER THE BRIDGE

Piezometer No. (1)	*Piezometer Elevation.* (2)	*Pressures in feet of water for prototype.* *Full Discharge.* (3)	*Three-fourth Discharge.* (4)	*Half Discharge.* (5)	*Quarter Discharge.* (6)
		Right Face.			
		First Row.			
1	5244.83	0.67	— 0.58	— 0.83	— 0.83
2	5242.83	— 2.33	0.92	— 0.06	— 0.08
3	5239.83	— 4.58	— 1.33	— 1.83	0.17
4	5237.86	— 0.83	— 3.58	— 1.08	0.17
5	5234.83	— 15.08	— 9.33	— 4.83	— 1.83
6	5232.83	— 17.08	— 5.08	— 8.83	— 3.58
7	5229.83	— 11.08	— 7.83	— 5.33	— 2.83
8	5227.83	— 1.08	— 0.33	0.92	1.17
		Second Row.			
9	5244.83	1.17	0.67	0.92	0.92
10	5244.83	0.61	— 1.08	— 0.83	— 1.08
11	5239.83	5239.83	— 0.08	— 0.17	0.42
12	5237.83	0.17	0.17	0.92	0.17
13	5234.83	— 1.58	— 2.08	— 2.33	— 2.08
14	5232.83	3.42	2.42	1.42	0.17
15	5229.83	4.42	4.83	2.67	1.17
16	5227.83	5.92	5.67	4.17	3.17
		Third Row.			
17	5244.83	— 2.58	— 0.08	— 0.33	— 3.58
18	5242.83	0.97	0.67	0.42	0.17
19	5239.83	— 1.93	— 0.83	0.17	— 0.08
20	5237.83	— 2.83	— 2.08	— 0.83	— 0.08
21	5234.83	— 7.08	— 6.33	— 5.08	— 0.83
22	5232.83	— 4.33	— 3.08	— 2.83	0.17
23	5229.83	— 0.68	— 0.83	— 1.83	— 0.08
24	5227.83	2.17	7.17	1.17	— 0.33

STATEMENT I—(*cont.*)

KUNDAH HYDRO-ELECTRIC SCHEME—PRESSURE STUDIES FOR THE LEFT EXTREME PIER UNDER THE BRIDGE—(*contd.*)

Piezometer No. (1)	*Piezometer Elevation.* (2)	*Pressures in feet of water for prototype.* *Full Discharge.* (3)	*Tthree-fourth Discharge.* (4)	*Half Discharge.* (5)	*Quarter Discharge.* (6)
		Left Face.			
		Fourth Row.			
25	5244.83	0.67	9.33	0.67	— 1.33
26	5242.83	0.17	— 9.58	1.17	— 3.08
27	5239.83	— 5.58	— 1.83	— 0.33	— 2.08
28	5237.83	— 2.83	— 0.33	— 0.58	— 0.58
29	5234.83	— 12.05	— 10.83	— 12.58	— 2.08
30	5232.83	— 6.33	— 8.08	— 7.33	— 7.83
31	5229.83	— 5.58	— 5.08	— 4.83	— 2.83
32	5227.83	— 0.83	— 0.83	— 0.83	— 0.83
		Fifth Row.			
33	5244.83	1.42	1.42	1.17	1.17
34	5242.83	1.67	1.17	1.17	1.17
35	5239.83	0.08	0.17	0.17	0.42
36	5237.83	0.17	0.17	0.42	0.42
37	5234.83	0.17	0.17	0.17	0.17
38	5232.83	— 0.83	— 0.58	— 0.83	— 4.42
39	5229.83	— 0.33	— 0.33	— 0.58	— 0.08
40	5227.83	— 0.83	0.42	1.17	— 0.08
		Sixth Row.			
41	5244.83	1.67	1.42	1.42	1.17
42	5242.83	0.67	1.17	0.67	0.72
43	5239.83	— 0.58	0.17	0.17	— 0.08
44	5237.83	0.67	0.67	1.42	0.67
45	5234.83	— 0.33	— 0.08	— 0.83	— 0.08
46	5232.83	— 0.33	0.67	— 0.83	0.17
47	5229.83	0.08	0.67	0.17	0.17
48	5227.83	1.92	2.17	2.17	1.92



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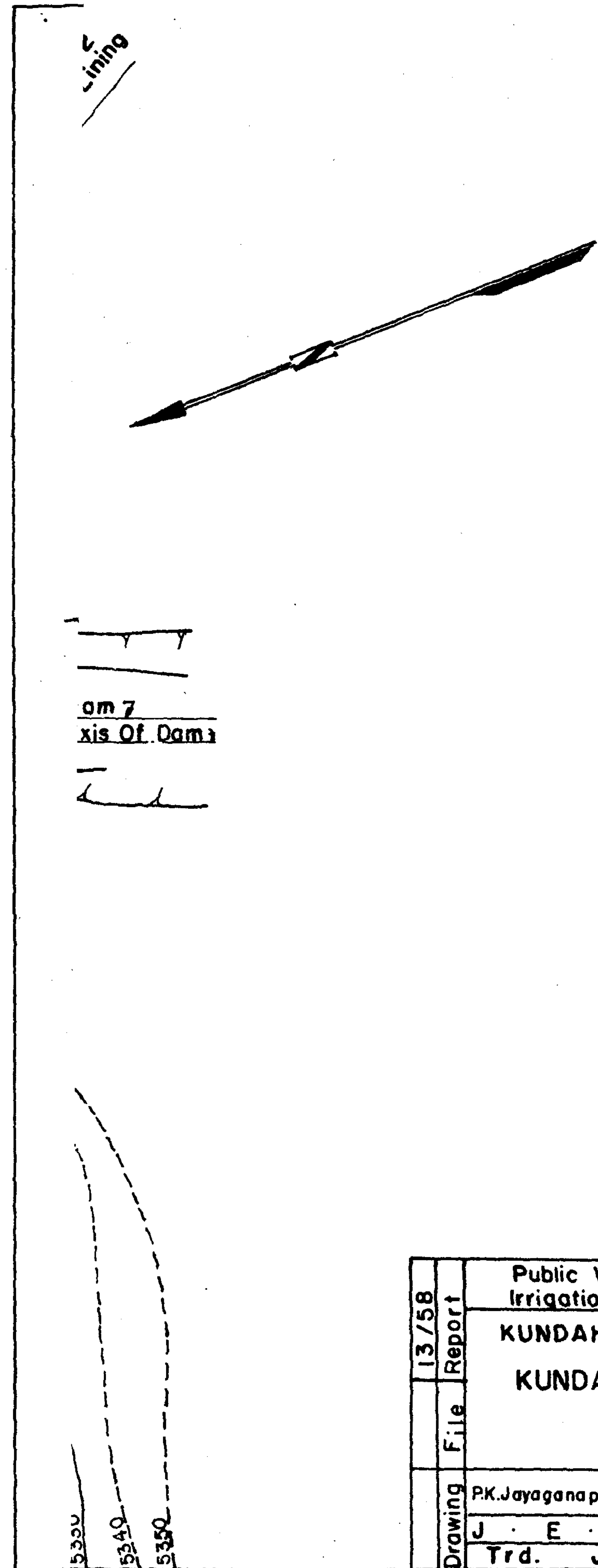
ITEM No	DRG No



File 13/58 Report

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

KUNDAH HYDRO ELECTRIC SCHEME

KUNDAHPALAM DIVERSION DAM

GENERAL PLAN

Drawing		
P.K.Jayaganapathy.	C' Kuppuraj	R.R. Reddy.
J · E · R	A · E · R	E · E · R
Trd. J·J	Ckd.	Fig. 2

ANNUAL REPORT OF '58

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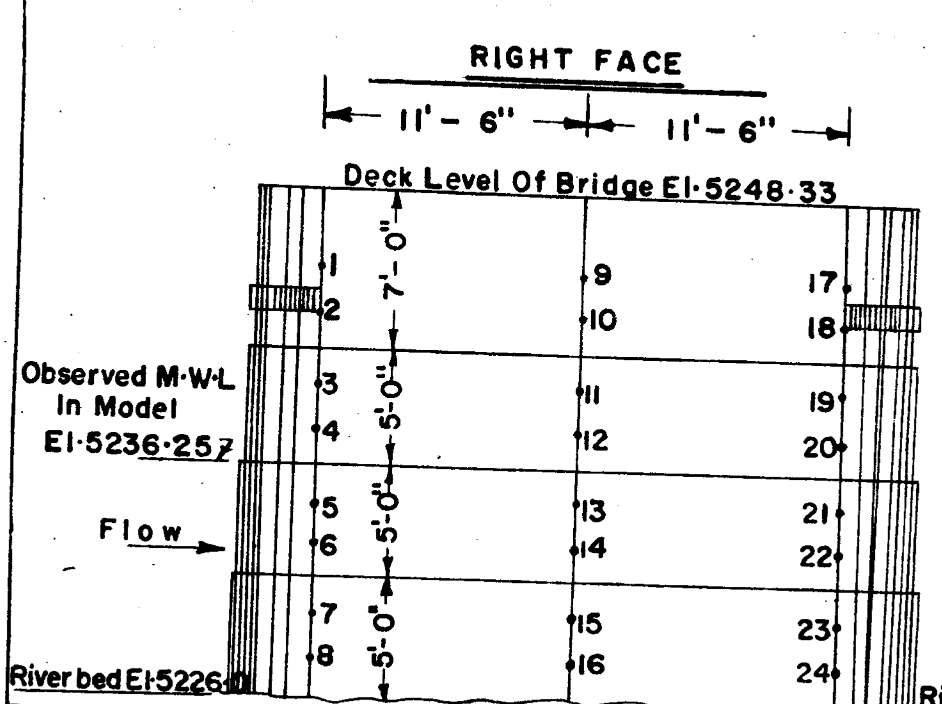
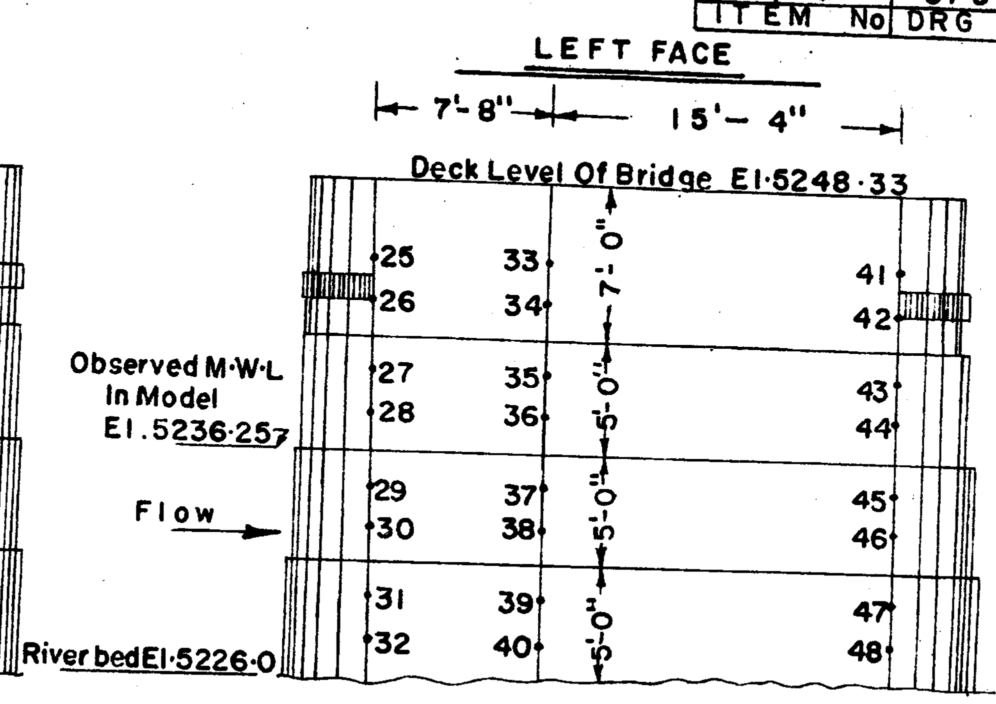
RIGHT FACE



LEFT FACE



Piezometer Points Nos						Piezometer Elevation
1st row	2nd row	3rd row	4th row	5th row	6th row	
1	9	17	25	33	41	5244·83
2	10	18	26	34	42	5242·83
3	11	19	27	35	43	5239·83
4	12	20	28	36	44	5237·83
5	13	21	29	37	45	5234·83
6	14	22	30	38	46	5232·83
7	15	23	31	39	47	5229·83
8	16	24	32	40	48	5227·83

NOTE :-

1. To be read with statement no I I·R·S. report no 13/58
2. Shape of cut water_ semicircular

Scale In Feet

100 0 100 200

Report I 3/58	Public Works Department, MADRAS. Irrigatión Research Station, POONDI.
File	KUNDAH HYDRO ELECTRIC SCHEME KUNDAHPALAM BRIDGE—PRESSURE STUDIES Plan Showing The Location And Elevation Of Piezometer Points

Drawing	P.K.Jayaganapathy.	C. Kuppuraj	R. R. Reddi.
	J · E · R	A · E · R	E · E · R
	Trd JJ	Ckd	Fig 3

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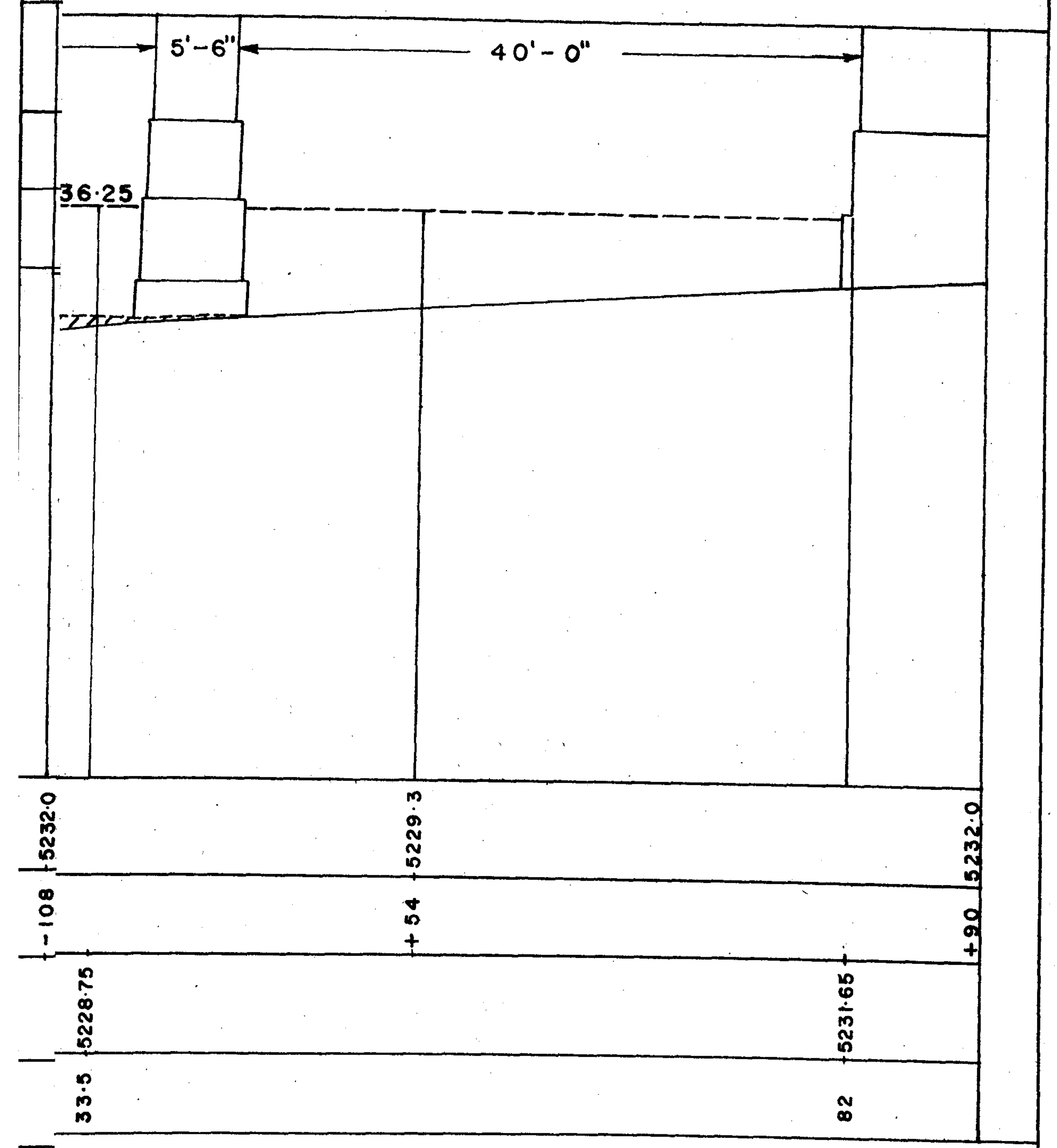
ITEM No	DRG. No



13/58 Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	KUNDAH HYDRO ELECTRIC SCHEME KUNDAH RIVER TRAINING WORKS CROSS SECTION THROUGH ℄ OF BRIDGE (C·S· AT 900 D/S OF DAM)		
Drawing	P.K.J.Ganapathy	C·Kuppuraj	R.R.Reddy,
	J·E·R	A·E·R	E·E·R
	Trd. J·J	Ckd.	Fig. 4

20

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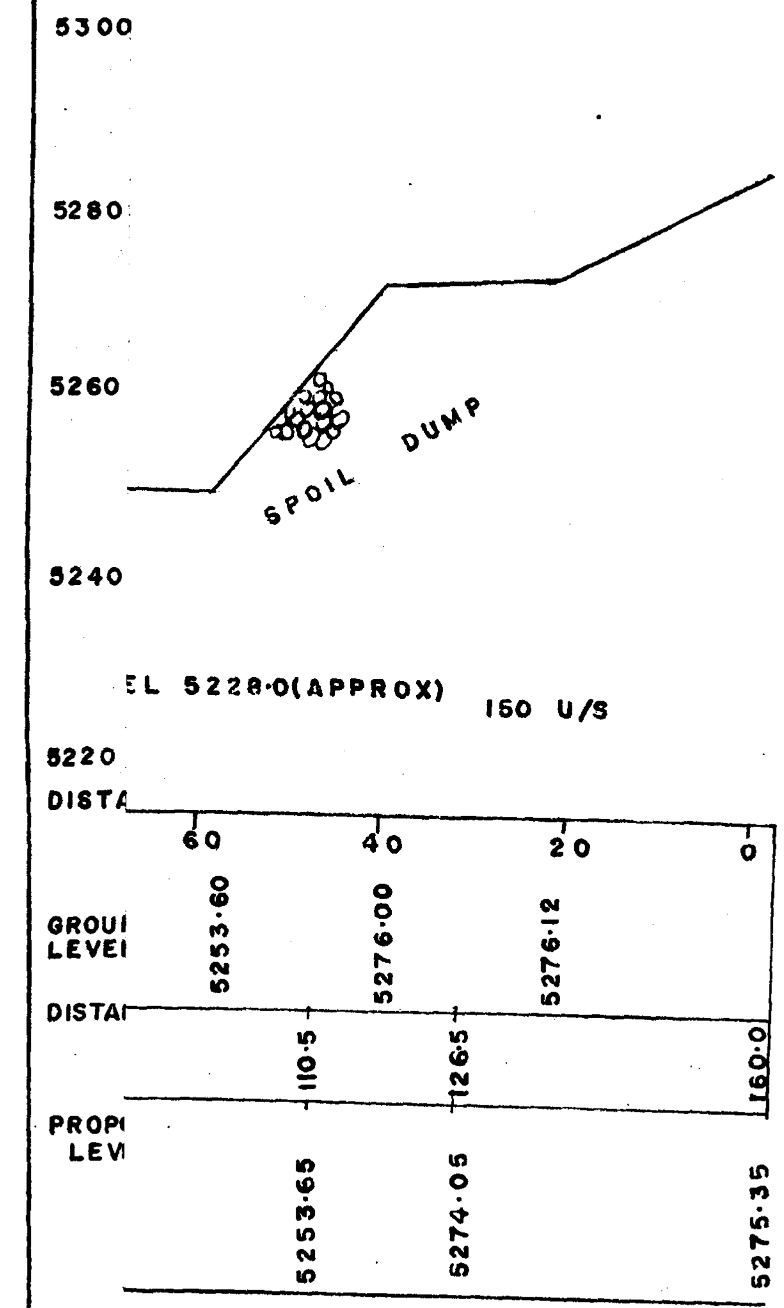

Report 13/58	Public Works Department – Madras. Irrigation Research Station–Poondi.
File	KUNDAH HYDRO ELECTRIC SCHEME KUNDAHPALAM DAM D·S· 200–1000 PROPOSALS FOR RIVER TRAINING

Drawing		
P.K. Jayaganapathy.	C·Kuppuraj	R.R. Reddi.
J·E·R	A·E·R	E·E·R
Trd: T.K.S	Ckd:	FIG 5

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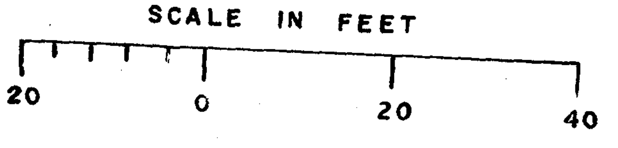
Item No.	Drg No.



Drawing File. Report 13/58.

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

KUNDAH HYDRO ELECTRIC SCHEME
KUNDAPALAM DAM. D.S 200—1000
PROPOSALS FOR RIVER TRAINING.

P.K. Jayaganapathy	C. Kuppuraj	R.R. Reddy.
J.E.R	A.R.O	E.E.R
Trd: TKS	Ckd:	FIG... 6.

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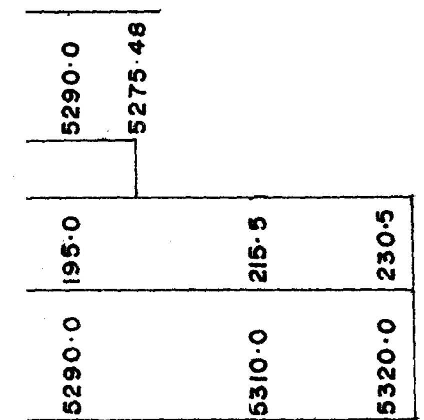

Report 13/58

File

Public Works Department— Madras
Irrigation Research Station–Poondi

KUNDAH HYDRO ELECTRIC SCHEME

KUNDAHPALAM DAM D·S·200–1000

PROPOSALS FOR RIVER TRAINING

Drawing

P.K.Jayaganapathy	C·Kuppuraj	R.R. Reddi
J·E·R	A·E·R	E·E·R
Trd: V·V·	ckd:	FIG 7

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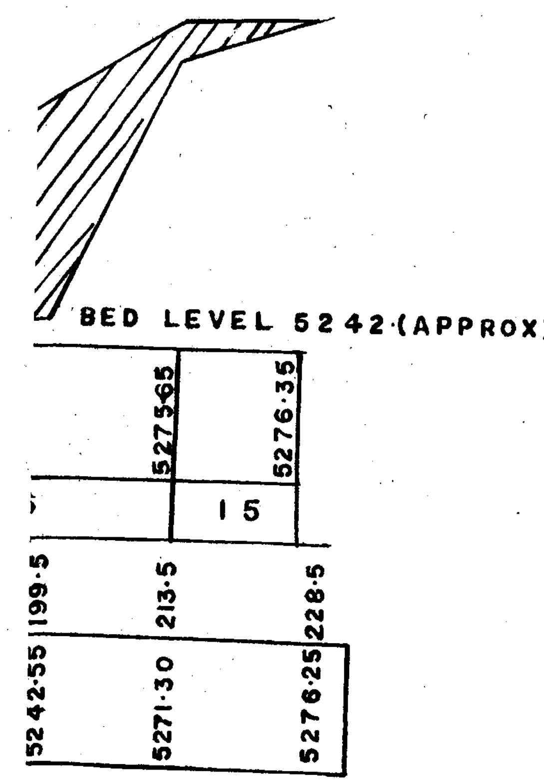

13/58 Report	Public Works Department—Madras. Irrigation Research Station—Poondi.		
File	KUNDAH HYDRO ELECTRIC SCHEME KUNDAHPALAM DAM D·S·200–1000 PROPOSALS FOR RIVER TRAINING		
Drawing	P.K.J.GanaPathy	C.Kuppuraj	R.R. Reddy.
	J·E·R	A·E·R	E·E·R
	Trd: T.K.S	Ckd:	FIG.... 8

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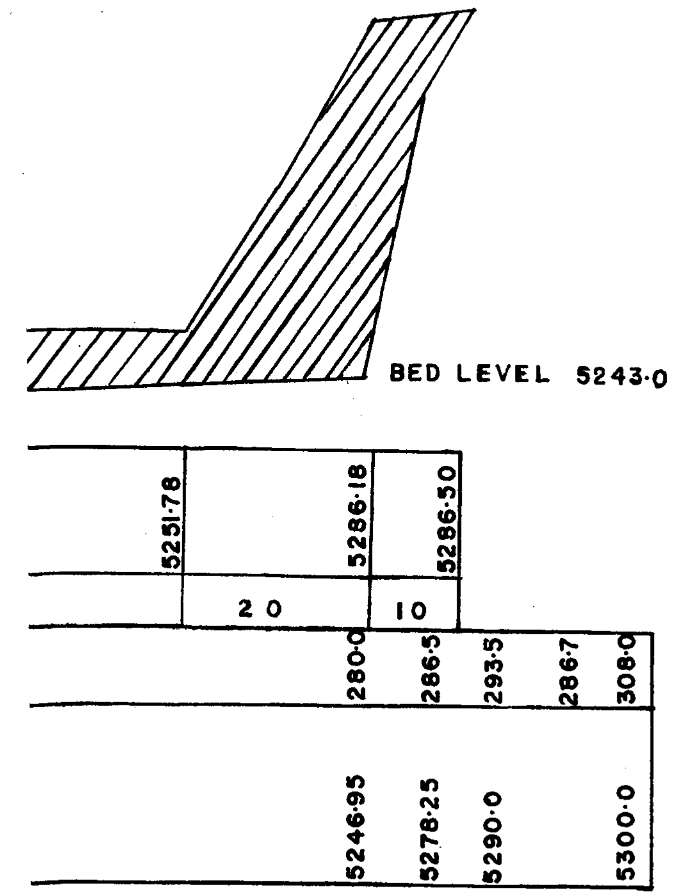
Item No	Drg No



Public Works Department—Madras.
Irrigation Research Station Poondi.

KUNDAH HYDRO ELECTRIC SCHEME.
KUNDAPALAM DAM D.S 200—1000
PROPOSALS FOR RIVER TRAINING

Report 13/58.
File
Drawing

P.K.J. Ganapathy	C. Kuppuraj.	R.R. Reddy.
J. E. R	A. R. O	E. E. R
Trd: TKS	Ckd:	FIG...... 9

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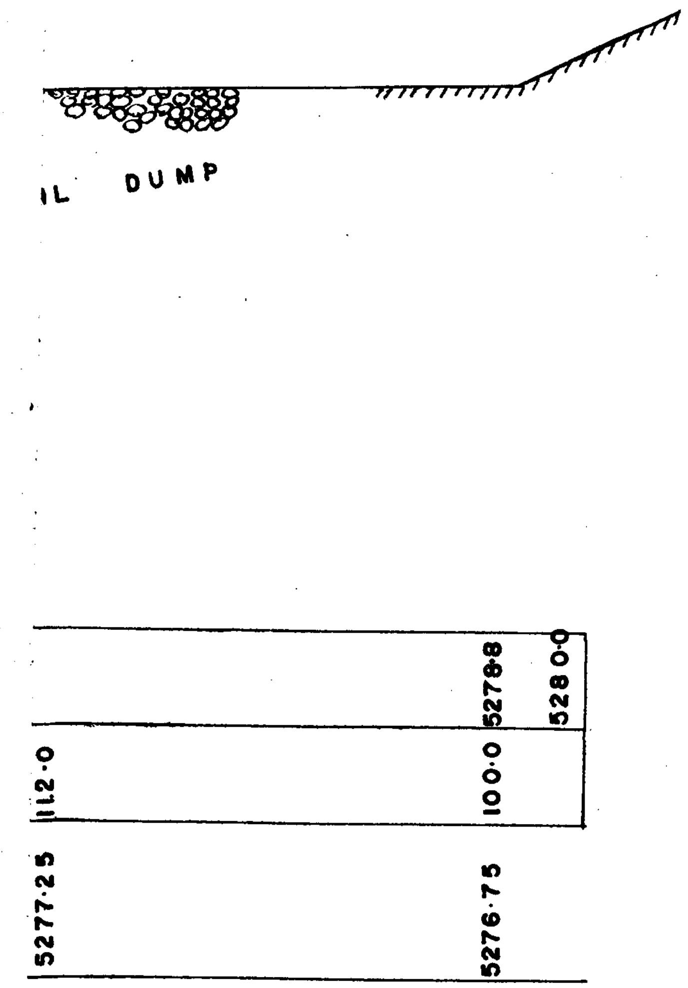
Item No	Drg No



13/58	Public Works Department - Madras. Irrigation Research Station - Poondi		
Report	KUNDAH HYDRO ELECTRIC SCHEME KUNDAHPALAM DAM D·S· 200-1000		
File	PROPOSALS FOR RIVER TRAINING		
Drawing	P.K.J. Ganapathy	C. Kuppuraj	R. R. Reddi.
	J. E. R	A. E. R	E. E. R
	Trd: T.K.S	Ckd:	FIG... 10.

