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(GOVERNMENT OF INDIA)

**CENTRAL WATERPOWER, IRRIGATION,
& NAVIGATION COMMISSION**

CWIN RESEARCH STATION POONA

Director : D. V. Joglekar, B.E.

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1948



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Central Waterpower, Irrigation & Navigation Commission.

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INTRODUCTORY NOTE

By

A. N. Khosla, I. S. E.

Consulting engineer to the government of India and chairman, Central Waterpower, Irrigation and Navigation Commission, New Delhi,

1. As reported last year, the expansion in connection with the organisation of the Central Research Station was intended to be carried out at Khadakwasla as far as possible and the balance at Okhla near Delhi. Subsequently, however, it was decided to restrict the expansion of the Station to Khadakwasla for the present. The Bombay Government was accordingly approached to acquire 109 acres of the land adjoining the Research Station. 90 acres of the land have already been acquired and taken possession of by the Research Station. The remaining 19 acres will be acquired during the current year.

Arrangements for supplying water to the newly acquired area are nearing completion. Detailed estimates of levelling of the area for purposes of rating tank, workshops and field hydraulic laboratory have been prepared and submitted to the Government of India for approval. It is proposed to locate some staff quarters at Khadakwasla.

Estimates for carrying out experiments with model of deep-seated sluices with a view to studying the effects of various shapes of gates on the co-efficient of discharges and cavitation etc. have been submitted to the Government of India for approval.

(ii)

2. The requisite staff for the expanded station was recruited during the year. They are receiving practical training on the Hirakud and Kosi projects preparatory to taking up regular research at Khadakwasla or Poona.

3. Most of the equipment required for the various laboratories was ordered during the year and some of it has already arrived. Early steps are being taken to set up these laboratories.

4. The Research Station has been increasingly in demand for the solution of diverse problems, more particularly those relating to development of river resources in India. Requests for carrying out experiments have been received from some of the neighbouring countries. Problems relating to navigability of major waterways near Ports are engaging the special attention of the Research Station.

Advice, based on accumulated experience of the station, was given to Provinces and States.

5. Notable among the experiments carried out and in progress during the year are those pertaining to the improvement of the navigability of the Hooghly estuary, the Cochin Harbour, the training of the Ganga river at Garhmukteswar, the Tapti at Surat, the Kshipra at Ujjain and the Kuakhai at the proposed bridge near Baliana (near Bhuvanesar). Experiments have also been carried out on spillways, dams and appurtenant works.

Also the wave recording apparatus is being developed at the Station for accurate reproduction and recording of wave characteristics. This apparatus will be very useful in tackling the problems of the ports and harbours where waves play an important part. It is immediately required for the model of the Madras Harbour.

6. The Research Station was visited in July 1948 by the Standing Committee of the Cabinet for the Ministry of Works, Mines and Power along with the Hon'ble Minister and Secretary for Work, Mines and Power Member (Irrigation) Central Waterpower, Irrigation and Navigation Commission and Officers of the Central P.W.D.

7. Shri K. K. Framji continued to be the Director till 3-7-48 when he had to leave consequent on his appointment as Director General of Ordnance Factories. Shri D. V. Joglekar took over as Director from 4-7-1948.

8. The work of the Station has been creditably handled by the Officers and staff at Khadakwasla and satisfactory progress towards the expansion of the Station has been maintained.

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2. STAFF AND THEIR ACTIVITIES.

Director : Shri K. K. Framji, M. B. E., B. A. (Bom.), B. Sc. (Lond.) A. K. C.
(Lond.), A. M. Am. Soc. C. E., M. I. E., I. S. E. up to 3-7-1948.
Shri D. V. Joglekar, B. E. from 4-7-1948.
Deputy Director : Shri D. V. Joglekar, B. E. up to 3-7-1948.

Research Officers.

1. Shri G. M. Panchang, B. A. (Hons.), M. Sc. (Lond.) from 1-10-48.
2. ,, S. T. Ghotankar, M. Sc.
3. ,, C. V. Gole, B. E.

Assistant Research Officers.

1. Shri S. N. Mulekar, B. Sc. (Elec. & Mech.)
2. ,, S. V. Chitale, B. E.
3. ,, P. K. Kulkarni, B. Sc. (Elec. & Mech.)
4. ,, G. T. Wadekar, B. E. from 29-11-1948.
5. ,, K. R. Datye, B. E. (Extra Assistant Director)

Research Assistants.

1. Shri G. T. Wadekar, B. E. up to 28-11-1948.
2. ,, C. G. Tamhankar, B. Sc. (Elec. & Mech.)
3. ,, P. M. Damle, B. Sc.
4. ,, Baldev Kumar, M. A. (Math.) from 26-7-1948.
5. ,, T. K. Subramanyam, M. Sc. (Ag.) Soil Mech. ,, 27-7-1948.
6. ,, V. V. Venkataswarlu, M. Sc. (Physics.) ,, 27-7-1948.
7. ,, T. V. Iyengar, M. Sc. (Physics) ,, 4-8-1948.
8. ,, R. Rangarajan, M. A. (Statistics) ,, 9-8-1948.
9. ,, V. Chidambaram, M. Sc. (Math) ,, 9-8-1948.
10. ,, P. S. Chowdhari, M. Sc. (Statistics) ,, 20-8-1948.
11. ,, S. D. L. Luthra, B. Sc., A. M. I. S. E. ,, 23-9-1948.
12. ,, N. S. Vasudeva Rao, B. Sc., B. E. ,, 18-5-1948.

Overseers.

1. Shri N. K. R. Iyengar
2. ,, W. N. Damle from 10-7-1948.

Head Draftsman.

1. Shri H. N. Shahani, B. E. (Civil) from 27-9-1948.

Computors "A"

1. Shri S. B. Barye, i/c Laboratory.
2. „ R. P. C. Varma, B. Sc.
3. „ Balasaran Das Baijal, B. Sc. from 2-8-1948.
4. „ Ram Prakash Malhotra, B. Sc. from 2-8-1948 to 23-12-1948.
5. „ R. C. Narsinhan, B. Sc. from 6-8-1948.
6. „ S. R. Gokhale, B. Sc. „ 6-8-1948.

Silt Analysts.

1. Shri P. S. Kapoor, M. Sc. from 19-7-1948.
2. „ D. R. Bapaye, B. Sc. „ 28-7-1948.
3. „ J. T. Karira, B. Sc. „ 25-8-1948.

Office Superintendent.

1. Shri D. T. Deshpande.

In March 1948, the Director (Shri K. K. Framji) and the Deputy Director along with their assistants visited the Tansa Lake in connection with the proposed installation of automatic gates to increase the storage for

Bombay Water Supply. Later during the year the Director (Shri D. V. Joglekar) and the assistant officers concerned visited 12 places which are shown in figure 1. The practice of inspecting the sites of various problems under investigation at the Station has proved very beneficial in enabling the staff to visualise the prototype conditions for natural reproduction in models.

In July 1948 Shri K. K. Framji, the Director of the Station was transferred to Calcutta as Director General of Ordnance Factories. He handed over charge to Shri D. V. Joglekar. At the end of July, the Standing Committee attached to the Ministry of Works, Mines and Power held a meeting at the Inspection Bungalow at Khadakwasla. The Honourable Minister Shri N. V. Gadgil with the Standing Committee Members, accompanied by Shri B. K. Gokhale, Secretary, Works, Mines and Power, Sardar Mansingh, Member, Central Waterpower, Irrigation & Navigation Commission, Shri Puri, Chief Engineer, Central P. W. D., New Delhi, Shri Chopra, C. E., Delhi and other officers, arrived at Khadakwasla in the morning on 24th July. Shri K. K. Framji, the former Director of this Research Station who happened to be in Poona on his routine office tour in his new capacity was also invited by Shri B. K. Gokhale to attend the meeting.

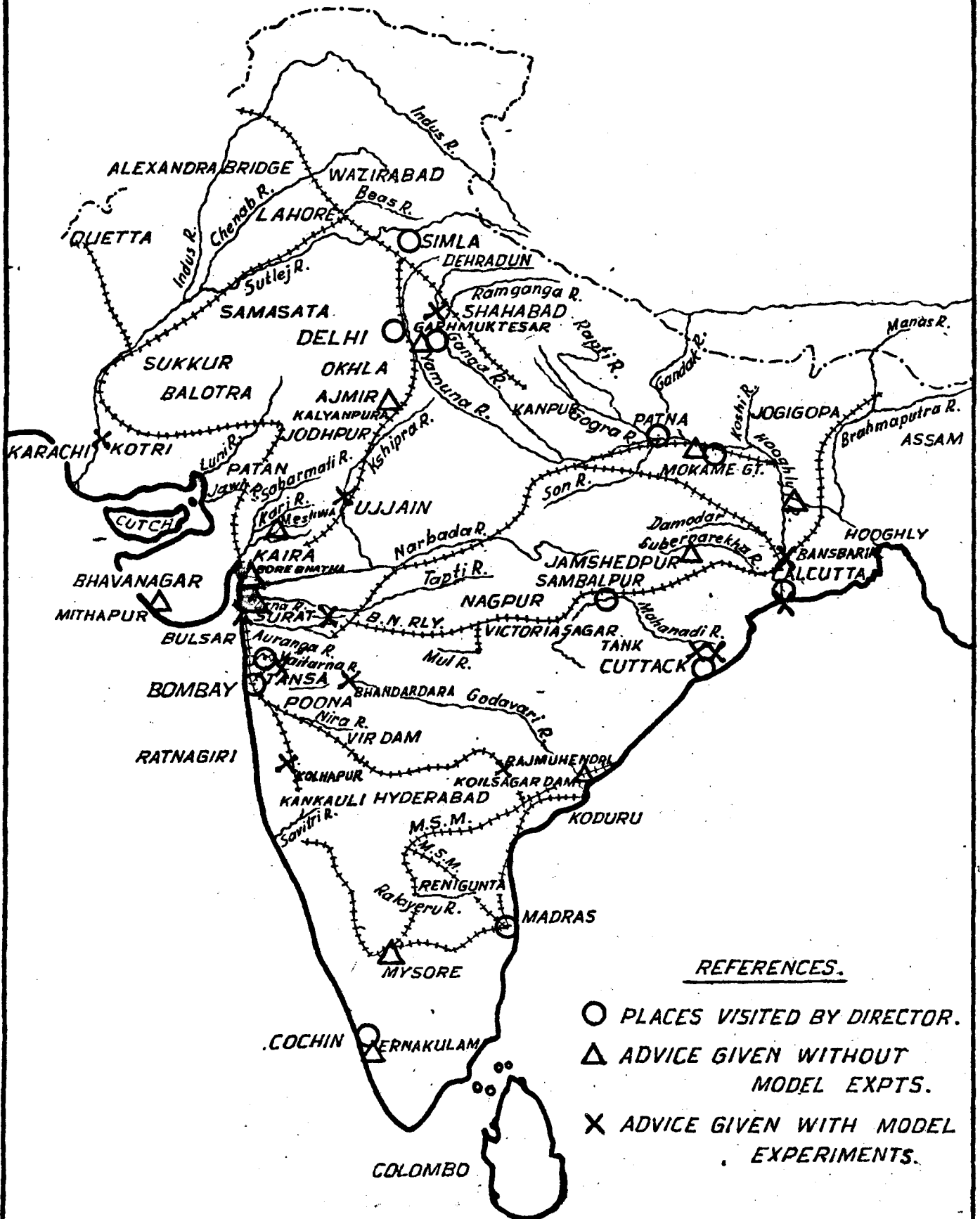
The visitors arranged in two batches were shown round the Research Station—one party being led by Shri D. V. Joglekar and the other led by Shri K. K. Framji. The members and the officers were keenly interested in the present activities of the Station. The new area of 106 acres being

FIG. 1

MAP OF INDIA.

1948

SCALE:- 1" = 320 MILES.



REFERENCES.

- PLACES VISITED BY DIRECTOR.
- △ ADVICE GIVEN WITHOUT MODEL EXPTS.
- × ADVICE GIVEN WITH MODEL EXPERIMENTS.

acquired under the urgency clause for the expansion of this Station was also seen by the visitors and they were satisfied that any amount of additional area could be secured as and when found necessary. Shri A. N. Khosla could not join the party being detained in Europe.

Expansion of the C.W.I. and N. Research Station :

In pursuance of Government's decision to expand the activities of the Research Station a proposal to acquire an area of 106 acres under the urgency clause was put up. An area of 82 acres has been taken possession of and the remaining is being made over in the next few months.

In September and October the Federal Public Service Commission recruited officers for the expanded Station.

Equipment for the various laboratories has been ordered and is being received.

During the year under report advice based on previous experience was given in 15 cases, a synopsis of which has been embodied in the body of the report. Besides this, 18 specific cases were dealt with by model experimentation in addition to basic studies of scour below rigid structures, flow in incoherent sand, meandering of channels etc.

A number of parties of students from Madras, Bangalore, Poona, Bombay, Baroda, Patna and a large number of visitors visited this Station during the year; prominent visitors were:—

The Hon. Shri N. V. Gadgil, Minister, Works, Mines and Power.

The Hon. Shri Bhupati Mujumdar, Minister for Irrigation, West Bengal.

Shri B. K. Gokhale, Secretary, Works, Mines and Power.

„ B.S. Puri, Chief Engineer, C. P.W.D.

„ Chopra, Chief Engineer, C. P.W.D.

„ K. K. Framji, Director General, Ordnance Factories, Calcutta.

„ S. N. Majumdar, Administrator, Damodar Valley Project.

Mr. J. B. Murray and Mr. Mitter, Officers, Port Trust, Calcutta.

Mr. H. L. Vadera, Director, Central Designs.

The Hon. Chief Justice of Bombay.

Shri L. V. Lalwani, Superintending Engineer, Central Circle, Calcutta.

„ S. P. Sarathy, Liaison Officer, Port Trust, Calcutta.

ANNUAL REPORT (TECHNICAL) YEAR 1948.

Shri Mathure, Sanitary Engineer, Gwalior.

The Chief Engineer, i/c Hydroelectric Scheme at Radhanagari,
Kolhapur.

Sardar Mansingh, Member, Central Waterpower, Irrigation &
Navigation Commission, New Delhi.

Shri Khanna, Assistant Consulting Engineer, Central Water-
power, Irrigation & Navigation Commission.

„ S. V. Diwatia, Superintending Engineer, Deccan Irrigation
Circle, Poona.

„ Moose, District Judge, Poona.

SECTION "A" - HYDROLOGY.**1. Rainfall and Run-off.**

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(i) Sources and Yield.

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2. Estimation of run-off from Vaitarna Catchment of the proposed dam site (Bombay).

1. INTRODUCTION.

Owing to the large increase in the population and industries of the Bombay, the contents of the Tansa Lake even with installation of four feet high automatic gates are totally inadequate to meet its requirements. In order to augment the present water supply it has been decided to construct 235 ft. high gravity type cement concrete dam across the Vaitarna river (Fig. 1) which will impound about 6,000 m.cft. of water. The Special Engineer of the Bombay Municipality referred the Vaitarna Scheme to the Station for advice on the following points:—

1. Capacity of the spillway to pass the maximum probable flood discharge.
2. Correlation ratio between the yield of Tansa and Vaitarna Catchments. With a view to determine statistically whether the anticipated minimum annual run-off will be obtained from Vaitarna Catchment.
3. The type of spillway to be provided and the necessary flood lift with the type considered suitable.

Director's opinion on the first two of the above points were given as below, while the third point is dealt with elsewhere in the Report.

2. (a) *The maximum probable flood discharge of Vaitarna Catchment.*

Independent estimates were made according to various methods as described below.

(i) *Observed data.*

Surface velocities were measured by floats up to a depth of 11 ft. and converted to mean velocities by using a conversion factor of 0.82. Velocity observations were not available for depths greater than 11 ft. but surface slopes were recorded up to 26 ft. depth which enabled to work out velocities according to Chezy formula. With the velocities thus obtained and the computed sectional areas, discharges at different water levels were calculated and a discharge:gauge curve was prepared by the Special Engineer. By extrapolation of this curve he estimated the discharge at the highest recorded depth of 31.5 ft. in 1941 to be 96,900 cusecs.

In the opinion of the Director, there are several shortcomings in the method adopted by the Special Engineer in constructing the discharge : gauge curve.

Where the reach between the observations from which surface slope is derived is not uniform, even if it is straight, it is the *energy* slope rather than the surface slope which must be taken for obtaining the correct value of the velocity. By taking energy slope instead of surface slope, less/greater discharge would be obtained according as the downstream section is less/more than the upstream section. As, however, the cross-sectional area at the downstream end of the reach was said to be not available it was not possible to verify the resulting error in estimate of the flood discharge by taking surface slope instead of energy slope.

Part of the discharge : gauge curve extrapolated by the Special Engineer was not quite correct and a more accurate estimate according to the curve modified by the Station gave the discharge as 90,000 cusecs as against 96,900 cusecs calculated by the Special Engineer.

Velocities were also calculated by the Station according to

$$(A) \text{ Manning's formula } V = \frac{1.486 R^{2/3} S^{1/2}}{N}$$

$$(B) \text{ The Lacey general flow formula } V = 16R^{2/3} S^{1/3}$$

and (C) Lacey's absolute rugosity formula

$$V = \frac{1.3458 R^{3/4} S^{1/2}}{N_a}$$

For obtaining Lacey's N_a , a curve of N_a : depth was prepared from available data.

The velocities and corresponding discharges for depth of 31.5 ft. according to these formulae are as under:

Table No. 1

Showing computed velocities and discharges according to various formulae
Depth = 31.5 ft.

Formula.	Velocity in ft/sec.	Discharge in cs.
Manning's (N = 0.035)	14.6	89,000
Lacey's general flow	15.1	92,000
Lacey rugosity	16.85	1,03,000
Chezy's — extrapolation by the Station	14.80	90,000

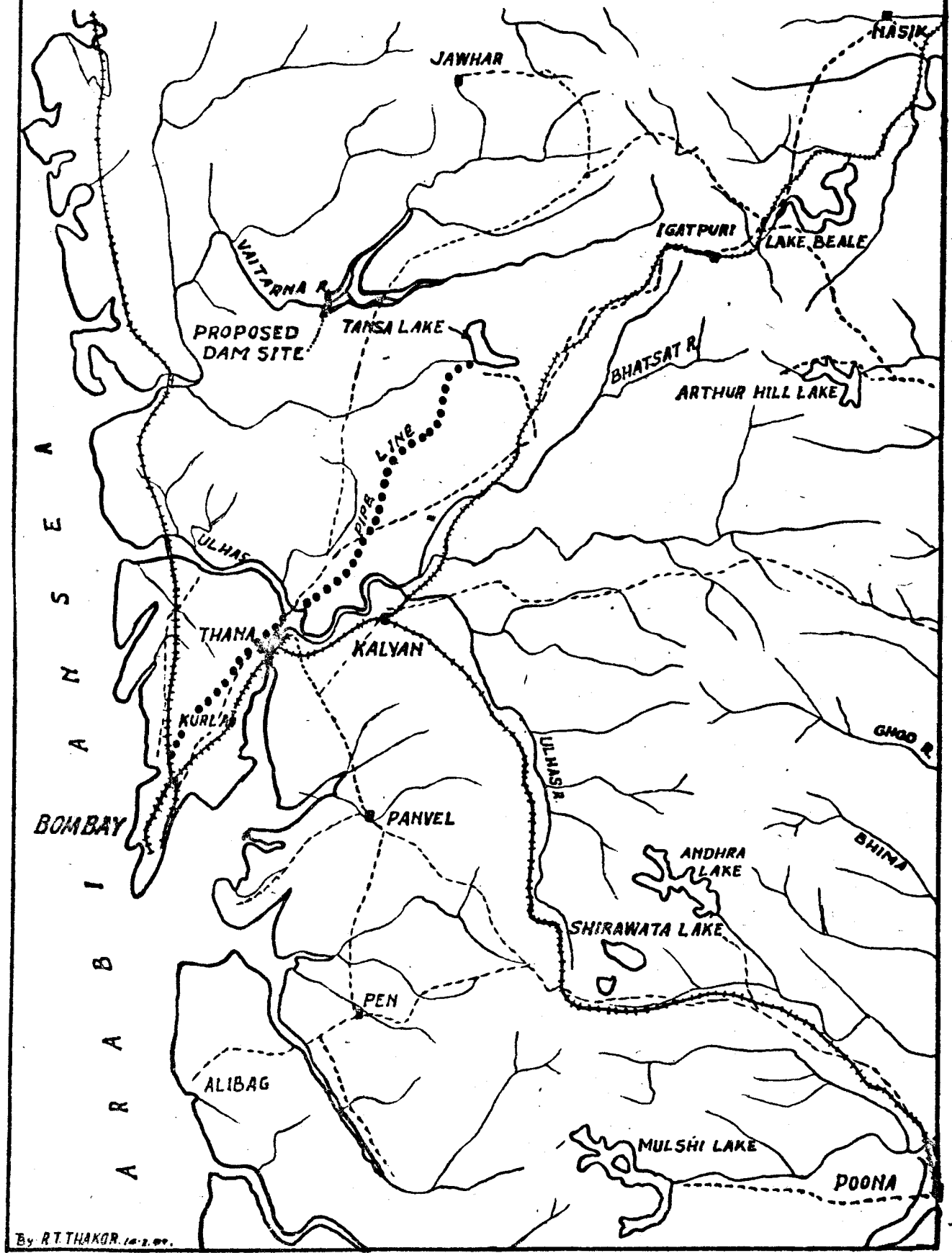
FIG :- I

INDEX PLAN

SHOWING TANSA AND VAITARNA

DAM SITE

SCALE :- 1" = 16 MILES.



(ii) *Fan Catchment Formula.*

In the final analysis, a fan-shaped catchment must be taken as one in which the run-off from the farthest extremity concentrates at the dam (or towards the end of the catchment) practically simultaneously. Maximum inflow occurs when the duration of the rainfall of maximum intensity equals or exceeds the time required for flow from the farthest part of the catchment to reach the dam site. It is in light of these considerations that it is possible to decide whether a particular catchment is fan-shaped or requires allowance for its elongated shape, which precludes the concentration of discharge of maximum intensity, there being an overall retardation effect.

In the case of the 1941 flood with 31½ feet depth, the duration and the intensity of the rainfall leading to this flood are unfortunately not known. For these conditions, the time concentration and the average intensity of rainfall during that period are roughly worked out to be about 12.20 hours and 1.08" per hour respectively —

Appendix I vide Appendix I. Intensity data not being available, whether these conditions were actually obtained or not cannot be strictly verified. The maximum daily rainfall in 1941 of 13.02", when compared with the average of the maximum rainfall for all years for which data are available (6.78") suggests that the figure of computed intensity may be approximately correct. The time of concentration being only 12.2 hours — which is also approximately the duration of rainfall of maximum intensity — no allowance for divergence from fan shape can justifiably be made. According to the fan catchment formula, the maximum flood discharge would be 91,500 cusecs, say, 92,000 cusecs.

(iii) *Correlation of run-offs of Tansa and Vaitarna Catchment.*

The catchment of Vaitarna is 175 sq. miles while that of Tansa 52½ sq. miles, i. e. both are small fan shaped catchments. Both are of similar topography and are adjacent to each other. Hence, rain storms of nearly equal intensity may be expected in the two catchments, with due allowance for lag. As flood discharge is proportional to the square-root of the catchment area based on the maximum flood discharge (observed plus absorption) of Tansa of 50,000 cusecs, the flood discharge of Vaitarna would be

$$50,000 \times \sqrt{\frac{175}{52.5}} = 91,500 \text{ cusecs.}$$

The correlation between the maximum daily run-off in each year, averaged, over three consecutive days, in the Tansa and Vaitarna catch-

ments was examined because for reasons given above a significant correlation may be expected. As will be seen from Fig. 2, a good correlation exists between these run-offs, the statistical equation being

$$Q_T = .0152Q_V^{1.3508}$$

When extrapolated for $Q_T = 50,000$ cusecs.—the maximum—ever at Tansa—the maximum discharge from Vaitarna catchment would be of the order of 83,600 cusecs. It is however, to be noted that simultaneous discharge data of both the catchments are not available beyond Tansa run-off mean of three consecutive days of 13,850 cusecs. = 7460 m.g.p.d. and extrapolation much beyond the observed range must be separately justified.

Recapitulating, the values of the estimating maximum discharge, as obtained by various methods, were as under:—

Method	Maximum Q
(i) From observed data corresponding to maximum observed depth of 31.5 ft.	Chezy 90,000 cs
	Manning 89,000 cs
	Lacey (General flow) 92,000 „
	Lacey (Rugosity) 101,000 „
(ii) Fan catchment formula	92,000 „
(iii) Proportional to root of catchment area	91,500 „
(iv) Correlation of run-offs of Tansa and Vaitarna	83,600 „

Having due regard to safety as to economy, a minimum discharge capacity of the weir of 92,000 cusecs appeared essential. It must be emphasized that this estimate does not provide for extraordinary cyclonic storm floods such as the one experienced in November 1948. Sufficient data about such floods not being available, it is not possible to predict their intensity and frequency.

APPENDIX I

Determination of the time of concentration of the run-off caused by the storm on 2-7-41 due to rainfall of 13.02 inches/day.

Formulae used:—

$$Q_{\max} = 1,000 K.i \quad \text{Where}$$

$$i = \frac{Rf(a)}{t+1}$$

$$\frac{t^3}{t+1} = \frac{C L^2}{K.S.R.f(a)}$$

Now $Q_m = 1,000 Ki$

Q_{\max} = maximum discharge in cusecs./1000 acres.

i = Average intensity of rainfall in inches/hour.

R = Rainfall coefficient.

$f(a)$ = A function of the area for the adjustment of the average intensity of rainfall = 0.96 in this case.

t = time of concentration in hrs.

K = the run-off coefficient = 0.84 in the present case.

$$\begin{aligned} \therefore 87,000 &= \frac{1,000 \times .84 \times i \times 175 \times 640}{1,000} \\ &= 94,000 i \end{aligned}$$

$$\therefore i = \frac{94,000}{87,000} = 1.08''/\text{hour}$$

C = a coefficient = $\phi(K,R)$

L = length of basin in miles = 45 miles.

S = Average slope of the catchment = 7.7 ft/1,000

$$\frac{t^3}{t+1} = \frac{C L^2}{K.S.R.f(a)}$$

$$\therefore t^3 = \frac{C L^2}{K.S.f(a)} \times \frac{t+1}{R}$$

$$\text{But } i = \frac{Rf(a)}{t+1} \therefore \frac{R}{t+1} = \frac{i}{f(a)} = \frac{1.08}{.96} = 1.125$$

$$\therefore t^3 = \frac{C \times 45 \times 45}{.84 \times .0077 \times 0.96} \times \frac{1}{1.125} = 2,90,000 C$$

Assuming $C = .0063$

$$t^3 = 2,90,000 \times .0063 = 1830 \therefore t = 12.20 \text{ hrs.}$$

To check value of C

$$i = \frac{Rf(a)}{t+1} \therefore 1.08 = \frac{R.96}{13.20} \therefore R = \frac{1.08 \times 13.20}{.96} = 14.85$$

A 2

$$\therefore KR = .84 \times 14.85 = 12.45.$$

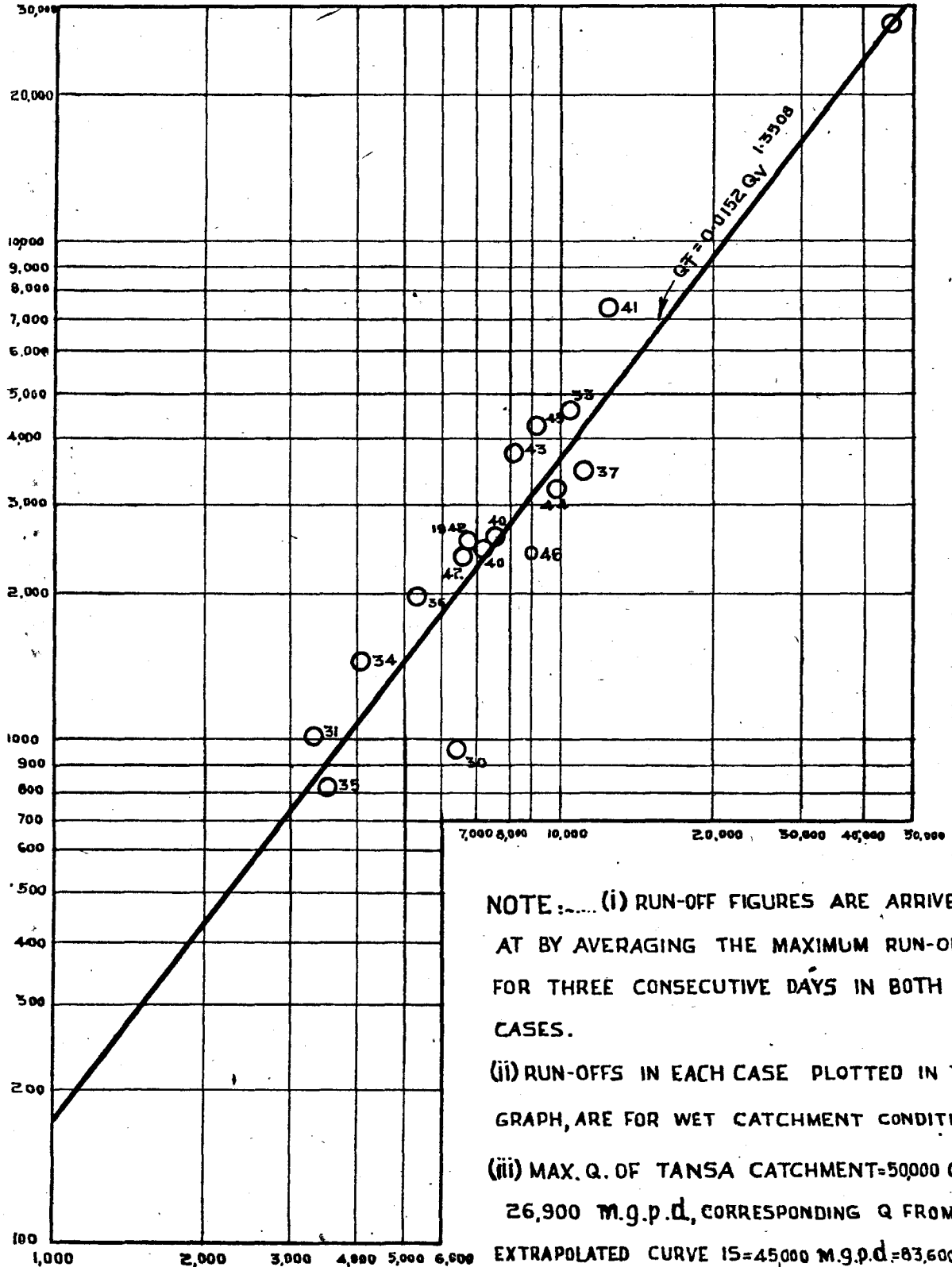
\therefore Value of C (from curve on page 45 of Richard's "Flood estimation & Control") = .0064

Thus

$$t = 12.20 \text{ hours; } i = 1.08 \text{ inches/hour and } Q_m = 87,000 \text{ cusecs.}$$

FIG. 2

SHOWING CORRELATION BETWEEN MAXIMUM
DAILY RUN-OFFS OF THE TANSA &
VAITARNA CATCHMENTS.



MAXIMUM DAILY RUN-OFF OF TANSA CATCHMENT IN MILLION GALLONS.

NOTE:..... (i) RUN-OFF FIGURES ARE ARRIVED AT BY AVERAGING THE MAXIMUM RUN-OFFS FOR THREE CONSECUTIVE DAYS IN BOTH THE CASES.

(ii) RUN-OFFS IN EACH CASE PLOTTED IN THIS GRAPH, ARE FOR WET CATCHMENT CONDITIONS.

(iii) MAX. Q. OF TANSA CATCHMENT = 50,000 C.S. = 26,900 M.G.P.D., CORRESPONDING Q FROM THE EXTRAPOLATED CURVE IS = 45,000 M.G.P.D. = 83,600 C.S.

MAXIMUM DAILY RUN-OFF OF VAITARNA CATCHMENT IN MILLION GALLONS.

2. (b) *Correlation ratio between the yield of Tansa and Vaitarna Catchments.*

For determining the total yearly yield at Vaitarna for which direct data are not available, the correlation between yields of Tansa and Vaitarna was attempted by the Special Engineer.

Statistical analysis of the available yield and rainfall data of Tansa and Vaitarna catchments is undertaken by the Station and the findings thereof will be compared with the results of the investigation of the Special Engineer for arriving at a final conclusion.

2. Coefficients for Conversion of Surface Velocities to mean Velocities in Natural Streams.

It is common to estimate mean velocity on a vertical from the observed surface velocity with the help of some appropriate multiplying factor p . For small streams with rough beds the maximum value of the coefficient

$$p = \frac{\text{mean velocity}}{\text{surface velocity}}$$

is 0.89, minimum 0.78, and the average 0.84. For average streams (American) in moderate flood $p=0.90$ and in floods 0.90 to 0.95.⁽¹⁾ Observations on Hemavathi river in Mysore State, with fairly soft bed and vertical banks the coefficient varies within 0.85 to 0.88. In cauvery at Chunchinkatta with a rocky bed and sloping banks the coefficient ranges from .77 to .79⁽²⁾ On the Indus p varies between .80 and .89.⁽³⁾ The coefficient is thus not constant but variable, — the variation depending on the stage, depth and nature of the bed. It varies directly as the depth and velocity and inversely as the rugosity.

Available data in the variation of 'p' with depth and bed and side roughness is set out below.⁽⁴⁾

Table I

Value of 'p' ($= \frac{V_m}{V_s}$) for different values of n and depths.

Depth in ft.	Rugosity coefficient n						
	0.03	0.025	0.020	0.0175	0.015	0.013	0.010
0.9	—	—	0.83	0.86	0.88	0.89	0.91
1.0	0.78	0.82	—	—	—	—	—
1.1	—	—	0.84	0.87	0.89	0.90	0.91
1.25	—	—	0.85	0.87	0.89	0.91	0.91
1.50	—	—	0.87	0.88	0.90	0.91	0.92
2.0	0.80	0.86	—	—	—	—	—
3.0	0.83	0.88	—	—	—	—	—
5.0	0.85	0.89	—	—	—	—	—
10.0	0.86	0.90	—	—	—	—	—
15.0	0.87	0.91	—	—	—	—	—
20.0	0.88	0.92	—	—	—	—	—

(1) Parker, 'Control of Water', p. 55.

(2) 1946 Report of Hydraulic Research Station, Krishnarajsagar, p. 22.

(3) 1944 Report of Develop & Research Division, Sind, p. 15.

(4) Parker, 'Control of Water', p. 64.

Table II. Values of the rugosity coefficient 'n' in Kutter's and Manning's formula frequently exceed 0.03. *vide* Table II below⁽⁵⁾ and data are lacking for 'n' greater than 0.03.

Table II

Values of 'n' for natural Streams.

No.	Surface	Condition of stream bed			
		Best	Good	Fair	Bad
1	Clean, straight bank, full stage, no rifts or deep pools.	0.025	0.0275	0.030	0.033
2	Same as (1) but some weeds & stones.	0.030	0.033	0.035	0.040
3	Winding, some pools and shoals clean.	0.033	0.035	0.040	0.045
4	Same as (3), lower stages, more ineffective slopes & sections.	0.040	0.045	0.050	0.055
5	Same as (3), some weeds & stones.	0.035	0.040	0.045	0.050
6	Same as (4), stony sections.	0.045	0.050	0.055	0.060
7	Sluggish river reaches, rather weedy or with very deep pools.	0.050	0.060	0.070	0.080
8	Very weedy reaches.	0.075	0.100	0.125	0.150

According to Grumsky,⁽⁶⁾ 'p' depends only upon the ratio

$$\frac{W}{d} \left(= \frac{\text{width of stream}}{\text{mean depth}} \right)$$

Fig. 3 This dependent variation is shown in Fig. 3.

The coefficients however need to be varied with those actually obtaining on the site, say, by vertical velocity curve or the two point method, but the present note has been written only by way of interim information on the subject.

(5) Pages 373 and 863 Engineering News, Vol. 75, 1916.

(6) Trans. Am. Soc. of C.E., Vol. 66, Page 123.

3 Water Supply for the Proposed Steam Power Station of the South Gujrat Electric Grid near Surat (Bombay)

1. Problem

To meet the cooling water requirements of the proposed steam power station of the South Gujrat Electric Grid, the Electrical Commissioner with the Government of Bombay proposed a pumping station on the left bank of the Tapti near Utran—Fig 4 about 2½ miles downstream of Surat. The Station advice was sought on the site for locating the pump house.

Fig. 4.

To ensure that the existing flow conditions would not be adversely affected by the contemplated pump house and a deep perennial channel free from trouble due to silting would be maintained, it was suggested that the shifting of the pump house site about ½ a mile upstream of the original location at the transition where the main flow of the river crosses from the left bank to the right, should be considered.

It was found that due to proximity of the Surat City, sufficient space for the Power Station would not be easily and economically available. Acquiring land for railway siding would be costly and disposal of ash would be difficult. It was therefore considered impracticable to shift the pump house and the power station site upstream by about half a mile as desired.

Fig. 4.

An alternative site on the right bank of the Tapti upstream of Surat near Utran—Fig. 4—was therefore proposed to be examined for its potentiality to yield sufficient, constant and permanent water supply.

2. Data

Sufficiency, constancy and permanency of the yield depends on several factors such as changes in the river bed and bed material in the vicinity, homogeneity and compactness of the strata, water table at low water levels in the river etc. All these factors have to be carefully examined before deciding in favour of the site. Shifting courses of the deep channel above and below the proposed site have to be studied from the plans of all the years for which data are available, and possibility of a layer of fine silt depositing with the first and last floods, thereby reducing the percolation water, have to be properly investigated by analysing the silt. Sand samples in the trial borings have also to be analysed to work out the transmission coefficient and a couple of cross-sections at half mile interval above and below the site scrutinised to verify the homogeneity of the water bearing strata.

All the data mentioned above together with the figures of immediate and contemplated future water requirements were asked for. The scrutiny and study of the data received, led to the following observations.

FIG. 3
VARIATION OF P $\left(\frac{V \text{ MEAN}}{V \text{ SURFACE}} \right)$
WITH $\frac{W}{D}$ $\left(\frac{\text{WIDTH}}{\text{MEAN DEPTH}} \right)$

SCALE: HOR. 1" = 15'
VER. 1" = 0.15'

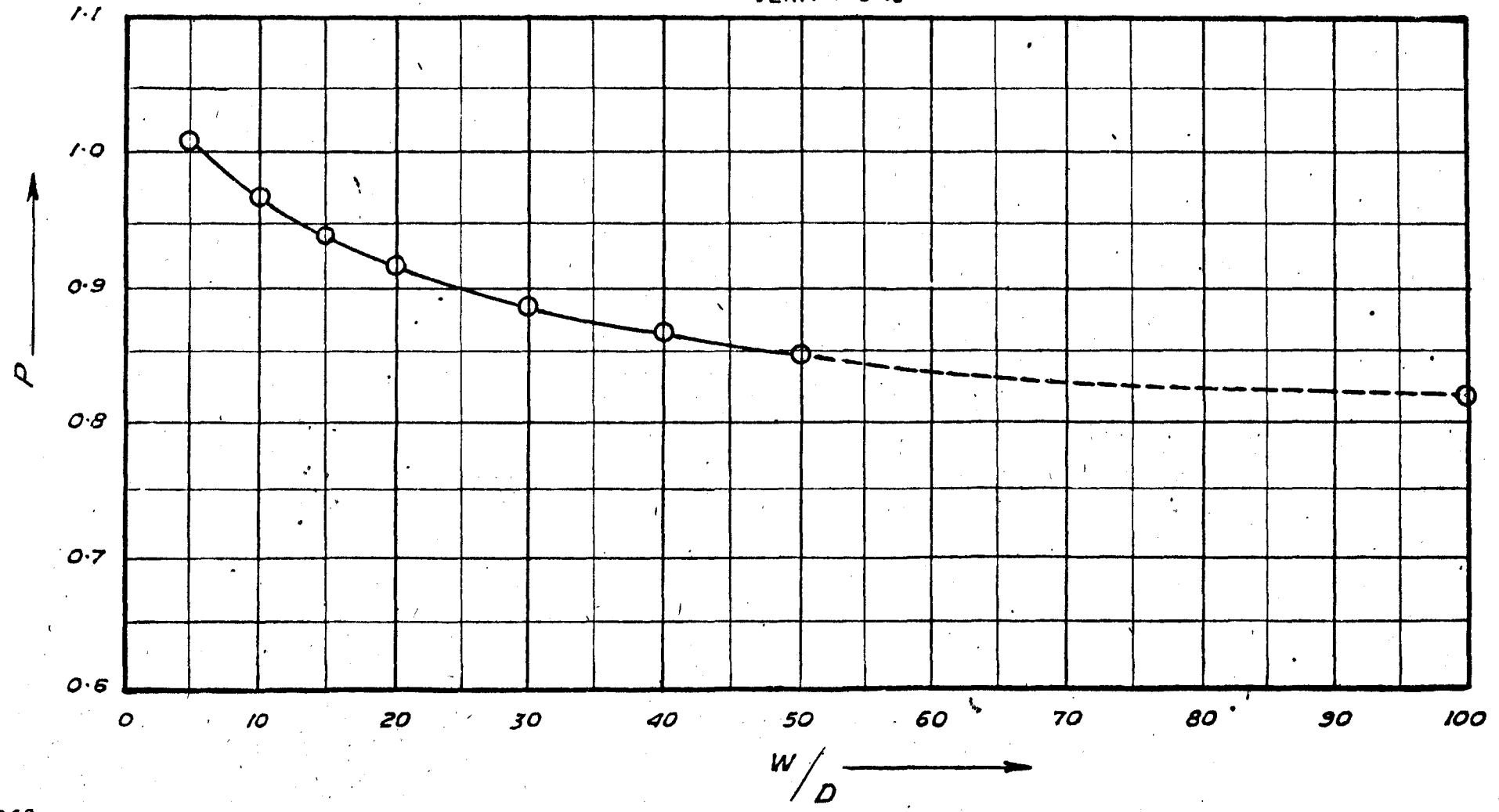
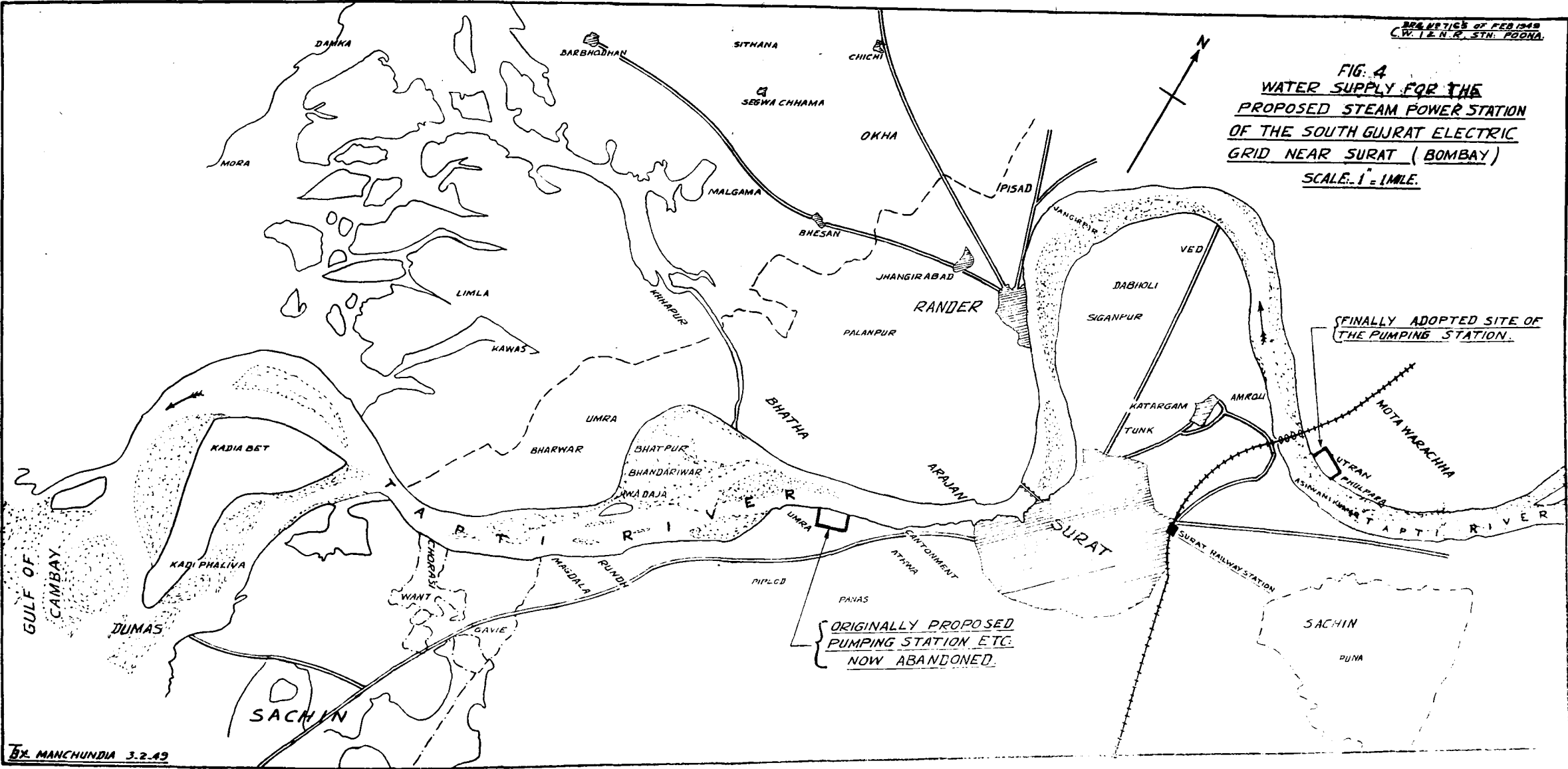


FIG. 4
WATER SUPPLY FOR THE
PROPOSED STEAM POWER STATION
OF THE SOUTH GUJRAT ELECTRIC
GRID NEAR SURAT (BOMBAY)
SCALE: 1" = 1 MILE.



3. *Analysis of data*

(a) *Yield tests*: It was apparent that the yields obtained were much in excess of the requirements — one of the bores with a small operating head discharged as much as 10 to 18 times the desired maximum.

In exhaustive tests the desired daily yield should be obtainable in 8 working hours with reasonable depression head and without permanent lowering of the water table. In the present case the depression heads had been so small and the consequent discharges so copious that though the tests were not sufficiently prolonged as they should be there did not appear much cause for apprehension regarding the availability of the required discharges. Also the discharge can further be augmented, if required, by safely increasing the depression head after providing a suitably designed inverted filter of stone over the bed of the well.

A note of caution, however, was sounded against placing entire reliance on the few tests made. It was emphasised that additional borings and yield tests at the proposed final sites of the wells would still be necessary as was evident from the available yield and bore-data showing that the strata formations and the yields at several bores within only about 400 ft. of each other varied considerably.

(b) *Type of wells*: Tube wells being much cheaper than and comparable in respect of discharging capacity with the percolation wells, were recommended for consideration.

(c) *Water-table*: The water table appeared to be at about the water level in the river. As even in years of worst rainfall, the Tapti does not totally dry up at Utran, the well will not be required to be taken too deep nor will the suction head be excessive.

(d) *Shifting courses of the deep channel above and below the proposed site*:

These data were not available. Utran, however, happens to be on an unnaturally sharp bend of the river caused by scour resistant hard formation of yellow soil. The changes in the river configuration here are therefore expected to be limited. The shifting of deep channel to the extent of jeopardizing the water supply is therefore not much likely.

(e) *Possibility of deposition of fine silt before and after the flood season*

Carpet of fine silt is found to have a pronounced effect on the percolation of water to the sub-surface water bearing strata especially at lesser depths as has been the experience at Ahmedabad, Ujjain etc.

No such deposition is found at least in the vicinity of the sandy island on which the bores were taken, and therefore likelihood of reduction in water supplies on this account appeared to be remote.

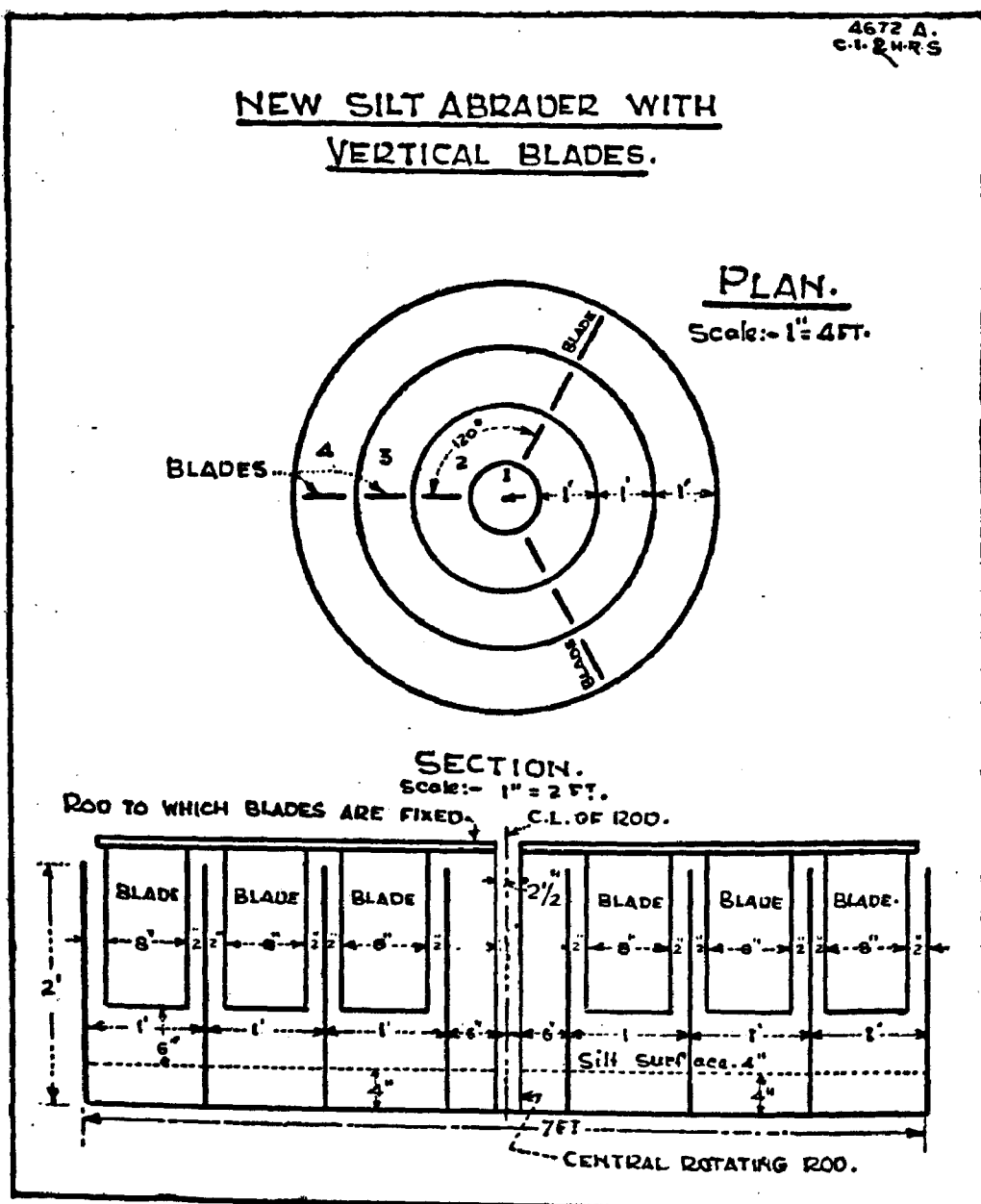
(f) *Analysis of sand supplies in the trial bores:*

It was desired to analyse the bore samples and work out the transmission coefficient and porosity, thus enabling the computations for ascertaining the size of the well necessary to ensure the particular discharge under specified operating head and through a limited zone of influence. The theoretical treatment was not however possible due to considerable diversity of the water bearing strata. This was not so important as actual yield tests were satisfactory.

Recommendations

In general the yield tests were reassuring and satisfactory. It was therefore recommended that the pumping station should be located at Utran as it would not have to face any difficulties about inadequacy of water supply.

Fig. 1.



B--THEORY OF FLOW AND DESIGN OF CHANNELS

1. Flow in rivers and canals.

(ii) FLOW IN CHANNELS IN INCOHERENT AND COHERENT ALLUVIUM

4. *Rate of Attrition in channels.*

The experiment, started in September 1938, to determine how far attrition was the true explanation of the progressive reduction of size of bed materials from the hills to the sea, was carried out in a circular tank in which water was revolved with a uniform velocity and hence experiments could be done with only one velocity at a time. These experiments have been described in the 1946 Annual Report; the description is repeated after adding this year's data—to make the note complete.

1943 Apparatus

In order to expedite results, a circular tank 7 ft. in diameter and 2 ft. deep was divided into 4 compartments. Vertical blades were spaced 120° apart, fixed to a central revolving shaft, with the bottom edge of the blades 10" above the bottom of the tank as shown in fig. 1. The mean velocity of the water in the three compartments was 2.38 ft./sec., 4.16 ft./sec. and 5.93 ft. sec. respectively.

Fig. 1.

This experiment was started on 3rd February 1943 using Brahma-putra sand, passing through a 60 mesh sieve and retained on a 80 mesh sieve; so that the range of variation of grades of sand in the mixture was less than normal.

$$m \text{ of original sand} = .185 \text{ mm.} \pm .10 \text{ (or } \pm 60\%)$$

$$m \text{ of sand used in Expt.} = .254 \text{ mm} \pm .063 \text{ (or } \mp 25\%)$$

The sand thus obtained was placed in each compartment in the tank to a depth of 4" (about 1150 lbs.) and water was maintained at a constant level.

The abrader was run for 1460 hours, Sand samples were collected from each compartment and after thoroughly mixing the sand, they were analysed by the Puri siltometer. The observations were repeated after 3770 hours (1944), 5650 hours (1945), 6550 hours (1946) and 8097 hours (1948) and sand samples were analysed for each period.

Table 1. Table 1 shows the percentage fraction of various grades to total quantity.

The data of "reduction in volume of sand": Velocity after 1460 hours, was analysed and tentative straight line relation was obtained. Similar

relations were also obtained after 3770 hours, 5650 hours, 6550 hours and 8097 hours of working.

Reduction in volume after 1460 hours	=	.00137 (V-2) ^{.39}
" " " " 3770 "	=	.00202 (V-1.2) ^{.39}
" " " " 5650 "	=	.0030 (V-.92) ^{.39}
" " " " 6550 "	=	.0038 (V-.89) ^{.36}
" " " " 8097 "	=	.0055 (V-.83) ^{.35}

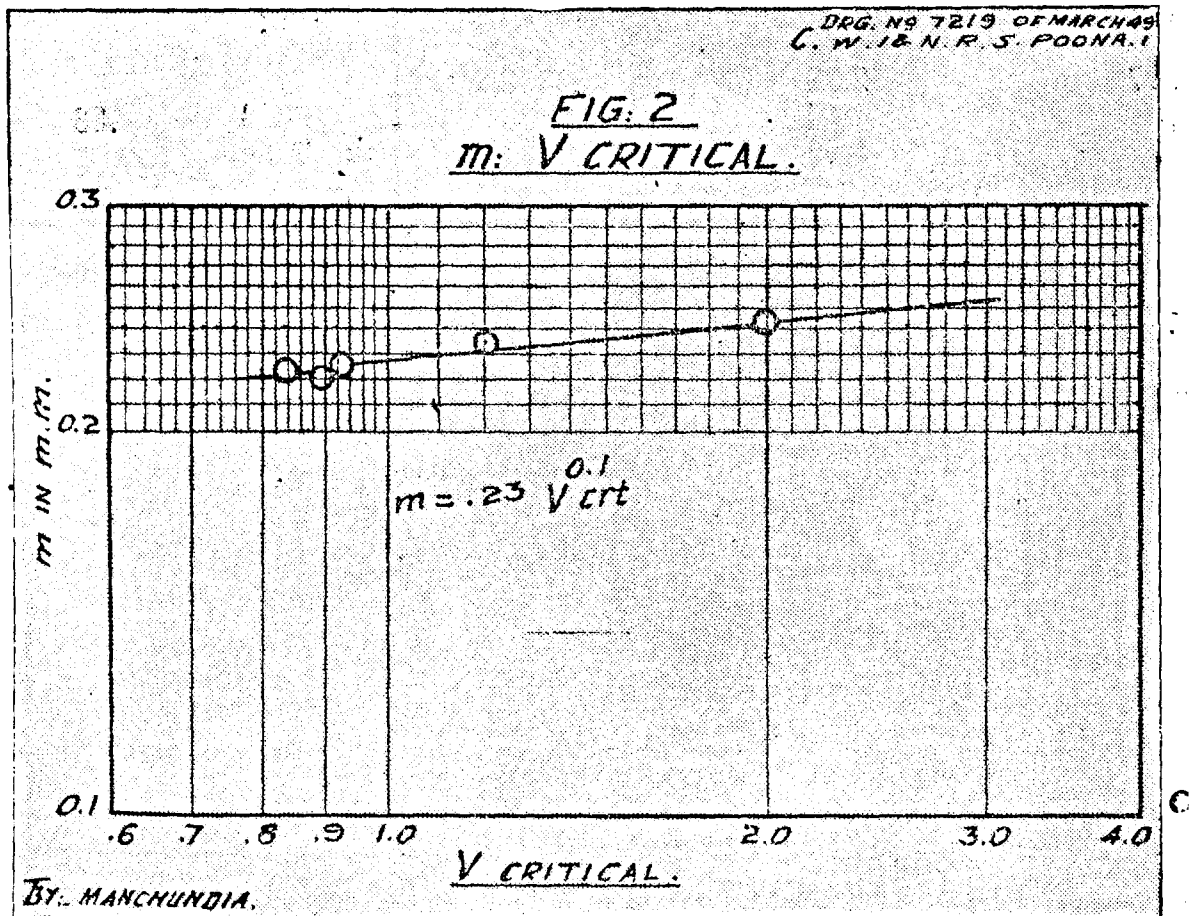
The data after 1460 hours indicates that the attrition started at velocity 2 ft./sec. The critical velocities to start abrasion after 3770 hours, 5650 hours 6550 hours and 8097 hours were 1.2, .92, .89 and .83 ft./sec respectively.

As size of sand decreased, the critical velocity to start attrition also decreased, thus showing that the critical velocity depends upon the grade of material, a lower critical velocity being required to start abrasion of finer grade material.

An approximate relation between \bar{m} and V critical, viz.

$$\bar{m} = 0.23 V_{crt}^{0.10}$$

Fig. 2 is shown in fig. 2 where \bar{m} = mean size in the three



compartments at the end of the five time intervals, shown
Table 2 in table 2.

Table 2

Showing critical velocities and mean diameter in m.m. at different periods.

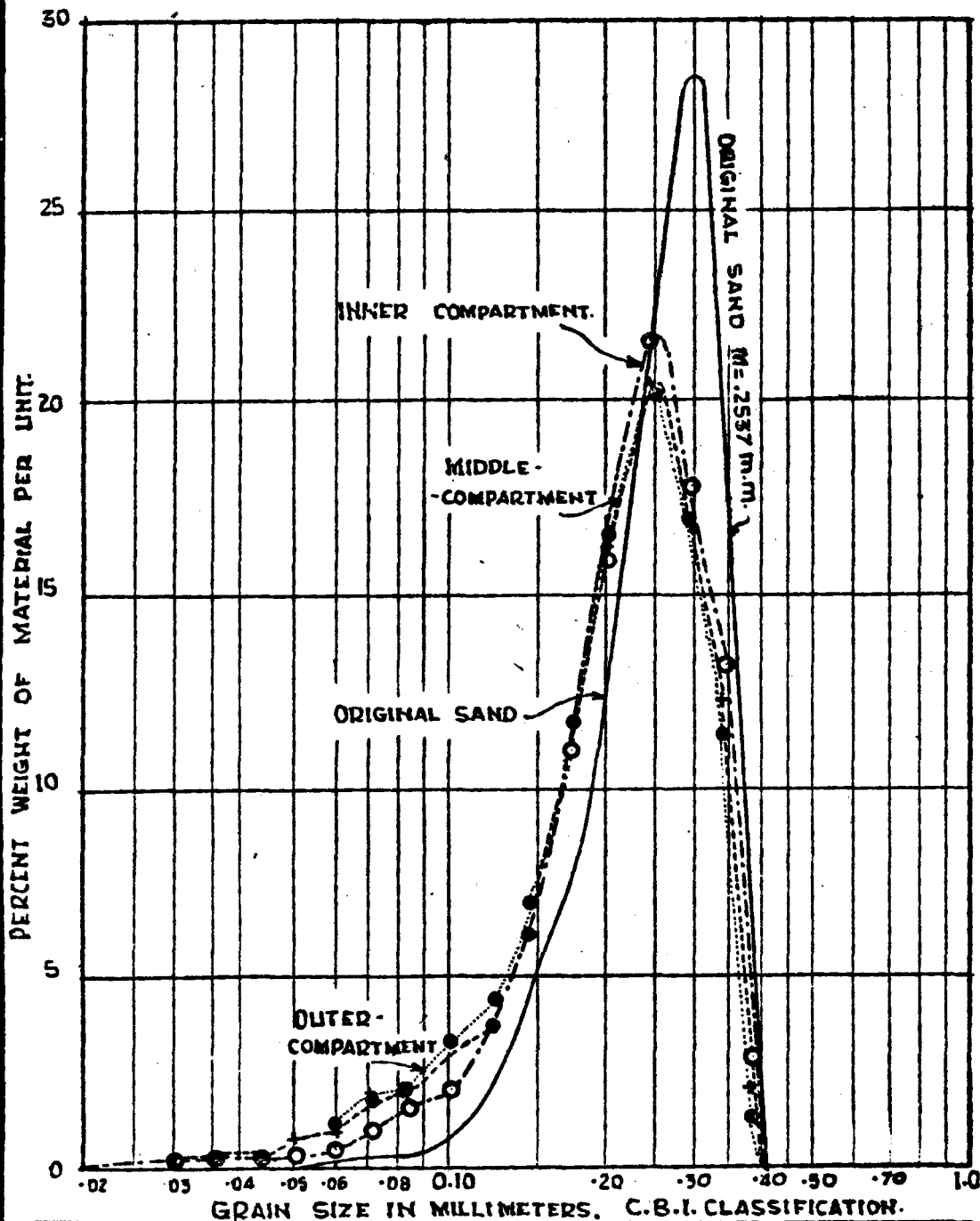
Period of working	Velocity in ft./sec.	m in mm. in each compartment.	Mean m in mm.	V _{crt} ft./sec.
1460	2.38	.2487		
"	4.16	.2438	.2444	2.0
"	5.93	.2408	± .0032	
3770	2.38	.2419		
"	4.16	.2358	.2368	1.2
"	5.93	.2328	± .00311	
5650	2.38	.2345		
"	4.16	.2267	.2280	.92
"	5.93	.2228	± .0045	
6550	2.38	.2317		
"	4.16	.2240	.2250	.89
"	5.93	.2194	± .005	
8097	2.38	.2308		
"	4.16	.2230	.2243	.83
"	5.93	.2190	± .0085	

FIG. 3

SIZE DISTRIBUTION CURVES (BY PURI SILTOMETER)
OF
BRAMHAPUTRA SAND COLLECTED IN
SAND ABRADER AFTER 8097 Hrs.

REF.~

ORIGINAL SAND	m = .2537 m.m.	—————
INNER COMPARTMENT.	m = .2308 "	-○- - - -
MIDDLE "	m = .2230 "	-+ - - - -
OUTER "	m = .2101 "	-●- - - -

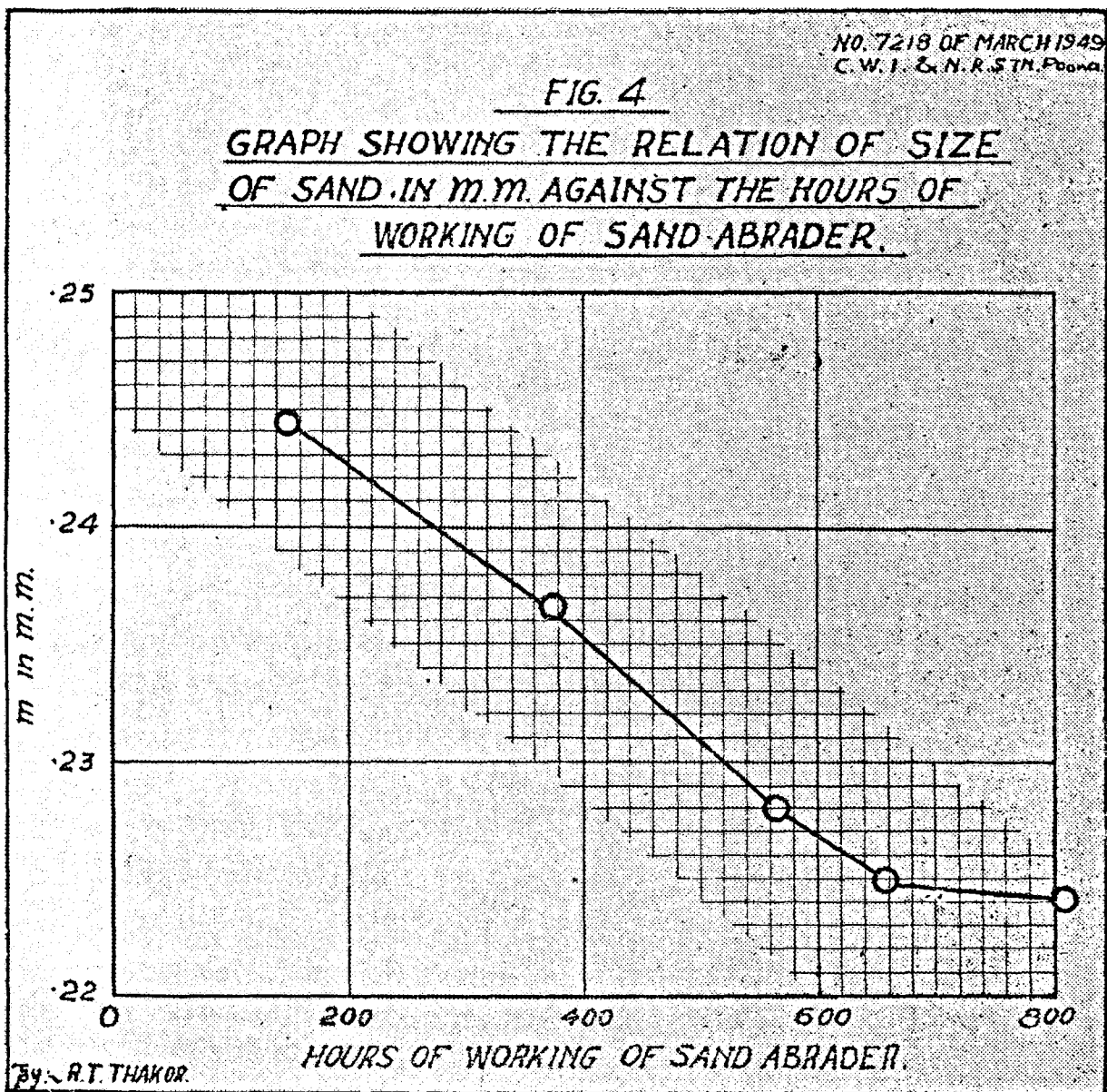


MEDIUM SILT 0.0156 - 0.0312	COARSE SILT 0.0312 - 0.0625	VERY FINE SAND 0.0625 - 0.125	FINE SAND 0.125 - 0.167	MEDIUM FINE SAND 0.167 - 0.25	MEDIUM SAND 0.25 - 0.50	COARSE SAND 0.50 - 1.0.
--------------------------------	--------------------------------	----------------------------------	----------------------------	----------------------------------	----------------------------	----------------------------

fillon

Fig. 3 Figure 3 shows size distribution curves of sand at the end of 8097 hours as compared with original size distribution. Abrasion is continually decreasing, as will be seen from

Fig. 4 fig. 4 and the rate is now so low that it is not considered worthwhile to continue these experiments.



5. Protecting the left bank of the Katjuri River at Khannagar (near Cattack) above the Katjuri Railway Bridge (Orissa).

According to the Director's initial advice based on previous experience and later confirmed by model tests, (1) a 1,400 ft. long spur pointing 60° upstream was constructed at the burning ghat with an apron round the nose before the floods of 1947. During the floods, spur was subject to a severe attack and a deep scour-hole formed at its nose. There, however, being clay in the bed, the scour-hole did not form to the natural angle of

repose of sand, with the result that the apron did not launch as it should and some length of the nose was washed out — see photos Nos. 5481 and 5486. Apart from this damage, however, the action of the spur in protecting the endangered left bank was very satisfactory. Detailed data of river bed and a channel scoured towards pier No. 12 from left abutment of the railway bridge — see photo No. 5496 — showed that main channel would be well away from the left bank in course of time.

Conditions after the floods have been asked for and are awaited.

6. Limitation in producing discharging capacity and flow conditions in full scale volute siphons based on results from scale models,

The volute siphon design was developed by Shree Ganesh Iyer, Director, Hydraulic Research Station, Mysore. This design was adopted for spillways in the Hirebhasgar Dam (Mysore). The construction of the Dam has been completed and the volute siphons have operated for the first time in August.

The design is novel in many aspects. The operating head i. e. difference between the crest of the bellmouth and the centre of the outlet is 57 ft. and the diameter of the vertical shaft is 18.0 ft. The discharge of each siphon was estimated at 12,000 cusecs and eleven such siphons were constructed to pass 1,32,000 cusecs, out of the maximum probable flood of 2,00,000 cusecs. These are perhaps the largest high-head siphon spillways

(1) Annual Report 1947, Page 43.

constructed so far in India and keen interest was evinced in their performance in the Engineering Circle.

Experiments with scale models of volute siphons as constructed at Markonhally (Mysore) were carried out at this Research Station⁽¹⁾ and from the study of these models improvements were suggested in some parts of the design. These were subsequently incorporated by Shree Ganesh Iyer in designing the volute siphons at Hirebhasgar.

The question of discharging capacity and performance of the volute siphons at Hirebhasgar was referred to this Station by P.W.D. Minister (Mysore) in 1945 and experiments were therefore carried out with a 1/24 scale model — as originally designed and modified later by Shree Ganesh Iyer. The exact shape and the volute dimensions were unfortunately not finally fixed till after a much later period. The volutes in the model were prepared according to the dimensions originally supplied. This discrepancy however, does not affect the priming depth.

In the 1/24 scale *model* the discharge of each siphon was equivalent to 11,900 cusecs for water level in the reservoir equivalent to 1.0 ft. above the crest of the bellmouth (formed at R.L.1776.0). Experiments done at Krishnaraj Sagar Research Station too, gave similar results. Thus if conclusions were to be entirely based on these results, the discharging capacity in the full scale siphon would be 11,900 cusecs.

Pressure observations made at this Research Station however, disclosed that pressures at the base of the bellmouth and also along the inside of the lower bend when converted to the prototype indicated cavitation in these regions and thus the equivalent discharge figures would not be correct and similitude between the model and full scale would no longer be valid.

In order to arrive at the discharging capacity of the full scale siphon the maximum negative pressure of -25 ft. was *assumed* (-27 ft. at sea level and lesser i. e. -25 and so on for higher elevations. This allows for vapour pressure and variations in pressure observed in the siphon), at the base of the bellmouth. Actual position of the maximum negative pressure will naturally depend upon the design of the siphon. This gave a discharge of each siphon of 9,800 cusecs. Calculations also indicated that the outlet would not run full. It may be pertinent to point out that in the scale models this possibility was not indicated as all the models were discharging full bore. It was, therefore, recommended by this Station that the outlet might be reduced as shown in fig 1, so as to prevent cavitation by causing back pressure.

Fig. 1

Station that the outlet might be reduced as shown in fig 1, so as to prevent cavitation by causing back pressure.

(1). Research Publication No. 10, A.R.T 1945, pages 72-78.

KATJURI RIVER AT KHANNAGAR SPUR—CUTTACK
(ORISSA).



PHOTO No. 5481 Looking downstream from left bank showing the damaged nose of the Khannagar Spur. Note also silting in the foreground, where previously there was a deep water channel.

KATJURI RIVER AT KHANNAGAR SPUR—CUTTACK
(ORISSA).



PHOTO No. 5486 Showing a close view of the damaged nose of Khannagar Spur. Taken from the downstream slope of the Spur looking towards the nose.

KATJURI RIVER AT KHANNAGAR SPUR—CUTTACK
(ORISSA).

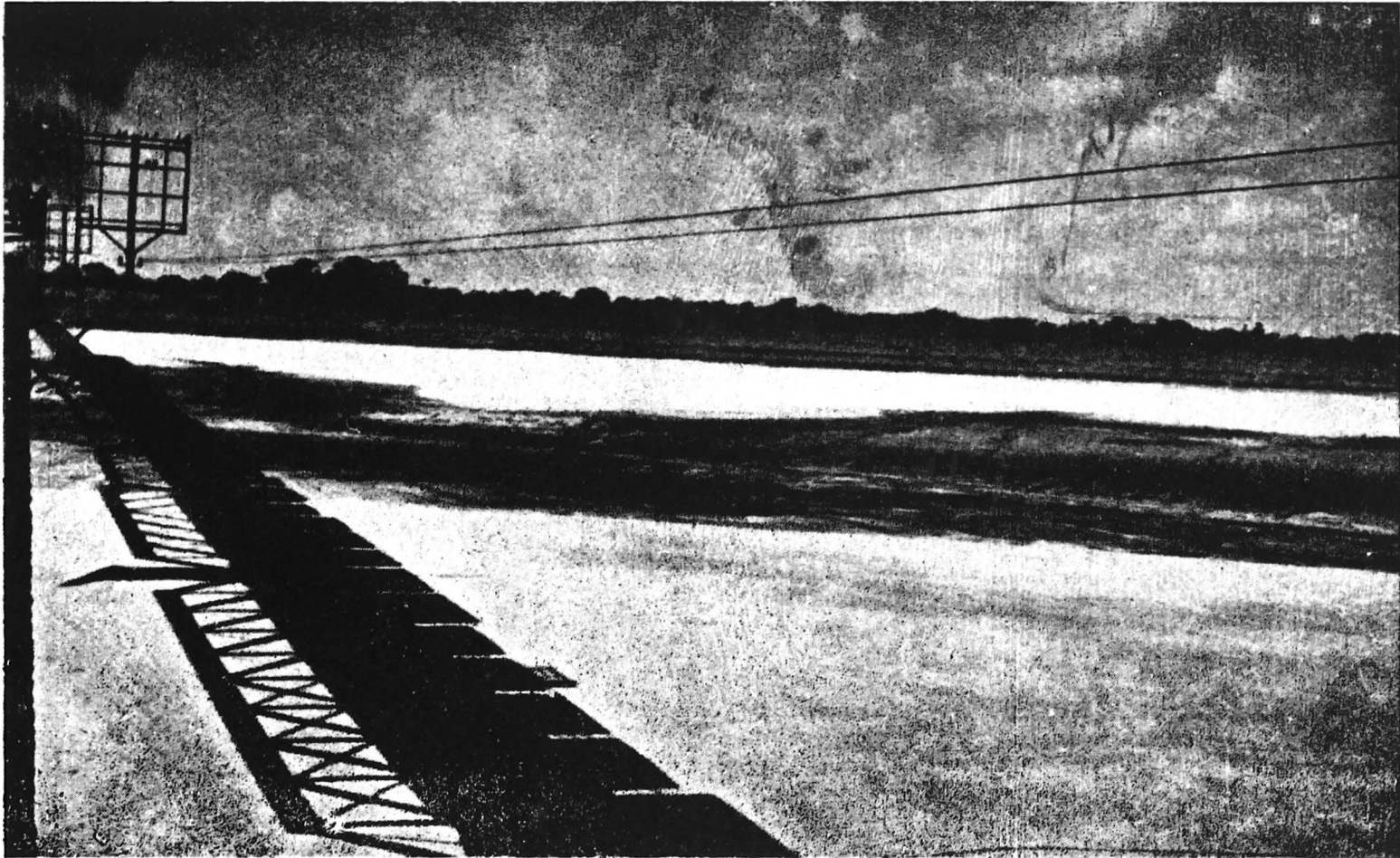
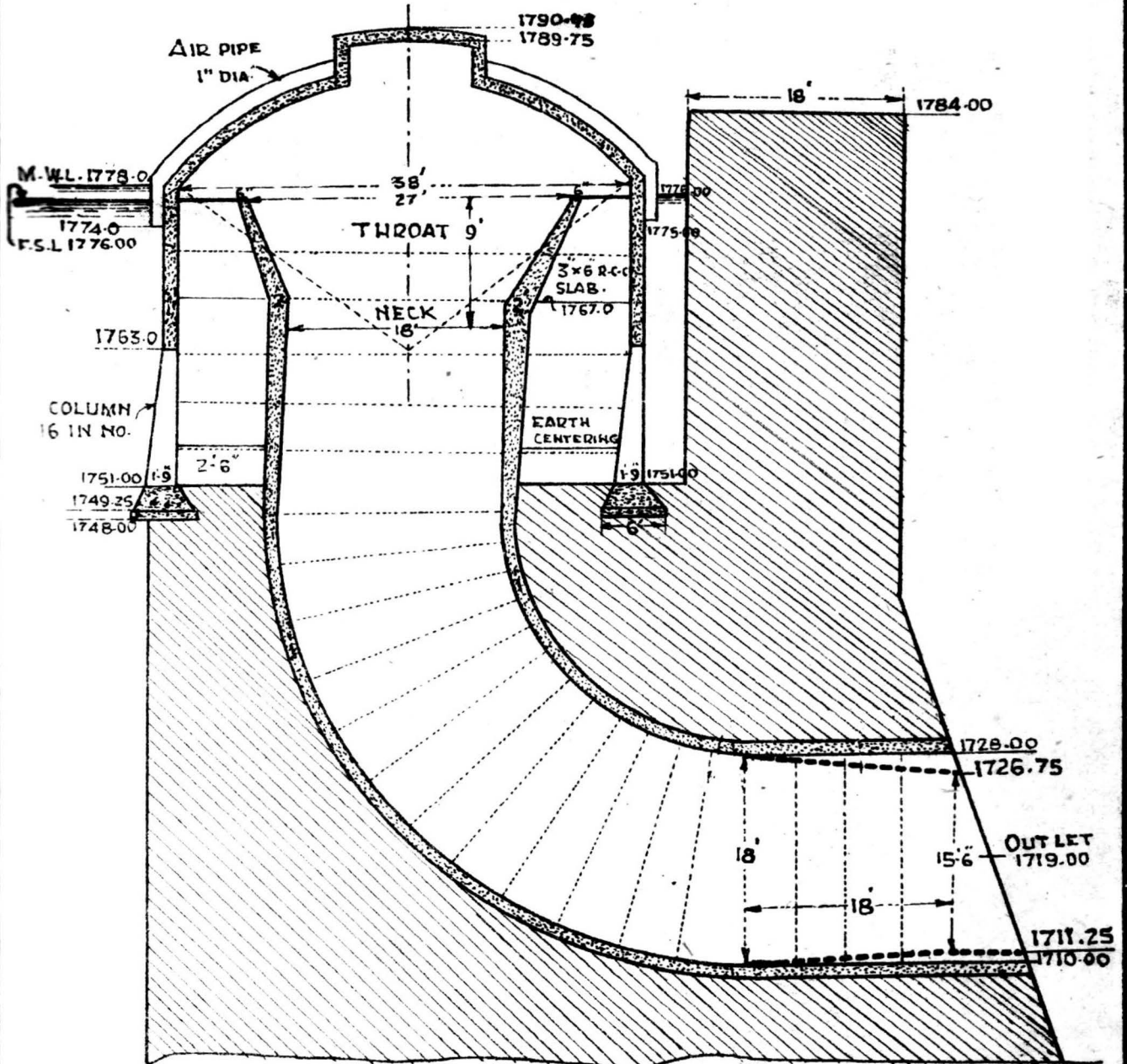


PHOTO No. 5496. Taken from the left abutment of the Katjuri Railway Bridge looking towards the right bank. Note the scour in the river bed as manifested by exposed clay bank in the background which runs to pier No. 12 from the left.

FIG. I

SIPHON HIREBHASGAR DAM.

Scale:- 1" = 18FT.



It was, however, decided later when Shree A. N. Khosla, the Chairman, CWINC inspected the volute siphons at Hirebhasgar that any of these recommendations may be deferred till the actual behaviour of the full scale siphons was known.

The operation of these siphons as reported so far confirms the conclusions reached and there is a close agreement in discharge figure as predicted from assumption of limiting negative pressure at the base of the bellmouth and as observed. The discharge of 18 ft. diameter volute siphon as observed, varied between 9,000 cusecs and 10,000 cusecs. The outlets too were partially discharging.

As experiments were carried out in Poona only with one scale model the priming depth of the full scale could not be estimated, as usually three different scale models are required for such prediction. The priming depth obtained in the Jog Siphons was 1.55 ft. and is slightly higher than anticipated, but by proper design of the bell-mouth and volutes, this could be reduced to under one foot.

