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ANNUAL REPORT FOR

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VOLUME-I

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INTRODUCTION

The Annual Report for the year 1959 presents the record of the research work done by the Irrigation Research Station, Poondi during the year 1959. This report contains seven sections classified as per the C.B.I & P. Classification and appears in a single volume.

During this year, several important problems connected with the irrigation and Hydro-Electric projects under execution in this State, were investigated with the aid of the hydraulic models and useful suggestions and recommendations have been made based on the model tests. investigations have also been taken up on the various problems connected with the Parambikulam Aliyar Project so as to verify experimentally the soundness of the designs and make suitable recommendations.

Studies were also conducted for the determination of the discharge characteristics of the spillways of the Amaravathy and Vaigai Dams and optimum gate-operation charts have been prepared for the guidance of the field engineers.

Various problems encountered in the maintenance of hydraulic structures have also been studied in hydraulic models and suitable redmedial measures have been recommended. The following deserve special mention:

(1) Cavitation in Mettur Dam River sluices.

(2) Protection of left abutment of Vilangudi bridge.

(3) Silt exclusion from canal off-takes.

Some headway has also been made in the field of basic and fundamental reserch. Experimental verification of the existing theories on the phenomenon of sub-soil flow has been made utilising both the hydraulic model and the electrical analogy tray. Investigations were also carried out on the characteristics of flow through Circular Notches and over bye-washes. The evaporation studies to determine the utility of the cetye alcohol for minising evaporation losses were also continued. Studies were also in progress to flush the Cooum river with increased tidal influx by suitable measures.

The research staff are very thankful to the valuable help and guidance rendered by the Superintending Engineer (Designs) and the Chief Engineer (Irrigation) in carrying out the studies successfully.

2. Personnel:

Chief Engineers:—

Sri J. G. Abraham, B.E., M.I.E. (Ind.) M. AM. SOC. C.E.

Sri K. V. Ekambaram, B.E., M.I.E. (Ind.) I.S.E.

Superintending Engineers :—

Sri V. R. S. Pandian, B.E., A.M.I.E. (Ind.)

Sri D. Henry David, B.E., A.M.I.E. (Ind.)

Sri V. A. Rangarajan, B.E., A.M.I.E. (Ind.)

Executive Engineer:—

Sri P. K. Kandaswamy, B.E., M.SC. (Iowa.) M. ASCE. (from 12-9-'58 f.n.)

Assistant Engineer (Research):—

1. Sri R. Sridharan, B.E. (till 1-7-'59 f.n.)
2. Sri J. James, B.E., (from 1-7-'59 f.n.)

Assistant Research Officer.—

1. Sri M. Balasubramaniam, B.E. (till 5-3-'59 f.n.)
2. Sri U. V. Balakrishnan, B.E. (from 5-3-'59 f.n.)

Junior Engineers:—

1. Sri Albert Nathaniel Raj, B.E. (till 17-8-'59 f.n.)
2. Sri P. Krishnaswamy, B.E. (till 17-10-'59 f.n.)
3. Sri R. N. Krishnan, B.E. (from 8-10-'58 f.n.)
4. Sri Rajaratnam, B.E. (Hons.), M.SC. (Engg.) (till 30-7-'59)
5. Sri J. James, B.E. (till 1-7-'59 f.n.)
6. Sri S. Krishnamurthi, B.E.
7. Sri M. R. Vedagiri, B.E.
8. Sri A. Shanmugam, B.E. (from 14-12-'59 f.n.)
9. Sri C. Raghavendra Rao, B.SC. (Engg.)
10. Sri R. Narayanan, B.E. (from 14-10-'59 f.n.)

Junior Research Assistants:—

1. Sri B. Thomas, B.SC. (Hons.) (from 23-3-'59 a.n.)
2. Sri S. Meenakshisundaram, M.A.

Technical Assistant (Research)

Sri S. Krishnamurthi, B.E.

Section Officer (Works)

1. Sri N. Valmikinathan (till 6-10-'59 f.n.)
2. Sri K. S. Kasiviswanathan (from 3-11-'59 f.n.)

3. Visitor's Remarks—

Name and address of the visitor	*Date of visit.*	*Remarks.*
1. Shri D.V. Joglekar, Adviser, Central Board of Irrigation and Power, Poona.	28——'59	Sri Kandaswamy is now incharge of the Research Station. He has taken two years training in U.S. and I am sure he with the help of his colleagues will raise the status of the research station still higher. I am very glad to find many problems of basic nature are being studied in addition to numerous specific problems and I hope this switch to basic and fundamental study will be sustained by ensuring continuity in research personnel. (Sd) D. V. Joglekar.
2. Shri J. P. Naegamvala, Superintending Engineer, Central Designs Organisation, Bombay.	4—2—'59	I had visited the Station in 1949. In last ten years it has expanded considerably. It is so gratifying to see that our young engineers and scientists are manning this Station so efficiently and take such keen interest in their work. I wish them all every success.
3. Mr. Chlaw Ratanaeon, Royal Irrigation Dept., Bangkok, Thailand.	30—7—'59	We are Thai Engineers from Thailand. We had have an opportunity to visit and attend the Officers of Poondi Irrigation of Irrigation models and Research work. This exploitation will be the most useful to us. We should like to appreciate the Government of India and Madras which treated us very kindly and sincerely.
4. Shri D. V. Joglekar, Adviser, Central Board of Irrigation and Power, Poona:	22-10-'59	Since my last visit in January 1959, great progress has been made in fundamental and basic research problems. There is a great improvement in the layout of models, garden layout etc., and the surroundings are ideal for research.
5. Mr. Walter J. Smith, C/o. Montreal Engineering Co., P. O. Box No. 250 Place D'Acmes Montreal P.O.	5-11-'59	I have very much enjoyed the interesting evening learning about your very extensive model research and seeing the methods used in studying all phase of hydraulic works. It is an excellant research station and I appreciate having the opportunity to see it and enjoy your hospitality.
6. Mr. L. G. Grandy, Ganadian Engineer, C/o Kundah Hydro-Electric Project.	5-11'59	It is a great honour and pleasure to visit your station and see the excellant works you are doing in hydraulic Engineering.

SUMMARY OF THE ANNUAL REPORT FOR 1959

A summary of the results of the hydraulic model expriments conducted at the Irrigation Research Station, Poondi, during the year 1959 is furnished below, arranging the items as per the Central Board of Irrigation and Power Classification.

Item Number.	*C.B.I. and P. Clsssification Number.*	*Desription.*
1	A-1 (i)	... *Flow throw Circular Notches.*—The circular and semi-circular notches have certain inherent advantages and they can be used for the measurement of water in hydraulic laboratories and Research Station Studies were conducted with a view to promote the use of these notches for the measurement of water discharge. Stevens' evolusion using Elliptic function for the theortical quation of flow through the circular notches was given. Another method solution styied as "Function" was introduced with the evolution of a simplified chart for direct solution. Further studies were conducted to compute the discharges in the non-moulder or submerged stage.
2	A-1 (i)	... *Bye—Washes.*—Bye—washes are normally provided as flood escapes for tanks. At present the discharge over a bye-wash is calculated by means of empirical formulae. Studies were conducted to find the effect of different variables such as upstream slop, downstream slope, top width, head of over, flow etc., on the discharge coefficient. The data obtained were analysed to formulate a design criteria for the design of bye-washes.
3	A-1 (i)	... *Flow through open channel obstructions.*—Studies were initiated to investigate the hydraulics of flow through open channel obstructions such as wires, anicuts, bridges, regulators etc., and the mechanism of scour around bridge piers and also to predict the afflux caused by astructure of certain obstruction ratio for any given flood discharge under field conditions and vice versa. As a first step, experiments were completed to study the nature of flow through bridge with square ended yiers and piers with simi-circular out waters. Further studies are in progress with piers of equiangular ends.
4	A-1 (iv)	... *Discharge characteristics of Amaravathy spillway.*—Studies were conducted with a view to determine the discharge characteristics of Amaravathy spillway. A detailed description of the investigations carriedout both for controlledled and uncontrolled flow over the spillway has been given in the final respect on the studies. The trend of variation of the discharge coefficient for free flow over the spillway has been studied. The variation of the discharge coefficient with the gate opening and the number of gates operated, in the case of controlled flow, has also been studied. From the results of the experiments, an "Optimum gate operation chart" has

Item Number.	C.B.I. and P. Clsssification Number.	Description.
		been developed for various under levels upstream in the reservoir. A working chart also has been developed so as to find out the amount of opening by which the gates should be operated to pass a flood of known magnitude. Examples have been worked out to illustrate the use of the chart.
5	A-1 (iv)	*Discharge characteristics of Vaigai Spillway.*—Studies were conducted in a comprehensive model constructed to scale of 1/60 to obtain the spillway rating curve for the Vaigai Dam. Experiments were conducted both for free and gate controlled flows. Optimum gate operation charts to pass the floods over the spillway on the basis of least consumption of power were developed for different water levels in the Reservoir. A working chart was also evolved so as to find out the amount of gate opening required with seven shutter operation to pass the flood knowing the rate of rise of water in the Reservoir.
6	A-1 (vi)	*High Coefficient Weirs*—It was proposed to increase the storage capacity of some of the Irrigation tanks in this State by raising the Crest level of the surplus weirs. Studies were conducted to arrive at a suitable profile for the surplus weirs to pass the estimated surplus quantity without increasing the M. W. L appreciably and the length of the weir. The Profile for the weirs were so selected as to modify the existing structures to the minimum extent possible. It was found that by suitably modifying the profile of the weir the discharge coefficient was increased considerrably and hence the weir can pass the estimated discharge for the same length, without increasing the Maximum Level much.
7	B-1 (ii)	*Aliyar Dam-Training of the Surplus course*—A three-dimensional model of Aliyar Dam has been construcred to a scale of 1/60 showing the river course, spillway section, a portion of the earth dam and non-overflow section of masonry dam, the bridge across the river downstream of the dam etc. Studies are being conducted for (i) arriving at the best radil for the bends in the surplus course for guiding the flow satisfactorily ; (ii) observing the water surface profile in the surplus course from the tow of spillway upto the existing anicut at about 1500 feet downstream of the dam and (iii) to effect even distribution of flow through the vents of the bridge constructed across the rivers.
8	B-1 (ii)	*River training work for Periyar at the site of Oumbum Valley Panchayat Water supply Scheme*—The Periyar river at the site of Oumbum Valley Panchayats water supply scheme follows a serpentive course consisting of two S-curves. Hence it is feared that the river may cut across the site of Pump-house causing damages to the latter. The problem has been referred to this Station to device some river training work to avoid damage to the pump-house. The model construction has been taken up.

Item number.	C.B.I. and P. Classification number.	Description.
9	B-5 (i)	*Dindugal Water supply scheme—River sluice—Energy dissination arrangements*—Studies were conducted in a geometrically similar model constructed to scale of 1/15 to arrive at a suitable energy dissipation arrangement for the Earth-Dam River Sluice of the Dindugal Water supply scheme. The finalised arrangement consists of a stilling basin of baffle type with a baffle wall three feet high and two rows of staggered blocks 2'0" x 1"0" x 1'0" fixed at the end of the ridid apron. A roof deflector at the exit of the sluice barrel has been recommended.
10	B-5 (i)	*Cavitation Studies on Mettur Dam river sluices*—Studies were conducts to determine the pressures obtaining along the sides of the barrels of the Mettur dam High Level and Low level sluices in rear of the gate slots and to suggest suitable remedies to avoid cavitation damages to the barrels. By rounding off the sharp corners and by the provision of air vent pipes at suitable levels in the deflector plates the high negative pressures were reduced in the model to a safe range.
11	B-6	*Aliyar Dam—Energy dissination studies*—A sectional model of Aliyar Dam spillway to a scale of 1/60 has been constructed to devise the most efficient and economic energy dissipation arrangements necessary at the toe of the spillway for both the conditions of smooth surface and rough rock surface of the chute spillway. A sunk basin type of energy dissipating arrangement has been found to be the best.
12	B-6	*Aliyar Dam—Pressure Studies*—A sectional model of Aliyar Dam Spillway to a scale of 1/60 has been constructed incorporating the finalised energy dissipating arrangements to evolve a cavitation-free profile for the spillway. Suitable curves at the beginning and at the end of the chute spillway are being evolved in order to reduce the negative pressures developed at three points.
13	B-6	*Morning Glory Spillway*—Model studies were initiated to find out the hydraulic performance of the morning glory spillway designed for the Siruvani Reservoir in Coimbatore District. The head versus discharge relationship for the spillway has been established by model experiments. The pressures obtaining at various points along the profile of the spillway have also been determined for various heads over the spillway crest. Further studies are in progress.
14	B-6	*Palar anicut—Energy dissination arrengements in rear of anicut*—The problem of devising suitable energy dissipation arrangments in rear of the Palar anicut in order to protect the rear aprons was studied in this Station. A sloping apron which will form a sunk basin of 6 feet depth has been found to give a good hydraulic performance.

Item number.	C.B.I. and P. Classification number.	Description.
15	B-8	*Palar Anicut—Silt exclusion studies*—It was proposed to imporove the existing Palar anicut and to find out the most effective silt-exclusion device for the given conditions. Model experiments have been conducted to find out a suitable device for exclusion of silt from the channels taking off from the anicut.
16	B-8	*Grand Anicut Canal—Silt exclusion studies—(Full width model)*—Studies were conduct for excluding silt from the Grand Anicut Canal. A divide wall was put up near the Grand Anicut Head Sluice and studies were conducted to arrive at the optimum length and width of the pocket. Further studies are in progress to find out the efficacy of other types of silt excluders.
17	B-8	*Grand Anicut Canal—Silt—Silt exclusion studies—Part width model*—Studies were conduct to exclude silt from entering into the Grand Anicut Canal by allowing a part of the discharge through an additional regulator, drawing discharge from the uplong drain. Then, along with the same proposal the sill level of the existing head regulator was raised and the studies conducted. Studies were also conducted with a view to observe the performance of the sand vents and the curtain wall proposed for construction in Grand Anicut Canal at R.D. 2/34 for diverting a portion of the bed sand which the canal carried from its upper reaches.
18	B-8	*Nathampatti Supply Channel—Silt exclusion studies*—Studies were conducted with different lengths of groynes in different position, and finally a 100 feet long placed perpendicular to the head sluice was found to be effective in reducing the entry of silt into the canal and the same was recommended for adoption. Then the groyne was tested for its affectiveness, for low discharges in the river with and without the curved wali of height 1/3rd the depths with its upstream face vertical and downstream face given a slope of 1 to 5, on the upstream side of the 100 feet long groyne (suggestion of Sri D. V. Joglekar, Adviser, Central Board of Irrigation and Power). But by eddies it was found that the groyne, was nor as effective as it was, for low discharges. Studies were completed.
19	B-8	*Srivakundam Anicut— Equitable discharge—Further studies*—In the studies previously conducted for a dominent discharge in the river, a set of three groynes were used to shift the deep course of the river from the left margin towards the centre. Then studies were conducted to find out whether the deep course shifted will get obliterated for higher discharges. It was found out that the performance of the groynes in shifting the deep course holds good even for higher discharges.

Item number	C.B.1. and P. Classification number	Description
20	B-12	*Vilangudi Bridge—Protection of the left abutment*—Studies were conducted to protect the left abutment of a bridge which has been constructed at the neck of a S-curve and at the junction of three streams. Repelling groynes were placed on the upstream of the bridge on the left side and it was found that the abutment was saved from deep scour to a considerable extend. Studies were conducted to arrive at the optimum length and location of the groyne. Further studies are in progress to distribute the flow uniformly through all the vents of the bridge.
21	C-2 (ii)	*Kundah Hydel Project—Surge tank in tunnel No. 2*—A Surge shaft has been provided in tunnel No. 2 of the Kundah Hydel Scheme at the end of the tunnel. Studies are in progress to find out the maximum and minimum surge levels, in the surge tank for the condition of instantaneous closure of the control valve.
22	C-2 (ii)	*Sandynallah Project—Spillway—Energy Dissipation studies*—Hydraulic model studies were conducted in a sectional model to evolve a suitable energy dissipation studies. A number of trials were made with different heights of baffles, different stilling basin floor elevations and with or without friction blocks. Finally, in view of the occurrences of a soft pocket within the stilling basin as reported from field, a 100 feet long basin with 9 feet depth was recommended for adoption.
23	C-2 (ii)	*Sandynallah Project—Spillway—Pressure studies*—The profile designed for the spillway of Sandynallah Dam was tested for pressures in a sectional model constructed to scale 1/50. This profile was revised three times. The finally revised profile was found to be devoid of any negative pressure and the same was recommended for adoption.
24	C-2 (ii)	*Upper Bhavani spillway—Energy dissination studies*—A sectional model of the Upper Bhavani Spillway was constructed to a scale of 1/100 with flip bucket type of energy dissipation arrangement. The studies have been conducted by altering the radius of the bucket along with the flip angle so that a wide range of trails can be analysed. The relative merits of the buckets are also being studied based on the pressures occuring in the bucket for various operations of the spillway.
25	C-E (ii)	*Upper Bhavani spillway —Pressure studies*—A sectional model of the Upper Bhavani spillway was constructed to a scale of 1/100 with piegometric arrangements. Studies were conducted for different uncontrolled flows as well as controlled flows over the crest of the spillway. The occurrence of the subatmospheric pressures on the profile was far below the allowable limit. Hence this original profile was recommended for adoption.

Item number	C.B.I. and P. Classification number	Description
26	C-2 (ii)	*Kundah Power House—2-Tail Race proposals Pressure studies for the measuring weir and cellibration of the weir*—A comprehensive model of the design proposels was constructed to a scale of 1/20 representing the outlet conduits of the units, the common tail race pool, the measuring weir and the outfall chute way. The model revealed certain peculiar phenomenon of flow pattern upstream of the measuring weir. Hence, before attempting to calibrate the weir, it was found imperative to improve the flow pattern upstream of the wairfor all conditions of operations of the units so that the head on crest was uniform. Studies were conducted to improve the flow pattern by trying various devices. It was recommended that an alternative to the measuring weir should be adopted in construction such us flow meters, for testing the efficiency of units. However the weir profile was improved based on pressure studies and the same was recommended for adoption, if necessary.
27	D-4	*Evaporation Losses from open storages*—For assessing the utility of Hexadecanol for reducing the loss due to evaporation from large open storages, experiments were carried out in a medium sized tank called Buderi tank situated about two miles south-west of Irrigation Research Station, Poondi. Various quantities of the solution of Cetyl alcohol dissolved in Mineral Turpentine were sprayed daily on the tank-water surface for a certain period in each case with a view to find the maximum amount of savings in evaporation that can be achieved by the use of this chemical and also to find the amount of chemical required to give optimum results. The effect of wind, rain etc., on the chemical film was also studied. Studies were conducted by installing what are known as "Seepage meters" to assess the quantity of seepage through the bed of the tank.
28	E.1 (ii)	(a) *Sub-Soil Flow—Earth Dams*—The seepage through earth dams depends on many factors which include the type of material used for the construction, the type of foundation available, the depth and location of cut off walls, the location of internel core walls and the drainage measures provided etc. Studies were conducted to find out the effects of the various factors on the seepage quantity. These results were compared with the results obtained by the electrical analogy methods and with the computed values based on some of the existing theories.

Item number	C.B.I. and P. Classification number	Description
29	F-1 (iv)	*Palar Anicut—Design of the two zoned channel section*—In order to carry away the silt that enters the channel through the head sluice even during low flows in the channel, a new type of design of channel was evolved. The design consists of a narrow inner section provided within the usual trepezoidal section and berms of suitable widths. The hydraulic model studies on this channel section is in progress.
30	H-3	*Flushing the river Cooum by tidal action*—A sand bar gate formed at the mouth of the river Cooum, by the action of the littoral drift along the East Coast and as there is no flow in the river for a major part of the year it is cut off from the sea. Any water that is tapped in the river becomes stagnant and with the inflow of some sewage that overflows the city sewers, the water in the river emits a bad odour. It was proposed to cut open the bar by dredging to make the river tidal and to channelise the river so that the tidal effect is felt to the greatest extent possible, for effective flushing of the river. Experiments were conducted in a model constructed to a scale of 1/200 H and 1/20 V. A tidal mechanism to produce a tidal range corresponding to four feet in a period of 12 hours in the prototype was devised and made use of for the studies. Studies were conducted tn finalise the various aspects such as the optimum width of the section, depth of the section, narrowing the width towards the upper reaches, reducing the width of the North Arm of the River Cooum, the optimum width of the cut to be maintained in the bar site in keeping with economy and efficiency etc. These were analysed both theoretically and by model studies for comparison.

FLOW THROUGH CIRCULAR NOTCHES

1. SYNOPSIS

The hydraulics of flow through circular and semicircular notches in rectangular flumes is presented with a view to promote the use of these notches for the measurement of water discharge in Hydraulic Laboratories and Research Stations. A brief resume of the early literature is presented. The theoretical equation of flow through circular notches is derived. Steven's solution for this equation using Elliptic functions is given. Another method of solution, styled as " ϕ Function" is introduced with the evolution of a simplified chart for direct solution. Based on considerable amount of carefully conducted experimental work, separate discharge equations have been derived both for Steven's solution and the ϕ Function and the coefficients are also presented in the form of simplified charts. Further, this paper gives a method of computation of discharge in the non-modular or submerged stage.

2. INTRODUCTION

Sharp-crested weirs and notches of various types are being used in Hydraulic Laboratories and Research Stations for the measurement of water discharge. In this country the circular notch has not come into the common use. The circular notch, has certain inherent advantages. Firstly, the crest of the notch can easily be turned and levelled with precision in a lathe. Secondly, the weir crest does not have to be levelled and hence the point of zero flow is easily determined. Thirdly, it gives an appreciable head of flow even for small discharges, an advantage which is also shared by the V-notch.

The theoretical equation of flow through a circular notch is difficult, if not impossible, to integrate. As a result, there are a number of equations some highly mathematical and others profusely empirical. The most notable solution to this posing problem was given by J. C. Stevens (1) M. ASCE., in the form of an Elliptic Function. Due to the lack of experimental data, Stevens assumed a constant value of O. 59 for the discharge coefficient. But it is to be Generally expected that the coefficient may vary with the head of flow and the approach conditions. So, it is proposed to develop a discharge coefficient equation for Stevens' formula. Further, it is aspired to develop a simpler yet equally accurate theoretical discharge equation and also a chart for the discharge coefficient to obtain the actual discharge for a set of given conditions. The discharge characteristics of this notch in the submerged or non-modular range has been studied. Detailed investigations on these lines have been made in the Irrigation Research Station, Poondi, for the case of semicircular notches fixed in rectangular flumes.

3. REVIEW OF LITERATURE

H. V. Cone was the early investigator in the United States of America. He experimented with two circular and two semi-circular notches. Extensive experiments on flow through circular, parabolic and triangular notches are conducted by F. V. Greve at the Engineering Experiment Station of Purdue University. Eldon R. Dodge developed a trignometrical series formula and also curves for discharge coefficients. Gulliver did experiments with a small circular notch, the differential equation was mechanically integrated and the results were given by curves. Thijsse gives a different formula for discharge through circular notches and he has used these notches extensively for the measurement of water in his laboratory. J. C. Stevens (1), M. ASCE., has presented a new solution for finding the discharge through circular notches.

4. THEORY OF FLOW

The adjoining sketch is a schematic representation of the flow through a circular notch.

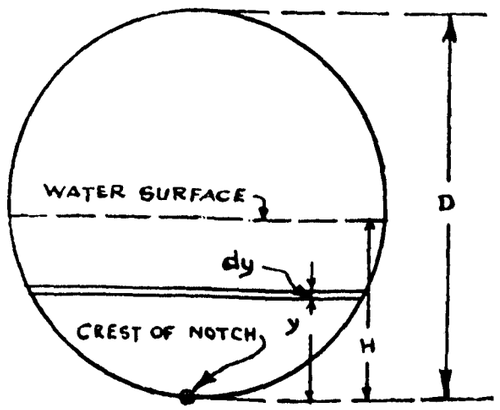
D is the diameter of the notch,

H is the head of flow,

Considering a strip of water or depth dy at a height of y above the crest as shown in the sketch, the width of the strip = b = $2\sqrt{y\,(D - y)}$

the area of the strip = $\int A$

$2\sqrt{y\,(D - y)}\,dy$



Schematic representation of Flow through a circular notch

The elemental discharge through this strip =

$$dq = 2\sqrt{2g}\;\sqrt{y(D - y)}\;(H - y)\,dy \qquad (1)$$

This expression holds good whether H is greater or less than the radius of the notch.

4. 1. Steven's Elliptic Function

With considerable assistance from Dr. W. E. Milne. J. C. Stevens, M. ASCE., obtained a complicated solution of this complex differential equation which is given as

$$Qt = \frac{4}{15}\sqrt{2g}\;D^{5/2}\left[2\,(1 - k^2 - k^4)\,E - (2 - k^2)\,(1\,k^2)\,K\right] \ldots (2)$$

were Qt is the theoretical discharge,

D is the diameter of the notch

$k = \sqrt{\frac{H}{D}}$,

H and D are as defined in the sketch, and E and K are Elliptic Functions to be taken from a Table of Integrals.

The above equation is further simplified and written

as $Qt = A\Delta \quad \ldots \quad \ldots \quad \ldots \quad (3)$

where $A = \frac{4}{15}\sqrt{2g}\;D^{5/2}$

$= 2.14\;D^{5.2} \quad \ldots \quad \ldots \quad (4)$

and $\Delta = \left[2\,(1 - k^2 - k^4)\,E - (2 - k^2)\,(1 - k^2)\,K\right] \quad \ldots \quad (5)$

Table I gives the values of Δ to four decimals for the values of k2 or H/D ranging from 0 to 100.

4. 2. ϕ Function

The differential equation of flow through a circular notch can be put in the form,

$$Qt = 2\sqrt{2g}\;\int_0^H \sqrt{y\,(D - y)\,(H - y)}\;dy$$

This can be expanded into an infinite series and be integrated term by term and the final equation can be shown to be

$$Qt = 2\sqrt{2g}\;\pi\,D^{5/2}\left[\frac{1}{8}\,Z^2 - \frac{1}{32}\,Z^3 - \frac{5}{1024}\,Z^4\right] \quad \ldots \quad (6)$$

where $Z = H/D = k^2$ where k is the parameter used by Stevens.

Equation (6) can be simplified and given as

$$Qt = \phi\,(Z)\;D^{52} \quad \ldots \quad \ldots \quad \ldots \quad (7)$$

where $\phi\,(Z) = 2\sqrt{2g}\;\pi\left[\frac{1}{8}\,Z^2 - \frac{1}{32}\,Z^3 - \frac{5}{1024}\,Z^4\right] \quad \ldots \quad (8)$

neglecting further terms inside the bracket.

Values of ϕ (Z) for values of H/D upto 1.0 have been computed for notches of diameter upto 60 inches and the theoretical discharges are plotted against Z as given in Figure-1 and 2 so that the theoretical discharge for any particular notch and given head can easily be read off. Alternatively Qt can also be calculated with the aid of a Table which gives the value of the function ϕ(Z) for various values of Z, multiplying ϕ (Z) by $D^{5/2}$, similar to Table-I but is not given in this paper.

5. MODEL SET UP AND EXPERIMENTS

5. 1. Model set up

The experiments for free flow were conducted in a masonry flume 20′ 0″ x 3′0″ x 2′0″. Water was let into the flume at the upstream end and measured with a suppressed rectangular weir which was already calibrated using Rehbocks's formula. The semi-circular notch was fixed at the downstream end of the flume as shown in Figure-3, so that it discharged in a free state. Experiments for free flow were conducted with four different diameters of notches, the diameters being 2′0″, 1′6″, 1′3″ and 1′0″ respectively. The experiments for submerged flow were conducted in another masonry flume 40′0″ x 2′0″ x 2′0′ and submergence studies were done only for 1′0″ diameter notch.

5. 2. Experiments for free flow

Firstly the 2′0″ diameter notch was fixed centrally in the 3′0″ wide flume with its crest at a height of 8 inches above the bed of the approaching level floor. Various discharges, measured at the weir plate were passed through the notch. The head over the circular notch was measured by a foot-gauge situated in a gauge well located at a distance of 4 Hmax from the notch where Hmax is the maximum probable head of flow over the notch. Secondly, the height of the crest was fixed 7 inches above the floor and the studies repeated. Similarly, experiments were conducted for six more different heights of crest varying from 6″ to 0″ at intervals of one inch. This scheme of experimentation was gone through with the other three semi-circular notches also as given in Table-II.

For each run, the coefficient of discharge was computed both by Stevens' Function and ϕ Function. The discharge coefficient obtained by Stevens' Function is denoted as $C\Delta$ and that obtained by ϕ Function as $C\phi$.

5. 5. Experiments for submerged flow

The free and submerged flow characteristics of the 1'0" diameter notch installed in the 2'0" wide flume were studied, using a V-notch for discharge measurement. For each particular discharge, the tail water was varied in steps and the corresponding upstream head was measured. This was repeated with five or six discharges. The variation of the reduction factor 'f' given by the general equation

$$\frac{Q'a}{Qa} = f \quad \ldots \quad \ldots \quad (9)$$

was studied.

Where Qa is the actual discharge in the free flow or modular stage

Q'a is the actul discharge in the submerged or non-modular stage and

f is the reduction factor.

6. DISCUSSION OF RESULTS

Free Flow Analysis—Effect of velocity of approach

The results of the experiments on the four notches were plotted with Qa, the actual discharge as abscissae and $\frac{H}{D}$, the ratio of head of flow to diameter of notch as ordinate as shown in Figure-4. On a close analysis it can be seen that the points for anyone notch diameter are seen to follow the same trend of variation. The coefficients $C\Delta$ and $C\phi$ are shown plotted against $\frac{H}{P}$ in figure 5 and 6, where P is the height of the crest above the floor.

It is evident that the values of $C\Delta$ and $C\phi$ tend to increase as the values of H/P progressively increase ; but it is difficult to draw definite conclusions on this aspect because of the limited range of H/P that was possible of investigation under the present scheme of study. It may be of particular interest to investigate further as to how the discharge coefficient may vary for increasing values of H/P ranging from O to ∞. Nevertheless, it may be taken that the discharge coefficient remains more or less constant for the range of H/P (namely, upto about 6) investigated under the present scheme.

For both types of functions under consideration, the discharge coefficients have been shown plotted against H/D in Figure-7 and 8. It is seen the reform that the coefficients increase for increasing values of H/D. Also, it is note-worthy to observe that the wide of the flume relative to the diameter of the notch has considerable effect on the discharge coefficients.

6.2. Submerged Flow Analysis

It has been shown that for weirs and notches functioning in the submerged stage, upto the Modularity limit any increase in tail water level does not affect the upstream head. The submergence ratio of h/H, (where h is the height of tail water above the crest and H the head of water above the crest) at the modularity limit varies from a negative value for sharp crested rectangular weirs to a high value of about 0.80 for long-throated weirs. From the experimental results plotted in Figure-10, at the modularity limit the submergence ratio was found to be 0.30, and the variation of the reduction factor f with h/H is also shown in the same figure.

Knowing the reduction factor and the discharge in the modular range as taken from Figure-8, the discharge in the non-modular range can be computed from equation (2).

Dr. Ville monte (2) has proposed a formula for computing the discharge in the non-modular range.

$$\frac{Q}{Q_1} = k\left[1 - \frac{Q_2}{Q_1}\right]^m \quad \ldots \quad \ldots \quad (14)$$

where Q is the actual discharge,

Q_1 is the modular discharge for the given head,

Q_2 is the modular discharge for a head equal to the hight of the tail water above the crest

& K and m are constant and exponent respectively. According to Dr. Villeimonte K can be assumed to be unity and the exponent m can be given an average value of 0.385. The variation of the reduction factor f with h/H by the Ville monte formula is also given in the same figure. It seems to predict 'Instantaneous submergence' i.e., the non-modular stage sets in the moment the tail water rises above the crest. This is in contradiction to the experimental results obtained at this station. In support of the conclusion drawn in this paper, the submergence curve for the V-notch (3) is reproduced in Figure-10 and it may be seen that it also has a value of h/H = 0.30 at the modularity limit.

7. MEASUREMENT OF DISCHARGES WITH CIRCULAR NOTCH

Computation Procedures

7.1. Method-I

The computation procedure given below applies only to the case of circular or semi-circular notches located centrally in rectangular flumes.

Procedure

(1) Find out the head of flow - H

(2) From the diameter of the notch, compute $k^2 = H/D$

(3) From Table I, read off the value of Δ for the computed value of k^2.

(4) Calculate $A = 2.14\ D^{5/2}$

(5) Then, the theoretical discharge Qt is given by the equation $Qt = A\Delta$.

(6) C can easily be read off from Figure-7

(7) The actual discharge is given by the formula
$$Q_a = Q_t \times C_\Delta$$

(8) If the tail water rises above the crest, compute h/H, the sumbergence ratio.

(9) If h/H, is less than 0.30, than the actual discharge can still be obtained using the equation $Q_a = Q_t\ C_\Delta$

(10) If h/H is greater than 0.30, find out the corresponding reduction factor f from Figure-10.

(11) The actual discharge is given by the equation
$$Q'_a = f\ Q_t \times C_\Delta$$

Example—1

A semi-circular notch 2'0" diameter is fixed centrally in a rectangular flume of width 4'0". Find out the discharge when the head over the crest is 1'0" (by Steven's Method)

(a) in the free flow stage ;

(b) when the tail water rises above the crest such that h/H = 0.50.

Solution

(a) The theoretical solution Q_t is given by the equation $Q_t = A\Delta$

$A = 2.14\ D^{5/2} = 2.14 \times 2.0^{5/2}\ 12.10$

For $k^2 = H/D = \frac{1.0}{2.0} = 0.5$

From Table-1. $\Delta = 0{,}6354$.

Hence the theoretical discharge $= A\Delta$
$= 12.10 \times 0.6354$
$= 7.70$ cfs.

From Figure-7, C_Δ is found to be 0.600.

Hence the discharge in the free flow stage $= 0.60 \times 7.70$
$= 4.61$ c.f.s.

(b) Given submergence ratio h/H = 0.5.

From Figure-14, the reduction factor f = 0.96.

The actual non-modular discharge is given by the equation

$$\frac{Q'a}{Qa} = f$$

$Q'a = 0.96 \times 4.61 = 4.42$ c.f.s.

Discharge in the free flow or modular stage = 4.61 cfs.

Discharge in the non-modular stage = 4.42 cfs.

7.2. Method - II ϕ Function Method :

(1) Find out the head H

(2) From the diameter of the notch, find Z = H/D

(3) From Figure-1 or 2, read off the theoretical discharge for the given values of Z and D

(4) $C\phi$ can easily be read off from Figure-8

(5) The actual discharge is given by the equation
$$Q_a = Q_t \times C_\phi$$

(6) If the tail water rises above the crest, calculate the submergence ratio h/H

(7) If h/H is less than 0.30 the actual discharge given by the equation $Qa = Q_t\ C\phi$ still holds good.

(8) If h/H exceeds 0.30 then the performance becomes non-modular, and the discharge is given by the equation

$$\frac{Q'a}{Qa} = f$$

where f is a reduction factor to be read from Figure-10

Example-2

Solve example-1 by the ϕ function method

Solution

(a) Given $H/D = \frac{1.0}{2.0} = 0.50$

From Figure-2 $Q_t = 7.70$ cfs.

From Figure-10 $C\phi = 0.59$

Hence, the actual discharge $= 0.59 \times 7.70 = 4.54$ cfs. $= Qt \times C\phi$.

(b) Given the submergence ratio h/H = 0.5

The discharge in the non-modular range is given by the equation

$$\frac{Q'a}{Q_a} = f$$

From Figure-14, for h/H = 0.5, f = 0.96.

Hence, the discharge in the non-modular range = 4. 54 x 0. 96 = 4.36 cfs.

Discharge in the modular or free flow stage = 4.54 cfs.

Discharge in the non-modular stage = 4.36 cfs.

8. CONCLUSION

The circular and semi-circular notches have certain inherent advantages and they can be used for the measurement of water in Hyderaulic Laboratories and Research Stations. But because of the difficulty in solving the theoretical discharge equation and the non-availability of any suitable rigid practical solution, these notches have not come into general use in this country and many other countries alike. A remarkable solution to this problem was furnished by J. C. Stevens, M. ASCE., by the use of Elliptic functions and it is given as $Q_t = A\Delta$. But, because of the lack of experimental data, a constant value of 0.59 was assumed for the discharge coefficient to obtain the actual discharge. But it is generally known that this coefficient various with the approach conditions. Hence detailed investigations were carried out in this Research Station for semi-circular notches located centrally in rectangular flumes. Simplified curves have been developed for C_Δ

Secondly the theoretical equation of flow through circular notches has been integrated by expanding it into an infinite series and a method of solution named "ϕ Function" has been developed. This method is given in the form of a simplified chart so that the theoretical discharge for any set of given conditions can easily be read off. Curves for $C\phi$ have been developed.

Thirdly, studies were conducted to find out the modularity limit and also the variation of the reduction factor 'f' with the submergence ratio h/H. The modularity limit was found to occur when h/H = 0.30 and the variation of 'f' with h/H is also given. This variation of 'f' with h/H is slightly different from that given by Dr. Ville monte's formula which predicts 'Instantaneous submergence'.

It can be seen that the results or the studies on semi-circular notches will also hold good for full diameter notches. The results of the studies given in this paper facilitate easy and accurate measurement of water passing through circular and semi-circular notches located centrally in rectangular flumes. It is hoped that this paper will enable the circular and semi-circular notches to be used extensively in the different laboratories and Research Stations in this country and abroad.

9. REFERENCES

(I) Stevens J.C.,M. ASCE., "Flow through Circular Notches" Paper 1455 Proc. ASCE., December 1957.

(2) Stevens J.C., M. ASCE., Closure on paper 1455. Proc. ASCE January 1959.

(3) Smith C.D., "Open Channel water measurement with Broad-crested weir". Annual Bulletin of the International commission on Irrigation and Drainage 1958.

TABLE-1

Values of Δ

K^2	0	1	2	3	4	5	6	7	8	9
.00	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0001	0.0001	0.0002	0.0002
.01	0.0003	0.0004	0.0004	0.0005	0.0006	0.0007	0.0008	0.0008	0.0009	0.0011
.02	0.0012	0.0013	0.0014	0.0015	0.0017	0.0018	0.0020	0.0021	0.0023	0.0025
.03	0.0026	0.0028	0.0030	0.0032	0.0034	0.0036	0.0038	0.0040	0.0042	0.0044
.04	0.0047	0.0049	0.0051	0.0054	0.0056	0.0059	0.0062	0.0064	0.0067	0.0070
.05	0.0073	0.0075	0.0079	0.0082	0.0085	0.0088	0.0091	0.0094	0.0098	0.0101
.06	0.0104	0.0108	0.0111	0.0115	0.0119	0.0122	0.0126	0.0030	0.0134	0.0138
.07	0.0142	0.0146	0.0150	0.0154	0.0158	0.0163	0.0167	0.0171	0.0176	0.0180
.08	0.0185	0.0189	0.0194	0.0199	0.0203	0.0208	0.0213	0.0218	0.0223	0.0228
.09	0.0233	0·0238	0.0243	0.0249	0.0254	0.0259	0.0265	0.0270	0.0276	0.0281
.10	0.0287	0.0293	0.0298	0.0304	0.0310	0.0316	0.0322	0.0328	0.0334	0.0340
.11	0.0346	0.0353	0.0359	0.0365	0.0372	0.0378	0.0385	0.0391	0.0398	0.0404
.12	0.0411	0.0418	0.0425	0.0432	0.0439	0.0445	0.0453	0.0460	0.0467	0.0474
.13	0.0481	0.0489	0.0496	0.0503	0.0511	0.0518	0.0526	0.9533	0.0541	0.0549
.14	0.0557	0.0564	0.0572	0.0580	0.0588	0.0596	0.0604	0.0612	0.0621	0.0629
.15	0.0637	0.0646	0.0654	0.0662	0.0671	0.0679	0.0688	0.0697	0.0705	0.0714
.16	0.0723	0.0732	0.0741	0.0750	0.0759	0.0768	0.0777	0.0786	0.0795	0.0805
.17	0.0814	0.0823	0.0833	0.0842	0.0852	0.0861	0.0871	0.0881	0.0890	0.0900
.18	0.0910	0.0920	0.0930	0.0940	0.0950	0.0960	0.0970	0.0980	0.0990	0.1001
.19	0.1011	0.1022	0.1032	0.1042	0.1053	0.1064	0.1074	0.1085	0.1096	0.1106

Table-I (Contd.)

K^2	0	1	2	3	4	5	6	7	8	9
.20	0.1117	0.1128	0.1139	0.1150	0.1161	0.1172	0.1183	0.1194	0.1206	0.1217
.21	0.1228	0.1240	0.1251	0.1263	0.1274	0.1286	0.1297	0.1309	0.1321	0.1332
.22	0.1344	0.1356	0.1366	0.1380	0.1392	0.1404	0.1416	0.1428	0.1440	0.1453
.23	0.1465	0.1477	0.1490	0.1502	0.1515	0.1527	0.1540	0.1552	0.1566	0.1578
.24	0.1690	0.1603	0.1616	0.1629	0.1642	0.1655	0.1663	0.1681	0.1694	0.1708
.25	0.1721	0.1734	0.1747	0.1761	0.1774	0.1788	0.1801	0.1815	0.1828	0.1842
.26	0.1856	0.1870	0.1883	0.1897	0.1911	0.1925	0.1939	0.1953	0.1967	0.1981
.27	0.1995	0.2010	0.2024	0.2038	0.2053	0.2067	0.2081	0.2096	0.2110	0.2125
.28	0.2140	0.2154	0.2169	0.2184	0.2199	0.2113	0.2228	0.2243	0.2258	0.2273
.29	0.2288	0.2304	0.2319	0.2334	0.2349	0.2364	0.2380	0.2395	0.2410	0.2426
.30	0.2441	0.2457	0.2473	0.2488	0.2504	0.2520	0.2535	0.2551	0.2567	0.2583
.31	0.2599	0.2615	0.2631	0.2647	0.2663	0.2679	0.2696	0.2712	0.2728	0.2745
.32	0.2761	0.2777	0.2794	0.2810	0.2827	0.2844	0.2860	0.2877	0.2894	0.2910
.33	0.2927	0.2944	0.2961	0.2978	0.2995	0.3012	0.3029	0.3046	0.3063	0.3080
.34	0.3098	0.3115	0.3132	0.3150	0.3167	0.3185	0.3202	0.3220	0.3237	0.3255
.35	0.3272	0.3290	0.3308	0.3326	0.3343	0.3361	0.3379	0.3397	0.3415	0.3433
.36	0.3451	0.3469	0.3487	0.3506	0.3524	0.3542	0.3560	0.3579	0.3597	0.3616
.37	0.3634	0.3653	0.3671	0.3690	0.3708	0.3727	0.3746	0.3765	0.3783	0.3802
.38	0.3821	0.3840	0.3859	0.3878	0.3897	0.3916	0.3935	0.3954	0.3973	0.3993
.39	0.4012	0.4031	0.4050	0.4070	0.4305	0.4109	0.4128	0.4148	0.4167	0.4187
.40	0.4207	0.4226	0.4246	0.4266	0.4286	0.4305	0.4325	0.4345	0.4365	0.4385
.41	0.4405	0.4425	0.4445	0.4465	0.4486	0.4506	0.4526	0.4546	0.4567	0.4587

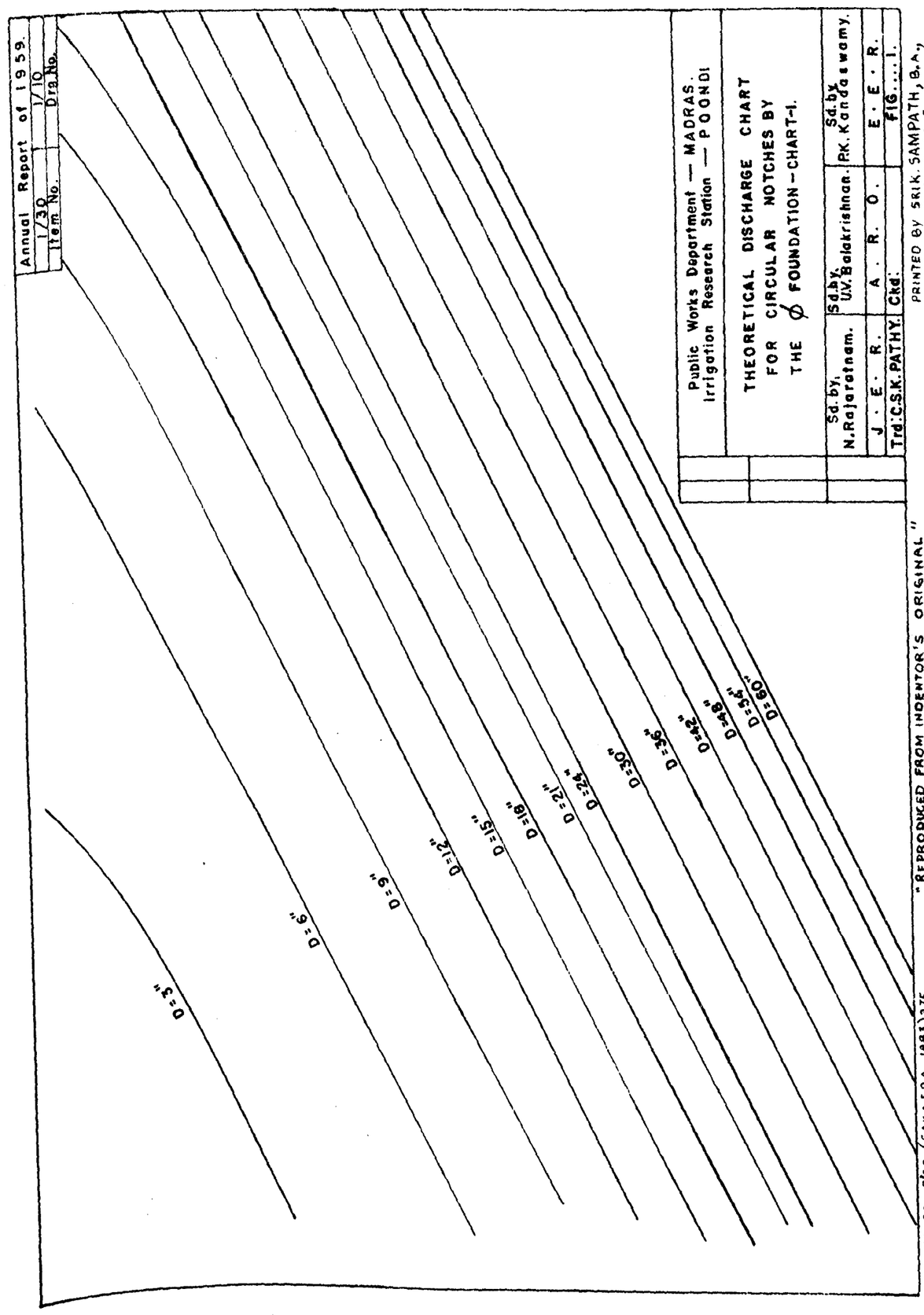
.42	0.4607	0.4628	0.4648	0.4669	0.4689	0.4710	0.4731	0.4751	0.4772	0.4793
.43	0.4814	0.4834	0.4855	0.4876	0.4897	0.4918	0.4939	0.4960	0.4981	0.5002
.44	0.5023	0.5044	0.5066	0.5087	0.5108	0.5129	0.5151	0.5172	0.5194	0.5215
.45	0.5236	0.5258	0.5280	0.5301	0.5323	0.5344	0.5366	0.5388	0.5410	0.5431
.46	0.5453	0.5475	0.5497	0.5519	0.5541	0.5563	0.5585	0.5607	0.5629	0.5651
.47	0.5673	0.5696	0.5718	0.5740	0.5762	0.5785	0.5807	0.5830	0.5852	0.5875
.48	0.5897	0.5920	0.5942	0.5965	0.5987	0.6010	0.6033	0.6056	0.6078	0.6101
.49	0.6124	0.6147	0.6170	0.6193	0.6216	0.6239	0.6262	0.6285	0.6308	0.6331
.50	0.6354	0.6377	0.6401	0.6424	0.6447	0.6470	0.6494	0.6517	0.6541	0.6564
.51	0.6587	0.6611	0.6634	0.6658	0.6682	0.6705	0.6729	0.6753	0.6776	0.6800
.52	0.6824	0.6848	0.6872	0.6895	0.6919	0.6943	0.6967	0.6991	0.7015	0.7039
.53	0.7063	0.7087	0.7112	0.7136	0.7160	0.7184	0.7208	0.7233	0.7257	0.7281
.54	0.7306	0.7330	0.7355	0.7379	0.7404	0.7428	0.7453	0.7477	0.7502	0.7526
.55	0.7551	0.7576	0.7601	0.7625	0.7650	0.7675	0.7700	0.7725	0.7749	0.7774
.56	0.7799	0.7824	0.7849	0.7874	0.7899	0.7924	0.7949	0.7975	0.8000	0.8025
.57	0.8050	0.8075	0.8101	0.8126	0.8151	0.8177	0.8202	0.8227	0.8253	0.8278
.58	0.8304	0.8329	0.8355	0.8380	0.8406	0.8431	0.8457	0.8483	0.8508	0.8534
.59	0.8560	0.8586	0.8611	0.8637	0.8663	0.8689	0.8715	0.8741	0.8767	0.8793
.60	0.8819	0.8845	0.8871	0.8897	0.8923	0.8949	0.8975	0.9001	0.9027	0.9053
.61	0.9080	0.9106	0.9132	0.9258	0.9185	0.9211	0.9237	0.9264	0.9290	0.9317
.62	0.9343	0.9369	0.9396	0 9423	0.9449	0.9476	0.9502	0.9529	0.9555	0.9582
.63	0.9609	0.9635	0.9668	0.9689	0.9716	0.9742	0.9769	0.9796	0.9823	0.9850
.64	0.9877	0.9903	0.9930	0.9957	0.9984	1.0011	1.0038	1.0065	1.0092	1.0119
.65	1.0147	1.0174	1.0201	1.0228	1.0255	1.0282	1.0309	1.0337	1.0364	1.0391
.66	1.0418	1.0446	1.0473	1.0500	1.0528	1.0555	1.0583	1.0610	1.0637	1.0665

Table-I (contd.)

K^2	0	1	2	3	4	5	6	7	8	9
.67	1.0692	1.0720	1.0747	1.0775	1.0802	1.0830	1.0858	1.0885	1.0913	1.0940
.68	1.0968	1.0996	1.1023	1.1051	1.1079	1.1107	1.1134	1.1162	1.1180	1.1218
.69	1.1245	1.1273	1.1301	1.1329	1.1357	1.1385	1.1413	1.1441	1.1469	1.1497
.70	1.1524	1.1552	1.1580	1.1608	1.1637	1.1665	1.1693	1.1721	1.1749	1.1777
.71	1.1805	1.1833	1.1861	1.1889	1.1918	1.1946	1.1974	1.2002	1.2031	1.2059
.72	1.2087	1.2115	1.2144	1.2172	1.2200	1.2229	1.2257	1.2285	1.2314	1.2342
.73	1.2370	1.2399	1.2427	1.2456	1.2484	1.2512	1.2541	1.2569	1.2598	1.2626
.74	1.2656	1.2683	1.2712	1.2740	1.2769	1.2798	1.2826	1.2855	1.2883	1.2912
.75	1.2941	1.2969	1.2998	1.3026	1.3055	1.3084	1.3112	1.3141	1.3170	1.3198
.76	1.3227	1.3256	1.3285	1.3313	1.3342	1.3371	1.3400	1.3428	1.3457	1.3486
.77	1.3515	1.3544	1.3572	1.3601	1.3630	1.3659	1.3688	1.3716	1.3745	1.3774
.78	1.3803	1.3832	1.3861	1.3890	1.3918	1.3947	1.3976	1.4005	1.4034	1.4063
.79	1.4092	1.4121	1.4150	1.4179	1.4208	1.4237	1.4266	1.4294	1.4323	1.4352
.80	1.4381	1.4410	1.4439	1.4468	1.4497	1.4526	1.4555	1.4584	1.4613	1.4642
.81	1.4571	1.4700	1.4729	1.4758	1.4787	1.4816	1.4845	1.4874	1.4903	1.4932
.82	1.4961	1.4990	1.5019	1.5048	1.5077	1.5106	1.5135	1.5164	1.5193	1.5222
.83	1.5251	1.5280	1.5309	1.5338	1.5367	1.5396	1.5425	1.5454	1.5483	1.5512
.84	1.5541	1.5570	1.5599	1.5628	1.5657	1.5686	1.5715	1.5744	1.5773	1.5802
.85	1.5831	1.5860	1.5889	1.5918	1.5947	1.5976	1.6005	1.6034	1.6063	1.6092
.86	1.6121	1.6150	1.6178	1.6207	1.6236	1.6265	1.6294	1.6323	1.6352	1.6381
.87	1.6410	1.6438	1.6467	1.6496	1.6525	1.6554	1.6583	1.6611	1.6640	1.6669
.88	1.6698	1.6726	1.6755	1.6784	1.6813	1.6841	1.6870	1.6899	1.6928	1.6956
.89	1.6985	1.7014	1.7042	1.7071	1.7100	1.7128	1.7157	1.7185	1.7214	1.7242
.90	1.7271	1.7300	1.7328	1.7357	1.7385	1.7414	1.7442	1.7470	1.7499	1.7527
.91	1.7556	1.7584	1.7612	1.7641	1.7669	1.7697	1.7726	1.7754	1.7752	1.7810
.92	1.7839	1.7867	1.7895	1.7923	1.7951	1.7980	1.8008	1.8036	1.8064	1.8092
.93	1.8120	1.8148	1.8176	1.8204	1.8232	1.8260	1.8287	1.8315	1.8343	1.8371
.94	1.8399	1.8427	1.8454	1.8482	1.8510	1.8537	1.8565	1.8593	1.8620	1.8648
.95	1.8675	1.8703	1.8730	1.8757	1.8785	1.8812	1.8840	1.8867	1.8894	1.8921
.96	1.8649	1.8976	1.9003	1.9030	1.9057	1.9084	1.9111	1.9138	1.9165	1.9192
.97	1.9219	1.9245	1.9272	1.9299	1.9325	1.9352	1.9379	1.9405	1.9442	1.9458
.98	1.9484	1.9511	1.9538	1.9563	1.9589	1.9616	1.9642	1.9668	1.9694	1.9720
.99	1.9745	1.9771	1.9797	1.9823	1.9848	1.9874	1.9899	1.9924	1.9950	1.9975
1.00	2.0000									

TABLE—II

Sl. No. (1)	Description (2)	Diameter of notch (3)	Height of crest above floor level (4)	Remarks (5)
1.	Free flow studies		P = 0″	
			= 1″	
	Semi-circular notch	2′0″	= 2″	
			= 3″	
			= 4″	
			= 5″	
2.	Free flow studies		P = 1″	
			= 2″	
	Semi-circular notch	1′6″	= 3″	
			= 4″	
			= 5″	
			= 6″	
			= 7″	
			= 8″	
3.	Free flow studies		P = 1″	
			= 2″	
	Semi-circular notch	1′3″	= 3″	
			= 4″	
			= 5″	
			= 6″	
			= 7″	
			= 8″	
4.	Free flow studies		P = 0″	
			= 1″	
	Semi-circular notch	1′0″	= 2″	
			= 3″	
			= 4″	
			= 5″	
			= 6″	
			= 7″	
			= 8″	
5.	Submerged flow studies Semi-circular notch	1′0″	P = 1″	



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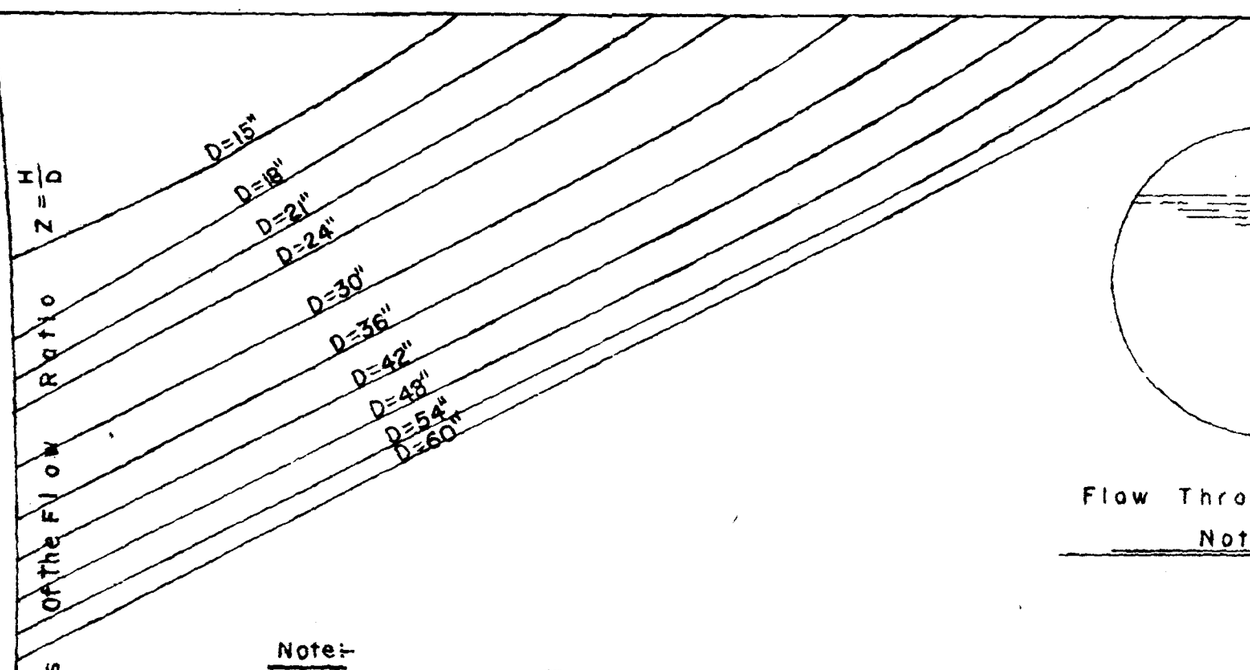

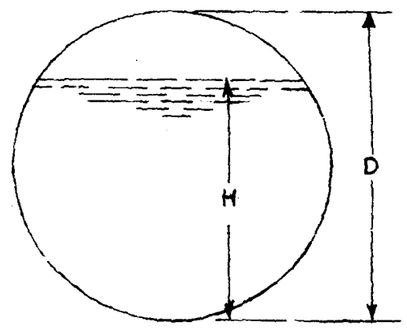

Flow Through A Circular Notch

Note:-

$$Q = \phi\ (Z)\ D^{5/2}$$

$$\text{Where } \phi(Z) = \frac{2}{3}\sqrt{2g}.\ \pi\left[1/8 Z^2 - 1/32 Z^3 \cdot 5/1024 Z^4\right]$$

$$Z = H/D$$

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

Theoretical Discharge Chart For Circular Notches By The ϕ Function — Chart - 2

192/59			
Drawing	N. Rajaratnam		
	J. E	A. R. O	E. E. R
	Trd: J.V	Ckd:	Fig. 2

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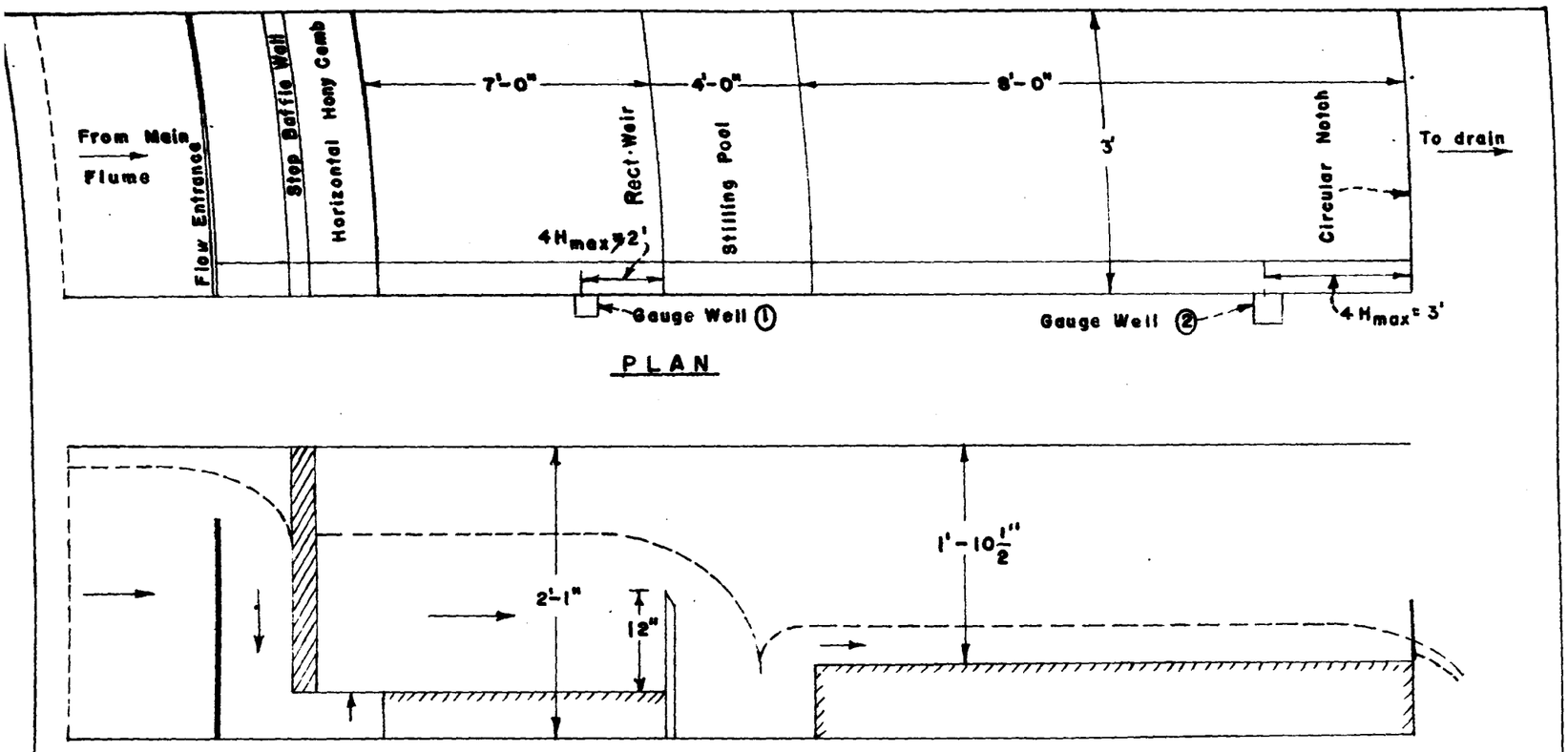

SECTIONAL VIEW OF MODEL SET UP

(Not To Scale)

Report	Public Work Department – Madras Irrigation Research Station – Poondi		
File	FLOW THROUGH CIRCULAR NOTCHES MODEL SET UP		
193/59 Drawing	N. Rajaratnam	U.V. Sadashiva	P.K. Kandaswamy
	J · E · R	A · R · O	E · E · R
	Trd: V·V·	ckd:	FIG···· 3

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Item No	Drg. No

Reg. No 24 -E'62 (Saka Era 1882)—375.

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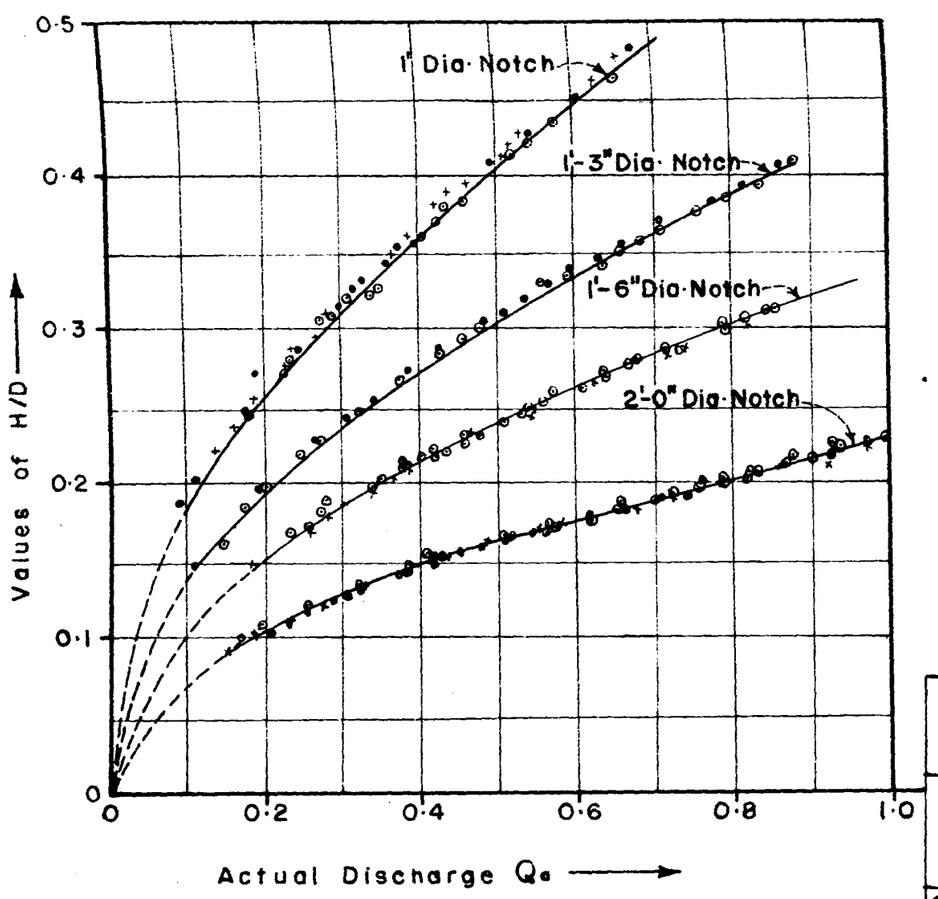
Item No	Drg. No



Report	Public Works Department – Madras Irrigation Research Station – Poondi		
File	FLOW THROUGH CIRCULAR NOTCHES PLOTTING OF Qa With H/D		
194/59 Drawing	N. Rajaratnam	U.V. Balakrishna	K. Ramaswamy
	J. E. R	A. R. O	E. E. R
	Trd: V.V.	chd:	FIG ..... 4.

Reg. No. 25 – E'62 (Saka Era 1882) – 375

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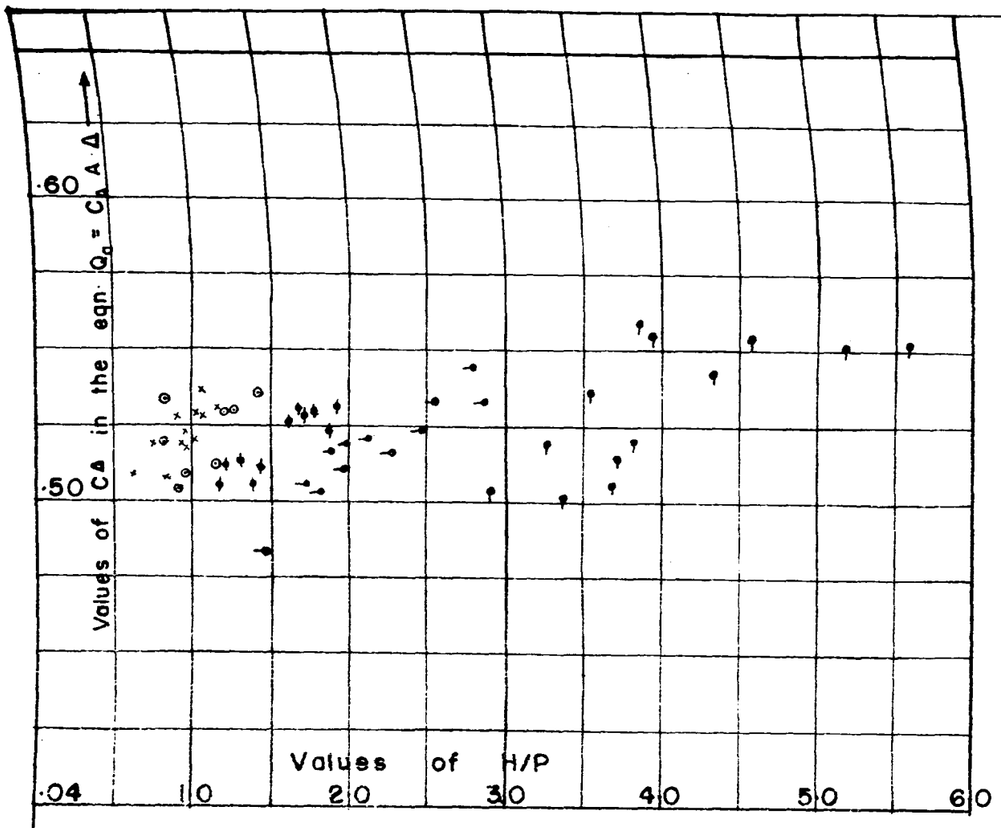

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LEGEND:

1'-0" Dia. Notch		P = 1"
"		P = 2"
"		P = 3"
"		P = 4"
"		P = 5"

Public Works Department - Madras
Irrigation Research Station - Poondi

FLOW THROUGH CIRCULAR NOTCHES
VARIATION OF C_A With H/P

Report			
File			
195/69 Drawing	N. Rajaratnam		
	J. E. R	A. R. O	E. E. R
	Trd: V.V.	ckd:	FIG. 5

REG NO. 26 E'62 (SAKA ERA 1883)-325
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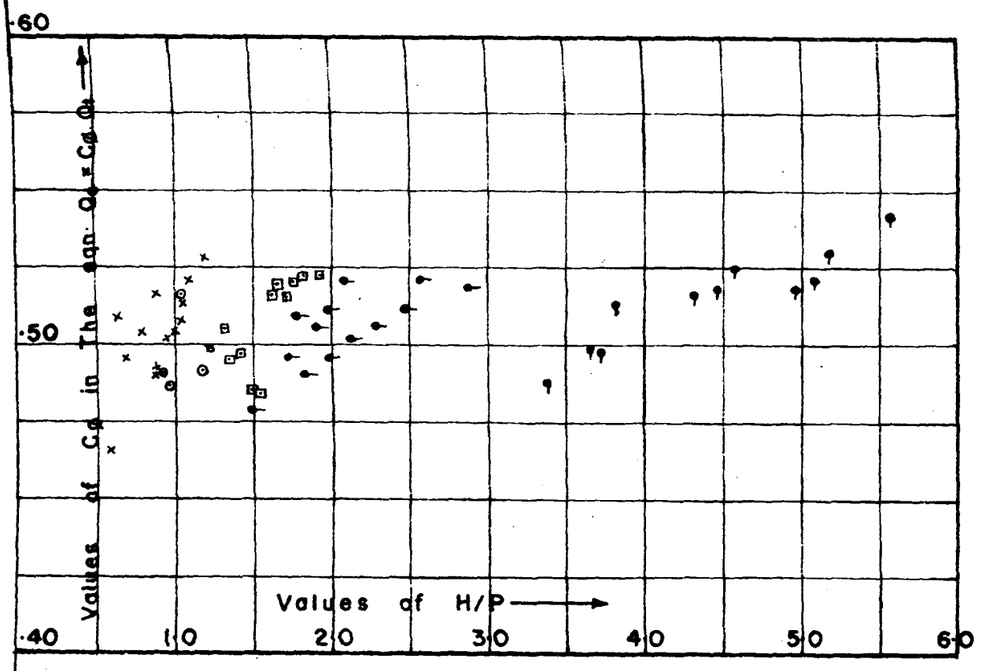

LEGEND:

●	1'-0" Dia.	P = 1"
●	"	P = 2"
⊡	"	P = 3"
⊙	"	P = 4"
×	"	P = 5"

Report	Public Works Department – Madras Irrigation Research Station – Poondi		
File	FLOW THROUGH CIRCULAR NOTCHES VARIATION OF Cø WITH H/P		
196/59 Drawing	N. Rajaratnam.	U.V. Sadasivam	B.K. Ramaswamy
	J. E. R	A. R. O	E. E. R
	Trd: V.V.	ckd:	FIG..... 6

REG NO 27 E'62 (SAKA ERA 1883)-375

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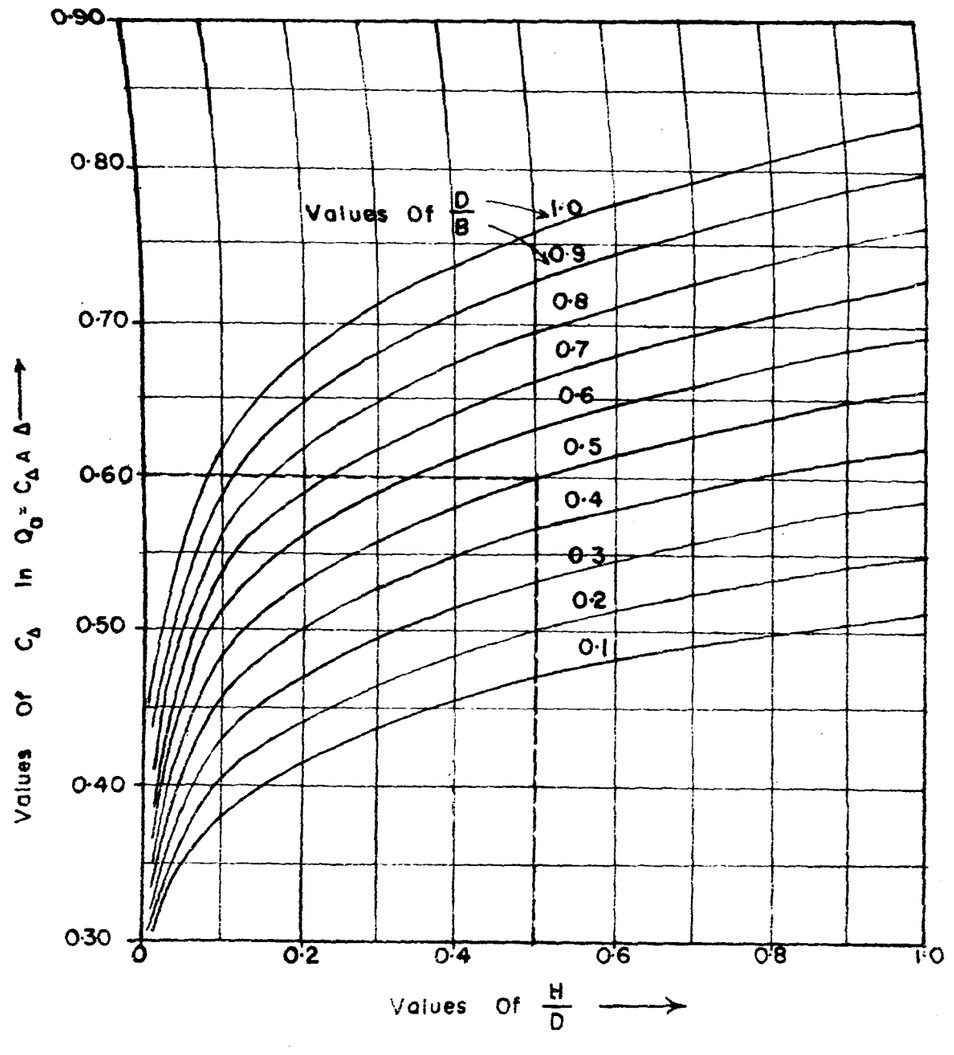
Item No	Drg No



199/59	Report	Public Works Department — Madras. Irrigation Research Station — Poondi.		
	File	FLOW THROUGH CIRCULAR NOTCHES. SIMPLIFIED CHART FOR C_Δ FOR STEVENS SOLUTION.		
	Drawing	N. Rajaratnam	U.N. [illegible]	[illegible]
		J·E·R	A·R·O	E·E·R
		Trd: T.K.S	Ckd:	FIG 7

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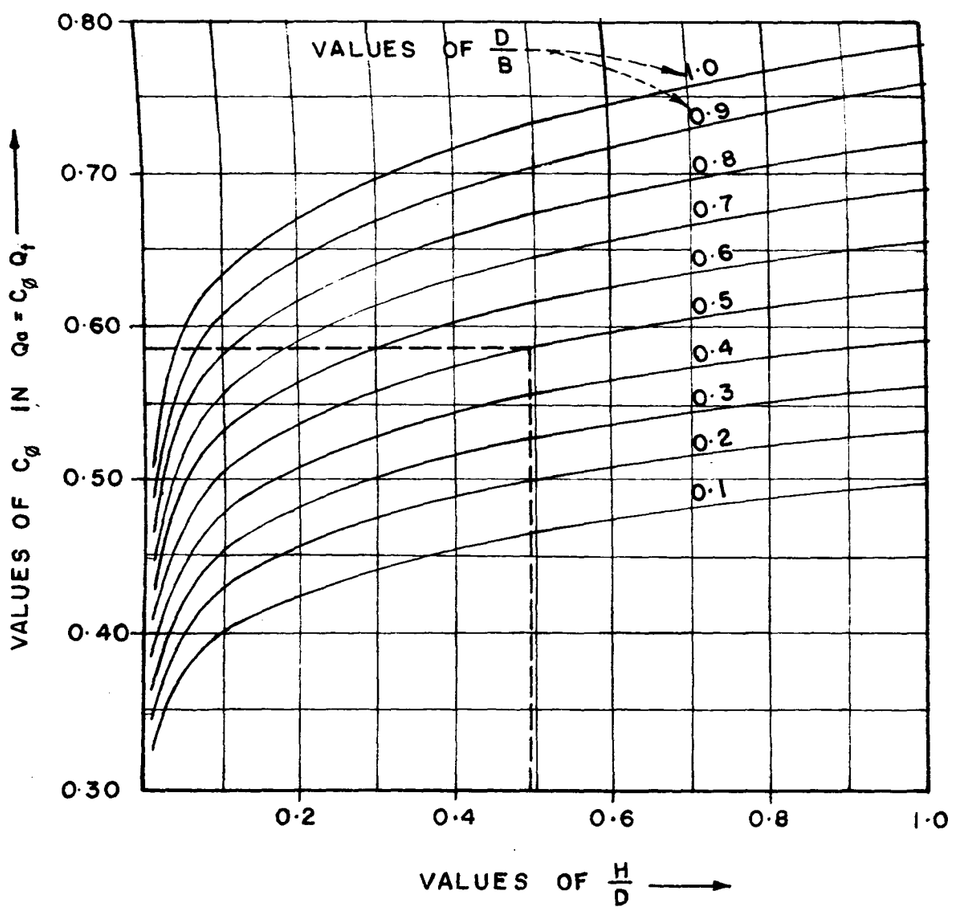

Report	Public Works Department — Madras Irrigation Research Station - Poondi.		
File	FLOW THROUGH CIRCULAR NOTCHES SIMPLIFIED CHART FOR Cø FOR Ø FUNCTION		
202/59 Drawing	N. Rajaratnam	G.V. [illegible]	[illegible]
	J · E · R	A · R · O	E · E · R
	Trd: V·V·	ckd:	FIG ···· 8

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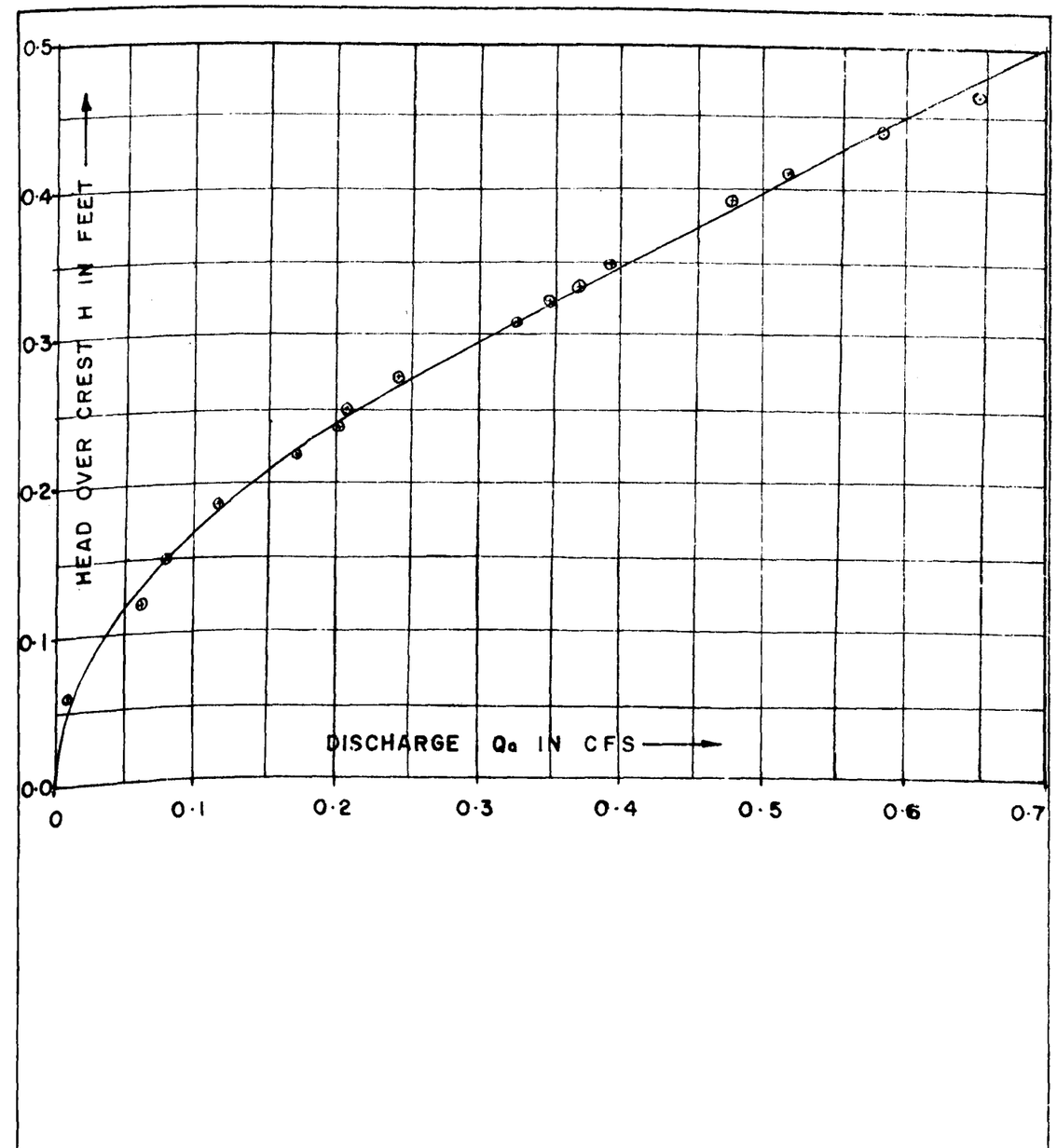

Dia· of Notch = 1'–0"

Report	Public Works Department–Madras Irrigation Research Station–Poondi		
File	FLOW THROUGH CIRCULAR NOTCHES STUDIES FOR SUBMERGENCE		
203/59 Drawing	N·Rajaratnam	U.V. Sadakwil	B.K. Rao Sarma
	J·E·R	A·R·O	E·E·R
	Trd: T·K·S	ckd:	FIG··· 9·

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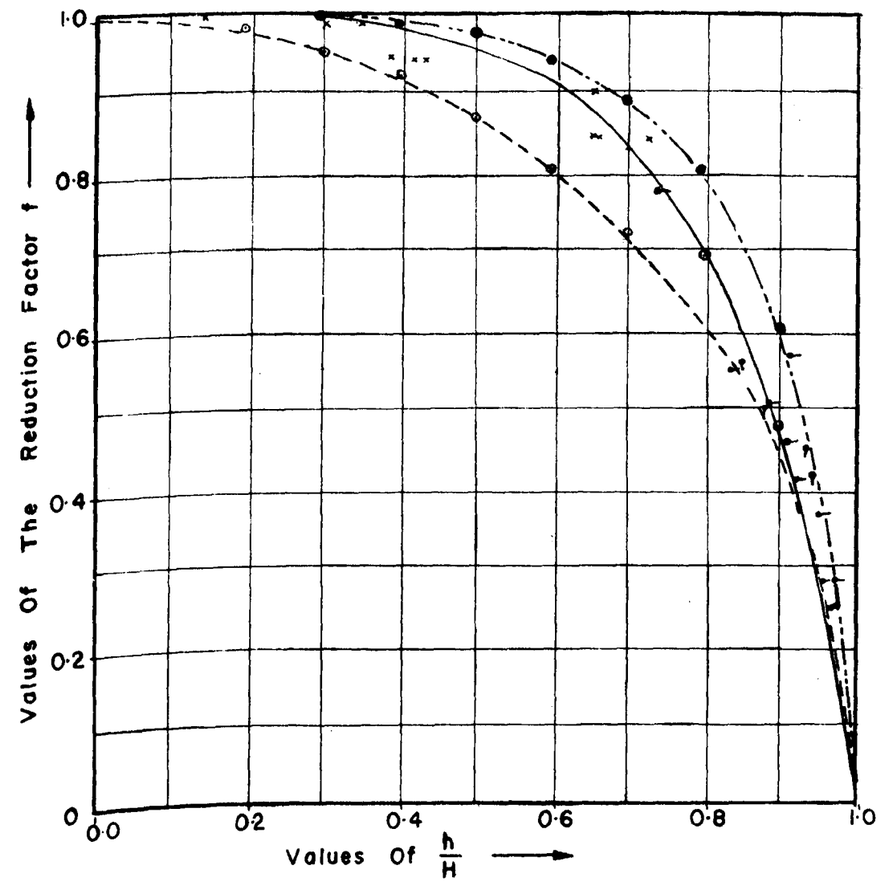

LEGEND

- Villemontes Curve
- Poondi Curve
- V— Notch Curve

Report	Public Works Department—Madras. Irrigation Research Station—Poondi.		
File	FLOW THROUGH CIRCULAR NOTCHES REDUCTION FACTOR CHART		
Drawing 204/59	N. Rajaratnam		
	J·E·R	A·R·O	E·E·R
	Trd: T.K.S	Ckd:	FIG.... 10.

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BYE-WASHES

INTRODUCTION

A bye-wash may be defined as a flush escape provided in the irrigation tanks for surplussing the floods. It was proposed to evolve a rational formula for the design of bye-washes with the experimental process. This report is in continuation of the previous report published as item 3 in the Annual Report for the year 1958.

THEORY

A bye-wash may be taken as a broad crested weir with certain upstream and downstream slopes. In the case of the broad crested weir the discharge formula is expressed as

$$Q = C\,B\,H^{3/2}$$

where Q = discharge in Cft.

C = co-efficient of discharge

B = length of weir normal to the direction of flow

H = head in feet over the crest.

In the free flow of a broad crested weir within the modular range the co-efficient c varies mainly with H/L where L is the width of the crest, and is independent of the ratio H/P where P is the height of the crest above the approaching floor. In the case of a bye-wash the upstream and downstream slopes are very flat. Hence the co-efficient 'c' may vary with the ratio H/P.

In order to analyse the discharge formula various factors depending on the shape of the bye-wash and other physical factors have to be considered. As already explained in the Annual Report for 1958,

$$Q = f\,(B, S_u, L, P, S_d, H, r, \quad , n)$$

where B, L, P, H have been defined already

S_u = upstream slope

S_d = downstream slope

r = fluid weight per unit volume

= mass density

n = co-efficient of rugosity

By dimensional analysis it can be shown that

$$Q = \phi(F, R, S_u, S_d, H/P, H/L, n)$$

where F = Froude's number

R = Reynold's number

This report presents the results of the experiments conducted to study the variation of the discharge co-efficient 'c' with H/P for different upstream slopes (S_u) and for different crest widths of bye-wash (L).

MODEL SET-UP AND EXPERIMENTS

Three masonry flumes each 3′ 0″ wide and 26′ 0″ in length were constructed with separate discharge measuring devices (V-notches) for each flume. The models were constructed to a scale 1/4 to study the complete range of photo-type dimensions (Figure-1).

Experiments were conducted for bye-washes having crest widths of 1′0″, 0′6″ and 0″ and in each case the upstream slope was varied from 1/100 to 1/1000, the down-stream slope of 1150 being constant. Discharge over the bye-wash was measured by the V-notch and in each case the value of the co-efficient of discharge was found from the formula $C = \frac{Q}{B H^{3/2}}$. The value of H/P was varied for a fixed upstream and downstream slope and I0th of the crest by varying the head over the crest(H). The variation of H/P values with the discharge co-efficient was calculated and is shown plotted in the figures 2 to 6.

DISCUSSION OF RESULTS

From the graphs plotted connecting C and H/P, it is seen that as the value of H/P increased the co-efficient of discharge 'C' also increases. The value of 'C' adproximately ranges from 2.8 to 4.

The increase in the value of 'C' with H/P is steeper in the case of steep slopes that in the flat slopes. This indicates that for flat slopes, the value of H/P should be greater than for steep slopes, for a certain discharge co-efficient.

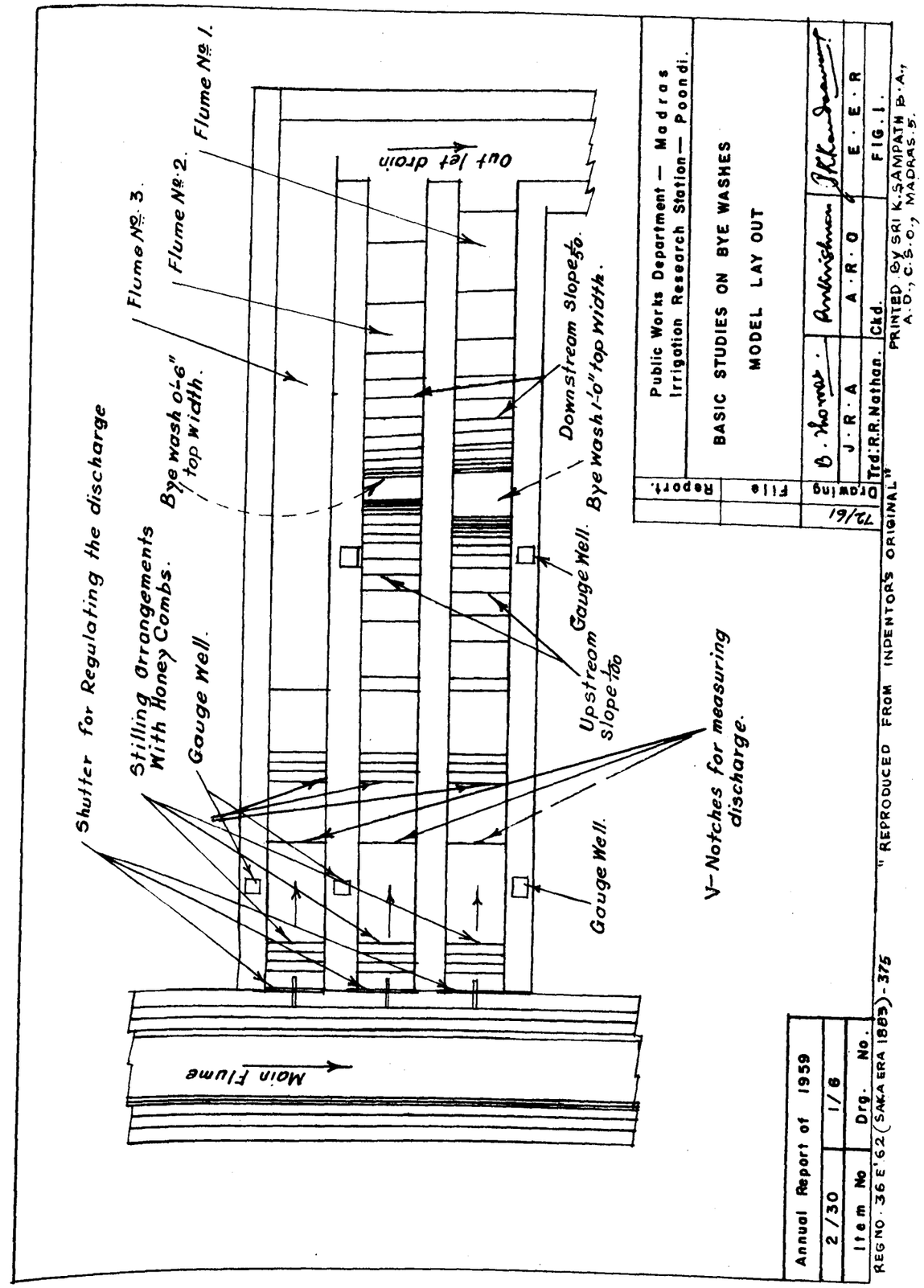
In Figure-7 the variation 'C' against H/P values are shown for different crest width of the bye-washes, the other dimensions remaining the same. In this case it is found that for larger crest with the co-efficient of discharge is more for a particular value of H/P.

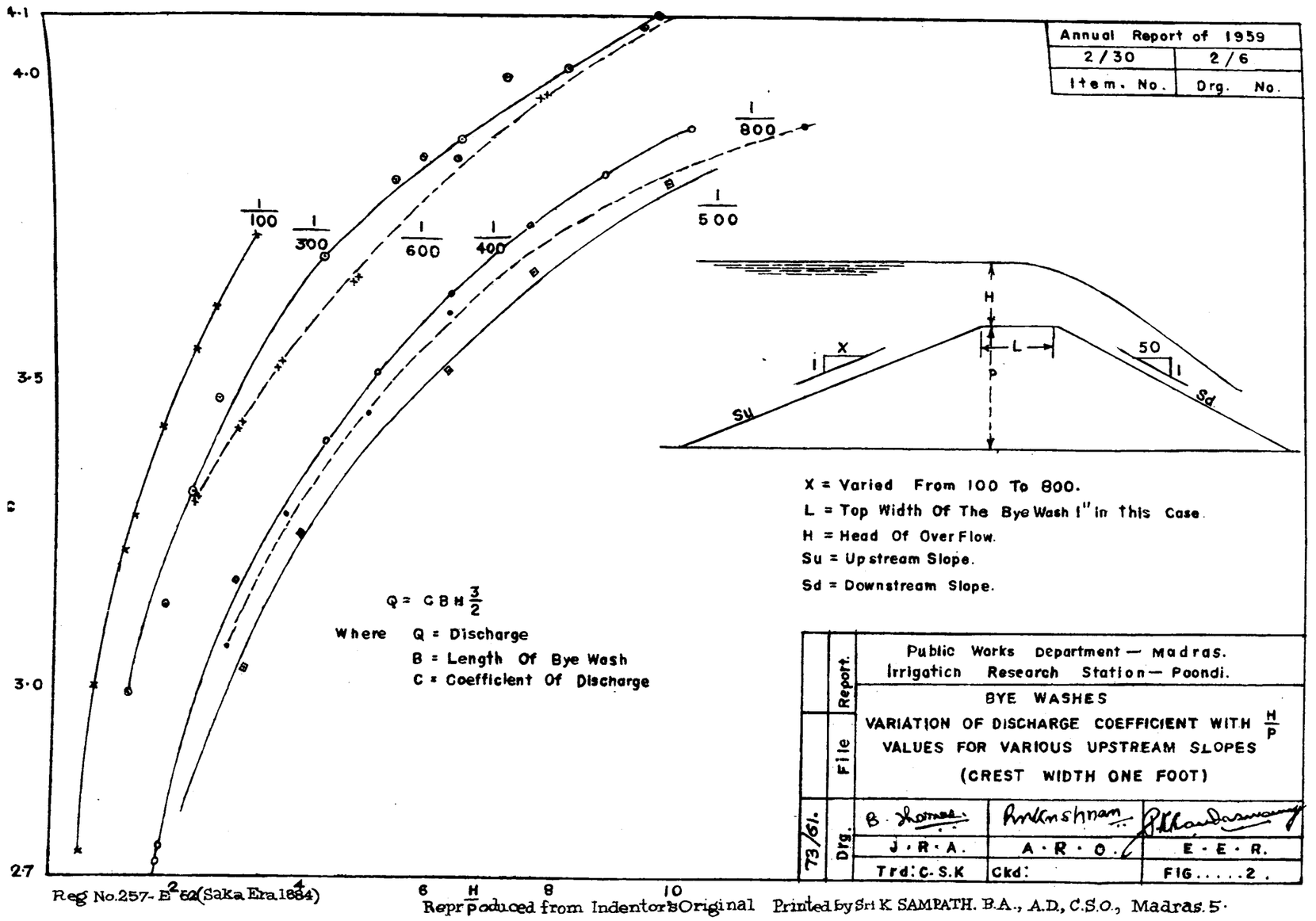
A typical flow profile for the triangular-sectioned bye-wash (crest width = 0) is shown in Figure-8. There is a sudden fall in the profile of the flow in the vartical plane of the apex.