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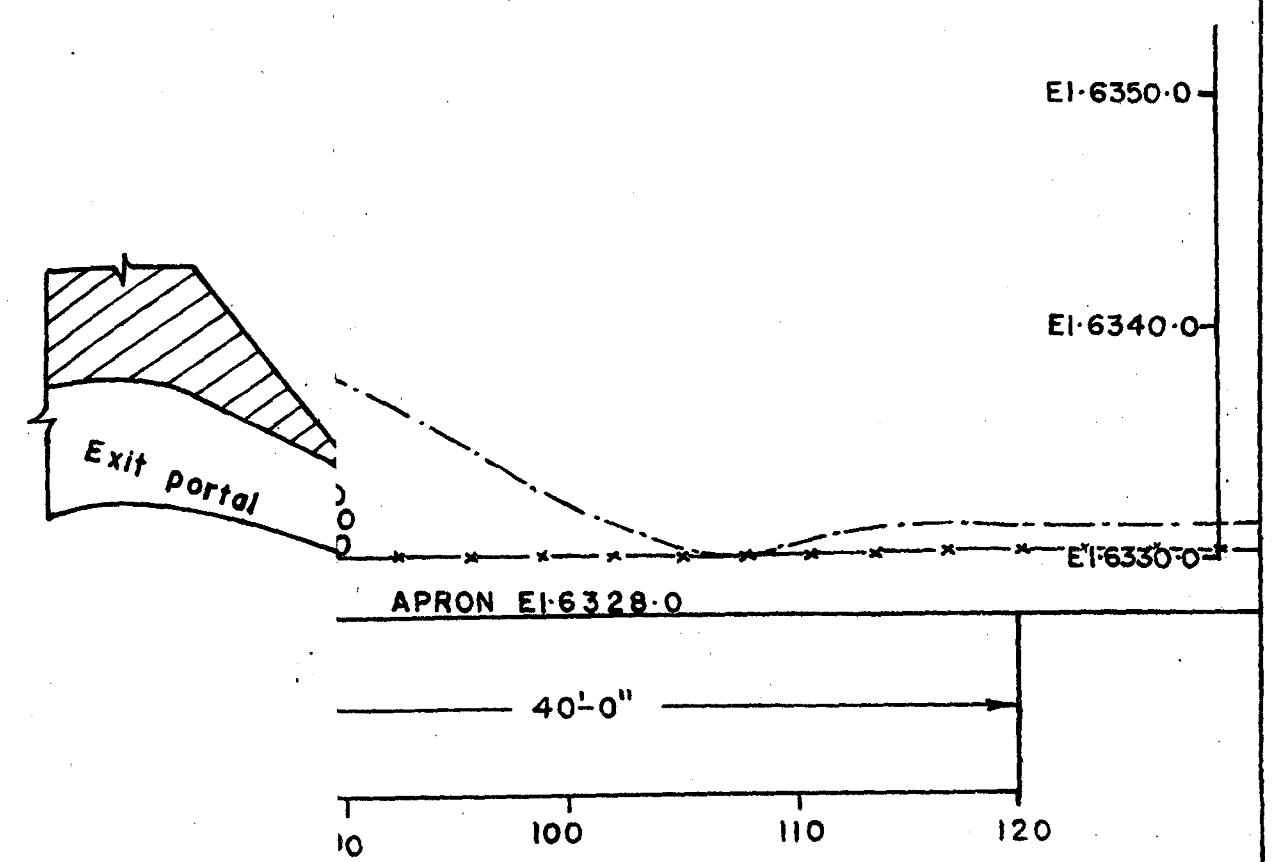
…HER …CULARS	MAX· HUMP	VELOCITY AT 120' FROM EXIT·	
…s of …3'x4' …om toe	El·6341·4	21'/Sce·	Design Recommended in I·R·S Report No·11/58·
…lock	El·6369·66	15·36'/Sec·	Design as Finalised by S·E·T·C & Communicated in his letter No·S·E·T·C/E5/A3/3159/Kun 40AII/15-12-58



Report 2/59	Public Works Department - Madras Irrigation Research Station Poondi
File A-12-0	Kundah Hydro-electric Scheme Emerald Scour Vent Energy Dissipation Arrangements FLOW PROFILES FOR ALTERNATIVES TRIED.
Drawing 15	P.K. Jayaganapathy — J·E·E; Trd: V.V · [signature] — A·E·R; ckd · [signature] — E·E·R; FIG ··· 1.

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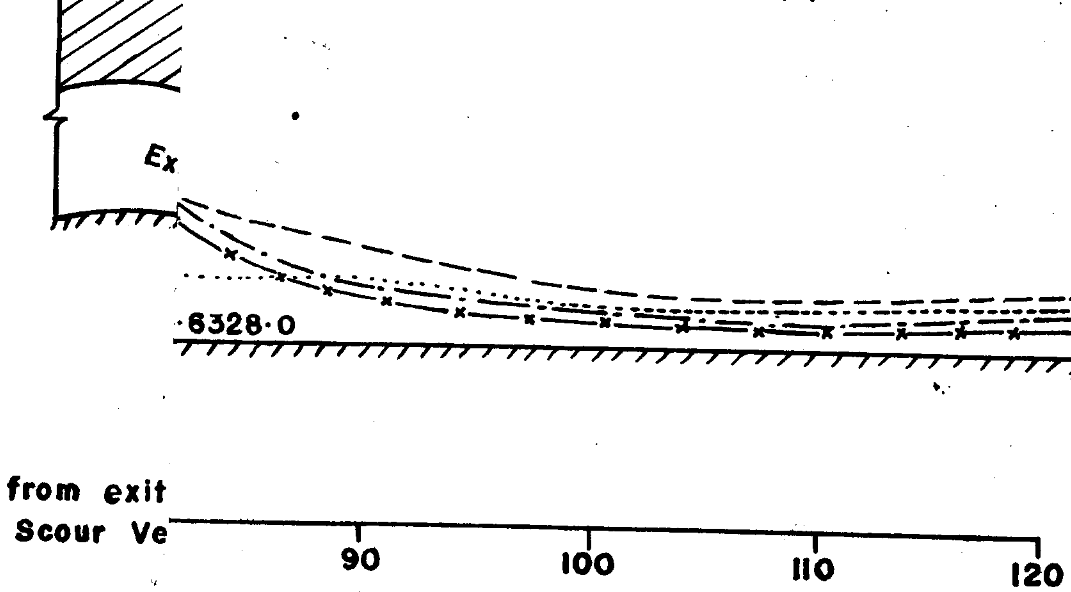
No.	LEGEND	
1	————	(15'+20') 10'
2	········	(20'+15') 10'
3	—·—·—	(20'+20') 10'
4	—x—x—	(20'+20') 9' Finalised design recommended now for adoption

NOTE: To be read with statement No·1



Distance from exit
Scour Ve

2/59 Report	Public Works Department–Madras Irrigation Research Station–Poondi		
A-12-a File	Kundah Hydro-electric Scheme Emerald Scour Vent Energy Dissipation Arrangements FLOW PROFILES FOR ALTERNATIVES TRIED		
16 Drawing	P.K. Jayaganapathy	Nirtharan	S.S. Kandaswamy
	J E E	A · E · R	E · E · R
	Trd· V·V	chd:	FIG··· 2

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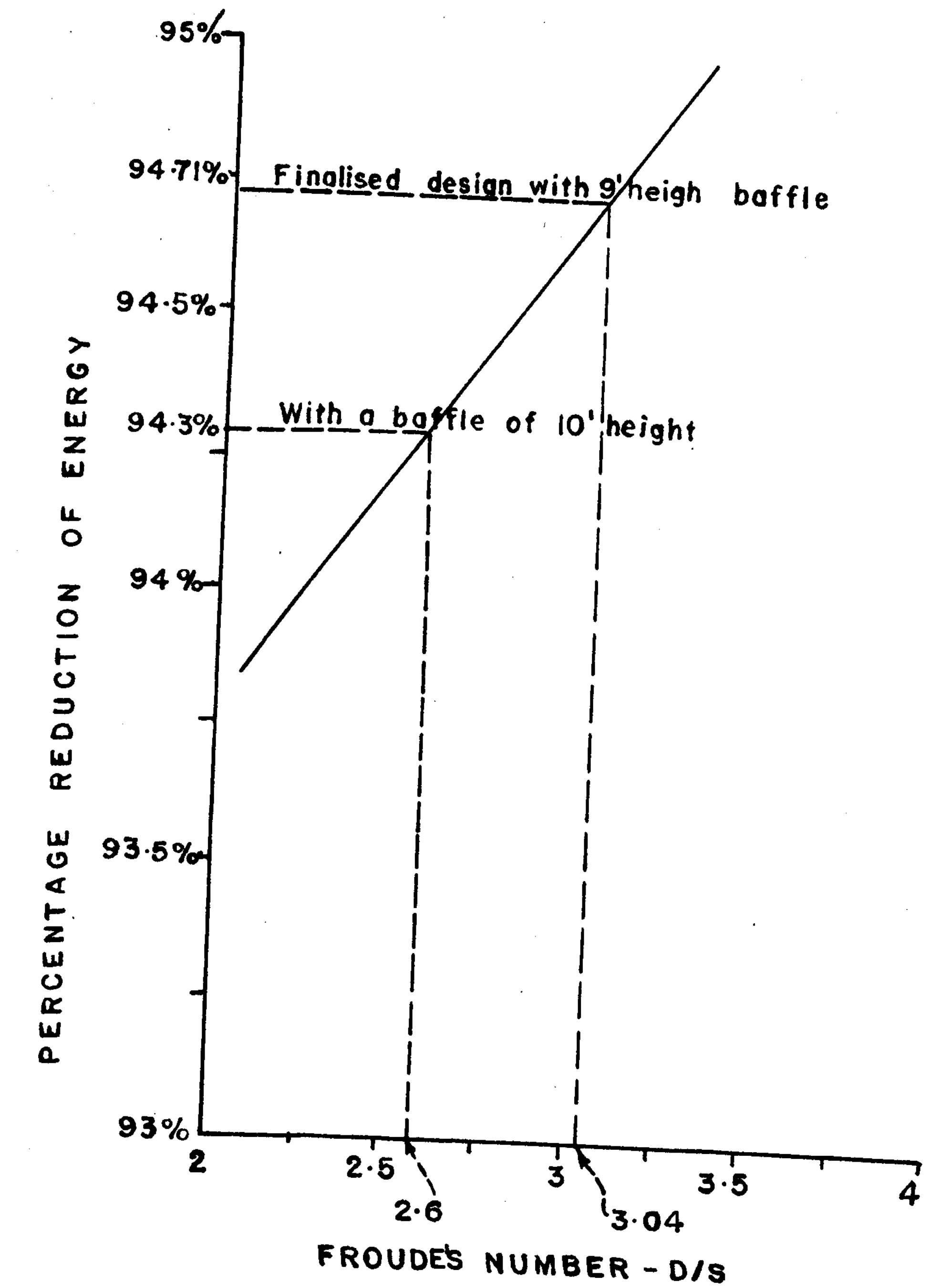

2/59 Report	Public Works Department - Madras Irrigation Research Station - Poondi
A-12-a File	Kundah Hydro-electric Scheme Emerald Scour Vent GRAPH SHOWING THE PERCENTAGE REDUCTION OF ENERGY VS FROUDE'S NUMBER

18 Drawing	P.K. Jayaganapathy	[signature]	[signature]
	J·E·E	A·E·R	E·E·R
	Trd: V.V	ckd	FIG···4

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Item No.	Drg. No.

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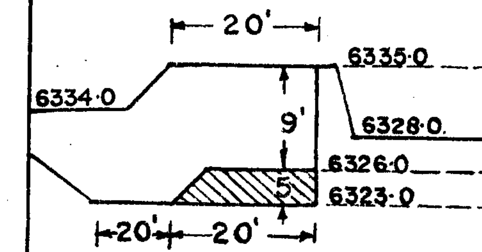
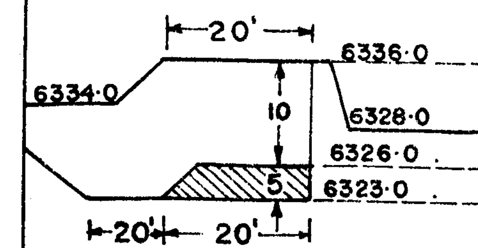
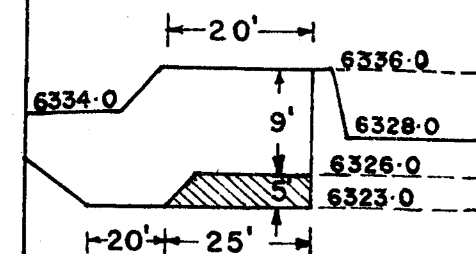
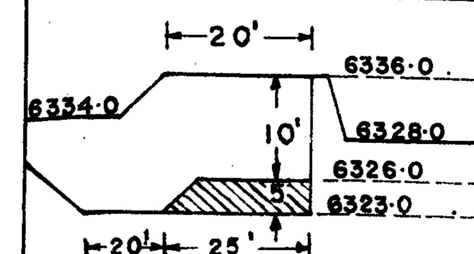
Statement Nº I.(Cont)

No.	1	2	3	4	5
7		El·6349·6	20·20'/Sec	93·46%	
8		El·6351·0	20·20'/Sec	93·13%	
9		El·6349·0	21·24'/Sec	92·85%	
10		El·6350·0	19·38'/Sec	93·74%	

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File	A-12-a
Drawing	14

PUBLIC WORKS DEPARTMENT-MADRAS
IRRIGATION RESEARCH STATION-POONDI·

Kundah Hydro-electric Scheme
Emerald Scour Vent
Energy dissipation studies
ALTERNATIVE DESIGN FOR STILLING BASIN.

P.K. Jayaganapathy	Mithanan	[signature]
J·E·E	A·E·R	E·E·R
Trd: V·V	ckd:	FIG···5

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REPORT ON PERIYAR IRRIGATION TUNNEL—EROSION IN REAR OF HEAD SLUICE.

1. Introduction :

1.1. River Periyar rises in the Western slopes of Western Ghats in the Kerala State and flows through mostly rocky country before falling into the Arabian Sea. River Vaigai has its source very near that of Periyar and flows eastward to the Bay of Bengal. Unlike Periyar, river Vaigai is not perennial but is of a flashy variety, draining its catchment rapidly during storms and running dry most of the year. This state of affairs was not conducive to growth of irrigation in the Vaigai basin.

1.2. In the year 1882, Major Pennyquick submitted a proposal to create a storage reservoir across Periyar and divert the water to the Vaigai basin. The proposal was approved and a site was chosen and a straight gravity dam was completed in the year 1887.

1.3. A rock tunnel 5,887 feet long was drilled through the ridge of the Western Ghats. This tunnel, which will henceforth be called Irrigation Tunnel, had a rough interior and was non-uniform in cross section and bed slope. Bed slope of tunnel was very steep, having a fall of 78.67 feet in 5,887 feet. Water from Periyar lake is drawn through a rocky approach channel 7,600 feet long and discharged into the tunnel in regulated amounts by a head sluice which had a vent 9′ 6″ wide and 12′ — 6″ high. There are two shafts over the tunnel one at 165 feet and the other at 1,713 feet from the start of the tunnel.

1.4. *Proposals by Electricity Department.*—Water issuing from Irrigation Tunnel falls through about 1,200 feet before reaching the plains. The Madras State Electricity Board, the then Electricity Department, proposed to utilise this drop for power generation. The proposals were to construct a forebay dam with a 45 feet long spillway, a scour vent and a penstock of 13 feet diameter, to drill a power tunnel 4,188 feet long, etc. They included enlarging and trimming with 1 foot thick reinforced concrete, the Irrigation Tunnel to a regular shoe section of area 151.67 sq.ft., and installing a remote controlled gate at shaft No. 1. The Tunnel was designed to carry 1,600 cusecs discharge. The existing head sluice was to be utilised as an emergency gate in future.

1.5. *Later proposals by Public Works Department.*—It was later proposed to lower the sill of the tunnel entry by 5.14 feet, to line the approach channel and to carry out lowering the invert of tunnel upto 700 feet from start of tunnel. The head sluice vent was proposed to be enlarged to a section 9′ 6″ wide and 15′ 9″ high.

1.6. *Execution.*—During the year 1957, most of the length of the Irrigation Tunnel was lined. Lining in the first 22 feet length of tunnel and a short length upstream of shaft was not completed.

2. Problem :

2.1. During the irrigation season of the year 1957, the existing head sluice upstream of the start of tunnel was operated as the remote controlled gate had not been installed. During that season a maximum discharge of 1,850 cusecs is reported to have been drawn.

2.2. During inspection of the interior tunnel face, during a brief closure period, it was observed that the lining in the reach between shaft No. 1 and head sluice had worn out exposing reinforcement in the concrete and in few spots dents were also noticed and that the wear was conspicuous on the floor and the junction of side wall and invert.

2.3. It was proposed to conduct model studies to find out the reasons therefor. Incidentally it was proposed to find out the velocity near the emergency gate and head sluice (the present head sluice gate and shaft No. 1).

3. Data :

Tunnel—

Width	14′ 0″
Overall height	12′ 6″
Designed discharge	1,600 cusecs.

Head Sluice (Vide Figure No. 3)—

Vent	9′ 6″ × 15′ 9″
Sill	El. 2,810.81 (M.S.L.).

Shaft Gate (Vide Figure No. 4)—

Vent	Tunnel section.
Sill	El. 2,810.35
Cross section of shaft in plan	11′ 8″ (Along) × 14′ 0″ (Across)

4. Model :

The existing model of the Periyar Irrigation Tunnel to a geometrically similar scale 1 : 36 was utilised, for these experiments. Relevant portions were produced in teak wood with the top half of tunnel removable so that velocity observations with a pitot tube might be made. Gravitational discharge scale ratio was adopted for the experiments.

5. Experiments and Discussion :

5.1. During the studies to analyse the flow characteristics of the Periyar Irrigation Tunnel, it was observed that flow will be an open channel flow in the supercritical region mostly. For a discharge of 1,600 cusecs the flow in rear of head sluice was found to be an open channel flow, when the water level in forebay was at El. 2,814.0 or below. As in the case in rear of a shutter where the velocity must be $C_V\sqrt{2g^H}$, where Cv is a coefficient and H is the head of water over tail water level here also it was calculated to be in the order of about 46 feet/second. It was thought that this high velocity which was expected to be of the order of 46 feet/second, might have been the probable factor causing erosion. Hence, velocity observations were made in the model. Velocities in rear of head sluice and shaft No. 1 are given in Statements I and II. Negative pressure occurred to a tune of 11.35 feet at a point noted in the sketch.

It is understood that steel lining is provided for gate grooves. The construction work is also completed. If the curve can be made into one of a larger radius the negative pressure is likely to get reduced.

5.2. As the velocities are high, it was proposed to reduce by placing baffles across the tunnel. To allow low discharges to be passed the said baffles were proposed to be vented baffles, with suitably sized vents at the invert level. Due to the construction of water way, a standing wave forms within the shaft and head sluice shaft and experiments were conducted to observe the depths at various sections. Piezometers were connected to the head board and depths were observed from the water elevations noted in the piezometers. Velocity observations were made at different cross sections with a pitot tube without any baffle end with various proposals tried. Velocity observations in rear of shaft gate and head sluice are given in statements. The depth of the water at the various reaches are marked in the L.S. of the tunnel for 1,600 cusecs for 1,850 cusecs discharge (as got from the model). The calculated velocity in rear of the gate and the one that is got from model studies tally. The calculations are shown in Appendices I and II for this discharges of 1,600 cusecs and 1,850 cusecs. As the field officers expressed their difficulty in constructing baffles and removing them when the forebay comes into operation, the idea of providing baffles was dropped.

6. Conclusions and Recommendations :

The next alternative, that could be thought of, for creating a standing wave, was the provision of a sunk basin in the tunnel in rear of the gate. This was also considered as not feasible in view of the advanced stage of execution of improvements to the tunnel. Therefore the only remedy that could be suggested for overcoming the effect of high velocities is to adopt cutstone flooring in place of the proposed reinforced concrete flooring both in rear of Head sluice and Emergency gate. To decide the length upto which such cut stone flooring need be provided, velocity observations with F.R.L. in the Periyar Lake and F.R.L. in the forebay, were recorded for discharges varying from 1,600 cusecs to 4,000 cusecs. The results are furnished in the Statement IV from which it may be observed that the dispersion of the jet below the gate in shaft No. 1 gets dissipated considerably within a length of 50 feet from the gate. Therefore it is recommended that the flooring for the tunnel may be paved with cutstones set in cement mortar from Ch. 180 to Ch. 240. If any portion of this reach has already been connected and if any difficulty is experienced in removing the concrete so laid, the cutstone paving may be done wherever it is feasible within this reach, so that the necessity or otherwise of this paving might be examined after watching the performance of the flooring of the tunnel for a couple of seasons.

Regarding the emergency gate similar condition of velocity are going to exist and in order to safe guard the floor where the velocity is a maximum, it is quite necessary to pave the floor with cutstones of suitable thickness to take the high velocity of 46 feet/second. This high velocity will transit into the normal velocity in a certain zone and it is in this zone the floor is to be protected.

STATEMENT I

Velocity Observations Near The Shaft No. 1.

Sl. No.	Particulars.	Pilot tube observations. Ch. 190	Ch. 215	Ch. 240	Ch. 265	Ch. 290	Prototype velocities. Ch. 190	Ch. 215	Ch. 240	Ch. 265	Ch. 290	Gate opening in feet.	Remarks.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
		(IN INCHES.)					(IN FEET PER SECOND.)						
I.	*Discharge* 1,600 *cusecs*												
1	Free flow ...	13.1	12.4	11.5	11.2	11.6	50.26	48.75	47.00	46.25	46.00	3.15	
2	With skimmer baffle 4' high at 30' from gate (i.e., ch. 213).	13.2	Reverse flow.	0.2	2.1	3.5	50.40	Return flow.	6.22	20.50	21.82	3.15	Condition 1 (2) velocities near the roof at Ch. 215—22 feet/second.
3	With skimmer baffle 5' high at 30' from gate (i.e., Ch. 213).	11.8	Do.	2.4	3.2	3.1	47.50	Do.	21.50	24.80	24.40	3.5	Condition 1 (3) velocity near the roof at Ch-215 equal to 40.00 feet/second.
II.	*Discharge* 1,850 *cusecs*												
1	Free flow ...	14.7	13.1	12.6	10.6	10.1	53.06	50.25	49.25	45.00	43.90	3.75	
2	With skimmer baffle 4 ft. high at 30' from gate.	12.8	Reverse flow.	Nil.	4.2	4.1	49.50	Return flow.	Return flow.	28.40	28.20	4.0	Condition II (2) velocity near the roof at Ch. 215—20.68' and at Ch. 240—18.60'/sec.
3	With skimmer baffle 5' high at 30' from gate.	11.6	Do.	Nil.	2.2	2.8	47.12	Do.	Do.	20.56	23.16	4.3	Condition II (3) velocity near the roof at Ch-215—20.68' and at Ch. 240—20.10'/sec.

(1) Velocities observed at the central line of the tunnel. (2) Model scale 1/36. (3) M.W.L. maintained 2,861.10. (4) Sill of head sluice 2,810

STATEMENT II

Velocity observations near the head sluice

Sl. No.	Particulars	Pitot tube observations Section I	Section II	Section III	30'D/S of Section III	Section IV(50' D/S of Section III	Prototype velocities Section I	Section II	Section III	30'D/S of Section III	Section IV (50'D/S of Section III)	Gate opening in feet	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
I.	*Discharge* 1,600 *Cusecs*—												
1	Free flow ...	19.4"	19.3"	14.8"	12.0"	11.6"	61.00	60.75	53.25	48.00	47.12	4.1'	1. Velocities observed at the centre line of the tunnel.
2.	With skimmer baffle 5 ft. high at 25' downstream of gate position, Ch. 208.	Not taken		...	9.2"	9.2"	...	...	...	42.00	42.00	4.1'	2. Model scale 1/36. 3. M.W.L. maintained at 2861.10-
3.	Skimmer baffle 4 ft. high at 25 ft. downstream of gate position.	Shooting flow.				...	...	...	...	...	...	...	4. Sill of head sluice 2810.82.
II.	*Discharge* 1,850 *Cusecs*—												5. For position of Sections I to IV vide Blue print PR. No. 221/1957.
1	Free flow ...	19.4"	19.4"	12.9"	12.3"	12.3"	11.9"	61.00	61.00	49.60	48.50	4.8'	
2	Skimmer baffle 5' high at 25' downstream of gate position, Ch. 208.	Not taken			9.4"	9.2"	...	...	...	42.40	42.00	4.8'	

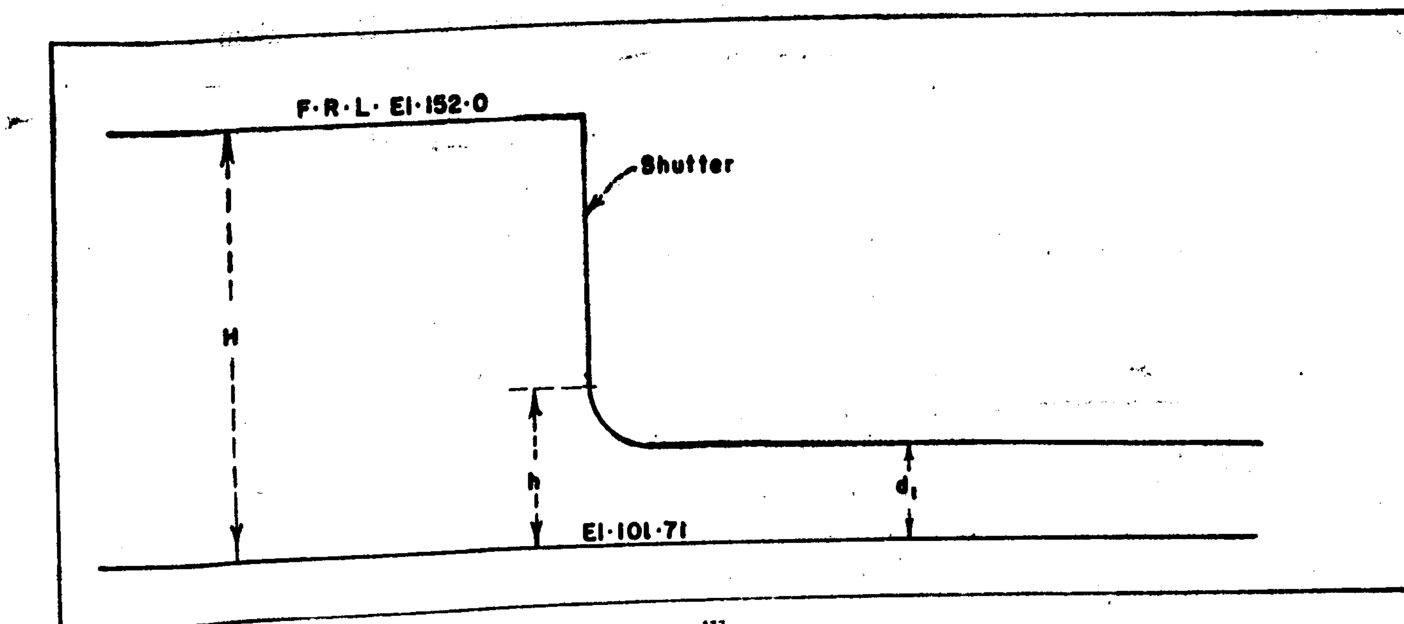
STATEMENT III

DEPTH OF FLOW AT VARIOUS SECTIONS OF THE TUNNEL

Sl. No.	Arrangement in rear of Shaft No. 1	Elevation of water surface in rear of Shaft No. 1	Gate opening in feet	Piezometer tap point numbers 5	7	9	10	11	12
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
		(IN FEET)				(IN FEET)			
Discharge 1,600 *Cusecs*—									
1.	Free flow	5.90	3.00	3.97	4.15	4.90	4.65	4.65	4.48
2.	Baffle 4′ high at 27′ from gate position	9.36	3.35	5.47	4.65	5.05	5.65	4.65	3.95
3.	Baffle 4′ high at 80 feet	10.08	3.35	5.47	4.65	4.90	5.65	4.65	4.73
4.	Skimmer baffle 4 ft. high at 27 feet	7.56	3.00	5.77	4.65	4.90	3.65	5.65	3.98
5.	Skimmer baffle 4 feet high at 80 feet	7.92	3.00	5.72	4.90	5.15	9.83	4.65	3.12
6.	Baffle 5′ high at 27 ft. ...	13.93	3.50	5.47	4.90	5.65	5.65	4.65	4.98
7.	Baffle 5′ high at 30 ft.	14.29	3.50	5.47	4.65	5.40	5.65	4.65	4.98
8.	Skimmer baffle 5′ high at 27 feet ...	11.30	3.60	5.72	4.65	4.90	5.65	4.65	4.48
9.	Skimmer baffle 5′ high at 30 feet ...	10.44	3.60	5.72	4.75	4.90	5.65	4.65	5.98
Discharge 1,850 *Cusecs*—									
1.	Free flow	7.74	4.00	4.72	4.65	5.85	5.55	5.65	5.68
2.	Skimmer baffle 5 ft. high at 27 ft. ...	16.05	4.10	5.97	4.15	5.15	4.85	4.90	5.98
3.	Skimmer baffle 5 ft. high at 30 feet	10.84	3.95	6.97	5.40	5.40	6.15	5.15	4.98

APPENDIX I.