The scale models are thus an aid to engineering skill but limitations of the models should always be considered before predicting the behaviour of the full scale work.

7. Redistribution of sand load and discharge between the Mahanadi and Katjuri system of Cuttack (Orissa).

1. *Introduction*:—During the year under report, in continuation of the experiments reported in A.R.T. 1947, experiments were conducted .

- (1) to test if the Khannagar spur would be necessary in view of the pitched embankment on the left bank of the Katjuri—Series III.
- (2) to evolve the shortest embankment on the left bank of Katjuri river below Naraj which will give the desired results of deep water channels on the southern side of Cuttack—Series IV.

2. *Experiments and their results*:—to enable comparison, all the experiments were carried out under the same conditions of flood discharge and sand charge.

Before the construction of the Khannagar spur was undertaken, a point was raised as to whether spur was at all necessary for the protection of Khannagar bank, particularly in view of the embankment on the left bank downstream of the Naraj weir proposed by this Station.

Experiments were therefore done with the embankment alone. Fig. 2 shows the low water channel obtained with embankment. From this it will be seen that though deep water channel conditions have improved above Cuttack, yet the general curvature of flow and the channel below the Kuakhai offtake remained practically the same with the result that at Khannagar, diving flow and consequent bank under-cutting and erosion persisted. This meant that even inspite of the embankment, the Khannagar spur was essential for giving local protection to the Khannagar bank. Further the pitched embankment and other alternatives proposed for redistribution of sand charge would take some time to come into operation. Whereas, Khannagar protection was an immediate necessity; it was therefore, recommended that the Khannagar spur should be immediately constructed.

Having decided this point experiments of Series IV, to evolve short-embankment which would serve the purpose were undertaken; with the Khannagar spur constructed.

In Series III, a number of pitched embankments were tested and modified from time to time in conformity with visual observations noted during the experiments.

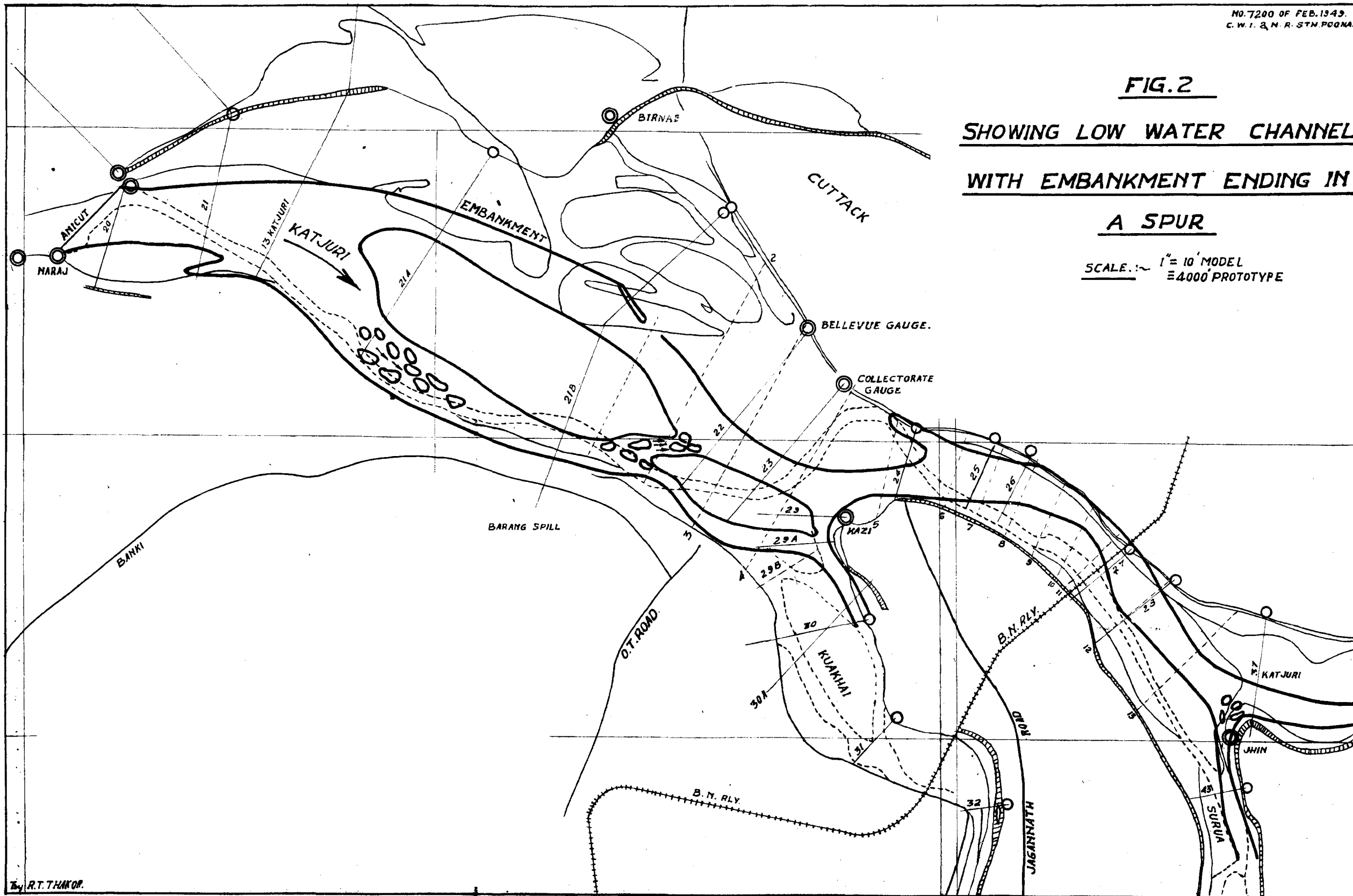
Figs. 3 and 4 show the various embankments tested in the model with Naraj weir waterway reduced by 15.5% from its right and the Banki Road spill discharge reduced by 50%. The embankments were provided with high level opening at its crossing with the Birnasi gap channel. The length of the high level opening in the embankment was equal to the Biranasi channel width with the crest level at R. L. 82.⁽¹⁾

The best result of channel conditions was obtained with embankment No. 7. The low water channel as obtained after experiment with embankment No. 7 is shown in Fig. 5 and photo No. 5229. It will be noticed that the low channel crossed over to the left bank a little below the Collectorate gauge whereas under existing conditions it crosses over well downstream.

(1) A. R. T. 1947, page 35.

FIG. 2
SHOWING LOW WATER CHANNEL
WITH EMBANKMENT ENDING IN
A SPUR

SCALE: 1" = 10' MODEL
= 4000' PROTOTYPE



R.T. THAKOR.

NO. 7138 OF FEB. 1949
C.W. I. & N.R. STR

FIG. 3

SHOWING THE VARIOUS
EMBANKMENTS ENDING
IN A SPUR TESTED
IN THE MODEL.

SCALE 1" = 4000'

REFERENCES:—

- | | |
|------------------|-----------|
| EMBANKMENT NO. 1 | ————— |
| " NO. 1a | - - - - - |
| " NO. 2 | - · - · - |
| " NO. 3 | · · · · · |

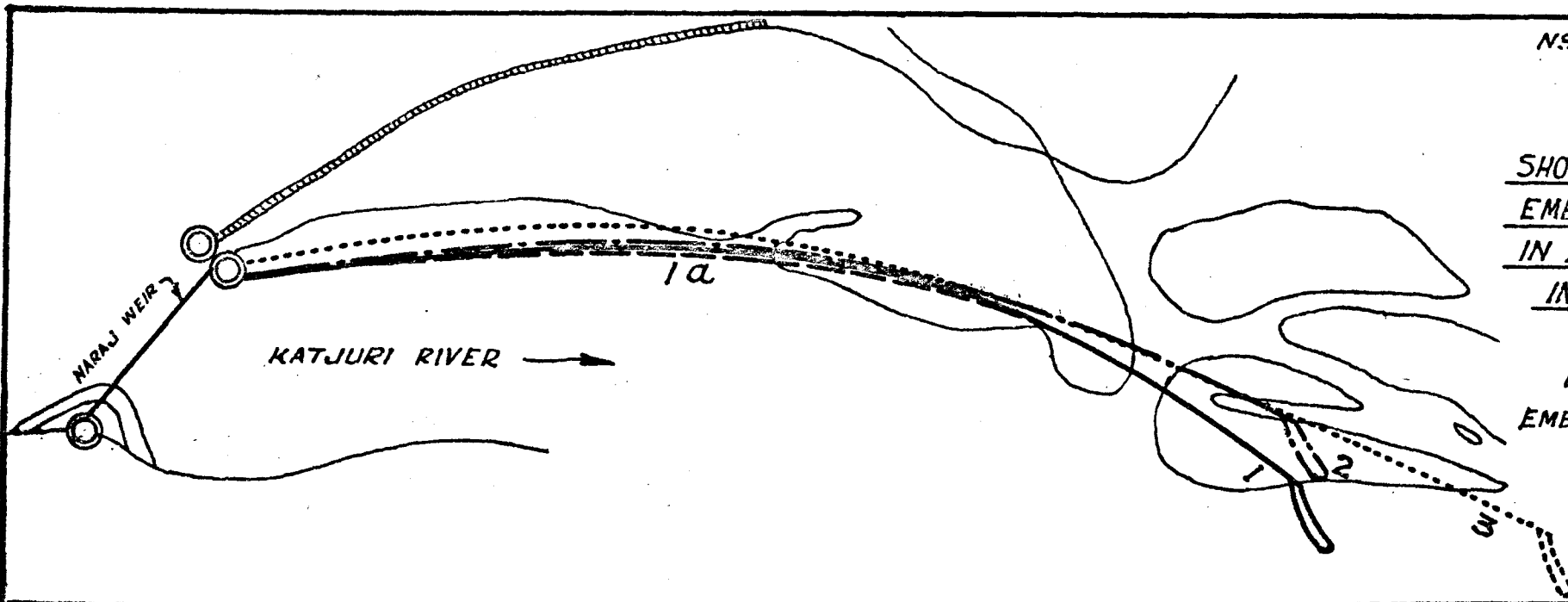


FIG. 4

SHOWING THE
EMBANKMENTS TESTED
IN THE MODEL.

SCALE 1" = 4000'

REFERENCES:—

- | | |
|------------------|-----------|
| EMBANKMENT NO. 4 | - - - - - |
| " NO. 5 | ————— |
| " NO. 6 | - · - · - |
| " NO. 7 | · · · · · |

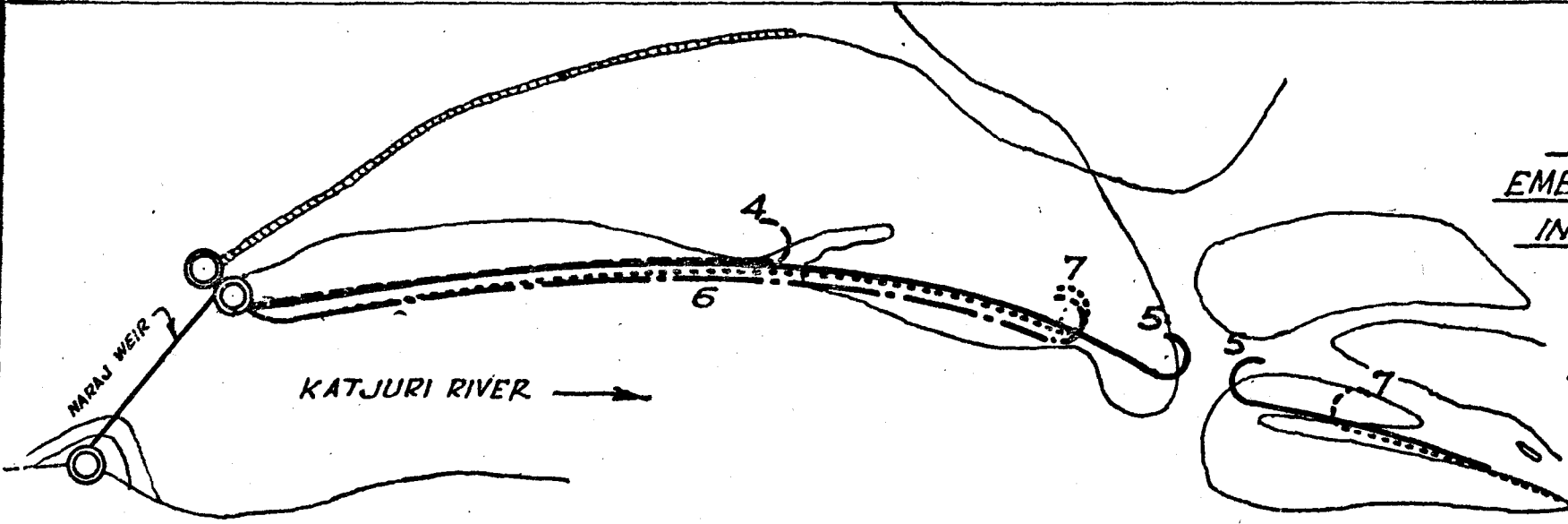
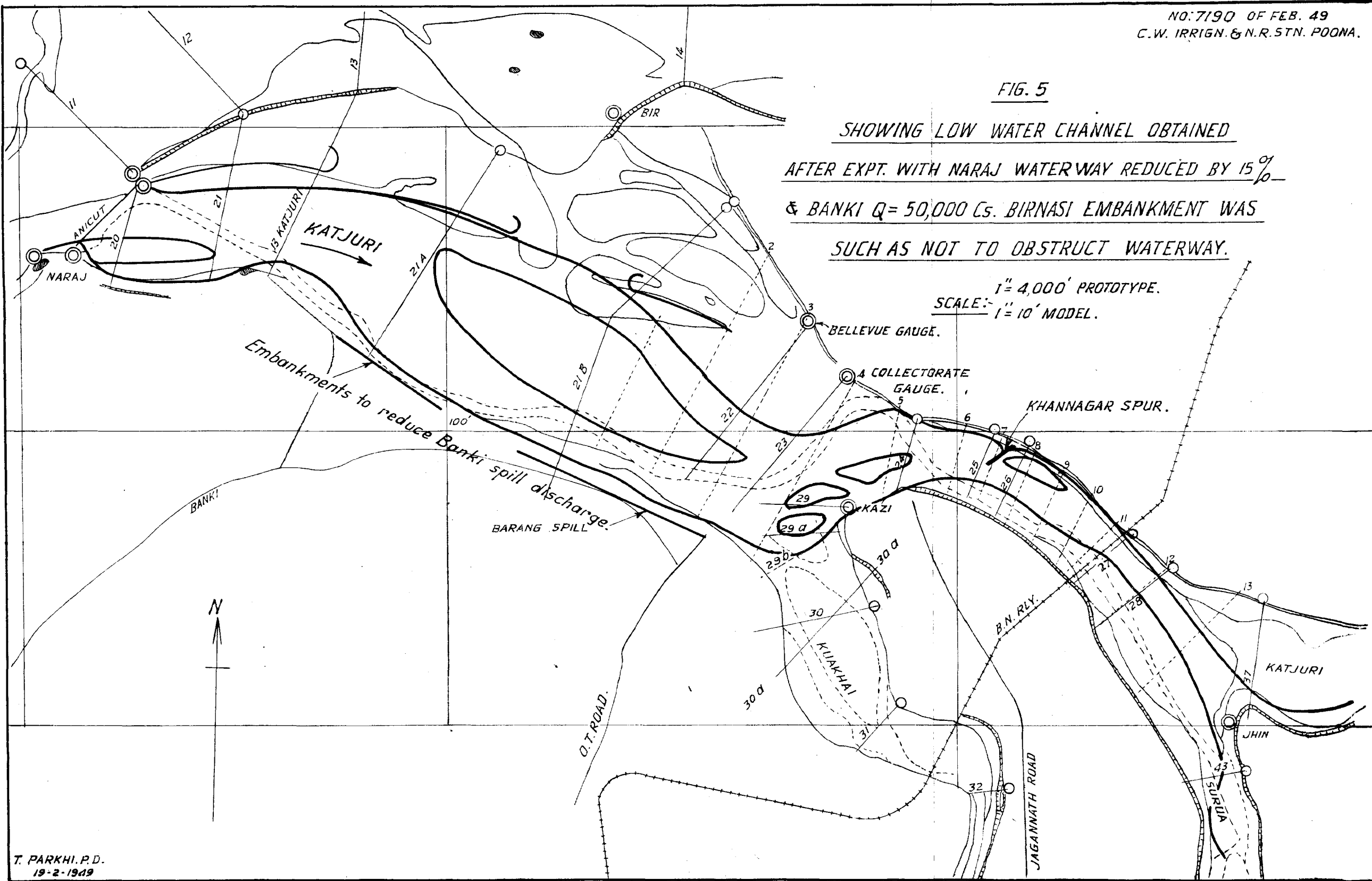


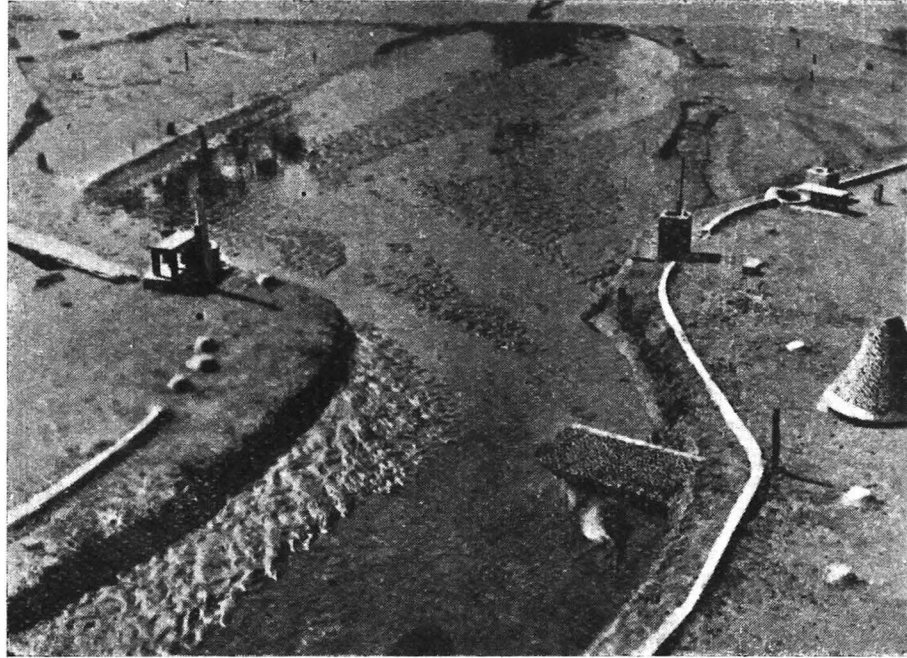
FIG. 5

SHOWING LOW WATER CHANNEL OBTAINED
AFTER EXPT. WITH NARAJ WATERWAY REDUCED BY 15%
& BANKI Q = 50,000 C.S. BIRNASI EMBANKMENT WAS
SUCH AS NOT TO OBSTRUCT WATERWAY.

1" = 4,000' PROTOTYPE.
SCALE: 1" = 10' MODEL.



T. PARKHI, P.D.
19-2-1949



BELLEVUE GAUGE.
COLLECTORATE GAUGE.

KHANNAGAR SPUR.

PHOTO No. 5229. Looking upstream
from Katjuri Railway Bridge.
Photo showing low water channel obtain-
ed in experiment with embankment
No. 7, Banki Spill discharge reduced to
50,000 cusecs and Naraj weir waterway
reduced by 15.3% at its right end.

3. *Effect of the proposed Hirakund dam on conditions of the Katjuri river at Cuttack.*

The safe flood gauge at Naraj has been accepted as 90.1 when the total flood discharge at Naraj is 12,15,000 cusecs. Provision has been made in the Hirakud Dam Project to control the floods so as not to exceed the safe maximum flood gauge at Naraj. In other words, after the Hirakud dam is constructed the peak flood discharge of the Mahanadi at Naraj would not exceed 12 lakh cusecs. As meander length is a function of the discharge it was necessary to examine the effect of the reduction of discharge on the Katjuri system and to see whether the pitched embankment etc., would at all be necessary under post-Hirakud Dam conditions.

With this object in view an experiment was done in which the controlled flood cycle of 1933 was reproduced without bed charge as would be the case after the dam is constructed.

Fig. 6. The data of controlled floods of the Mahanadi at Naraj was taken from the June 1947 Hirakud Dam Project Report.

The model was laid according to the 1946 survey with the Khannagar spur constructed, and discharge cycles as in fig. 6 were repeated till the model bed stabilised. fig. 7 shows the low water channel obtained in the Katjuri River. It will be

Fig 7. noticed that the apex of the meander of the channel, which under existing conditions was at the Kazipatna gauge, site below the offtake of Kuakhai, moved upstream and was about 4000 up stream of the O. T. road upstream of the head of Kuakhai. This experiment showed that though under existing conditions, an embankment as shown in fig. 4 with Naraj weir waterway reduced by 15.5% and Banki Road spill discharge reduced by 50% would give the desired result, yet under the post Hirakud Dam conditions, the desired aim could be achieved without doing anything except the construction of Khannagar spur for local protection.

8. *Traiding of the Ganga River at Garhmukteswar.*

Member Engineering and Director, Civil Engineering, Railway Board, inspected the river Ganga at Garhmukteswar on the 13th September 1948. It was considered that the river had come dangerously close to the track and immediate remedial measures must be taken to maintain the lines of communications.

It was proposed to extend the left guide bund by 2,000 ft. with a mole head before the monsoon of 1949.

The Director, Central Waterways, Irrigation & Navigation Research Station was consulted in the matter and he agreed to the proposals. Later on, the Chief Engineer, E.I. Railway put forth a proposal of constructing a series of spurs fig. 8 — to arrest further inroads of the river towards the track.

The Railway Officers also had a proposal of diverting the track $\frac{1}{2}$ a mile away from the existing line. On considerations of the imminent danger of break in communications and the cost involved in the construction of the spurs, the Director inspected the site with the Dy. Chief Engineer, E.I. Railway and it was considered better and safer to divert the track extend the left guide bund and await results of model tests for further training works for providing a smoother approach channel.

Besides the dangerous approach of the river to the railway track, the river is also attacking squarely the bank near the village Lakheri and Jigri. A model to evolve suitable training measures is being constructed.

9. Erosion of the banks of the Bhagirathi River near Purbhashali.

When the Director visited the Ganga at Garhmukteswar, the Dy. Chief Engineer E.I. Railway discussed the erosion of the Bhagirathi bank at Purbasthali.

Fig. 9 shows the courses of the Bhagirathi river in the past and Fig. 10 shows the present conditions. It will be seen that the river is now only 161 feet away from the track and the Dy. Chief Engineer discussed the feasibility of protection of the railway line by pitching the slopes or diversion of the line.

The Director suggested that if the line had to be safe, it should be retired at least by about one mile because retiring it by 400 ft. as proposed would not be enough. In case of the other alternative of protecting the line by pitching with an apron, the cost of protection would be very heavy. It would not only mean a considerable amount of recurring expenditure on maintenance, but the length of the arc and depth of scour go on increasing requiring more and more protection of the bank. This would also be instrumental in keeping the river in this position permanently.

FIG. 6
SHOWING THE 1933 FLOOD PEAK, AS IT WOULD HAVE
BEEN UNDER POST DAM CONDITIONS
AND MODEL PEAK CYCLE.

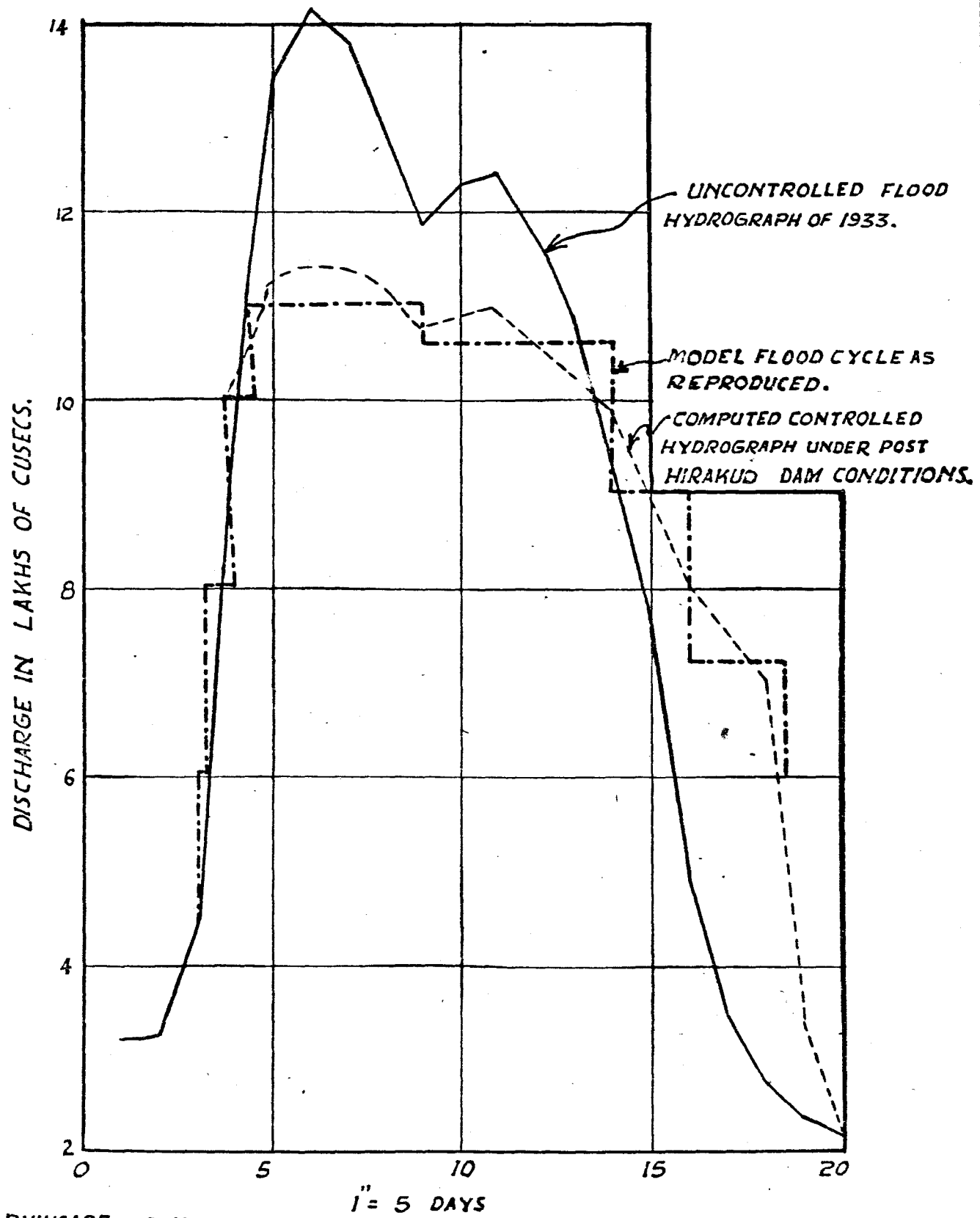


FIG. 7

SHOWING LOW WATER CHANNEL OBSERVED WITH
MODEL LAID ACCORDING TO 1946 CONDITIONS OF
KHANNAGAR SPUR, DISCHARGES RUN BEING -
ACCORDING TO THE PORT HIRAKUD DAM -
PROJECT CONDITIONS.

SCALE: 1" = 10' = 4000'

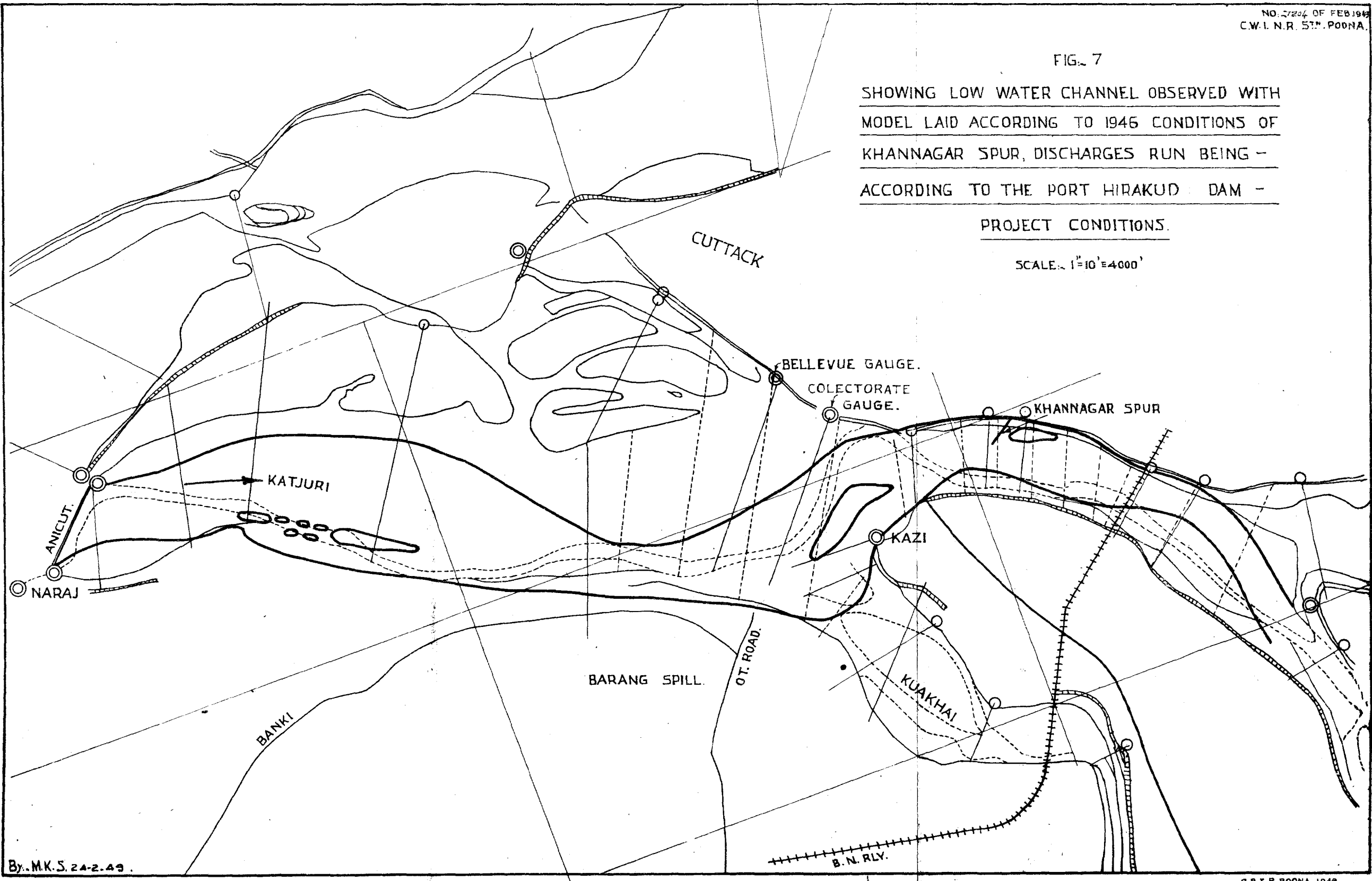


FIG. 8

PLAN SHOWING THE
RIVER GANGA AND
PROPOSED TRAINING
WORKS AT MILE 39-40

(E.I.R. DELHI BRANCH)

SCALE 1" = 600'

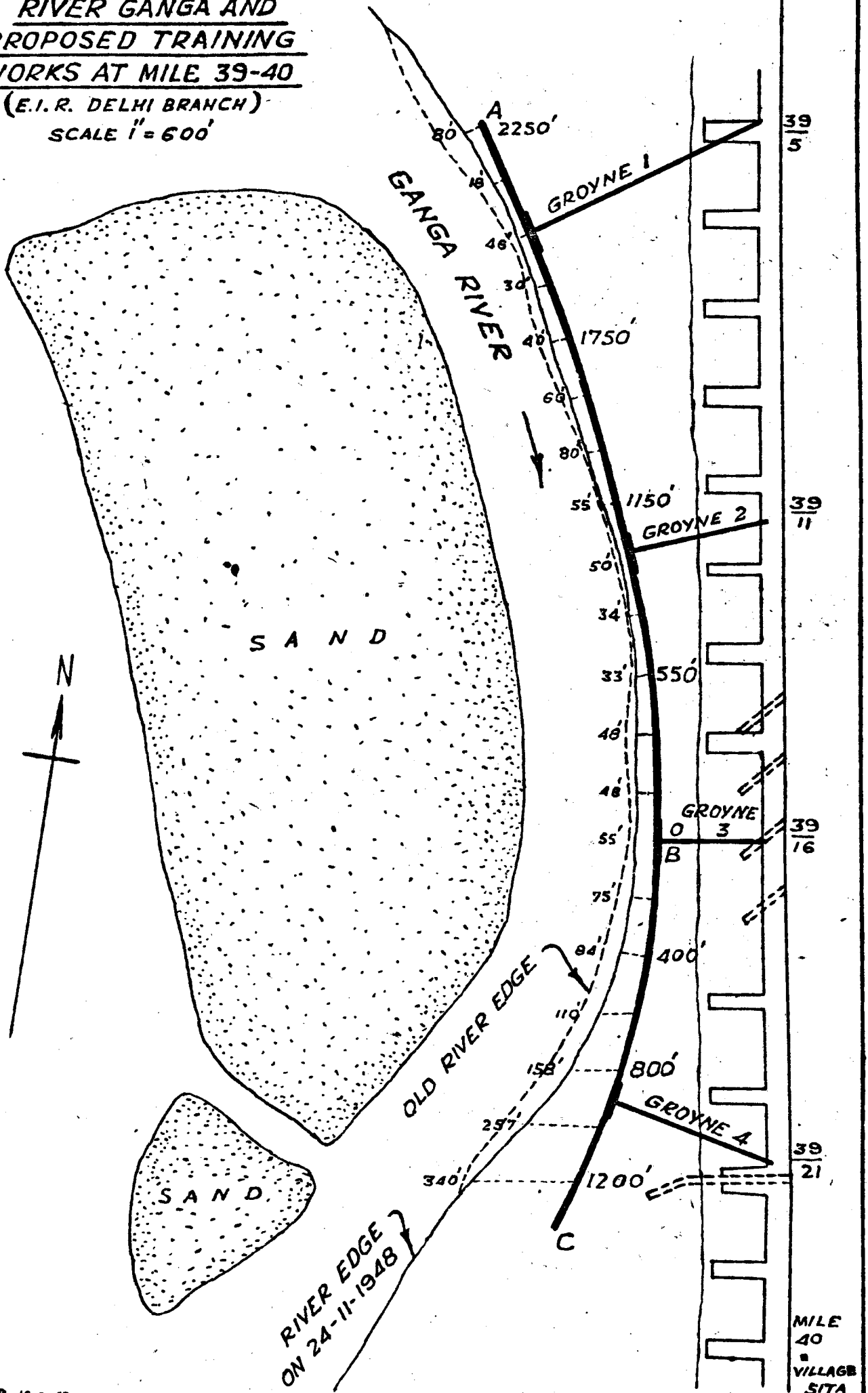


FIG. 10

EROSION OF BHAGIRATHI RIVER BANK

BETWEEN

NABADWIPDHAM & PURBHASTHALI M: 60 TO 70

SCALE. 1600 FT TO AN INCH.

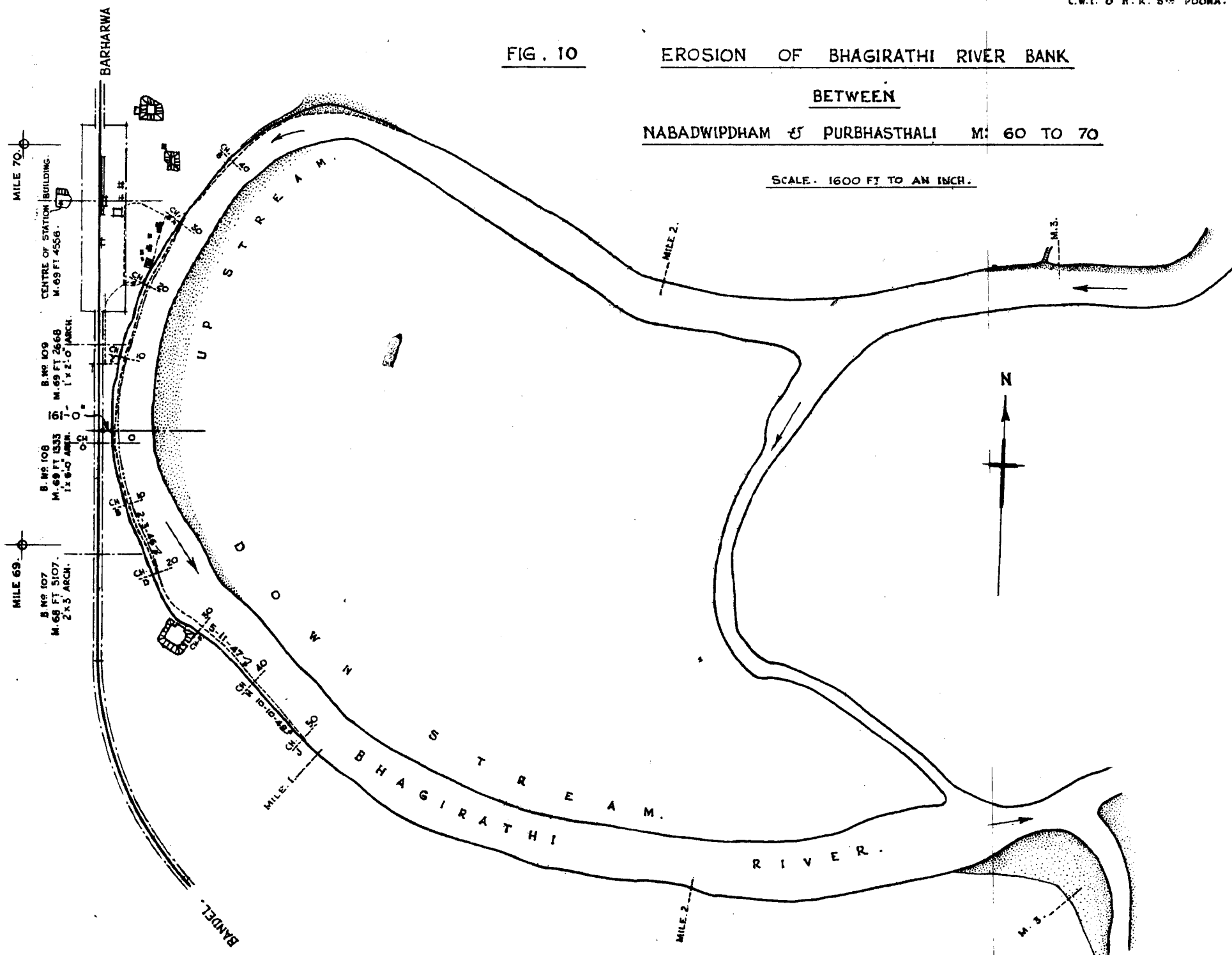
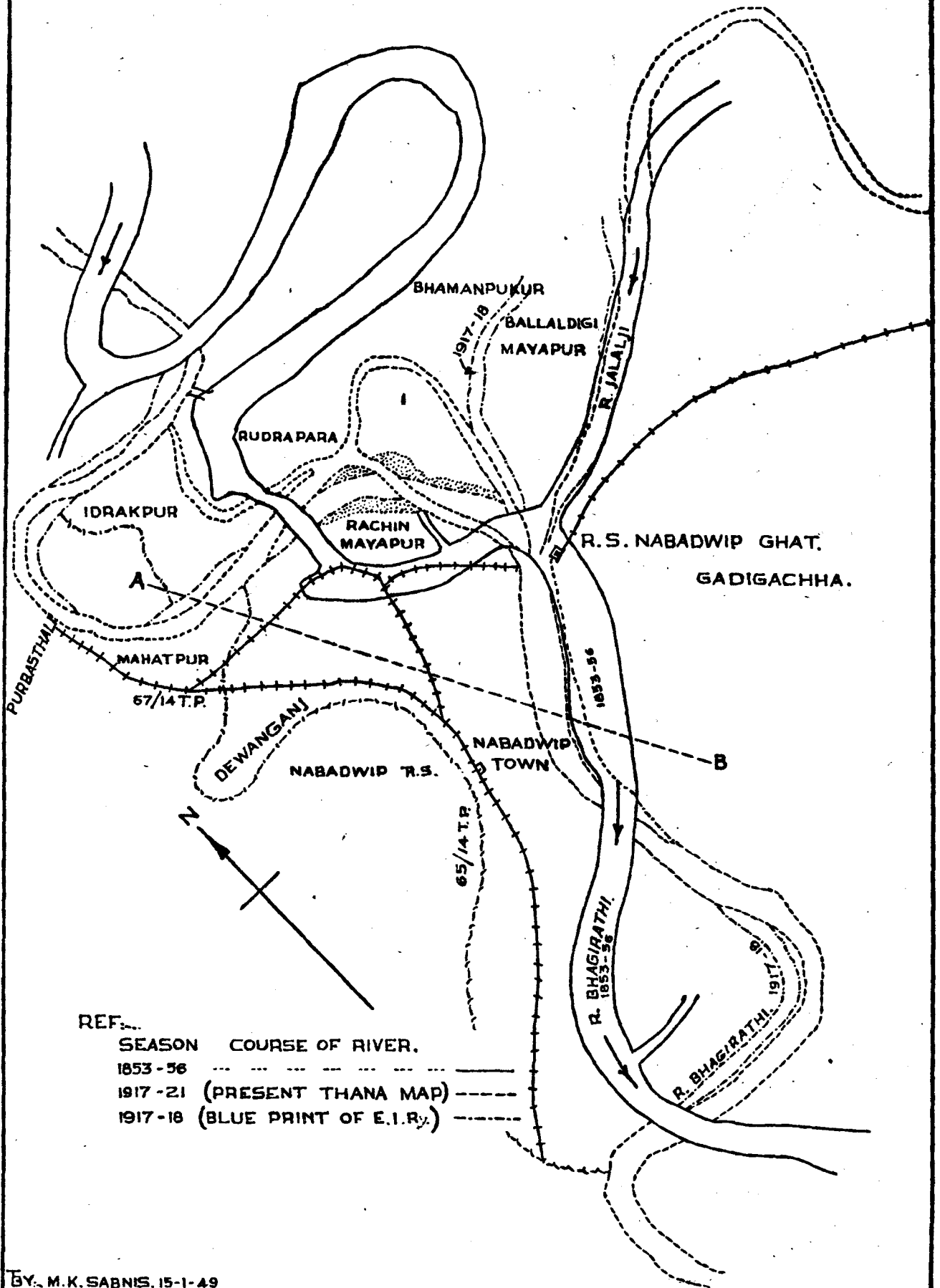


FIG. 9

INDEX MAP OF RIVER BHAGIRATHI
SHOWING THE RIVER COURSE OF-
DIFFERENT YEARS NEAR NABADWIP TOWN.

SCALE: 1" = 1.5 MILE



As the rate at which the river has increased its length in the past varies considerably (viz. 15 ft. a year to 50 ft. a year), great reliance cannot be placed on the average rate and it would be hazardous to predict what the erosion next year will be.

Considering the previous year's record, it appears that the line would be safe for at least one year more during which period, the Director suggested, it would be possible to retire the line by one mile away from its present position and in his opinion, this appeared to be the only practical solution.

10. Training of the Kshipra River at Ujjain—Union of Madhya Bharat—(Gwalior State).

1 History.

(The problem was described in last year's Annual Report, but the history of the the case is repeated in this year's report to present a complete note).

1.1. The City of Ujjain draws its water supply from an infiltration gallery situated in the bed of the adjoining Kshipra River. The infiltration gallery was constructed to a length of 200 ft. in the years 1901-1903, and due to the increased demand of water, as a result of tripling of the town population, the length of the gallery was increased to 600 ft. The gallery is constructed on rock foundations; it is rectangular in section (4'×7') with an arched pucca top. Dry stone masonry is built between side pillars spaced 6 feet apart to collect the filtrate from sides. In order to have an assured depth of water over the infiltration gallery, throughout the cold and hot seasons, a barrage was constructed in 1931, half mile downstream of the gallery. The Barrage impounds 14 feet of water by means of a double row of 6" high kurries, staunched on the upstream face with mud to prevent leakage. There are only two mechanically operated scouring sluices in the Barrage which have proved very inadequate. During the floods only top 3 feet of kurries can be removed thus having a permanent ponding of 11 feet of water over the Barrage sill. This, however, created a serious problem of accretion of fine silt and clay occurring upstream of the Barrage on falling floods. To get sufficient water and infiltration head ponding is inevitable. On falling floods, fine silt and clay settles over the gallery; and as choking is felt in the gallery, the rate of pumping is increased with the result that the static head over the gallery and the dynamic head of pumping together exceed the critical head* of filtration. The choking, as the rate of pumping increases, becomes severe and the supply drops down very considerably. During such periods the infiltration gallery works only intermittently.

Fig. 11:

* In an ordinary sand filter the head does not exceed 3 to 4 ft. and even in a rapid filter the head does not exceed 10 ft. In this case the head exceeds 14 ft. in falling floods

1.2. Further, the gallery being situated on the convex side at the apex of the meander, the river has been gradually receding away from it and eroding its concave right bank. This called for the protection of the right bank, particularly of the historical observatory known as 'Jantar Mantar'; which was done by paving the side slopes with reinforced concrete slabs and stone pitching. To protect the right bank from scour and to push the flow over the infiltration gallery, a series of spurs were proposed by the local officers and advice regarding their efficacy was sought from this Station in November, 1943.

1.3. In consideration of all aspects of the case this Station suggested the construction of a single, attracting spur (No. 3) on the left bank opposite a point midway between spurs No. 7 and 8 on the right bank. The local officers, however, thought that the single spur might take a long time to act and might worsen conditions at 'Jantar Mantar'. So they constructed, before the floods of 1944, repelling spurs No. 1 and 2 on the right bank and spur No. 3 on the left bank. This attracting spur No. 3 suggested by the Station was not given sufficient time or a fair chance to come into action because after the first low floods of 1944, it was cut down by a length of 150 ft. from its nose; while after the peak floods it was cut further down by 80 feet. Later, spurs No. 4, 5, 6 and 7 were constructed on the right bank.

Fig. 12.

1.4. The Government of Madhya Bharat (formerly Gwalior State) are now planning to pump raw water direct from the river. The site of this pumping Station is to be on the right bank in the reach AB which has, therefore, to be protected from erosion. This scheme will take some time to materialise and in the interim period the infiltration gallery has to be kept clear from choking,

Fig. 11.

1.5. With this twofold object in view, (a) of protecting the right bank from erosion, and (b) of preventing deposition of fine silt over the infiltration gallery, a model of the Kshipra River was constructed. The model extends $3\frac{1}{2}$ miles upstream of the Barrage and $\frac{1}{2}$ mile downstream. The model reach is shown in Fig. 11.

2. Hydraulic Data ⁽¹⁾

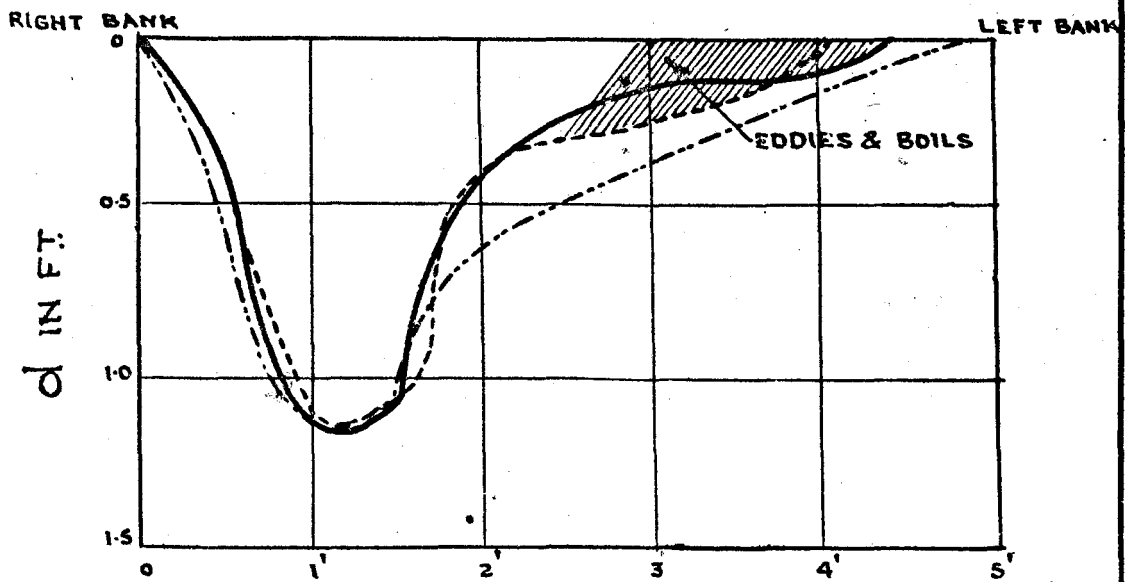
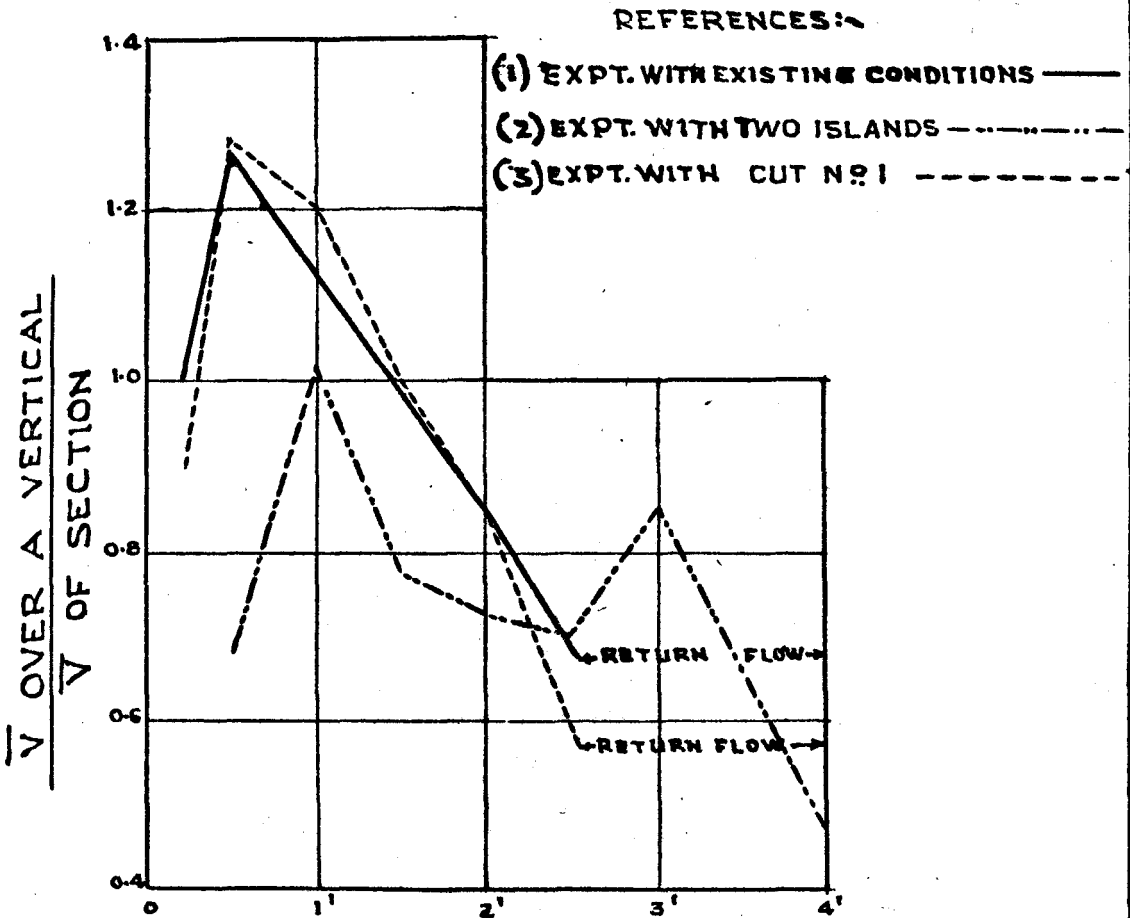
As in the majority of cases the available hydraulic data is very meagre

2.1. Bed Sand ⁽¹⁾

$$m_1 = 3.8 \text{ m.m.}$$

$$m_2 = 5.75 \text{ m. m.}$$

FIG.12
1/170 : 1/30 KSHIPRA RIVER MODEL
VELOCITY DISTRIBUTION CURVES
AT CROSS SECTION JANTARMANTAR



2.2. *River Slopes* ⁽¹⁾ Bed slope = 1 in 2500 (approximately).
Surface slope = 1 in 4400 (approximately
in the reach under invest-
igation.)

2.3. *Flood Discharge.* ⁽¹⁾ The Station estimates of flood dis-
charges are, $Q_{\max.} = 1.36$ lacs cusecs.
 $Q_{\text{dom.}} = 0.70$ lacs cusecs.

3. *Model Scales.* ⁽¹⁾

The model scales adopted were:—

L. Scale = 1 : 170
d „ = 1 : 30
V. E. = 5.67
Slope scale ratio = 1 : 2
Time scale = 1 : 780
Discharge scale = 1 : 28900

4. *Proving the Model.*

4.1. In the preliminary tests carried out to prove the model, the model was laid with B.S.E. = V.E. = 5.67.

Photo 5220 On adjusting $Q_{\max.} = 1.36$ lacs cusecs, with the corresponding water level at the Barrage = R. L. 1585.0, it was noticed that the main flow short-circuited along the left bank instead of hugging the concave curve along the right bank. At and below the Pump-House down to a point about 300 ft. upstream of 'Jantar-Mantar' there was very slack flow and water was almost quiescent. On reducing $Q = 0.70$ lacs cusecs and adjusting the water level at the Barrage = R. L. 1567.5, almost the same conditions of flow prevailed except that there was a slight tendency for the flow to shift towards the right bank. The bed movement on the shoal opposite 'Jantar-Mantar' and the Pump-House was very excessive. When the discharge was 70,000 cusecs a severe 'boil' with a vertical axis formed opposite 'Jantar-Mantar'; however the severity of this boil decreased gradually on increasing the discharge to the maximum stage.

4.2. This is an incised river with a low ratio of
 $\frac{\text{surface width}}{\text{depth}} \left(\frac{W}{d} = 54.6 \right)$. The slope should, on theoretical consideration be flatter for such a type of river. The tendency of the flow to cut across

(1) For details, see A.R. (T) for 1947.

the shoal must have been due to steeper slope exaggeration and it was considered from previous experience that B.S.E. (bed slope exaggeration) should be reduced to 3.0. On effecting this change of B.S.E. to 3.0 from B.S.E. = V.E. = 5.67, conditions improved though not satisfactorily. The B.S.E. was further reduced to 2.0 when flow conditions similar to prototype were obtained in the model. The factual evidence also corroborated the similarity, in the model and prototype, regarding the regions of attack and high velocities and the flow pattern.

Fig. 15

4.3. With the adjustment of B.S.E. = 2.0, similarity of flow conditions was obtained so far as the sharp bend of the meander was concerned. In prototype, the main flow, after hugging and eroding the right bank leaves it at the Shiva's Temple yard shoots across to the left bank. It strikes the left bank just upstream of the Barrage where excessive erosion has occurred. This similarity was obtained only when the top of the submerged Barrage was raised by $\frac{1}{2}$ " ($\frac{1}{2}$ " = 1'-3" in proto). This is evident from the fact that co-efficient of discharge through the Barrage is higher in model than in prototype due to foreshortening. Raising the top of the Barrage reduced the waterway over it thus neutralising the excess curvature induced in the model. This resulted in higher velocity near the left bank upstream, of the Barrage, leading to the erosion of that bank. Thus in the model, not only was the flow pattern reproduced correctly but erosion of the left bank upstream of the barrage was also visible.

4.4. The discharges were then altered to find out severe stage of flood attack on the right bank. It was observed that with $Q = 0.70$ lacs cusecs and corresponding water level at the Barrage \approx R.L. 1575.5 severe conditions were visible. The following experiments were, therefore, made with maximum and dominant discharges.

5. *Experiments.*

5.1. It has been established that accretion of fine silt and clay occurs over the infiltration gallery due to the non-availability of waterway through the Barrage. At present there are only two mechanically operated scouring sluices which are very inadequate to flush the upstream. The only solution to get rid of this accretion difficulty is to have additional scouring sluices (which can be operated mechanically). This will, however, tend to increase the attack on the right bank due to steeper slope. This was confirmed by the model.

5.2. The other aspect of the problem viz. prevention of erosion of the right bank was then considered. The first suggestion indicated that if the discharge during floods is by-passed, considerable reduction in the attack may occur. The heaviest was observed when the discharge stage was at 70,000 cusecs. Further, the river showed a tendency to cut across the shoal when the discharge increased above 70,000 cusecs.

(1) A cut, shortening the sharp meander of the river, therefore, appeared a probable solution.

(2) Another probable solution was a device which would attract the flow towards itself thus causing reduction of concentration of flow along the right bank. An attracting island was, therefore, tried. As the deflection of flow was to be continued throughout, along the left bank towards the Barrage, two islands acting in chain were tried. To prevent the shoal occurring between the two islands, a divide wall was given a trial. The following were the experiments done:—

- (a) with cut No. 1—across the shoal.
- (b) with cut No. 2—across the shoal.
- (c) with two pitched islands.
- (d) two islands and a divide wall.

5.3. Discussion of experiments.

5.3. 1. Experiment (a)—with cut No. 1.

The dimensions and alignment of cut are shown in Appendix I and photo 5234 and 5235 Figs. 12, 13 and 14. The cut carried 20% discharge at the dominant stage. As it was submerged at higher floods the discharge carried could not be distinguished from the discharge carried by the parent river. In the parent channel, the velocities reduced considerably and the attack on the right bank also reduced.

Experiment (b) with cut No. 2 was planned to observe the conditions which may occur in case a smaller initial cut were to develop into a main channel in future, in the absence of a pacca inlet. As the cut carried 56% discharge at the dominant stage, the flow in the parent channel became very quiescent but indicated a danger of the infiltration gallery failing to get enough supply of water, due to silting. This indicated the desirability of having a pacca inlet.

5.3.2. Experiment (c) was made with two pitched islands (i) and (ii) design is given in Appendix III and the positions are indicated in Fig. 17. The assymetrical island (i) was kept upstream with the steeper side towards the shoal to have as much throw off on the left bank. Island (ii) was aligned in such a position that the flow attracted by island (i) was directed towards island (ii) to deflect it still further on the left bank. There was only a partial reduction of attack on the right bank and no accretion occurred owing to the turbulence created by the islands. This experiment indicated a solution o

Appendix III
Figs. 17
(a to d)

Photos
5221, 5226.

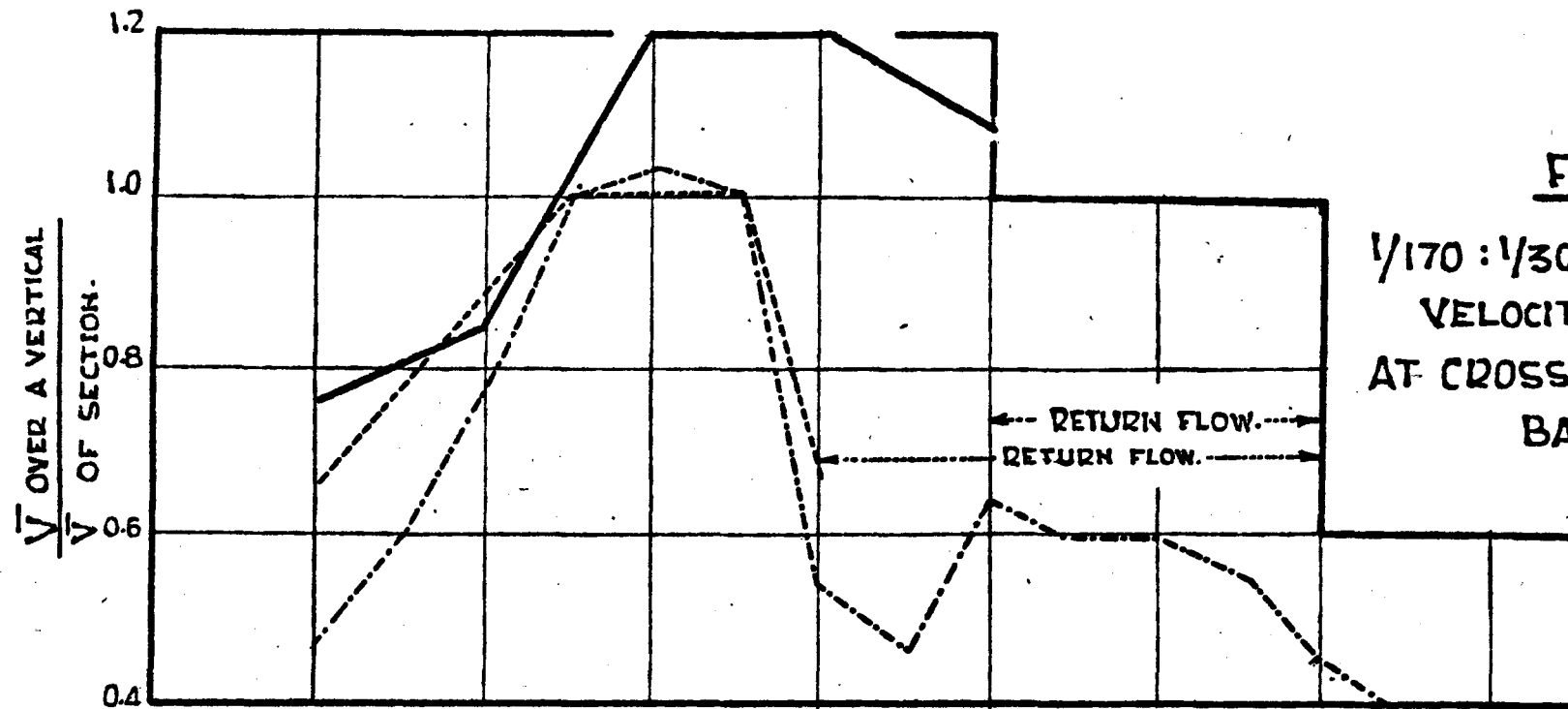
Fig. 18 the problem with the two fold objective balanced equally. Experiment (d) was made with two islands and a divide wall in between. As the islands in this position threw a heavy sandy shoal in between the two, a divide wall was put in to obviate this difficulty. The result was, however, not satisfactory; on one side of the wall-towards R.B.-a channel was scoured out while on the other side a heavy shoal was formed. The stability of the wall would also be constantly in danger owing to the unbalanced forces on the two sides.

6. *Recommendations.*

In the model, the Cut No. 1 as well as the islands have given satisfactory results. As a matter of fact the islands have indicated a solution that solves the problem with the two-fold objective balanced equally viz. preventing shoal formation over the infiltration gallery and protecting the right bank from the pump house to the Barrage. But, is feared that, in prototype, the islands will take a long time to come into action. In the absence of sufficient flood hydrograph data the number of flood years cannot be predicted. The behaviour of islands will depend upon the nature of subsequent flood years. This is an incised river and the floods are of very short duration. Consequently, the islands may not get a fair chance to act during the floods though they constitute a definite long term solution. On the other hand, the working of the cut will be definite. With the installation of the raw water pumping station in the next few years, the infiltration gallery and the cut will be quite efficient and sufficient to protect the right bank from erosion. Therefore, a cut is recommended. The detailed design of the cut including the design of a pacca inlet and energy dissipating devices downstream is kept pending awaiting the data of levels and bores asked for.

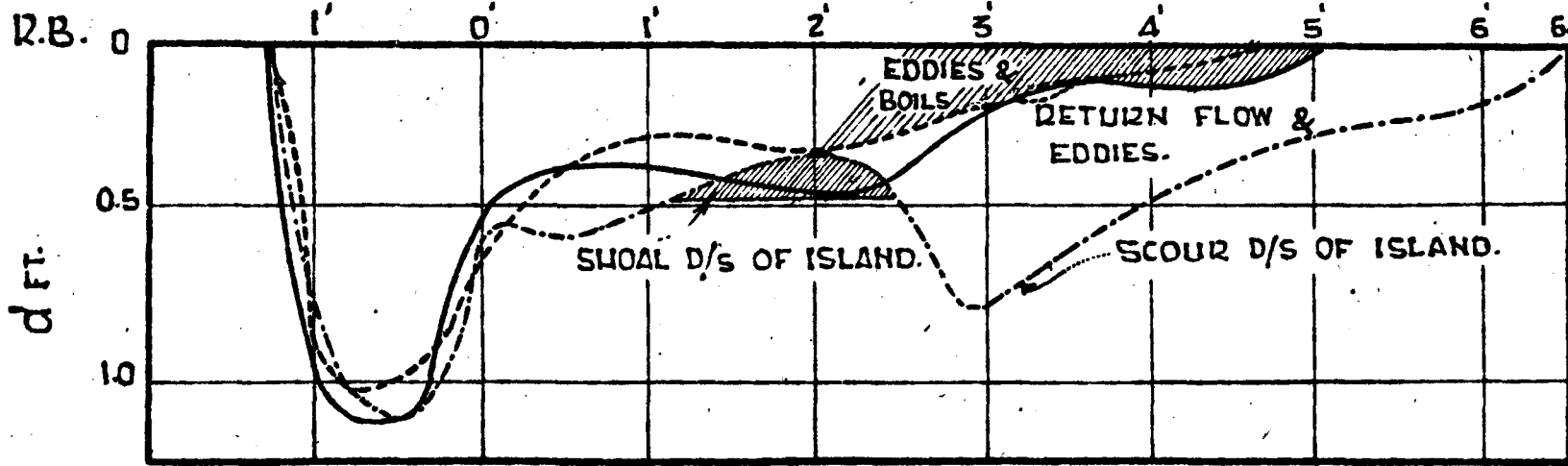
FIG. 13

1/170 : 1/30 KSHIPRA RIVER MODEL.
VELOCITY DISTRIBUTION CURVES.
AT CROSS SECTION 1/2 MILE U/S
BARRAGE (7-7)



REFERENCES.

- EXPTS. WITH.
- 1) EXISTING CONDITIONS. ———
 - 2) CUT NO. 1. - - - - -
 - 3) TWO ISLANDS. - · - · -



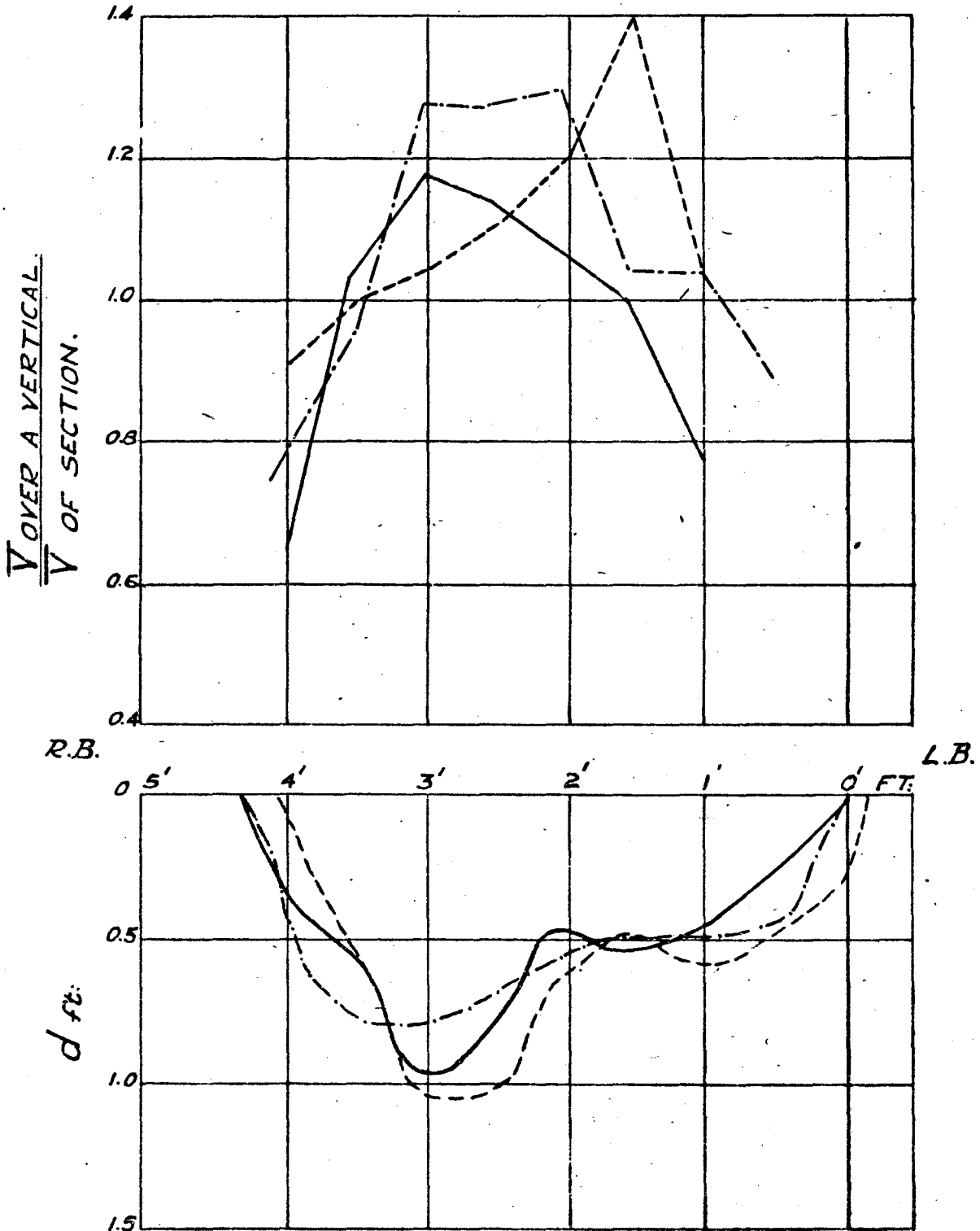
L.B.

FIG: 14

1/170: 1/30 KSHIPRA RIVER MODEL,
VELOCITY DISTRIBUTION CURVES
AT 340 FT. U/S BARRAGE.

REFERENCES:-

- EXPT: WITH 1) EXISTING CONDITIONS —————
 2) CUT - - - - -
 3) TWO ISLANDS - · - · -



By: MANCHUNDIA 19.1.49

1/170, 1/30. KSHIPRA RIVER MODEL

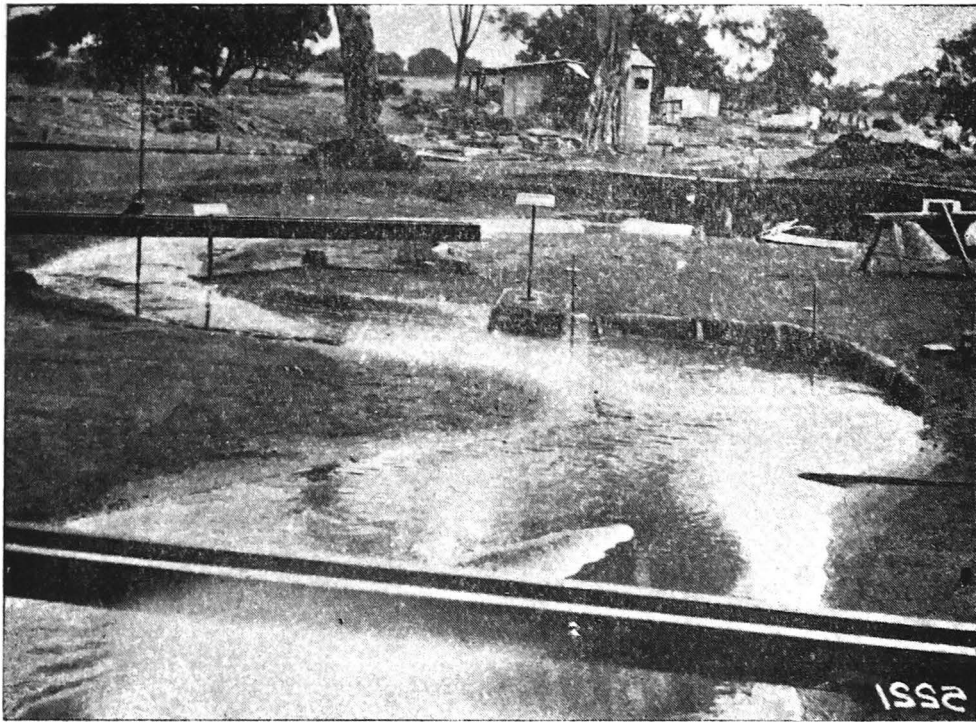


PHOTO No. 5220. General lay-out of the model looking downstream.

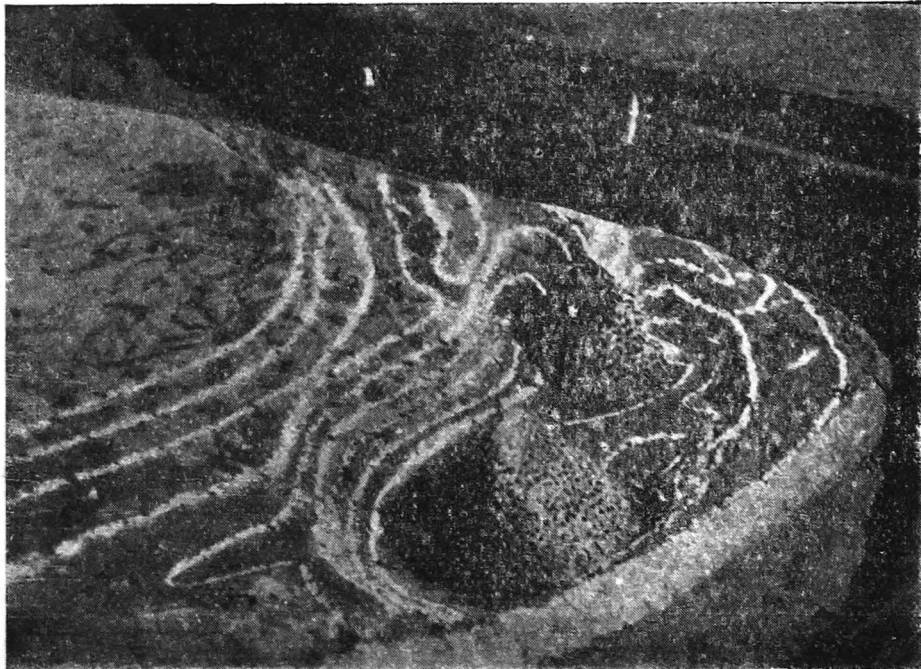


PHOTO No. 5221. Showing two pitched islands with contours of scour-hole. Pitching of 'Jantar-Mantar' in background.

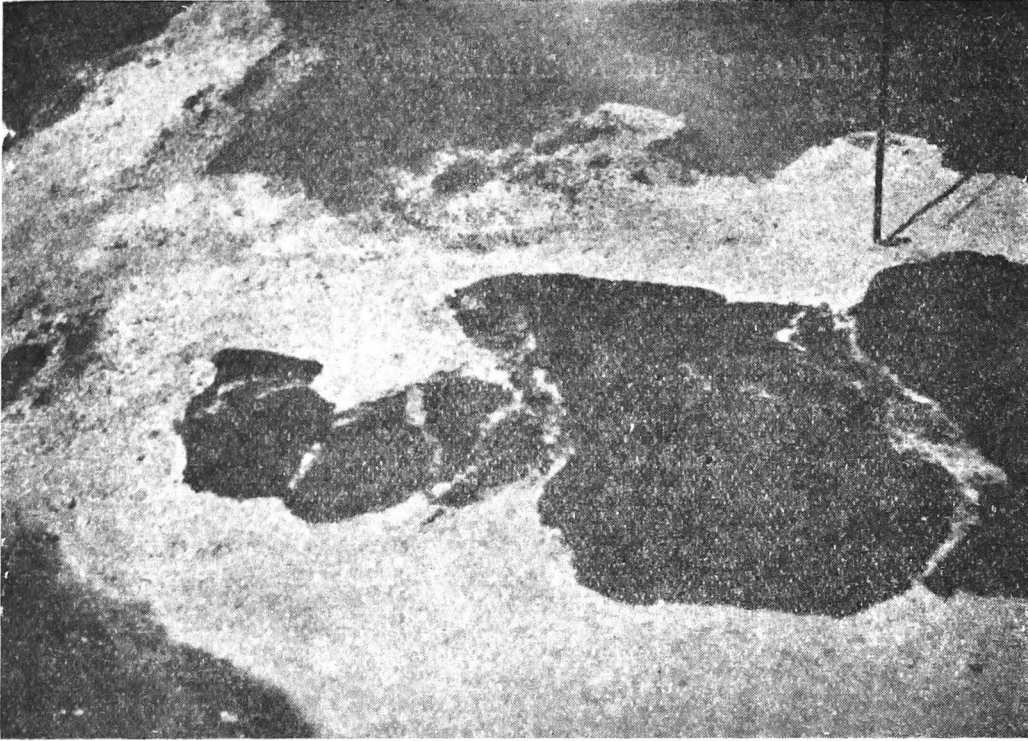


PHOTO No. 5227. Showing eddies and boils opposite 'Jantar-Mantar'.



PHOTO No. 5226. Two pitched islands in relation to the infiltration gallery alignment represented by the rods.

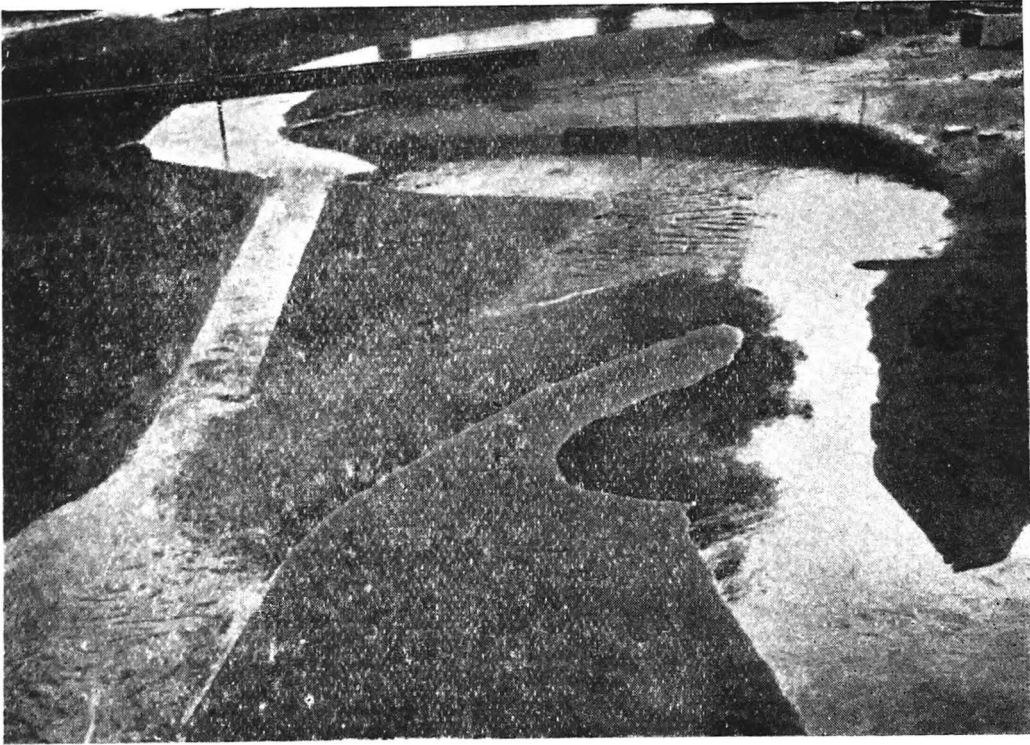


PHOTO No. 5234. Showing the cut across the shoal at $Q = 70,000$ cusecs discharge.