CONCLUSION

From the results presented in this report it can be seen that the co-efficient of discharge depends on the upstream slope of the bye-wash. So, a correction factor for the influence of the upstream slope on the co-efficient of discharge is absolutely essential to the discharge formula. This factor can be determined by further analysis of the data. Similar corrector factors could be determined for the influence of the crest width, downstream slope, etc., on the co-efficient of discharge by further experiments.

The charts furnished in the report will be great use in the design of a bye-wash, because the correct co-efficient of discharge of a bye-wash the other factors being known can be found out from the chart.





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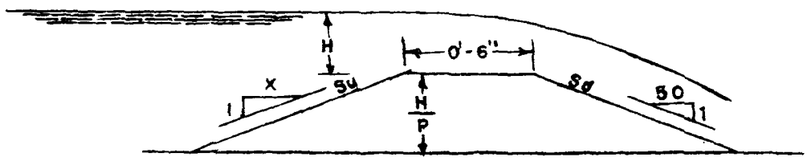

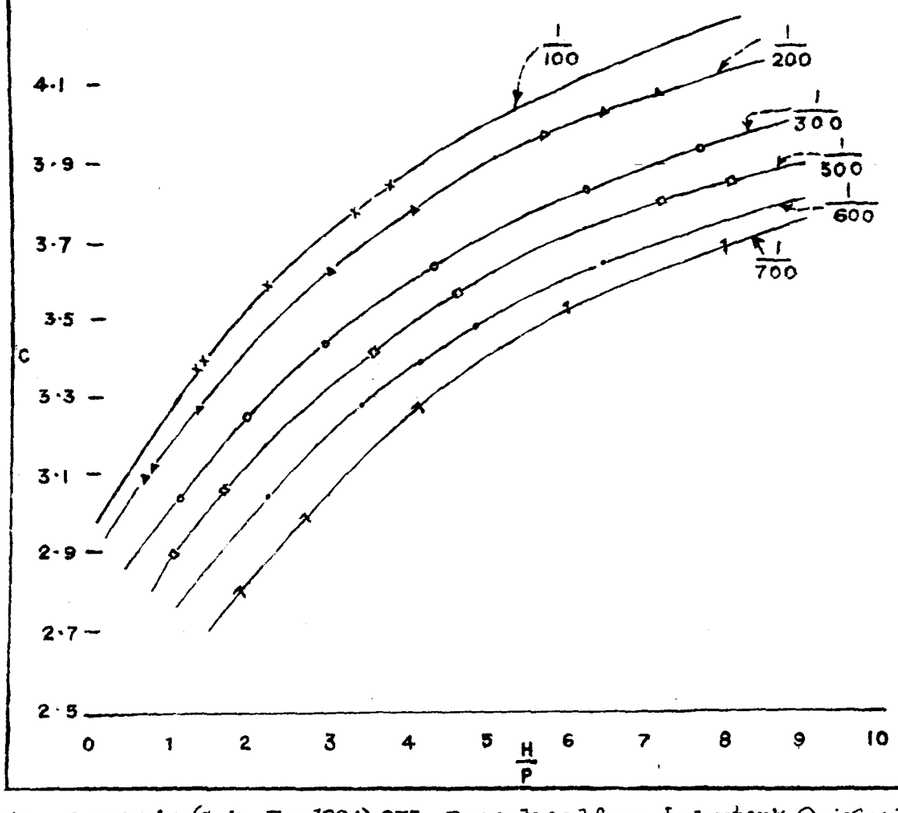

L = Top Width Of Bye Wash (0'-6" In This Case.)

l = Height Of Bye Wash

H = Head Of Over Flow.

Su = Upstream Slope

Sd = Downstream Slope

X = Varied From 100 To 700

Report.	PUBLIC WORKS DEPARTMENT—MADRAS. IRRIGATION RESEARCH STATION—POONDI.		
File	BYE WASHES VARIATION OF DISCHARGE COEFFICIENT WITH $\frac{H}{P}$ VALUES FOR DIFFERENT UPSTREAM SLOPES. ($\frac{1}{2}$' CREST WIDTH)		
74/61 Drg.	B. Thomas	Krishnan	Chandramani
	J·R·A	A·R·O	E·E·R.
	Trd: C.S.K.	Ckd:	FIG. - - - 3

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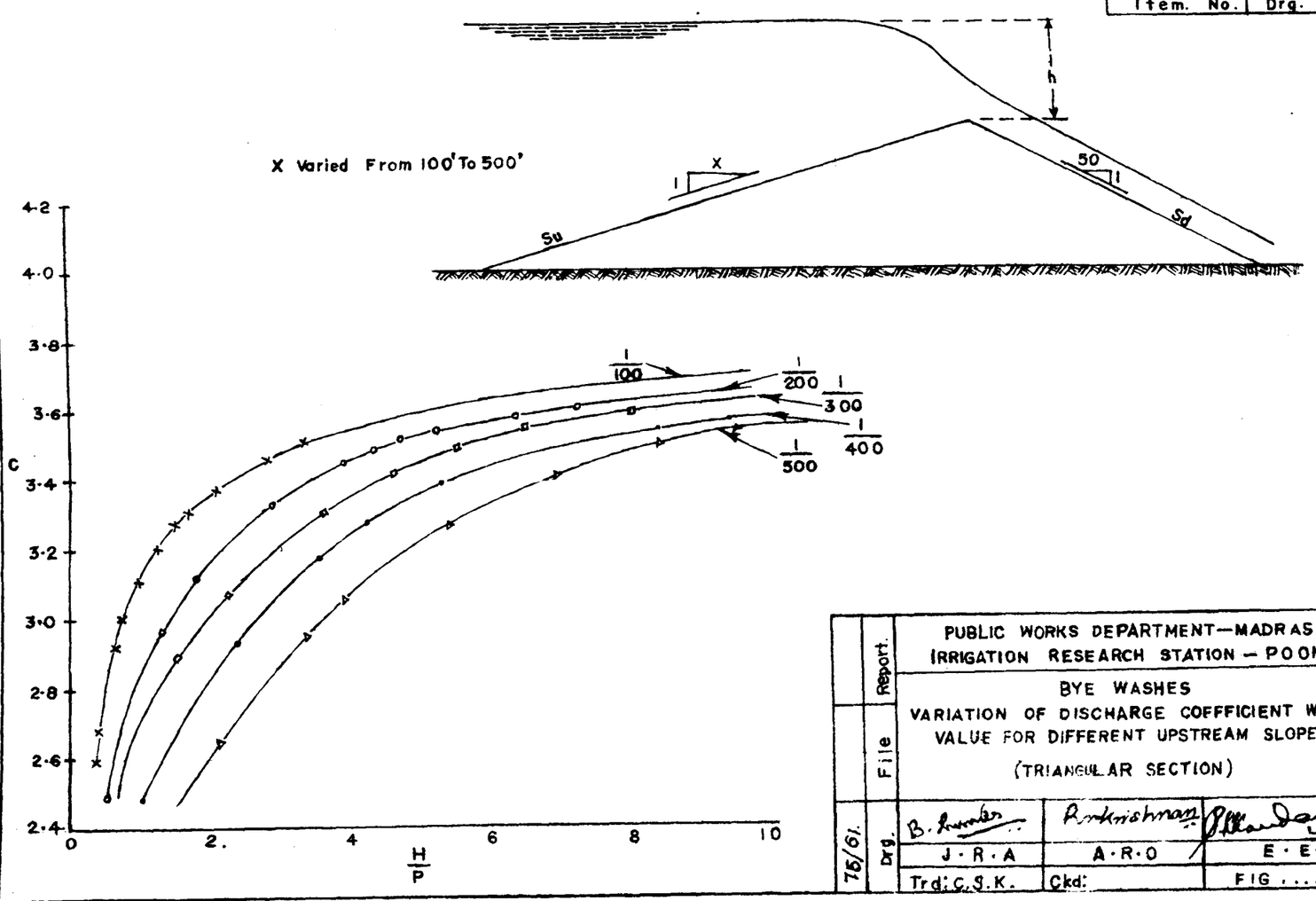

Report.	PUBLIC WORKS DEPARTMENT—MADRAS IRRIGATION RESEARCH STATION — POONDI.		
File	BYE WASHES VARIATION OF DISCHARGE COEFFICIENT WITH H/P VALUE FOR DIFFERENT UPSTREAM SLOPES (TRIANGULAR SECTION)		
75/61. Drg.	B. Ramaiah	P. Krishnan	P. Ramaiah
	J·R·A	A·R·O	E·E·R.
	Trd: C.S.K.	Ckd:	FIG ....4..

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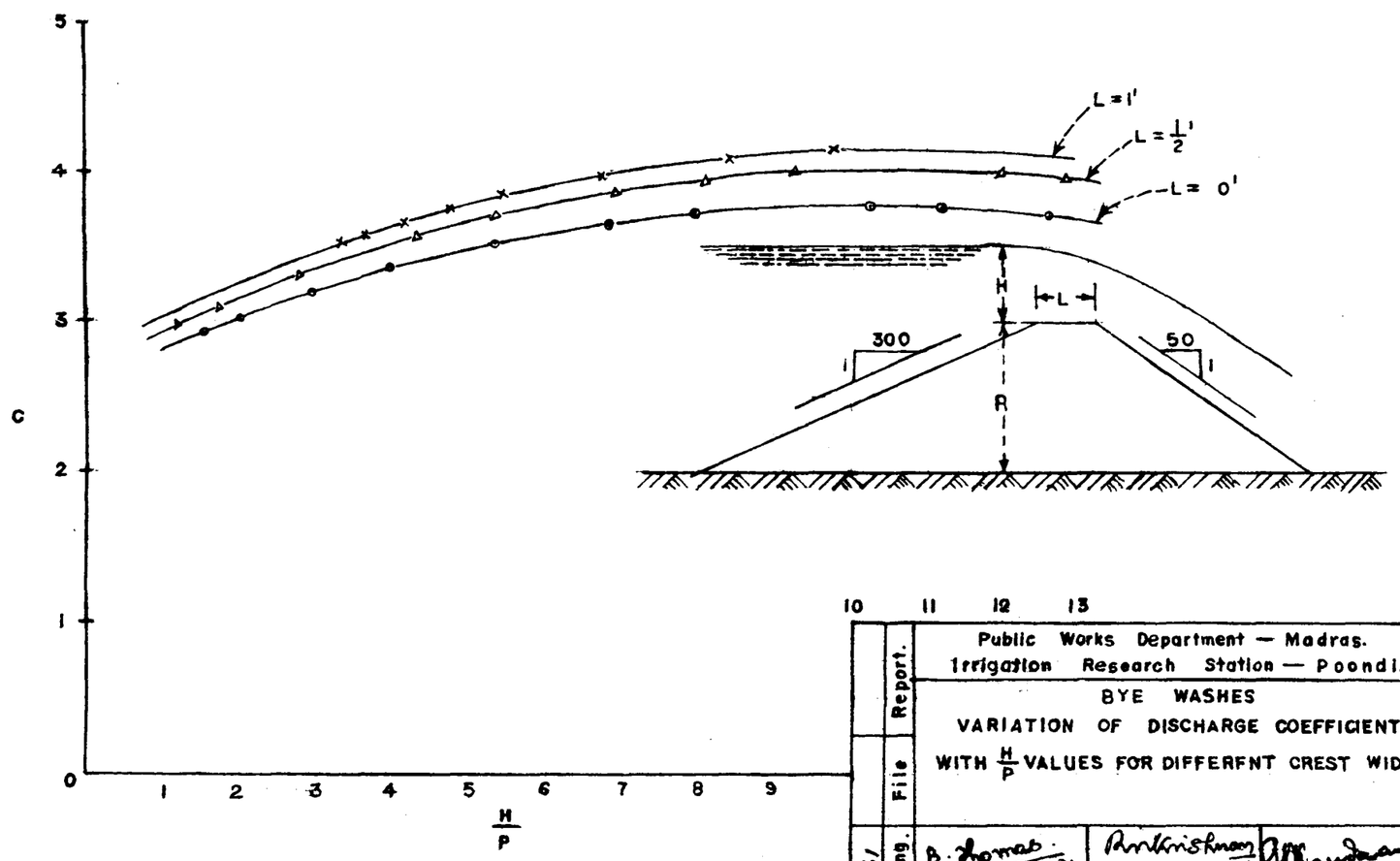

Report.	Public Works Department — Madras. Irrigation Research Station — Poondi.		
File	BYE WASHES VARIATION OF DISCHARGE COEFFICIENTS WITH $\frac{H}{P}$ VALUES FOR DIFFERENT CREST WIDTHS.		
76/61 Drawing.	B. Thomas	Ponkrishnan	
	J · R · A	A · R · O	E · E · R
	Trd: C.S.K. Pathy	Ckd:	FIG. . . . 5.

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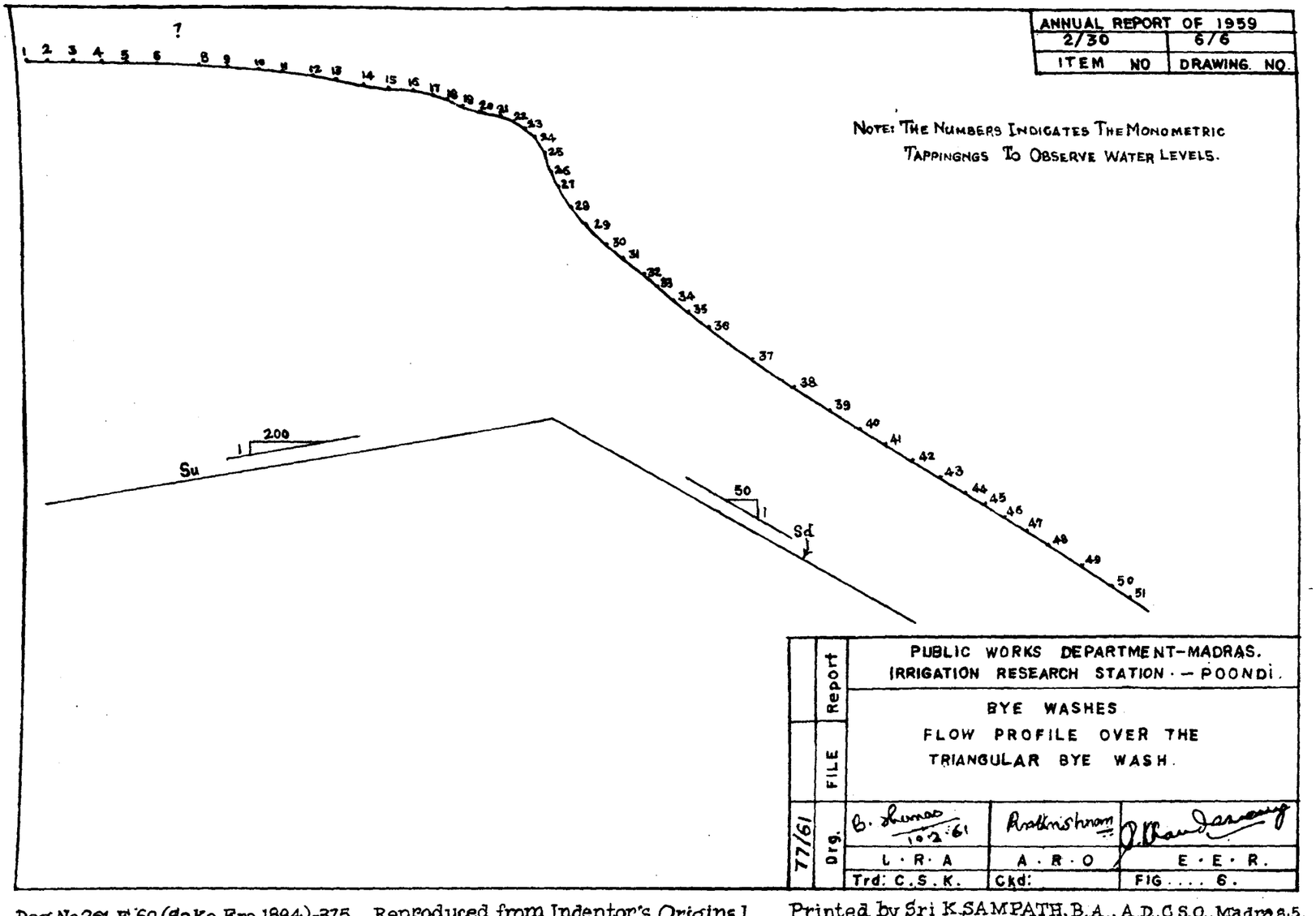

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FLOW THROUCH OPEN CHANNEL OBSTRUCTION

Introduction:

The investigation of the flow through open channel obstructions was proposed to be made in order to study the important significances of this phenomenon namely the mechanism of the scour produced in the stream bed due to the creation of high and erdsive velocities near the piers, the afflux that can be produced at the time of heavy floods passing through the piers and the estimation of the flood discharge but the heading up near the piers.

Theory

The flow through the bridge vent was normally analysed by the orifice concept or the orifice weir concept. But if the contraction is large such as to cause critical depth and the standing wave then the discharge can be computed by means of broad crested weir formula in the modular range. When the submergence ratio cxceeds what is known as modularity limit a reduction factor can be found out to find out the correct discharge.

Scheme of studies :

The construction of piers in the flow through open channels provides obstruction to the flow. The obstruction percentage to the flow is denoted by the symbol 'm' and defind as

$$m \equiv \frac{(1-b)}{B} \; 100$$

$$\equiv \frac{(B-b)}{B} \; 100$$

$$\equiv \frac{t}{B} \times 100$$

where B — original bed width of the river

b — contracted bed width

t — total thickness of the piers.

In the first stage of work a horizontal paved bed was assumed. For different obstruction ratios the variation of $\frac{Y1}{Yn}$ and $\frac{H3}{Yn}$ with Fn has to be studied (Fig.1).

where Y_3 - Y_n - the normal depth of flow. This Y_n has to be measured further downstream away from the region of eddies created by the tail of the piers.

Y_1. This is the headed up depth. This should be measured at a distance not less than the wicth of a simple opening from the nose of the piers on the upstream side.

Y_2. Thid is the depth of flow at the section of the maximum contraction. This may be measured along the pier length or a little further downstream.

H_3. - Y_1 - Y_n or Y_1 - Y_3 and this is generally known as the afflux, caused by the construction of the bridge. F_n is the Fraude's number computed for tht normal, section

Model Construction:

A masonry flume about 100' long was constructed and rails were laid in the side walls for a trolly fitted with gauge-angle to take observation of water levels in the flume. (A rectangular notch 1'2" high and 3'0" long was fixed in the flume (Fig.2). A tail gate of screw gearing type was constructed in the flume to maintaion different tail waters. The piers were made in teak wood, having a central piece with a constant length of about four times the thickness. To this the nose and tail can be attached by means of screws. For having two spans there was one full central pier and two half piers. A certain discharge measured with a weir on the upstream side was sent in. Suitable stilling arrangements and sufficient length of the flume were provided to establish the normal velocity distribution.

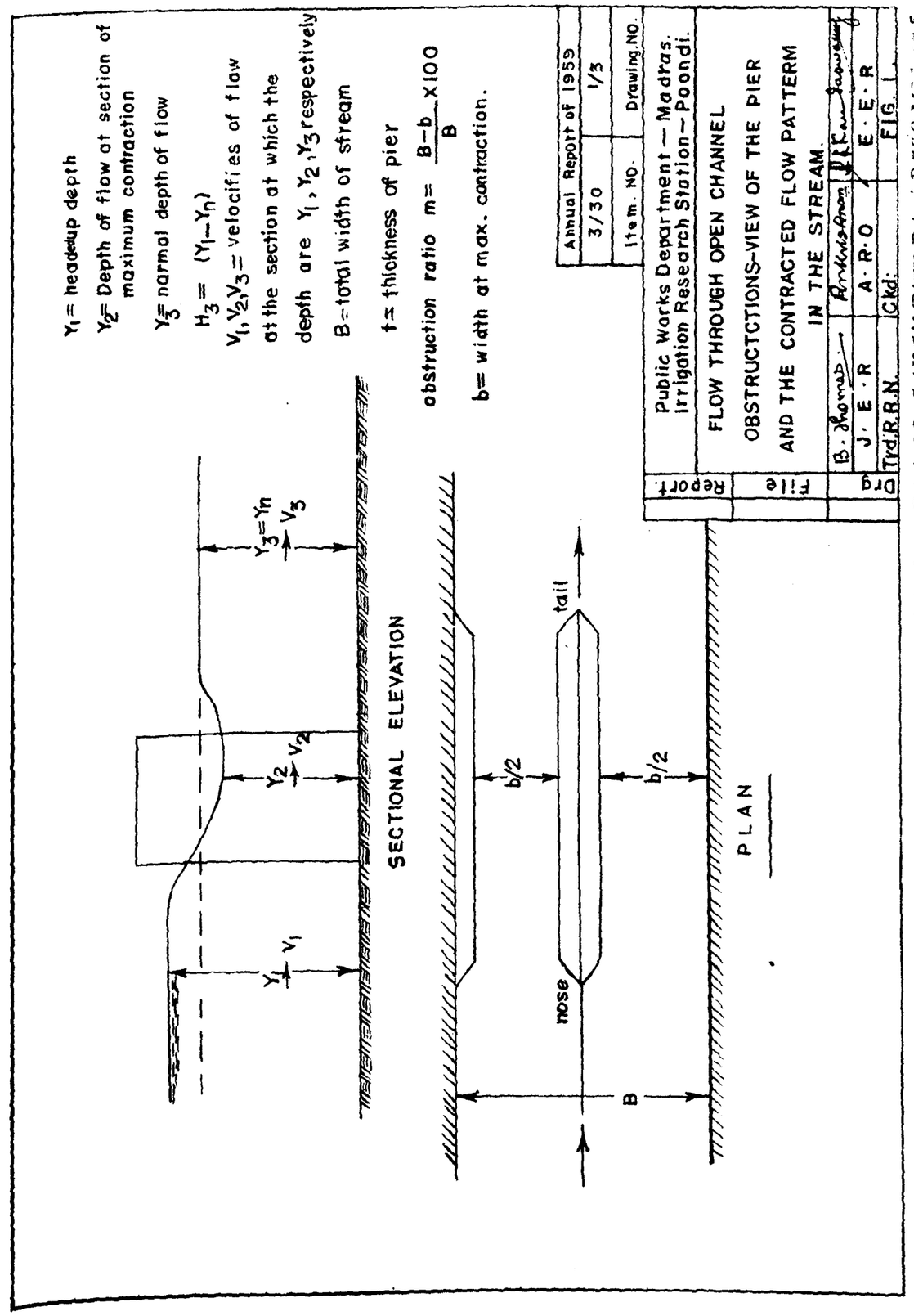
2. Experiments and results:

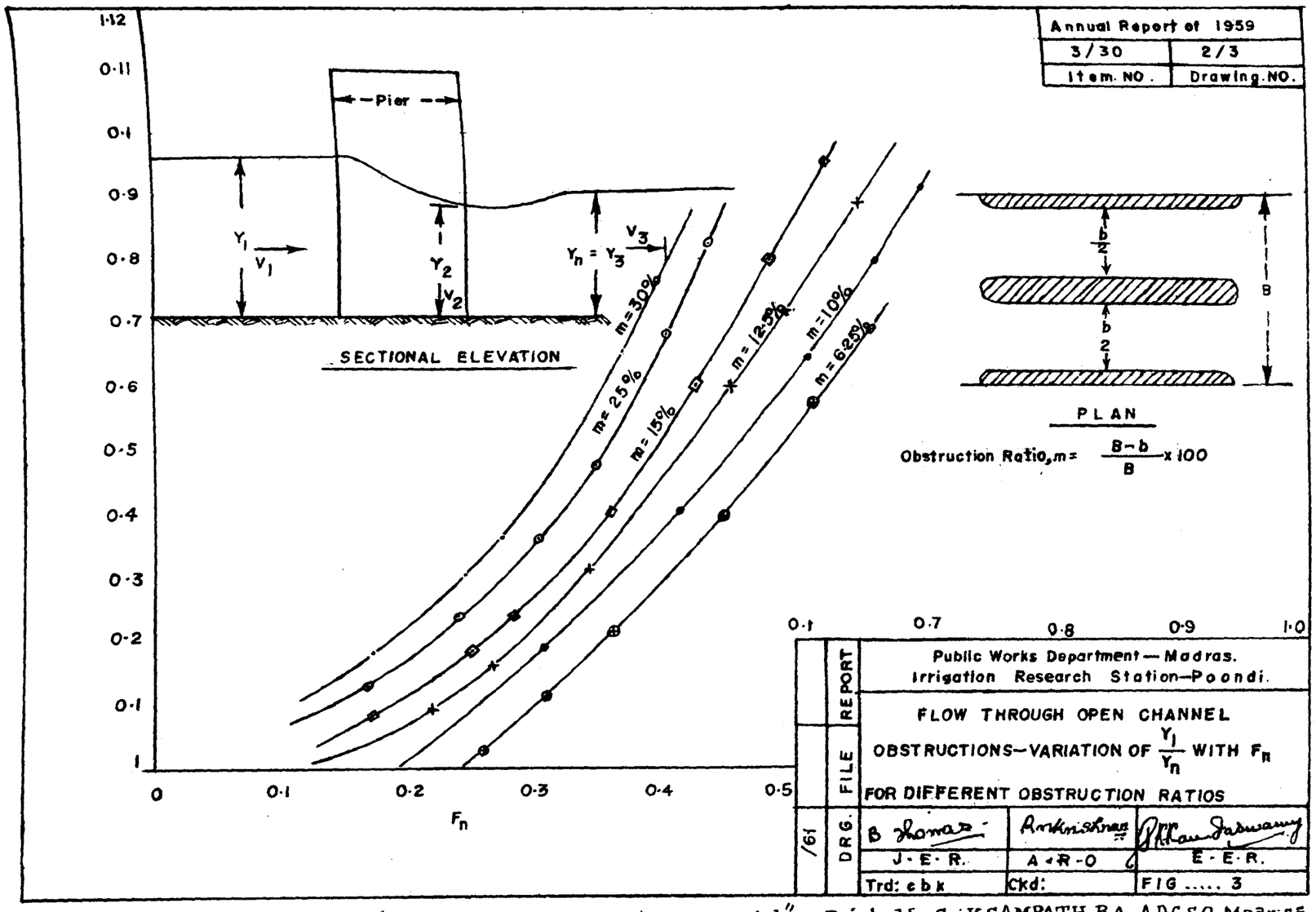
Experimehts were carred out for different obstruction ratios, and for diferent discharges. in each case finding out the factors Y_1, Y_2, and Y_3. The nose and tail of the piers used were semi circular in shape. The variation of Y_1/Y_n with F_n for different/'m' values are shown plotted in Fig.3.

It was found that for a particular obstruction ratio, the ratio Y_1/Y_n increases with the increase in Froude's number. For a fixed Froude's number the value of Y_1/Y_n increases with the increase of the obstruction ratio. The chart is useful in predicting the discharge through, the bridge openings once the obstruction ratio and afflux due to the bridge piers are known.

Further Studies:

Studies are to be conducted with the rectangular and equiangular piers for different obstruction ratios. The mechanism of scour produced in a mobile bed model has to be investiaged, taking into account the time and size of the sand represented by its settling velocity. The effect of the boundary in the production of the scour will be studied with mobile boundary models.





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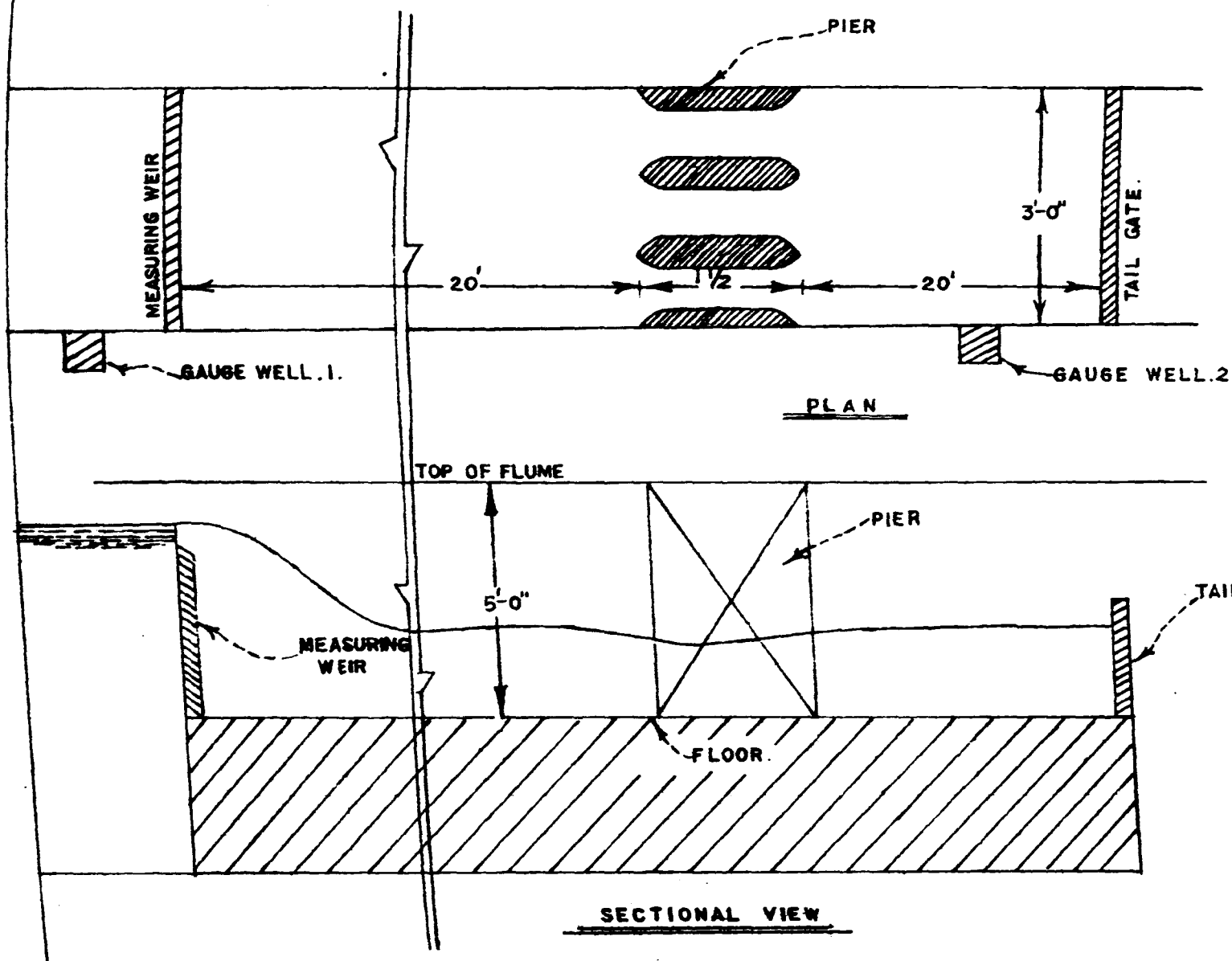
Item. NO.	Drawing. NO.



NOT TO SCALE

Report	Public Works Department — Madras Irrigation Research Station — Poondi		
File	FLOW THROUGH OPEN CHANNEL OBSTRUCTIONS PLAN & SECTIONAL VIEW OF THE MODEL		
Drawing	B. Thomas	Ranganathan	P.K. Ramadoss
	J · E · R	A · R · O	E · E · R
	Trd: R. Ranganathan.	Ckd.	FIG 2.

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AMARAVATHY SPILLWAY

Discharge Characteristics

1. SYNOPSIS

Studies were conducted to obtain the spillway rating curves for the Amaravathy Dam by means of scale model tests. Experiments were conducted for both controlled and uncontrolled flows, The model set up has been briefly described. The procedure followed in conducting the experiments had also been presented. Results obtained both for free flow and gate controlled flow have been discussed. Optimum gate-operation charts to pass floods over the spillway on the basis of least consumption of power have been developed for different water levels in the reservoir. A working chart has also been evolved so as to find out the amount of gate opening required with nine shutter operation to pass the flood knowing the rate of rise of water level in the reservoir,

2. INTRODUCTION

A dam has been constructed across the river Amaravathy, an important tributary to the river Cauvery at a place about 13 miles west of Udumalpet in Coimbatore District. A spillway 324 feet long is provided in the dam to surplus the flood waters. Vertical lift shutters are provided on the spillway to store up water to the maximum water level of the reservoir. It is to be noted that the shutters are located three feet downstream of the crest of the spillway. This report deals with the studies conducted to determine the discharge characteristics of the spillway for both controlled and uncontrolled flows.

3. DATA

Hydraulic Particulars

a)	Crest of spillway ...	El. 1150,00
b)	Maximum water level ...	El. 1175.00
c)	No. of vents in spillway	9 Nos.
d)	Sive of each vent ...	36' x 25'
e)	Maximum flood discharge	1,40,000 cusecs
f)	Pier thickness	9 feet

Figure-1 shows the details of the spillway and gate location.

4. MODEL SET UP

The three dimensional model of the Amaravathy spillway which had already been constructed to a scale of 1/60 was made use of for the present studies. The discharges over the spillway were measured over a V-notch fixed in the leading flume. Suitable baffle arrangement was provided to obtain a uniform distribution of flow in the spillway approach channel. The heads over the crest of spillway and sill of V-notch were measured by means of point gauges reading correct to 0.001 foot. The piers and the spillway shutters were made of teakwood and painted to avoid swelling of the teakwood. The shutters were opened to the correct amount by means of teakwood wedge blocks. The details of the plan of the model set up is given in Figure-2.

5. EXPERIMENTS

5.1. In the first place, experiments were conducted for conditions of free flow over the spillway. The central vent alone was kept open, all the other vents being closed with the teakwood shutters. The discharges over the spillway for various overflow heads were observed. The experiment was done for the following symmetrical operation :

(1) Central one vent open

(2) Central three vents open

(3) Central five vents open

(4) Central seven vents open

(5) All nine vents open.

5.2. Secondly, experiments were conducted to study gate controlled flow over the spillway, maintaing M.W.L. in the reservoir. Keeping all the other shutters closed, the central shutter alone was operated to different gate-openings. The discharges over the spillway were observed for each gate opening. Care was taken to see that M.W.L. was maintained in the reservoir. The above set of experiments was repeated for all the five symmetrical gate-operations mentioned before.

5.3. Thirdly, experiments to study gate controlled flow were reported keeping different water levels upstream in the reservoir for all the five symmetrical gate operations. Water levels corresponding to 22 feet, 20 feet, 15 feet and 10 feet heads over the spillway crest were maintained in the reservoir and observations of discharge for various gate openings recorded

6.1. Free flow

The model date have been reduced by the use of the Commonly-accepted discharge formula for free flow over a spillway crest namely,

$$Q = C L H^{3/2}$$

Q is the discharge in cubic feet per second

C is the discharge coefficient

L is the length of crest in feet

Hiš the head of water over crest in feet

The computed values of 'C'tin the above formula vary with the heads of overflow and the number of vents kept open. In Figure-3, a graph connectiug the discharge coeffleient 'C' and the ratio H/H_D has been drawn, were H is the actual head of overflow ; H_D is design head, (25 feet in this case),

It is seen from the graph that the discharge coefficiert increases firstly with the head of overflow and secondly with the number of vents kept open. For instance, there is percentage increase of 6.11 in the value of 'C' for the nine vent operations over the one for single vent operation for a value of H/HD =1.

Figure-4 shows the variation of the coefficient with the number of vents kept open for overflow heads of 10 feet, 12 feet, 15 feet, 20 feet and 25 feet. The coefficient increases uniformly as more number of vents are opened for any particular overflow head.

6. 2. Gate controlled flow

The discharge over a controlled spilway is given by the equation ;

$$Q \equiv C L (H_1^{3/2} - H_2^{3/2})$$

Q is the discharge in cusecs

C is the discharge coefficient

L is the length of crest in feet

H_1 is the head of water over crest of spillway in feet

H_2 is the head of water measured from gate-bottom in feet

$\equiv H_1 - G$ where G is gate opening in feet,

The discharge coefficient 'C' has been computed for different gate operations for various water levels in the reservoir. Figure-5 represents the variation of the coefficient 'C' with the gate opening (G) for the nine shutter operation at M.W.L. It is seen from the graph that the cofficient decreases with increasing gate opening upto a certain limit beyond which it steadily inereases. At a gate opening of 20 feet the water surface just touches the gate and for all greater openings the flow over the spillway is not controlled by the gate but is a free overflow.

For a particular gate opening, the effect of the number of gates operated on the discharge coefficient can be seen Figure-6. The coefficient increases with increasing number of gates operated for all gate openings. The 15 feet opening gives the minimum discharge cofficient, no matter how many shutters are opened. Figure 7 represents the spillway rating curve for the nine shutter operation, The variation of discharge with the head on crest of spillway for gate openings ranging from 3 feet to 15 feet is represented is

the rating curve. The free-flow curve from whice the gate-controlled flow curves take off gives the discharge when the flow is not controlled by the gate. The point where the controlled flow curve takes off from the free-flow curve gives the head on the spillway at which the flow becomes free for a particular gate opening. The spillway rating curve has been made use of for the preparation of the optimum gate-operation chart given in Figure-15 to pass any known flood.

6.3. Optimum gate operation to pass flood discharges

Figure-8 shows the different comdinations of gate operations to pass a flood of known magnitude at M.W.L. The curves have been drawn with the data collected during the experiments. Optimum gate operation charts have been drawn for the Amaravathy spillway based on the following two criteria:—

(1) Gate openings of less than one foot are to be avoided.

(2) The economy in gate operation increases with increasing number of shutters operated.

In actual practice, the shutters may have to be operated to pass the flood discharges even before M.W.L. is reached. Hence these charts drawn for other water levels in the reservoir have also been included in the present report Figure 9 through 12 represent the optimum gate operation charts for M.W.L. 22 feet, 20 feet, 15 feet and 10 feet heads over crest of spillway.

The area-capacity curve of the Amaravathy reservoir is given in Figure-13. In times of floods, the water level in the reservoir rises, the rate of rise depending upon the net inflow and the waterspread area at the reservoir elevation. The rate of rise

$$\frac{dH}{dT} \equiv O/A$$

where Q- Net inflow in cusecs.
A - Water spread area in Square feet at a particular reservoir elevation.

Figure-14 represents the rate of rise of water level in feet/hour for various net inflows at elevations of 1160, 1165, 1170 and 1175 in the case of Amaravathy reservoir. Knowing the rate of rise of water level at any reservoir elevation the net inflow is known. The amount of gate-opening to pass the flood for the nine shutter operation can be had from Figure-7. Based on this, a chart has been developed correlating the rate of rise of water level in the reservoir at any reservoir elevation with the gate-opening required to pass the flood for the nine-shutter operation. The data are presented in Figure-15. The use of the chart is illustrated below by an example :

Problem-1.

The observed rate of rise in the reservoir is 2.5 feet/hour. The average reservoir elevation is 1165 duriug the period of observation. what is the gate opening required to pass the flow for the nine-shutter operation?

Referring to Figure 15, it is seen that nine shutters are to be opened by 10 feet to pass the required discharge. The corresponding net inflow is 56,000 cfs as taken from Figure-14.

Problem-2

Suppose all the nine shutters are already opened by 10 feet as in the previous example, and a rate of rise of water level of 3-33 feet/hour is then observed, the average reservoir elevation during the period of observation being 1170, by how much more the gates are to be opened to pass the increasing flood ?

Referring to Figure-14, the net inflow into reservoir is found to be 30,000 cfs. The gate opening required to pass the flood is 11 feet as can be seen from Figure-15. Hence all the nine shutters should be raised by one-more foot.

Thus knowing the rate of rise of water level and the reservoir elevation the amount of gate opening can be read off directly from Figure-15. Only the nine shutter operation has been taken for consideration as this operation involves the least consumption of power.

7. CONCLUSION

The experiments were conducted mainly with a view to develop a chart which will readily give the amount of gate-opening to an operator on the field, to pass a known flood over the spillway. Undue strees need not be laid on the necessity of opening the spillway gates to the correct amount in order to maintain the water level in the reservoir or to allow it to increase at a specified rate. It is hoped that the chart in Figure-15 may be of considerable use for this purpose. Only two informations are to be known for making use of the chart:—

(1) Rate of rise of water level

This can be observed in the field by noting the readings on the gauges installed in the reservoir during a known interval of time, say 15 minutes.

2) Reservoir elevation

The average reservoir elevation also can be had from the gauge readings recorded at the beginning and end of the time interval.

This chart is recommended for use in practice in the operation of spillway gates in the Amaravathy reservoir. It may be of interest to receive the critical comments after the actual use of this chart during flood times.

The studies have been made largely guided by the similar experiments done on the Lower Bhavani Spillway[1] and other experiments done on the spillways at TVA doms and presented in proceedings of the American Society of Civil Engineers separate No. 626[2].

8. REFERENCES

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2. Proceedings of American Society of Civil Engineers Separate No. 626 - February 1955.

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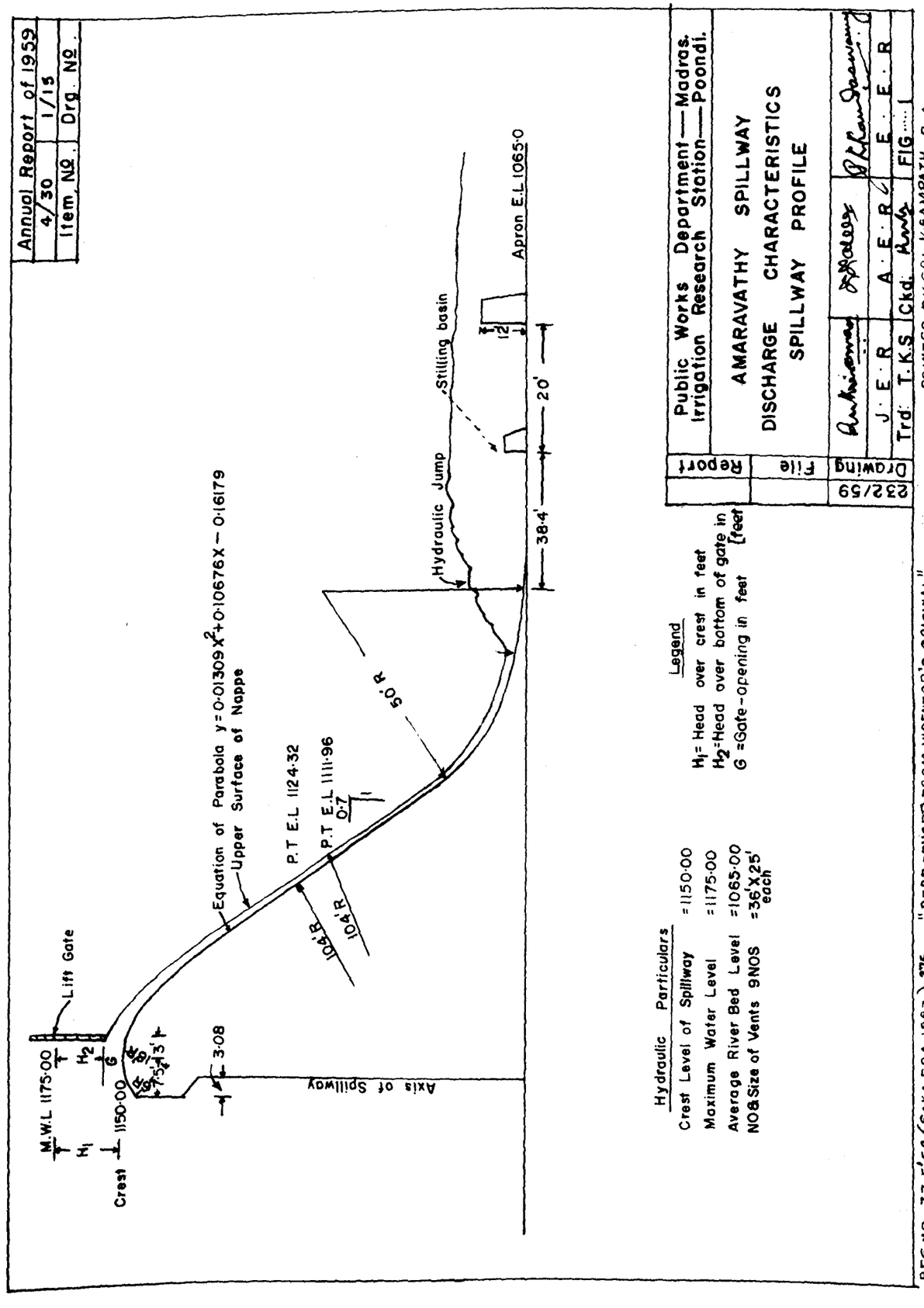
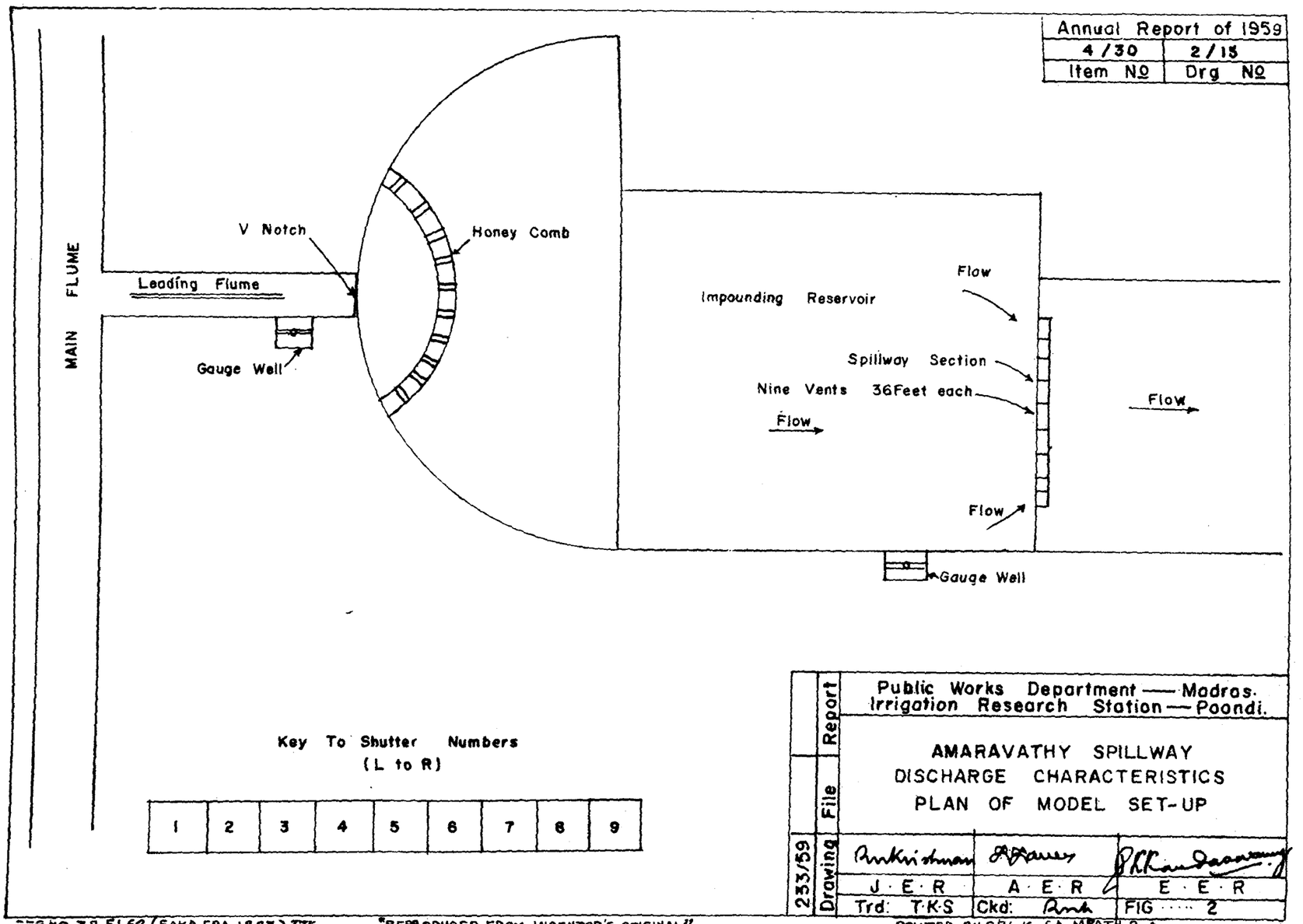
2 Engineering for dams by William P. Creager Julian He and and Joel D. Justin - Volume II.





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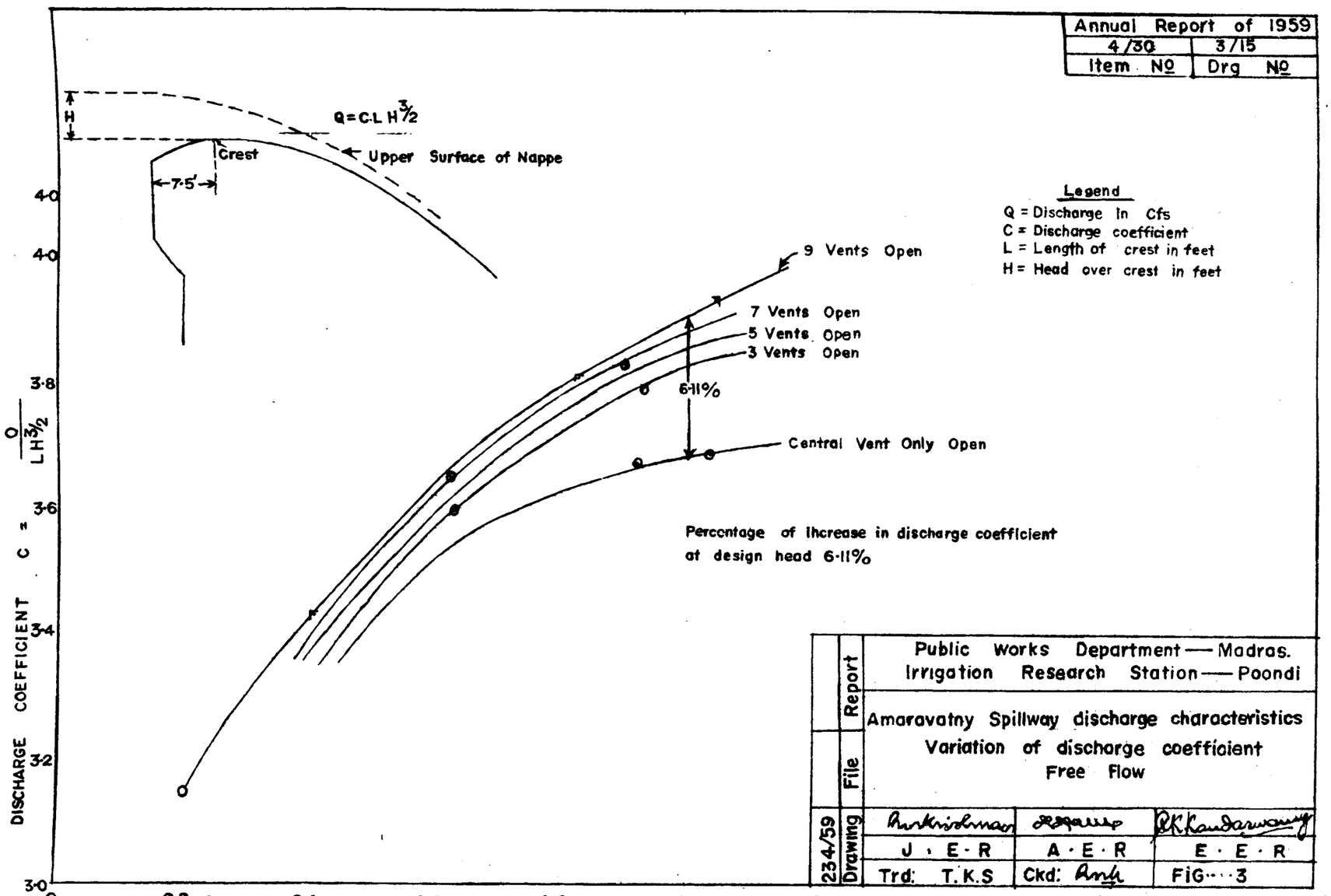

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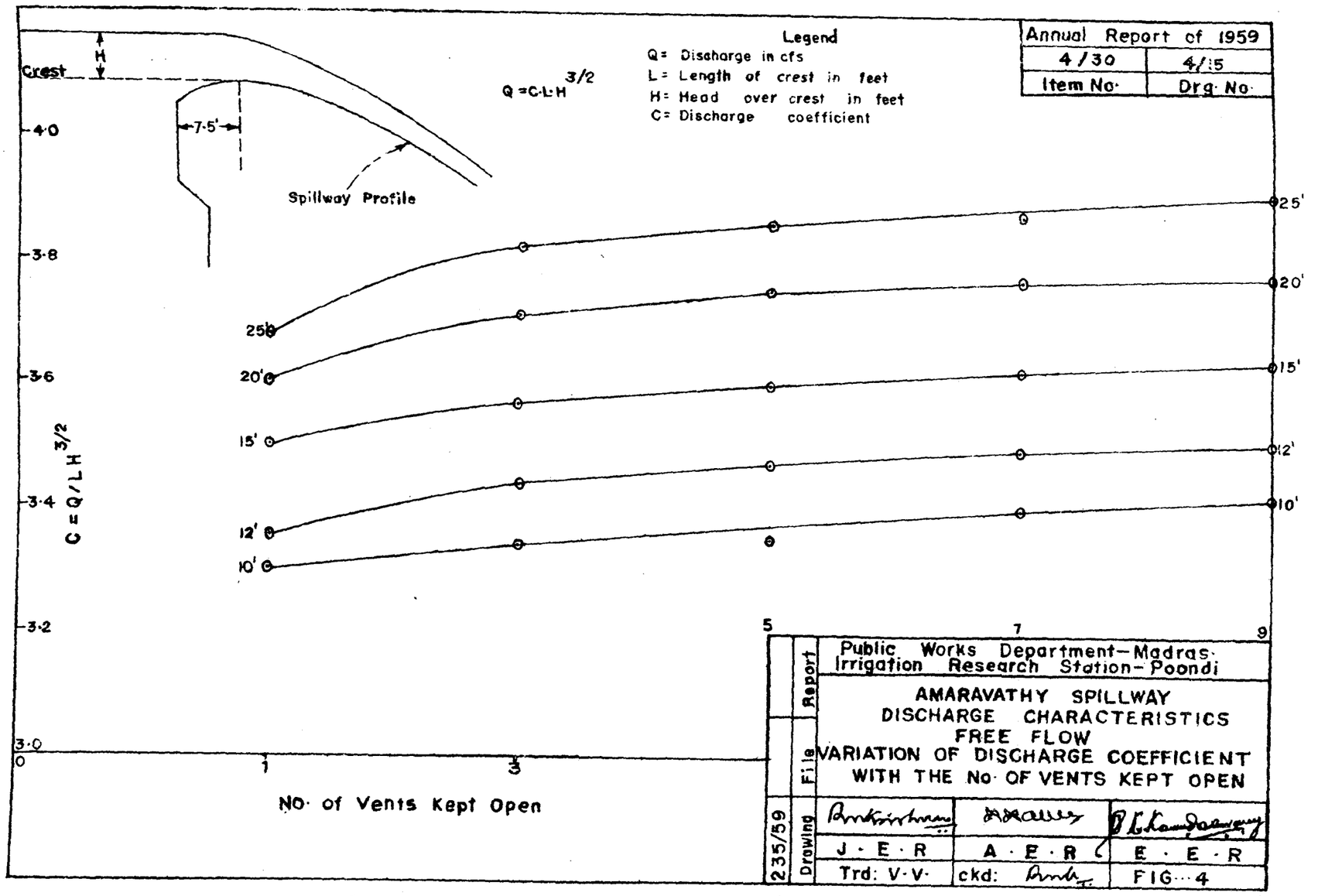

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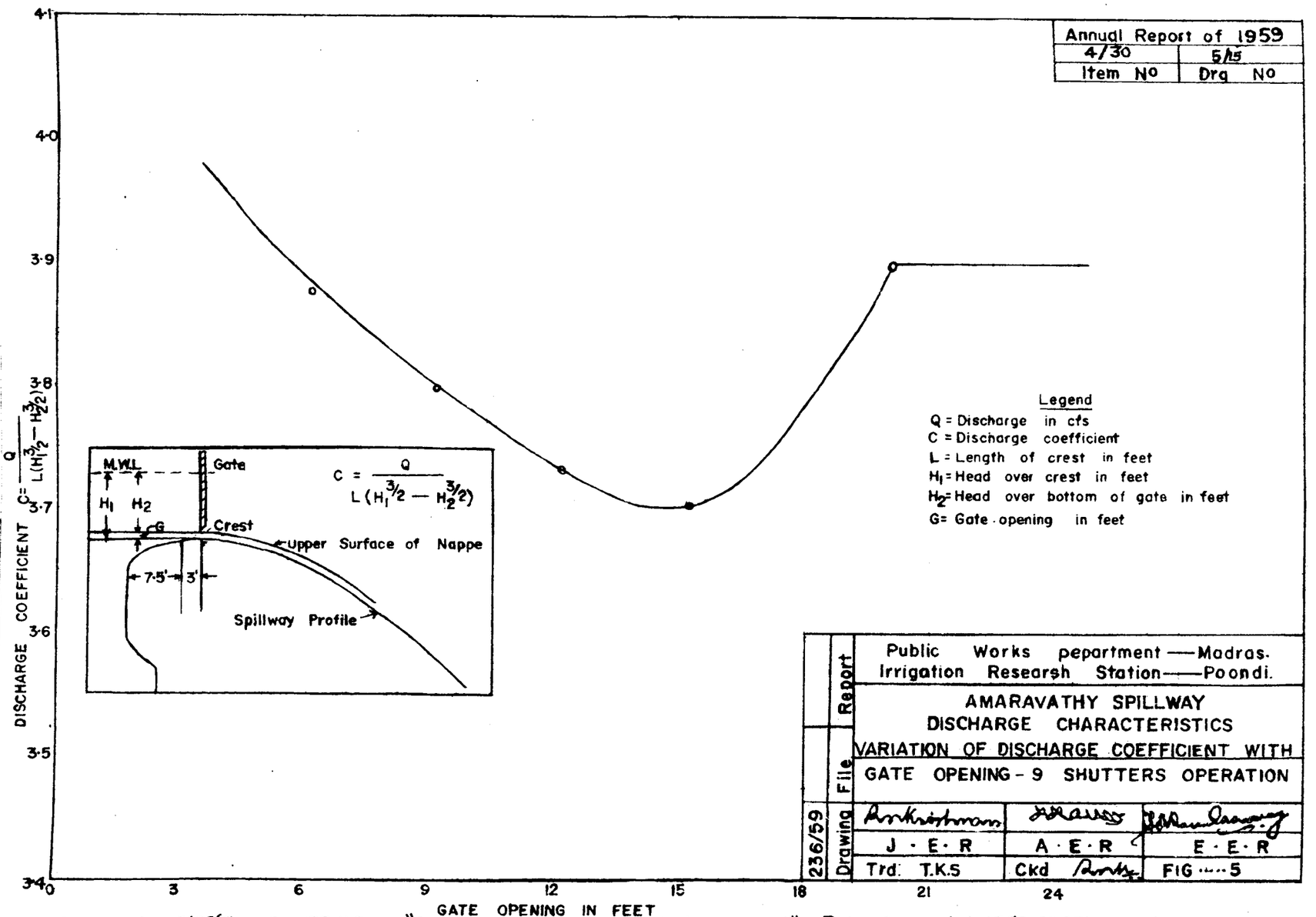

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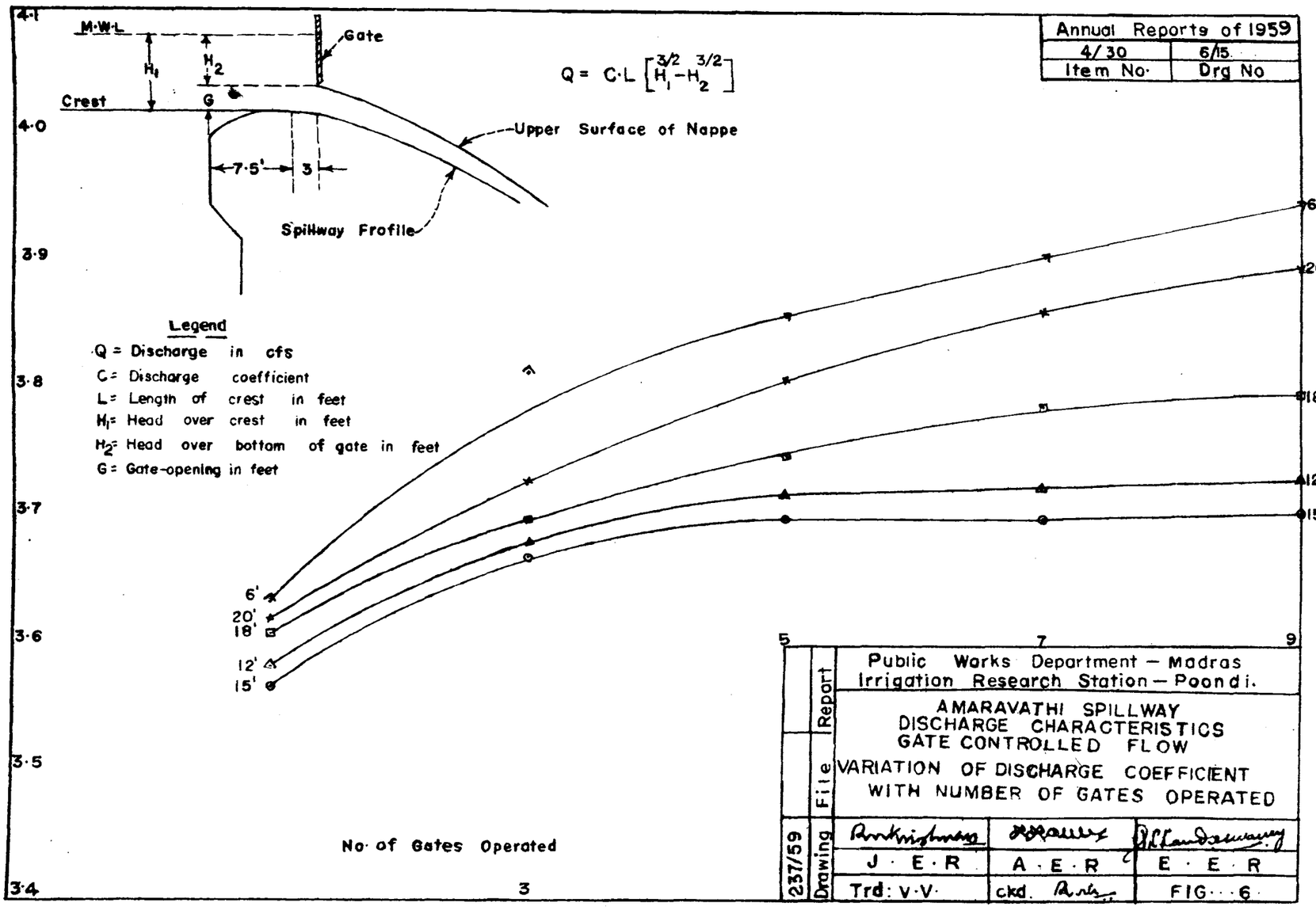

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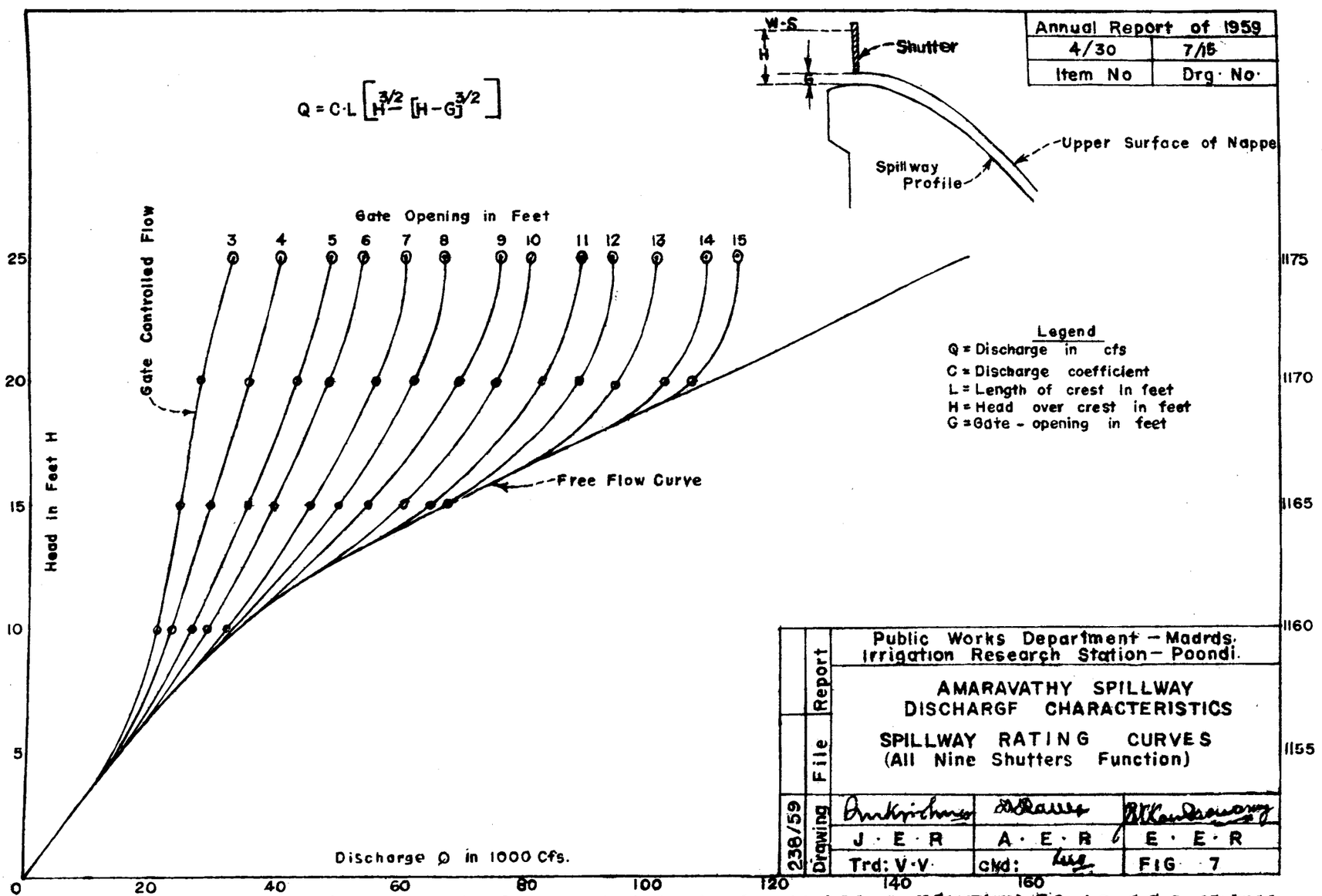

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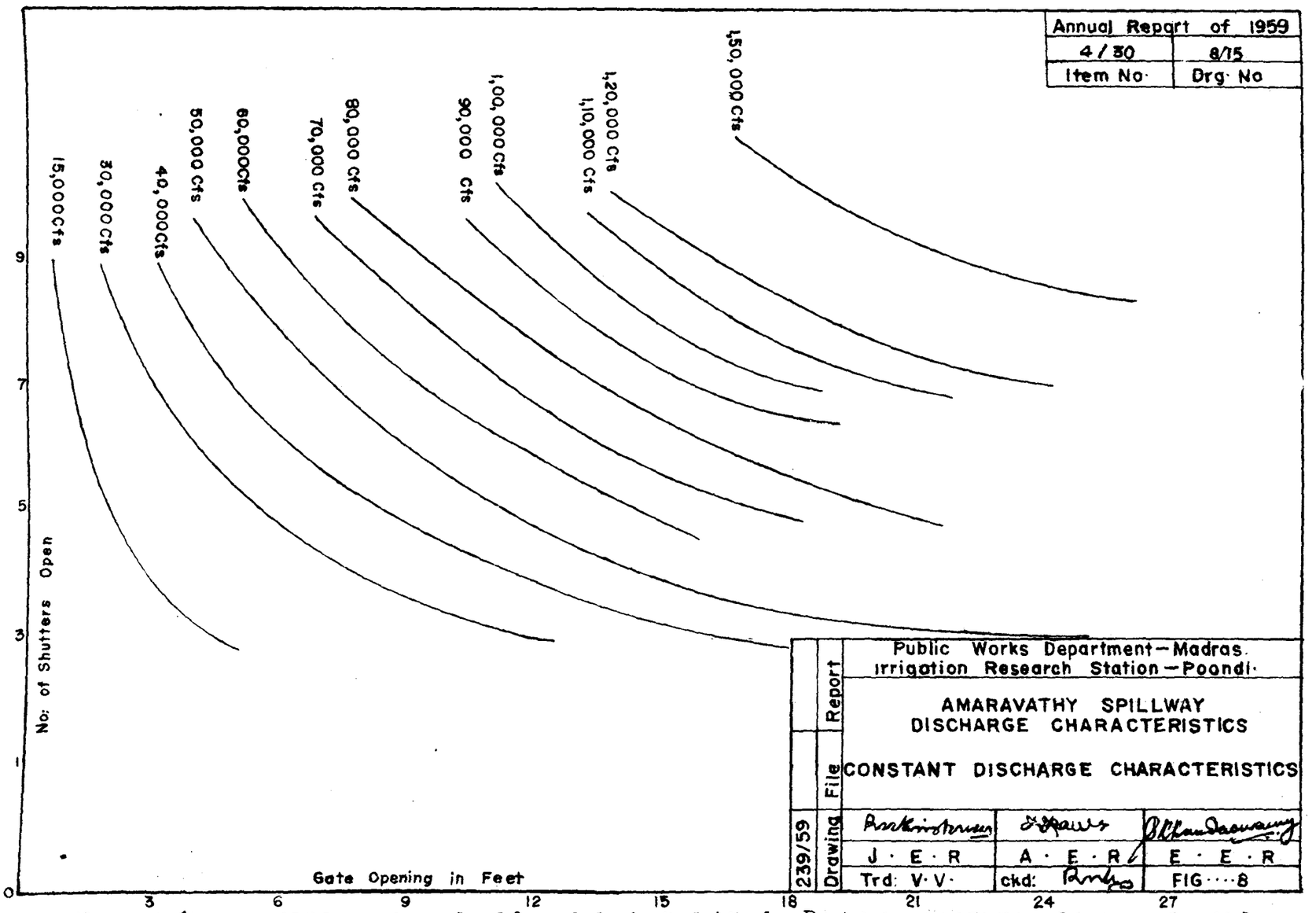

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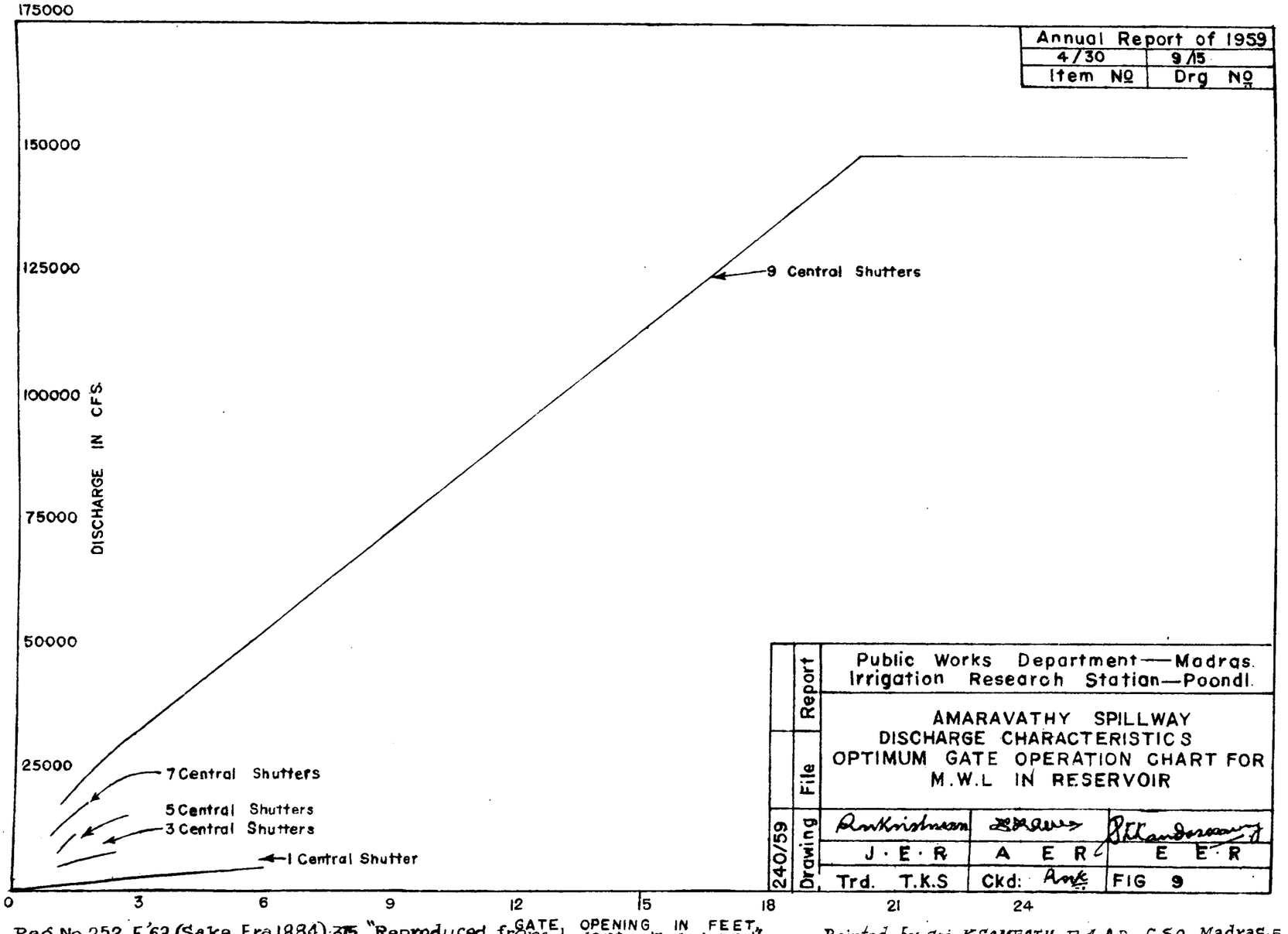

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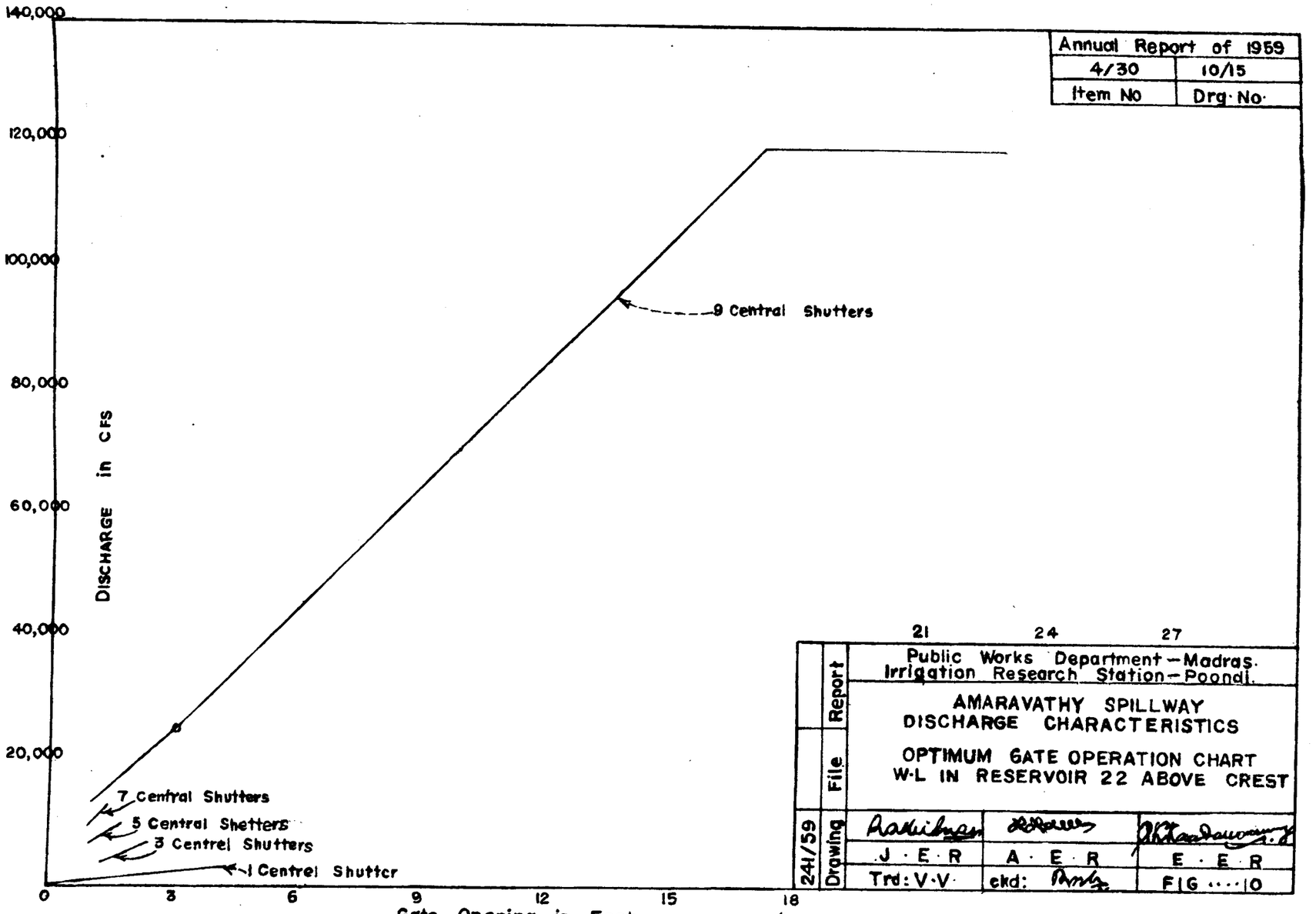

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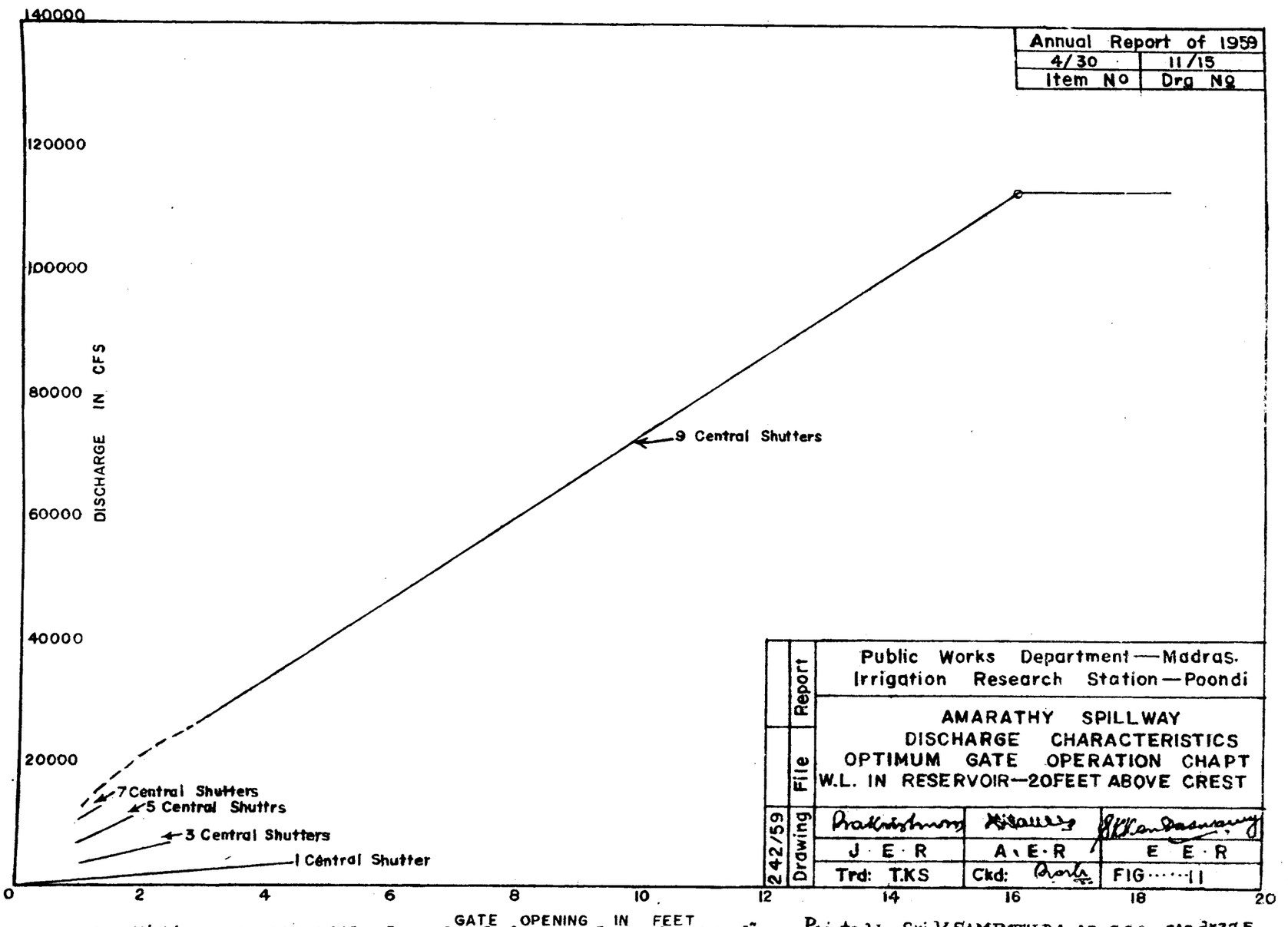

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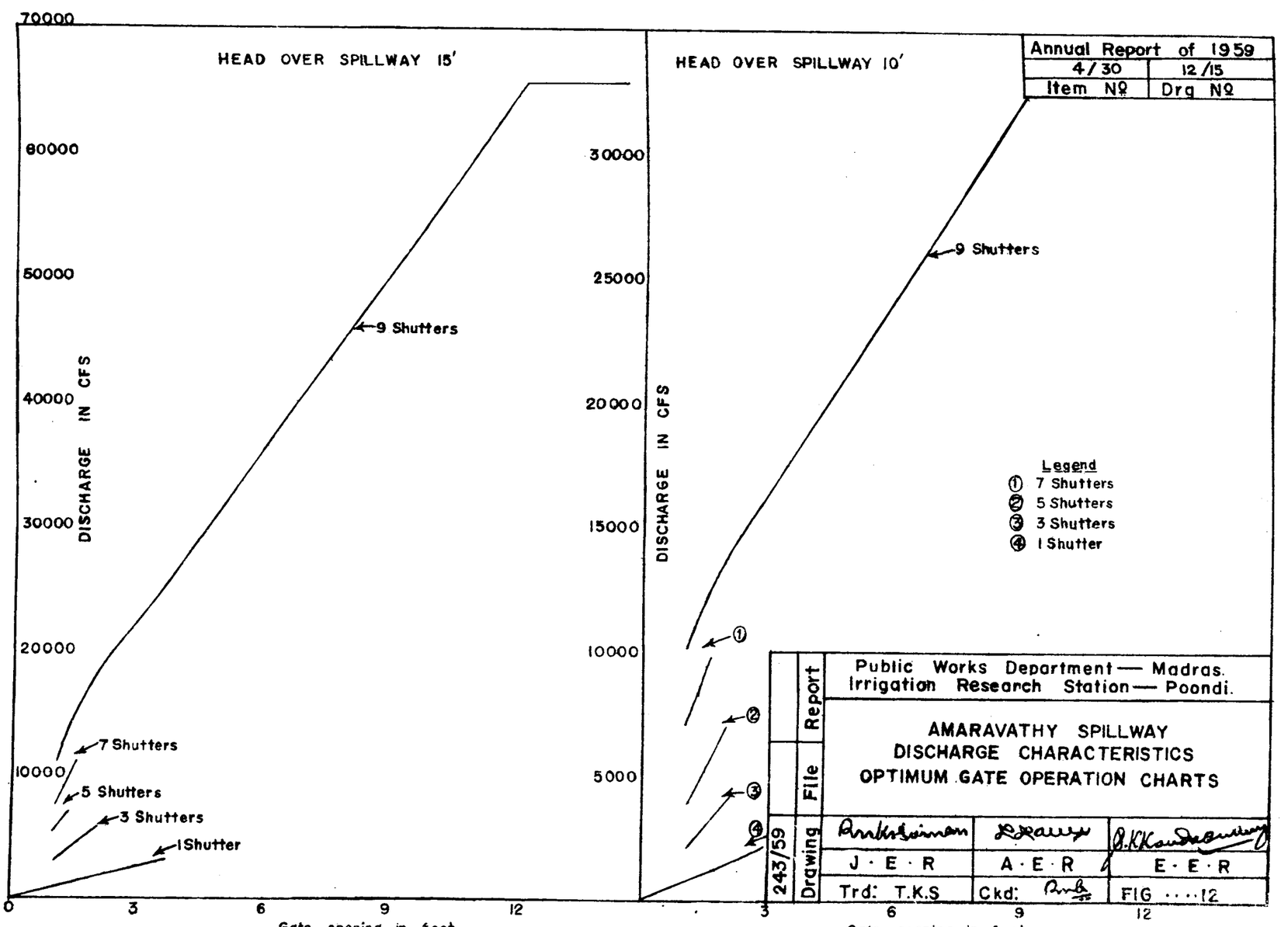

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Item Nº	Drg Nº

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

AMARAVATHY SPILLWAY
DISCHARGE CHARACTERISTICS
OPTIMUM GATE OPERATION CHARTS

Report	File	Drawing 243/59		
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		Trd: T.K.S	Ckd:	FIG ····12

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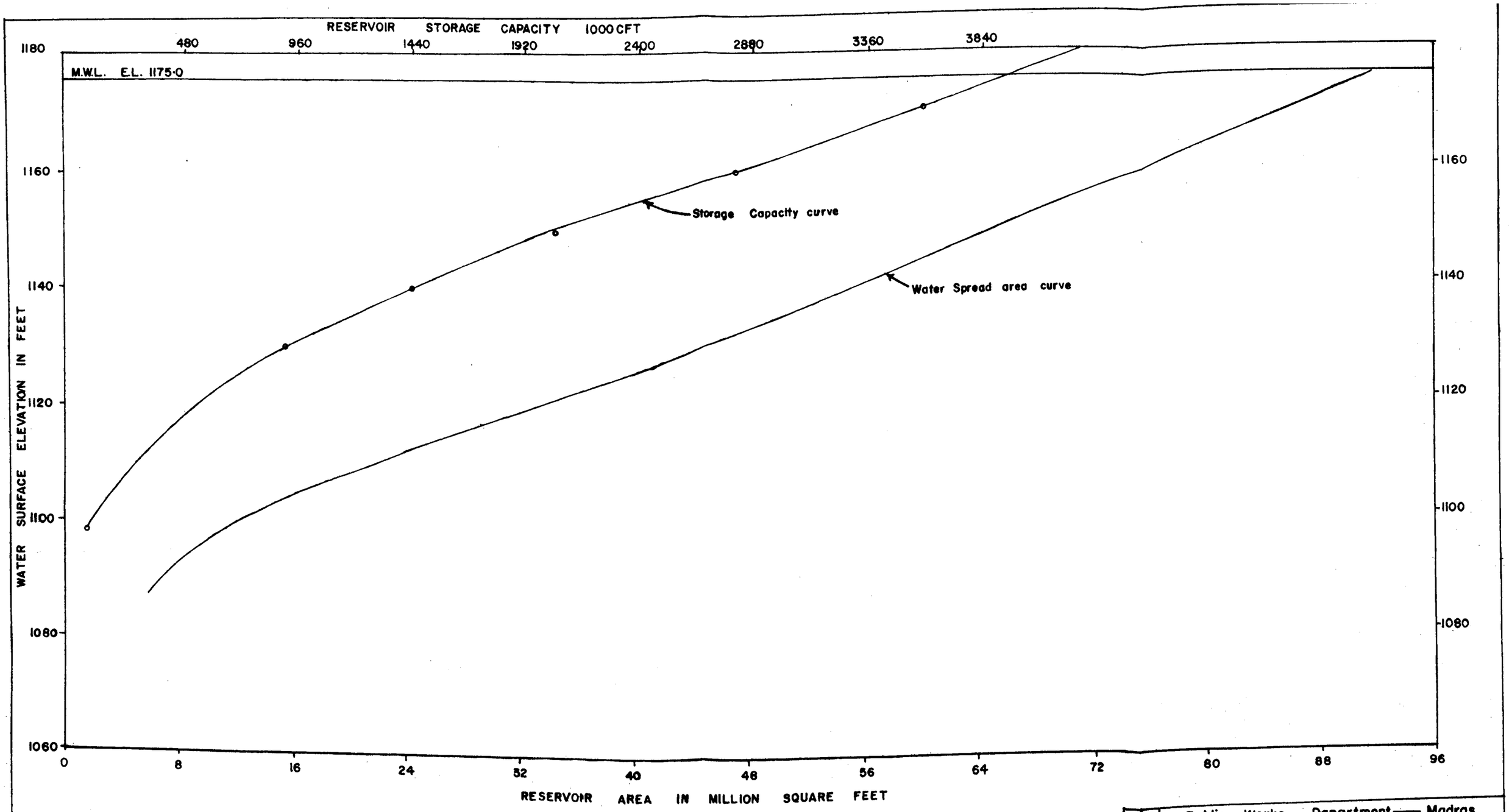

Report	Public Works Department —— Madras. Irrigation Research Station —— Poondi.	
File	AMARAVATHY RESERVOIR AREA CAPACITY CURVE (As Received) FROM DESIGNS OFFICE	
244/59 Drawing	J · E · R	A · E · R	E · E · R
	Trd: T.KS	Ckd:	FIG--- 13

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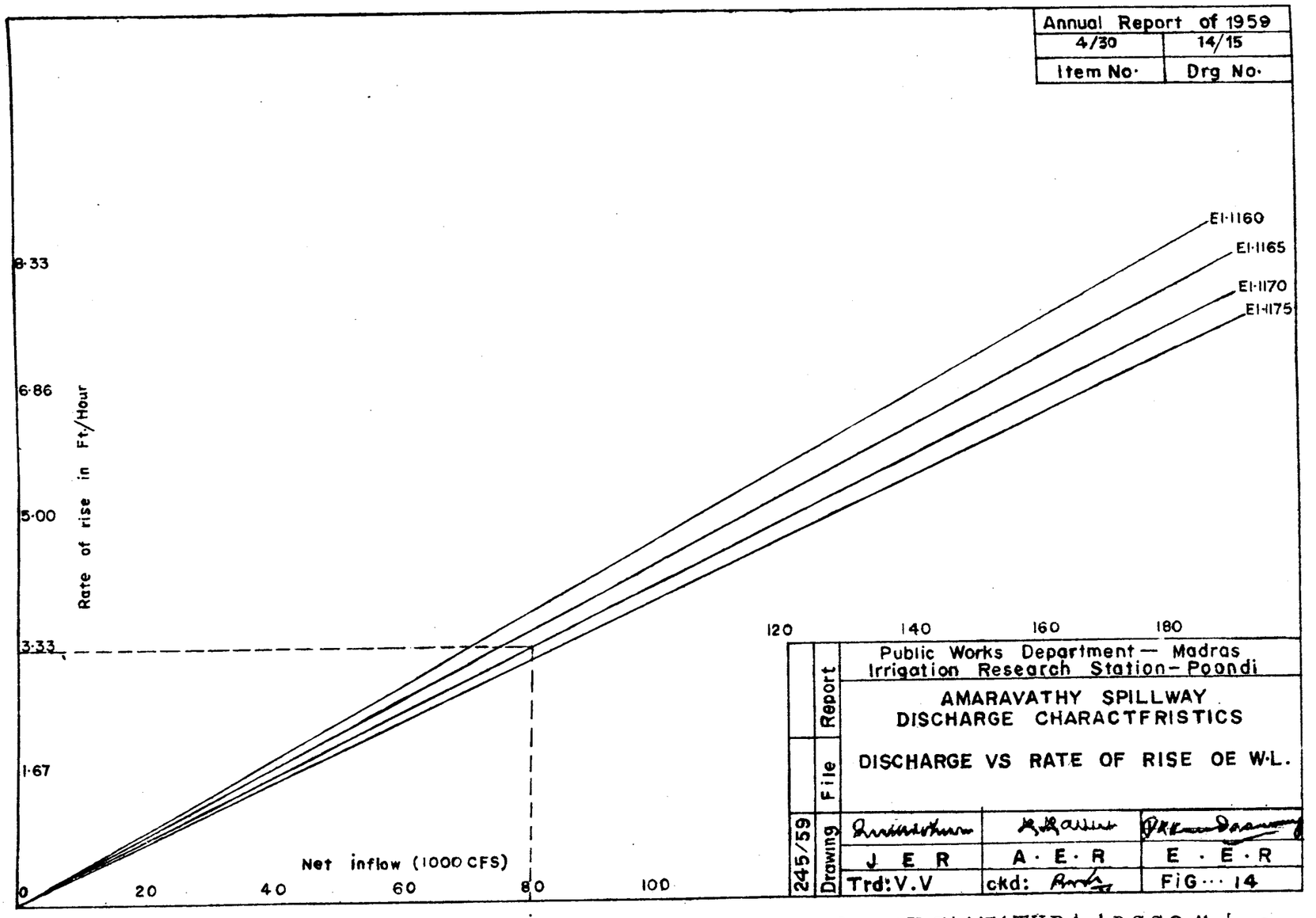

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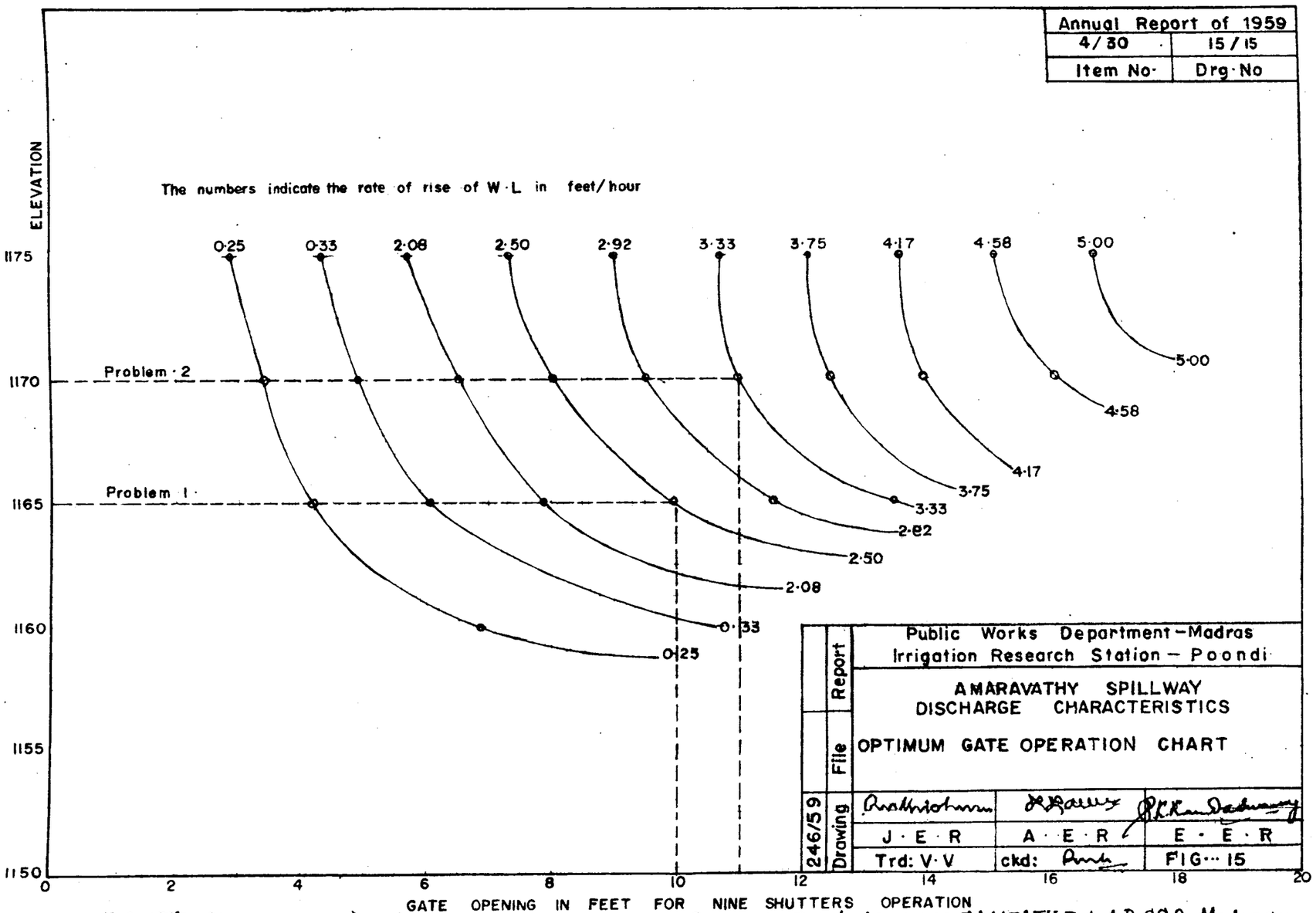

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VAIGAI DAM SPILLWAY

Discharged Characteristics

1. SYNOPSIS

This report deals with the discharge characteristics of the Vaigai Spillway, A brief introduction of the Vaigai Dam has been given and the model set up also briefly described. Studies have been conducted both for free and gate controlled flows and the variation of the discharge coefficients for both the flows has been analysed. Based on the results of the experiments an efficient and economic method of passing floods over the spillway has been evolved by drawing "Optimum operation Charts" for different water levels in the reservoir. Another chart has also been developed giving the amount of the gate opening required for seven shutter operation to pass any flood knowing the rate of rise of water level in the reservoir at a particular reservoir elevation.