Periyar Irrigation Tunnel—Computation of Velocities Downstream of Head Sluice And Shaft No. 1



H = Depth of water over sill

H_1 = Effective head

= H

ϕ = Velocity coefficient 0.96 assumed

$d_1 = h, \quad = 0.62$ assumed

$H = 152.0 - 101.71$

$= 50.29$

Velocity $v_1 \quad = \phi/2_g\,(H - d_1)$

effective head, $H = d_1 \dfrac{vl^2}{2_g}$

$$= \frac{d_1}{H} \qquad \frac{\phi^2\,(H - d_1)}{H}$$

$$= (1 - \phi^2)\,\frac{d_1}{H}\,\phi^2$$

$$= \frac{(1 - .96^2)d_1}{50.29}\ 0.96^2 \ = \frac{0.08d_1}{50.29}\ 0.92$$

Assume h $\quad = 4.2'$ then $d_1 = 4.2 \times 0.62 = 2.6$

then $\quad = \dfrac{0.08 \times 2.6}{50.29}\ 0.92$

$= 0.924$

Discharge 1,600

discharge $= 1{,}600$ cusecs

Discharge per foot width $= \frac{1600}{12} = 133$ cusecs.

$133 = Q = d_1$ (Effective head H1 is taken into account).

$= h.\ \phi \sqrt{2g\,(H_1 - d_1)}$

$133 = 0.62\ h \times 0.96 \sqrt{64.4\,(50.29 \times 0.92 - 0.62h)}$

$= 0.62 \times 0.96\ h \sqrt{64.4\,(46.6 - 0.62\ h)}$

$\frac{133^2}{0.96^2 \times 0.92^2 \times 64.4} = h^2\,(46.6 - 0.62h)$

$777 = h^2\,(46.6 - 0.62\ h)$

$h = 4.2$ satisfies the above equation

$\therefore$ the assumption $h = 4.2$ made in the beginning is correct.

$d_1 = 4.2 \times 0.62 = 2.6$

Velocity $= \frac{1600}{12 \times 4.2 \times 0.62}$

$= 51$ ft./sec.

Velocity at the bottom of the tunnel. $= 51 \times 0.9$ approx.

$= 46$ ft./sec.

Discharge 1,850 *cusecs.*

$Q = \frac{1850}{12} = 154$ cusecs.

$154 = 0.96 \times 0.62 \times h\ \sqrt{64.4\,(H_1 - 0.62\ h)}$

$\frac{154^2}{0.96^2 \times 0.62^2 \times 64.4} = h^2\,(H_1 - 0.62\ h)$

$H_1 = H$

$= \frac{0.08\ d_1}{50.29}\ 0.92$

Assume $h = 4.9$ then $d_1 = 4.9 \times 0.62 = 3.0$

Then $= \frac{0.08 \times 3.0}{50.29} = 0.92$

$= 0.925$

$H_1 = 46.7$

Substituting this value in the above equation we get

$\frac{154^2}{0.96^2 \times 0.62^2 \times 64.4} = h^2\,(46.7 - 0.62\ h)$

$h = 4.9$ satisfies the above equation

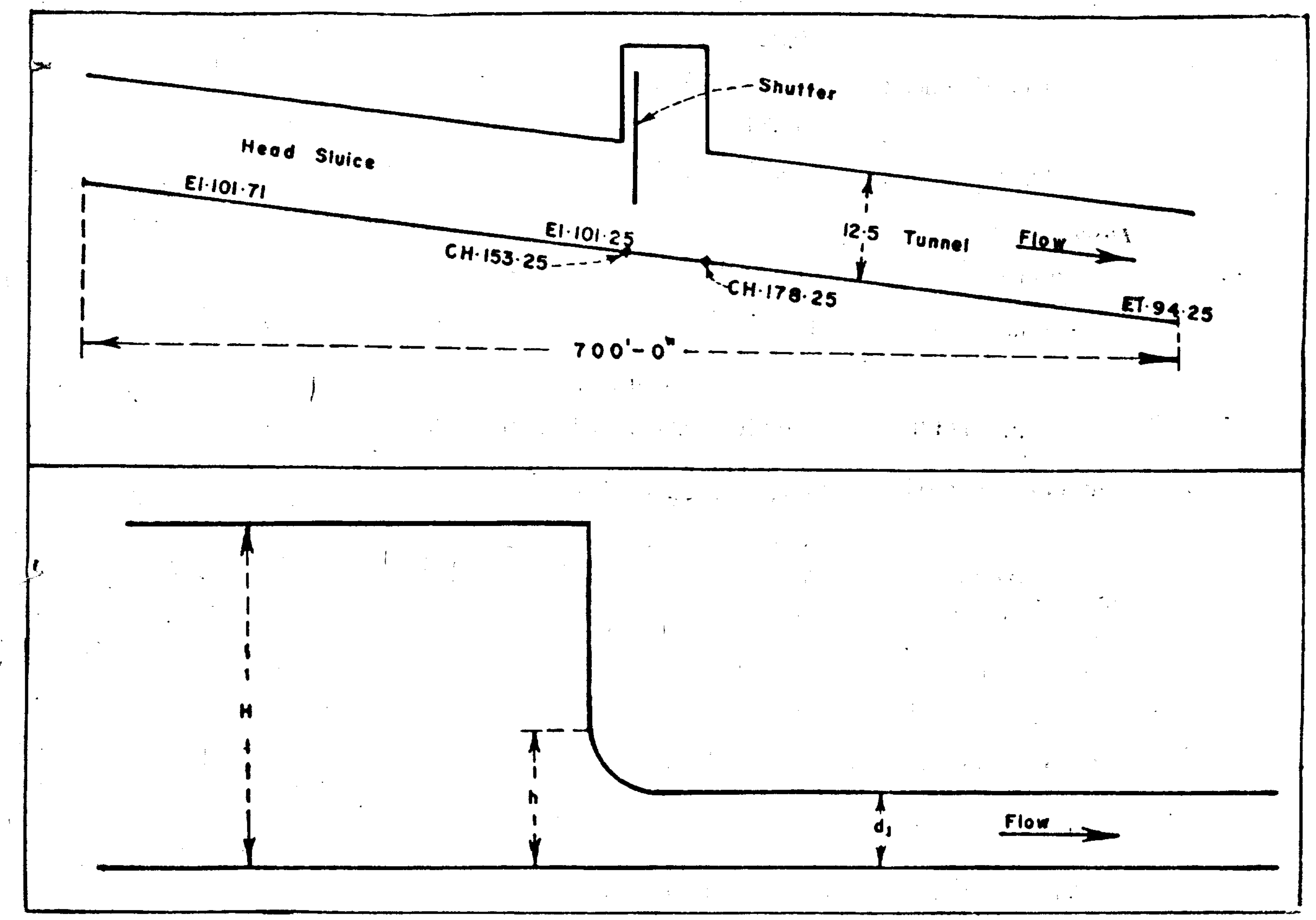
$\therefore$ the assumption that $h = 4.9$ is correct

$d_1 = 4.9 \times 0.62 = 3,0$

Velocity of flow $= \frac{1850}{12 \times 3.0} = 50.6$ ft./sec.

Velocity at the bottom may be in the order of $0.9 \times 50.6 = 46$ ft./sec.

Shaft No.1.



$d_1 = h$ is assumed as 0.62 as before

ϕ velocity coefficient assumed as 0.96 as before.

$H = 152.00 - 101.25$

$= 50.75$

Effective head $= H$

where $= (1 - \phi^2)\ \frac{d_1}{H}\ \phi^2$

$= \frac{0.08}{50.75}\ d_1\ 0.92$

Assume h $= 3.6$

$d_1 = 3.6 \times 0.62$

$= 2.23$ feet

Therefore $= \frac{0.08 \times 2.23}{50.75}$ 0.92

$= 0.923$

Effective head $= 0.923 \times 50{\cdot}75$

$= 46.84$

Operating head $= 46.84 - 0.62$ h

Discharge 1600.

Discharge $=$ 1,600 cusecs.

Discharge per foot width $= \frac{1600}{14}$

$=$ 114.3 cusecs.

$\therefore\ 114.3 = 0.62\,h \times 0.96 \sqrt{64.4\,(46.84 - 0.62\,h)}$

$h^2\ (46.84 - 0.62\,h) = \frac{114.3^2}{0.96^2 \times 0.62^2 \times 64.4}$

$= 574$

$h^2 - \frac{0.62\,h^3}{46.84} = \frac{574}{46.84}$

$h^2 - 0.0013\,h^3 = 12.26$

h $=$ 3.6 satisfied the equation

$\therefore$ The assumption that h $=$ 3.6 is correct

Velocity $= \frac{1600}{14 \times 3.6 \times 0.62} = 51.25$ ft./sec.

Velocity at the bottom $= 51.25 \times 0.9$

$= 46.125$

$=$ 46 ft./sec.

Discharge 1850.

$q = \frac{1850}{14}$

$=$ 132 cusecs.

$= \frac{0.08\,d_1}{50.75}$ 0.92

Assume h $= 4.2$

$d_1 = 4.2 \times 0.62 = 2.6$

$= \frac{0.08 \times 2.6}{50.75}$ 0.92

$= 0.924$

$H_1 = 0.924 \times 50.75$

$= 46.9$

$132 = 0.96 \times 0.62\,h \sqrt{64.4\,(46.9 - 0.62\,h)}$

$h^2\,(46.9 - 0.62\,h) = \frac{132^2}{0.96^2 \times 0.62^2 \times 64.4}$

$= 763.7$

$h^2 - \frac{0.62}{46.9}\,h^3 = \frac{763.7}{46.9}$

$h^2 - 0124\,h^3 = 15.3$

Put h $= 4.2$

then left hand side $= 4.2^2 - 0.0184 \times 4.2^3$

$= 17.64 - 0.92$

$= 16.72$

Put h $= 4.0$

then left hand side $= 16 - 0.79$

$= 15.21$

$\therefore$ h $=$ 4.0 correct

$d_1 = 4.0 \times 0.62 = 2.48$

Velocity $= \frac{1850}{14 \times 2.48} = 53.3$ ft./sec.

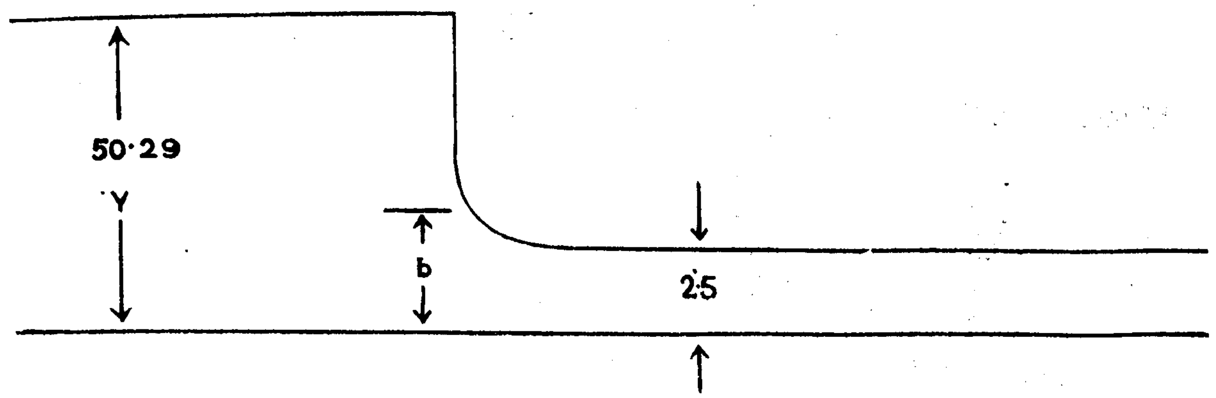
Velocity at bottom $= 53.3 \times 0.9 =$ 48 ft./sec.

APPENDIX II

Periyar Head Sluice.

(Rough Method.)

Head sluice gate height opened : Discharge 1,600 cusecs.



$Q = C \; b \; B\sqrt{2gy_1}$ $\quad y_1 = 50.29$

$B = 12$

$C = 0.57$

From page 537. Engineering Hydraulics—

Assume $b = 3.5$

$\frac{y_1}{b} = \frac{50.29}{3.5} = 14.4$

$C = 0.57$

$1600 = 0.57 \times b \times 12 \times \sqrt{64.4 \times 50.29}$

$b = 4.1$ feet $\quad d_1 = 0.62 \times 4.1 = 2.54$

Shaft No. 1, *Gate height opened.*

$1600 = 0.57 \times b \times 14\sqrt{64.4 \times 50.29}$

$B = 3.51 \quad d_1 = 0.62 \times 3.55 = 2.2$

NOTE.—The gate openings have been checked by detailed calculations and are found to tally.

For a discharge of 1850 — *Shaft No.* 1.

$Q = C \; b \; B\sqrt{2gy}$

$y_1 = 50.29$

$b = ?$

$B = 14$

$C = 0.58$

Assume $h = 4.8'$

From page 537, Engineering Hydraulics.

$\frac{y_1}{4.8} = 10.5 \qquad e = 0.58$

$1850 = 0.58 \times b \times 14 \sqrt{64.4 \times 50.29}$

$b = 4.0$

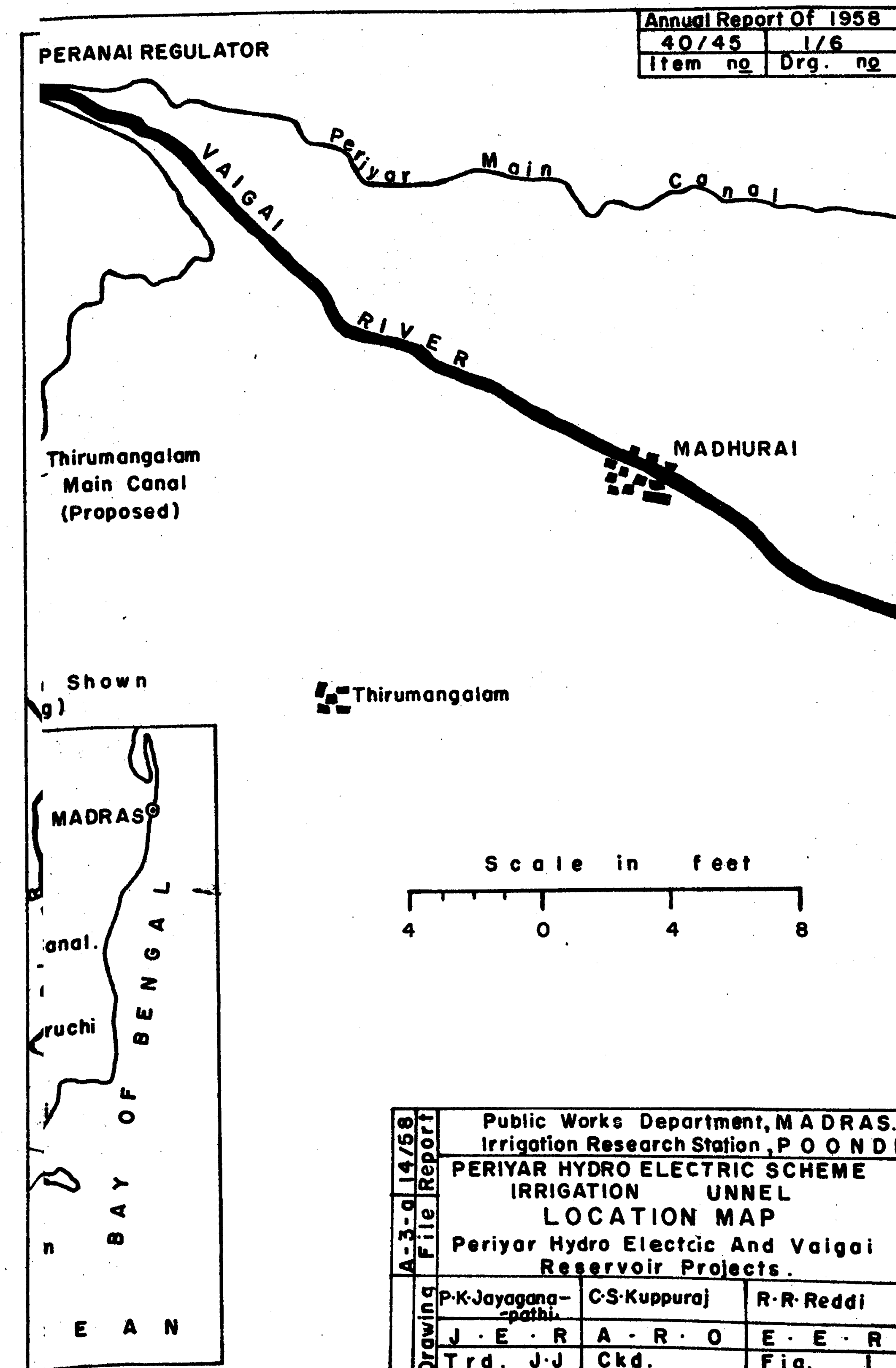
Versus 4.00 as given previously

Head Sluice No. 1

$1850 = 0.58 \times b \times 12 \sqrt{64.4 \times 50.29}$

$b = \frac{4.0 \times 14}{12} = 4.67$

Versus 4.9 feet got from calculations, exactly as shown previously.



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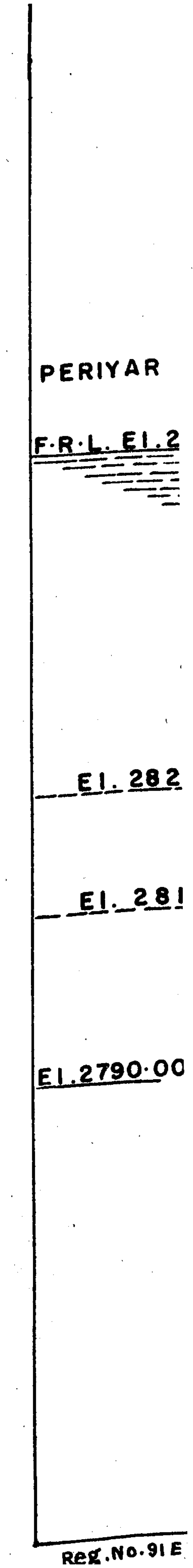
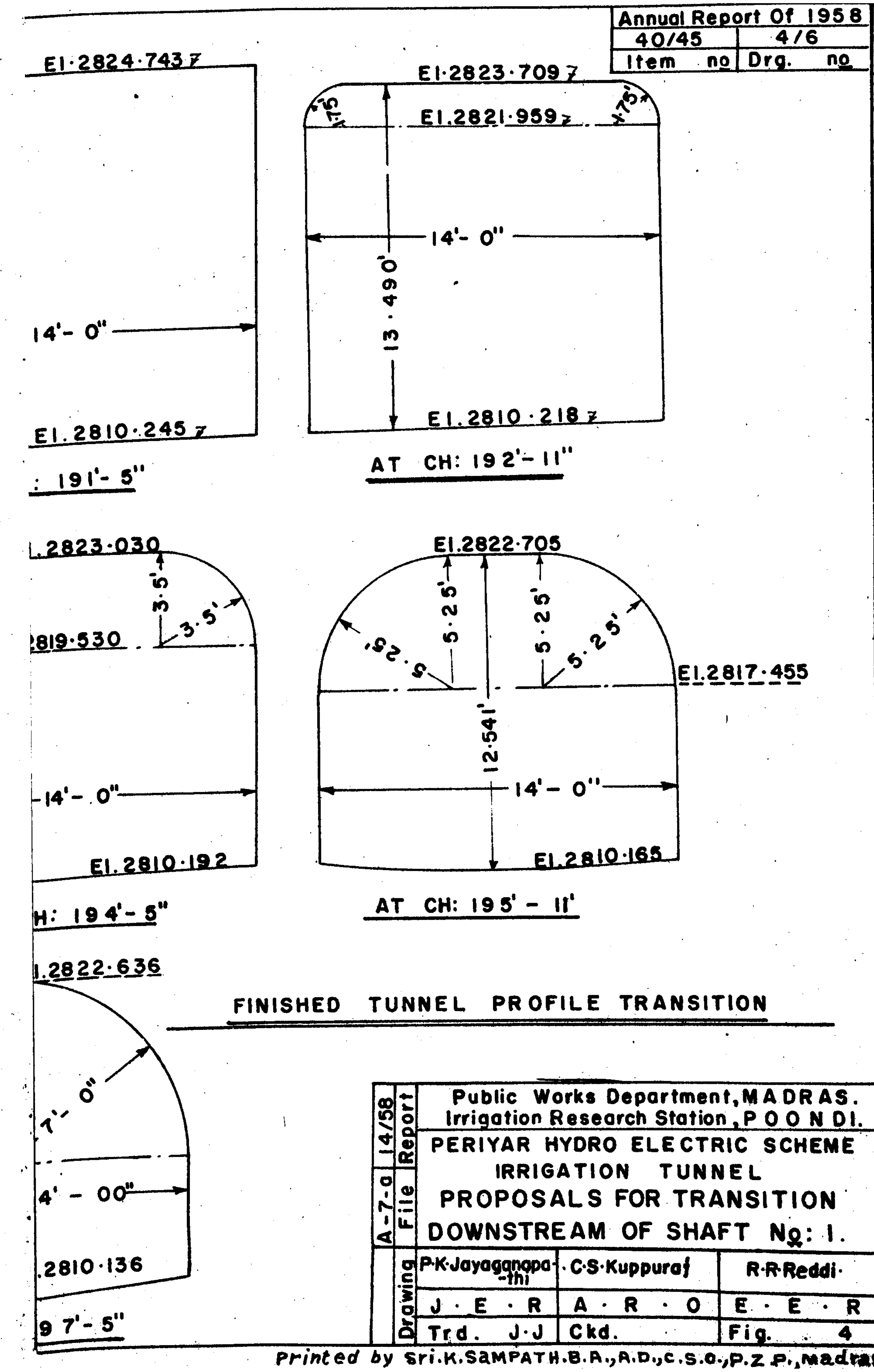

PERIYAR
F·R·L. El.2
El. 282
El. 281
El.2790·00
Reg.No·91E


Annual Report Of 1958
40/45 Item no
4/6 Drg. no
El·2824·743
14'- 0"
El. 2810·245
: 191'- 5"
El·2823·709
El. 2821·959
14'- 0"
13·490'
El. 2810·218
AT CH: 192'- 11"
.2823·030
3·5'
3·5'
2819·530
-14'- 0"
El. 2810·192
H: 194'- 5"
El. 2822·705
5·25'
5·25'
5·25'
5·25'
El. 2817·455
12·541'
14'- 0"
El. 2810·165
AT CH: 195'- 11"
.2822·636
7'- 0"
4' - 00"
.2810·136
9 7'- 5"
FINISHED TUNNEL PROFILE TRANSITION
A-7-a File
14/58 Report
Public Works Department, MADRAS.
Irrigation Research Station, POONDI.
PERIYAR HYDRO ELECTRIC SCHEME
IRRIGATION TUNNEL
PROPOSALS FOR TRANSITION
DOWNSTREAM OF SHAFT No: 1.
Drawing
P·K·Jayagganapa-thi
C·S·Kuppuraj
R·R·Reddi·
J · E · R
A · R · O
E · E · R
Trd. J·J
Ckd.
Fig. 4
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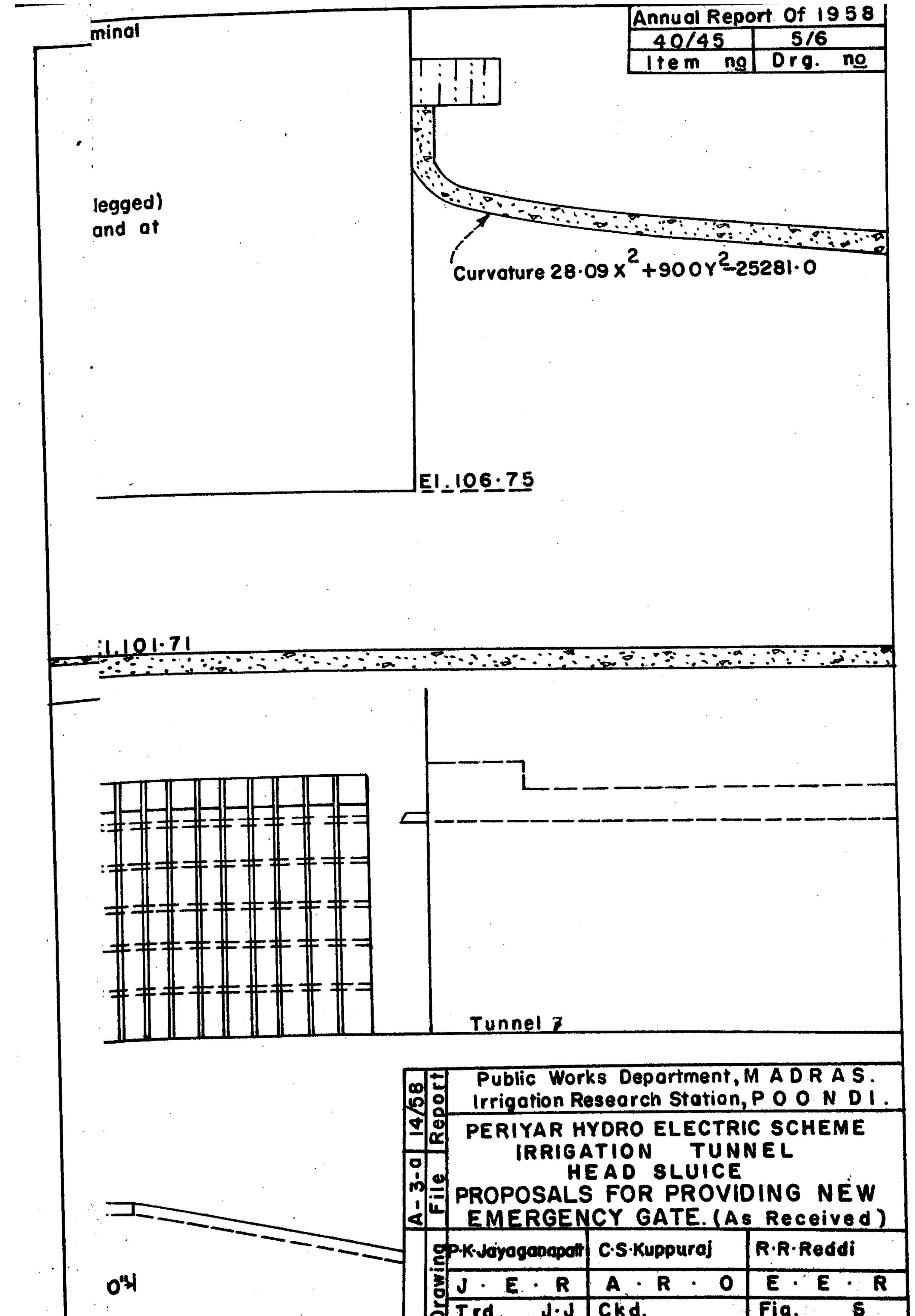

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PARSHALL FLUME.