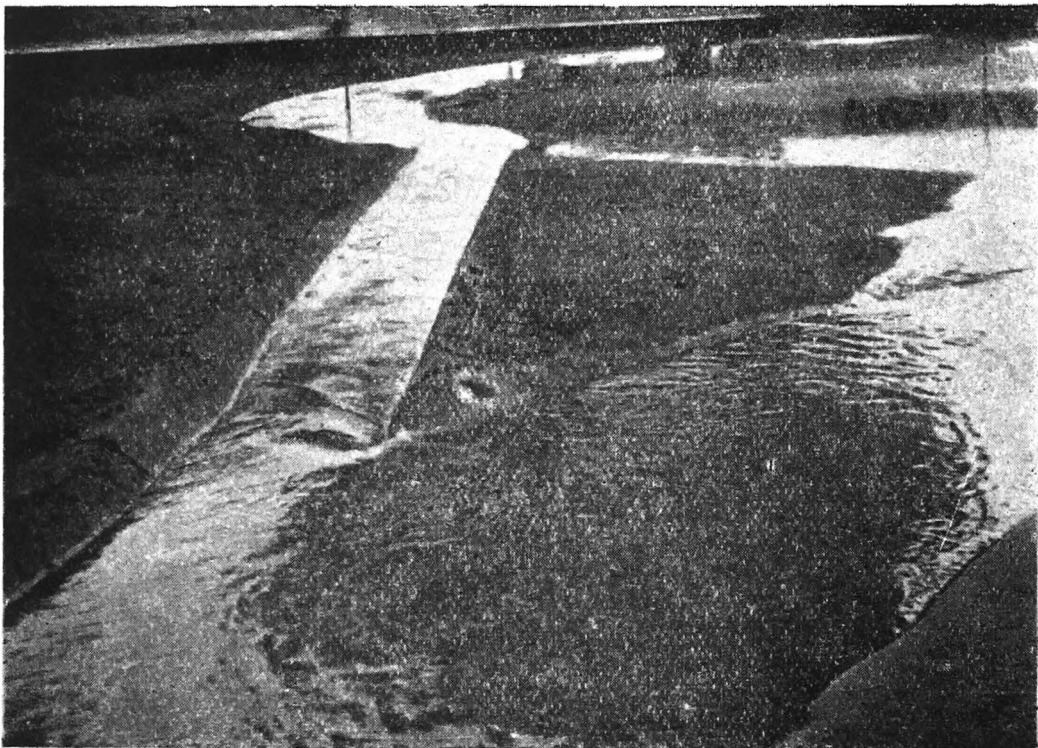
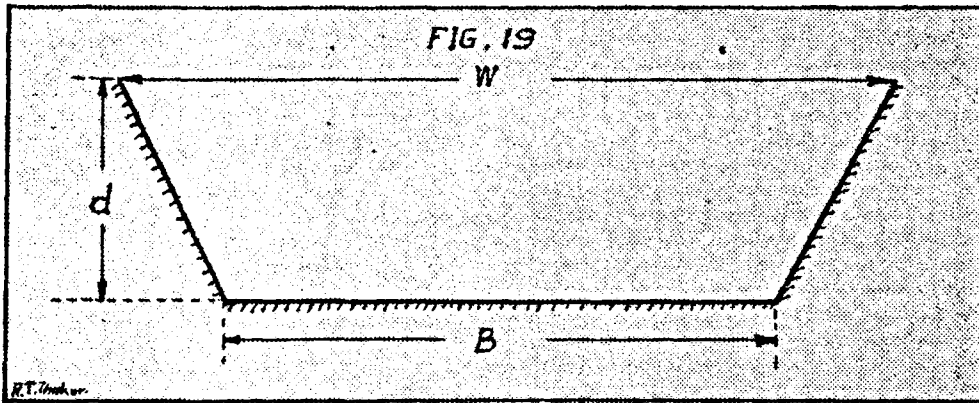


PHOTO No. 5235. Close-up of the Cut looking downstream showing turbulence at inlet.

APPENDIX I (Cut No. 1)

Dimensions of the Cut.



In model
 $B_m = 9''$
 $W_m = 15''$
 $d_m = 6''$
 $l_m = 16'.0''$
 $P = 22.4 \text{ in.}$

In proto
 $B_p = 127''$
 $W_p = 144''$
 $d_p = 15''$
 $L_p = 2720'$

$A = 72 \text{ square in.}$

$$\therefore R = \frac{A}{P} = \frac{72}{22.4} = 3.22 \text{ in.}$$

$S = 0.0013 \text{ (observed)}$

Theoretical carrying capacity of the cut—

$$V = \frac{1.486 R^{\frac{2}{3}} S^{\frac{1}{2}}}{N} \quad \dots \quad \dots \quad \dots \quad \text{Manning's formula}$$

$N = 0.02$

$= 1.02 \text{ ft. per second,}$

$$Q = A \cdot V = \frac{72 \times 1.02}{144} = 0.51 \text{ cusecs.}$$

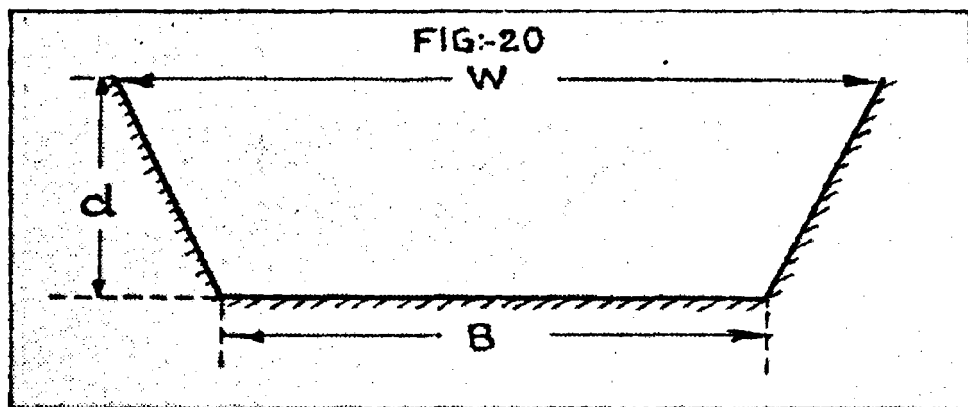
$$\therefore \text{Percent discharge carried by the cut} = \frac{0.51}{2.5} \\ \text{(at dominant stage)} \quad \quad \quad = 20.4\%$$

The velocity distribution diagram is shown in Fig. 20 (a) from which V (observed) works out to 0.98 ft. per second, almost as the theoretical.

$$\therefore \text{Actual } Q \text{ carried by the cut} = 0.48 \text{ cusecs} \\ = 19.2\%$$

APPENDIX II (Cut No. 2)

Dimensions of the cut.



In model

$$\begin{aligned} B_m &= 14' \\ W_m &= 22' \\ d_m &= 8'' \\ l_m &= 16'-0'' \\ P &= 31.9 \text{ in.} \end{aligned}$$

In proto

$$\begin{aligned} B_p &= 197' \text{ say } 200' \\ W_p &= 217' \\ d_p &= 50' \\ l_p &= 2720' \end{aligned}$$

$$A = 156 \text{ square inches.}$$

$$\therefore R = \frac{A}{P} = 5.01 \text{ in.}$$

$$S = 0.0013 \text{ (observed)}$$

$$V = \frac{1.486 R^{\frac{2}{3}} S^{\frac{1}{2}}}{N}$$

$$= 1.324 \text{ ft./sec.}$$

$$\begin{aligned} \therefore \text{Percent discharge carried by the cut} &= \frac{1.42}{2.5} \times 100 \\ &= 56.8\% \end{aligned}$$

Fig. 20 (b) The velocity distribution curve is shown in fig. 20 (b).

$$V \text{ (observed)} = 1.20 \text{ ft./sec.}$$

$$\text{Actual } Q \text{ carried by the cut} = \frac{156 \times 1.2}{144}$$

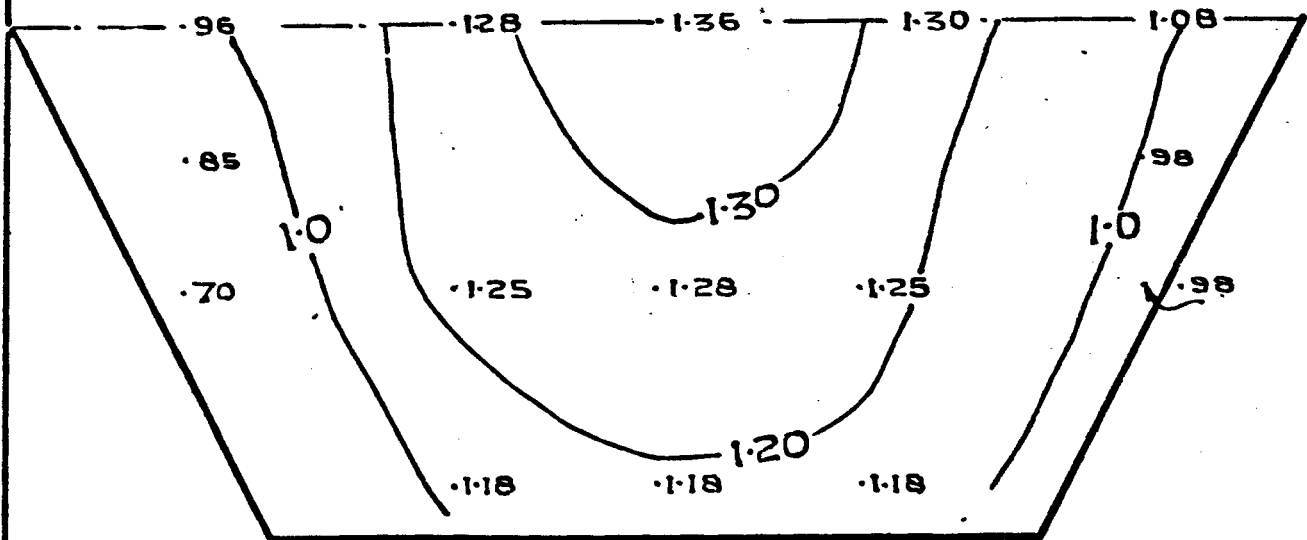
$$= 1.3 \text{ cusecs.}$$

$$= 52.0\%$$

FIG. 20 (a)

VELOCITY DISTRIBUTION IN

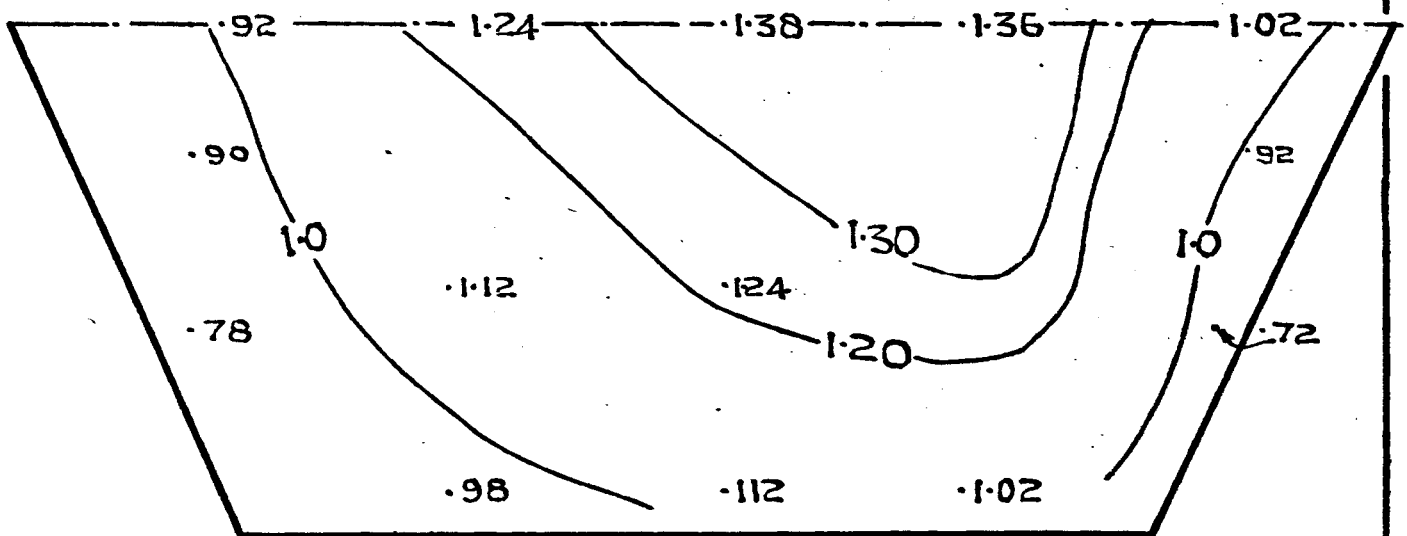
CUT NO. 1.



CROSS SECTION OF CUT NO. 1

SCALE:- 1" = 3" (IN MODEL.)

(b) VELOCITY DISTRIBUTION IN CUT NO. 2

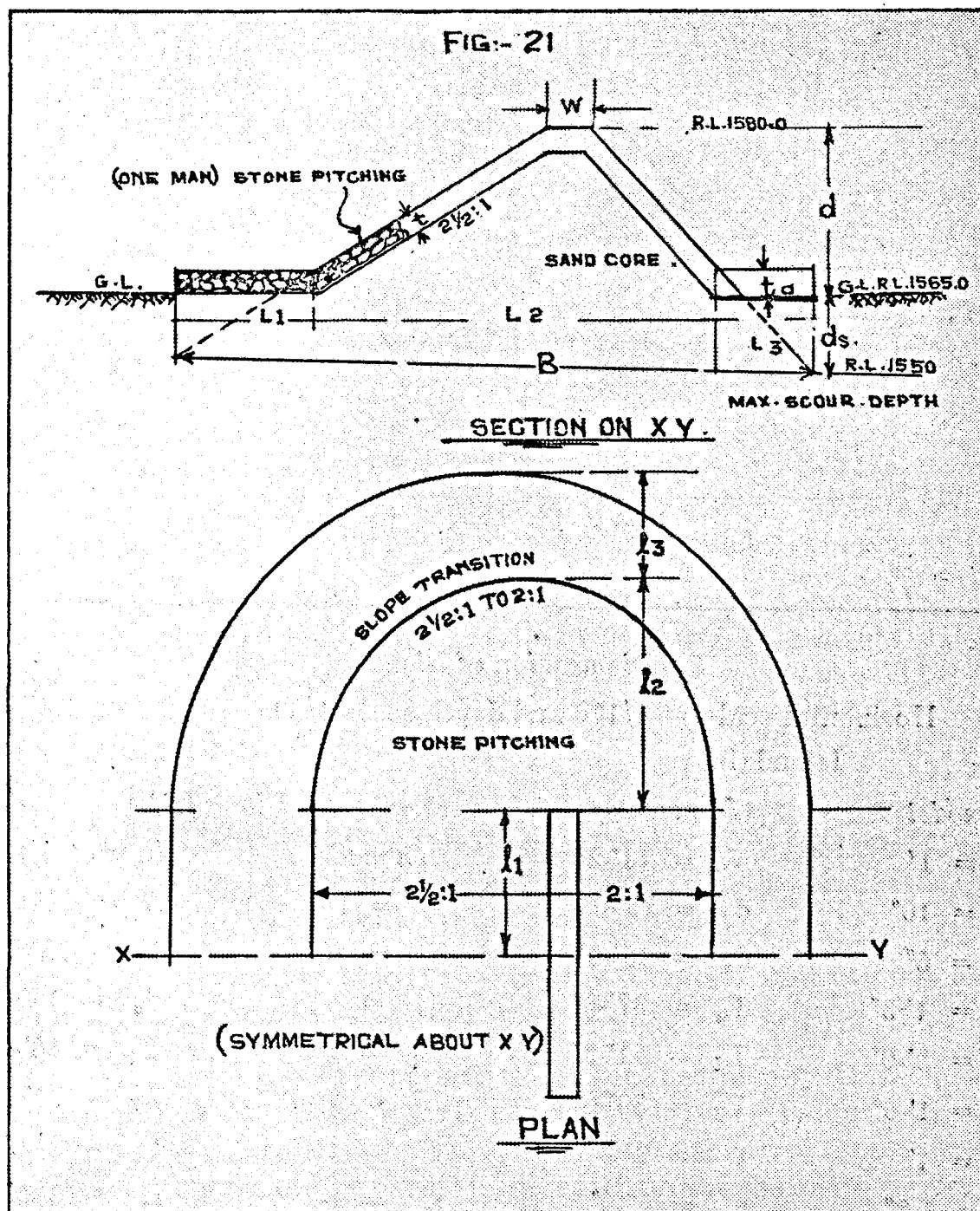


CROSS SECTION OF CUT NO. 2

SCALE:- 1" = 4" (IN MODEL.)

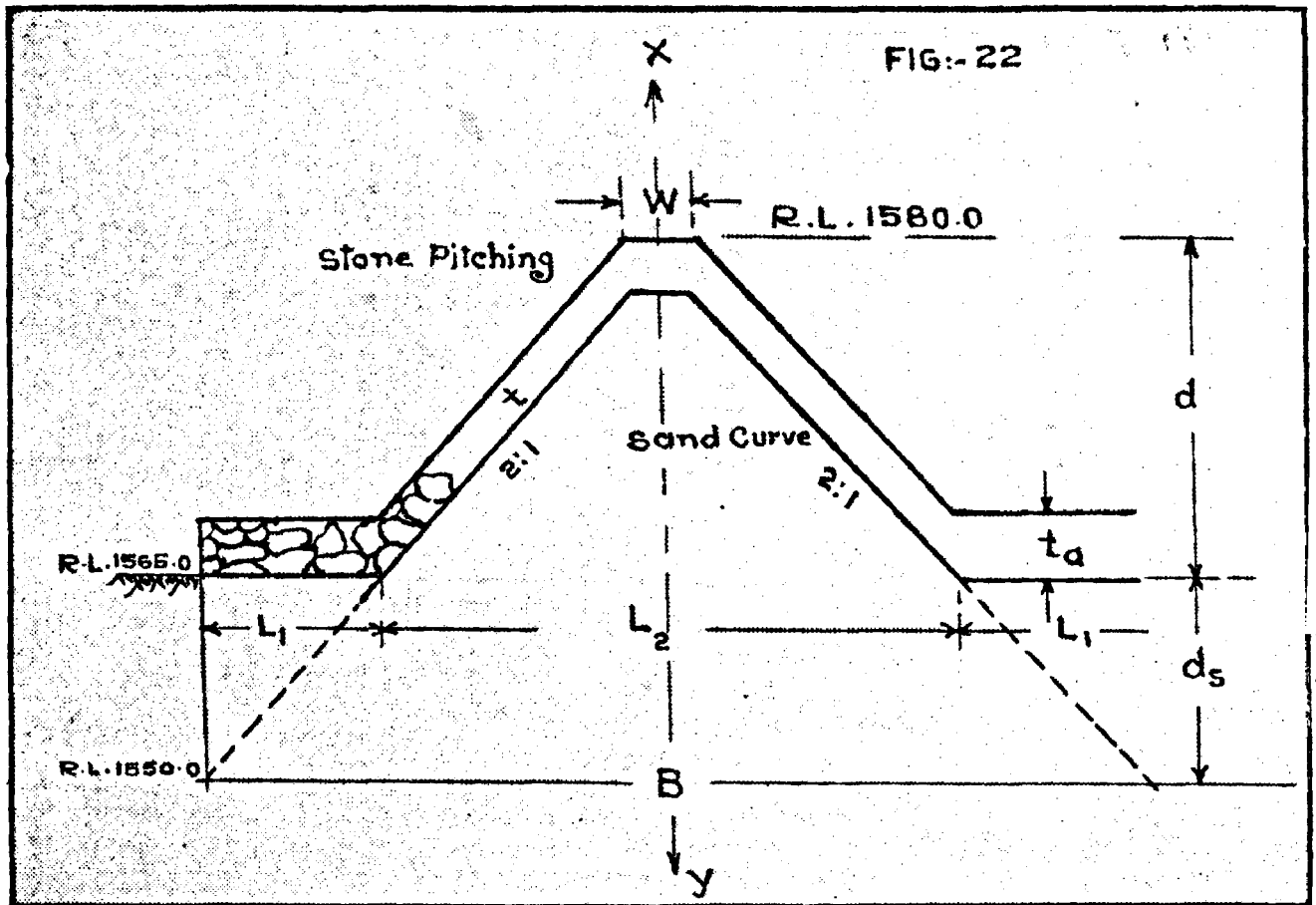
APPENDIX III (Two pitched islands)

Design of islands:— Island (i) (Sketches not to scale)



Note:— R. L. of maximum scour-hole with model was $\bar{\bar{}}$ so apron is designed for scour down to R. L. 15550.0

Island (ii) — (symmetrical about axis XY)



Dimension of islands.

Horizontal scale = 1/170 and depth scale 1/30

Island (i)

<i>In model</i>	<i>In prototype</i>	<i>In model</i>	<i>In prototype</i>
$W = 1''$	$W = 14'-2''$	<i>Island (ii)</i>	
$d = 10''$	$d = 25'-0''$	<i>In model</i>	<i>In prototype</i>
$d_s = 6''$	$d_s = 15'-0''$	$W = 1''$	$W = 14'-2''$
$L_1 = 1'-3''$	$L_1 = 37'-6''$	$d = 10''$	$d = 25'-0''$
$L_2 = 2'-0''$	$L_2 = 81'-2''$	$d_s = 6''$	$d_s = 15'-0''$
$L_3 = 1'-0''$	$L_3 = 30'-0''$	$L_1 = 1'-0''$	$L_1 = 30'-0''$
$t = 1''$	$t = 2'-6''$	$L_2 = 2'-0''$	$L_2 = 74'-2''$
$t_a = 1.2''$	$t_a = 3'-0''$	$t = 1''$	$t = 2'-6''$
$B = 4'-3''$	$B = 149'-2''$	$t_a = 1.2''$	$t_a = 3'-0''$
$l_1 = 0'-3''$	$l_1 = 42'-6''$	$B = 4'-0''$	$B = 134'-6''$
$l_2 = 1'-10\frac{1}{2}''$	$l_2 = 37'-6''$	$l_1 = 0'-3''$	$l_1 = 42'-6''$
$l_3 = 1'-1\frac{1}{2}''$	$l_3 = 33'-9''$	$l_2 = 1'-8''$	$l_2 = 30'-0''$
		$l_3 = 1'-0''$	$l_3 = 30'-0''$

11. Training of the Tapti River at Surat — Bombay.

1. Introduction.

The Station was approached in 1944 for advice regarding protective measures necessary for arresting the bank erosion on the right bank of the Tapti river between Rander and Hope Bridge at Surat and the severe attack on the town walls below the bridge on the left bank. Subsequently erosion of the fertile and valuable lands in the Bhatpur bight on the right bank about 5 miles downstream of Surat was also referred to the Station for suggesting suitable remedial measures.

History of the case is given at length in 1944 and 1947 Annual Reports (Tech.) of the Station.(1)

Explanatory tests were carried out to devise suitable training works and the promising measures referred to in the 1947 Annual Report (Tech.) were subsequently intensively studied in the model.

The results of these tests are presented below.

In testing out various training measures in the model, the financial aspect of the problem and practical and local difficulties in carrying out the work were naturally taken into account. These considerations led to the adoption of the following general principles in devising the necessary work.

(i) Whenever a city or a big town is endangered, to make the training works economical, they are to be designed to provide only local relief and protection. All the alternative training works should be studied in detail so as to evolve the final measure which will provide sufficient security comparable with its costs.

(ii) Protective works for local bank erosion are very rarely justifiable on financial considerations. Only under exceptionally favourable circumstances, expenditure on training works will be permissible, when these should be designed with the aid of the model.

Considerations which preclude the adoption of protective measure against local bank erosion are the following.

Bank erosion is never restricted or confined to any definite reach in the river. It is very likely that the forces let loose by the protective works built at places where erosion at present is experienced will cause damage elsewhere. This is more apparent in the reaches near the sea where the river slope, owing to draw-down effect, is steep and the tendency for bank erosion is accentuated.

(1) Annual Report (Tech.) of I. W. E. Station Poona, 1944, p. 87—90.

“ “ “ “ “ “ “ “ 1947, p. 55—57.

Real remedy against bank erosion on an all time basis in a comprehensive scheme, lies controlling and regulating flood flows by providing flood storage reservoirs, so that by their judicious operation flood discharges below them would be appreciably reduced and rendered almost harmless.

2. *Experiments observation and results :—*

(i) *Repelling groyne for protection of Rander town and river bank in the vicinity.*

To start with a repelling groyne 1' = 330' long and pointing 60° upstream was tested.

(A) *Positioning of the groyne.*

The groyne was tested at three positions in succession, shown in fig 23. The following considerations governed the choice of the positions.

Merits of the upstream position.

(a) The undermining on the left bank below Hope bridge is due to the current not following the curve at Surat which instead shoots straight across the bridge causing excessive scour and undermining in the part of the bank between Makhi Khadi and Nanpura Post Office. It is desirable that any protective works at Rander should help the river to flow more along the left bank. This was better achieved by the groyne at the upper position, as indicated by better approach conditions upstream of the Hope Bridge.

(b) Due to the constriction of waterway and deflection of the current away from the right bank, the opposite left bank below the groyne, may perhaps be eroded. Surat City being not less than 1½ miles downstream of Rander town slight erosion opposite the groyne, it is hoped, would be permissible. It is however safer and hence desirable that any erosion caused by the groyne should be as far upstream of Surat as possible. The upper location of the groyne is desirable from this point of view.

Demerits of the upstream position.

(a) For equal throw-off of the main flow the length of the groyne required at the upper position is more than at the lower position.

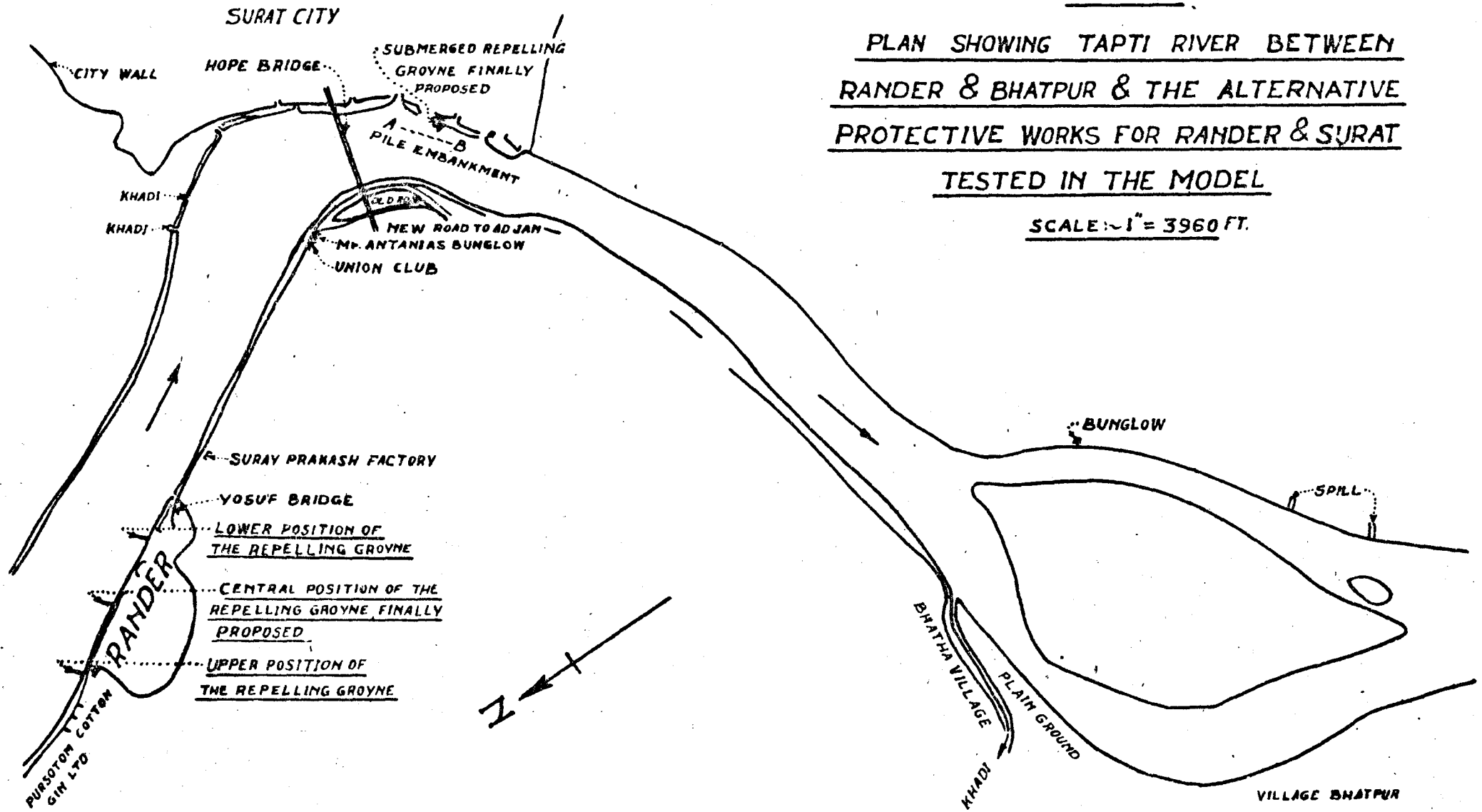
(b) The extreme upstream position being just below the bend, comparatively higher velocities and deeper bed levels would necessitate stronger and costlier groyne.

As a compromise solution, meeting the above conflicting requirements the central position of the groyne near Mandir Ghat was finally decided upon.

FIG. 23

PLAN SHOWING TAPTI RIVER BETWEEN
RANDER & BHATPUR & THE ALTERNATIVE
PROTECTIVE WORKS FOR RANDER & SURAT
TESTED IN THE MODEL

SCALE: 1" = 3960 FT.



By: A. J. THORNTON

(b) Length and orientation of the groyne

Different lengths of the groyne were tested at the central position.

With a longer groyne, the deflection of the high velocity flow away from the bank was more. The scour hole at the nose of the groyne was also removed further inside the river.

But the constriction of the waterway and the consequent scour at the nose was more, and the cost of the longer groyne with the necessary heavier apron was more.

Given a groyne of certain length, the desired throw-off can be effected by aligning it suitably with respect to the flow lines.

Model experiments indicated that for providing adequate protection to the Rander bank and to obtain the throw-off of the current at a safe distance away from the 300 ft. long groyne pointing 70° upstream would be necessary.

(C) Type of the groyne

Silt charge in suspension is relatively low in the deep and narrow, incised rivers — of which the Tapti is an example — than in wide and shallow rivers in flood plains like the Indus, the Ganga etc. The chances of a permeable tree groyne ultimately becoming solid by entrapping silt are therefore very remote in the Tapti and hence this comparative cheap substitute for an impermeable groyne was ruled out.

Impermeable groyne of 300 ft. length and a free board of 2 ft. with necessary apron protection was designed in light of the results of model tests. The cost of such a groyne together with the heavy apron protection at the nose necessary for preventing the damage to the groyne was enormous, about thirteen lakhs of rupees.

The cost of the full height groyne was said to be prohibitive considering the stakes involved and therefore a similar but submerged groyne was tested. With submerged groyne other complications however crept in. At high discharges with a submerged groyne with top at half flood level i.e. at R.L. 79 it was observed that a drowned wave with boils and turbulence occurred below the groyne. These undesirable conditions were ameliorated with lowering the top of the groyne, and ultimately it was found that with a groyne and with top of at R.L. 66.00, the flow could be effectively thrown off the Rander bank without causing unduly heavy turbulence and consequent erosion below the groyne.

The results showed that though not so effective as the full height groyne the submerged groyne would still provide sufficient protection to the river bank at Rander.

(D) *Model observations with the submerged groyne.*

The model observations with and without the submerged groyne are in figs. 24 and 25, which reveal that with the groyne the maximum velocity upstream of it and near the bank will be reduced by about 12%. Below the groyne the main channel will be deflected away from the bank — see photo No. 5219 and the maximum velocity near the bank will be lowered by about 8%.

(E) *Details of the submerged groyne.*

In spite of increase in length by 100 ft. the height of the submerged groyne being $\frac{1}{4}$ th of the full height groyne, the cost will be reduced to about 15% of the full height groyne i. e. about 2 lakh rupees. Detailed design of the submerged groyne is given in Appendix 1.

(ii) *Protective works for preventing undermining on the left bank below Hope bridge.*

The model showed that the Rander groyne would partially improve the flow conditions on the left bank at Surat below Hope bridge which at present experiences heavy damages due to undermining caused by square attack of the river. Some direct remedy is however warranted to ensure protection of this reach of the bank.

(A) *Various alternative training tested.*

In the preliminary experiments a repelling full height groyne, similar to one at Rander was found very promising. The deep channel however runs along the Surat bank and such a groyne with the minimum freeboard of say two feet would have to be about 67 ft. high, i.e. 15 ft. higher than than the Rander full height groyne will be expensive. Alternative remedies such as attracting spurs, pile embankments etc., were therefore next tested.

Attracting spur on the opposite bank had an advantage of being in the shallow water and hence considerably cheaper and easier to build. An attracting spur was therefore tested at various positions and at different orientations but was found to have only local effect and consequently was not efficacious in pulling away the strong frontal attack away from the left bank.

A continuous single line of piles across the main flow was next built on the alignment AB as shown in Fig. 23. The length of the pile line was 600 ft. i.e. about 44% of the length of the bank it was designed to protect. It was so effective that most of the attack on the bank was

FIG. 24
PLAN OF TAPTI SHOWING
EFFECT OF THE PROPOSED GROUYNE
NEAR RANDER ON THE FLOW LINES.
SCALE: 1" = 8' = 2640.

REF:
SURFACE LINES OF FLOW ———
BED " " " ·····

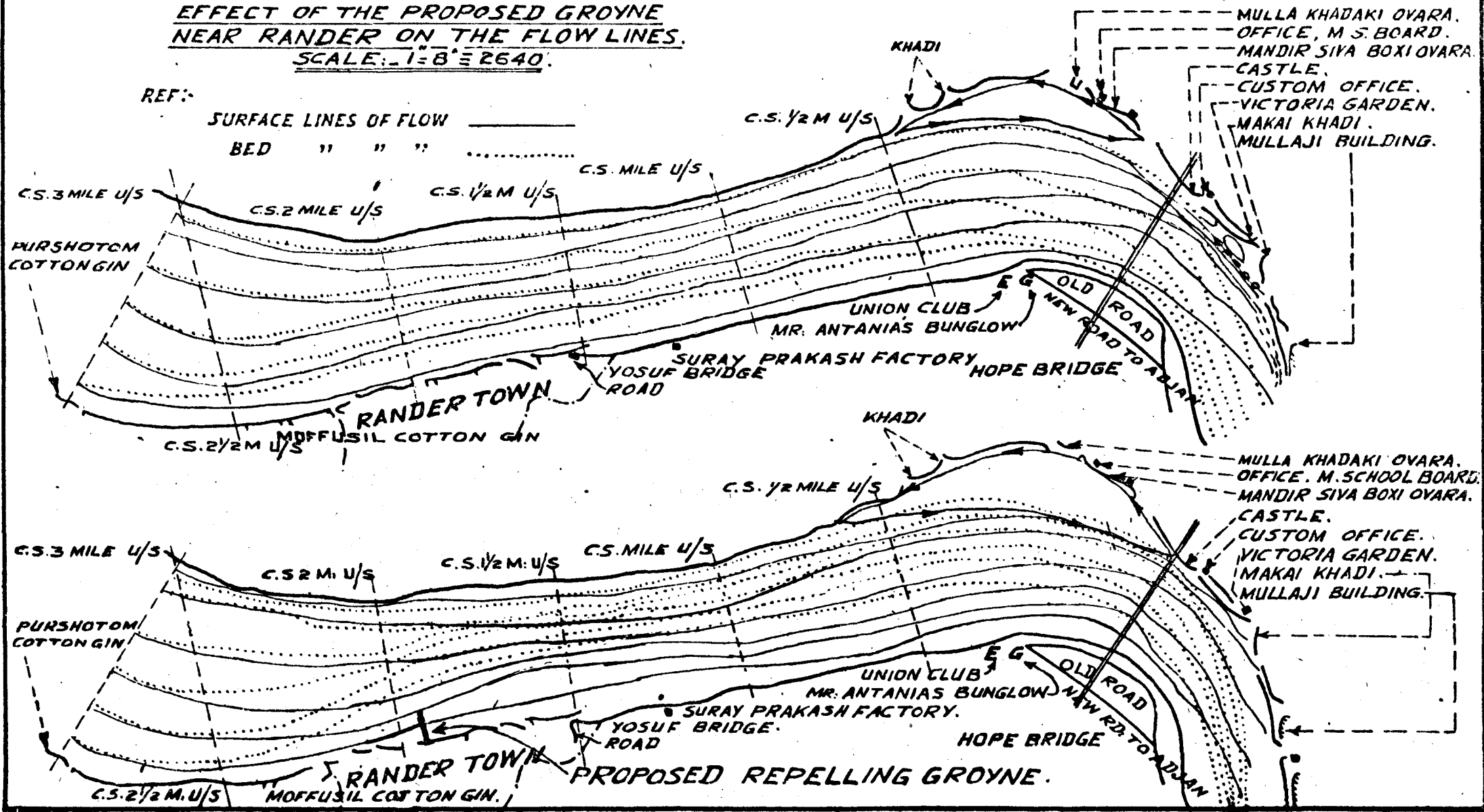
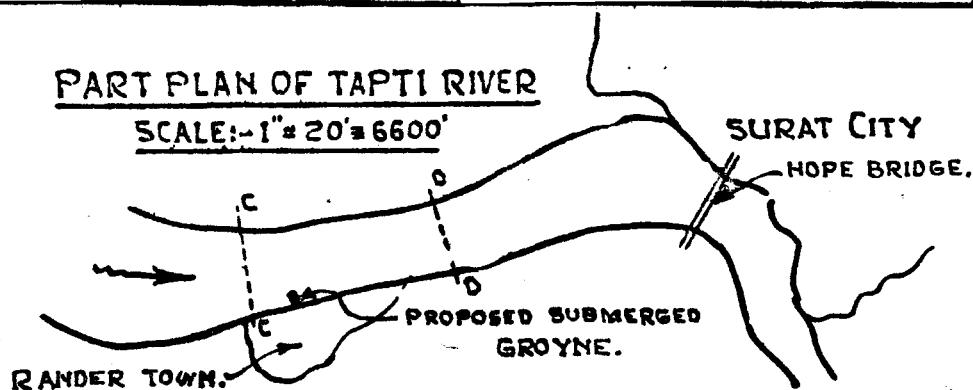


FIG. 25

VELOCITY SECTIONS ACROSS TAPTI AT RANDER

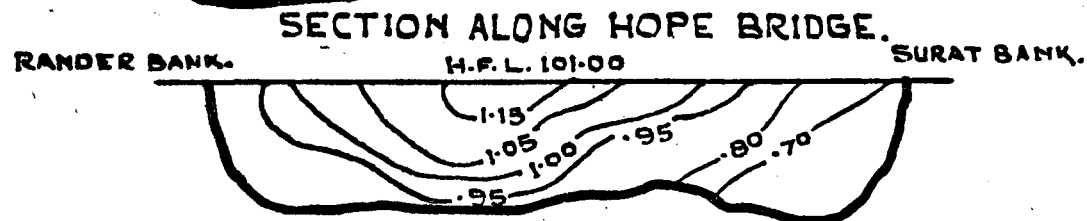
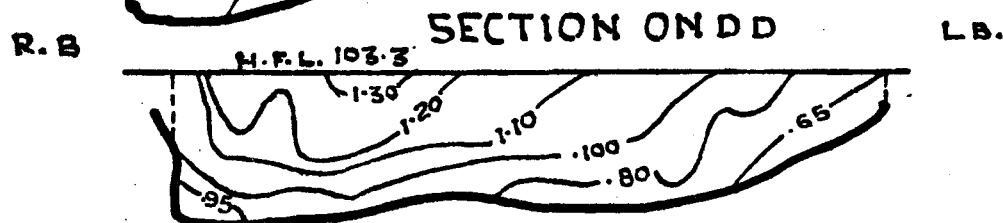
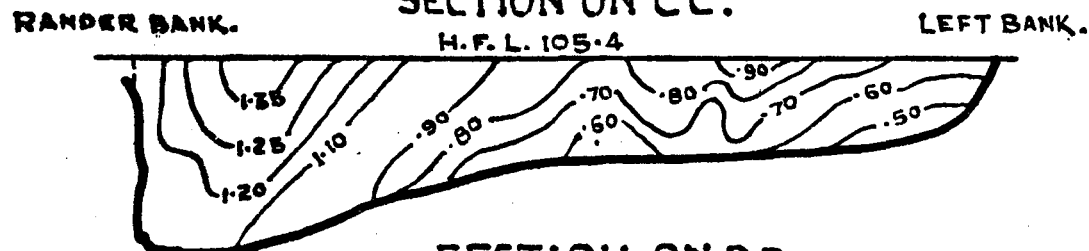
WITH AND WITHOUT GROUYNE

SCALE: HOR. 1" = 2' = 660'
VER. 1" = 1' = 72'

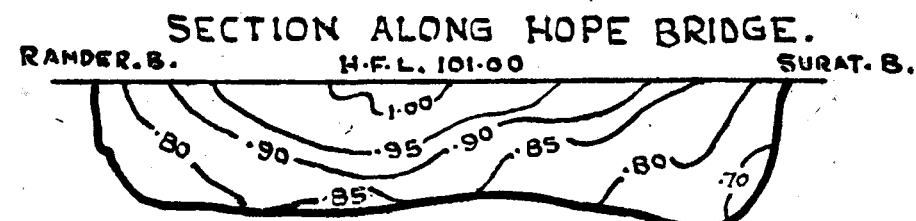
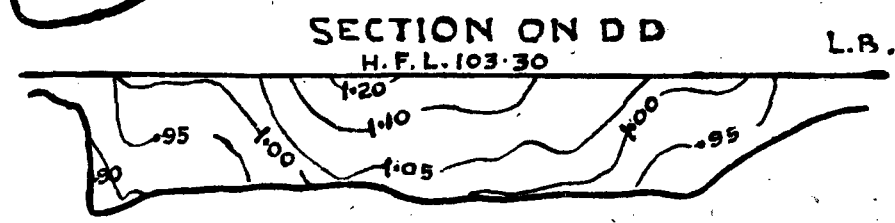
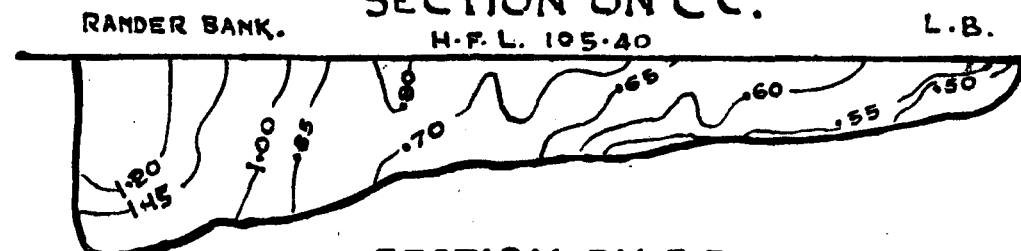


VELOCITY SECTIONS

WITHOUT GROUYNE
SECTION ON C C.



WITH GROUYNE
SECTION ON C C.



shielded by it. Due to the attraction local scour was caused along the pile line and the deep channel previously close to the bank was pulled away and made fairly central in line with the embankment — see photos 5251 and 5236.

The flow conditions at the pile embankment were however noticed to be very severe. Rollers with horizontal axis on either side of the piles formed which owing to their high velocities and consequent turbulence created dangerous conditions for the boats plying about. Also the deep scour caused by the rollers will require very long piles about 125 ft. long to ensure adequate fixity after scour. As the length of available piles is about 50 ft. to 60 ft., $2\frac{1}{2}$ length of pile lines with overlap would be necessary and hence the embankment would be prohibitive in cost and construction very difficult.

A submerged pile line was expected to reduce the cost appreciably, at the same time improve the flow conditions at the pile embankment. The submerged embankment was however not so efficient in reducing the undermining. Its efficacy was found to be disproportionately low with the economy.

Photo 4249

An alternative of open pile line projecting above high flood was tested but with little success.

Attracting groynes and pile embankments thus having given poor results, attention was refocussed on the repelling spur. This was already found very effective, the main objection against it being its high cost. Hence submerged groyne similar to one at Rander was examined.

Various alternative positions, orientations and lengths of the groyne were tested as in the case of Rander groyne. Ultimately 250 ft. long groyne pointing 65° upstream and submerged about half flood level was found to meet the requirements.

(B) Results with the submerged groyne.

Below the Makhi Khadi, along the Surat bank the deep channel which was previously bugging the bank was pushed out almost to the centre. Maximum velocities could not be reduced but the high velocity current responsible for the damage on the bank was shifted away to follow the deep channel.

Photo 5306

Figs 26 & 27

Along the right bank opposite the groyne the velocities increased by about 40% but the net velocities were about the same as those on the left bank. The right bank here being on the inside i. e. convex side of the bend severe erosion at this bank, is not anticipated in the post-groyne period. It is not, however, unlikely that the river will develop some tendency for

erosion, being constricted in an extraordinary narrow reach. This, however, is unavoidable, lest Surat cannot otherwise adequately be protected. And if unavoidable, it was learnt, that erosion upto about 100 ft. inside the bank opposite and a little further downstream is permissible.

Below Nanpura Post Office, the effect of the groyne was feebly felt and flow conditions reverted to existing conditions.

Appendix II Details of the Surat groyne are given in Appendix II.

(iii) Protective works for arresting erosion on the right bank near Bhatpur.

Very valuable fertile land, about one square mile in area, and costing about 3,000 rupees per acre has been eroded by the river in the embayment near Bhatpur. Main cause of this erosion is strong prolonged action in the flood, though tidal action is partially responsible in accentuating the damage. With a view to stop further inroads into the bank different types of training works were tested in the model.

The river below Umra gradually widens to about 3 times its average width to reunite below Bhatpur. The main channel of the river here bifurcates forming deep channels along the two banks with a low dividing island in the centre. With the present configuration of the river, protective measures for Bhatpur should aim at either reducing the discharge and thus the intensity of attack of the Bhatpur channel or else receding the deep section and eroding current of the Bhatpur arm away from the Bhatpur bank.

Various experiments on the above lines were conducted and are being continued to evolve a suitable measure which will provide sufficient protection commensurate with its cost.

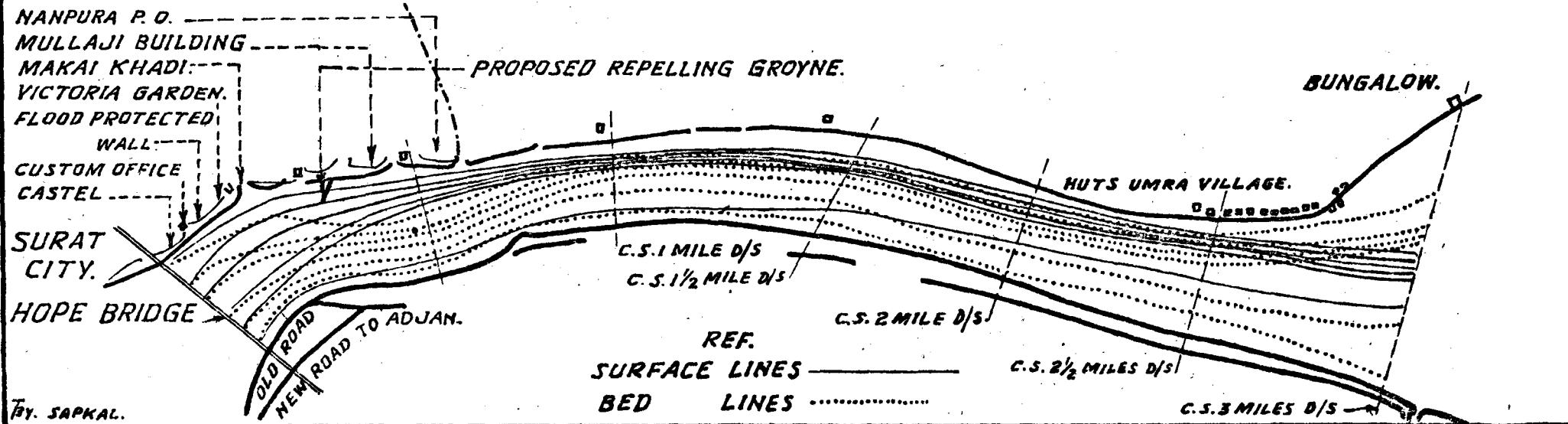
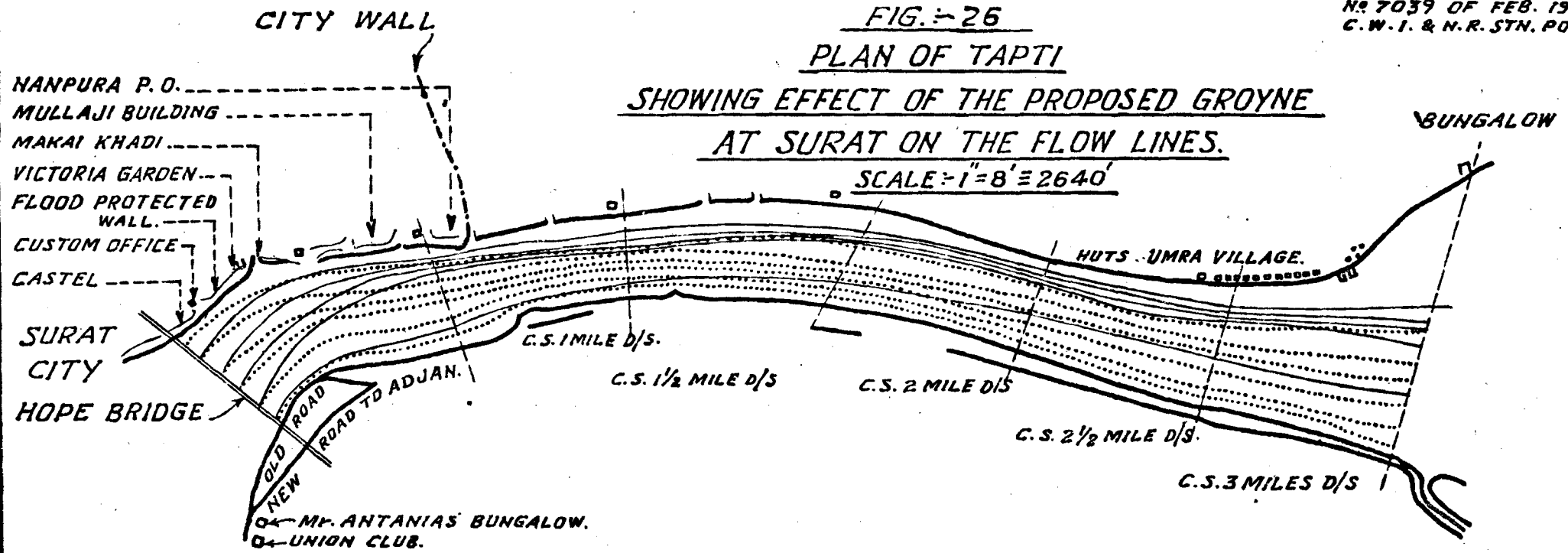
3. Recommendations:—

(1) For local protection along the river bank at Rander and Surat, submerged type of repelling groynes according to detailed designs given in Appendices I and II should be adopted.

(2) For protection of the bank in the Bhatpur embayment a reasonably economic measure has not so far been evolved. Model experiments are continued to test out exhaustively all possible alternatives.

FIG. - 26
PLAN OF TAPTI
SHOWING EFFECT OF THE PROPOSED GROUYNE
AT SURAT ON THE FLOW LINES.

SCALE = 1" = 8' = 2640'



BY. SAPKAL.

FIG. 27

No. 7037, OF FEB. 1949
C.W.I.N.R. STN. POONA

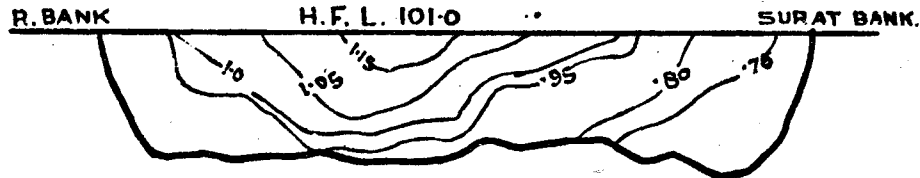
VELOCITY SECTIONS ACROSS TAPTI AT SURAT.

SCALE: HOR. 1" = 2' = 660'
VER. 1" = 1' = 72'

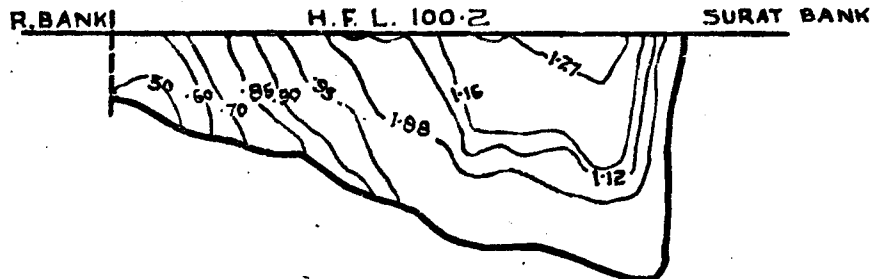
VELOCITY SECTIONS

WITHOUT GROUYNE

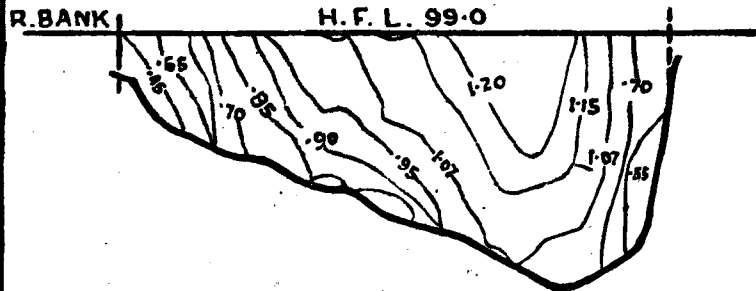
SECTION ALONG HOPE BRIDGE



SECTION ALONG A-A

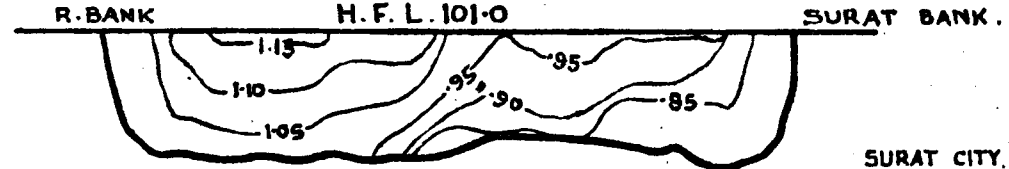


SECTION ALONG B-B

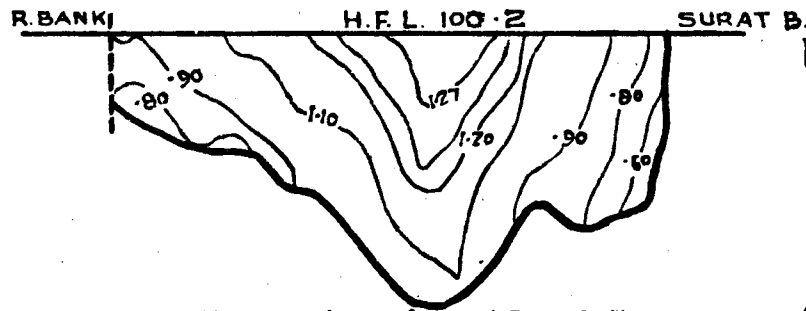


WITH GROUYNE

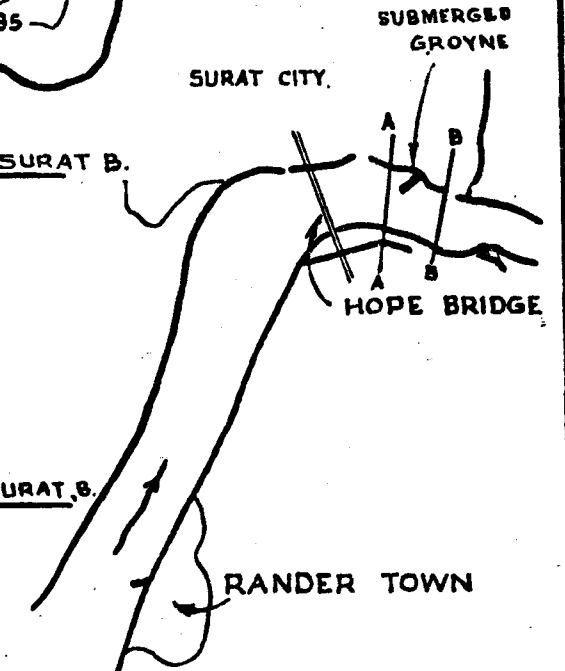
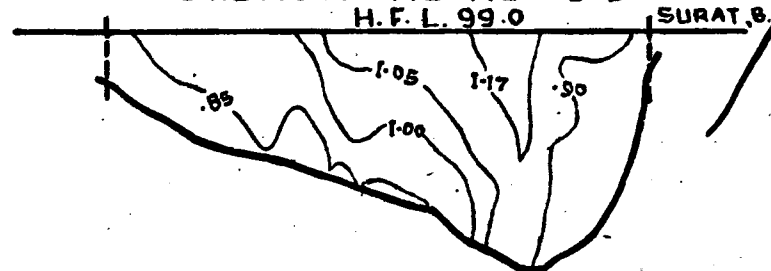
SECTION ALONG HOPE BRIDGE



SECTION ALONG A-A



SECTION ALONG B-B



PART PLAN OF TAPTI.

SCALE: 1" = 20' = 6600'

By. M.K. SABNIS.

THE TAPTI AT SURAT

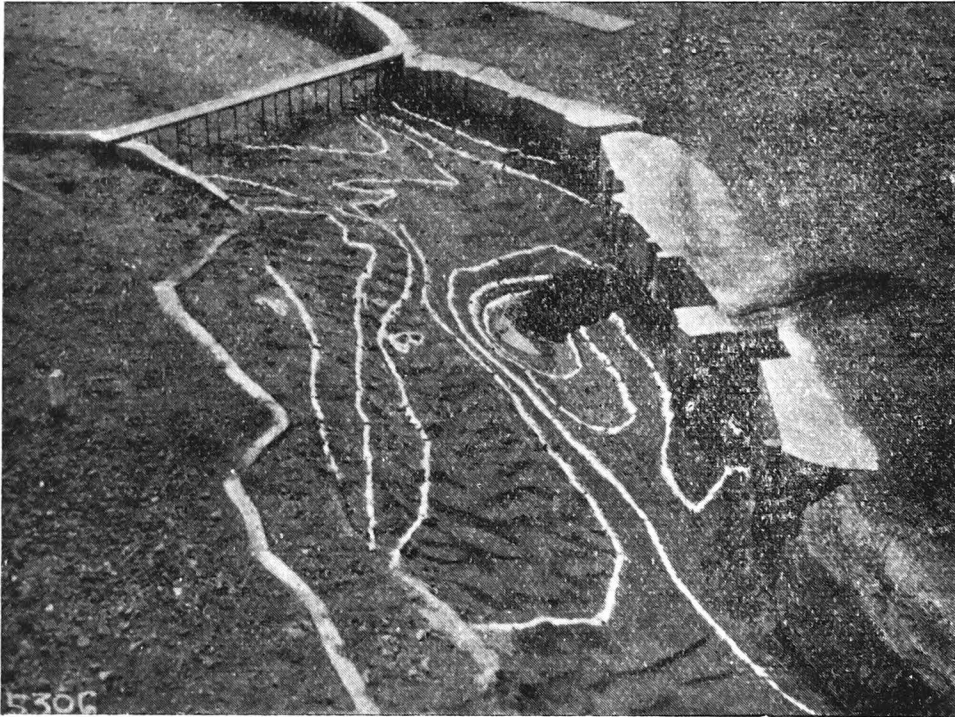


PHOTO No. 5306. Showing efficacy of the submerged repelling groyne below Makhi Khadi in protecting the Surat bank from frontal attack & consequent deep scour along the left bank.

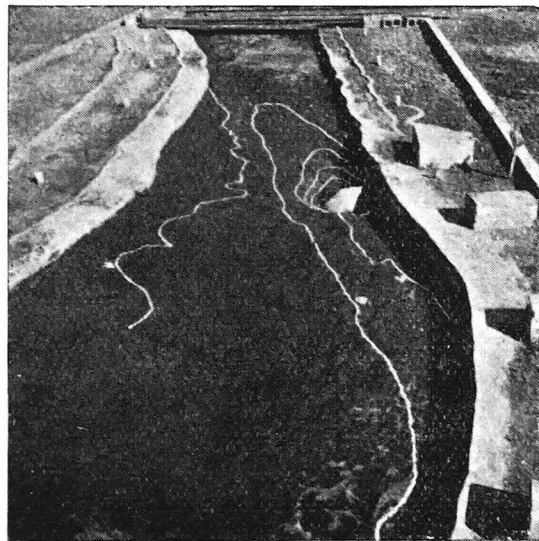


PHOTO No. 5219. Showing deflection of the deep channel away from the Rander bank after the submerged groyne came in operation in the model.

THE TAPTI AT SURAT

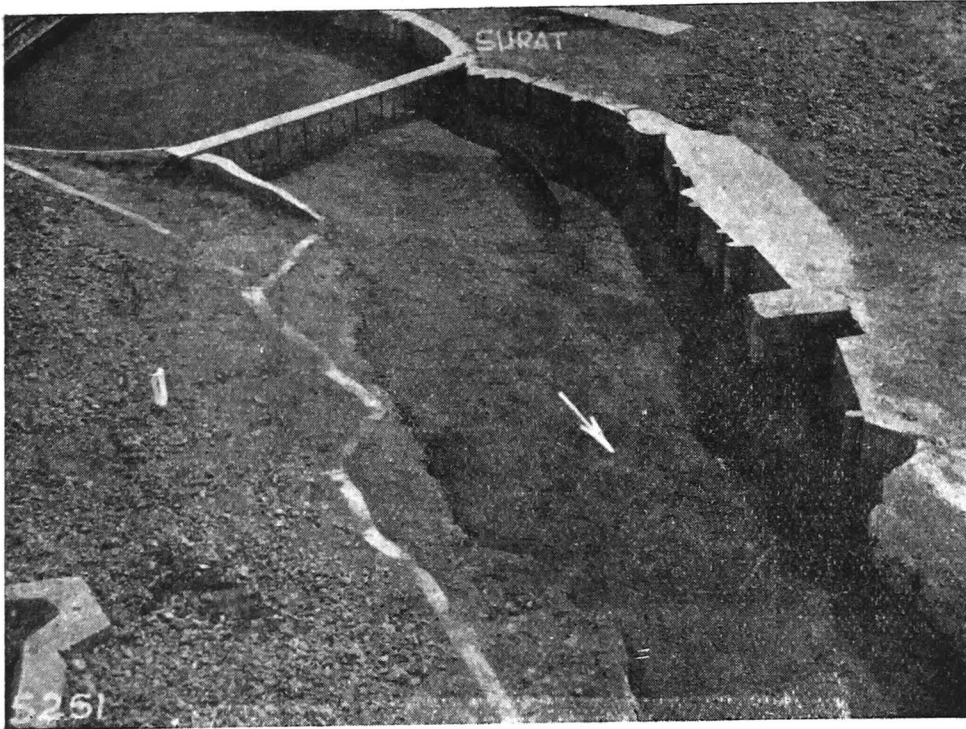


PHOTO No. 5251. Model as laid. The deep channel along the Surat bank below Hope Bridge is clearly visible.

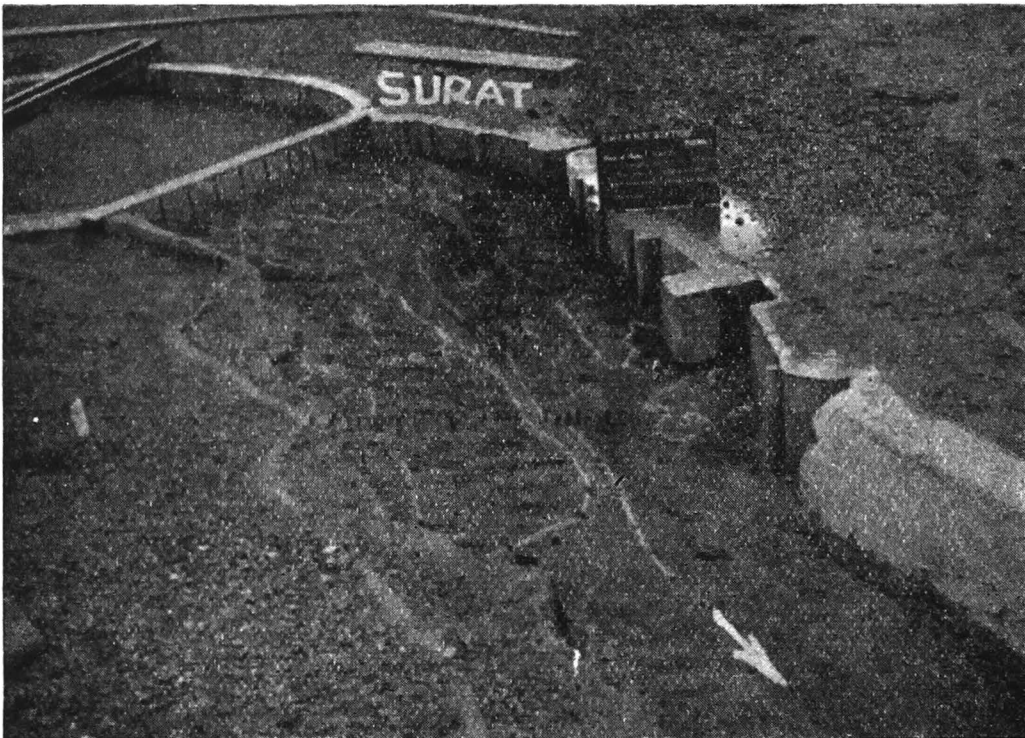


PHOTO No. 5236. Model after running with the continuous pile embankment protruding above H.F.L. The photo shows the deep channel being pulled away from Surat bank and made central by the pile line.

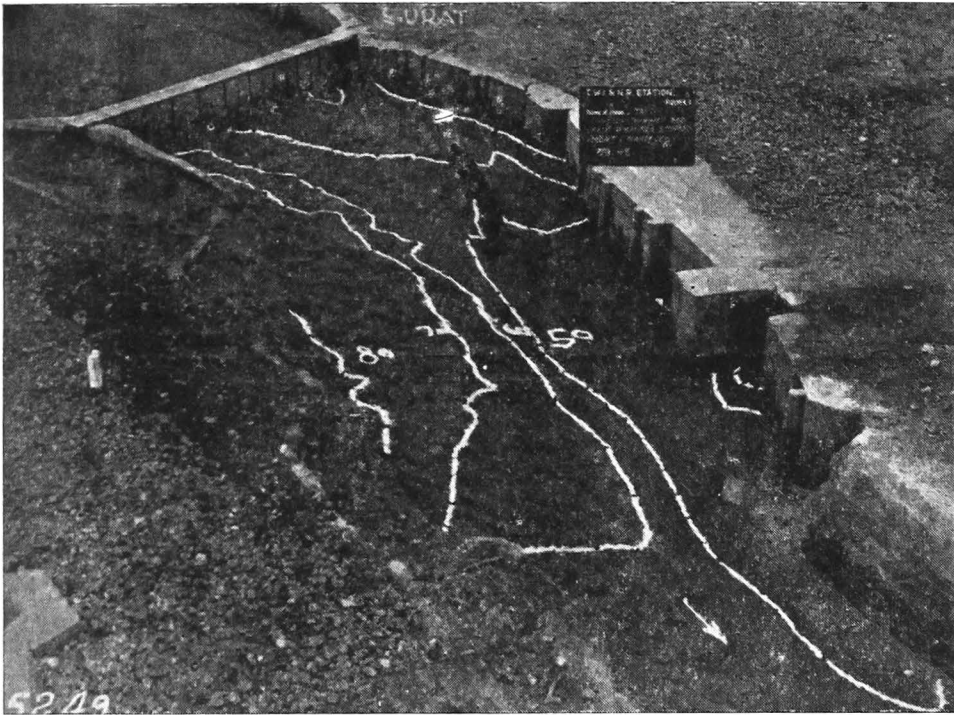


PHOTO No. 5249. The submerged pile embankment was inefficacious in attracting the deep channel on to itself.

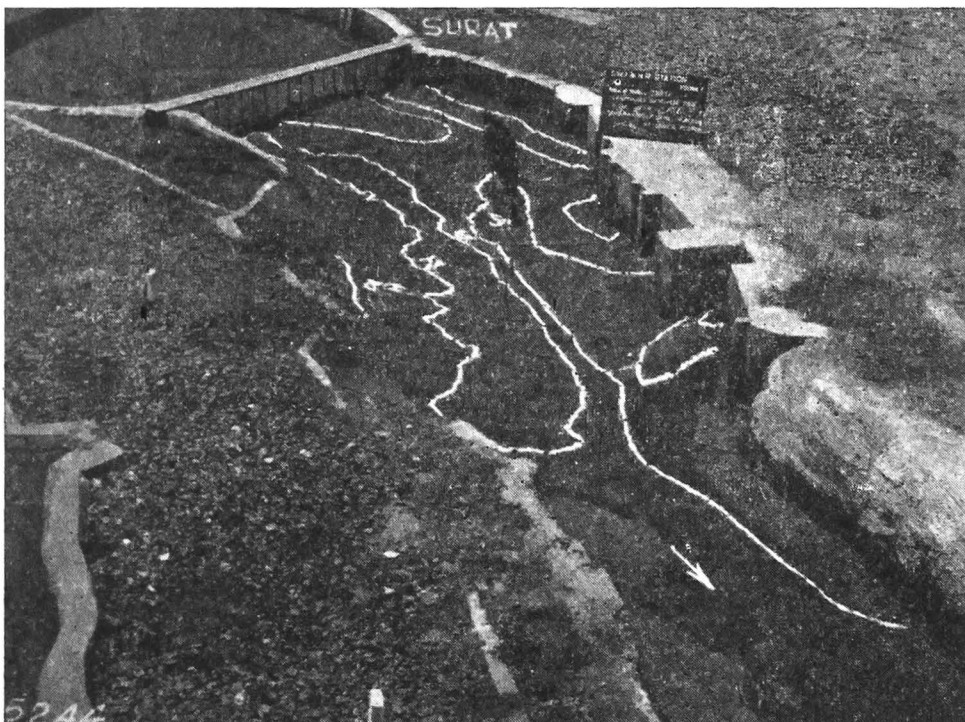


PHOTO No. 5244. The open pile embankment with top above H.F.L. was unsuccessful in pulling away the deep channel from Surat bank.

APPENDIX I

Details of the proposed submerged repelling groyne at Rander.

The groyne should be positioned, aligned and oriented
 Fig. 28. as shown in Fig. 28.

Shank of the groyne.

Length of groyne at top	400 ft.
Top width	10 ft.
Top level at half flood level i.e.	R.L. 66.00
Side slopes of the shank	1½ to 1.
Slope of the nose	3 to 1.

The bed level of the river at the groyne is R.L. 53.00 and the proposed R. L. for the top of the groyne is 66.00. The low tide level being 64.00, the groyne will have to be constructed in water and should be wholly of concrete blocks or stones. The stones for the core may vary in size from quarry refuse to about 1 Cft. but the slopes of the shank and the nose should consist only of heavier 'one man' stones to a depth of 3 ft.

Alternatively it can be constructed by dumping sand in gunny bags upto L.T.L. for the groyne core and finishing off the slope faces by stones or blocks of concrete for a thickness of four to five feet from the faces Above L.T.L. it should become possible to build up the sand core properly and facing stones for 3 ft. thickness should be hand pitched. The nose should be only in stone whether the groyne is a sand core groyne or full stone groyne. Another modification necessary with a sand core groyne will be about side slopes which should be not steeper than 2½ to 1.

The alternative proposal of sand groyne will have to be considered and compared with the stone groyne and the more economical of the two should be adopted.

Apron design.

Apron should be provided to prevent the slipping of the stones of the groyne nose and the shank into the scour-hole caused by them. For this reason the apron stone should be sufficient after launching to shield the sloping faces of the sand bank from below the bottom of the stone groyne down to the deepest bed level after scour. The stones in the apron should invariably be heavy, weighing not less than 120 lbs. each.

In the model, the maximum scour obtained with the full-height groyne was 75 ft. below H.F.L. i. e. to R.L. 30.00 while with the submerged groyne

the maximum scour was 70 ft. below H.F.L. i. e. to R.L. 35.00. Thus the depth of scour was reduced appreciably, by about $\frac{1}{2}$.

The analysis and study of the available data about scour depths at the noses of the full-height groynes suggest that the maximum depth below H.F.L. to be anticipated is about $2R$ where R is Lacey depth given by the formula:

$$R_{\text{Lacey}} = .473 \left[\frac{Q}{f} \right]^{\frac{1}{3}}$$

Q being maximum discharge in cusecs and 'f' the silt factor defined as $1.76\sqrt{m}$, where 'm' is mean diameter of the bed material in mm.

$$\therefore \text{Maximum depth of scour} = 2R$$

$$= 2 \times .473 \left[\frac{Q_{\text{max}}}{f} \right]^{\frac{1}{3}}$$

$$Q_{\text{max}} = 7,50,000 \text{ cusecs}$$

$$m = 0.33 \text{ mm.}$$

$$\therefore f = 1.76\sqrt{.33} = 1.0$$

$$\begin{aligned} \therefore 2R &= 2 \times .473 (7,50,000)^{\frac{1}{3}} \\ &= 85.6 \text{ or say } 86 \text{ ft.} \end{aligned}$$

Therefore lowest bed level in the scour-hole at the nose of the groyne will be 16 ft. below H.F.L. Highest recorded H.F.L. at the Hope bridge is 101.00 in 1944. With surface slope of .391 ft./1,000 ft., corresponding H.F.L. at the proposed site of the groyne, which is about 2 miles upstream. of the Hope bridge works out to

$$\text{R.L. } 101.00 - \frac{5280 \times 2 \times .391}{1,000} = \text{R. L. } 105.13.$$

Therefore lowest bed level in the scour-hole at the nose of the groyne = $105.13 - 86.00 = 19.13$ or say R. L. 19.00. With 7% reduction for less scour with submerged groyne depth of scour = $86 - 6.02 = 79.98$ or say 80 ft.

This scour depth is thus 10 ft. more than observed in the model. The divergence being small, in the interests of safety, it is advisable to design the apron for depth of 80 ft.

The quantity of stone per foot should be sufficient to cover the prospective slope of the scour-hole with an adequate thickness of the stone layer, say $2\frac{1}{2}$ ft..

$$\therefore \text{Cubic contents per foot} = 125 \times 1.3 = 164 \text{ cft/ft.}$$

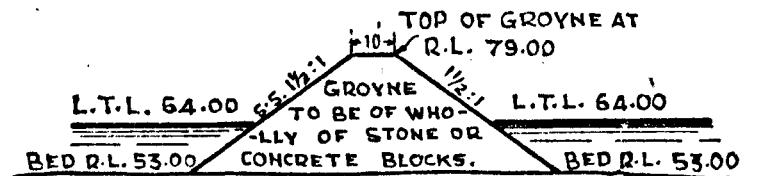
FIG. 28

TAPTI RIVER AT RANDEK
SHOWING DETAILS OF PROPOSED
SUBMERGED REPELLING GROUYNE
ON THE RIGHT BANK.

Scale:-1"=600.

SECTION OF THE PROPOSED SUBMERGED-
-REPELLING GROUYNE AT RANDEK ACROSS
THE SHANK.

H.F.L. 103.13.



Scale:-1"=60FT.

SECTION A-A.

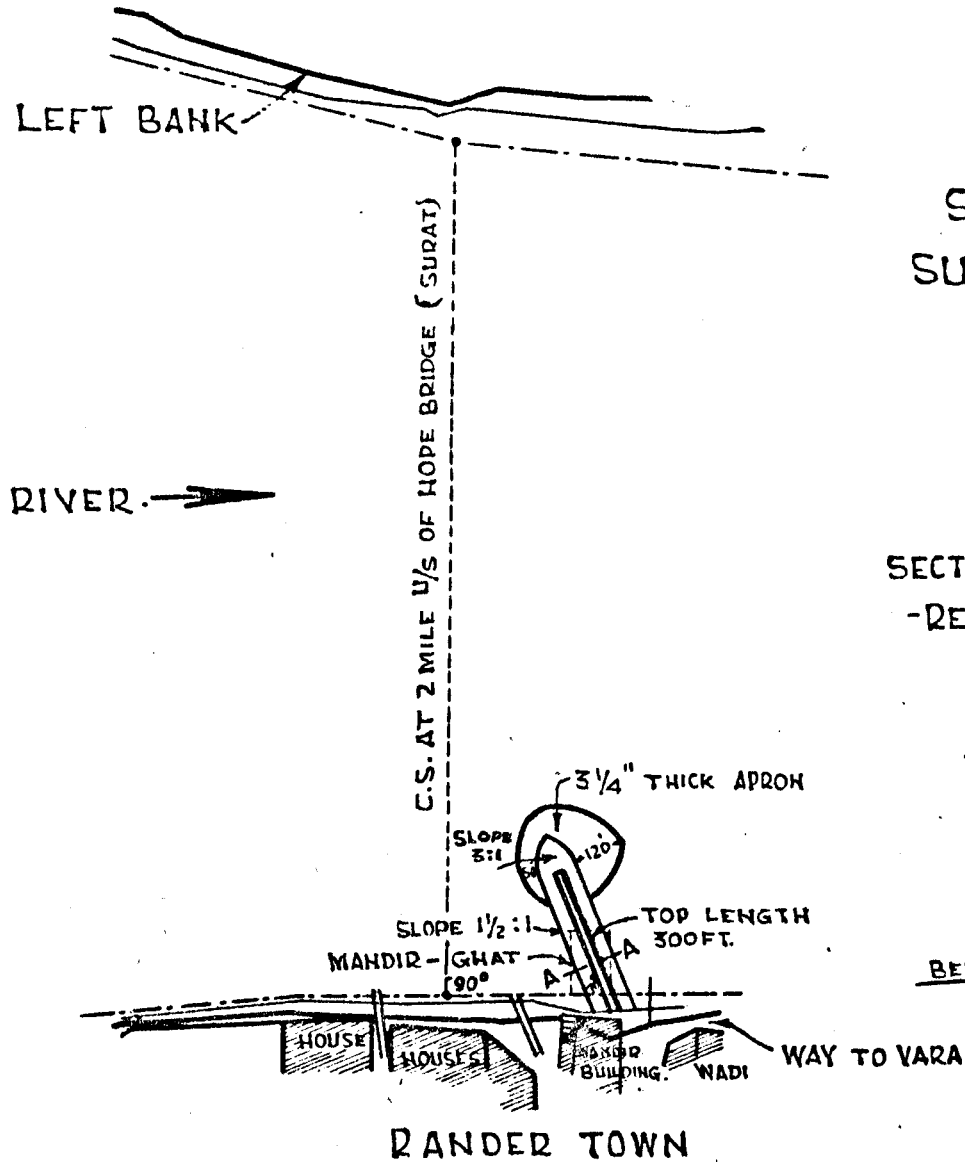
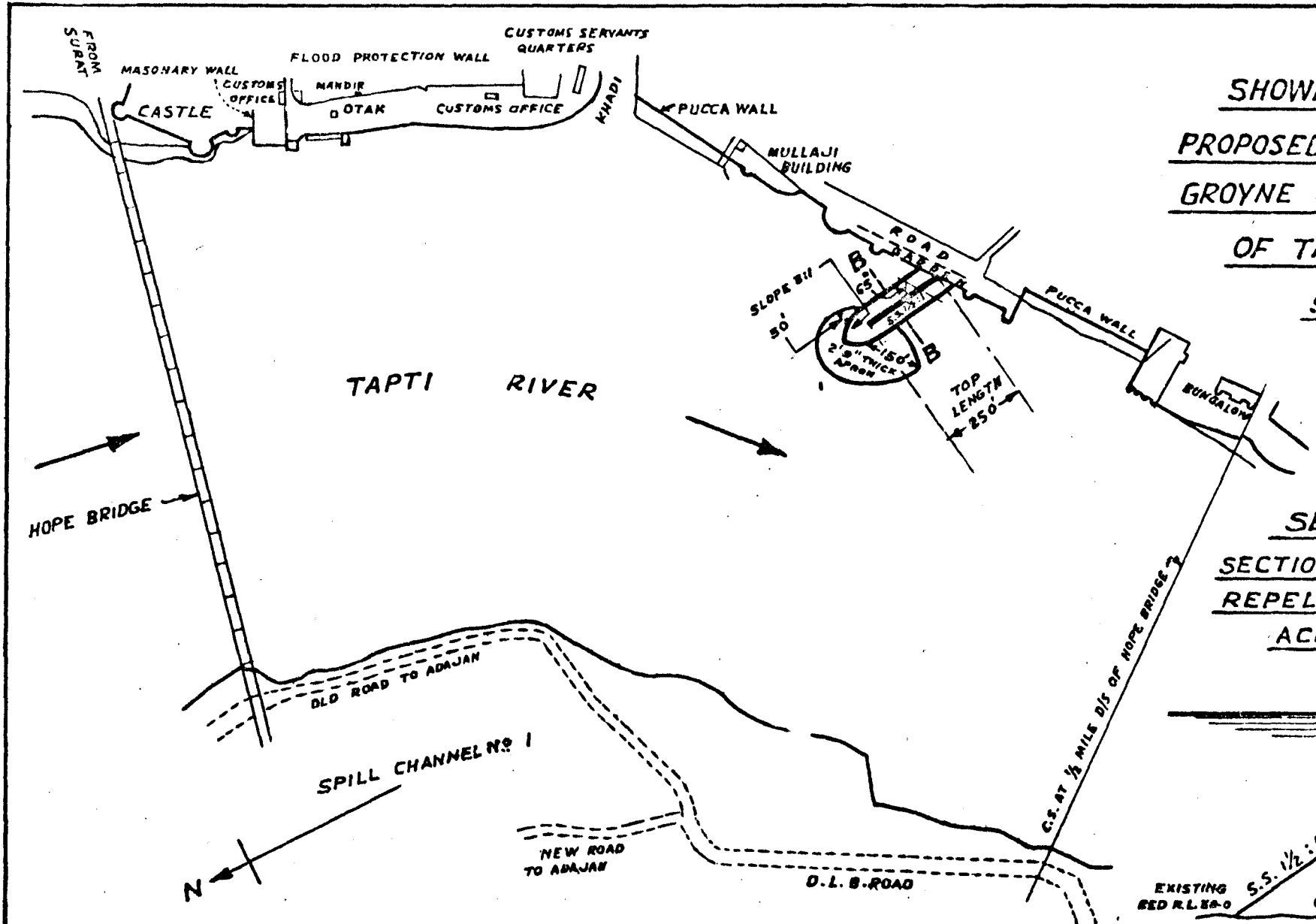


FIG. 29

**SHOWING DETAILS OF THE
PROPOSED SUBMERGED REPELLING
GROYNE ON THE LEFT BANK
OF TAPTI AT SURAT.**

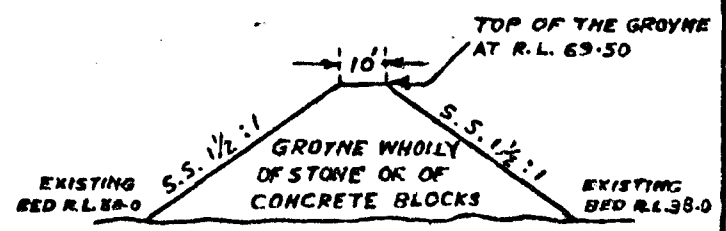
SCALE 1" = 600'

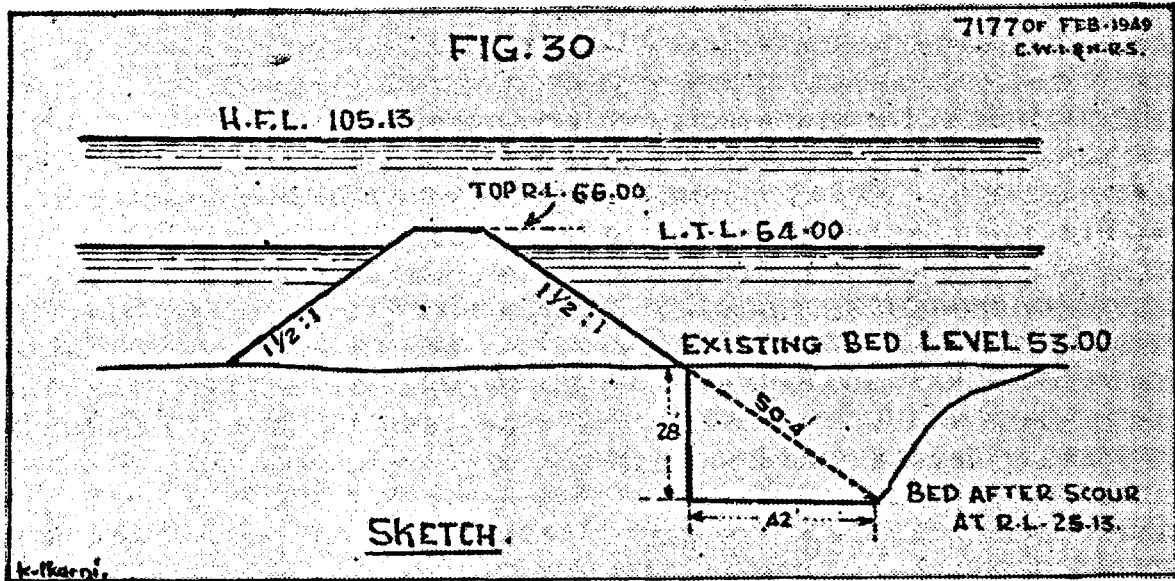


**SECTION ON BB
SECTION OF THE PROPOSED
REPELLING GROYNE AT SURAT
ACROSS THE SHANK**

SCALE 1" = 60'

H.F.L. 100.70





So as to ensure adequate protection at all points where scour is expected and especially where deeper scour is anticipated heavier apron is provided at the downstream face of the nose. The apron shape in plan, so designed, fairly conforms to the shape of the scour-hole as indicated in the model.