2. INTRODUCTION

Vaigai Dam has been constructed across River Vaigai in Madurai District at a distance of about 40 miles trom the city of Madurai. The dam is mainly meant for storing up the flood waters of the River Vaigai and also the tail race water of Periyar Power House, which can be used in Periyar Main Canal to irrigate an additional ayacut of 20,000 acres in the Districts of Madurai and Ramanathapuram. The spillway section of the dam has been designed to surplus a flood discharge of 59,170 cusecs with a design head of 15 feet. The spillway portion consists of seven spans of 40 "0" clear each, with piers of 9"0" thickness. Vertical lift gates have been fitted over the crest to control the flow and the gates have been located three feet downstream of the crest. The profile of the spillway is given in Figure-1. Studies have been conducted in this Station to find out the most efficient and economic method of passing any flood discharge over the spillway by way of preparing charts styled as "Optimum Gate Operation Charts".

3. DATA

Catchment area	970 Sq. miles
Capacity of the reservoir at M. W. L. ...	6878 M. Cft.
Water spread area at M.W.L....	260-5 M. Sft.
Size of Vent ...	40"0" × 15"0"
No. of Vents	7 Nos.
Pier thickness	9"0"
Crest level of the spillway ...	El. 901,00
M.W.L, of Reservoir ...	El. 916,00
Spillway discharge	59,170 cusecs

4. MODEL SET-UP

The three-dimensional model, constructed to a scale of 1/60, was made use of for conducting these studies. The water level in the reservoir portion was maintained accurately by measuring the levels by means of a foot gauge reading correct to 1/1000 of a foot. The discharge passing over the spillway was measured over a right-angled V-notch fixed at the end of a leading flume, well downstream of the dam. Gate openings were adjusted accurately by means of teakwood wedge blocks of required height. The model set-up is shown in Figure-2.

5. EXPERIMENTS AND RESULTS

5.1. Studies were conducted to analyse the discharge characteristics of the spillway both the free and gate controlled flows. While conducting the studies for the free flow; only the central vent was kept open at the first instance and experiments were conducted for different heads over the spillway. Then the experiments were repeated by keeping open three central vents, five central vents and finally all the seven vents.

5.2. The discharge coefficient was calculated from the formula $Q = C L H^{3/2}$

where Q — Discharge in cusecs over the spillway

C — Coefficient of discharge

L — Length of the spillway in feet

H — Head of overflow in feet.

Figure-3 gives the values of the discharge coefficients for the free flow plotted against H/H_D where H is the head of overflow and H_D is the design head which is 15 feet in the case of Vaigai Dam. From the figure it can be seen that the dischare coefficient steadily increases with the head of overflow and with the number of shutters operated. Though the value of the discharge coefficient for the Seven shutters operation is not very much different from that for the single shutter operation, there is some definite incraese in the value of the discharge coefficient for the seven shutter operation over the single shutter operation, but, practically there is no difference in the values of the discharge coefficient between the five and seven shutters operation. It is seen from the figure that there is a percentage increase of 7.25% in the value of the discharge coefficient for the seven shutter operation over the single shutter operation when $H/H_D = 1$.

5.3. Figure-4 gives the variation of the discharge coefficient with the number of shutters operated for different overflow heads. It is seem from the figure that the discharge coefficient increases with increased number of gate openings.

5.4. Secondly, studies were conducted for gate controlled flow. In this case also the same pattern of operation of the shutters, namely, one shutter, three shutters, five shutters and all the seven shutters, was followed. Readings were taken for different gate openings keeping the water level at M.W.L. The discharge coefficint was calculated by using the formula

$$Q = C L (H_1^{3/2} - H_2^{3/2})$$

where Q, C and L stand for the same notations as in the case of free flow

H_1 — Depth of water in feet over the crest of the spillway

H_2 — Depth of water in feet from the bottom of the gate

H_2 — $(H_1 - G)$ where G - Gate opening in feet.

Similarly studies were conducted for different water levels corresponding to 12 feet, 11 feet, 10 feet and 5 feet of water over the crest.

5.5. The variation of discharge coefficient with the gate opening for the seven shutter operation with M.W.L in the Reservoir is presented in Figure-5. The discharge coefficient decreases with any increase in the gate opening upto a certain limit (in this case 5 feet) and then steadily increases. When the gate opening is 10 feet the flow just touches the bottom of the gate and for any further opening of the gate the flow is no longer controlled by the gate and it becomes a free flow.

5.6. The spillway rating curve for seven shutter operation is given in Figure-6. The qnantity of discharges passing for different gate openings ranging from two feet to ten feet and for different water levels over the spillway is given in this figure. The free flow curve from which the controlled flow curves branch off gives the discharge when the flow is not controlled by the gate for any head of water over the spillway. The point where the controlled flow curve takes off from the free flow curve gives the head over the spillway at which the flow becomes free for a particular gate opening.

5.7. The Discharge Chart in Figure-7 gives the different combinations of possible operation for passing off any known flood for the seven shutter operation at M.W.L. Before operating the shutters to dispose of any flood two important points should be borne in mind, the first one being that gate opening less than one foot should be avoided as this will create large amount of negative pressure on the spillway surface and the second one being that the economy in gate operation increases with increasing number of shutters operated. Based on these considerations a chart styled as "Optimum Gate-operation Chart" is given in Figure-8 to pass any flood at M.W.L. But in practices shutters may have to be operated even before the maximum water level condition is attained. Therefore, similar charts have been drawn for different upstream water levels corresponding to 12 feet, 11 feet, 10 feet and 5 feet of head over the spillway and are presented in figures 9 through 11.

5.8. But these charts can be made use of only if the incoming floods are known. It will be very difficult at the time of floods to gauge the net-inflow into the reservoir. So, another method was adopted as has been done in the case of Amaravathi Dam to pass the discharge by arriving at the net inflow from the rate of rise of water level at any particular reservoir elevation. In Figure-12 the area-capacity curve for the Vaigai Dam is given. At times of floods, the water level in the reservoir rises, the rate of rise being a function of the net inflow and the water spread area at that particular reservoir elevation.

$$\frac{dH}{dT} = \frac{O}{A}$$

where $\frac{dH}{dT}$ = Rate of rise in water level

Q = Net inflow at particular reservoir elevation

A = Water spread area at that particular reservoir elevation.

Figure-13 gives the rate of rise of water level in ft/hour for various net inflows at different elevations of the Vaigai Reservoir namely 906,00, 911,00, 912,00, 913,00 and 615,00, the crest level of the spillway being 901,00. By referring to this figure, the net inflow into the reservoir can readily be assessed if the rate of rise of water level in the reservoir is noted for a particular time. Knowing the net inflow, the amount of gate opening to pass the discharge with the seven shutters operation can be had from the Figure-6. Combining all these things a chart has been developed giving the amount of gate opening to pass the flood with seven shutter operation for any rate of rise of water level in the reservoir at any reservoir elevation (vide Figure-14). For example, if the water level in the reservoir rises at a rate of 0.5 ft/hour when the reservoir elevation is 912,00 then to pass the discharge at that instant all the seven shutters should be opened by 7 '00" (from Figure-14). The corresponding net inflow into the reservoir will be 32,500 cusecs as taken from Figure 13.

6. CONCLUSION

Studies were conducted with a view to develop a method to pass a known flood discharge over the spillway. A chart has been given correlating the rate of rise of water level at any reservoir elevation with the gate opening required to pass the discharge for the seven shutter operation. The reservoir elevation at any instant can be seen from the gauge post fixed in the reservoir and the rate of rise of water level can be noted for a short period say 15 minutss, and the rate or rise for one hour can be calculated. Knowing these two details the gate opening requirod to surplus to inflow at that instant can be known from the chart presented in Figure-14. The net inflow can be had from Figure-13. This chart has been developed with the idea that it may be of much use in the field for operating the shutters at times of floods.

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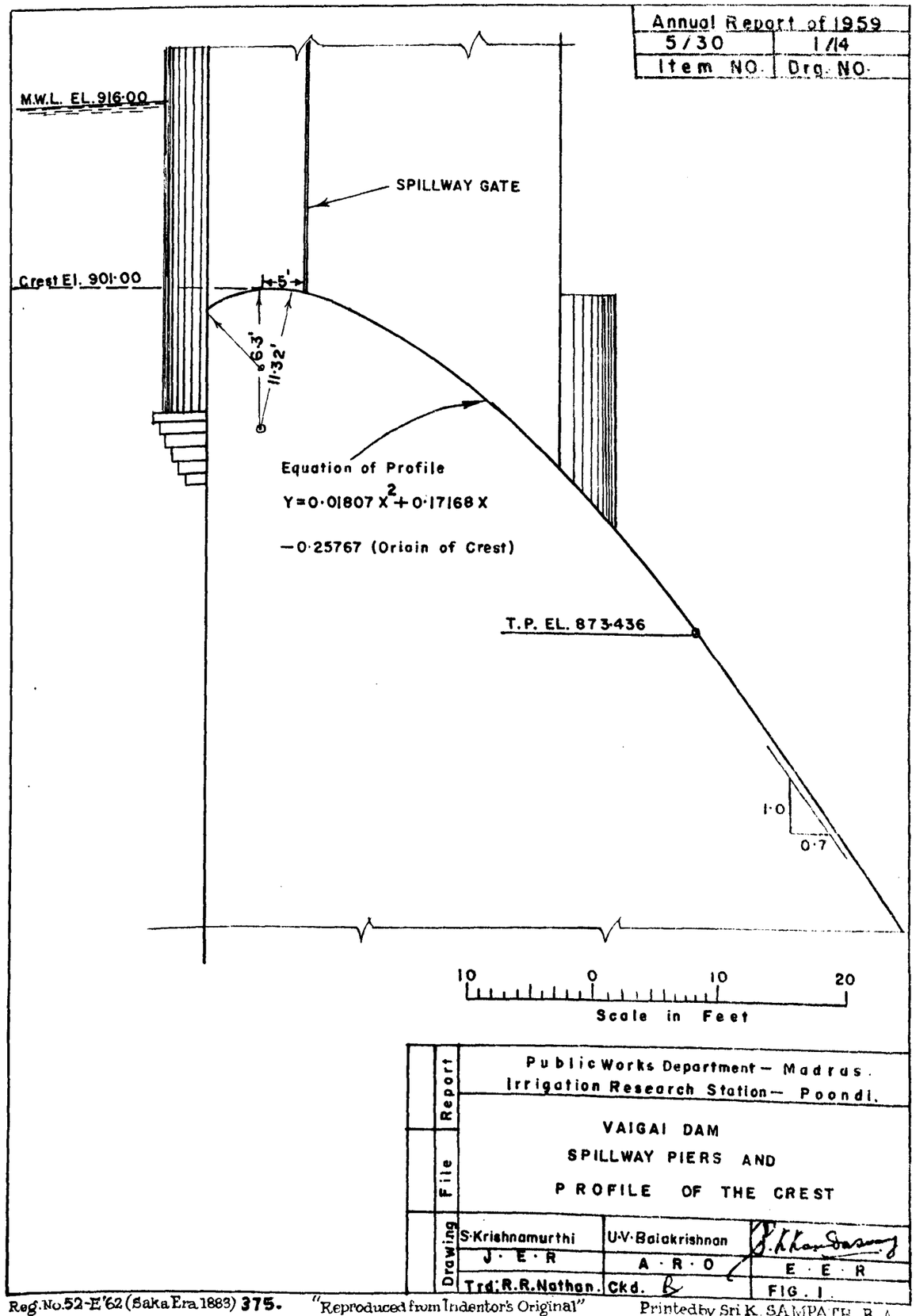
(2) Discharge characteristics of Amaravathi Spillway I. R. S. Report No. 23/59.



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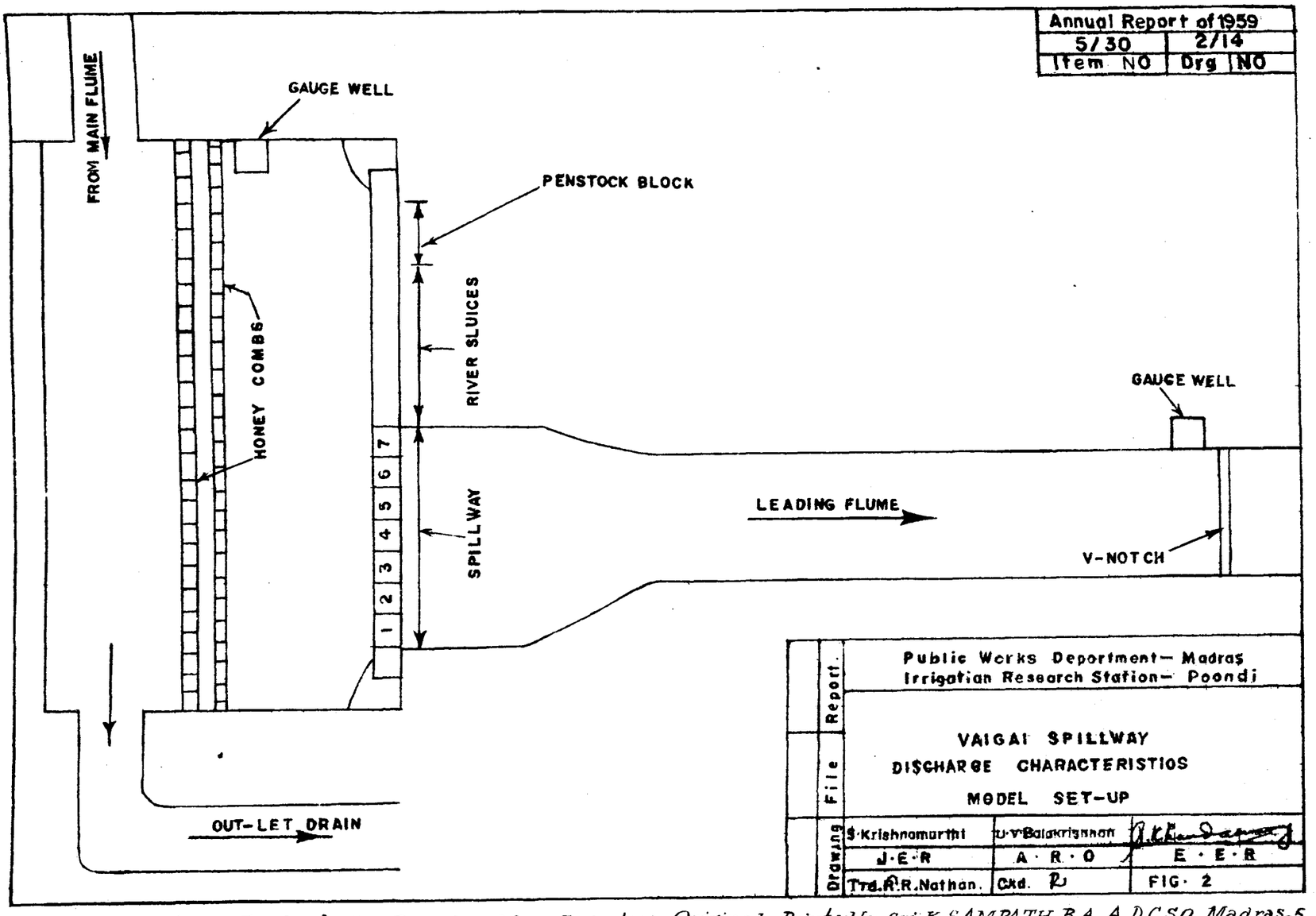

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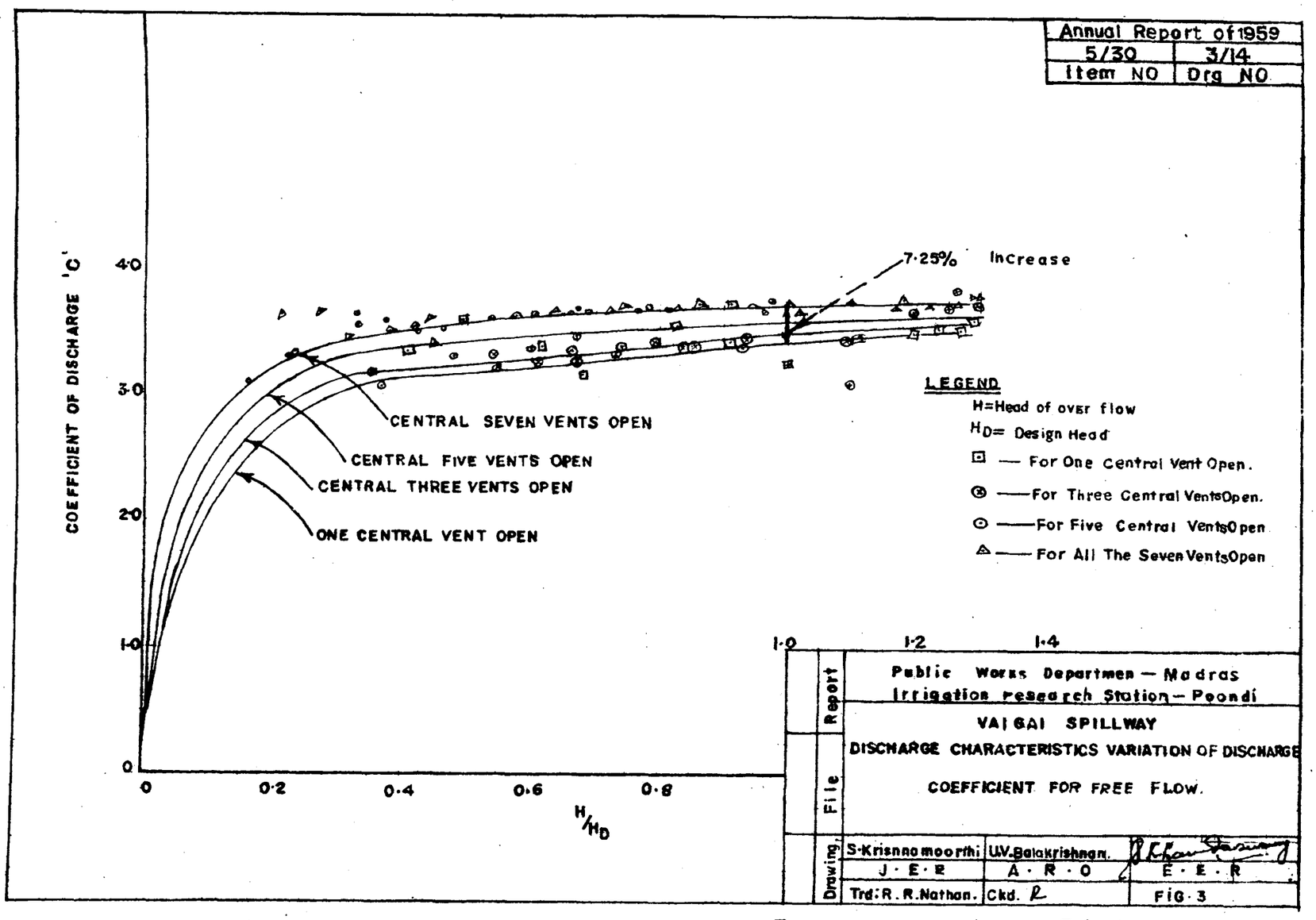

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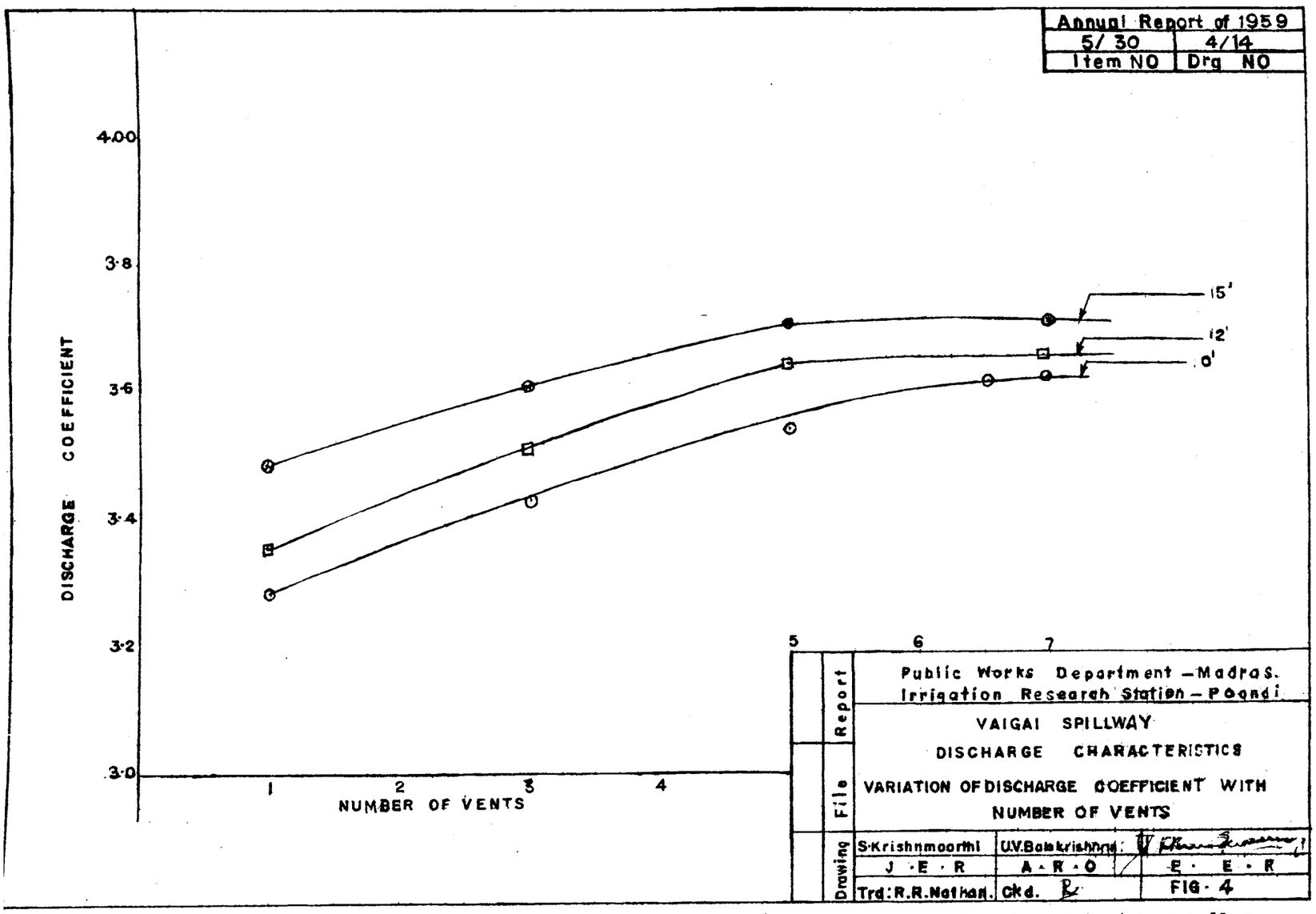

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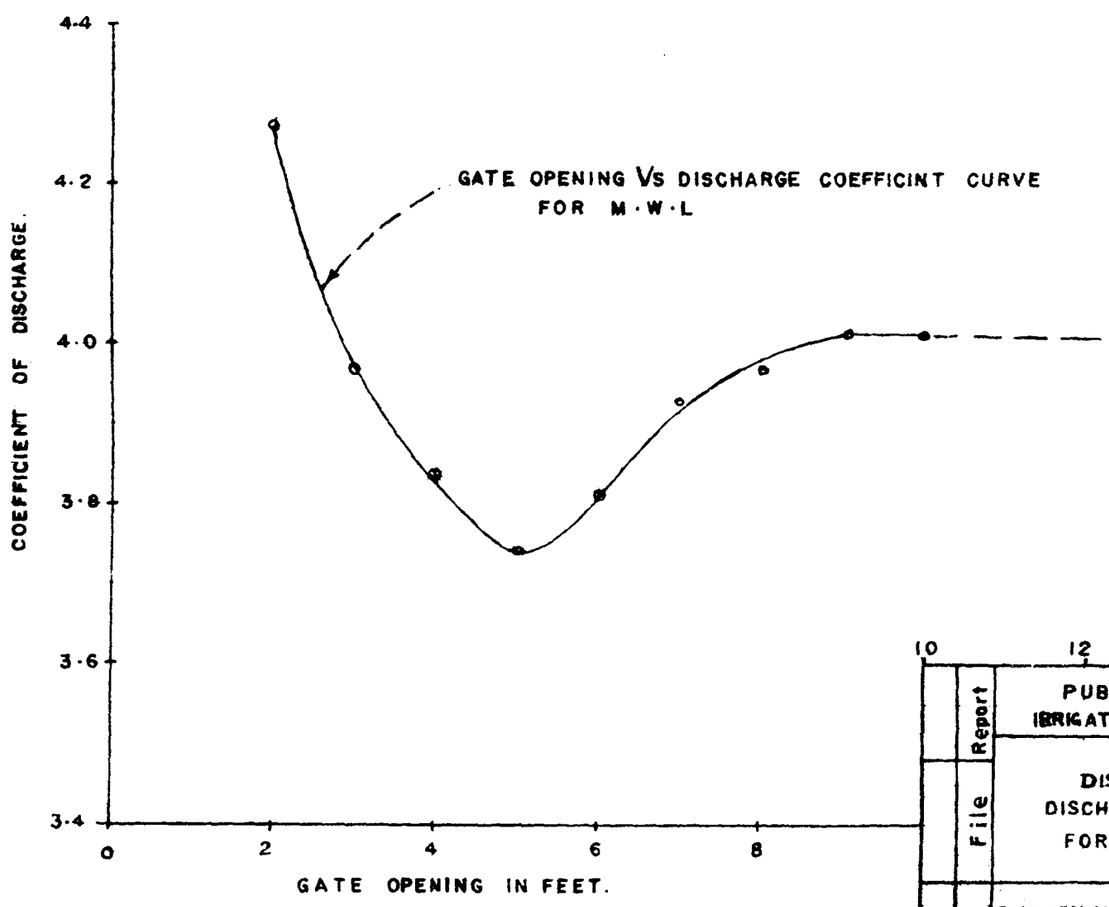

Report	PUBLIC WORKS DEPARTMENT—MADRAS. IRRIGATION RESEARCH STATION — POONDI.
File	VAIGAI SPILLWAY DISCHARGE CHARACTERISTICS DISCHARGE COEFFICIENT Vs GATE OPENING FOR 7 SHUTTERS OPERATION WITH M·W·L
Fig.	S. KRISHNAMURTHY / U·V·Balakrishnan / J·E·R. / A·R·O / E·E·R. / Trd: C.S.K. / Ckd: / Fig. 5.

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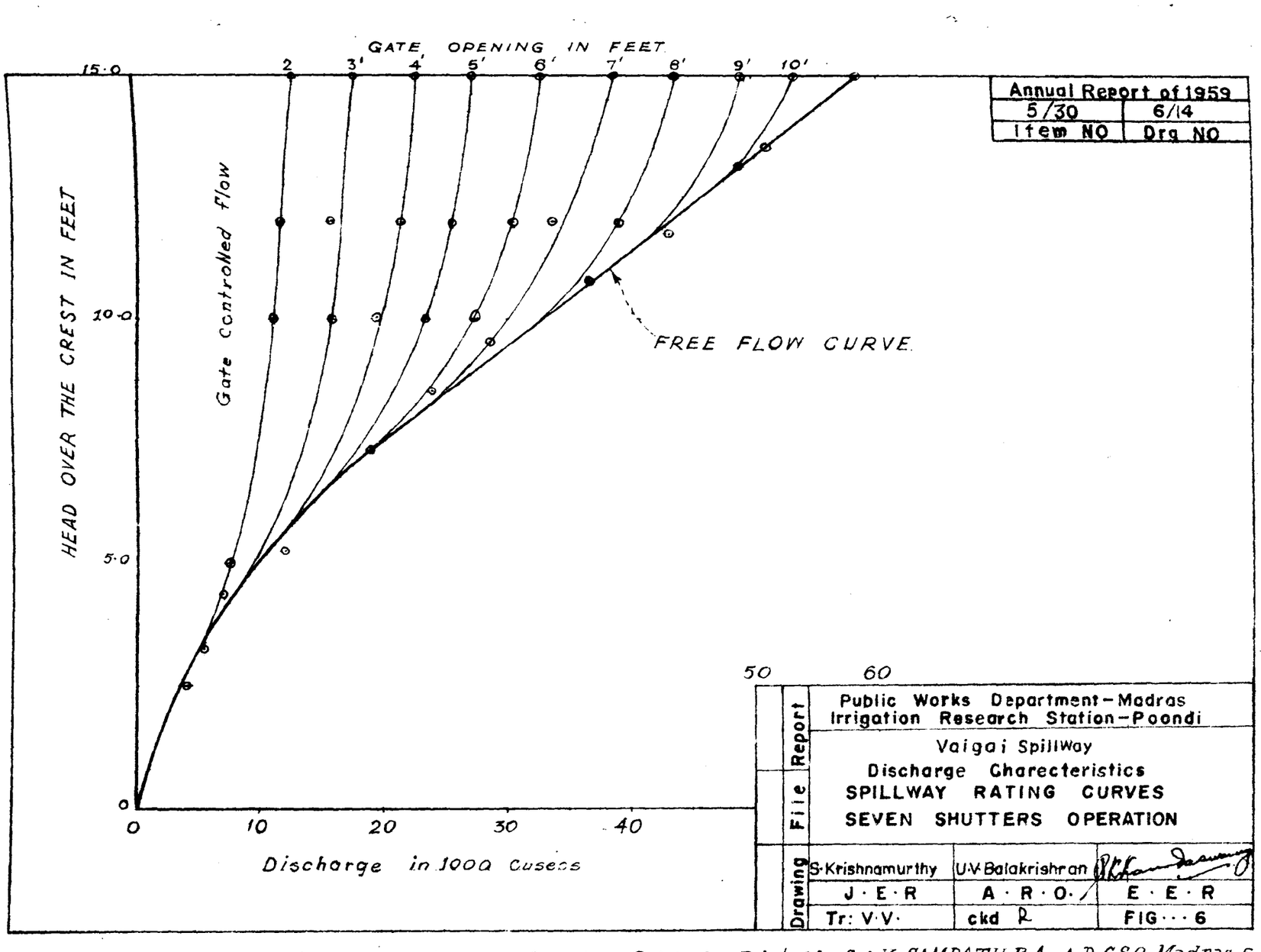

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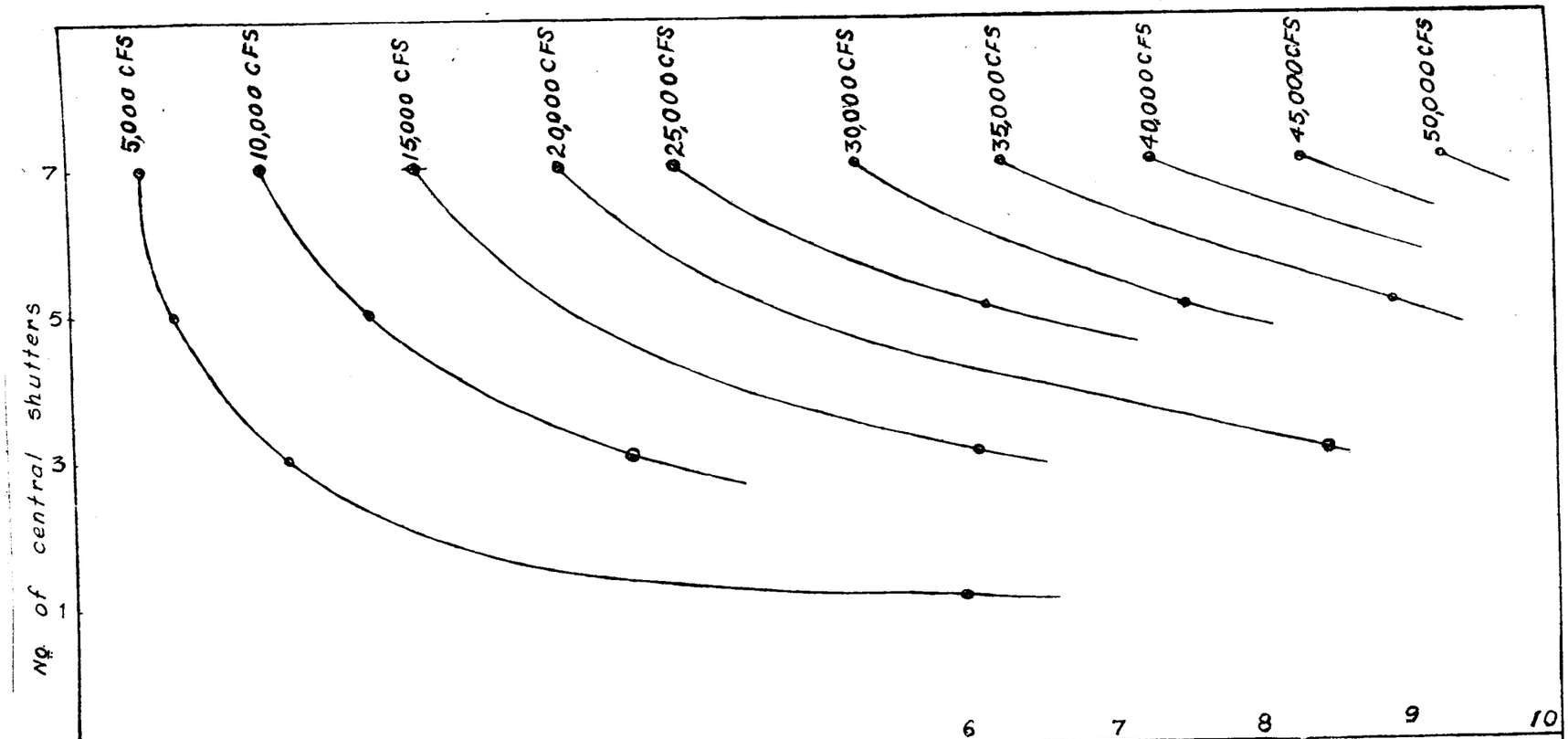

Report	Public Works Department – Madras Irrigation Research Station – Poondi		
	Vaigai Spillway Discharge Charecteristics		
File	CONSTANT DISCHARGE CHARECTERISTICS		
Drawing	S.Krishnamurthi	U.V. Balakrishnan	
	J · E · R	A · R · O	E·E·R
	Tr: V V·	ckd· R	FIG···7

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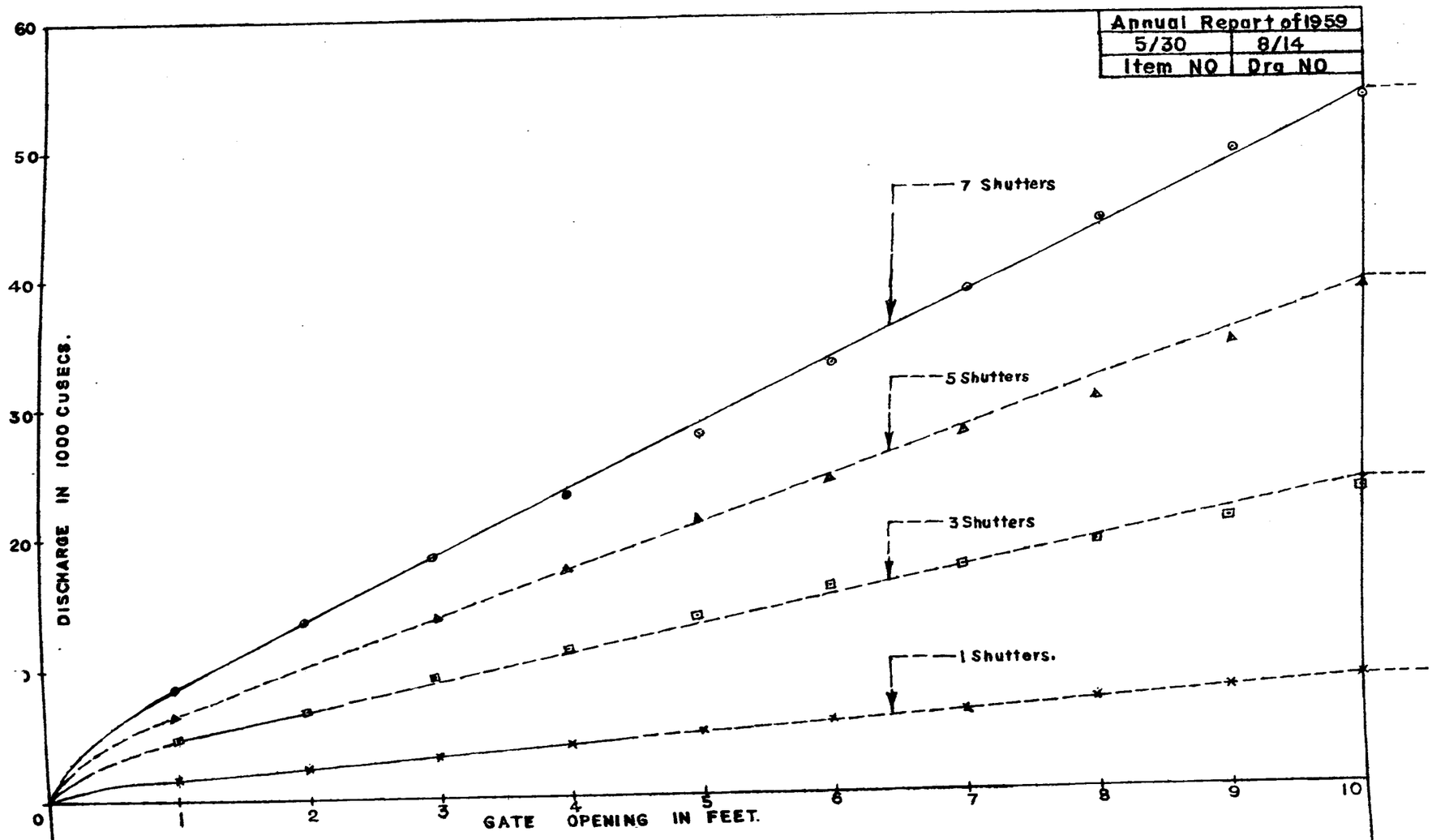

/60	File Report.	PUBLIC WORKS DEPARTMENT—MADRAS. IRRIGATION RESEARCH STATION—POONDI.		
		VAIGAI SPILLWAY. DISCHARGE CHARACTERISTICS OPTIMUM GATE-OPERATION CHART FOR M·W·L AT 916·00		
	Drg.	S. KRISHNAMOORTHI	U·V·Balakrishnan	P. Kandasamy
		J·E·R.	A·R·O	E·E·R.
		Trd: C.S.K.	Ckd:	Fig... 8

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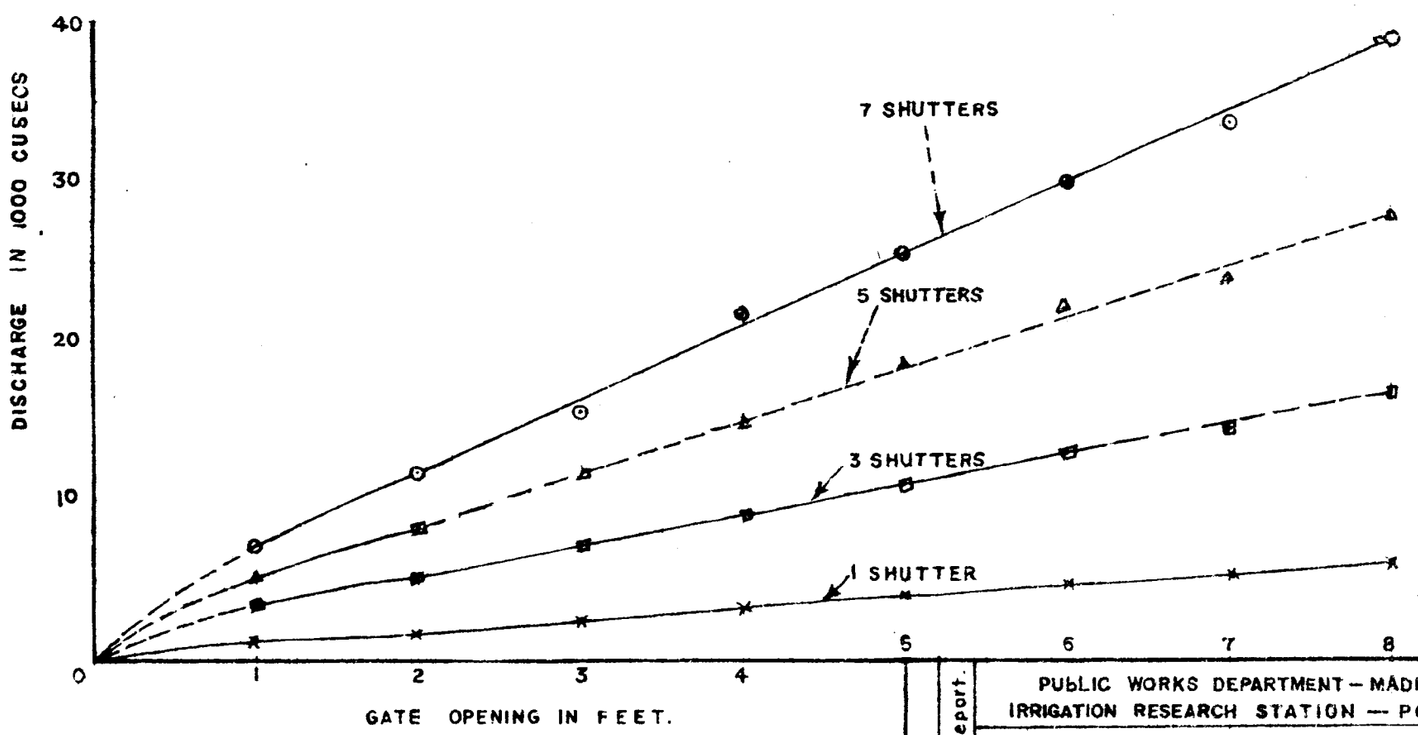

PUBLIC WORKS DEPARTMENT — MADRAS
IRRIGATION RESEARCH STATION — POONDI.

VAIGAI SPILLWAY.
DISCHARGE CHARACTERISTICS.
OPTIMUM GATE OPERATION CHART
FOR W.L AT 918·00

Report.			
File			
/60 Drg.	S.Krishnamoorthi	U.V.Balakrishnan	[illegible]
	J.E.R.	A.R.O	E.E.R.
	Trd: C.S.K Pathy	Ckd	Fig. 9

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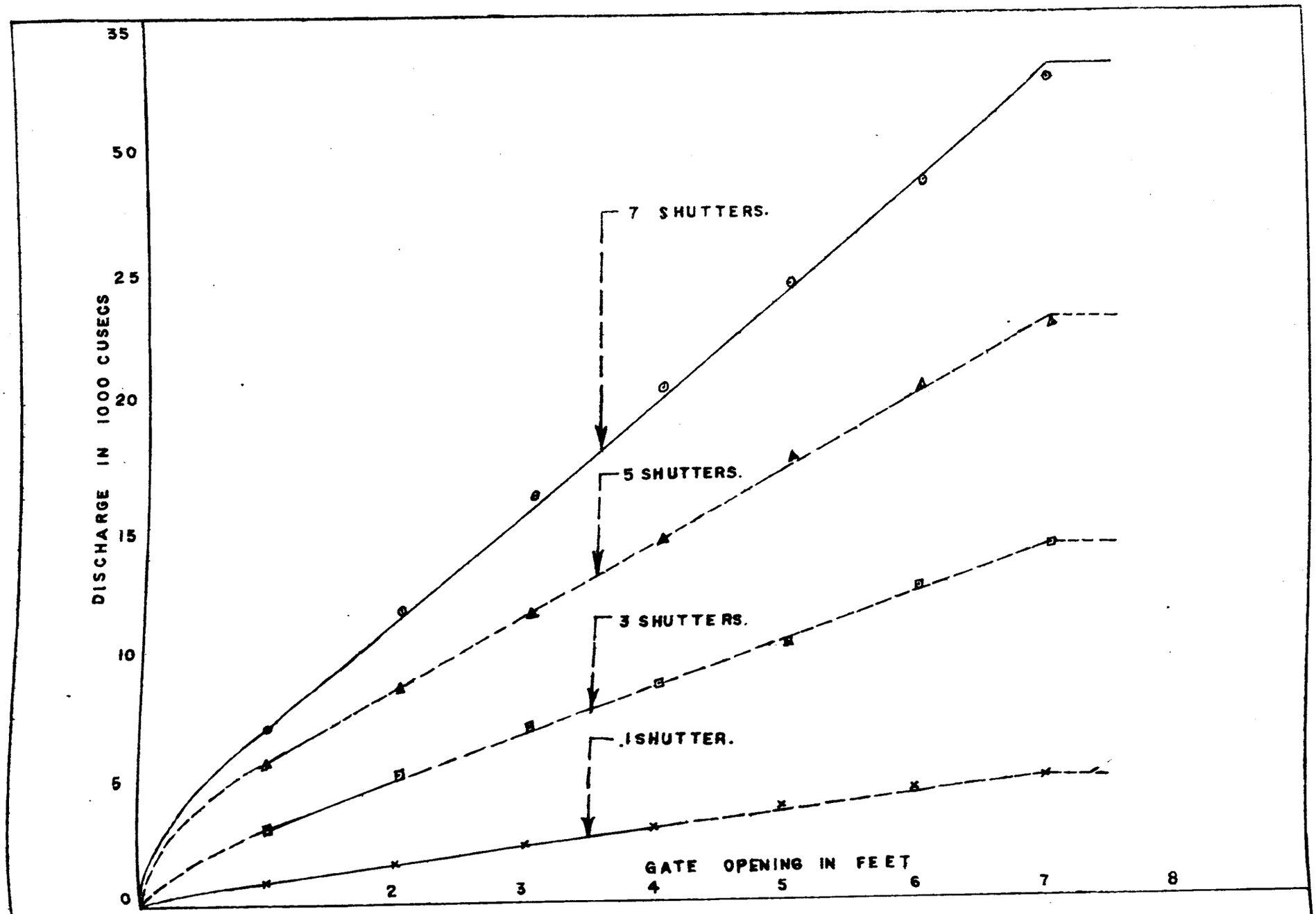

PUBLIC WORKS DEPARTMENT—MADRAS IRRIGATION RESEARCH STATION — POONDI.		
VAIGAI SPILLWAY DISCHARGE CHARACTERISTICS OPTIMUM GATE—OPERATION CHART FOR W·L AT 912.00.		
S.Krishnamurthi	U·V·Balakrishnan	
J·E·R.	A·R·O	E·E·R.
Trd: C.S.K.	Ckd:	Fig. 10.

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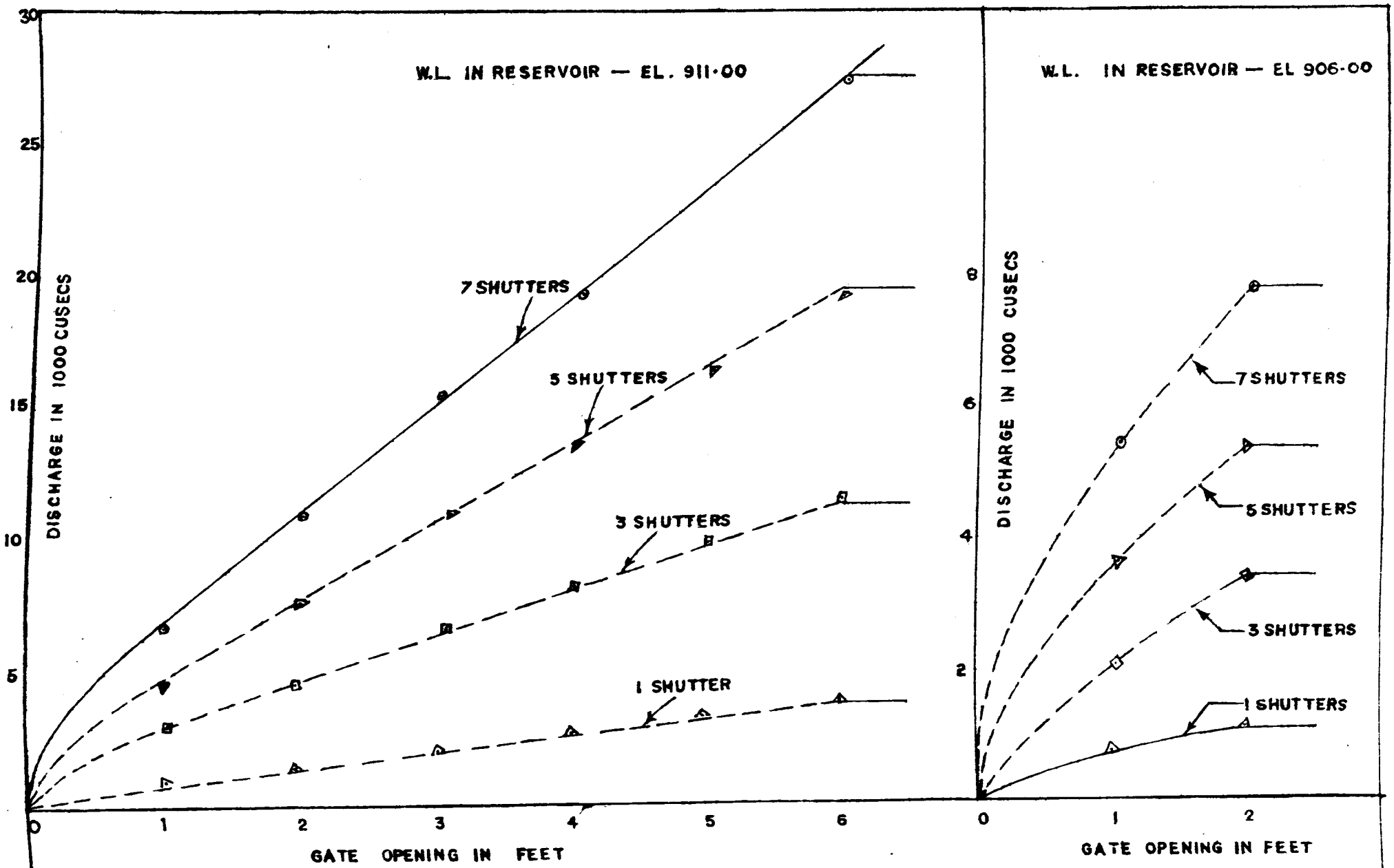

Report	Public Works Department — Madras. Irrigation Research Station — Poondi.		
File	VAIGAI SPILLWAY DISCHARGE CHARACTERISTICS OPTIMUM GATE OPERATION CHART FOR. W.L.S AT 911·00 AND 906·00		
Drawing	S·Krishnamurthi	U·V·Balakrishnan	
	J·E·R	A·R·O	E·E·R
	Trd: R.R. Nathan.	Ckd.	FIG·11

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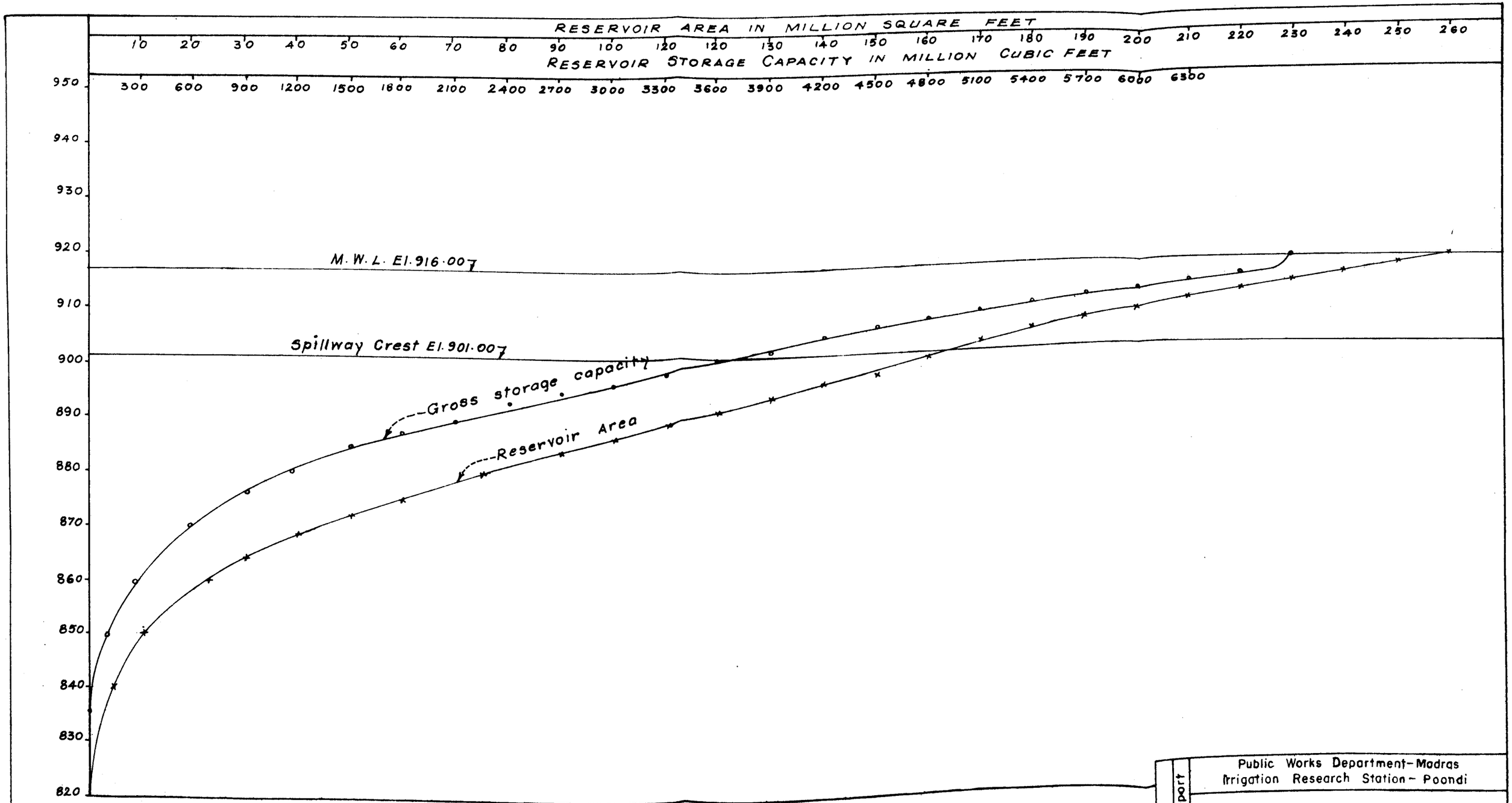

Public Works Department - Madras
Irrigation Research Station - Poondi

Vaigai Spillway

AREA CAPACITY CURVES

Drawing	S. Krishnamurthy	U.V. Balakrishnan	
	J.E.R	A.R.O	E.E.R
	Tr: V.V.	ckd:	FIG. 12

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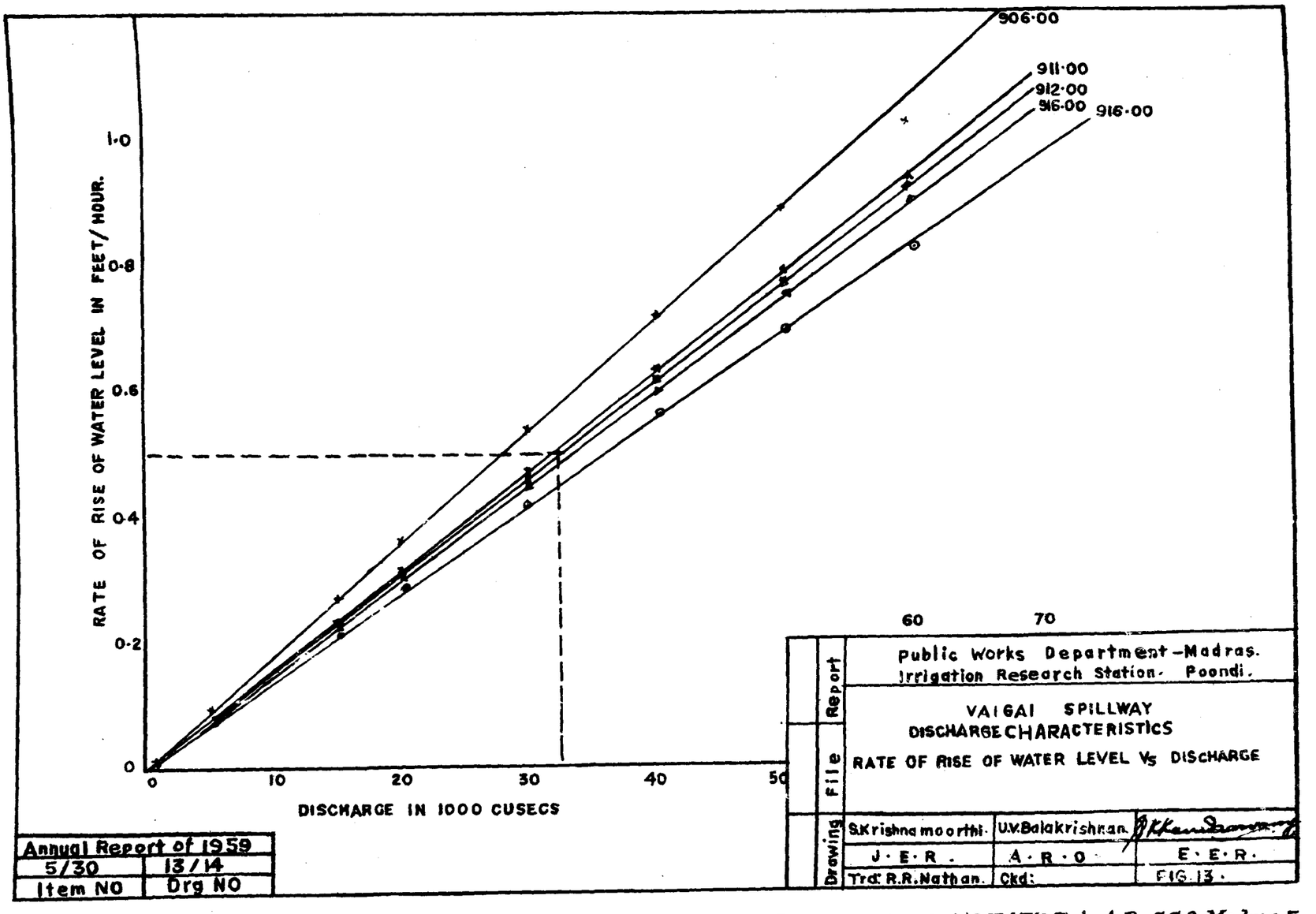

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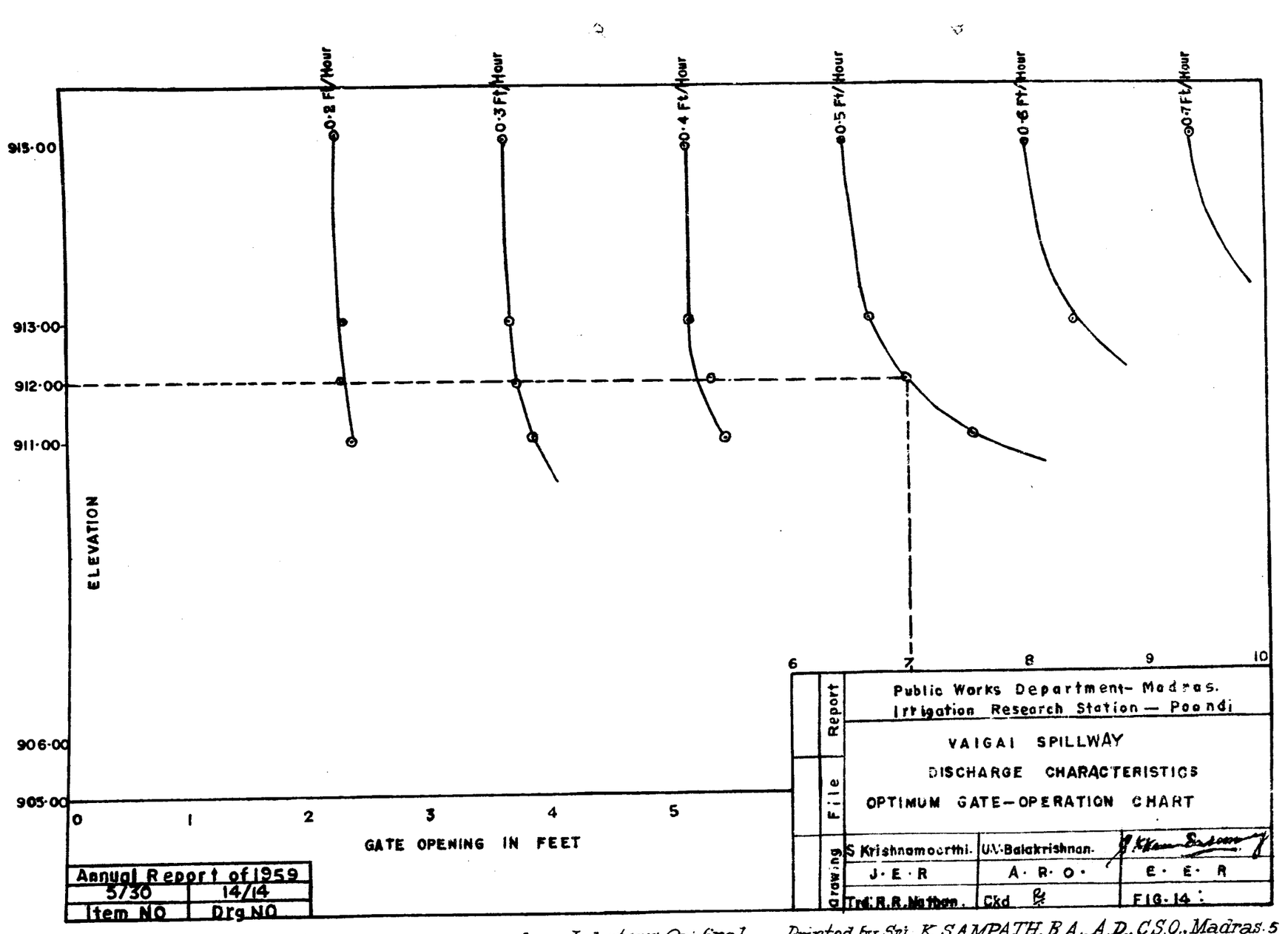

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High Coefficient Weirs

1. SYNOPSIS

This report deals with the studies conducted to convert the existing profiles of three surplus weirs of tanks into high coefficient weirs. The relation between the Maximum Water level, length of the weir and run-off to be surplussed over the weir is briefly discussed and the factors affecting the discharge coefficient of a weir have been given. A short description of the model set-up is also given. The method of experimentation has been explained and the results analysed. The advantage of adopting a high coefficient profile is brought out by working out an example.

2. INTRODUCTION

2.1. Generally agricnlture in South India, particularly in Madras State, mainly depends on tank irrigation System and there are innumerable tanks, mostly small ones used for irrigation purposes. So much so, the problems of storage, maintenance and safety of such tanks require the utmost attention. Periodical repairs to the tank bunds and restoration of the tank capacities once in a few years will satisfy the maintenance part of the tanks. The safety of a tank depends to a larger extent on the efficiency of the "Surplus Weir" (an important componant of a tank) meant for the disposal of the run-off which is in excess of the maximum storage capacity of the tank.

2.2. Every tank has been designed for a particular maximum storage capacity and in flow. Thus, the Maximum Water Level for the tank is fixed and correspondingly the T. B. L. is also fixed allowing sufficient free board. Water can be stored in the tank upto the crest level of the surplus weir i.e. F. T. L. and the excess should be discharged over the surplus weir. If it is desired to increase the storage capacity of the tank, the only and simple possible way is to raise the crest level of the surplus weir. In such a case, in order to pass the estimated run off, either the M. W. L. should be raised as the head over the crest of the weir is reduced owing to the rise in the crest level, or the length of the weir should be increased to pass to the estimated run-off with the original maximum water level.

2.3. The explanation for the above statement is as follows:

$$Q = C\,L\,H^{3/2} \text{ assuring free flow condition}$$

where

C - Coefficient of discharge

L - Length of weir

H - Head of overflow over the crest of the weir.

Since the expense of water in the tank is very large and also the water is stored up in the tank, the velocity of approach is assumed to be negligible,

2.4. Now for a particular type of weir the discharge is a funtion of the length of the weir (L) and head of overflow (H) assumbing the coefficient of discharge (C) to be constant. As previously mentioned if the crest level of the weir is raised then for that fixed maximum water leval the head over the crest of the weir will be reduced and it will not be possible to pass the estimated discharge, unless the length of the weir is increased. On the other hand if it is desired to keep the length of the weir constant then the maximum water level will have to be raised. In such cases the only possible way to pass the estimated discharge over the surplus weir without either increasing the length of the meir or raising the maximum water level appreciably is to increase the coefficieut of discharge of the weir by some means or other.

2.5. The coefficient of discharge for a weir depends on several factors, the more important ones being:—

(1) The shape of the weir

(2) The height ef the weir

(3) The head of flow over the crest of the weir

(4) Downstream depth of water

(5) Velocity of approach.

2.6. The variation of the coefficient of discharge with the ratio $\frac{H}{H+P}$ where H is the head of overflow and P is the height of the weir is very small and can be neglected. The deviation in the value of the coefficient is about 4% for $\frac{H}{H+P}$ ratio within the range of 0.3 to 0.6. In almost all the tanks the height of the weir will be within about 4"0" and the head of overflow will be about 2"0". Hence the ratio $\frac{H}{H+P}$ is about 0.33 (within 0.3. to 0.6) and the deviation in the coefficient of discharge is negligible. So, this aspect need not be considered.

2.7. The coefficient of discharge varies slightly with the head of overflow. For the design purposes, normally the maximum head is taken and the variation of the coefficient in the neighbourhood of the maximum head is usually vary small and can be neglected. In the case of many tanks the site conditions are more or less such that only free flow condition is established and the submergence of the nappe is a rare case. Therefore, the discharge characteristics of the submerge flow is not discussed in this report. As already mentioned, the velocity of approach in the case of tank weirs is very small and can be neglected.

2.8. The main factor that influences the coefficient of discharge is the shape of the weir section. By changing the shape of the weir suitably, the coefficient of discharge can be increased considerably. So, the only easiest way of increasing the coefficient of discharge for a tank weir is to

adopt a high coefficient profile for the new constructions or to alter suitably the shape of the existing weirs without meddling much with the already existing structure.

Now the problem to be solved by the model studies is to convert the existing profiles of three surplus weirs of VakadaiMukur Hisa Tank in North Arcot District, into high coefficient profiles to pass the maximum run-off for a raised F. T. L. without raising the maximum water level appreciably. As a result of raising the F. T. L. of tank, the storage capacity of the tank is proposed to be increased.

3. MODEL SET-UP

The experiments were conducted in a flume of width 2' 6". The model for one weir was constructed to a scale of 1/5 and for the other two to a scale of 1/10 and studies conducted. All the requisite appurtenances for flow measurements and gauge readings etc., were fixed in the flume. The details of the model set up are given in Figure-1.

4. EXPERIMENTS AND RESULTS

4.1. The studies were started to arrive at a high coefficient profile for the existing weir, bearing in mind that the existing structure should not be tampered with as far as possible. To begin with, a modal of a typical weir with a rectangular cross-section was constructed and the variation of coefficient of discharge with the head of overflow was observed by allowing different discharges (Vide Figure-2). The discharge coefficient of the weir can be increased by altering the shape of the weir in two ways, one by giving a smooth curve on the downstream side and the other by giving an upstream alone. If the downstream shape of the weir conforms to smooth curve, preferably to the shape of the lower nappe of the falling water it will establish a smooth flow over the weir thereby increasing the discharging capacity of the weir. Therefore, a smooth curve from the raised F. T. L. was given on the downstream side of the weir and the variation of the discharging coefficient was found out for different over flow heads. Of course, this has increased the discharge coefficient, but to a very small extent. If an ogee curve is given then the coefficient of discharge would have been increased still further, but to achieve this the existing structure will have to be completely altered, which is not desirable. It is obvious from this that the downstream curve aids more in attaining a smooth flow rather than getting a higher coefficient. Secondly an upstream slope was tried. Without changing the downstream shap of the weir only an upstream slope was given by connecting the original crest level on the upstream side and the proposed crest level on the downstream side. It was observed that this resulted in considerable increase in the discharge coefficient (Vide Figure-2).

4.2. By referring to Figure-2 it can be concluted that an upstream slope has much effect in increasing the discharge coefficient while the downstream curve side not so much in increasing the cofficient of discharge as in establishing a smooth flow over the weir. Therefore, in the further studies to be conducted with respect to the three surplus weirs mentioned above, it was proposed to combine these two modifications to get a higher discharge cofficient as well as a smooth flow. Moreover it was proposed to adopt an

upstream slope of 45° as it has been proved that an upstream slope of this angle will have the maximum effect in increasing the coefficient of discharge.

4.3. The model of the first weir was constructed to a scale of 1/5 with the existing profile. Diffeaent discharges were allowed to flow over the weir and the corresponding overflow heads were measured. The discharge coefficients for different discharges and the corresponding overflow heads were calculated. A curve showing the variation of the discharge coefficient with the head of overflow was drawn (vide Figure-3). Then the profile of the weir was mobified as proposed. An upstream slope of 45° was given from the original crest level of the weir till it meets the proposed crest level and from that point onwards a smooth circular curve was given to join with the downstream end. Studies were conducted as before and the variation of the discharge coefficient with the head of overflow was plotted in the same figure. It can be found from the curves that for the modified profile the coefficient of discharge has been increased considerably. Moreover, from the figure it can be noted that the existing structure is not altered much.

4.4. Studies were conducted exactly on the same lines in the case of the other two weirs also, construeted to a scale of 1/10 and the modified profiles were found to increose the discharge coefficient considerably (vide Figures 4 and 5). But in the case of the second weir (vide Figure-4) for the first proposed high coefficient profile the increase in the coefficient of discharge was not much. So a secend profile as shown in the figure was tried and this gave a better result eventhogh the existing structure is to be tampered with slightly.

4.5. The advantage of using a high coefficient profile is illustrated by the followiug example:

Taking the case of the first weir (Figure-3).

As per the original condition,

M. W. L. ... El. 367.35

F. T. L. ... El. 364.62

Head of overflow = 367.35—364.62

= 2.73 ft.

From the discharge coefficient curve for the original profile, the discharge coefficient for a head of 2.73 seet = 2.68.

The discharge per foot run of the weir

$$= 2.68 \times (2.73)^{3/2}$$

$$= 12.1 \text{ cuses}$$

As per the modified condition,

proposed M. W. L. ... El. 368.35

proposed F. T. L. ... El. 365.85

The head of overflow ≡ 363.35—365.85

≡ 2.50 feet.

Suppose the same profile is adopted for the proposed weir also. Then the coefficient of discharge for a head of 2.5 feet is 2.67 as taken from thecurve for the original profile.

The discharge per foot run of the weir

$$\equiv 2.67 \times (2.5)^{3/2}$$

$$\equiv 10.5. \text{ cusecs.}$$

This discharge per foot run of the weir is less than the original discharge of 12.1 cusecs.

It means that it will not be possible to pass the estimated run off unless the maximum water level is raised still higher or the length of the weir increased further.

Alternately, if we adopt the modified high coefficient profile, the corresponding discharge coefficient for a head of 2.5. feet is 3.9 (taken from the curve for the modified profile.)

The discharge per foot run of the weir

$$3.9 \times (2.5)^{3/2}$$

16.4 cusecs.

which is greater than the original discharge of 12.1 cusecs.

Therefore, it is obvious that with the modified profile the maximum water level will not rise even to the proposed level of El. 368.35 with the same length of the weir to pass the discharge of 12.1 cusecs per foot length of the weir,

4,6. Likewise, the utility of the high coefficient profile can be proved in case of the other two weirs also. A comparative statement showing the increase in the discharge for all the three weirs is given (vide statement-1),

5. CONCLUSION

It has been proposed to increase the Storage capacity of a tank by raising the crest level of three surplus weirs in the tank. Studies were conducted in this station with a view to modify the existing profiles into high coefficient ones without much altering the existing structure so that the estimated run-off can be passed the same length of the weir and without approciable increase in the maximum water level. The curves showing the variation of the dircharge coefficient with the head of overflow both for the original profile and the modified profile in the case of all the three weirs are given. Also a statement showing the comparative increase in the discharge is furnished.

6. REFERENCES

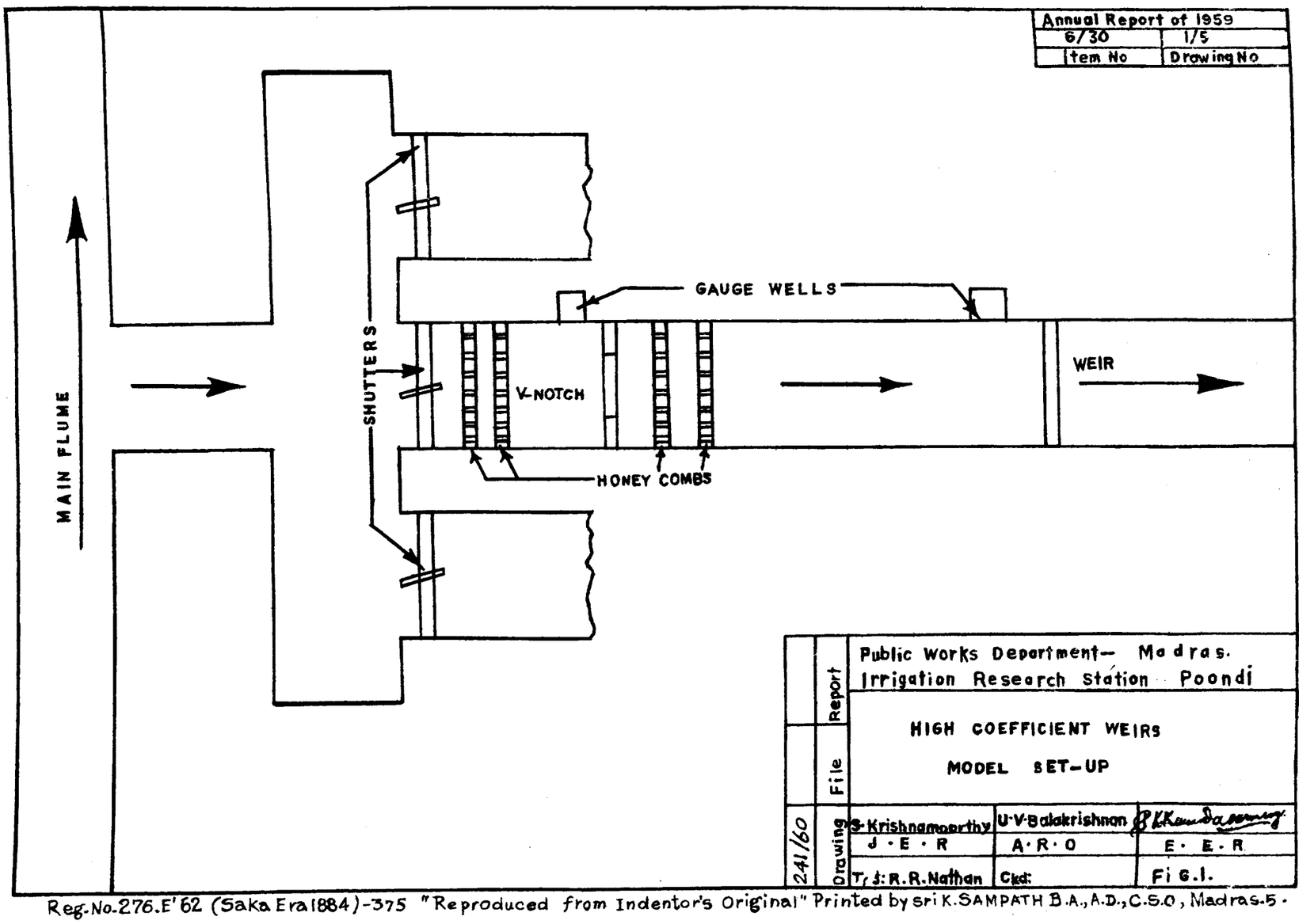
(1) Simple profiles for tank weirs—Irrigation Research Station—Annual Report for 1953,

(2) Studies of Crests for overfall dams—Bulletin 3. Boulder Canyon Project—Final Reports—United States Bureau of Reclamation.

STATEMENT 1

Weir No.	Design Head (H) M.W.L. — F.T.L.		Discharge Coefficient		Discharge $q = C H^{3/2}$ (in cusecs)		REMARKS.
	Original	Proposed	Original	Proposed	Original	Proposed	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	367.35-364.92 2.73 feet	368.35-366.85 2.5 feet	2.63	3.9	12.1	15.4	The maximum water level will not rise even to the proposed level.
2	367.35-364.80 2.55 feet	368.35-365.85 2.5 feet	2.63	3 9	10.9	15.4	
3	367.35-364.79 2.66 feet	363.35-365.85 2.5 feet	2.74	3.82	11.2	15.2	

N B.: The proposals of the surplus weirs in respect of the M.T.L. and Maximum water level are as per data furnished by the field officers.



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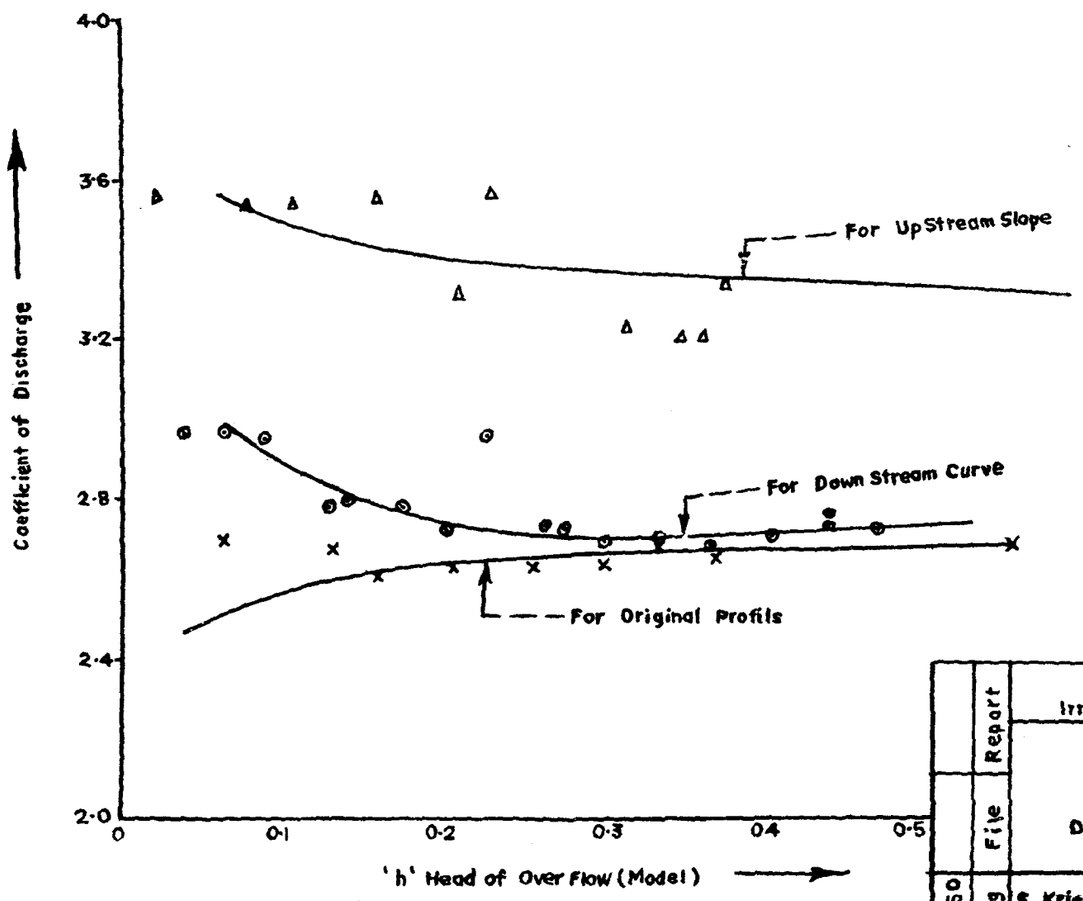
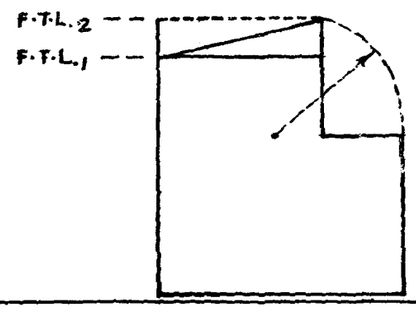
6/30	2/5
Item No	Drawing No





Report	Public Works Department — Madras. Irrigation Research Station - Poondi.	
File	HIGH COEFFICIENT WEIRS. DISCHARGE COEFFICIENT Vs HEAD	
244/60 Drawing	S. Krishnamoorthy / U.V.B. Lakrinan.	P.K. Ramaswamy
	J·E·R. / A·R·O·	E·E·R
	Trd C.S.K.Pathy. / Ckd:	Fig. 2.