(C.B.I. & P. No. D-3.)

1. Synopsis :

Parshall Flume is an improvement over the Common Venturi Flume. It can be used for measuring the Irrigation supplies in the water courses from the smallest ditch to a canal carrying as much as 200 cusecs. One such flume was designed to carry a maximum discharge of 3 cusecs in the modular region and constructed in the laboratory in connection with the installation of such flumes for the purpose of measuring the outflow in irrigation works. Experiments were conducted in the modular and non-modular ranges to find out the accuracy of the Parshall Flume.

2. Introduction :

Parshall Flume is an improvement over the usual Venturi Flume. It can be used for the measurement of water in the water courses from the smallest ditch to a canal carrying as much as 200 cfs. It functions with less loss of head than the weirs and the modularity level is also high, as much as 60 per cent so that only one head measurement need be made for measuring the flow in most of the cases. It has the advantages that it can be made portable, silt will not settle in the flume and that the accuracy of measurement is also within the allowable limits.

One such flume was designed and constructed for measuring a maximum flow of 3 cfs. in the free flow stage. Experiments were conducted both for free and submerged flow and the results are presented in this report.

3. Design of a Parshall Flume for 3 cfs.

It is proposed to install the Parshall Flume in a masonry flume of width two feet. Taking Manning's coefficient of rugosity to be 0.013 for concrete surface, the normal depth of flow can be found out by the application of Manning's formula. The normal depth was found to be 0.87 ft. For the maximum discharge of three cusecs, and for a head " Ha " of 1.0 foot, the throat width was found to be 9 inches from Tables (Table 7, Reference 1).

For the free flow, the afflux in the flume

$= 0.4$ Ha

$= 0.4$ ft.

So, increased depth of flow

$= 0.87 \quad 0.4$

$= 1.27$ ft.

So, the height by which the floor of the flume should be raised is 1.27 — 1.00 = 0.27 ft. The other dimensions of the flume can be taken from any standard Text-book on Irrigation Engineering. The details of 9 inches throat width Parshall Flume are given in Figure 1.

4. Experiments :

The Parshall Flume was constructed in a two feet wide flume as shown in Figure 1. The discharge into the flume was measured with a V-notch at the upstream end of the flume. The same discharge passing through the Parshal flume was also calculated using the Standard Tables for Parshall Flume. A regulator was put up downstream of the Parshall Flume and the discharge was assessed by observing the water levels upstream and downstream of the regulator and calculating the discharge by the empirical formula. $Q = 6 A \sqrt{h}$ where A is the area of flow and h is the head causing discharge. All these three discharges are shown plotted in Figure 2.

For free flow, the Standard Tables always give a discharge greater than the V-notch discharge and the formula $Q = 6 A \sqrt{H}$ always gives a discharge lower than the V-notch discharge. It is easily seen that the discharge measured by Parshall Flume is more accurate than that assessed by the formula $Q = 6 A \sqrt{h}$ as the curve for Parshall discharge is the discharge curve furnished by the formula $6 A \sqrt{h}$.

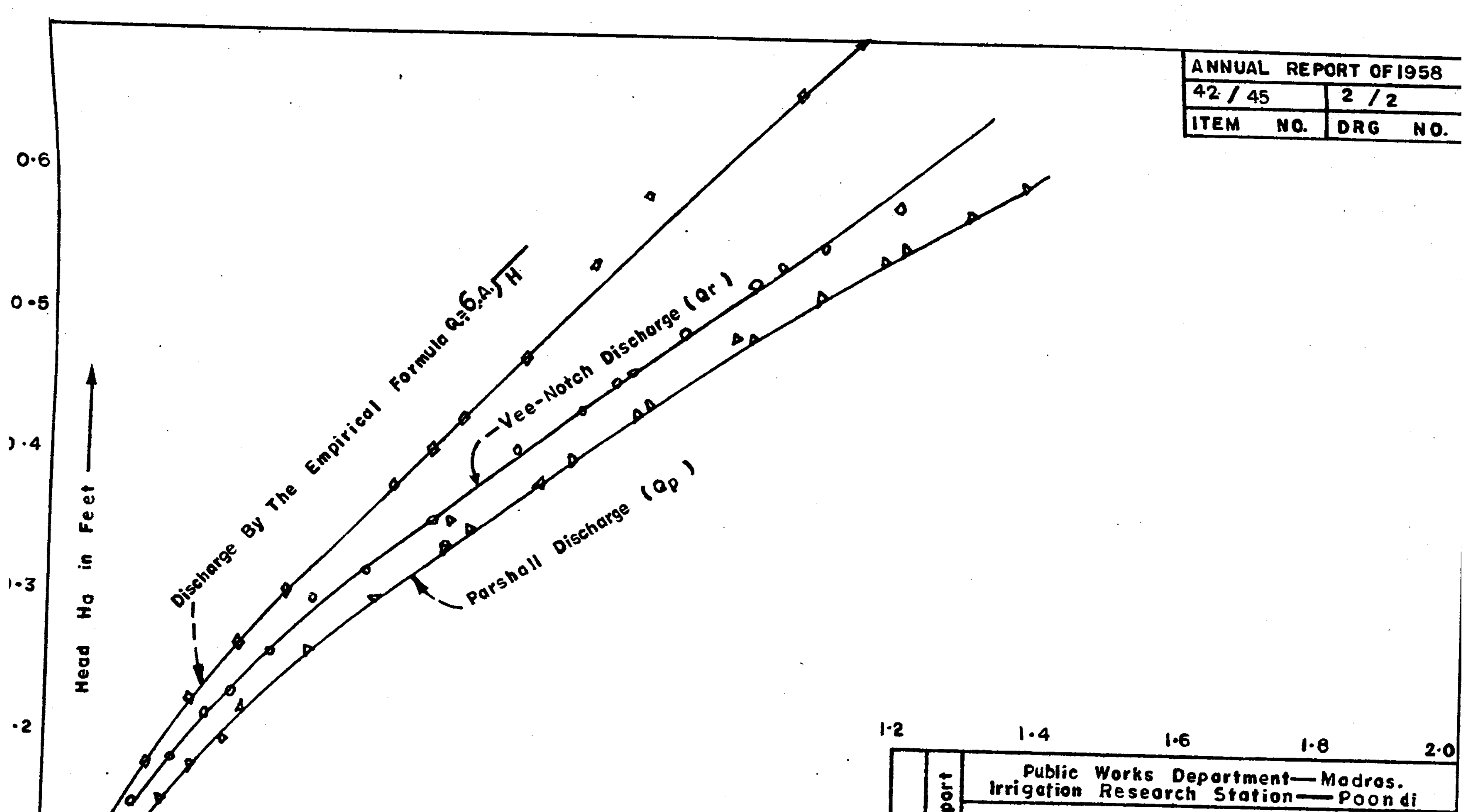
For submerged flow, the submergence correction given by Standard curves (Reference 2) is so very approximate that it was found not necessary to calibrate the Parshall Flume for submerged flow.

5. Conclusion :

Parshall Flumes can advantageously be used for the measurement of water in small water courses used for irrigation purposes. They can be made even portable. They can be designed for any discharge range by referring to any standard Text Book on Irrigation Engineering. In free flow range, the accuracy obtained is far better than the discharge assessed by the formula usually used, i.e., $Q = 6 A \sqrt{h}$

6. References :

1. Israelson, Orson W.—Irrigation Principles and Practices—Joh Wiley & Sons, New York.



ANNUAL REPORT OF 1958	
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ITEM NO.	DRG NO.

Public Works Department—Madras.
Irrigation Research Station—Poondi

CALIBRATION OF A 9" PARSHALL FLUME FOR FREE FLOW.

Report			
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Reg.No.48 E'61.(Saka Era 1882) 375.

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CONTROL OF EVAPORATION LOSSES FROM OPEN STORAGES.

(C.B.I. & P. No. D-4.)

Synopsis :

It has been concluded from experiments conducted in Australia and other countries that a monomolecular film of Cetyl Alcohol spread over the water surface arrests evaporation to a certain extent. The applicability of the above process to any particular place requires data regarding wind, weather, temperature, humidity, etc., as the success of this process depends on these various factors. In order to collect these data and to find the effect of these changing factors on the monomolecular film experiments were conducted in a medium sized tank near the Irrigation Research Station, Poondi. This paper sets forth the details of the experiments conducted and the difficulties met with during these experiments. The experiment is being continued for collection of more data for drawing definite conclusions regarding the efficacy of the chemical as an effective agent to control the loss due to evaporation from water surface.

1. Introduction :

1.1. Evaporation is the process by which a liquid or a solid is changed into a 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 molecules of water is that the faster moving molecules near the surface escape into the atmosphere. There will be also a return of these molecules back into the parent medium, i.e., water body. If on the whole, the number of molecules escaping 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, Thus evaporation is necessarily a cooling process.

Immediately adjacent to the water surface is a thin layer of air whose temperature is the same as that of the water. This gets saturated and if the vapour pressure of the air above is less than that in this film, the vapour near the water surface will be dispensed through diffusion, convection of wind section and consequently evaporation will continue. The rate of evaporation depends upon the difference between these vapour pressures or upon the pressure gradient. This principle is known as Dalton's Law and is expressed by the formula.

$$E = c\,(p_w - p_a)$$

where E is the rate of evaporation,

P_w is the vapour pressure of the film adjacent to the water surface,

P_a is the vapour pressure in the air above

c is a constant which varies with upon

(1) the depth of the water surface ;
(2) the wind velocity ;
(3) the seasonal variations ;
(4) the topography of the surroundings ;
(5) the barometric pressure and
(6) the humidity gradient of the air above.

A number of methods are available for assessing this rate of evaporation from water surfaces.[1] They are,

(1) The storage equation,
(2) Measurement in auxiliary pans,
(3) Evaporation formulae,
(4) Humidity and wind velocity gradients and
(5) Measurement of insolation.

Since evaporation causes a considerable loss of useful storage especially in tropical countries, various measures are being tried to reduce the loss of water due to evaporation.

1.2. As a result of the studies conducted in Australia[2], it has been concluded that a monomolecular film of Cetyl Alcohol, otherwise known as Hexadecanol, spread over water surface reduces the loss due to evaporation. Studies are being conducted in this Station in a medium-sized tank to find the feasibility of the economic usage of this chemical for conservation of water from large open storages for the different meteorological conditions obtaining in this part of the State based on the information available on the technique and procedure followed in Australia.

1.3. For the spreading of the chemical Cetyl Alcohol over the water surface the following two methods are available : (1) Solid method and (2) Solution method (i.e., the chemical Cetyl Alcohol is dissolved in a suitable solvent and this solution is sprayed on the water surface). Of these two methods, it has already been settled by Australian experiments that the solid method is not capable of application to areas of water spread more than two acres[3] and hence the solution method has been recommended as being fruitful for large application.

2. Laboratory Experiments :

2.1. Before starting the field experiements in Poondi preliminary experiments were conducted in the Concrete and Soil Research Laboratory at Madras for arriving at a suitable solvent for the chemical. Solvents such as S.B.P. spirit 40/60 S.B.P. spirit 55/115°C mineral turpentine, etc., were tried. As regards the S.B.P. spirit it was found that though the Cetyl Alcohol is fairly easily soluble in this spirit, these solvents volatilised off at an extremely rapid rate with the result the hexadecanol would not spread out as rapidly as would be desirable and besides these, was a tendency for the Cetyl Alcohol to be thrown out as a Crystal over the water surface. Also, other technical difficulties encountered in storing large quantities of these solvents for which

special licence is required ruled out the possibility of the selection of these solvents for large scale application Mineral turpentine is not so highly volatile as the others but as regards its solvent power for Cetyl Alcohol it is a bit low. But after actual trials it was found that at the usual atmospheric temperature obtaining here the Cetyl Alcohol was sufficiently soluble in Mineral Turpentine and this solvent was used in the field experiments conducted. Studies were also conducted in the laboratory for arriving at the concentration of the solution to be used and the rate of frequency of the application. After these experiments, a concentration of 13 grains of Hexadecanol in one gallon of solvent was arrived at as a suitable proportion. The rate and frequency of application being dependent on many variables such as wind, waves, etc., it was considered impracticable to arrive at any conclusions by means of small scale laboratory experiments and hence these aspects were reserved for field studies.

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.[2] In the absence of any definite information on the exact method of determination of the surface pressure, a few indicator oils such as, cocoanut oil, castor oil and oleic acid were tentatively used to indicate surface pressures below 17 dynes/sq.c.m., 17 to 30 dynes/sq.c.m. and 30 to 60 dynes/sq.c.m., respectively. Later Shell High Speed Diesel oil, Shell vitrea oil-13, Shell vitrea oil-21, Shell ensis fluid-256 were used to indicate surface pressure of 13 dynes, 16 dynes, 24 to 40 dynes respectively.

3. Field Studies :

3.1. Two medium sized irrigation tanks, viz., Uthukottai and Sengaral tanks were selected initially for conducting experiments on a comparative basis by treating one of the tanks with the chemical solution and the other untreated. Due to late arrival of the chemical and difficulty in procuring the solvent, the studies could not be started well in time as anticipated. By the time the chemical and solvent were procured one of the tanks got dried up and so only the other tank was utilised for conducting the experiments.

In this tank also many difficulties were met with. The enormous growth of weeds in the tank was not conducive to the effective formation of the film. Fishing operations by local villagers and the activities of stray animals and birds in the tank contributed towards the disturbance and rupture of the film with the result that at no time was the film found to be fully covering the whole area of the tank. Moreover the outflow from the tank which could be measured with great difficulty when it was only through sluices was made, impossible due to the baling out of water directly from the tank for irrigation purposes. Above all, this tank also got dried up considerably and the venue of experiments had to be shifted to a more suitable site near the Research Station at Poondi.

3.2. *Experimental set up in Buderi Tank.*—After due consideration, 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 was selected (vide Figure 1). This tank has a waterspread area of about 20 acres and a maximum depth of about 11 feet of F.T.L. This tank has the added advantage in that the water of this tank is not used for any irrigation purposes and in summer there is no inflow into or outflow from the tank which is a very desirable feature as far as the evaporation experiments are concerned. For purposes of these studies a field laboratory was set up on the northern foreshore of the tank containing (*a*) a Maximum and Minimum thermometer for recording temperatures, (*b*) a wet and dry bulb thermometer for humidity observations, (*c*) a rain gauge, (*d*) an Anemometer for knowing the wind velocity, (*e*) a wind vane for knowing the direction of wind,, (*f*) two land pans of the standard U.S. Weather Bureau type. In addition to this three, four feet diameter pans were installed to float on the water surface of the Buderi Tank. A gauge well was fixed in the tank to take observations of water levels from which losses 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 pans and one gauge well were installed in the adjacent Poondi Reservoir for recording similar observations. Floating rafts were used for the purposes of spraying, taking readings of water levels from the floating pans, replenishing the water in the floating pan and for testing the film strength at various places on the tank surface. The location of all these experimental appurtenances in and around the tank are shown in Figure 2.

3.3. *Experiments.*—In the present set of experiments the object is to collect data regarding :

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

(2) The extent upto which the loss due to evaporation can be reduced by spreading the chemical Cetyl Alcohol on the water surface.

(3) The effect of wind and rain on the chemical film.

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

Before starting the regular experiments, with a view to clear the water surface of dirt and scum a large quantity of the solution, i.e., 9 lbs. of Cetyl Alcohol in 20 gallons of Mineral Turpentine was sprayed on the Buderi water surface. The chemical while spreading locks in its film all the dirt and scum and this is swept along with the film by the wind towards the leeward shore. The area covered by the solution and the actual spreading of the chemical was easily visible to the naked eye in that the microwaves and ripples got quelled over the treated surface. Varied high concentration solutions were sprayed for the following few days to get the water surface practically clean. Systematic experiments were then started.

3.4. *Surface pressure of film.*—The surface pressure of film being the main contributing factor in arresting the evaporation, attempts were made to assess the actual surface pressure exerted by the film spread on tank water surface. Due to the erratic behaviour of the indicator oils, viz., Cocoanut oil, Castor oil and Oleic acid, when used on the treated water surface, it was not possible to precisely assess the surface pressure of the film. It was next decided to test the same by taking the treated water of the tank in a sauce pan and conducting similar tests. This method was also found unsatisfactory as the film got ruptured or dislodged if the sample was not taken carefully. Next an enclosure 3′ 0″ diameter open at top and bottom was suspended in the surface

and tests were carried out within these enclosure. This proved to be more effective than the other methods and hence was adopted for testing of film strength. The oily patches of surface were carefully removed after every test.

Later the following indicator oils were used for testing the strength of the film as these oils gave sharper indications of the surface pressure of the film.

			Surface pressure in dynes/sq.c.m.
(1) Shell H.S.D. Oil	...	...	13
(2) Shell Vitrea Oil-13	...	...	16
(3) Shell Vitrea Oil-21	...	...	24
(4) Shell Ensis Fluid-256	...	...	40

3.5. *Concentration of the chemical solution.*—To arrive at the optimum concentration of the solution, various proportions of the chemical and the solvent were applied and the observations were recorded for a certain period in each case. Treatment with the chemical solution was given to the tank surface to one of the floating pans provided in Buderi Tank and to one each of the pans of 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 five 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 relevant meteorological data and the loss of water from the pans as well as the tank were recorded during the period of experimentation.

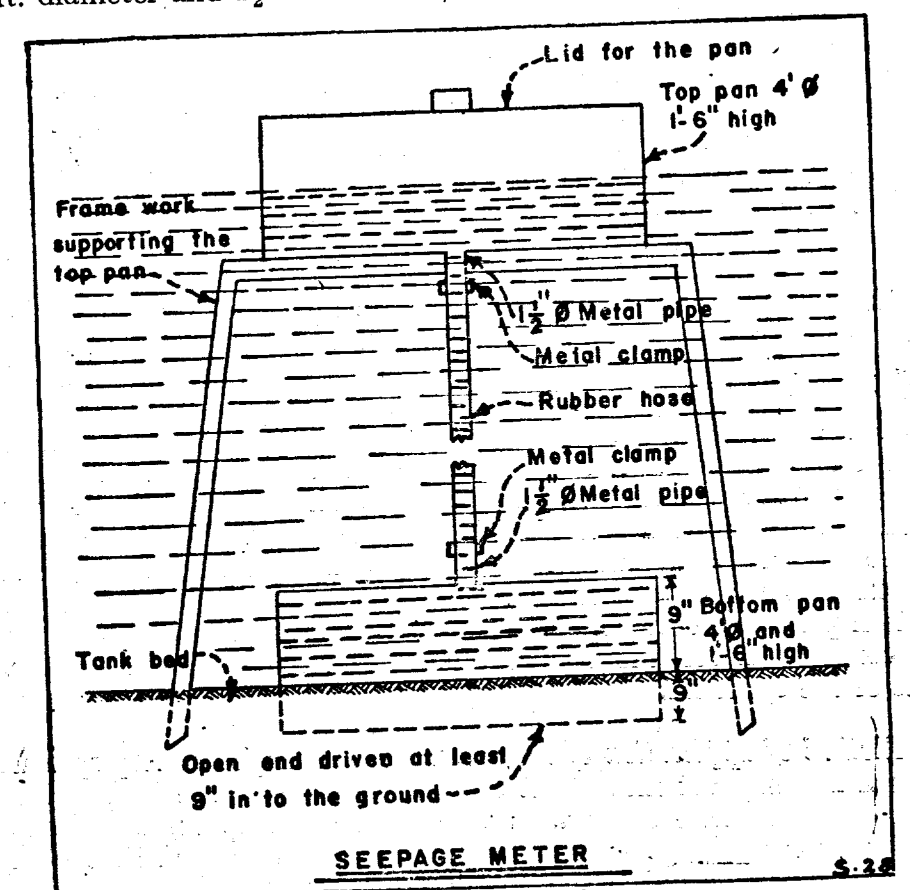
In the next series it has been proposed to apply various quantities of the solution to the tank water surface keeping the concentration of the solution (18 lbs. of Cetyl Alcohol to 40 gallons of Mineral Turpentine) the same—Various quantities of this solution from 1/2 a gallon to 3 gallons in increments of 1/4 gallon is being applied to the tank water surface every day for a certain period in each case and the relevant observations are being recorded. By this means it is proposed to find the amount of solution required for getting the maximum savings in the evaporation loss from which the amount of chemical required per acre to obtain the maximum savings in evaporation can be worked out. Observation without any treatment to the tank is also proposed to be recorded for a certain period in between these series so that this result can be used as a basis of comparison for the results obtained after treatment with various quantities of the chemical.

3.6. *Method of application of the chemical.*—In the beginning spraying was done from a floating raft moving at right angles to the direction of the wind. Later spraying was done along with windward shore by means of spraymax 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 shore as soon as it was applied to the surface. Hence it was decided to have the chemical dripping from dispensing units made out of tins with a pin hole each at the bottom. These were mounted on stands along the shore. But it was observed that this method also had some drawback the lateral dispersion of the film was not sufficient to cover the entire tank. The number of such units and also number of dispensing points per unit were then increased to overcome this drawback to the extent necessary to have the entire tank surface covered with the film.

3.7. *Effect of wind.*—The wind was observed to have beneficial as well as detrimental effect on the process as a whole. To clean the surface as a preliminary operation to the regular operations and to spread the chemical the wind was beneficial, whereas it was detrimental in what it carried away the film towards the leeward shore, thus vitiating the effects of the film 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.

3.8. *Seepage loss.*—For calculating the loss of water due to evaporation alone, it is necessary that the loss due to seepage from the tank is known. In order to assess the quantum of water lost due to seepage from the tank, a device called the seepage motor was installed in the Buderi Tank.

The device consists of two cylindrical pans of equal diameter and size (i.e., 4 ft. diameter and $1\frac{1}{2}$ ft. diameter) is drilled in the middle of the pans and



short metal pipes about 4" long and of the same diameter as 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 slipping

the ends of a rubber hose on them. One of these pans is inverted and rammed into the bed such that at least 9" of its sides penetrate into the soil and the rest is above the tank bed. The other pan with its open end facing upwards is supported on a framework above the first pan with its bottom at 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 that of water in the tank. As the pans are connected to each other the hollow portion of the bottom pan will also be full of water. When the water from the bottom pan seeps 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.

This meter is being tried in the tank and no definite conclusion has yet been arrived at. Further experiments are proposed to be conducted in the two or more similar devices installed at various places in the tank. There may be variations in the bed material in the different places in the tank and consequently the amount of seepage may also vary from place to place in the tank bed. It is proposed to study this aspect by taking observations simultaneously on two or three such meters set at different places in the tank. Further, observations in a number of meters instead of in only one will also be helpful in acertaining the correctness of results obtained. Hence conclusive results can only be obtained after these proposed studies are completed.

3.9. *Collection of data.*—During the experiments, the following data are being collected daily :

(*a*) Maximum and minimum temperature.

(*b*) Humidity at 8 a.m., 12 noon and 4 p.m.

(*c*) Rainfall measurements, if any.

(*d*) Intensity and direction of wind.

(*e*) The water temperatures of the four land pans and three floating pans daily at 8 a.m., 12 noon and 4 p.m.

(*f*) Water temperature of Poondi Reservoir and Buderi Tank at 8 a.m., at 12 noon and 4 p.m.

(*g*) Water levels in all land and floating pans.

(*h*) Water levels in Poondi and Buderi Tanks.

(*i*) Water level in the seepage meter.

4. Conclusion :

From the data collected the savings in evaporation loss obtained due to the spreading of the chemical Cetyl Alcohol over the water surface can be worked out. The temperature and humidity observations will facilitate (1) Calculation of evaporation loss theoretically from formulae, based in Dalton's Law and compare the results from other computations made from field observations.

(2) Preparations of charts like humidity *vs.* Evaporation loss, Temperature *vs* Evaporation loss, etc., and (3) arriving at conclusions as to the effect of vapour pressure of air and wind velocity on evaporation loss.

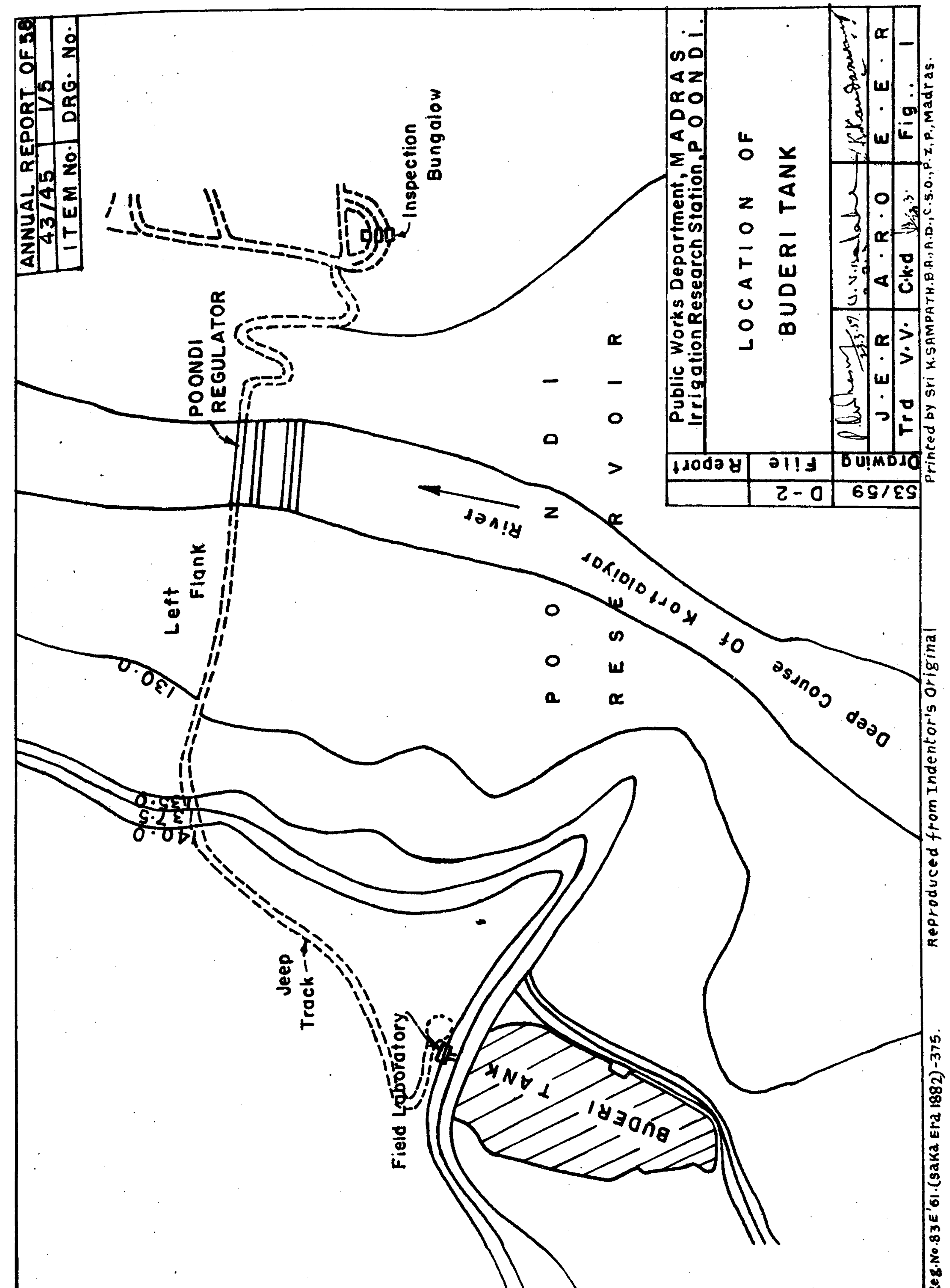
More data need be collected before arriving at definite conclusions regarding the practical utility or otherwise of the proposed treatment with the chemical, Cetyl Alcohol.