The width of the apron may be 50 ft. Hence thickness should be $\frac{164}{50} = 3.28$ or say $3\frac{1}{4}$ ft.

APPENDIX II

Details of the proposed submerged repelling groyne at Surat

The position, orientation and alignment of the groyne should be as shown in fig. 29.

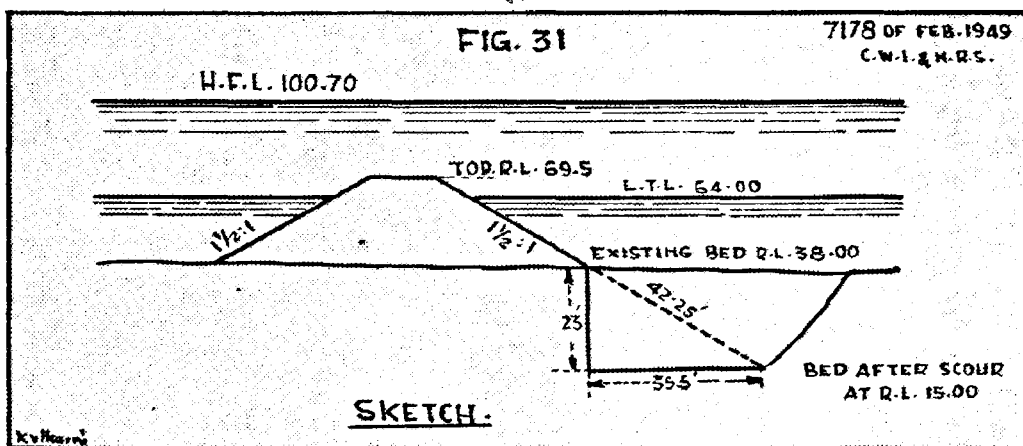
Shank of the groyne.

Length of the groyne at top	...	250 ft.
Top width	...	10 ft.
Top level should be at half flood level i.e.	...	R. L. 69.50
Side slopes of the shank	...	1½ to 1
Slope of the nose	...	3 to 1

The groyne to be either of full stone type or else with sand core, similar to the Rander groyne. Pitching on slopes above L.T.L. should also similiary be hand laid.

Apron Design.

The maximum depth of scour with the submerged groyne was equivalent to 81 ft. as against 84 ft. with the full-height groyne while according to 2D_{Lacey}, the maximum depth works out to 86 ft. below H.F.L. As all these figures are nearly equal the design of apron is based on depth of scour of 86 ft.



Cubic contents per foot = $42.25 \times 2.5 = 105.50$ cft. And with an allowance of 30% for stones being washed away, cubic contents per foot = $105.5 \times 1.3 = 137$ cft/ft.

The width of the apron = 50 ft. Hence thickness should be $= \frac{137}{50} = 2.74$ or say 2 ft.-9 in.

12. Training the Kuakhai River at the Proposed Road Bridge site near Baliana (Orissa).

Introduction.

The Kuakhai river takes off from the right bank of the Katjuri river opposite Cuttack. The off-take being about 6 miles below Naraj. (fig. 32). On its right bank the Katjuri spills about a lakh of cusecs—part of the discharge flowing into the Kuakhai above the Railway bridge and part finding its way into the Kuakhai below the Railway bridge through the Barang Nala and other culverts in the Railway embankment. Below the Railway bridge the Kuakhai spills over its right bank and this spill together with that coming through the culverts flows down towards the Mancheswar Gap where a portion of this discharge spills through it to flow down to Puri district, and the remainder finding its way back into the Kuakhai. During the last four years (as stated by the S. E., E. C., Cuttack) embankments have been reconstructed along the Kuakhai on both sides. As a result of this the flood levels have risen between Kazipatna and Baliana. As such very little spill now passes through the right embankment to the Mancheswar Gap and the escape now works mostly by the Barang Nala discharge and by the flood from the river joining it just opposite the escape. The left bank of Kuakhai has irrigation embankments in certain lengths. But due to the Jaggannath Puri Road embankment which acts as a second continuous embankment, no spill generally occurs on the left bank.

The proposed national highway crosses the Kuakhai river, 1.5 miles above the bifurcation of the Kuakhai into Kushabhadra and Bhargovi, where the road bridge to cross the river is to be constructed. The Consulting Engineer, Bridges referred this case to this Station for

“Investigation of Hydrological problems of the locality and determination of.

- (ii) Exact discharge to be expected into the Kuakhai itself at the Bridge Site.
- (ii) Discharge expected through Mancheswar Gap.
- (iii) Necessity or otherwise of protective works on either banks.”

The Model

As the Kuakhai branch of the existing 1/400:1/66 V. E. model of the Mahanadi and its branches at Cuttack constructed to study the redistribution of sand charge and discharge in the Mahanadi and the Katjuri did not extend sufficiently downstream and as this area was urgently required for the model of the river Hooghly it was decided to construct the Kuakhi model

elsewhere. This new model constructed to the same scale ratio viz 1/400:1/66 extended from the Naraj weir down to about 1/2 a mile above the Katjuri Railway bridge, with the Kuakhai off-take in-between. The reach of the Katjuri with the Banki Road spill on the right and the Birnasi spill on the left was reproduced to serve the entry length which had been proved in the old Mahanadi model. Above the Naraj weir arrangements were made to simulate correct flow conditions by drawing off the proper share of discharge by the Mahanadi. The Kuakhai reach extended from the Kuakhai head down to about 3 miles below the proposed Road Bridge Site lengthwise and from the Jagannath-Puri Road to Railway embankment laterally. Arrangements were also made to collect and measure the spill discharges from the Mancheswar Gap, Kushabhadra and Bhargovi.

Procedure.

Before each experiment the model bed was moulded to 1946 river conditions and floods rising from 6 lakhs cusecs above Naraj to 15 lakhs cusecs and falling back to 6 lakhs cusecs were run with injection of bed material.

The proportions of discharge drawn by the Mahanadi and Katjuri and the sand charge were those obtained during the previous Mahanadi experiments reported in A.R.T. 1946 and 1947. Observations of velocities, flow lines, flow conditions, contours etc., were made in each experiment.

Preliminary Tests.

The Irrigation and Flood embankments were first constructed in puddle according to the cross sections supplied. Near the bridge site on the right bank, the bund is a little distance away from the river edge with a high level ground (fig. 33) which spills over only during the peak flood. In the model correct throw-off and curved flow could not be obtained till the bund slope was made 1:1 keeping the mid-depth point correct (fig. 33) which meant sacrificing reproduction of the slope of the right bank.

Fig. 33

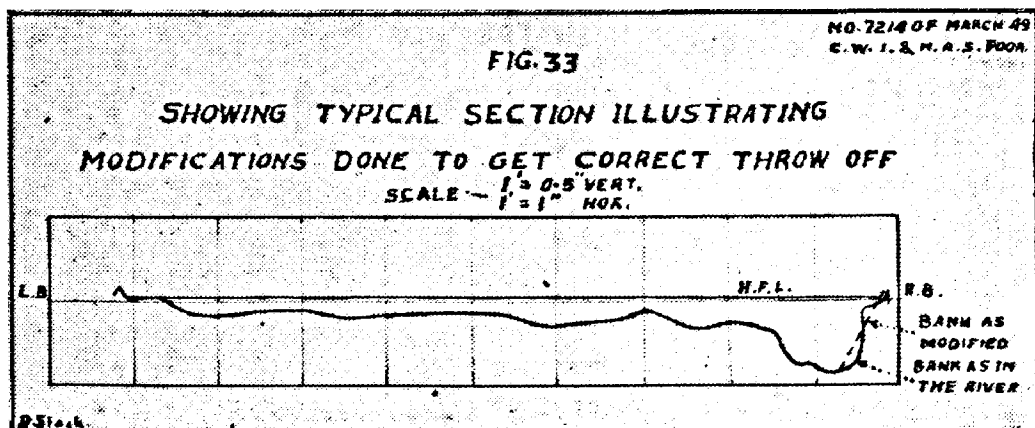
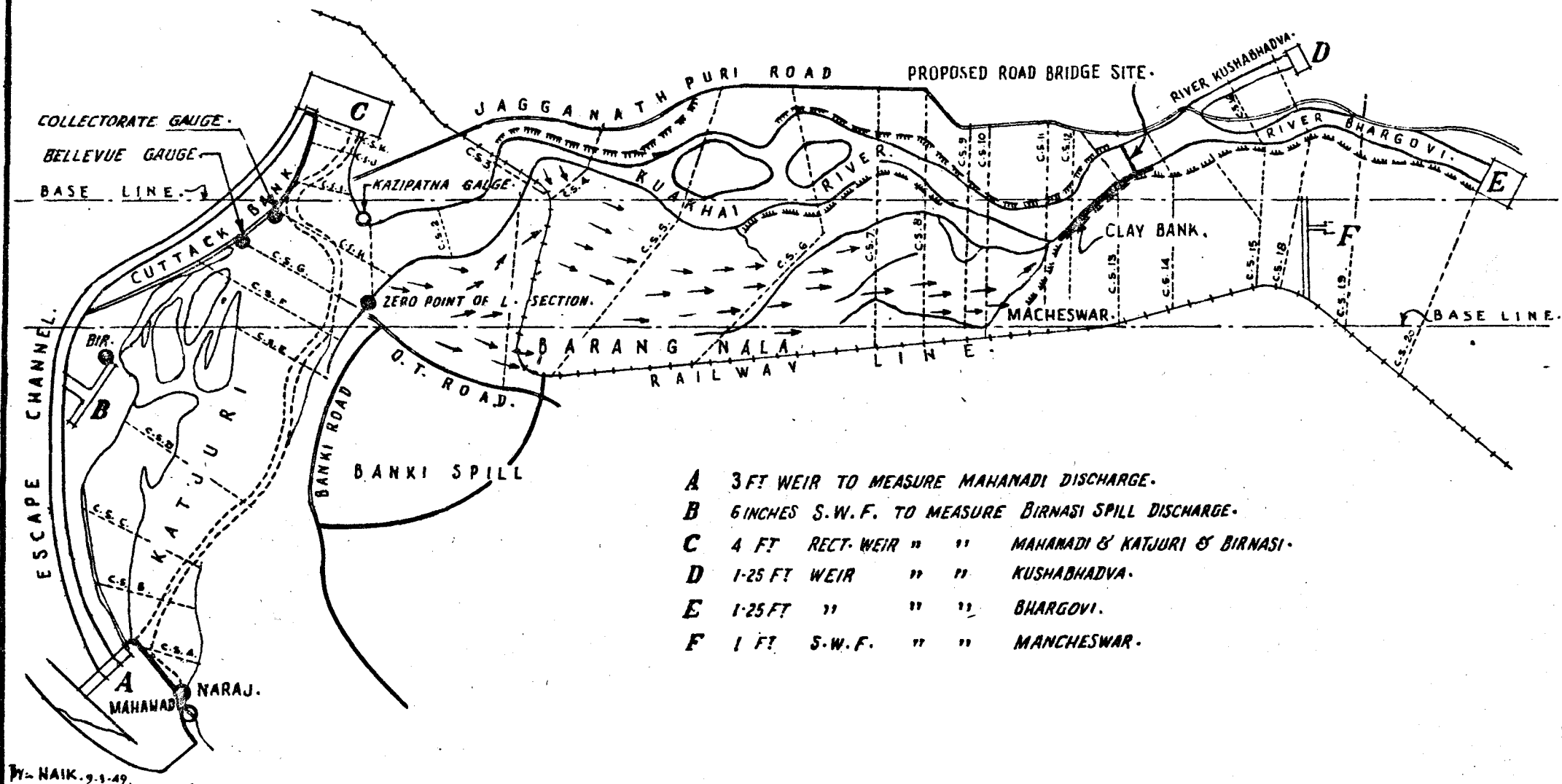


FIG. 32 $\frac{1}{400} = \frac{1}{66}$ MODEL OF RIVER KUAKHAI
 SHOWING GENERAL LAYOUT OF THE MODEL.

NO 7222 - (9 MARCH 1949)
 C. W. I. & N. R. STN POONA.

SCALE :- 12000 FT TO AN INCH.



- A 3 FT WEIR TO MEASURE MAHANADI DISCHARGE.
- B 6 INCHES S.W.F. TO MEASURE BIRNASI SPILL DISCHARGE.
- C 4 FT RECT. WEIR " " MAHANADI & KATJURI & BIRNASI.
- D 1-25 FT WEIR " " KUSHABHADVA.
- E 1-25 FT " " " BHARGOVI.
- F 1 FT S.W.F. " " MANCHESWAR.

By- NAIK. 9.1.49.

This is yet one of the many examples of compromising the bank slopes in V. E. models to get correct throw-off.

Experiment No. 1. Was done with existing river conditions.

Experiment No. 2. Was done with the proposed road bridge in position.

Discussion of Results.

Figs. 34 and 35 show the lines of flow observed with a flood of 8 lakhs cusecs above Naraj in experiment Nos. 1 and 2 respectively.

It will be seen that under existing conditions the effect of curved flow existed in that the surface flow of top water followed the right bank near the bridge site whereas the bed flow was deflected away from it. There was greater concentration of surface flow at the right bank near the bridge site in experiment 1 which decreased with the bridge in experiment 2.

With the bridge the concentration of flow at the bridge section was further away from the Right bank than under pre-bridge conditions.

Thus with the bridge, the general effect on the flow was to cause a translation of the channel further away from the right bank in the neighbourhood of the bridge site which is also seen from fig. 36 which shows the bed conditions obtained in the model in experiment 1 and 2.

Visual observations indicated attack and diving flow at the right bank near and above C.S. 12 both in experiment 1 and 2. This portion, in the river is already protected by juck work and if the local protection provided at this point in the past is adequate no further increase in the trouble due to the bridge may be expected at this point.

Figs. 37(a) and (b) show isovels (equal velocity contours) observed at C.S. 2ft. upstream of bridge site, and at C.S. 14 down stream of bridge site, observed with a flood of 8 lakhs cusecs above Naraj. The effect of the bridge in shifting high velocity to the left is clearly seen—fig. 37 (a).

Effect of bridge on the discharge of Kushabhadra & Bhargovi.

Under pre-bridge i. e. existing conditions the Kushabhadra took 39% and the Bhargovi 61% of the peak flood of 2.68 lakhs cusecs in the Kuakhai, but due to the shifting of the channel more to the left, the Kushabhadra percentage of peak flood increased to 45% while that of Bhargovi dropped to 55%.

Discharge of the Mancheswar Gap.

The discharge of the Mancheswar Gap in the model would depend on maintaining the correct H.F.L. at the Gap and the surface slope downstream of the gap. When the C.E. Bridges saw the model working he was of the opinion that in the model there was too much of ponding, downstream of the Mancheswar Gap and suggested to obtain relevant gauge data at Bhuvaneshwar; the Director with his assistants later visited the bridge site and on enquiries at site the water level attained below the Mancheswar Gap, near the bridge site was found to be 2 ft. below the top of right embankment i.e. R.L. 65.8. On return from tour these experiments were carried out and the observed discharge was as shown in fig. 38 with different slopes downstream.

Fig. 38

According to S. E., E. C.'s letter giving the data of the gauge at the road crossing of the Gangua interian drainage channel on the Bhuvaneshwar Daya road the the surface slope of the Mancheswar Spill is of the order of 2.94' per mile i.e. the water level below the Mancheswar gap near the road bridge site works out to R.L. 62.4 as against 65.8, as obtained on enquiry at site. Perhaps it may not be quite correct to interpolate the slope as there may be a number of small channels draining into the Mancheswar Gap discharge; and hence it appears that an estimate of spill discharge of 1.02 lakh cusecs based on the slope of 2.94 ft. per mile appears to be excessive and that a discharge = 68,000 cusecs based on the water level data near the bridge site obtained from enquiry might be of the correct order.

Scour at Bridge Piers.

At site the right abutment is well away from the river edge with high level ground in between. As, with the bridge, the channel tends to shift away to the left; the right abutment needs no apron protection. As regards the left abutment, the velocities near it were observed to be slack moreover being on the inside flow, no attack need be anticipated on it. As such no protection is needed for the left abutment also.

As regards scour round the piers the mean value of Lacey 'f' i.e. $f = 1.76 \sqrt{\text{mean diameter of sand in mm.}}$, worked out to 1.11 for sand samples collected across the bridge site section, which gives

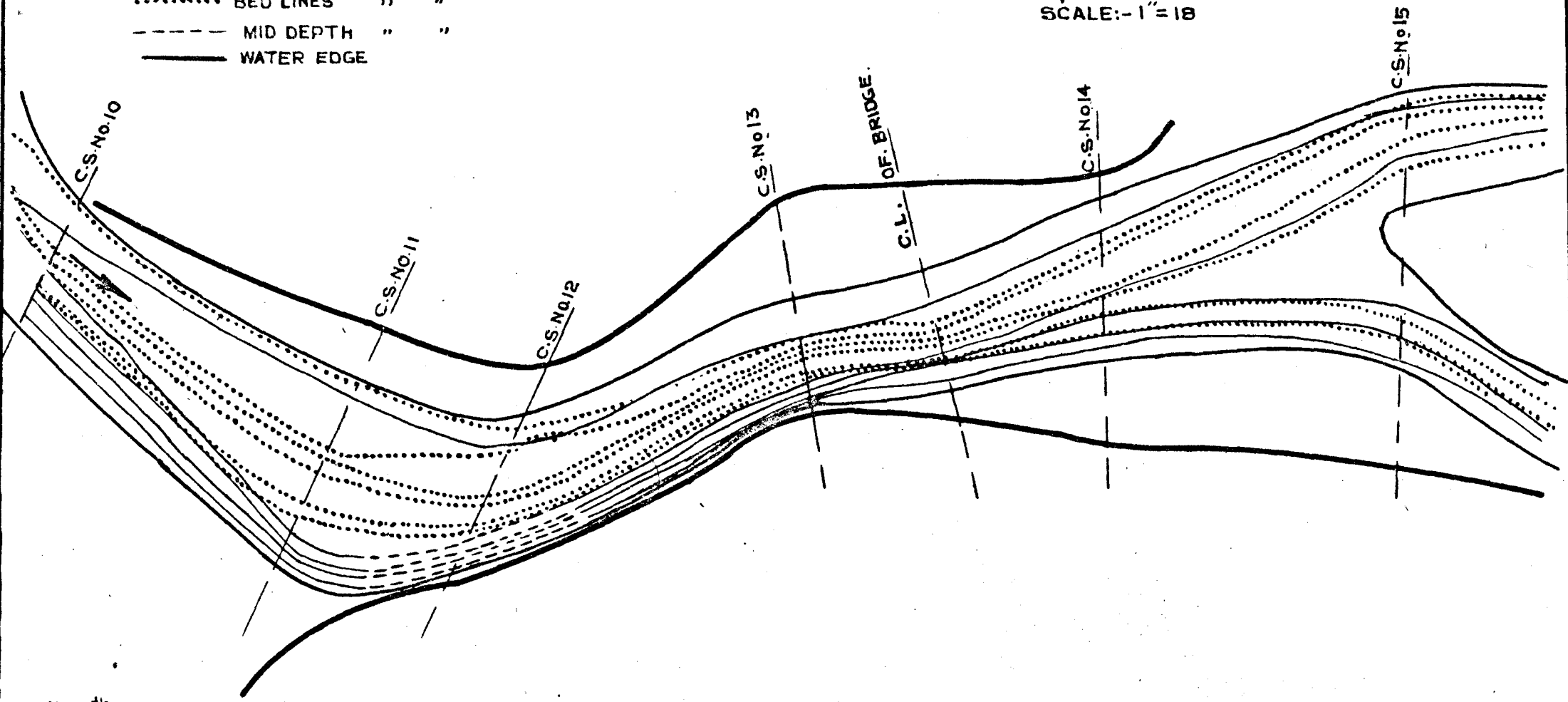
$$D_{\text{Lacey}} = 0.473 \left(\frac{Q}{f} \right)^{1/3} = 0.473 \left(\frac{2.68 \times 10^5}{1.11} \right)^{1/3} \\ = 29.45 \text{ ft.}$$

FIG: 34

1/400 : 1/66 MODEL OF RIVER KUAKHAI
SHOWING THE LINES OF FLOW
WITH EXISTING CONDITION

$Q_p :- 8,0000$ CU SECS.
SCALE: - 1" = 18'

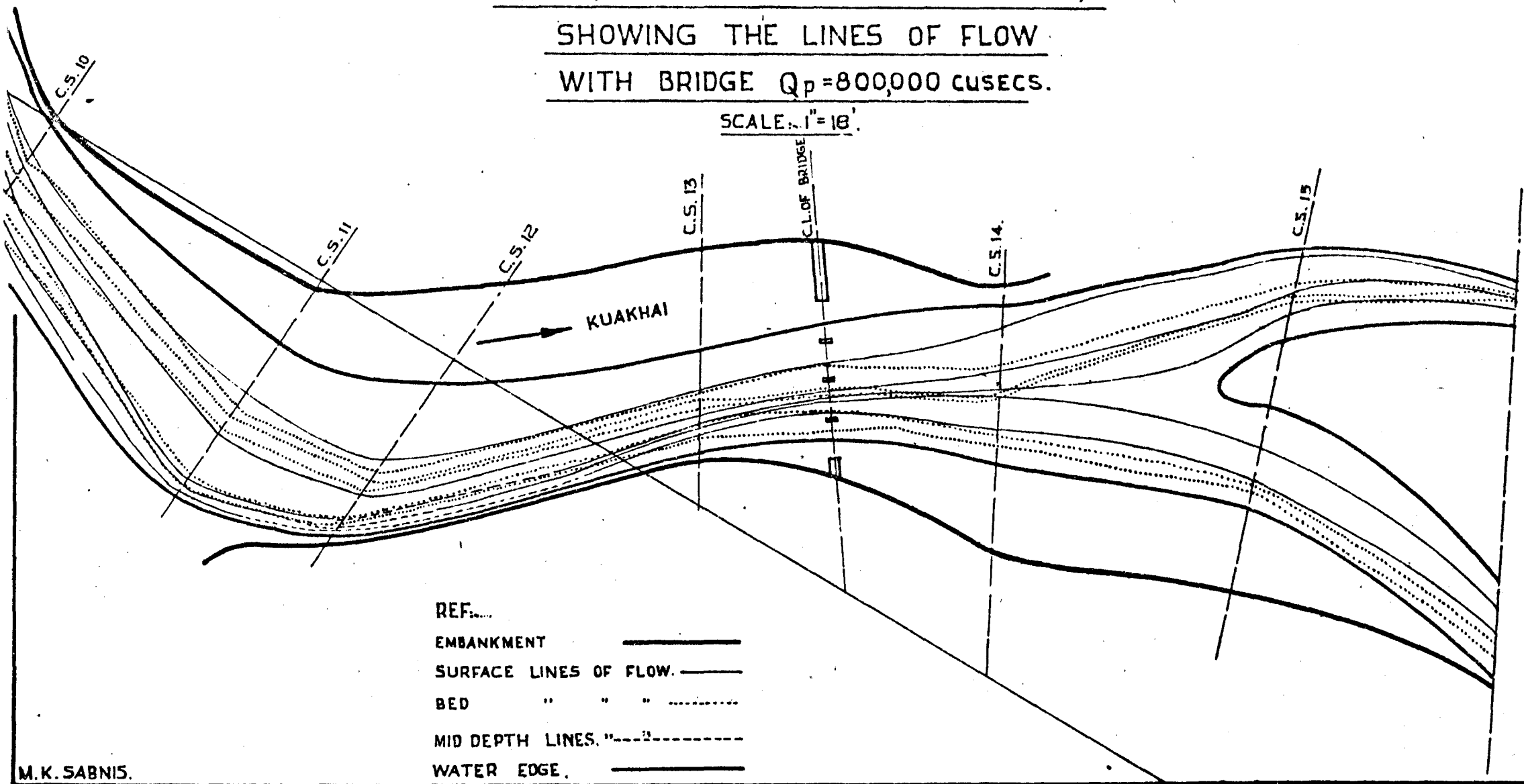
- Ref:-
- EMBANKMENT
 - SURFACE LINES OF FLOW
 - BED LINES " "
 - - - MID DEPTH " "
 - WATER EDGE



R.K. Kankar

FIG. 35
 $\frac{1}{400} : \frac{1}{65}$ MODEL OF RIVER KUAKHAI,
SHOWING THE LINES OF FLOW
WITH BRIDGE $Q_p = 800,000$ CUSECS.

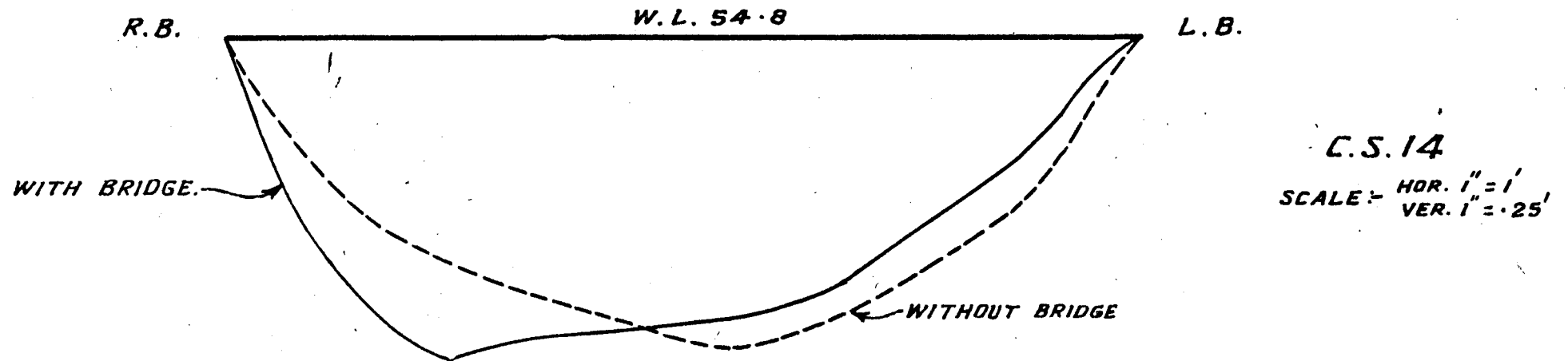
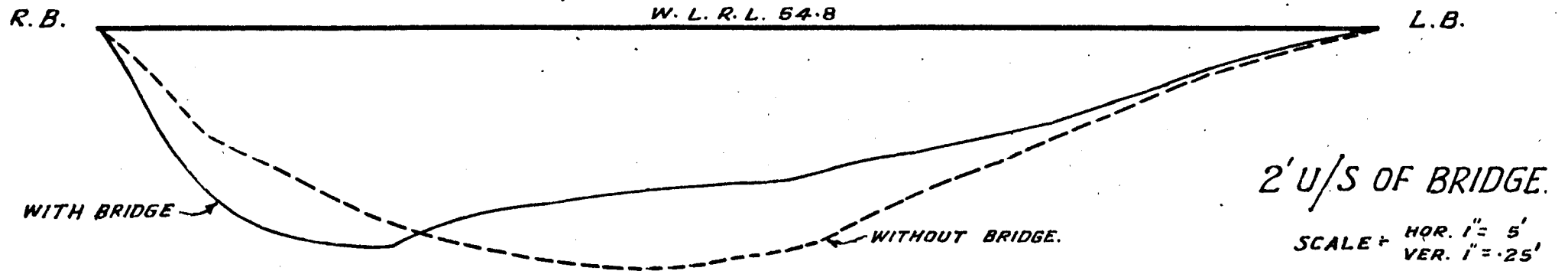
SCALE: 1" = 10'



REF.
EMBANKMENT ———
SURFACE LINES OF FLOW. ———
BED " " "
MID DEPTH LINES. " " " - - - -
WATER EDGE. - - - -

M.K. SABNIS.

FIG. 35
1/400:1/66 MODEL OF THE RIVER KUAKHAI
SHOWING THE CROSS SECTIONS 2' U/S OF BRIDGE & C.S.14
AS OBTAINED WITH & WITHOUT BRIDGE
WITH 8 LAKHS Cs. FLOOD.



By. SAPKAL

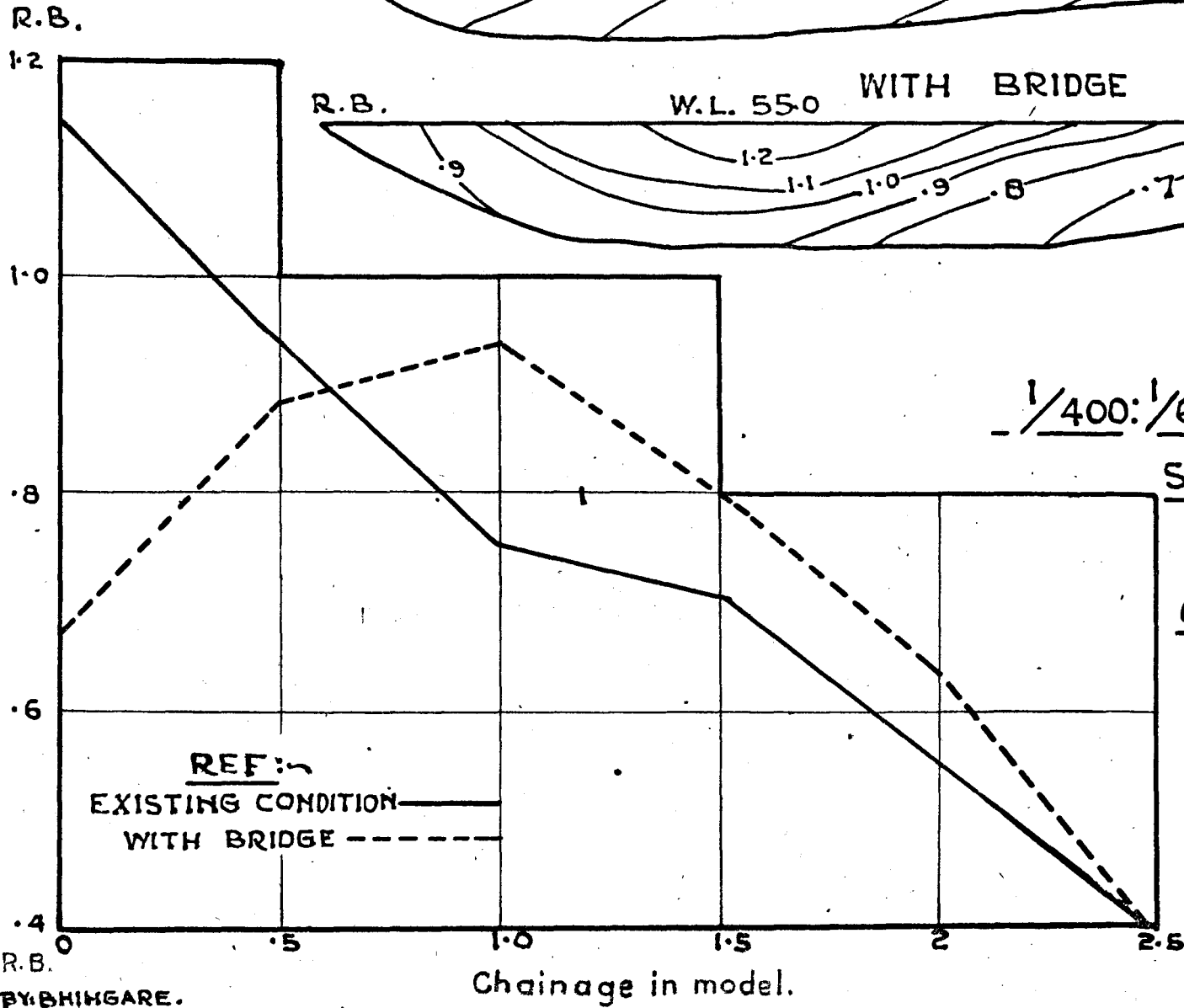
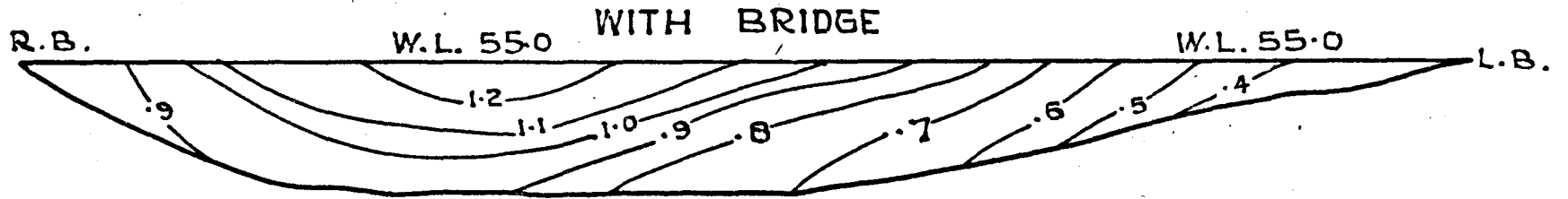
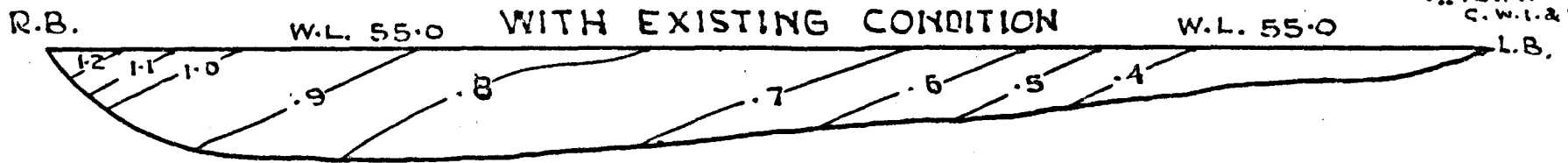


FIG. 37 (a)

1/400:1/66 MODEL OF THE RIVER KUAKHAI
SHOWING VELOCITY DISTRIBUTION

AT 2' U/S OF BRIDGE

OBSERVED WITH Q = 8 LACS CU SECS.

ABOVE NARAJ

SCALE:- HOR. 1" = .5'
VER. 1" = .5'

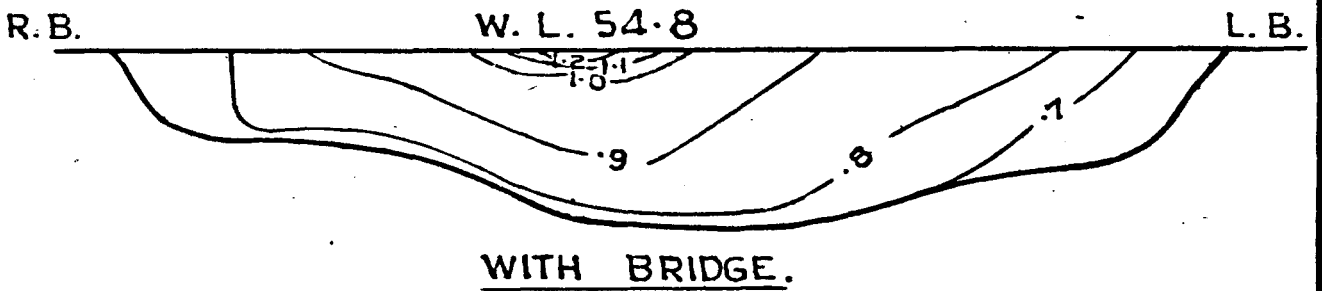
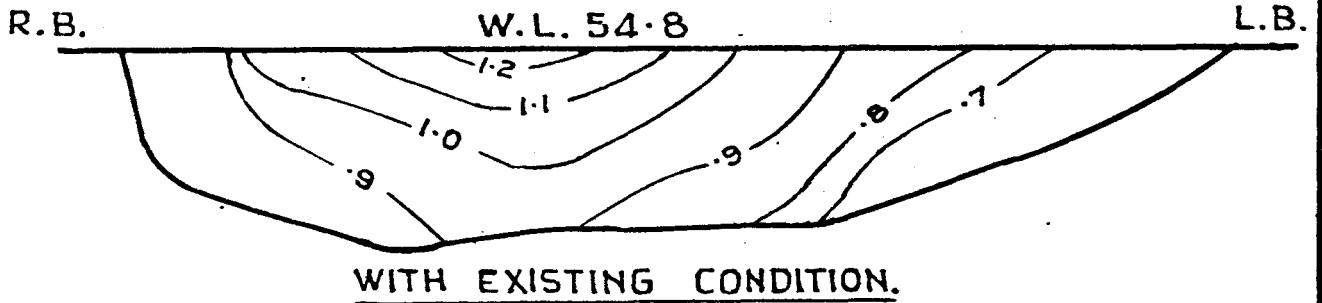
V_m = mean velocity on vertical.

L.B. \bar{V}_0 = mean velocity of the section.

FIG. 37 (b)

1/400: 1/66 MODEL OF THE RIVER KUAKHAI
SHOWING VELOCITY DISTRIBUTION AT C.S. NO 14
OBSERVED WITH Q = 8 LACS. ABOVE NARAJ.

SCALE: - $\frac{\text{HOR. 1''=1'}}$
 $\frac{\text{VER. 1''=.50'}}$



R.B. SCALE: - $\frac{\text{HOR. 1''=1'}}$ L.B.
 $\frac{\text{VER. 1''=.20'}}$

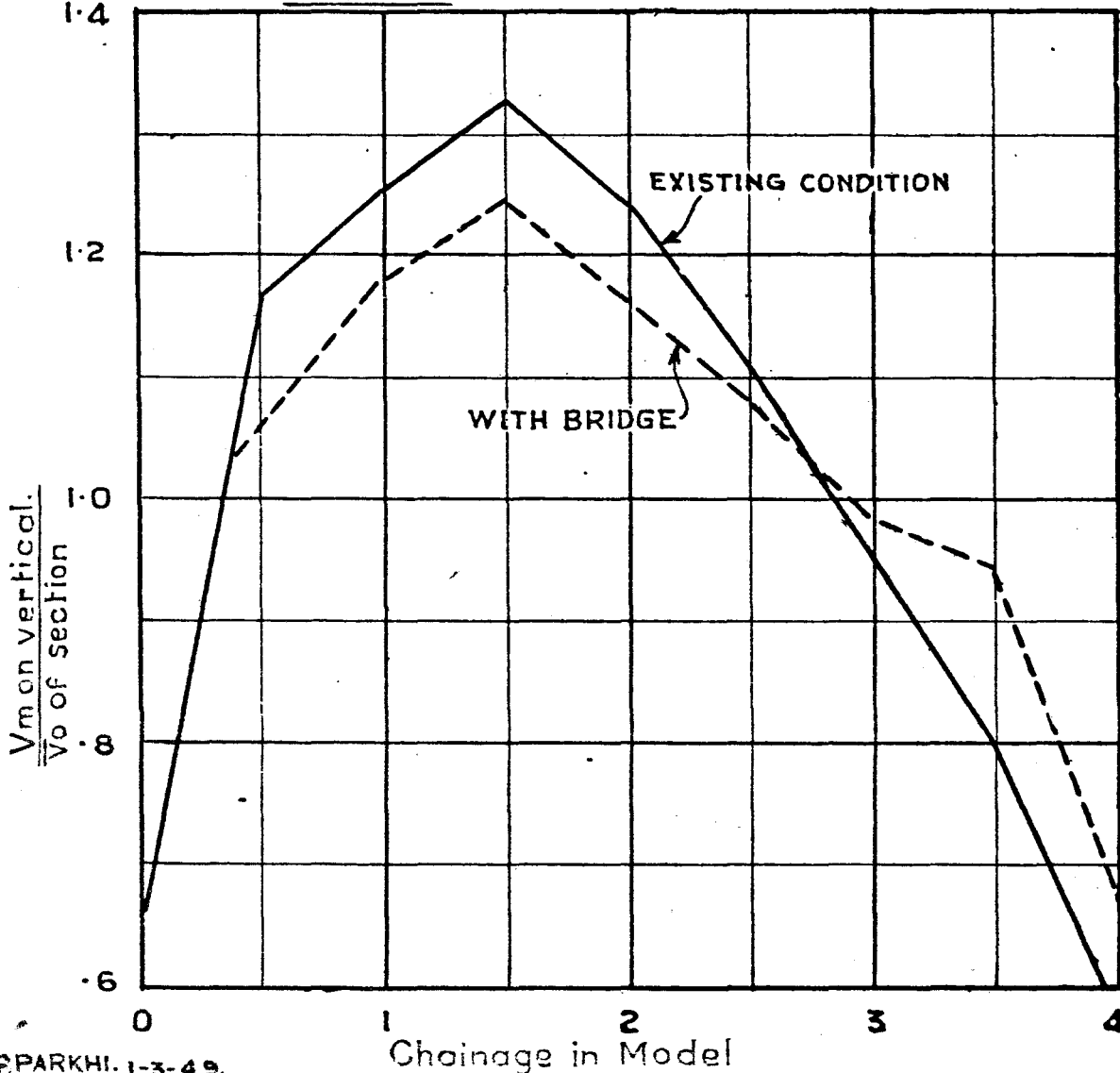
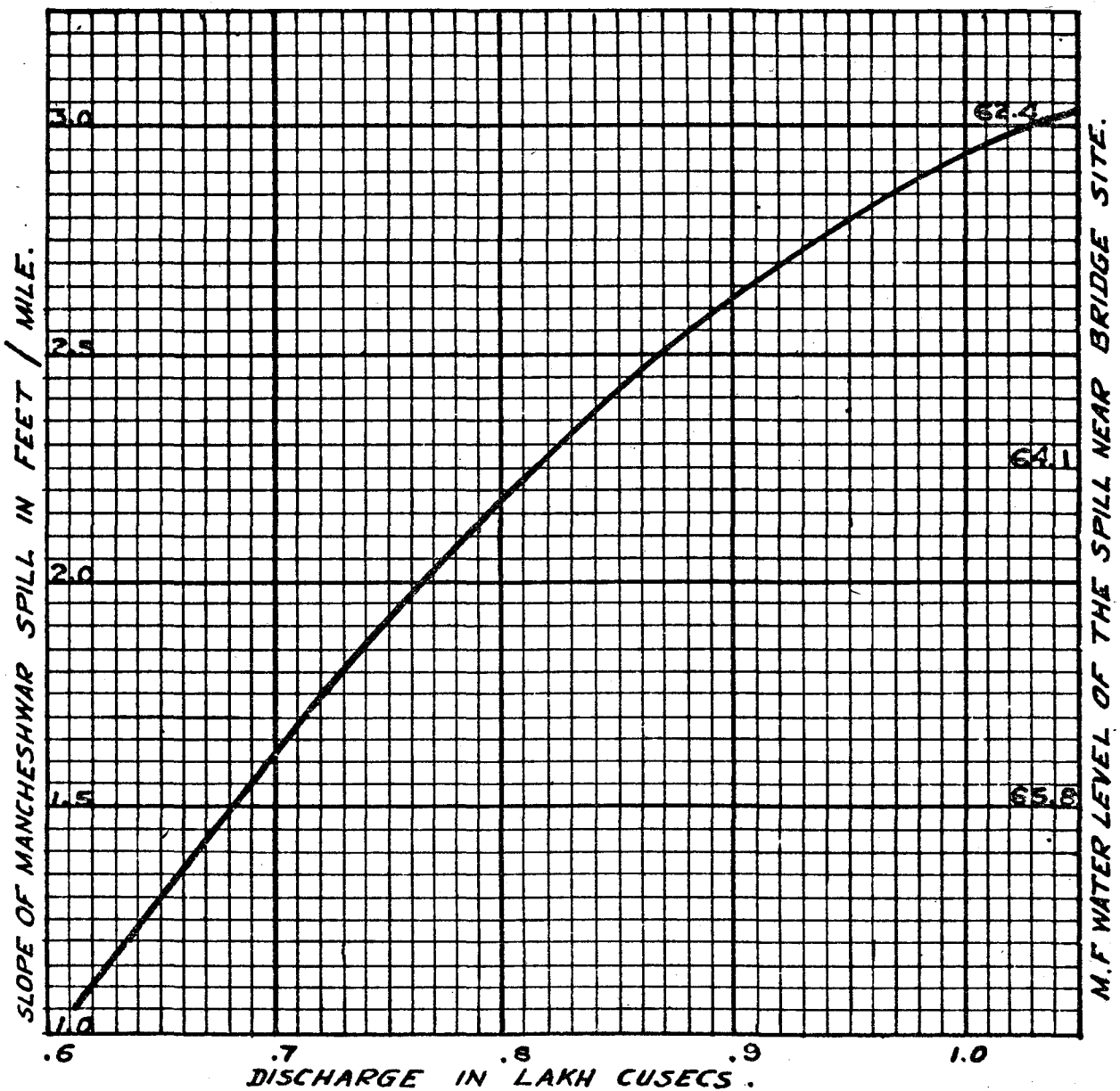


FIG:- 38

1/400 ; 1/66 MODEL OF THE RIVER
KUAKHAI SHOWING THE RELATION SHIP
BETWEEN THE DISCHARGE OF MANCHESHWAR
SPILL AND SLOPE OF THE SPILL OR M.F.L
OF THE SPILL NEAR THE BRIDGE SITE.



BY: MANCHUNDIA. 2.3.49.

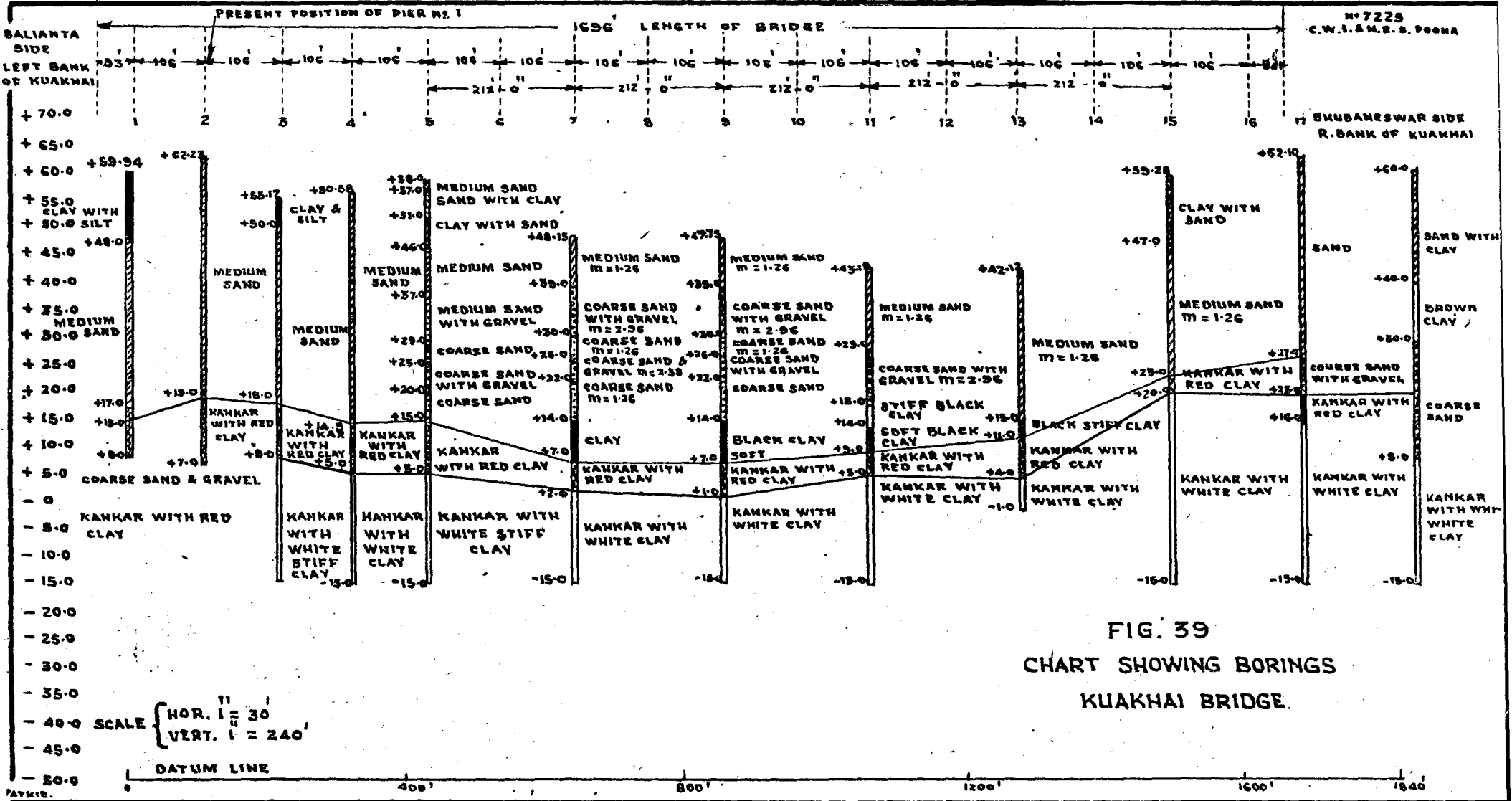


FIG. 39
CHART SHOWING BORINGS
KUAKHAI BRIDGE.

The maximum scour round bridge piers according to this Station analysis⁽¹⁾ is $2D_{\text{Lacey}}$ i.e. 58.9 or say 59. The M.F.L. at the bridge site being R.L. 65, the bottom of scour-hole would be at R.L. 6.

Considering however, the data of borings shown in fig. 39, the river bed consists of medium sand on the top and coarser and harder layers of gravel and sand, kankar and clay etc. Underneath, the actual scour would be appreciably less than when the entire bed is of sand i.e. scour would be less than $2D_{\text{Lacey}}$.

Model observations showed that the maximum depth of scour obtained was equivalent to 36 ft. which equals to $1.22 D_{\text{Lacey}}$. The lowest R.L. was 29.00 as against pier foundation level near right bank at R.L. 15.0. There is thus sufficient grip and there is no necessity of apron protection round piers.

Waterway of the Bridge.

The existing design of the Bridge provides a waterway of 1596, giving C in $W = C \sqrt{Q} = 3.08$ with a discharge of 2.68 lakhs cusecs. It was considered that as construction on the left bank has already started one span on the right could have been reduced giving a waterway of 1490 ft. which gives $C = 2.88$. As however the bridge design is of balanced suspension and cantilever type reduction of one span was not possible and hence it was agreed that the waterway may be kept at 1596 ft..

Conclusions.

- (1) The selected bridge site is suitable
- (2) No training measures are necessary.
- (3) No apron protection for piers or abutments appears necessary.
- (4) The waterway of the bridge may be 1596 ft.
- (5) The maximum discharge through the Mancheswar Gap may be taken as 68,000 cusecs, the maximum discharge through the Kuakhai being 2.68 lakhs cusecs.

(1) Annual Report (Technical) 1944, page 74.

13. RAIL-CUM-ROAD BRIDGE ACROSS THE YAMUNA BELOW THE PROPOSED BARRAGE AT PURANA KILLA

Fig. 40

In connection with the proposed circular railway round greater Delhi, it was proposed to construct a railway-cum-Road bridge across the Yamuna below the proposed barrage at Purana Killa—fig.40—. It was considered advantageous to combine the barrage and the bridge, provided the barrage sited in the lower reaches would serve its purpose. A bridge in the higher reaches was not considered convenient.

The Director, Civil Engineering, Ministry of Railways (Railway Board) sought the advice of this Station in regard to the suitability of the bridge site.

None of the proposed Railway-cum-Road bridges were considered suitable because both were on curved reaches of the river having a wide meander belt. A bridge or a gorge or other fixed exit has a tendency to double the meander belt above it. Hence construction of a bridge in such reaches was bound to affect the regime of the river both upstream and downstream.

It was also considered that in the event of deterioration of curvature, the Agra canal may also be prejudicially affected.

With regard to the Barrage, model experiments⁽¹⁾ had shown that a Barrage at Purana Killa could, but not without some difficulty, be made to perpetuate favourable conditions at the pumping station inlets at Delhi Gate by means of a curved-pitched embankment. Moreover, the effect of a Barrage at this site on the river below and above it can be minimised by suitable measures as shown by the model tests. Whether a Barrage at a lower site can be made similarly harmless to the effects of the pitched embankment at Delhi Gate and/or the river above and below it, could not be definitely stated without specific model experiments. The Barrage-cum-Bridge at the proposed alternative site, further downstream, would have other disadvantages too, as it would make sand exclusion from off-taking canal, above the new barrage — a most difficult problem and was likely to cause heavy accretion between this and Okhla Weir with very probable detrimental results on the Agra Canal.

It was therefore suggested to explore the possibilities of a bridge crossing at

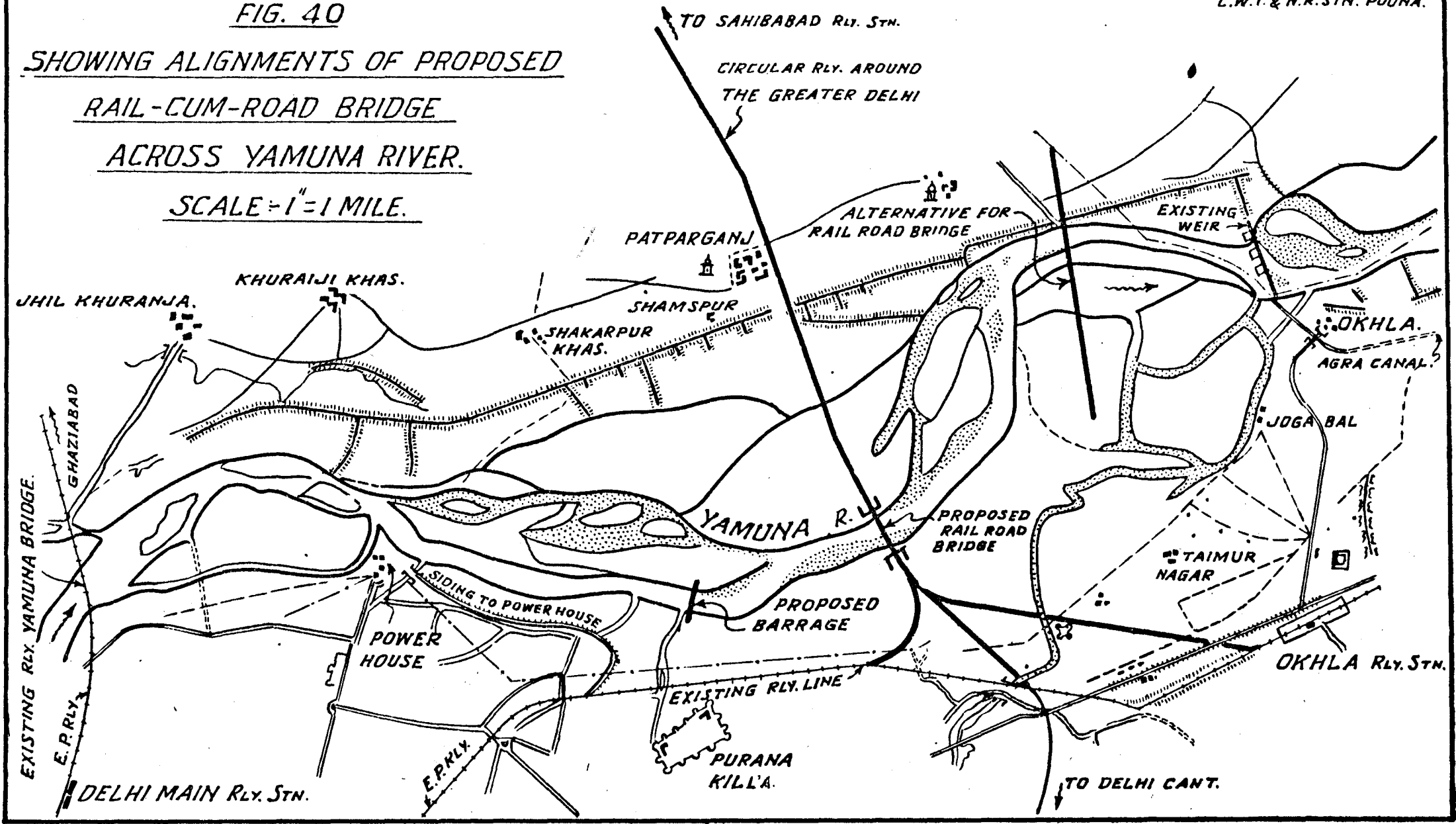
- (a) the Okhla Weir
- or (b) the proposed Barrage at Purana Killa.

(1) Annual Report (Tech.) of work done during the year 1946, page 60.

FIG. 40

SHOWING ALIGNMENTS OF PROPOSED
RAIL-CUM-ROAD BRIDGE
ACROSS YAMUNA RIVER.

SCALE = 1" = 1 MILE.



14. Preliminary note on the Purna River (Bombay).

Introduction.

1.1 During recent years, chiefly after sustained floods of 1944, severe erosion is taking place near the village Desai Falia situated along the right bank of the Purna river, Surat District (Bombay Province). Now the river has come so close to the village that some of the houses standing on the bank are already damaged and quite a few are threatened such that permanent protective measures appear necessary.

Fig. 41.

Proposal of the local Officers.

1.2 The local officers had proposed the following measures to ameliorate the conditions at Desai Falia:—

(a) short groynes 40 ft. to 90 ft. long mainly along the right bank in the vicinity of Desai Falia.

(b) removal of the sand bar on the inside convex curve of the river to reduce the attack at Desai Falia.

(c) few short groynes on the left bank near the village Pelad Buhari.

The cost of these proposals was roughly Rs. 50,000.

These proposals were forwarded by [the Secretary, Bombay P.W.D. for further scrutiny and technical advice of the Director of the Station.

Remarks by the Director.

1.3. It was pointed out that short groynes 40 ft. to 90 ft. long may arrest the erosion temporarily; but they could not be depended upon. The experience of the Station has been that short groynes when placed on the outside of the bend, at the point of attack, tend to peg down, if not actually pull, the high velocity flow on that bank; in other words they behave much the same as "attracting groynes". The river being relatively narrow and deep for its discharge (B/D ratio = 20 and maximum estimated discharge = 1.13 lakh cusecs), there would be a deficiency of sand and silt in suspension, which would have otherwise lodged upstream and downstream of these groynes; the tendency, therefore, would be to accentuate scour at the noses of groynes due to relatively sand free water, and consequently it would be very difficult to maintain these groynes (which would be 40 ft. deep).

As for (b) para 1.2 merely removing sand from the convex side would not ease the situation; as the bar would reform due to natural tendency of the river. This tendency would be accentuated particularly since the right bank groynes would attract flow to their scoured noses and the money spent over this proposal would be wasted.

A long repelling groyne at the end of the concave curve on the right bank would have been an ideal solution had it not been for the existence of another village Pelad Buhari on the opposite bank. Under these circumstances, it was difficult to suggest the best alignment, length and position of the groyne; it was therefore suggested that model experiments should be carried out.

(2) *Hydraulic data.*

(i) Representative samples of bed sand collected from the bed near the village when analysed gave mean grade varying from 2.15 to 5.77 mm in diameter.

(ii) The river surface slope is 0.90 ft. per 1,000 in the reach under investigation.

(iii) *Maximum flood discharge*: As there are no records of velocity and discharge the maximum flood discharge is computed by the following methods.

(a) From fig. 41 it will be seen that the meander length is 10,150 ft. which gives from the station formula

$$M_L = 30\sqrt{Q_{\max}}$$

$$Q_{\max} = 1,12,800 \text{ cusecs.}$$

whereas the dominant discharge is given by the formula

$$M_L = 40\sqrt{Q_{\text{dom}}} \quad \therefore Q_{\text{dom}} = 63,000 \text{ cusecs.}$$

These figures are, however, based on only one meander length available and, therefore needs corroboration.

(b) Q_{\max} from Lacey's general flow formula

$$V = 16 R^{2/3} S^{1/3}$$

Fig. 42 $R = 22.10$ ft. and $S = 0.90$ ft. in 1,000 near C.S. 42 shown in fig. 42. where the river has a straight reach.

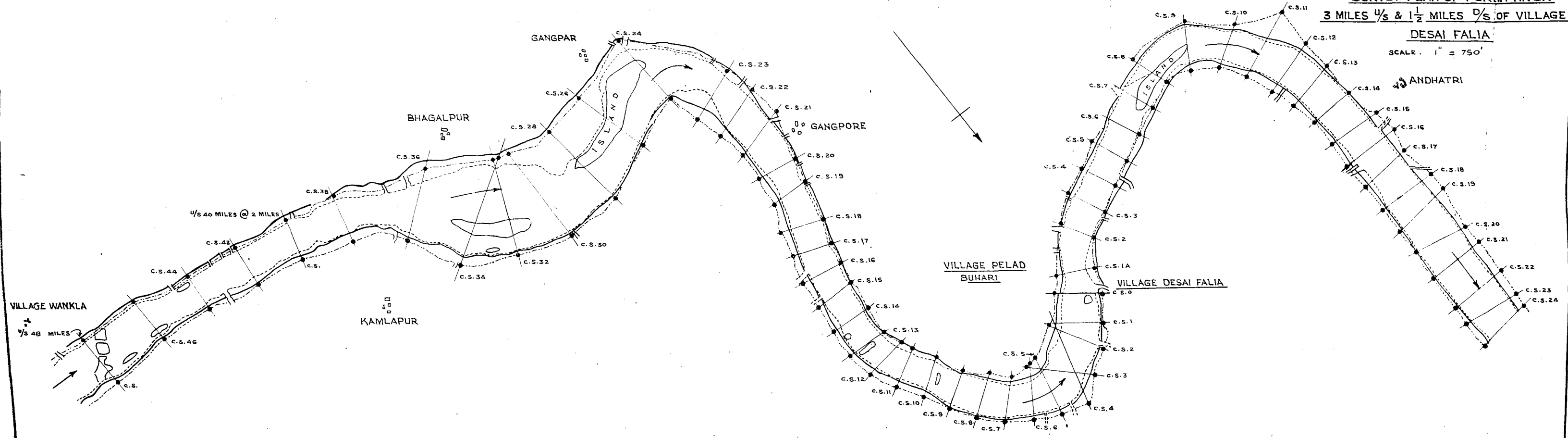
$$V = 11.50 \text{ ft. per second}$$

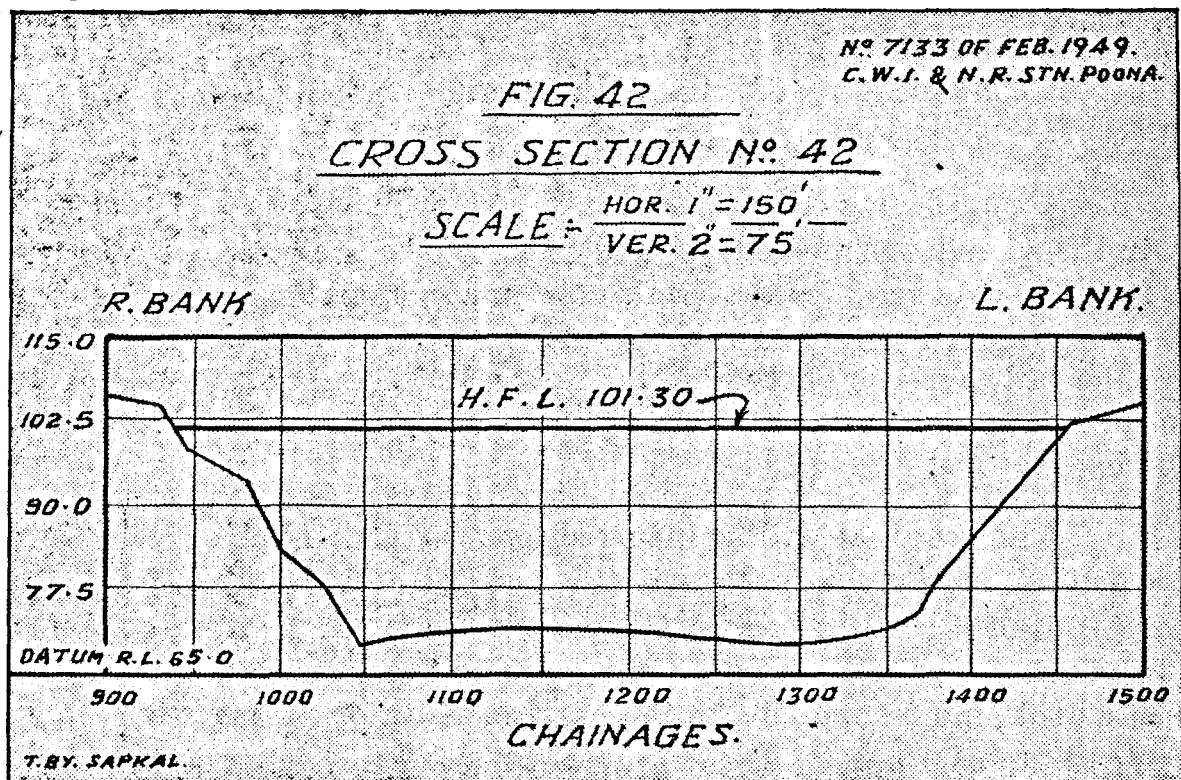
$$\therefore Q = A \times V = 1,38,000 \text{ cusecs.}$$

We may take the mean of the two figures or $Q_{\max} = \text{say } 1,25,400 \text{ cs.}$

FIG. 41
RIVER TRAINING WORKS
SURVEY PLAN OF PURNA RIVER
3 MILES $\frac{4}{5}$ & $1\frac{1}{2}$ MILES $\frac{0}{5}$ OF VILLAGE
DESAI FALIA

SCALE. 1" = 750'





The model is now under construction.

15. Prevention of erosion in the Narbada at Borbhata downstream of railway bridge near Broach

1. Introduction.

Director's advice was sought regarding protective measures against bank erosion near the village Borbhata on the left bank opposite Broach and on the sides of the channel now widened due to floods in the Narbada river. The other problem was prevention of flooding of the lands on the left bank as shown in fig. 43. This flooding apparently is not due to the river spilling over the whole of the left bank but is caused by the on-rush of river water into the channel in floods which then overflows the channel banks and inundates the low lying area.

Fig 43

2. Director's opinion:

Regarding flooding of the low lying lands this can be prevented only by throwing a strong bund across the mouth of the channel if only the channel banks are low. If however the river also spills over its left bank, flood embankment along the river bank, wherever low, will have to be constructed to constrain the river to the Khadir proper; this will however lead to prohibitive expenditure not justifiable by the protection and relief it would afford and hence was not recommended. Multipurpose project for the Narbada is under active consideration for some time past and when it

will materialise in the near future it is expected that the moderated river discharge at Broach will be $\frac{1}{2}$ to $\frac{1}{3}$ of its present magnitude; the floods will then be contained in the river Khadir and no spills over banks will then occur.

3. About damage due to erosion by river floods, though only the area in the vicinity of the village was desired to be protected, it was apparent from the data supplied that bank erosion is not confined to the neighbourhood of the village but is eating into the left bank continuously from above the railway bridge down to far below Borbhata. Protective works even for local relief in this case are not advisable, though feasible, for the following reasons:

(i) The cost that will be incurred to construct any effective training work will not be commensurate with the benefits provided by it;

(ii) Borbhata being opposite the city of Broach anything done on the left bank for protection of the village will have its repercussions on the opposite bank, more so, owing to the narrow width of the river here; and

(iii) in post-project period bank erosion is expected to be appreciably reduced due to reduction in discharge as mentioned in para 2 above.

Thus to ameliorate the present conditions no remedial measures could be suggested because besides being costly, they would have affected flow conditions on the Broach bank.

16. Protection of Railway bridge Piers on the Godavari at Rajahmundry

The Railway Bridge on the Godavari river at Rajahmundry, 65 miles from the sea and five miles above Dowlaiswaram anicut, crosses the river width of 9096 feet by 56 Spans of 160 ft. each and one of 40 feet. All the piers are of masonry founded on wells. River bed consists generally of coarse sand overlying a bed of stiff clay except on the left bank. Near the left bank in spans 1 and 2, sand stone and conglomerate occur and in the deep channel in spans 3 to 9, only sand is met with at least 117 feet below H.F.L. 62.80 i. e. to R.L.—54.50 The deep channel in the left end spans, remarkable for its persistency from the time even before construction on the bridge does not continue far upstream or downstream, and has been caused by abnormal scour due to water impinging against the sandstone exposed on the left flank. The natural curvature of the river also helps to maintain the deep channel along this bank. Local concentration of discharge in the rest of the spans, as in the left side spans, cannot be accounted for due to lack of data. The only surveys available are of 1924 and 1938. Yearly soundings round the bridge piers, from 1942 to 1948, reveal the capricious nature of the river behaviour, evident from the depth of bed scour varying from a minimum of 15 ft. to a maximum of about

Fig. 44

FIG. 43

INDEX PLAN SHOWING EROSION OF
THE NARBADA RIVER AT BROACH
AND THE LOW LEVEL LANDS ON
LEFT BANK SUSCEPTIBLE TO FLOODING

SCALE 1"=1 MILE

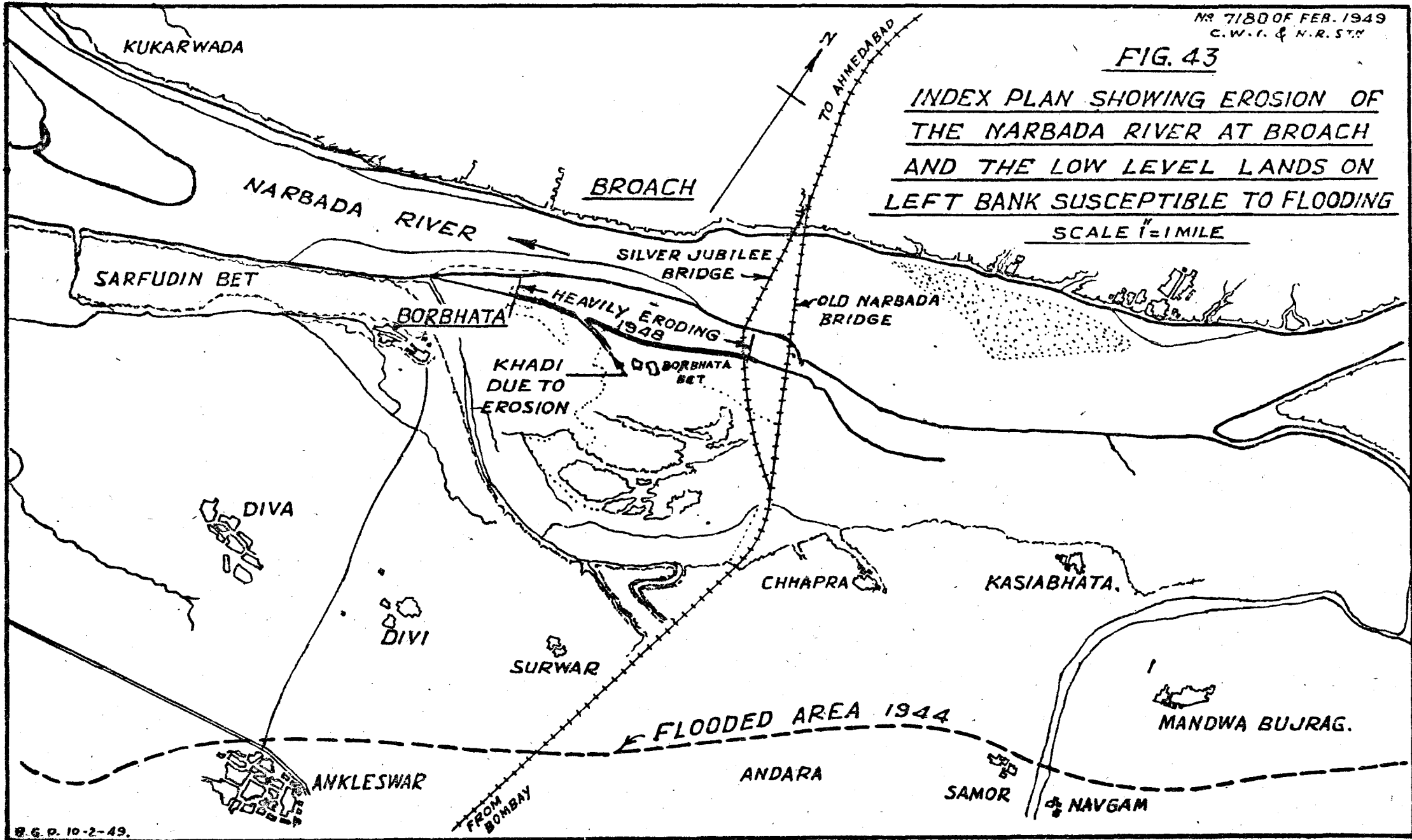
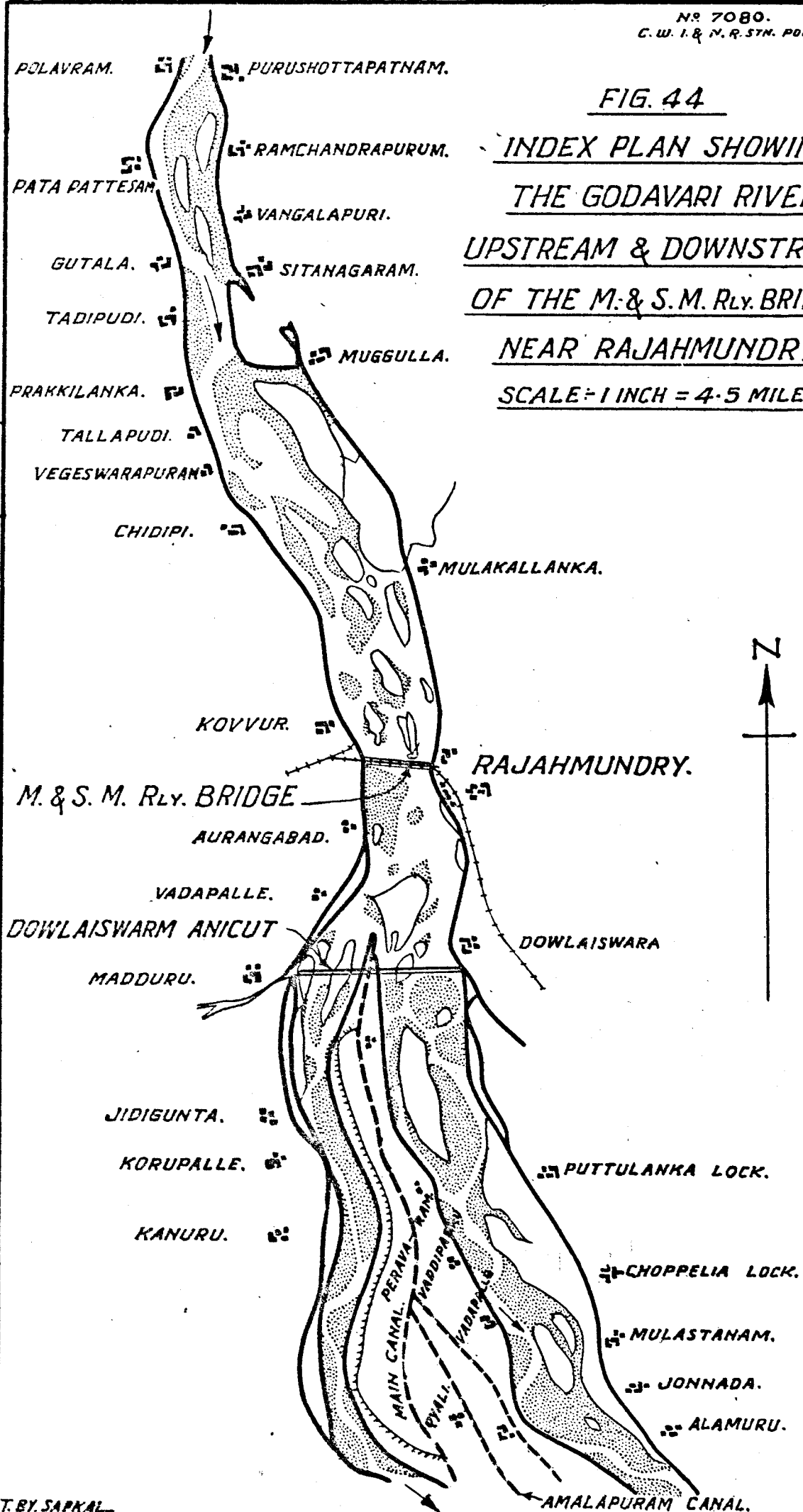


FIG. 44

INDEX PLAN SHOWING
THE GODAVARI RIVER
UPSTREAM & DOWNSTREAM
OF THE M. & S. M. RLY. BRIDGE
NEAR RAJAHMUNDRY.
SCALE: 1 INCH = 4.5 MILES.



T. BY. SARKAL

45 feet, almost throughout the river width. An ample margin of safety is therefore warranted in respect of anchorage to be provided for the piers.

In Technical Paper No. 86, on the construction of the Godavari Bridge written in 1908 by Mr. Walton, Engineer-in-chief of Godavari Bridge Works has reported scour at Pier Nos. 4 and 5 of 8ft. 3" and 77 ft. 11" below high flood level (or say R.L. - 18 and R. L.—15 respectively). In a long sustained flood the scour may even be deeper than this. To reduce the scour round the piers in the deep channel and to induce the river to deepen, elsewhere, stone pitching at the piers in the deep channel portion was provided. Due to heavy scour inspite of the stone protection to about R.L. 3.00 caused in 1943, standing instructions were given for taking soundings in January in each year round all the piers by the Railway Chief Engineer to ascertain the depth of scour and for arresting scour at the specified warning levels by dumping in stones or concrete blocks.