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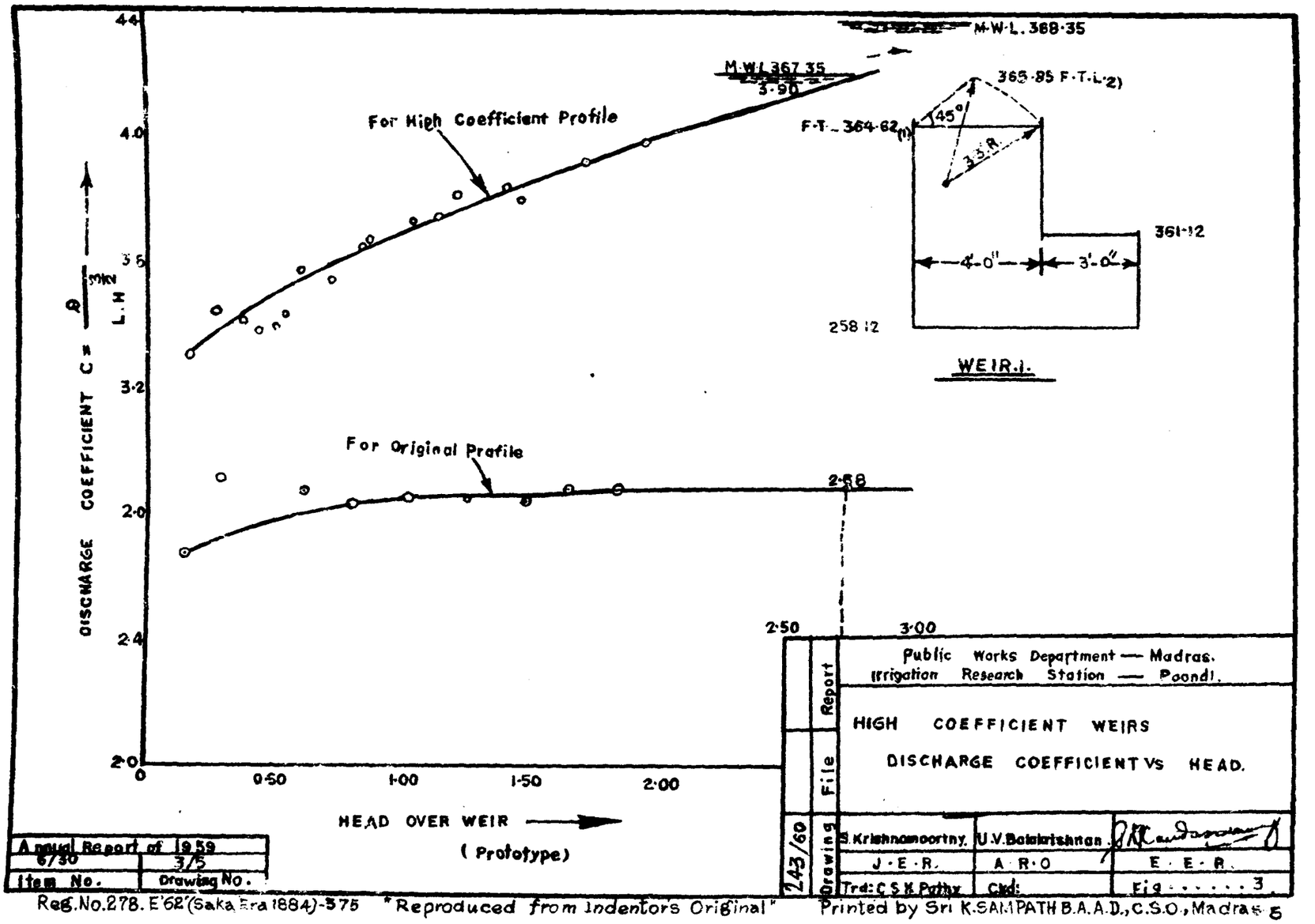

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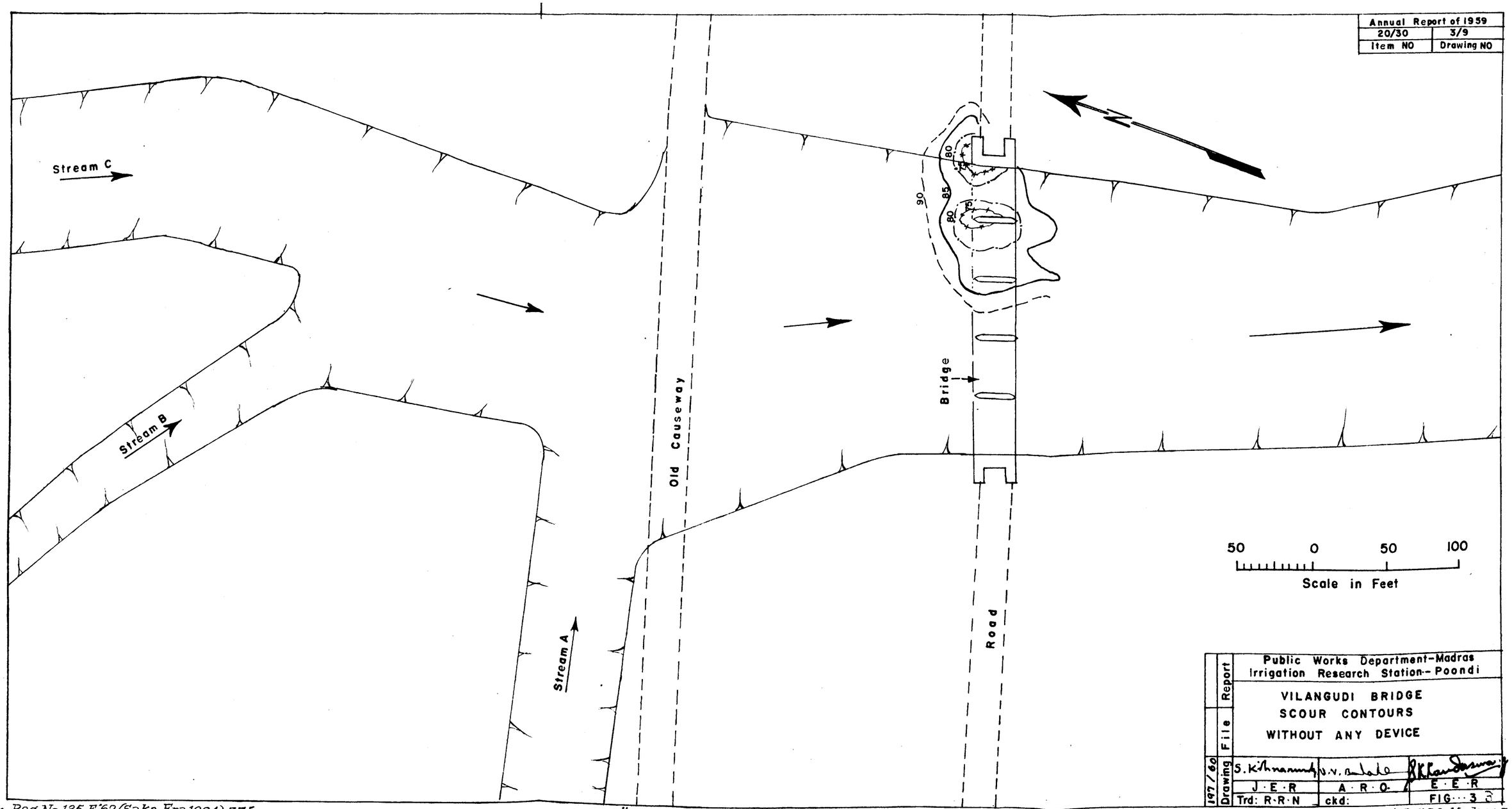

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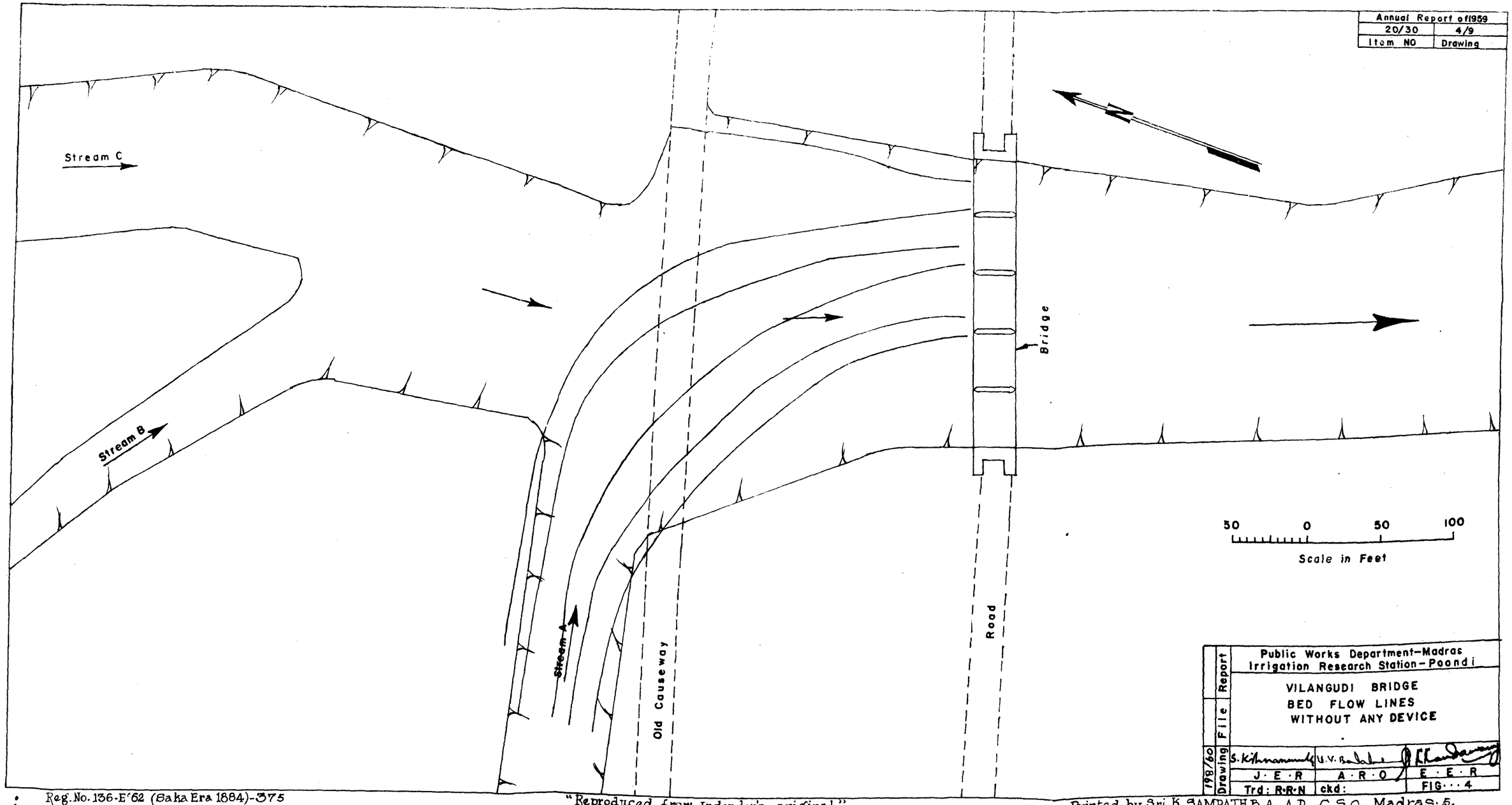

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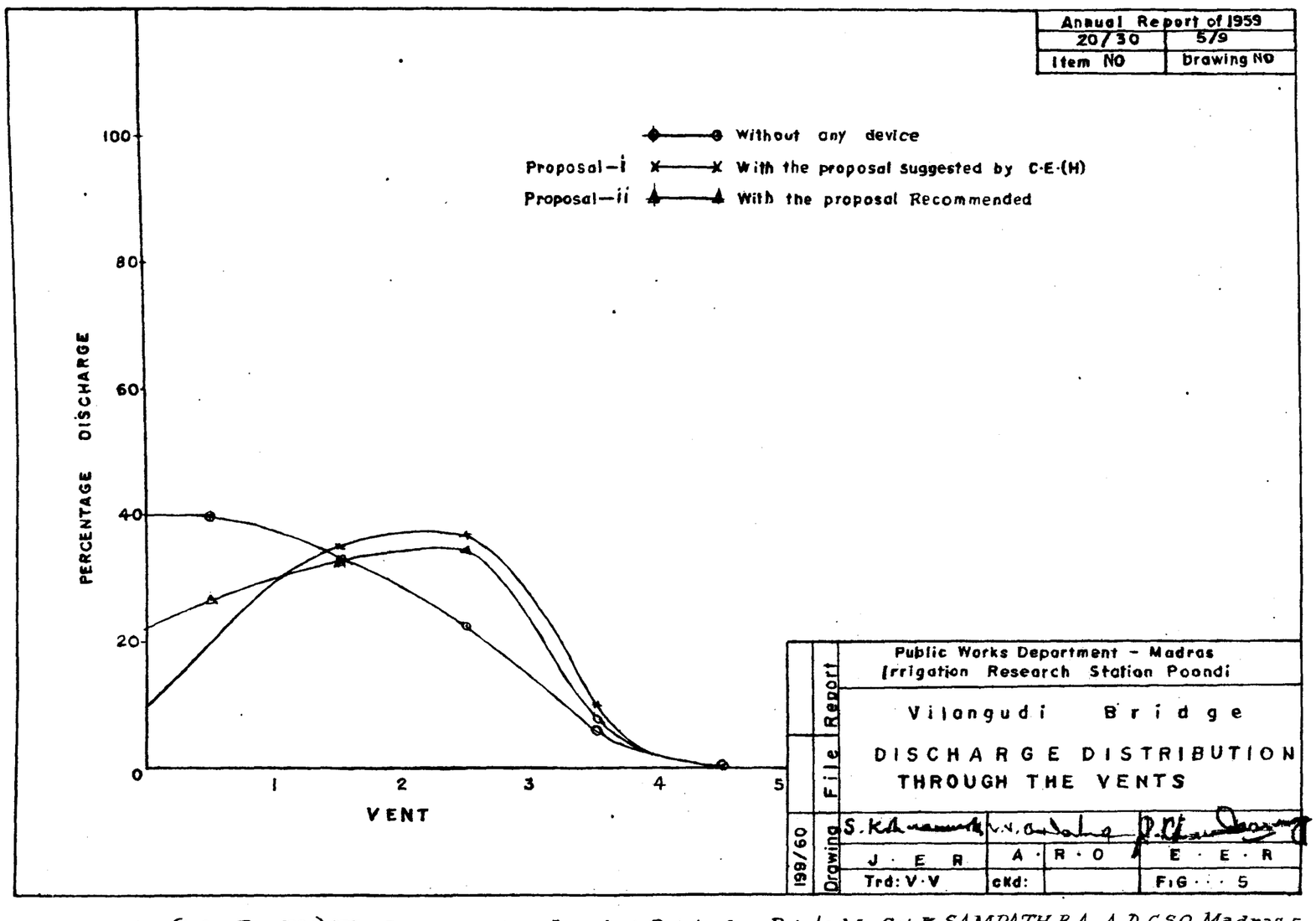

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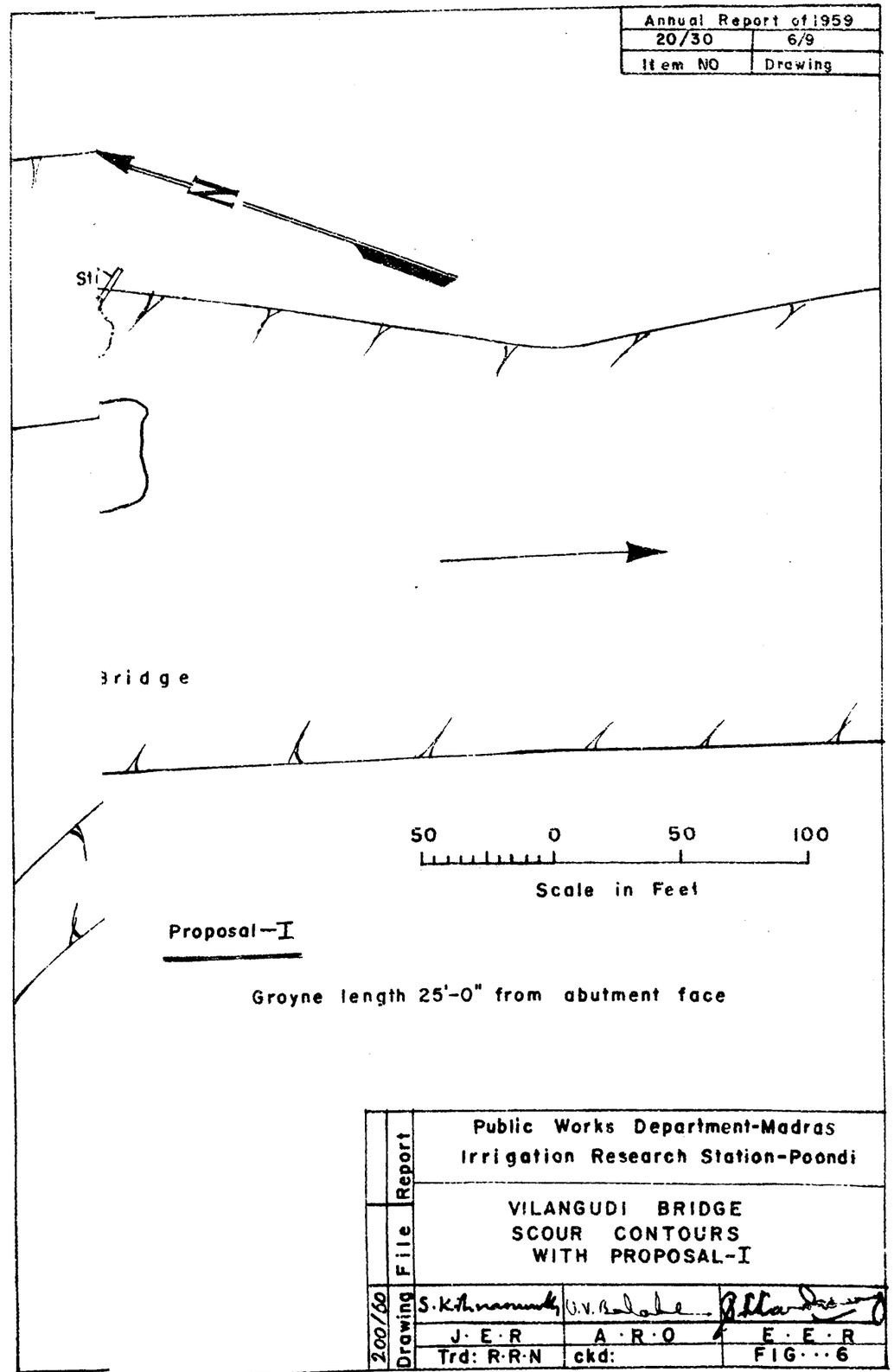
Item NO	Drawing



Report	Public Works Department-Madras Irrigation Research Station-Poondi		
File	VILANGUDI BRIDGE SCOUR CONTOURS WITH PROPOSAL-I		
200/60 Drawing	S. Krishnamurthy	U.V. Balaba	
	J·E·R	A·R·O	E·E·R
	Trd: R·R·N	ckd:	FIG···6

Reg. N

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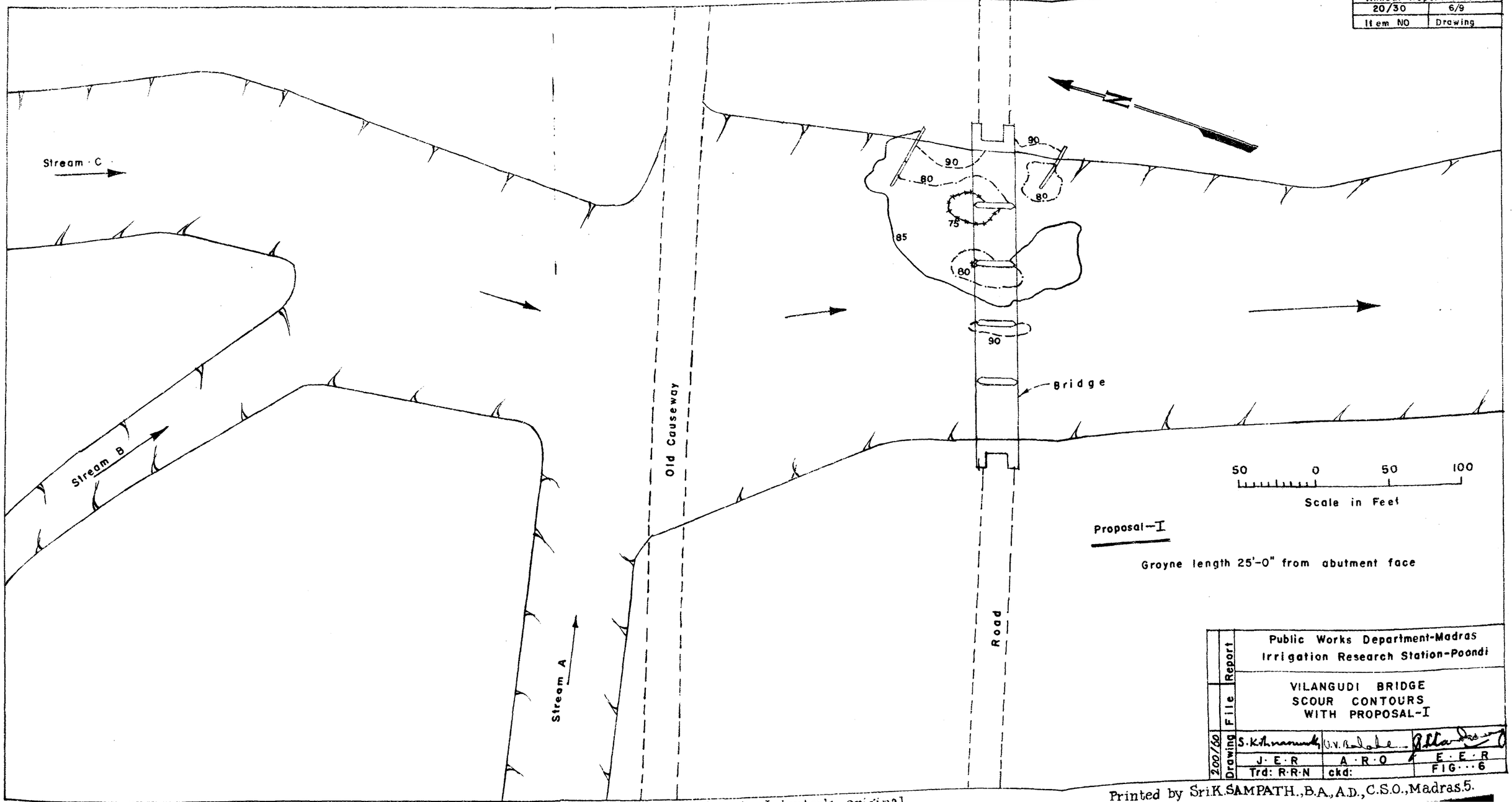
Annual Report of 1959	
20/30	6/9
Item NO	Drawing



Report	Public Works Department-Madras Irrigation Research Station-Poondi	
File	VILANGUDI BRIDGE SCOUR CONTOURS WITH PROPOSAL-I	
200/60 Drawing	S.K.Thanmunk	G.V.Rao
	J·E·R	A·R·O
	Trd: R·R·N	ckd:

E·E·R
FIG···6

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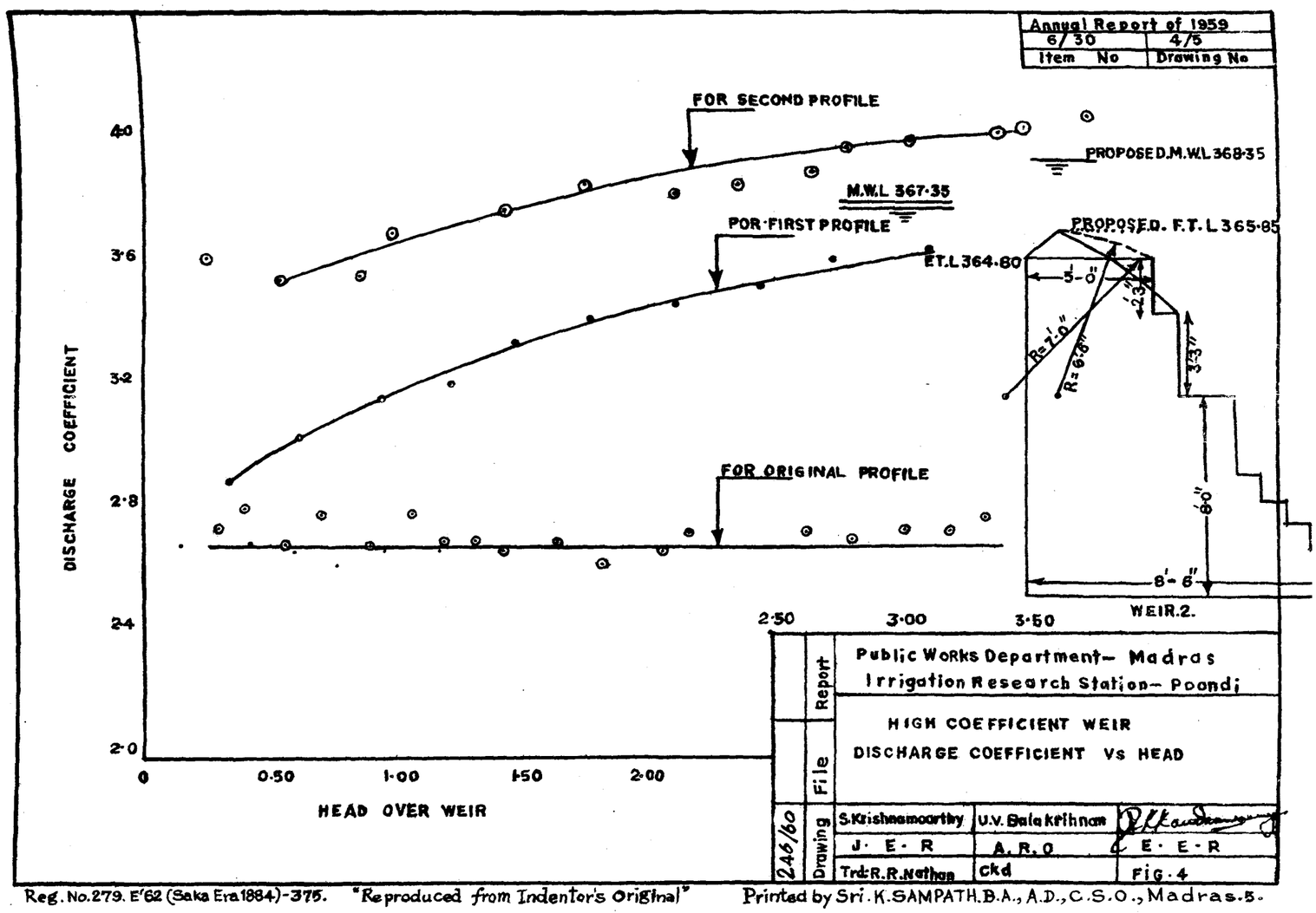

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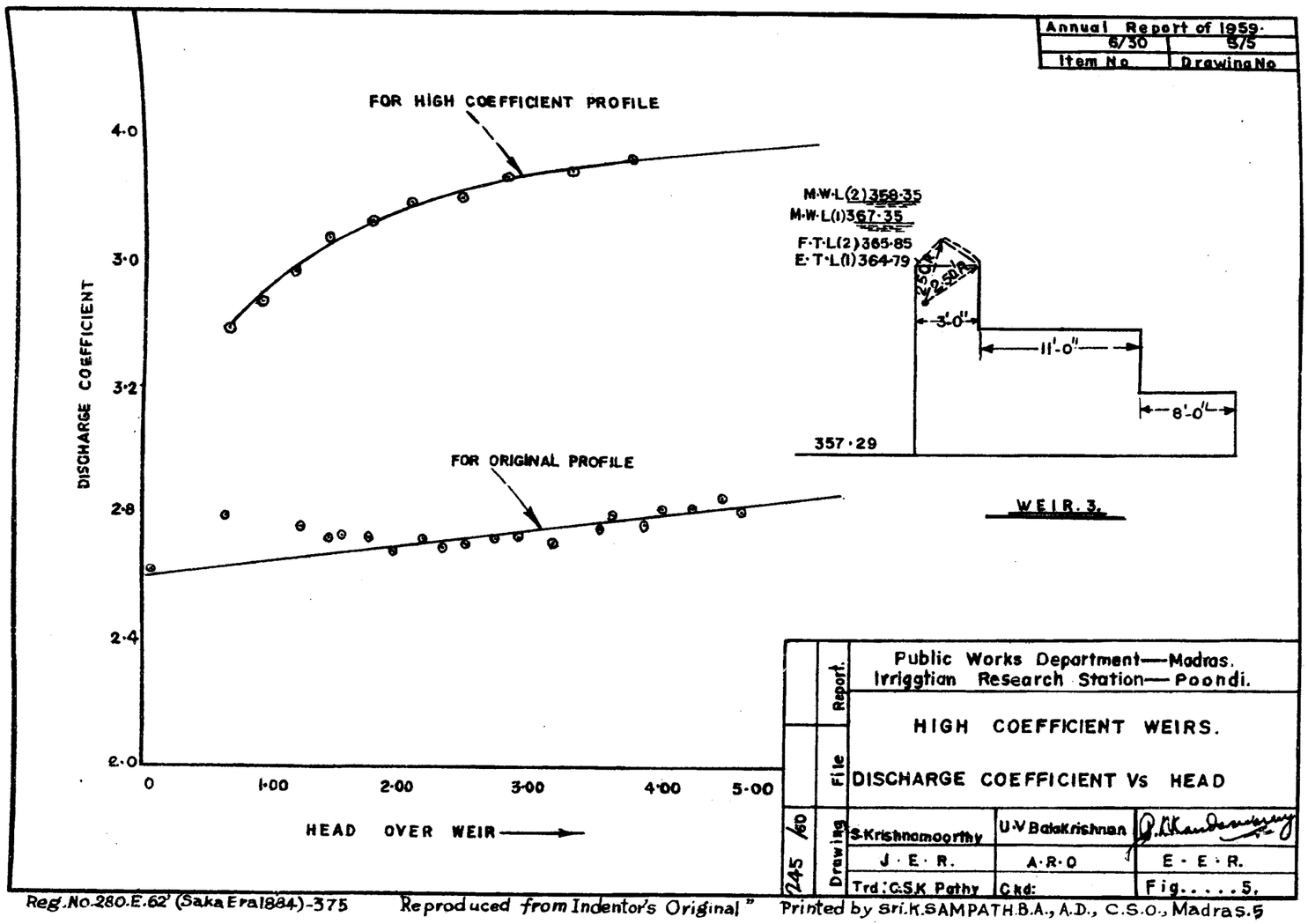

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PARAMBIKULAM—ALIYAR PROJECT

ALIYAR DAM SPILLWAY

MODEL STUDIES

River Training Works And other Allied Aspects

1. Synopsis:

The spillway of Aliyar Dam is located on the right fiank of the Aliyar River, unlike the conventional way of locating it in the river course. The spillway consists of ogee section and chute section separated by a horizontal apron, the height of the former being about 1/5th the height of the latter. A new surplus course is proposed to be excavated, connecting the spillway and the existing Aliyar River. It is desired to study the river training works, the flow patterns downstream of the spillway upto the Pallivalangal cnicut and other allied problems.

2. Introduction:

The Aliyar Dam is essentially a part of the comprehensive Parambikulam-Aliyar Project. Apart from impounding the available Aliyar flows this dam serves as a balancing reservoir to receive and utilise the waters directed from Birar, Sholayar and Parambikulam basins for irrigating the ayacut on the eastern plains of western ghats through two canals, viz. upper canal and Pollachi canal, taking off from the left and right flanks of Aliyar Dam respectively. Fig 1. The spillway of Aliyar Dam is located on the right flank of the Aliyar River so that the existing high mound of rock outcrop is taken advantage of the economising the height of spillway to be constructed. Fig. 2 This proposal proved to be cheaper than the one of having the spillway in the river course,

The spillway consists of ogee section for a height of 18′ from the crest El. 1040.00 down to El. 1022.00 to El, 930.00 seperated by a horizontal apron 25′0″ long at El. 1022.00. Fig. 3. In the chute, spillway the existing solid rock will be dressed to a slope of 2: 1. There are 11 vents of 30′0″ each separated by 9.00 thick piers in the spillway. The spill channel at the toe of the spillway is 420′ wide and it mediates a bed of about 90° and a change of width to 200′ and runs parallel to the axis of the dam till its conflence with the Aliyar river. A bridge consting of 7 vents of 30′ each has already been constructed across the Aliyar River downstream of the dam at about 300′ below the point of confluence of the new surplus course with the Aliyar River, to carry the diverted road connecting Pollachi and Valparai Anicut known as Pallivalangal anicut exists across the Aliyar River at about 700′ downstream of the bridge and it serves as the poinl of control for the flow characteristics in the surplus course downstream of the spillway.

3. Problem :

The scheme of studies is as follows :

(*a*) To observe the effectiveness and efficiency of the energy dissipating arrangements for smooth surface in the chute spillway section finalised in the sectional model, in 3-Dimensional model and to effect any change in the arrangement if found necessary.

(*b*) To observe the water surface profiles from the toe of the spillway upto the anicut for maximum flood discharge conditions.

(*c*) To evolve suitable radii of curvature at th e bends in the surplus course.

(*d*) To correct the flow pattern at the bend in the river course and to snake the water evenly pass through the vents of the bridge without endangering its safety.

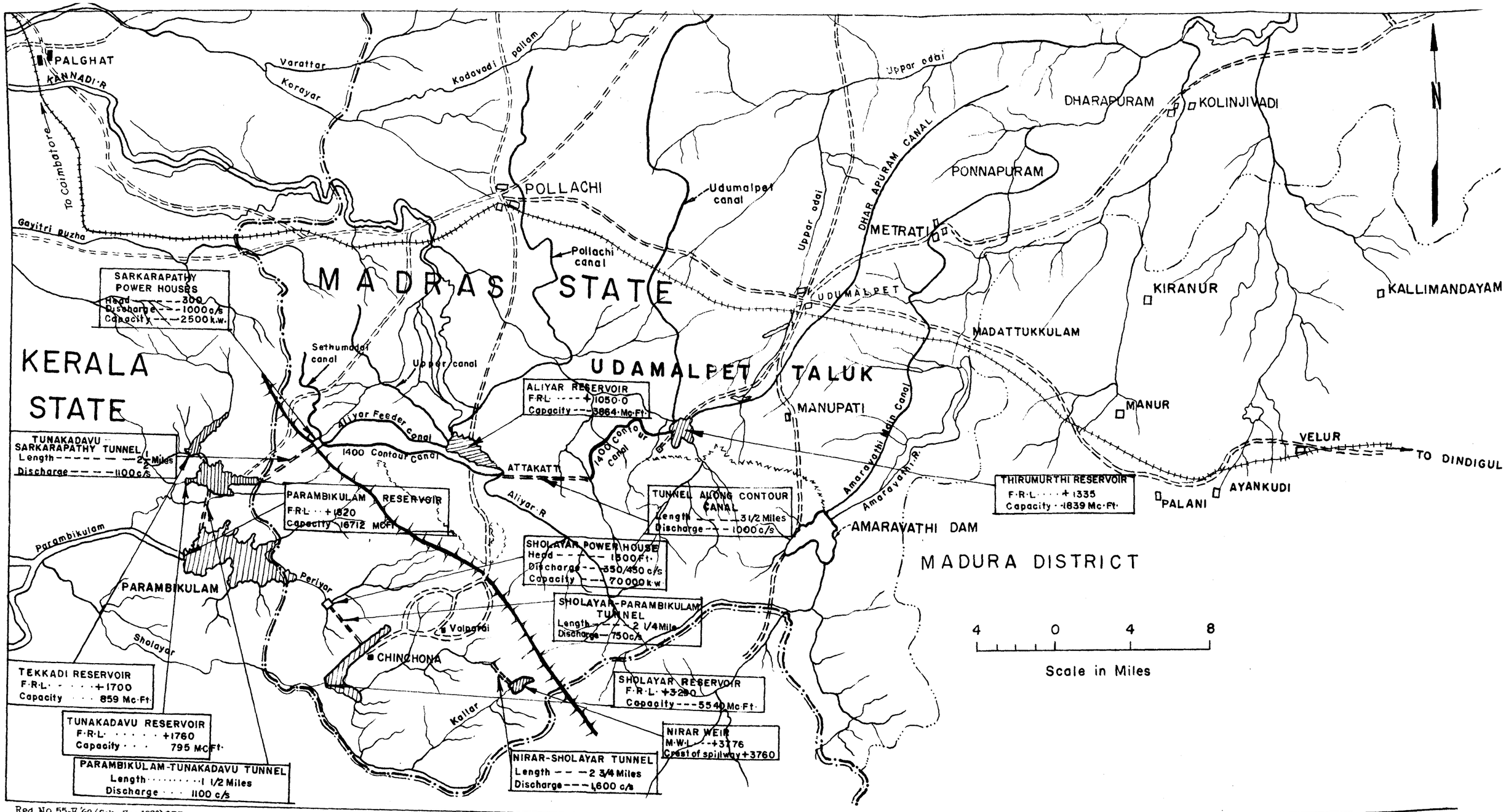
(*e*) To test the efficiency of the energy dissipating arrangements proposed for the smooth surface with 2: 1 slope in the chute section for the condition when the existing rock surface forms the profile of the spillway in the chute section.

(*f*) To analyse the possibility of having the horizontal apron connecting the ogee spillway section and the chute spillway section in different levels along the length of the spillway to suit the rock levels obtained in the prototype with spacial reference to minimising the cost of concreting for the apron.

(*g*) To observe the distribution of pressure along the surface of the spillway when the existing rock surface forms the profile in the chute section.

4. Model :

A geometrically similar 3-dimensional model was constructed to a scale of 1/60. A portion of earth dam on the left and a portion of the masonry dam on the right of the spillway were also represented in addition to the spillway section of 11 vents of 30'-0" wide each separated by piers 9'0" thickness. The profile of the spillway adopted for section model was followed in the 3-D model also. The equiangular easewaters evolved in the sectional model were adopted in rear of the spillway piers. The surplus course in rear of the spillway, the connection between the surplus channel and the Aliyar River the bridge constructed dowustream across the Aliyar River, the existing Pallivalangal anicut below the bridge and river course for half a mile length below the Pallivalangal Anicut were represented in the model. Suitable gauges and weirs fixed in the model at the required places for the measurement of head of flow and discharge. The river sluices were also constructed on the left side adjacent to the spillway section.



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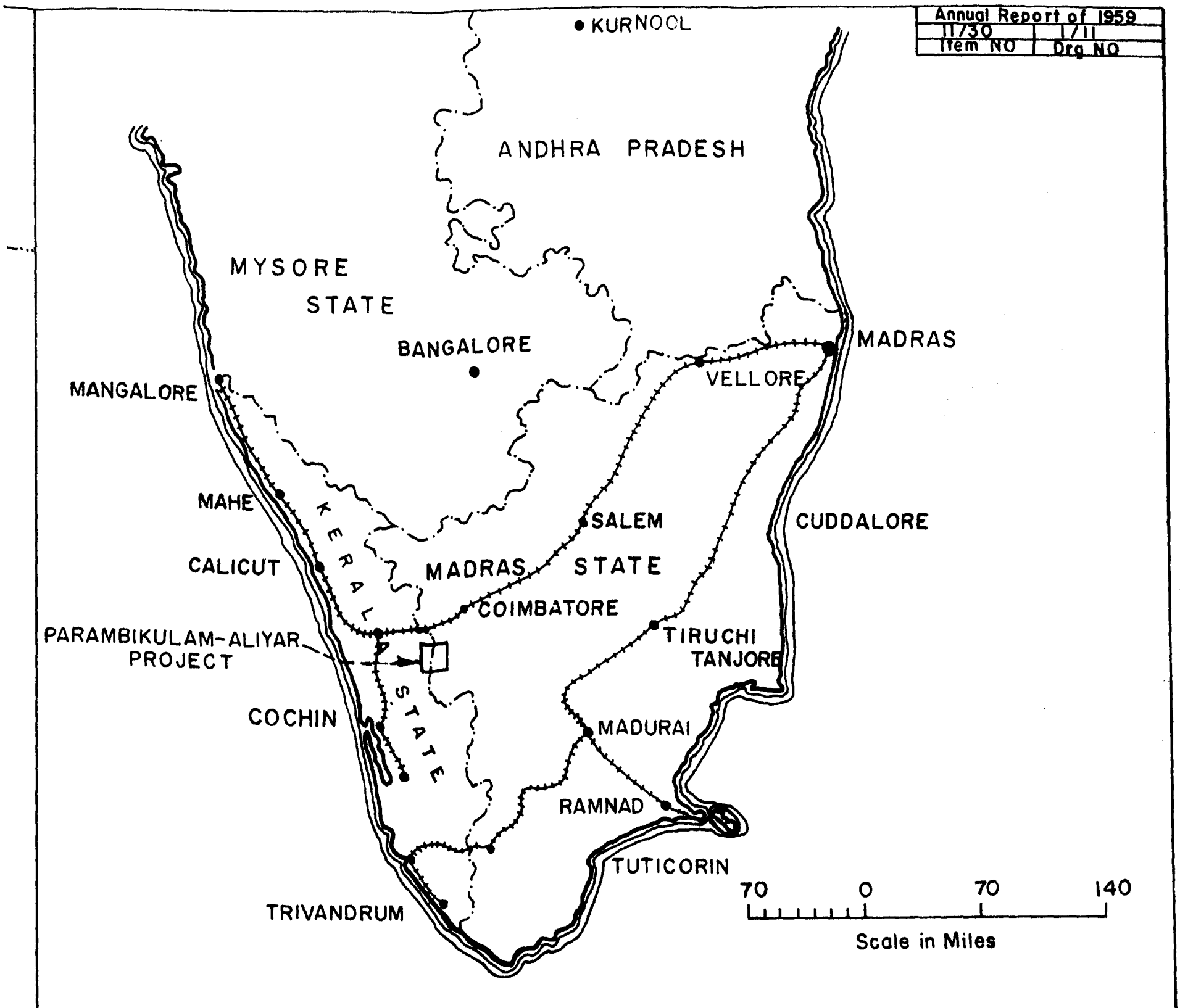

Public Works Department–Madras
Irrigation Research Station–Poondi

PARAMBIKULAM ALIYAR PROJECT
INDEX MAP

21/6/60	Drawing	A. Shanmugam	
		J·E·R	A·E·R
		Trd: V·V·	ckd:

		E E R
		FIG ··· 1

Report
File

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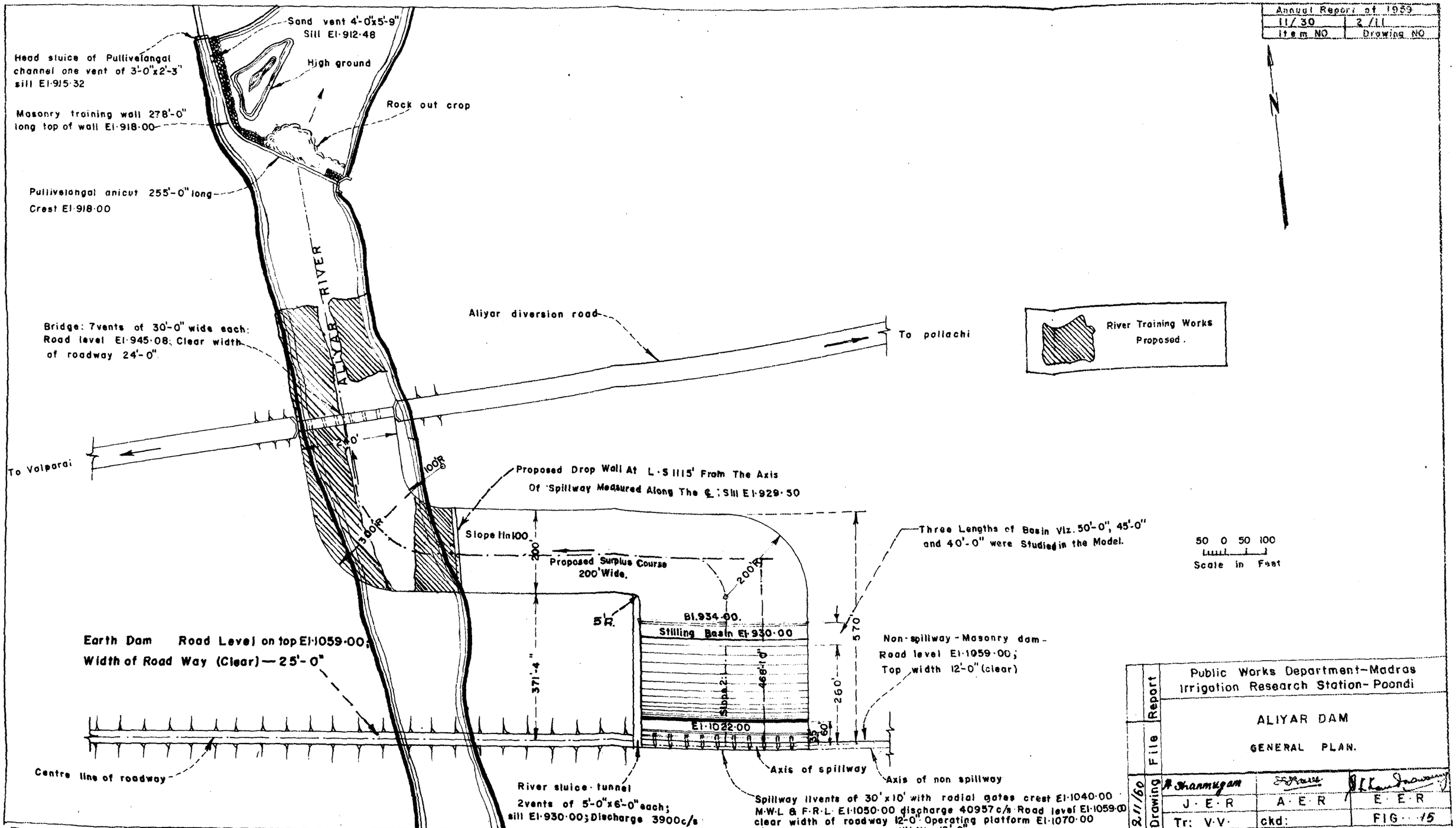

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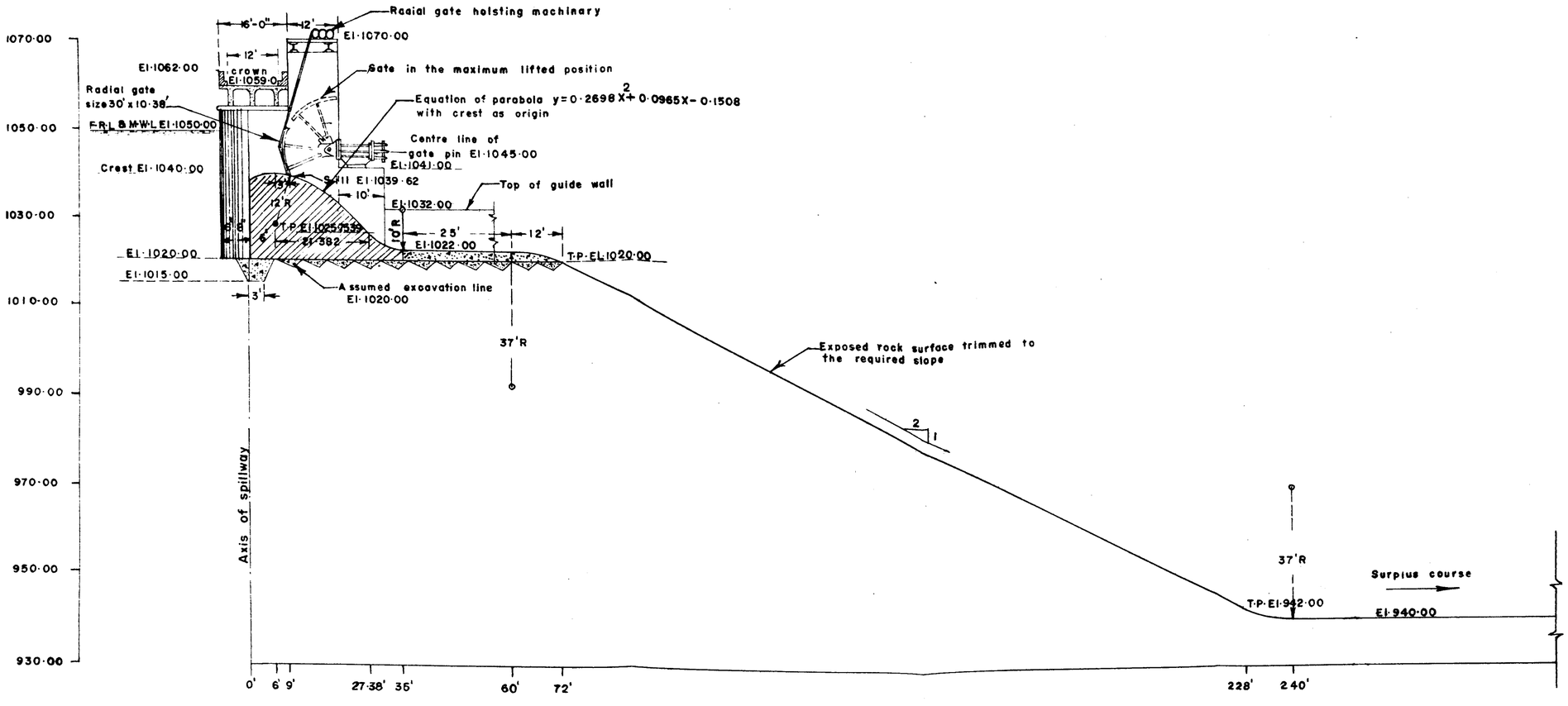
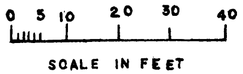


NOTE

1. There is an average depth of 5' of loose boulders to be removed suitable leveling of the surface is to be done during execution
2. The actual height of body wall will vary along the length of spillway. The max section is indicated in this drawing.



Report	Public Works Department – Madras Irrigation Research Station-Poondi		
File	ALIYAR DAM SPILLWAY SECTION (Original design)		
2/2/60 Drawing	A Shanmugam		
	J·E·R	A·E·R	E·E·R
	Tr: V·V·	ckd:	FIG···3

Reg No 50-E'62 (Saka Era 1883) – 375.

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PERIYAR RIVER TRAINING WORK AT THE SITE OF CUMBUM VALLEY PANCHAYATS WATER SUPPLY SCHEME

1. Introduction

1.1. The water supply scheme for the Cumbum Valley Panchayats envisages the utilisation on small flow of about 2 cusecs from the tail race discharge issuing from the turbine units of the Periyar Power house. The headworks of the water supply scheme have been located in a reach of the Periyar River which is meandering in a serpentine course. The course of the stream takes a double curve of acute nature at this site where it runs with steep bed slope of 1 in 250 in a deep valley. Consequently the deep course of the stream for normal flow in the river hugs to the outside of the numerous bends wherever they occur, shifting from one side to the other alternatively. It is the fear that this deep course may take a natural cut across the slender bank on which are located the intake structure sump well and pump house of the water supply system. If such a contingency arises not only will these structures be in danger of collapse, but also the intake structure will be cut away from the deep course that would be thus formed difeating the very purpose of the water supply system. In order to obviate such a contingency, the course of the stream will have to be trained in a suitable way both for protecting the structures of the water supply schemes and for feeding the intake structure constantly with adequate quantity of water. The droblem was referred to this station to devise suitable river training works for the purpose.

2. Data

2 1. *Periyar River*:

(*a*) Maximum flood discharge ... 3200 cusecs

(*b*) Normal discharge ... 1600 cusecs

(*e*) Bed slode ... ... 1 in 250

2.2. *Water Supply Seheme*:

(*a*) Discharge required for water supply purposes ... 2 cusecs

(*b*) Pumping quantity ... 760 gallons per minute.

3. Model Set-up

A geometrically similar model to a scale of 1/36 is proposed to be put up with the bed and banks of the river moulded with a mixture of clay and sand in the proportion 1 : 3. Model construction is in progress.

DINDUGAL WATER SUPPLY SCHEME
RIVER SLUICE—ENERGY DISSIPATION

1. Synopsis

This report deals with the energy dissipating arrangements finalised for an earth-dam river sluice of the Dindugal Water Supply Scheme. The arrangement consists of a stilling basin of baffle type with a baffle wall three feet high and two rows of staggered blocks 2'0"x 1'0"x 1'0" fixed at the end of the rigid apron.

2. Introduction

An earth dam has been constructed across River Kodavanar, near Athoor, with intake tower arrangements to supply water to Dindugul town. In order to allow water to the irrigable lands downstream of the earth-dam a river sluice in the earth-dam has been designed and constructed to pass a discharge of 200 cusecs. Since the head of water available at the upstream of the sluice is about 52 feet under M. W. L. conditions, the velocity at the downstream portion of the sluice will be considerable and so necessary energy dissipating arrangements are to be provided.

3. Model

A geometrically similar model to a scale of 1/15 was constructed representing the entire length of the sluice in a flume of section 5'0"x 3'0". A rigid apron of length 25 feet was also provided beyond the stilling basin with solid masonry side walls. The bed was moulded with 40-60 sand for taking scour observations. A V-notch for measuring the discharge through the sluice and a tail gate to control the tail water level, when found necessary were also provided.

4. Experimeuts and Discussions

4.1. Originally model was constructed with a rectangular barrel 5'0"x 2'9" as per the plan No. P. H. 8/56 P. H. (N) 6/59 received from the field. A number of experiments were conducted to arrive at the best energy dissipating arrangement and the one which consists of a baffle wall three feet high with two rows of staggered blocks 2'x 1'x 1' at the end of the rigid apron and with a roof deflector at the exit end of the sluice found suitable and hence finalised—vide the interim rsport sent in this office No. 229 SE dated 25—5—'59.

4.2 Subsequently a new plan was received from the field with some alterations in the construction of the sluice. In this, a rectangular barrel 4'0"x 2'9" is provided, with R. C. beams 6"x 4" fixed at the intervals of six feet instead of a continuous covering of slab on top of barrel. Over this an arched roof four feet wide and three feet high has been provided for housing the water supply main, laid over the R. C. Beams—vide Figure-1. Hence fresh experiments were required to be conducted to finalise a suitable energy dissipating arrangement for this sluice.

4.3. The already existing model was reconditioned as per Figure-1. The originally finalised proposals (recommended in the interim report) were proposed to be tried and suitable alterations made as found necessary. So, a baffle wall three feet high was fixed at a distance of 37 feet from the exit end of the sluice. A rigid apron 25 feet long was provided and two rows of staggered blocks were fixed at the end of the apron the first row being fixed at a distance of 18 feet from the front face of the baffle wall. The roof deflector was omitted since the adoption of the same is not feasible in the present case. A duration of 30 minutes was taken for each trail and the model was run under M. W. L. conditions. At the end of the experiment scour profile was taken and the velocity downstream of the apron was noted. The maximum scour produced was not deep and the velocity was also low. This performance is as good as the one obtained for the energy dissipation arrangements

already finalised in the interim report. Hence, for the latest design of sluice barrel (design as furnished in Figure-1 received from the field subsequently) the already recommended energy dissipation arrangements but without the roof deflector will hold good, as one alternative.

4.4. Since the flow in the barrel is a conduit flow, the sluice will run full under high heads. Hence the R. C. beams and the water supply main will cause obstruction to flow. These obstructions will cause considerable turbulence inside the barrel especially when the velocity is high due to high upstream head. This may endanger the safety of the pipe and the stability of the R. C. beams. Moreover, this will result in a greater loss of energy and hence a reduction in discharge.

4.5. So, as the other alternative. it was proposed to have a continuous slab throughout instead of the R. C. beams at intervals thus separating the sluice barrel from the arch portion which is meant to house the water supply main alone. In this case the pipe can be carried over the continuous slab and the arch barrel can be made as a separate unit for housing the water supply main. These modifications were carried out in the model and with the same energy dissippating arrangements, experiments were conducted to compare the performances. Eventhough the depth of scour and the velocity of flow were low, severe return flow was formed inside the stilling basin. The discharge obtained was observed to be more than in the previous case eventhough the cross-sectional area of flow was less than that of the previous case. This was due to the fact that there were no obstructions to the flow which caused a reduction in velocity, with R. C. beams in position. But due to the formation of the return flow provision of a roof deflector was found necessrry.

4.6. To prevent the formation of the return flows a roof deflector projecting to a horizontal length of two feet from the exit end of the sluice with a clear opening of 2′6″ at the end and with a smooth circular curve of radius 2′6″ at the top was provided to spread the flow uniformly. Experiments were conducted under M. W. L. conditions with the same type of energy dissipating arrangements as before. In this case the scour was found to be less deep and the velocity low. Comparing the scour indicies, this proposal was found to be the best of all the three proposals tried with same type of energy dissipating arrangements. The comparative performances of the three proposals tried are given in statement-1 vide Figure-2. The finalised arrangements for energy dissipation are shown in detail in Figure-3.

5. Conclusion

As far as the energy dissipating arrangements are concerned for the sluice barrel with arched roof carrying the water main inside over R. C. beams, the arrangement, with a baffle wall three feet high fixed at a distance of 37 feet from the exit end of the sluice with two rows of staggered blocks 2′x 1′x 1′ fixed on the apron 25 feet long, the first row being fixed at a distance of 18 feet from the front face of the baffle (as already recommended in the interim report but without the roof deflector) will hold good. But the beams and the pipe inside the barrel will function as obstructions to flow of water and cause turbulance inside the barrel which will endanger the safety of the pipe and the stability of the beams. Therefore if possible it is preferable to have a continuous R. C. slan right through instead of the R. C. beams at intervals, thereby enclosing the flow passage into a rectangular barrel 4′0″x 2′9″ and separating the arched portion housing the water supply main. The following energy dissipating arrangements are suitable in this case:

(*a*) a roof deflector as shown in figure-3.

(*b*) a baffle wall three feet high at a distance of 37 feet from the exit end of the sluice.

(c) Staggered blocks 2′0″x 1′0″x 1′0″ in two rows the first row being fixed at a distance of 18 feet from the front face of the baffle wall, the clear space between the two rows being two feet.

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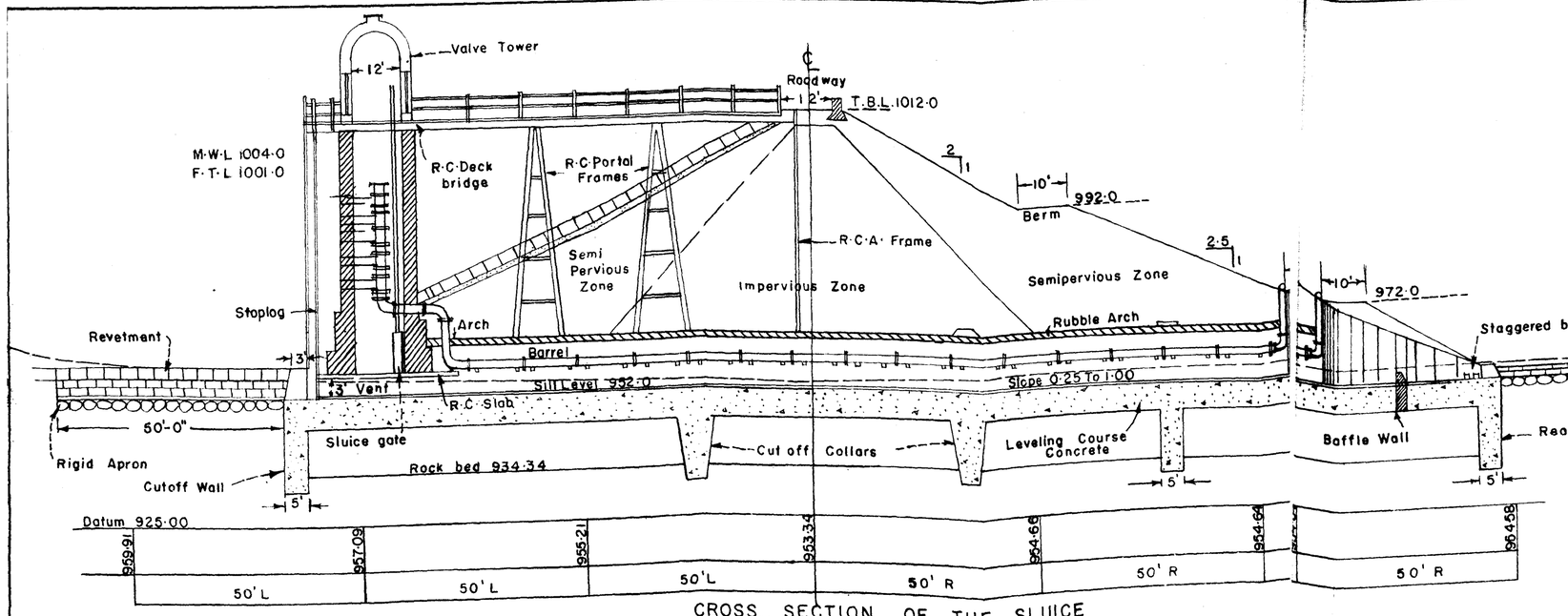
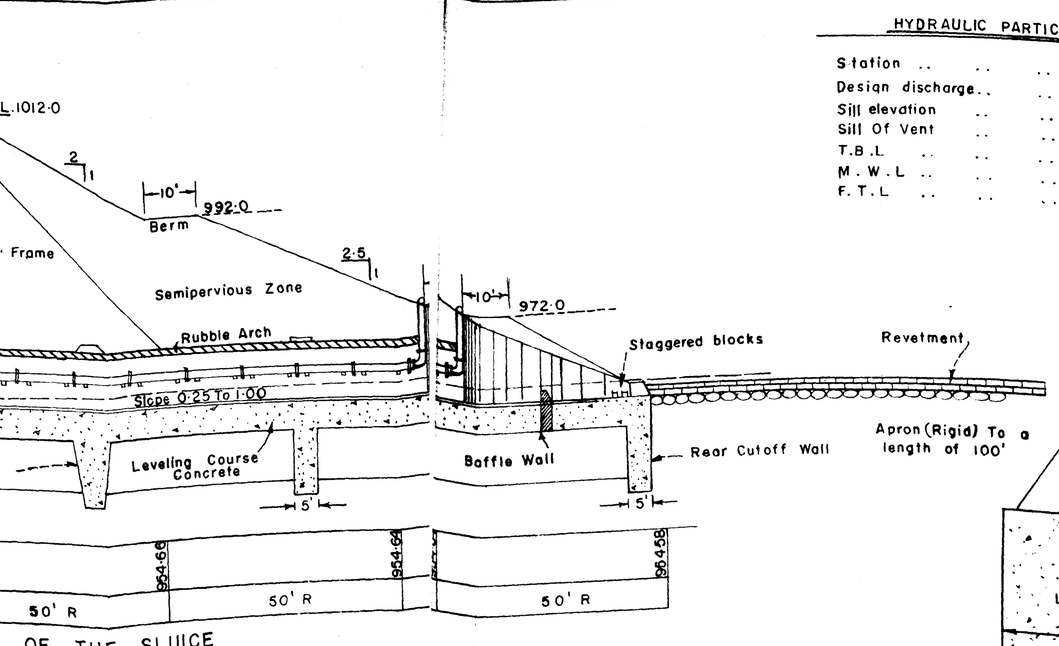


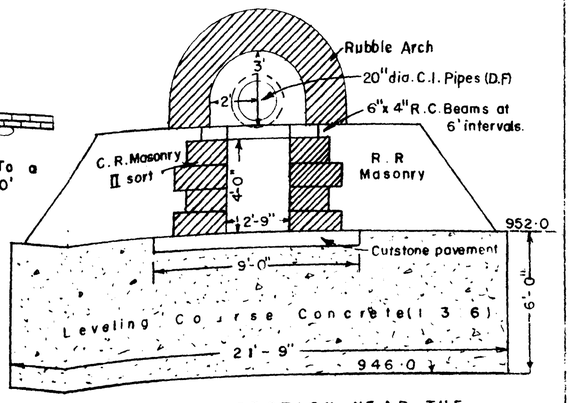
CROSS SECTION OF THE SLUICE

SCALE 1" = 20'



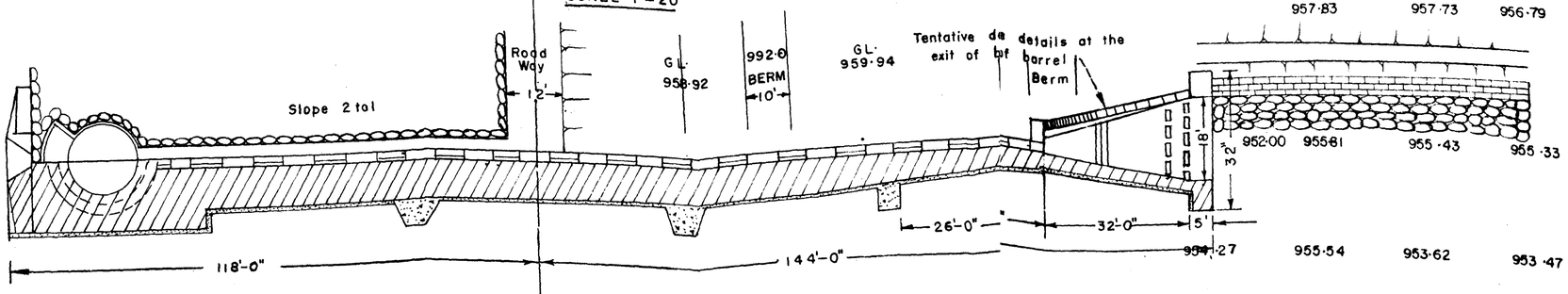
HYDRAULIC PARTICULARS

Station	.. L.S. 8165'
Design discharge..	.. 200 cusecs
Sill elevation ..	.. 952.0
Sill Of Vent ..	.. 3'-0"x 2'-9"
T.B.L	.. 1012.0
M.W.L	.. 1004.0
F.T.L	.. 1001.0



CROSS SECTION NEAR THE JUNCTION OF THE VENT AND BARREL

Scale : 1" = 4'



PLAN

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI	
File	DINDUGAL WATER SUPPLY SCHEME CONSTRUCTION OF RIVER SLUICE AND INTAKE ARRANGEMENTS. (As Received)	
Drawing 136/59	S. V.	U.V. Sadasivan / K. Ramaiah
	J. B. R	A. R. O / E. E. R
	Trd. V.V	Ckd. / Fig. 1

Reg. No.41- E.'62 (Saka Era 1883)- 375.

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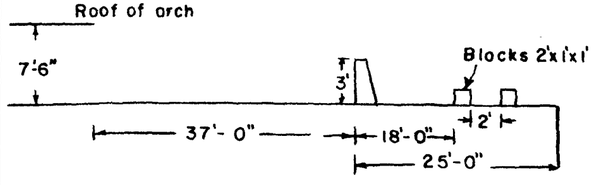
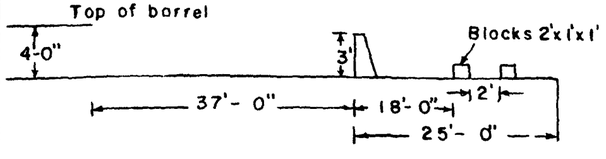
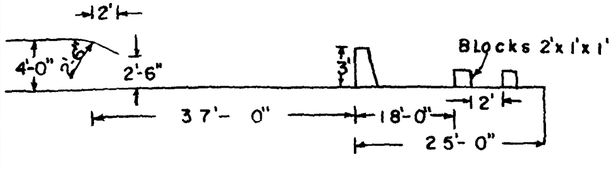
Annual Report Of 1959	
9/30	2/3
ITEM No	DRG No

STATEMENT No I

Sl. no	Device	Type of sluice	Performance	Scour Details: End scour	Max. scour	Distance of max. scour	Scour index	Velocity D/S	Hump height
1.		Sluice barrel with arched roof with water supply main inside and over R.C beams _ Vide fig. 1	The flow at the exit is not smooth, but turbulant. Very low velocity inside the stilling basin. A stable secondary jump is formed. The scour is reduced. But the max. scour is formed nearer to the apron.	0·495'	2·22'	5·625'	14·8	6·55'/sec	5·61'
2.		Rectangular sluice barrel 4'-0"x2'-9" with continuous slab on the top.	Severe return flow is formed as soon as the sluice is put under operation. The return flow is eleminated by external means. and the observations taken.	0·54'	2·295'	10·625'	8·25	5·65'/sec	6·06
3.		Rectangular sluice barrel 4'-0"x2'-9" with continuous slab on the top and with a roof deflector at the exit.	Uniform jump is formed inside the stilling basin. A stabble secondary jump is formed. The scour below the apron is much reduced. – FINALISED –	0·465'	1·77'	8·125'	4·43	6·08'/sec	6·24

Report	Public Works Department, MADRAS. Irrigation Research Station, POONDI.		
File	DINDUGAL WATER SUPPLY SCHEME RIVER SLUICE _ ENERGY DISSIPATION STATEMENT _ I		
137/59 Drawing	J·E·R	A·R·O	E·E·R
	Trd. J·J	Ckd.	Fig. 2

Reg. No. 42 - E '62. (Saka Era 1882) - 375.

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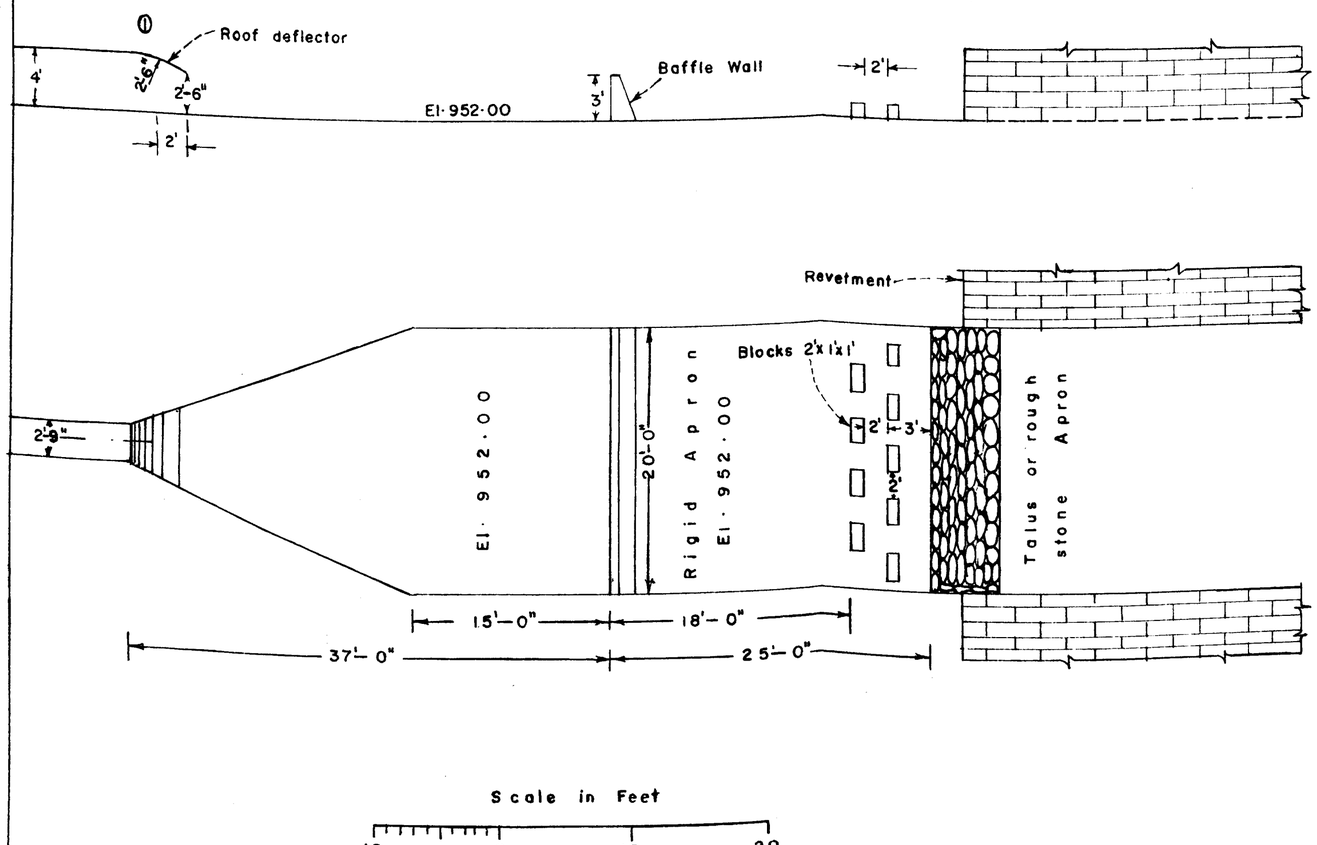
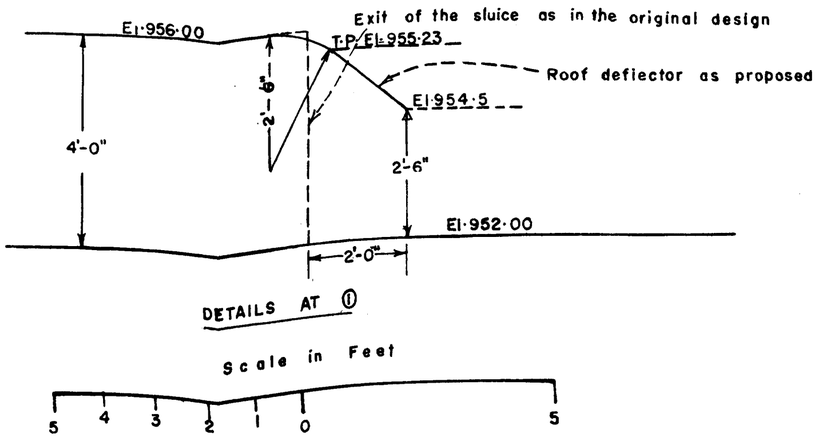
Annual Report of 1959	
9/30	3/3
Item Nº	Drawing Nº





Report	Public Works Department-Madras Irrigation Research Station-Poondi	
File	DINDUGAL WATER SUPPLY SCHEME	
	RIVER SLUICE-ENERGY DISSIPATION (Finalised Design)	
138 Drg.	J·E·R	A·R·O	E·E·R
	Trd: V.V.	ckd:	FIG··· 3

Reg. No. 43 - E '62 (Saka Era 1882) - 375

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CAVITATION STUDIES

On Mettur Dam River Sluices

1. Synopsis

Studies were conducted with a view to suggest suitable remedial measures to prevent cavitation damages to the sides of the barrel of the Mettur Dam River Sluices. A brief note on the problem has been given. The model set up has been briefly explained. The method of experimentation has been presented. The results of the experiments have been analysed and the remedial measures to be adopted have been given. The limitations of the study have also been discussed and scope for further reasearch has been indicated.

2. Introduction

2.1. The Mettur Dam across the Cauvery river has been provided with two sets of river sluices with sills at different levels, one set at a higher level being known as High Level Sluices and the other at a lower level being known as Low Level Sluices (1). There are eight High Level Sluices of size 10'6"x 16'0" each with sill level at El. 720,00 and five low level sluices of size 7'0"x 14'0" with sill level at El. 670.00. Figures 1 and 2 show the plan and cross section of the Low Level and High Level Sluices respectively.

2.2. Each vent of the sluices is provided with a mildsteel free-roller, culvert sluice, gate working against hard, cast iron, free rollers moving in a cage on machined paths in cast iron grooves. The gate is suspended from the centre line of the dam and is made to fit into a hollow cast iron frame with protected side grooves (1).

2.3. Both the sets of sluices have been provided with a bell mouthed entry upstream. Downstream of the gate groove the vents are lined with cut-stone and have concrete, semi-circular, arched roof. The floors slope towards the rear on a gradient of 1 in 15 from a point 5 feet behind the gate. Air vents have also been provided at the top in the rear connecting them to drywells.

2.4. After a few years of operation, some portions downstream of the gate slot were found to be pitted, especially the roller frames. Hence, in order to protect the roller frames from the onslaught of Cavitation attack, a deflector plate was provided in 1942 (Vide Figure-3). This addition only shifted the affected area a little downstream. The cast iron frame of the gate slot and the cutstone-sides of the sluices have been badly eroded (vide photographs and statement enclosed),

2.5. This problem was referred to this Station for model study and for suggesting suitable remedies for preventing further damages.

(3) The roughened surfaces of the eroded cast iron and masonry portions once formed will be responsible for producing further high negative pressures.

6.2.2. Regarding the third reason, considerable research has been done to assess the cavitation potential of roughened surfaces. In the article on "Cavitation damage of roughened concrete surfaces"(4) by Donald Colgate M. ASCE, the evaluation of the Cavitation potential of various types of roughened surfaces has been discussed. Two specimens of roughened concrete surfaces in the Davia Dam spillway bucket have been taken and laboratory experiments conducted to asses their cavitation potential.

From the data obtained in the laboratory, a chart has been prepared plotting the average velocity near the roughened surface against the depth of flow (or pressure head) for the condition of incipient cavitation for the two tested surfaces. The chart is reproduced in Figure-11 in this report.

6.2.3. A similar chart may have to be prepared for the roughened surfaces existing at Mettur Dam river sluices by conducting similar laboratory experiments. But because of the non-availability of the test apparatus required for conducting the above studies, the curves obtained for the Davis Dam specimen may be tentatively used to determine approximately if the flow conditions prevailing at Mettur Dam river sluices would cause cavitation in the presence of the roughened surfaces.

6.2.4. To apply the Davis Dam chart for the conditions at Mettur Dam, the average velocity of flow has been computed for various gate-openings from 1 foot upto 10 feet by using the formula $V \equiv C_v \sqrt{2gH}$.

where V ≡ average velocity of flow in feet/sec.

C_v = Coefficient of velocity (taken as 0.9)

H ≡ Head of water above the centre of the opening on the upstream side.

The depth of flow in rear of the gate has been assumed to be nearly equal to the gate-opening itself. The variation of velocity for different gate-openings or depths of flow is shown in Figure-11,

6. 2. 5. Taking the curve shown for specimen No. 2 (Figure-11) (being the worst case) it can be seen that the conditions existing at Mettur Dam would cause cavitation in the low level sluices at a gate opening of 1 foot for all reservoir elevations of 780.00 and more. Also for M. W. L. in the reservoir cavitation is likely to occur in the low level sluice for all gate openings upto full bore operations.

A similar chart for the High Level Sluice is shown in Figure-12, From this chart, it can be seen that cavitation is likely to occur in the High Level Sluice at a gate opening of 1 foot for all reservoir elevations of 780,00 and more. For maximum water level in the reservoir cavitation is likely to set in for all gate openings of 5.5 feet and less.

6. 2. 6. In order to provide remedial measures for the reasons enumerated as (1) and (2) in para 6. 2. 1,, the following two contrivances were incorporated in the model and their effects studied.

Remedy I — Rounding off the sharp corners marked A and B in Figure-3

Remedy II — Providing air vent pipes at suitable places as shown in Figure-6.

6. 3. The pressure readings at the various points with the remedy I alone incorporated are shown in tables 3 and 4 for the two sets of sluices. Pressure contours have also been drawn and are presented in Figures 13 and 14. By rounding off the sharp corners, the negative pressure have been considerably reduced from 13 feet to 8 feet in the case of low level sluices and from 16 feet 4 feet in the case of High Level Sluices.

6. 4. Pressure observations were taken for all the points with both remedial measures incorporated simultaneously and the data are presented in Tables 5 and 6. Pressure contours drawn for this condition are also presented alongside the pressure contours with Remedy-I only for comparison (Please see Figures 11 and 12). The maximum negative pressures have been further reduced to a value of about 3 feet in the low level sluice and 2 feet in the High Level Sluice.

7. Recommendations

On the basis of the experimental data, the following remedial measures are snggested in order te reduce, if not to prevent, further cavitation damages in the two sets of sluices.

1) The sharp corners downstream of the deflector plate may be eased out as shown in Figure-6. The material to be used for this purpose may be of stainless steel.

2) Two air vent pipes about 4 inches to 6 inches diameter each may be provided at the centre of the deflector plate one at a height of 2 feet from the sill level and another at 5 feet from the sill level. If structurally possible these air vent pipes may be conveniently connected to the existing air vent pipes leading to the dry wells.

3) Cast iron being brittle, is a very bad material to resist cavitation attack. Hence, highly resilient materials like mild steel, may be used in the place of cast iron, for future constructions.

4) The sluices may be lined with steel sheet for a short distance downstream of the gate slot covering the existing cavitation zones to avoid erosion of the masonry,

8. Conclusion

It is but pertinent to mention here about the limitations of the present studies. In the first place, the pressures under which the model studies were conducted were not reduced to scale. Hence the pressure readings observed and the pressure reduction found are only of a qualitative nature and the

results cannot be interpreted quantitatively. Secondly, the diameter of the air vent pipe cannot be determined accurately by model studies as the quantity of air demand cannot be accurately measured. The size of the pipe recommended is only from practical considerations. Thirdly, the resisting property of the materials recommended as substitutes for cast iron cannot be studied in the hydraulic model and they have been suggested only as a recommendation in a general way. In spite of the above limitations, it may be safely concluded that the remedial measures suggested herein will reduce the extent of cavitation damages on the side walls of the high level and low level sluices.

9 Scope for Further Research

9.1. The evaluation of the cavitation potential of roughened surfaces has been briefly indicated in Para 6.2.2. of this report. Plaster moulds of the roughened surfaces existing at Mettur Dam river sluices may be taken and casts made from the mould to represent the prototype roughness for evaluating their cavitation potential as described in the article on Cavitation Damage of roughened concrete surfaces[4]. A chart similar to the one reproduced from the article and shown in Figure-11 and Figure-12 may be beveloped for the conditions at Mettur Dam river sluices. From such a chart, we may be able to determine the critical gate-openings at which cavitation may set in and attempt to restrict the operation of gates to the safe range indicated by the chart thereby preventing further damages.

9.2. Moreover, it has been found out that if the approach to the cavitated zone was made rougher than at present, the average velocity of the stream necessary to prevent cavitation would be less than if the approach surface is smooth. Hence, the degree of roughness to be given to the approach surfaces may also be determined by further research so that the average velocity of stream necessary to prevent cavitatation is less than the average velocity obtaining at present in the Mettur Dam river sluices.

10. References

1. History of "Cauvery Mettur Project" by C. G. Barber.
2. "Hydraulic Characteristics of Gate Slots" by J. W. Ball-Proceedings of the American Society of Civil Engineers"—Volume 85 No. HY 10, October 1959 Part I.
3. Engineering Hydraulics" by Hunter Rouse.
4. "Cavitation Damage of roughened Concrete Surfaces" by Donald Colgate, M. A.S.C.E, Proceedings of the American Society of Civil Engineers—Volume 85 No. HY 11, November 1959.

STATEMENT SHOWING THE DETAILS OF EROSION IN HIGH LEVEL SLUICES, METTUR DAM

Sl. No. (1)	Vent No. (2)	*Left Side* Length (3)	*Left Side* Height (4)	*Right Side* Length (5)	*Right Side* Height (6)	Remarks (7)
1.	Vent - 1	1'0"	3'9"	1'0"	5'9"	
2.	Vent - 2	1'0"	5'6"	1'0"	5'6"	
3.	Vent - 3	1'6"	5'0"	1'6"	5'0"	
4.	Vent - 4	2'0"	4'0"	1'6"	5'0"	
5.	Vent - 5	2'0"	5'6"	2'0"	5'6"	
6.	Vent - 6	2'0"	5'0"	2'0"	5'6"	
7.	Vent - 7	1'0"	2'0"	0'9"	1'6"	
8.	Vent - 8	1'6"	4'6"	1'6"	4'6"	

CAVITATION STUDIES ON METTUR DAM RIVER SLUICES HIGH LEVEL SLUICE PRESSURES OBTAINING IN THE MODEL

TABLE-1

Piezometer		Water level kept at El. 790·00			Water level keen at El. 770·00		
Number	Elevation	Gate opening in feet			Gate opening in feet		
		3 feet	4 feet	5 feet	3 feet	4 feet	5 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	721.50	29.75	30.00	33.25	20.75	19.75	25.50
2.	720.00	—16.00	—8.75	—10.50	—14.50	—13.75	— 8.50
3.	721.00	—6.25	—4.75	— 0.25	— 4.50	— 2.75	5.25
4.	721.00	—5.75	—2.75	— 9.50	— 3.00	1.00	6.00
5.	721.00	—5.75	—1.50	— 2.00	— 3.50	0.25	1.00
6.	721.00	—1.25	—1.75	0.75	— 0.75	—0.25	0.00
7.	721.50	1.00	16.75	24.50	0.50	17.50	21.50
8.	722.00	—4.75	—13.00	—9.50	— 6.25	—13.75	—8.25
9.	722.00	—2.00	—2.00	2.50	0.00	—0.50	1.75
10.	722.00	—1.00	2.00	2.00	0.75	—0.50	2.25
11.	722.00	0.25	0.50	3.00	0.00	0.25	3.30
12.	722.00	0.00	0.75	1.25	0.00	1.00	1.00
13.	723.50	0.00	0.25	12.00	0.75	0.50	8.50
14.	723.00	—0.25	—4.25	—4.50	—0.50	—3.50	—3.75
15.	723.00	0.25	0.25	0.75	0.00	0.00	1.00
16.	723.00	0.00	1.00	1.50	0.00	0.00	1.25
17.	723.00	1.50	—3.00	0.75	1.00	—2.00	0.00
18.	723.00	0.50	1.00	—3.00	0.75	1.00	2.00
19.	724.50	—0.25	—0.50	1.25	0.25	—0.50	—0.75
20.	724.00	0.00	—0.25	—0.75	0.00	0.00	—0.50
21.	724.00	0.00	—0.25	0.00	0.00	0.25	0.00
22.	724.00	0.50	—0.25	0.50	0.25	0.25	0.25
23.	724.00	0.25	0.00	—0.50	0.25	0.25	0.00
24.	724.00	0.25	—0.25	—0.75	0.50	0 25	0.25
25	725.50	0.25	—0.25	—0.25	0.25	0.25	0.00
26.	725.00	0.25	0.00	0.00	0.25	0.25	0.00
27.	725.00	—0.25	0.00	0.00	0.25	0.25	0.25
28.	725.00	0.25	—3.25	—3.25	0.25	0.25	0.25
29.	725.00	0.25	—3.00	—3.00	0.25	0.25	0.25
30.	725.00	0.25	0.00	0.25	0.25	0.25	0.25
31.	726.00	0.25	0 25	0.25	0.25	0.25	0.25
32.	726.00	0.50	0.25	0.25	0.50	0.25	0.25
33.	726.00	0.50	0.25	0.25	0.50	0.25	0 25
34.	726.00	0.50	0.25	0.25	0.50	0.25	0.25
35.	726.00	0.75	—0.25	—0.25	0.25	0.25	0.25
36.	726.00	0.25	0.25	0,75	0.25	0.25	0.25
37.	727.50	0.25	9.25	0.25	0.50	0.50	0.50
38.	727.00	0.25	0.25	0.25	0.25	0.50	0.25
39.	727.00	0.25	0.25	0.25	0.25	0.25	0.25

Table-2 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
42	677.00	0.00	0.25	0.00	0.25	0.25	0.25
43	678.00	0.50	0.50	0.50	0.25	0.25	0.25
44	678.00	0.25	0.25	0.25	0.25	0.25	0.25
45	678.00	0.25	0.25	0.25	0.25	0.25	0.25
46	678.00	0.50	0.50	0 50	0.00	0.25	0.25
47	678.00	0.25	0.25	0.00	0.00	0.00	0.25
48	684.00	105.25	104.25	103.75	86.00	85.75	84.75
49	684.75	104.50	104.50	103.00	85.00	84.75	83.75
50	685.50	103.00	103.50	101.25	84.00	83.75	82.75
51	686.25	103.50	101.25	98.50	83.50	82.00	79.75
52	686.75	103.00	101.25	99.50	83.00	82.00	80.50
53	670.00	1.60	3.25	12.75	0.25	3.00	10.50
54	670.00	—13.00	—10.00	—13.00	—10.00	—13.00	—13.00
55	670 00	0.50	—0.25	—0.75	—1.75	—1.00	—0.75
56	670.00	—	—	—	—5.00	—5.75	—2.75
57	670.00	—0.50	8.25	18.75	0.50	7.00	15.75
58	670.00	—1.25	2.00	8.50	1.75	4.00	8.75
59	670.00	+2.25	3.25	4.75	2.75	3.00	4.25
60	670.00	—	—	—	—	—13.00	—13.00
61	670.00	109.50	97.25	89.25	90.50	81.50	75.00
62	670.00	118.50	117.00	114.25	98.75	97.75	95.75
63	670.00	119.00	118.57	117.00	99.00	98.75	97.75
64	669.93	119.07	118.57	118.32	98.07	99.32	98.82
65	669.70	119.50	119.05	119.30	99.55	99.80	99.80
66	671.00	94.25	92.50	83.50	89.25	78.25	70.75
67	671.00	117.75	116.00	113.75	97.50	96.25	94.75
68	671.00	117.75	116.75	115.25	98.00	96.50	96.70
69	671.00	118.25	117.50	117.00	98.25	88.25	97.75
70	671.00	118.45	117.25	116.75	98.25	98.25	97.75
71	671.00	118.50	117.75	117.75	98.50	98.50	98.50

CAVITATION STUDIES ON METTUR DAM RIVER SLUICES HIGH LEVEL SLUICE—PRESSURES OBTAINING AFTER THE INTRODUCTION OF REMEDY—I

TABLE - 3

Piezometer		Water level kept at El. 790·00			Water level kept at El. 770·00		
		Gate opening in feet			Gate opening in feet		
Number	Elevation	3 feet	4 feet	5 feet	3 feet	4 feet	5 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	721.50	—	—	—	—	—	—
2.	721.00	—	—	—	—	—	—
3.	721.00	—0.50	4.00	5.00	3.00	4.00	5.00
4.	721.00	—0.50	5.00	7.75	2.25	4.25	6.25
5.	721.00	—1.00	1.25	3.00	0.00	1.50	3.00
6.	721.00	—1.00	—0.50	0.75	—0.50	0.25	1.00
7.	722.50	—	—	—	—	—	—
8.	722.00	—	—	—	—	—	—
9.	722.00	—4.00	—1.75	1.25	—1.75	1.25	2.25
10.	722.00	—4.25	—4.50	—1.25	—2.00	—2.00	—0.50
11.	722.00	2.75	3.25	4.00	2.75	3.00	3.50
12.	722.00	0.75	1.25	1.25	1.25	1.25	1.25
13.	723.50	—	—	—	—	—	—
14.	723.00	—	—	—	—	—	—
15.	723.00	6.50	7.25	10.50	4.50	4.50	9.25
16.	723.00	0.00	—0.25	—0.25	0.00	0.00	1.00
17.	723.00	0.75	1.00	1.00	1.75	1.75	2.50
18.	723.00	0.00	0.00	0.25	0.00	0.75	1.50
19.	724.50	—	—	—	—	—	—
20.	724.00	—	—	—	—	—	—
21.	724.00	0.25	0.00	0.50	0.25	0.25	—0.25
22.	724.00	0.25	0.25	0.50	0.25	0.50	0.75
23.	724.00	—0.25	—0.50	1.25	0.00	—0.25	—0.75
24.	724.00	0.50	0.50	0.75	0.50	0 50	0.50
25.	725.50	—	—	—	—	—	—
26.	725.00	—	—	—	—	—	—
27.	725.00	—0.25	0.00	0.00	0.25	0.25	—0.50
28.	725.00	0.00	0.00	0.25	0.25	0.25	0.00
29.	725.00	0.00	0.00	0.00	0.25	0.00	0.00
30.	725.00	0.50	0.50	0.50	0.25	0.50	0.50
31.	726.50	—	—	—	—	—	—
32.	726.00	—	—	—	—	—	—
33.	726.00	0.25	0.25	0.50	0.25	0.25	0.25
34.	726.00	0.25	0.25	0.25	0.25	0.25	0.25
35.	726.00	0.00	0.25	0.25	0.25	0.25	0·25
36.	726.00	0.50	0.25	0.25	0.25	0.25	0.25
37.	727.50	—	—	—	—	—	—
38.	727.00	—	—	—	—	—	—
39.	727.00	0.25	0.25	0·25	0.25	0.25	0.25
40.	727.00	0.25	0.25	0.25	0.25	0.25	0.25

Table-3 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
41.	727.00	0.25	0.25	0.25	0.25	0.25	0.25
42.	727.00	0.25	0.25	0.25	0.25	0.25	0.25
43.	725.00	—	—	—	—	—	—
44.	725.00	0.25	0.25	0.25	0.25	0.25	0.25
45.	725.00	0.50	0.25	0.50	0.25	0.25	0.25
46.	725.00	0.25	0.25	0.25	0.25	0.25	0.25
47.	725.00	0.25	0.25	0.25	0.25	0.25	0.25
48.	736.00	51.75	52.25	51.75	33.00	32.75	32.00
49.	736.00	51.00	51.25	51.00	31.75	32.00	31.00
50.	737.00	50.50	50.75	50.50	31.25	31.50	30.50
51.	738.00	49.00	49.00	48.75	30.00	30.25	30.00
52.	720.00	1.50	8.00	13.25	3.00	8.00	11.00
53.	720.00	3.25	7.75	12.00	4 25	7.25	9.50
54.	720.00	1.50	2.75	5.00	2.25	3.25	5.00
55.	719.98	—	3.98	5.48	2.48	2.23	1.48
56.	720.00	10.25	14.75	19.25	8.75	12.25	12.75
57.	720.00	6.75	10.00	14.25	5.75	8.75	11.00
58.	720.00	3.00	4.25	6.75	3.75	4.00	15.75
59.	719.98	6.23	6.23	4.98	4.73	4.98	3.98
60.	720.00	49.75	47.25	45.75	35.00	33.25	32.50
61.	720.00	66.25	64.75	63.25	47.00	46.00	44.60
62.	720.00	67.75	66.75	66.10	48.50	47.15	46.75
63.	719.90	68.60	67.85	67.85	48.86	48.60	47.85
64.	719.90	69.05	68.55	68.30	48.30	49.30	48.50
65.	721.00	49.25	45.00	44.25	35.00	33 00	41.75
66.	721.00	65.00	64.00	61.50	46.75	47.00	43.75
67.	721.00	67.50	66.75	66.25	47.25	46.00	45.75
68.	721.00	67.25	66.00	65.75	47.25	47.75	45.75
69.	721.00	69.00	68.75	69.00	47.75	48.75	47.00

REMEDY-1 Rounding off corners as shown in Figure-6

CAVITATION STUDIES ON METTUR DAM RIVER SLUICES LOW LEVEL SLUICES—PRESSURES OBTAINING WITH REMEDY-I

TABLE - 4

Piezometer		Water level kept at El. 790.00			Water level kept at El. 770.00		
		Gate opening in feet			Gate opening in feet		
Number	Elevation	1 foot	2 feet	3 feet	1 foot	2 feet	3 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	—	—	—	—	—	—	—
2	—	—	—	—	—	—	—
3	671.00	8.75	—4.50	11.75	6.75	7.00	10.25
4	671.00	4.00	7.25	6.75	3.25	3.50	5.75
5	671.00	5.25	4.50	4.25	3.75	2.75	3.75
6	671.00	2.00	1.25	—0.25	2.50	1.25	1.00
7	—	—	—	—	—	—	—
8	—	—	—	—	—	—	—
9	672.00	1.00	4.25	5.50	1.25	2.75	4.50
10	672.00	2.00	5.00	6.25	1 75	3.25	4.50
11	672.00	0.75	—1.75	2.50	1.00	3.75	2.25
12	672.00	—1.25	1.75	—3.00	—0.50	—0.75	1.00
13	—	—	—	—	—	—	—
14	—	—	—	—	—	—	—
15	673.00	1.00	0.00	0.75	0.00	0.25	0.50
16	673.00	0.00	0.50	0.50	0.00	0.25	1.00
17	673.00	0.50	1.00	1.75	0.50	0.50	1.50
18	673.00	0.50	0.75	1.50	0.75	0.50	1.00
19	—	—	—	—	—	—	—
20	—	—	—	—	—	—	—
21	674.00	0.50	0.00	0.00	0.00	0.00	0.00
22	674.00	0.25	0.20	0.00	0.25	0.25	0.00
23	674.00	—0.25	0.35	—0.25	0.00	0.00	0.00
24	674.00	—0.50	0.25	0.50	0.25	0.50	0.50
25	—	—	—	—	—	—	—
26	—	—	—	—	—	—	—
27	675.00	0.25	0.25	0.50	0.25	0.25	0.25
28	675.00	0.25	0.00	0.00	0.25	0.25	0.25
29	675.00	0.25	0.00	0.00	0.00	0.25	0.25
30	675.00	0.25	0.25	0.25	0.25	0.25	0.50
31	—	—	—	—	—	—	—
32	—	—	—	—	—	—	—
33	676.00	0.25	0.25	0.25	0.25	0.25	0.50
34	676.00	0.25	0.25	0.00	0.25	0.50	0.50
35	676.00	0.25	0.25	0.00	0.55	0.50	0.75
36	676.00	0.50	0.25	0.25	0.25	0.50	0.50
37	—	—	—	—	—	—	—
38	—	—	—	—	—	—	—
39	677.00	0.25	0.25	0.25	0.25	0.25	0.25
40	677.00	0.25	0.25	0.25	0.00	0.25	0.25
41	677.00	0.00	0.00	0.25	0.25	0.50	0.25
42	677.00	0.00	0.25	0.25	0.25	0.25	0.25
43	—	—	—	—	—	—	—

Table-4 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
44	678.00	0.25	0.25	0.25	0.50	0.50	0.50
45	678.00	0.50	0.50	0.25	0.25	0.25	0.25
46	678.00	0.50	0.50	0.50	0.25	0.25	0.50
47	678.00	0.25	0.25	0.25	0.25	0.25	0.50
48	684.00	105.00	104.25	103.75	86.00	85.75	84.75
49	684.75	105.00	104.50	103.00	85.00	84.75	83.75
50	685.50	103.75	103.50	101.25	84.00	83.75	82.75
51	686.25	103.50	101.25	98.75	83.50	82.00	80.50
52	686.75	103.00	101.50	99.00	83.00	82.00	80.50
53	670.00	1.50	3.25	12.50	0.25	3.00	10.50
54	670.00	—8.00	—7.00	—8.00	—7.50	—8.00	—8.00
55	670 00	—0.25	—0.25	—0.25	—1.00	—0.50	—0.25
56	670.00	—	—	—	—4.00	—3.25	—1.00
57	670.00	—0.25	8.25	18.75	0.50	7.00	15.75
58	670.00	—0.75	2.00	8.50	2.00	4.00	5.75
59	670.00	2.25	3.25	4.50	2.75	3.25	4.25
60	670.00	—	—	—	—	5.00	8.00
61	670.00	109.50	97.25	89.25	90.50	81.50	75.00
62	670.00	118.00	117.00	114.00	98.75	97.75	95.75
63	670.00	119.00	118.00	117.00	99.00	98.75	97.65
64	669.93	119.07	118.57	119.32	98.07	99.32	98.82
65	669.70	119.50	119.50	99.59	99.80	99.80	99.80
66	671.00	94.00	92.50	83.50	89.25	78.25	70.75
67	671.00	118.00	116.00	113.75	97.57	96.25	94.75
68	671.00	118.00	116.75	115.25	98.00	96.50	96.75
69	671.00	118.25	117.50	117.00	98.25	98.25	97.75
70	671.00	118.25	117.25	116.50	98.25	98.25	97.75
71	671.00	118.50	117.75	117.50	98.50	98.50	98.50

Remedy—I Rounding off the corners as shown in Figure-6

CAVITATION STUDIES ON METTUR DAM RIVER SLUICES—HIGH LEVEL SLUICES—PRESSURES OBTAINING WITH REMEDIES I & II

TABLE - 5

Piezometer		Water level kept at El. 790·00			Water level kept at El. 770·00		
		Gate opening in feet			Gate opening in feet		
Number	Elevation	3 feet	4 feet	5 feet	3 feet	4 feet	5 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1.	721.50	—	—	—	—	—	—
2.	721.00	—	—	—	—	—	—
3.	721.00	1.00	4.00	—	—	—	—
4.	721.00	0.50	5.25	7.25	1.00	4.00	5.00
5.	721.00	—0.75	—1.50	3.00	0.75	1.75	3.00
6.	721.00	—0.50	—0.25	9.50	—0.50	0.50	1.00
7.	722.50	—	—	—	—	—	—
8.	722.50	—	—	—	—	—	—
9.	722.00	3.00	—0.25	9.50	—2.00	0.75	2.50
10.	722.00	—3.50	—1.50	—1.25	—2.25	—1.75	—0.25
11.	722.00	13.00	3.00	4.50	2.00	4.00	3.50
12.	722 00	1.00	1.00	1.50	1.00	1.50	1.00
13.	723.50	—	—	—	—	—	—
14.	723.00	—	—	—	—	—	—
15.	723.00	5.00	8.00	11.25	3.75	5 50	9.50
16.	723.00	0.00	—0.50	0.25	0.00	0.25	0.75
17.	723.00	0.50	0.25	3.00	0.75	1.00	2.50
18.	723.00	0.00	0.25	0.75	0.00	0.00	0.75
19.	724.50	—	—	—	—	—	—
20.	724.00	—	—	—	—	—	—
21.	724.00	0.00	0.00	0.75	0.25	0.00	—0 50
22.	724.00	0.25	0.25	0.75	0.50	0.50	1.00
23.	724.00	0.25	—1.00	—0.75	0.00	0.00	—0.75
24.	724.00	0.50	0.50	0.50	0.50	0 50	0.75
25.	725.50	—	—	—	—	—	—
26.	725.00	—	—	—	—	—	—
27.	725.00	0.25	0.00	—0.25	0.25	0.25	—1.00
28.	725.00	0.25	0.00	0.00	0.25	0.25	—0.25
29.	675.00	0.25	0.00	0.00	0.25	0.25	—0.25
30.	725.00	0.25	0.50	0.50	0.50	0.25	0.75
31.	726.50	—	—	—	—	—	—
32.	726.00	—	—	—	—	—	—
33.	726.00	0.50	0.50	0.25	0.50	0.50	0 50
34.	726.00	0.25	0.25	0.50	0.50	0.50	0.25
35.	726.00	0.50	1.50	0.50	0.50	0.50	0·00
36.	726.00	0.25	0.50	0.50	0.50	0.50	0.50
37.	727.50	—	—	—	—	—	—
38.	727.00	—	—	—	—	—	—
39.	727.00	0.25	0.25	0·25	0.50	0.25	0.25
40.	727.00	0.25	0.25	0.25	0.50	0.25	0.25
41.	727.00	0.25	0.25	0.25	0.50	0.25	0.25
42.	727.00	1.00	1.00	0.25	0.50	0.25	0.25

Table-3 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
43.	728.00	—	—	—	—	—	—
44.	728.00	0.25	0.25	0.50	0.50	0.50	0.50
45.	728.00	0.25	0.25	0.50	0.50	0.50	0.50
46.	728.00	0.25	0.25	0.50	0.50	0.50	0.50
47.	728.00	0.25	0.25	0.50	0.50	0.50	0.50
48.	736.00	53.25	52.75	51.00	33.75	33.25	32.50
49.	736.50	53.50	51.75	50.75	33.25	32.50	32.50
50.	737.00	51.75	51.25	50.00	32.75	31.75	28.00
51.	738.00	50 00	50.00	48.75	31.25	30.75	29.00
52.	728.00	1.50	8.25	13.75	—1.75	7 25	10.50
53.	728.00	3.50	7.75	11.75	3.00	6.75	9.25
54.	728.00	1.25	3.00	5.00	1.75	3.30	4.75
55.	719.98	—4.75	—4.23	—4.48	—2.48	—2.23	—1.23
56.	720.00	10.75	15.50	9.50	8.00	12.00	14.75
57.	720.00	6.75	10.50	14.00	5.50	8.00	10.25
58.	720.00	3.25	4.00	6.70	2.75	3.75	5.25
59.	719.98	—6.28	—6.48	—4.98	—3.48	—3.73	—0.98
60.	720.00	50.25	47.50	45.00	35.50	34.25	32.75
61.	720.00	67.00	65.25	62.50	48.25	46.50	45.00
62.	720.00	68.25	67.50	65.50	48.25	48.00	47 25
63.	719.00	69.10	68.35	67.10	48.85	48.85	48.35
64.	719.70	69.55	69.05	68.00	50.30	48.55	49.05
65.	721.00	48.75	46.25	44.00	34.00	33 25	33 00
66.	721.00	66.00	64.50	62.00	46.50	45.75	44.25
67.	721.00	61.25	60.50	58.50	47.50	47.50	46.75
68.	721.00	67.75	67.00	65.25	48.75	48.50	48.00
69.	721.00	67.75	67.50	66.25	37.25	37.25	37.00

REMEDY I Rounded Corners &
REMEDY II Introduction of Air Vent Pipe

CAVITATION STUDIES ON METTUR DAM RIVER SLUICES LOW LEVEL SLUICE—PRESSURES OBTAINING WITH REMEDIES I & II

TABLE - 6

Piezometer		Water level kept at El. 790.00			Water level kept at El. 770.00		
		Gate opening in feet			Gate opening in feet		
Number	Elevation	1 foot	2 feet	3 feet	1 foot	2 feet	3 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	—	—	—	—	—	—	—
2	—	—	—	—	—	—	—
3	671.00	8.75	—1.50	11.75	6.75	7.00	10.25
4	671.00	4.00	7.25	6.75	3.25	3.50	5.75
5	671.00	5.25	4.50	4.25	3.75	3.75	3.75
6	671.00	2.00	1.25	0.00	2.50	1.25	1.00
7	—	—	—	—	—	—	—
8	—	—	—	—	—	—	—
9	672.00	1.00	4.25	5.50	2.25	2.75	4.50
10	672.00	2.00	5.00	6.25	1.75	3.25	4.75
11	672.00	0.75	1.75	2.50	1.00	3.75	2.25
12	672.00	—0.25	—0.50	—1.00	0.00	0.00	0.00
13	—	—	—	—	—	—	—
14	—	—	—	—	—	—	—
15	673.00	1.00	0.00	0.75	0.00	0.25	0.50
16	673.00	0.00	0.50	0.50	0.00	0.25	0.00
17	673.00	0.50	1.00	1.75	0.50	0.25	1.50
18	673.00	0.50	0.50	1.50	0.75	0.50	1.00
19	—	—	—	—	—	—	—
20	—	—	—	—	—	—	—
21	674.00	0.50	0.00	0.00	0.00	0.00	0.00
22	674.00	0.25	0.00	0.00	0.25	0.25	0.00
23	674.00	0.00	0.25	0.25	0.25	0.25	0.00
24	674.00	0.00	0.25	0.50	0.25	0.50	0.50
25	—	—	—	—	—	—	—
26	—	—	—	—	—	—	—
27	675.00	0.25	0.25	0.50	0.25	0.25	0.25
28	675.00	0.25	0.00	0.00	0.25	0.25	0.25
29	675.00	0.25	0.25	0.25	0.50	0.50	0.50
30	675.00	0.25	0.25	0.25	0.25	0.25	0.25
31	—	—	—	—	—	—	—
32	—	—	—	—	—	—	—
33	676.00	0.25	0.25	0.25	0.25	0.25	0.50
34	676.00	0.50	0.25	0.25	0.25	0.25	0.50
35	676.00	0.25	0.25	0.25	0.25	0.50	0.75
36	676.00	0.25	0.25	0.25	0.25	0.25	0.50
37	—	—	—	—	—	—	—
38	—	—	—	—	—	—	—
39	677.00	0.25	0.25	0.25	0.25	0.25	0.25
40	677.00	0.25	0.25	0.50	0.00	0.25	0.25
41	677.00	0.00	0.00	0.25	0.25	0.50	0.25
42	677.00	0.00	0.25	0.25	0.25	0.25	0.25
43	—	—	—	—	—	—	—

Table-6 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
44	678.00	0.25	0.25	0.25	0.50	0.50	0.50
45	678.00	0.50	0.50	0.50	0.25	0.25	0.50
46	678.00	0.50	0.50	0.50	0.25	0.25	0.25
47	678.00	0.25	0.25	0.25	0.25	0.25	0.25
48	684.00	105.00	104.25	103.75	86.00	85.75	84.75
49	684.75	105.00	104.25	103.00	85.00	84.75	83.75
50	685.50	103.75	103.50	101.25	84.00	83.75	82.50
51	686.25	103.50	101.25	98.75	83.50	82.00	79.50
52	686.75	103.00	101.50	99.00	83.00	82.00	81.00
53	670.00	1.50	3.25	12.50	0.55	3.00	10.50
54	670.00	—3.00	—3.25	—3.00	—2.50	—2.50	—2.00
55	670.00	0.00	0.00	0.00	0.00	0.00	0.00
56	670.00	—	—	—	—1.00	—1.00	—0.25
57	670.00	—0.25	8.25	18.75	0.50	7.00	15.75
58	670.00	—0.50	2.00	8.50	2.00	4.00	8.75
59	670.00	2.25	3.25	4.50	2.75	3.25	4.25
60	670.00	—	—	—	—	—2.00	—2.00
61	670.00	109.50	97.25	89.25	90.50	81.50	75.00
62	670.00	110.00	117.00	114.25	98.75	97.75	95.75
63	670.00	119.00	118.00	117.00	99.00	98.75	97.75
64	669.93	119.07	118.57	118.32	98.07	99.32	98.82
65	669.70	118.50	119.50	119.30	99.05	99.80	99.80
66	671.00	94.00	92.50	83.50	89.25	78.25	70.75
67	671.00	118.00	116.00	113.75	97.50	96.25	94.75
68	671.00	118.00	116.75	115.25	98.00	96.50	96.75
69	671.00	118.25	117.50	117.00	98.25	98.25	97.75
70	671.00	118.25	117.25	116.50	98.25	98.25	97.75
71	671.00	118.50	117.75	117.50	98.50	98.50	98.50

REMEDY I Rounding of the corners.
REMEDY II Introduction of air vent pipe (vide Figure-6).