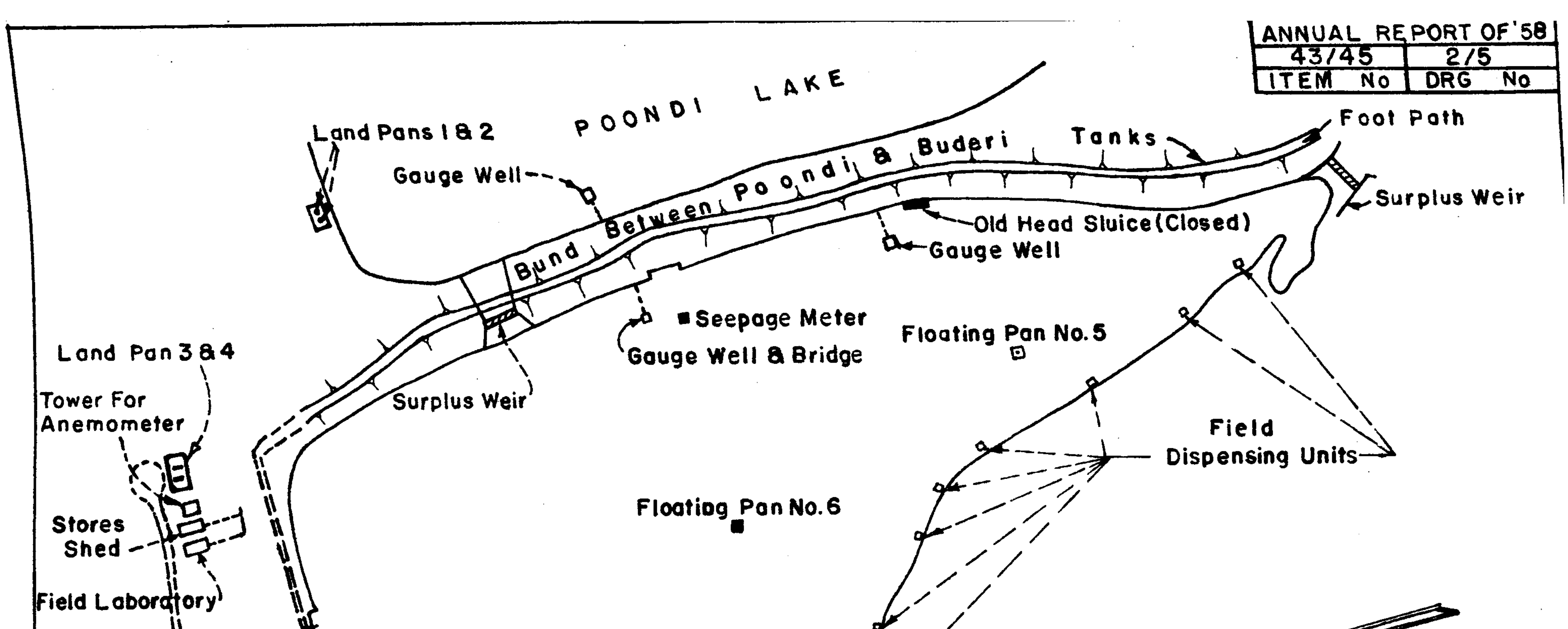
5. References :

1. " Hydrology " by C. O. Wisler and E. F. Brater. Chapman and Hall, Limited.

2. " Influence of Monolayers on the Natural Rate of Evaporation of Water " by W. M. Mansfield, Australia.

3. C.S.I.R.O. Leaflet Series No. 15—" Saving Water in Dams."





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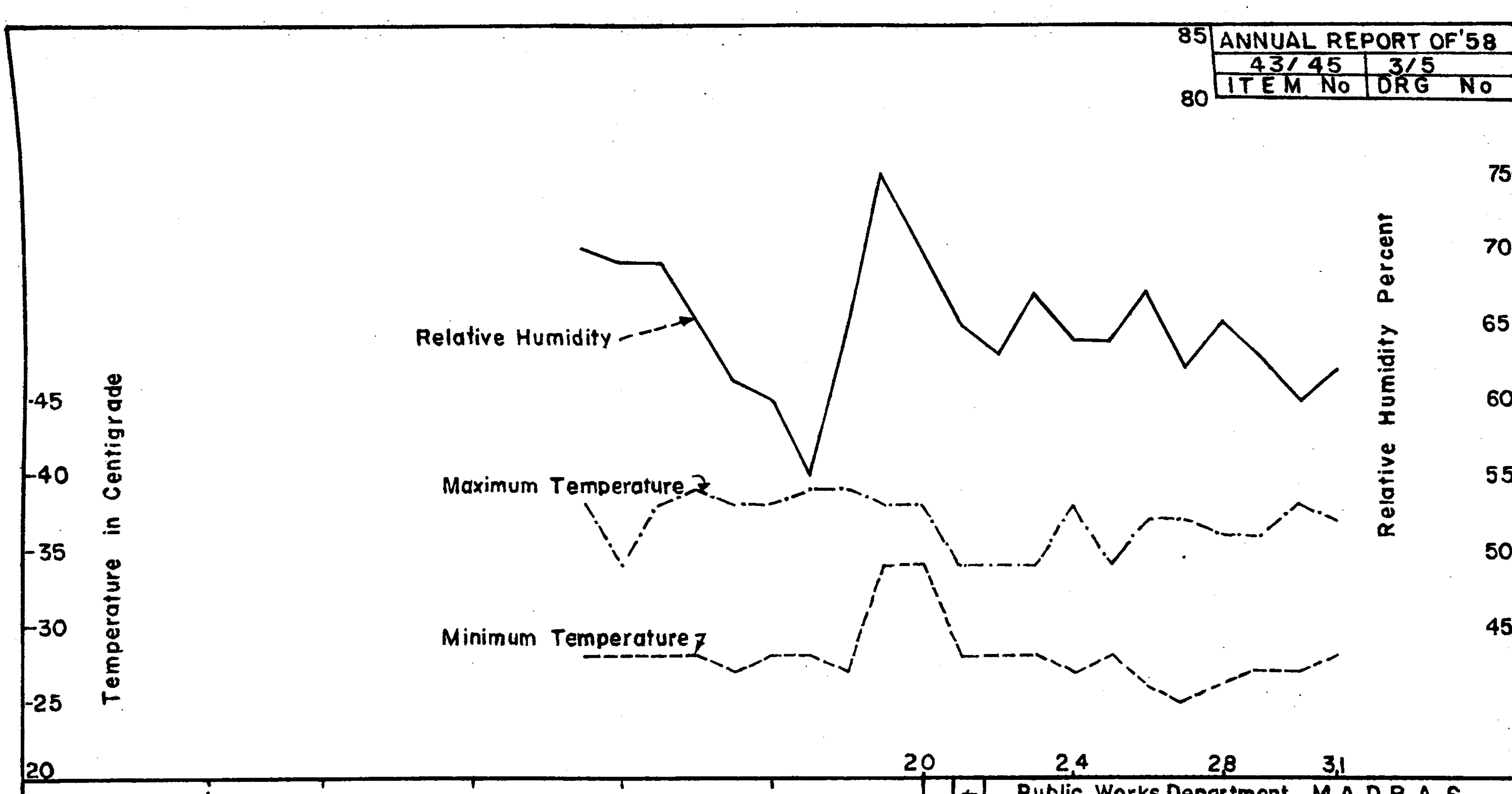

Public Works Department, MADRAS
Irrigation Research Station, POONDI

Maximum & Minimum Temperatures
And Relative Humidity—May, 1958

Report			
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Drawing 55/59	J·E·R	A·R·O	E·E·R
	Trd V·V	Ckd	Fig...... 3

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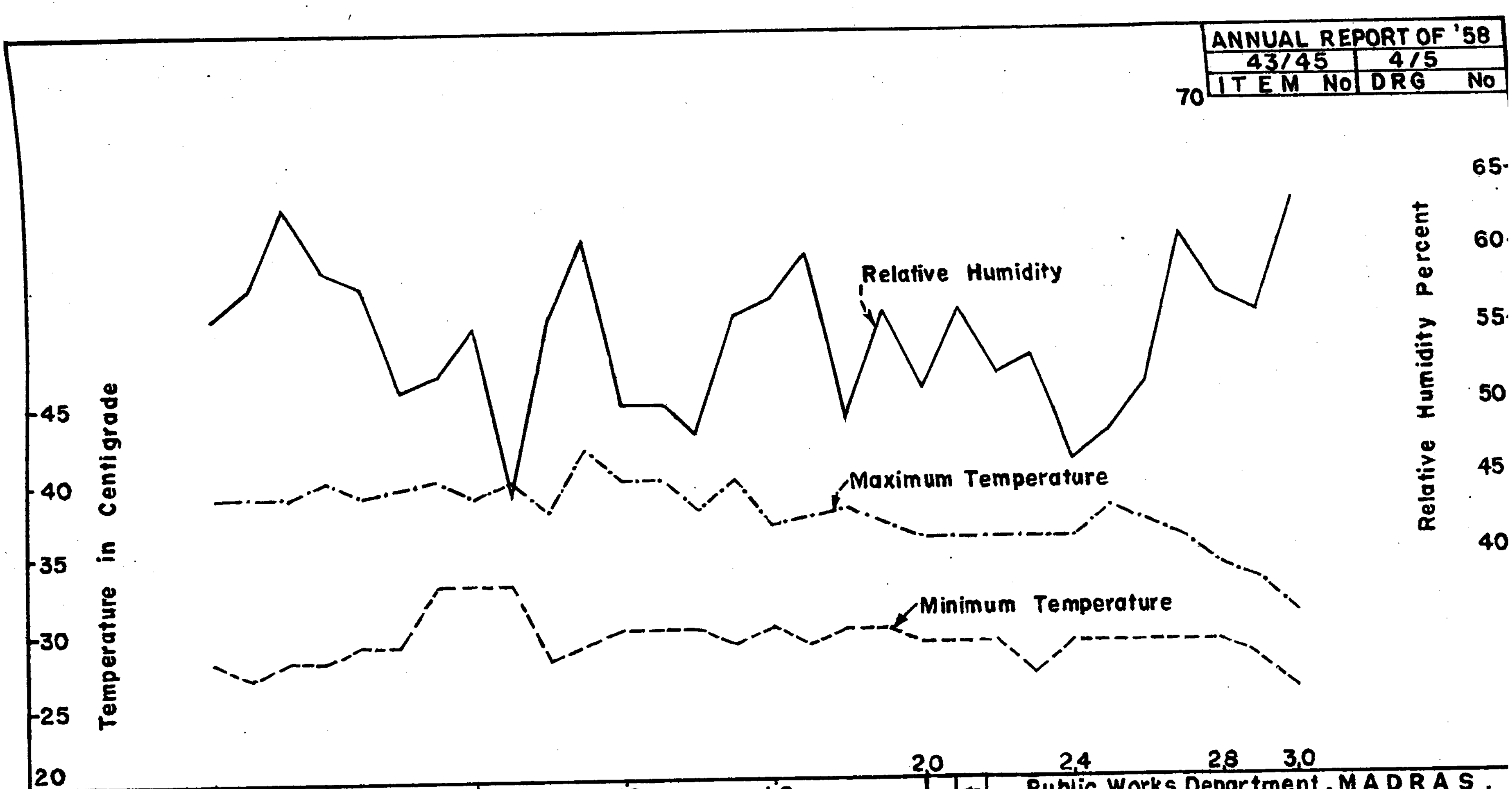

56/59	Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI		
	File	Maximum & Minimum Temperatures And Relative Humidity – June, 1958		
	Drawing	J·E·R	A·R·O	E·E·R
		Trd V·V	Ckd	Fig..4

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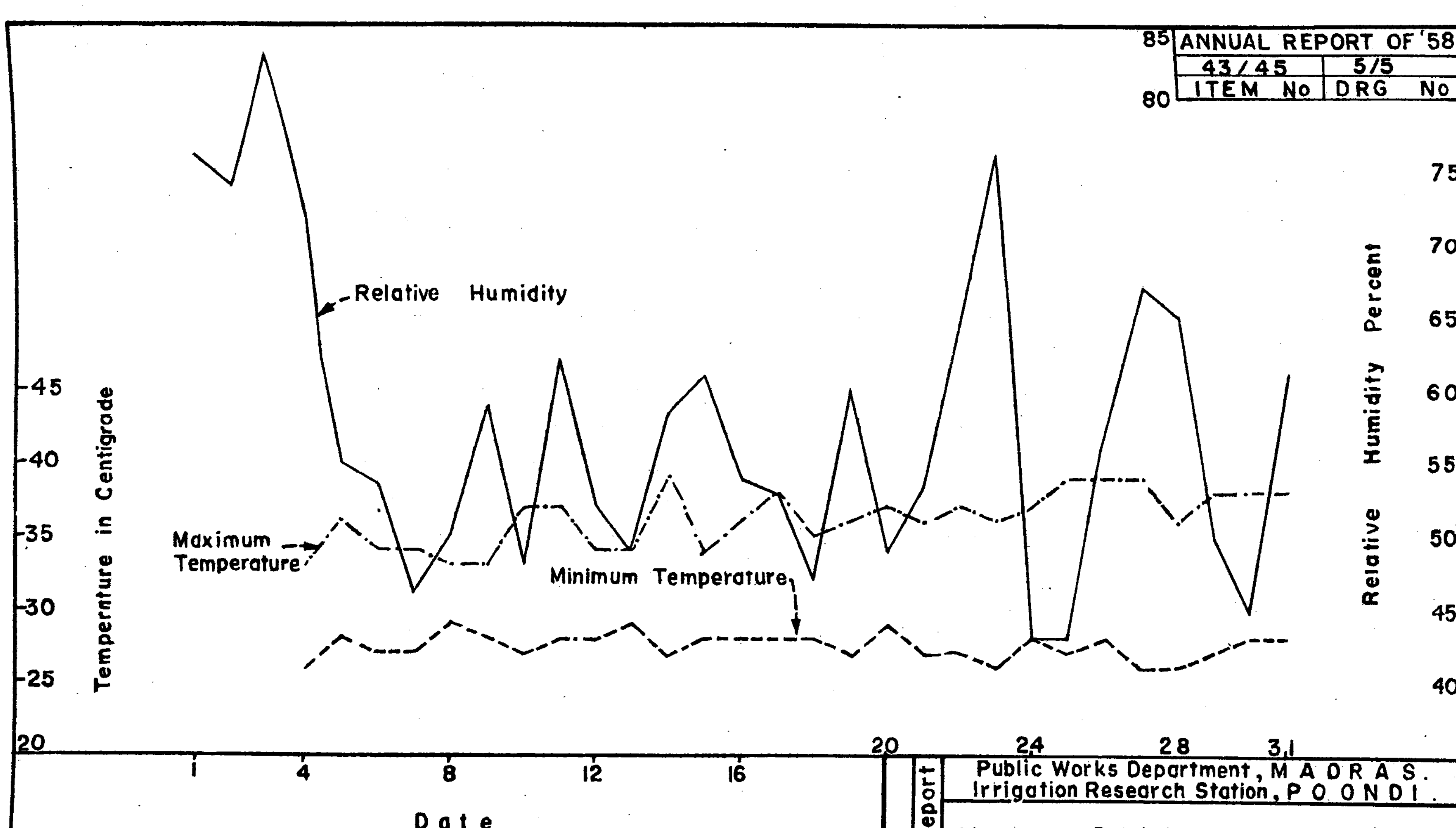

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	Maximum & Minimum Temperatures And Relative Humidity — July, 1958.		
Drawing 57/59	J · E · R	A · R · O	E · E · R
	Trd V·V	Ckd	Fig 5

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SUB-SOIL FLOW—EARTH DAMS.

(C.B. I. & P. No. E-1 (iii) (*a*))

Synopsis :

This report contains a detailed scheme of studies proposed to be conducted on seepage through earth dams and the previous foundations on which they are founded. The methods of analysis and solution of the problem on the two aspects of Phreatic line and quantity are indicated. Based on the principles of dimensional analysis, all the variables have been assembled into one " General Function ". In addition, this report also gives a detailed description of the Electrical analogy set up and a detailed design of the Hele-Shaw Channel. The constant head and variable head permeameters designed and constructed in this Laboratory for measuring the permeabilities of the different samples of soils used are also described in full detail.

1. Introduction :

1.0. Earth dams have been used for storing irrigation waters from times immemorial and certain earthen bunds in India and Ceylon are of prehistoric originand unknown antiquity. When water is stored by the construction of an earth bund, a certain amount of water flows through the pores of the soil, of which the bund is made and is thus lost. This flow of water through the earth bund is called seepage flow.

The quantity of seepage depends upon many factors. They include the type of material used in the construction of the bund and the method of construction adopted, the type of the foundation available, the type of cut off provided and the drainage measures provided, etc.

The free surface of flow through the bund or the line of saturation is called " The Phreatic Line ". If the soil is very fine, there may be a negligibly small amount of flow through the " Capillary Fringe " region also. The seepage phenomenon has got the following important points of significance :—

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

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

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

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

(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 and thereby may be rendered unfit for cultivation purposes.

To compute the quantity of seepage, at present there are a few empirical formulae based upon many assumptions and approximations. A number of experimental techniques of a precise nature are being used now-a-days in this study. If these studies could be conducted on a rather wide range and thorough basis, a reasonably sound theory can be proposed for this phenomenon of seepage. Furthermore, rational formulae and simplified charts could be evolved. With this objective in view, the studies to be described in the following pages are being conducted in this Station in collaboration with the Physics and Soil Mechanics Laboratory, Chepauk. This scheme of study is sponsored by the Central Board of Irrigation and Power under the scheme of " Basic and Fundamental Research."

2.0. *Programme of work.*—It is proposed to study the section on Earth dam under four divisions as below :—

(1) Homogeneous sections, (2) Zoned sections, (3) Composite Rockfill dams and (4) Hydraulic-Fill dams. Each section will be studied assuming the foundation at different depths.

(1) *Fixing the Phreatic line.*—This will be done in (1) A Hydraulic model and (2) An Electrolytic tank model.

The seepage line obtained above will be checked and compared with that obtained by the analytical methods.

(1) Relaxation method.

(2) Casagrande's method.

(3) Principle of the " Complex Potential ".

(2) *Computing the quantity of seepage flow.*—This will be obtained for each section studied by the following different methods :—

(1) Hydraulic models. Sandy models. Models having the prototype material.

(2) Electrolytic tank method.

(3) Hele-Shaw Viscous channel.

(4) Casagrande's method.

(5) Dupuit's method.

(6) Dr. Dachler's method.

(7) Pavlovsky's method.

(8) Computation from flow net. (Prof. Forcheimar's method.)

3. *Dimensional Analysis.*—The various factors affecting the seepage are :

(1) Permeability of the material of the dam k (the horizontal permeability is assumed to be the same as vertical permeability),

(2) the top width of the dam b,

(3) the bottom width V,

(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 permeability of the foundation material k_2,

(10) the factor to represent foundation stratification,

(11) depth of cut-off provided D,

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

(13) the specific weight of fluid (water),

(14) the dynamic viscosity of water,

(15) the surface tension of water,

(16) the acceleration due to gravity, and

(17) the velocity of flow.

Writing down the different quantities in the form of a function :

$$f\{ q, k, b, H, h, m_1, m_2, d, k_2\, \alpha, D, \beta, \gamma, \mu, \sigma, \eta, \nu \} = 0$$

combining these 17 physical quantities by the principle of dimensional analysis,

$$q = \phi \left\{ F, R, W, K_1, K_2, \frac{b}{H}, \frac{B}{H}, \frac{h}{H}, m_1, m_2, \frac{d}{H}, k_2, \alpha, \frac{D}{H}, \frac{\beta}{H} \right\}$$

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 is to be studied by putting a number of sand models.

From the data obtained, it is proposed to evolve a rational expression for the quantity of seepage. Secondly, it is proposed to develop an equation for the phreatic line. Thirdly, attempts will also be made to evolve simplified charts to fix the seepage line and to compute the seepage quantity.

The same scheme of studies will be repeated for Rockfill dams with such suitable modifications as may be found necessary.

3. Hydraulic Models :

Description of the model :

The first section chosen for study was that of a zoned section of a dam constructed on a (sandy) foundation of 20 feet depth.

Details of the zoned section :

Impermeable rock level	El. 0.00
Top of sandy foundation	El. 20.00
M.W.L.	El. 112.00
Top of dam	El. 120.00

Cut-off trench :

Bottom width of the cut-off trench	10 sq. ft.
Side slopes of the cut-off trench	1 on 1.
Top width of cut-off trench	55 feet.
Width of the cut-off wall	5 feet.
Height of the cut-off wall	10 feet.

Details of the core-wall :

Top width	10 feet.
Elevation of the top	112.0
Side slopes on both sides	1 on $\frac{1}{2}$
Bottom width of core-wall	102 feet.
Material at the core	Silty clay.
Clay—70% fine sand	30%

Details of the shell :

Top width	20 feet.
Upstream slope	1 on 3.
Downstream slope	1 on 1 upto El. 90.
	1 on 2.5" El. 60.
	1 on 2.5" El. 20.
With 2 numbers of berms	15 feet wide each.
Free board for the dam	8 feet.

Details of the Rock Toe :

Bottom width	10 feet.
Side slope upto G.L.	1 on 1.
Top of rock toe	El. 435.0.

Material of the toe is properly graded to satisfy the requirements of a film.

Description of the model :

The model was constructed to a scale of 1/60. This scale was arbitrarily fixed. Anyway, it was assumed that the capillary rise and the discharge through the capillary fringe will be negligibly small.

The model was constructed inside a glass flume shown in Figure 1 in the main laboratory. The impermeable foundation was simulated by a horizontal bed plastered with rough cement mortar. The cut-off wall was constructed in cement mortar. The trench was filled up with silty clay and rammed to the specifications to be given later under the item on the construction of the core.

The core wall and the shell portion were raised simultaneously. The material for the core wall was silty clay having 70 per cent of clay and 30 per cent of fine sand. The material for the shell was silty loam having 70 per cent of fine sand and 30 per cent of clay. A moisture content of 10 percent was taken. For consolidation, every layer was spread to a depth of 3" and rammed by means of wooden rammers to an uniform depth of 2".

Fixing the phreatic line :

For obtaining the phreatic line, 23 piezometers were embedded in the body of the dam at the required spacings as given in Figure 3. Each piezometer is made up of a 1/4" diameter vertical copper tube of suitable height with a horizontal copper tube of 1/8" diameter at the bottom. These L-shaped piezometers are embedded in the body of the dam and are connected to the manometer board by means of suitable rubber tubings. The manometer board is fixed so as to give the phreatic line directly.

Water-supply to the model :

A separate pipe line was laid to ensure constant uninterrupted water-supply to the model. The dam was kept loaded with water tight through for so many days. The water level on the spstream was maintained constant at the M.W.L. by means of a rectangular weir constructed on the upstream side.

Measurement of seepage :

To measure the quantity of seepage through the earth dam, a sump of 18" × 18" × 9" was constructed on the downstream side and the seepage water was led into it by a suitable leading channel. A gauge plank set up has been erected to enable discharge measurements.

To find the permeability of the shell and core materials, a constant head and variable head permeameters have been designed and fabricated in the local workshop. The permeability of the shell and core materials have been found out. A detailed description of these instruments and the statements of the readings are given elsewhere in the report.

The difficulty experienced with the model is that the phreatic line has not been established itself. The water column in piezometers is raising very slowly and it may take a few more weeks to get the final phreatic line.

Hydraulic model—For a homogeneous section :

A homogeneous section on an impermeable foundation is chosen for the studies. The details of the section are shown in Figure 4.

The model is to be constructed with 14-18 sand. The permeability of the 14-18 sand has been found out by means of the constant head permeameter. The statement of the readings is given at the end. The model is under construction inside a masonry flume. It is proposed to investigate the function,

$$q=\phi\left\{F, H, W, k_1, k_2, \frac{b}{H}, \frac{B}{H}, \frac{h}{H}, m_1, m_2, \frac{d}{H}, k_2, \alpha, \frac{D}{H}, \frac{\beta}{H}\right\} \text{in this model.}$$

The scale of the model is fixed as 1/20 so as to give a three feet height of the model. This may reduce the capillary rise and the resulting distortion of the phreatic line to a minimum. This fact has to be established by putting a number of models to different scales till a constant pattern of the phreatic line is obtained.

Methods proposed for fixing the phreatic line :

1. By the manometric set up as before.
2. By an electrical method.

Description of the electrical method : After the flow is established, glass tubes 1/4" in diameter will be inserted at different points along the centre line and a graduated pointer will be inserted. This pointer will be connected to an electric circuit with a calling bell as shown in Figure 5. When the pointer touches the seepage water inside, the circuit is completed and the bell rings. By this method, the seepage line can be fixed accurately. A low voltage battery of 6 volts is proposed to be used for this circuit.

3. *Chemical Methods* :

The principle that a solution of silver nitrate gives a curdy white precipitate with a solution of lead nitrate, will be made use of, for fixing the phreatic line in the sandy model.

4. *Electrical Analogy Studies* :

Two dimensional electrical analogy studies are usually conducted to obtain the equi-potentials and hence the flow net for the different sections tested.

Firstly, a homogeneous section shown in Figure 6 was taken up for study. The model was constructed in teak-wood to a scale of 1/100. The phreatic line has to be fixed approximately in the electrical model. This was accomplished using Casagrande's method. This phreatic line was simulated in the model. Further studies could not be taken up as the amplifier set-up was in repair and had been sent out for rectification.

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.[1] In figure, the phreatic line is to be fixed for all the sections in hydraulic models. Photos show the electrical analogy model and the set up available in this laboratory.

5. **Heleshaw Channel.**[2]

This apparatus takes its name from Prof. Heleshaw of Liverpool, England, who more than fifty years ago, built a similar apparatus to demonstrate to his students, stream line flow patterns around boundaries of different shapes. If two plates are spaced only a small distance apart, fluid flow between them is laminar and hence forms a two dimensional potential field. Since flow is a porous media also represents a potential flow field in accordance with Darcy's law that the velocity is proportional to the slope, the channel can be employed as a model analogy of flow in porous media.

The model has the advantage of enabling accurate measurements of porous-media flow to be made without encountering the difficulties of a porous media. The free surface is clearly visible : a variable zone of partial saturation above the free surface is replaced by a calculable capillary zone ; variable porosities and permeabilities are avoided and velocities can be measured directly by dyes or volumetric measurement. Also the free surface for steady flows can be scaled directly without resorting to piezometers and motion pictures can be used to record unsteady flows. A suitable channel has been designed for the zoned section described under " Hydraulic models " and the computations are given below.

Design for the flow of water—Principle of design :

For the earth dam considered as the prototype,

$v_p = k.i.$ (Darcy's law)

where v_p is the discharge velocity,

k is the transmission constant for the material of the dam and

i is the hydraulic gradient.

For flow between two parallel plates, spaced very closely,

$$v_m = i \frac{g}{z} \frac{B^2}{12}$$

where v_m is the velocity in the channel,

i is the pressure gradient,

g is the acceleration due to gravity,

z is the kinomatic viscosity of the fluid, and

B is the spacing between the fluids.

Since Reynolds law is the criterion for these model studies, the velocity ratio $vr = 1/\angle r$.

$$\therefore \frac{i}{\angle r} = \frac{vm}{vp} = \frac{i. \frac{G}{z}\frac{B^2}{12}}{k.\ i.}$$

For given values of k and z,

B can be calculated.

Computations for water :

$$Vr = \frac{1}{Zr} = 60 \text{ (for the model proposed).}$$

Maximum value of vP is controlled by R = 1 (according to K. B. Khuslani[3])

$$1 = \frac{V_P}{z} d.$$

$$= \frac{V_P}{.826 \times 10^5} \times \frac{.07}{2.54}$$

When $\frac{0.07}{2.54}$ represents the size of the material (40-60 size) chosen.

Vp = 0.0003 ft./sec.
m = 60 × .0003
= 0.018 ft./sec.
= 0.02 ft./sec.

Area of flow = 1/16 × 1/12 × 2
= 1/96 sq.ft.

using a nominal spacing of 1/16"

Q = a x v
= .02 × 1/96 = 0/.08 gpm.