Fig. 45. Being provided yearly with heavy pitching, the piers in the deep channel have not given cause for much concern and anxiety. The river, however, deepened elsewhere in spans 15 to 20 in 1948 to below the specified warning levels. The case was therefore referred to the Poona Station for advice for the action to be taken against this exceptional scour.

2. Data

For thorough scrutiny of the case, the data, *inter alia*, of survey plans discharge hydrographs, sand samples etc., were asked for. Of these, the only plan available was that for the year 1924, besides the topo sheet prepared from survey of 1938. No discharge data at the bridge could be obtained and hence Station opinion has necessarily to depend on the discharges observed in 1924 to 1946 at the Dowlaiswaram anicut, 5 miles downstream of the bridge, already available with the Station. The sand samples from the river bed at pier Nos. 4, 10, 18, 36 and 45 were collected and furnished for analysis as requested.

3. Analysis of data and findings thereof.

Depth of scour in a river depends on various diverse factors, important among these being the river configuration, maximum intensity of discharge, sand charge and the nature of the bed material.

In para 2 the scour in the deep channel near the left bank is already explained as due to the favourable, though not prominent, river curvature and occurrence of sandstone formations. The ratio of B/D (river width/river depth) at the bridge site is 353 against Lacey B/D ratio of 65 for maximum discharge of 21.08 lacs cusecs. The river thus is relatively much wider and shallower as compared with Lacey section. Unless the

bridge is sufficiently shortened to bring it nearer to the Lacey width—allowing for wide or narrow nature of the river—by use of suitable training works, it is common experience that unrestricted scour and silting occur in any of the spans due to changing formations of sand islands above the bridge changing the entry conditions.

The deep scour in the left end spans is attributable also to the weir action caused by the pitching laid in between the piers, (i.e. across the deep channel). The whole of the river bed in these spans is thus built up which must be causing deep scour below the bridge where the pitching blocks must be slipping down and thus being carried away and lost.

Survey plans and other data covered too short a period to indicate any other reasons leading to local abnormal scour and its probable extent in the rest of the river width excepting in the left side deep channel, as has been observed between piers 16 to 22 in January 1948. Actual scour depths in flood must have been appreciably more than these as substantial reduction in scour depths due to subsequent silting and sanding till January must have occurred. To obtain maximum depths of scour, soundings should have been taken during falling flood stages.

Maximum river discharge at the bridge site was estimated at 1,500,000.⁽¹⁾ The available discharge hydrographs however showed the recorded maximum discharge of 2,107,660 cusecs in 1942. In between the bridge and the anicut, no appreciable addition to the river discharge due to contribution by tributaries or deduction due to withdrawals by canals etc.² was apparent from the plans supplied and hence the peak river discharge at the anicut were presumed to be practically the same as those obtaining at the bridge. Thus though the maximum discharge figure was available distribution of discharge across the river giving discharge concentration was not known.

Bed sand samples along a line 200 feet upstream of the bridge and parallel to it and opposite pier Nos. 4, 10, 18, 36 and 45 were analysed. The mean weighted diameter of sand particles in the various samples varied within a wide range—0.37 and 1.58 m.m.

Data of scour round bridge piers of existing bridges have been analysed at the Poona Station⁽²⁾ which showed that the depth of scour was sufficiently closely related with Lacey depth defined by $47 [Q/f]^{1/3}$, Q being discharge and f the silt factor = $1.76 \sqrt{\text{mean sand diameter}}$. According to the present practice in Bridge Engineering⁽³⁾ of providing a grip to

(1) Technical paper 86 by Mr. Walton.

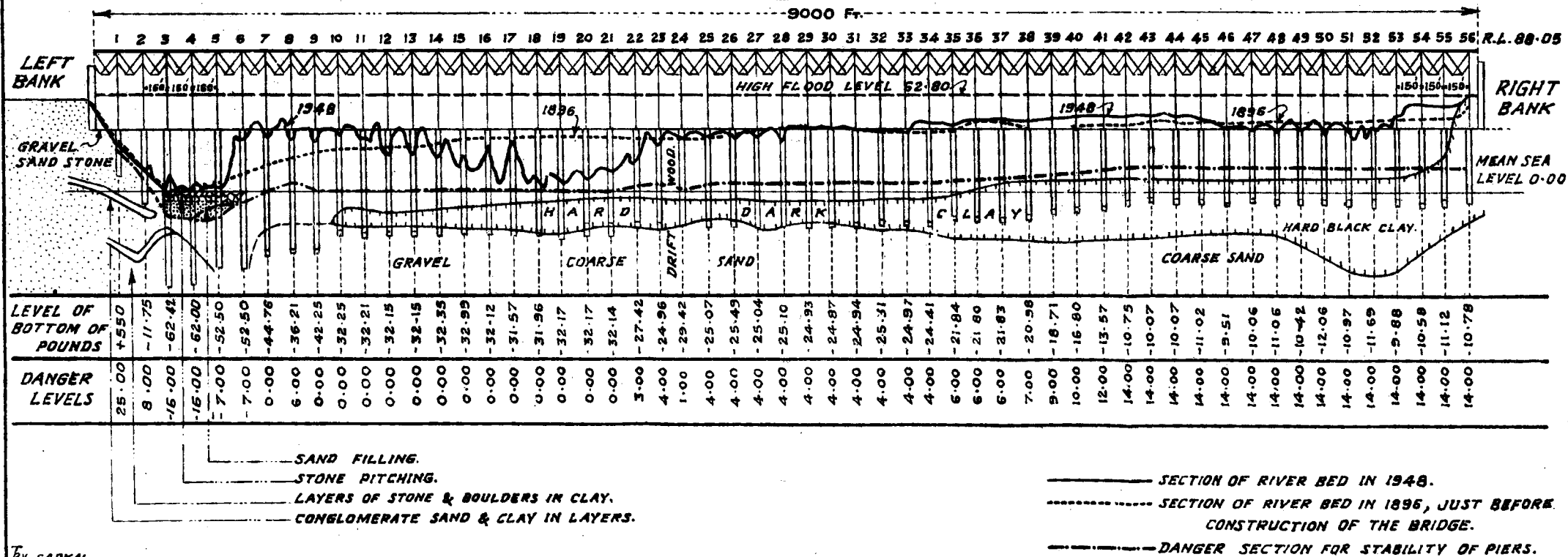
(2) Page 78 of A. R. (T) 1944 of the I. W. E. Station, Poona.

(3) Railway Board Technical Bulletin No. 78 "Well foundations for Bridges".

FIG. - 45
GODAVARI BRIDGE
ON
M. AND S. M. RAILWAY. SOUNDINGS
FOR
1896 AND 1948

SCALE - $\frac{HOR. 1" = 800'}{VER. 1" = 80'}$

1937-1938
GENERAL ELEVATION.



By SAPNAL.

the piers equal to half the anticipated maximum scour depth below H. F. L. assuming the scour depth = $2D_{Lacey}$, the lengths of piers in sand of sizes occurring at the site necessary to ensure adequate grip works out as below.

Table I

Pier No.	Sand Dia. in m.m.	D_{Lacey} ft.	Length of pier = $3D_{Lacey}$ ft.	H.F.L. (62.80) — $3D_L$ i.e. R.L. of bottom of foundation ft.	Actual foundation Levels. ft.
4	0.37	59.3	177.9	- 115.1	- 62.00
10	1.15	49.1	147.3	- 84.5	- 32.25
18	0.75	52.7	158.1	- 95.3	- 31.96
36	1.44	47.3	141.9	- 79.1	- 21.80
45	1.58	46.5	139.5	- 76.7	- 11.02

The above safe foundation levels according to $3D_{Lacey}$ criterion are based on the assumption that the sub-surface strata are wholly of incohesive sand. Actually piers No. 1 and 2 have been sunk through and founded on sandstone, piers 10 to 33 are sunk through clay and founded on coarse sand and piers No. 34 to 56 are sunk through sand and founded on hard clay. Only piers No. 3 to 9 are sunk wholly through sand. Thus grip length = D_{Lacey} criterion can be said to be applicable strictly to piers 3 to 9 only. But even in the case of these piers since the estimate of scour depths is based on surface sand only it cannot be expected to be reasonably correct. For fairly accurate assessment of depth of scour, sand samples at all depths down to anticipated scour, not only at the surface, ought to be examined. But these were difficult to obtain in the present case, nor were they of any practical use since pier foundations cannot now be deepened to provide the necessary grip of D_{Lacey} in the light of the analysis of sub-surface sand samples. Hence in the existing circumstances to ensure safety the scour at the piers should be held up at $2/3$ pier depth by providing pitching.

Piers No. 1 and 2 being sunk through and founded wholly in sand stone, it was not necessary to provide them with any protection.

Piers No. 3 to 9 are wholly sunk through sand and hence the danger levels should have been in accordance with $1/3$ pier depth criterion. The danger levels fixed by the Chief Engineer, M. & S.M. Railway, have been obtained from Spring's curve which arbitrarily provides grip length. These however, in the present case were approximately the same as these according to $1/3$ depth rule and hence were not modified. The warning levels

at which pitching top should be maintained are 16 ft. higher than danger levels for piers No. 3 and 4 and 10 ft. for 5 and 6. This precaution of checking scour at warning levels, 10 to 16 feet higher than danger levels appears essential as the likelihood of the pitching stone or blocks maintained at danger levels getting washed away in floods, thus endangering the pier, cannot be ruled out. Actually observations since 1945 showed that concrete blocks weighing as much as 1/2 ton each used to be carried away by river in spans 1 to 9 and hence were required to be renewed in successive years.

Model experiments at the Poona Station in connection with the protection round bridge piers have shown that,

“Stones should at the lowest practicable level to avoid the formation of high mounds of stone which tend to obstruct the Waterway of the bridge.

The greater the depth at which the stone is laid the more stable it is, i. e. the more severe attack, it can withstand without being carried away.

It is necessary to place stone at such a level, and to use stone of such size and weight that it will not be disturbed with the worst attack for which allowance has to be made. Stone laid at higher levels is of little value.”

Since the top level of block pitching could not be lowered in the present case for reasons explained above the alternative remedy was to try heavier blocks.

For the rest of the piers — i.e. piers No. 10 to 56 — the danger levels were worked out as 1/3 pier depths and an allowance of 10 feet was made between danger and warning levels which as already said was necessary to safe-guard against sudden and local scour.

As long as scour is arrested at warning levels there would generally be no cause for anxiety.

Conclusions and Recommendations.

(1) At all the piers, the existing practice of maintaining stone protection at warning levels below which scour should not be allowed, will have to be continued for the safety of the piers.

(ii) In the spans where the pitching blocks get washed away yearly, concrete blocks heavier than at present, should be tried. At the same time practice of pitching the river bed in between the piers as in the left end spans, should be discontinued and protection should be restricted only to the piers.

FIG. 46
BALL BEACON TO BUND MARK
MOYAPUR
SURVEYED 23rd TO 27th OCTOBER 1947.
SHOWING THE PROPOSED ALIGNMENT OF THE
WALL & THE EROSION AT THE BIRLA JUTE MILL
 SCALE: 1" = 1 STATUTE MILE

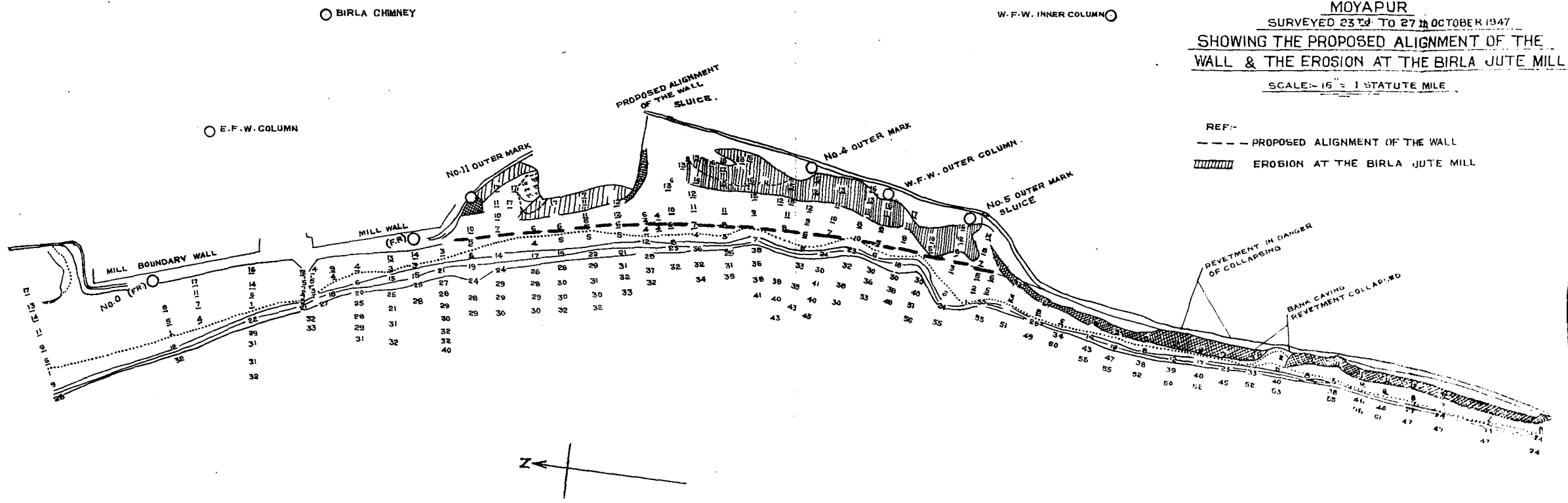
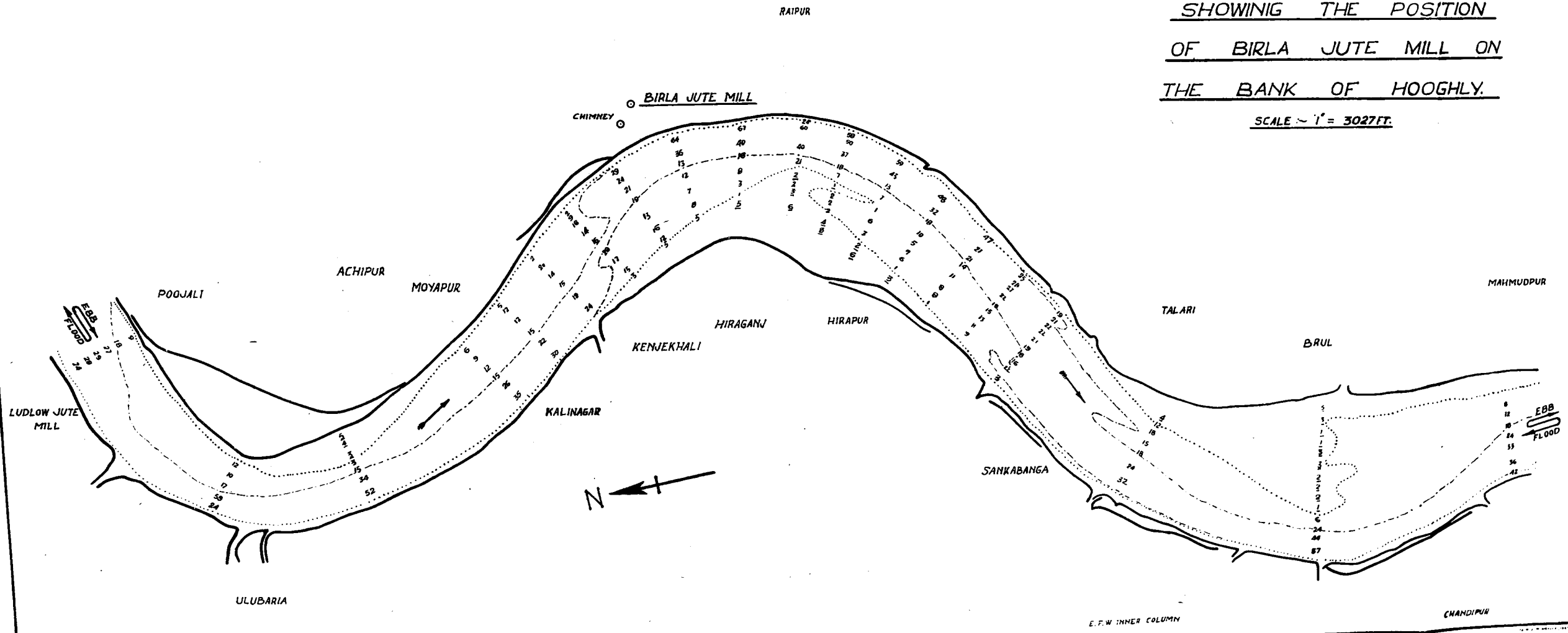


FIG. 47
SHOWING THE POSITION
OF BIRLA JUTE MILL ON
THE BANK OF HOUGHLY.

SCALE - 1" = 3027 FT.



E.F.W. INNER COLUMN

(iii) As long as scour is arrested at warning levels no apprehension need be felt about safety of the piers against scour.

(iv) To provide timely protection and also to obtain deepest scour observations, soundings should be made say at 2/3 falling flood-stage water level instead of in January.

Erosion at Birla Jute Mill on the Hooghly River

Introduction.

A severe bank erosion was reported by the Port Commissioners at the Birla's Jute Mills situated on the bank of the Hooghly as shown in fig. 46 and 47. The Director was requested to suggest means to prevent this erosion. After analysing the curvature of the river at this place it was found that the erosion was not due to sharp curvature for the discharge, but it was due to the fact that the Mill is situated on the concave side (the more so being at the apex of the meander) of the curvature, as shown in fig. 47.

Figs. 46 & 47

Findings & recommendations.

It is stated that the inflow hits the bank on which the mill is situated and the maximum inflow discharge is estimated to be 8,00,000 cusecs. The width of the river near the Mill is 3,700 ft.

Using Lacey's formula for finding the depth of scour - though not strictly applicable to tidal flow -

$$D = 0.9 \times \left\{ \frac{q^2}{f} \right\}^{1/3}$$

where, D = the maximum depth of scour in feet
 q = the intensity of discharge per foot and
 f = silt factor = 1.76 √m

m = 0.14 m.m.
 f = 0.66

The depth of scour as shown in table below was obtained.

Inflow (cusecs)	Discharge intensity in cusecs per foot allowing for 50% concentration.	D _{Lacey} (feet)	Maximum depth of scour expected = 2D _{Lacey} (feet)
1	2	3	4
8,00,000	324	48.5	97.0
			or 77 ft. below K. O. D. S. (Kidderpore old Dock Sill).

The above figure of 77 ft. in Col. 4 may be compared with 41 ft., the present maximum depth of scour below K.O.D.S.

Fig. 46

It was recommended that a sea wall be constructed as shown in fig. 46 with the foundation taken as deep as practicable — preferably more than 70 ft. below K.O.D.S. with the longest piles obtainable.

Tidal model of the Hooghly Estuary

1. Introduction:—

The three rivers Bhagirathi, Bhairab-Jalangi and the Matabhanga Churni, which take off from the Ganga river with their net work of creeks and khals are known as the Nadia Rivers. These are the principal fresh water feeders of the Hooghly. Bhagirathi and the Bhairab Jalangi form the Hooghly below their confluence at Nadia — fig. 48 — about 92 miles upstream of Calcutta. The Matabhanga -Churni meets the Hooghly at Chakdaha about 50 miles north of Calcutta. The Hooghly continues its meandering course through alluvial soil past Calcutt and meets the Bay of Bengal about 85 miles downstream of Calcutta. The Damodar and the Rupnarain meet the Hooghly at Fulta point and Hooghly point respectively downstream of Calcutta besides the Rasulpur and Haldia rivers which meet it further downstream. Of these the Rupnarain and Damodar form the main tidal compartments of the Hooghly.

Fig. 48

The tidal range of the Hooghly at Saugar is of the order of 17 ft. and the tidal effect reaches up to Swarupganj about 84 miles north of Calcutta.

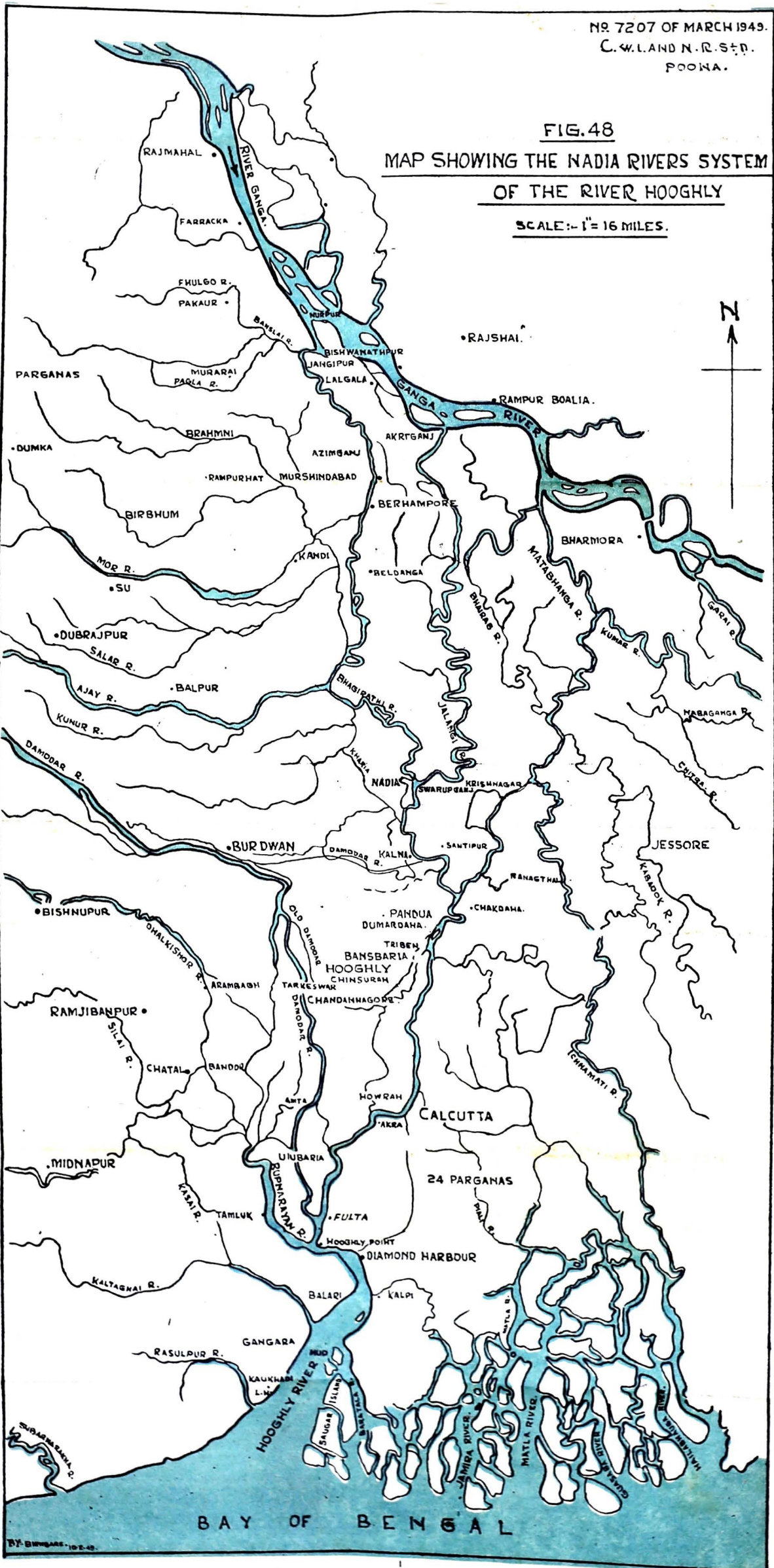
From November to February, the flood tide is stronger than ebb and the preponderance of the former gains much more over the latter during March to June, being maximum at equinoxes in March. During the period, from July to October, the ebb gradually gets preponderance and during the Monsoons, the ebb current preponderates so that at neap tides, sometimes depending on the freshet discharge; at Calcutta the tidal flood current disappears altogether and its only effect is manifest in backing up the freshet waters.

During the eight month of preponderance of the flood tide, the river bed conditions in the estuary adjust themselves to conform with the direction and curvature of the flood currents. During the four months however, when the ebb preponderates, the channels formed by the ebb are not the same as those created by the flood in many places with the result that Cul-de-sac channels and bars are thrown up and dredging has to be resorted to, to maintain navigable channel across the bars. Fig. 49 shows the various bars in the river. Besides the Panchpara, Pir Sarang, Poojali, the Mayapur and Roypur crossings, the

Fig. 49

FIG. 48
MAP SHOWING THE NADIA RIVERS SYSTEM
OF THE RIVER HOOGHLY

SCALE:- 1" = 16 MILES.



BAY OF BENGAL

Fulta point is peculiar in that due to the confluence of the Damodar there is a concave curvature (at Fulta) followed by another concave curvature at Ninan; which in its turn causes the serious trouble at the Western Gut and the Eastern Gut channels at Hooghly point,

There is a belief that the condition of the Hooghly estuary is deteriorating especially in the reach above Diamond Harbour, and to facilitate navigation it has been proposed to construct a ship canal linking Diamond Harbour to Calcutta costing 15 crores of rupees.

Before considering seriously such a costly project, the Port Authorities consulted Sir Claude Inglis as to whether it was possible to improve the navigability of the Hooghly by training works. In his "Report on the Prospects of improving the navigability of the Hooghly", Sir Claude recommended the construction of the following model experiments to evolve the best training measures for the purpose.

Model I—Extends from 25 miles downstream of Saugar island to Bansbaria (scales of $1/2,000$ horizontal and $1/160$ vertical). This model would be expected to give valuable qualitative information regarding the conditions of flow, the movement of material and consequently the changes in the river and the estuary. With the scales adopted, the depths, widths and velocities in the River in the higher reaches may, however, be too small to give reliable and definite information. This model would, therefore, be considered as a pilot model, so far as the reach is concerned.

Model II—This is to extend from Balari to Konnagar to horizontal scale of $1/400$ and vertical scale of $1/60$. This will give information about the conditions in the upper reaches and because of larger scales as compared to Model I is expected to give results of greater significance than the first one.

2. Though the Port Commissioners intend to construct their own model at Calcutta at a later stage, they decided that this Research Station should tackle the problem. The points to be investigated as stated by the Port Authorities are as under.—

"(i) The object of the investigation is to determine by means of Models the possibility of improving the River Hooghly to an extent which will enable ships of 26 feet draft to navigate the River on any day of the year.

(ii) The depths in the River are such that under the prevailing conditions,

- (a) Below Diamond Harbour drafts are limited by the Balari Bar with a depth of about 14 ft. at L. W. O. S.
- (b) Above Diamond Harbour drafts are limited by the seasonal deterioration of various bars and in particular

the Eastern Gut and Ninan with depths which may fall to 6 ft.

(iii) The lack of depth of water above Diamond Harbour can be met by the construction of a canal.

(iv) If the River below Diamond Harbour can be improved to the desired depth and the River above Diamond Harbour cannot be proportionately increased, then the canal becomes a necessity if the required drafts are to be accommodated.

(v) If the River above Diamond Harbour can be improved to the same extent as the lower reaches, no canal is necessary.

(vi) If the area between Fulta Point and Diamond Harbour can be improved to the desired extent, the remaining bars can from experience be maintained at the required depth by dredging.

(vii) The depths required on the various bars to meet the required conditions are as follows:—

(a) Panchpara Sankral Munikhali Pir Serang Poojali	} 22 ft. 6 ins. necessary. 24 ft. 0 ins. desirable.
(b) Mayapur	20 ft. desirable. 18 ft. adequate.
(c) Roypur	18 ft. 6 ins. desirable. 16 ft. 6 ins. adequate.
(d) Ninan and Eastern Gut	16 ft.
(e) Balari Auckland	17 ft.
Entrance Bar	17 ft. plus allowance for swell + 18" to 5'.

(viii) In addition to the main question of depth required for navigation, the following points in geographical order are for investigation:—

- (a) The erosion of the right bank above Howrah Bridge.
- (b) The improvement of the inland vessels wharves above Howrah Bridge on the left bank.
- (c) The alignment of both banks between Howrah Bridge and Garden reach including
 - (i) the formation of bars in the River off Outram Ghat, Fort Point, Hastings and Mutiabruz.
 - and (ii) the silting of the entrance of K. G. Dock with particular reference to the movement of the Mutiabruz sand.
- (d) The shoaling of Panchpara Crossing.
- (e) The shoaling of Saukral and Munikhali during the predominant flood tide season and its effect on Pir Serang and Poojali when clearing out during the freshet season.

- (f) The effect of the gradual closing of the mouth of the Damodar River and the desirability of artificial closing of the mouth.
- (g) The stabilisation and improvement of the Balari Bar.
- (h) The stabilisation of the entrance channel with a least depth of 17 ft. at L. W. O. S.
- (i) For all bars, an investigation into the present methods of dredging and the position of the dumping grounds. This should include the dumping of spoil in the Port proper.

(ix) The maintenance of the Head Water supply is considered vital to the existence of the Port. At present very little information is available and no control can be exercised by the Commissioners. Existing conditions must therefore be accepted. (It is understood that the Ganges has already been robbed of 40% of its dry weather flow before it reaches the offtakes of the feeder rivers of the Hooghly). The consideration of the maintenance of this supply and its improvement is a necessity but may be deferred till the more urgent problems have been investigated."

3. *The Model.*

The model extends from Bansbaria down to reef with all the tributaries i.e. Rupnarain, Damodar, Haldia and Rasulpur reproduced -- see Photo No. 5410. The model bed is proposed to be laid in the Hooghly sand at Ninan ($m = 0.095 \text{ mm}$)

4. *Design of the tide reproducing arrangements for field model:*

To maintain equivalent rate of accretion due to suspended silt in the model the vertical velocities have to be relatively increased in the ratio of $L/d^{3/2}$ where $1/d$ and $1/L$ are the depth and length scale ratios of the model. In the case of the Hooghly pilot model this ratio works out to 0.988. Hence, the model rate of sedimentation will have to be equal to that of the prototype; for which proper salinity is proposed to be maintained by means of potash alum.

For reasons of cost it is necessary to use the same saline water over and over again allowing for losses etc., for which a re-circulation arrangement has been designed.

When, however, it is not necessary to run saline water, an alternative arrangement has been provided for using fresh water by gravity flow.

The design of the tidal arrangement.

The sump which is fed by gravity flow has a capacity of 5,000 cft. which takes into account any subsequent changes of depth scales requiring greater discharge, capacity for initially filling the Model, etc.

Water from the sump will be pumped into the entrance channel whose main function is to neutralise any jetting and turbulence caused by concentrated delivery from the four delivery pipes of the pumps.

Water flows over the weir provided along the downstream end of the entrance channel into the stilling pool which is provided with 5 symmetrically situated standing wave flumes for measuring the discharge dropping down into the Tidal Chamber. Stop gates have been provided along the side end walls of the stilling pool to adjust any fluctuations that may occur in pumping.

The tidal chamber is provided with sliding weirs which will be manually operated till electrical equipment is received and set up for automatic generation of tides.

A constant discharge measured by standing wave flumes drops into the tidal chamber from where it can flow, wholly or partially as required,

(a) into the model producing flood tide;

or (b) into the escape downstream of the sliding weirs when ebb is being reproduced;

by suitable manipulation of the sliding weirs.

Thus, when the flood tide is being reproduced, the sliding weir will be suitably raised, thus reducing the escape discharge and increasing the tidal influx, causing rise in the river.

On the ebb, the weirs which will be gradually lowered will escape out the ebb flow and also the constant discharge pumped into the tidal chamber from the sump.

When clear fresh water will be run for preliminary runs for designing cam etc., the sump will act as the head chamber in which the gravity flow water will be stilled and escaped into the entrance channel by means of the four screw gates to work the model tides as explained before. The discharge escaped by the sliding weirs will, in this case, be returned to the Mutha Right Bank Canal.

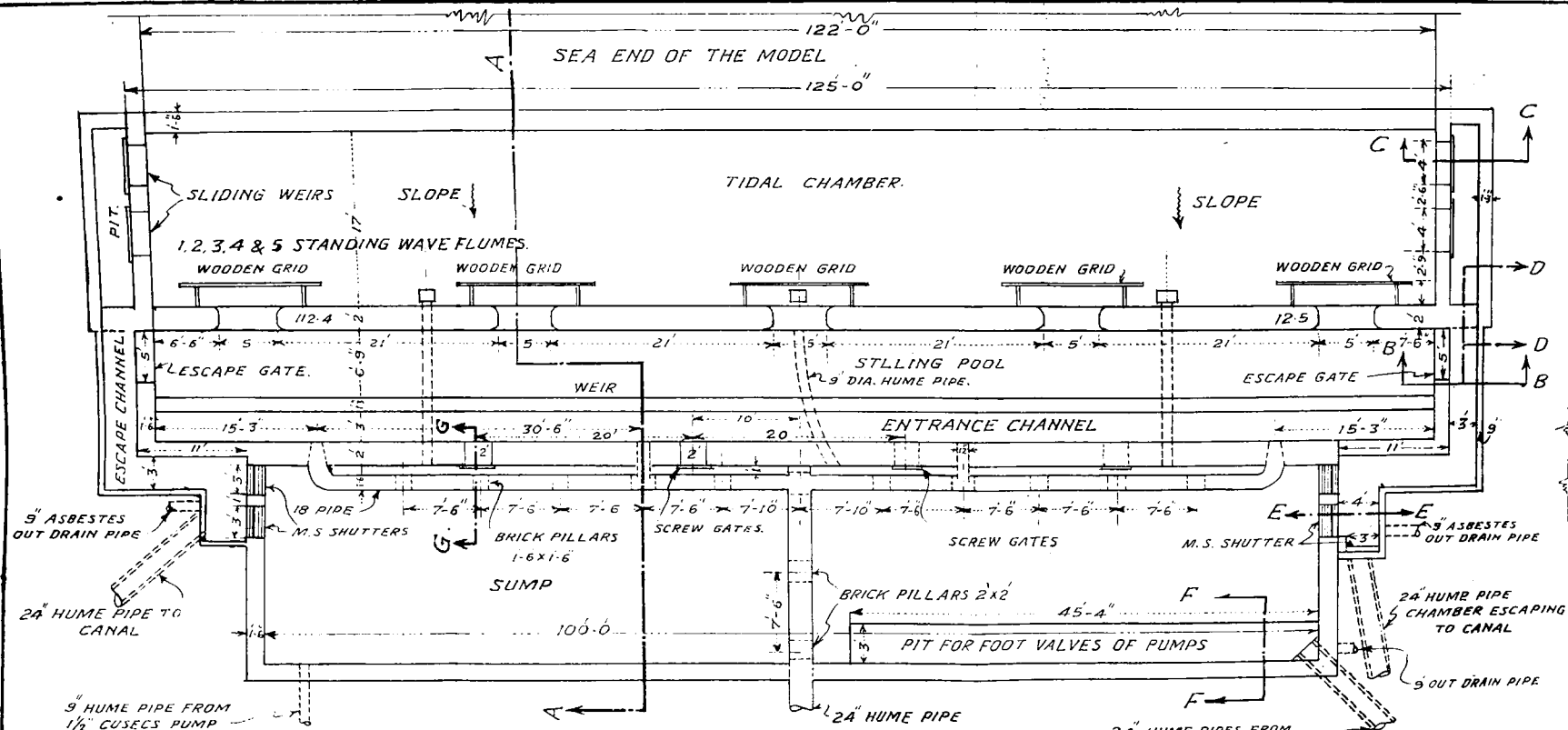
River Discharge:—The river discharge required for the Hooghly, Rupnarain, Damodar will be stored in the subsidiary tank 15 ft. × 10 ft. × 3 ft. from where it will be led to the various rivers by 3" pipes and valves.

This water will ultimately find its way into the tidal chamber from where it will be pumped back duly with fresh water pipe supply into the tank referred to above.

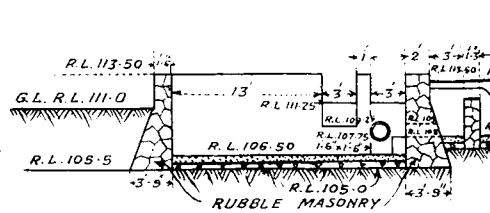


PHOTO No. 5410. Showing Hooghly Model

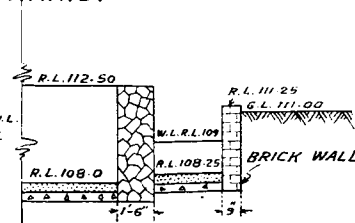
FIG. -50
HOOGHLY MODEL 1:2000/1:160



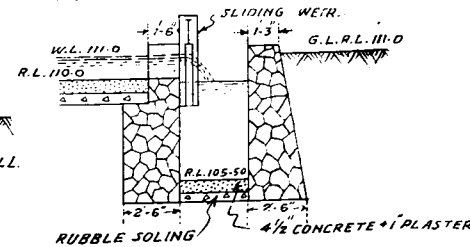
PLAN OF TIDE GENERATING MECHANIS.
SCALE = 1" = 12'



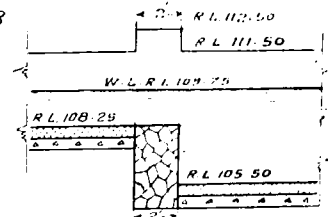
SECTION ON A-A.
SCALE = 1" = 6'



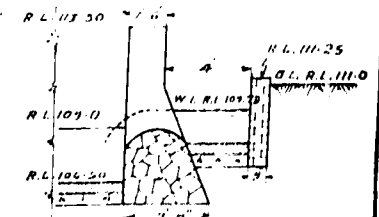
SECTION ON B-B
SCALE = 1" = 6'



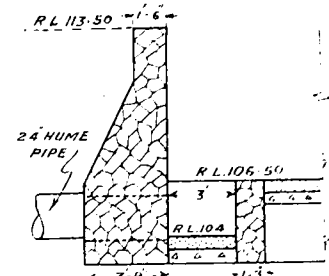
SECTION ON C-C
SCALE = 1" = 6'



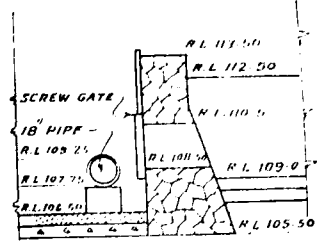
SECTION ON D-D
SCALE = 1" = 6'



SECTION ON E-E.
SCALE = 1" = 6'



SECTION ON F-F
SCALE = 1" = 6'



SECTION ON G-G.
SCALE = 1" = 6'

APPENDIX I

Calculations to fix the capacity of the sump.

Fig. 51. The maximum tidal inflow in the model with 1/2000, 1/160 scale ratios of the models is 12.35 cusecs. The constant discharge supplied into the tidal chamber will thus be 12.35 cusecs. Fig. 51 shows the model influx discharges for the spring tide of March 1940. The ordinate of the curve represents the discharge going into the model whereas the offsets of the ordinates between the curve and the horizontal line of $y = 12.35$ cusecs represent the discharge escaping into the sump viz. the escape channel. The net depleted discharge therefore is given by the ordinates of the curve.

The minimum capacity of the sump therefore, should be sufficient to accommodate the quantity of water represented by the shaded portion under the curve. This works out to 950 cft. approximately.

If the depth scale were changed to 1/80 the minimum capacity computed on the same lines as above should be 2,400 cft. Add to this the capacity required to initially letting water into the model which will be about 2,400 cft.

Making allowance for leakage and evaporation etc., the capacity may be fixed at 5,000 cft. This will enable operating the model even if the depth scales have to be changed from 1/160 to 1/80.

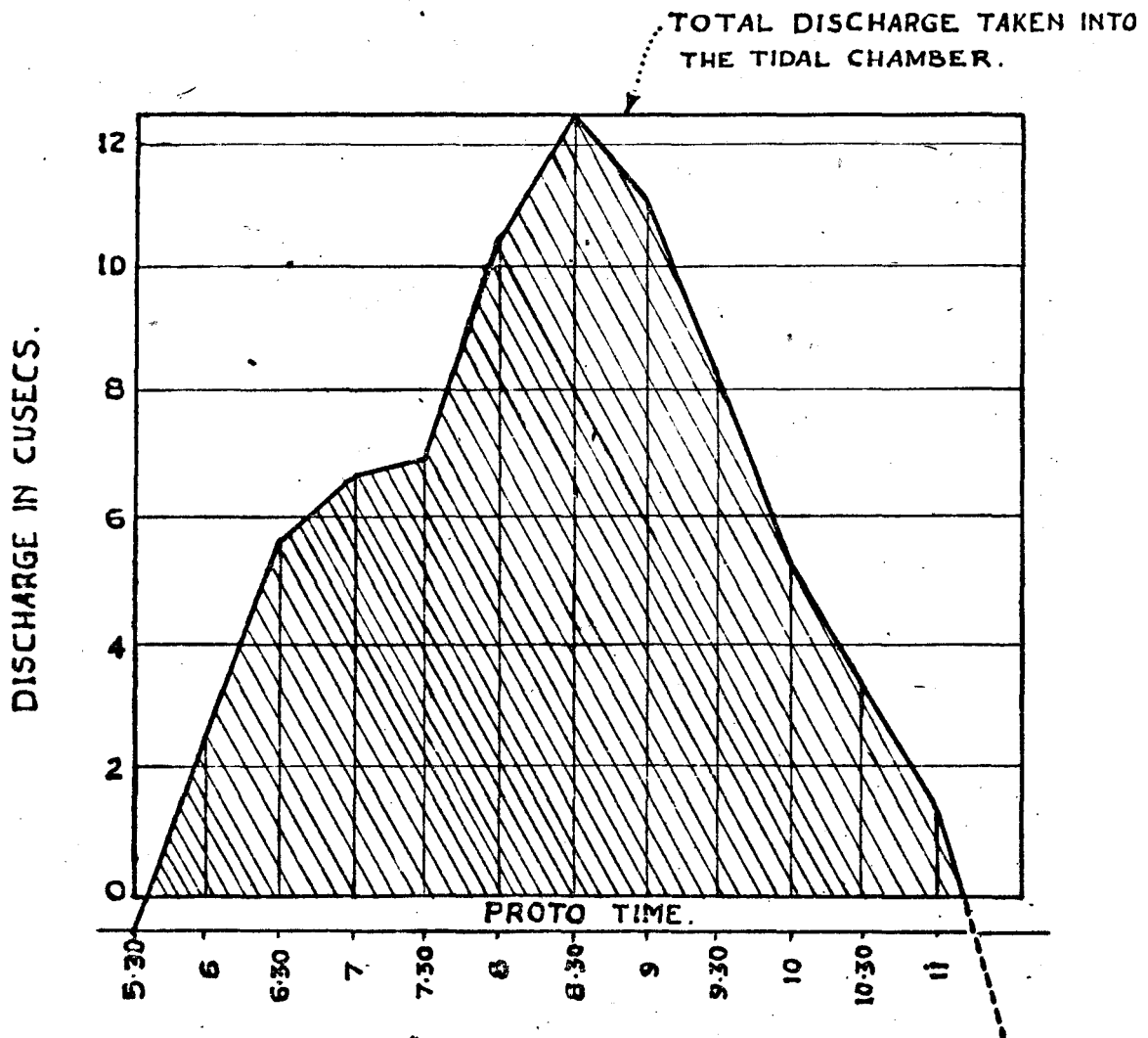
FIG. 51

HOOGLY MODEL EXPERIMENTS.

SHOWING $1/2,000$: $1/160$ MODEL INFLUX

DISCHARGE FOR THE SPRING TIDE OF

MARCH 1940.



$t = 20$ SECONDS IN MODEL

$$= 20 \times \frac{2000}{\sqrt{160}}$$

OR 3162.28 SECONDS

i.e. 52.70 MINUTES IN PROTO.

19. Preliminary note on the design of scale ratios of the Model of the Hooghly River from Bansbaria down to Reefs i. e. 15 miles below Saugar

Introduction :—The model of the Hooghly Estuary covers a reach of 115 N. Miles from Bansbaria down to Reefs 15 miles below the Saugar Light House, the object of model experiments is to investigate the means of improving the navigability of the River.

Length scale Ratio: Considering the area available immediately, the length scale of 1/2000 suggested by Sir Claude Inglis is suitable and is adopted.

Depth scale ratio :—Sir Claude Inglis had proposed a depth scale of 1/160; presumably this was based on Lacey's formulae which gives

$$\text{depth scale} = (\text{length scale ratio})^{2/3}$$

Table 1 attached gives the computations for Reynolds criterion Allen's criterion etc. for different depth scales.

According to Allen's method given on page 283 of his book "Scale models in Hydraulic Engineering", columns 4 to 7 of table 1 give the model depth in inches for Saugar, Gangra, Balari, Hooghly point and King George Docks with different depth scales ranging from 1/80 to 1/200. Columns 8 to 11 give the model velocities and their equivalent prototype values worked out by the formulae

$$V = 1.30 (\phi - \phi)^{0.27} L^{0.27} M^{-0.27} R^{0.23}$$

where

V = velocity in ft./second.

ϕ = sp. gr. of sand used in the model.

ϕ = sp. gr. of water assumed unity in this case.

L = length of the side of a cube having the same weight as the average grain (inches).

M = Uniformity modulus of material.

h = depth of water in inches.

The average grain size was taken to be the weighted mean diameter of the Ninan sand viz. 0.094 m.m. with which the model bed is proposed to be moulded. Fig. 52 shows the sand distribution graph from which the modulus of uniformity of Ninan sand was worked out to be 0.483.

The prototype max. flood velocities in the March 'spring tide of 1940 at Saugar, Gangra, Balari, Hooghly point and King George Docks have been tabulated in column 3.

According to Allen, if computed velocities—columns 8 to 11—prove to be not greater than the anticipated maximum flood or ebb velocity (Col. 3)

Table

Showing the computations for working out scale

1	Mean d with maximum flood. 2	Maximum flood v in proto ft./sec. 3	h=depth in model corresponding to col. 2 with d scale			
			1/80 4	1/120 5	1/160 6	1/200 7
	ft. in.		in inches			
Saugar	... 25 0	5.20	3.756	2.496	1.884	1.500
Gangra	... 26 1	7.41	3.912	2.616	1.956	1.572
Balari	... 25 0	5.40	3.756	2.496	1.884	1.500
Hooghly Point.	27 4	7.50	4.116	2.748	2.064	1.644
King George Docks.	35 0	10.70	5.196	3.492	2.628	2.100

Depth scale. 16	$h \equiv 16.33$ ft. below Saugor feet 17	h^3 18	e 19	$\frac{\sqrt{H/30}}{\sqrt{16.33/30}}$ 20	$h^3 e \sqrt{H/30}$ 21	Vertical velocity ratio $\frac{L/d^{3/2}}{22}$
1/80	0.204	0.0025	25.00	0.737	0.157	2.80
1/120	0.136	0.0025	16.67	„	0.0307	1.52
1/160	0.102	0.0011	12.50	„	0.0101	0.988
1/200	0.082	0.0006	10.00	„	0.00442	0.706

1

ratios of the 1/2000 Hooghly Model.

V* in ft. per second with d scale In model and its equivalent in prototype				Model Reynolds $Vd/10^5$ with d scale of			
1/80	1/120	1/160	1/200	1/80	1/120	1/160	1/200
8	9	10	11	12	13	14	15
0.513 ≡ 4.588	0.467 ≡ 5.116	0.443 ≡ 5.604	0.416 ≡ 5.883	0.1666	0.0971	0.0695	0.0520
0.520 ≡ 4.651	0.472 ≡ 5.170	0.442 ≡ 5.591	0.420 ≡ 5.940	0.1695	0.1029	0.0720	0.0550
0.513 ≡ 4.588	0.467 ≡ 5.116	0.443 ≡ 5.604	0.416 ≡ 5.883	0.1606	0.0971	0.0695	0.0520
0.524 ≡ 4.687	0.478 ≡ 5.236	0.447 ≡ 5.654	0.424 ≡ 5.996	0.1707	0.1095	0.0769	0.0581
0.554 ≡ 4.955	0.505 ≡ 5.532	0.473 ≡ 5.983	0.449 ≡ 6.350	0.2426	0.1469	0.1036	0.0786

Max. flood Q in model with various d scales		Q in model ≡ 50×10^6 cusecs.	Time scale ratio and tidal period.	
d scale	Q scale = $\frac{L}{d^{3/2}}$		Time scale ratio.	Model tidal period ≡ 12 hrs. 18 mins.
23	24	25	26	27
1/80	$1/1.43 \times 10^6$	35.00	1/223	3.30 mins.
1/120	$1/2.63 \times 10^6$	19.00	1/183	4.03 ,,
1/160	$1/4.05 \times 10^6$	12.35	1/158	4.66 ,,
1/200	$1/5.66 \times 10^6$	8.82	1/141	5.24 ,,

*—Computations for $V = 1.30 (\phi - \phi)^{0.37} L^{0.27} M^{-0.27} h^{0.23}$
 $= (1.3 \times 1.148 \times 0.2094 \times 0.822) h^{0.23}$
 $= 0.3785 h^{0.23}$

in the model, then there is a very good chance that the proposed vertical scale will be satisfactory from the aspect of bed movement.

The lower limit of velocities shown in columns 8 to 11 are the velocities computed by Allen's formulae which is based on flume experiments where curvature of flow did not exist. When, however, there is curved diving flow, the distribution of velocities across the section is far from normal and bed movement is expected to occur at even lower velocities than those mentioned in columns 8 to 11.

Taking this fact into account, it will be seen that depth scale of 1/160 would be about right and hence it is proposed to be adopted.

20. Proposed Bridges and Reclamation between Ernakulam, Thanthonni and Mulavukad Islands (Cochin)

1. Introduction.

The Chief Engineer, Cochin State, has proposed to construct bridges between Ernakulam and Wallarpot and Figs. 53 and 54 the reclamation of the shallow areas surrounding Thanthonni Island and a part of Ernakulam. The area thus reclaimed will be of great benefit in as much as it will provide for the expansion of the Ernakulam town which is already congested. The Director was, therefore, requested to advise on the effects this proposal would have on the tidal regime of the back waters.

2. Findings and advise.

2.1. As seen from the plan, the deep channels and therefore presumably the main current, appear to be along the west bank between Thanthonni and Mulavukad Islands, but a subsidiary deep channel also exists between Thanthonni and Ernakulam.

2.2. It was pointed out that the waterway of the two channels, before and after bridging, dredging and reclamation should be compared separately for the two channels and not jointly. It will be seen from table 1 that under post-bridge conditions the waterway of the Ernakulam-Thanthonni Channel will be increased by 29% and the waterway of the Thanthonni Mulavukad Channel will become 2% less than at present. The existing deeper channel, however, is the Thanthonni Mulavukad Channel. This, therefore, indicates that if the discharge distribution and current sets remain substantially unaltered, the excess waterway of the Ernakulam Channel is likely to silt up rapidly and the aggregate waterway may fall short of the existing overall waterway. Thus, on the one hand there is wasteful dredging proposed in the Ernakulam Channel while, on the other, it is inadequate on the Mulavukad side.

SHOWING SUMMATION CURVE OF
NINAN SAND FOR
COMPUTING UNIFORMITY MODULUS.

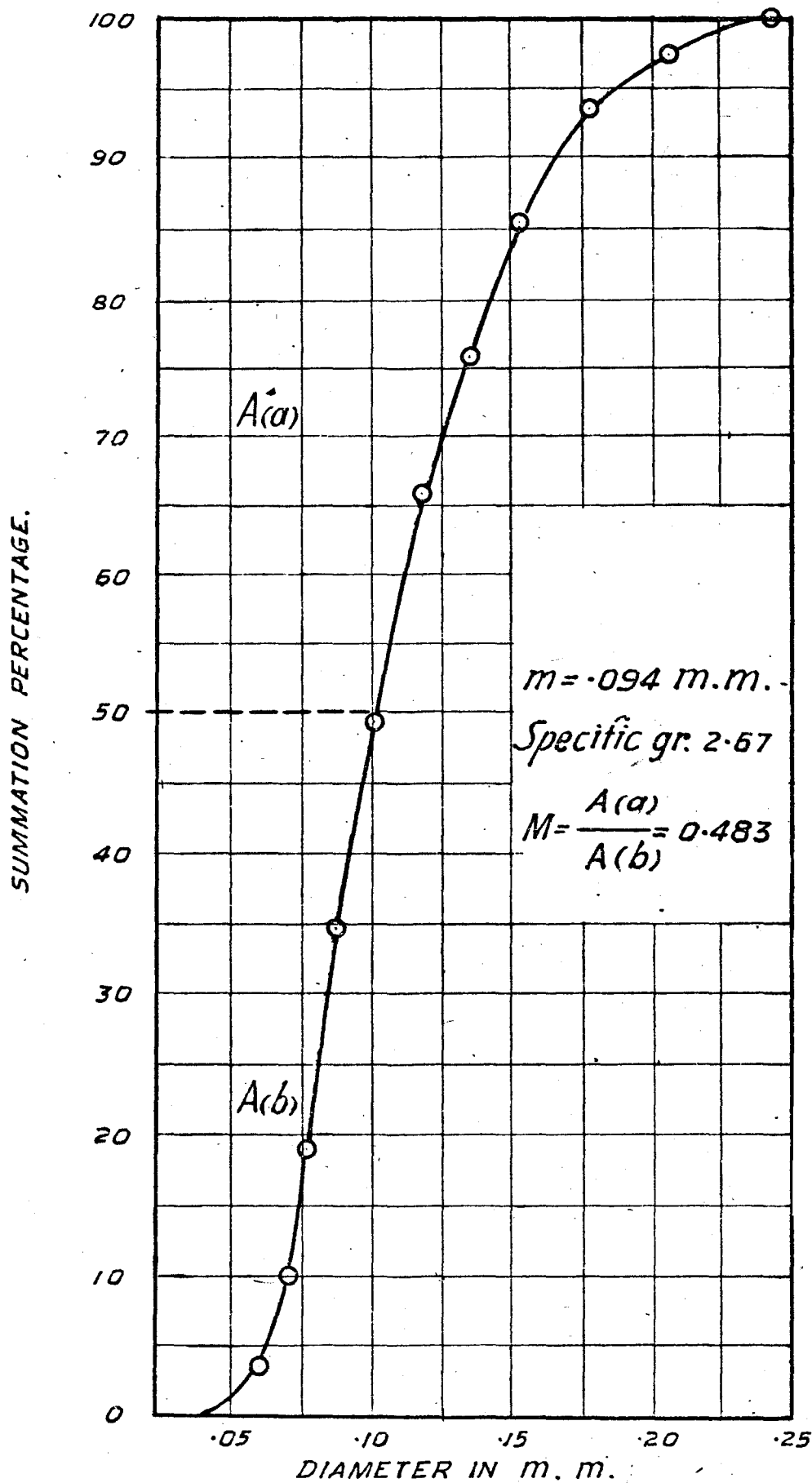


FIG. 53

SHOWING THE CENTRE LINE OF THE PROPOSED BRIDGE
THE AREA TO BE RECLAIMED.

SOUNDINGS ARE IN FEET BELOW LOW WATER O. S. T.

SCALE:- 1" = 990'

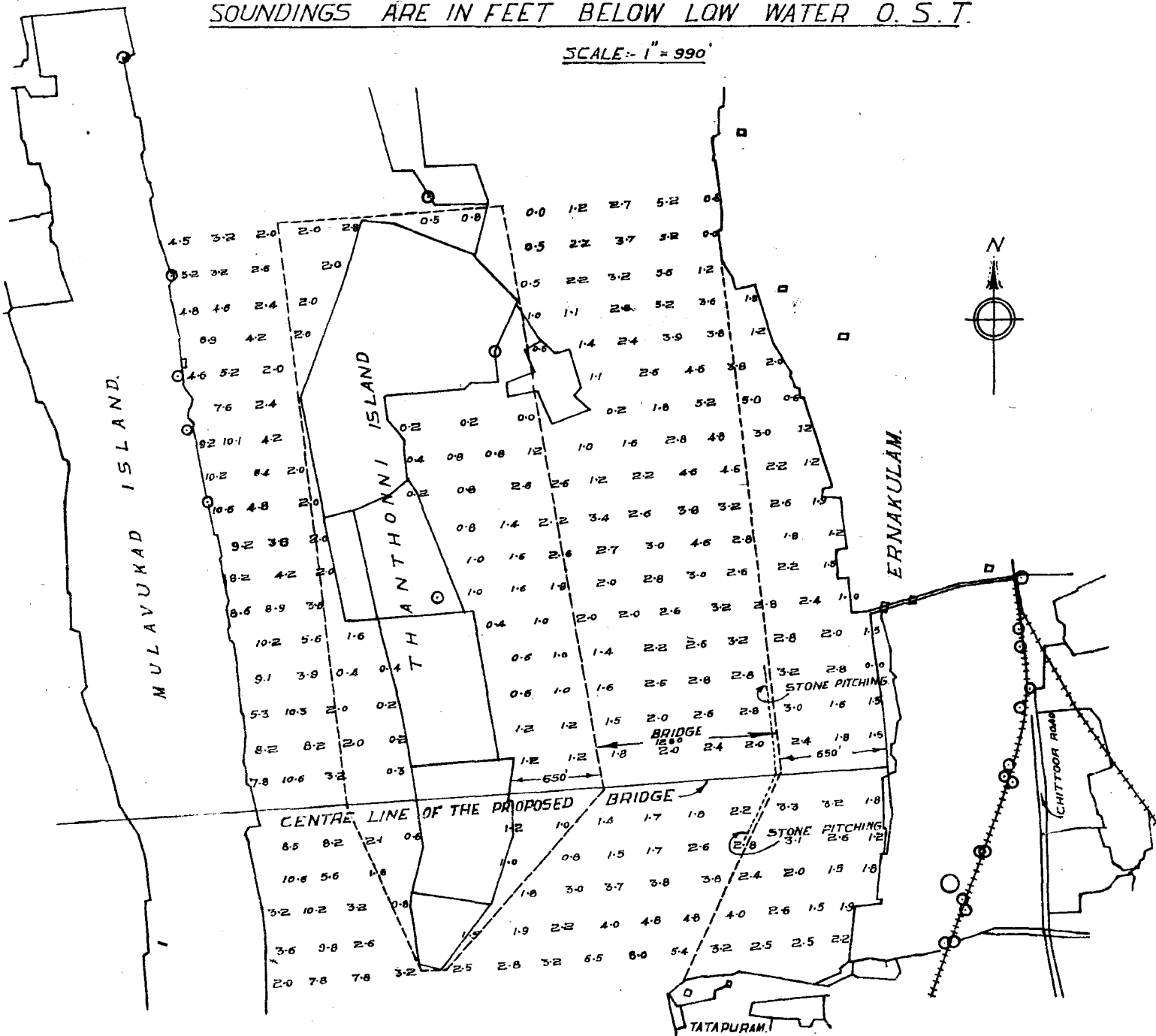


FIG. 54

SHOWING THE PROPOSED ALIGNMENT OF
BRIDGE BETWEEN ERNAKULAM AND
WALLARPADAM ISLAND AND THE
RECLAMATION PROPOSAL.

SCALE 1" = 4000'

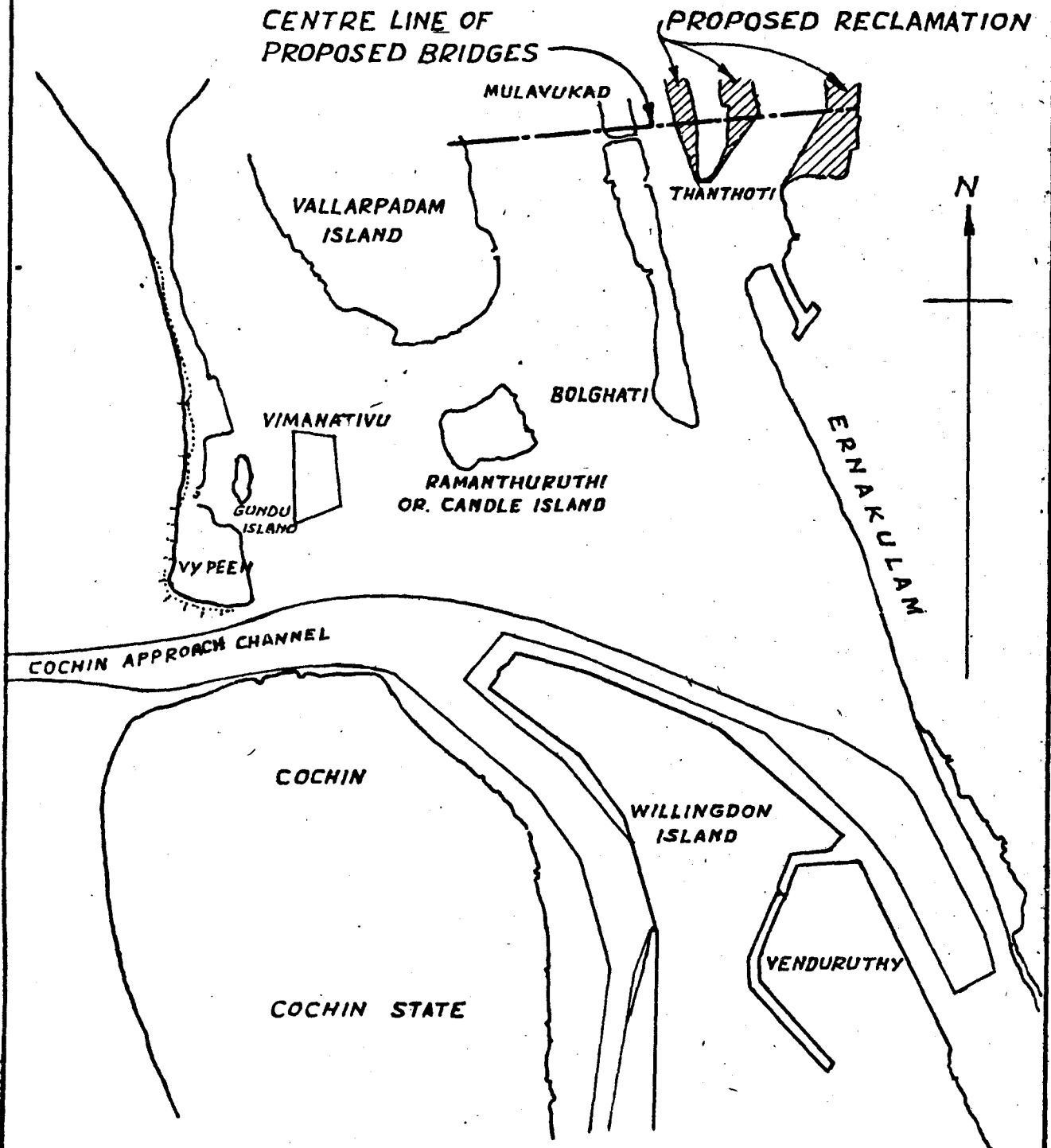


Table I

Showing waterway before and after bridging, dredging and reclamation

	Between Ernakulam and Thanthonni Isladns.			Between Thanthonni & Mulavukad Islands.		
	Post- Bridge	Pre- Bridge	Col 1 Col. 2	Post- Bridge	Pre- Bridge	Col. 4 Col. 5
	1	2	3	4	5	6
(1) Area of bridge 'section in sq. ft.	20,870	15,335	1.36	8,973	8,592	1.04
(2) Area of obstruction assumed proportional to bridge length in sq. ft.	1,122	—	—	510	—	—
(3) Net area of flow 'A' in sq. ft.	19,748	15,335	1.29	8,463	8,592	0.985
(4) Width of channel 'B' in ft.	1,280	2,580	0.50	650	1,280	0.51
(5) $\frac{\text{Net Area A}}{\text{Width B}} = \text{Mean depth}$ in ft.	15.4	5.94	2.59	12.8	7.8	1.64

2.3. The best scheme appears to be to least disturb the established regime of the two channels and obtain the best value by dredging in such a way that the areas of the two channels would have, approximately, the same ratio as the existing areas; i.e. 15,335 : 8,592 or 1.79 : 1.

Accordingly the net dredged area of the Ernakulam Channel would be about 18,091 sq. ft. against 19,748 sq. ft. proposed and that of the Mulavukad channel 10,120 sq. ft. against 8,463 sq. ft. at present. The corresponding depths in each channel can be worked out and will then stand a better chance against resiltting by virtue of their conforming to existing natural depths.

2.4. It is presumed that the large amount of dredging proposed is to utilise the spoils in reclaiming the land, (the cut balancing the fill) otherwise it would have been possible to reduce the dredging.

2.5. The percentage obstruction caused by the bridge is small, as table 2 shows, particularly after the proposed dredging.

Table 2

Showing percentage obstruction caused by proposed bridging.

Channel between	Original waterway before bridging sq. ft.	Net waterway after dredging, bridging & reclamation sq. ft.		Area of obstruction sq. ft.	Obstruction as % of cross section.		
		As proposed by C.E. Cochin	As suggested in 2.3 above		Existing	As proposed	As suggested in 2.3 above
1	2	3	4	5	6	7	8
Ernakulam Thanthonni	15,335	19,748	18,091	1,122	7.3	5.38	6.2
Thanthonni- Mulavukad	8,592	8,463	10,120	510	5.94	6.02	5.03

Comparing this with Gibson's results⁽¹⁾ of similar experiments the percentage obstruction being small the effect on H. W. spring tide, L. W. spring tide and delay in time of high water and slack water period is expected to be small.

2.6. In spite of this, due to curved flow and restricted widths the Eastern Ernakulam Channel, in particular, will have to be pitched in concave portions — shown in plan in dotted line — so as to resist erosion.

21. Silting of salt water intake channel at Mithapur Okhamandal, Kathiawad.

1. Introduction.

1.1. The Tata Chemicals Ltd. have established a factory at Mithapur about 7 miles from Okha. Chemicals are manufactured from common salt which is obtained by evaporating sea water taken through an intake channel into two shallow water lakes.

(1) "An experimental investigation of the effect of Bridge piers and other obstructions on the tidal levels in an Estuary" Excerpt journal of the institution of civil Engineers, Session 1937-38.

2. History.

Fig. 55

2.1. In the year 1933, a barrage with 8 flood gates was built at the entrance to the Outer Sagar and an intake channel 66 ft. wide and 140 ft. long was built in rubble, which extended up to the then shore line.

2.2. The tidal range at Mithapur is 12 ft. (i.e. from R.L. 2 to R.L. 14). The sill level of the barrage is R.L. 10.7. Sea water is admitted into Outer and Inner Sagars by opening the barrage gates when the tide level is above R.L. 10.7.

Fig. 56

2.3. After the construction of the intake channel, it was observed that silting occurred on both sides outside the channel and the toe of the sea shore advanced. After 1933, practically every year, the intake channel was extended towards the sea to get a clear entrance; but the silting on both sides of the channel and creep of the shore toe kept pace with the wall extension. At present, the Northern wall of the intake channel extends to a length of 515 ft. and the Southern wall to 615 ft from the barrage.

3. Problem.

3.1 The above mentioned process of silting and advance of the shore still continues and it is feared that if this encroachment is not checked, the channel would eventually be completely blocked, thus becoming unsuitable or prohibitively costly for maintenance.

3.2. The silting is not only confined to the entrance of the intake channel but some deposition of sand is said to take place in the intake channel itself up to the barrage as well as some portion of Outer Sagar.

Fig. 55

3.3. The Company is contemplating to construct another boat cum intake canal, the tentative alignment of which is shown in Fig. 55. This has to be aligned and designed to preclude or minimise similar sanding trouble and subsequent expenditure over heavy dredging in future.

4. Analysis of the problem.

4.1. The problem is two fold.

- (a) Silting on both sides of the intake channel and advancing of the shore.
- (b) Silting of intake channel and Outer Sagar

4.2. Any extension in the sea portion is usually accompanied with silting problem. Bed sand is thrown in suspension by wave action, is

carried by tidal currents and this is deposited in return flow or slack velocity regions produced by any structure in the sea. Conditions at the intake channel are not dis-similar to these.

4.3. Diagnosis of the silting in the intake channel and part of Outer Sagar is comparatively simple. The barrage is apparently responsible, as it prevents the free movement of tidal flow. Some of the heavy sand charge must be finding its way with sea water drawn into the Outer Sagar when barrage gates are opened at the time of high tide; and in normal times the material carried by the flood tide deposits in the intake channel.

5. Remedial Measures.

5.1. The problem of realigning the present intake channel and boat cum intake canal, to avoid present trouble, is a complex one and opinion can be expressed only after the results from model experiments are available.

5.2. Silting of intake channel however suggests an obvious solution of flushing it occasionally. The following recommendations were made by the Station.

5.3. Though practical measures for complete elimination of sanding are difficult to suggest, measures for its reduction can be put into effect immediately and they may perhaps obviate the subsequent necessity of more costly measures. With this view, it was suggested that another set of regulating gates should be refixed at the old site of Sagar Dam, where these gates used to be, before they were removed to the present position. The loose stone rubble walls in the approach channel to Outer Sagar should be raised above water level and made pucca, similar to the walls on the sea side of the new gates. The new gates at the Barrage have very shallow foundations; these will have to be strengthened to permit the flow back to the sea with maximum head equal to the difference between the pond level in the compartment and the crest level of this regulator. A line of sheet piles or an impermeable curtain wall carried into rock at the sea end followed by a loose stone apron designed for maximum scour would be all that is required.

Fig. 55

These proposals were made with a view to give added control by dividing the compartment area into two parts, so that by keeping the second set of gates shut, the Barrage gates open, the Outer Sagar between the two sets of gates can be flushed during monsoon with no loss of storage from the Inner Sagar and when another freshet comes, the Outer Sagar could be replenished. This flushing would not only help to keep the Outer Sagar from rapidly accreting but could be effectively used to flush out the approach channel above the Barrage gates right down to its rocky bed. With the provision of second set of gates, these flushing operations should be practicable, at opportune periods of the seasons, without any loss of useful storage.

FIG. 55
PLAN SHOWING OUTER & INNER
SAGAR & PRESENT INTAKE
CHANNEL
SCALE. 1" = 1800'

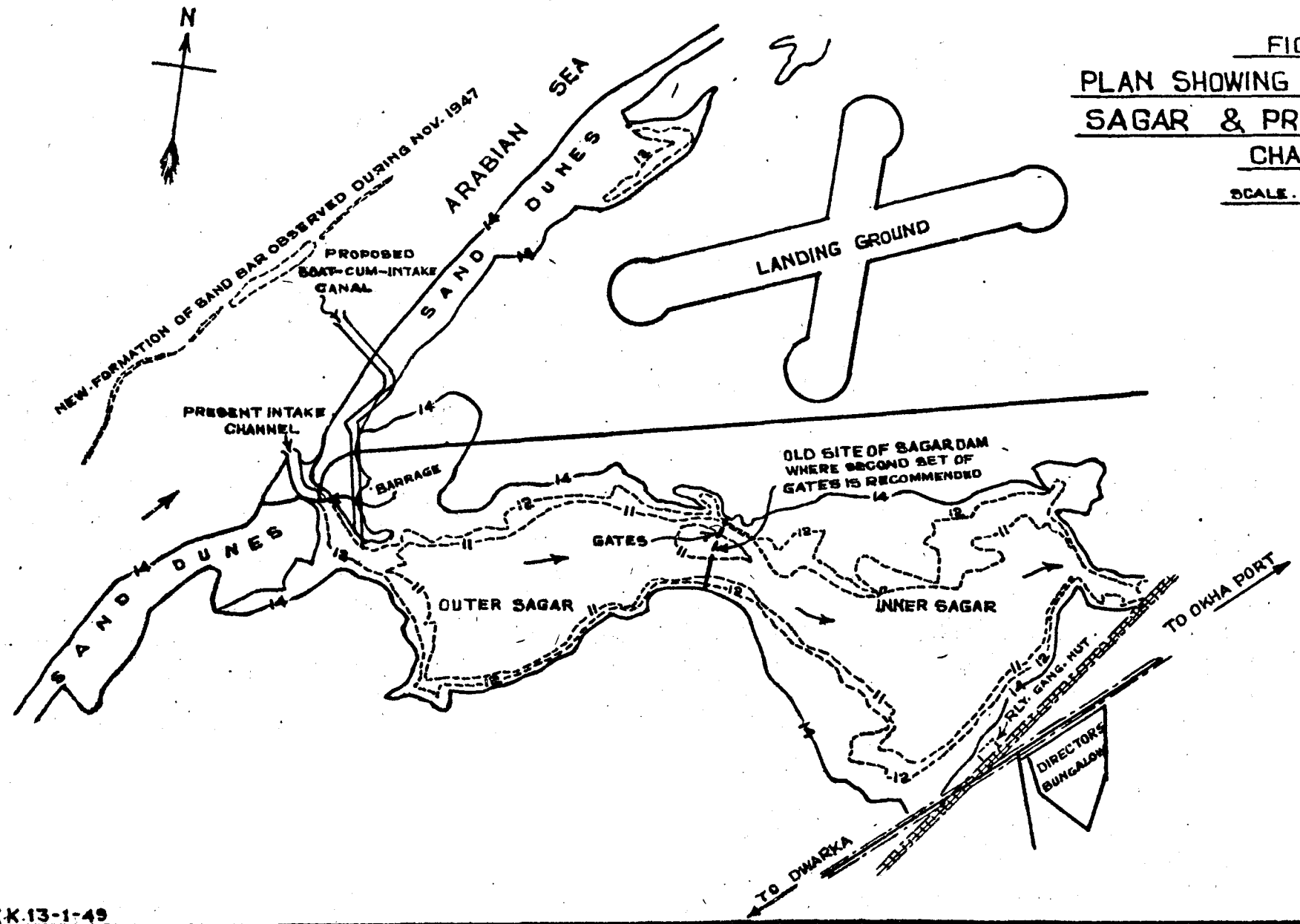


FIG. 56
'PLAN SHOWING EXTENSION OF
INTAKE CHANNEL SINCE 1933 AND
CORRESPONDING SHORE ADVANCE
SCALE: 1" = 120'

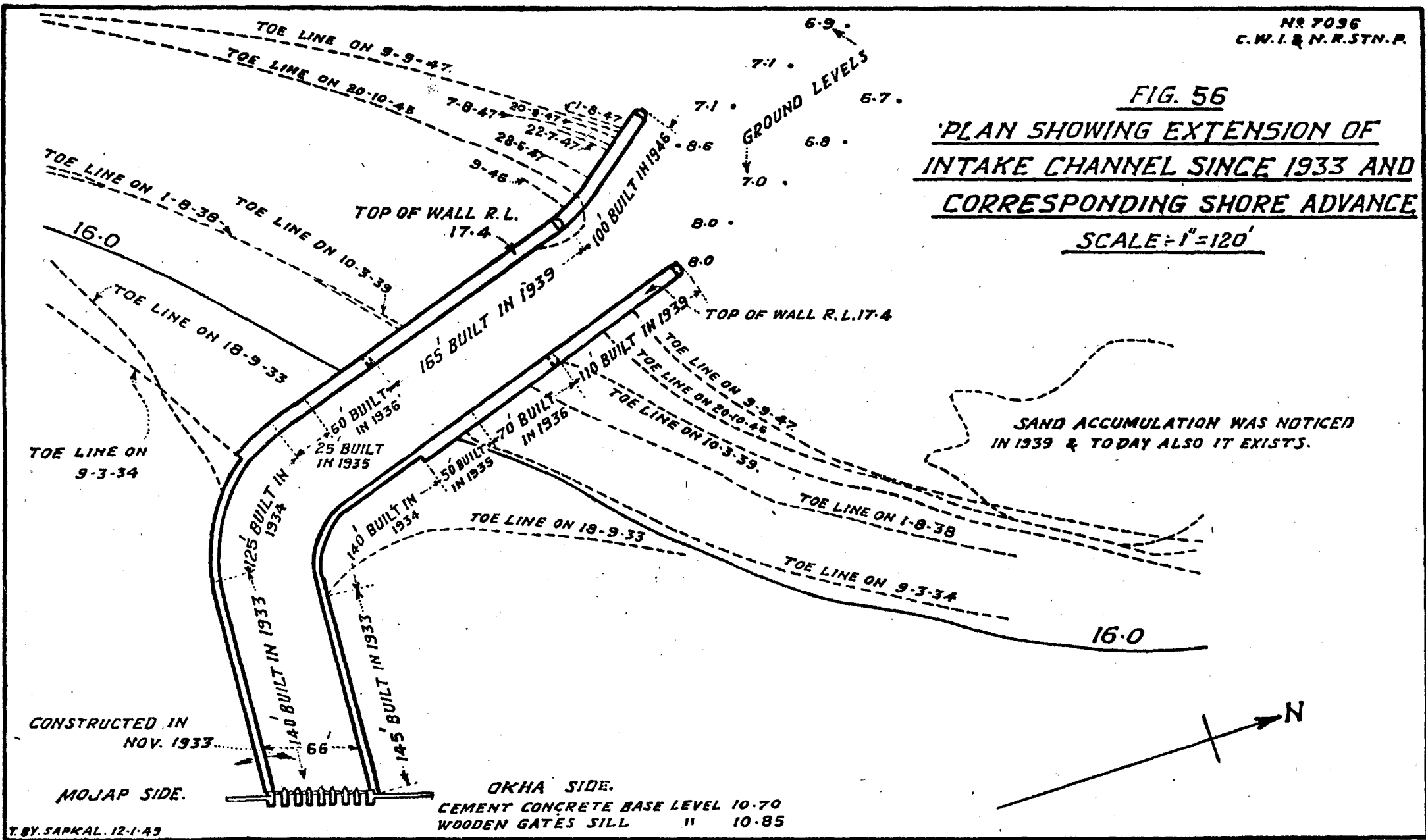
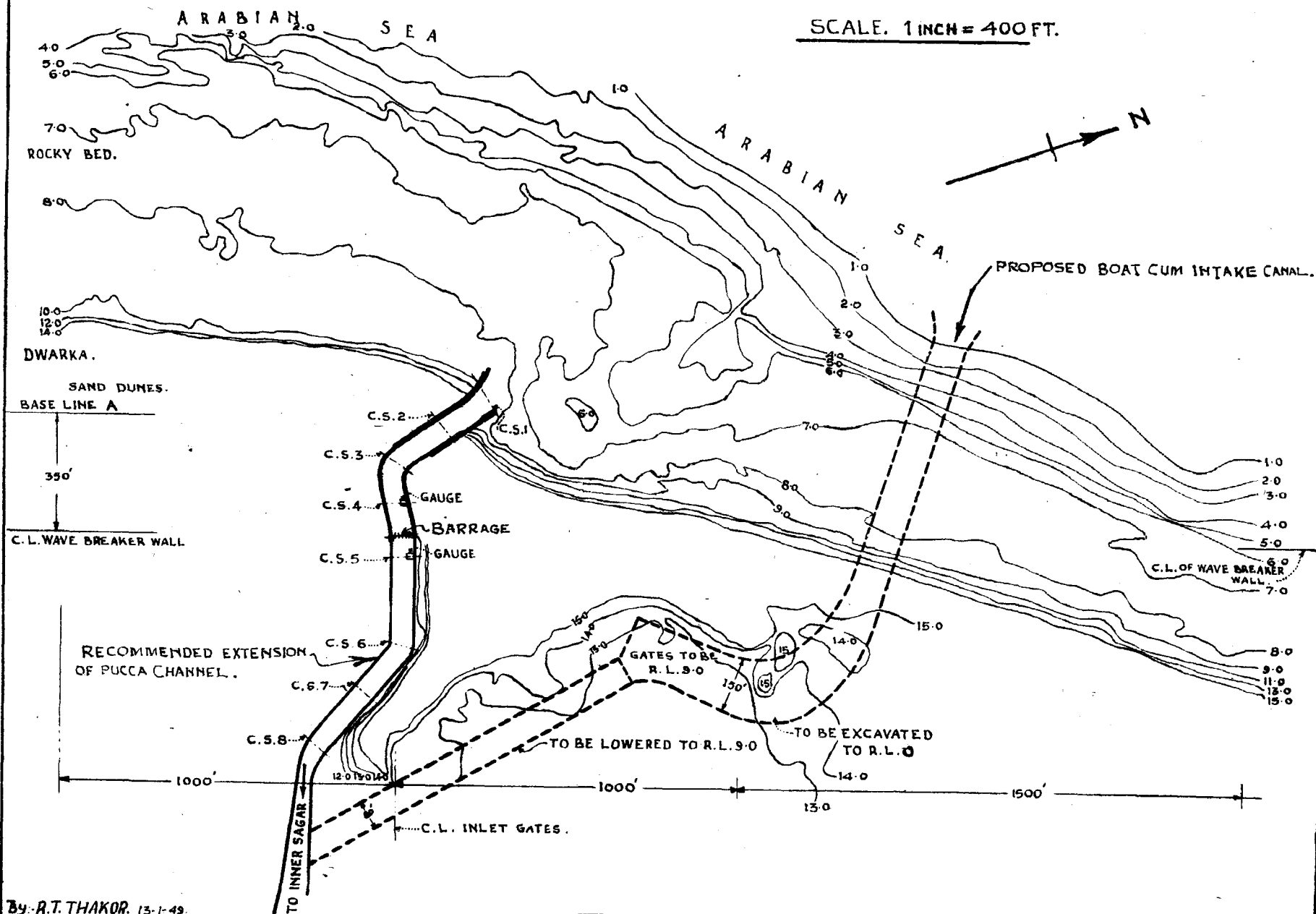


FIG. 57

TATA CHEMICALS LTD.