METTUR DAM HIGH LEVEL SLUICES (Vent No. 4)

Photographs showing the Damages in the Masonry and C. I. Frame

PHOTO No. 1
Left Side
(View looking downstream)

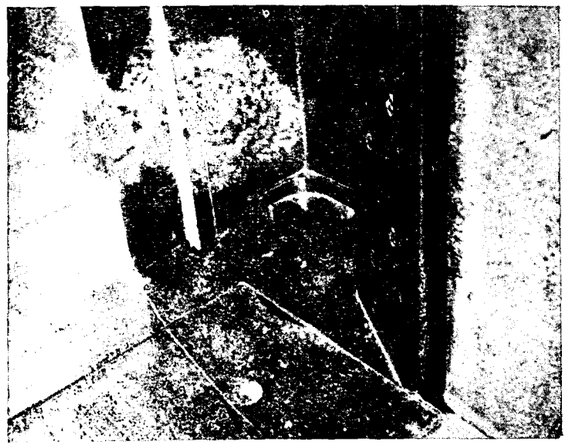

PHOTO No. 3
Left Side
Looking Downstream
(Photograph showing the dispositions of the Deflector Plate & the Shutter)

PHOTO No. 2
Right Side
(View looking upstream)

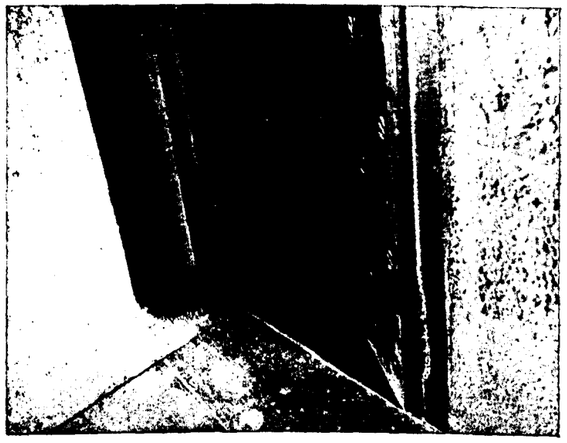

PHOTO No. 4
Enlarged and close up of Photo No. 3

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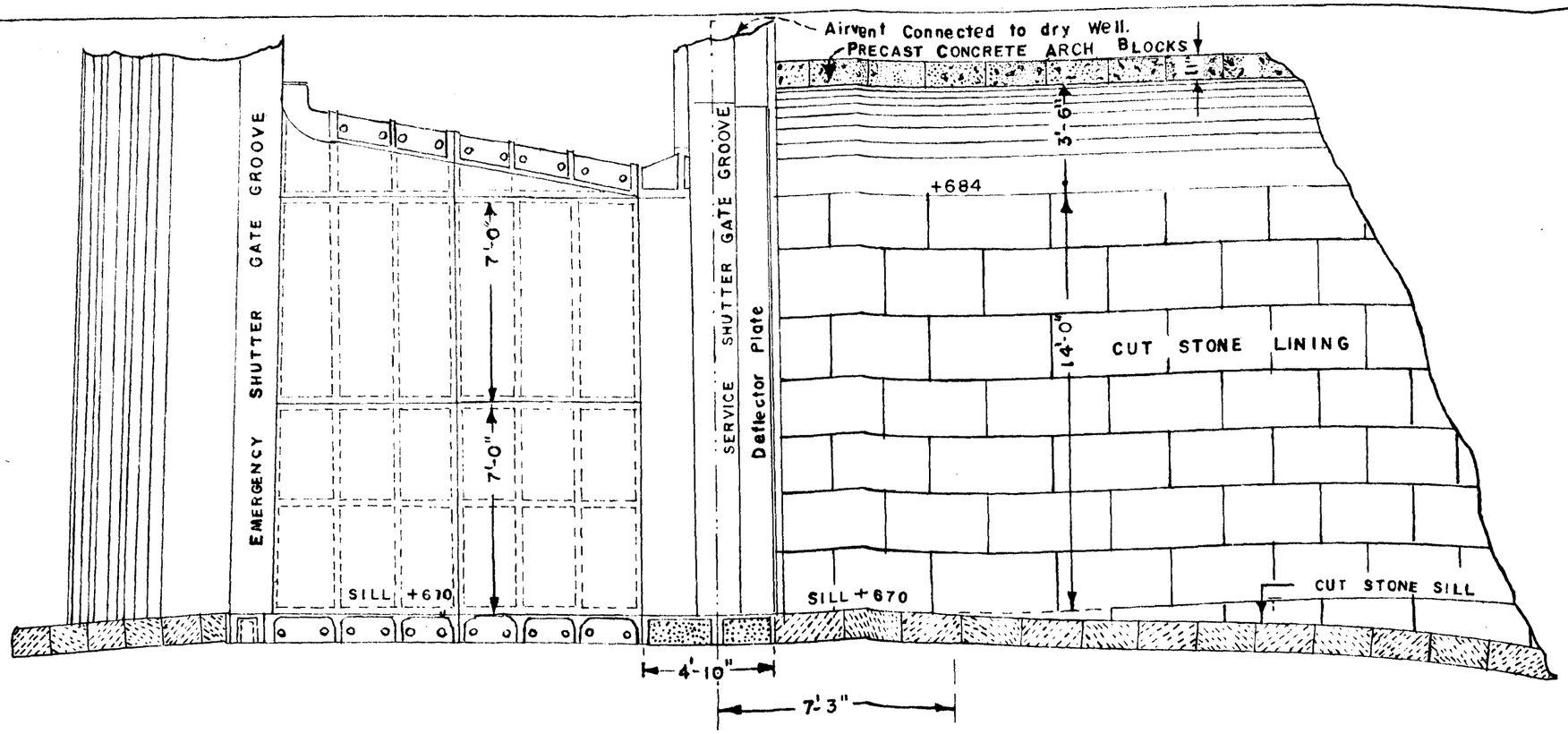
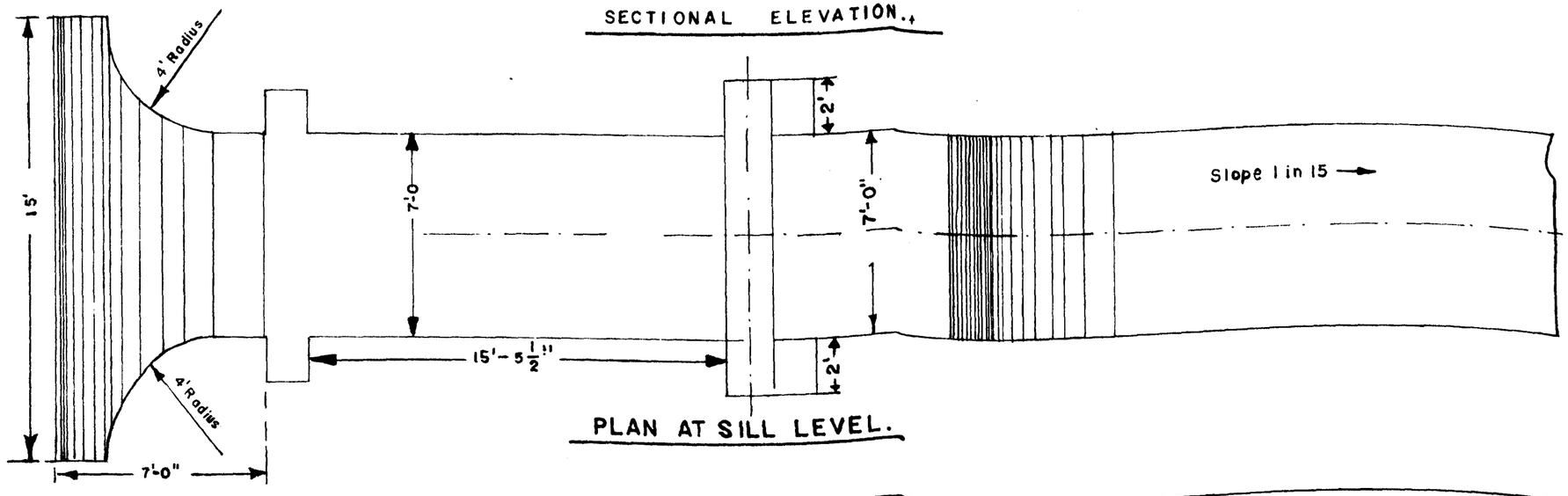
ITEM NO	DRG. NO.



SECTIONAL ELEVATION.

Scale In Feet

4 0 4 8



PLAN AT SILL LEVEL.

44/60	Report	Public Works Department — Madras. Irrigation Research Station — Poondi	
		Cavitation Studies On Mettur Dam River Sluices.	
	File	LOW LEVEL SLUICE (As Received)	
	Drawing	J. E. R.	A. E. R. / E. E. R.
		Trd: C. S. K.	Ckd: / Fig. . . . . .1.

Reg. No. 31- E '62 (Saka Era 1883)375

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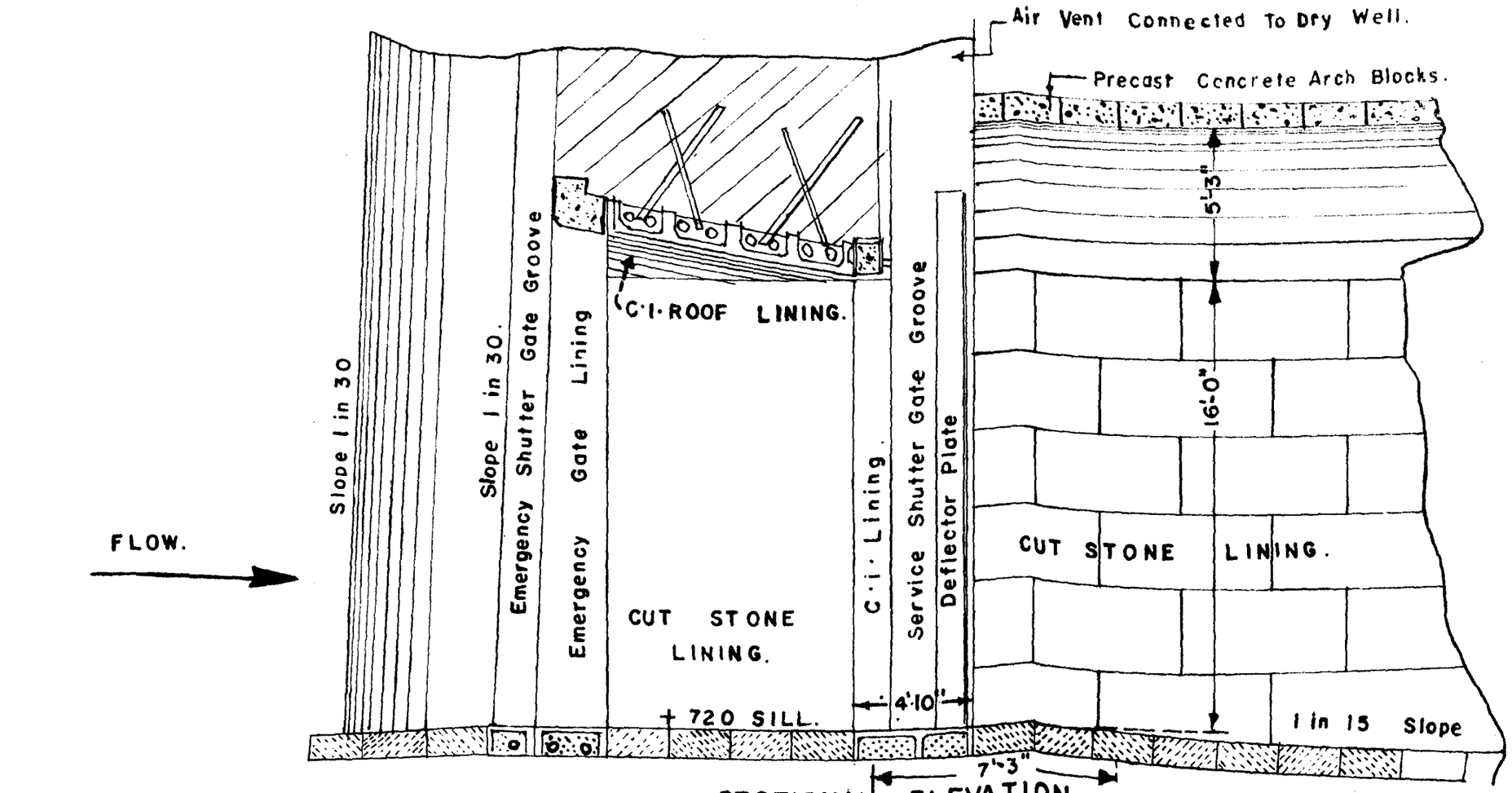
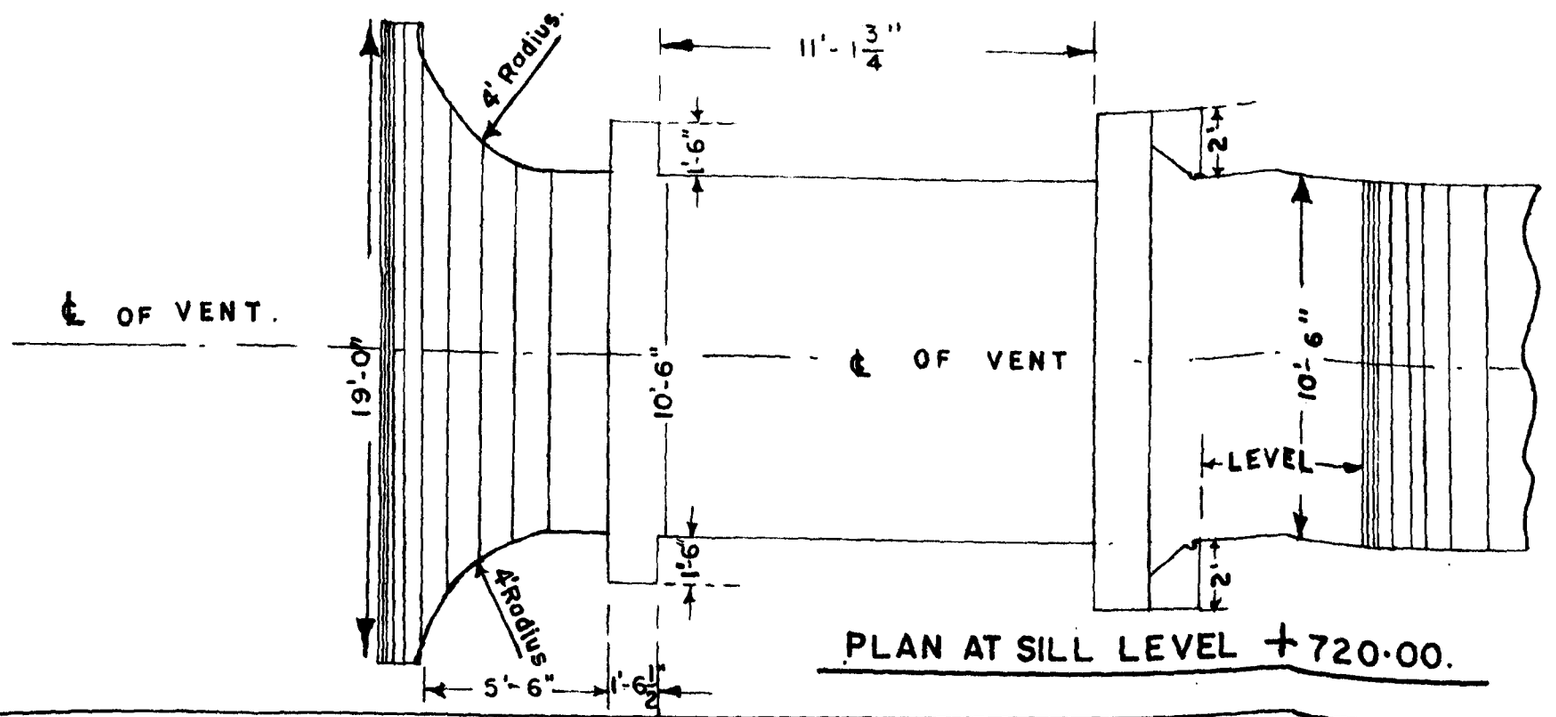
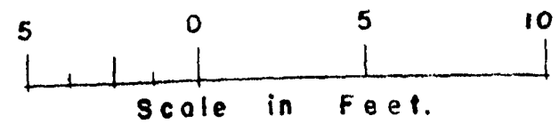


SECTIONAL ELEVATION.

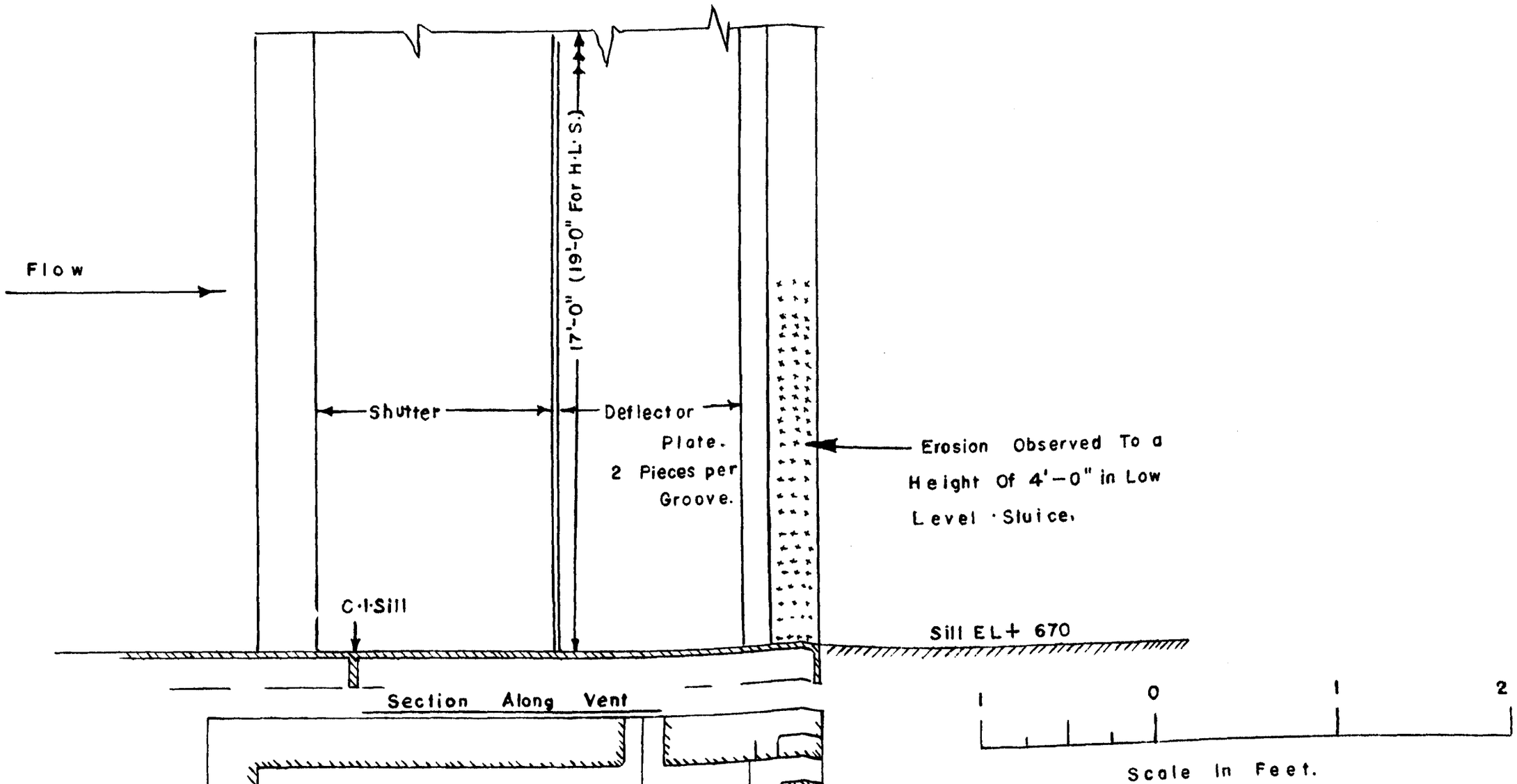


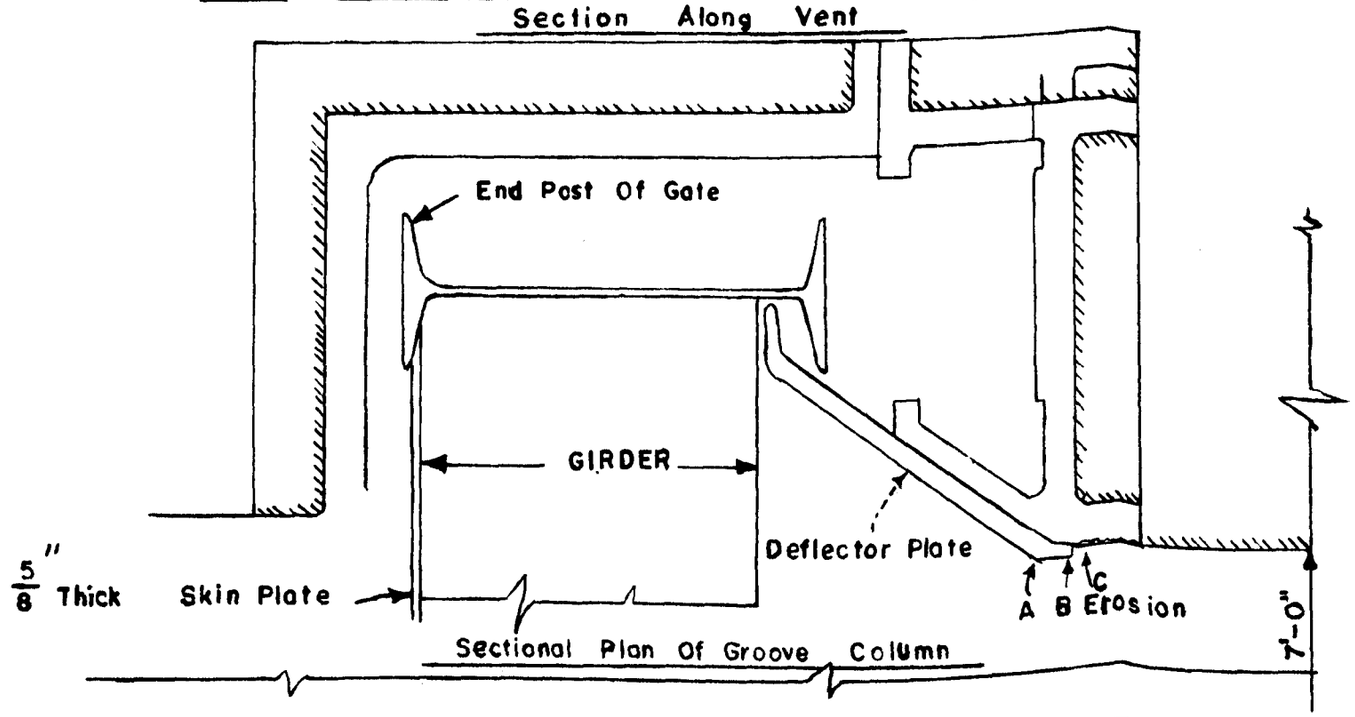
PLAN AT SILL LEVEL +720·00.



Report	Public Works Department — Madras. Irrigation Research Station — Poondi.		
	Cavitation Studies on Mettur Dam River Sluices.		
File	HIGH LEVEL SLUICE. (As Received)		
45/60 Drawing			
	J·E·R	A·E·R	E·E·R.
	Trd: A·S·D	Ckd:	Fig No: 2.

Reg. No. 32 - E'62 (Saka Era 1883) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH, B.A., A.D., C.S.O., Madras. 5.





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10/30	3/14
ITEM. NO.	DRG. NO.

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

Report

Cavitation Studies On Mettur Dam River Sluices

LOW LEVEL SLUICE.

EROSION IN FRAME.
(Details As Received From Field.)

File

47/60 Drawing

J.E.R	A.E.R	E.E.R
Trd: A.S.D.	Ckd:	Fig..... 3.

Reg. No. 33 - E '62. (Saka Era 1883) 375 "Reproduced from Indentor's Original" Printed by Sri K. SAMPATH, B.A, A.D., C.S.O. Madras. 5.
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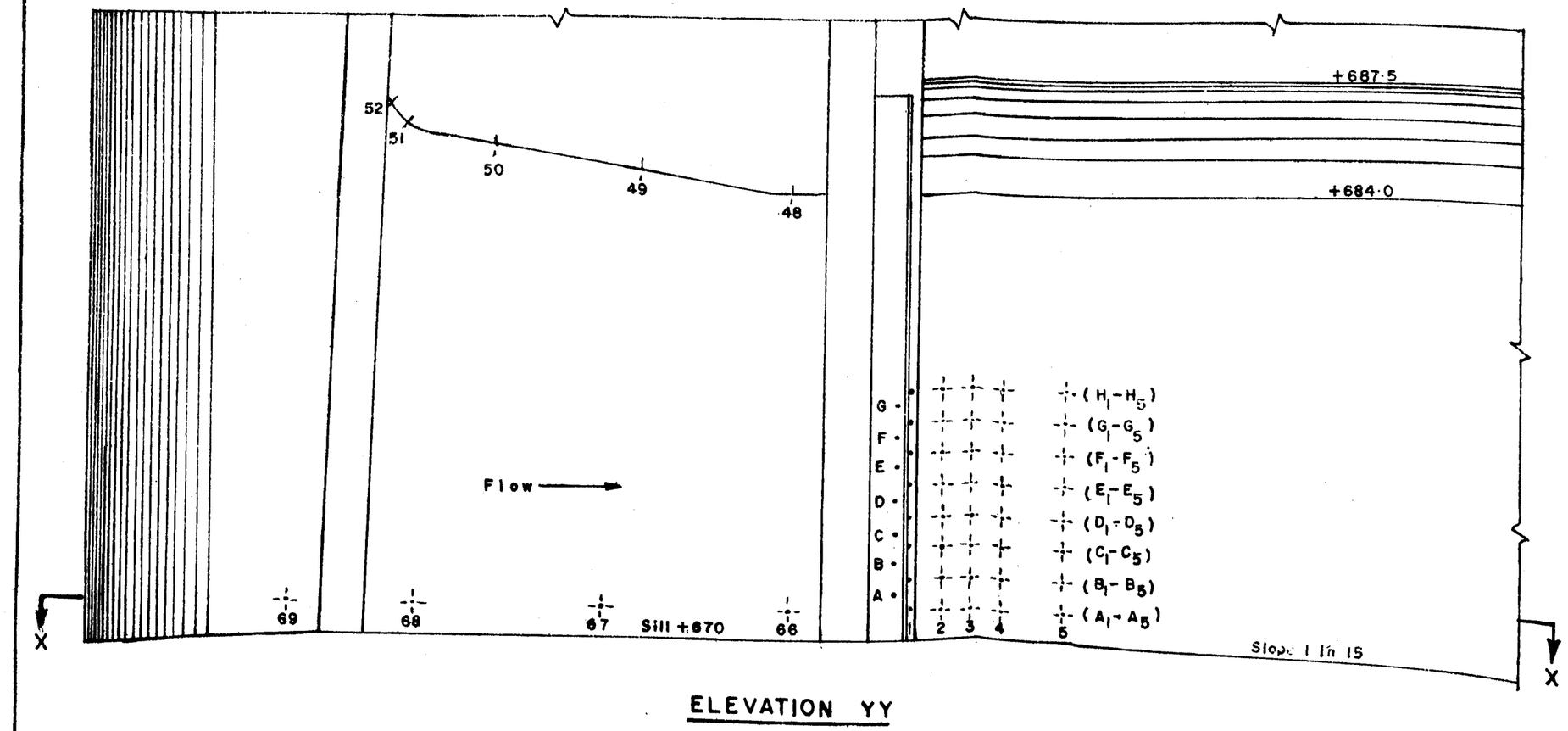
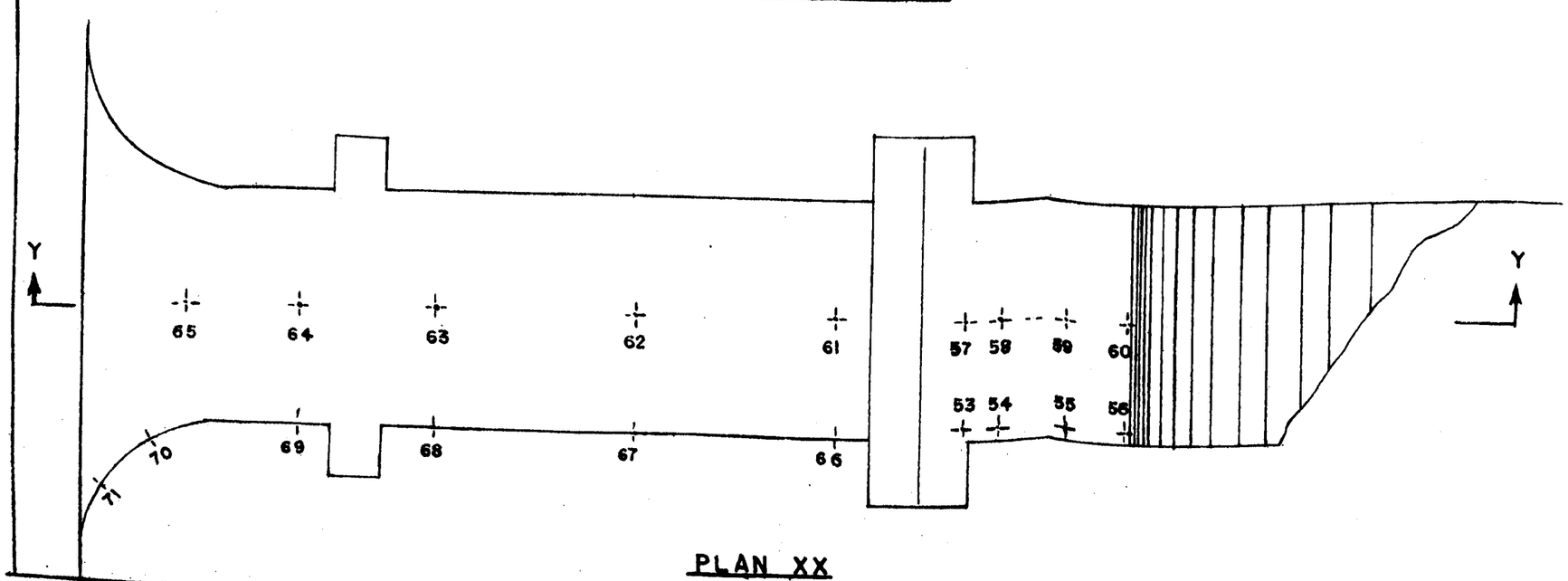
ITEM NO	DRG. NO



ELEVATION YY

Piezometer		Elevation	Piezometer		Piezometer	
Letter	Nos		No	Elevation	No	Elevation
A	1	671·50	48	684·00	61	670·00
A_1-A_5	2 to 6	671·00	49	684·75	62	670·00
B	7	672·50	50	685·50	63	670·00
B_1-B_5	8 -12	672·00	51	686·25	64	669·93
C	13	673·50	52	686·75	65	669·70
C_1-C_5	14-18	673·00	53	670·00	66	671·00
D	19	674·50	54	670·00	67	671·00
D_1-D_5	20-24	674·00	55	670·00	68	671·00
E	25	675·50	56	670·00	69	671·00
E_1-E_5	26-30	675·00	57	670·00	70	671·00
F	31	676·50	58	670·00	71	671·00
F_1 F_5	32-36	676·00	59	670·00		
G	37	677·50	60	670·00		
G_1-G_5	38-42	677·00				
H_1-H_5	43-47	678·00				



PLAN XX

Scale in Feet

4 0 4 8

Read This Drawing With Fig···No·1

Public Works Department — Madras.
Irrigation Research Station — Poondi

Cavitation Studies on Mettur Dam River Sluices

LOW LEVEL SLUICE
PIEZOMETER TAPPINGS
&
ELEVATION

J·E·R	A·E·R	E·E·R
Trd: A·S·D	ckd:	FIG···4

42/50 Drg. File Report

Reg.No.44-E'62(Saka Era 1883) 375.

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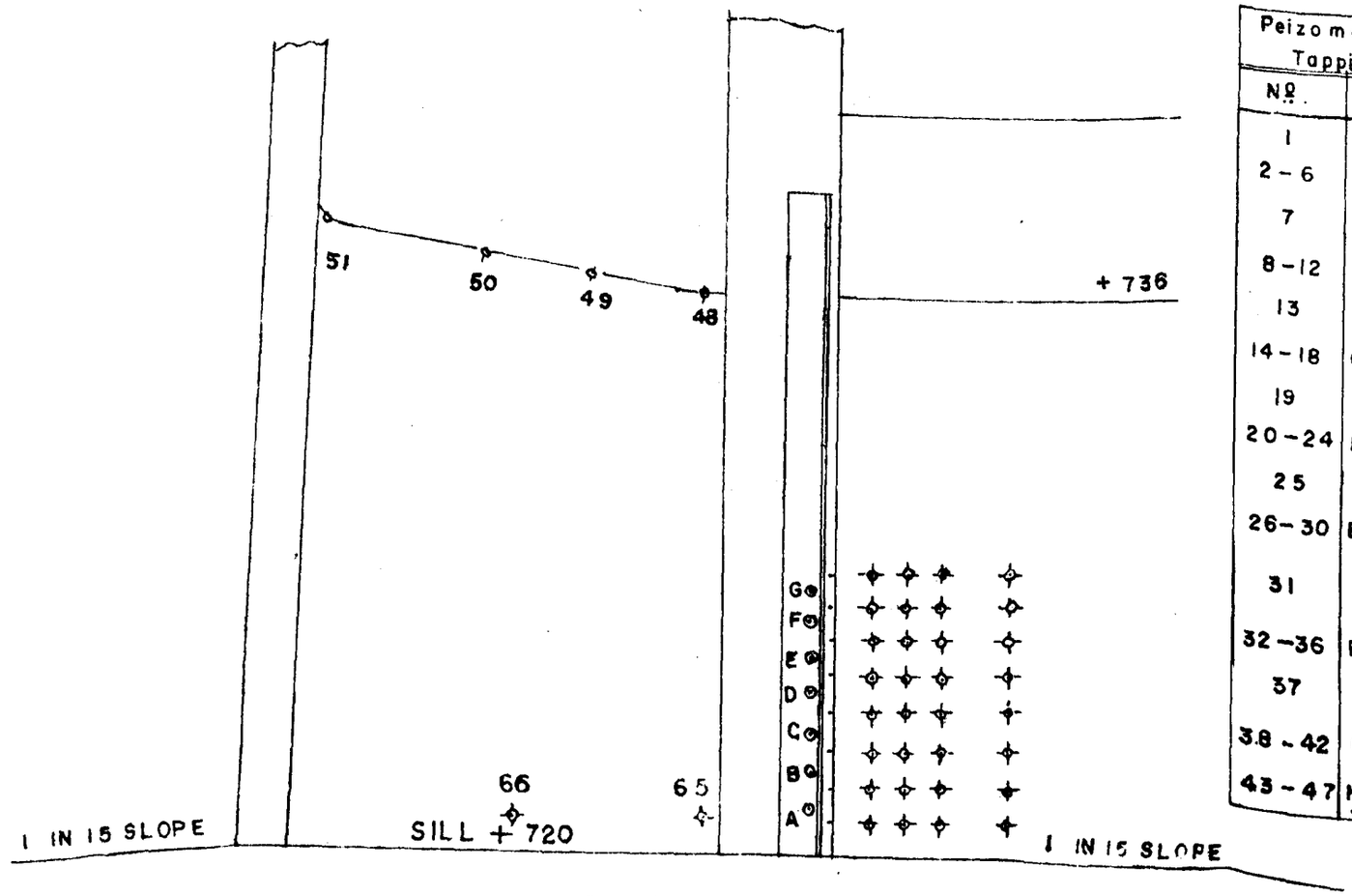
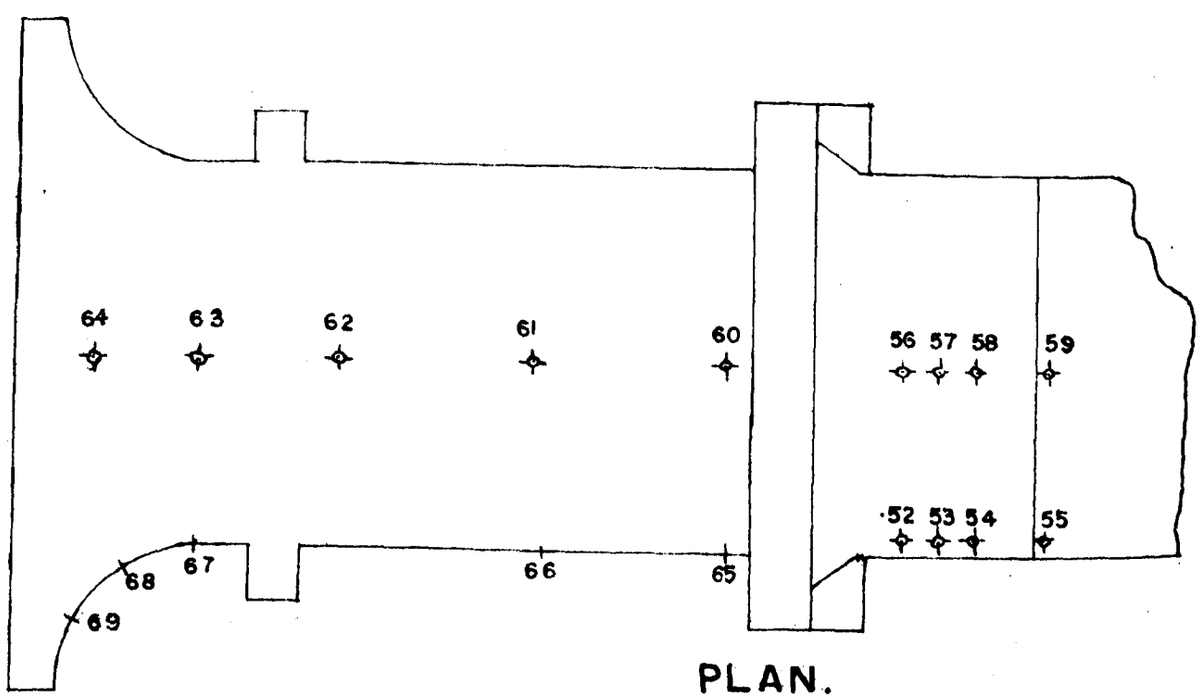


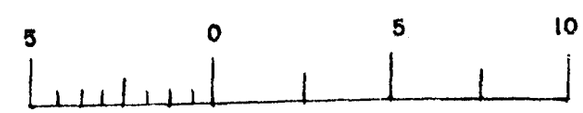
SECTIONAL ELEVATION

Peizometric Tappings.		Pz. Elevation	Peizometric Tappings.		Peizometric Tappings.	
No.	Letters.		No.	EL.	No.	EL.
1	A	721·50	48	736·00	63	719·90
2 – 6	A_1 – A_5	721·00	49	736·00	64	719·70
7	B	722·30	50	737·00	65	721·00
8 – 12	B_1 – B_5	722·00	51	738·00	66	721·00
13	C	723·50	52	720·00	67	721·00
14 – 18	C_1 – C_5	723·00	53	720·00	68	721·00
19	D	724·50	54	720·00	69	721·00
20 – 24	D_1 – D_5	724·00	55	719·98		
25	E	725·50	56	720·00		
26 – 30	E_1 – F_5	725·00	57	720·00		
31	F	726·50	58	720·00		
32 – 36	F_1 – F_5	726·00	59	719·98		
37	G	727·50	60	720·00		
38 – 42	G_1 G_5	727·00	61	720·00		
43 – 47	H_1 – H_5	728·00	62	720·00		

Read this drawing with Fig. 2.



PLAN.



SCALE IN FEET.

Annual Report of	1959
10/30	5/14
ITEM. NO	DRG. NO.

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

Cavitation Studies on Mettur Dam River Sluices

HIGH LEVEL SLUICE.
PEIZOMETRIC TAPPINGS
& ELEVATION.

55/60 Drg. File Report

J.E.R.	A.E.R.	E.E.R.
Trd: A.S.D.	Ckd:	Fig: ..... 5.

Reg. No 45-E 62 (Saka Era 1883) 375.

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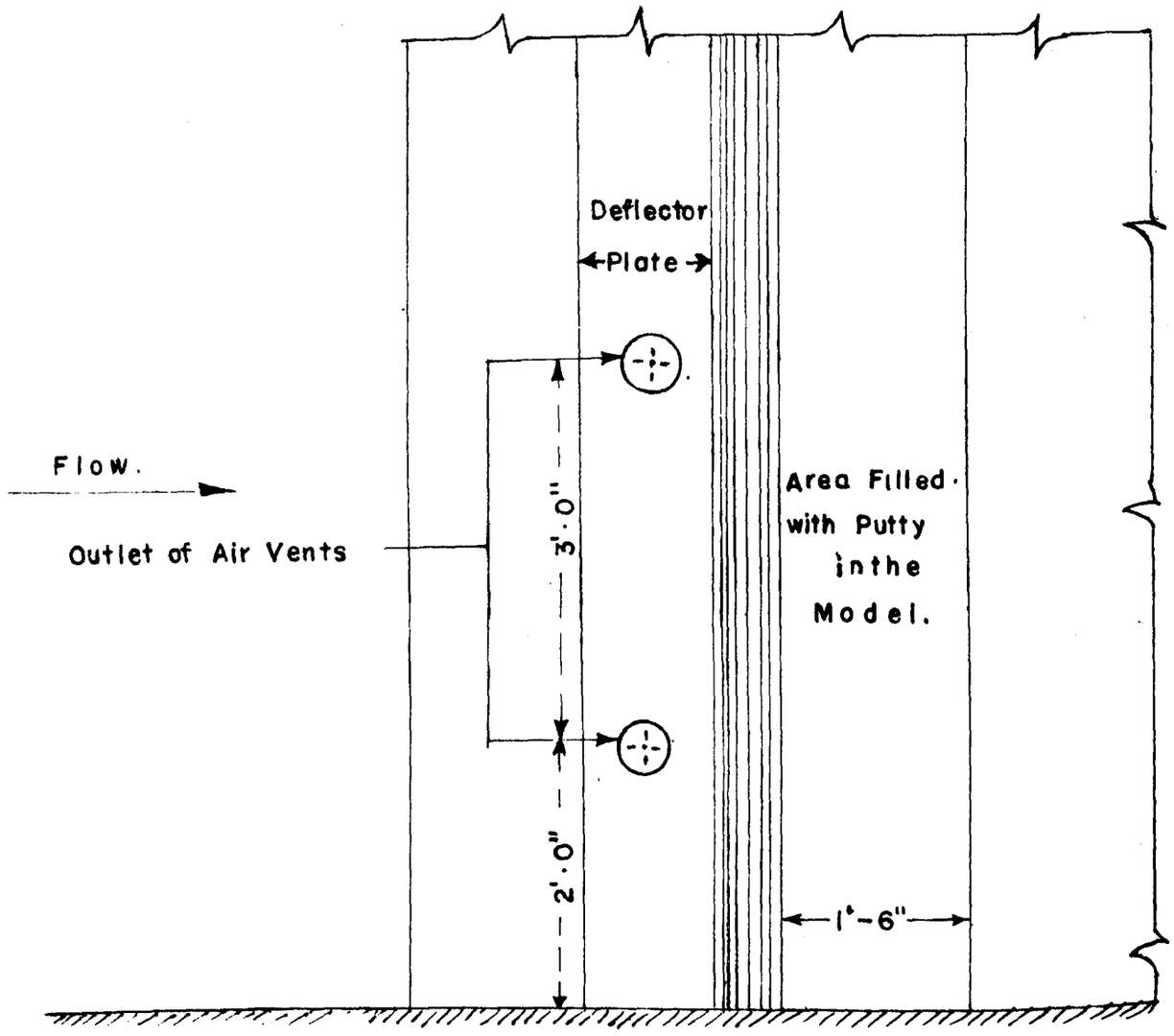
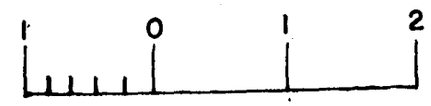


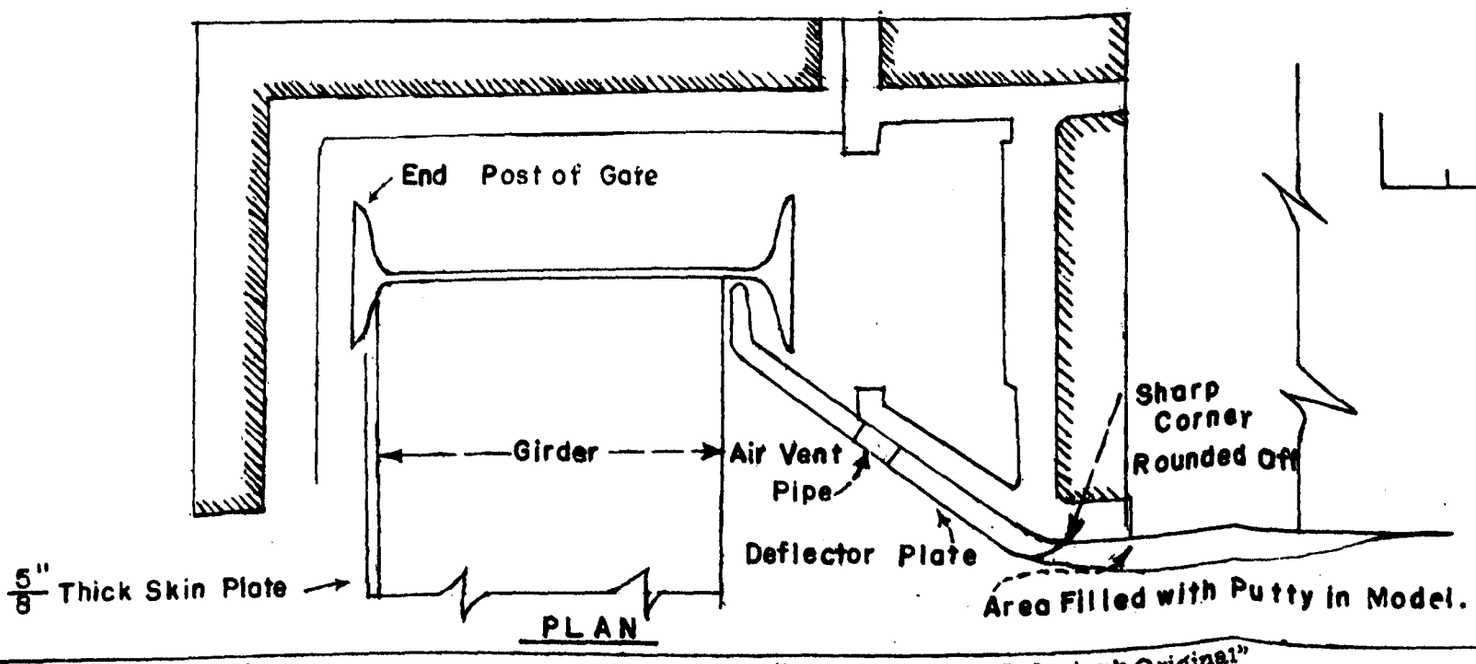
SECTION.



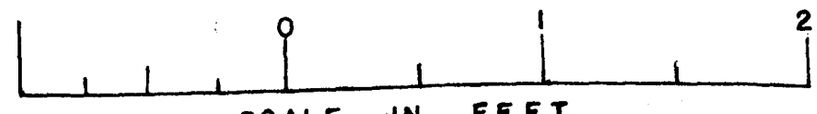
SCALE IN FEET.

Remedy i —— Rounding off the Corners

Remedy ii —— Providing Air Vents at places Shown in figure.



PLAN



SCALE IN FEET.

	Report.	Public Works Department — Madras. Irrigation Research Station — Poondi.		
		Cavitation Studies On Mettur Dam River Sluices.		
	File	DETAILS OF REMEDIES i & ii		
54/60	Drawing			
		J · E · R.	A · E · R.	E · E · R.
		Trd: A · S · D	Ckd:	Fig... No.

Reg No. 46-E'62 (Saka Era 1883) 375

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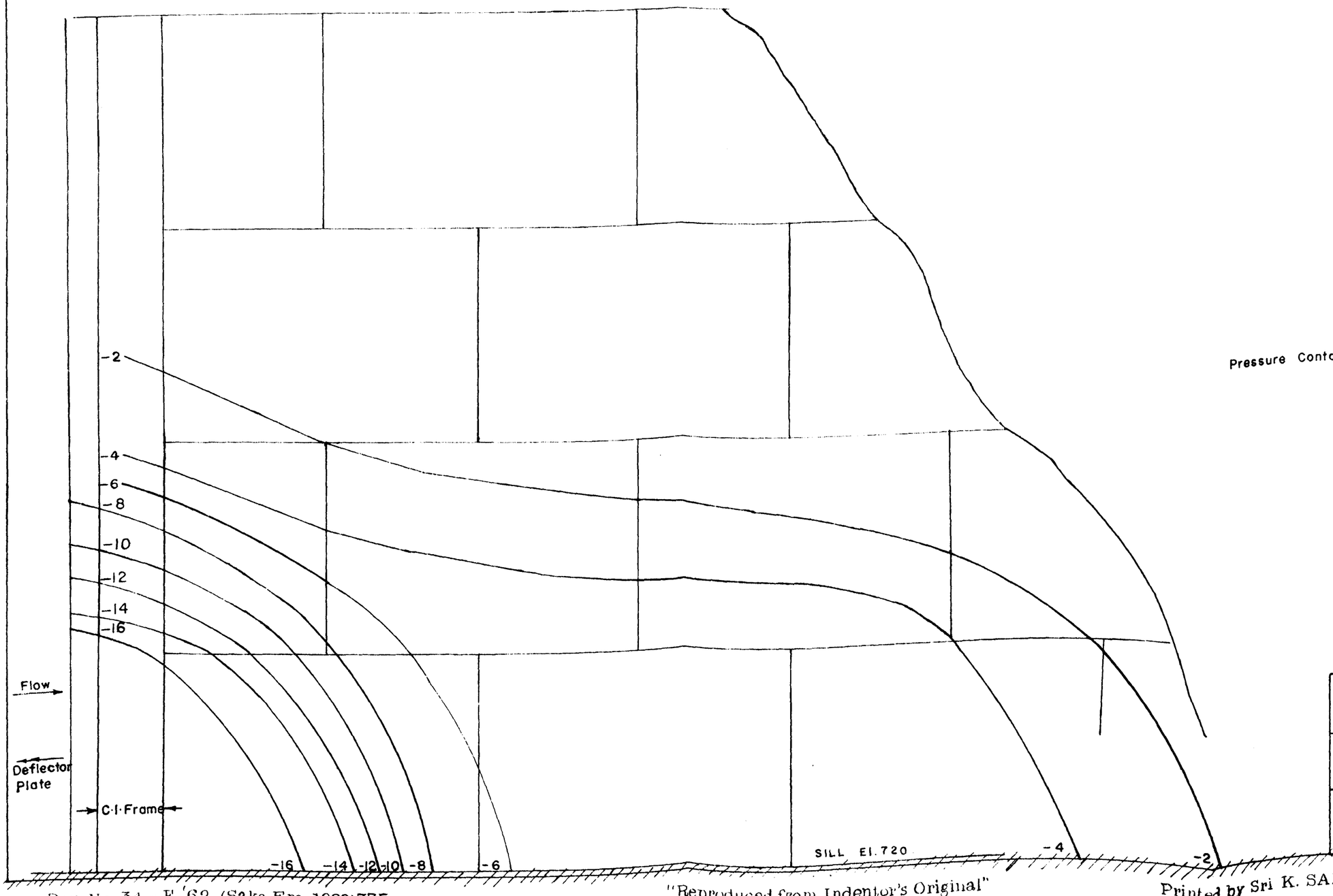
Item No	Drg. No.



Pressure Contours For → Water Level El. 790 (M.W.L.)
→ Gate Opening 3 Feet

Scale in inches
5 0 5 10

Report	Public Works Department—Madras Irrigation Research Station-Poondi		
	Cavitation Studies on Mettur Dam River Sluice		
File	HIGH LEVEL SLUICE PRESSURE CONTOURS FOR M.W.L IN RESERVOIR		
41/60 Drawing			
	J.E.R	A.E.R	E.E.R
	Trd: A.S.D.	ckd:	FIG... 7

Reg. No. 34.- E '62 (Saka Era 1883) 375

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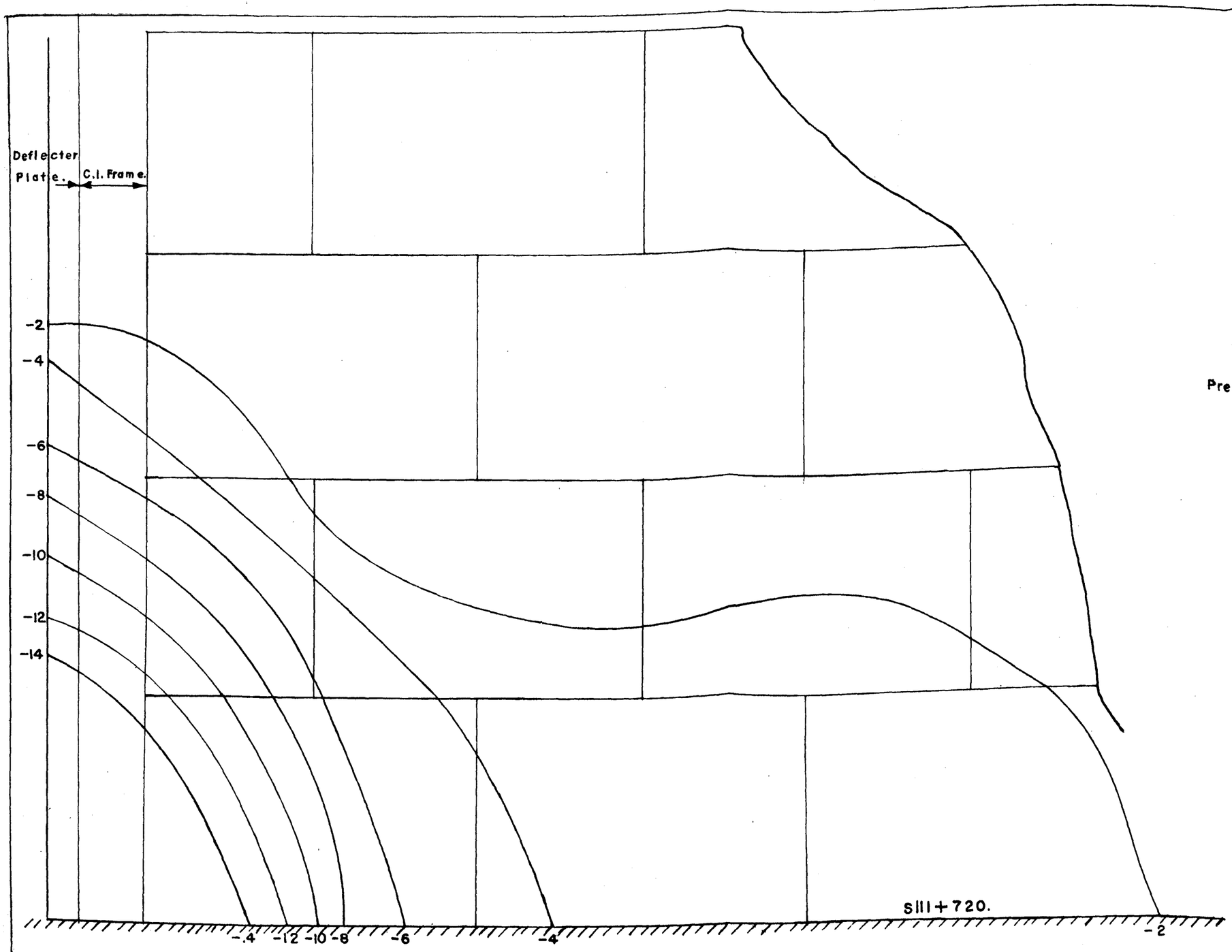
ITEM. NO	DRG. NO.



Pressure Contours For — Water Level El·770 / Gate opening 3 Feet

Scale in inches

5 0 5 10

Report	Public Works Department – Madras Irrigation Research Station – Poondi		
	Cavitation Studies on Mettur Dam River Sluices		
File	HIGH LEVEL SLUICE PRESSURE CONTOURS FOR W L +770·0 IN RESERVOIR		
40/60 Drawing	J · E · R	A · E · R	E · E · R
	Trd: C·S·K	ckd:	FIG · · · 8

Reg. No. 47-E'62 (Saka Era 1883) 375.

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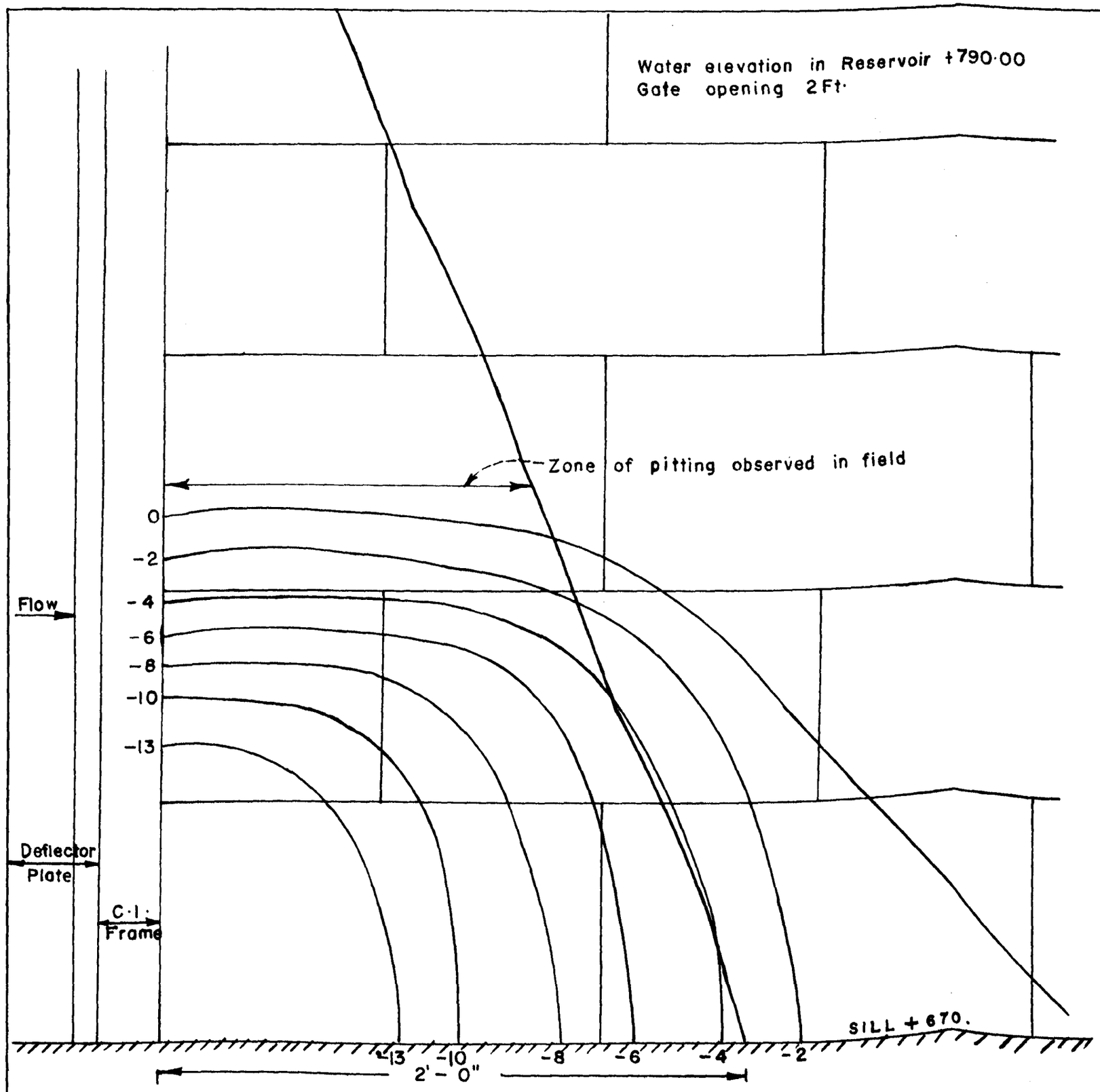

Report	Public Work Department—Madras. Irrigation Research Station—Poondi		
File	Cavitation Studies on Mettur dam River sluices Low Level Sluice PRESSURE CONTOURS FOR M·W·L· IN RESERVOIR		
58/60 Drawing			
	J·E·R	A·E·R	E·E·R
	Trd: A·S·D	ckd:	FIG···9

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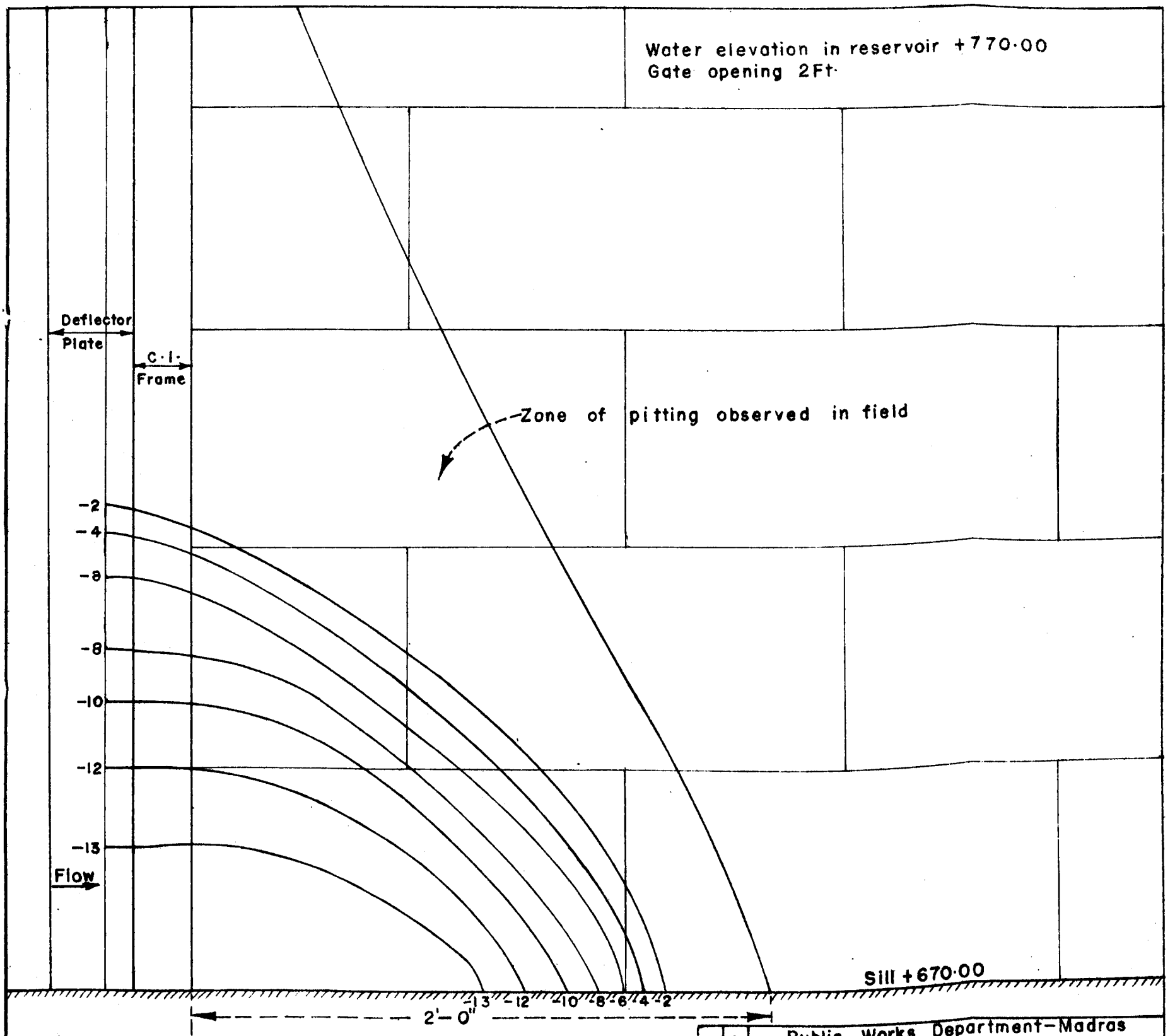

Report	Public Works Department—Madras Irrigation Research Station—Poondi		
File	Cavitation Studies on Metur dam river sluices Low Level Sluice PRESSURE CONTOURS FOR W·L· +770·00 IN RESERVOIR		
59/60 Drawing			
	J · E · R	A · R · O	E · E · R
	Trd: v·v·	ckd:	FIG···10

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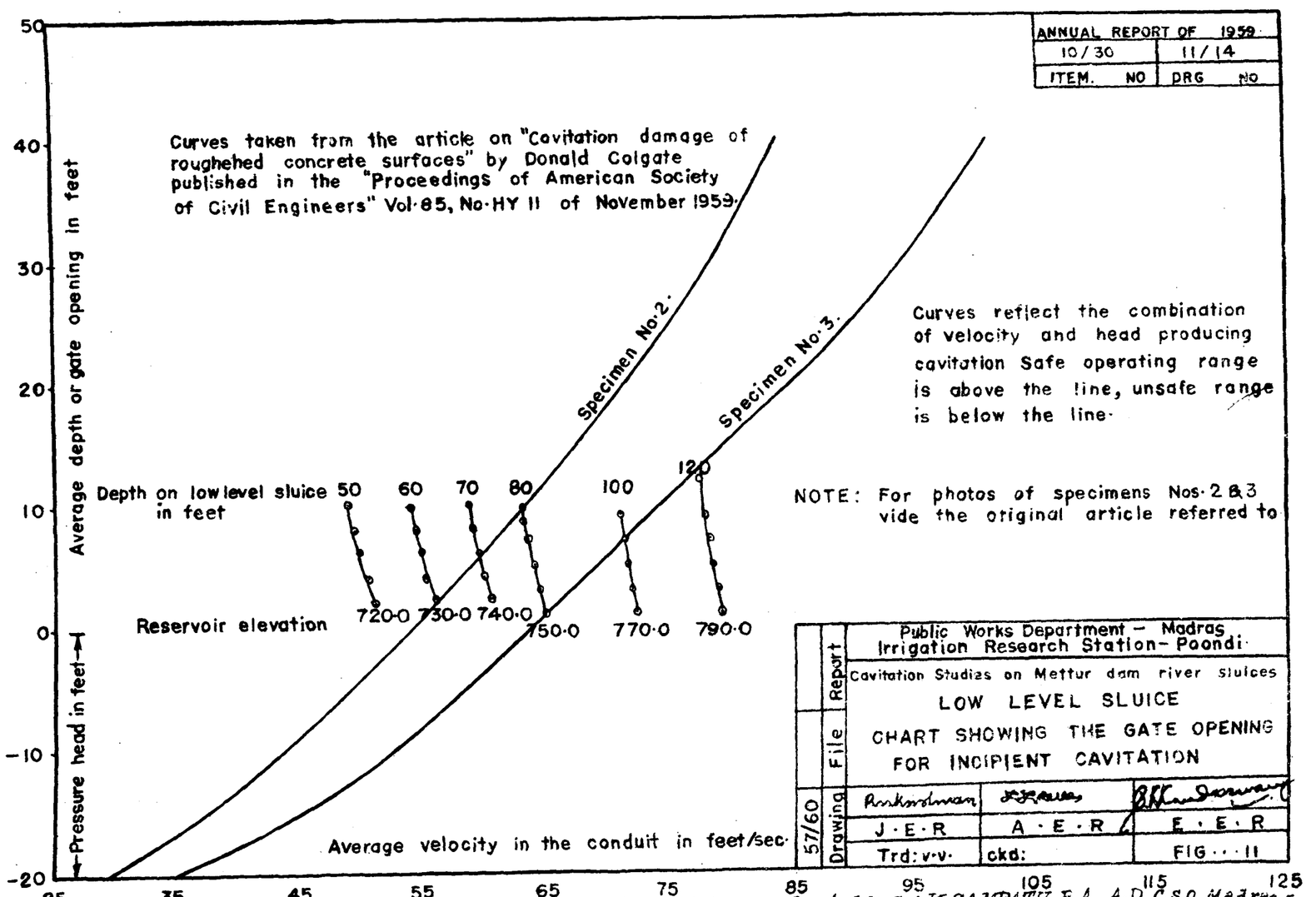
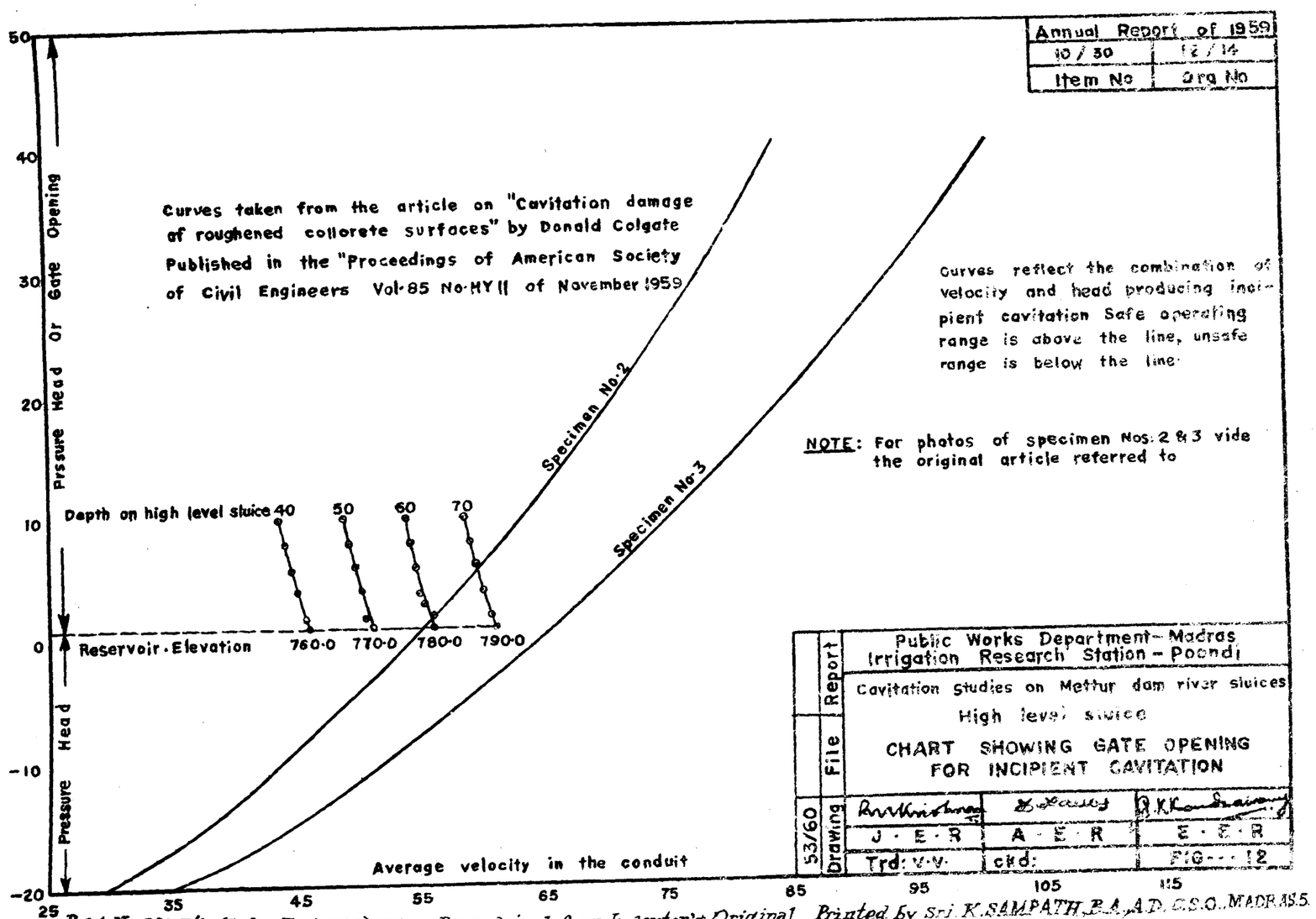
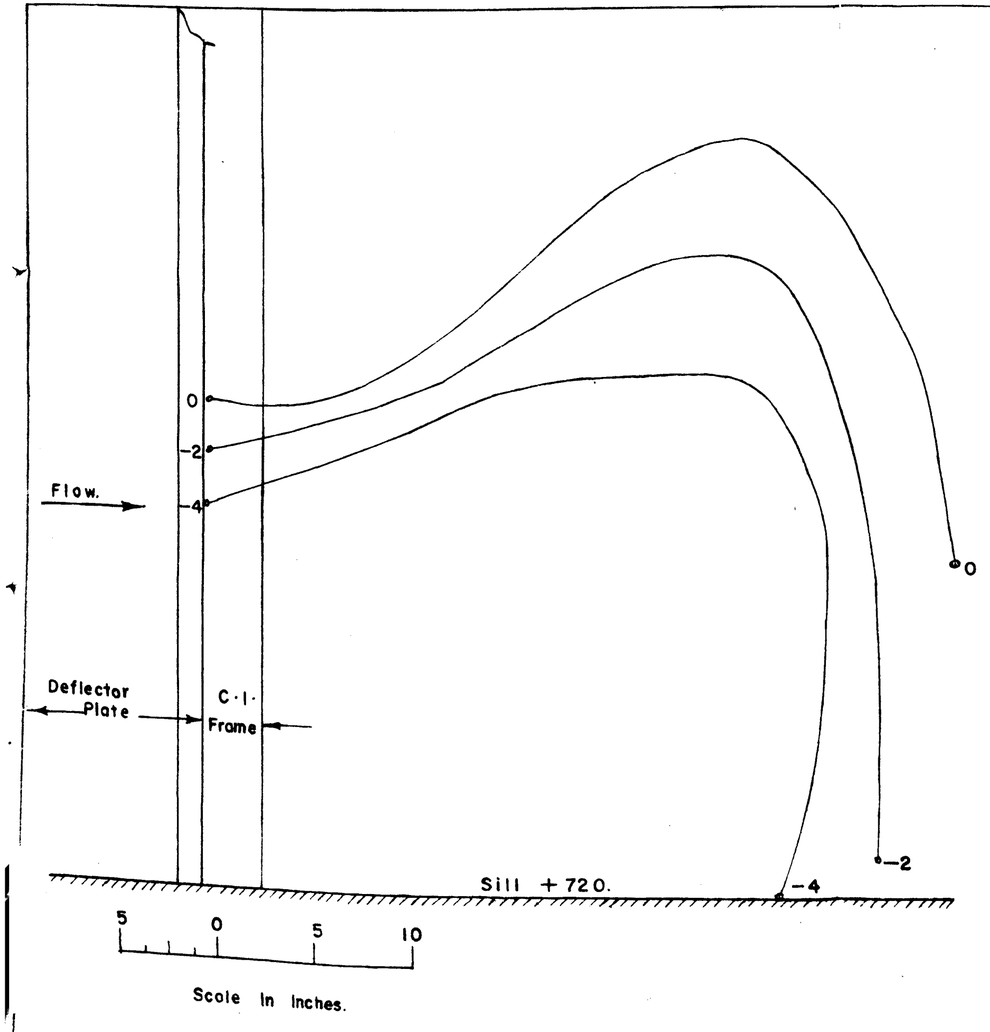


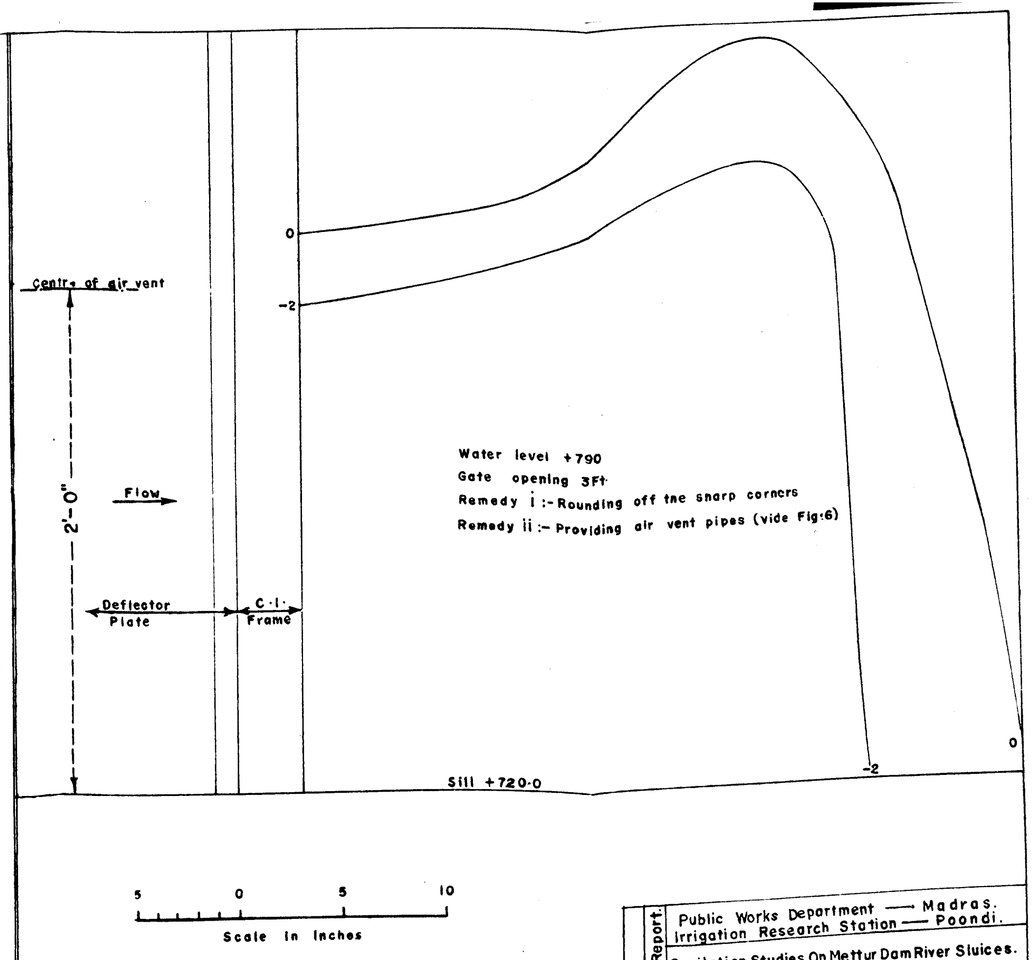
Reg. No 281-E'62(Sa.Ra.Era 1884) 375. Reproduced from Indentor's Original Printed by Sri. K. SAMPATH, B.A., A.D., C.S.O., Madras-5



Reg. No. 282 B'62 (Saka Era) 884)·375 Reproduced from Indentor's Original Printed by Sri K. SAMPATH, B.A., A.D. G.S.O., MADRAS.



Water Level.... E.L. +790
Gate Opening..... 3 Ft.
Remedy i = Rounding Off The Sharp Corners
(Vide Fig 6.)



Report:	Public Works Department → Madras. Irrigation Research Station → Poondi.		
File.	Cavitation Studies On Mettur Dam River Sluices. HIGH LEVEL SLUICE. PRESSURE CONTOURS ..WITH REMEDIES I & II.		
Drawing. 52/60	J·E·R.	A·E·R.	E·E·R.
	Trd: C·S·K.	Ckd:	Fig..No.....13

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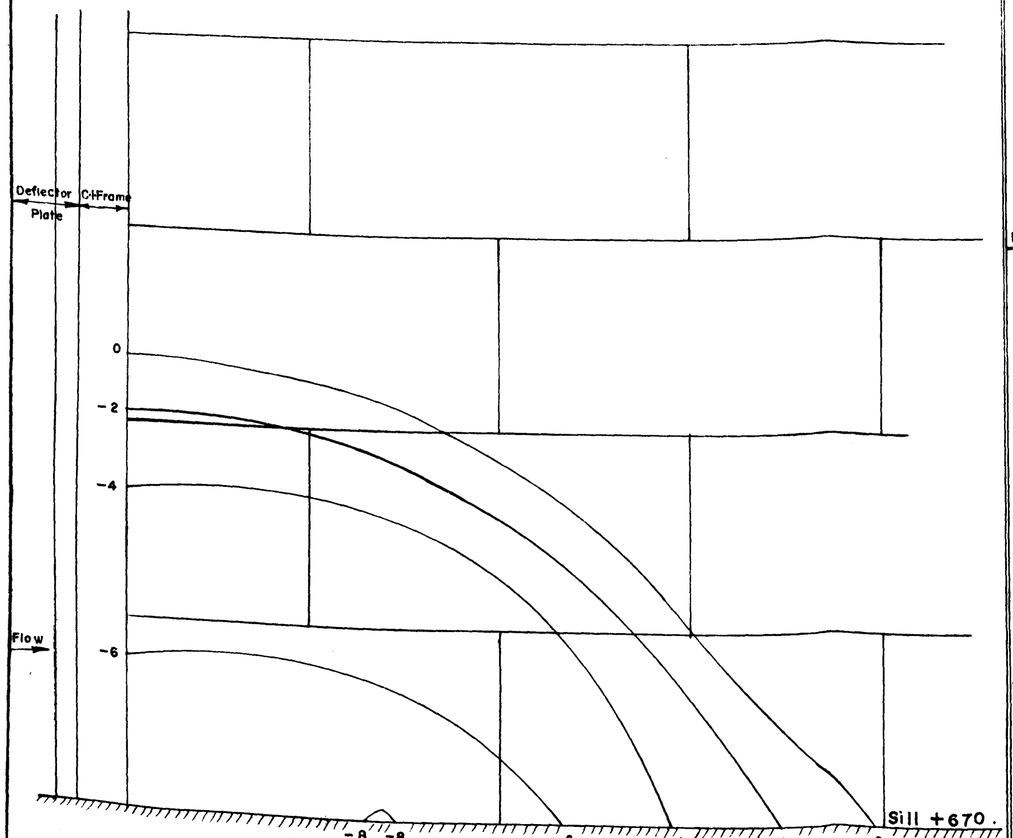
ITEM. NO	DRG. NO

Remedy I :- Rounding off the Sharp Corners (vide) Fig·6

Water Elevation in Reservoir +790·00

Gate Opening - 2 Feet·

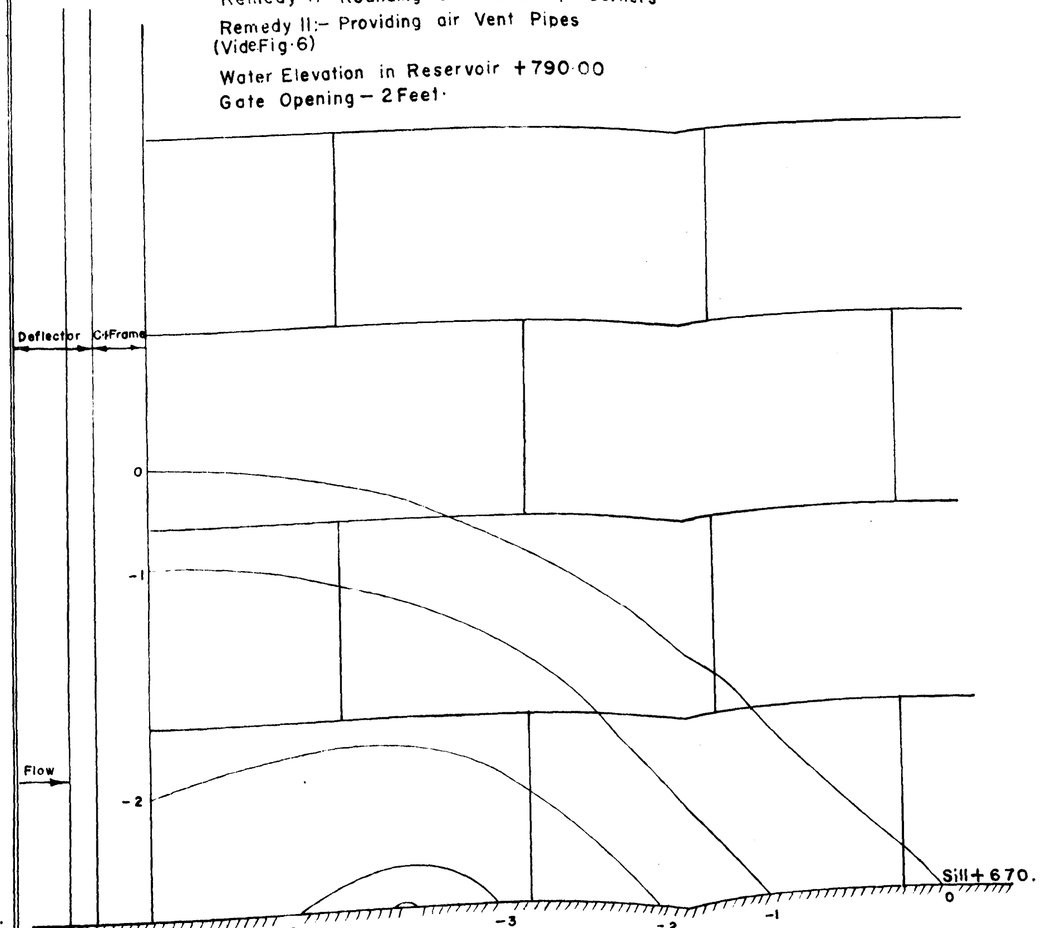


Remedy I :- Rounding off the Sharp Corners

Remedy II :- Providing air Vent Pipes (Vide Fig·6)

Water Elevation in Reservoir +790·00

Gate Opening - 2 Feet·



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

Cavitation Studies on Mettur Dam River Sluices.

LOW LEVEL SLUICE
PRESSURE CONTOURS WITH
REMEDIES I & II.

Report	File	Drawing 46/60	
		J·E·R	A·E·R
		Trd: C·S·K.	ckd:

E·E·R
FIG. 14.

Reg No.54 E'62 (Saka Era 1883) - 375.

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PARAMBIKULAM ALIYAR PROJECT

ALIYAR DAM SPILLWAY

Model Studies

1. SYNOPSIS

The spillway of Aliyar Dam is located on the right flank of the Aliyar river, unlike the conventional way of locating it in the river course. The spillway consists of ogee section and chute section separated by a horizontal apron, the height of the former being about 1/5th the height of the latter. A new surplus course is proposed to be excavated, connecting the spillway and the existing Aliyar river. It is desired to study the following aspects in the model, viz., energy dissipating arrangement at the toe of chute section of the spillway, pressure observation on the spillway section, river training works, the flow pattern downstream of the spillway upto the existing anicut and other allied problems.

2. INTRODUCTION

2.1. The Aliyar Dam is essentially a part of the comprehensive Parambikulam-Aliyar Project-apart from impounding the available Aliyar flows, this dam serves as a balancing reservoir to receive and utilise the waters diverted from Nirar, Sholayar and Parambikulam basins, for irrigating the ayacut on the eastern plains of western ghats through two canals, viz. Upper canal and Pollachi canal, taking off from the left and right flanks of Aliyar Dam respectively, (Figure-1). The spillway of Aliyar Dam is located on the right flank of the Aliyar river so that the existing high mound of rock outcrop is taken advantage of in economising the height of spillway to be constructed (Figure-2). This proposal proved to be cheaper than the one of having the spillway on the river course by 30 lakhs.

2.2. The spillway consists of ogee section for a height of 18 feet from the crest El. 1040.00 down to El. 1022.00 and chute section for a height of 92 feet from El. 1022.00 to El. 930.00, separated by a horizontal apron 25′0″ long at El. 1022.00 (Figure-3). In the chute spillway, the existing splid rock will be dressed to a slope of 2 : 1. There are 11 vents of 30′0″ each separated by 9′0″ thick piers, in the spillway. The spill channel at the toe of the spillway is 420 feet wide and it negotiates a bend of about 90° and a change of width to 200 feet and runs parallel to the axis of the dam till its confluence with the Aliyar river. A bridge consisting of 7 vents of 30 feet each has already been constructed across the Aliyar river downstream of the dam at about 300 feet below the point of confluence of the new surplus course with the Aliyar river, to carry the diverted road connecting Pollachi and Valparai. An anicut known as Pullivelangal anicut exists across the Aliyar river at about 700 feet downstream of the bridge and it serves as the point of control for the flow characteristics in the surplus course downsteam of the spillway.

3. DATA

3. 1. *Spillway.*

Crest of spillway	...	El. 1040.00
F. R. L. and M. W. L.	...	El. 1050.00
Free Board	...	9.0′
Road Bridge level over spillway	...	El. 1059.00
Width of roadway over dam between ogee spillway and chute spillway	...	1240′
Top of parapet	...	El. 1062.00
Level of horizontal apron between ogee spillway and chute spillway	...	El. 1022.00
Length of horizontal apron	...	25 feet
B. L. of channel at the toe of spillway (tentative)	...	El 940.00
Spillway vents	...	11 Nos. of 30′x 10′ each
Thickness of pier	...	9 feet
Max. flood discharge (calculated)	...	40957 cusecs
Discharge through the vents (Provided)	...	40,430 cusecs
Length of spillway including piers	...	420 feet

3. 2. *Surplus Course.*

Width of surplus course at the toe of spillway	...	420 felt	
		Original Design	Revised Design
Width of surplus course downstream of the bend	...	210 feet	200 feet
Bed slope	...	1 in 70	I in 180
Full supply depth	...	9′0″	12.4′
Velocity (calculated)	...	82.05′/sec	16.63′/sec
Capacity (computed)	...	41,680 c/s	41,270 c/s

3. 3. *Bridge.*

No. of vents and size	...	Nos. of 30′ wide each
M. F. L. calculated	...	El. 938.00
Free Board	...	3 feet
Level of bottom of deck beam	...	El. 941.00
Road level	...	El. 945.08
Width of roadway (Between kerbs)	...	24.0 feet

3. 4. *Anicut.*

Length of masonry anicut	...	255.0′
Length of training wall on the left	...	278.0′
Crest level of masonry anicut	...	El. 918.00
Top of training wall	...	El. 918.00 (average)
Top width of masonry anicut	...	6.0 feet
Top of right abutment, wing walls etc.	...	924.28
Average bed level of river downstream of anicut	...	El. 914.00
Top width of training wall	...	3.0′

4. PROBLEM

4. 1. The problems are intended to be studied in three stages and the scheme of studies is as follows :

PART I—STUDIES ON ENERGY DISSIPATION ARRANGEMENTS

(a) To evolve the most efficient and economic energy dissipating arrangements downstream of the spillway.