Head to be pumped against = 2.0′ + 3′ 0″ + 5′ 0″ hf.
= 10′ 0″ neglecting the friction loss of hf. ft.

$$\text{H.P. required} = \frac{.124}{550}$$

= a very small quantity.

So, a fractional H.P. Motor of 0.5 H.P. can be used with a pump to circulate the water. If the suitable pump is not available, an air compressor available in the workshop can be made use of.

Capacity of over-head tank (2 hours supply)
= .08 × 60 × 2
= 10 gallons
= 1.6 c.ft.

A small tank of 24″ diameter and 24″ height can be used.
Capacity of the sump at the bottom is fixed as 40 gallons.
Length of the pipe line required—50 feet.
Suitable gauges have to be provided for the measurement of water levels.

Similar computations have also been made for crude oil of $z = 796 \times 10^{-6}$ sq.ft./sec.

The construction of the channel is to be taken up soon and the finalised design is shown in Figure 8.

6. Mathematical Method :

Relaxation method[4] :

The relaxation method developed by South-well is being used to obtain the flow net for the different sections analysed. Computations for the homogeneous section have been taken up and may take some more time for completion.

Complex potential :

Using the principles of " Theory of Functions " the solution of the stream line function ψ and the potential function ϕ are being attempted and if ψ can be solved for the phreatic line, then that will give the seepage quantity per foot width of the dam.

Using the principles of " Conformed Transformation ", mathematical solutions for the seepage line and seepage quantity are being attempted.

7. Permeameters :

Variable head permeameter :

This is used for finding out the permeability of soils of very low permeability like silty clay or clay.

Derivation of the formula involved :

Let 1 be the thickness of the sample,
A be the area of cross section of the sample,
a be the area of cross section of the stand pipe.

If he is the head over the sample at any instant, and if dh is the drop in head in an interval of time of ' dt '

$$-a.dh = A.v.dt.$$

$$= k \frac{h}{l} dh. A.$$

$$v = k \frac{h}{l}$$

where v is Darcy's discharge velocity and k is the permeability of the material.

Simplifying

$$dt = -\frac{a}{A} \frac{1}{k} \frac{dh}{h}$$

Integrating

$$T = -\frac{a}{A} \frac{1}{k} \log_e \frac{h_2}{h_1}$$

$$= \frac{a}{A} \frac{1}{k} \log_e \frac{h_1}{h_2}$$

where h_1 and h_2 are the heads at the beginning and end respectively of the time interval of T

Rewriting the above equation

$$k = \frac{a}{A} \times \frac{1}{T} \times 2.3 \log_{10} \frac{h_1}{h_2}$$

Description of the apparatus :

Referring to Figure 9, it consists of a steel cylinder 6 inches internal diameter and 8 inches length. It is surmounted by a cone at the top. A stand pipe is fixed at the top of the cone. It is provided with a stop cock. The wooden scale which 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 165 inches should be added to every reading.

At the bottom, the water is collected in a circular tank 12 inches in diameter and 4 inches in height. It is also provided with an overflow pipe to maintain a constant water level steel plates with 1/8" diameter holes, closely spaced are provided at the top and bottom of the sample, instead of the conventional porous plates.

Method of experimentation :

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 the intervals of half an hour for 24 hours.

The permeability is computed by using the formula

$$k = \frac{a}{A} \times \frac{1}{T} \times 2.3 \log_{10} \frac{h_1}{h_2}$$

The average of the several calculated values is taken as the permeability value for the material of the dam.

In the same way, the permeability of the sample in the vertical direction also can be found out with certain modifications.

A sample sheet of computation is given in the statements attached.

Constant head permeameter :

This is used for finding the permeability of sandy soils. It consists of a steel cylinder 6 inches internal diameter and 8 inches length. Two copper tubes, each 1/4" diameter are provided at a central spacing of 6 inches. At the top and bottom, suitable covering lids are provided with porous plates covered with fine mesh. Suitable manometric set up has been constructed to find out the head lost between the two tappings.

Method of experimentation :

The sample whose permeability is to be found out is taken in the cylinder. It is filled with water and saturated. The connections are made and water is allowed to flow up through the model. After 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 found out from the manometer.

Now $v = \frac{kh}{l}$ (Darcy's law) ϕ

where v is Darcy's discharge velocity,

k is the permeability of the sample,

h is the difference in head between the two manometric tappings

and l is the distance between the two manometric tappings.

$$v = k\frac{h}{l}$$

$$v = \frac{\text{volume collected}}{\text{time taken} \times \text{area of the sample.}}$$

$$= \frac{V}{t \times A}$$

$$k\,\frac{h}{l} = \frac{V}{t \times A}$$

$$k = \frac{V}{t \times A} \quad \frac{l}{h}$$

Permeability experiments have been conducted for 24-40 and 14-18 sands. 14-18 size sand is being used for the model of the homogeneous section. The computations and the results are given in the statements attached.

8. Conclusion :

At this stage, it is really impossible to give any conclusions from the results obtained. But still the following points are brought out :—

(1) For hydraulic models, there is no necessity to use the same material as that used in the prototype structure. Because, if it is used in the model is takes so many months for the phreatic line to be established. So, usually sand or granular slag can be used in hydraulic models.

(2) It is really laborious and also not very accurate to fix the phreatic line by means of the electrolytic tank method.

(3) In the matter of choosing scales for the earth dam models, care must be taken to see that the capillary rise is small. Secondly, the material of the model should not be so very coarse as to cause turbulent flow in the pores.

(4) The validity of Darcy's law for a gradient of 3 to 5 has been checked up and found to apply as well.

(5) A criterion for transferring the seepage quantities in the model to the prototype values is to be established.

References :

1. " Electrical Analogy "—Central Board of Irrigation and Power Publication.

2. Flow in Porous Media, studied by Hele Shaw Channel by David K Todd, Civil Engineering, February 1955.

3. " Irrigation Engineering ", by K. B. Kuslani, Vol. III,

STATEMENT No. 1.

PERMEABILITY COMPUTATIONS FOR THE SHELL MATERIAL—SILTY LOAM.

Apparatus used : Variable head permeameter zoned section—Hyd. Model 1.

Formula used : $k = zx \frac{a}{A} \times \frac{1}{T} \times 23 \log_{10} \frac{h_1}{h_2}$

No. (1)	h_1 *inches.* (2)	h_2 *inches.* (3)	*T hours.* (4)	*k in inch/hour* (5)
1	44.125	40.875	0.50	.00240
2	44.125	36.00	1.50	.00187
3	44.125	34.060	2.00	.00179
4	44.125	32.375	2.50	.00179
5	44.125	29.875	3.00	.00179
6	44.125	29.500	3.50	.00160
7	44.125	29.437	4.00	.00141
8	44.125	27.062	4.50	.00150
9	44.125	24.000	6.00	.00141
10	44.125	22.875	6.50	.00139
11	44.125	22.312	7.00	.00135
12	44.125	21.625	7.50	.00131
13	44.125	20.935	8.00	.00130
14	44.125	20.187	8.50	.00128
15	44.125	19.437	9.00	.00126
16	44.00	41.312	0.5	.00178
17	44.00	38.938	1.0	.00172
18	44.00	36.875	1.5	.00162
19	44.00	35.062	2.0	.00157
20	44.00	33.500	2.5	.00151
21	44.00	32.062	3.0	.00147
22	44.00	29.687	3.5	.00155
23	44.00	29.500	4.0	.00139
24	44.00	27.312	4.5	.00146
25	44.00	26.687	5.00	.00139
26	44.00	24.562	6.5	.00124
27	44.00	23.625	7.0	.00123
28	44.00	22.875	7.5	.00121
29	44.00	22.125	8.0	.00119
30	44.00	21.325	8.5	.00116

Average value of k = 0.00149 inch/hour.

STATEMENT No. 2.

PERMEABILITY COMPUTATION FOR THE CORE MATERIAL (i.e., SILTY CLAY).

Apparatus used : Variable head permeameter.

Formula used : $k = \frac{a}{A} \times \frac{1}{T} \times 2.3 \times \log_{10} \frac{h_1}{h_2}$

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	33.625	32.875	6.00	.000666
3	33.625	32.813	6.50	.000504
4	33.625	32.781	7.00	.000467
5	33.625	32.688	7.50	.000437
6	33.625	32.688	8.00	.000410
7	33.625	32.625	8.50	.000386
8	33.625	32.563	9.00	.000365
9	41.500	40.688	5.00	.000706
10	41.500	40.625	6.00	.000657
11	41.500	40.625	6.50	.000606
12	41.500	40.500	7.80	.000466
13	41.500	40.433	7.50	.000436
14	41.500	40.375	8.00	.000409
15	41.500	40.313	8.50	.000386
16	37.5	36.313	8.00	.000411
17	37.5	36.281	8.50	.000388
18	37.5	36.188	9.00	.000367

Average value of k = 0.000494 inch/hour.

STATEMENT No. 3.

Permeability Computations for 24—40 Size Sand.

Apparatus used: Constant head permeameter.

Formula used: $k = \frac{V}{t \times A} \times \frac{1}{h}$

No. (1)	Head lost h feet (2)	Vol. of water V in c.c. (3)	T in secs. (4)	k in ft./hour (5)
1	.100	420	60	5.94
2	.105	395	60	5.66
3	.095	380	60	5.65
4	.085	350	60	5.82
5	.080	350	60	6.17
6	.080	350	60	6.17
7	.080	346	60	6.10
8	.080	345	60	5.82
9	.080	330	60	5.82
10	.080	330	60	5.82
11	.080	330	60	5.82
12	.080	330	60	5.82
13	.080	330	60	5.82

Permeability of 24—40 size sand is taken as 5.82 ft./hour.

STATEMENT No. 4

Seepage Observation for the Zoned Section

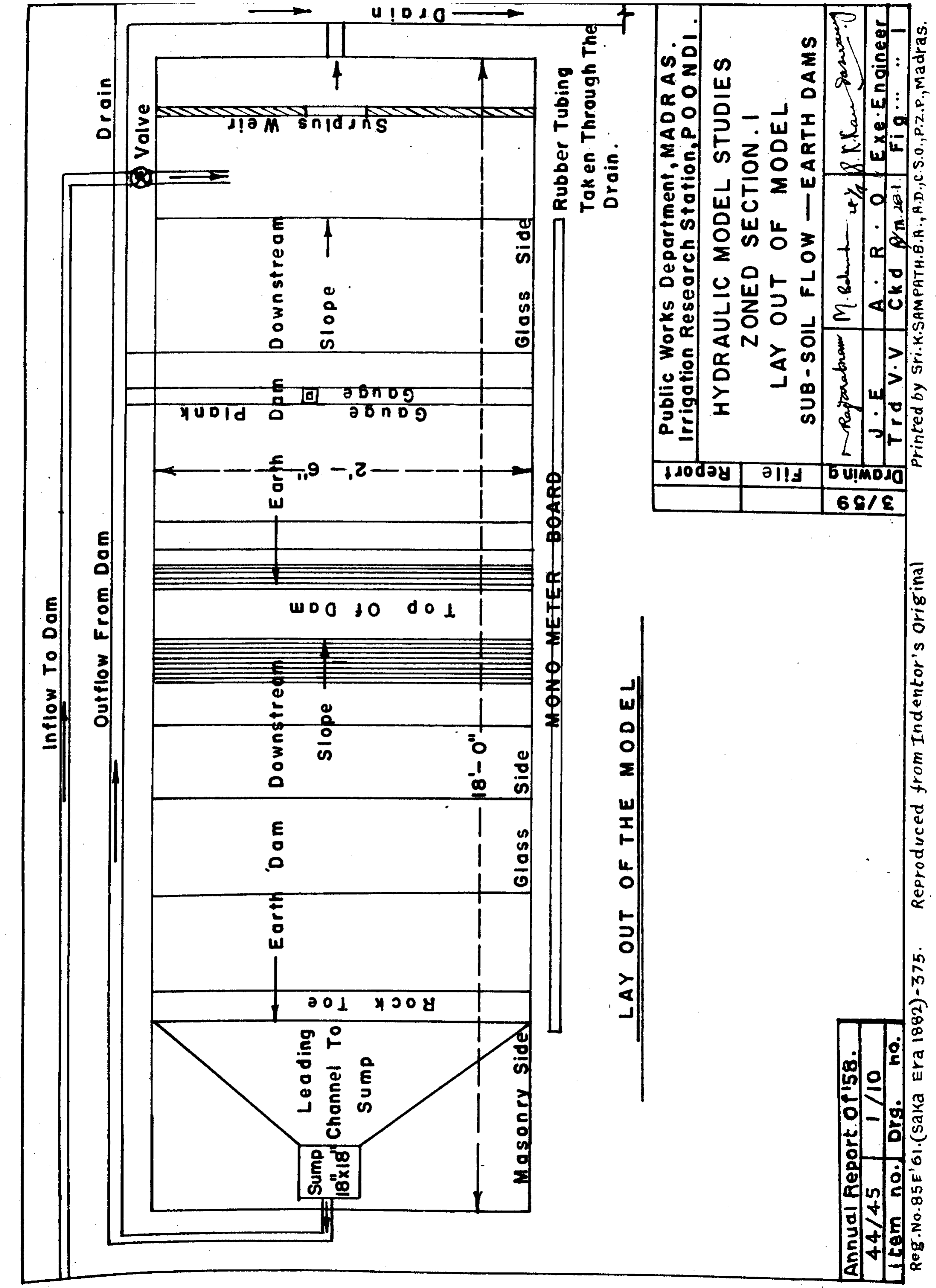
Date and No. (1)	Time. (2)	Water level in sump (3)	Difference in feet. (4)	Qty. of Seepage C.ft. (5)	Rate of Seepage in C.ft./hour. (6)	Seepage in C.ft./hour/foot length. (7)
16–1–1959—						
1	11-00 a.m.	92.0	...	...	...	...
2	11-30 a.m.	109.8	.178	.400	.800	0.320
3	12-00 noon	127.0	.179	.403	.808	0.322
4	2-15 p.m.	80.7	...	...	...	...
5	2-30 p.m.	90.2	.095	.214	.856	0.342
6	2-45 p.m.	99.7	.095	.214	.856	0.342
7	3-00 p.m.	109.0	.093	.209	.836	0.334
8	3-15 p.m.	118.6	.096	.216	.864	0.336
9	3-30 p.m.	127.0	.084	.189	.756	0.302
10	3-40 p.m.	82.7	...	...	...	...
11	3-55 p.m.	92.0	.093	.209	.836	0.334
12	4-10 p.m.	100.8	.083	.198	.792	0.317
13	4-25 p.m.	110.9	.101	.226	.904	0.362
17–1–1959—						
14	7-45 a.m.	79.3	...	...	...	...
15	8-00 a.m.	92.3	.100	.225	.900	0.360
16	8-15 a.m.	99.3	.100	.225	.900	0.360
17	8-30 a.m.	109.3	.100	.225	.900	0.360
18	8-55 a.m.	101.8	...	...	...	...
19	9-10 a.m.	111.8	.100	.225	.900	0.360
20	9-25 a.m.	121.8	.100	.225	.900	0.360
21	9-40 a.m.	131.8	.100	.225	.900	0.360
22	9-55 a.m.	141.8	.100	.225	.900	0.360
23	10-00 a.m.	99.3	...	...	...	...

STATEMENT No. 4—(*contd.*)

Seepage Observation for the Zoned Section—(*contd.*)

Date and No. (1)	*Time* (2)	*Water level in sump.* (3)	*Difference in feet.* (4)	*Qty. of Seepage C.ft.* (5)	*Rate of Seepage in C.ft./hour.* (6)	*Seepage in C.ft./hour/foot length.* (7)
17-1-1959—(*contd.*)						
24	10-15 a.m.	109.6	.103	.232	.928	0.371
25	10-30 a.m.	119.6	.100	.225	.900	0.360
26	10-45 a.m.	139.6	.100	.225	.900	0.360
27	11-00 a.m.	139.6	.100	.225	.900	0.360
28	11-15 a.m.	102.6	...	...	...	...
29	11-30 a.m.	112.9	.103	.232	.928	0.371
30	11-45 a.m.	122.9	.100	.225	.900	0.360
31	12-00 noon	133.8	.109	.246	.964	.386
32	12-15 p.m.	143.9	.101	.228	.912	.365
33	2-00 p.m.	100.0	...	...	...	...
34	2-15 p.m.	110.8	.108	.243	.962	.385
35	2-30 p.m.	84.3	...	...	...	...
36	2-45 p.m.	94.3	.100	.225	.900	.360

Average value for the seepage loss in C.ft. per hour per foot length of the Dam = 0.352.



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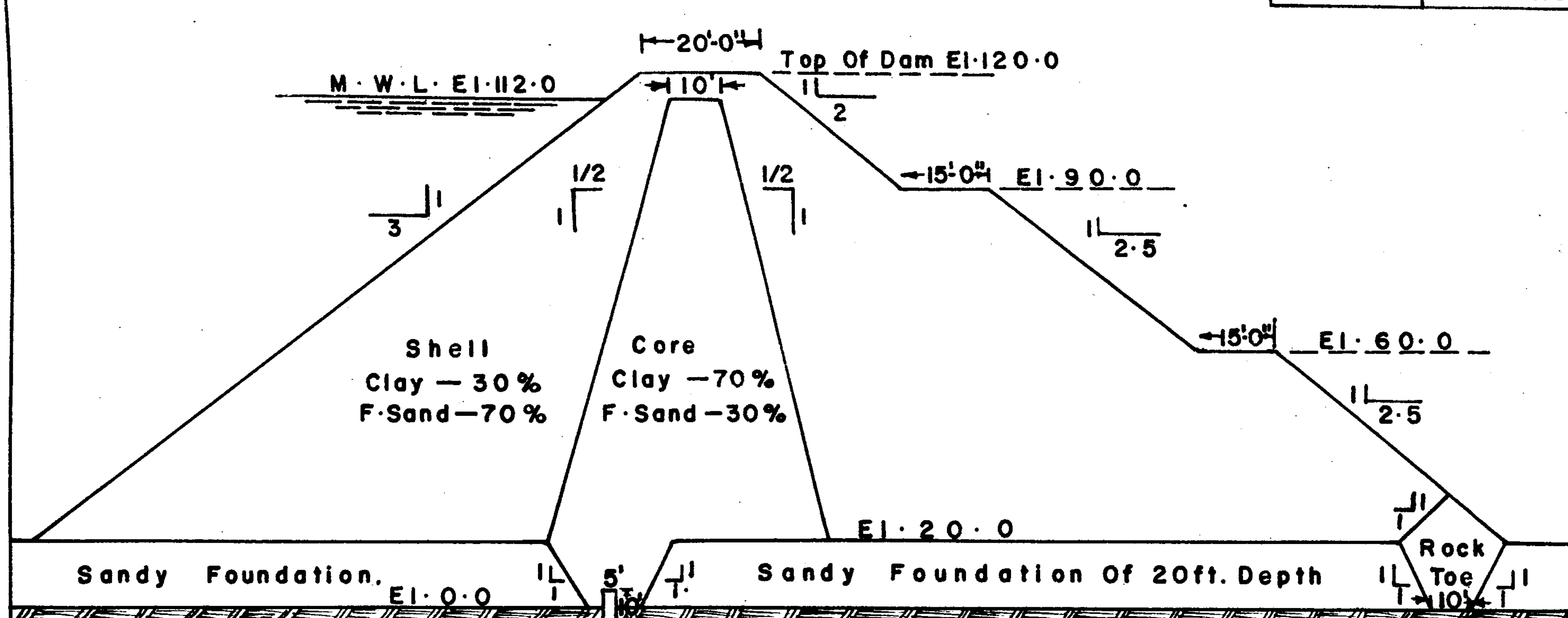
ITEM No	DRG No



DETAILS OF SECTION OF THE ZONED DAM
(Not To Scale)

Report / File / Drawing 4/59	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
	ZONED SECTION OF DAM HYDRAULIC MODELS-I		
	SUB-SOIL FLOW—EARTH DAMS		
	J·E	A·R·O	Exe·Engineer
	Trd V·V	Ckd	Fig·· 2

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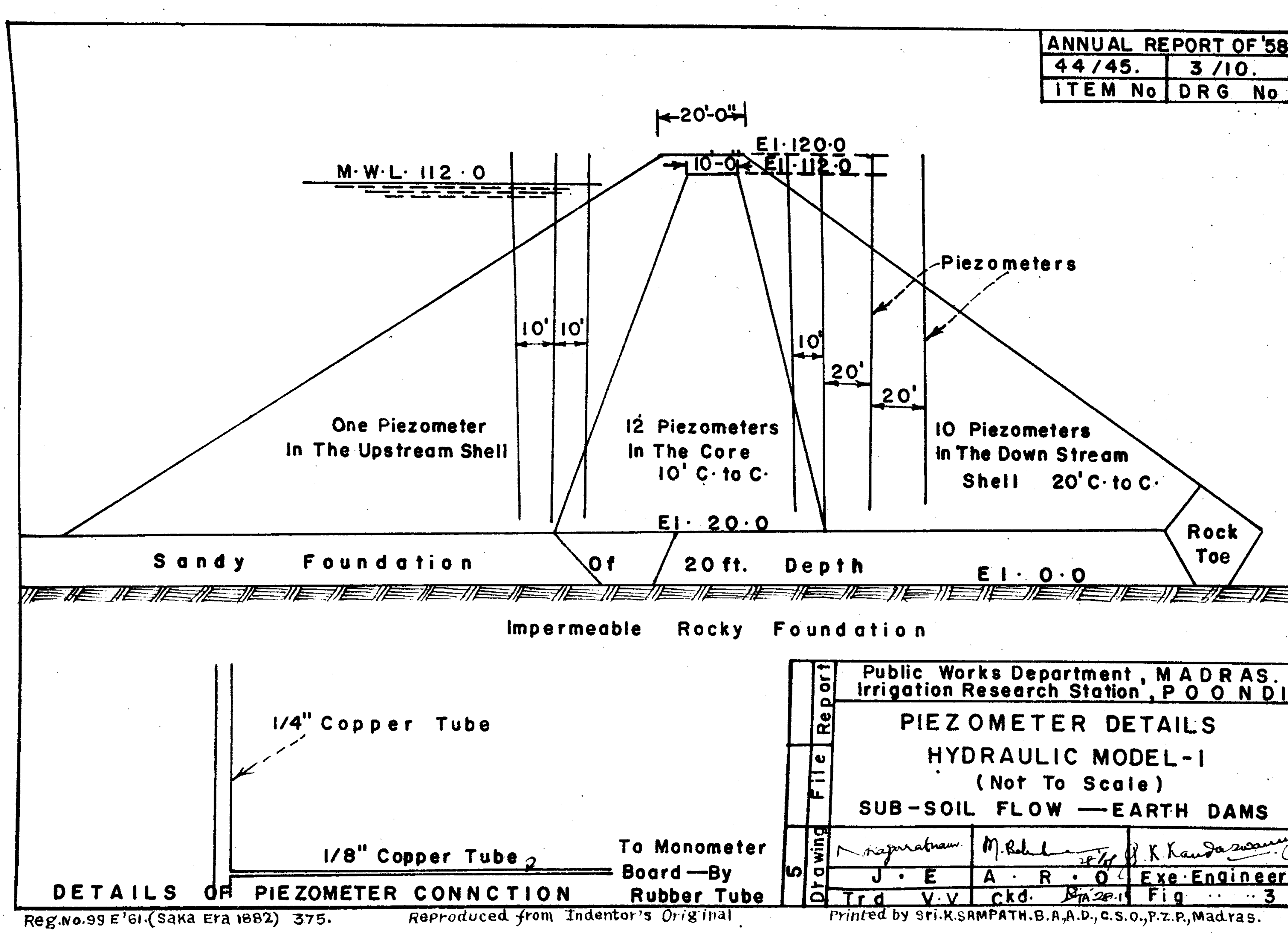

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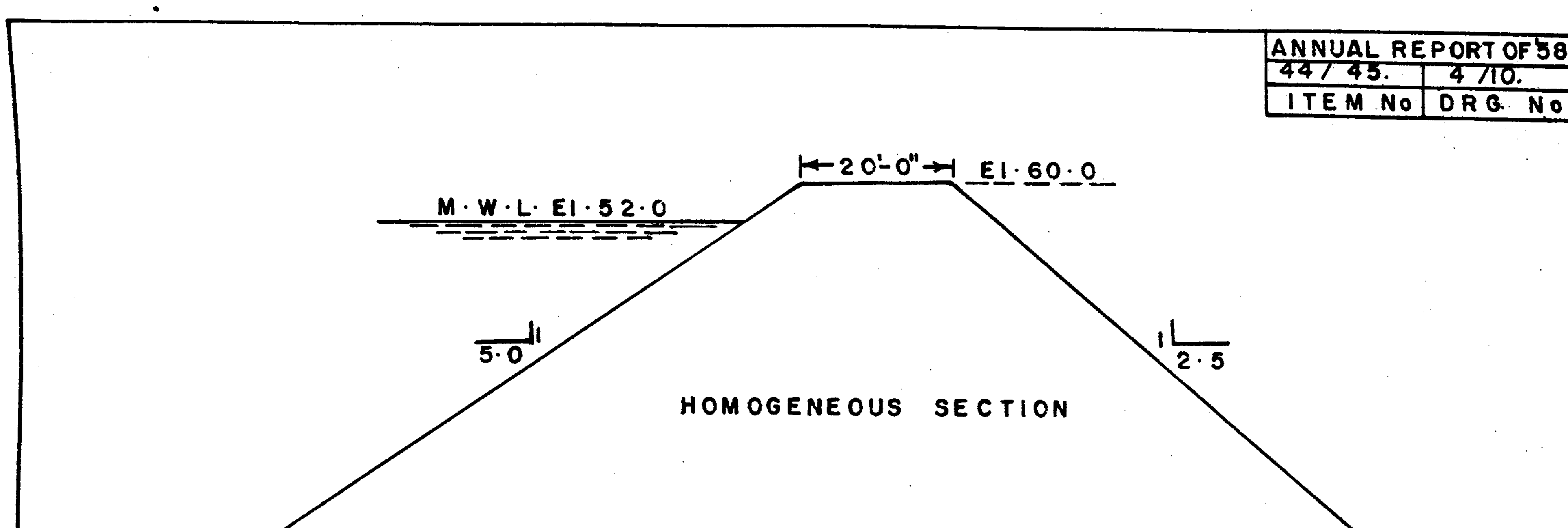

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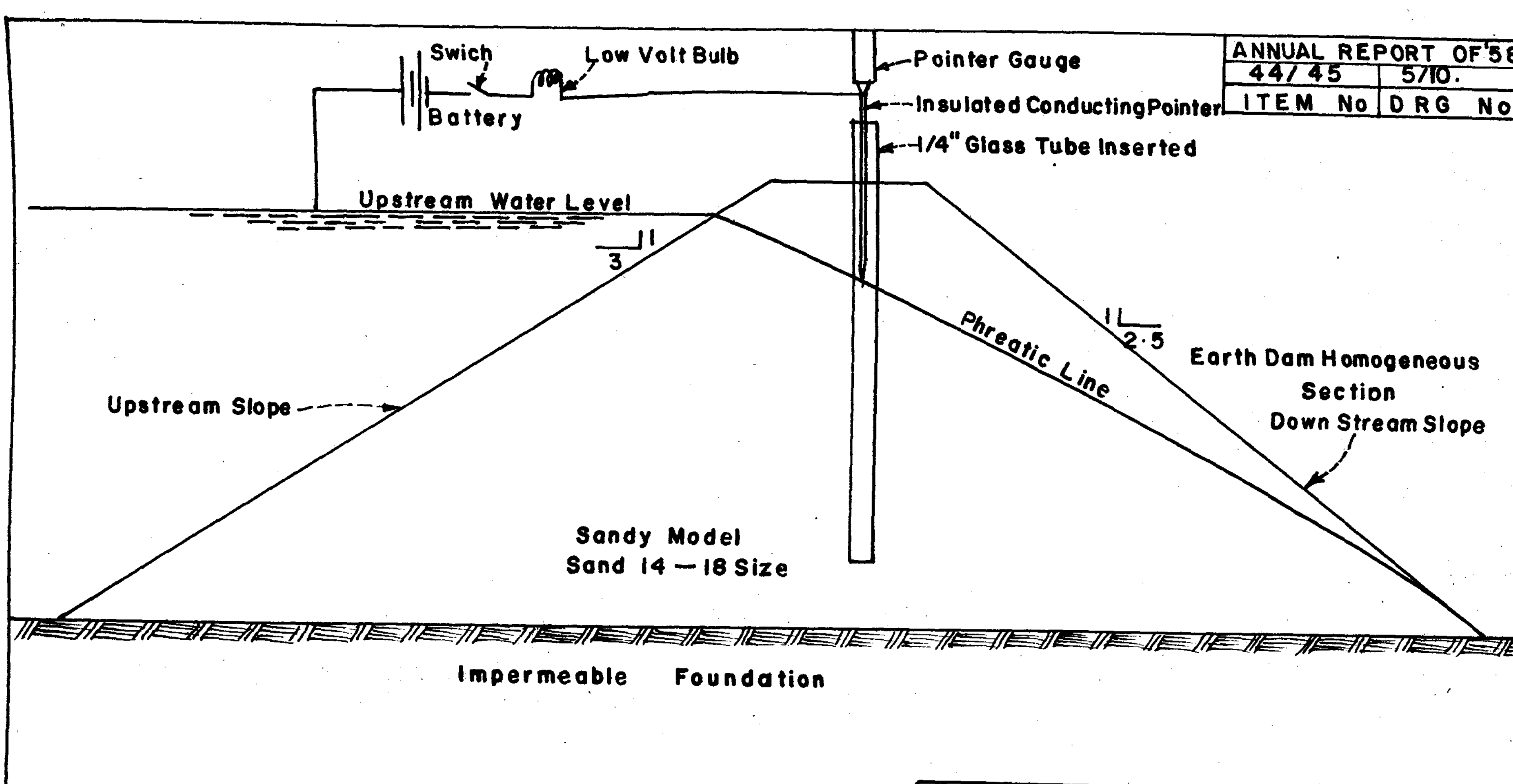