PLAN SHOWING TENTATIVE POSITION OF PROPOSED
BOAT-CUM-INTAKE CANAL AND RECOMMENDED
EXTENSION OF PUCCA CHANNEL IN OUTER SAGAR

SCALE. 1 INCH = 400 FT.



6. *Feasibility of scouring operation.*

6.1. Length of pucca channel in Outer Sagar according to above suggestions would be about 1,200 ft. For the scouring of this channel and the present intake to be effective, sufficiently high velocities must be obtained and the duration should be appreciable.

6.2. Velocities which are likely to be attained are given in Table 1 in table 1 and are based as follows.

Water level in Outer Sagar in the beginning of the scouring operation would be R.L. 14.0. Loss of head in 1,200 ft. channel i.e. up to the barrage gates is assumed to be 0.5 ft. and Manning's rugosity coefficient = 0.02. Velocities and discharges in the proposed pucca channel for different water levels in Outer Sagar are given in columns (8) and (9) of table 1.

Scouring velocities for Mithapur sand (mean diameter = 0.41) for different depths according to Kennedy's formula are given in column (11).

It will be seen that the velocities in the channel would be higher than the scouring velocities. Discharges, for fully opened barrage gates with different water levels, are given in column (12), which are higher than the discharges required to produce scouring velocities.

Duration of scouring operation will be the time required for water level in the Outer Sagar to drop from R.L. 14 to R.L. 12. This is about 2.23 hours. The scouring operation will be, therefore, quite satisfactory.

Velocities in the present 615 ft. long intake channel would be considerably higher, being a free flow to sea and the channel can be effectively flushed to the bed rock.

7.1. The Tata Chemical Ltd. have been informed that model experiments, with two different scale models have to be done to arrive at an all time satisfactory solution for the present intake channel as well as for the proposed boat cum intake channel.

Table 1

No.	Water level in the Outer Sagar above mean sea level.	W. L. at the gates, assuming 0.5 ft loss in the 1200 ft. channel.	Mean depth of water in the channel, ft.	A Area of cross section of channel width 66 ft. sq.ft.	P Wetted Perimeter. ft.	R Col.(5) Col.(6) ft.	$V = \frac{1.486R^{2/3}}{N} S^{1/2}$ ft./sec.	N = 0.02 (assumed)	Q in the channel cusecs.	* $V_0 = 0.8d^{0.64}$ ft./sec.	Scouring velocity $1.4V_0$ ft./sec.	Discharge through gates when fully opened for W L.s in Col.(3) = CBD 1.5 cusec	Remarks.
1	2	3	4	5	6	7	8	9	10	11	12	13	
1.	14.0	13.5	3.0	198.0	72.0	2.75	2.88	570	1.7	2.38	600		
2.	13.5	13.0	2.5	165.0	71.0	2.3	2.56	420	1.5	2.1	450		
3.	13.0	12.5	2.0	132.0	70.0	1.89	2.25	298	1.31	1.83	310		
4.	12.5	13.0	1.5	99.0	69.0	1.44	1.86	184	1.09	1.52	190		
5.	12.0	11.5	1.0	66.0	68.0	0.97	1.44	95	0.84	1.18	91.5		

*
 No. of gates—8
 Width of each gate...5.33ft.
 ∴ Total discharge =
 $CBD^{1.5} = 3.0 \times 8 \times 5.33 \times D^{1.5}$
 Column 12 shows that the gates are capable of passing the discharges in Col-(5).
 * Kennedy formula for movement of sands of mean dia. above 0.4 mm. Mean dia. of sand at Mithapur = 0.4 mm.

Table 2.

Serial No.	Particulars	Water contents in Outer Sagar M. Cft.	Mean Discharge Cs.	Time required Hours	Remarks
1	2	3	4	5	6
1.	Water level between R.L. 14 and 13.	1.0	435	0.64	For discharges see Table 1, Col. 9.
2.	Water level between R L. 13 and 12.	1.13	198	1.59	
				2.23 hours.	

22. Tide Generating Apparatus in the Hooghly Model.

Introduction.

1.1. In tidal compartments of rivers the flow of water is in both directions; when the flow is in the same direction as the river flow, it is known as ebb tide and when the flow is in opposite direction to the river flow or when the sea water flows into the river it is known as flood tide. During flood tide and ebb tide, water levels at all places in the tidal compartments of rivers change with time. At any place water levels rise during flood tide and gradually fall during ebb tide depending on the time of flood and ebb tide, the shape of the river and the sea and river discharges.

1.2. In connection with the Hooghly Model Experiments it was found necessary to devise an apparatus, which would produce the reversal of flow and the same rise and fall of water level to model scales as in the river. As electrical power was not available, the mechanical drive of the clock work was used for the operation of a cam to make the reproduction of tide automatic. In the absence of the electrical relays, a simple device of a cone, operating as a valve and controlled by a needle, was developed and the whole relay system was worked by the hydraulic power as described in detail below.

This device was very briefly described in Annual Report (Tech.) 1940.⁽¹⁾

(1) Page 19, Annual Report (Tech.) 1940-41.

2. The apparatus.

2.1. The apparatus mainly consists of three parts having different functions. The main parts are (a) the recorder (b) needle valve operated by a cam and (c) the cone valve operated by a lever and float.

2.2. The recorder consists of a rotating drum operated by its own clock-work mechanism. It is fitted with a stylus which carries a pencil and can slide along the length of the rotating drum. To one side of the stylus is attached a float by means of a brass wire passing over a pulley and to the other side is attached a counterbalance weight of the float in a similar manner. The float moves in a gauge well which is connected to the model and always keeps the same water level as in the model. The float records water level in the model at any instant, communicates it to the pencil and the rotating drum takes care of time. The recorder thus checks up the model tidal curve and helps in modifying the cam.

Photo 5164.

2.3. Working of the apparatus.

Functions of (b) & (c) will be clear from the working of the apparatus which is as follows:—

The clock-work mechanism rotates a cam, obtained by the method of trial and error, as shown in photo 5149 and Fig. 58. One complete rotation of the cam requires the same time as one tidal curve in the model. For 1/300, 1/60 Hooghly Model one tidal curve required 19 min. \cong 12 hrs. 18min. in the river and so the cam is rotated at the rate of 1 revolution in 19 minutes. The rotation of cam lifts or lowers the pulley which is in contact with it and the movement of the pulley operates the needle valve which is connected to it by a rigid frame.

Photo 5149.

Fig. 58.

Photo 5157.

The needle chamber in which a constant water level is maintained by a overflow arrangement is connected to the float chamber as shown in Fig. 59. The operation of the needle, therefore, controls the discharge into the float chamber, or the water level in the float chamber. The float is thus brought into action.

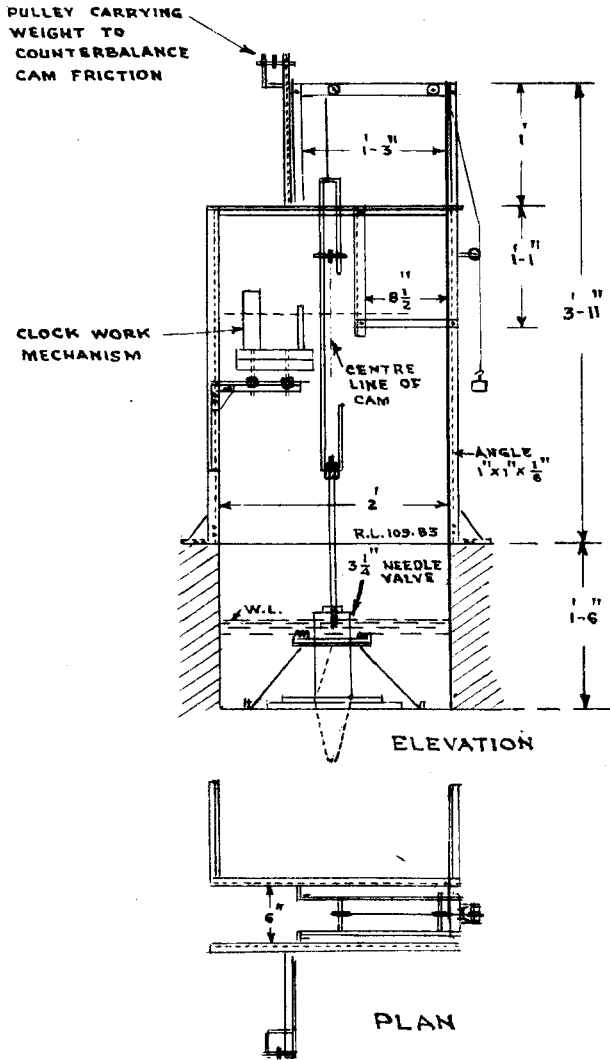
Fig. 59.

This float is connected to a lever, the other end of which carries the cone valve, as shown in Fig. 60. The movement of the cone valve, thus, even though apparently controlled by float is taken care of by the cam and if the intermediate links are kept in mind it can now be said that the cam controls the cone valve.

Photo 5155

Fig. 60.

FRAME SHOWING
CLOCK WORK & NEEDLE VALVE
SCALE 1" = 20"



DETAILS OF 3 1/4" NEEDLE VALVE
FOR TIDAL APPARATUS
SCALE: 1/4" FULL SIZE

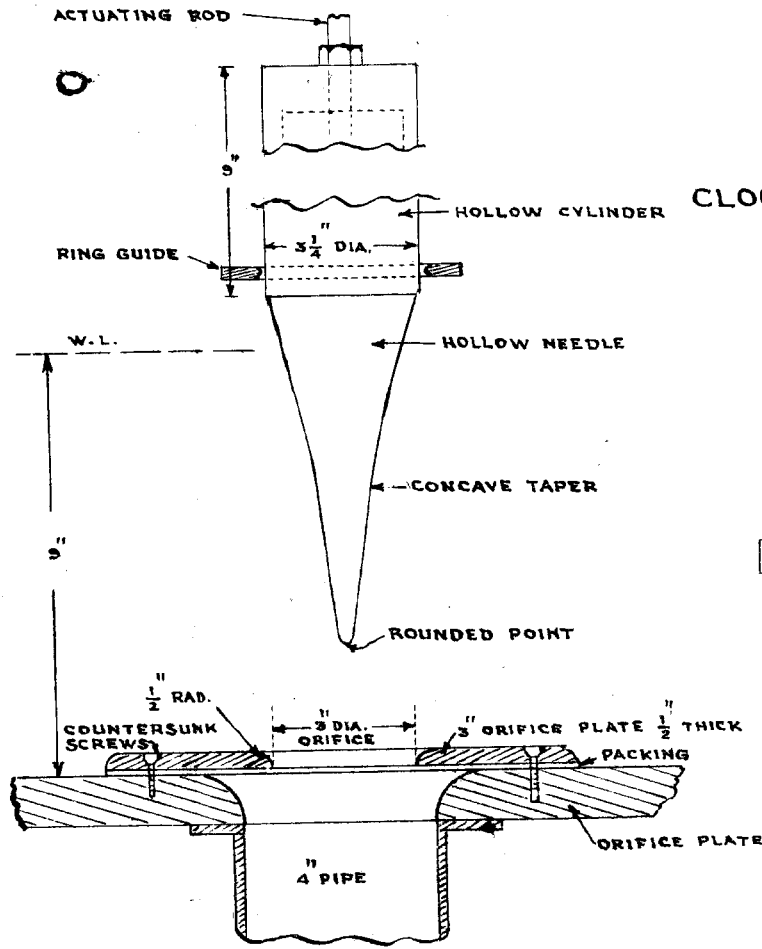


FIG. 58

CLOCK WORK OF TIDAL APPARATUS
(N. T. S.)

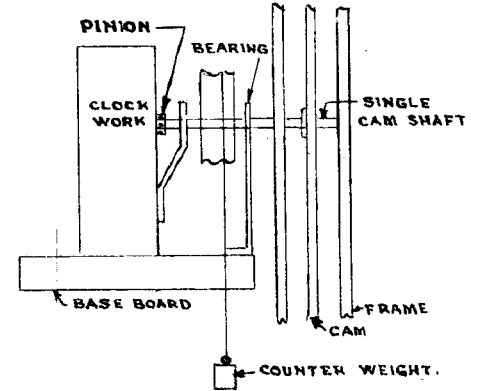
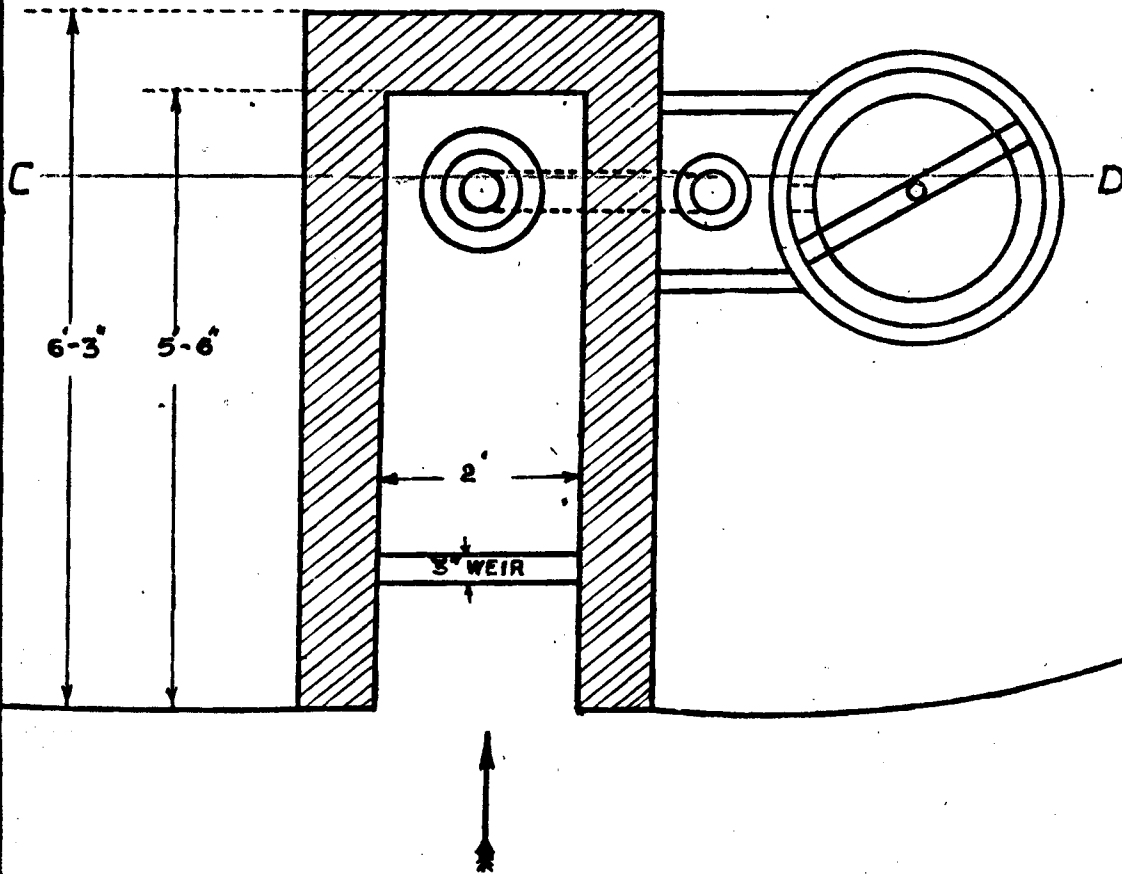
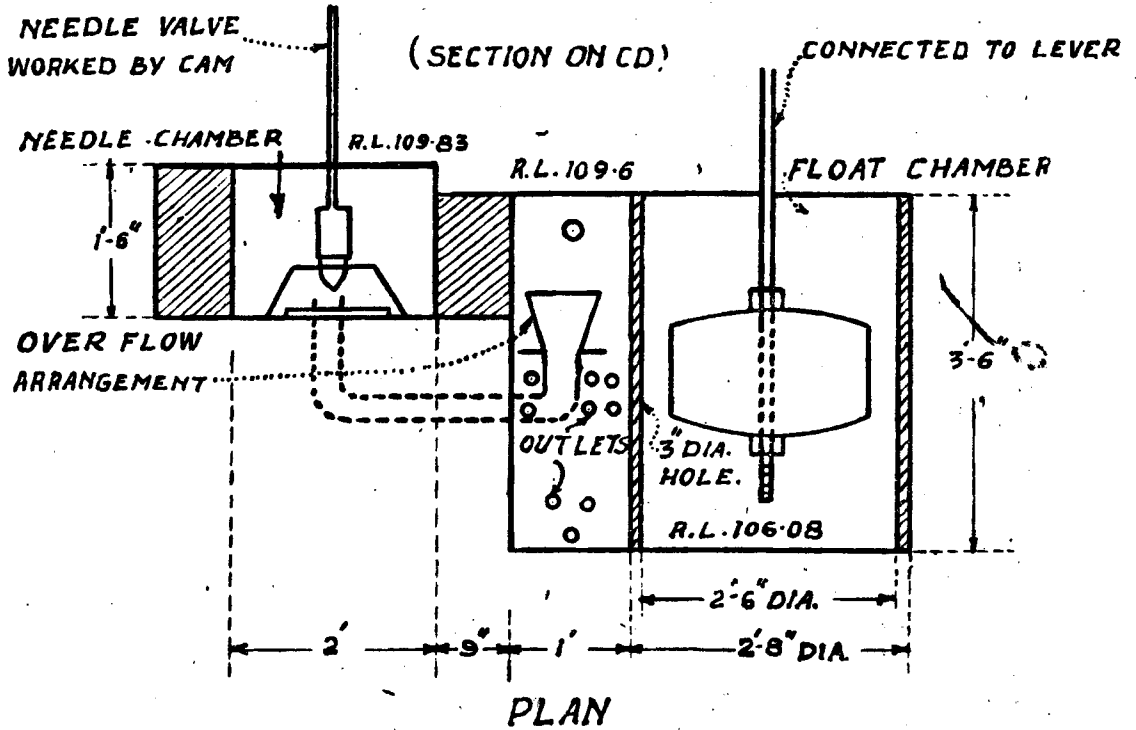


FIG 59 FLOAT CHAMBER AND NEEDLE VALVE CHAMBER.

NO. 7113
C.W.I. & N.R. STN
POONA.

VALVE CHAMBER.

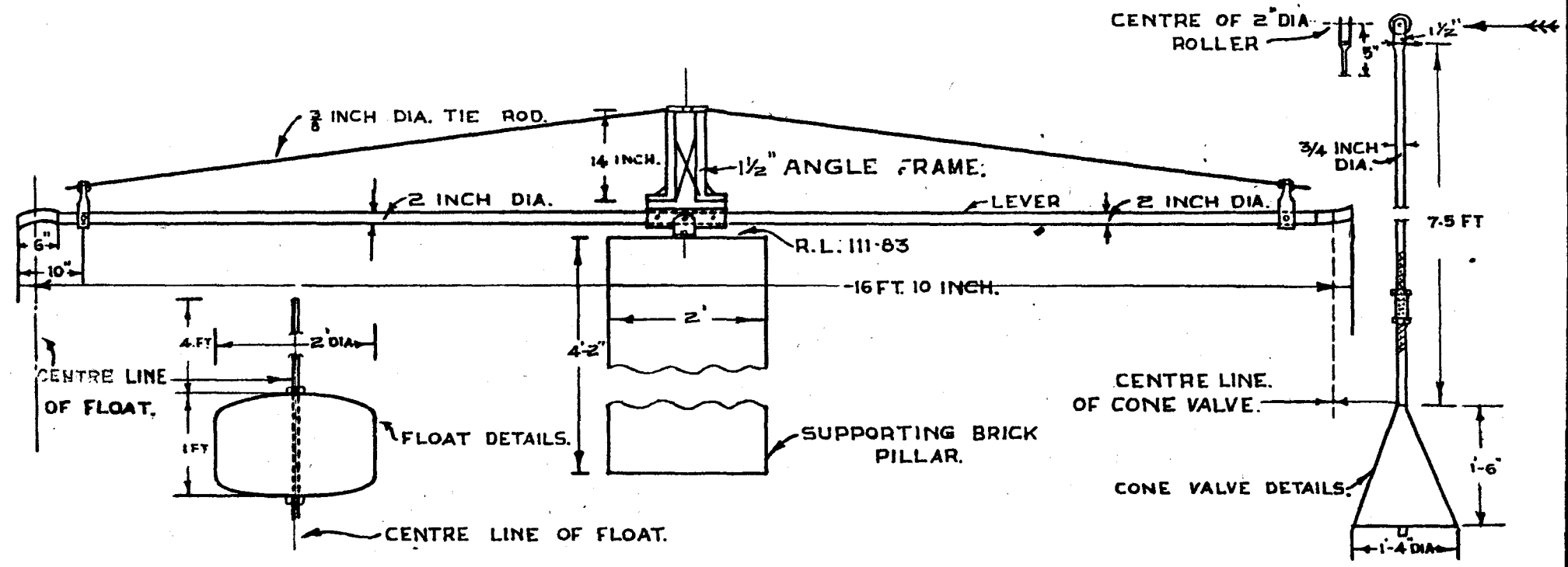
SCALE $\sim 1" = 2'-6"$



LEADING CHANNEL
TO D/S S.W.F.

FIG. 60
TIDE GENERATING APPARATUS
LEVER OPERATING THE CONE VALVE & FLOAT

SCALE: 1" = 30"



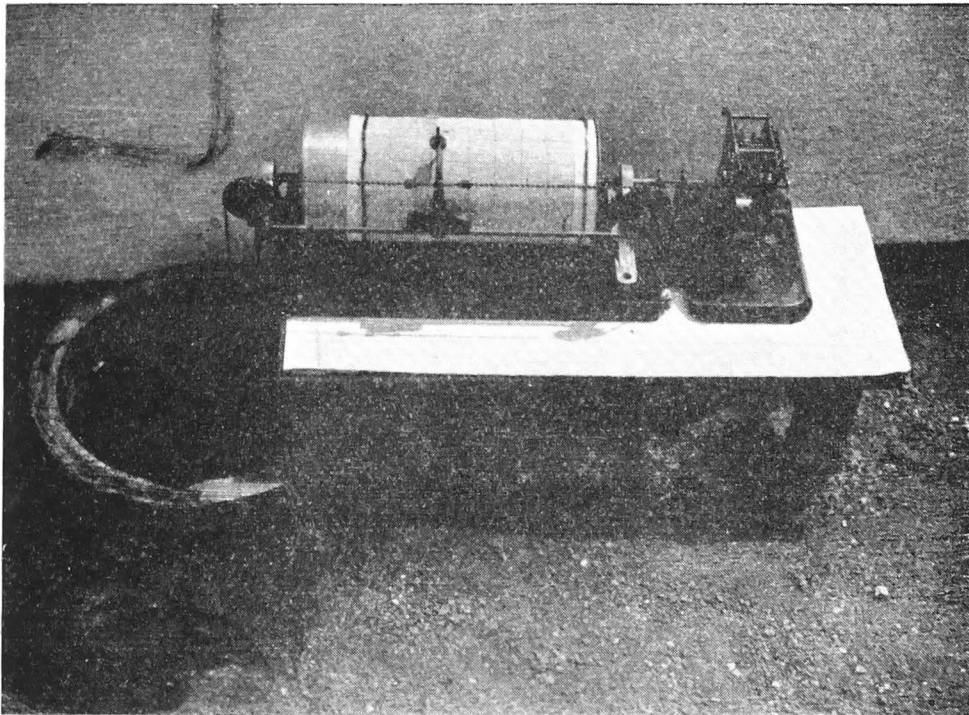
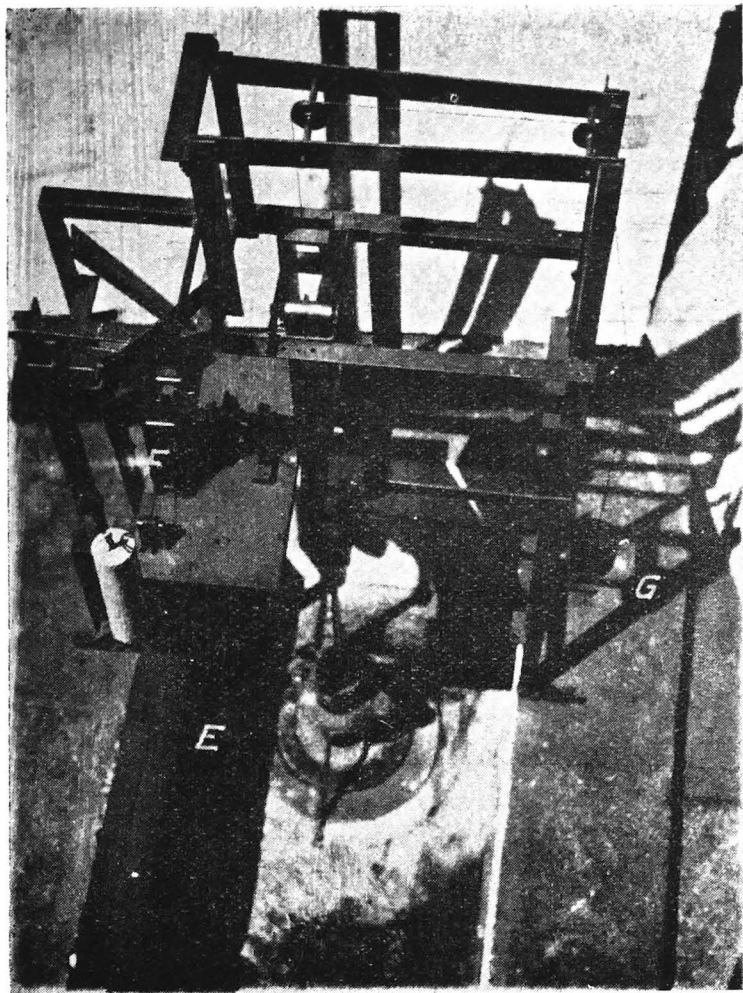


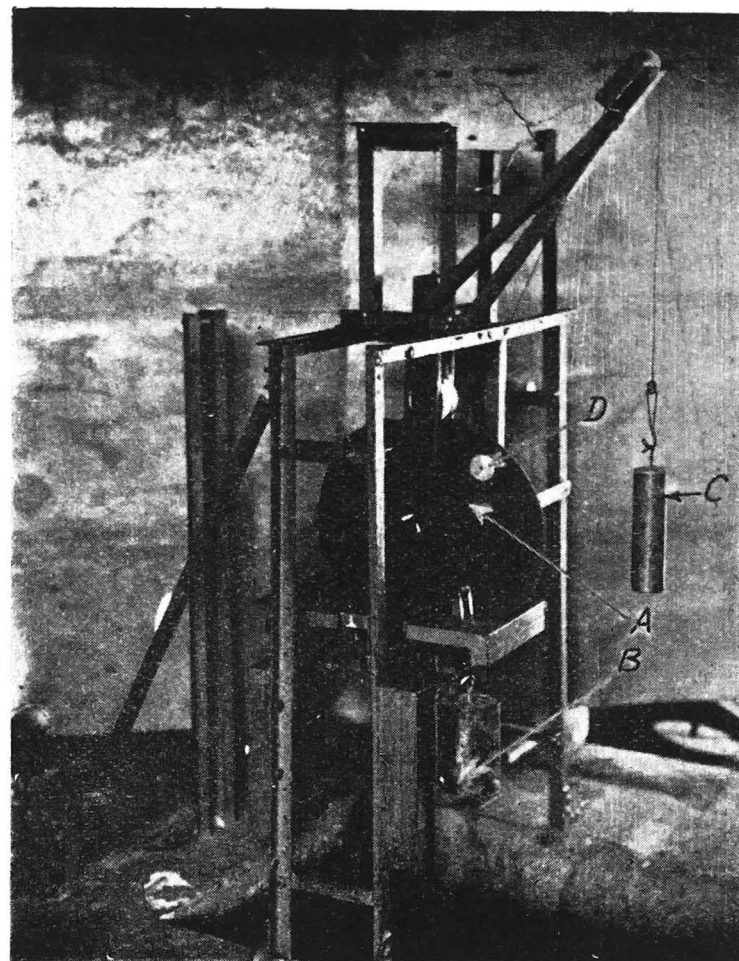
PHOTO No. 5164. Showing Recorder,
stylus and clockwork mechanism.

PHOTO No. 5157.



E—Needle valve.
F—Clockwork mechanism of the cam.
G—Counter-balance weight of the needle valve and the frame.

PHOTO No. 5149.



A—Cam.
B—Weight operating the clockwork mechanism.
C—Additional weight for driving cam.
D—Balancing weight of the cam.

PHOTO No. 5155.

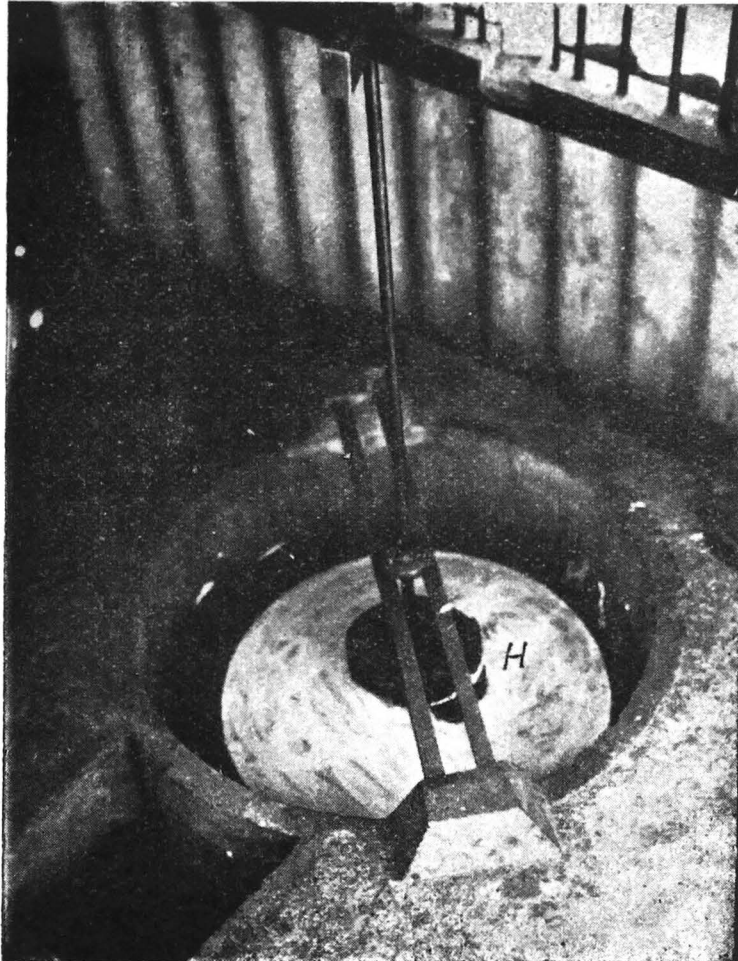
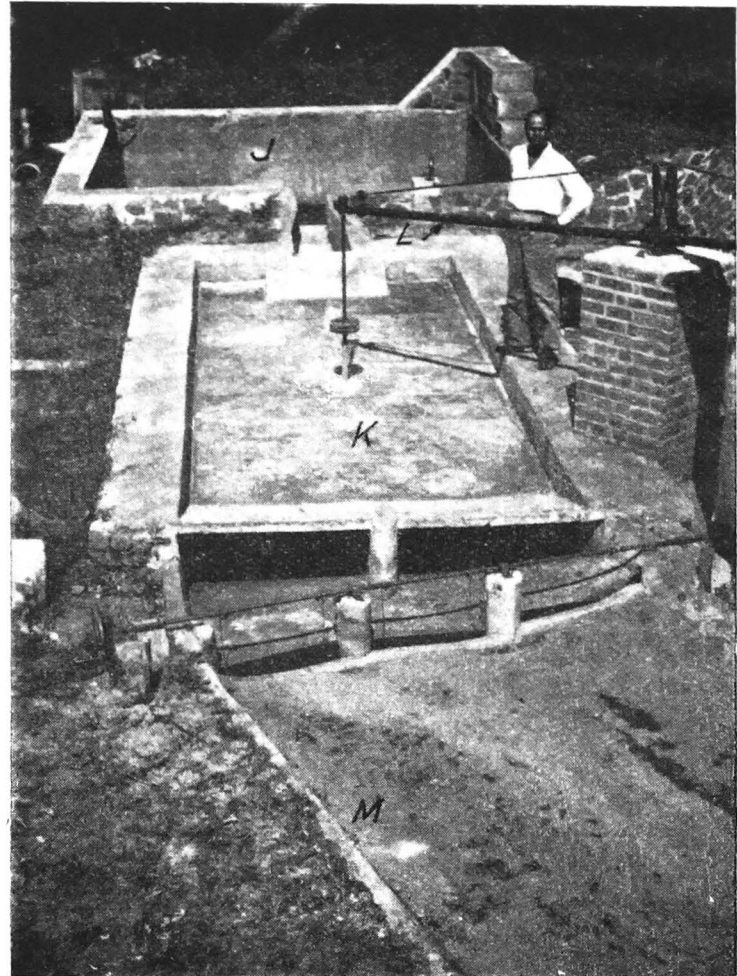


PHOTO No. 5156.



H—Float in the float chamber.
J—Downstream Chamber.
K—Upper Chamber covered with slabs.
L—Lever connecting cone valve.
M—Model.

Photo 5156 The cone valve moves within the two chambers shown
Fig 61. in Fig. 61. Water is let into the upper chamber from the
 downstream chamber representing sea water. When the
 cone valve moves in the downward direction the annular opening between
 the lower and upper chamber increases and allows water of the upper
 chamber to flow into the lower chamber and then into escape channel. This
 enables the model water to flow into the upper chamber and produce ebb
 tide.

When the cone valve moves in the upward direction and finally
 closes the opening, water in the upper chamber cannot escape and the
 additional discharge coming in from the down stream chamber finds its way
 in the model. Flood tide is thus reproduced. The upper chamber is particu-
 larly covered with slabs so as to force water in the model under pressure.

In brief the cam raises the needle valve, admits water in the to floa
 chamber, raises the float, lowers the cone valve, allows water to escape
 produces and ebb tide.

For the reproduction of flood tide the cam closes the needle valve,
 stops water going to the float chamber, lowers the float, raises the cone valve
 and does not allow water to escape to waste but finds its way into the model.

The float chamber is provided with outlets of varying
Fig. 59. diameter and at different depths to maintain the desired
 water level in the float chamber. They also serve as an
 additional control in shaping the cam.

3. *The clock-work mechanism of a cam.*

It is worked by weights, as against the usual method of helical
 springs, which drive the main toothed wheels and in turn move the subsidiary
 wheels connected to the main wheels. The gear ratio of the toothed wheels
 is so arranged as to get one complete rotation of the cam in the same time
 as required for a tidal curve in the model. The clock work mechanism of
 the recorder is quite separate from that of cam and both of them are well
 synchronised before starting the apparatus.

4. *Design of a cam.*

4.1. The cam is so designed as to get correct rise and fall of the
 needle valve. Rate of rise of needle has to take into consideration the time
 required for emptying the upper chamber with the various annular openings
 due to the cone valve movement. The rate of fall of the needle requires the
 consideration of time for filling the upper chamber.

4.2. Considering the number of variables viz. the opening of the
 needle valve, the variable discharge admitted into the float chamber, changing

head in the upper chamber, the variable annular opening due to cone valve and the friction in the needle frame and the lever, any mathematical approach to the design of cam is not possible.

4.3. To set a correct shape of a cam, therefore, a method of trial and error has to be resorted to. To start with the needle valve is manually operated to get the correct reproduction of tidal curve, which requires skilled operation and supervision with a perfect synchronising of all the recorders in the model.

Lift of the needle valve is measured at regular intervals of $\frac{1}{2}$ minute or 1 minute. On a circular disc the intervals are marked and the lift of the needle valve is plotted. The cam is then shaped accordingly.

4.4. The cam is fitted in position and tested under working conditions. If the correct reproduction of the tidal curve is not obtained either due to slight error in plotting or due to friction between the pulley and the cam, it is reshaped and tested. This procedure is repeated till correct reproduction of tidal curves is obtained. The clock-work must be sufficiently powerful to work the cam.

4.5. The cam is balanced by a balancing weight to reduce the jerky motion and ensure smooth working.

4.6. Fig. 62 shows the shape of the cam which produced a freshet spring tide curve at Cossipore. From the zero position of the cam to about 2 minutes the needle valve stops any discharge going into the float chamber.

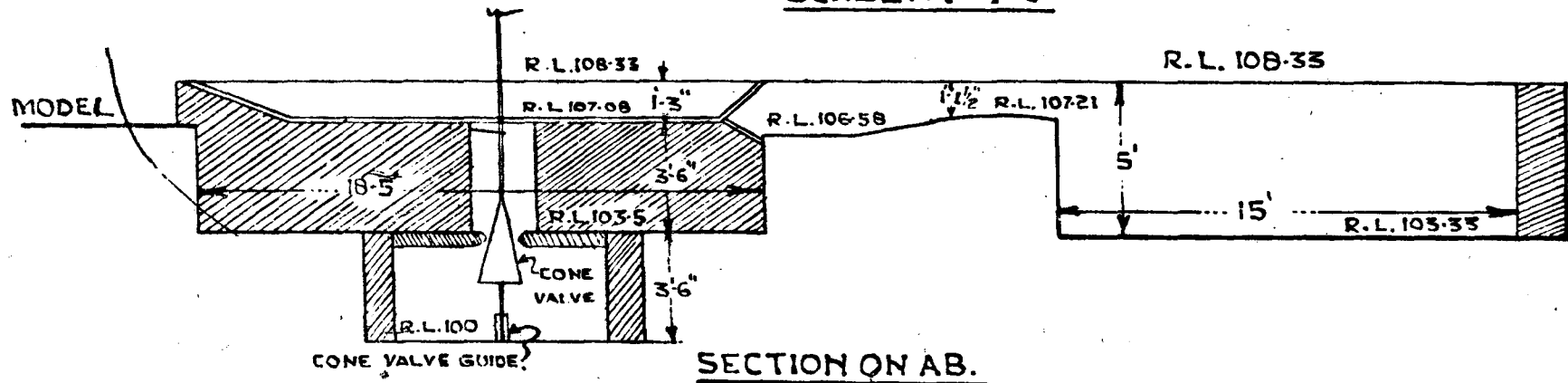
Fig. 62 The needle valve is then gradually lifted up to about 5 minutes when the water level in the float chamber has sufficiently been raised to lower the cone valve to such an extent as would enable the upper chamber to be emptied to admit the ebb discharge of the model. It would thus be clear that even though the needle valve is lifted after two minutes the flood tide continues for 5 minutes and the gradual lift of the needle valve from 2 minutes to 5 minutes allows for the filling of the float chamber to raise the float and to lower the cone valve so as to empty the upper chamber. From 5 minutes to about 15 minutes the needle valve is in a raised position, the float is raised and the cone valve is lowered to get the ebb tide curve correctly reproduced. After 15 minutes, the needle valve is slowly lowered thus raising the cone valve, reducing the annular opening and slowly ponding up water in the upper chambers for the flood tide to follow.

4.7. In the present arrangement the cam reproduces the same tidal curve over and over again whereas in nature all the tidal curves are different from one another. Improvement of this arrangement to simulate nature is under consideration in which case different cams for different tidal curves would come below the pulley in succession.

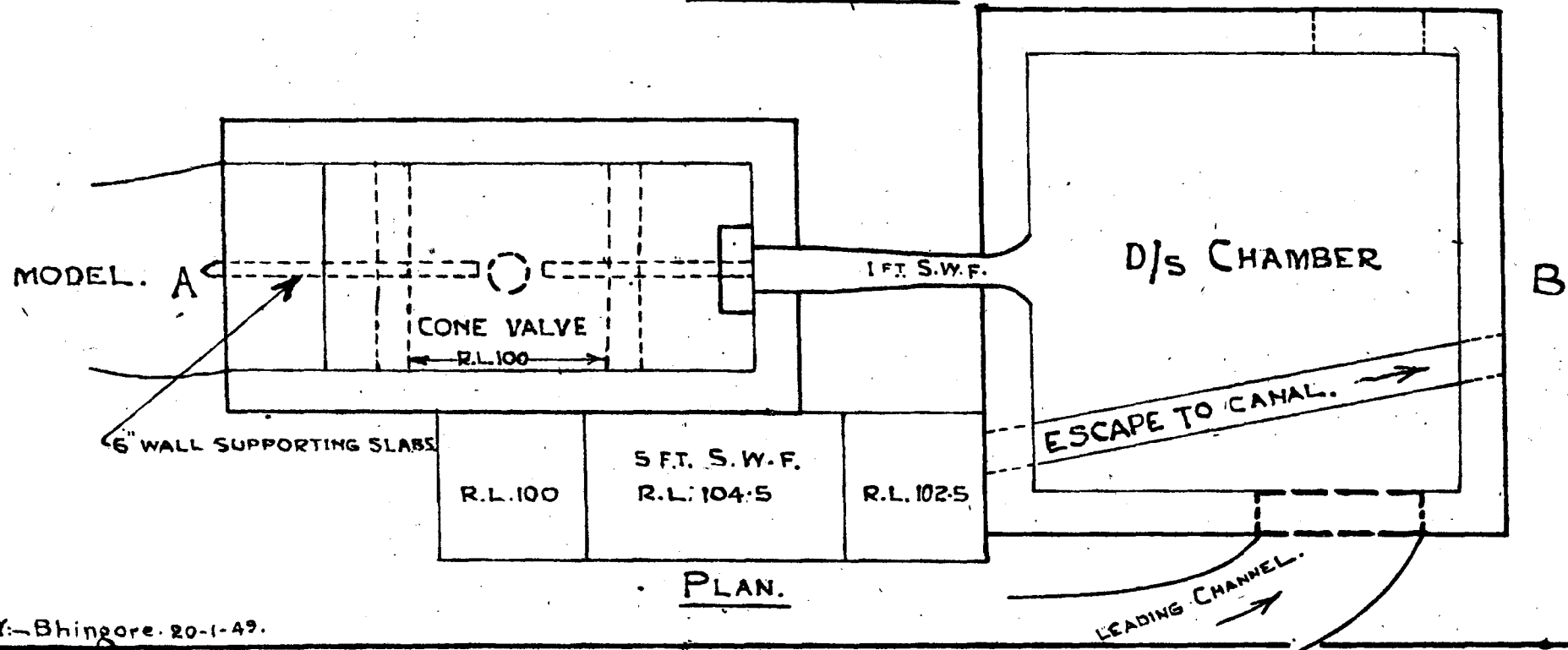
FIG. 61 CONE VALVE CHAMBER

NO. 7122
C.W.I.&N.R. STATION.
POONA.

SCALE: 1" = 7'-6"



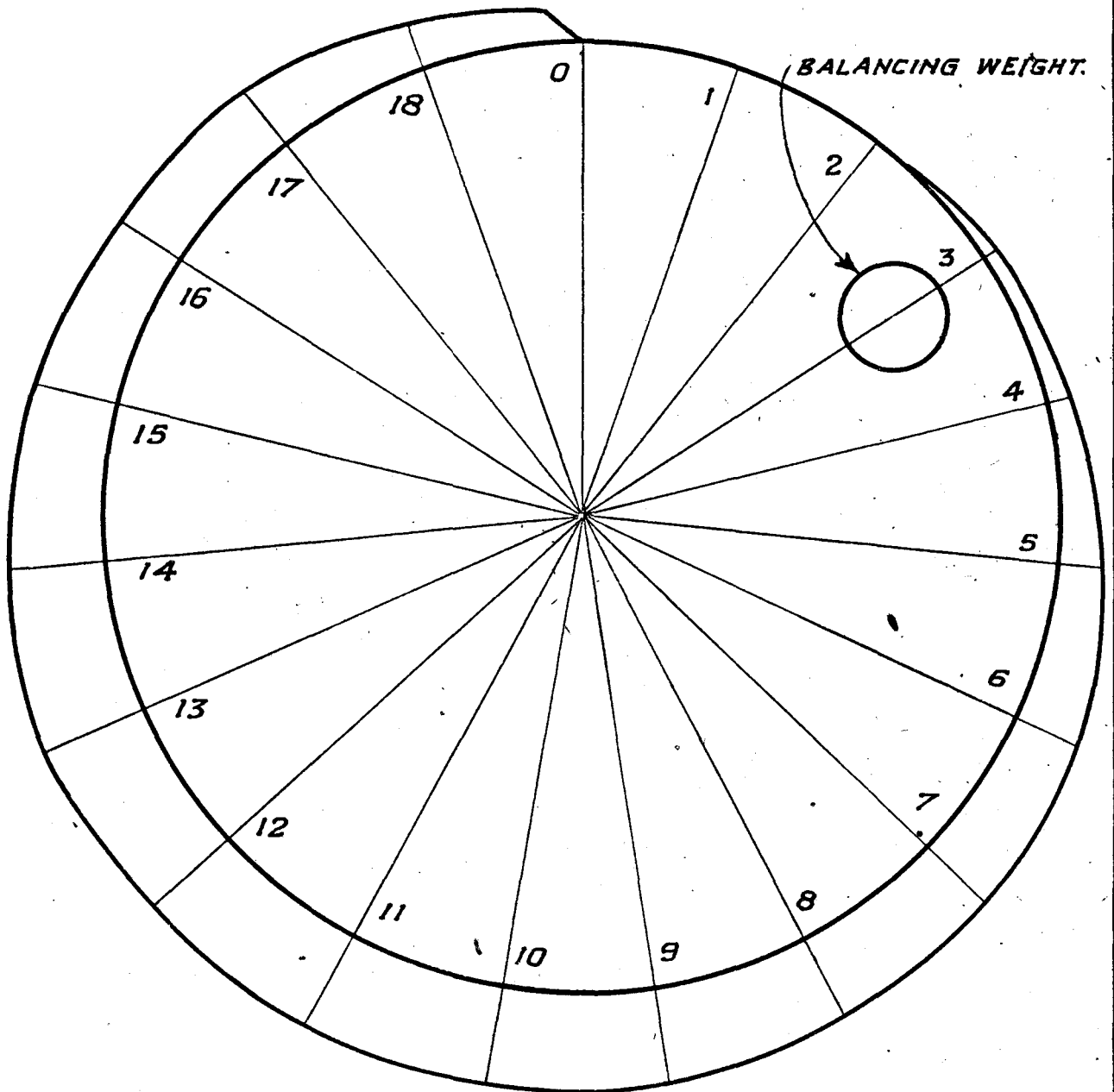
SECTION ON AB.



PLAN.

BY: Bhingore. 20-1-49.

FIG. 62
TIDE GENERATING APPARATUS
CAM, PRODUCING FRESHET SPRING TIDE
AT COSSIPORE
SCALE = 1/3 SIZE.



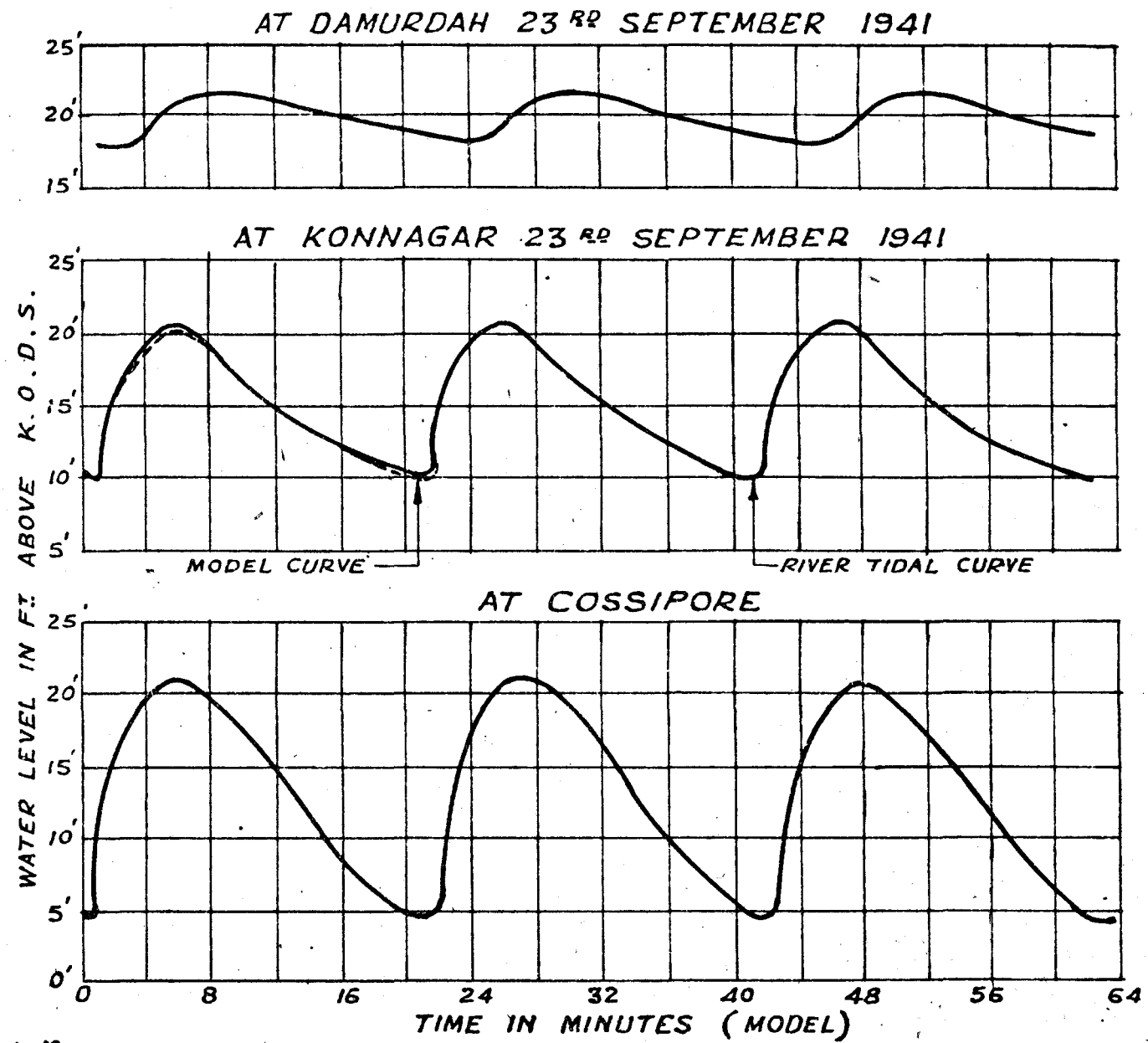


FIG. 63
HOOGHLY RIVER MODEL
TIDAL CURVES AT
DAMURDAH, KONNAGAR
AND COSSIPORE.

5. Hooghly river model.

5.1. The model included a length of 33 miles from Damurdah to Cossipore. Length scale of the model was $1/300$ and the depth scale = $1/60$ thus having a distortion of 5. Experiments with the model have been described in Annual Reports for the year 1940-41, 1941-42, 1942-43, 1943, 1944, 1946, and 1947.

5.2. The tide generating apparatus is installed at the Cossipore end of the model. Water levels at Damurdah end are controlled by a sliding Cippoletti weir. In the river the flood tide is absorbed by a certain length of the river depending on the intensity of discharge. In the model, however, as the whole tidal compartment is not reproduced, the surplus discharge is allowed to escape by a sliding Cippoletti weir. During ebb tide, in the river, the absorbed discharge returns back to sea along with the river discharge. In the model, therefore, to make up the escaped discharge during flood tide, water is fed in at the Damurdah end by raising the Cippoletti weir. During the flood tide and ebb tide the weir is lowered and raised respectively, to get the correct reproduction of tidal curve at Damurdah.

5.3. Fig. 63 shows the tidal curves at Cossipore, Konnagar and Damurdah. Tidal curve at Cossipore end was obtained to set the correct reproduction of the curve at Konnagar and was only slightly different from the tidal curve in the river as shown in Fig. 63.

23. Wave Recording Apparatus.

1. Introduction.

1.1. New problems, such as Madras and Cochin Harbours, Mithapur Intake, in which waves created by wind in sea, are one of the major natural forces involved, are being investigated at the Station. Model studies of these require accurate reproduction and recording of wave characteristics such as period, length and shape. An apparatus is, therefore, being developed to observe the above mentioned features of waves produced in models.

2. Outline of the apparatus.

2.1. If two wires are dipped in water and constant potential is applied, variation in current will occur with change in depth of immersion of wires; by placing a number of such units (hereafter called resistance units) in a line at right angles to the wave front and recording simultaneously variations in currents, for a short time, all pertinent information such as length, shape and period of wave will become available.

Fig. 64(a).

3. Investigations.

3.1. *Capillary effect.* It is desirable that the wires of the resistance unit should be as thin and as close as possible for least distortion of waves to be recorded. This distance, however, cannot be made indefinitely small as capillary film forms between the wires, if placed too close, and rapid variations in water levels are not faithfully followed,

With two nichrome wires, each 0.072" dia., it was observed that the optimum distance was 0.06". It might be possible to reduce this using thinner wires.

3.2. *Polarisation.* When the resistance unit was dipped in water (Khadak wasla lake water, Temp. 24.5°C.) to 0.4 ft. depth and connected to a 6 volt battery, 20 milliamps current passed through the unit. Polarisation occurred and bubbles formed on both the wires when current was passed for about 10 seconds. Due to this, value of current for same immersion in water varied which was undesirable for accurate interpretation of depths from values of currents.

Experiments showed that a maximum current of 3.0 milliamps. could be passed, through the resistance unit described, without causing any noticeable bubbles on the wires. There was, practically, no decrease in the value of the current even when passed for about 5 minutes.

Photo 5415. Maximum current for further experiments, was therefore limited to 3.0 milliamps by interposing a potentiometer in the circuit and applying reduced voltage across the unit.

3.3. *Linear relation.* To obtain directly, wave height and wave form from records of current variations, it is important that the relation between the depth of immersion and current should be linear.

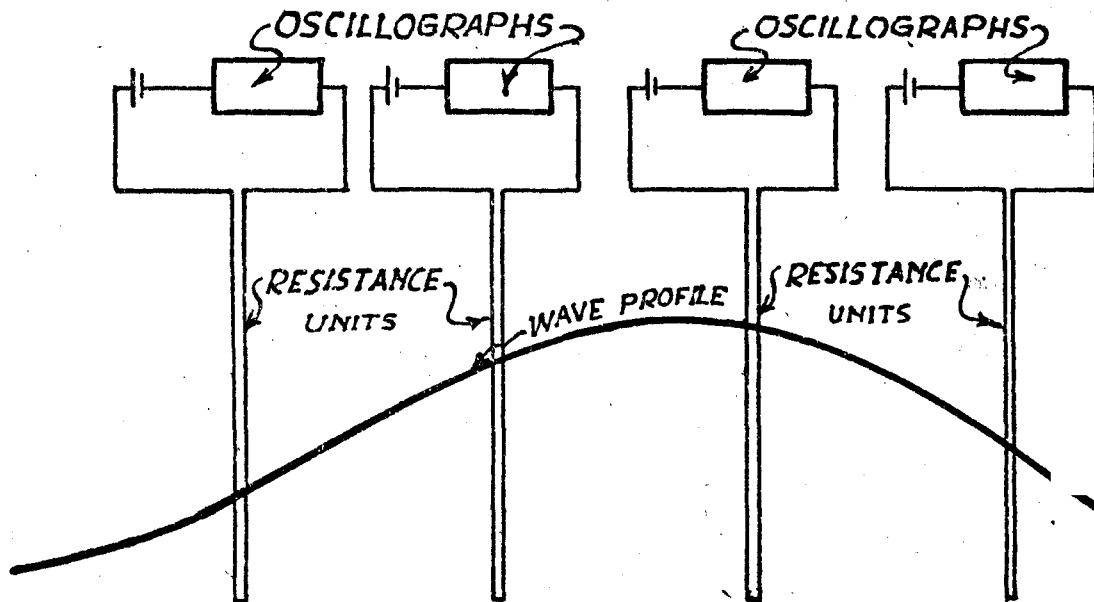
In the first instance, the resistance unit with immersed ends of two wires kept open was tested. Variation in current is plotted against depth of immersion in water in Fig. 65(a) (dotted line). It will be seen that the points practically lie on a straight line for depths from 0.1 ft. to 0.4 ft. For depths below 0.1 ft. this relation no longer held. This was due to the fact that leakage paths at the ends considerably affected the resistance at low depths.

To reduce this effect, the ends were sealed by a piece of ebonite and wax as shown in photograph 5409 and Fig. 65(c). This improvement gave a linear relation throughout the range of observation i.e. 0.01 ft. 0.4 ft. depths.

The actual apparatus to record varying currents simultaneously on a common paper camera will be more elaborate. Messrs. Cambridge Instrument Co., 13, Crossvenor Place, London S.W.1, have been approached

FIG. 64
WAVE RECORDING APPARATUS

(a)



(b)

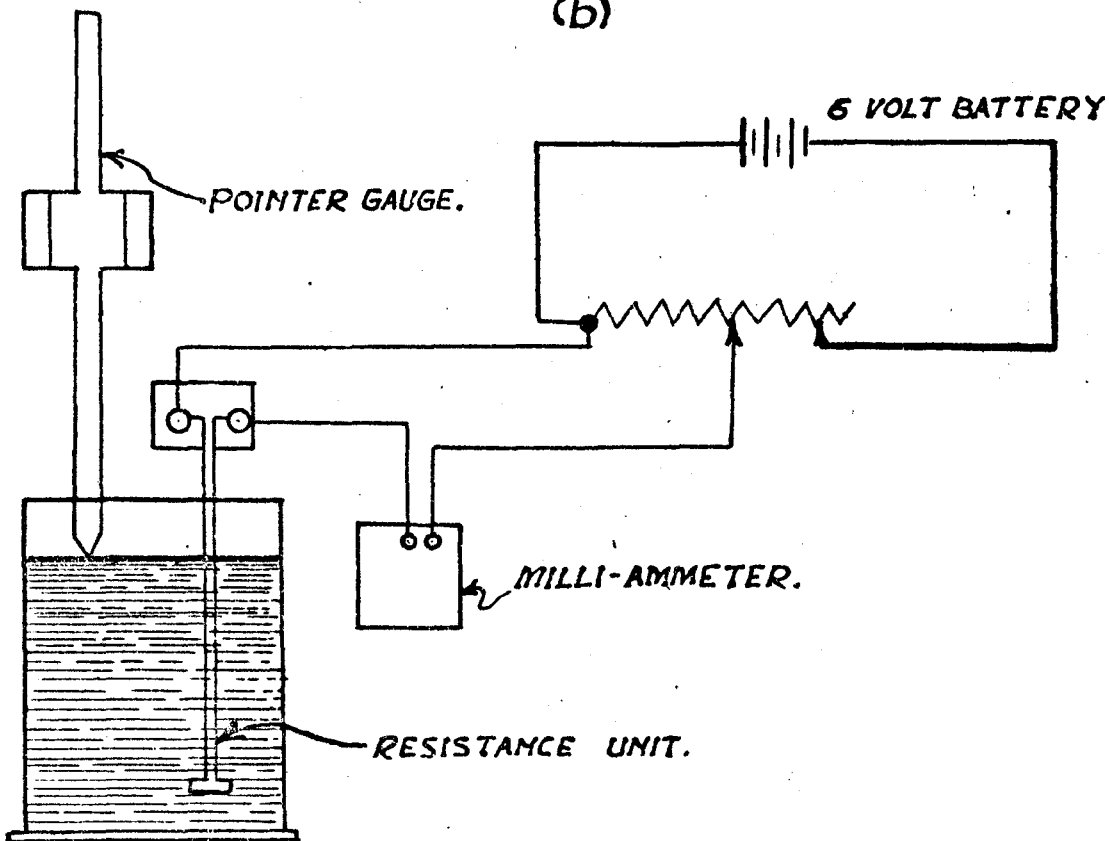
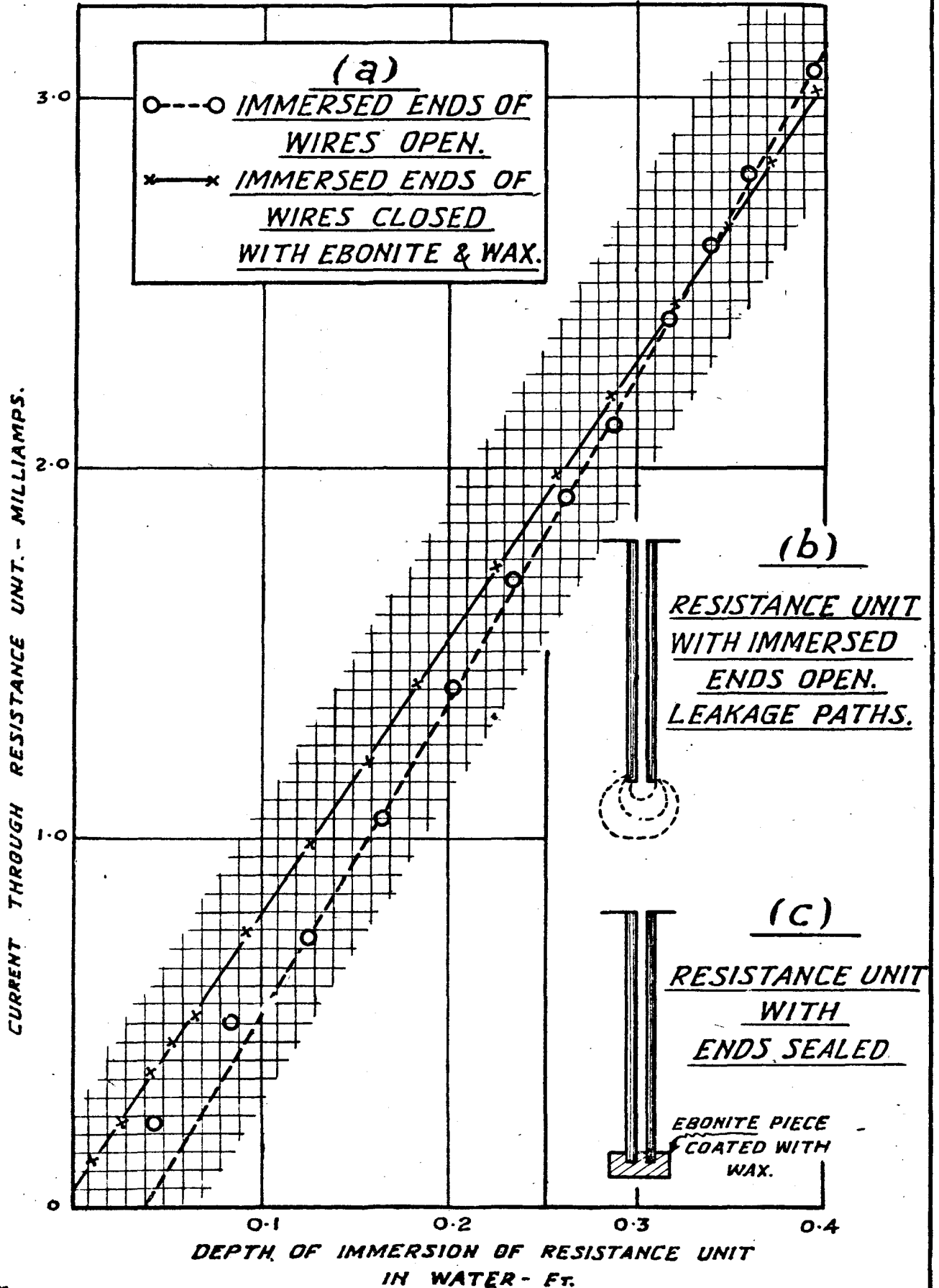


FIG. 65 (a) TO (c)
WAVE RECORDING APPARATUS.



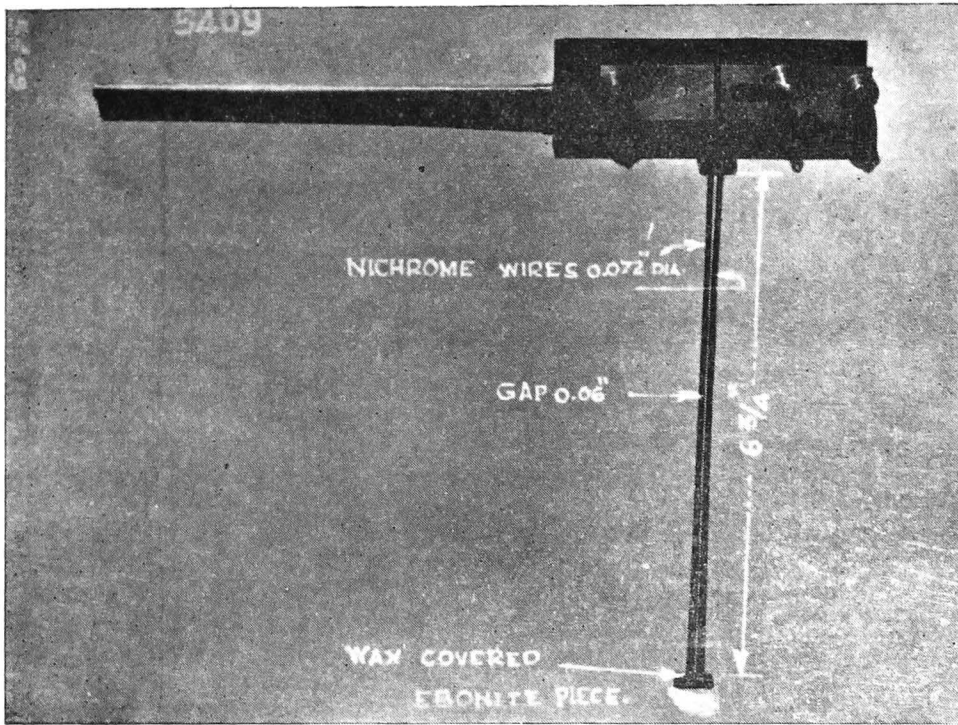


PHOTO No. 5409. Resistance unit of Wave Recording Apparatus.

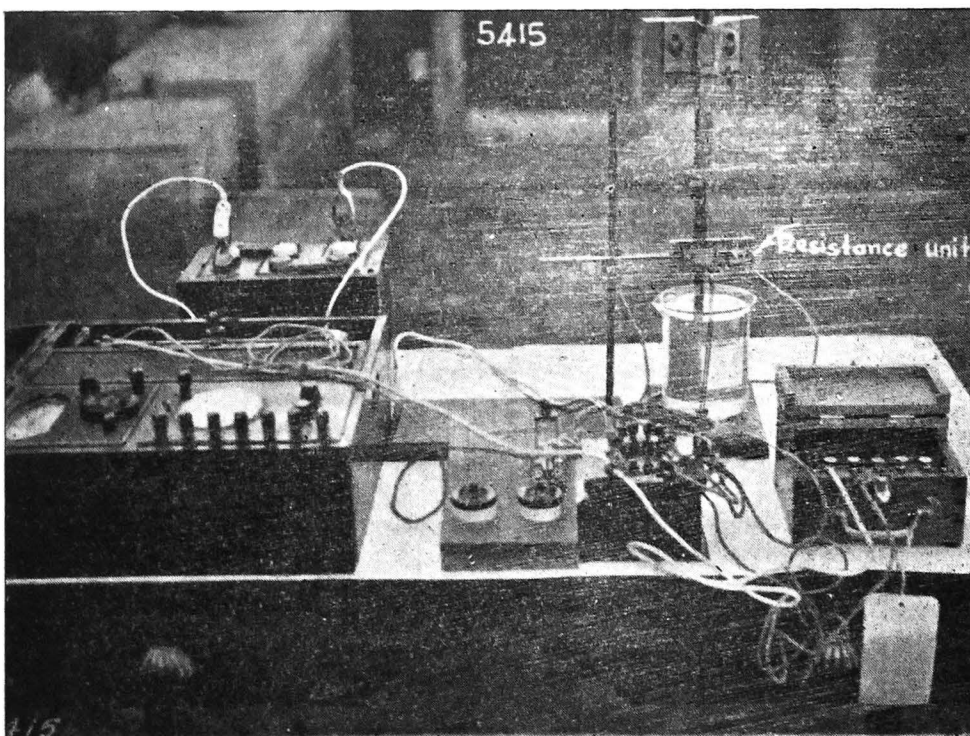


PHOTO No. 5415. Testing of Resistance Unit for Current and depth of immersion relation,

for a suitable 'Low Frequency Oscillograph' Messrs. Poona Laboratory Apparatus Works are also trying to develop a mirror type galvanometer of low resistance and sufficiently quick period to record the varying phenomenon and it is hoped to evolve a working model in the near future.

24. Proposed design of Lower Sind Barrage at Kotri for effective sand exclusion from Canals, Sind.

1. Introduction.

1.1. The experiments reported in Annual Report (Technical) 1947 (p. 116) covered the following salient features of the design of the barrage.

- (a) Sills of head regulators;
- (b) design of the left and right pocket and the optimum number of spans;
- (c) long pocket versus short pocket;
- (d) design of guide banks;
- (e) location of the left abutment;
- (f) left and right pocket divide walls;
- (g) river conditions after the construction of the barrage;
- (h) regulation of the barrage and the necessity or otherwise of keeping pocket gates higher than barrage gates; and
- (i) necessity or otherwise of paving the bed of the pockets.

1.2. This year's work (upto May 1948) mainly covers the following points:—

- (i) Testing the applicability of the scour factors for the left and right guide banks and the central island as proposed in the design;
- (ii) reorientation of the alignment of the right arm of the barrage;
- (iii) testing 'gulleting' on the flat side slope of the central island;
- (iv) tests to see whether the modified alignments of the right guide bank moved, sufficiently river-wards, would help to offset the apprehended inordinate shoaling near right bank; and
- (v) tests for standing the suitability of the alternative design of a single barrage at or upstream of the site finally adopted for the double barrage design.

All the experiments, except for (iii) above, were carried out in 1/250:1/50 vertically-exaggerated model; the experiments for 'gulleting' however were carried out in 1/10) geometrically similar model.

1.3. After the visit of the Chief Engineer, Lower Sind Barrage, to the Station in December 1947, the above mentioned additional experiments were carried up to May 1948 and are-described in this year's Report. Owing to the shortage of space and staff and other urgent experiments on the waiting list, these experiments were discontinued after May 1948. with the approval of the Chairman, Central Waterpower Irrigation and Navigation Commission. Though the problem, unhappily, belongs to other dominion and is not of specific interest is given for information and guidance of designers dealing with an ancillary type of work.

2. *Testing the applicability of scour factors for the left and right guide banks and the Central Island.*

2.1. *Left guide bank.* Appendix I shows the reduced levels of deepest scour obtained in the model and the scour factor recommended as compared to the R.L. of deepest scour expected in the design.

When high flood discharge $\cong 7,50,000$ cusecs was run in the model, it was observed that scour along the left guide bank—although it is on the concave side of the curve—was not severe and it appeared that a scour factor of 1.5 R provided in the design was rather on the high side. The level of bottom of deepest scour factor of 1.25 should be sufficient all along the left guide bank.

The reasons for this lower scour than expected in design, along the left guide bank, are that—as evinced by the the model—(a) the flow concentrates in the middle of the river on account of the attraction by the Central Island; and (b) the high river discharges tend to flow straighter, following a course of flatter curvature.

In spite of the pronounced curvature on the left bank, upstream of the Sukker Barrage, berms have formed along that bank; conditions at Kotri being somewhat similar, there is further reason to believe that, a scour factor of 1.25 R should be sufficient.

2.2. *Right guide bank.*

The reach A B (fig. 66) of the right guide bank was subject to severe attack. The bottom level of deepest scour observed in this reach was R.L. 12.0, showing that the factor of 1.5 R provided in the Sind Note was necessary. The flow in the reach B C was, as anticipated, very slack and there was little scour observed below the existing bed in this reach and so the scour factor of R, adopted in the Sind Note, might be retained.

Fig. 66.

The level of the bottom of deepest scour observed in the models in the reach C D (i. e. upstream of the Kalri Baghar Feeder) was R. L. 35.0. Scour factor R_2 was reasonable.

2.3. *Nose of the Central Island.*

Due to the flat side slope of 1 in 6 proposed for the nose of the Central Island, it was observed that the deepest scour was not lower than = R.L. 10.0, and hence a scour factor of 2 R (giving bottom of scour at R. L. 16.90) appeared a little excessive; the scour factor might be reduced to 1.75 R. This would also counterbalance, to some extent, the extra cost of the island arising from the flatter side slope recommended for the nose of the island. The levels of bottom of deepest scour observed for the left and right shanks of the Central Island were R. L. 10.0 and 25.0 respectively.

For the same reasons the scour factors of 1.75 R_1 and 2 R_2 Appendix I — justified by the model — might be adopted for these shanks, respectively. (see Appendix I)

2.4. The Sind note provided for an apron, assuming a uniform launching slope of 3:1 (3 horizontal : 1 vertical) for maximum scour. This was not correct. The slopes of launched apron depend upon the varying severity of attack, as shown by the model. It was recommended that the apron should be designed to launch to a slope of 3:1 for the Central Island and reach A B of the right guide bank, 3.5 : 1 for the whole of the left bank and reach C D of the right guide bank, and 2:1 for the reach B C of the right guide bank.

(ii) *Alignment of the right arm of the Barrage—*

With a view to making the flow central in the right arm channel, downstream of the barrage, and to reduce the curvature of the Kalri Baghar Feeder just downstream of the Head Regulator, it was proposed to reorient only the right arm of the barrage anti-clockwise.

3.1. Experiments were accordingly carried out with the right arm of the barrage reoriented, anti-clockwise by 7° i. e. the magnetic bearing of the right arm of the barrage was made $276^\circ - 30'$, while that of the left arm of the barrage was kept the same as before, viz. $283^\circ - 30'$.

With the barrage in one line, it was observed that the flow after leaving the right arm of the barrage impinged on the right bank at c.s. mile 2.5 and then was deflected away from the bank. With the altered orientation, the flow was more central and followed better the existing curvature of the right bank.

After running four consecutive rising and falling cycles and exaggerating the silting near the right guide bank,—to compensate for the model

limitation in respect of shoaling — lines of flow were Figs 66 & 67; observed from c. s. mile 5.5 and c. s. mile 7.5; the exclusion of sand was found to be as good as obtained with the previous alignment of the right arm.

It was also observed that no change in the alignments either of the right guide bank or of the central island was necessary.

4. (iii) *Experiments to test "gulleting" on the flat side slopes of the Central Island in 1/100 scale geometrically similar model.*

A very flat side slope of 1 in 6 has been recommended for the nose and a part of the right arm of the Central Island, partly with a view to preclude the bed sand being thrown into suspension by turbulence at the nose by differential depths and extent of scour on the left and right arms of the Central Island.

As "gulleting" is a phenomenon observed to occur in cases of flat side slopes — the reproduction of which is only possible in a geometrically-similar model — experiments to assess the likelihood of gulleting were carried out in 1/100 scale geometrically-similar model; the scale selected was large enough and suitable from the point of view of availability of discharge and space.

The model was laid in Koregaon sand ($m = 0.3$ mm.) according to depths, obtained in the 1/250 : 1/50 scale vertically-exaggerated model for $Q \cong 5,00,000$ cusecs. Correct velocity distribution, as observed in 1/20 : 1/50 model, was enforced by slightly moulding the model reach between c. s. miles 6 and $5\frac{1}{2}$ (See figs. 68 & 69).

The side slope of the Central island was pitched with $\frac{3}{16}$ " dia. gravel (1) equivalent to "one man stone".

Being a geometrically-similar model, it was initially laid with a bed slope of 0.1 ft. per 1000, i. e. same as the slope in the prototype. It was interesting to note that after the bed slope exaggeration as well as surface slope exaggeration observed varied from 4.50 to 5.20.

Thus, while laying a geometrically-similar model, precaution has to be taken to lay it to a slope steeper than the prototype slope as proportionally greater frictional resistance has to be overcome in such a model. Assuming a value of $N = 0.035$ in the Manning's formula

$V = \frac{1.486}{N} R^{2/3} S^{1/2}$, V & R being calculated for $Q \cong 5,00,000$ cusecs after moulding the cross section at mile $5\frac{1}{2}$ so as to have correct velocity distri-

(1) Annual Report (Technical) 1839-40 Poona. Pages 50, 51.

FIG. 66 & 67
PLAN SHOWING LINES OF FLOW
FOR Q = 200,000 Cs. AFTER ALTERING
THE RIGHT ARM OF THE BARRAGE AT KOTRI
WITH SILTING ALONG THE RIGHT GUIDE BANK

SCALE 1" = 7.5' = 1875'

REF.
SURFACE WATER —————
BED WATER ·······

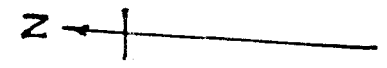


FIG. 66. OBSERVED FROM C.S. MILE 5.5

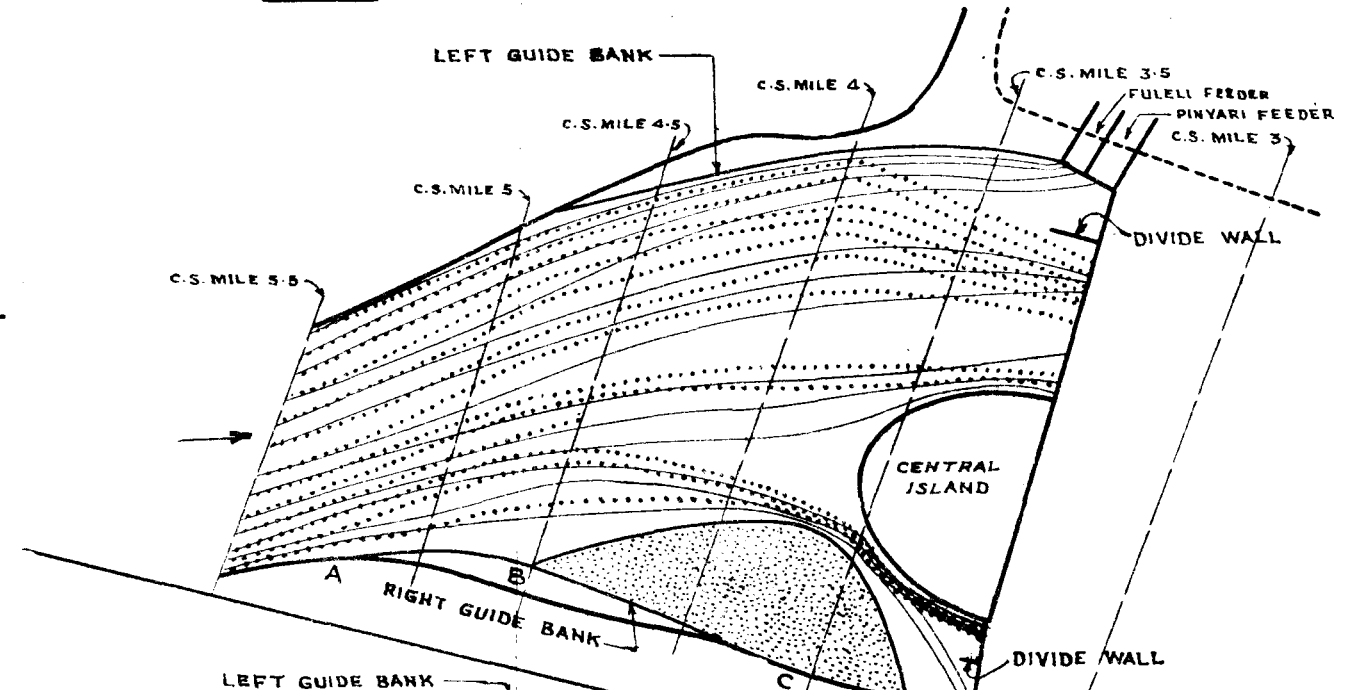


FIG. 67 OBSERVED FROM C.S. MILE 7.5

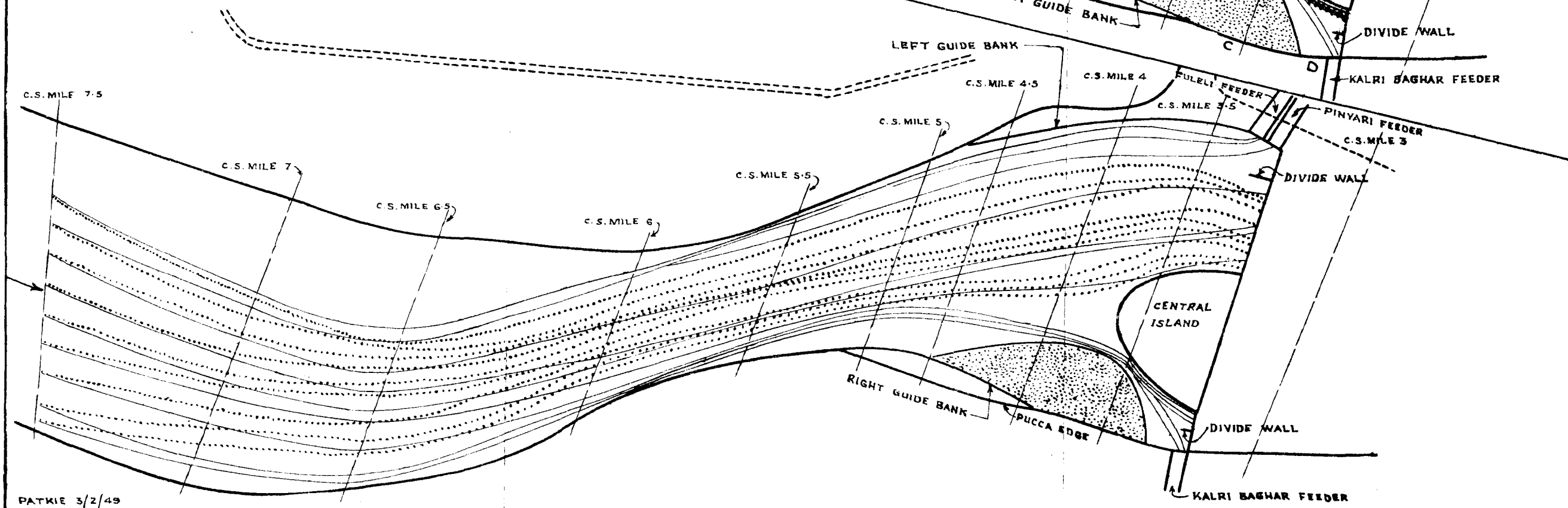


FIG. 68
SHOWING VELOCITY DISTRIBUTION
AT C.S. MILES 6 & 5½ IN 1/250:1/50 & 1/100
G.S. MODEL FOR Q ≅ 500,000 Cs.