(b) To study the flow pattern in rear of the blunt end of the piers of the road bridge over spillway and to design efficient case waters.

PART II—PRESSURE STUDIES

(a) To observe the pressures developed along the surface of the spillway section and to recommend a cavitation free profile which will ensure non-occurrence of sub-atmospheric pressures for all free flow and gate controlled flow conditions.

PART III—RIVER TRAINING WORKS AND OTHER ALLIED PROBLEMS

(a) Te observe the effectiveness and efficiency of the energy dissipating arrangements, finalised in the sectional model, in 3-dimensional model and to effect any change in the arrangement if found necessary.

(b) To observe the water surface profiles from the toe of the spillway upto the anicut for maximum flood discharge conditions.

(c) To evolve suitable radii of curvature at the bends in the surplus course.

(d) To correct the flow pattern at the end in the river course and to make the water evenly pass through the vents of the bridge without endangering its safety.

PART I—ALIYAR DAM SPILLWAY—STUDIES ON ENERGY DISSIPATING ARRANGEMENTS

5. SECTIONAL MODEL

5. 1. A geometrically similar sectional model to a scale of 1/60 was constructed, representing only 5 vents out of the 11 vents in the prototype. The profile of the spillway section received from the Designs Office was adopted for the construction of the model (Figure-3). The spillway consists of ogee section from the crest El. 1040.00 to El. 1022.00 and chute section

with a slope of 2: 1 from El. 1022.00 to El. 940.00 seperated by a 25' long herixontal apron at El. 1022.00. The equation of the perabolic curve of the ogee section $i-Y-O.0298x^2+0.0965x-0.1508$ with the crest as origin. The radii of the circular curves connecting the ends of the chute section to the hortzontal floors at El 1022.00 and at El. 940.00 were not given in the original design and a radius of 37 feet had been adopted for both the curves by this station.

5.2. Trial pits excavated at the toe of the spillway revealed the presence of sound and hard rock at El. 930.00 only. Calculations for the depth of the stilling basin required a depth of 4.0'. Appendix I. Accordingly, the bed level of the surplus course below the stilling basin should be at El. 934.00. Good solid rock was also net with at El. 934.00 itself at a place about 50 feet below the toe of the spillway, the probable end of the stilling basin. Therefore it was decided to keep the bed level of the surplus course downstream of the basin at El. 934.00 which would provide a depth of 4.0' above the basin level of 930.00, (Figure-4) suitable alterations were done in the model to incorporate the above changes in bed levels (Photo-1).

5.3. The spillway section was built in masonry and its surface was finished smooth. A slope of 1 in 70 was adopted for the surplus course. The width of the surplus course below the bed and the curvature of the bend near the toe of the spillway were all simulated in the same ratio of the length of the spillway in the model to that in the prototype. A length of 900 feet from the axis of the spillway was represented in the model. The bed and the vertical sides of the surplus course were moulded rigid. The piers, road bridge on top of spillway and operating platform were made of timber and placed over the crest of the masonry spillway in the model in order to simulate the prototype conditions of flow through the ventway. Radial gates were provided for the ventway. Figure-5 shows the plan of the model set up. Levels were painted on the side walls of the fluxe to facilitate the measurement of water levels upstream of the spillway sufficiently far away from the spillway crest to avoid the effects of flow through the vents of the spillway. Figure-6, presents a comprehensive picture of the model lay-out.

6. EXPERIMENTS AND OBSERVATIONS.

6.1. The spillway section received from the Designs Office Figure-3, was tested in the model for maximum flow conditions with M.W.L. at El. 1050.00. Shooting flow was observed at the toe of the spillway which created a sudden unsteady and skew jump across the surplus course at the bend downstream of the spillway. Photo No. 2 shows this pattern of flow which indicates the necessity of a sunk type of stilling basin for energy dissipating arrangement.

6.2. The depth of basin required to produce the hydrollic jump is 4.0' vide calculations in Appendix 1. The values of the sequent depths of the hydraulic jump are 1.37' and 18.00'. The length of the basin may be 3 times the height of the jnmp which works out to 50.0'. One advantage of having a sloping spron at the toe of the spillway is that the length of basin can be reduced by making the jump to advance over the sloping spron. In this Aliyar Dam, the flat slope of 2.1 of the chute spillway plays the role of sloping apron and it was considered worthwhile to conduct studies for different lengths of basin lesser than 50 0'. Therefore three lengths of basin *viz*: 40 feet, 45 feet and 50 feet were proposed to be studied in the model.

Studies were also proposed to be conducted using friction blocks inside the basin in order to reduce the depth of flow and velocity of water leaving the basin.

6.3. It was observed in the model, that the curvature of the bend on the right side did not at all affect the flow conditions along the bend, but the curvature on the left side affected the formation of the jump and the jump appears to be pulled downstream on the right side of the channel. Hence it was considered better to have a straight back on the left side as for as possible in order to facilitate the formation of the hydraulic jump at right angles to the direction of flow. A nominal radius of 50 feet and a minimum radius of 5 feet were adopted for the right side and left side curves of the bend respectively, (Figure-7) which ensured the formation of the jump as desired.

6.4. Experiments were conducted for maximum flood discharge conditions keeping the M.W.L. at El. 1050.00 with and without the friction blocks of size 4' x 3' x 3' each at 10 feet centres for the three lengths of basin, *viz*: 40 feet, 45 feet and 50 feet. Velocities and depths of flow were observed along the centre line of spillway and the surplus course. The difference between the specific energies just before and after the jump accounts for the loss of energy in the basin. The percentage loss of energy obtained in each of the six trials was computed and this was considered as the primary criterion in deciding the best energy dissipating arrangement. The results are in Statement No. 1.

6.5. In all the trials it was observed that the trial water level was built up slowly as the intensity of discharge was increased from the minimum to the maximum. A steady and uniform hydraulic jump was formed in the case of 50 feet long basin. The formation of the jump was more steady and uniform in the cases of 45 feet and 40 feet long basins (Photo 3). The hump of the hydraulic jump perfectly merged with the tail water level in the first two cases whereas a fall of about 2 to 3 feet from the hump elevation to the tail water level was observed in the third case. Reduction of the order of 0.50 ft. (average) in the water level after the jump was noticed when friction blocks were used inside the stilling basin.

6.6. At the bend below the basin the flow appeared to be very slow with appreciably small velocities ranging from 3 to 4 feet/second. After the bend the flow acquired greater velocity of the order of 19 to 22 feet/second. The water surface profiles taken along the centre line of the spillway and the surplus course are plotted in Figures 8 and 9. Transeverse flow occurred in the surplus course below the bend and at the downstream end of the left side curve there was a sudden drop in the flow coupled with return flows. The flow was uniform and steady along the right side curve.

6.7. The flow pattern on the spillway section in rear of the blunt end of the spillway piers was observed to be quite unsatisfactory and it was felt obsolutely essential to device an efficient case water at the blunt rear end of the spillway piers. As originally designed, the cut water at the front end of the pier is equiangular one with circular curves of 6.8 feet radius, (Figure-10). and this is found to be quite alright. Three alternate devices were proposed for the case waters in rear of the spillway piers, *viz*: (1) Equi-angular case waters with circular curves of radius epual to the thickness of the pier attached to the originally proposed blunt end, (2) semi-circular case waters with radius equal to half the thickness of the pier attached to the originally proposed blunt end rounded off with circular covers of 3 feet radius. The

flow profiles in rear of the spillway piers for all the above four proposals including the original design, were traced out and plotted in Figur-11.

7. DISCUSSION OF RESULTS

7.1. Since sound and hard rock is met with at the toe of the spillway, it is considered uneconomical to use friction blocks inside the stilling basin which will entail the recurring expenditure in the maintenance of these friction blocks because of its vulnerability to the attack of the fast moving column of water added by the inevitable imperfectness of the connection between the concrete blocks and the rocky floor of the basin. Entering Statement No. 1, it is seen that the percentage loss of energy is greater if the friction blocks are used. But the increase in the efficiency obtained by using friction blocks is not considerably appreciable to warrant the extra expenditure involved in the construction of the friction blocks. Hence, it is found economical to adopt a sunk type stilling basin without any friction blocks. An analysis of the percentage loss of energy obtained for the three lengths of basin without friction blocks *viz*: 40 feet, 45 feet and 50 feet reveals that the 45 feet long basin is efficient. In this case, at a point 350 feet below the axis spillway the Vol. 1930′/second the depth of flow is 14.22 feet and the maximum hump elevation is only El. 948.80 which are the minimum compared with those of the other cases. It is observed that the velocity and the depth of flow in the surplus course downstream of the spillway are the minimum in the case of 45 feet long basin. Hence, a 45 feet long and 4 feet deep basin is recommended for adoption as an energy dissipating device at the toe of the Aliyar Dam Spillway.

7.2. Referring to Figure-11, it is observed that the elevation of the water surface is minimum in the case of equangular case waters with circular curves of radius equal to the thickness of the pier *i.e.* 9 feet and the same is recommended for adoption in rear of the spillway piers.

8. CONCLUSION

8.1. The following recommendations are offered in respect of the energy dissipating arrangements at the toe of the Aliyar Dam Spillway.

(*a*) A sunk type stilling basin of 45 feet length and 4 feet depth may be adopted at the toe of the spillway. The bed levels at the basin and at the beginning of the surplus course held the basin may be at El. 930.00 and El. 234.00 respectively.

(*b*) An equiangular case water with circular curves of radius equal to the thickness of the pier *i.e.* 9 feet may be adopted in rear of the originally proposed blunt end of the spillway piers.

8.2. The above recommendations are found to be efficient only for the smooth surface of the chute spillway section between 1022.00 aud El. 930.00, with a slope of 2:1. It is gathered from the field that the existing rock outcrop will be left with the surface as obtained by only removing the overburden soil and loose bounders alone, in which case the surface of the chute section will be uneven and rugged. The efficacy of these arrangements will have to be tested by simulating the conditions of the rock surface obtained in the prototype. It is also essential to test the performance of these arrangements in a three dimensional model where any change in the flow pattern can be studied with reference to the side effects of the spray wall and the finalised design of the surplus course.

9. BIBLIOGRAPHY

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2. Irrigation Engineering—Volume II—Projects, Conduits and Structures by Ivan E. Houk, curve losses in open conduits, pages 108 and 109.

3. "Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers", April 1959 — Paper on "TWO METHODS TO COMPUTE WATER SURFACE PROFILES" by Joe M. Lara and Kenneth. B. Schoedar, pages 79 to 82.

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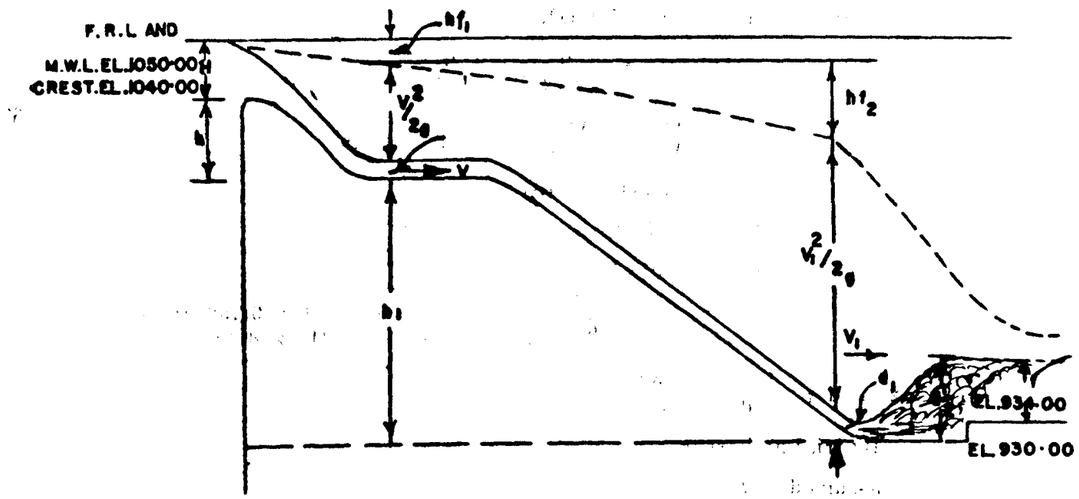
APPENDIX

1. Calculations for depth of Hydraulic Jump.



Considering the spillway into two portions (i) Ogee spillway between El. 1040.00 and El. 1022.00 and (ii) chute spillway between El. 1022.00 and El. 930.00.

(i) Ogee spillway between El. 1040.00 and El. 1022.00. Applying Bernoulli's Thereom,

$$H + h = d + \frac{v^2}{2g} + hf_1 \qquad \ldots (1)$$

where

q — discharge in Cft/sec. per linear foot of channel width

d — the depth upstream from the jump in feet.

g — acceleration due to gravity in feet/sec. — 32.2 ft./sec.^{2}

But v — q/d and by Mannings formula.

$$v = \frac{1.486}{n} R^{2/3} S^{\frac{1}{2}}$$

$$S_{\frac{1}{2}} = \frac{vn}{1.486\, R^{2/3}}$$

Assum R — d, for the vide and shallow channel.

$$S^{\frac{1}{2}} = \frac{vn}{1.486\, d^{2/3}}$$

$$S - \frac{v^2 n^2}{(1.486)^2 d^{4/3}} = \frac{v^2 n^2}{2.21 d^{4/3}}$$

$$S \times h = hf_1 = \frac{v^2 n^2 n}{2.21 d^{4/3}} = \frac{n^2 q^2 h}{2.21 d^{10/3}}$$

Substituting in equation (1),

$$H + h = d + \frac{q^2}{2gd^2} + \frac{n^2 q^2 h}{2.21 d^{10/3}}$$

$$q = \frac{Q}{b} = \frac{40957}{429} = 97.52 \text{ cusecs.}$$

$H = 1050.00 = 1040.00 = 10.0'$

$h = 1040.00 = 1022.00 = 18.0'$

$n = 0.013$ for concrete surface.

$$10.0' + 18.0 = d + \frac{97.52^2}{2 \times 32.2\, d^2} + \frac{0.013^2 \times 97.52^2 \times 18}{2.21 \times d^{10/3}}$$

$$28 = d + \frac{147.7}{d^2} + \frac{13.09}{d^{10/3}}$$

By trial and error,

assumed = 2.44

Substituting, $28 = 2.44 + \frac{147.7}{2.44^2} + \frac{13.09}{2.44^{10/3}}$

$28 = 2.44 + 24.50 + 0.6692 = 2791$

$d = 2.44'$

$$v = \frac{Q}{A} = \frac{80957}{420 \times 2.44} = 39.96'/\text{sec.}$$

(ii) Chute spillway between El. 1022.00 and El. 930 00
Applying Bernoulli's Theorem;

$$d + \frac{v^2}{2g} + h_1 = d_1 + \frac{v_1^2}{2g} + hf_2 \quad \ldots (3)$$

i.e. $$d + \frac{v^2}{2g} + h_1 = d_1 + \frac{q2}{2gd_1^2} + \frac{n^2 q^2 h_1}{2.21\, d_1^{10/3}} \quad \ldots (4)$$

$d = 2.44'$

$v = 39.96'/\text{sec.}$

$h = 1022.00 - 930.00 = 92.00$

$g\ n = 0.017$ for dressed ashlar surface.

Substituting in equation (4),

$$2.44 + \frac{39.96^2}{2 \times 32.2^2} + 92.0 = d_1 + \frac{97.52^2}{2 \times 32.2 d_1^2} +$$

$$+ \frac{0.017^2 \times 97.52^2 \times 92}{2.21\, d_1^{10/3}}$$

$$2.44 + 24.80 + 92.0 = d_1 + \frac{147.7}{d_1^2} + \frac{114.4}{d_1^{10/3}}$$

$$119.24 = d_1 + \frac{147.7}{d_1^2} + \frac{114.4}{d_1^{10/3}}$$

By trial and error,

assume $d_1 = 1.37$,

$$119.24 = 1.37 + \frac{147.7}{1.37^2} + \frac{114.4}{1.37^{10/3}}$$

$119.24 = 1.37 + 40.05 + 79.70$

$\underline{119.24} = \underline{120.12}$

$d_1 = 1.37$

$$v = \frac{97.52}{1.37} = 71.19'/\text{sec.}$$

$$= -\frac{d_1}{2} \sqrt{\frac{2q^2}{gd_1} + \frac{d_1^2}{4}}$$

$$= -\frac{1.37}{2} \sqrt{\frac{2 \times 97.52^2}{32.2 \times 1.37} + \frac{1.37^2}{4}}$$

$= -0.685 + \sqrt{431.1 + 0.4691}$

$= -0.685 + \sqrt{431.5691}$

$= -0.685 + 20.78 = 20.095$ or 20.10.

2. Design of Surplus course Downstream of the bend near the toa of the Spillway

The width of the surplus course is restricted to 210 feet as a maximum in order to avoid rock outting on the right side of the channel, and to economise the construction of retaining wall at the rear toe of the Earth Dam. The slope is also decided with reference to the rock levels obtained along the alignment of the surplus course.

The bed level of the surplus course at the beginning is kept at El.934.00 where hard rock is met with. The bed level of the surplus course at the junction with the river is kept at El. 922.00 Length of the surplus course from the downstream end of stilling basin to the junction of river course is 340 feet.

The total fall is 934.00 - 922.00 - 12.0'

Bed slope - $\frac{12.0}{840}$ = 1 in 70

Assume a Section 210' x 9.0' Area A - 210' × 9' - 1890 sq. ft.

Wetted perimeter - p - 210 + 2 x 9.0 - 210 + 18 = 228

Hydraulic mean radius - R - A/P - $\frac{1890}{228}$ - 8.29

By Manning's formula $v = \frac{1.486}{a} S^{\frac{1}{2}} R^{2/3}$

$$= \frac{1.486}{0.033} \left(\frac{1}{70}\right)^{\frac{1}{2}} \times 8.29$$

= 22.05'/sec.

Discharge - AV - 1890 x 22.05 - 41,680 c/s.

against a required discharge of 40,957 c/s.

Hence; adopt a section 210' x 9' with a slope of 1 in 70.

3. Design for the Depth and Length of Stilling Basin.

The channel at the toe of the spillway is 420 feet wide; it negotiates a bend of about 90° and its width also is reduced to 210 feet below the bend. The depth of tailwater at the toe of the spillway is computed by using Bernoulli's Theorem.

Consider the two sections at which the change of width is effected i.e., one at 365 feet downstream of the axis of spillway and the other at 645 feet downstream of the axis of spillway.

Section I. At 365 feet downstream of the axis of spillway :

Width of channel b_1 — 420 ft.

depth of flow d_1 —

Discharge — Q — 40957 cusecs.

Velocity - $V_1 = \frac{Q}{A_1} = \frac{Q}{b_1 d_1} = \frac{40957}{420 \times d_1}$

$$= \frac{97{\cdot}52'}{d_1} \text{ sec.}$$

By Manning's formula, $V_1 = \frac{1.486}{n} \times R_1^{\frac{2}{3}} \times Sf_1^{\frac{1}{2}}$

$$Sf_1^{\frac{1}{2}} = \frac{Vn}{1{\cdot}486 \; R_1^{\frac{2}{3}}} = \frac{Qn}{1{\cdot}486 \; A_1 \; R_1^{\frac{2}{3}}}$$

$$Sf_1 - \text{friction slope} - \left[\frac{Qn}{1{\cdot}486 \; A_1 \; R_1^{\frac{2}{3}}}\right]$$

$A_1 = b_1 d_1 = 420 \times d_1 = 420 \; d_1$

$B_1 = d_1$ (for wide and shallow channel).

n = 0·033 for blasted rock surface.

$$Sf_1 = \left(\frac{40957 \times 0{\cdot}033}{1{\cdot}486 \times 420 \; d_1 \times d_1^{\frac{2}{3}}}\right)^2$$

$$= \frac{4.685}{d_1^{10/3}}$$

Section II. At 645' downstream of the axis of spillway width of channnel — b_2 - 210 ft.

Depth of flow - d_2 - 9·0'

Discharge — Q - 40957 c/s.

Velocity - $V_2 = \frac{Q}{A_2} = \frac{q}{b_2 d_2}$

$$= \frac{40957}{210 \times 9} = \frac{40957}{1890} = 21.67'/\text{sec.}$$

$$Sf_2 - \text{friction slope} - \left[\frac{Qn}{1.486 \times A_2 \times R_2^{\frac{2}{3}}}\right]^2$$

$R_2 = b_2 d_2 = 210 \times 9 = 1890$ Sq. ft.

$R_2 = d_2 = 9.0'$ (for wide and shallow channel).

n = .033 an increase of 0.001 per 20°

Curvature is adopted in the value of *n* to include bend losses 3, 4.

$$= 0.033 + \frac{90}{20} \times 0.001 = 0.033 + 0.0045$$

$$= 0.0375$$

$$Sf_2 = \left(\frac{40957 \times 0{\cdot}0375}{1{\cdot}486 \times 1820 \times 9^{\frac{2}{3}}}\right)^2 = 0.01597$$

Loss of head due to friction — h.—

$L\left(\frac{Sf_1 + Sf_2}{2}\right)$ where L is the distance between the two sections.

$$= \frac{280}{2} \times \left(\frac{4{\cdot}683}{d_1^{10/3}} + 0.01597\right)$$

$$= 140 \left(\frac{4.683}{d_1^{10/3}} + 0.01597\right)$$

$$= \frac{656.32}{d_1^{10/3}} + 2.23580$$

As the above expression included the bed losses also h_f

other losses $= \frac{656.32}{d_1^{10/3}} + 2.2358$

Applying Bernaulli's Theorem between sections I and II,

$$Z_1 + d_1 + \frac{V_1^2}{2g} = Z_2 + d_2 + \frac{V_2^2}{2g} + (h_f + \text{other losses})$$

Where Z_1 and Z_2 are the datum heads in the respective sections and other losses—bend loss, eddy loss, bridge pier loss, etc. In this case there are no bridge pier loss and eddy loss (as the velocity head is only increased towards the reach) and hence only bend loss is taken into consideration.

Taking the bed level of the surplus course at Section-II as datum and substituting the values,

$$\frac{250}{70} + d_1 + \frac{97.52^2}{d_1^2 \times 2 \times 32.2} = 0 + 8.0 + \frac{21.67^2}{2 \times 32.2} + \frac{656.32}{d_1^{10/3}} + 2.2358$$

$$4 + d_1 + 147.7/d_1^2 = 9.0 + 7{\cdot}29 + 656.32/d_1^{10/3} + 2.2358$$

$$d_1 + \frac{147{\cdot}7}{2} - \frac{656.32}{d_1^{10/3}} = 18.5258 - 4.0 = 14.5258$$

By trial and error, assumed $d_1 = 13.9$

$13.9 + 0.7643 - 0.1046 = 14.5258$

i.e. $14.6643 - 0.1016 = 14.5268$

i.e. $14.5627 \simeq 145258$

$d_1 = 13.9'$

Tail water depth $= t = 13.9'$

Depth of hydraulic jump as calculated in item 1 above — 20.1'

Model studies have revealed that the depth of the pool required for the hydraulic jump to occur is only 85% to 90% of the theoretical value obtained by using the formula for the depth of the hydraulic jump.

$$= 20.1 \times \frac{90}{100} = 18.00 \text{ or } 12.1'$$

Tail water depth $= t = 13.9'$

Depth of stilling basin required — Depth of jump (d_1) —

Tail water depth (t) = 18.1 — 13.9 = 4.2' or 4.0'

Hence about 4.0' deep basin. The basin level may be kept at El. 930.00 and the bed level of the surplus course below basin may be at El. 934.00.

***Note*:—For Figures 1 to 3,**

Refer Figures 1 to 3 of Item 7.

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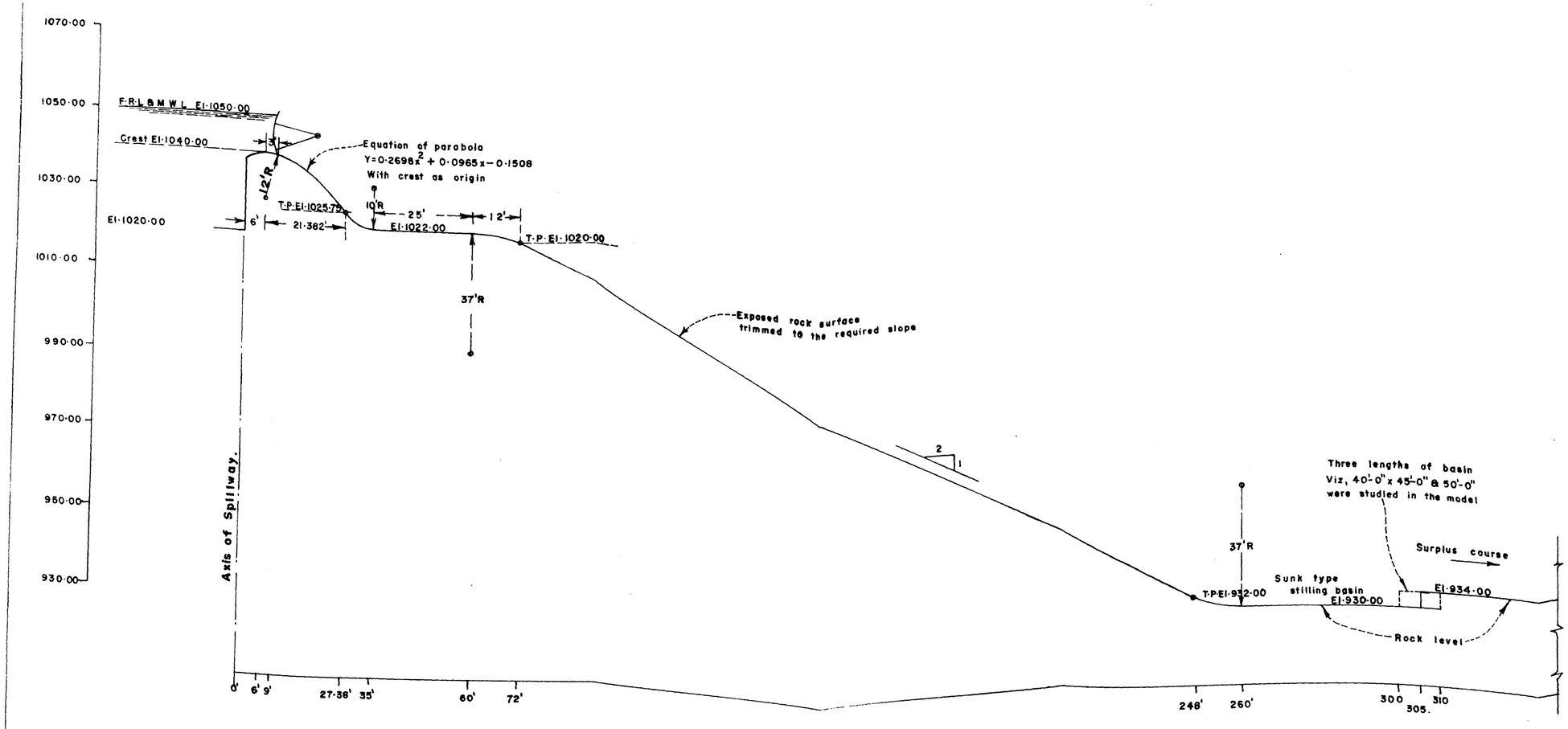
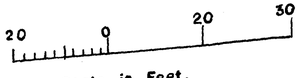
11/30	4/11
Item NO	Drg NO





Scale in Feet.

Report	Public Works Department — Madras Irrigation Research Station — Poondi		
File	ALIYAR DAM SPILLWAY SECTION (Revised design)		
2/8/60 Drawing	A. Shanmugam		E · E · R
	J · E · R	A · E · R	E · E · R
	Tr: V·V·	ckd:	FIG ··· 4

Reg.No.51-R'62 (Saka Era 1883) - 375.

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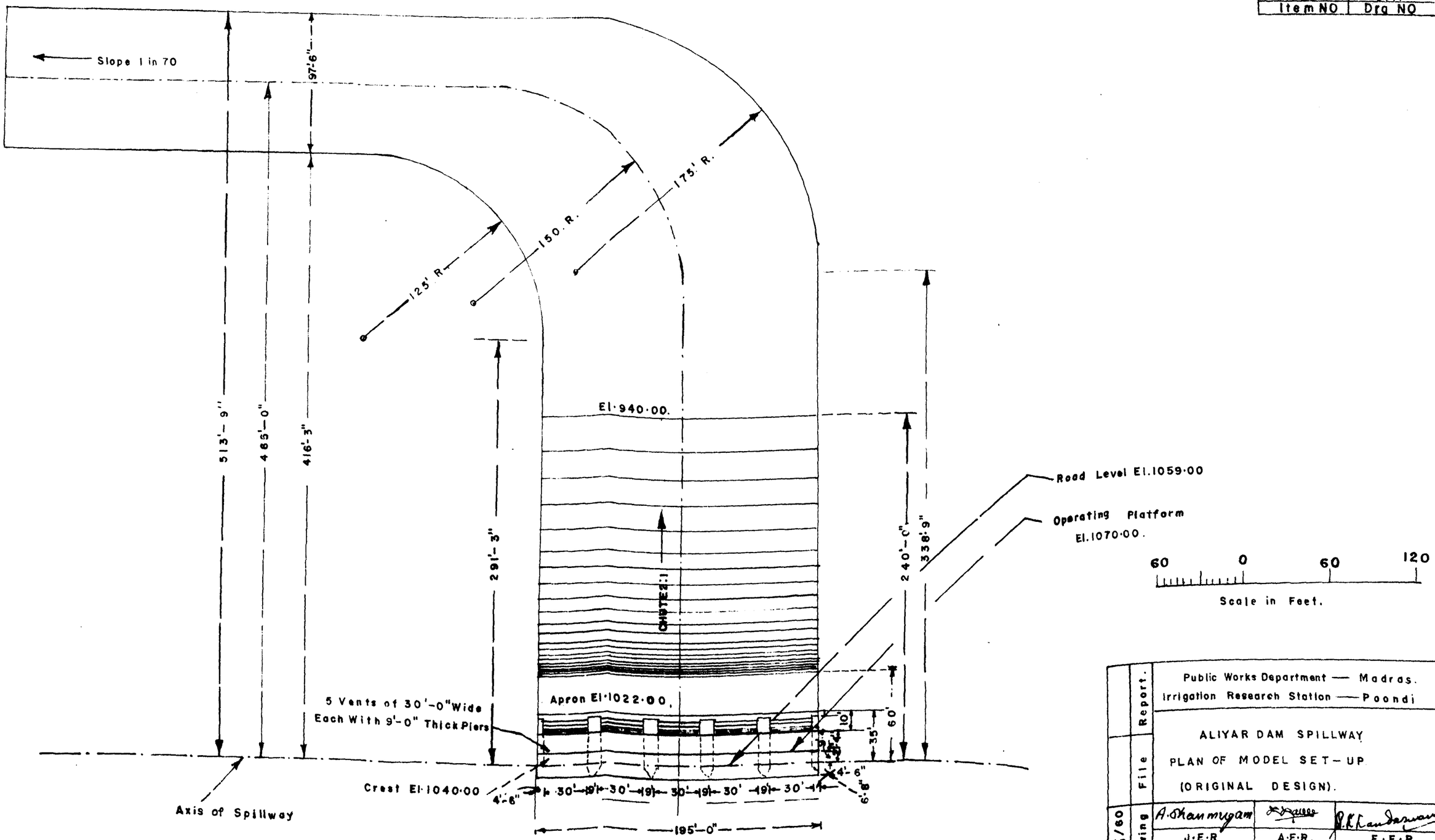
Annual Report of 1959	
11/30	5/11
Item NO	Drg NO



Report.	Public Works Department — Madras. Irrigation Research Station — Poondi	
File	ALIYAR DAM SPILLWAY PLAN OF MODEL SET-UP (ORIGINAL DESIGN).	
Drawing	A. Shanmugam / J.E.R	A.E.R. / E.E.R.
	Trd: C.S.K. Pathy	Ckd: / Fig. No. 5.

2.15/60

Reg. No. 57-E'62 (Saka Era 1883)875.

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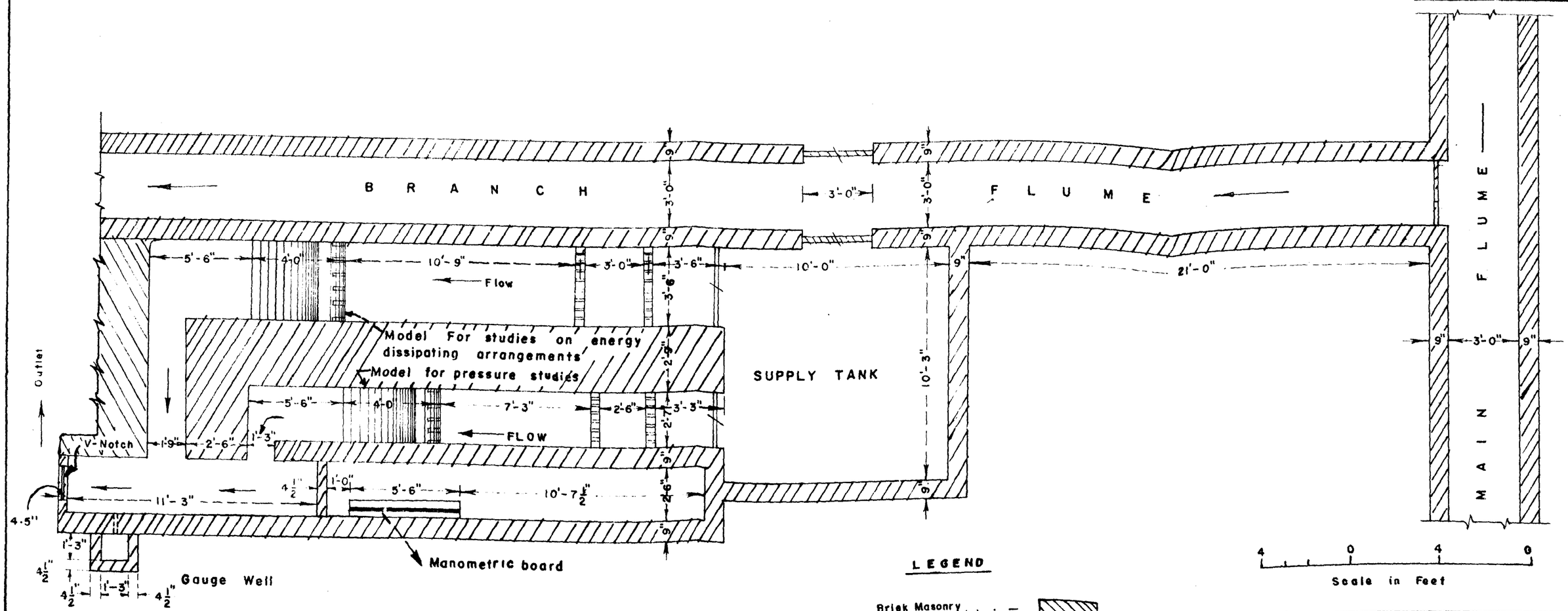
Annual Report of 1959	
11/30	6/11
Item NO	Drg NO



LEGEND

Brick Masonry . . . . —

Hony — Comb Work . —

Steel Shutter With Lifting Arrangements —

Public Works Department — Madras
Irrigation Research Station — Poondi.

ALIYAR DAM SPILLWAY
MODEL LAY-OUT

Report			
File			
2/6/60 Drawing	A. Shanmugam		
	J·E·R	A·R·O	E·E·R
	Tr: V·V·	ckd:	FIG···6

Reg. No. 58-E 62 (Saka Era 1883) 375.

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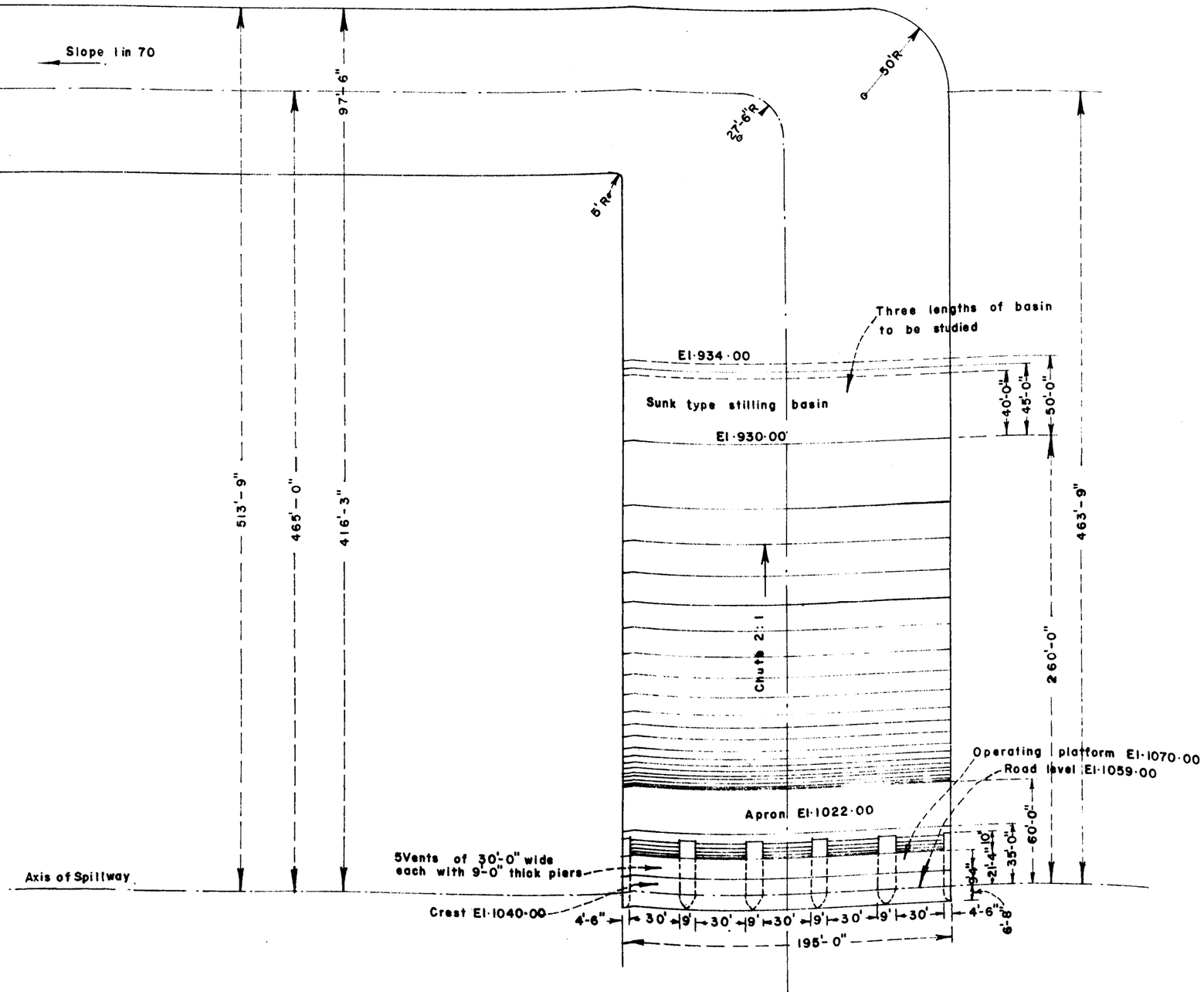
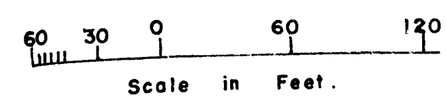
Annual Report of 1959	
11/30	7/11
Item NO	Drawing NO





PUBLIC WORKS DEPARTMENT-MADRAS IRRIGATION RESEARCH STATION-POONDI		
ALIYAR DAM SPILLWAY PLAN OF MODEL SET-UP (Revised design)		
A. Shanmugam		
J·E·R	A·E·R	E·E·R
Tr: V·V·	ckd:	FIG···7

Report / File / Drawing 277/60

Reg. No. 59-E'62. (Saka Era 1882) 375.

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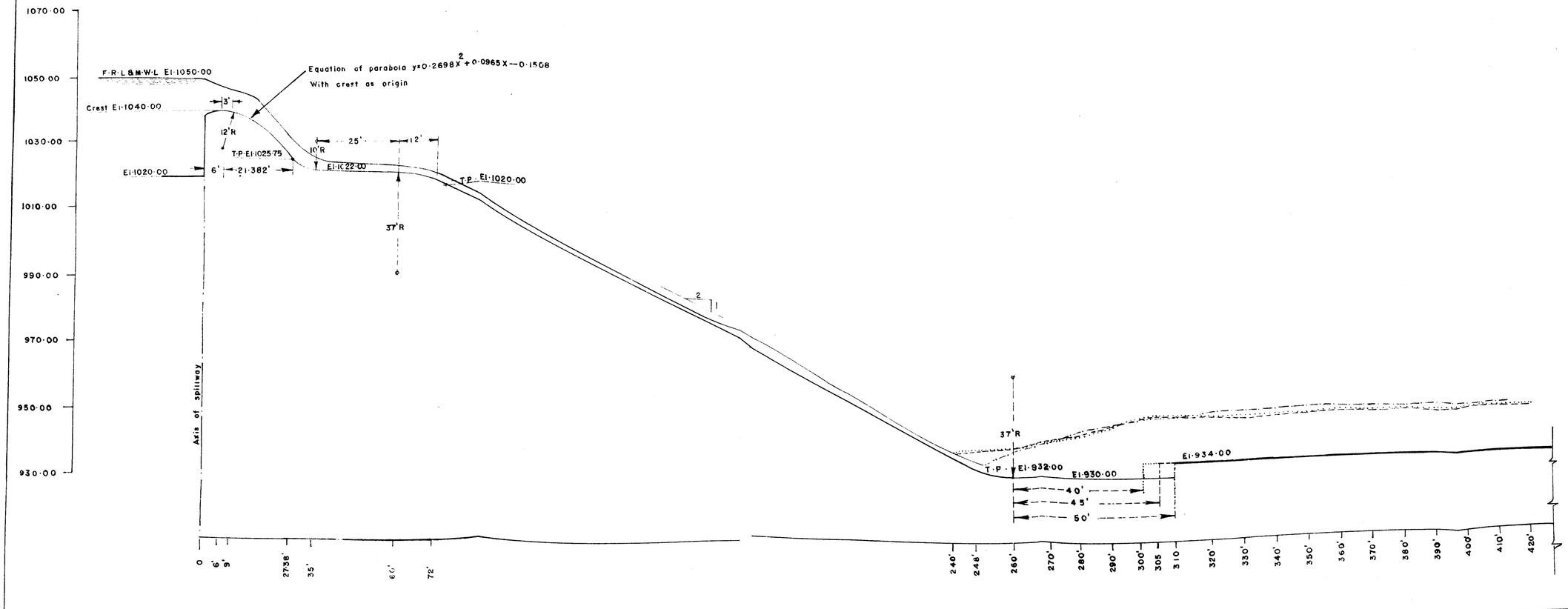
Annual Report of 1959	
11/30	8/11
Item NO	Drawing NO



LEGEND

WATER SURFACE PROFILES FOR

.....40'-0" Long basin

------45'-0" Long basin

-.-.-.-50'-0" Long basin

20 0 20 40

Scale in Feet

Report	Public Works Department — Madras. Irrigation Research Station — Paondi		
File	ALIYAR DAM SPILLWAY ENERGY DISSIPATING ARRANGEMENTS Water surface profiles		
Drawing	A Shanmugam		
	J·E·R	A·E·R	E·E·R
	Tr: V·V·	ckd:	FIG···8

2/8/60

Reg. No. 85-E'62 (Saka Era 1884) 375.

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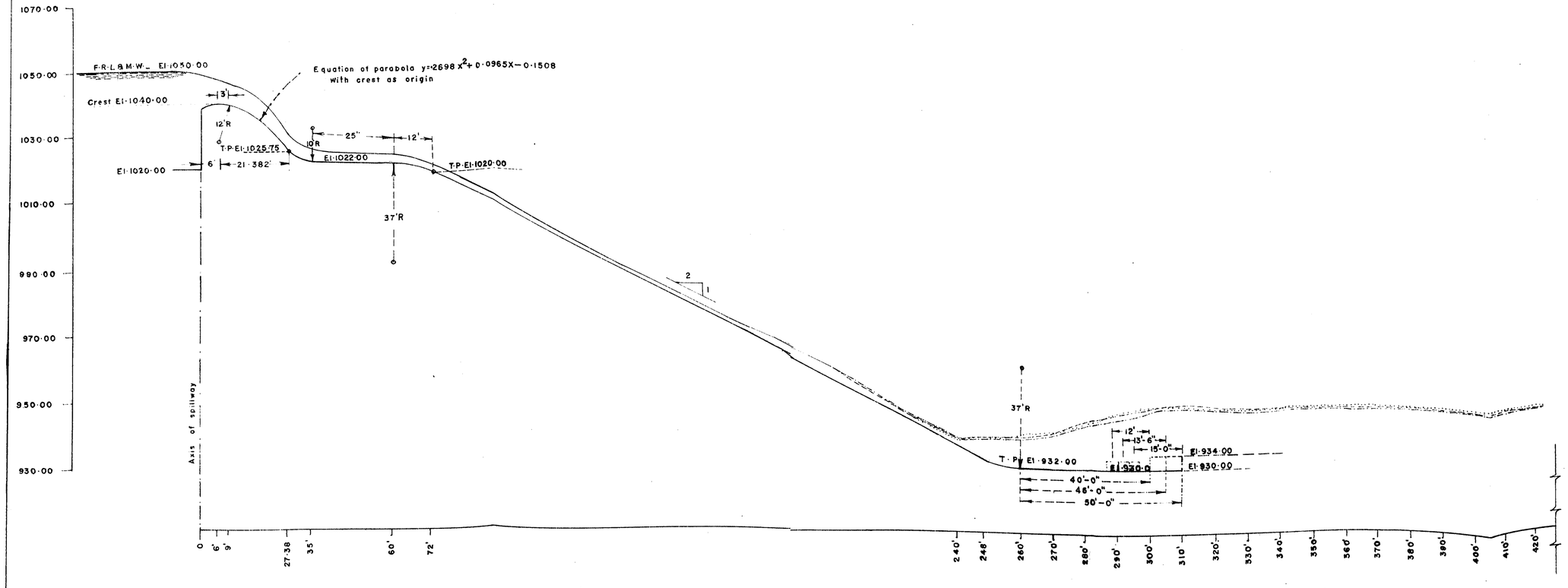
Annual Report of 1959	
11/30	9/11
Item NO	Drawing NO



LEGEND

WATER SURFACE PROFILES FOR

40'-0" Long basin with 4'x 3'x 3' blocks at 10'-0" crs· each placed at 12'-0" from the end of Basin.

45'-0" Long basin with similar blocks placed at 13'-6" from the end of Basin.

50'-0" Long basin with similar blocks placed at 15'-0" from the end of basin.

20 0 20 40

Scale in Feet

Report	Public Works Department – Madras· Irrigation Research Station – Poondi·		
File	ALIYAR DAM SPILLWAY		
	ENERGY DISSIPATING ARRANGEMENTS Water surface profiles		
2/9/60 Drawing	A. Shanmugam		
	J·E·R	A·E·R	E·E·R
	Tr: V·V·	crd:	FIG ···9·

Reg. No 86-E'62 (Saka Era 1884) 375.

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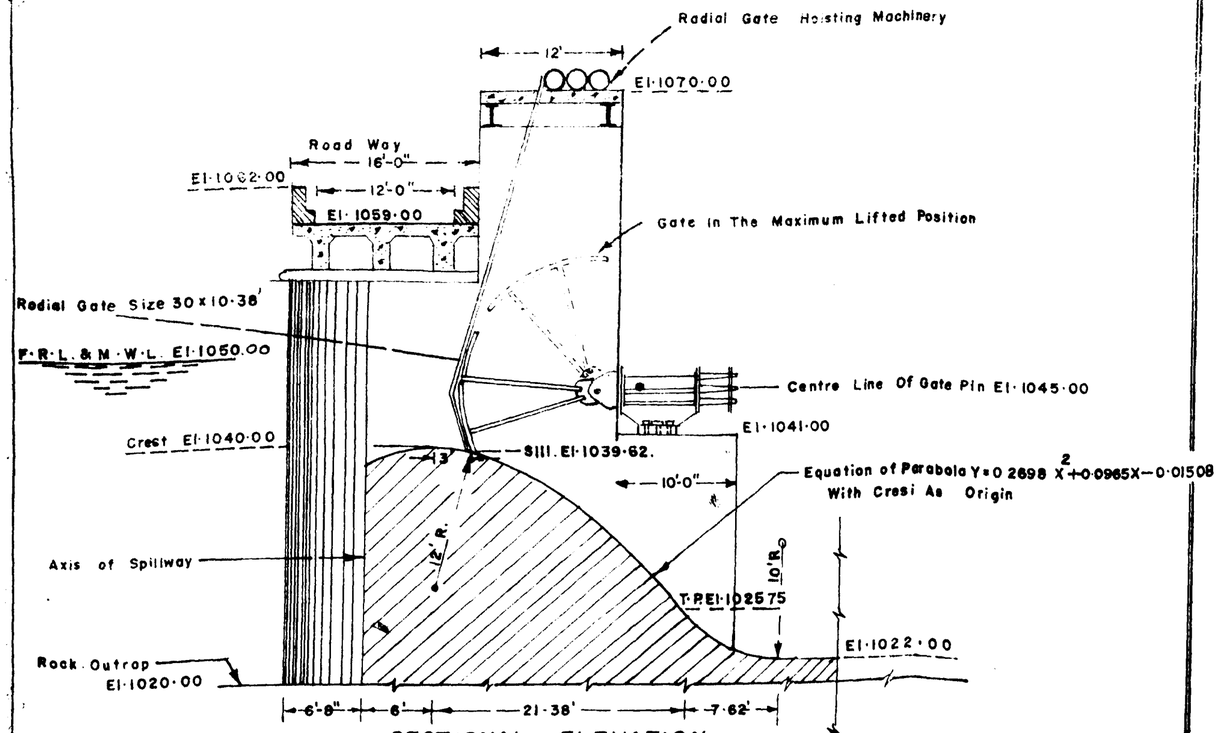
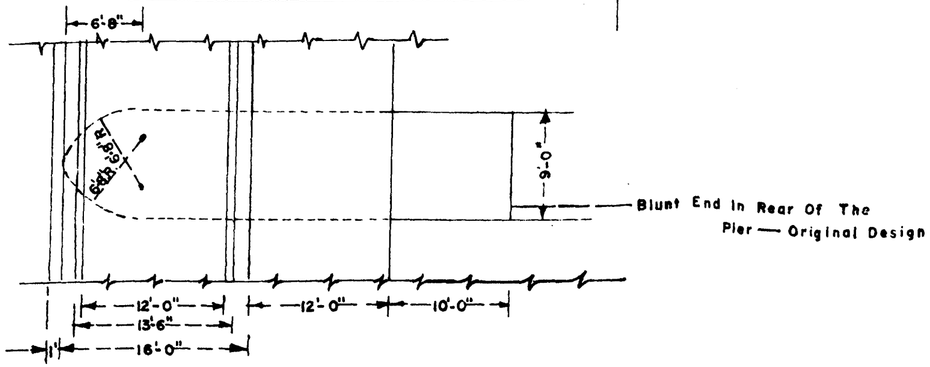
Annual Report of 1959	
11/30	10/11
Item NO	Drg NO

Different Types Of Ease Waters Studied In Rear Of Spillway Piers.

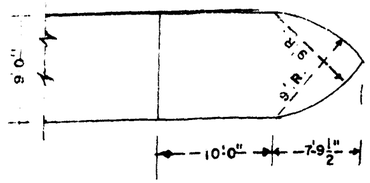


SECTIONAL ELEVATION.

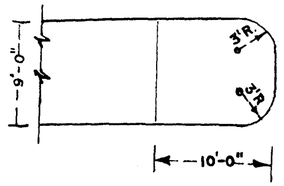


PLAN

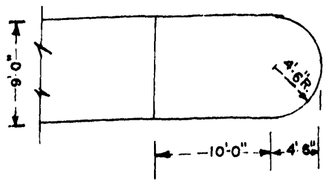
1. EQUIANGULAR EASE WATER



3 CORNERS ROUNDED OFF WITH CURVES OF 3'-0" RADIUS.



2. SEMICIRCULAR EASE WATER.



Scale in Feet.

Report	Public Works Department — Madras. Irrigation Research Station — Poondi.		
File	ALIYAR DAM SPILLWAY. EASE WATERS IN REAR OF THE PIERS. (Original Design And Different Types Studied)		
220/60 Drg	A Shanmugam		
	J·E·R	A·E·R	E·E·R
	Trd: C·S·K. Pathy	Ckd:	FIG. . . . 10.

Reg. No. 82-E '62 (Saka Era 1884)-375

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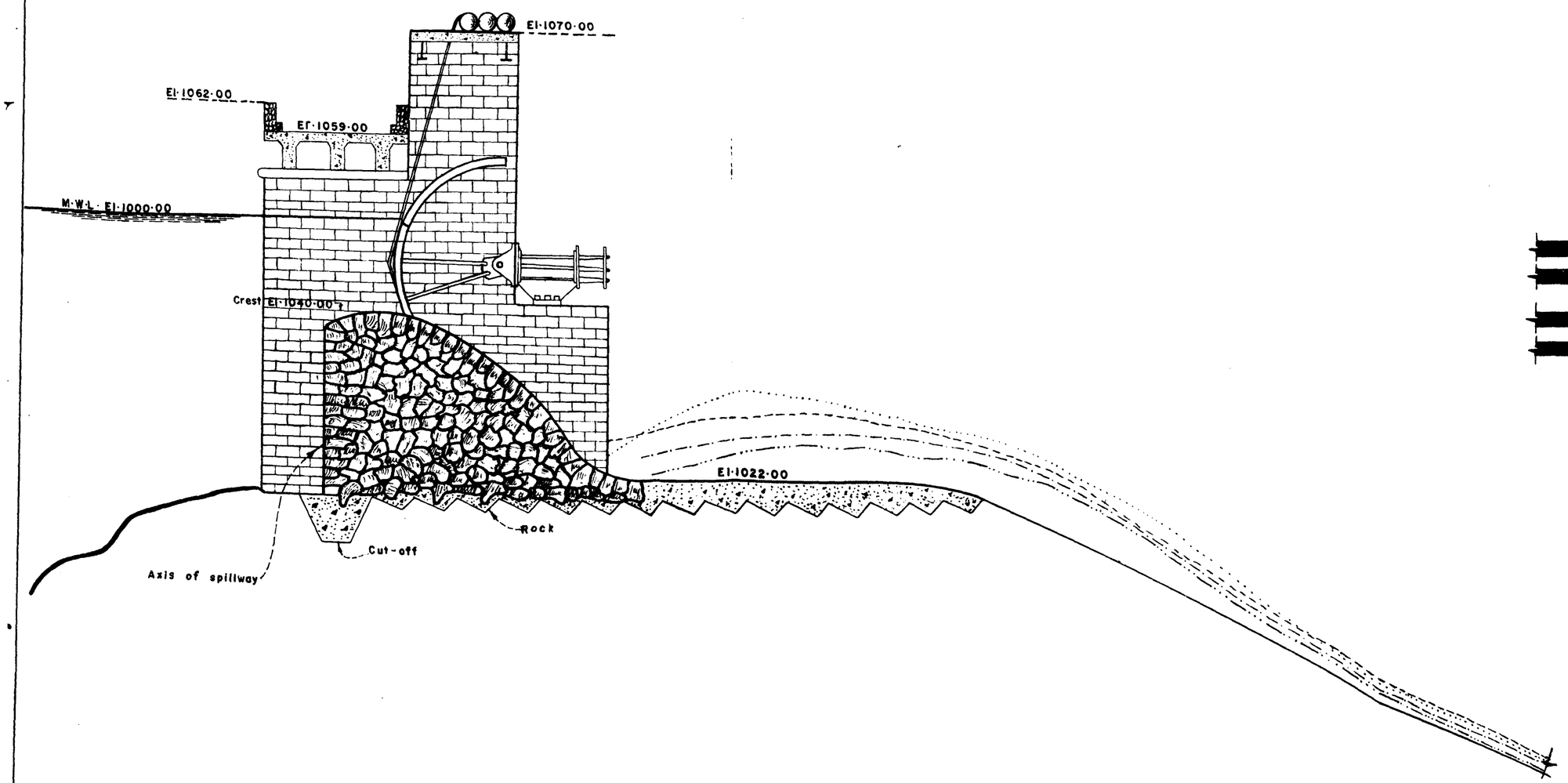
Annual Report of 1959	
11/30	11/11
Item NO	Drawing NO



LEGEND

WATER SURFACE PROFILES IN REAR OF SPILLWAY PIER

1· Blunt end

2· Corners rounded of with circular curves of 3'-0" radius

3· With semicircular ease water

4· With equiangular ease water

10 0 10 20

Scale in Feet

Report	Public Works Department-Madras Irrigation Research Station-Poondi·		
	ALIYAR DAM SPILLWAY		
File	DESIGN OF EASEWATERS IN REAR OF SPILLWAY PIERS		
	Water Surface Profiles		
83.1/60 Drawing	A·Shanmugam		
	I·E·R	A·E·R	E·E·R
	Tr: V·V·	ckd:	FIG···11.

Reg. No 83. E'62 (Saka Era 1684) 375

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PART II

Aliyar Dam Spillway - Pressure Studies

5. MODEL

5.1. A geometrically similar sectional model representing only 4 vents out of the 11 vents in the prototype, was constructed to a scale of 1/60 (Photo-4). The original section received from the Designs Office was adopted in the first instance, in the construction of the model (Figure-3). The equation of the paralolic curve of the ogee section is $Y = 0.02693\,x^2 - 0.0966\,x - 0.1508$ with the crest as origin. The radii of the circular curves connecting the ends of the chute section to the horizontal floors at El. 1022.00 and El. 940.00 were not given in the original designs and a radius of 37' had been adopted for both the curves by this station. It was found out by the excavation of trial pits at site that the solid rock was available only at El. 930.00 at the toe of the chute section. Accordingly alterations were done in the model in order to lower the toe of the chute section to El. 930.00 (Figure-12). As decided by the studies on energy dissipating arrangements 4'0" deep and 45'0" long sunk type stilling basin was adopted at the toe of the chute section.

5.2. To observe the pressure variation from one end to the other end of the ventway due to the effect of contraction caused by the piers, piezometric points were located on three sections, viz. : sections AA and BB at 5'0" from the ends and section CC along the centre line of the ventway (Figure-12). The piezometric points on these three sections were also staggered in order to get a comprehensive idea of the pressures developed at all representative points on the spillway section. At the piezometric points 1/8" diameter copper tubes were embedded in the masonry of which the spillway section was constructed. These copper tubes were connected by means of 1/8" dia. rubber tubes to a battery of glass tubes arranged in the respective places and elevations of the piezometric points on the spillway section and collected on a manometer board containing the profile of the spillway section drawn to the actual dimensions of the model (Photo 5). Levels at intervals of 1.0' were written on the manometer board to facilitate observation of the pressures developed along the spillway section at the piezometric points directly in foot of water of prototype dimension. The elevation of water levels obtained in the glass tubes was observed. To measure the head of flow over the spillway, levels were pointed on the side walls of the flume, sufficiently far from the spillway crest to avoid the effects of the flow through the vents of the spillway.

5.3. The surface of the spillway was finished smooth. A slope of 1 in 70 was adopted for the surplus course. The width of the surplus course below the bend and the curvature of the bend near the toe of the spillway were all simulated in the same ratio of the length of the spillway in the model to that in the prototype. A nominal radius of 50 feet and a minimum radius of 5 feet were adopted for the outer and inner curve of the bend respectively. The bed and the vertical sides of the surplus course were moulded rigid. The piers road bridge on top of spillway and operating platform were made of

timber and placed over the masonry spillway in the model in order to simulate the prototype conditions. Radial gates were provided for the ventway. Figure-6 represents a comprehensive picture of the model lay-out inside the laboratory.

6. EXPERIMENTS AND OBSERVATIONS.

6.1. Experiments were conducted for observing the pressures developed along the spillway section for both free flow conditions and gate controlled flow conditions. In the former case, observations of pressures developed along the spillway surface were recorded for various conditions in the water level starting from the minimum head of 1 foot over the crest level of El. 1040.00 and keeping the shutters open clear all the time. The observations of pressures were continued for heads upto 13 feet over the crest i.e., 3 feet or 30% over the design head of 10 feet with a view to study the pressure characteristics for any contingency of excess flow over the design head. Similarly observations of pressure were recorded for various gate opening starting from a minimum opening of 1 foot and keeping the water level in front of the spillway at F.R.L. and M.W.L. El. 1050.00. When the gate opening exceeded 6 feet the flow was no longer/controlled one, as free flow was observed in the model. Hence, observations of pressures for higher gate openings were not taken. The results of observations are tabulated in statement Numbers 2 and 3.

7. DISCUSSION OF RESULTS

7.1. Entering statement No. 2 negative pressures are observed for low heads of 1 foot of flow over the crest, the maximum value of the negative pressure being 1 foot of water in the prototype, obtained near the toe of the chute section. The pressure generally increases as the head of flow over the crest is increased, the maximum value of the positive pressure being 14.5 feet of water in the prototype, obtained at the concave circular curve joining the toe of the chute section to the horizontal floor of the stilling basin at El. 930.00. For heads of free flow of 13 feet negative pressures are observed at the convex. Circular curve joining the horizontal apron at El. 1022.00 to the top of the chute section, the maximum value of the negative pressure being only 0.75 feet of water in the prototype.

7.2. Entering Statement No. 3 it is observed that the pressure developed along the surface of the spillway section decrease near the crest and increase at other points, as the gate opening is increased. A maximum positive pressure of 12.75 feet of water in the prototype is realised at the concave circular curve joining the parabolic ogee section to the horizontal apron at El. 1022.00. No sub-atmospheric pressure is found to have occurred anywhere along the spillway section for gate controlled flow.

8. CONCLUSION.

8.1. The profile of the spillway section in Figure-4 is found to be free from the perils of cavitation for the conditions of smooth surface of the spillway as the maximum subatmospheric pressure developed is only 1 foot of water in the prototype which is well within the allowable limits. It is ascertained from the engineers in charge of the execution of the project, that the rock outcrop of the chute section will be left with the surface as obtained by removing the overburden soil and loose boulders alone in which case the surface of the chute section will be only uneven and nigged and not smooth. It is considered to be of paramount importance and essential to simulate in the model the surface of the spillway obtained in the prototype and find out whether any undesirable sub-atmospheric pressure occur on the surface of the chute section for various conditions of flow over the spillway. Suitable recommendations will have to be offered in order to protect the uneven rocky surface of the chute section from the danger of cavitation if found essential.

STATEMENT I

Percentage loss of Energy in the various proposals of energy Dissipating arrangements.

Serial No. (1)	Length of basin in feet. (2)	Depth of basin in feet. (3)	Basin level. (4)	Bed level of surplus course below basin. (5)	Details of Friction blocks. (6)		Distance from the axis of spill way in feet. (7)	Depth of flow in feet. (8)	Velocity in feet per second. (9)	Velocity head $V^2/2g$ in feet. (10)	Specific energy. $d + \frac{v^2}{2g}$ in feet. (11)	Loss of energy in feet. (12)	Percentage loss of energy. (13)	Maximum hump elevation. (14)	Sketch showing the Energy dissipating arrangements. (15)	Observations and remarks. (16)
...	...	...	...	...	...		230	1.40	69.66	75.36	76.76*	...	...	...	...	*The specific energy is calculated just before the formation of the jump neglecting the the loss due to friction.
1(a)	50	4	El. 930.00	El. 934.00	No Blocks	...	350	14·40	1.936	0.06	14.46	62.30	81.17	El. 949.14	...	A more steady and somewhat uniform hydraulic jump is formed. The hump of the jump perfectly merges with the tail water level.
1(b)	50	4	El. 930.00	El. 934.00	4′ x 3′ x 3′ size blocks each at 10 feet centres placed with their centres at 15′ from the end of basin	...	350	13.68	2.866	0.13	13.81	62.95	82.02	El. 948.58	...	
2(a)	45	4	El. 930.00	El. 934.00	No Blocks	...	350	14.22	1.936	0.06	14.28	62.48	48.81	El. 948.30	...	A steady and more uniform hydraulic jump is formed. The jump satisfactorily merges with the tail water level.
2(b)	45	4	El. 930.00	El. 934.00	4′ x 3′ x 3′ size blocks each at 10′ centres placed with their centres at 13′ 6′ from the end of basin	...	350	13.62	2.866	0.13	13.75	63.01	82.10	El. 948.84	...	
3(a)	40	4	El. 930.00	El. 934.00	No Blocks	...	350	14.52	2.014	0.06	14.58	62.18	81.02	El. 948.52	...	A somewhat steady and uniform hydraulic jump is formed. There is a fall between the hump and the tail water level.
3(b)	40	4	El. 930.00	El. 934.00	4′ x 3′ x 3′ size blocks each at 10′ centres placed with their centres at 12′ 0″ from the end of basin	...	350	13.32	3.253	0.16	13.48	63.28	82.43	El. 948.83	...	

STATEMENT II

ALIYAR DAM SPILLWAY—PRESSURE STUDIES—FREE FLOW CONDITIONS

Piezo metric points Nos.	Location of piezo-metric points.	Distance of piezometric points from the axis of spill way in feet.	Elevation of piezometric points.	Pressure in feet of water in prototype — Head of flow over the crest						
				1 foot	3 feet	5 feet	7 feet	9 feet	10 feet	13 feet
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	A	0	1038.50	2.5	4.25	5.5	5.75	6.0	6.0	5.5
2	B	5	1040.00	0.75	2.25	3.25	4.00	4.75	4.75	5.25
3	C	10	1039.50	0.25	1.25	1.75	2.0	2.25	2.25	6.75
4	A	15	1037.00	0.5	0.75	1.25	1.50	1.50	1.50	1.25
5	B	20	1033.00	0.25	1.0	1.0	1.0	0.75	0.75	0.50
6	C	25	1028.25	0.5	1.75	2.0	2.0	2.75	3.0	5.25
7	A	30	1023.50	Nil.	1 0	4.25	6.5	11.25	13.0	17.25
8	B	35	1022.00	0.5	0.25	0.75	2.0	5.75	7.0	12.50
9	C	40	1022.00	0.5	0.25	0.5	0.75	1.75	2,5	5.25
10	A	45	1022.00	0.5	1.0	1.75	2.25	3.0	3.5	4.75
11	B	50	1022.00	0.5	0.5	0.75	1.0	1.25	1.75	3.0
12	C	55	1022.00	0.5	0.5	0.75	1.0	1.75	1.95	2.25
13	A	60	1082.00	0.5	0.75	1.5	1.75	2.25	2.25	2.5
14	B	65	1021.50	0.5	0.5	0.25	0.25	0.25	Nil.	0.25
15	C	70	1020.50	0.5	0.25	0.75	0.5	0.25	Nil.	0.75
16	A	75	1018.60	Nil.	0.5	0.5	0.5	1.0	0.75	0.75
17	B	80	1016.00	0.25	0.25	0.5	0.75	1.0	1.25	1.50
18	C	85	1013.50	Nil.	0.2	0.25	0.5	1.0	1.00	1.25
19	A	90	1011.00	0.5	Nil.	0.25	1.0	1.5	1.5	2.0
20	B	95	1008.50	0.5	Nil.	Nil.	0.5	0.5	1.0	1.25
21	C	100	1006.00	Nil.	0.25	0.5	0.75	1.0	1.25	2.00
22	A	105	1003.50	0.25	Nil.	0.25	0.5	1.5	1.5	2.25
23	B	110	1001.00	0.5	Nil.	0.25	0.5	1.25	1.5	3.25
24	C	115	998.50	0.25	Nil.	Nil.	0,25	0.25	0.25	0.75
25	A	120	996.00	0.5	0.75	1.75	2.25	3.75	3.5	4.75
26	B	125	993.50	0.5	Nil.	Nil.	0.25	1.0	0.75	1.5
27	C	130	991.00	Nil.	1.0	1.75	2.0	2.75	2.75	3.25
28	A	135	983 50	0.75	Nil.	0 25	0.75	1.25	1.75	2.0
29	B	140	986.00	0.75	Nil.	Nil.	Nill	0.75	0.25	1.0
30	C	145	983.50	0.25	0.75	1.5	1.75	2.5	2.5	2.75
31	A	150	981.00	0.75	0.25	1.25	2.0	4.0	0.75	4.5
32	B	155	978.50	0.5	Nil.	0.25	0.25	0.75	0.75	1.5
33	C	160	976.00	Nil.	0 3	1.0	1.25	1.75	1.73	2.75
34	A	165	973.50	0.5	0.5	1.5	2.25	3.5	3.5	4.0
35	B	170	971.00	0.25	0.25	0.5	0.75	1.25	1.2	2:5
36	C	175	968 50	Nil.	0.25	0.25	0.25	0.25	0.25	0.75
37	A	180	966.00	0.5	0.5	2.5	3.25	5.25	5.25	5.75
38	B	185	963.50	Nil.	Nil.	0.25	0.25	0.5	0.5	0.5
39	C	190	961.00	0.5	0.5	1.0	1.0	2.5	2.5	3.5
40	A	19	958.50	0.5	Nil.	0.5	0.75	1.75	1.75	2.5
41	B	200	956.00	0.25	0.25	1.25	1.5	2.5	2.5	2.25
42	C	205	953.50	Nil.	0 25	0.75	0.75	1.0	0.75	1.5
43	A	210	951.00	1.0	Nil.	0.25	0.75	1.25	0.75	1.0
44	B	215	948.50	Nil.	0 25	1.0	1.25	1.25	1.5	1.5
45	C	220	956 00	0.5	0.75	1 0	1.25	1.5	0.5	1.0
46	A	225	943.50	0.5	Nil.	0 75	1.25	2.5	2.5	3.0
47	B	230	941 00	Nil.	1.75	2 25	1.75	2.5	2.75	3.0
48	C	235	938.50	0.25	0 5	1 0	1.5	3.25	2.75	3.75
49	A	240	936.00	0 5	1 0	2.0	3.0	3.75	4.0	5.75
50	B	245	933.50	1.75	3.25	2.75	3.5	4.5	5.35	6.0
51	C	250	931.50	3.75	4.5	3.25	2:75	4.75	7.50	14.50
52	A	255	930.50	4.75	6.25	4.5	4.25	4.0	6.25	10.75
53	B	260	930.00	5.25	7.0	7.0	7.75	8.25	9.75	12.25
54	C	265	930.00	5.25	7.0	6.75	6.75	3.25	4.75	7.25
55	A	270	930.00	5.25	7.0	8.75	6.75	2.25	3.25	4.75
56	B	275	930.00	5.25	7.5	8.5	7.75	4.25	6.75	8.75
57	C	280	930.00	5.25	7.75	9.25	9.0	4.25	5.25	6.75

STATEMENT II

ALIYAR DAM SPILLWAY—PRESSURE STUDIES

Rate controlled flow conditions with F.R.L. & M.W.L. at El. 1060.00.

Pieno-metric point Nos.	Location of piezometric points.	Distance of piezo-metric points from the exis of spillway in feet.	Elevation of piezometric points.	1 foot.	2 feet.	5 feet.	7 feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	A	0	1008.49	11.29	10.75	9.25	6.0
2	B	5	1010.00	2.72	2.80	6.70	3.0
3	C	10	1029.5	0.5	1.0	1.73	2.23
4	A	15	1531.0	0.75	0.5	0.75	1.75
5	B	20	1023.0	1.0	0.75	0.50	0.75
6	C	25	1028.25	2.0	2.0	2.05	3 25
7	A	30	1052.5	5.0	3.75	2.5	12.75
8	B	33	1038 25	0.5	0.75	8.85	6.75
9	C	40	1032 0	0.5	0.75	1.0	2.5
10	A	45	1029.0	1.0	1.25	3.0	2.5
11	B	80	1029.0	0.5	1.0	3.25	2.0
12	C	85	1028.0	0.75	1.0	7.5	1.75
13	A	80	1022.0	1.0	0.0	2.85	2.86
14	B	86	1081.5	0.75	0.5	Nil.	0.25
15	C	70	1080.5	0.75	0.5	Nil.	0.96
16	A	75	1018.5	0.25	0 75	0.75	0.75
17	B	80	1010.0	0·5	0.75	1.85	1.25
18	C	85	1012.5	0.5	0.5	1.0	1.0
19	A	90	1011.6	Nil.	1.5	1.5	1.5
20	B	95	1050.0	Nil.	Nil.	0.95	1 0
21	C	100	1006.0	0.75	1.25	1.25	1 25
22	A	106	1003.5	Nil.	1.0	1.85	1.78
23	B	110	1001.0	0.25	0 75	1 05	1.8
24	C	116	998.8	Nil.	0.25	0.25	0.28
25	A	120	996.0	0.75	3.5	3.5	4.25
26	B	125	988.5	Nil.	0.5	1.0	1.28
27	C	130	991.0	0.25	2.75	3.0	7.75
28	A	135	988.5	Nil.	1.0	1.25	1.75
29	B	140	986.0	Nil.	0.25	0.5	0.70
30	C	145	963.5	1.75	2.5	2.75	2.5
31	A	150	961.0	0 5	3.35	3.5	4.5
32	B	155	978.5	Nil.	0.5	0.75	1.0
33	C	160	976.0	1.25	1.5	1.75	1.75
34	A	165	970.50	0.5	2.75	3·25	4.0
35	B	170	971.0	0 5	2.60	1.25	6.5
36	C	175	968.0	0 25	Nil.	0.25	1.0
37	A	180	968.0	1.0	4.0	4.75	5.75
38	B	185	960.6	Nil.	0.25	0.25	0.8
39	C	190	961.0	1.0	1.75	2.0	2.75
40	A	195	968.5	0.25	1.25	1.75	2.25
41	B	200	968.0	1.25	2.0	2.5	2.75

Pieno-metric point Nos.	Location of piezometric points.	Distance of piezo-metric points from axis of spillway in feet.	Elevation of piezometric points.	1 foot.	2 feet.	5 feet.	7 feet.
42	C	205	963.5	0.6	0 75	1.0	1.28
43	A	210	901.0	Nil.	0.75	1.0	1.6
44	B	215	948.5	0.75	1.25	1.25	1.25
45	C	220	966.0	1.0	1.75	2.0	2.78
46	A	225	943.5	0.5	1.75	2.25	2.78
47	B	230	941.0	1.0	2.0	2.27	2.78
48	C	235	908.5	1.25	1.75	2.0	2.0
49	A	240	906.0	1.25	1.75	2.0	2.0
50	B	245	938.6	2.5	2.25	4.75	5.25
51	C	250	931.5	3.75	3.25	3.75	7.25
52	A	255	920.5	6.25	3.25	3.25	5.25
53	B	260	930.0	6.75	7.25	3.0	9.25
54	C	265	930.0	7.0	3.25	3.0	3.75
55	A	270	930.0	8.75	2.25	1.5	1.75
56	B	275	930.0	7.25	5.0	4.0	5.0
57	C	280	930.0	9.25	8.0	3.75	3.75

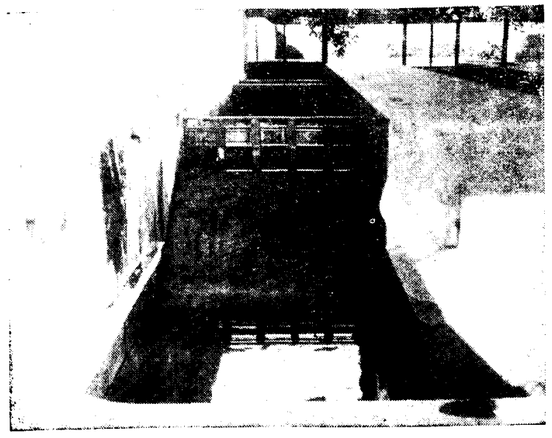

PHOTO 1. Aliyar Dam Spillway—Energy Dissipating arrangements Model set-up looking from downstream.

PHOTO 2. Aliyar Dam Spillway – Original design — Formation of hydraulic jump below spillway.

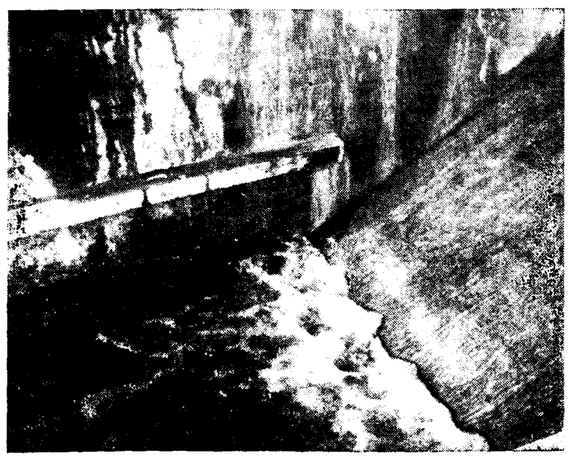

PHOTO 3. Aliyar Dam Spillway — Energy dissipating arrangements. Revised design — Formation of hydraulic jump for 45'0. long and 4'0" deep basin.

PHOTO 4. Aliyar Dam Spillway — Preesure studies—Model set-up looking from downstream.

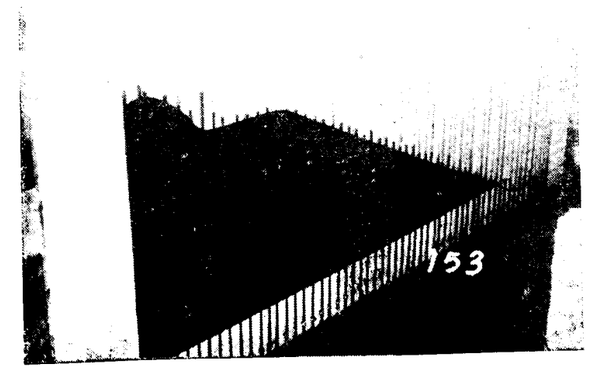

PHOTO 5. Aliyar Dam Spillway — Pressure studies — Manometric Board set-up.

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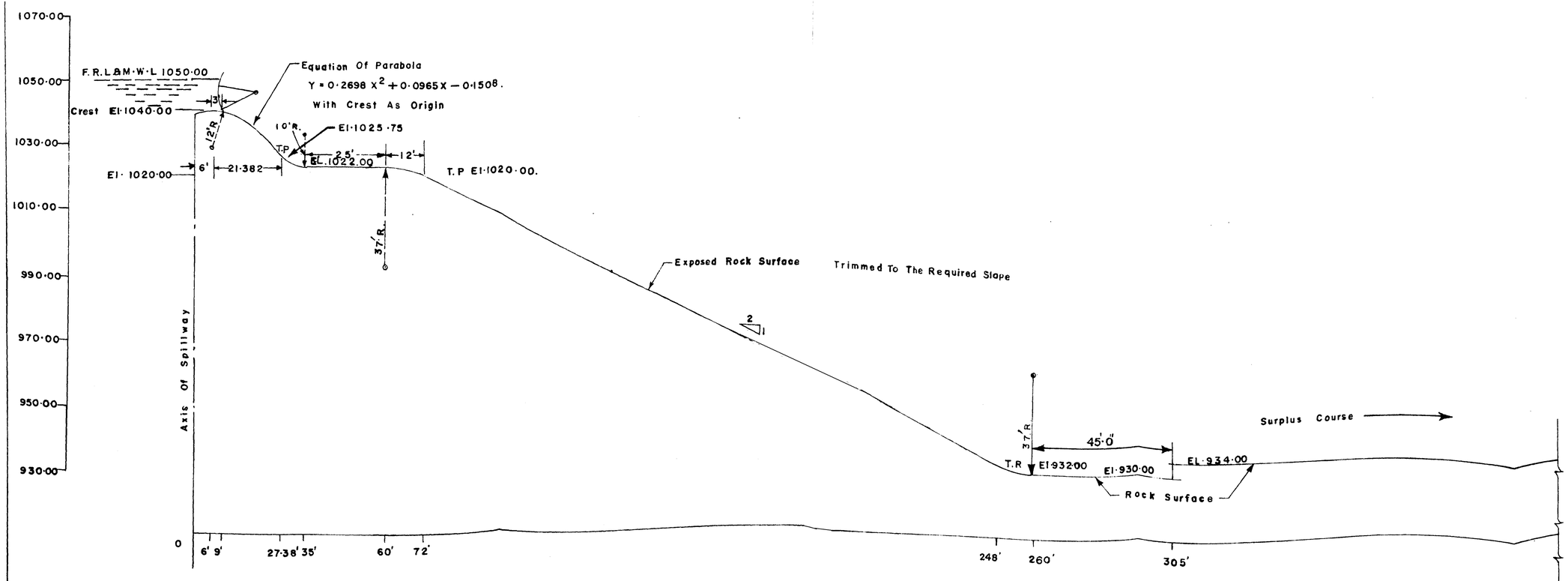

Public Works Department—Madras. Irrigation Research Station—Poondi.		
ALIYAR DAM SPILLWAY SECTION. (Revised Design)		
A. Shanmugam		
J·E·R.	A·E·R	E·E·R.
Trd: R·R·N.	Ckd:	Fig...12.

Reg No 89-E'62 (Saka Er. 1884)-375.

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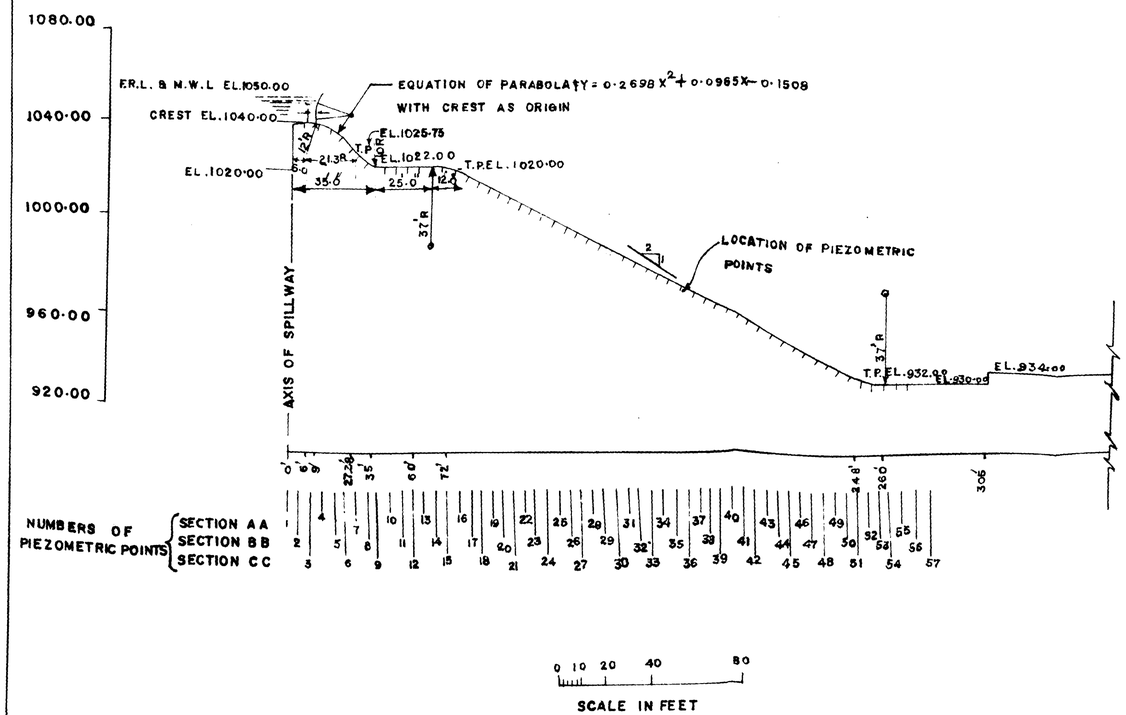

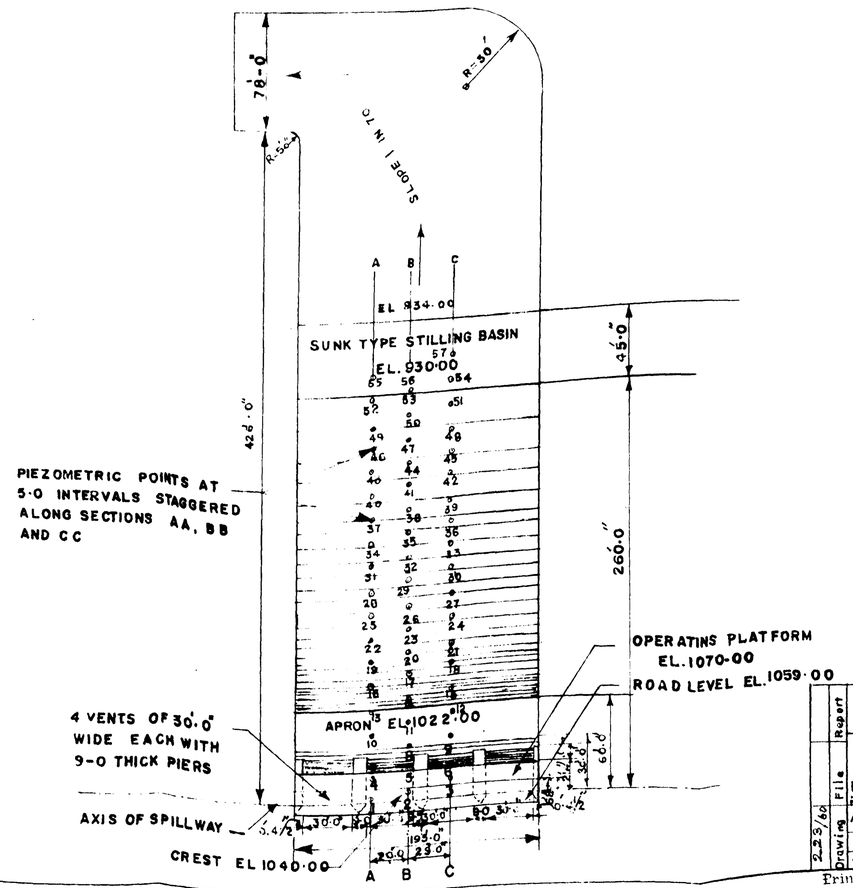

Report	Public Works Department — MADRAS Irrigation Research Station. POONDI.	
File	ALIYAR DAM SPILLWAY PRESSURE STUDIES PLAN SHOWING THE LOCATION OF PIEZOMETRIC POINTS IN MODEL SET UP	
Drawing 223 S/60	A. Shanmugam	
	J. E. R	A. E. R	E. E. R
	Trd: R.R.N	Ckd:	FIG. 13

Reg No 90-E'62 (Saka Era 1884) 375.

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Morning Glory Spillway

1. INTRODUCTION

The morning glory spillway cum intake lower for the water supply scheme of Coimbatore town was designed for the Siruvani Reservoir in Coimbatore district. The plan and section by the spill as designed are shown in Figure 1. The spillway discharges at its exit and into a tunnel, about 5,000 feet along, which carries the discharge across the hills to be let down into a natural stream feeding the waterworks of the Coimbatore town. Studies were initiated to determine the hydraulic performance of the spillway.

2. DATA

Crest of spillway	...	El. 2881.0
M E.L.	...	El. 0892.0
Sill of tunnel	...	El. 2808.0
Size of tunnel	...	8′ × 6′
Length of tunnel	...	5000 feet.
Size of spillway shaft	...	3′ 3″
Diameter of the spillway at crest level	...	15′ 0″
Total drop from M.W.L. to sill at tunnel	...	84 feet.

3. MODEL SET UP

A teakwood model of the spillway was made to a scale of 1/10 and this was installed in one of the trays of the high head tank. Sufficient number of piezometric tappings were provided along the spillway face, from the crest down to the exit end of the spillway so as te facilitate a study of the pressure conditions inside the spillway. 1/8″ diameter copper tubes were inserted into the piezometer holes and these werc connected by means of a rubber tubes to a battery of glass tube fixed to a manometer board. The manometer board was graduated and fixed in such a way that points representing a certain elevation in the teakwood model and the board were at the same level.

A V-notch was fixed on the downstream side below the exit end of the spillway in order to measure the discharge passing over thc spillway. Sufficient stilling arrangemcnts were provided upstream of the V-Notch. The heads of water over the crest of spillway and over the sill of V-Notch were measured by means of pointer gauges reading correct rate 0.001 feet.

4. EXPERIMENTS

In the first place, experiments were conducted to determine the discharge characteristics of the spillway. For this purpose, water was allowed to pass over the spillway and sufficient time was allowed so that a constant water level was obtained over the crest of the spillway. The heads of the water over the crest of the spillway. The heads of water over the crest of the spillway and over the sill of the V-Notch were measured simultaneously. The experiments were repeated for heads varying from 0.1 feet over the crest of the spillway up to M.W.L. The discharge over the spillway was then calculated for the corresponding head over the crest. The results are presented in statement No. 1.

A study of the pressure conditions inside the spillway for different heads over the crest of spillway has been also taken up and is in progress.

STATEMENT No. 1

VARIATION OF DISCHARGE WITH HEAD ON CREST OF SPILLWAY.

S. No.	Head over Spillway in Model.	Head over Spillway in prototype.	Discharge in model in c/s.	Discharge in prototype in c/s.	Remarks.
1	0.147	1.47	0.759	239.8	
2	0.182	1.82	1.037	327.7	
3	0.200	2.00	1.126	374.4	
4	0.227	2.27	1.39	430.2	
5	0.263	2.63	1.452	458.8	
6	0.638	6.38	1.488	470.2	
7	0.655	6.55	1.502	474.6	
8	0.714	7.14	1.521	480.6	
9	0.935	9.35	1.558	492.3	

PALAR ANICUT—ENERGY DISSIPATION STUDIES

1. Introduction :

The Palar anicut has been constructed across the Palar river about four and a half miles below the town of Arcot in N. A. District. This anicut has been provided with stepped aprons in the rear and these aprons are partly of solid masonry and partly of drystone work. The first apron in the rear consists of cut-stone work and varies in width from 10 feet at some places to 20 feet at some other places. The second apron consists of solid masonry and is also varying in width. A row of reinforced concrete piles is provided at the end of the second apron. The third apron is of rough stone dry packing and is provided with a retaining wall at its end. A concrete cart track has been formed over the third apron just in rear of the second apron and its width varies from 10 feet to 14 feet. The fourth apron is also of drystone work and is provided with a retaining wall at its end. The top of this last retaining wall is kept at El. 481.50 and it is made of roughstone drypacking. A typical section is given in Fig. 1.

2. Problem :

It is reported that during every flood, when water-level rises by 5 feet and more over the crest of the anicut, severe damages are caused to the rear aprons. After the reconstruction of the anicut in 1905, flood damages to the rear aprons had occurred during the years 1909, 1911, 1916-17, 1930-31, 1935-36, 1937-38, 1943 and 1946 (as many as 8 times during the period of 42 years.

In order to prevent the recurring damages caused to the aprons in times of floods, suitable energy dissipation arrangements have to be designed for the conditions at the Palar anicut. Hence hydraulic model studies were conducted to evolve suitable energy dissipation devices.

3. Data :

Maximum flood discharge over anicut	...	1,70,000 cusecs.
Front M.F.L.	...	El. 504.23 feet.
Length of anicut	...	2628 feet.
Effective length of anicut	...	2555 feet.
Maximum discharge per foot run of the anicut	...	67.7 cusecs.
Average crest level of anicut	...	El. 495.38.

35952

4. Model Set Up:

A sectional model of the Palar anicut and the rear aprons was constructed to a scale of 1/24 in a masonry flume 4 feet wide. A typical section of the Palar anicut was taken for model studies vide Fig. 1. The anicut and the solid aprons were constructed by bricks in cemeat mortar, plastered over and finished with neat cement. The drystone work (lakes portion) was simulated by pebbles set in soil cement in the ratio of 75 soil to 1 cement. The bed of the river beyond the aprons was moulded with sand passing through 24 mesh sieve and retained on 30 mesh sieve in order to facilitate a steady of the scour pattern. Arrangements were provided in the model for determining the flow and scour profiles and for maintaining the upstream and downstream water-levels.

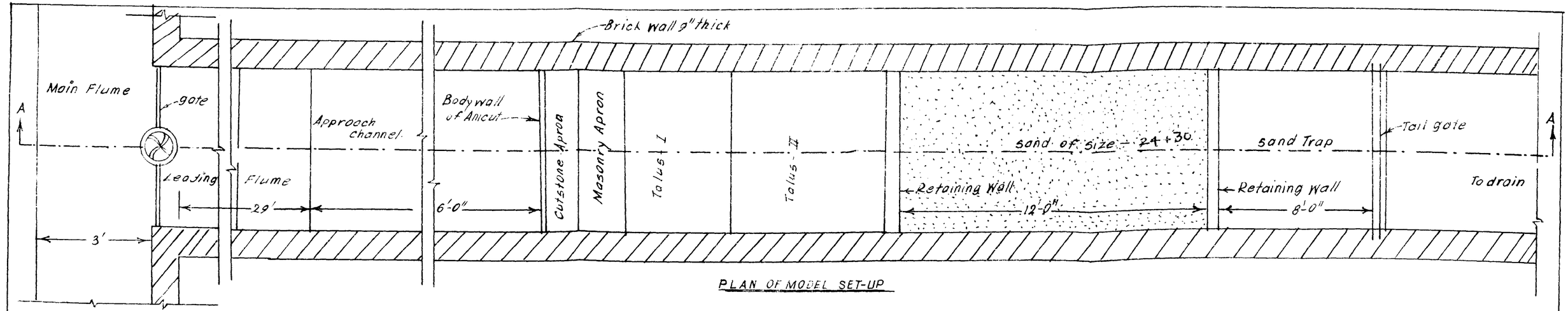
5. Experiments:

Experiments were conducted on the following two types of energy dissipation arrangements.

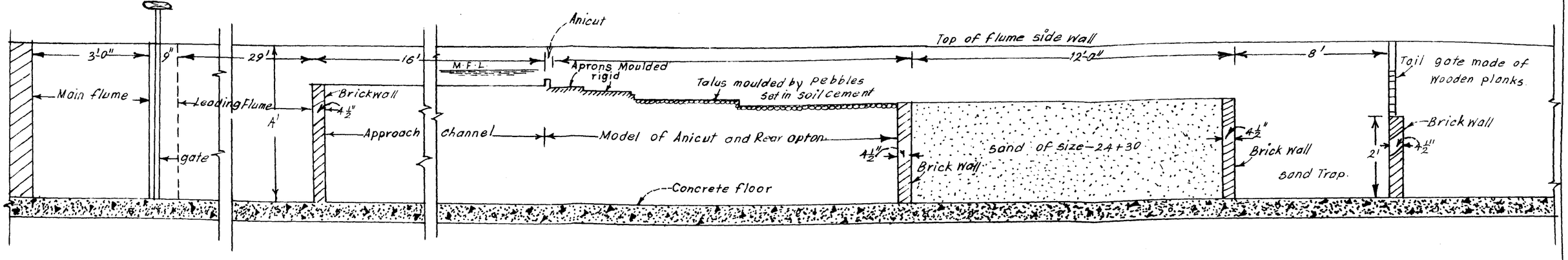
(1) Baffle walls and friction blocks.