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44/45	5/10.	
ITEM No	DRG No	

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
	EARTH DAM — HYDRAULIC MODELS — II		
File	A SIMPLE METHOD OF FIXING THE PHREATIC LINE		
	SUB-SOIL FLOW — EARTH DAMS		
Drawing 7	N. Raganathan	M. ... 28/7/58	R. K. Ram...
	J. E	A. R. O	Exe · Engineer
	Trd V·V	Ckd ...	Fig 5

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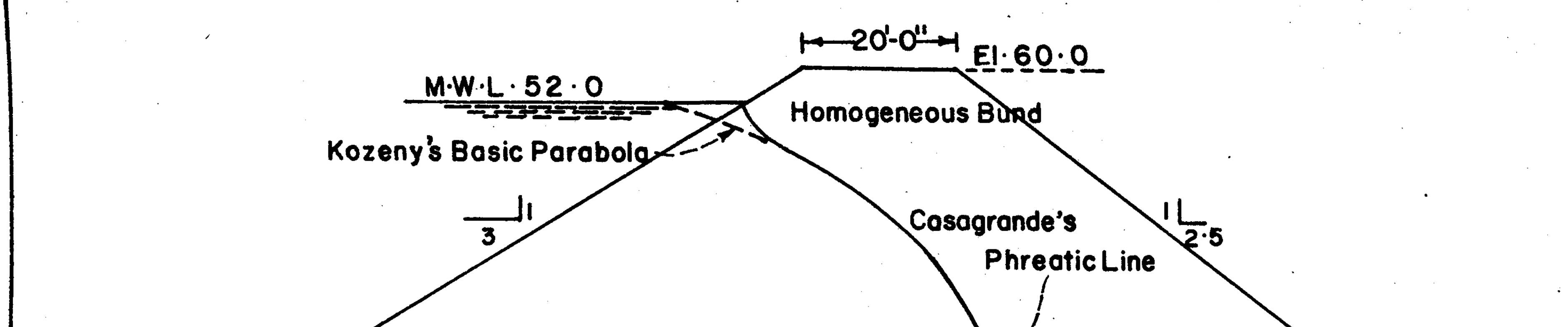
44/45.	6/10.
Item No	Drg No



Computations For Phreatic Line (Casagrande's Method)

X	Y
−3	10·6
−7	0
10	21·8
20	27·4
30	32·1
40	36·2
50	39·9
60	43·2
70	46·4
80	49·4
91	52·4

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

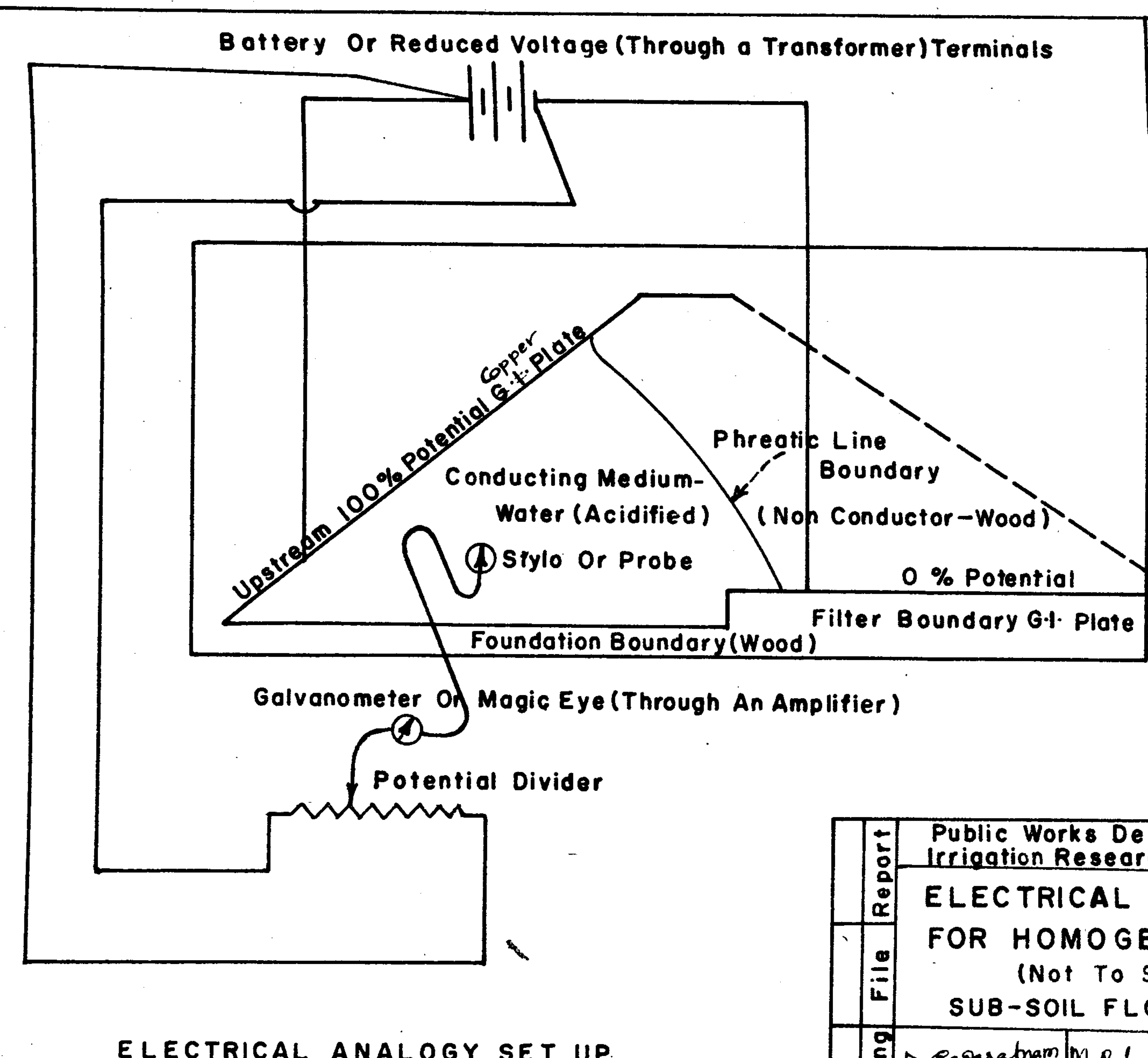
HOMOGENEOUS BUND FOR ELECTRICAL ANALOGY TESTS-I
(Not To Scale)

SUB-SOIL FLOW — EARTH DAMS

Report	File	Drawing 8
N. Rajaratnam	M. Balasubramaniam	G. K. Kandaswamy
J·E	A·R·O	Exe·Engineer
Trd V·V	Ckd	Fig 6

Reg. No. 102 E'61. (Saka Era 1882) 375.   Repoduced from Indentor's Original   Printed by Sri. K. SAMPATH. B.A., A.D., C.S.O., P.Z.P., Madras.

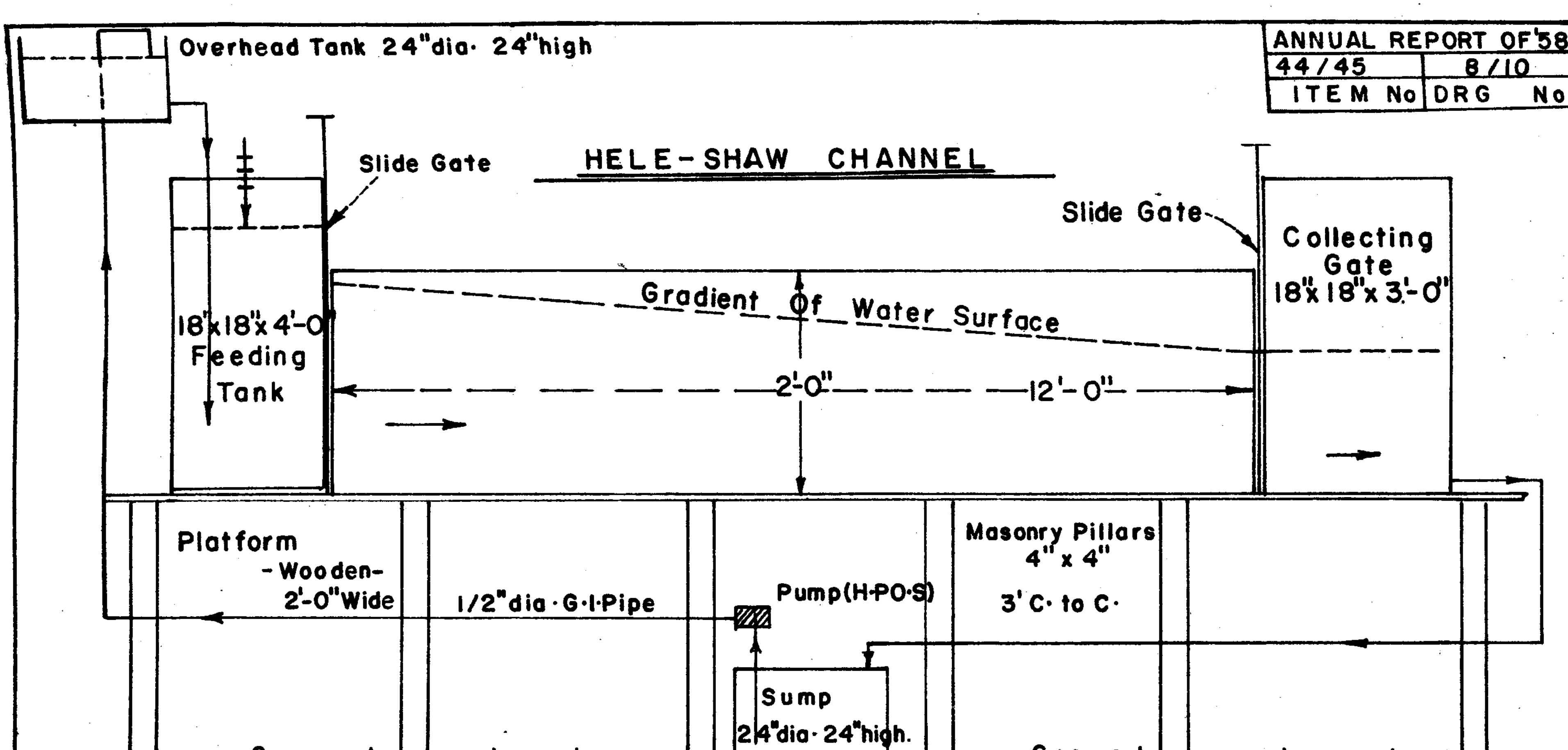
ANNUAL REPORT OF '58	
44/45	7/10.
ITEM No	DRG No



ELECTRICAL ANALOGY SET UP

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	ELECTRICAL ANALOGY SET UP FOR HOMOGENEOUS SECTION (Not To Scale) SUB-SOIL FLOW — EARTH DAMS		
Drawing 9	N Rajaratnam	M. Bahl...	B.K. Kandaswamy
	J · E	A · R · O	Exe · Engineer
	Trd V · V	Ckd	Fig 7

Reg. No. 103. E'61. (Saka Era 1882) 375.   Reproduced from Indentor's Original   Printed by Sri. K. SAMPATH, B.A., A.D., C.S.O., P.Z.P., Madras.



ANNUAL REPORT OF '58	
44/45	8/10
ITEM No	DRG No

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.
File	DESIGN OF HELE-SHAW CHANNEL (Not To Scale) SUB-SOIL FLOW — EARTH DAMS
Drawing 10	J·E · A·R·O · Exe·Engineer
	Trd V·V · Ckd · Fig ·· 8

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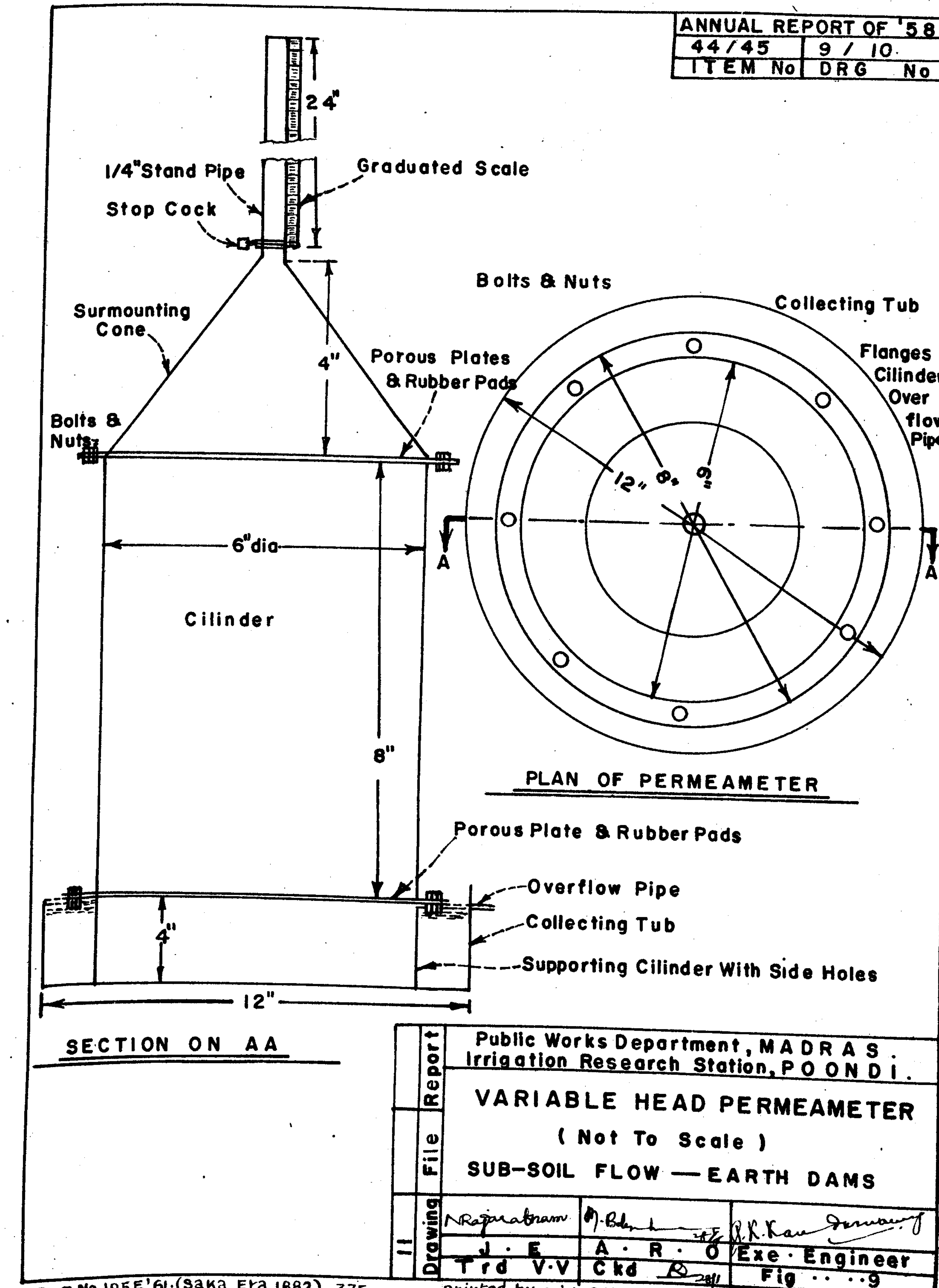

Reg. No. 105 E'61. (Saka Era 1882) 375. Printed by Sri. K. SAMPATH, B.A., A.D., C.S.O., P.Z.P., Madras.
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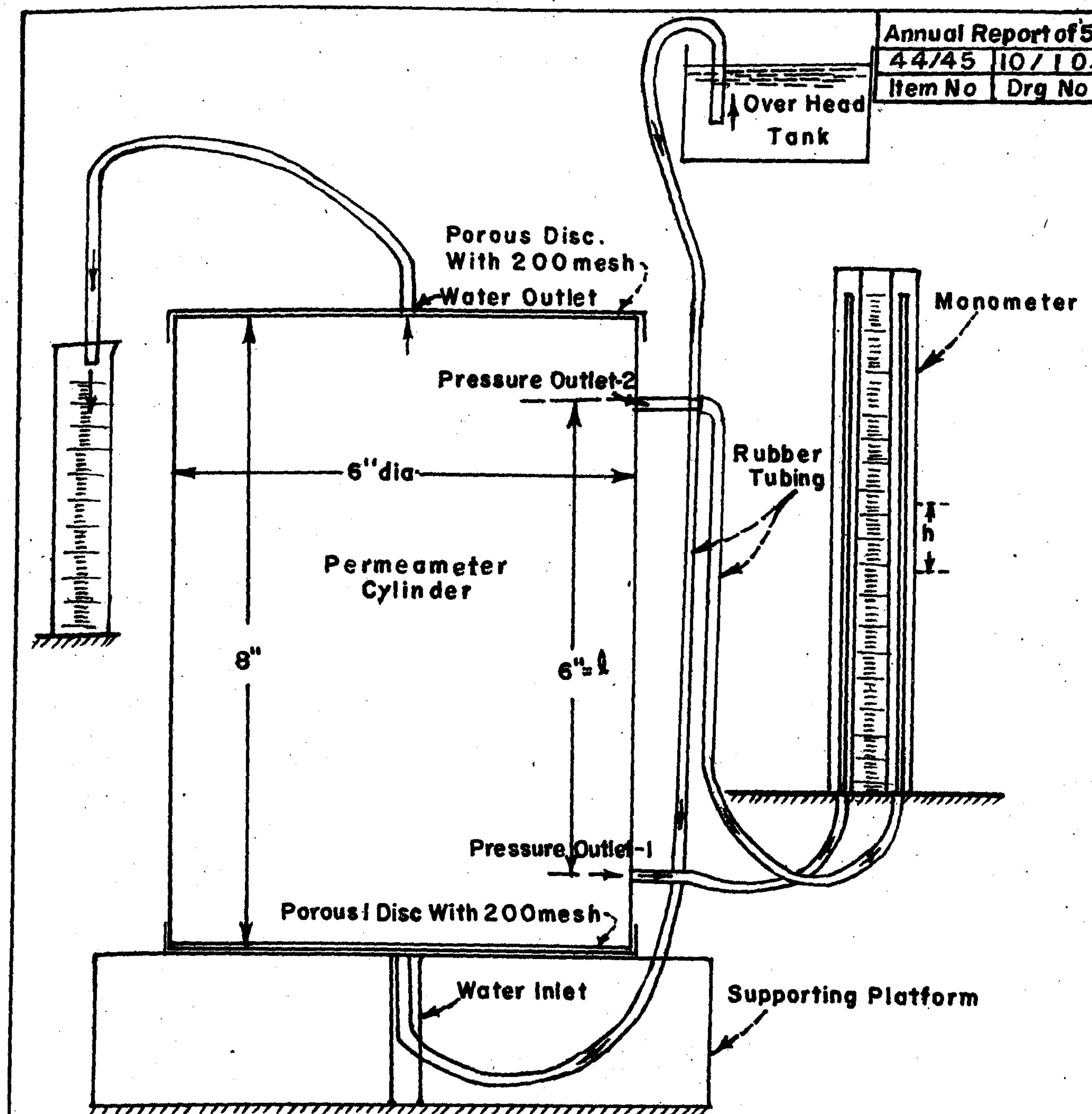

NOTE The Arrows Inside The Tubing And The Cylinder Indicate The Direction Of Flow Of Water.

12 Drawing File Report

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

CONSTANT HEAD PERMEAMETER
(Not To Scale)

SUB-SOIL FLOW—EARTH DAMS

J. E.	A. R. O	E. E. R
Trd V.V	Ckd	Fig. 10

Reg. No. 106-E'61.(Saka Era 1882)-375. Printed by Sri. K. SAMPATH, B.A., A.D., C.S.O., P.Z.P., Madras.

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LAND RECLAMATION IN PULLARAMBAKKAM.

(C.B.I. & P. No. F-4 (i) & (ii).)

Synopsis :

This report deals with the reclamation of about 300 acres of wet and dry lands of the Pullarambakkam village in Madras State. Water-logging and salinity of soils are explained. The causes of water-logging are analysed and suitable measures have been worked out for arresting further seepage into the lands. Shallow open drains have been designed to lower the water table to the required level. The operation of removal of salinity has been given in three stages. This report also presents an approximate estimate of the cost of the proposed remedial measures.

2. Introduction :

Magnificient, extensive and crop-insured irrigation systems, commendable water-supply systems, gigantic hydro-electric projects and large flood control reservoirs have been developed by the construction of multivarious types of hydraulic structures across the natural water courses. But not always are the good acts of man unaccompanied by evil effects at another end and the storage reservoirs, constructed for the permanently profitable uses of man-kind, at times, result in the submersion of fertile lands, prosperous villages, industrial towns, resourceful mines, rich reserve forests, thrilling game sanctuaries, antique archaeological sites and in the undue rise of the ground water table of the adjoining lands, thereby causing water-logging and salinity. One such case of water-logging and land reclamation has been referred to this Station.

3. Problem and its Location :

About 300 acres of wet lands and dry lands within the limits of the Pullarambakkam village in Madras State (Figure 1) lie on the rear side of the eastern bund of the Poondi Reservoir which was commissioned in 1944. Before the construction of this reservoir, the wet lands getting their supply from the Periya Isa tank were giving remarkable yield and the rainfed dry lands also were giving fairly good yield for one crop. After the Poondi Reservoir was filled up, due to the excessive seepage from the Reservoir, there was a continuous rise of the ground water table in the adjoin-ing lands. In a few years, the ground water table came up to the ground surface and as a result, the whole area became water-logged.

Followed by the movement of the ground water upwards, due to the residual head and also aided by the soil evaporation, the injurious salts of Sodium and Calcium which have been lying before at considerable depths, have come up to the root zone of the crops thereby affecting the yield of the crops. To state it briefly, the 300 acres of adjoining lands have become water-logged and saline. The problem was referred to this Research Station for suggesting remedial measures.

4. Water logging and Salinity of Soils

4.1. *Water-logging.*—(1) A soil is said to be water-logged when the soil pores within the root zone of the normal crops are saturated with water to cut off the normal supply of air from the atmosphere. This state of affairs depends not only upon the depth of the ground water table but also on the height of the capillary fringe which varies with the type and structure of the soil. If a land is to be free from water-logging, the depth of the top of the capillary fringe should not be less than about two feet below the ground surface. So, if the height of the capillary rise is, say, X feet depending upon the texture of the soil, then the ground water table should be at a depth of not less than (2 × 4) feet.

According to Dr. McKenzie Taylor, the factors responsible for the infertility of the water-logged soil are —

(1) Anaerobic conditions created,

(2) The difficulty of carrying out cultivation operations,

(3) The competition between the crop and the natural flora to exist and

(4) The possibility of concentration of the injurious salts in the root zone of the normal crops.

4.2. *Salinity.*—Saline soils are coils having excess soluble salts which make the ground water sufficiently concentrated to injure the plants and impair soil productivity. The term alkali soil is applied to soils which have an excess of exchangeable sodium either with or without excessive total soluble salts. Among all the salts, Sodium Carbonate is the most injurious one. It is also known as black alkali. It has been found out that even a trace of black alkali in the root zone is injurious to the plants. When the water table is rising, the salts move up along with the water and get deposited on the surface when the water is evaporated. During dry seasons, the ground water moves down leaving the salts in the root zones. When the concentration of the salts exceeds a maximum safe limit, the plants will wither away. Various temporary and permanent measures have been proposed for reclaiming saline lands.

5.0 Analysis of the Problem

Causes of water-logging of the Pullarambakkam lands.—From a careful analysis of the site conditions, it was clear that the water-logging is due to the excessive seepage from the Poondi Reservoir. The seepage from the Poondi Reservoir into the adjoining lands may be through the earthern bund and the foundation as well. But on an inspection of the site, it has been observed that the upstream face of the earthern bund in this particular reach is rivetted with concrete slabs thereby making it impervious. So, there cannot be any appreciable quantity of seepage through the bund. Further, from Figure 2, it can be seen that the bund is constructed on a natural clay blanket which extends to a depth of a few feet, varying from 4 to 8 feet, overlying a porous strata. It was found out that during the construction of the bund, borrow pits were dug inside the water-spread area near the bunds thereby removing the clay blanket, which, as a result, exposed the porous strata occurring to the deleterious action of seepage with the full head of the stored up water. So, the entire seepage flow should be occurring through the porous strata partially blocked up by the irregularity aligned cut-off walls.

6.0. Remedial Measures :

To arrest the seepage from any reservoir into the adjoining lands, the following methods are available :—

(1) Land-side berms,

(2) Relief walls,

(3) Sub-levees,

(4) Impervious blankets on the upstream side,

(5) Cut-off walls,

(6) Drainage blankets and

(7) Drainage trenches.

Amongst these seven methods, land-side berms and sub-levees cannot be used for reducing the seepage through the foundation. Impervious blankets, if they are to be effective in reducing the seepage through the foundation to a considerable extent should be laid to a minimum length of 10 H feet on the upstream side of the bund, where H is the maximum depth of water in the reservoir. Hence this may prove to be very costly.

Cut-off walls can successfully be used for controlling the seepage through the foundation if they are taken to the impervious strata underlying the porous foundation. In this particular case, this may be impossible from the point of view of constructional difficulties.

Drainage blankets or normal filters in earth dams will only reduce piping, but cannot reduce the quantity of seepage and hence cannot be used in the present case.

Drainage trenches dug on the downstream side of the bund to the required depth, can be used for intercepting the seepage flow and leading it to some tank downstream, when the depth of the bottom of the porous strata is considerable, shallow drainage trenches can be used in conjunction with suitably spaced relief wells.

In this particular case, the thickness of the porous sub-stratum of sand and sandy loam varies from 1′ 0″ to 3′ 0″ and lies at a relatively shallow depth of 4′ 0″ to 6′ 0″. Hence the excavation of a drainage trench to a depth of about 5′ 0″ to 8′ 0″ from the ground level may thus completely penetrate the porous strata and hence will intercept the seepage flow completely. If suitable fall is given to this drainage trench, the entire quantity of intercepted seepage can be safely led out into low-lying tank situated down below.

If it is thought that in certain locations, the 11′ 0″ depth is excessive from this point of view of stability of the side slopes or the excessive top width of the trench and also from the point of view of giving a uniform grade at this depth, then the trench can be dug to a depth of about 5′ 0″ and relief wells sunk to a depth of 11′ 0″ at suitable intervals to completely intercept the under seepage.

It is very important to see that sufficient bed fall is given to the drainage trench, so that the intercepted seepage flow is readily drained out and let into an outlet down below.

P—II—16 End.