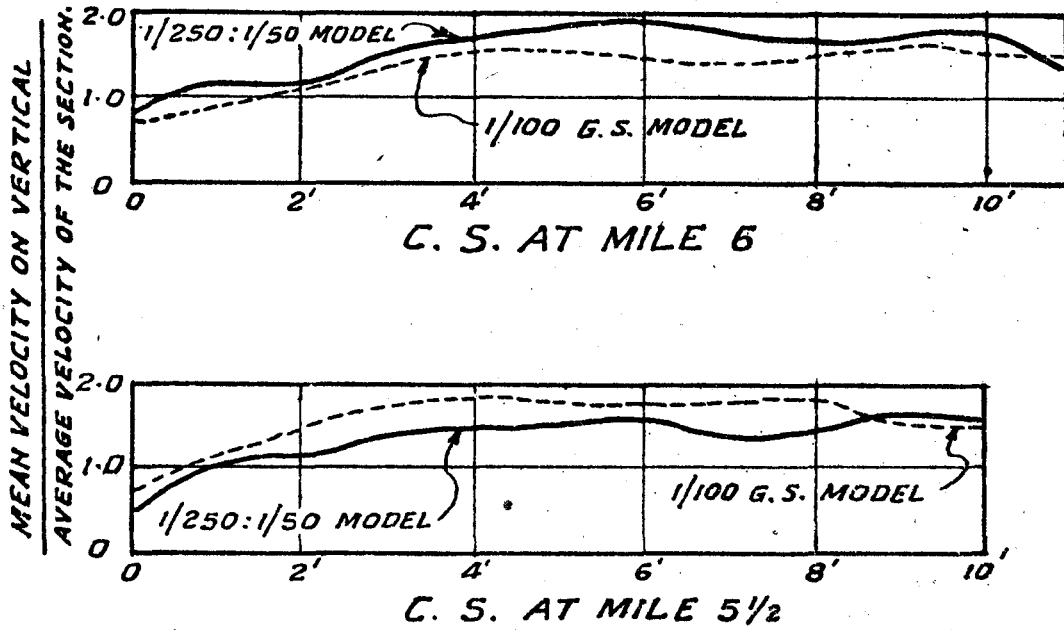
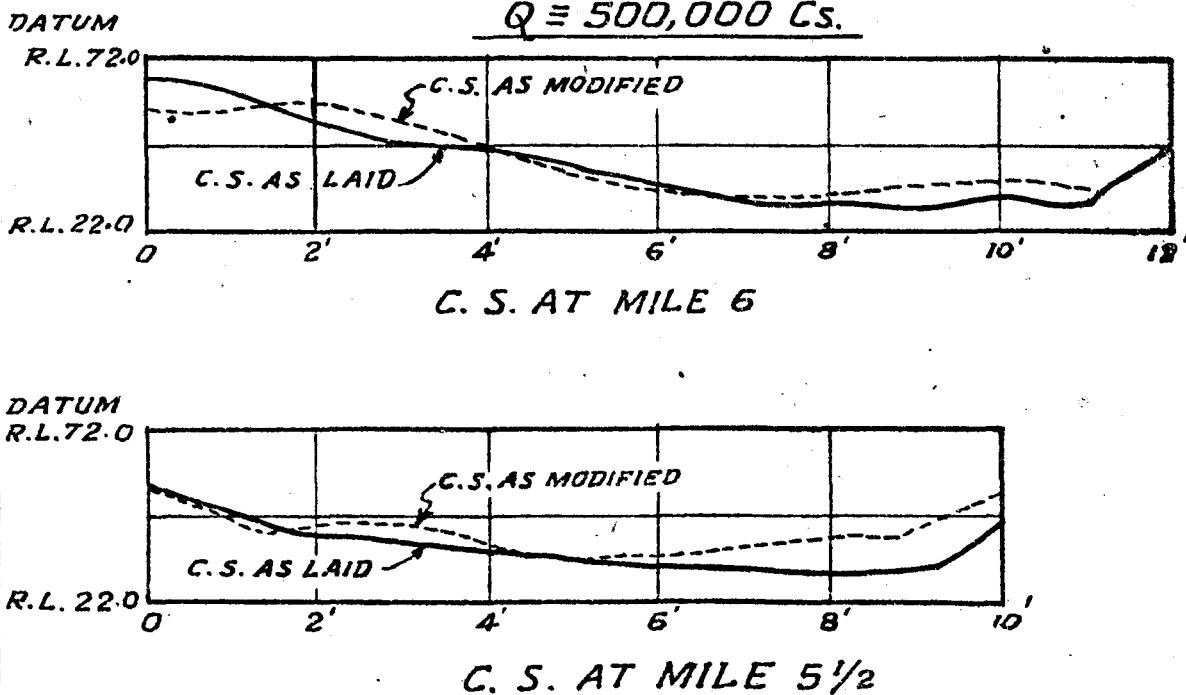


FIG. 69
SHOWING CROSS SECTIONS MILES 6 & 5½
AS LAID & AS MODIFIED FOR
Q ≅ 500,000 Cs.



bution slope works out 0.42 ft. per 1000 which shows very little divergence from the experimental results.

Discharge was gradually increased from $Q \equiv 5,00,000$ cusecs to $Q \equiv 750,000$ cusecs, allowing sufficient time for the bed to stabilise and adjust itself.

Little or no "gulleting" was observed either at the nose or the right arm of the Central Island with this size of stone.

Though no "gulleting" was observed, the pitching of the Central Island in the vertically-exaggerated model had been frequently washed out and there were indications of rather severe action.

In view of this, it is necessary to pitch the island with 'two-men stone' (each stone 150-200lbs.) in the region of likely "gulleting" and to have adequate apron protection as shown in fig. 70.

Fig. 70.

5. (iv) *Experiments to see whether the right guide bank could be moved riverwards.*

5.1. In answer to the criticism of likely inordinate shoaling against the right guide bank of the Double Barrage design, more detailed experiments were carried out with modified alignments of the guide bank moved sufficiently riverwards. Alignments tested in the model are shown in fig. 71.

Fig. 71.

5.2. As borings have disclosed the presence of clay at a substantially higher level in the right bye-river, the bed of the model of the bye-river (upstream of the barrage) was moulded in inerodible clay below R. L. 43.50 and all the experiments described below were carried out with this factor unchanged. This also enabled comparison of the results with those obtained previously with a wholly erodible bed.

One rising and falling cycle up to $Q \equiv 5,00,000$ cusecs was run with each of these guide bank alignments under still pond conditions, with left and right arm barrage gates opened equally. Figures 72 &

Figs. 72 & 73. 73 show the contours observed in the model with alignments numbers 1 and 3. Table 1 shows the afflux between the left and right arm barrages and the discharge entering the right arm channel upstream of the barrage.

Figure 74 shows the variation of the intensity of discharge observed along the cross sections 2500 ft., 4000ft. and 5500 ft. parallel with guide bank alignments numbers 1 and 3.

Fig. 74.

Table 1.—Showing the afflux the left and right arm barrage and discharge tapped by the right arm channel with right guide bank alignment numbers.

Discharge in cusecs	Right Guide Bank No. 1 (Station Design)		Right Guide Bank No. 2 (Intermediate)		Right Guide Bank No. 3 (Moved Riverwards)		Remarks
	Afflux feet	Discharge cusecs	Afflux feet	Discharge cusecs.	Afflux feet	Discharge cusecs.	
200,000	0.70	31,000	.80	30,000	1.00	27,800	The barrage was worked under still pond up to $Q \equiv 500,000$ cusecs. There was no enforced regulation between the left and right arm barrages.
250,000	0.70	38,000	.80	35,000	1.00	34,700	
300,000	0.65	47,100	.75	39,000	0.85	39,100	
350,000	0.90	57,800	0.90	55,300	1.05	40,000	
400,000	0.80	69,600	1.00	61,400	1.45	55,900	
450,000	0.65	78,400	1.20	73,400	1.50	62,700	
500,000	0.65	85,400	1.20	76,800	1.50	71,600	

FIG. 70

PLAN SHOWING THE CENTRAL
ISLAND WITH SLOPES
FINALLY RECOMMENDED.

SCALE = 1" = 500'
(6:1 ≡ 6 HORIZONTAL TO 1 VER.)

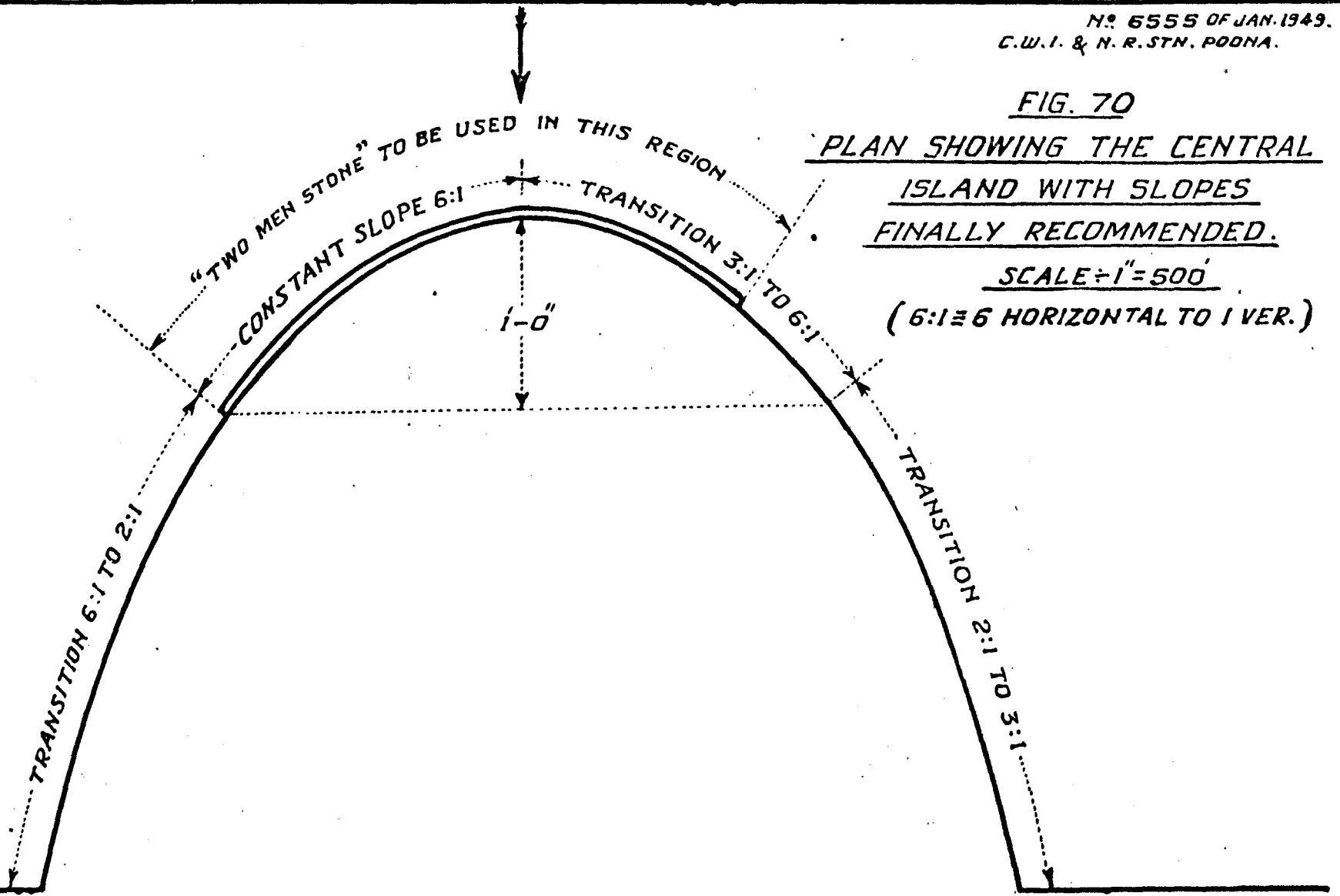


FIG. 71

LOWER SIND BARRAGE - KOTRI.
SHOWING ALIGNMENTS OF RIGHT
GUIDE BANK AS TESTED IN
THE MODEL.

Scale:- 1" = 5' = 1250'

7149 OF JAN. '49.
C.W.I. & N.R. S. POONA.

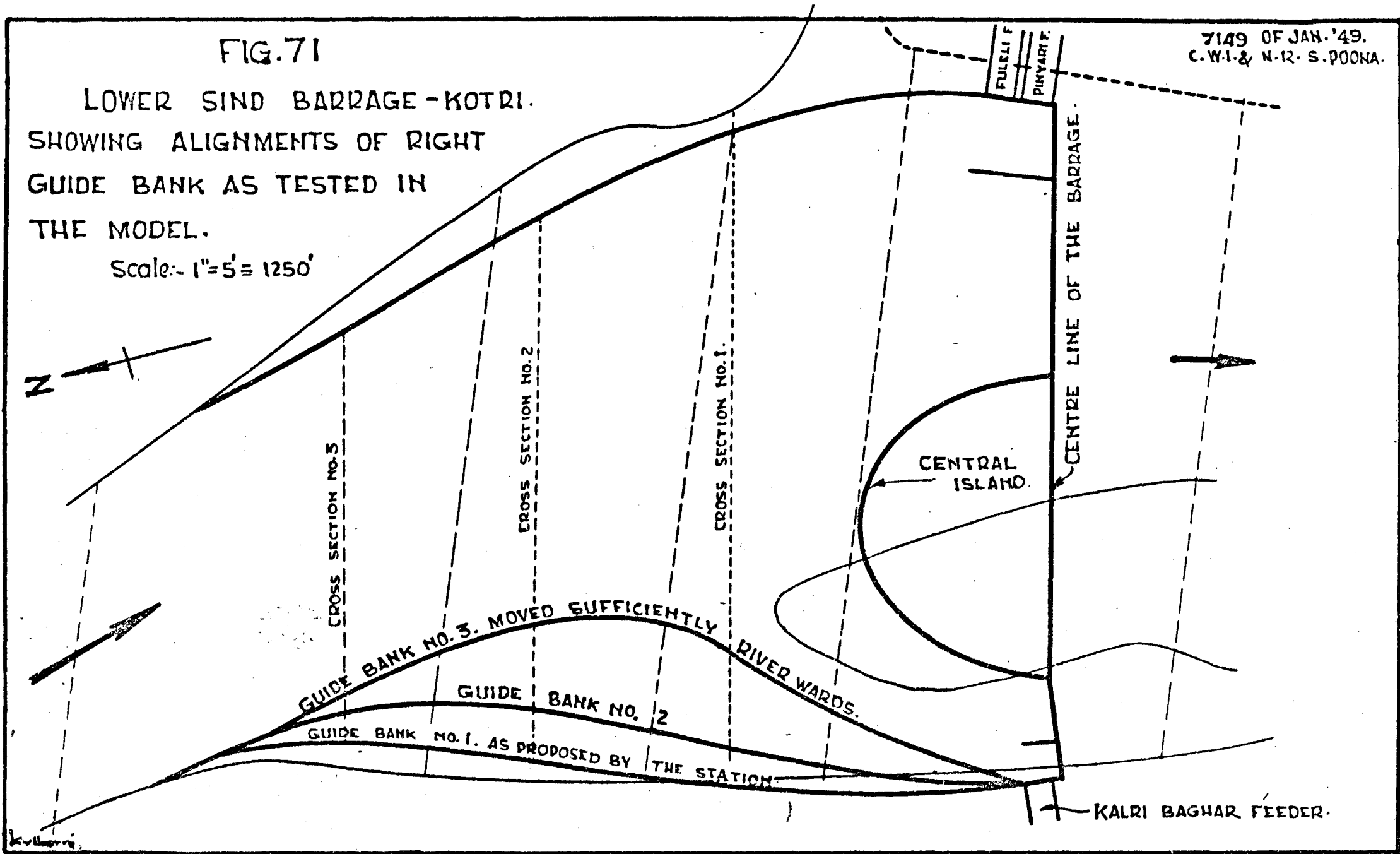


FIG. 72 & 73

SHOWING CONTOURS OBSERVED WITH
GUIDE BANKS NO. 1 & 3.
AFTER RUNNING A RISING & FALLING
CYCLE.

Scale: 1" = 75' = 1875 FT.

FIG. 72
GUIDE BANK.
NO. 1.

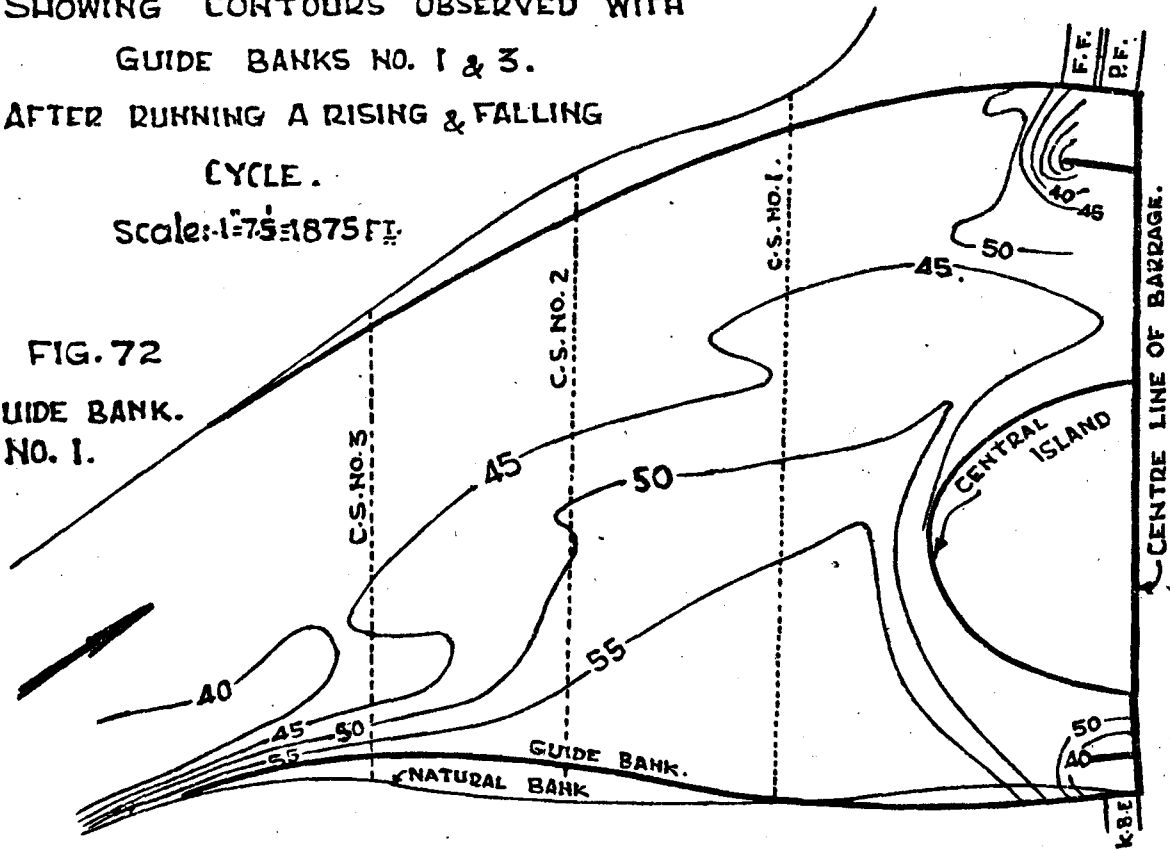


FIG. 73
GUIDE BANK
NO. 3.

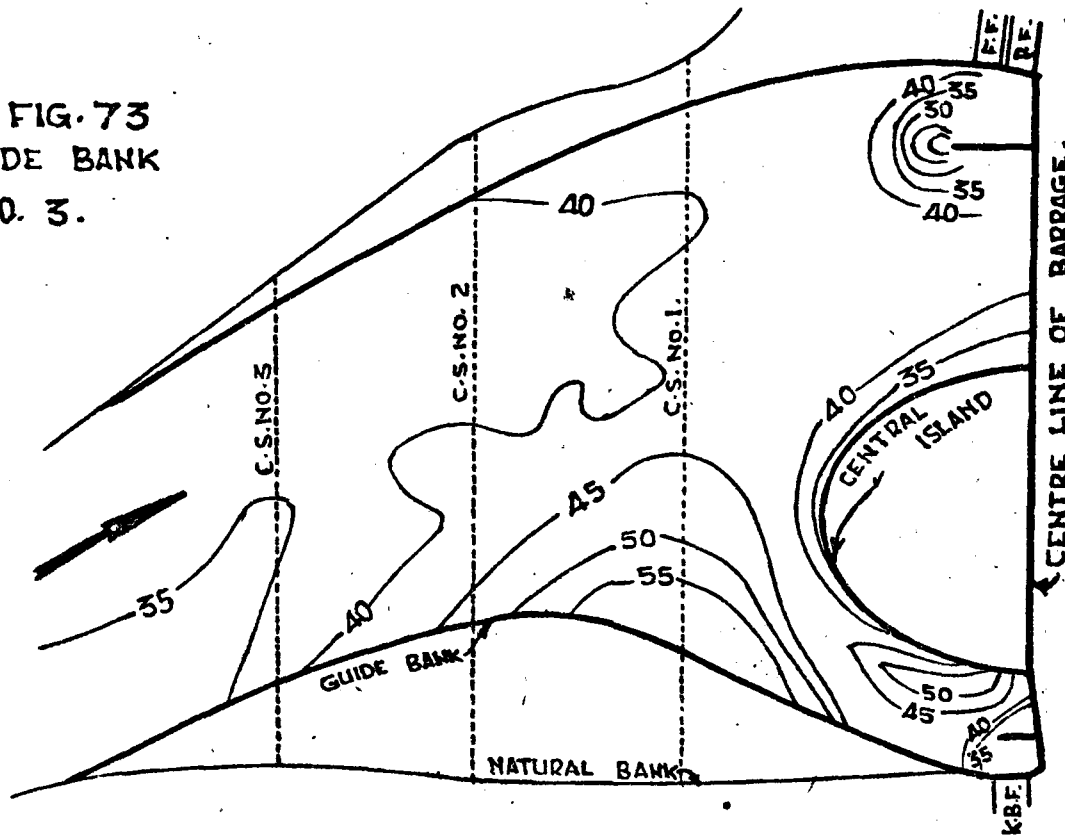


Table 2:—Showing the afflux between the left and right arm barrage and discharge entering the right arm channel, (With right guide bank No, 1— Station design)

Discharge cusecs	Cycle No. 1		Cycle No. 2		Cycle No, 3		Remarks,
	Afflux feet	Discharge cusecs	Afflux feet	Discharge cusecs	Afflux feet	Discharge cusecs	
200,000	1.40	22,000	0.70	27,800	0.70	29,100	same as in Table 1,
250,000	1.40	24,900	0.80	33,000	0.80	35,100	
300,000	1.55	26,700	0.90	42,200	0.80	45,000	
350,000	1.60	30,000	1.0	50,200	0.90	55,200	
400,000	1.75	39,700	0.95	62,800	0.90	69,000	
450,000	1.75	46,300	0.95	71,900	0.85	80,300	
500,000	1.90	55,600	1.10	84,700	0.60	90,900	

5.3. The analysis of this data brings out the following points :—

(a) The Station design gave least afflux and with the right guide bank moved more and more riverwards the afflux progressively increases (See Table 1.)

(b) There was also a substantial reduction in the discharge tapped by the right arm barrage, with alignment numbers 2 and 3, under unregulated conditions.

[Note :— Previous experiments were made with the bed of the right bye-channel moulded in erodible clay below R. L. 43. 50; with an inerodible bed, it was observed that, with the same alignment of the guide bank, the afflux is greater than that obtained previously, owing to the depth and extent of the scour round the nose of the central island and the consequential 'diminished " pull "]

(c) It will be seen from fig. 74 showing the variation of intensity of discharge that, guide bank alignment number 3 there was a marked tendency of an increase in the concentration of discharge towards the left, which would necessitate stronger protection upstream and downstream of the left barrage arm and the lowering of the downstream cistern level.

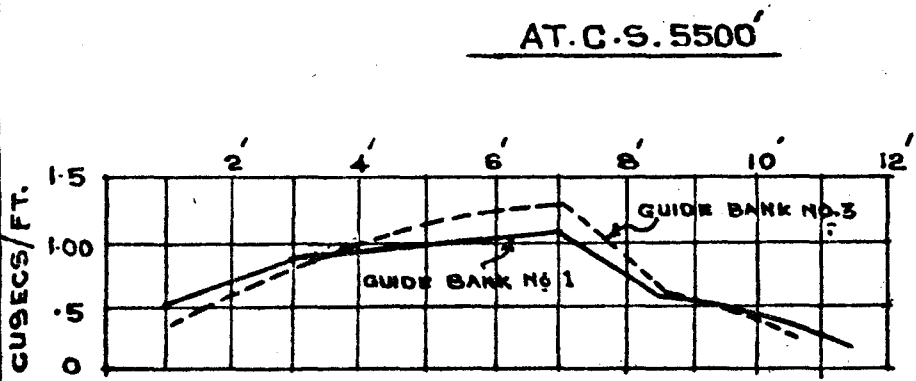
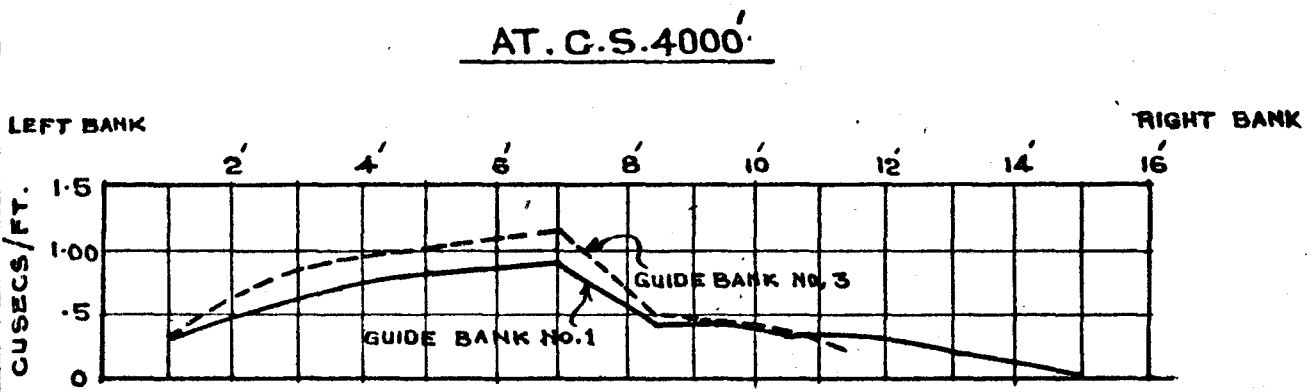
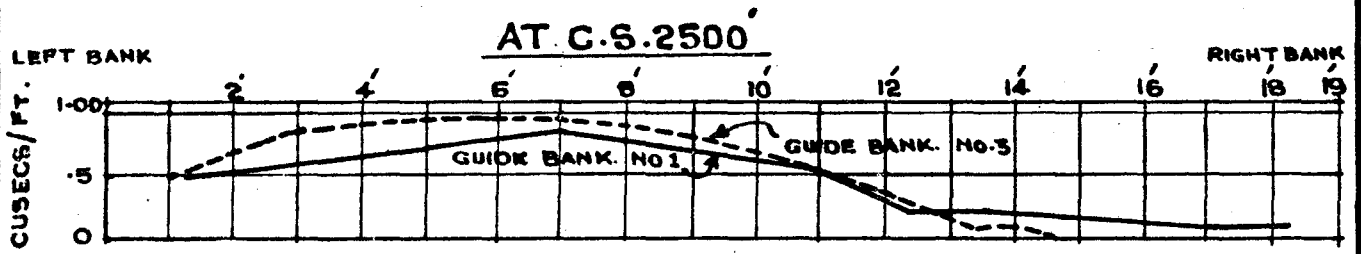
(d) The pacca pavement of the extreme left end spans of the right arm barrage, with alignment number 3, was observed to be covered with sand, proving conclusively inferiority of such a guide bank ; because the downstream shift of the point of transition of the curve from concavity to convexity, inherent in alignment number 3, adversely affects the distribution of sand charge between the two arms of the Barrage. Thus alignment number 3 with increased sand charge superimposed over reduced discharge was less, not more, favourable as expected, from point of diminished risk of accumulation of sand on the right arm channel.

(e) It is also not unlikely that, owing to the restricted Waterway between the right shank of the central island and the right guide bank opposite to it, the development of unusually deep scour in the sandy beds would create turbulence in the right pocket and, thus, excess bed sand would be drawn by the R.B. Feeder.

(f) Moreover, the shift in the axis of the bye-river, caused by so much shift of the right bank riverwards, must in the long run cause significant changes, which can not all be foreseen and which may not be as favourable as the natural conditions which are being perpetuated by the guide bank alignment number 1.

FIG. 74

SHOWING VARIATION OF DISTRIBUTION OF
INTENSITY OF DISCHARGE
ALONG C.S. 2500 FT, 4000 FT. & 5500 FT.
PARALLEL TO & 1/4 OF THE LEFT BARRAGE



SCALE:- HOR. 1" = 4' = 1000'
VER. 1" = 2 cs./ft. = 500 cs./ft.

5.4. As the shift of the guide bank riverwards showed that it would do more harm than good, further experiments were carried out, with the guide bank alignment number 1 as proposed by the Station, to ascertain whether the worst shoaling along it would lead to equally adverse conditions as would be obtained with guide bank of alignment Number 3.

Photo 5251

A shoal of fine red cohesive clay was laid roughly along the contour R.L. 55.0, obtained in experiments with guide bank alignment Number 1 up to R.L. 68.0 (see photo 5251). Four rising and falling cycles were run and the afflux between the left and right arm channels and the discharge entering the right channel were observed.

Table 2

See Table 2.

Fig. 75 &
Photo 5252

It is evident from the table that with every cycle there was a substantial, progressive improvement in afflux and the discharge entering the right arm channel. Figure 75 and photo 5252 show the remnant of the shoal after the fourth cycle. In spite of the little shoal still remaining along the guide bank the exclusion of sand from the R.B. Feeder was quite satisfactory.

5.5. Conclusion.

A shift of the guide bank riverwards, on the misguided presumption that no further shoaling would take place beyond the line of this proposed guide bank, would rob the bye river of its natural, valuable waterway, would impose a significant shift in its axis and cause greater afflux, less tapping of discharge and more 'sanding' in the right arm channel. Instead, it would be wiser to let nature alone, to slightly adjust the axis of the bye-river to suit itself, by building a small shoal along the guide bank if the post-barrage conditions so warrant it, as the model has shown times out of number that sand exclusion from the R. B. Feeder will not be adversely affected by this formation of shoal.

6. (v) Alternative designs of a single Barrage.

6.1.1. Mainly with a view to save cost—the additional cost in the duplication of flank walls and the provision of the Central Island in the Double Barrage design being Rs. 30 lakhs roughly — the Chief Engineer Lower Sind Barrage suggested alternative designs of a single barrage upstream of the site finally adopted for the double Barrage design.

The second criticism raised against the Double Barrage design was that it would lead to inordinate shoaling along the right guide bank near the entrance to the right arm; but model tests have shown that this

shoaling at its worst, can do no foreseeable harm and will probably improve the sand-exclusion from the Right Bank Channel.

6.1.2. The positions and designs of the Single Barrage at other sites than that selected for the Double Barrage were soon proved on the model to be obviously unsuitable, mainly owing to the insufficiently developed curvature at these upper sites. The Chief Engineer, Sind, agreed to drop these summarily out of consideration.

6.1.3. The Chief Engineer, however, considered the arguments (mentioned in para 6.1.1.) in favour of a Single Barrage design to be sufficient, in spite of the several proved merits of the Double Barrage design, to justify detailed model tests.

6.2. *Experiments and discussion of the results of the Single Barrage Design.*

6.2.1. Experiments were, therefore, carried out, in the 1/250:1/50 V. E. model, with the following arrangement of spans —

8 left pocket spans.	Crest R. L. 52.0
28 barrage spans.	Upstream pavement R. L. 46.0.
4 right pocket spans.	Downstream cistern R.L. 37.0.

The alignment of the left guide bank, the position of the left abutment and the withdrawals of the left and right bank canals were maintained the same as finally adopted in the Double Barrage design.

The design of the right guide bank was as suggested by the Chief Engineer viz. 4000' long with a very flat curve of $R = 11000'$ and ending in a 120° curved head of $R = 500'$. This design of the guide bank nose was expected to act as an attracting bank and thus improve the adverse conditions for sand exclusion from the Right Bank Feeder.

6.2.2. After running four rising and falling cycles up to $Q \equiv 5,00,000$ cusecs (i.e. after deeps and shoals were stabilised in the model), it was observed that, in spite of the low withdrawal of the Right Bank Feeder and the higher ratio of $V_{\text{river}} : V_{\text{pocket}}$, at the most difficult river stages ($Q \equiv 2,00,000$ cusecs), the unfavourable offtake at the end of the wrong (convex) curvature of the right guide bank could not manifestly be corrected by the more favourable $V_{\text{river}} : V_{\text{pocket}}$ ratio. On the contrary, bed sand was visibly agitated into suspension and sucked into the Right Bank Feeder in large quantities.

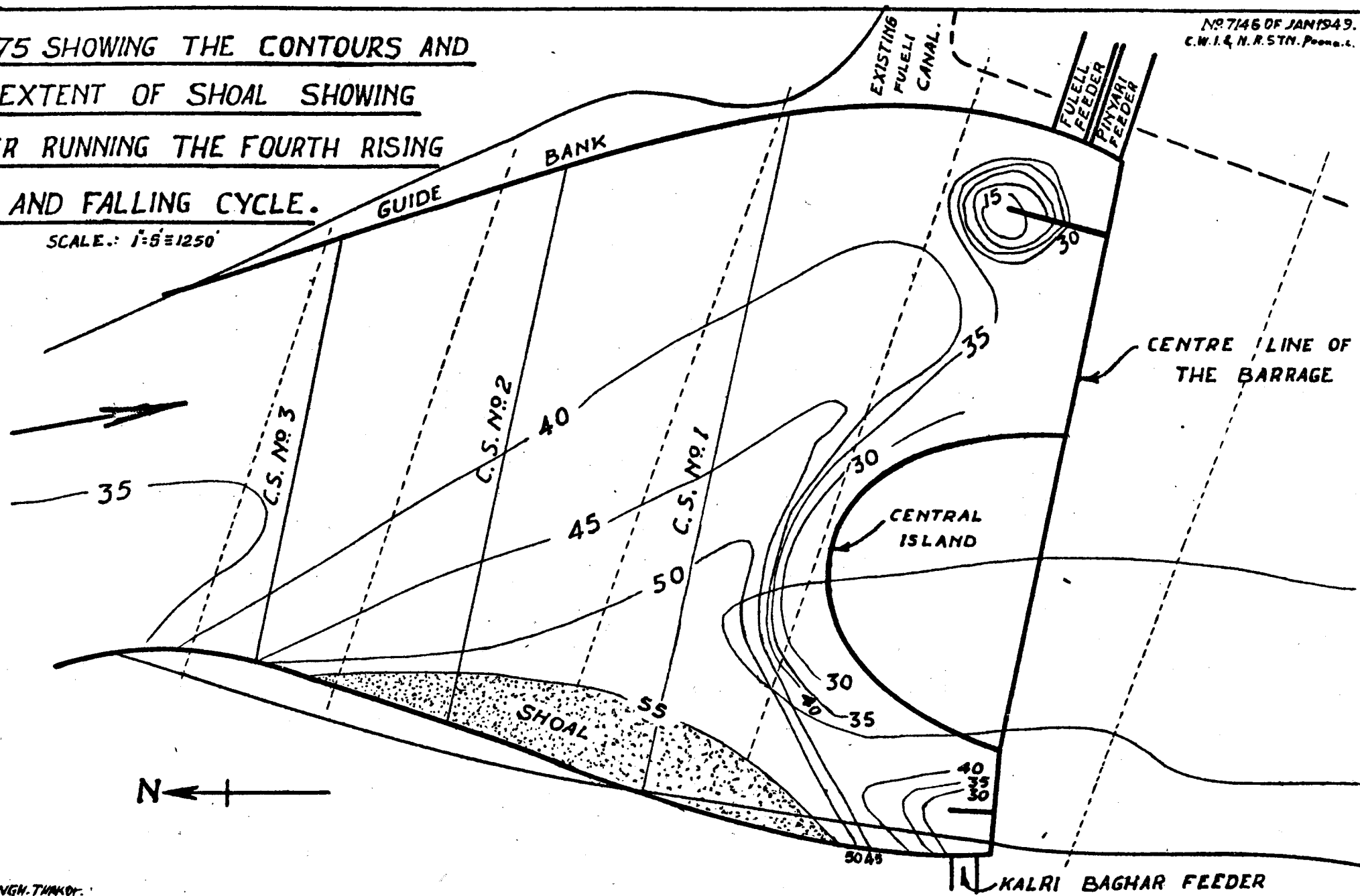
6.2.3. The contours in fig. 76, observed after four rising and falling cycles, show the marked tendency of shoal formations just in front of the right pocket and against

Fig. 76.

FIG.75 SHOWING THE CONTOURS AND
EXTENT OF SHOAL SHOWING
AFTER RUNNING THE FOURTH RISING
AND FALLING CYCLE.

SCALE: 1" = 5' = 1250'

NO. 7146 OF JAN 1949.
 C.W.I. & N.R. STN. Poona. I.



By RANJITSINGH THAKUR.

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MODEL OF LOWER SIND
BARRAGE—KOTRI.

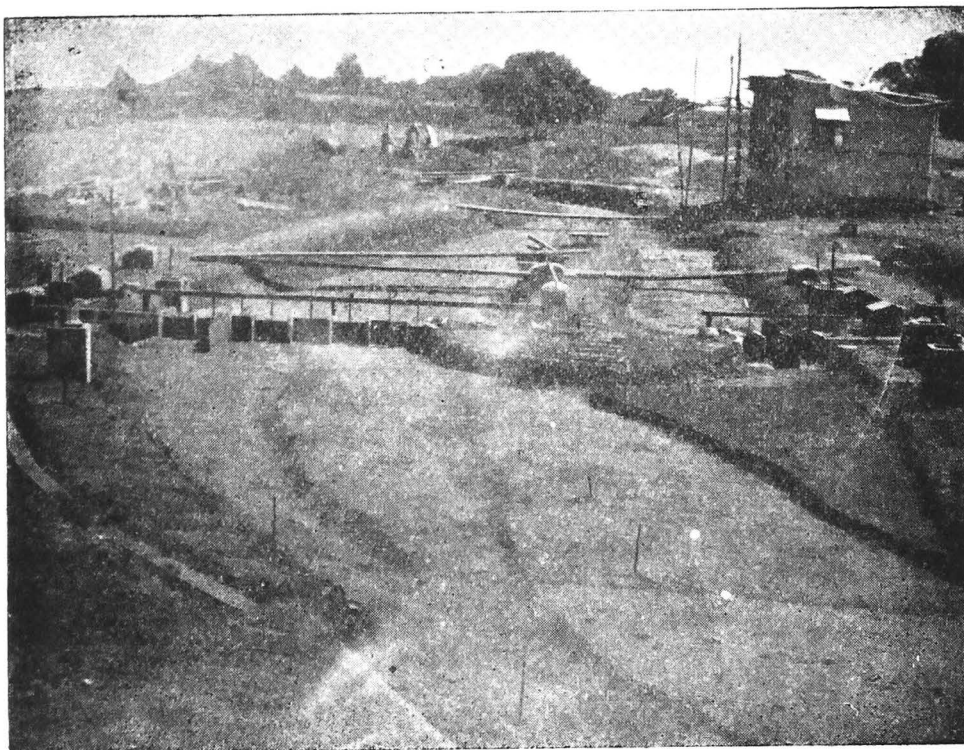


PHOTO No. 5251. Showing shoal as laid in the model against the right guide bank roughly along contour R.L. 55.0 up to R.L. 68.0.

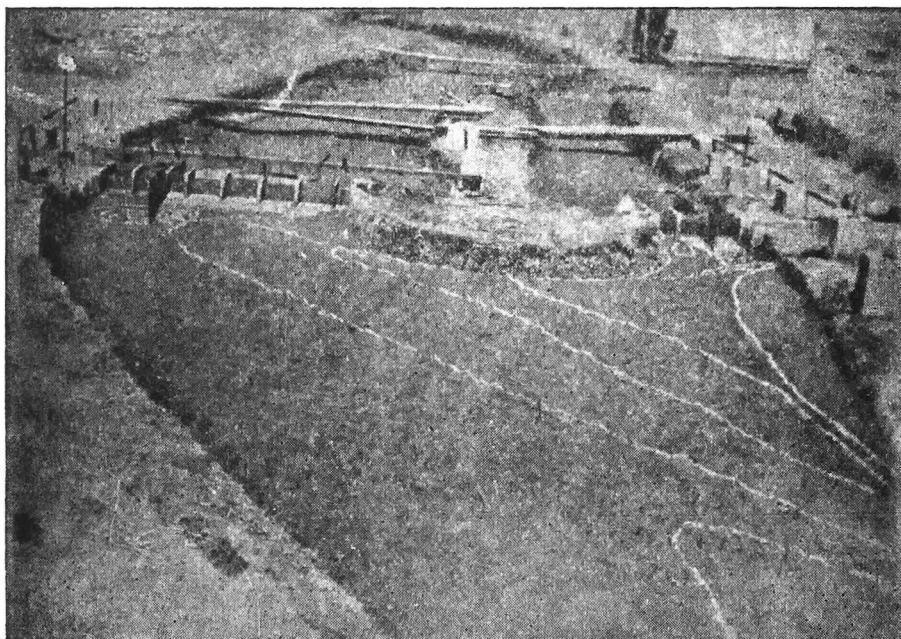


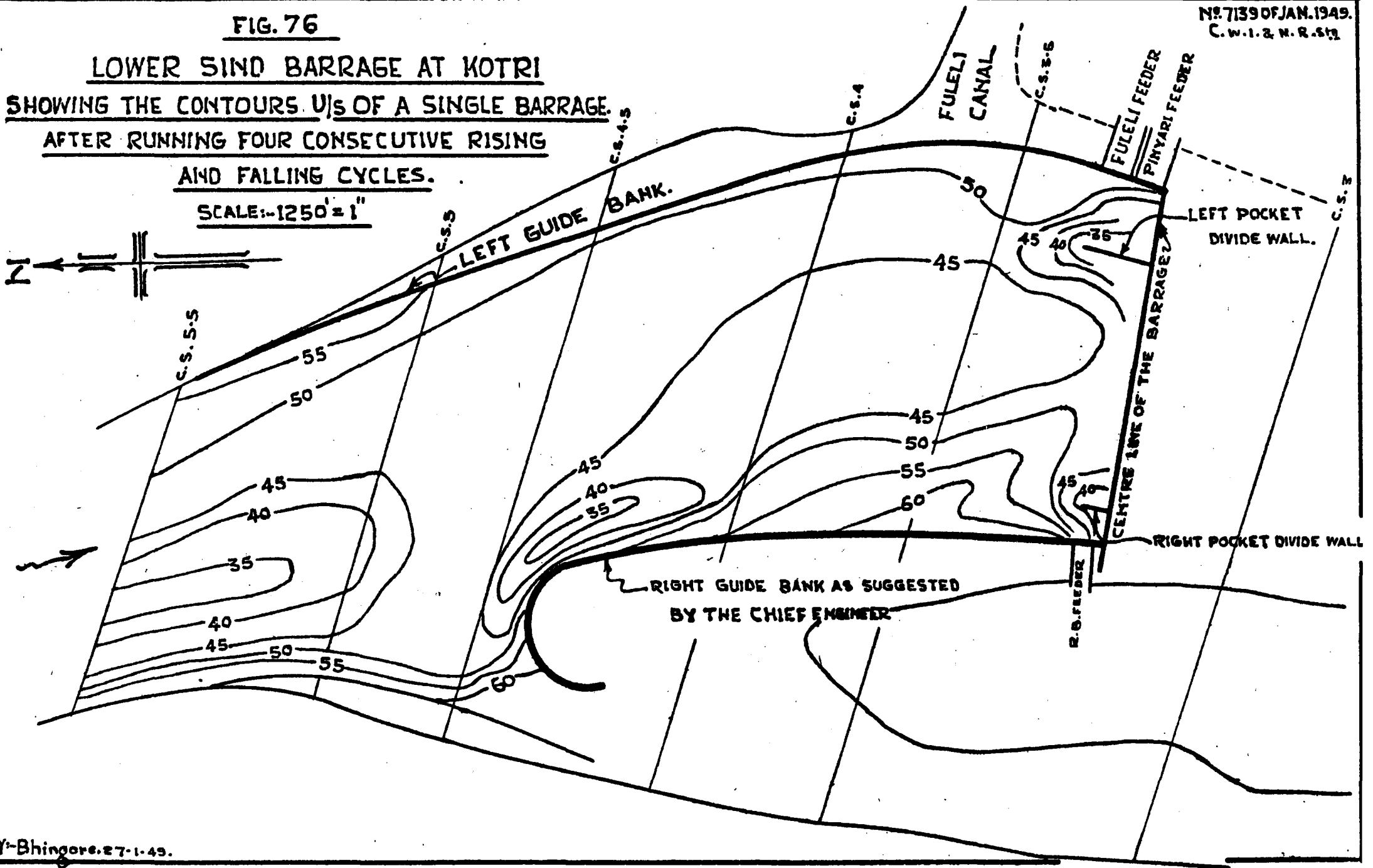
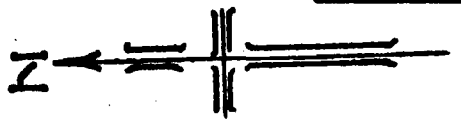
PHOTO No. 5252. Showing the contours and remnant of the extent of shoal against the right guide bank after running the fourth rising and falling cycle,

FIG. 76

№ 7139 OF JAN. 1949.
C.W. 1.2 N.R. 512

LOWER SIND BARRAGE AT KOTRI
SHOWING THE CONTOURS U/s OF A SINGLE BARRAGE.
AFTER RUNNING FOUR CONSECUTIVE RISING
AND FALLING CYCLES.

SCALE:- 1250' = 1"



the right guide bank. The consequences of these shoals will be far worse than the counterparts in the double Barrage design; while the model showed some improvement in exclusion after shoaling in the Double Barrage design, the very reverse was manifest, in the model of the Single Barrage design, owing no doubt to the improvement of curvature in the former and the worsening in the latter *after shoaling*.

6.2.4. Figure 77 shows the lines of flow, for $Q \approx 2,00,000$ cusecs, after running four rising and falling cycles. These also clearly showed how disproportionate would be the distribution of bed sand relative to the discharge which would be shared by the extreme right end spans of the Barrage. The upstream pacca pavement of the right side of the barrage—as well as the Head Regulator of the Right Bank Feeder was seen fully covered with bed sand owing to the concentration of a substantial percentage of bed charge on the right side of the river which was not observed in experiments with the Double Barrage design.

Moreover, in no experiment carried out with the Double Barrage design, as much sand was ever collected in the model in the silt pit downstream of the R.B. Feeder, as in experiments with the *Single Barrage* design.

This would not only lead to shoals upstream of the barrage as already brought out in para 6.2.3. above but also affect the regime of the river downstream of the barrage owing to the heavy concentration of discharge on the left side of the barrage. Water drawn by the left barrage spans, being comparatively clear of the bed sand, would either seriously erode the Jamshoro Bund downstream of the barrage and/or carve out a deeper channel on the left side of the river with increase in concentration of discharge following the sanding of the right side of the river downstream; this progressive deterioration would lead to adverse conditions downstream; which might further deteriorate the conditions upstream.

6.2.5. With a view to assess the attracting effect of the proposed nose of the guide bank, experiments were carried out alternatively with,

Fig. 77 (a) Right Guide Bank as suggested by the Chief Engineer (See fig. 77).

Fig. 77 (b) Right Guide Bank alignment same as in (a) above up to 4000' upstream of a barrage and then taken straight to link up with the right bank of the river. (Also fig. 77).

After running a rising and falling cycle for discharges upto $Q \approx 5,00,000$ cusecs, the velocity distribution was observed at the cross section 4000' upstream of the barrage for both these alignments.

Figure 78 shows the percentage of discharge and the velocity distribution along the C.S., 4000' upstream of the barrage.

Fig. 78.

It will be seen from the same figure that the scour-hole at the nose of the guide bank had a strong attracting effect as expected by the designer. But, in spite of this stronger attraction by the nose, no difference was observed in sand exclusion from the Right Bank Canal. On the other hand, this attraction had a little adverse effect on the Left Bank in that it pulled away a percentage of flow from the left bank and thus slackened the velocities and reduced the curvature along the left guide bank.

Thus, an attracting nose, if successful, would in all probability reduce the prospects of a deep channel along the left bank—desirable for satisfactory sand exclusion from the left bank canals which is difficult in spite of the favourable curvature, because of their proportionately large withdrawals. If too effective, the attracting nose may pull too large a fraction of the river across to it, necessitating subsequent measures to counteract this effect. Moreover, the unfavourable location of this attracting nose, in relation to the river meanders, would cause it to pull not the top water, as is desirable, but also bed sand.

6.2.6. It was also observed in the Model that ejectors would not prove useful, for two good reasons:—

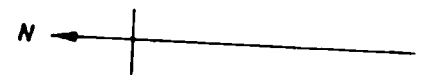
(a) The Canal offtake being on the convex (i.e. inside) of the curve, with the increased Pocket discharge following the provision of escapable discharge for the ejectors, a demonstrably larger amount of bed sand was drawn into the canal.

(b) The river conditions downstream of the barrage were not satisfactory for flushing away the ejected bed sand from the tail of the supercharged outfall channel; being on the inside (i.e. the silting side) of the curve downstream of the barrage, the outfall would get plugged with accumulated sand unless continuously removed by dredgers.

It is essential, therefore, to locate the canals on both banks to have favourable concave curvature which can be secured by means of the Double Barrage Design.

6.2.7. In the Double Barrage Design, owing to the concave curvature on the right bank, assisted by the central island, not only exclusion was satisfactory but it was also possible to maintain "semi open" flow regulation. In the Single Barrage Design "semi open" flow would be impracticable, as with it, the model showed an increased amount of bed sand was drawn by the R. B. Feeder.

FIG. 77
LOWER SIND BARRAGE AT KOTRI
PLAN SHOWING THE LINES OF FLOW
FOR Q = 200,000 CUSECS AFTER RUNNING
FOUR CONSECUTIVE RISING & FALLING CYCLES.
SCALE 1" = 1250'



REFERENCES:—

- SURFACE LINES OF FLOW.
- BED LINES OF FLOW.

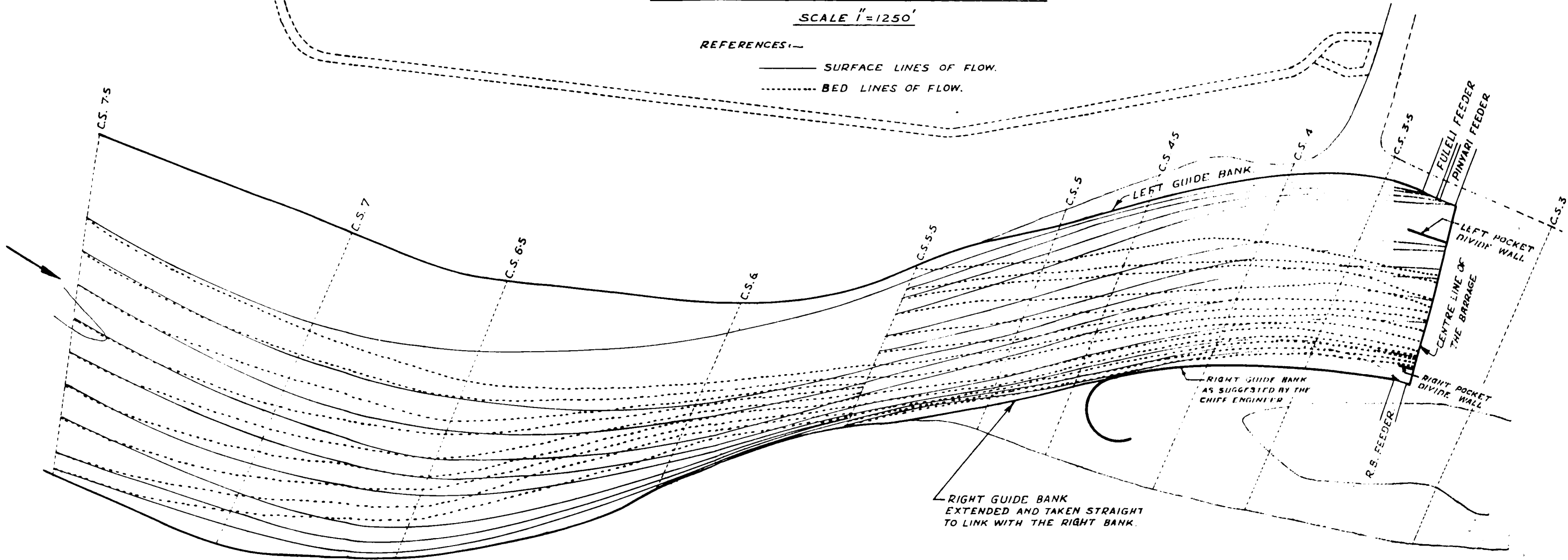
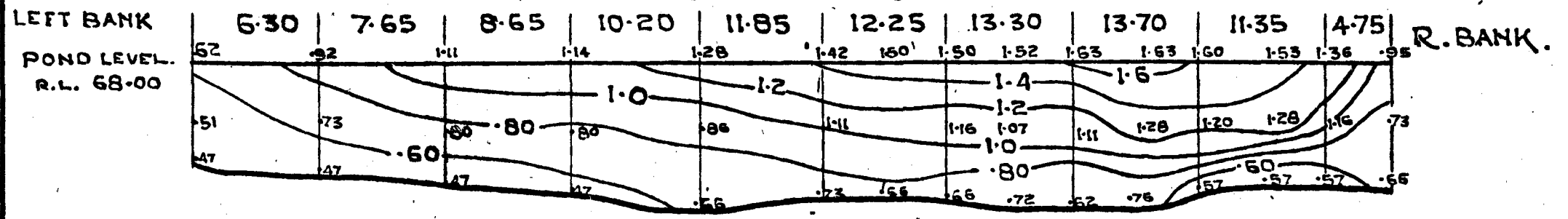


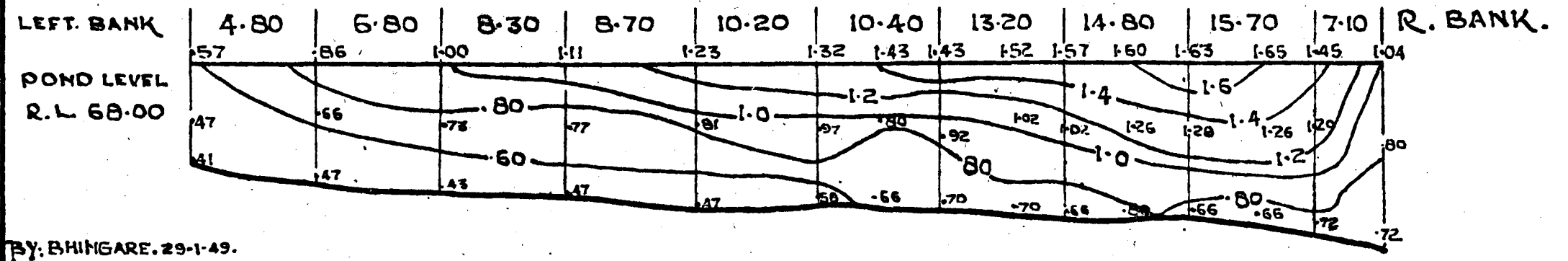
FIG. 78 LOWER SIND BARRAGE AT KOTRI
CROSS SECTION AT 4000 FT. U/S OF SINGLE BARRAGE.
SHOWING ISOVELS AND PERCENTAGE OF DISCHARGE FOR $Q=300,000$ cfs.

SCALE:— HOR. 1" = 1/2" = 375'
 VER. 1" = 9' = 37 1/2'

WITH RIGHT GUIDE BANK EXTENDED TO LINK UP WITH THE R. BANK.
 PERCENTAGE OF DISCHARGE.



WITH RIGHT GUIDE BANK AS SUGGESTED BY THE CHIEF ENGINEER.
 PERCENTAGE OF DISCHARGE.



By: BHINGARE. 29-1-49.

If "still pond" is to be rigidly observed it would necessitate raising the pocket gates by another 2ft. to 3ft. over those already recommended for the Right Pocket of the Double Barrage Design.

"Still pond" is only a temporary palliative and cannot be depended upon as an all-time remedy for arresting the excess influx of bed sand into the canal, unless it is aided by fully established favourable curvature; and the conditions at Kotri are likely to become eventually worse or, at any rate the same as those obtained near the right bank at Sukkur in 1939. Besides such shoaling as at Sukkur should never be permitted at Kotri because no similar training measure for improving the curvature will be possible at Kotri, there being no surplus waterway as was available at Sukkur.

6.2.8. The only possible alternative for satisfactory exclusion from the R. B. Feeder with a single barrage design would be to locate the off-take at the favourable curvature, some $2\frac{1}{2}$ miles upstream of the barrage. But this arrangement will neither be convenient nor satisfactory as the off-take will be far upstream of the control and fixity of the barrage. As the river upstream of this point cannot be regarded to be equally stable, this alternative cannot be recommended.

6.3. *Other demerits of the Single Barrage Design.*

Other disadvantages of the Single Barrage Design worthy of note are as under:—

6.3.1. The Single Barrage is unnatural to the site and such a distortion of natural conditions must inevitably produce far reaching changes, not all of which are foreseeable or can be reproduced by the undesirable consequences of distortion of existing conditions.

6.3.2. As the Single Barrage Design would not be in conformity with the natural existing waterway of the left arm channel to prevent inordinate concentration it would entail heavy cutting of the Keti island downstream of the barrage, estimated roughly at Rs. 15 lakhs; as the effective increase in waterway by scour along the island cannot be entirely depended upon owing to the existence of clay at a substantially higher level as already revealed by the borings.

6.3.3. Stone is on right bank; to bring it to site, a sufficiently strong bridge will have to be constructed across the bye-river. This will add to the cost and also the cost of materials will be increased owing to the increase in lead by about $\frac{1}{2}$ a mile. This presumes that the construction will be carried in the Ring Bund by diverting a part of the river through the bye-channel.

To avoid the bridge and diversion if the construction is done in cofferdam the difficulties will still be aggravated by the maximum scour caused by the great restriction of waterway in the Single Barrage Design.

Thus, from the point of view of the construction difficulties and programme, the Double Barrage Design, worked for the Right Bank as discussed at Poona, is apparently the superior proposition.

6.3.4. Owing to the loose design of the Sukkur Barrage there was sufficient margin for the provision of an approach channel for ameliorating the trouble in the right bank due to the unfavourable curvature at the offake. The Kotri Barrage design being comparatively very tight no such improvement will be feasible at a subsequent stage. The model showed that the conditions on the Right Bank at Kotri were as bad as, if not worse than, at Sukkur.

4. Conclusion.

Permanent advantages of dependable and efficient exclusion of sand on both banks and the superior control secured in the Double Barrage Design, with a definite saving in running costs, should not be sacrificed to the saving first cost expected from the Single Barrage Design. When a proper estimate is made of the comparative costs the difference may not be as much as expected because the Single Barrage design will involve heavy cutting of the 'Keti', stronger loose stone protections against greatest concentrations, etc.; but considerations of all time-efficiency must in any case prevail over those of additional first cost.

APPENDIX I

Showing depths of scour below water level and values of R-calculated for the main river, left and right arms of the river.

The maximum scour is calculated for an intensity of superflood discharge of 7,50,000 cusecs distributed over the minimum stable width thus

$$q = \frac{Q}{2.67\sqrt{Q}} = \frac{\sqrt{Q}}{2.67} = \frac{865}{2.67} = 324 \text{ cusecs/ft.}$$

$$\therefore R = 0.96 (324)^{2/3} = 45.3 \text{ ft.}$$

Below the nose of the Central Island the super flood discharge was distributed into two parts, 5,62,000, cusecs and 1,87,000 cusecs in left and right arms respectively. Consequently, the scour in the two arms would also depend upon the intensity of the respective discharge, so that the depth of scour in the left river would be,

$$R_1 = \left(\frac{\sqrt{5,62,000}}{2.67} \right)^{2/3} \times .96 = 41.2 \text{ ft.}$$

Similarly

$$R = \left(\frac{\sqrt{1,87,000}}{2.67} \right)^{2/3} \times .96 = 28.6 \text{ ft.}$$

Table 3. Main river upstream

	Left guide bank.	Reach AB of right guide bank.	Reach BC of right guide bank.	Nose of the Central Island.	Reach CD of the right guide bank.	Left arm of the Central Island.	Right arm of the Central Island.
1. H. F. L. super flood.	73.70	73.70	73.70	73.70	73.70	73.70	73.70
2. Scour factor recommended.	1.25	1.50	1.0	1.75	15.R ₂	17.5R ₁	2R ₂
3. Normal scour R (ft.).	45.30	45.30	45.30	45.30	28.60	41.2	28.6
4. Adopted R (ft.)	56.60	67.95	45.30	79.30	42.90	7.20	57.2
5. R. L. of bottom of scour thus recommended for adoption.	+17.10	+ 5.75	+28.40	-5.60	+30.80	+1.7	+16.5
6. R. L. of deepest scour observed in the model.	+25.0	+12.0	+35.0	+10	+35.0	+10.0	+25
7. R. L. of deepest scour proposed in the Sind Design.	+5.75	+ 5.75	+28.40	-16.90	+30.80	*	*

* Not worked out in Sind Design.

25. Investigations in connection with Radhanagari Hydro-electric Works— Kolhapur.

1. Introduction.

The scheme costing about two crores of rupees which is already under way envisages the construction of 115 ft. high gravity type masonry dam across the Bhogawati river near Radhanagari village in Kholhapur State, for impounding 8,000 m. cft. of gross storage. This storage is sufficient for generation of 5,000 K. W. continuous supply of electric power. The tail race discharge of 500 cusecs will be available for lift irrigation on the river banks downstream and will also provide adequate water supply for the Kolhapur City.

Out of an estimated maximum flood discharge of 40,000 cusecs, 75% is proposed to be escaped through the deep seated sluice outlets, the balance being passed over an emergency overflow waste weir. Five sluices, each eight ft. wide and 18' - 9" high, excluding four ft. radius arch as the top, is expected to have a total capacity of not less than 30,000 cusecs with discharge coefficient of 0.6. The sluices will be provided with special converging entrance pieces at the mouth to reduce the losses at entry. These entrance pieces will be 22' - 9" x 12' at the dam face tapering down to the conduit size of 18' - 9" x 8' in a distance of 9' - 4½". Exact coefficient of discharge with such a sluice entrance not being known, accurate estimation of the discharging capacity of the sluice outlets was not possible. The Poona Station was therefore approached for determination of discharge coefficient of the sluices which would then determine the capacity of the emergency weir. Besides these two problems, several others, such as determination of the top level of the flank walls below the sluices, protective works to prevent scour due to falling jet through the outlets, prevention of sand bars in the tail race have also to be subsequently tackled.

2. Investigations and results.

(i) Sluice outlets.

The original project design with tapered sides did not appear to be ideal for natural curved flow entry conditions. For obtaining maximum possible coefficient for the sluice entrance consistent with safety, the extreme water filaments have to be made to follow the sluice entrance sides without causing any cavitation. Such a design with positive pressures on the sluice walls can be obtained by shaping the entrance profile according to the profile of a free jet up to 'vena contracta' issuing from a sharp-edged orifice of the same size as the entrance section. In the present case the design on the above lines was however not permissible as the permanent and temporary sluice gates forming obligatory sections have already been ordered and some of their parts have actually arrived on site.

Photo
5398 &
Fig. 80.

It was still felt that a profile intermediate between the ideal bellmouth and straight tapered sides, would perhaps yield better results. Actually the results given by experiments 2 & 3 shown in fig. 80 were not satisfactory. The pressures obtained with these profiles were not only negative but at some points below -26 ft. i. e. cavitation pressures were obtained and even then the increase in discharge was inappreciable. The extra cost for providing centring for curved bellmouth in lieu of straight tapered entrance was thus not justified. It was therefore decided to recommend the original design.

In Experiment No. 2, the effect of aeration vents on discharge was examined. Aeration vents were proposed to prevent cavitation. It was found that so long as cavitation pressures were not reached, the air vents reduced the discharge due to interference. With tapered sides negative pressures were not recorded anywhere and hence the provision of air vents was considered unnecessary.

To determine the coefficient of discharge, experiments will be done in $1/40$ scale brass model.

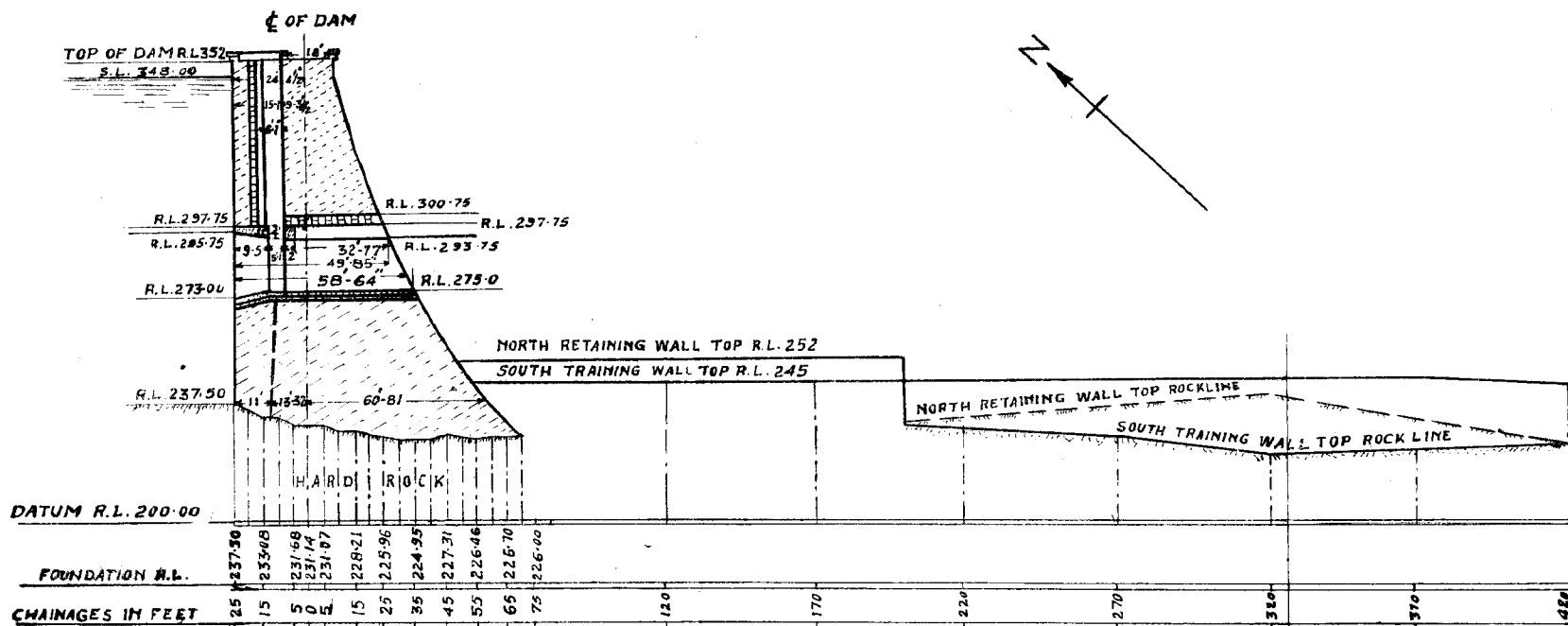
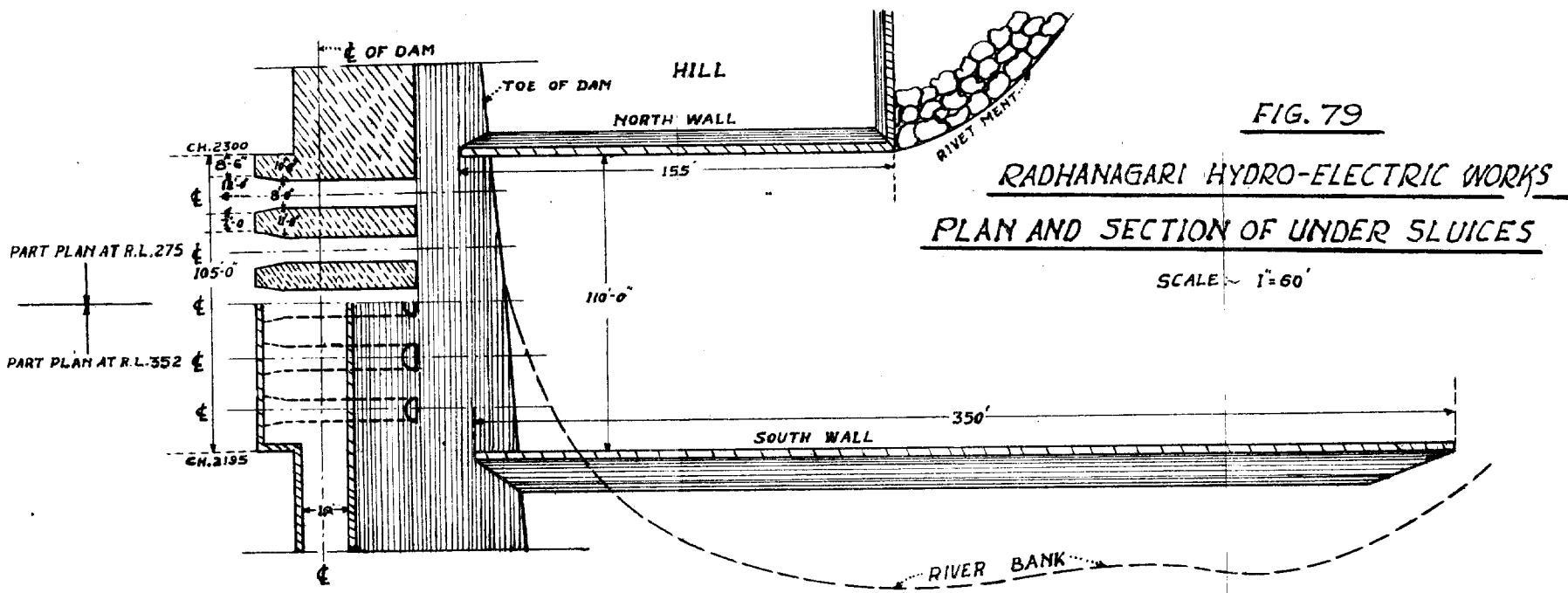
(ii) *Emergency Waste Weir*

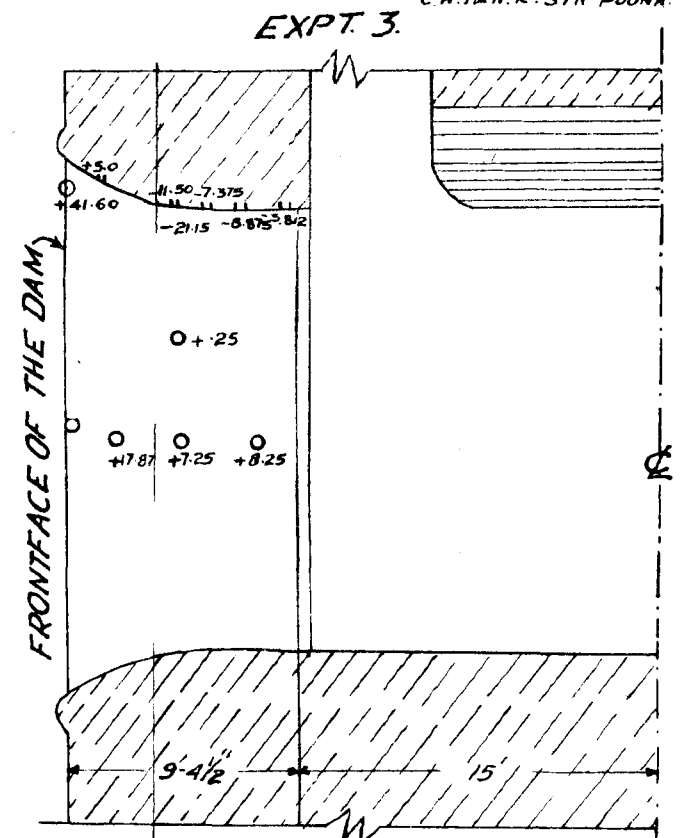
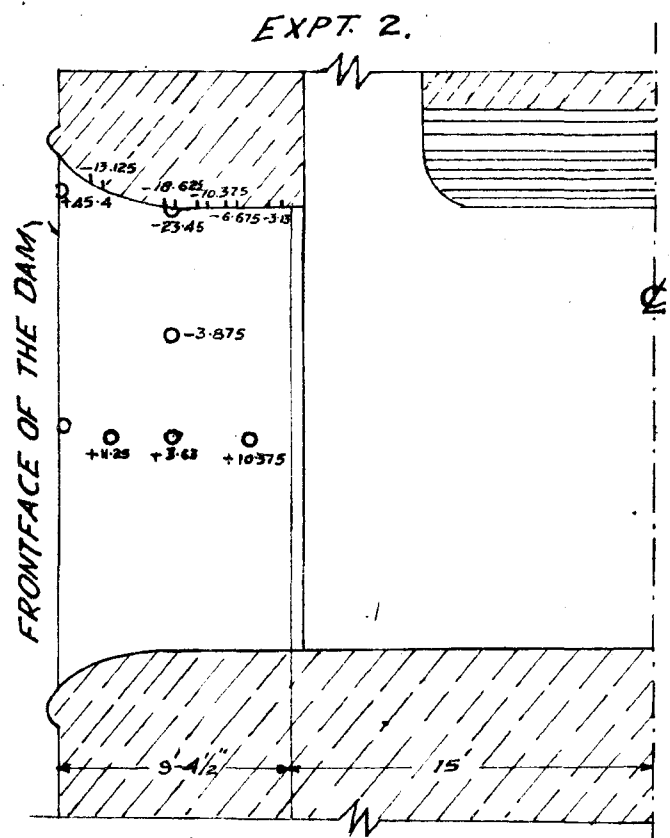
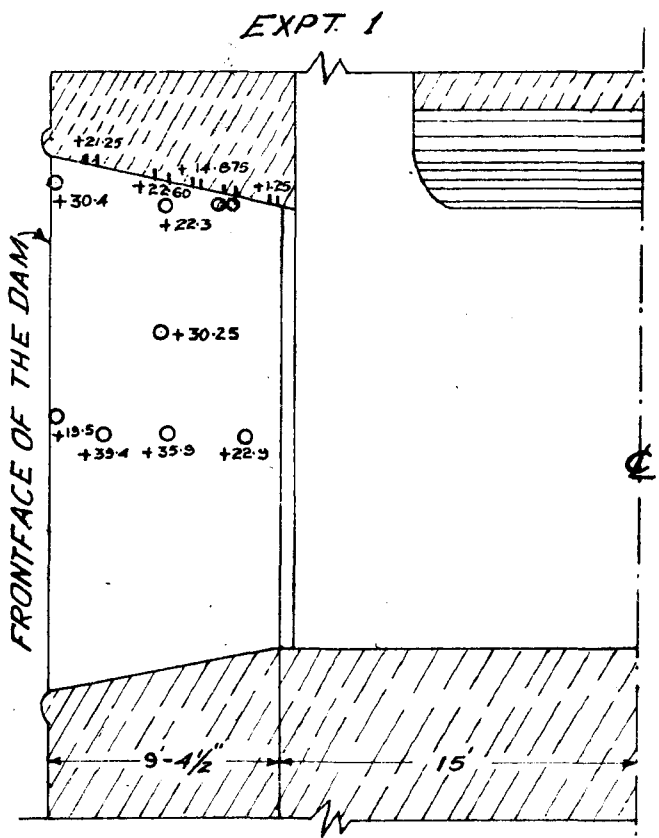
The section of the emergency waste weir proposed by the project engineer had a trapezoidal section with 6 ft. top and 18 ft. bottom. This was suitably modified to increase the coefficient and thus reduce the length of the weir substantially.

The maximum coefficient of 3.98 is possible with the profile proposed by the Station without causing any subatmospheric pressures on the spillway face. The coefficient can be still further raised by shaping the weir profile so that negative pressures down to, but not equal to or below, the cavitation pressures are caused.

The weir section to give the coefficient of about 3.98, suggested for adoption was as shown in fig. 81. This design is suitable to pass 10,000 cusecs with a head of four feet over a length of about 330 ft. After the exact discharging capacity of the sluices is determined by model sluices in brass, the emergency weir design will be finalised.

Fig. 81.





INCREASE IN DISCHARGE OVER THAT IN EXPT. 1 = 2.49 %

INCREASE IN DISCHARGE OVER THAT IN EXPT. 1 = 1.15 %
PRESSURES ON SIDES SLUICE ENTRANCE AND SIDES IN EXPTS 1, 2, & 3.

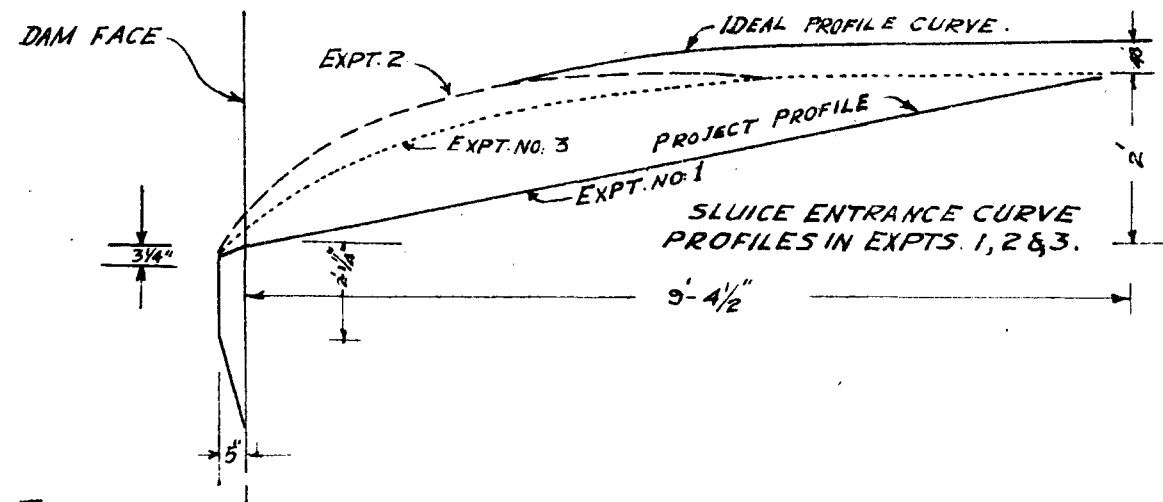


FIG. 80
RADHANAGARI HYDRO ELECTRIC SCHEME
EXPERIMENTS WITH DIFFERENT PROFILES
FOR SLUICE ENTRANCE.

SCALE OF PLAN. 1" = 8 FEET.

NOTE.
EXP. NO. 1. WITH TAPERED STRAIGHT PROFILES AS IN THE ORIGINAL PROJECT DESIGN
EXP. NO. 2, & 3. WITH CURVED PROFILES APPROXIMATING THE IDEAL BELL MOUTH SHAPE MODIFIED TO FIT IN WITH THE OBLIGATORY GATE DIMENSIONS.