(2) Sunk basin type of stilling arrangement.

The first type of energy dissipation arrangements was found to be unsatisfactory for the conditions existing in Palar anicut. Thefore, the sunk type of stilling basin was tried. A sloping apron which will form a sunk basin of 6 feet depth was found to give a good hydraulic performance. Further confirmatory studies are in progress.



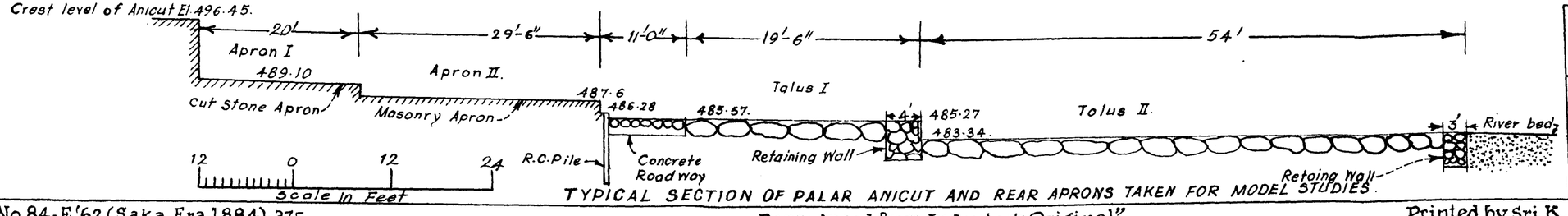
PLAN OF MODEL SET-UP



SECTION ON AA

M.F.L. El. 504.23

Scale in Feet: 0, 4, 8, 12



TYPICAL SECTION OF PALAR ANICUT AND REAR APRONS TAKEN FOR MODEL STUDIES.

ANNUAL REPORT OF 1959	
14/30	1/1
ITEM NO	DRAWING NO

Public Works Department — Madras
Irrigation Research Station — Poondi.

PALAR ANICUT
ENERGY DISSIPATION STUDIES
PLAN OF MODEL LAY-OUT.

Report.	File	Drawing	26/61
J.E.R	A.E.R	E.E.R	
Trd. R.R. Nathan.	Ckd.	FIG. 32.	

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PALAR ANICUT—SILT EXCLUSION STUDIES

1. Introduction :

1.1. There is an existing anicut across the river Palar, about four and half miles below the town of Arcot in N. A. District. The short-lined floods of Palar bring in a large quantity of coarse bed load and in spite of the existence of thirty nine scouring sluices in the body wall, of the anicut, the anicut has been silted up to its crest. The four channels taking off from the anicut Mahendravady and Kaveripauk on the left flank and Sakkaramallur and Dusi on the right flank have also drawn a lot of coarse bed load resulting in the reduction of their carrying capacities. Hence research work was conducted on a hydraulic model of the Palar anicut and the channels to evolve suitable devices from maximum exclusion of silt entering the channels.

2. Data :

2.1. *Anicut* :

1.	Crest Level	...	El. 496.38
2.	Length of anicut	...	2628 feet
3.	Top width of anicut	...	4'.6"
4.	Bottom width of anicut	...	8' to 9'

2.2. *Channels and head sluices* :

The data pertaining to the channels and the headsluices are given below :

Serial No.	Name of head sluice.		Sill level.	No. and size of vents.	Ayacut in acres.	Capacity at head in cusecs.	Length of main channel.	No. of tanks fed.
					Each.			
1.	**Left flank** :							
	Kaveripauk	...	El. 488.18	18 Nos. 5'×5'	51.059	3443	10M 7F	159
2.	Mahendravady	...	El. 488.18	3 Nos. 5'×5'	4849	552	24M 7F	28
3.	**Right flank** :							
	Sakkaramallur	...	El. 489.33	3 Nos. 6'×5'	5105	624	20M 3F	21
4.	Dusi	...	El. 490-18	15 Nos. 5'×5'	19.903	1598	23M IF	109

2. 3. *Scour vents*: The details of the scour vents are furnished below :—

Serial No.	Description of Scour vent.	Sill level.	No. and size of vents.	Elevation of operating platform.
1.	Left flank—			
	First set	... El. 489.18	9 Nos. 5'×3' 6" each	Above M.F.L.
2.	Second set	... El. 488.18	10 Nos. 5'×4' each	497.50
3.	Right flank	... El. 488.18	20 Nos. 5'×4' each	Fifteen vents 497.50. Five vents Above M.F.L.

3. Model set-up :

3. 1. A mobile bed model of the Palar river to a horizontal scale of I/200 and a vertical scale of 1/40 was constructed. The length of the river for about 2 miles upstream of the anicut and for about 1/2 mile downstream was reproduced in the model. The headsluices of the four channels were made in teakwood and fixed in their relative positions properly. G. I. shutters were provided for the headsluicer. The channels downstream of the headsluices were moulded rigid whereas the bed of the river was moulded to the cross-section templates already fixed along the length of the river course, with sand of size-24, 30 i.e., sand passing through 24 meshes sieve and retained on 30 mesh sieve. Scour vents were provided in the anicut on both the flanks with G.I. shutters to control the flow through them. In orderto measure the discharges in the river and the four channels, V-Notches were provided at the appropriate places. Other usual appurtenances like the back-feeding arrangements, tailgate control, silt traps etc., were provided in the model.

4. Experiments :

Experiments were conducted on this model to evolve devices for exclusion of silt from the four channel mentioned above. Improvements to the existing anicut, by way of lowering the sill levels of the scour vents on the two flanks of the anicut; provision of divide walls of optimum length between the scour vents and the headsluices of the channels etc., were tried in the model.

GRAND ANICUT CANAL—SILT EXCLUSION STUDIES.

1. Synopsis.

This report deals with the model studies conducted for evolving suitable remedial measures for reducing the silt entry into the Grand Anicut Canal. A short history of the development of the problem and the various field measures and the model studies undertaken to solve this problem right from the opening of the canal till date have been furnished. Details of the model studies conducted and the various devices tried such as divide wall with different widths of pocket and intermittant operation of the enclosed vents have been fully described and a solution has been evolved. Detailed recommendations for incorporating this solution have been furnished. A brief note on the further research work that has to be done has also been given.

2. Introduction.

Grand Anicut is an anicut irrigation structure constructed across the prehistoric breach called Ullar, on the left side of the River Cauvery. The Anicut is situated 12 miles from Tiruchirapalli. At this site, Cauvery and Vennar branch off and water is drawn into these rivers through regulators (vide Figure-I). Ullar joins with the river Coloroon some distance downstream of Grand Anicut. After the construction of the Mettur Dam for irrigation proposes in Tanjore District, a canal known as Grand Anicut Canal designed to take a discharge of 4,300 cusecs was taken off, from the River Cauvery at the site of Grand Anicut. The regulators act as diversion works and no substantial amount of water is stored as a reserve. During flood seasons, the Grand Anicut is opened and the flood water is let into the River Ullar.

3. Hydraulic Data.

(i)	Average bed of the River Cauvery at Grand Anicut ...	185.00
(ii)	Normal supply level ...	El. 194.95
(iii)	Sill level of Grand Anicut ...	El. 189.00
(iv)	Size and number of vents in Grand Anicut ...	30 vents of 32' each
(v)	Sill level of scouring sluice ...	El. 179.25
(vi)	Size and number of vents in the scouring sluice ...	5 vents of 20' each
(vii)	Sill level of the Cauvery Regulator ...	El. 184.25
(viii)	Size and number of vents ...	42 vents of 10' each
(ix)	Sill level of Vennar Regulator ...	El. 184.25
(x)	Size and number of vents ...	33 vents of 10' each
(xi)	Sill level of Grand Anicut Canal head regulator ...	Old crest : El. 194.19 Raised sill El. 185.86
(xii)	Size and number of vents of the G.A.C. head regulator ..	6 vents of 10'x5'4" each
(xiii)	Full supply discharge of Grand Anicut Canal ...	4,300 cusecs.
(xiv)	F. S. L. Depth ...	7'0"
(xv)	Bed width of the canal ...	180 feet
(xvi)	Bed slope ...	1.5 in 10,000
(xvii)	Upland drainage sill level ...	El. 185.00
(xviii)	Size and number of vents ...	6 vents of 30' each.

Problem.

As early as 1934, that is one year after the opening of the Grand Anicut Canal, this canal was drawing lot of silt with the result that the bed is being progressively silted up. Due to excess of silting up, the full supply discharge is not realised in the canal. A number of remedial measures were tried at the field to remedy this problem but all were futile. So, model studies were resorted to at this Station to find out an effective solution for this chronic problem.

4. History.

The Grand Anicut Canal was constructed in the year 1933. Even as early as 1934, the bed of the canal was getting silted up and proposal for excluding silt from the Grand Anicut Canal was receiving much attention from the department. Various suggestions were made and some of them were carried out in the field.

In the year 1937, a detailed report about the silting problem was furnished by the Superintending Engineer to the Chief Engineer who also suggested a groyne 300 feet long in between Grand Anicut Canal and the upland drain as one of the remedial measures. This problem was also referred to the Research Station, Poona for model studies.

In the year 1938, Mr. C. C. Inglis, C.E.E., B.A.I., M.Inst., C.E., M.ASCE., the then Director, Central Irrigation Research Institute, Poona, suggested the following :

(1) A groyne from the left bank and above the water level and a long submerged vane from the left bank.

(2) A long curved vane running out from the village Thohur.

The Director of the Research Station expressed the opinion that the silt trouble was due to the designed slope of the canal being inadequate and the canal was building its own slope. He was also of the opinion that if the silt excluders and ejectors could be used, it would be easier, better and cheaper than controlling silt from Cauvery river. The field Superintending Engineer, thereupon reported that it was not possible to increase the slope but it was possible to eject silt into the Vennar River at HD 0/40.

By the end of the year 1939, the Chief Enginner (Irrigation) informed the Director, Central Irrigation Research Station, that the silting at the head of the canal was perhaps due to radial shutters fitted at the head and mentioned that it was proposed to put in lift shutters (vide item 8 of figure-3). He also informed the Director that they were not immediately interested in solving the silt troble in the canal as development of ayacut was slow and that sanding was not likely to cause any difficulty as the head of water could be raised by holding up the water to a higher level and force the requisite supply into the canal, even with the sanded bed.

In the year 1940, the field Superintending Engineer, expressed the unsuitability of providing removable shutters downstream of the radial shutters. The Chief Engineer, then instructed that a drop wall or a curtain wall might be built in front of the Grand Anicut Canal head regulator (vide item 4 of figure 3) including three or four vents on the right side which can be used to scour the basin in rear of the curtain wall and above of the head sluice of the Grand Anicut Canal. But this was not constricted.

In the year 1943, a rough stone groyne was constructed for a length of 300 feet at an angle of 35° to the upland drainage bridge with the top width as 4'0" (vide item 3 of figure-3). This was the same as the proposal suggested by the field Superintending Engineer in the year 1937. This did not give the desired effect in excluding silt entry into Grand Anicut Canal.

In the year 1944, the Chief Engineer and the Superintending Engineer, after a joint inspection, decided to put another groyne a little downstream of the existing one. This was also constricted. It was decided that nothing should be done except operating the Vennar end vents as undersluices and no silt clearance was to be done. Only shoals and marginal accretions were to be removed and the behaviour of the canal had to be closely watched.

The Executive Engineer, Grand Anicut Canal reported in the year 1947, that the silting of the canal bed had always been on the increase and the silt which only travelled upto R.D. 1/30 in 1939 had travelled upto 2/20 in the year 1947. The Chief Engineer, then, suggested the following procedure to be adopted in the field.

(1) As soon as the Mettur sluices were opened for the supply of water the Grand Anicut Canal regulator should be kept closed for a day or two and the end vents of the Vennar Regulator should be opened to wash out the shoal in front of the G. A. C. regulator as effectively as possible, the shoal being also ploughed before water is admitted into the rivers so as to facilitate the scouring section.

(2) After the above was done the Grand Anicut Canal and the Vennar vents might be both kept open continuously and soundings taken in the basin between the Grand Anicut Canal head and the Vennar end vents to watch the behaviour of the bed within the limits of the basin. If material changes occur by the way of either silting or scouring or if by any change of regulation was made so that a steady bed could be maintained it could be noted and followed.

As per the Chief Engineer's instructions the effects were :

(1) The fresh shoal was lowered a little.

(2) Some silting up took place in front of the regulator and below.

The problem was referred to the Irrigation Research Station, Poondi in the year 1947, for conducting model studies for reducing the silt entering into the Grand Anicut Canal. Meanwhile silt clearance works were also carried out in the field. Some experiments on scale models were carried out in the Research Station, Poondi for the construction of silt ejector at R.D.O./40 of the Grand Anicut Canal. A silt ejector was finalised and recommended along a bell mouthed entry at the Grand Anicut Canal head sluice to facilitate the drawing of more discharge. The construction of this was deferred.

In the year 1950, the construction of the curtain wall in front of the Grand Anicut Canal head sluice with its top at 188.00 from the pier in between the vents No. 9 and 10 of Vennar from South and extending to 300 feet into the river to a distance of 79 feet from the right abutment of Vennar regulator was carried out (vide item 3(a) of Figure-3). But it was later found that the rolling of bed silt had not been effectively stopped by this curtain wall because of the fact that the bed of the river upstream of Grand Anicut Canal head had progressively silted up, thereby, defeating the

purpose of the curtain wall. Proposals were put by the Superintending Engineer concerned to continue the above curtain wall beyond the Ceylon Island (which was formed in the year 1948 by removing the collapsed groyne) and to connect it with the existing groyne (just above the Grand Anicut Canal head) Refer item 6 of Figure-3), raising the top level of the curtain wall to El. 190.00. But the Chief Engineer for Irrigation instructed that the curtain wall need not be extended or raised. But later in the year 1951, the Chief Engineer (Irrigation) apporoved the extension of the above mentioned design. Accordingly, the curtain wall was continued beyond the island to connect it with the existing bank (vide 3b of Figure-3). The extension wall was built in random rubble masonary in clay and pointed with cement mortor.

Meanwhile after further experiments in the Research Station and after attending to Chief Engineers remarks on the proposal of silt ejector a revised design for the silt ejector at R.D.O/40 of canal was submitted in the form of a report No. 1/51.

In the year 1952, the then Executive Enginser suggested the raising of the curtain wall by one foot further upto El. 190.00. But this was disapproved by the Superintending Engineer. Meanwhile further silt clearance was carried out. The portion of the curtain wail which was constructed in Random Rubble in clay got damaged several times and had to be repaired often. Proposals were also submitted in the year 1954 for the construction of a new training wall with concave curve (vide item 7 of Figure-3) (instead of the curtain wall with convex curve in random rubble in clay as existing). The Chief Engineer for irrigation was of the opinion that the construction of the curtain wall might await till the experiments conducted at Poondi were over. The prototype observation showed clearly that the curtain wall had no effect in excluding silt entering the Grand Anicut Canal. Silt clearance was carried out and it was observed that after the silt clearance the designed discharge flowed into the canal without any reduction.

Then, the desirability of converting the right end six vents of Vennar regulator (vide item 9 of Figure 8) into scouring sluices by lowering the sill by 2' to 3' was asked to be examined by the Superintending Engineer.

Silt clearance works were taken up and done manually in the year 1958. While these operations were going on, the Executive Engineer, irrigation Research Division inspected the site and during his inspection several proposals were tried to desilt the basin in front of the Regulators. To effect the desilting, the shuttors of Cauvery and Vennar were opened as much as possible and a minimum water level in front of the regulators was maintained, at the same time closing the Grand Auicut Canal head regulator. It was clearly seen in the field that the sand was getting desilted at a very rapid rate. As this wss expected to remove a good amound of silt upstream of the regulators the Executive Engineer, Irrigation Research Division suggested that this type of regulation might be tried in the remaining period of closure. He, further suggeed, an inner cur method to desilt the canal but this was not approved by the Chief Engineer for Irrigation.

In the year 1958, the Chief Engineer for Irrigation instructed that necessary estimates might be prepared for raising the sill of the Grand Anicut Canal head regulator and also to provide a supplementary head sluice to draw extra supply from the upland drainage. Necessary model studies were carried out in the Irrigation Research Station, Poondi to find out

the efficacy of this proposal in excluding silt entry into the Grand Anicut Canal. The reduction in the silt entry was not appreciable and the result of experiments were furnished in the form of I. R. S. Report No. 4/59. Further studies were carried out in the same model with increased discharge as well as the full discharge drawn through the additional head sluice did not help much in reducing silt entry into Grand Anicut Canal. Results of these experiments were furnished in the I. R. S. Report No. 5/60.

Meanwhile it was proposed to conduct further model studies to arrive at remedial measures at the head of the regulator itself and hence the required field particulars were obtained and a new model representing the Cauvery river two miles upstream of Grand Anicut with all the regulators was constructed. This report deals with the studies conducted in this model.

Another suggestion that has been made to reduce the silt entry into the Grand Anicut Canal is to construct a new head sluice on the right bank of the Cauvery river about two miles above the present head (at old Vennar offtake) where favourable curvature of the river exists and to utilise the old Vennar river for the formation of a Canal to link up with the present one. Model studies on this proposal are being takenup.

5. Model set up :

A vertically exaggerated modal was constructed to a scale of 1/200 horizontal and 1/40 vertical. The upstream of the Cauvery river above Grand Anicut was represented upto two mile and the regulators of the Cauvery, Vennar and Grand Anicut Canal were represented together with the Grand Anicut and the scour vents. The river banks were formed with cement mortar. All the regulators were also constructed with cement mortar. G.I. sheet shutters were provided for all the regulators. The portion representing the padugai lands was made rigid with Cement mortor with a rough finish. The bed of the river alone was kept mobile. Rods mounted on wooden reepers were placed at different cross sections, the top level of the rods representing the bed level of the Cauvery river. This arrangement was adopted to facilitate the moulding of the bed, thereby, to simulate the existing bed level of the river. The sand used for the mobile bed was of size 40-60.

Silt traps were provided downstream of the regulators to collect the sand passing through regulators.

V-notches were placed up stream of the Cauvery river and downstream of the regulators for measuring the discharge let into the river and the discharge passing through each one of the regulators. Necessary honeycombs were provided as stilling arrangement. Gauges were fixed for measuring the discharges. The whole model set up has been shown in Figure-4.

6. Experiments and Results :

A discharge corresponding to 26,500 cfs was allowed in the river. This was a representative discharge taken from the water rigister of 1957, when the Grand Anicut Canal was discharging 4,000 cusecs. Corresponding to this discharge in the river, discharges of 1,800 cfs, 12,000 cfs, 9,000 cfs and 4,000 cfs were allowed in Ullar, Cauvery, Vennar and Grand Anicut Canal respectively. The Water level upstream of the regulators was maintained at about 195,00 and the rear water levels downstream of the regulators were maintained at El. 180. El. 196.66, El. 191.19 in scouring sluices, Cauvery, Vennar and G. A. ~~Canal respectively~~.

The bed was moulded with a sand of size 40-60. The model was run to find out the correct rate of sand injection so that the model would come to regime conditions. By trial and error, it was found that a rate of sand injection of 50 lbs/hour was suitable. During the basic run observations like the velocity distribution in front of the Grand Anicut Canal head regulator, (Figure 5) and tracing of the fllow lines (Figure-5) and also taking the cross sections of the regime bed after the basic run (Figure-6) were made. Hourly collections of the sand that was passing through the various regulators were made and the following statement gives the silt distribution through each of the regulators.

STATEMENT—1

Name		Quantity collected per hour	Percentage collection
Grand Anicut Canal	...	42 lbs.	76.2 %
Vennar	...	7½ lbs.	13.8 %
Cauvery	...	5 lbs.	9.5 %
Ullar	...	¼ lb.	0.5 %
TOTAL	...	54¾ lbs.	100 %

After attaining the regime condition, experiments were conducted to find out devices for silt exclusion in Grand Anicut Canal by trying some remedial measures in front of the head regulator. Firstly a divide wall of 300 feet length the enclosing the right six vents of the Vennar regulator and placed perpendicular to the regulator was tried.

The enclosed vents of the Vennar regulator were kept closed. The top of the divide wall was kept above the water level and studies were conducted to find out the efficacy of this proposal. The silt distribution through various regulators are given below :

PROPOSAL—1

STATEMENT—2

Divide wall length ... 300 feet.
No. of Vennar vents enclosed ... 6 Nos.

Name		Quantity collected per hour	Percentage collection
Grand Anicut Canal	...	22¼ lbs.	42.8 %
Vennar	...	22½ lbs.	43.2 %
Cauvery	...	6¾ lbs.	13.0 %
Ullar	...	½ lb.	1.0 %
TOTAL	...	52 lbs.	100 %

As mentioned in the basic run, the same observations like velocity distribution, flow lines and cross sections of the bed were taken separatively for various proposals. Two more proposals were tried by increasing the width of pocket enclosing 9 vents and 12 vents respectively. As before, the divide wall was kept at right angles to the Vennar Regulator and the length of the divide wall being the same as before (i.e.) 300 feet. The enclosed vents of the Vennar Regulator were kept closed.

The following tables give the distribution through various regulators.

PROPOSAL—2

STATEMENT—3

Divide wall length ... 300 feet
No. of Vennar Vents enclosed 9 Nos.

Name		Quantity collected per hour	Percentage collection
Grand Anicut Canal	...	17¾ lbs.	33.6 %
Vennar	...	32 lbs.	60.6 %
Cauvery	...	3½ lbs.	4.8 %
Ullar	...	¼ lb.	1.0 %
TOTAL	...	53¾ lbs.	100 %

PROPOSAL—3

STATEMENT—4

Divide wall length ... 300 feet.
No. of Vennar vents enclosed ... 12 Nos.

Name		Quantity collected per hour	Percentage collected
Grand Anicut Canal	...	14 lbs.	30.4 %
Vennar	...	29½ lbs.	64·1 %
Cauvery	...	1½ lb.	3.3 %
Ullar	...	1 lb.	2.2 %
TOTAL	...	46 lbs.	100 %

The closed vents of the Vennar regulator were operated intermittantly to function as scour vents for flushing out the silt in the pocket, keeping the G. A. Canal regulator closed during this time. The second proposal i.e. divide wall placed nine vents apart from the right end of the regulator was incorporated in the model and these tests were carried out. The enclosed vents were opened for a period of 48 minutes at regular intervals of 11 hours 24 minutes corresponding to one day in a fortnight in the prototype. The results are tabulated below :—

STATEMENT—5

Name		Quantity collected per hour	Percentage collected
Grand Anicut Canal	...	12½ lbs.	27.2 %
Vennar	...	30¼ lbs.	66.2 %
Cauvery	...	2¾ lbs.	6.0 %
Ullar	...	½ lb.	0.6 %
TOTAL	...	46 lbs.	100 %

The same experiments i.e. the operation of the enclosed vents of Vennar regulator for a period corresponding to one day in every 15 days in the prototype, were conducted in the model with the divide wall of 300 feet length at the second main pier (i.e) enclosing 6 vents of Vennar regulator The results are tabulated and given below :

STATEMENT—6

Name		Quantity collected per hour	Percentage collection
Grand Anicut Canal	...	11½ lbs.	23.7 %
Vennar	...	22 lbs.	45.7 %
Cauvery	...	11¾ lbs,	24.4 %
Ullar	...	3 lbs.	6:2 %
TOTAL	...	48¼ lbs.	100 %

As suggested by the Chief Engineer (Irrigation) a divide wall of length 232'6" equal to the length of the straight curtain wall was incorporated in the model, keeping the top of the divide wall below water level at El. 194.00. The spur and the curved curtain wall were also incorporated in the model. The silt distribution through various regulators is tabulated below :

STATEMENT – 7

Name.		Quantity.	Percentage.
Grand Anicut Canal	...		56.5 %
Vennar	...		38.4 %
Cauvery	...		4.74 %
Ullar	...		0.4 %
TOTAL	...		100 %

Observations were also made for the same divide wall of length 232' 6" but the top kept above water level. The spur and the curved curtain wall was removed. The results are tabulated below :—

STATEMENT—8

Name		Percentage collection.
Grand Anicut Canal	...	46.1 %
Vennar	...	42.4 %
Cauvery	...	8.2 %
Uller	...	2.7 %
TOTAL	...	100 %

The length of divide wall was increased to 500 feet and the top level of the divide wall was kept above water level. The same six vents of Vennar regulator were kept closed, the location of the divide wall being the same as in the previous case. The following table gives the results of the observation made on this proposal.

STATEMENT—9

Name.		Percentage collection.
Grand Anicut Canal	...	20.8 %
Vennar	...	52.8 %
Cavery	...	17.2 %
Ullar	...	0.2 %
TOTAL	...	100 %

7. Discussion of results.

The location of the Grand Anicut Canal head regulator with respect to the river course and the other regulator seems to be most unfavorable in that the canal happens to draw a disproportionately larger quantity of silt than its due share of discharge. In this connection it is to be pointed out that the offtake of Grand Anicut Canal branching off at right angles to the main river also aids more drawal of silt into the canal. In Joglekar's works "model experiments and field experience have definitely shown that an off take at right angles to the parent channel does, in no way help exclusion. On the contrary, much less too water, moving at high velocity is deflected into the offtake as compared to slow moving bed water which brings in a large amount of course bed materials "(1).

The right angled offtake of the canal combined with the general unfavourable curvature of the river aids in the movement of a large quantity of silt into Grand Anicut Canal and the inadequacy of the slope of the canal further aids in the deposition of material on the bed of the canal.

The results of the basin run clearly reveal the effects mentioned above and the quantity of silt passing through Grand Anicut Canal was 76.2 % (Table-1) of the total collected silt, while the discharge drawn by the canal was only 15.1 % of the total discharge in the river (400 cfs. out of 26,500 cusecs). Though these model results may not exactly tally quantitatively with the prototype values, yet they will furnish a qualitative appraisal of the situation.

The bed flow lines observed for this basic run show clearly (vide Figure-5 and photo 1 and 2) have the bed filaments curve towards the Grand Anicut Canal regulator.

Devices that are to be evolved for avoiding this silt entry into the canal may be divided into two types; (i) shifting of the offtake to a position where the river has a favourable curvature and (ii) providing suitable structures upstream of the head sluice to correct the flow to avoid silt entry into the canal. Item (1) is being considered separately (as mentioned in the history) and item (ii) is being discussed in this report. Other forms of solutions consist of providing silt ejectors in the canal to eject the silt in the canal into the Vennar nearby. Experiments were carried out already on this in the previous years and reports submitted.

As a remedial measure, divide wall upsteam of the regulator to isolate the canal head regulator from the main flow was found to very useful in excluding silt. Since the top level of the divide wall was above the water level and also the end vents of Vennar enclosed in the pocket were kept closed a still pond was created in the pocket from which canal drew its supply. The improvement in the exclusion of silt due to divide wall is based on the reason that there is a difference in the discharge intensities per foot run in the pocket and in the river.

Referring to Figure-3 and photos 1 and 2 it is seen that in the basic run the bed flow lines are converging towards the Grand Anicut Canal head regulator and this is one of the reasons why Grand Anicut Canal is drawing a lot of silt, as the flow lines are indicative of the movement of bed load. When the divide wall is introduced and the enclosed vents of the Vennar regulator are closed, a still pocket is formed which, in turn, reduces the velocity of water entering into Grand Anicut Canal. So the bed material is dragged towards the Vennar side where a higher velocity exists. It can be seen from the flow lines (Figure 7 and Photos 3, 4, 5 and 6) flow lines are deflected towards the river side. The still pocket provided, as it were, a concave curvature of flow and Grand Anicut Canal draws its supply from the outside of the curve and hence the reduction of silt is effected.

Referring to the velocity distribution diagram for the basic run (Figure-5) contours of higher velocities are entering into the Grand Anicut Canal head sluice and contours of lower velocities are entering into the Vennar regulator. This is one of the reasons why large quantities of silt are entering Grand Anicut Canal and Vennar is not taking its due share of silt.

But when the divide wall is introduced these higher velocities are damped by the still pond and comparatively lower velocity contours are entering into Grand Anicut Canal. This lower velocity of water near Grand

Anicut Canal head regulator is responsible for the reduction in the silt intake by Grand Anicut Canal.

The ratio of the velocity en the river side to the velocity on the Crand Anicut Canal side of the divide wall should be greater than one (Figure 5 and 6) and as the ratio increases the reduction in the percentage silt entry in Grand Anicut Canal will also increase. This large velocity on the Vennar side of the divide wall will help in drawing more silt through the Vennar regulator.

In the basic run it was found that the percentage of silt drawn by Grand Anicut Canal was 76.2% (vide statement 9 and Figure 8) and after the introduction of the divide wall when the width of the pocket was about 83 feet enclosing 6 vents of Vennar regulator, the percentage silt drawn by Grand Anicut Canal was reduced to 42.8 % (vide statement 9). As the width of the pocket was increased the stilling effect of the pocket was more pronounced and the reduction of the silt entry in Grand Anicut Canal was increased further. So, as against the first proposal, the silt exclusion was more pronounced in the second proposal. Referring to Figure (8), the curve becomes relatively flatter after the second proposal. Referring to Figure (8), the curve becomes relatively flatter after the second proposal and there is not appreciable difference in the sand entry between the second proposal and the third proposal. Hence, there is an optimum limit for the width of the pocket, beyond which any further increase in width may not result in proportionately increased reduction of sand entry into the Grand Anicut Canal.

As can be seen from the Statement-9, the divide wall is effective in diverting the silt towards the Vennar and Vennar can take up this silt without its bed getting silted up, as it is a natural river, directly having its infall into the sea.

A scour hole gets formed at the tip of the divide wall in the shape of an inverted cone. It is necessary that the scourhole should be there as it aids the sediment reduction in the pocket. Considering the extent of scour at the nose of the divide wall, the materials thrown into the scour, due to intense turbulence and due to greater velocity on the Vennar side are taken towards the Vennar regulator. The scour around the nose has got a good bearing in diverting the silt towards the Vennar side.

Occasional flushing of the pocket was tried and found to be effective in reducing the silt entry into Grand Anicut Canal. With the divide wall at the third main pier of the Vennar regulator from the right end and at right angles to it (second proposal) and with occasional flushing of the pocket, the silt entry was reduced, to 27. 2% as against 33. 6% (vide statements 2 and 5) for condition of no flushing. With the divide wall of 300 feet length at the second main pier enclosing 6 vants of Vennar regulator, the reduction of silt due to flushing effect was found to be much more (vide Statements 2 and 6). The duration of the operation of the scour vents of Vannar regulator was so arranged in the model as to correspond to one day's operation in every fifteen days in the prototype. It has to be mentioned that this study is only a qualitative study and it can be concluded that the intermittent operation of the closed vents, keeping the canal closed at the same time, helps to reduce the silt quantity entering into Grand Anicut Canal. In the field such operation to flush out the sand accumulated in the pocket can be done whenever it is feasible i.e. whenever it is possible to close the canal for a day or two.

DEEP COURSE

At times in the model, a narrow deep course got formed adjacent to the right bank of the Cauvery river upstream of Grand Anicut Canal regulator (vide Figure-6). After the formation of such a deep course an abnormal decrease in the collection of silt in the Grand Anicut Canal was observed and the percentage collection came to about 12%, It could not be ascertained from the model studies that the formation of such a deep course is due to the divide wall or not. It cannot also be clearly stated whether such a deep course when formed in the prototype will effect such a heavy reduction in the quantity of silt t hat enters into the Grand Anicut Canal.

In view of the case and cost of construction, it is proposed that the existing curtain wall (top El. 189.00) may be rsised to El. 196.00 (above water level) so that it forms a divide wall enclosing six vents of Vennar regulator in this case, no fresh foundation is necessary for a length of 232 '6' and this may be extended to 300 feet. As seen from the results of the experiments, the percentage of silt entry into the Grand Anicut Canal is 56. 5% when the divide wall top is below water level (El. 194.00) and its length is kept as 232' 6" (the same length as the existing curtain wall). When the top of the divide wall is raised above water level the percentage of silt entry into the Grand Anicut Canal is 56.5% when the divide wall top is below water level (El. 194.00) and its length is kept as 232'6" (the same length as the existing curtain wall). When the top of the divide wall is raised above water level the percentage of silt entry drops down te 46% (vide statement-8). The effect of still pond is more pronounced in the latter case in diverting the bed flow to the Vennar side aud thus inducing a higher velocity zone. Also in the Figure 9 the curve shows that when the length of the divide wall is increased, the percentage of silt entry into the Grant Anicut Canal decreases. For a length of 500 feet, the silt entry into Grand Anicut Canal gets still more reduced to 29.8%. The divide wall ef 300 feet length enclosing 6 vents of Vennar regulator may be tried in the field and if the performance comes upto expectations, then the fnnction of the divide wall may be improved by increasing the length of divide wall by another 200 feet making the total length of the divide wall as 500 feet.

8. Conclusion and Recommendations.

It is found from the results obtained, that a simple divide wall, whose top is above water level with the enclosed vents of Vennar regulator kept closed, will prevent silt entry into Grand Anicut Canal to a considerable extent.

Regarding the width of the pocket, it is found that as the width of the pocket, is increased reduction of silt entering into Grand Anicut Canal also gets increased (vide Figure-8 Statement-9). Based on this it can be concluded that it is better to apopt a wider pocket for an effective silt reduction. But there is already an existing curtain wall enclosing 6 vents of Vennar regulater. Raising this curtain wall above water level to function as a divide

wall will be economical and easy for construction. In view of this it is proposed to have this width of the pocket enclosing vents of Vennar regulator.

It is observed that the function of the divide wall is appreciably effective only when its top is above water level (Vide figure 9-Statement 9).

Considering the length of the divide wall it is found that the length of 500 feet gives the best performance. Based on these conclusion the following recommendations are made for this problem of silt exclusion in Grand Anicut Canal.

(i) A divide wall of 300 feet length is to be placed at the second main pier of the Vennar regulator from the right abutment and at right angles to it enclosing 6 vents of Vennar regulator.

(ii) The top of the divide wall should be kept above water level.

(iii) If any batter is necessary for the divide wall, the face of the divide wall should be kept vertical on the Vennar side and the batter may be given on the Grand Anicut Canal side.

(iv) The vents of the Vennar regulator which are enclosed in the pocket should be kept closed when the Grand Anicut Canal regulator is functioning.

(v) The enclosed vents of Vennar regulator may be operated to clear off the silt collection in the pocket with the Grand Anicut Canal head regulator kept closed. The duration of operation and the interval of days between two successive operations may be decided in the field according to the feasibility of closing the Grand Anicut Canal during rainy reason. It is desirable to have scouring operation for at least one day in about a fortnight.

(vii) At first instance it is suggested that no apron is made at the nose of the divide wall and after one season, the scour that may have occured at the nose may be pitched with loose boulders to the profile of the scourholes.

(viii) The end vents of the Vennar regulator just near the divide wall on the Vennar side may be operated so as to discharge larger quantity of water. But there is an optimum for this type of widge operation in-as-much as excessive steips would couse swirls; a ten per cent increase in discharge is generally the medium (2).

The finalised recommendations are furnished in Figure-10.

9. Scope for further studies.

(1) The functioning of the divide wall can be tested for various stages in the river including the maximum flood discharge and with various discharges through the regulators.

(2) Shifting of the offtake two miles upstream of the present Grant Anicut Canal head can be studied. It is mentioned that a favourable curvature of the river exists there. An offtake taken off from the outside of the curve may not draw much silt and hence the shifting of the offtake upstream to a place of favourable curvature may be helpful in solving the silt problem in Grand Anicut Canal on a permanent basis. Studies are to be taken up on this aspect.

PHOTO No. 1

PHOTO No. 2

BASIC RUN

The Bed Flow Lines during the Basic Run are shown here

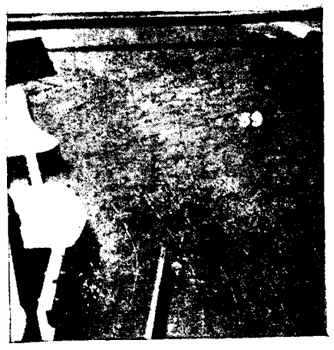

PHOTO No. 3

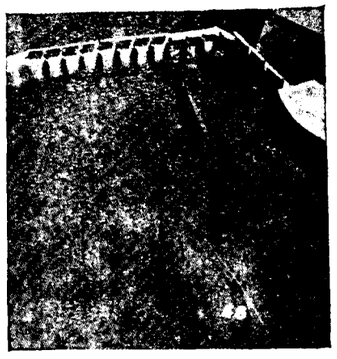

PHOTO No. 4

Proposal—II

Length of Divide wall	...	300
No. of Vennar Vents enclosed	...	6

PHOTO 3 and 4 show the bed flow lines after the introduction of the Divide wall.

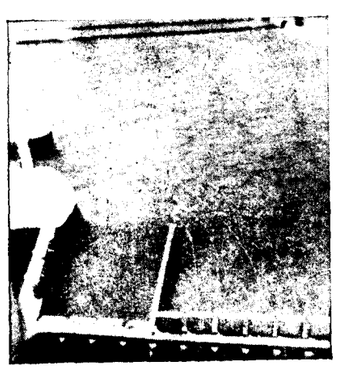

PHOTO No. 5

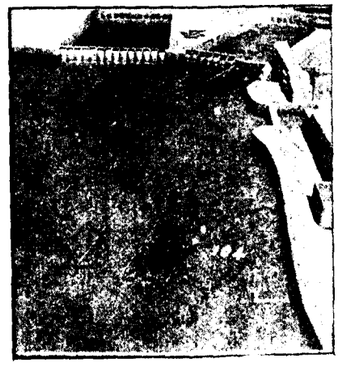

PHOTO No. 6

Proposal—III

Length of divide wall	...	300
No. of Vennar ents Venclosod	...	9

The Bed Flow Lines after the introduction of the Divide Wall are clearly seen.

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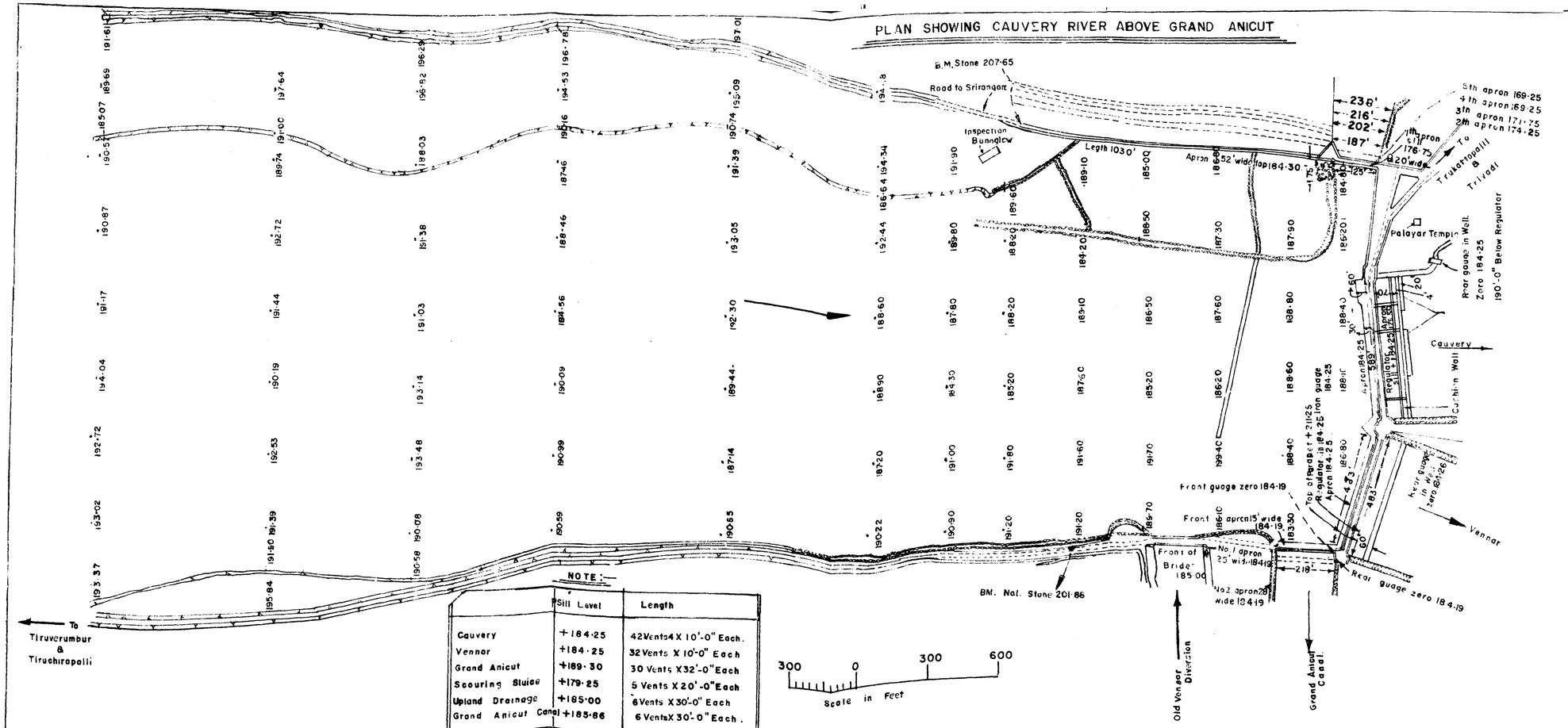
PLAN SHOWING CAUVERY RIVER ABOVE GRAND ANICUT



NOTE:—

	Sill Level	Length
Cauvery	+184·25	42 Vents X 10'-0" Each.
Vennar	+184·25	32 Vents X 10'-0" Each
Grand Anicut	+189·30	30 Vents X 32'-0" Each
Scouring Sluice	+179·25	5 Vents X 20'-0" Each
Upland Drainage	+185·00	6 Vents X 30'-0" Each
Grand Anicut Canal	+185·86	6 Vents X 30'-0" Each.

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

PLAN SHOWING CAUVERY RIVER ABOVE GRAND ANICUT.

/60	Drg.	R. Narayanan.	
		J.E.R.	A.R.O.	E.E.R.
		Trd: C.S.K. Pathy	Ckd:	Fi... I.

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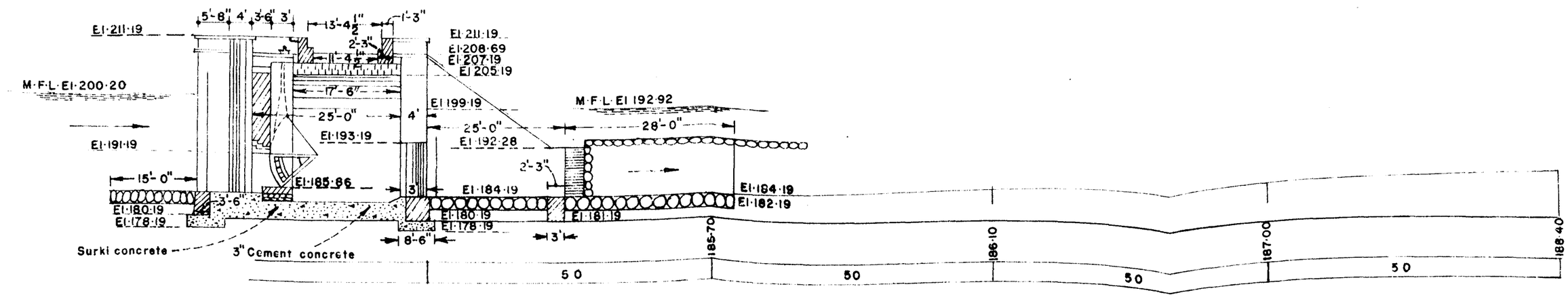
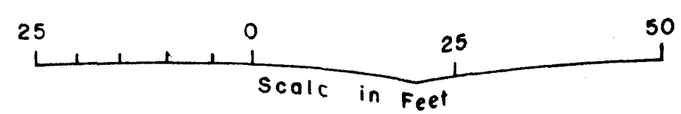


Sill level of G·A·C head regulator

Old crest = 104·19.

Raised crest = 185·86.



Size and No. of Vents of the G·A·C head regulator = 6 Vents of 10' x 5'-4" each.

Full supply discharge = 4 300 cfs

F·S·L depth . . . = 7'-0"

Bed width of canal . . . = 180'-0"

Bed slope = 1·5 in 10,000

Report	Public Works Department – Madras Irrigation Research Station – Poondi.		
File	SECTION OF GRAND ANICUT CANAL REGULATOR		
85/60 Drawing	R. Narayanan	U.V. Sahabi	Dharmadoss
	J·E·R	A·R·O	E·E·R
	Trd·C·S·K.	ckd:	FIG ··· 2.

Reg No 94-E'62 (Saka Era 1884) - 475.

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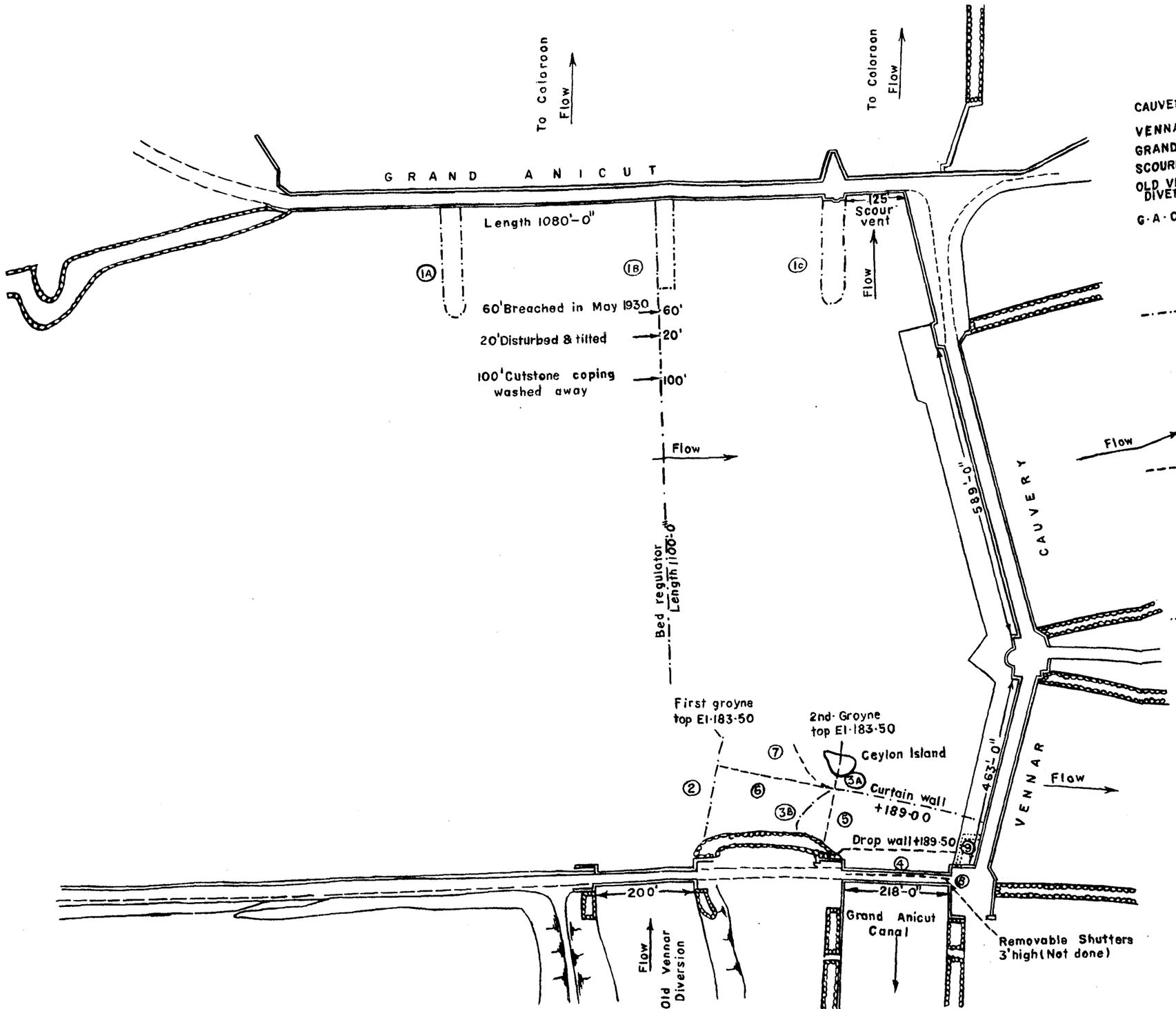

Scale 1/2000

		Sill
CAUVERY	42 Vents of 10'-0" each	El·184·25
VENNAR	33 Vents of 10'-0" each	El·184·25
GRAND ANICUT	33 Vents of 32'-0" each	El·189·30
SCOURING SLUICE	5 Vents of 20'-0" each	El·179·25
OLD VENNAR DIVERSION	6 Vents of 30'-0" each	El·185·00
G·A·CANAL	6 Vents of 30'-0" each	El·185·86

LEGEND

PROPOSALS CONSTRUCTED

1·Groynes and bed regulator perpendicular to the anicut (to avoid flow with high velocity parellel to the anicut)

2·First groyne

3A·Curtain wall

3B·Extension (Convex curve)

PROPOSALS SUGGESTED AND GIVEN OVER

4· Drop wall

5· Second groyne

6· Extension of the curtain wall (straight)

7· ——— do ——— (concave curve)

8· Removable shutters 3'-0" high

PROPOSAL UNDER SCRUTINY

9 Converting the right side 6 vents as scouring sluice and shifting them up

CEYLON ISLAND:— Furmed in april 1948 by removing groyne·5·

Report / File	Public Works Department-Madras Irrigation Research Station Poondi		
	DETAILS OF PROPOSALS CONSTRUCTED PROPOSALS SUGGESTED AND GIVEN OVER AND PROPOSALS UNDER SCRUTINY FOR SILT EXCLUSION IN G·A·CANAL		
Drawing	R. Narayanan.	U.V. Bolshe	[illegible]
	J·E·R	A·R·O	E·E·R
	Trd: c·s·k	ckd:	FIG·3

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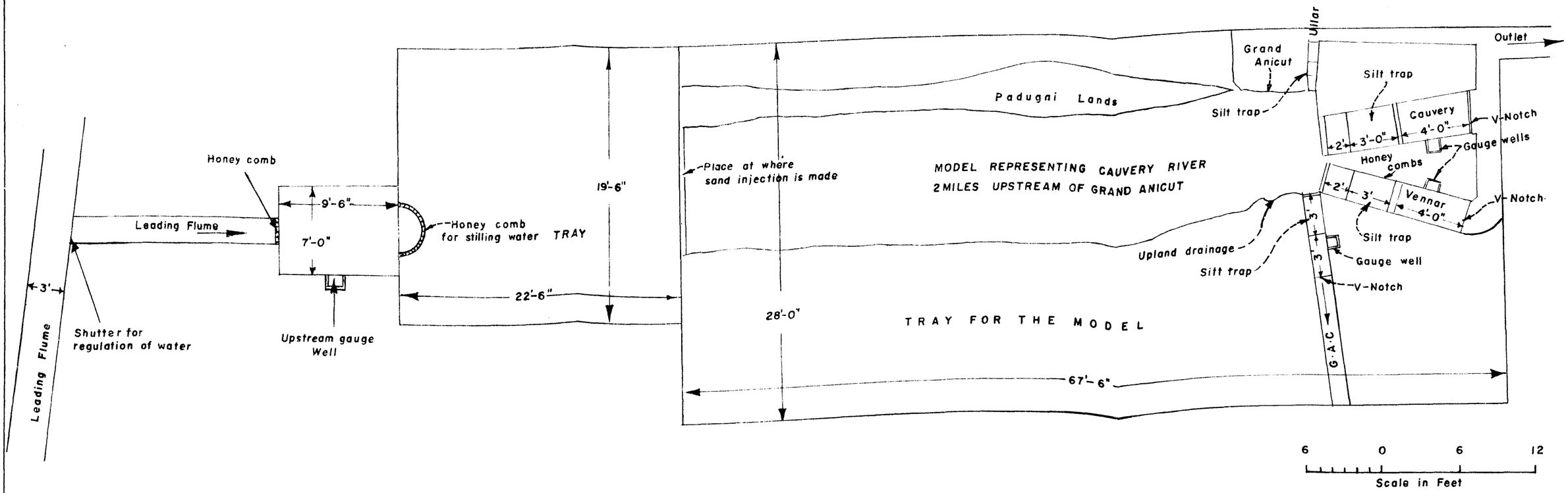

Report	Public Works Department—Madras Irrigation Research Station—Poondi	
File	MODEL LAYOUT FOR THE G·A·C SILT EXCLUSION STUDIES	
1071/60 Drawing	R. Narayanan	
	J·E·R	A·R·O	E·E·R
	Tr: R·R·N·	ckd:	FIG···4

Reg. No. 97-E'62 (Saka Era 1884) 375.

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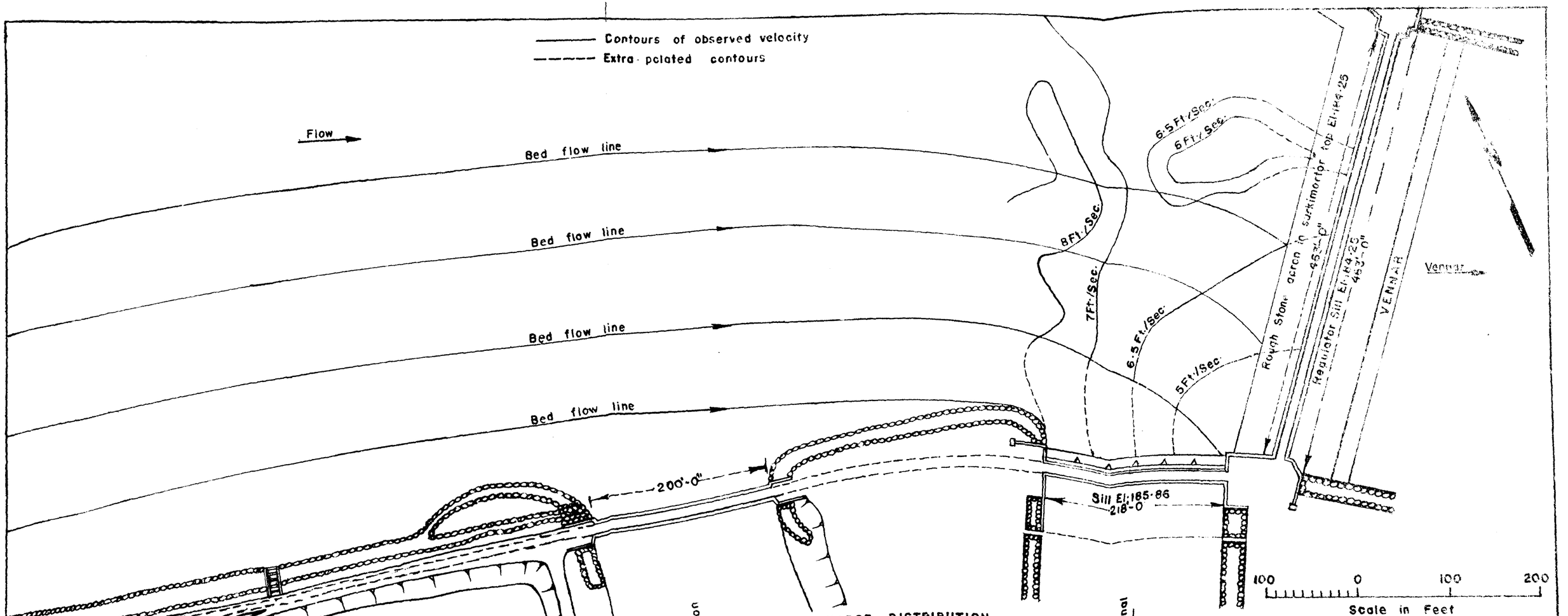

BASIC RUN WITHOUT ANY SILT EXCLUDING DIVICE
SILT DISTRIBUTION

Name	QTY. Coll. Per Hour.	Percent. of Total. Coll.
G.A.C	42. Lbs	76.2%
Vennar	7 1/2 Lbs	13.8%
Cauvery	5 LbS	9.5%
Ullar	1/4 Lbs	0.5%
Total	54 3/4 Lbs	100%

RATE OF SAND INJECTION 50Lbs/Hr.

DISCHARGE DISTRIBUTION

Name	Discharge	Porcen. of dis.
G.A.C	4 000	15.1%
Vennar	9 000	33.9%
Cauvery	12 000	45.3%
Ullar	1500	5.7%
Total. Dis.	26 500	100%

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Reg No.95-5'42(Sa)(a)Ra 1881) 575.

Public Works Department - Madras
Irrigation Research Station - Poondi

BED FLOW LINES AND VELOCITY CONTOURS FOR THE BASIC RUN

109/60			
Drawing			
	J.E.R	A.R.O	B.C.R
	Trd: v.v.	ckd:	FIG 5.

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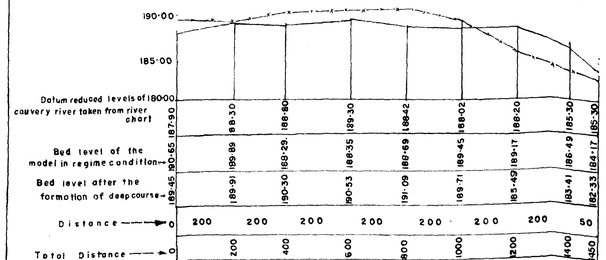

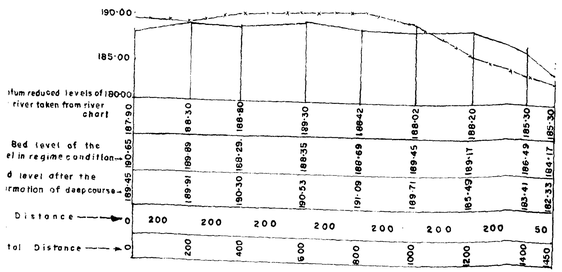

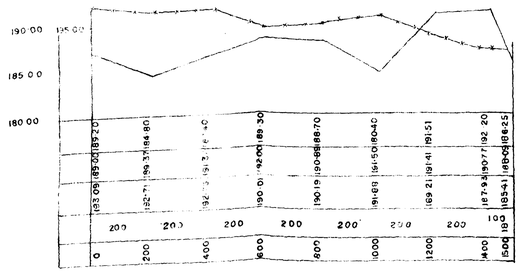

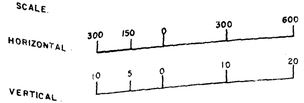

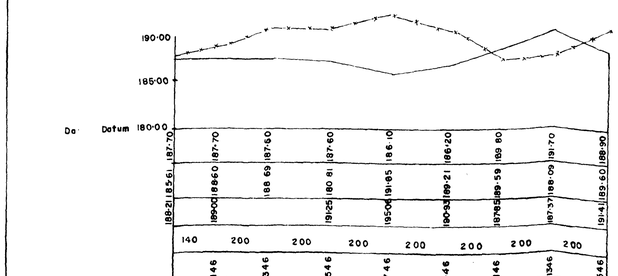

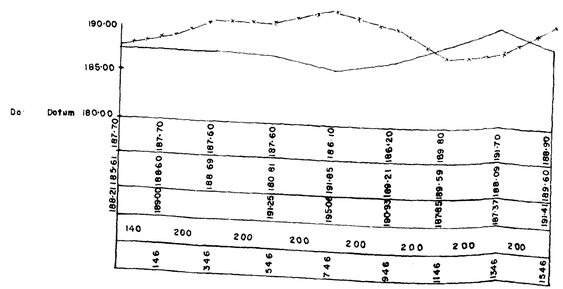

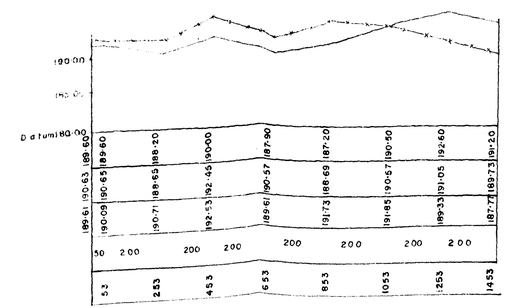

Report	Public Works Department — Madras. Irrigation Research Station — Poondi.
File	CROSS SECTIONS OF THE RIVER CAUVERY ABOVE GRAND ANICUT
	As per river chart after regime run in the model and after formation of the deep course with the divide wall
Drawing	R. Narayanan. U.V.G. Sastri / R.K. Sampathy
	J E R / A R O / E E-R
	Trd: R.R.N. / Ckd: / Fig: / G.A.

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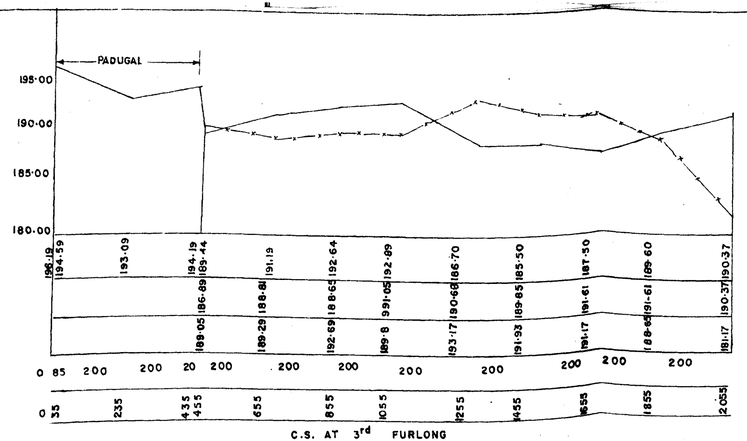
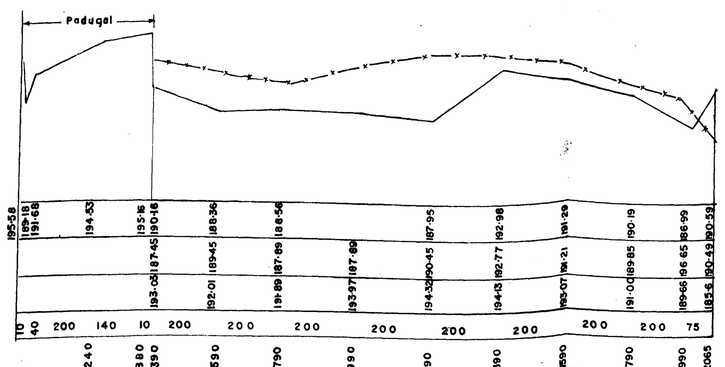
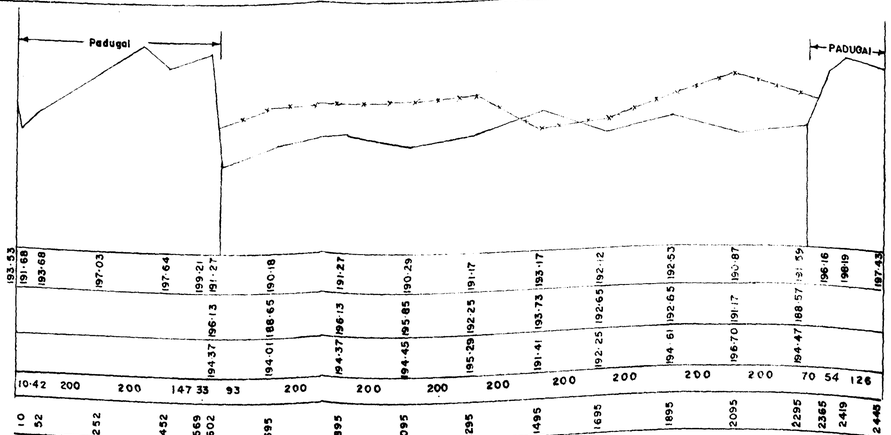


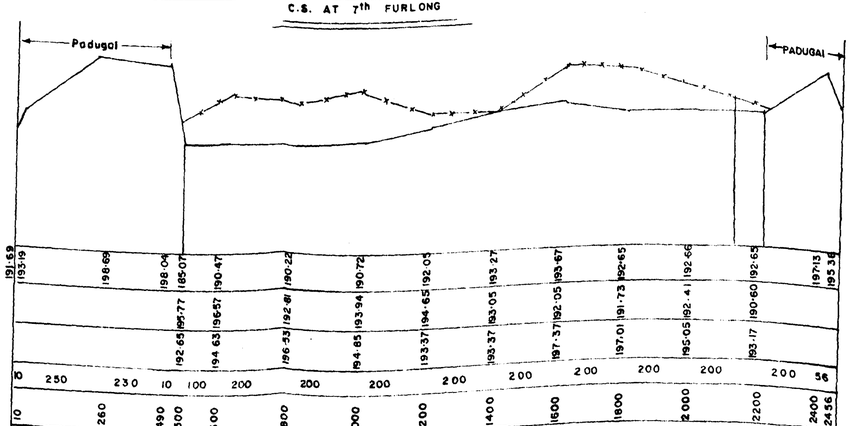
C.S. AT 3rd FURLONG



C.S. AT 5th FURLONG



C.S. AT 7th FURLONG



LEGEND

——— LEVELS OF CAUVERY RIVER ABOVE GRAND ANICUT AS OBTAINED FROM RIVER CHART. 1957–58.

—x—x— LEVELS AS OBTAINED FROM THE MODEL AFTER THE FORMATION OF DEEP COURSE.

SCALE.

HORIZONTAL. 300 150 0 300 600

VERTICAL. 10 5 0 10 20

Report	Public Works Department — Madras. Irrigation Research Station — Poondi.	
File	CROSS SECTIONS OF THE RIVER CAUVERY ABOVE GRAND ANICUT As per river chart after regime run in the model and after formation of the deep course with the divide wall.	
Drawings	R. Narayanan.	
	J. E. R	A. R. O	E. E. R
	Trd: R.R.Nathan.	Ckd	FIG 6-B

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FIG. 7

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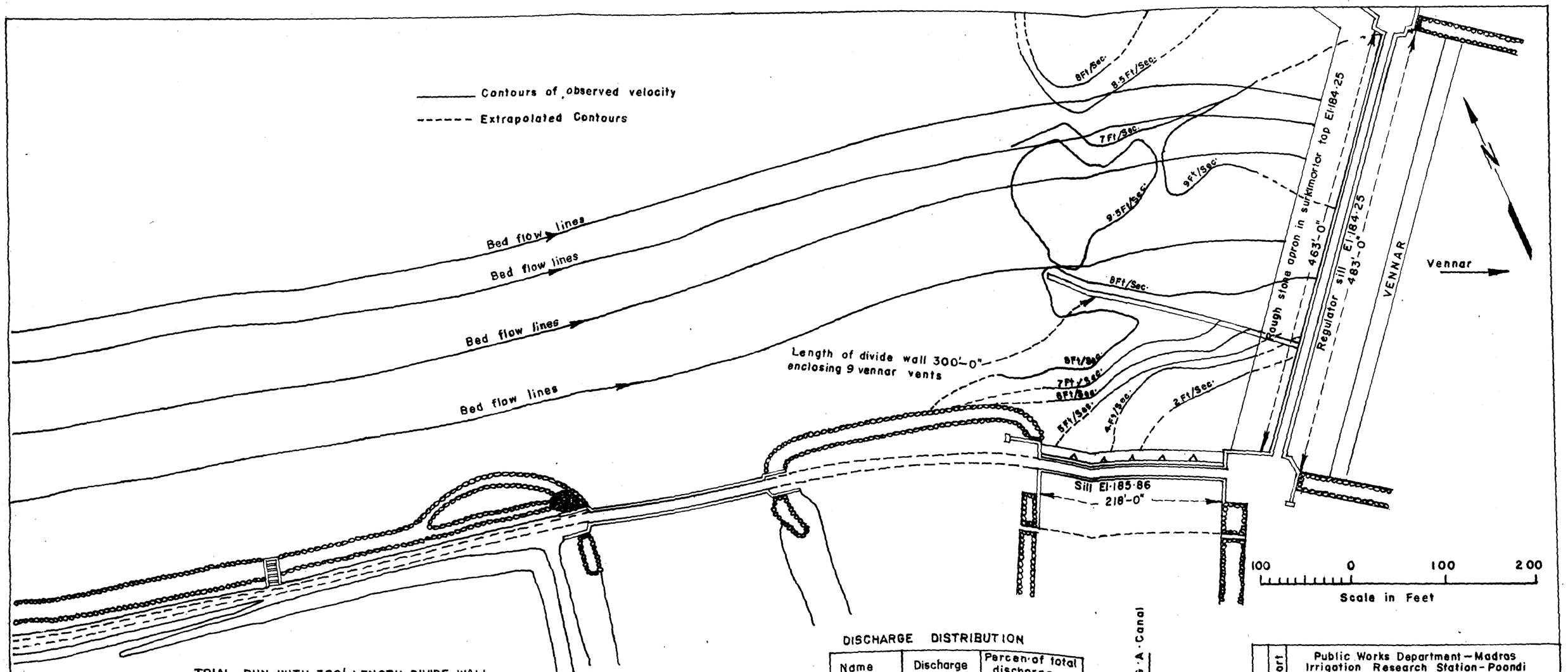

TRIAL RUN WITH 300' LENGTH DIVIDE WALL
RATE OF SAND INJECTION 50Lbs/hr.
SILT DISTRIBUTION

Name	Qty. Coll. Per hour	Percen. of total Coll.
G.A.C	17.75 Lbs	33.6%
Vennar	32 Lbs	60.6%
Cauvery	3.5 Lbs	4.8%
Ullar	0.5 Lbs	1.0%
Total	53.75 Lbs	100%

DISCHARGE DISTRIBUTION

Name	Discharge	Percen. of total discharge
G.A.C	4000 c/s	15%
Vennar	9000 c/s	33.9%
Cauvery	12 000 c/s	45.3%
Ullar	15000 c/s	5.7%
Total	26 500	100%

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Public Works Department — Madras
Irrigation Research Station - Poondi

FLOW LINES AND VELOCITY CONTOURS FOR THE TENTATIVE PROPOSAL OF PLACING THE DIVIDE WALL OF LENGTH 300' ENCLOSING 9 VENTS FROM THE RIGHT END OF THE VENNAR REGULATOR

Report | File | Drawing 107/60.

R. Narayanan. | J.N. Sahe | Ramdoss

J.E.R | A.R.O | E.E.R

Tr: V.V. | chd: | FIG...7

Reg. No. 98-E'62 (Saka Era 1884) 375.

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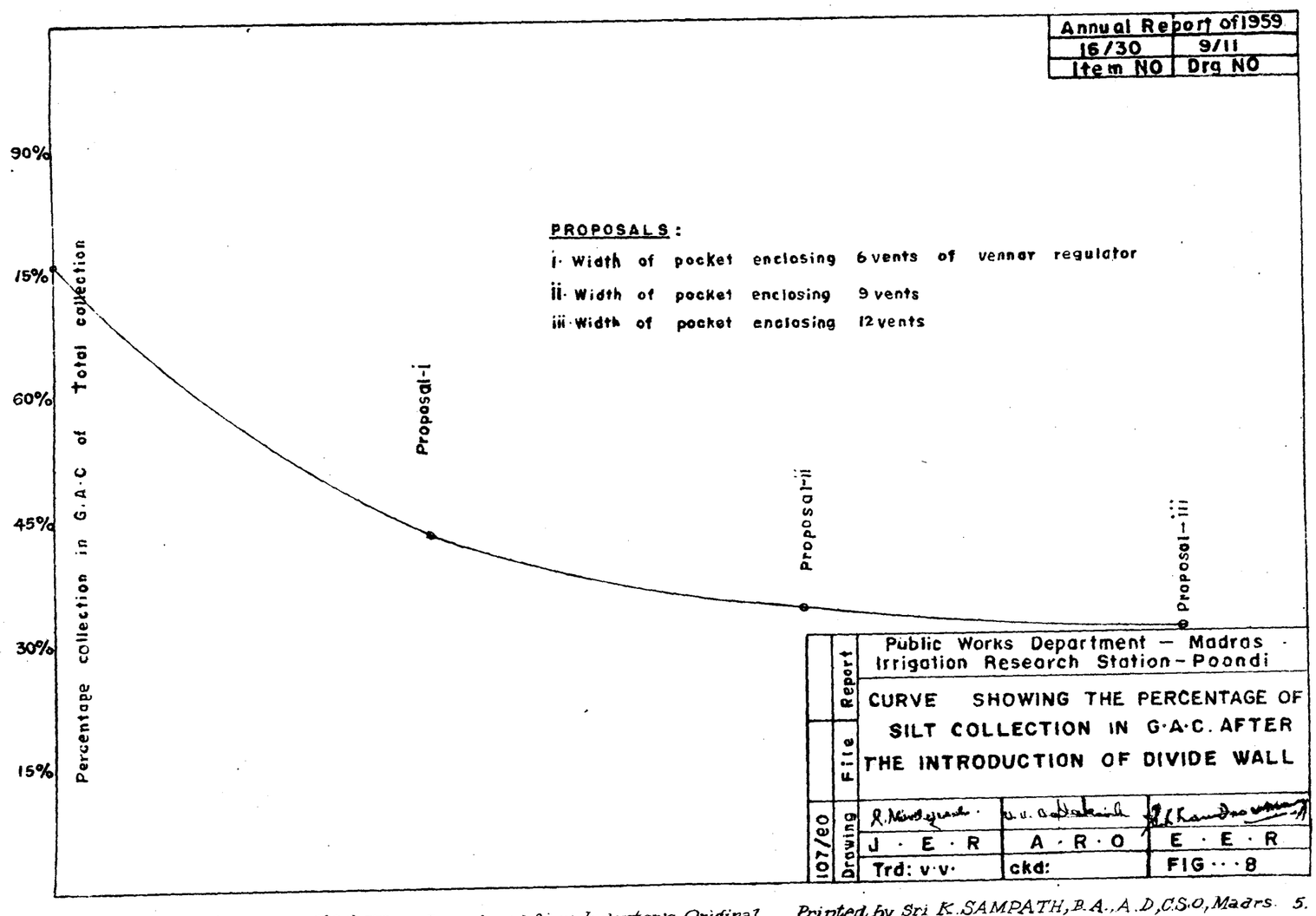

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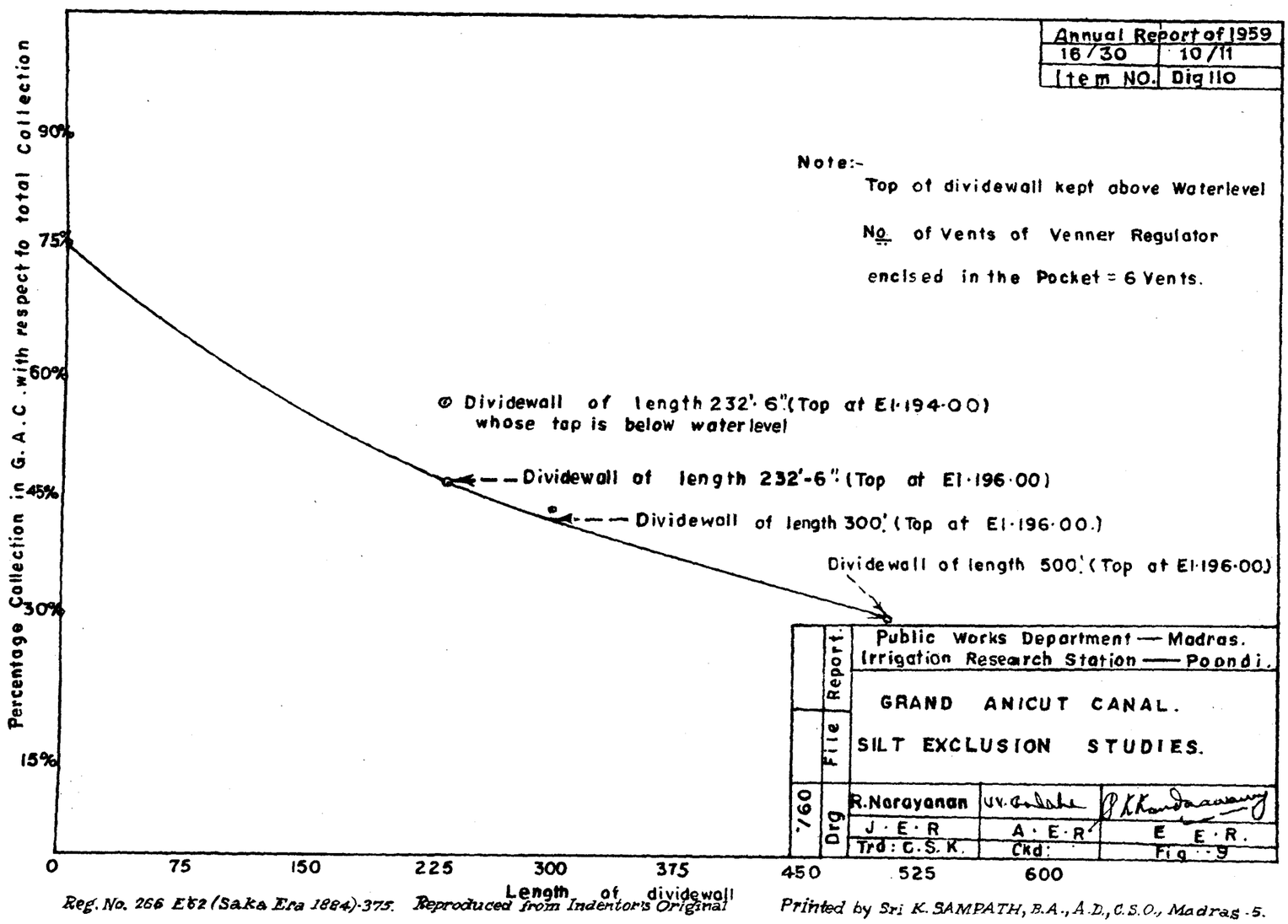

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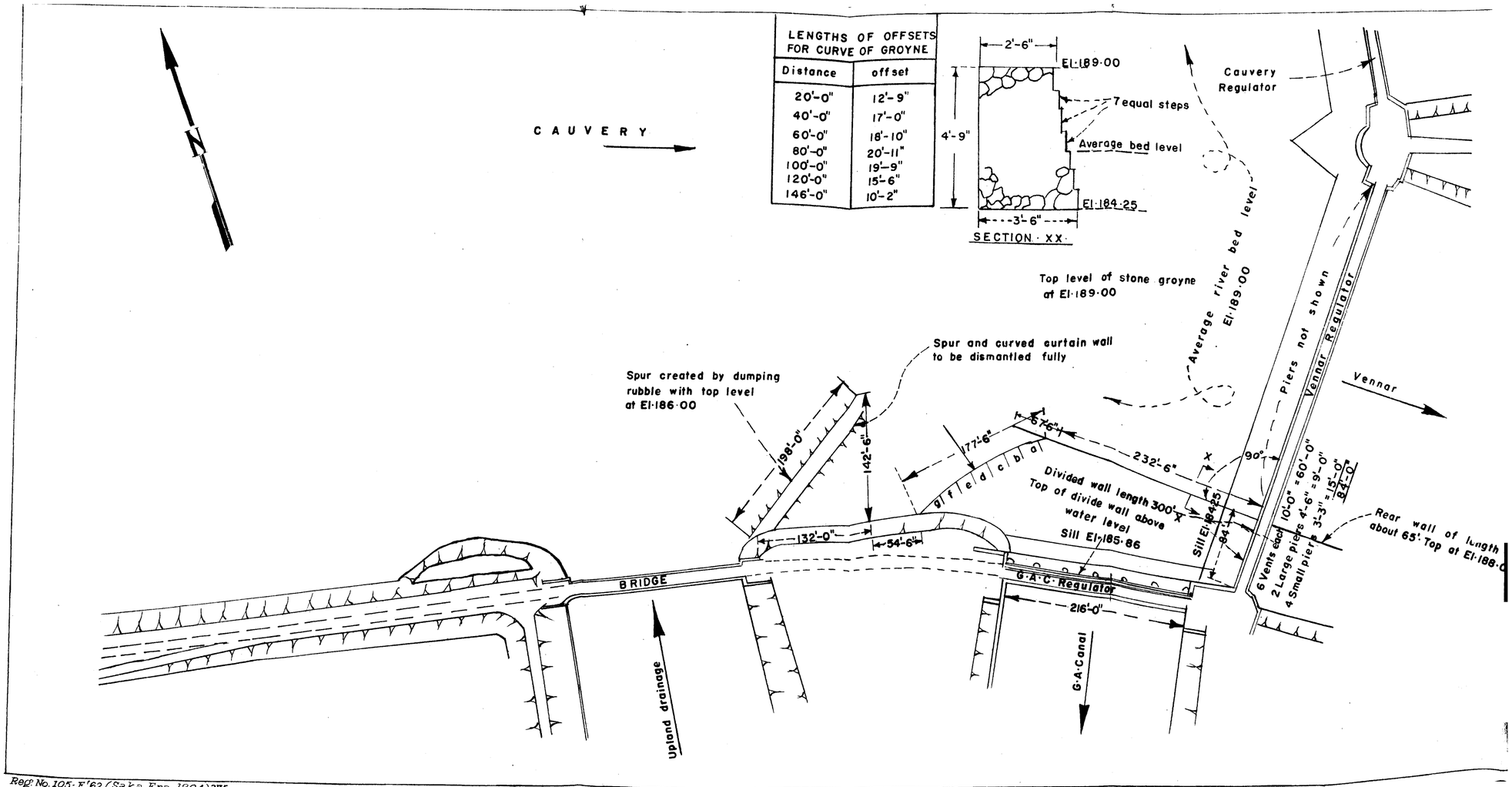

Report	Public Works Department — Madras Irrigation Research Station — Poondi		
File	GRAND ANICUT CANAL SILT EXCLUSION STUDIES FINALISED PROPOSALS		
Drawing	R. Narayanan.	U.V. Sohale	P. Kandasamy
	J · E · R	A · R · O	E · E · R
	Trd: R·R·N·	ckd:	FIG ··· 10.

Reg. No. 105-E'62 (Saka Era 1884) 375

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GRAND ANICUT CANAL—PROPOSAL FOR THE CONSTRUCTION OF AN ADDITIONAL REGULATOR FOR EXCLUSION OF SILT.

With sill of the existing regulator raised to El. 186.86.

PART I.

1. Synopsis.

In this part I, the proposal of raising the sill level of the existing head sluice of Grand Anicut Canal to El. 186.86 have been presented along with the performance of the proposed Additional Regulator to be constructed to compensate for the reduction in the area of flow through the existing head sluice. The performance of the additional regulator for various discharges passed through it has also been discussed. The results have been presented in the form of Statements and graph. Conclusions regarding the efficiency of the proposal have been arrived at.

2. Model.

The same model (part-width) as was used already was made use of for these studies.

The river bed and the bed of the upland drains were moulded in sand passing through No. 14 sieve to conform to the levels furnished in the chart of the Cauvery river for the year 1957-58. As per the suggestion of the Chief Engineer (Irrigation) the sill of the existing Grand Anicut Canal head sluice was raised by 1 foot and kept at El. 186.86. Silt traps were provided downstream of Vennar Regulator, Grand Anicut Canal head sluice and the Additional Regulator so that the quantity of sand passing through each of these three offtakes can be separately collected for the analysis of the results.

3. Studies and results.

Studies were conducted on the same lines as described in Part-II, for the following discharges through the existing head sluice and the balance through the Additional Regulator :

Experiment No.		Discharge through the Additional Regulator.	Dischasge through the existing head sluice.
1	...	30 %	70 %
2	...	50 %	50 %
3	...	75 %	25 %

Observations were made on the performance of the Additional Regulator in reducing the entry of silt into the canal. The percentage collection of sand for various discharges through the additional regulator is shown in Statement-1. A graph is also drawn for percentage of silt drawal in Grand Anicut Canal for various discharges drawn through the Additional Regulator (Figure-3).

4. Discussion.

In the experiments conducted incorporating the proposed Additional Regulator for the cases of drawing 30 % discharge and 50 % discharge through it, a stagnant pool was formed in the upland drain upstream of the new regulator, consequent on the backed up river water rendering the discharge almost silt free. 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. Considerable time was taken for the sand to move and reach the Additional Regulator as seen from the results obtained. (Only 10% to 15% of the total sand collected as seen from the Statement-1). But the upland drain was blanketed with silt to a maximum depth of 5 feet to 6 feet in prototype (vide Figure-1).

It was observed that the proposed Additional Regulator also started contributing to the sand drawal considerably into the canal during the later stage of the experiment.

To study the efficacy of the proposal in the matter of silt reduction, another experiment was conducted under the same conditions of operation with the Additional Regulator drawing 75% of the full supply and the existing regulator drawing the balance. The model was run and drawal of sand into a the canal was observed. The results obtained are tabulated also in Statement-1, and the bed conditions before and after the experiments have been presented in Figure-2.

A comparison of the results shows that there has been no appreciable reduction of silt due to the functioning of the Additional Regulator. 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, expecially 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. Figure-3 show the trend of sand drawal in the G. A. C. for increasing discharges ranging from 30 % to 75 % of the full supply drawn through the Additional Regulator.

In Statement II (a) and (b) the results are shown to reflect the sand in each of the two ways of flow (i.e. through the Grand Anicut Canal head sluice and through the Additional Regulator) for the three sets of combinations of the full supply discharge.

As the discharge increases, there is a corresponding increase in the sand drawal in both off-takes resulting in an over all increase in the sand drawal in Grand Anicut Canal as seen from Figure-3.

As such, the proposed Additional Regulator cannot be said to be efficient in so far as silt reduction is concerned. From the observations, it is seen that the 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 collection in the rear of the Additional Regulator could not be done as perfectly as desired. 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. But it is only the small quantity of sand, remaining in suspension because of the turbulence, that might have escaped the accounting for the Additional Regulator, but it had been included in the total sand collected for the Grand Anicut Canal. Relevent views of the model have been photograped and presented with suitable descriptions given thereon.

5. Conclusion.

As it is seen from the foregoing results that the Additional Regulator drawing either part of the discharge or full discharge, cannot be said to have much effect in reducing the sand that is moving into the Grand Anicut Canal. Hence its performance in reducing the incoming sand into the Grand Anicut Canal seems to be doubtful. But this regulator may serve to supply more discharge at times of need in case the existing regulator is not able to to draw the full supply.

STATEMENT I.

STATEMENT SHOWING THE QUANTITIES AND PERCENTAGE DISTRIBUTION OF SAND IN GRAND ANICUT CANAL.

Discharge through the Additional Regulator.		Percentage sand distribution			
		Vennar.	Grand Anicut Canal.	Additional Regulator.	Total in G.A. Canal.
(1)		(2)	(3)	(4)	(5)
30% discharge	...	57.70 %	31.90 %	30.40 %	48.30 %
50% discharge	...	54.59 %	30.41 %	14.9 %	45.31 %
75% discharge	...	51.37 %	7.52 %	40.61 %	48.13 %

Vide graph Fig. 3.

STATEMENT II.

STATEMENT SHOWING THE PERCENTAGE SILT CHARGE OF THE REGULATORS WITH RESPECT TO DISCHARGE DRAWN THROUGH THEM

The percentages are based on the total sand collected in Grand Anicut Canal including the existing head sluice and the Additional Regulator.

(a) ADDITIONAL REGULATOR.			(b) EXISTING REGULATOR.		
Discharge through the Additional Regulator.		Percentage silt through Additional Regulator.	Discharge through the existing regulator.		Percentage silt in Grand Anicut Canal existing.
(1)		(2)	(1)		(2)
30 % discharge	...	24.5 %	70 % discharge	...	76.5 %
50 % ,,	...	32.89%	50 % ,,	...	67.11%
75 % ,,	...	84.39%	25 % ,,	...	15.61%

SHOWING THE NATURE OF BED AT THE END OF STUDY FOR 30% DISCHARGE THROUGH THE ADDITIONAL REGULATOR FOR THE RAISED SILL IN GRAND ANICUT CANAL HEAD SLUICE

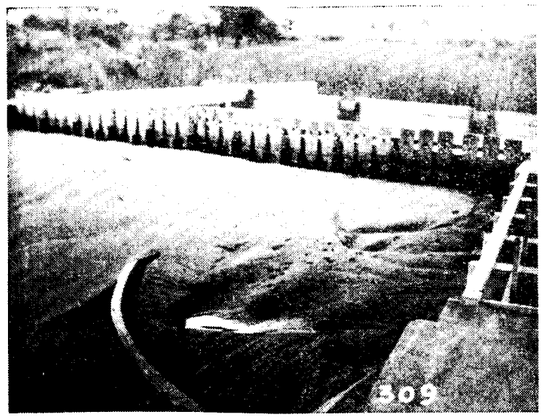


PHOTO No. 1
Up stream of existing Grand Anicut Canal Head Sluice

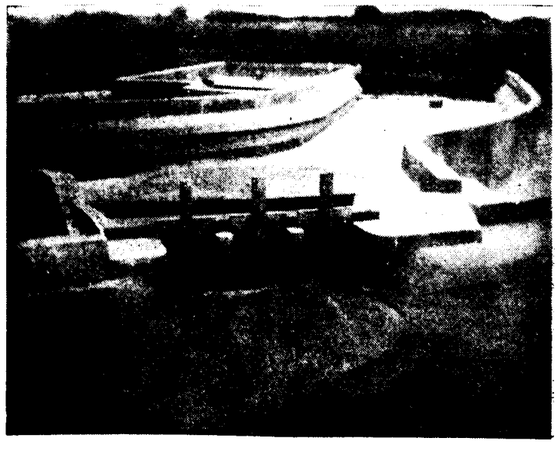


PHOTO No. 2
Near the proposed Additional Regulator

RUNNING CONDITION OF THE MODEL FOR 75% DISCHARGE THROUGH THE ADDITIONAL REGULATOR

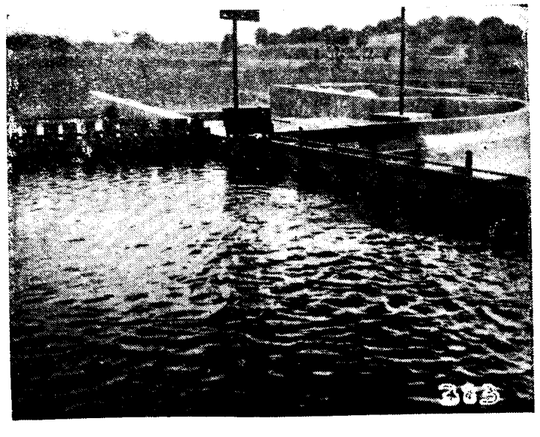


PHOTO No. 3

Up sterem of Grand Anicut Canal and Vennar Regulator

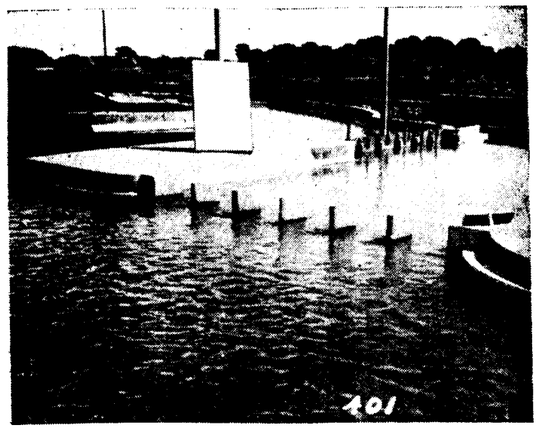


PHOTO No. 4

Near the Additional Regulator

PHOTOS SHOWING THE CONDITION OF MODEL AT THE END OF STUDY FOR 75 % DISCHARGED THROUGH THE ADDITIONAL REGULATOR

PHOTO No. 5
Additional Regulator in the Upland Drain

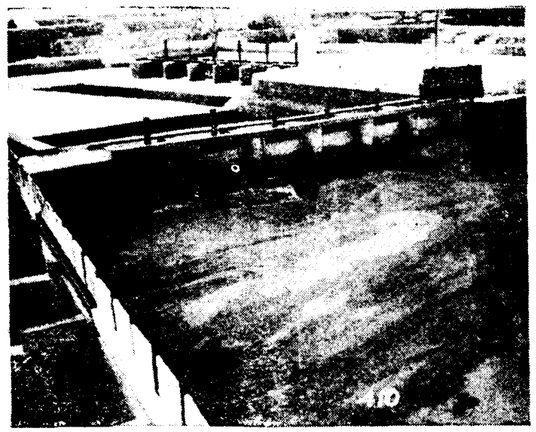


PHOTO No. 6
Existing Regulator and Vennar Regulator

GRAND ANICUT CANAL

PROPOSAL FOR THE CONSTRUCTION OF AN ADDITIONAL HEAD SLUICE FOR EXCLUSION OF SILT

(With sill of the existing regulator at El. 185.86)

Part—II

SYNOPSIS

This report deals with the problem of silt exclusion from the Grand Anicut Canal by way of constructing an additional head sluice with sill level kept at El. 187.86 on the common bank of the upland drain and the existing Grand Anicut Canal, so that a part of discharge will be drawn through the proposed additional regulator and the balance through the existing Grand Anicut Canal head sluice, the sill level of which is proposed to be raised by 1 foot to mitigate the quantity of bed load moving into the Grand Anicut Canal.

1. Introduction.

The Grand Anicut which is situated about 12 miles from Tiruchirapalli is an old irrigation structure across Ullar and is a prehistoric breach on the left margin of Cauvery river joining Coleroon some distance downstream from the said Anicut. At the site of the anicut, the river Cauvery divides into Cauvery, Vennar and Ullar. There are regulators across Cauvery and Vennar. After the construction of Mettur Dam, Grand Anicut Canal was taken off from the right bank of Cauvery river at the Grand Anicut site.

2. Hydralic Particulars.

		Existing G. A. Canal Headsluice	Proposed Additional Headsluice
1. Sill level	...	El. 185.86	El. 187.88
2. F. S. L.	...	El. 192.91	El. 192.91
3. Bed width	...	180 feet	110 feet
4. Bed fall	...	0·15/1000	0.15/1000
5. Existing discharge	...	4300 c/s	...
6. Existing size of vents	...	6 vents 30′x5′4″	...

3. Problem.

The bed of the Grand Anicut Canal in rear of the existing head sluice is blanketed with silt to an average depth of 3 to 4 ft. with the result that the present head sluice is not able to draw the required full supp y of 4300

cusecs. The Chief Engineer (Irrigation) has suggested that an additional head sluice may be considered on the right bank of the upland drain so that the backed up waters in the upland drain can be drawn to the Grand Anicut Canal through the proposed head sluice (Figure-1). It was felt that the supply thus drawn from the upland drain may not contain much of silt and it was decided to study in the model experiments the approximate silt reduction that can be attained by drawing (1) a discharge of 2150 cusecs (50% cusecs of the total discharge) through the additional regulator and the rest of the supply through the existing regulator and (2) full discharge through the additional regulator after closing the existing regulator. The latter suggestion was made based on the line of thinking that the accrued benefit of silt reduction if any for 30% discharge drawal through the additional regulator may be extended to the extreme case of drawing the entire supply through the proposed additional regulator (vide C.E.I's inspection notes dated 20-3-'59).