6.1. *Location and design of the drainage trench.*—There is an existing drain in the rear toe of the bund closer to the road. This drain has become almost blocked up by stones and by the growth of weeds and has lost its bed slope and consequently, is remaining as a dead pool. Further, the access to the culvert provided in the main road to draw off this seepage water has been completely blocked.

To intercept the seepage flow from the dam, this existing trench itself can be remodified to house in the following modifications :—

(1) The bottom width should be equal to 6 feet with side slopes of 1 on 1 suitably revetted with stones.

(2) The silt and stones deposited in the original trench should be removed and the trench is to be excavated to a depth of 5 feet at the end away from the culvert. (Marked in Plan No. 5 enclosed.)

(3) A longitudinal fall of 1 in 1,000 may be given.

(4) Suitable relief wells of 6′ 0″ diameter completely penetrating the porous strata should be dug at intervals of about 200 feet, and the portion of the wall upto the bottom of the longitudinal trench should be filled up with graded metal. The section of the proposed trench is shown in Figure 3.

(5) Suitable approach conditions should be given to the existing drainage culvert.

(6) From the culvert, a leading channel should be excavated upto the Orangal Madagu, to lead away all the seepage water.

If, after the construction of this drain, it is seen that all the seepage flow is not intercepted, then another set of relief wells of the same dimension as in the previous case can be provided at intervals of 100 feet. This arrangement is believed to intercept all the seepage flow.

7. Land Reclamation :

After arresting further seepage from the reservoir, the water-logged and saline lands have to be reclaimed. This has to be done in two stages. Firstly, the existing ground water table should be lowered. Secondly, the salinity of the ground should be removed.

7.1. *Lowering ground water table.*—For lowering the ground water table which, in this case, is at the ground level, one of the following methods of solution may be chosen :—

(1) Open drains—

  (*a*) Shallow open drains.
  (*b*) Deep open drains.

(2) Covered under drains—

  (*a*) Relief system.
  (*b*) Interceptor systems.

7.2. *Open drains.*—In the present case, the water table has to be lowered only by about 2.0 to 3.0 feet or so. Further, if deep open drains are adopted, they may give lot of trouble by way of slipping of the sides which may require revetting ; it may lower the water table to a depth much lower than the root zone and further, if they continue to be in operation after the reclamation is over, they may trap out a considerable portion of the deep percolating part of the irrigation supply.

On the other hand, shallow open drains do not have the disadvantages mentioned for deep drains. This will lower the water table to the required depth. They can be used at a later stage even for conveying the irrigation supplies. Hence open drains can be used in preference to the deep drains.

The closed tile drains are used sometimes to trap the percolating water which is in movement towards the lower terrain and also for lowering the ground water table. No doubt they have got the advantage that their trapping capacity is much more than the open drains. But it requires skilled labour for laying them, keeping them in the correct position against any possible settlement and if they get choked up, their relaying will entail considerable expenditure. In view of the above difficulties, it is preferable to use the shallow open drains for lowering the ground water table.

Design of the shallow open drains.—The design criteria is that the water table should be at least 2.60 feet below the ground level. Choosing a drain of trapezoidal shape, bottom wide 4′ 0″, depth 4′ 0″ and side slopes 1 on 1 and on a longitudinal gradient of 1 in 500, the spacing can be calculated for keeping the water table 2.60 ft., lower than the ground surface.

The flow of ground water is given by Darcy's equation $v = k \times i$

where v is Darcy's discharge velocity.
k is the coefficient of permeability and
i is the hydraulic gradient.

Assuming that the depth of flow in the channel is 1.00 ft., and referring to Figure 4,

$$v = \frac{k \times h_f}{r}$$

where h_f is the fall of bed upto the drain and
$2R$ is the spacing of the drains.

Substituting for $k = 2 \times 10^{-3}$ ft./sec.,
$h_f = 0.40$ ft.
$$v = 2 \times 10^{-3} \times \frac{0.40}{R}$$

Flow in the drain from both sides for a length of one foot is given by the equation

$$q = (a \times v)^2$$

" a " being the mean area of flow,

$$q = (1.20 \times 1)\ 2 \times 10^{-3} \times \frac{0.40}{R} \times 2$$

$$= \frac{0.00192}{R} \text{ c.ft./sec.}$$

and if L is the length of the plot parallel to the drain, the total seepage =

$$qL = \frac{.065}{R} \times L \text{ c.ft./sec.}$$

To find out the channel capacity to pass this discharge

$$v = c \sqrt{mi}$$

where v is the velocity of flow,

c is Chezy's constant,

m is the hydraulic mean depth, and

i is the energy gradient of flow.

Assuming that c = 80,

$$v = 80 \sqrt{\frac{5.0}{6.83} \times .002}$$

$$\text{Discharge } Q = iv$$

$$= .5 \times .80 \sqrt{\frac{5.0}{6.83} \times .002}$$

$$= 0.0152 \text{ c.fs.}$$

Equating the discharge capacity of the drain to the seepage from the drain

$$\frac{.00192}{R} \times L = 0.0152$$

In this case L = 0.75 mile = 3,960 ft.

$$R = \frac{0.00192 \times 3960}{0.0152} = 500$$

The spacing of the drains is 1,000 ft. For efficient functioning of the drain and quick lowering of the ground water table, the spacing can be taken as one furlong, so that in this case where the width of the affected plot is about 6 furlongs, 7 drains can be provided. The final alignment of the drains is shown in Figure 5.

7.3. *Removing the salinity of the soil.*—The affected lands seem to be having a high concentration of the ' black alkali ' or Sodium Carbonate which has been mainly responsible for the withering away of the crops. Anyway before a final solution is given, a thorough chemical analysis of the soil should be made.

The operation of reclamation can be carried out in three stages :

(1) *Leaching the surface crust.*—Since a large quantity of water is available from the Periya Isae tank, the objectionable salts which are deposited in the tcp layers and on the ground surface can be leached out and led into the shallow open drains, which will lead the saline water to the Orangal Madagu. This operation can advantageously be carried out in the rainy season.

(2) *Converting the harmful black alkali into harmless salts.*—If gypsum or calcium sulphate is added to Sodium Carbonate, the following reaction takes place :—

$$Na_2Co_3 + CaSo_4 \rightarrow Na_2\,So_4 + CaCo_3$$

and the black alkali is converted into harmless salts.

This operation of adding $CaSo_4$ can be carried out along with the ground water lowering operation so that the precipitated salts can also be lowered and washed into the drains. The exact quantity of $CaSo_4$ required per acre should be found out by experiments.

(3) *Land managements with tolerant crops.*—A few types of grass and cereals are to a certain extent, tolerant to the harmful alkalies. They can be grown for a season or two till even the residual quantity of the salts can be effectively removed. Table I gives some information regarding the plants in U.S.A. tolerant to alkalies. But work on similar lines should be carried out to suit the Indian conditions. The Department of Agriculture may be contacted on this point.

8. Approximate Cost of the Reclamation Scheme :

(1) Earthwork involved in the improvement of the existing toe drain (One unit = 1,000 c.ft.) ...	180 units.
(2) Earthwork involved in the excavation of 7 shallow open drains	800 units.
Total earthwork involved = 980 or 1,000 units. Cost of the above work at Rs. 18 per unit	Rs. 18,000
Cost of graded metal for toe drain, excavation of small field drains, revetment for open drains, etc. ...	Rs. 3,000
Probable cost of $CaSo_4$ for leaching operations, purchasing of suitable grass, expenditure of leaching operations, etc.	Rs. 9,000
Total cost of the scheme ...	Rs. 30,000

9. Conclusion :

About 300 acres of wet and dry lands of Pullarambakkam village have become water-logged and saline due to the excessive seepage, from the Poondi Reservoir. The problem was analysed critically and it was concluded that the excessive seepage was through the pervious strata below the bund which has been exposed to the deleterious action of seepage during the construction of the bund to arrest further seepage into the affected lands, it is reoommended to modify the existing toe drain so as to have a regular drainage channel of 6 ft., bottom width with 1 on 1 revetted sides and an average depth of 5 ft., and a bed fall of 1/1,000 with 6 ft. diameter relief wells at 200 ft. centres.

To lower the ground water table by about 2.60 ft., on the average shallow open drains of 4 ft. bottom width and 4 ft. depth with 1 on 1 sides may be excavated, at intervals of one furlong with a longitudinal slope of 1/500, parallal to the Trivellore-Othukottai Road. To remove the salinity of the lands, firstly, the leaching operation may be carried out followed by the treatment of the land with gypsum. If these measures are followed by the cultivation of certain 'tolerant crops' for one or two seasons, the injurious salts may completely be removed and the reclamation will be complete. The approximate total cost of the scheme was found out to be Rs. 30,000.

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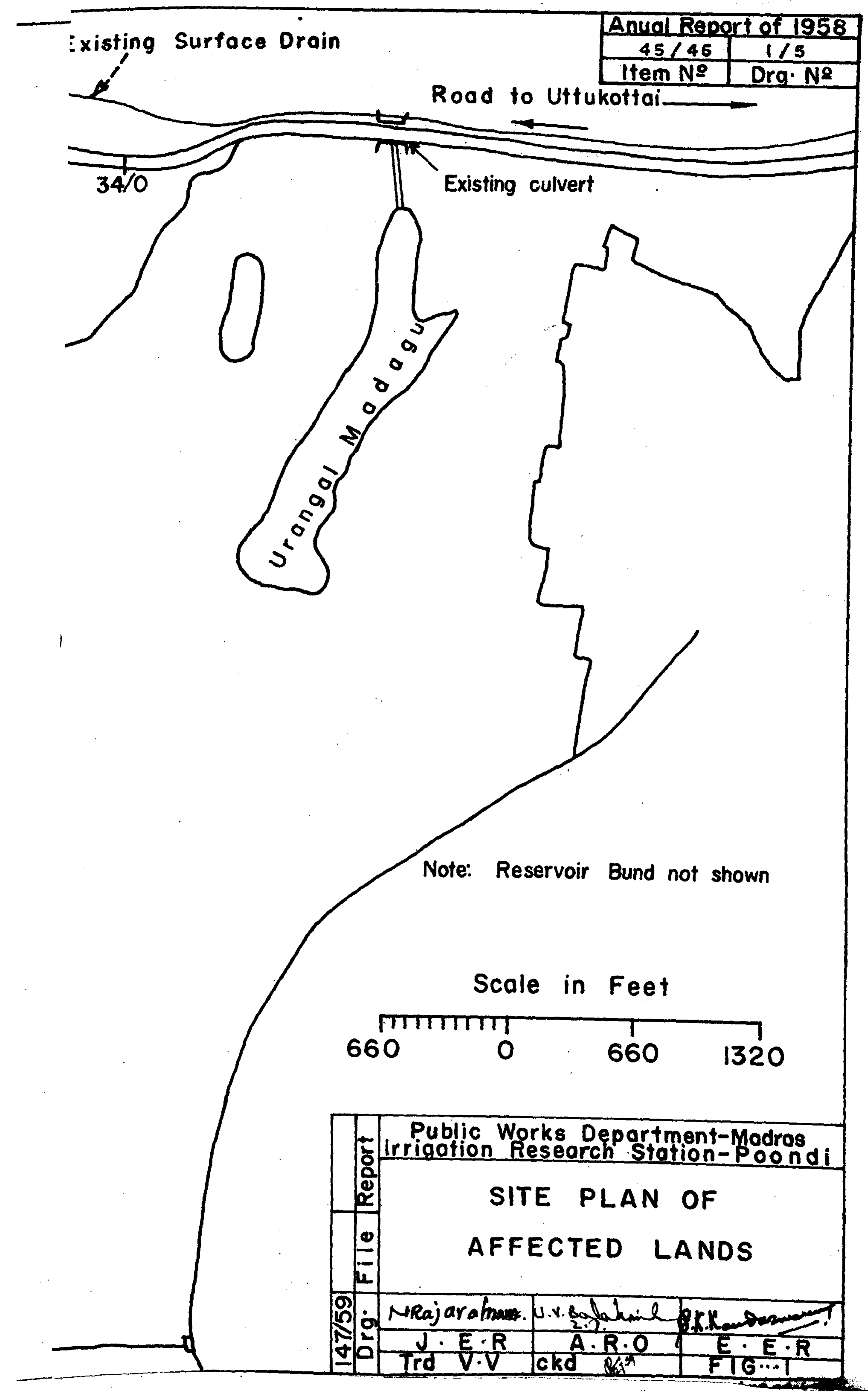
TABLE No. 1.

TOLERANCE OF THREE TYPES OF CROPS FOR SALINITY AS DETERMINED BY THE UNITED STATES REGIONAL SALINITY LABORATORY.

Under each of the three types of crops the most tolerant crops are listed first and the best tolerant last.

Type of crop.	*Salt tolerance.*			
	Good (Group I).	*Moderate (Group II).*		*Poor (Group III).*
Fruit.	Date Palm.	Pomegranite. Fig. Grape. Olive.		Crate fru it. Pear. Almond. Apricot. Peach. Plum. Apple. Orange. Lemon.
Field and truck.	Sugar beet. Garden beet. Nilo. Rape. Kale. Cotton. Barley.	Alfalfa. Flax. Tomato. Asparagus. Foxtail millet. Sorghum (grain). Rype (grain). Oats (grain). Rice.	Cantaloupe. Lettuce. Sunflower. Carrot. Spinach. Squash. Onion. Pepper. Wheat (grain).	Vetch. Peas. Colery. Cabbage. Artichoke. Egg plant. Sweet potato. Potato. Green beans.
Forage ...	Alkali sacton. Salt grasses. Nuttall Alkali. Bermuda. Rhodes. Rescue. Canada wild rye. Beardless wild rye. Western wheat.	While sweet clover. Yellow sweet clover. Perennial rye grass. Mountain-brome. Barley (hay). Birdsfoot trefoil. Strawberry cover. Dallis grass. Suedan grass. Hubam clover. Alfalfa (California common). Tail fescue. Rye (hay).	Wheat (hay). Oats (hay). Orchard grass. Blue grams. Meadow fescue. Reed canary. Big trefoil. Smooth brome. Tall meadow oat grass. Cicer milk vetch. Sour clover. Sickle milk vetch.	White Dutch clover. Meadow foxtail. Alsike clover. Red clover. Ledino clover. Burnet.

Solar Works, Madras-1.



Public Works Department-Madras
Irrigation Research Station-Poondi

SITE PLAN OF
AFFECTED LANDS

Report
File
Drg. 147/59

NRajaratnam.

J·E·R	A·R·O	E·E·R
Trd V·V	ckd	FIG·-I

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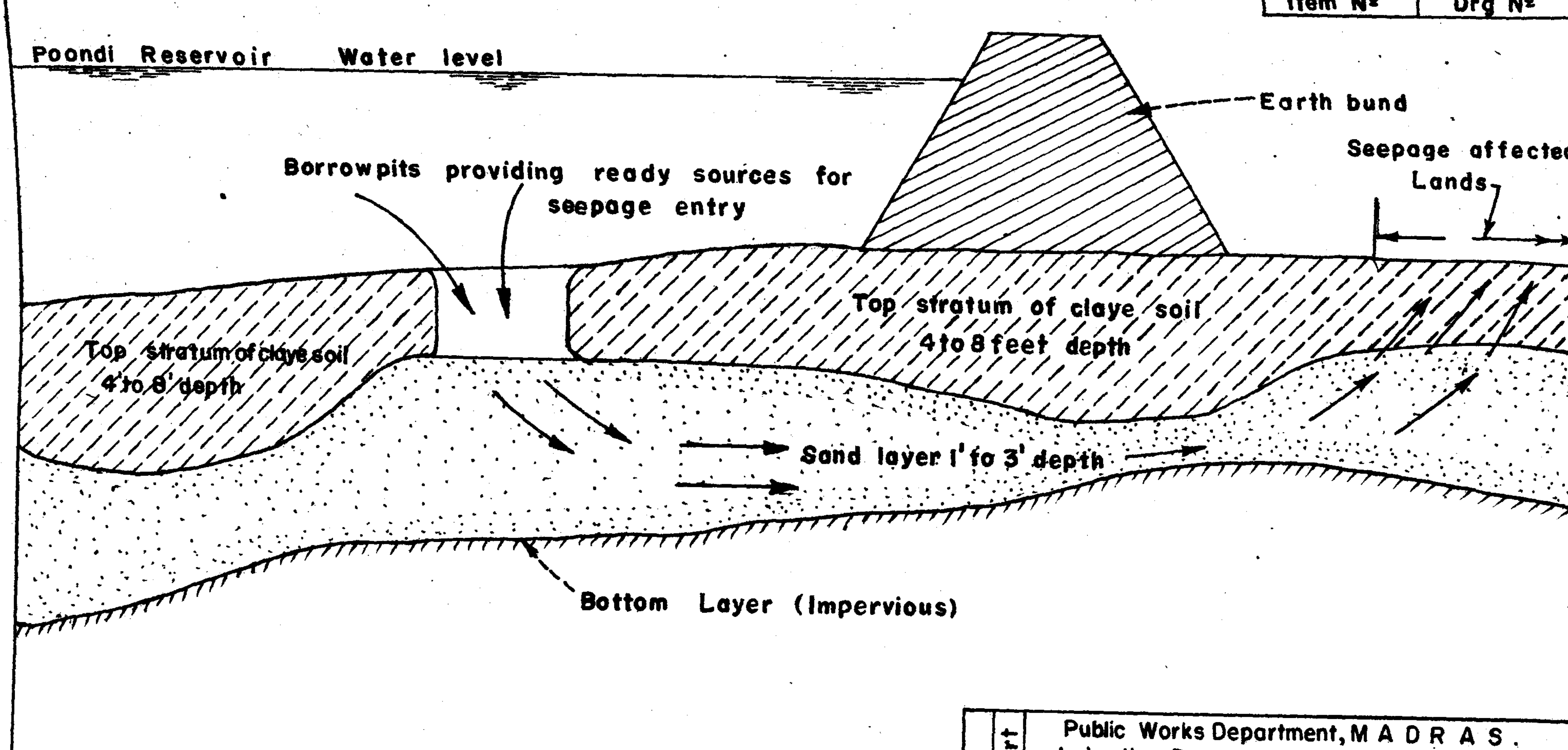

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	Land Reclamation in Pullarampakkam DETAILS OF STRATIFICATION BELOW THE BUND		
Drawing 148/59	Rajaratnam	V.V. 2.7.59.	
	J·E·R	A·R·O	E·E·R
	Trd. V·V·	Ckd.	Fig ... 2.

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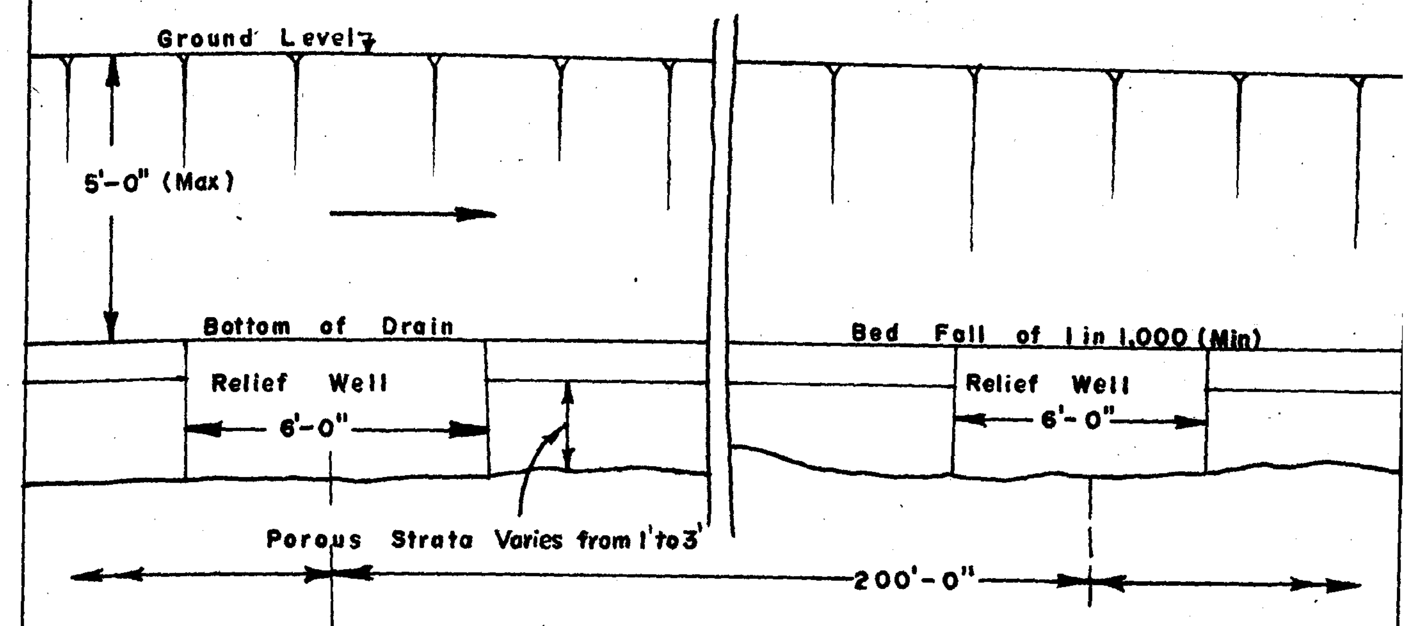
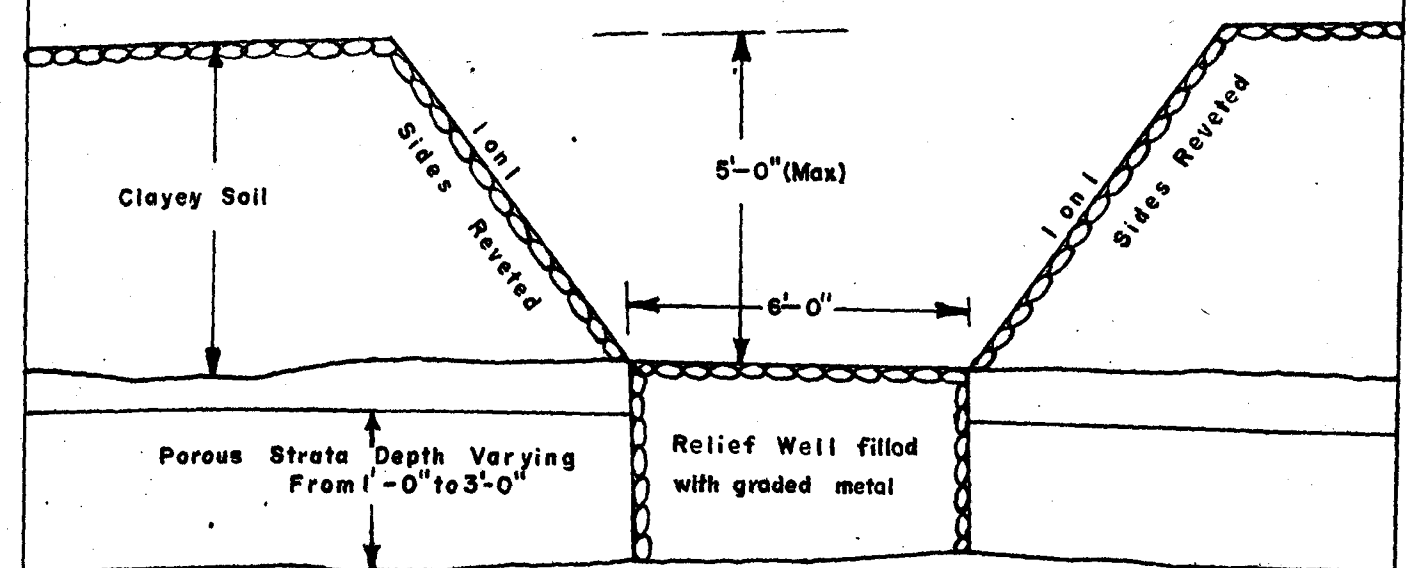


LONGITUDINAL SECTION OF THE TOE DRAIN



SECTION OF THE DRAIN AFTER IMPROVEMENT

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Report: Public Works Department — Madras
Irrigation Resear Station-Poondi

File: Land Reclamation in Pullarampakkam
DETAILS OF THE TOE DRAIN
(Not To Scale)

Drawing 149/59		
M. Rajarathnam.		
J·E·R	A·R·O	E·E·R
Trd: V·V.	ckd:	FIG ··· 3·

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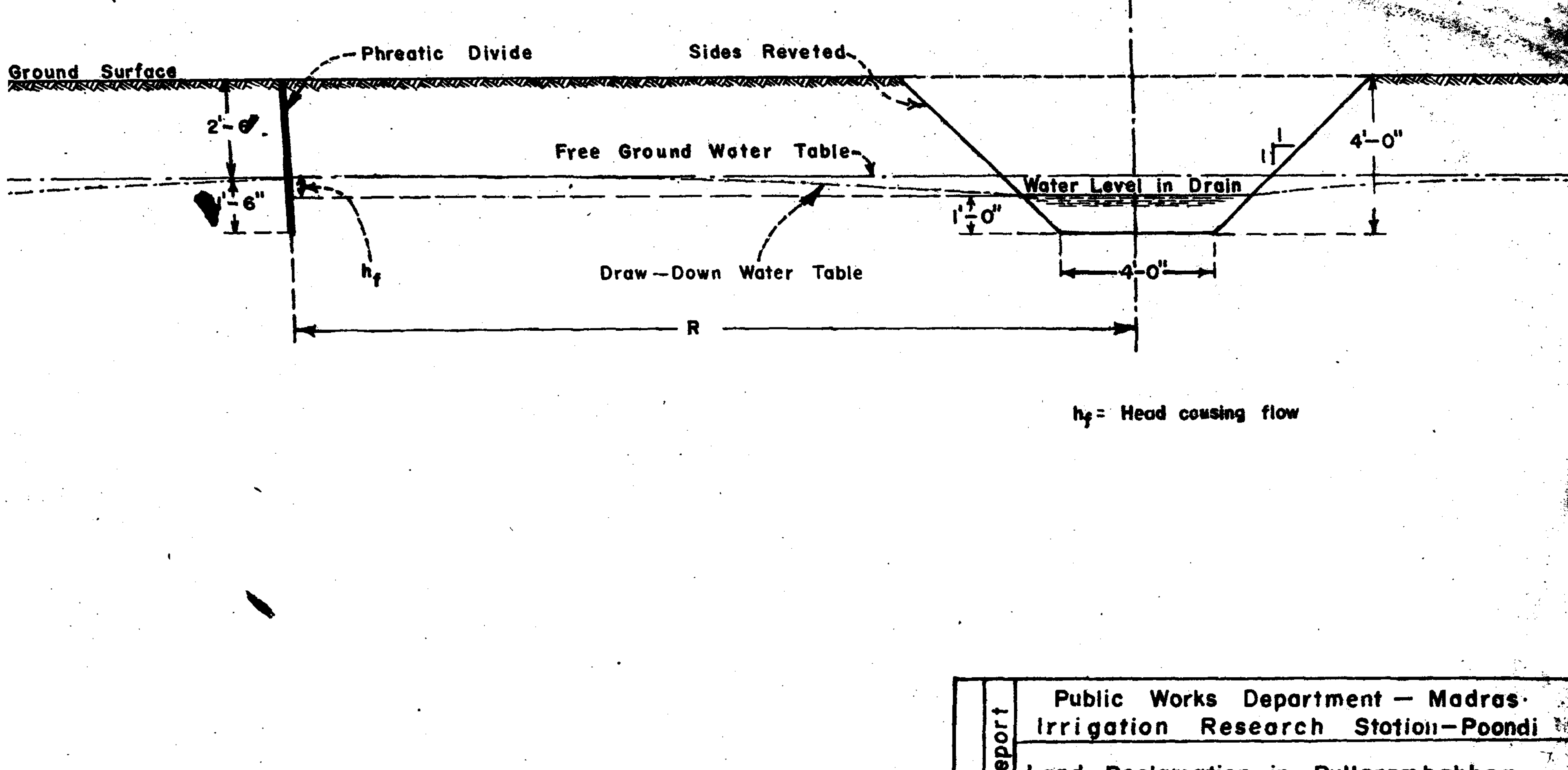

Report	Public Works Department — Madras. Irrigation Research Station—Poondi
File	Land Reclamation in Pullarambakkam SUB SOIL FLOW IN THE INTERCEPTING DRAIN
Drawing 150/59	N. Rajaratnam — J.E.R. ; A.R.O. ; E.E.R. ; Trd: V.V. ; ckd: ; FIG. 4