4. Model

A part width geometrically similar model to a scale of 1/24 was put up representing the right bank and a portion of the Cauvery river upstream of the anicut, same length of the upland drain, the Grand Anicut Canal and the head sluice. The river bed was moulded in sand passing through No. 14 sieve.

A rectangular weir was provided at the upstream and of the model for measuring the discharges 1st 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 each. Silt traps were provided downstream of Vennar regulator, G. A. 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.

Note:—As both the alternatives suggested by the Chief Engineer (Irrigation) namely 3 vents of 20 feet each and 2 vents of 30 feet each have the same total linear water way of 60 feet the studies were restricted to the first alternative (3 vents of 20 feet each) and the results obtained were taken as representative of both the alternatives. For the case of closing tne existing head sluice and sending the entire discharge of 4300 cusecs through the additional regulator, the required number of vents has been computed and fixed as 6 numbers of 34'0" each as shown in Figure-1.

5. Experiments and Results

Two experiments were conducted to study the effect of drawing partial and full discharges through the additional regulator. Through the additional regulator, 50% of the total discharge (i.e. 2150 cusecs) was allowed and the balance through the existing Grand Anicut Canal head sluice.

The canal water level upstream of the regulator (i.e.) in the Cauvery river was maintained at 196.11. As per the details received from the field the maximum discharge passed through the Vennar Regulator was 12,000 c/s and that in the Grand Anicut Canal was 4300 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 192.91 which

is the full supply level of the canal. The water level in Vennar was maintained at 188.28. Sand of the same size used for moulding the bed was injected at the rate of 1 cft/hour. The model was run under these conditions for 438 hours at the rate of 12 hours per day. The quantities of sand passed through Vennar and Grand Anicut canal at a place downstream of the additional regulator were collected at the end of each day and measured. The results presented here pertains to only that period during which the model attained steady conditions,

The following table gives the quantities and percentages of sand collected in the above case :

	Quantity of Sand Collected.	Percentage of Sand Collected.
Vennar ...	78.55 Cft.	66.5 %
Grand Anicut Canal and the Additional Regulator ...	39.67 Cft.	33.5 %

The quantity of sand passed through the proposed additional regulator into Grand Anicut Canal was separately collected and measured. It was only 3% of the total sand in Grand Anicut Canal collected at a place downstream of the Additional Regulator. From this it is clear that the silt charge in the flow drawn through the additional Regulator is less concentrated than the silt charge in the flow drawn through the Grand Anicut Canal head sluice. Therefore, it was propose to send the entire discharge of 4300 cusecs through the Additional Regulator so that it may perhaps result in considerable reduction in the sand drawal in Grand Anicut Canal ultimately.

The next experiment was carried out, drawing the full discharge of 4300 cusecs through the Additional Regulator alone after closing the existing regulator completely. The rest of the conditions such as upstream and downstream water levels, discharge through Vennar etc., remaining the same, only the size and member of vents provided for the Additional Regulator were altered to allow the full discharge (i.e. 6 vents of 34' x 5'). The model was run for 266 hours.

The table below gives the quantities and percentages of sand collected for the steady conditions obtained in the model

	Quantity of sand Collected.	Percentage of sand Collected.
Vennar ...	21.125 Cft.	45.31 %
Grand Anicut Canal ...	25.5 Cft.	54.69 %

6. Discussion on Results

In the first experiment conducted with the proposed Additional Regulator drawing a discharge of 2150 cusecs, a stagnant pool was formed in the upland drain upstream of the new regulator, because of the backed-up water and the discharge was almost silt free. But the existing head sluice of

Grand Anicut Canal began to draw and almost immediately after the experiment was started. After some hours of run sand was seen entering the upland drain slowly. Considerable time was taken for the sand to move and reach the Additional Regulator. Very little quantity of sand passed through the Additional Regulator as seem from the results obtained (only 3% of the total sand collected. But the upland drain was blanketed with silt to a considerable level corresponding to about 7 feet depth in prototype).

It was observed that the Additional Regulator also started contributing to the sand drawal into the canal during the later stage of the experiment. To study 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 drawing the full discharge of 4300 cusecs and the existing regulator completely closed. The model was run, and drawal of sand into the canal was observed.

The results obtained are tabulated in the enclosed statement showing the quantities and percentages distribution of sand to Grand Anicut Canal. These results have been presented in Figure-2 in the form of a graph.

A comparison of the results shows that there has been no appreciable reduction of silt due to the introduction of the additional regulator. 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 limitation, espesially 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.

As such the proposed additional regulator cannot be said to be efficient in so far as silt reduction is concerned.

From the observations it is seen that the 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 desired. 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 turbulence 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 junction of the Grand Anicut Canal. But it is only the small quantity of sand remained in suspension because of the turbulence, that might have escaped the accounting for the Additional Regulator, but it had been included in the total sand collected for the Grand Anicut Canal.

7. Conclusion.

As it is seen from the foregoing results that the Additional Regulator drawing either part of the discharge or full discharge does not seem to have much effect in reducing the sand that is moving into the Grand Anicut Canal. Hence its performance in reducing the incoming sand into the Grand Anicut Canal appears to be doubtful. But this regulator may serve to supply more discharges at times of need in case the existing regulator is not able to draw the full discharge.

STATEMENT SHOWING THE QUANTITIES AND PERCENTAGES DISTRIBUTION OF SAND IN GRAND ANICUT CANAL.

Name of river or channel.	Without the Additional Regulator functioning (Full discharges through existing regulator)		With the Additional Regulator drawing 1273 cusecs (30 discharge)		With the Additional Regulator drawing 2150 cusecs (30% discharge)		Without the existing regulator functioning (Full discharge through Additional Regulator)	
	Quantity of sand collected	Percentage of sand collected	Quantity of sand collected	Percentage of sand collected	Quantity of sand collected	Percentage of sand collected	Quantity of sand collected	Percentage of sand collected
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
VENNAR ...	77 Cft.	58%	85 Cft.	63%	78.55 Cft.	66.5%	21.125 Cft.	45.31%
GRAND ANICUT CANAL ...	55 Cft.	42%	50 Cft.	37%	39.67 Cft.	33.5%	25.5 Cft.	54.69%

Column (2) and (3) & (4) and (5) vide I. R. S. Report No. 12/59 Pages 3 & 4.

GRAND ANICUT CANAL

SYPHON AQUEDUCT—PROVIDING SAND VENTS

Part—III

1. Synopsis

Studies were conducted with a view to observe the performance of the sand vents and the curtain wall proposed for construction in Grand Anicut Canal at R. D. 2/34 for diverting a portion of the bed sand, which the canal carried from its upper reaches. A brief note on the problem has been given. The experimental set up has been explained and the results of the experiments are furnished. Conclusions arrived at based on these results are not encouraging for adopting the proposed construction at the site.

2. Introduction

The Chief Engineer (Irrigation) during the inspection of the Grand Anicut Canal suggested that the Syphon Aqueduct at R.D. 2/34 may be taken advantage of for constructing two vents in the left parapet wall of the aqueduct and a curved curtain wall in the bed to guide towards them, the silt carried by the Grand Anicut Canal, from its upper reaches.

The proposal envisages flushing of bed sand diverted by the curved curtain wall into the vents and thereon into the drain below which ultimately sill hoin Vennar within a short distance of its run. The quantity of discharge required for this flushing operation is preposed to be drawn in Grand Anicut Canal and ultimately let in Vennar through the drain at the aqueduct site. This proposal was thought of perhaps because the water levels in Grand Anicut Canal and Vennar at this site will permit this flushing operation through the sand vents. Also it was expected that the curved curtain wall will divert the bed sand into the sand vent effectively for successful flushing operation. Figure-1 shows the proposals as received from the field. It was required to test the effectiveness of this proposal by model studies at Poondi.

Accordingly, model studies for finding out the efficacy of the device suggested by the Chief Engineer (Irrigation) were conducted in a geometrically similar model and the results have been presented in this report.

3. Problem

It is required to determine from experiments, the best arrangement for optimum flushing operation of sand, with least obstruction to waterway in Grand Anicut Canal (i.e.) the dimensions and shape of the curtain wall must be such that it guides the bed load towards the sand vents, with minimum heading up of water in Grand Anicut Canal.

4. Data

Full supply discharge	...	4300 cusecs
Width of canal over the syphon	...	120 feet
Canal bed level	...	El. 182.32
Canal full supply level	...	El. 189.32
Size and number of sand vents 2 Nos.	...	8'0" x 1'0"
Sill level of vents	...	El. 182.32

5. Model

A full width model was constructed to a scale of 1/12 with a mobile bed moulded with sand of size passing through 14 sieve and retained on 18 sieve. The Grand Anicut Canal was simulated in the model for a length of about 150 feet upstream of the syphon aqueduct. This portion was constructed to a constant bed width of 180 feet (Prototype dimension) and the mobile bed was moulded with sand giving a suitable longitudinal slope.

The suphon portion and the two vents in the left parapet wall was constructed in masonry and plastered smooth with cement. Shutters were prepared in glavanised iron sheets and inserted in the grooves provided in the masonry. A curtain wall 2 feet high 1½ feet wide curved in plan as shown in Figure-2 was constructed in neat cement. Sand traps for collecting sand, arrangements for maintaining tail water levels in Grand Anicut Canal and V-notch to measure the discharge passing down the sand vents, were provided in the model.

6. Experiments

The maximum discharge of 4360 cusecs was allowed in Grand Anicut Canal maintaining the full supply depth of 7'0". The two sand vents were kept open by one foot. Discharge corresponding to the full supply and 50% of the supply were allowed in the model and the regime rate of sand injection in each was arrived at after a few trials. The distribution of silt between the sand vents and the canal and flow lines (bed and surface) for 100% and 50% of the sull supply discharge in the canal were observed for each of the two different heights of the curtain wall tried in the model (namely 1 foot and 2 feet height). Vide Figures-2 through 8.

7. Results and Discussion

The results of the observation are tabulated in the Statement below. The percentage of silt passing down the sand vents is found to be low as seen from the Statement.

STATEMENT

Discharge in canal		Percentage distribution of sand		Remarks.
		Grand Anicut Canal	Sand Vents	
Full supply discharge	...	98.8%	1.2%	
½ the full supply discharge	...	100 %	...	

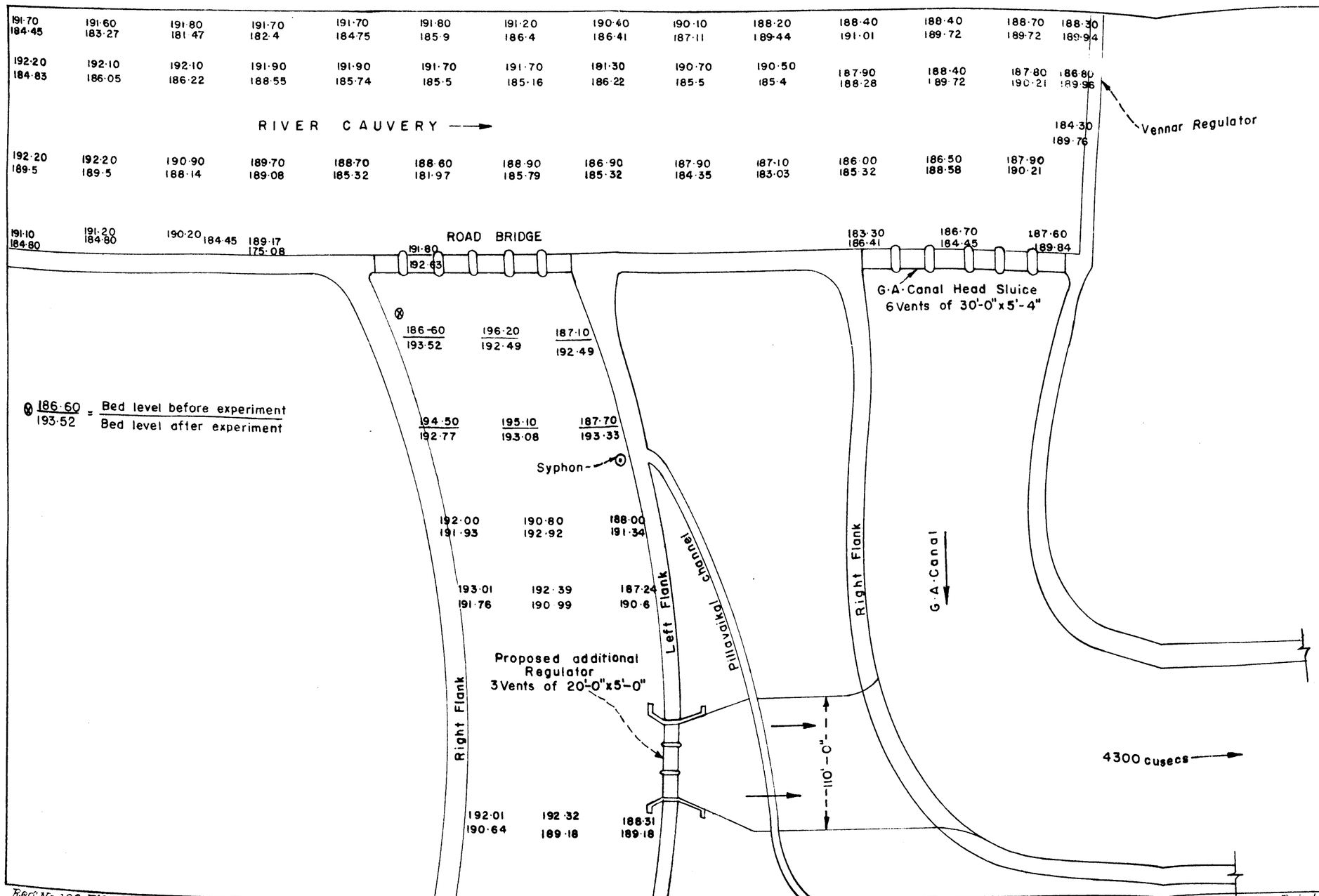
It is seen from Figures 2 through 8 that the flow lines entering the sand vents are very few or none at all as compared with the large number of them passing over the curtain wall. This has been observed to be so for both heights of the curved wall tried in the model. As the flow lines surve as an indication of the extent of bed sand movement one way or the other, fewer they are, the less will be the quantity of sand movement in the corresponding direction. The distribution of these flow lines shown in the Figures clearly brings out the reason for the begligible beneficial effect of the sand vents in flushing the bed sand.

Another important point is the turbulance caused by the introduction of the curtain wall in the zone of movement of the bed sand. As a result of the turbulance created, the sand particles are lifted up from the bed and kept in suspension only to jump over the curtain wall and not to get diverted by it, as is erroneously expected. This provides the reason as to why the combination of the curtain wall and the sand vents will not be an effective measure to get rid of even a portion of the bed sand moving along the Grand Anicut Canal bed.

8. Conclusion

From the results it is seen that only a negligible amonnt of silt is being flushed through the sand vents, by the curved curtain wall. Hence it is not advantageous to provide sand vents in the syphon aqueduct for the purpose of flushing the bed sand from Grand Anicut Canal.

Annual Report of 1969	
17/30	1/10
Item No.	Drg. No.



Discharge through additional Regulator	Percentage Sand reduction		
	Vennar	G·A·C + add·Regulator	Total G·A·C
30% of full Supply	57·70 %	31·90% +10·40%	42·30 %
50% of full Supply	54·69 %	30·41 % +14·9%	45·31 %

100 0 100 200
Scale In Feet

Report	Public Works Department – Madras Irrigation Research Station – Poondi		
File	G·A·CANAL AND THE UPLAND DRAIN WITH THE PROPOSED ADDL REGULATOR DRAWING 30% AND 50% OF THE FULL SUPPLY		
80/60 Drawing	J·E·R	A·E·R	E·E·R
	Trd: v·v·	ckd:	FIG··· 1

Reg. No. 106 E'62 (Saka Era 1884) 375

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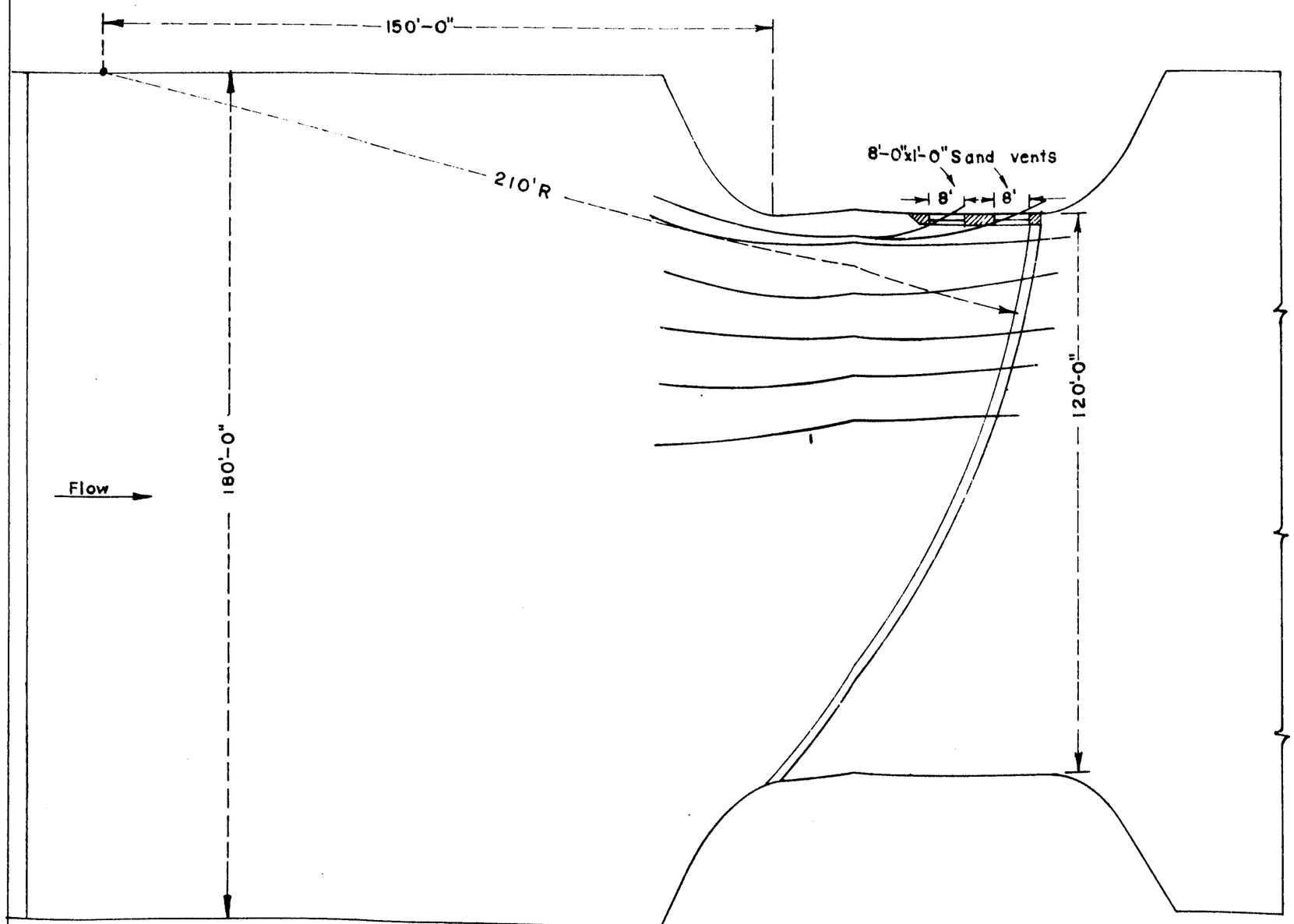

Sand vents	2Nos. of 8'-0"x1'-0"
Bed level of canal	182·32
Sill level of sand vent	182·32
Full supply level	189·32
Canal discharge Max.	300 cfs.
Height of curtain wall	2 Feet
Width of curtain wall	1'-6"

25 0 25 50
Scale in Feet

Public Works Department=Madras
Irrigation Research Station—Poondi

PROVIDING SAND VENTS TO THE SYRHON AT RD 2/34 OF G.A. CANAL SURFACE FLOW FOR MAX. DISCHARGE

J.E.R	A.E.R	E.E.R
Trd: V.V.	ckd:	FIG...2

78/60 Drawing File Report

Reg. No. 99-E'62 (Saka Era 1884) 375.

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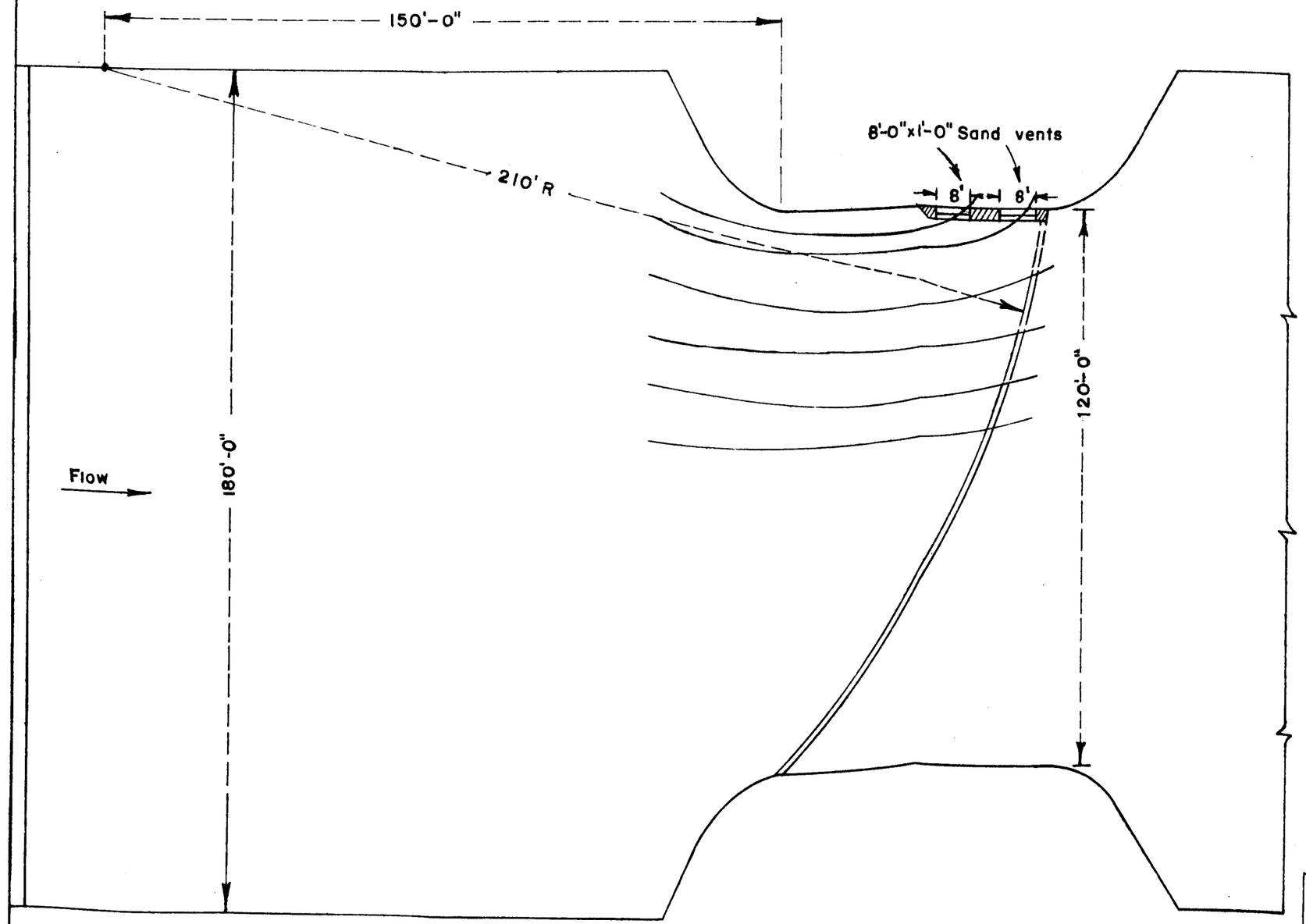

Sand vents	2 Nos. of 8'-0"x1'-0"
Bed level of canal	182.32
Sill level of sand vent	182.32
Full supply level	189.32
Canal discharge Max.	4300 cts.
Height of curtain wall	2 Feet
Width of curtain wall	1'-6"

25 0 25 50

Scale in Feet

Public Works Department - Madras
Irrigation Research Station - Poondi.

PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G.A.CANAL

BED FLOW LINES. MAX. DISCHARGE

Report			
File			
82/60 Drawing	M.Devadaja		P.K.Kan Sao
	J.E.R	A.E.R	E E R
	Trd: v.v.	ckd: —	EIG... 3

Reg. No. 100-E'62 (Saka Era 1884) 375

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17/30	4/10
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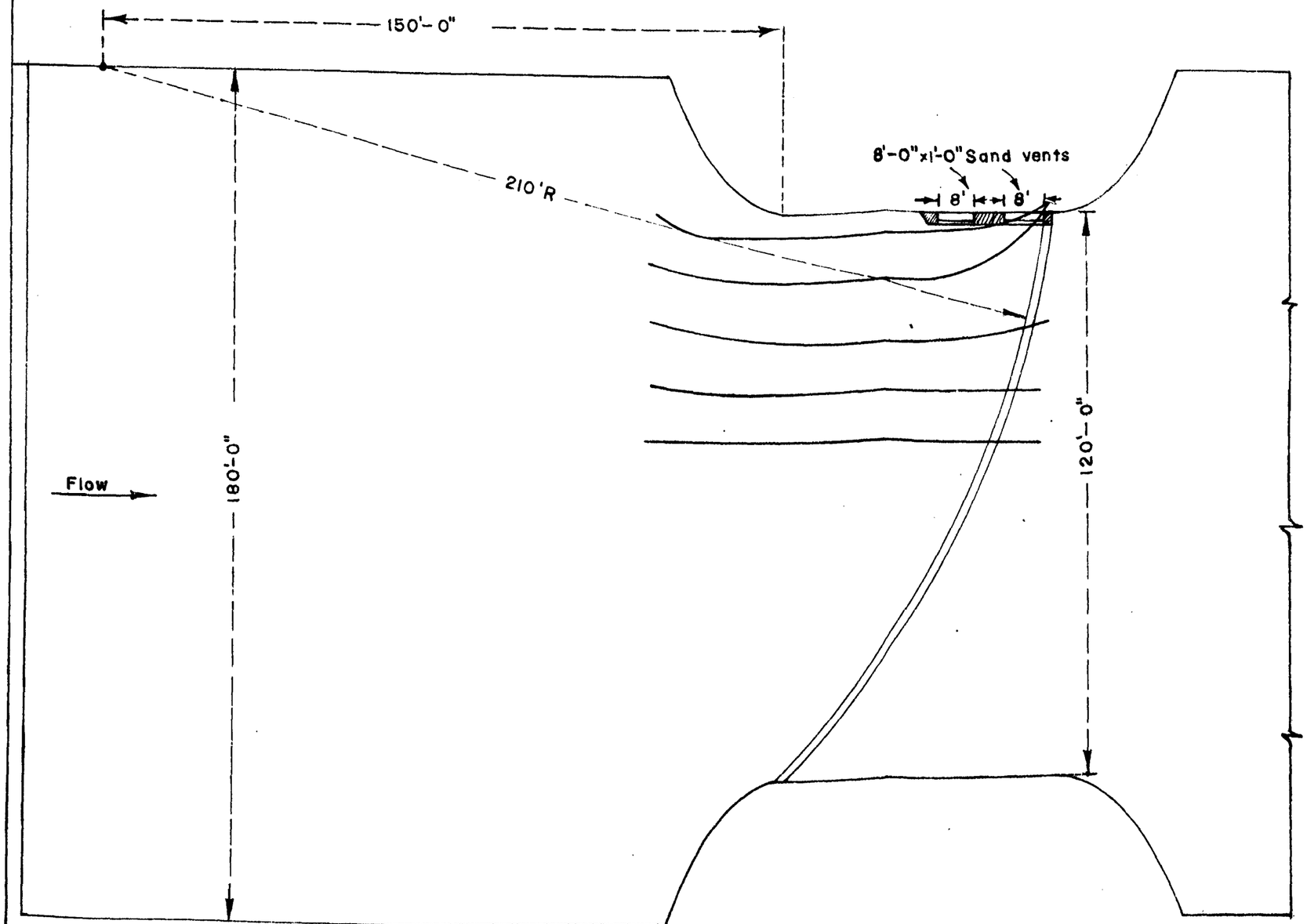

Sand vents	2 Nos. of 8'-0" x 1'-0"
Bed level of canal	182·32
Sill level of sand vent	182·32
Full supply level	189·32
Canal discharge Max.	4300 cfs.
Height of curtain wall	2 Feet
Width of curtain wall	1'-6"

25 0 25 50

Scale in Feet

Public Works Department — Madras
Irrigation Research Station — Poondi

PROVIDING SAND VENTS TO THE SYPHON AT RD 2/4 OF G·A·CANAL
SURFACE FLOW LINES. HALF MAX. DISCHARGE

74/60	Drawing	File	Report
J·E·R	A·E·R	E·E·R	
Trd: V·V·	ckd:	FIG···4	

Reg. No. 109·E'62 (Saka Era 1884) -375.

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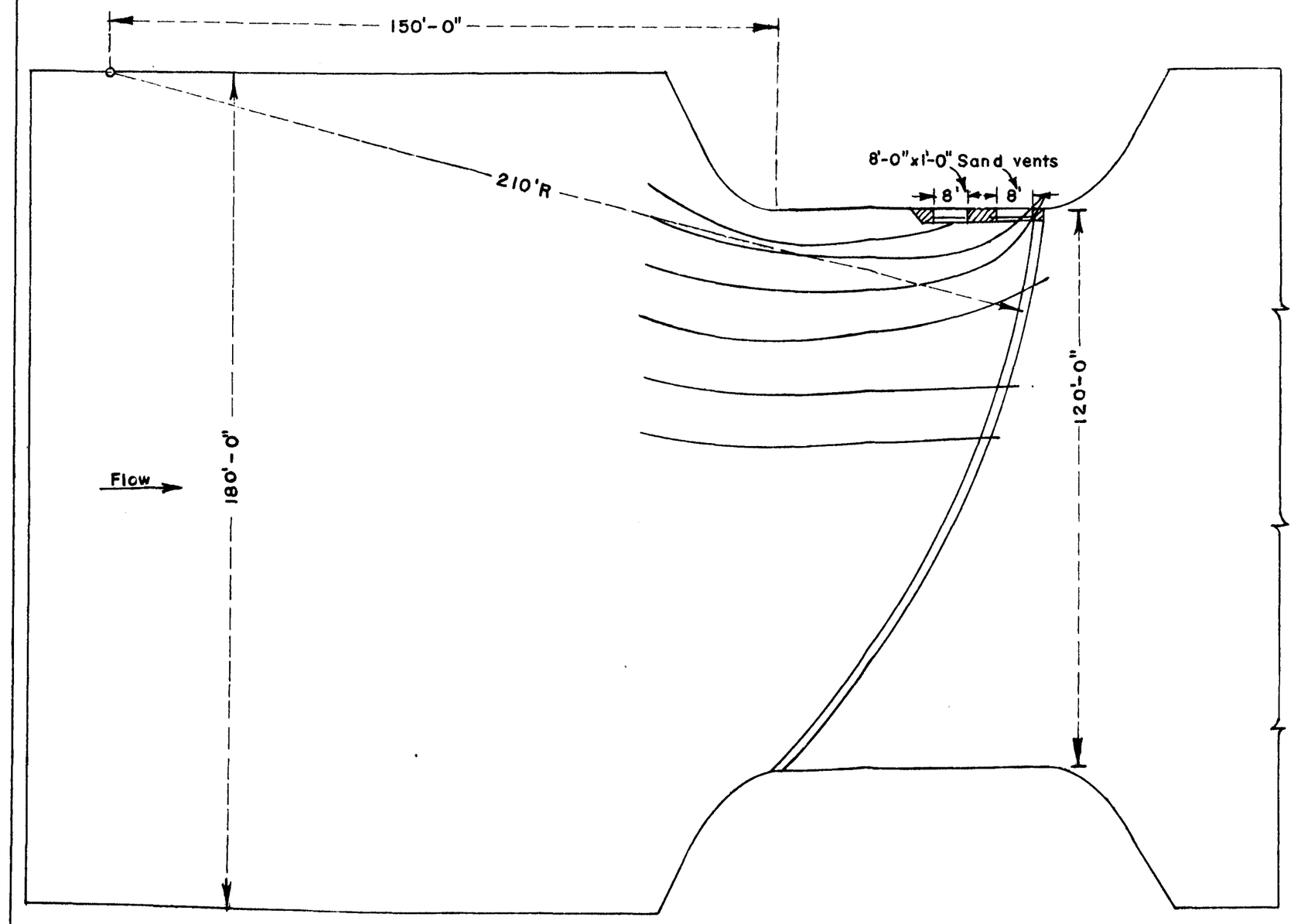
17/30	5/10
Item NO	Drawing NO



Sand vents	2Nos. of 8'-0"x1'-0"
Bed level of canal	182·32
Sill level of sand vent	182·32
Full supply level	189·32
Canal discharge Max.	4300 cts.
Height of curtain wall	2 Feet
Width of curtain wall	1'-6"

25 0 25 50

Scale in Feet

Public Works Department–Madras
Irrigation Research Station – Poondi.

PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G.A.Canal
BED FLOW LINES. HALF MAX. DISCHARGE

Report	File	75/60 Drawing		
		J. E. R	A. E. R,	E. E. R
		Trd: V.V.	ckd:	FIG ... 5

Reg. No. 110-E'62 (Saka Era 1884)-375.

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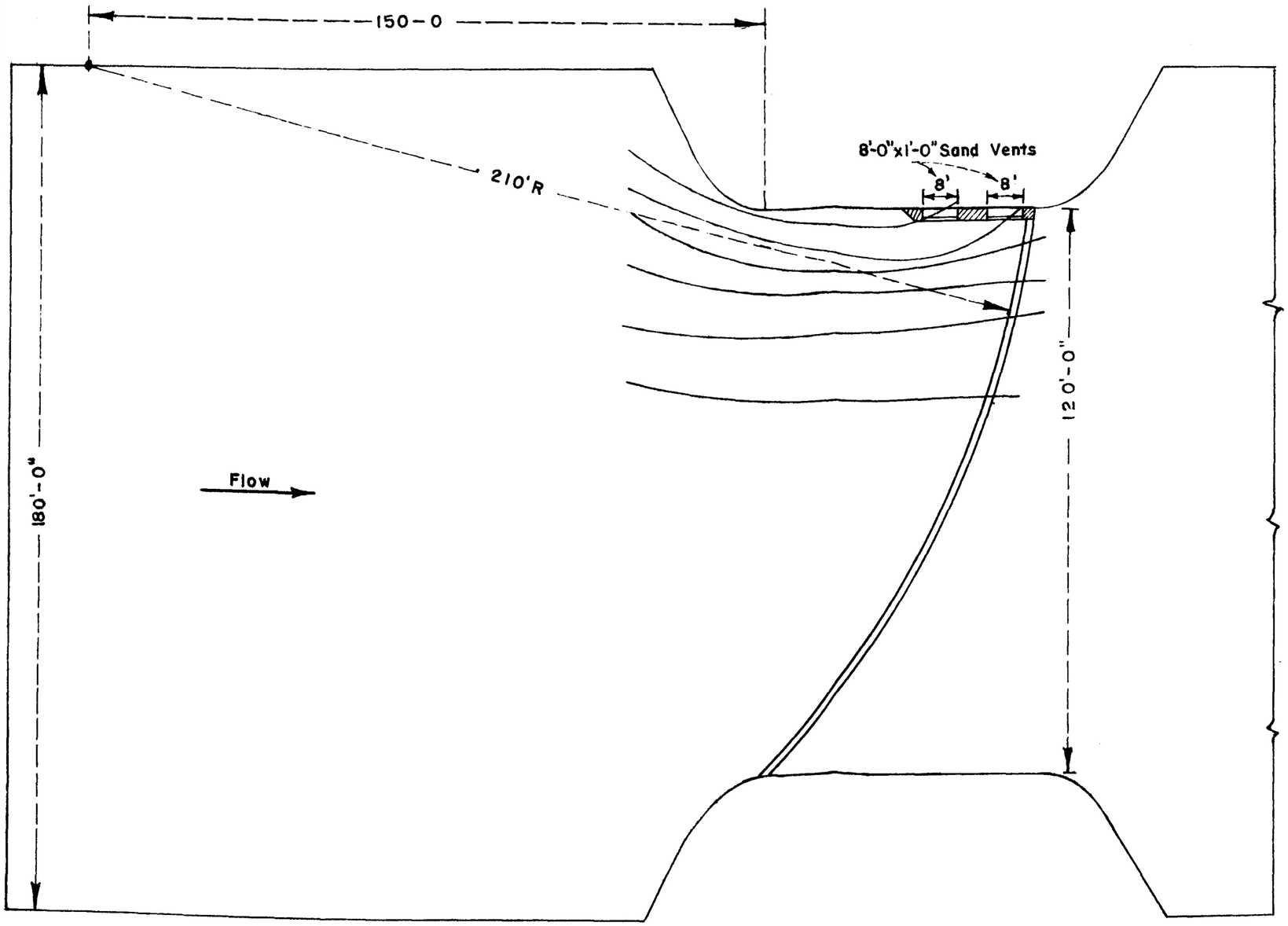
17/30	6/10
Item No.	Drg. No.



Sand Vents	2 Nos. of 8'-0"x1'-0"
Bed level of canal.	182.32
Sill level of sand vent	182.32.
Full supply level	189.32
Canal discharge Max.	4300cfs.
Height of curtain wall	1 Foot
Width of curtain wall	1'-6"

25 0 25 50
Scale in Feet

Public Works Department-Madras
Irrigation Research Station-Poondi

PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G.A. CANAL BED FLOW LINES, FOR MAX. DISHARGE

70/60			
Drawing	File	Report	
			Public Works Department-Madras Irrigation Research Station-Poondi
			PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G.A. CANAL BED FLOW LINES, FOR MAX. DISHARGE
			J E R / A.E.R / E.E.R
			Trd: V.V.I / ckd: / FIG...6

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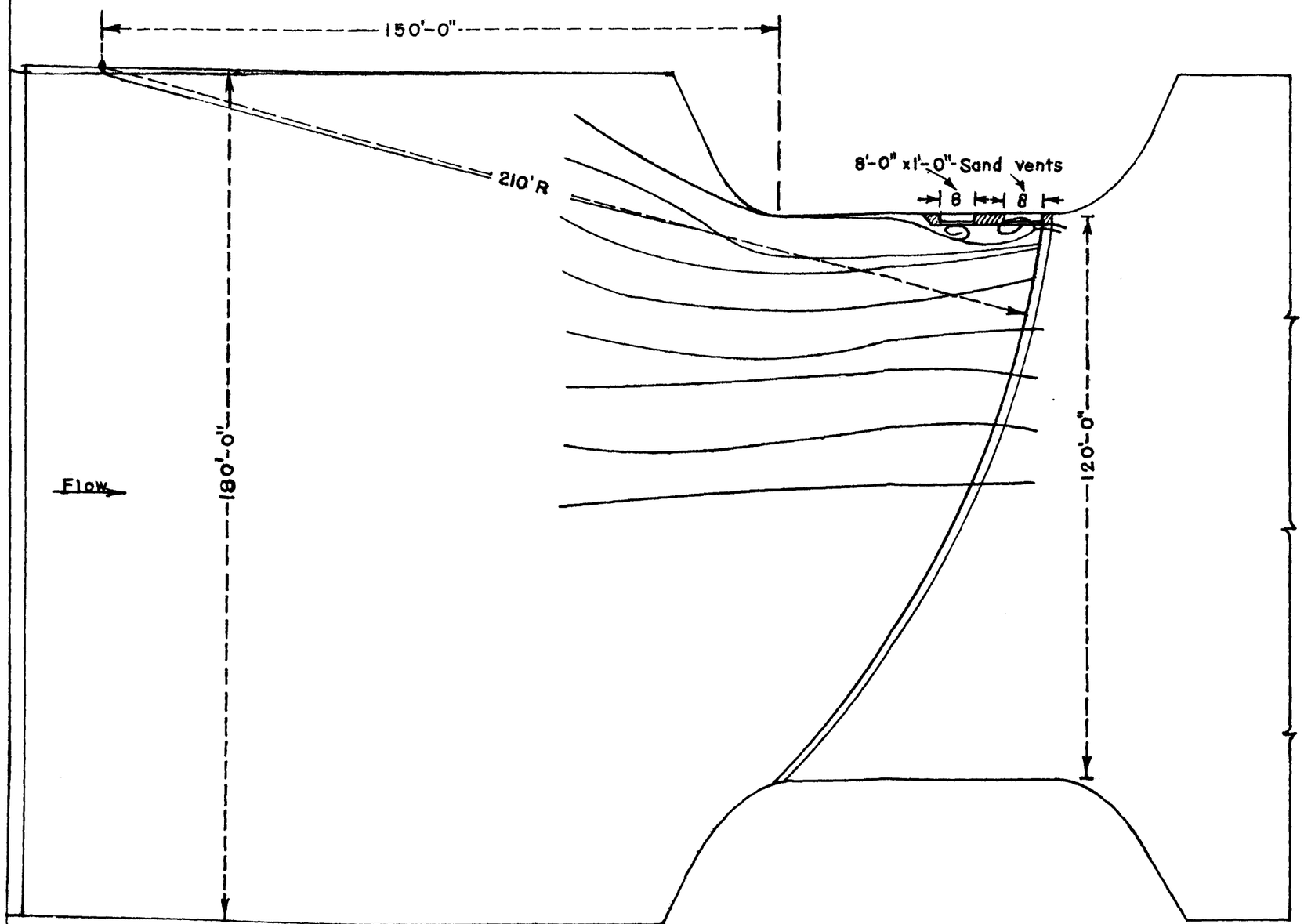
Annual Report of 1959	
17/30	7/10
Item NO	Drawing NO



Sand vents	2Nos of 8'-0"x1'-0"
Bed level of canal	182·32
Sill level of sand vent	182·32
Full supply level	189·32
Canal discharge Max·	4300 cfs
Height of curtain wall	1 Foot
Width of curtain wall	1'-6"

25 0 25 50

Scale in Feet

Public Works Department—Madras
Irrigation Research Station—Poondi

PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G·A CANAL

SURFACE FLOW LINES. HALE MAXIMUM DISCHARGE

Report	File	76/60 Drawing		
		J·E·R	A·E·R	E·E·R
		Trd:	ckd:	FIG···7

Reg No 112-E'62 (Saka Era 1884)-375.

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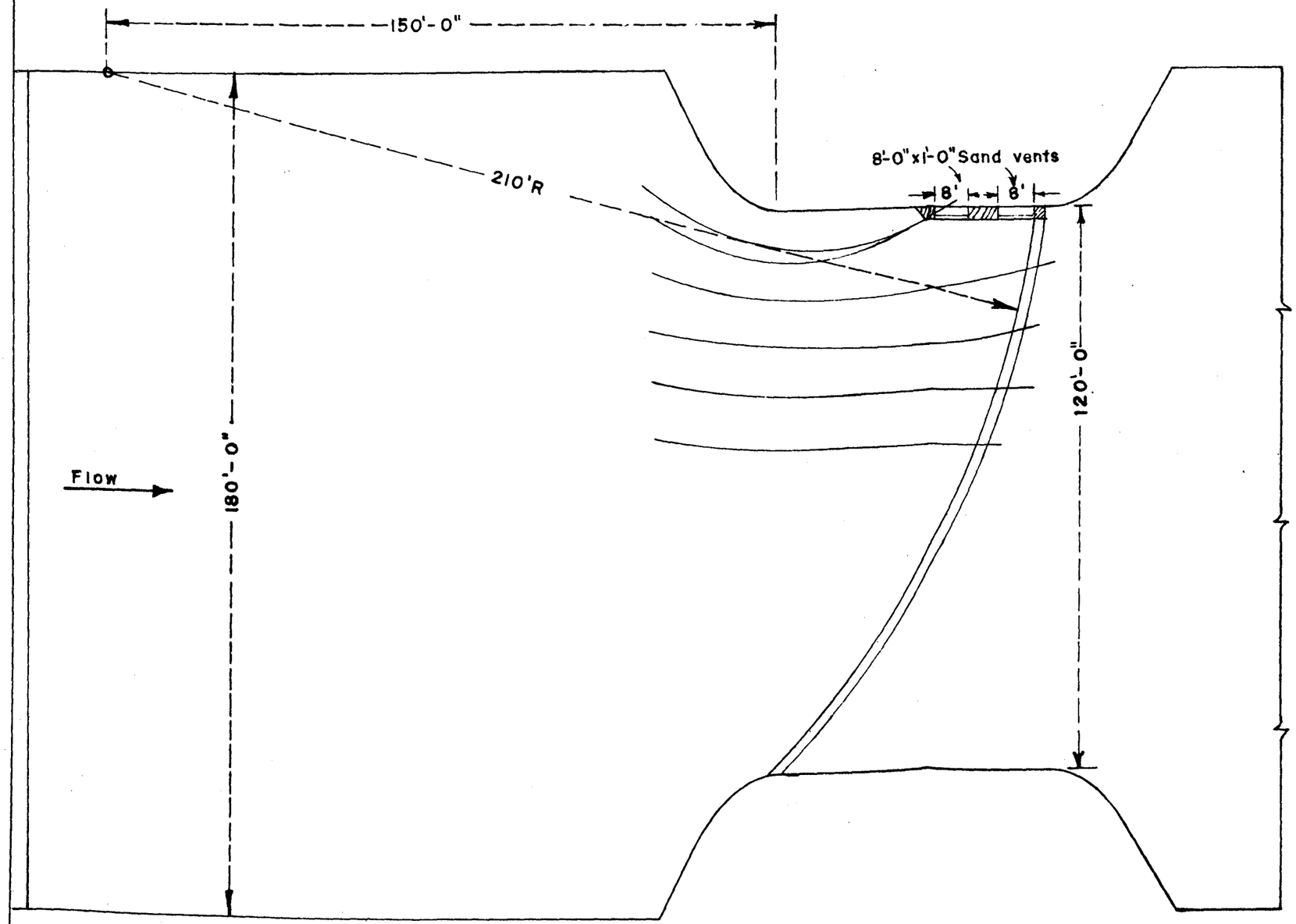
17/30	8/10
Item NO	Drawing NO



Sand vents	2 Nos. of 8'-0"x1'-0"
Bed level of canal	182·32
Sill level of sand vent	182·32
Full supply level	189·32
Canal discharge Max.	4300 cfs
Height of curtain wall	1 Foot
Width of curtain wall	1'-6"

25 0 25 50

Scale in Feet

Report	Public Works Department-Madras Irrigation Research Station-Poondi		
File	PROVIDING SAND VENTS TO THE SYPHON AT RD 2/34 OF G·A·CANAL BED FLOW LINES HALF MAXIMUM DISCHARGE		
77/60 Drawing			
	J·E·R	A·E·R	E·E·R
	Trd: V·V·	ckd:	FIG···8

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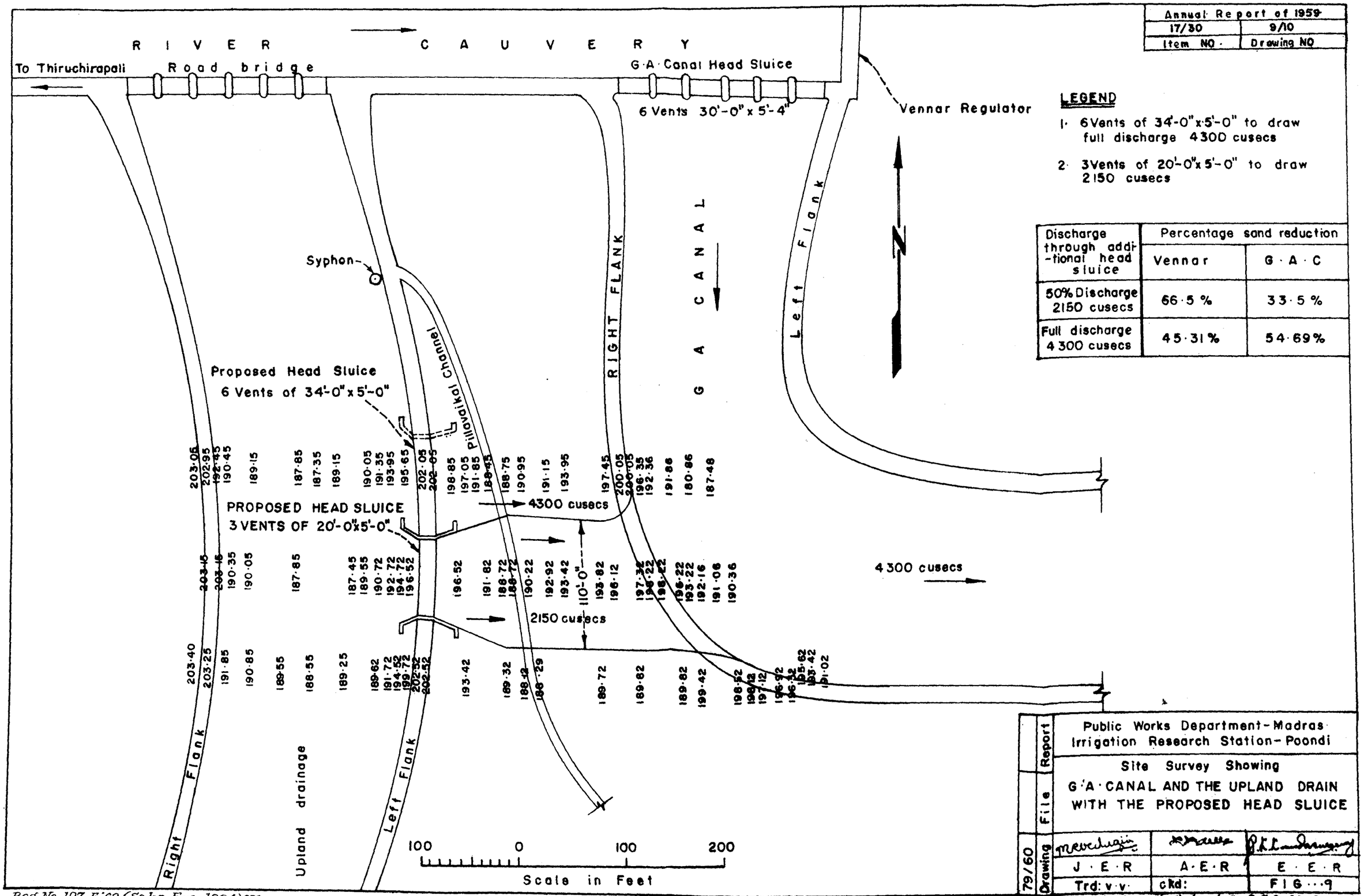
17/30	9/10
Item NO.	Drawing NO



LEGEND

1. 6 Vents of 34'-0" x 5'-0" to draw full discharge 4300 cusecs
2. 3 Vents of 20'-0" x 5'-0" to draw 2150 cusecs

Discharge through additional head sluice	Percentage sand reduction	
	Vennar	G·A·C
50% Discharge 2150 cusecs	66·5 %	33·5 %
Full discharge 4300 cusecs	45·31 %	54·69 %

Public Works Department - Madras
Irrigation Research Station - Poondi

Site Survey Showing
G·A·CANAL AND THE UPLAND DRAIN
WITH THE PROPOSED HEAD SLUICE

Report	File	Drawing 79/60		
		J·E·R	A·E·R	E·E·R
		Trd: v·v·	ckd:	FIG···9

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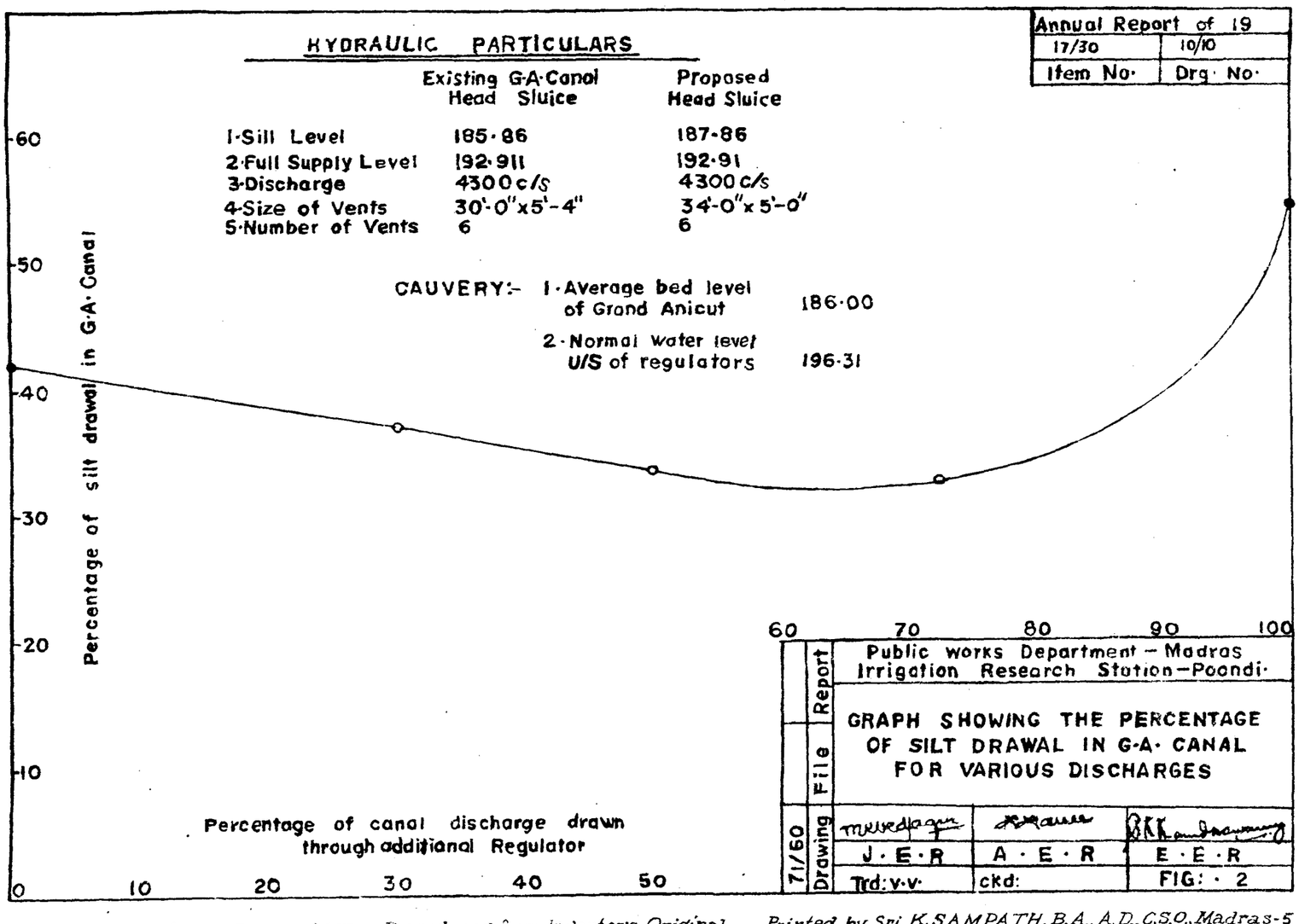

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NATHAMPATTI CHANNEL-SILT EXCIUSION STUDIES

To mitigate the quantiy of silt passing into the Nathampatti supply channel and to divert the silt into the Arjuna nadi, many renedial measures ware tried in the hydraulic model. Construction of head sluice at the off-take to control the flow in the channel, use of various lengths of groynes at various positions and divide walls of varicus lengths placed perpendicular to the head sluice were tried. Finally, the behaviour of 100 feet length divide wall placed perpendicular to the head sluice was found satisfactory for maximum discharge allowed in the river. Efficacy of this divide wall for low discharges has been examined and thus studies conducted are reported.

Experiments were conducted by allowing half the maximum discharge through the river and full supply in canal, halfthe maximum discharge in the river and half the full supply in the canal, quarter of the maximum discharge in the river and quarter of the full supply in the canal. In all the above cases as the flow in the river was less than the maximum flood discharge, the silt drawal in the canal was more. The divide wall was not an effective for low discharges in the river as it was for the maxmum flood discharge.

For low discharges curved wall of height 1/3rd of depth of the watar with its upstream face vertical and downstream face given a slope of 1 in 5, placed on the up streom side of the 100 feet length divide wall was used. But after a certain period of run it was observed that the curved wall was entirely covered up with silt and there was not appreciable reduction in the entry of silt into the canal.

Hence it can be concluded that the proposed divide wall placed perpendioular at the line of the head sluice will have pronounced effect only for the maximum discharges in the river. For low discharges, its effectiveness reducing the quantity of silt into the channel is less pronounced.

STATEMENT

Sl. No.	DISCHARGE	Percent distribution of sand in	
		River	Channel
(a)	Half the maximum discharge in the river and full supply through the canal	...	Full (100 %)
(b)	Half the maximum discharge in the river and half the full supply through the canal	73 %	27 %
(c)	1/4th the maximum discharge in the river and 1/4 the full supply through the canal	66.4 %	33.6 %
(d)	curved wall with half the maximum discharge in the river and half the full supply thro' the canal (vide Fig. 2).	70 %	30 %

For items a, b, and c vide Fig. 1.

SRIVAIKUNTAM ANICUT EQUITABLE DISCHARGE

In the studies already conducted three groynes used to shift the deep course of the river from the left towards the centre. The performance of the groynes for higher discharges was proposed to be studied in the model. Generally the performance of the groynes in shifting the deep course will only improve with the higher discharges.

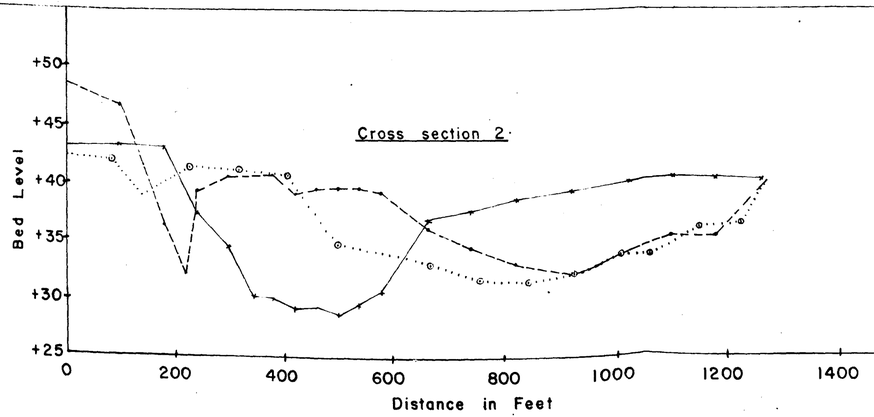
For conducting this study the already existing model construced to a scale of 1/180 Horizontal and 1/36 Vertical was taken up. As against the original discharges of 27,216 cusecs, a dischage of 40,824 cusecs which is nearly 1½ times the original discharges was allowed in the model. Before conducting studies to observe the peformance of the groynes, the sand injection rate for this higher discharge to keep the model in regime condition was to be fixed. So, the model was run with the original condition without the groynes, and diffent sand injection rates were tried. Finally a sandinjection rate of 17 lb/hour was arrived at to keep the river in the regime condition for a discharge of 40,824 cusecs.

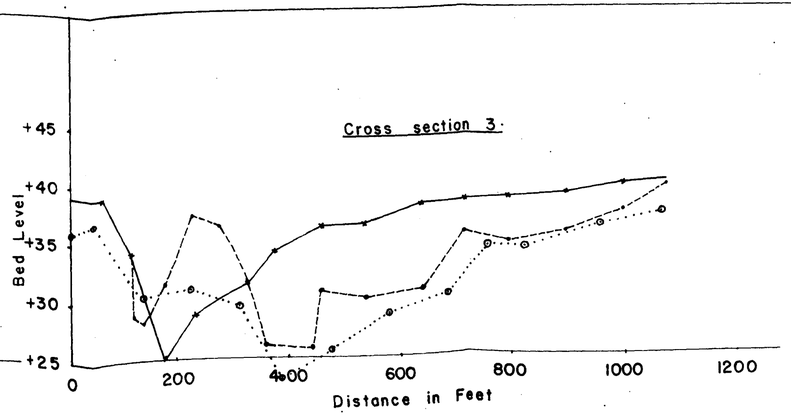
Then the three groynes were fixd in position as detailed in the Annual Report for 1959-Item No.25/45 (Vide Fig.-3 of Annual Report of 1958-Item No. 25/45). A discharge of 40,824 cusecs was allowed in the model with a sand injection rate of 17 lbs/hour. As before, the model was run for 54 hours. At the end of 54 hours the bed levels were taken along the cross-sections furnished already. The scour profile for the discharge of 27,216 cusecs with the groynes in position had already been taken and furnished in the previous (Vide figures 6 and 7 of Annual Report 1958-Item No.25/45). For this higher discharge of 40,824 casecs with the groynes in position the scour profiles were drawn along the same cross-section and compared with the previous ones (Vide Figure-1.)

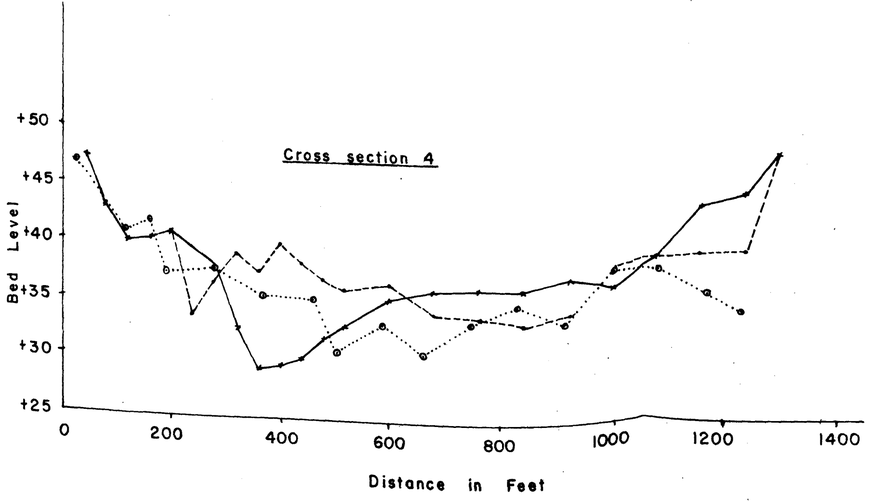
It can be seen from the figure that the deep course which got shifted towards the centre for a discharge of 27216 cusecs does not get obliterated for a higar discharge of 40,824 cusecs as was apprehended. Therefore, as already stated if the performance of the groynes in shifting the deep course of the river is alright for a certain discharges it will hold good for discharges higher also.

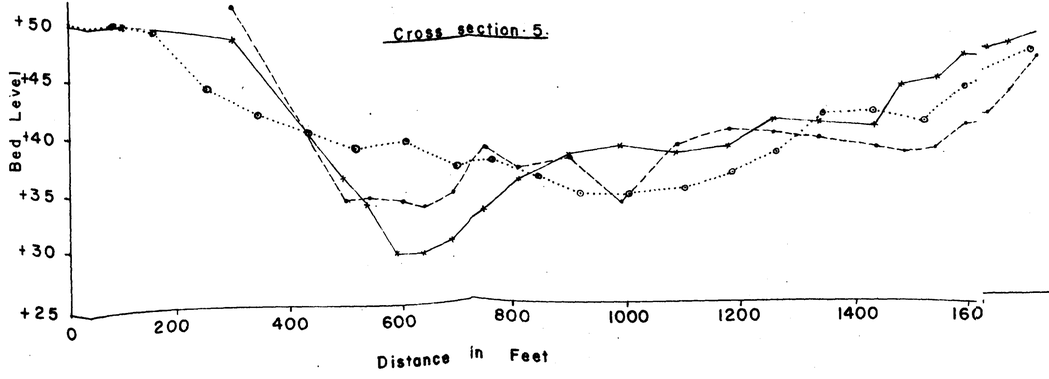
The details regarding the location of the groynes and the nose and the inclinations of the groynes to the river course are given in the annexed drawing (vide Figure-2). In the model, the top levels of the groynes were kept 1'0" (Prototype) above the water level obtained in the model for the discharge allowed. At the site the top levels of the groynes may be fixed one or two feet above the locally observed maximum flood level. Fixing the top and bottom widths of the groynes and the thickness and the width of the launching aprons are design aspects of the problem. With regard to the dimensions of the nose, a structure of 100 feet length, with a circular are of 10 feet radius at the bifurcation point (reduced to scals) was used in the model. The samc can be adopted at the site. The length of the nose can be increased if necessary to suit the site conditions. As has already been mentioned in page 8 of itm No. 25/45 of Annual Report for 1958, any rigid structure without sharp edges can be had for the nose, preferably with a circular facing.

Annual Report of 1959	
19/30	1/2
Item NO	Drawing NO









LEGEND

- Bed profile existing
- Bed profile for a discharge of 27,216 cusecs
- Bed profile for a discharge of 40,824 cusecs

Report	Public Works Dpartment — Madras Irrigation Research Station- Poondi.		
File	SRIVAIKUNTAM ANICUT BED PROFILE		
67/60 Drawing	S. Lakshmanan	U.V. Sadale	J. Khandrayan
	J·E·R	A·R·O	E·E·R
	Trd: v·v·	ckd:	FIG···1

Rcg No.114-E'62 (Saka Era 1884) 375.

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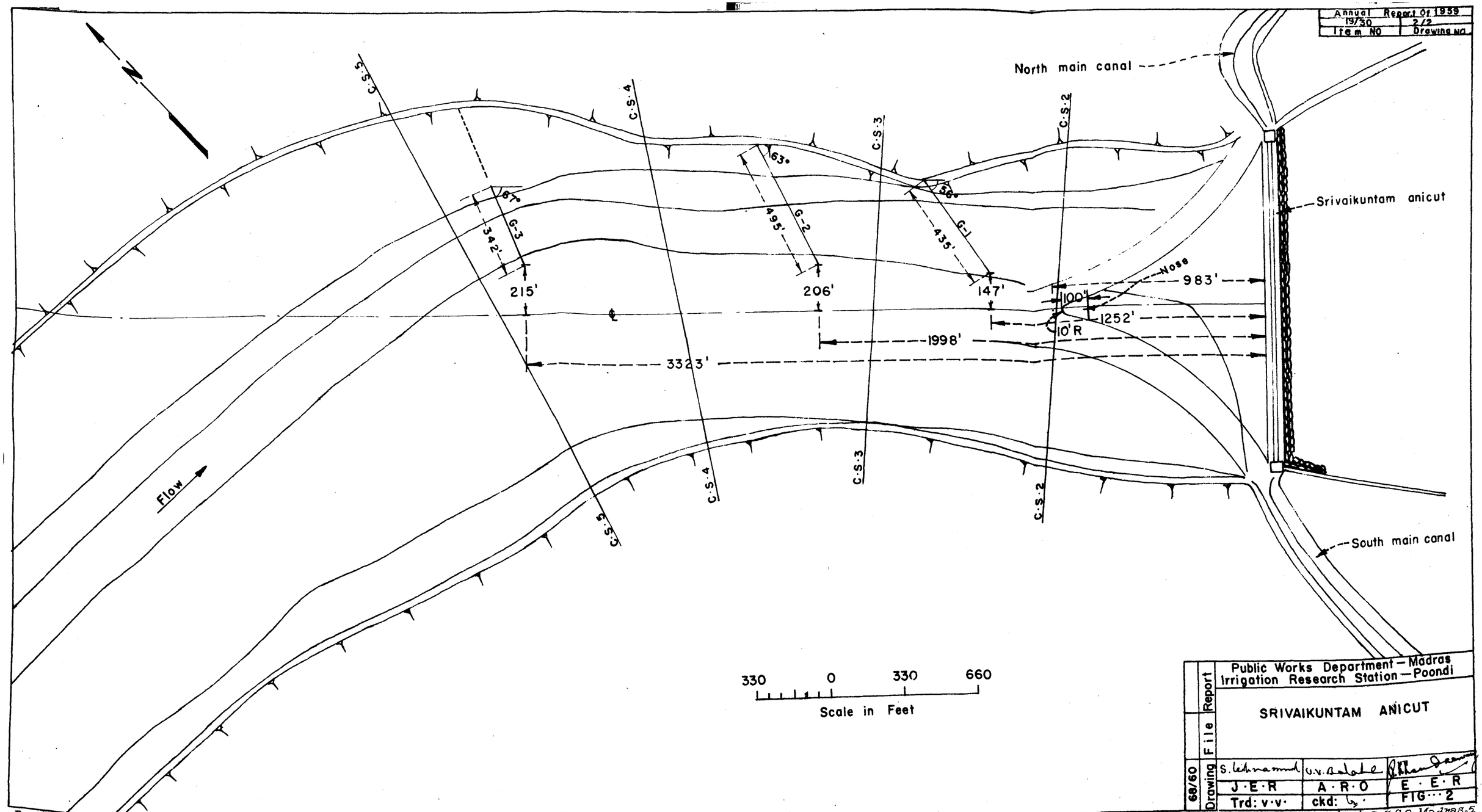

Reg. No. 115-E '62 (Saka Era 1884) 3'76

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Vilangudi Bridge

1. SYNOPSIS

This interim report deals with the studies conducted so far, in order to protect the left abutment of the Highway bridge constructed at the neck of a S-curve of a small river called Vilangudi Odai. A brief description of the model set up has been given. The details of the experiments are given in this report and results discussed. It is found from the results that an upstream repelling groyne fixed on the left margin, will solve the problem partly. The details of the further experiments that have been done partly to regulate the flow under the bridge are given in this report. The scope for further study is also indicated.

2. INTRODUCTION

2.1. Development of Highway is an essential requirement to any nation. In this country, particularly in Madras State, the development of Highways is progressing so rapidly that almost all the small villages may have the much-needed transport facilities. In the formation of Highways, construction of bridges is an important aspect. The main consideration that requires careful and special attention while proposing a bridge is the selection of proper site.

2.2. In the road leading to Chilamdaram from Tiruchi a causeway across one small river called Vilangudi Odai at about 45 miles from Tiruchi was recently replaced by a new bridge constructed downstream of that causeway. This Vilangudi Odai is a tributary to River Coloroon. Two other small streams join this main stream just upstream of the bridge. These three streams simultaneously get the maximum flow during the rainy season and the total maximum flood discharge is about 6500 cusecs. The slection of the site for this new bridge was made out in such a way that the bridge had to be located only on the neck of a S-curve just downstream of of the junction of the three streams, marked A, B and C for the sake of easy reference and convenience (vide Figure-1). The main stream A carries the the bulk of the discharge i.e. adout 5800 cusecs. Since the flow in this stream approaches the bridge along a curved course with the consequent concentration of flow on the left side, the left abutment of the bridge, gets attacked by (i) direct impinging of water due to the curved flow (ii) Curvature effect due to its being on the outer edge of the curve and (iii) concentration of flow on the left side. Naturally the right portion of the bridge, which is the innar side of the curve gets shoaled up on account of which there is very little or no flow through the left aputment of the bridge is under constant attack. The revetment and the bank connections are consequently damaged. Therefore, model studies were taken up in this Research Station to suggest some remedial measures to save the left abutment and the revetment from the attack of the floods and also to improve the flow condition through the

3. DATA

Maximum flood discharge in Stream-A	...	5825 cusecs
do Stream -B	...	258 ,,
do Stream-C	...	4.6 ,,
Total flood discharge in Vilangudi Odai	...	5579 ,,
Maximum water level	...	El. 92. 40

Details of the Bridge

No. of spans	...	5 Nos.
Length of span	...	32'0" clear
Thickness of pier	...	5'0" each
Road level	...	101. 50

4. MODEL SET UP

A geometrically similar model to a scale of 1/48 was constructed simulating all the three streams upstream of the bridge for a distance of about 1000 feet and the Vilangudi Odai downstream of the bridge for a distance of about 1300 feet. The bridge was made in teakwood and put up across the river as per correct orientation. The total discharges was measured in a main leading flume over a rectangular notch. The discharges in the streams 'B' and 'C' were also measured over V-notches fixed in the respective leading flumes and balance of the total discharge was allowed in the stream 'A'. The bed of the model was moulded with 14-24 sand as per the latest crosssection particulars received from the field. Arrangements had also been made by way of putting up tail gates at the downstream end of the model to adjust the water level in the model, if necessary (vide Figure-2).

5 EXPERIMENTS AND RESULTS

5.1. Studies were conducted mainly with a view to save the left abutment of the bridge from the attack of the floods and also to correct the flow condition under the bridge, thus avoiding concentration of flow through a few vents alone. The bridge site was inspected by the Chief Engineer (Highways) and instructions were given in the field to construct two groynes one at the up stream of the bridge and the other at the downstream of the bridge. It was decided to conduct studies first to determine the efficacy of this proposal.

5.2. Firstly, the model was run without any remedial measure incorporated in the model. The duration of run was fixed as one hour, based on the scour pattern produed during some initial trial runs as it was found that the extent of scour near the abutment and around the piers was almost constant after a duration of one hour. So, at the end of the one hour the scour readings were taken and the scour contours around the bridge were plottod. (Vide Figure 3). Also one flow lines were traced to assess the nature of glow approach towards the bridge from all the three streams and flow lines are given in Figure 4.

5.3. It was desired to pass the total discharge uniformly through all the vents as much as possible. So, for the original condition, the discharge through each vent was calculated by observing the velocity of flow through each vent and knowing the cross-sectional area. The discharge distribution curve was plotted and presented in Figure-5. The same three types of data i.e. scour contour, flow lines and discharge distribution were followed for all the trials made to assess the relative merits of each proposal.

Proposal-1.

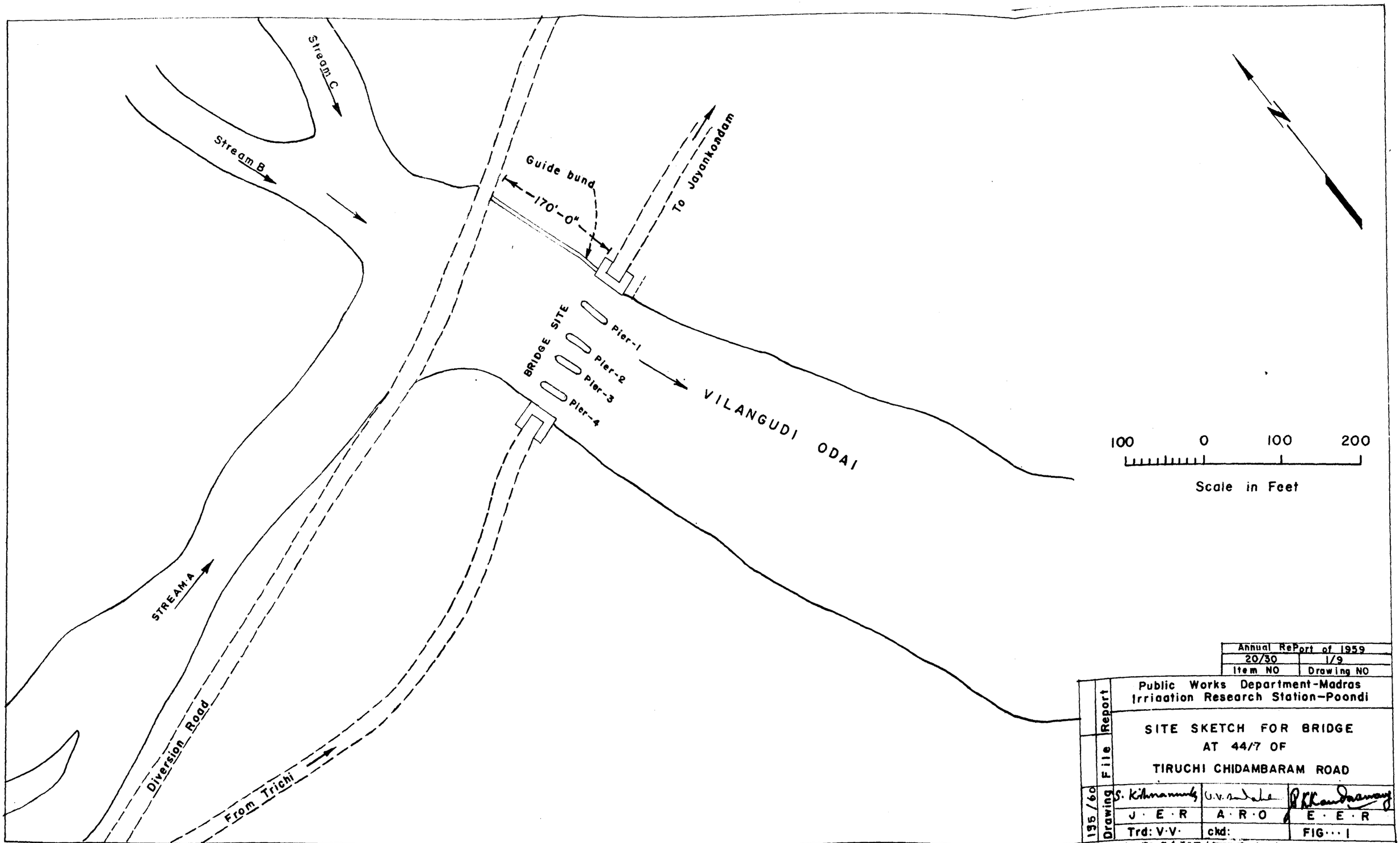
5.4. After conducting experiments on the lines mentioned above for original condition, the suggestions of the Chief Engineer (Highways) were incorporated in the model. An upstream groyne of length 25 feet from the face of the left abutment was fixed at a distance of 40 feet from the end of the abutment of the bridge and a downstream groyne of length 25 feet was fixed at a distance of 30 feet from end of the abutment of the bridge. Since the flow was to be diverted away from the left adutment, only repelling groynes as shown in Figure-6 were adopted. The repelling groynes were inclined at an angle of 60° to the bank. As before, the model was run for an hour. The scour readings were taken and the scour contours were plotted (vide Figure-6). The corresponding flow lines were traced and presented in Figure-7. The velocities were observed through each vent and the discharges were calculated. The distribution curve is given in Figure 5 along with the curve obtained for the original condition.

5.5. Based on the observed scour contours for the two trials i.e. with and without remedial measures as suggested by the Chief Engineer (Highways) the following conclusions are drawn. When the model was run as per the original condition without any device, deep scour was formed in the model around the abutment and the first pier as can be seen from the Figure-3 and the photos furnished in the same Figure. The 75 feet contour passes near the abutment and first pier indicating the deep scour in that region.

5.6. But referring to the Figure-6 in which the scour contours for the porposal-I with the remedial measures as suggested by the Chief Engineer (Highways) were drawn, it is seen that no deep scour has been formed near the abutment. The deep scour has been shifted to the first and the second pier. This can be clearly seen from the photos furnished in Figure-6.

The 80,00 contour is seen around the first and the second piers, but near the abutment only 90.00 contour is passing, thereby indicating that the abutment has been almost saved from deep scour by the up stream groyne.

5.7. From the flow lines traced for the original condition, it is observed that there is concentration of flow on the left side, the maximum flow being through the three left extreme vent and only very little flow passed through the fourth vent (vide Figure-4). This is mainly due to the curved flow in the stream A which carried the major part of total discharge. But after the introduction of the device the flow lines on the left side are diverted towards the right thus avoiding concentration on left extreme vent (vide Figure-7). Yet the maximum flow persist through the second and the third vent and there is practically no flow through the two right extreme vents as before.



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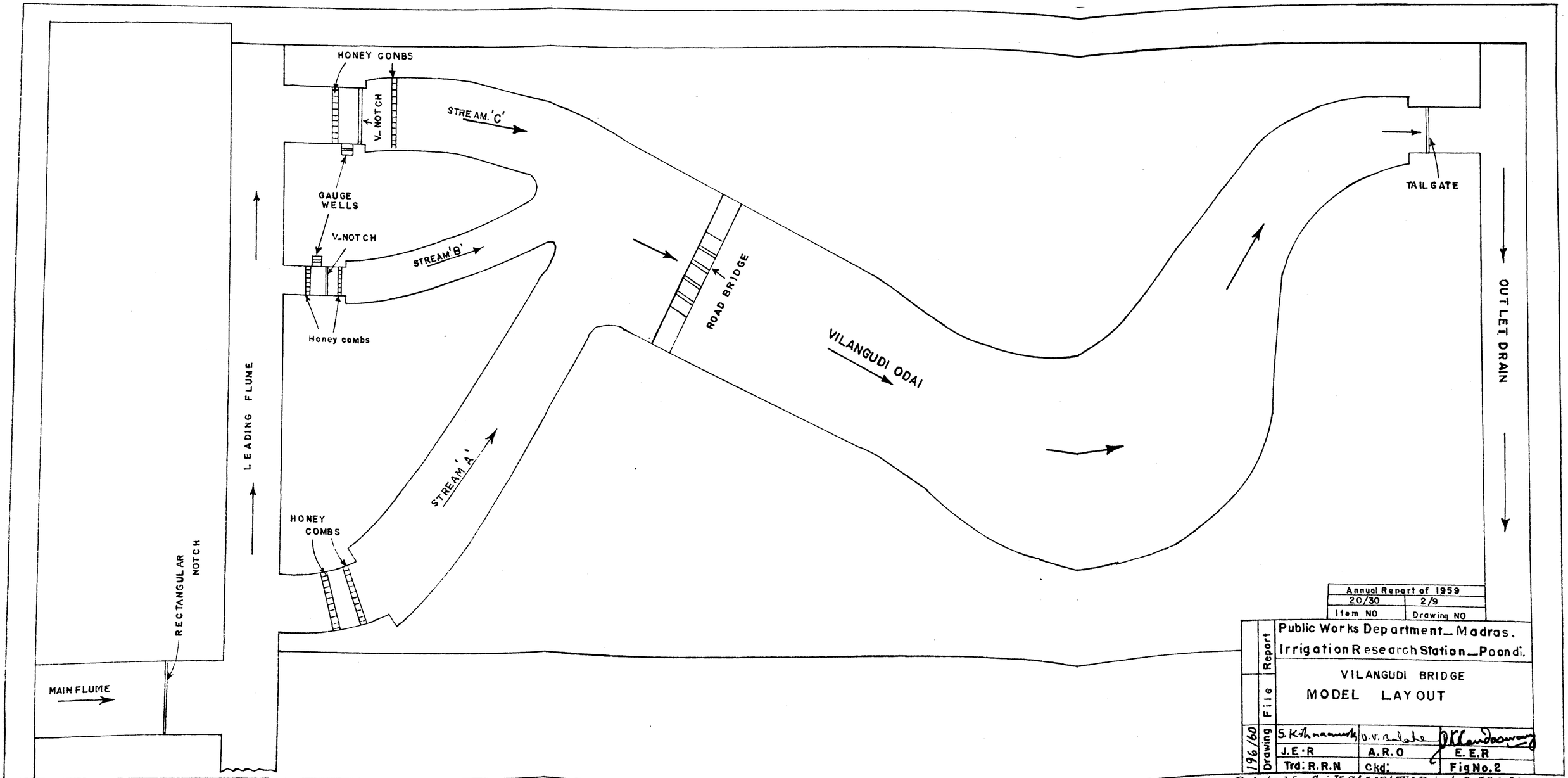

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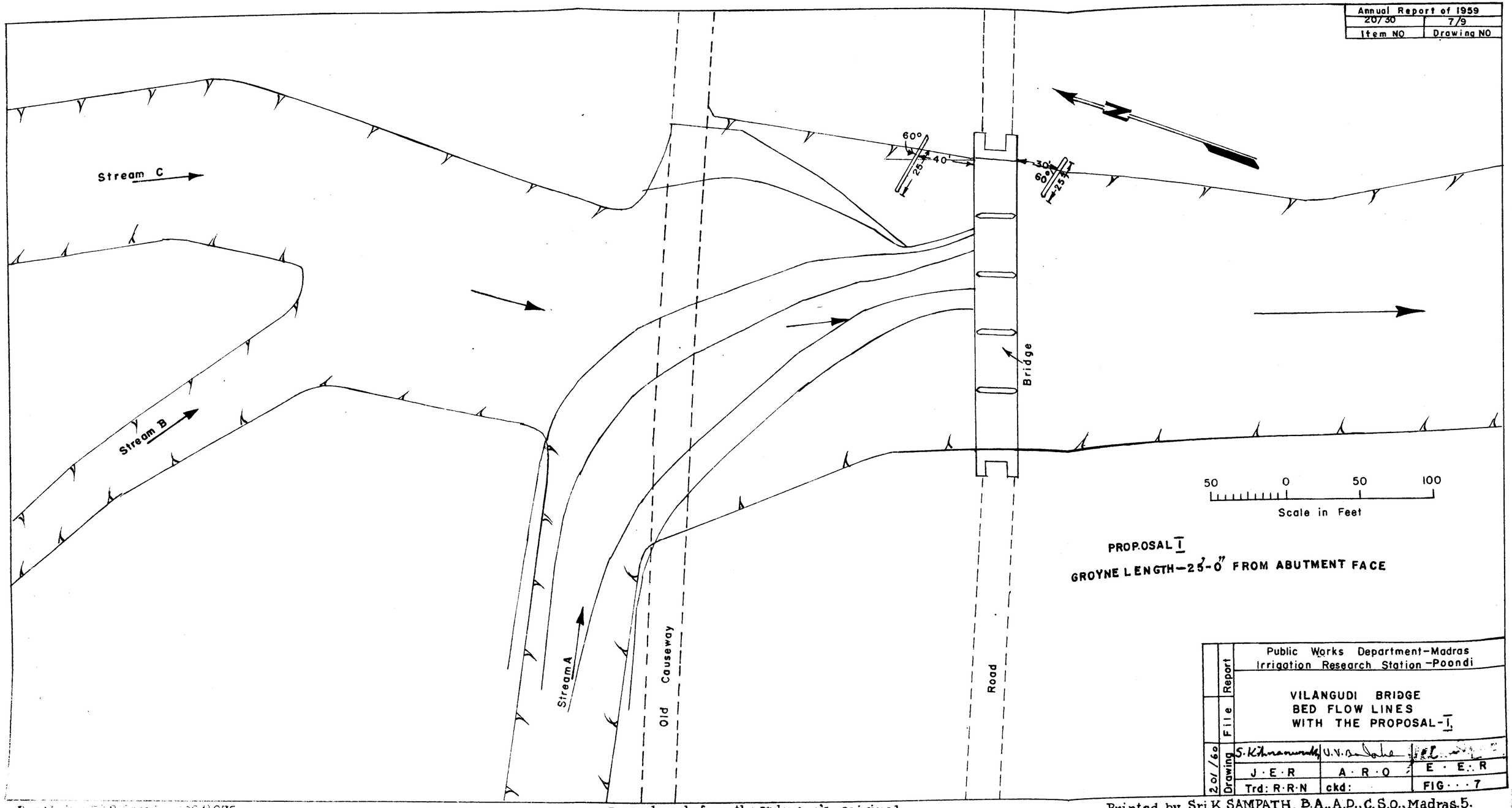
20/30	7/9
Item NO	Drawing NO



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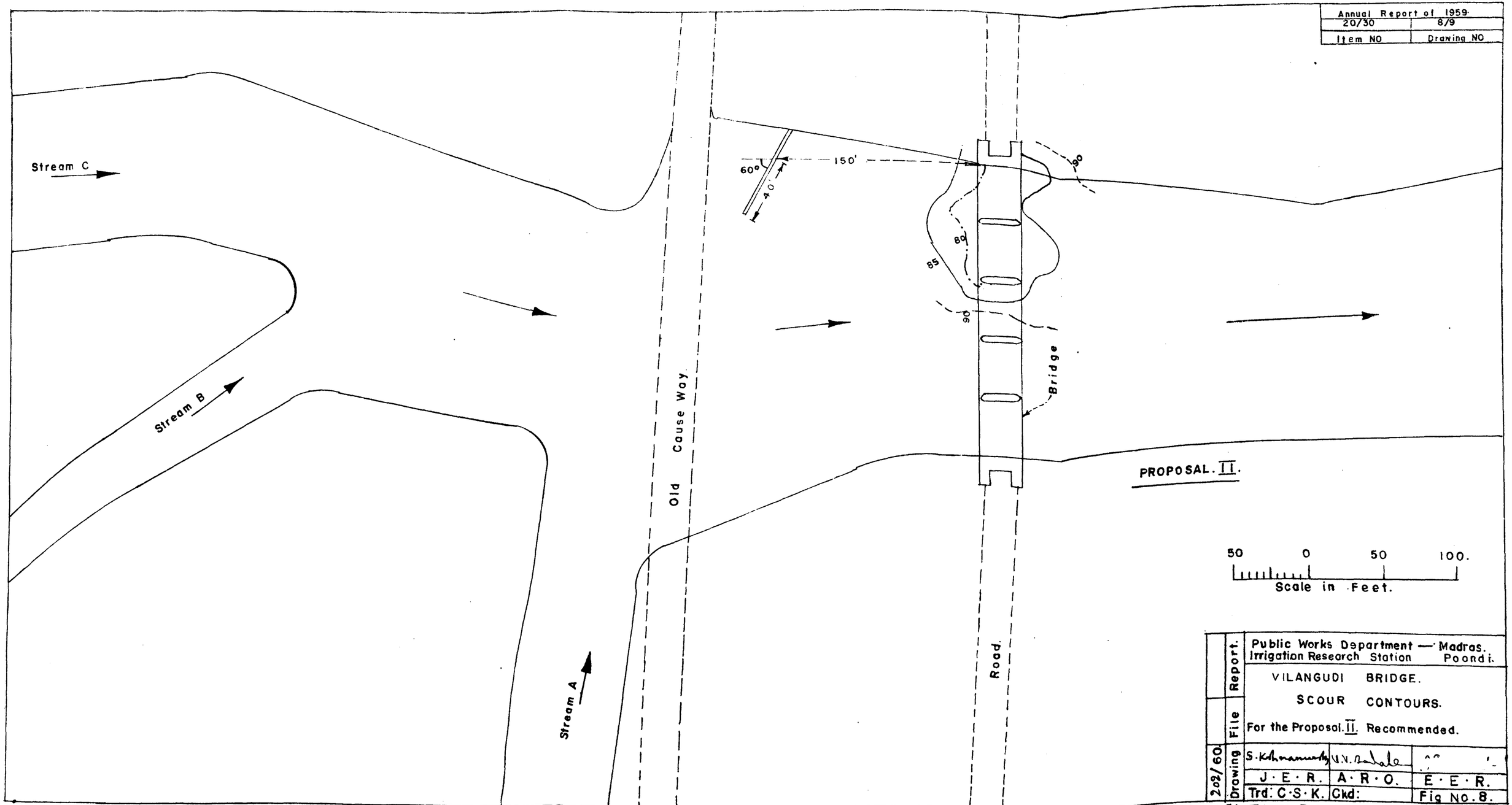

Report.	Public Works Department — Madras. Irrigation Research Station Poondi.		
	VILANGUDI BRIDGE. SCOUR CONTOURS.		
File	For the Proposal. II. Recommended.		
202/60 Drawing	S. Kalyanamurthy	N.V. Sahale	
	J·E·R.	A·R·O.	E·E·R.
	Trd: C·S·K.	Ckd:	Fig No. 8.

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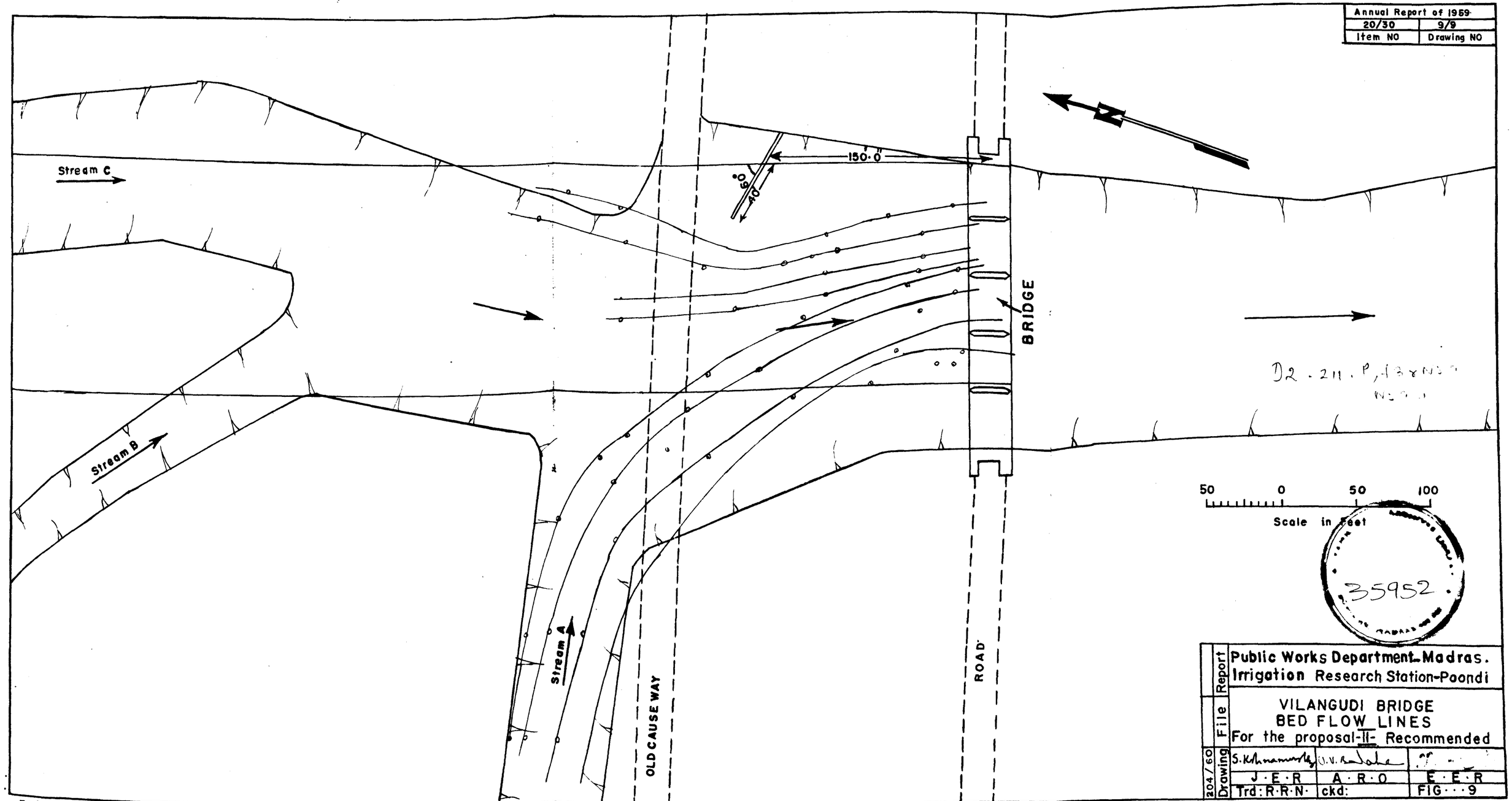
20/30	9/9
Item NO	Drawing NO



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35952

Report	Public Works Department-Madras. Irrigation Research Station-Poondi
File	VILANGUDI BRIDGE BED FLOW LINES For the proposal-II- Recommended
204/60 Drawing	S. Krishnamurthy / U.V. Badhe /
	J·E·R / A·R·O / E·E·R
	Trd: R·R·N· / ckd: / FIG···9

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