FIG. 81

SHOWING WASTE WEIR DESIGN

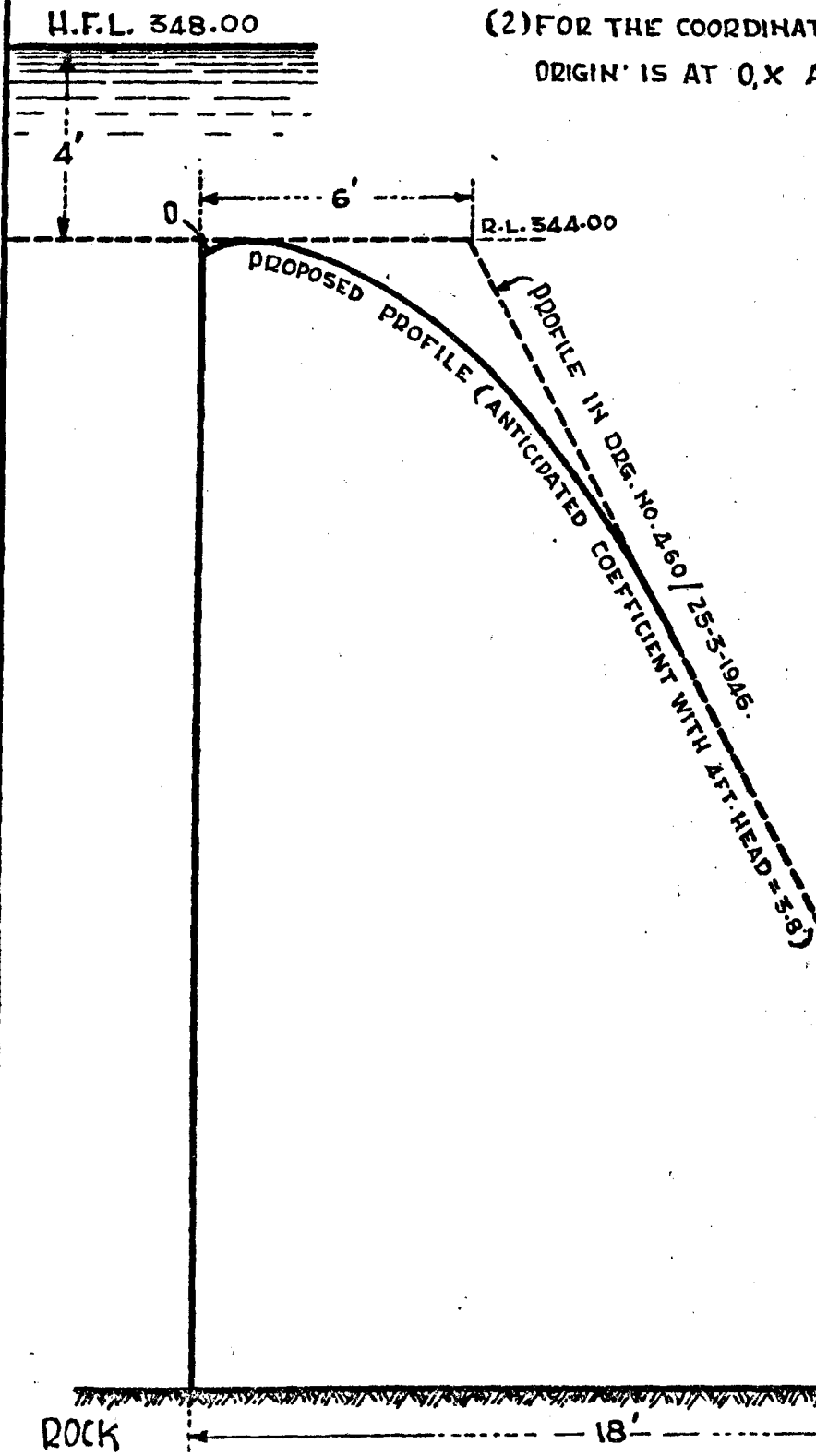
FOR

EMERGENCY SPILLWAY

Scale:- 1" = 5 FT.

NOTE.

- (1) BETWEEN $Y = 8'-5"$ & $24'-0"$ THE D/S FACE LINE OF THE WEIR IS STRAIGHT
- (2) FOR THE COORDINATES IN THE BELOW TABLE, ORIGIN IS AT O, X AXIS HOR. & Y AXIS VER.



—TABLE—
COORDINATES FOR WEIR-PROFILE.

X FT. INCHES.	Y FT. INCHES.
0-0	0-6
0-5	0-13/4
0-9 1/2	0-3/8
1-2 1/2	0-0
1-7 1/4	0-3/8
2-5	0-3
3-2 1/2	0-7 1/4
4-0	1-3/4
4-9 3/4	1-8
5-7 1/2	2-4 1/2
6-9 1/2	3-8 1/2
8-0	5-3
10-0	8-5
18-0	24-0

RADHANAGRI HYDRO—ELECTRIC
SCHEME.

Tank for maintaining constant W. L. at
R.L.—348.00.

Sluice fitted with several piezometer
tubes in the entrance portion.

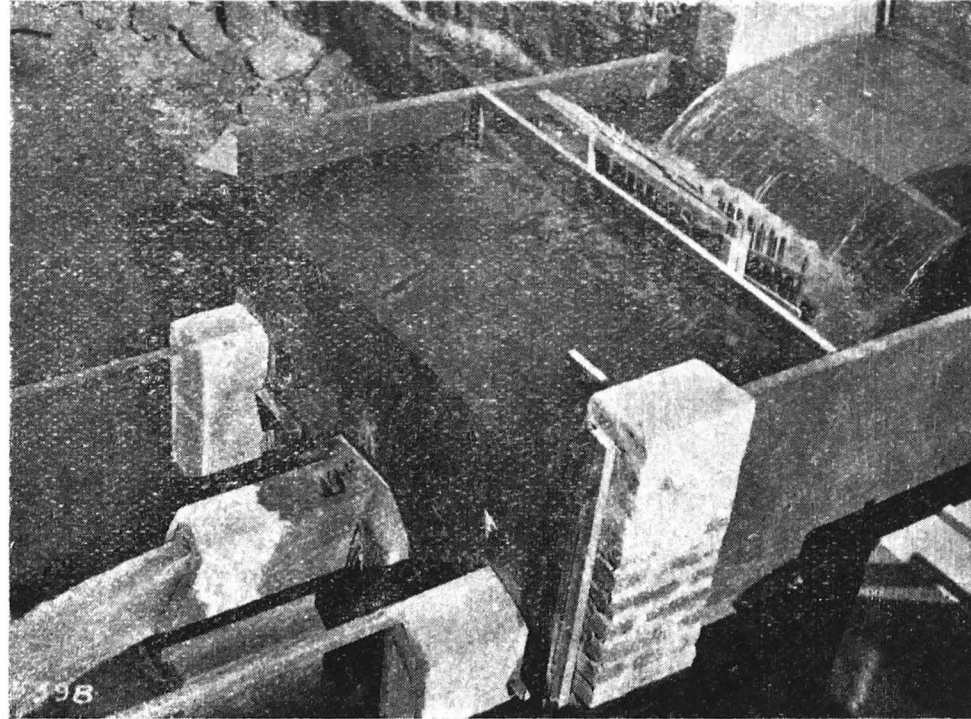


PHOTO No. 5398. Model set up for sluice
entrance design in connection with
Radhanagri Hydro-Electric Scheme.
Model scale 1/25.

26. Siphons and gates for Bhandardara waste-weir to raise the retention level.

1. Introduction.

1.1. Lake Arthur Hill, in the Western Ghats, stores water for the Pravara Canals. In the year 1926, when the dam was completed, the reservoir capacity was in excess of the irrigation requirements. The waste-weir consisted of a 656ft. wide cut (R.L. of waste-weir 260.0, dam top being at R.L. 270.0) through a hill forming the right flank of the dam. In 1937, a wall 2.5 ft. high was built on the waste-weir, as an emergency measure, to cope up with the increased irrigation demands. To obtain still greater storage, without exceeding high flood level, this Station suggested the construction of high coefficient weir (1) with sill at R.L. 265.45. This provided an overflowing capacity of 20,000 cusecs with 270.0 upstream water level. The weir was not built as recommended, but the old wall was increased in height to R.L. 265.7, in 1941.

1.2. The Irrigation requirements have increased since then and further increase in storage by raising the retention level to maximum possible i. e. R. L. 269.0 or 269.5 is felt desirable. This cannot be achieved by further increase in the height of the weir as the maximum high flood level R.L. 270.0 must not be exceeded. The alternative, therefore, is the installation of gates and siphons.

2. Discharge Capacity.

The catchment area of Lake Arthur is 47 sq. miles and the maximum run-off discharge by the fan catchment formula,

$$Q = \frac{7,000 A}{\sqrt{A-4}} \text{ is } 46,000 \text{ cusecs.}$$

The spillway will have to be designed for this maximum as there will be practically no flood absorption due to the narrow margin between the proposed retention level and high flood level.

3. Choice of Spillway:

3.1. The following are the four alternatives.

- (a) manually operated gates.
- (b) fully automatic gates.
- (c) siphons.
- (d) siphons and gates,

(1) Annual Report of work done during 1937-38, Central Irrigation & Hydro-dynamic Res. Station, Poona pp. 39-44.

Alternatives (a) and (b) were too costly, considering the additional storage obtained. Alternatives (c) was cheaper than (d) but did not afford the desired flexibility of operation. A combination of siphons and gates was therefore only considered. The following proposal was made by Mr. Thomas, then Executive Engineer, Ahmednagar Division.

3.2. Combination of siphons and gates.

This is technically a more attractive arrangement than either gates or siphons alone as it provides a proportion of the advantages of both. If gates were provided for one third of the discharge and siphons for two-thirds, the control obtainable with gates would be available and also much of the economy and reliability of siphons. The control provided by the gates would be useful in causing early priming of the siphons both in general practice during floods and for the purpose of tests and observations. Taking the capacity of siphons to be 36,000 cusecs (about 2/3 of 51,000 cusecs), 75 units would be required occupying 492 ft. For the gates, allowance made for flood absorption may be omitted to enable them to be used for priming siphons. Maximum discharge through the gates will be 15,000 cusecs and it would be necessary to provide a total width of at least $\frac{15,000}{83} = 181$ ft. as 83 cs/ft. is the maximum intensity over the cut. To allow for a reduced rate of discharge near right bank due to recessed approach and loss of head at gates, a total width of 205 ft. may be provided for gates and piers by widening the cut by 47 ft.

Siphons	497 0 ft.
Pier between syphon & gate	10.0 „
6 gates of 30' each	180.0 „
5 piers of 2'-3"	125.0 „
Side pier	2.0 „
Total.	<u>701.5 ft.</u>

Owing to the concentration of discharge approaching the limit for the approach channel, the minimum restriction should be placed on the flow. A sill 0.5 ft. high could however be provided. The gates would then be 9.5 ft. high. It would be desirable to lower the downstream level by 1 to 2 ft.

4. Economic Aspect of the Scheme.

4.1. Extra Storage.

Storage up to R.L. 269.5 (Proposed sill of siphon) ...	11,984 M. Cft.
Present storage up to sill of weir R.L. 265.7 ...	11,342 „
	<u>642 „</u>

FIG. 82
PLAN SHOWING THE EXISTING
WASTE WEIR AT BHANDARDARA
AHMEDNAGAR DIVISION.

REFERENCES:-
2437-63 ABOVE M.S.L. = R.L. 260-00
FULL ROCK DROP LINE ———
SOFT ROCK & SOIL LINE S.L. - - - -

SCALE 1" = 360' FT.

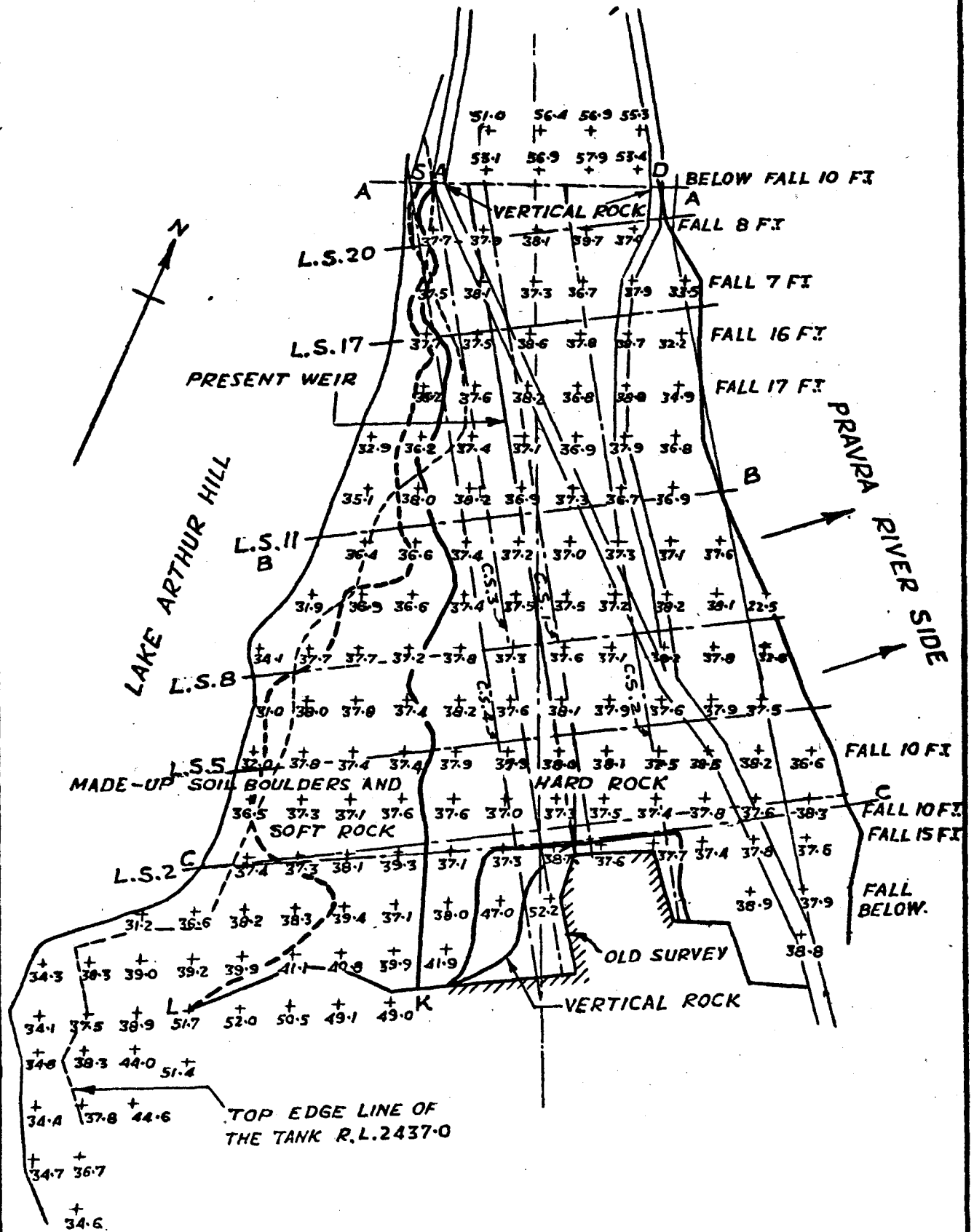
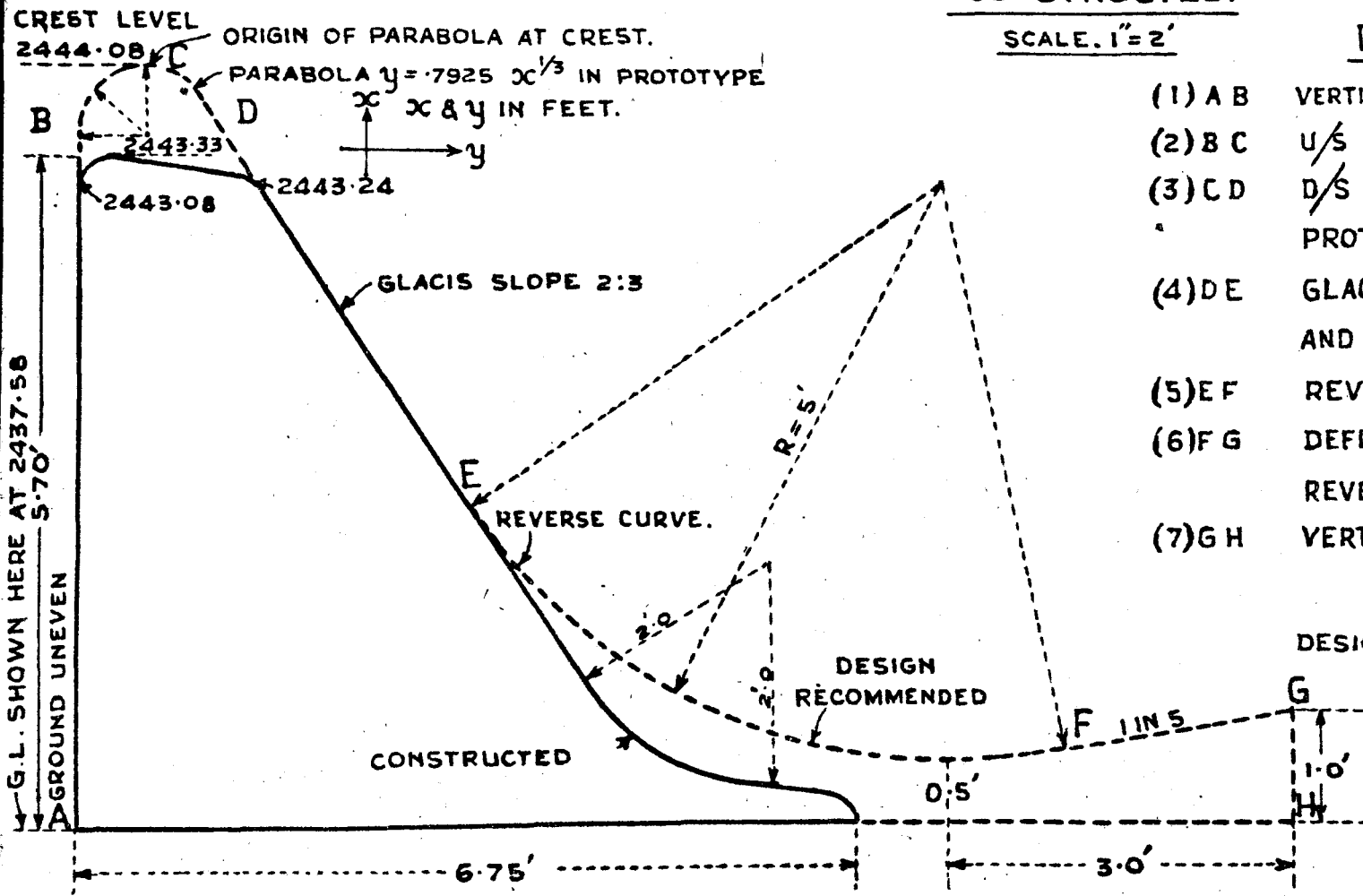


FIG. 83
SECTION OF H.C. WEIR PROFILE NO. VIII B
PROPOSED FOR RAISING WASTE WEIR OF LAKE ARTHUR
HILL, BHANDARDARA AND SECTION AS
CONSTRUCTED.



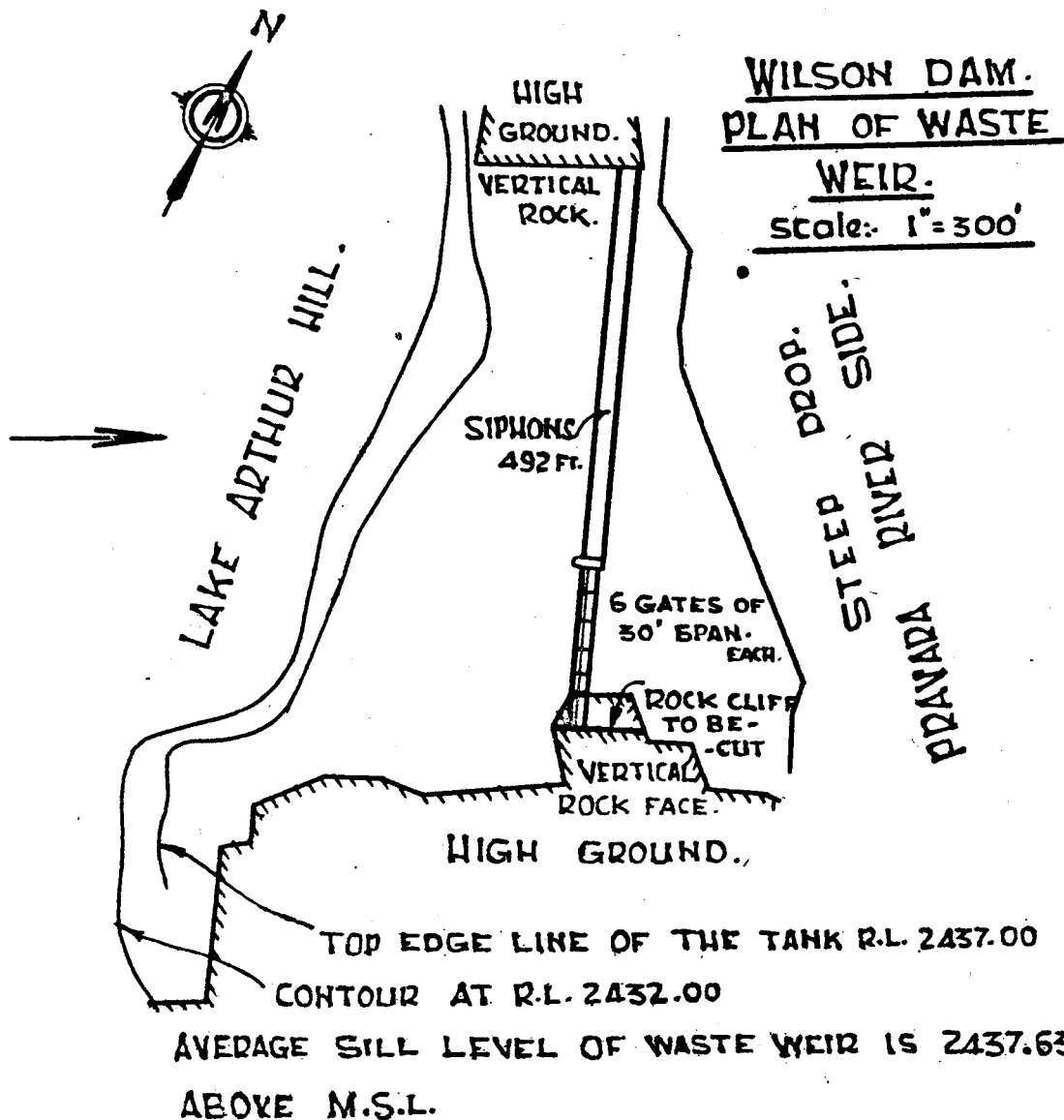
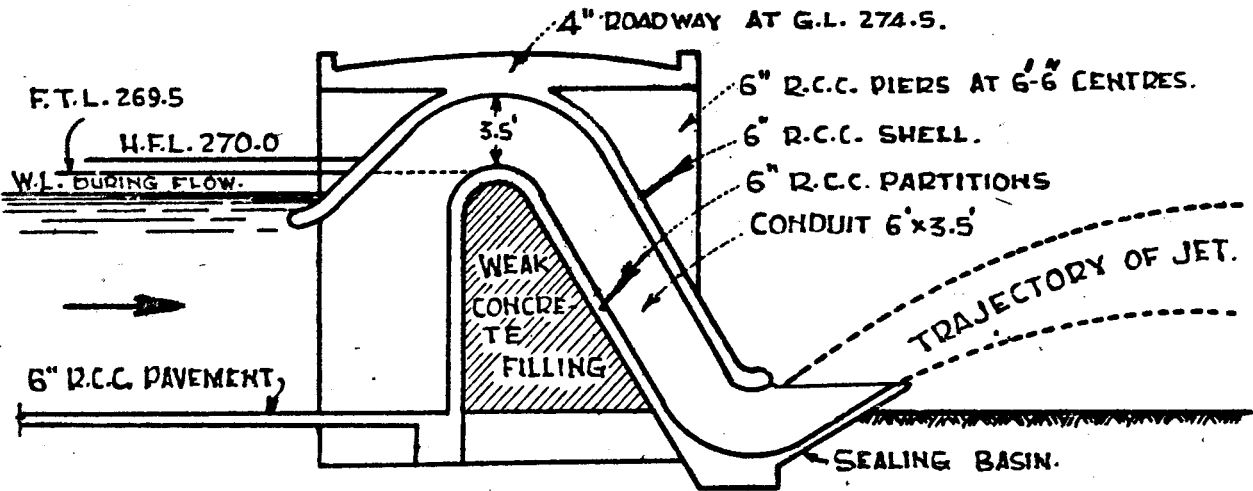
SCALE. 1" = 2'

DETAILS OF ORIGINAL DESIGN.

- (1) A B VERTICAL U/S FACE
- (2) B C U/S CREST QUADRANT OF A CIRCLE R=0.6'
- (3) C D D/S " PARABOLA $y = .7925 x^{1/3}$ FOR PROTOTYPE. (x & y IN FEET.)
- (4) D E GLACIS SLOPE 2:3 TANGENT TO PARABOLA AND REVERSE CURVE,
- (5) E F REVERSE CURVE R = 5'
- (6) F G DEFLECTOR 1 IN 5 TANGENT TO REVERSE CURVE
- (7) G H VERTICAL DROP 1'

PARKH. 20-1-49

3.5 FT. DIA. SIPHON OPERATING UNDER 8 FT. HEAD
DISCHARGE 480 CS. PER 6 FT. X 3 FT. UNIT.
Scale:- 1" = 10 FT.



Cost of siphons and gates (1947 figure) ...	4.78 lakhs
Value of additional storage at 3½% capitalization (Rs. 120 Per million Cft. as water charge) ...	22.5

Fig. 84. Figure 84 shows the design of siphon and the alignment of siphons and gates proposed by Mr. Thomas.

5. Experiments.

5.1. The investigation could be divided into two parts:—

(i) Experiments to determine the discharging capacity of the waste-weir cut, lines of flow and water profiles with different upstream water levels. This data was essential for assessing available head and depriming levels for siphon and optimum alignment of siphons and gates.

(ii) Design of siphons and gates to pass 80 to 83 cusecs per ft. so as to provide a total discharging capacity of 51,000 cusecs.

5.2. To carry out the first part of the investigation in a geometrically similar model of the whole waste-weir with as large a scale as possible was desirable. 1/30 scale was selected as it gave high Reynold's Number (= 35,000) and was the biggest that could be constructed in the available space. This also provided as an opportunity for comparing with the previous results of 1/30 G.S. Model.

5.3. Reproduction of roughness in model.

The waste-weir, on the whole, is very rough and the roughness varied considerably in different parts. This was an all important point and the reliability of the experiments depended upon correct reproduction in the model.

To attain exact hydraulic similitude between proto-type and model, the roughness in the model would have to be reduced to get scalar rugosity coefficient. This could only be done by altering bed roughness till water profiles and lines of flow were correctly reproduced in the model i.e. a process of trial and error.

Exact reproduction of bed with any number of cross-sections observed was practically out of consideration due to the extreme roughness of the bed, the undulations being as much as 2 ft. The following method was, therefore, adopted as the best approximation to correct representation of bed conditions.

Photographs
1295, 1296,
1300.

Fig. 82.

In the 1/30 geometrically-similar model, spot levels at 1.5 ft. ($\approx 5'$) intervals and few cross sections—see fig. 82—were reproduced. This gave the mean level of the waste weir. Protuberances in different parts of the waste weir were observed in the proto-type. They were reproduced in the model by sticking stone aggregate with sharp corners, on cement mortar bed. The size of stones varied from $\frac{1}{4}$ " ≈ 1.9 ft. to $\frac{3}{8}$ " ≈ 0.95 ft. In these experiments the existing waste weir wall was omitted

Photo

5266.

Upstream portion of the waste-weir, where the bed was not very rough and was slowly undulating with isolated boulders was represented by sticking gravel ($\frac{1}{4}$ "- $\frac{3}{8}$ " dia.) on the bed. A general view of the model is shown in Photograph 5266.

5.4. Discharges over the waste-weir were run with water levels in the lake varying from R.L. 263.5 to R.L. 270.3 and lines of flow and water profiles were observed for each discharge.

6. Results of experiments.

6.1. Discharge observations.

Table 1.

Table 1 shows water levels and discharges in the proto-type. Equivalent discharges are plotted against water depths above mean waste-weir level in fig. 85, shown by full line. These are surprisingly close to the previous results shown by dotted curve. The relation previously obtained $Q = 608 d^{1.95}$ where Q is the discharge in cusecs, 608 ft. is the waste-weir length and d depth of water, was thus confirmed.

Fig. 85.

6.2. Lines of flow.

Depths in the model being very small, and interference due to bed roughness was large, lines of flow were observed at the surface only by wool thread. Figure 86 shows lines of flow for discharges equivalent to 30,000 cusecs and 44,800 cusecs. These showed a tendency to shift towards the right, this being due to sudden expansion at the right bank.

Fig. 86.

(a) & (b)

Photos

5263, 5264

& 5265.

6.3. Manning's 'N' in model

Manning's 'N' for different parts of the model was assessed from water profiles for 44,800 cusecs discharge.

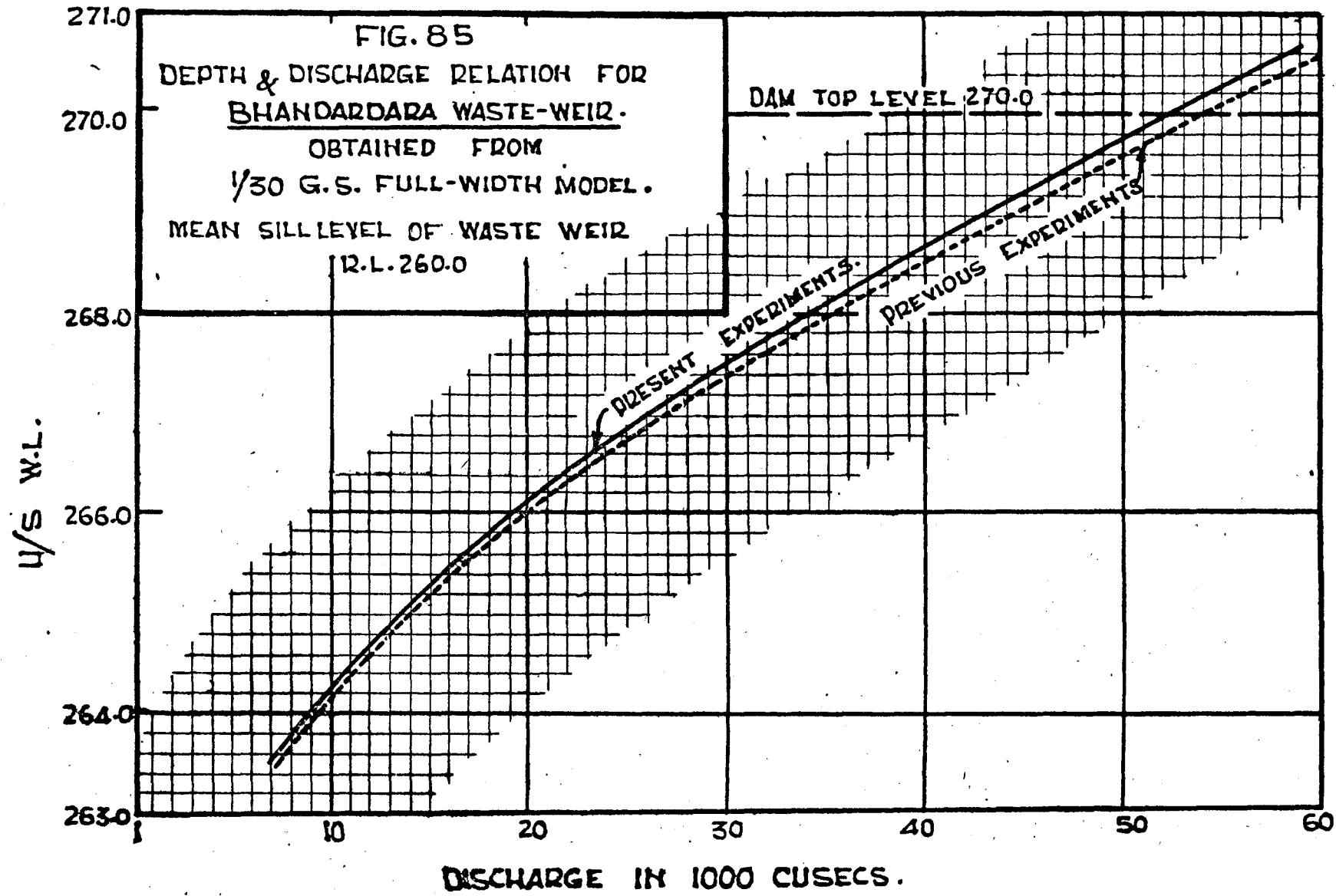


FIG. 86 (a)

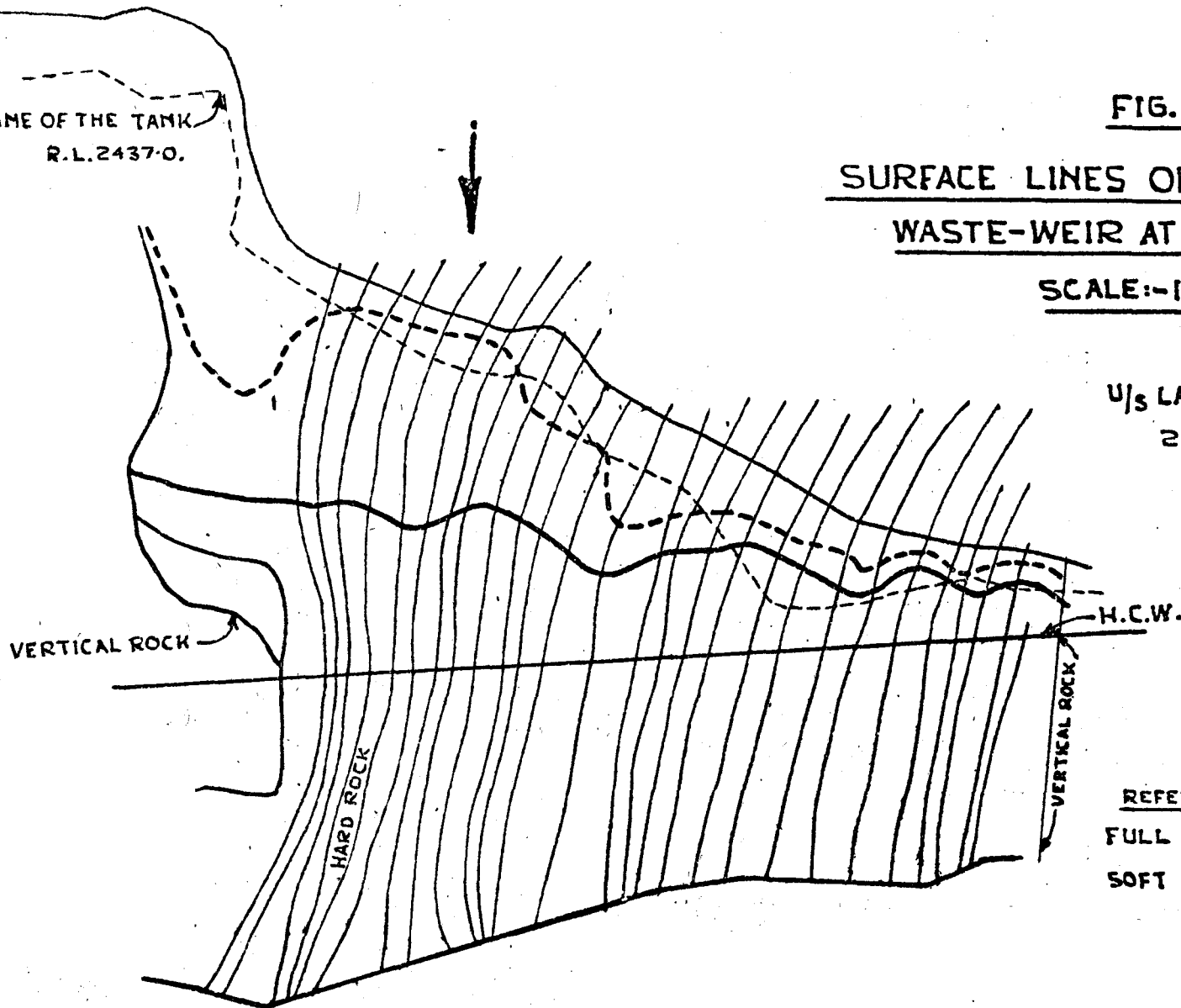
SURFACE LINES OF FLOW FOR
WASTE-WEIR AT BHANDARDARA.

SCALE:- 1" = 12' = 360'

REFERENCES:-

U/S LAKE LEVEL.
267.53

DISCHARGE.
30,000 CS.



REFERENCES:-

FULL ROCK DROP LINE, ————
SOFT ROCK & SOIL LINE, - - - - -

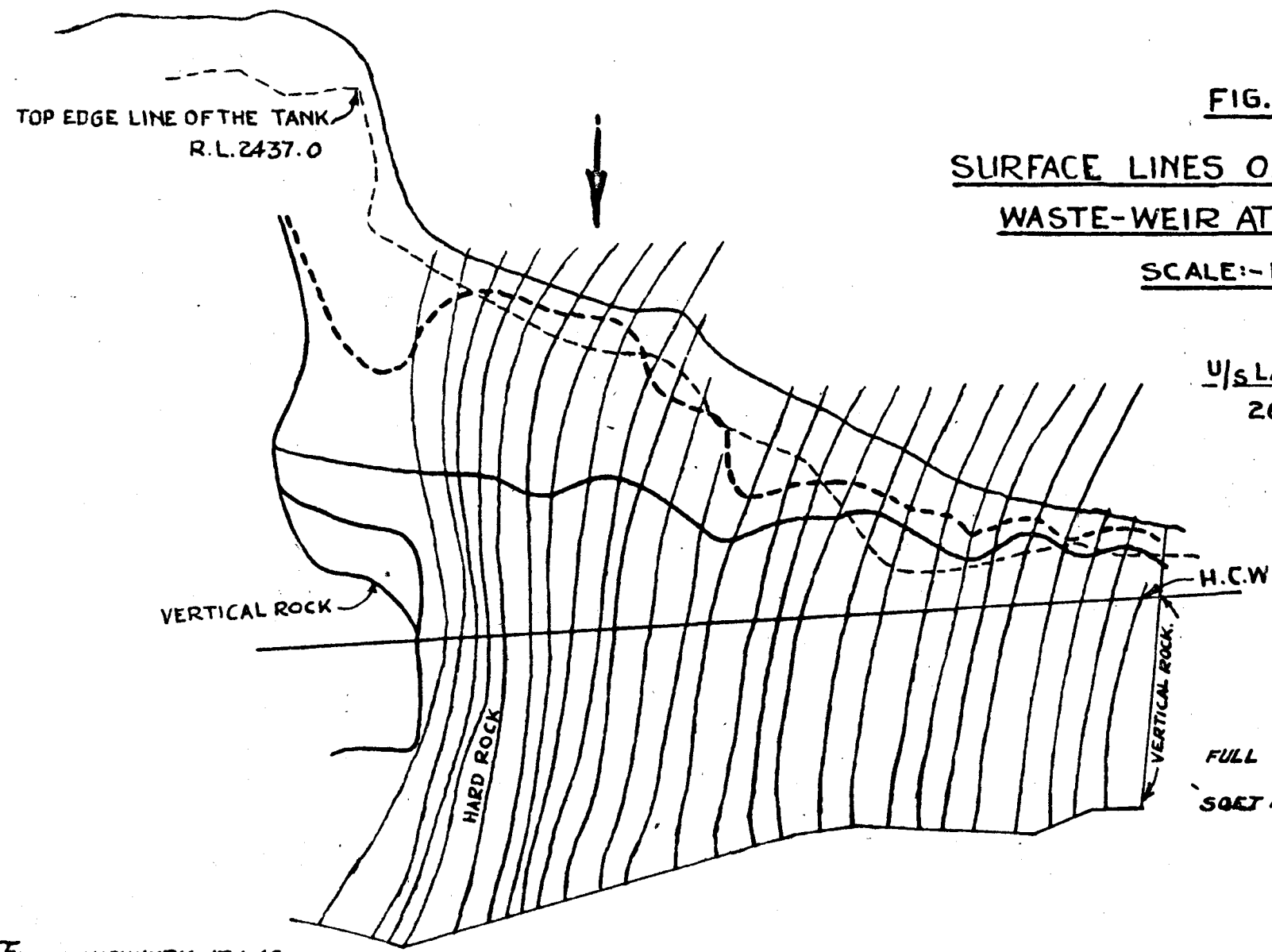
FIG. 86 (b)

SURFACE LINES OF FLOW FOR
WASTE-WEIR AT BHANDARDARA.

SCALE:- 1" = 12' = 360'

REFERENCES:-

<u>U/S LAKE LEVEL.</u>	<u>DISCHARGE.</u>
269.50	44,800 Cs.



FULL ROCK DROP LINE. ———
SOFT ROCK & SOIL LINE - - -

By MANCHUNDIA. 17.1.49

Sections 2, 5 and 8 were divided into two lengths of observation, (1) upstream portion which was comparatively less rough and (2) the downstream portion. These were considered separately. For sections 11, 17 and 20 which are short and more uniformly rough, one value of 'N' is obtained.

Water-levels upstream and downstream for each reach of observation are given in Table 2, column (3). Values of Manning's 'N' are given in column 11.

It will be seen that the weir is comparatively more rough on the left bank and downstream of the present weir. Average value of 'N' for the whole waste-weir comes to 0.059.

7. Siphon experiments.

7.1. Two important requirements of the siphons were :

(i) a low priming depth. This was essential as sufficient additional storage could only be obtained by increasing the retention level to as near as R.L. 270.0 i.e. the high flood level. To fix the retention level at R.L. 269.0, the lowest permissible, a priming depth less than 0.75 ft. is necessary.

(ii) The siphons must pass 80 cusecs /ft. to pass the maximum flood of 48,000 cusecs according to fan catchment formula, leaving a satisfactory margin of safety.

7.2. The preliminary design of siphon spillway suggested by Mr. Thomas, Executive Engineer had a very low discharge capacity. The design was modified by increasing the depth of the barrel from 3.0 ft. to 4.5 ft.

Fig. 84.

When experiments were carried out with a 1/6th scale model, it was found that the bucket provided at the end of the siphon was not effective in sealing the lower lip of the siphon during priming neither the jet projected as anticipated by Mr. Thomas. Due to these defects the priming depth obtained was very large viz. ≈ 2.5 ft. and the discharge intensity per foot width through the siphon was only ≈ 45.0 cusecs (with water level in reservoir ≈ 270.0 i. e. at M.F.L.)

Fig. 87.

In order to counter-balance the reduction in effective head in the first design. As the outlet of the siphon was lowered as shown dotted in fig. 87, thus increasing the difference between the crest of the siphon and lower lip from 9.0 ft. to 11.75 ft. In this second design though the discharge intensity per foot width increased to 50.0 cusecs; the priming depth was still large being ≈ 2.3 ft.; the discharge coefficient obtained was only 0.14.

Fig. 88.

7.3. The siphon design was then modified as shown in fig. 88. Two siphons, each 1 ft. \equiv 8.0 ft. were constructed in 2 ft. wide channel to a scale of 1/8. Head of the siphon was increased by cutting downstream rock to R. L. 248.0. The throat height was reduced to 4.0 ft. because it was observed in the previous experiments, with 4.5 ft. high throat, that water did not follow the upper curve. The downstream glacis was given a slope of 7° instead of 15° . It was anticipated that with steeper glacis water would leave the crest earlier and strike the opposite leg at a higher level thus causing early priming.

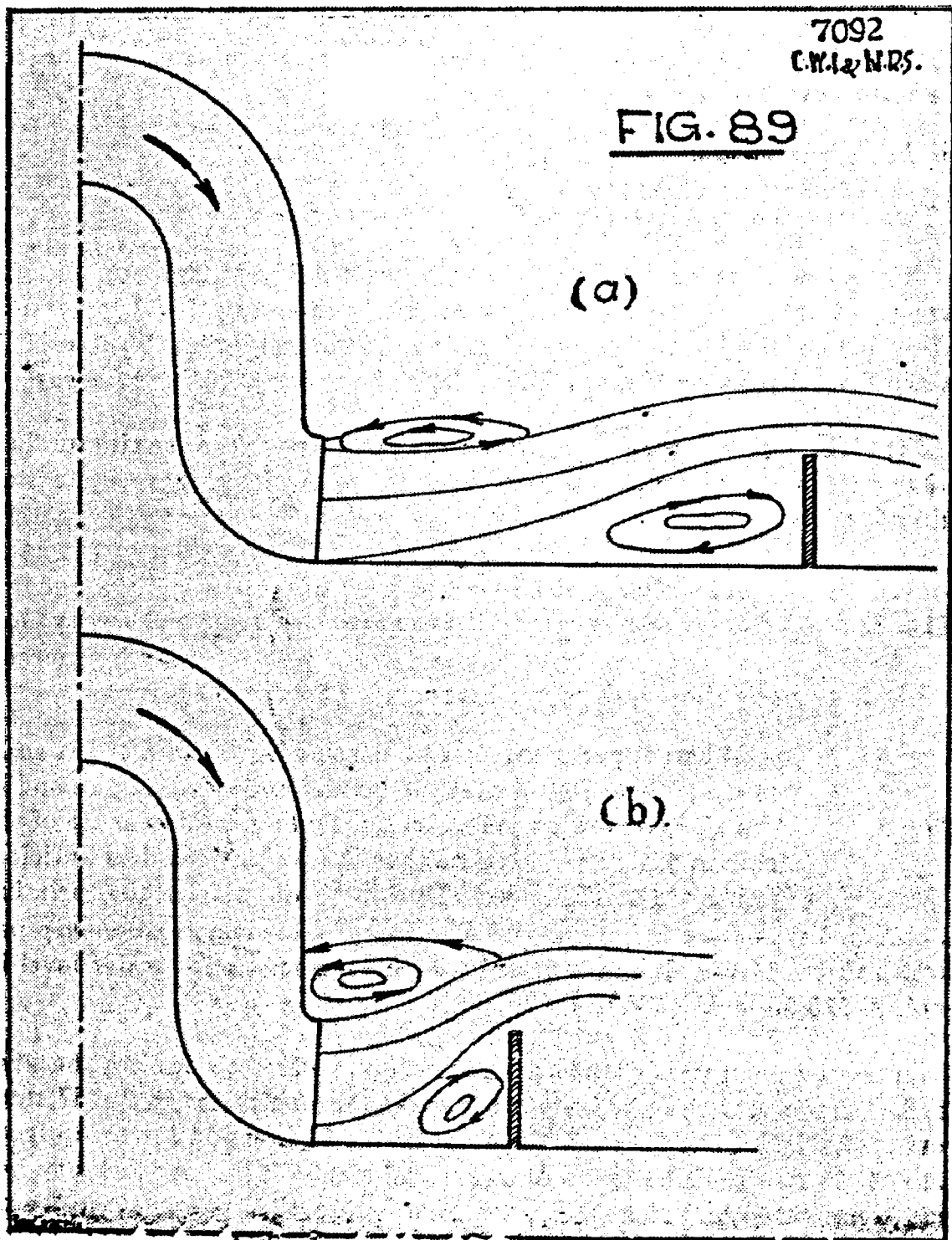


FIG. 87

BHANDARDARA SYPHON.

SCALE:- 1" = 10' = 5'

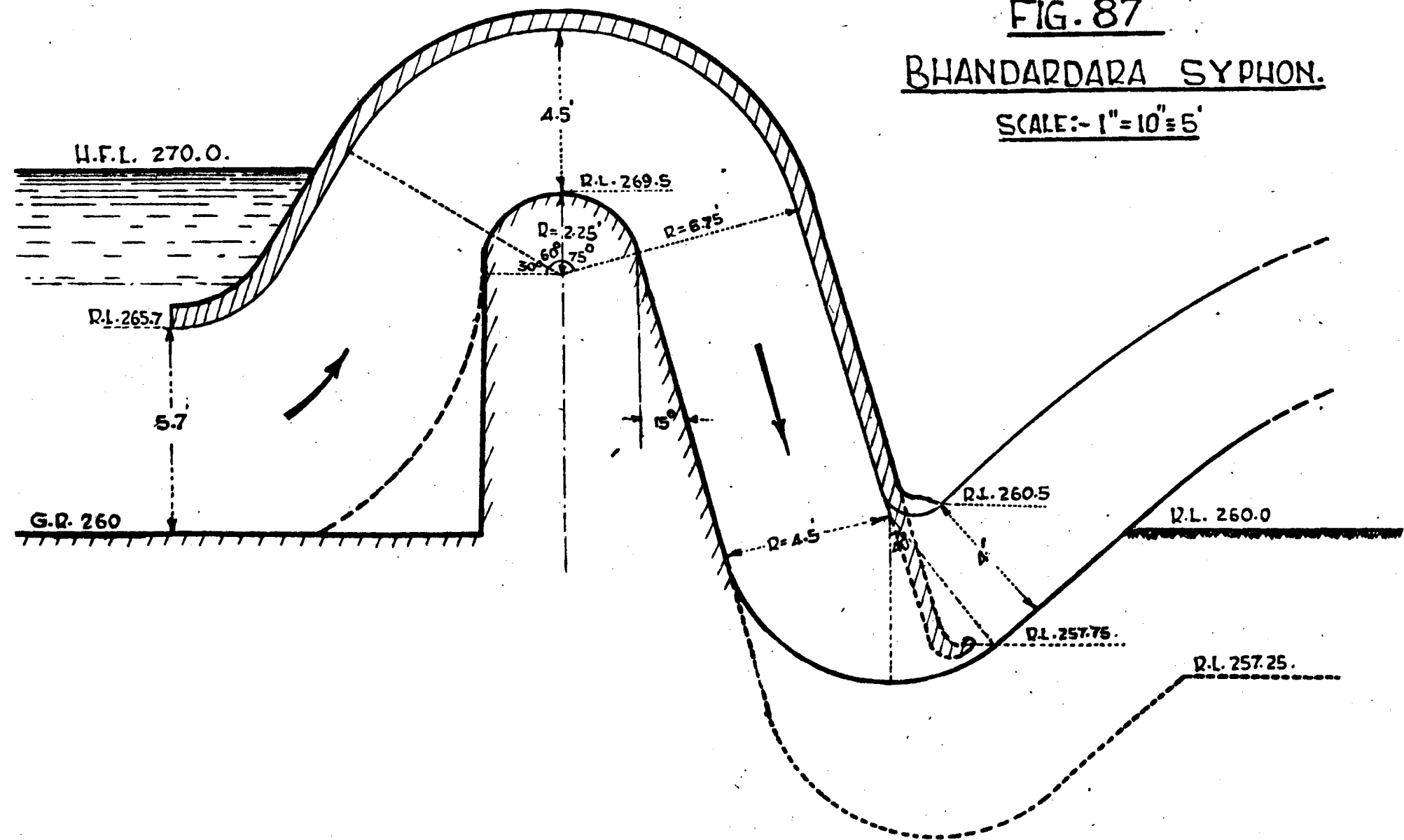
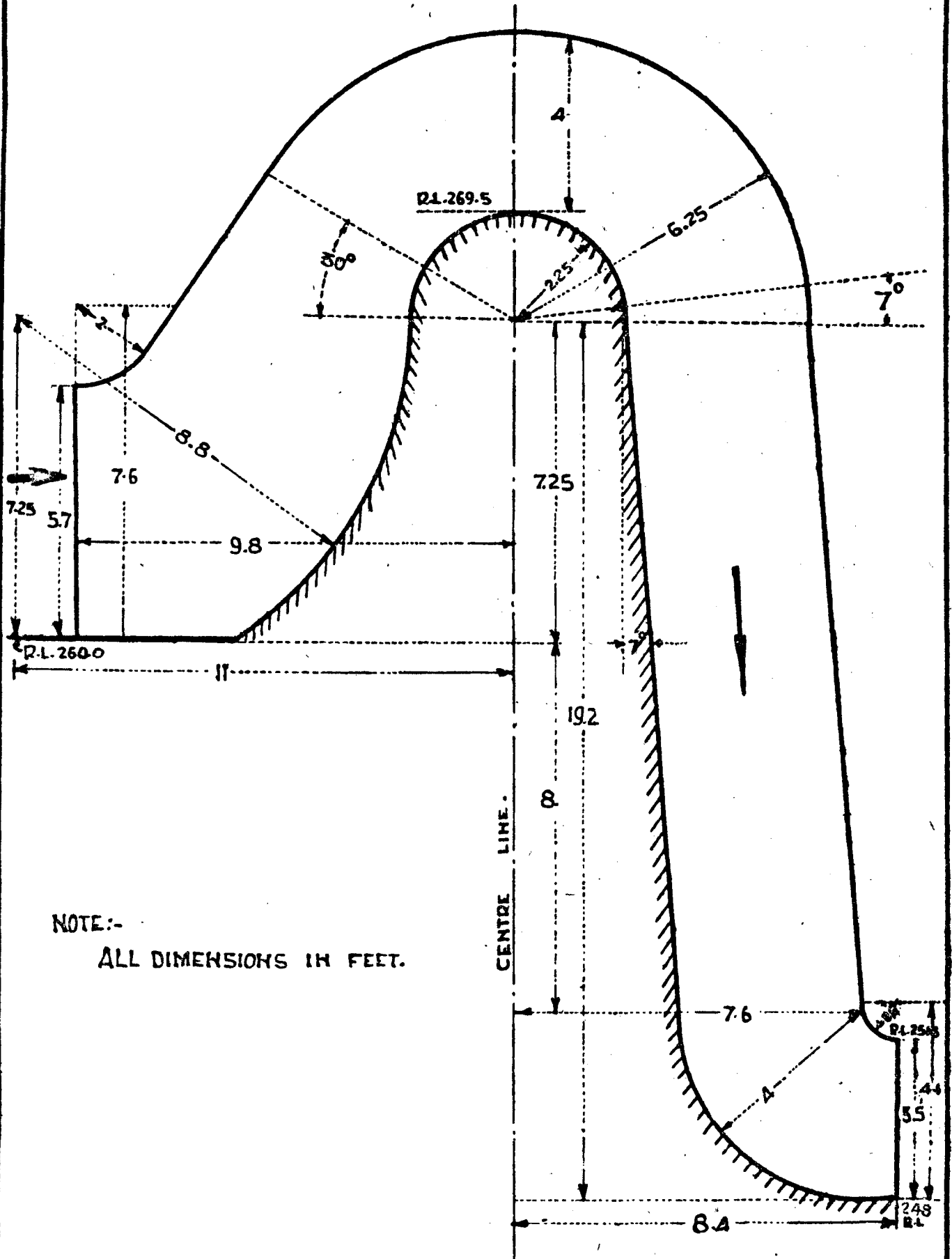


FIG. 88

BHANDARDARA SYPHON.

SCALE:- 1" = 0.5' = 4 FT.



NOTE:-
ALL DIMENSIONS IN FEET.

WASTE WEIR AT BHANDARDARA
(BOMBAY).

(Photographs were taken before H.C.
weir wall was constructed).

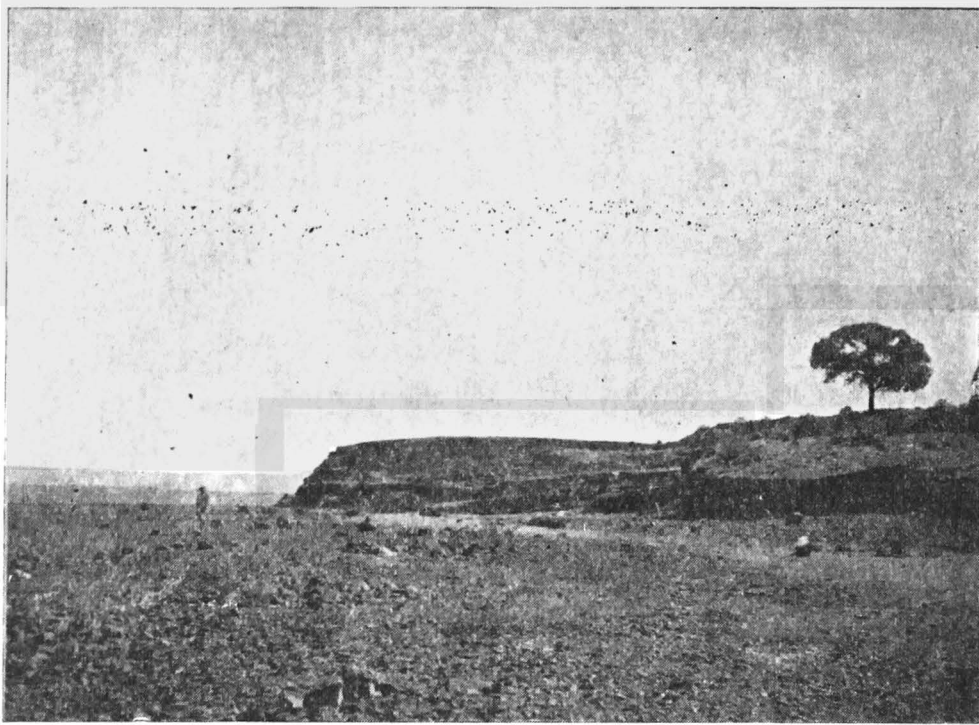


PHOTO No. 1300. Photograph showing
upstream portion of weir on Right Bank.



PHOTO No. 1296. Photograph showing
downstream portion of weir on Right
Bank.



PHOTO No. 1295. Photograph showing
downstream portion of weir on Left
Bank.

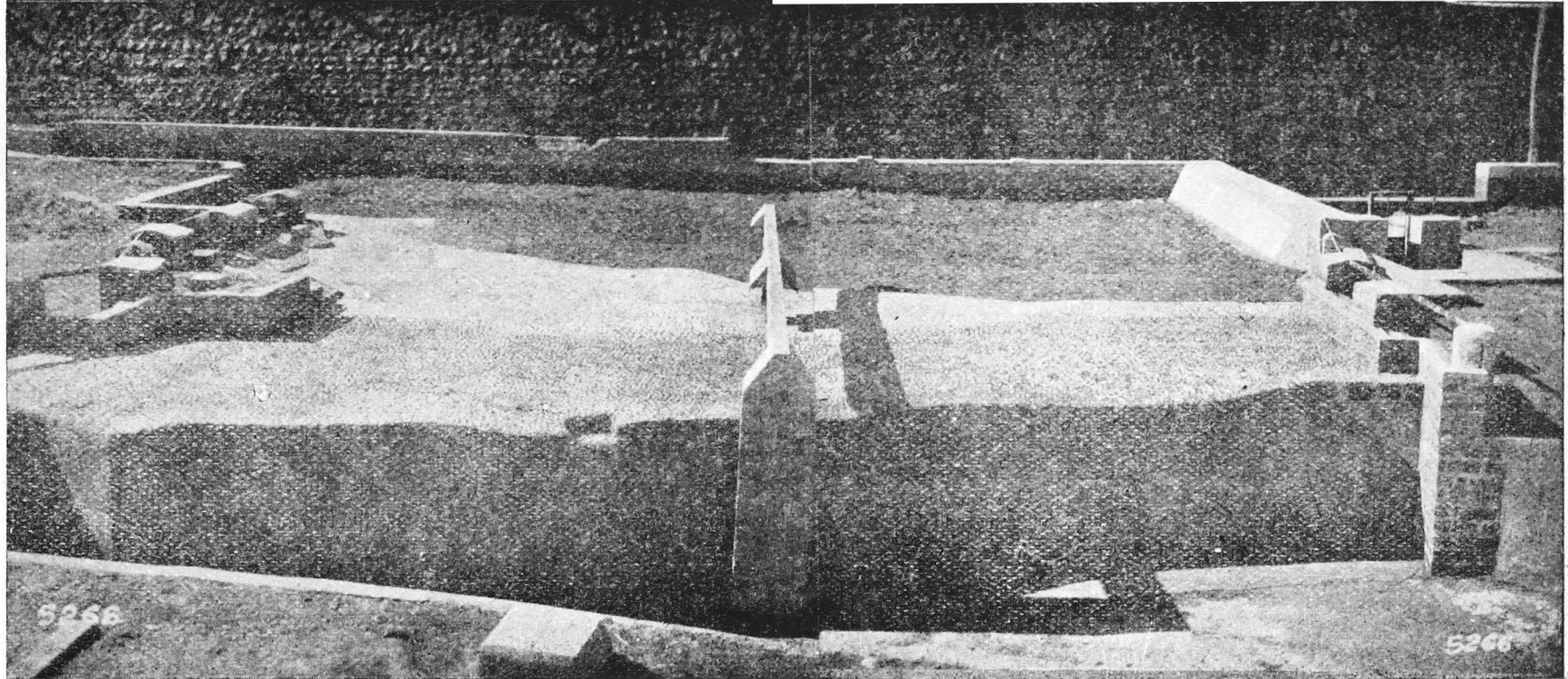


PHOTO No. 5266. Photograph showing bed-roughness in 1/30 G. S. Model of Waste-weir at Bhandardara.

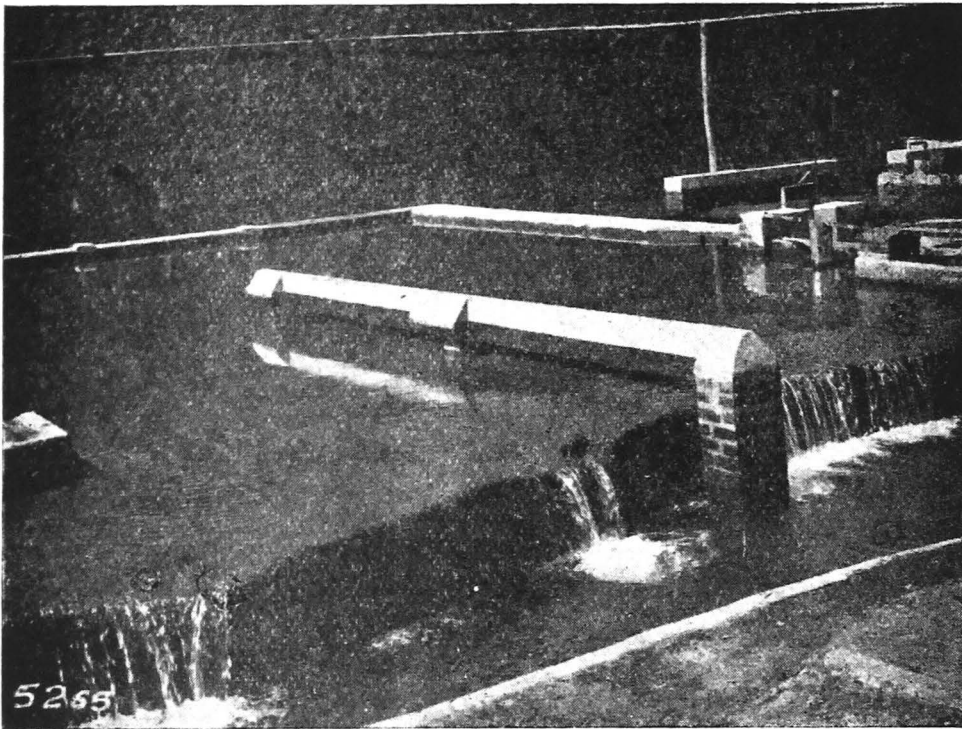


PHOTO No. 5265. 1/30 G. S. Model of
Waste-weir at Bhandardara discharging
6,825 Cs.

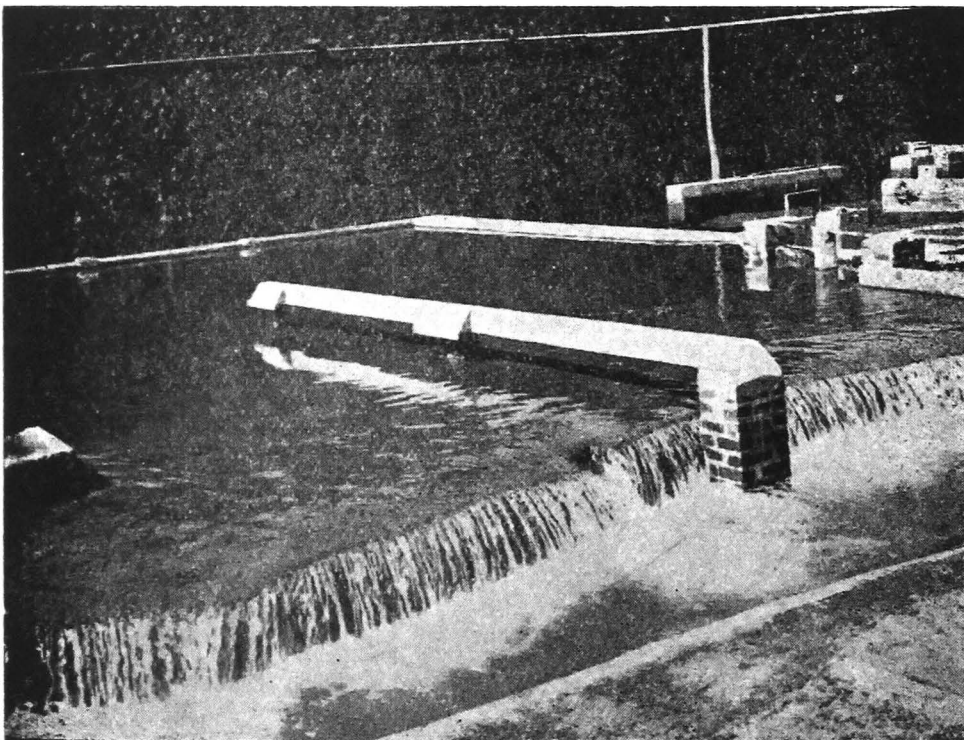


PHOTO No. 5263. 1/30 G. S. Model of
Waste-weir at Bhandardara discharging
36,700 Cs.

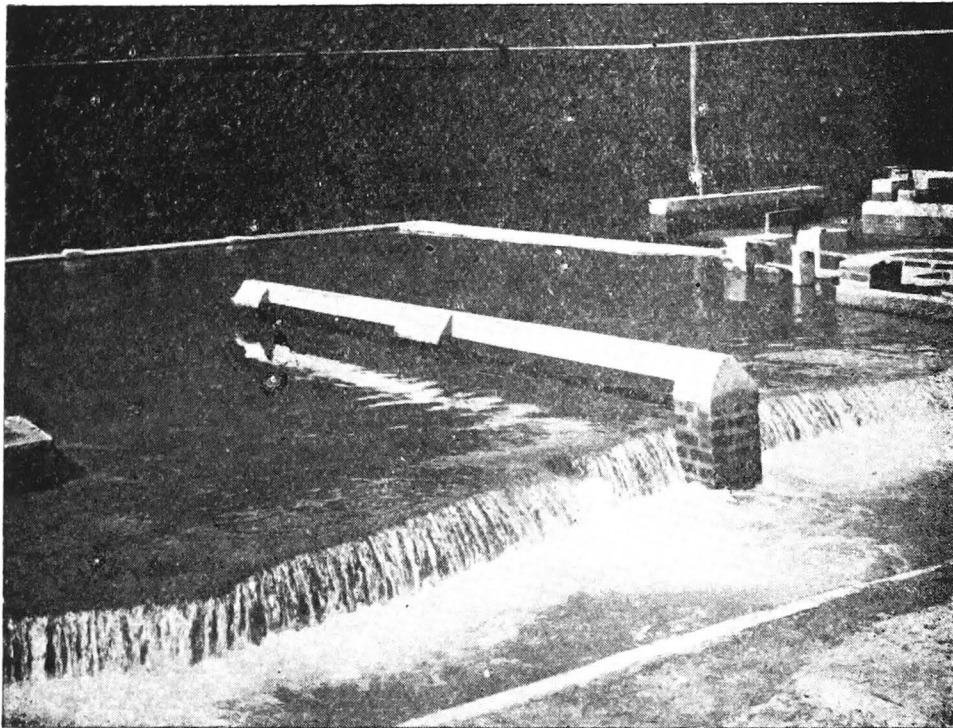


PHOTO No. 5264. 1/30 G. S. Model of
Waste-weir at Bhandardara discharging
52,000 Cs.

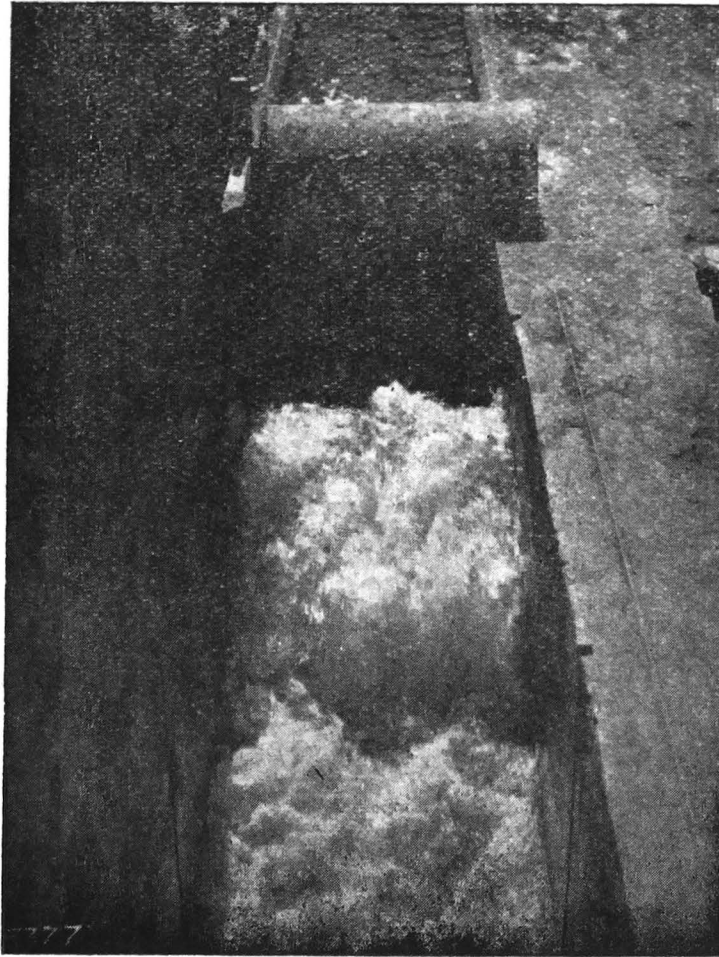


PHOTO No. 5377. Conditions downstream of 1/8 scale syphons. Priming weir 3.92 ft. high at 20.8 ft. downstream of syphons.

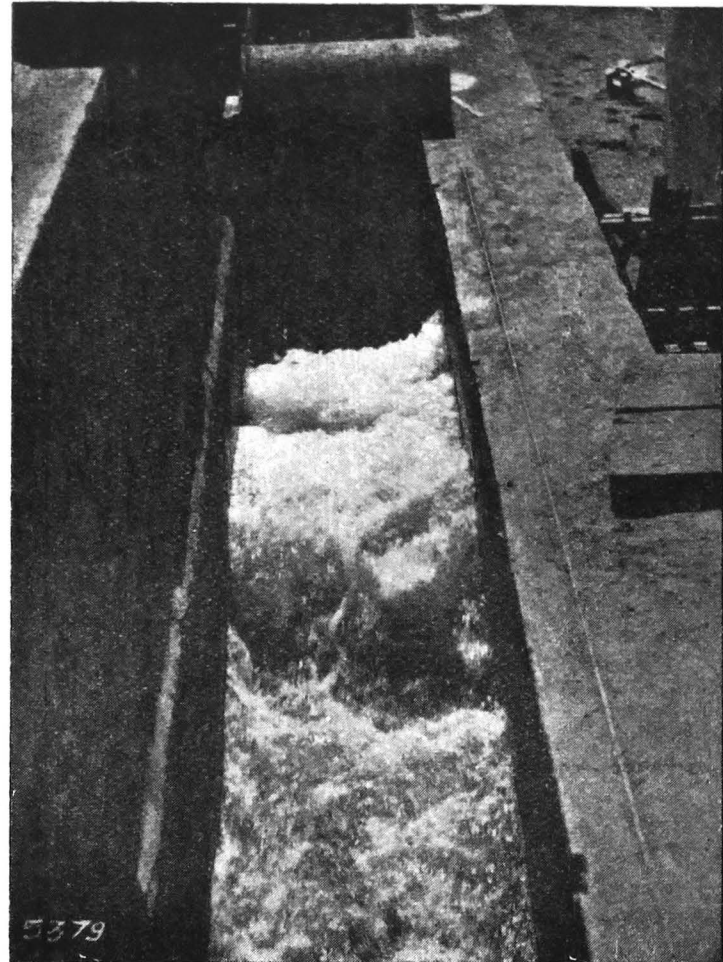


PHOTO No. 5379. Conditions of downstream of 1/8 scale syphons. Priming weir 4.4 ft. high at 61.6 ft. downstream of syphons.

Priming weir (in the form of vertical planks, to be later properly designed) were tested and found to be better than priming buckets.

8. Results of experiments.

8.1. Results are given in Table 3. The priming depth with this design considerably improved, being ≈ 0.64 ft. with a priming weir 3.92 ft. high, fixed 18.8 ft. downstream of the siphon, the discharge was ≈ 59.5 cusecs/ft. A discharge of 68.9 cusecs/ft. could be passed with a priming weir 4.4 ft. high at 61.6 ft.; the priming depth however slightly increased to ≈ 0.72 ft.

Fig. 89. 8.2. As one would expect, the priming weir when (a) & (b) placed nearer the siphon improved the priming qualities, and Photos but reduced the discharge by causing back pressure, than 5377 & 5379 when the weir fixed further downstream. (See Fig. 89)

9. Conclusion.

(i) Though the design is satisfactory from the point of view of low priming head, its discharge has to be increased to 80 cusecs/ft. Experiments are in progress to improve the discharging capacity.

(ii) It was also noted that to get the necessary head, rock will have to be cut to a depth of 10 ft. or so and in order to reduce the quantity of rock cutting, the siphons will have to be constructed nearer the downstream end of the waste weir than previously anticipated. Economics of the proposals will thus have to be carefully investigated.

Table 1.

Sl. No.	Upstream water level in the lake. ft.	Depth of water over waste-weir (Mean waste weir sill level 260.0) ft.	Discharge cusecs.	Discharge cs./ft. (Length of waste-weir 610').	Remarks.
1	2	3	4	5	6
1.	263.53	3.53	6,825	11.2	
2.	264.53	4.53	11,150	18.3	
3.	365.52	5.52	16,850	27.6	
4.	266.51	6.51	23,000	37.8	
5.	267.53	7.53	30,000	49.3	
6.	268.49	8.49	30,700	60.02	
7.	269.5	9.5	44,800	73.5	

Table 2

Serial No.	Section No.	Part of section for which drop in water level is considered.	Length of section. ft.	Depth of water at u/s & d/s of section. ft.	R Mean depth of water. ft.	Drop in water level. ft.	S Surface slope ft./1000 ft. Col. 7 x1000 Col. 4	Average q.ft. for waste weir. cs./ft. 9	V= Velocity in ft. sec. Col. 9 Col. 6	N (Mannings) 1.486 R ^{2/3} S ^{1/2}
1	2	3	4	5	6	7	8	9	10	11
1.	C.S. 2	+310 to +40	270	9.48	8.76	1.44	5.33	44,800	8.4	0.055
2.	.. 2	+40 to -135	175	8.04	6.58	2.91	16.5	610	11.15	0.06
3.	.. 5	+250 to +10	240	5.13	8.59	1.47	6.12	73.5	8.55	0.0565
4.	.. 5	+10 to -165	175	9.31	6.84	2.04	11.6		10.75	0.0535
5.	.. 8	+210 to 0	210	7.86	8.52	1.08	5.15		8.63	0.0511
6.	.. 8	0 to -145	145	5.82	6.7	2.55	17.5		11.0	0.063
7.	.. 11	+120 to -120	240	7.98	6.9	2.94	12.2		10.65	0.056
8.	.. 17	+75 to -110	185	5.43	7.63	2.18	11.8		9.65	0.064
9.	.. 20	+45 to -145	190	8.73	6.69	3.48	18.3		11.0	0.065
				6.54						
				8.43						
				4.95						

*Distance u/s High Co-efficient of weir are marked positive & d/s negative.
 (High Coefficient weir wall was not reproduced in these experiments).

Table 3

Sl. No.	Distance of weir from centre line of siphon.	Height of priming weir.	Priming water level, gauge 320 ft. u/s of siphon centre line.	R. L. of sill of siphon.	Priming depth.	Water level 320' u/s of siphon centre line.	Discharge q	Water level 33' u/s of centre line of siphon.	Velocity 33' u/s of syphon.	D'S lip level of siphon.	II. Total effective head Col.9+ Col.10- Col.11	Coefficient of siphon. C
1	ft. 2	ft. 3	4	5	ft. 6	7	8	9	ft./sec. 10	11	ft. 12	13
<i>1/6 Scale Syphon Model.</i>												
	No priming weir but a bucket.	-	272.2	269.5	2.5	270.0	45.0	269.36	0.36	260.5	9.22	0.41
		-	271.8	269.5	2.3	270.0	54.7	269.30	0.45	257.75	12.0	0.44
	Priming weir at 36.0	4.0	270.42	269.5	0.92	270.0	Not observed.					
<i>1/8 Scale Syphon Model</i>												
1.	61.6	4.4	270.22	269.5	0.72	270.0	68.9	268.82	0.95	251.5	18.27	0.49
2.	30.8	3.92	270.18	269.5	0.68	270.0	62.0	268.98	0.75	251.5	18.23	0.453
3.	18.8	3.92	270.14	269.5	0.54	270.0	59.5	269.16	0.67	251.5	18.27	0.435
4.	24.0	3.52	270.16	269.5	0.66	270.0	6.33	268.98	0.73	251.5	18.26	0.462

27. Experiments for dissipation of energy below the proposed Vaitarna dam spillway (Bombay).

1. Introduction.

1.1. The Special Engineer Bombay Corporation requested the opinion of the Director regarding the type of waste weir to be provided and the value of coefficient of discharge to be adopted in working out the depth of flood for passing the maximum flood discharge in connection with the proposed Vaitarna dam, contemplated to be constructed to cope with the increased demand of water supply to the city of Bombay.

1.2. Special Engineer after considering the merits and demerits of different types of spillways had come to the conclusion of a central spillway with a high coefficient weir (and 20 ft. high automatic shutters) as the cheapest proposition.

It was pointed out by the Director that having regard to the width of the gorge the proposal of siphon spillways should be reviewed in light of its overwhelming advantages such as automatic but dependable action, low first and maintenance costs, simpler and economical effective measures for dissipation of energy inspite of its disadvantages such as difficulty in raising the dam and F.S.L. at a later stage if found necessary and loss of flood absorption capacity as compared to that available in a broad crested weir.

1.3. The siphon spillways are now completely ruled out of serious consideration by the Special Engineer as it would be difficult to modify the design at a later stage if found necessary when the syphons were constructed. This was essential on account of paucity of data regarding discharge observations for sufficient number of years and hence he finally considered that the central high coefficient weir with crest at T.H.D. (i.e. Town Hall Datum) R.L. 55.10 should be now constructed and installation of automatic gates postponed till the estimated maximum discharge of 92,000 cusecs is substantiated by the actual measurement of the discharge over the weir.

1.4. After discussion with the Special Engineer, it is proposed to carry out only the following experiments at the Research Station in connection with the Vaitarna Scheme on behalf of the Bombay Corporation:—

- (a) To derive effective measures for scour protection downstream of the spillway.
- (b) To develop the shape of the weir crest to give the maximum coefficient of discharge with a water depth of 20 ft. over the weir.

- (c) To determine the pressures on the downstream profile of the spillway.
- (d) To ascertain the shape and size of guide walls downstream of the spillway to localise the flow within the bed width of the river.

For experiments (e), a full-width geometrically-similar model of the whole dam covering a reach $\frac{1}{2}$ a mile upstream and downstream of the dam is under construction. The experiments during the year were mainly carried out in a 1/30 geometrically-similar model for devising satisfactory measures for dissipating energy below the spillways and are given below.

In view of the constructional difficulties it is considered economical to locate the spillways centrally for passing a discharge of 92,000 cusecs over the net length of 280 ft. with 20 ft. depth above the sill (R.L. 515.0) Figure 90. shows the profile of the weir as proposed by the Special Engineer. The length of the weir viz. 280 ft. is fixed consistent with the existing width of the deep channel of the river. It is also proposed as stated above to install four automatic gates of 70 ft. each on the top of the spillway at a later stage.

Fig. 90.

As the tail water level, owing to the installation of the automatic gates will always be lower than levels at which dissipation of energy through hydraulic jump will take place satisfactorily the following two devices were considered possible in this case:—

- (a) To construct a short upward curving bucket so as to throw the sheet of water sufficiently away from the toe of the dam and thus prevent scour taking place in the immediate vicinity of the dam.
- (b) To construct a stilling pool by having a subsidiary weir.

As, however, there is solid rock in the bed of the river alternative (a) is considered cheaper, economical and quicker in construction.

The following experiments were carried out with $Q = 92,000$ cusecs for devising a suitable upward curving bucket:—

Experiment I. Without bucket.

Test 1. — with downstream water level R.L. 342.2

Test 2. — with downstream water level R.L. 326.7

Experiment II. With an upward curved bucket of radius $\cong 60'$

and height ≈ 5.0 ft for downstream water levels same as for experiment I.

Experiment III. With bucket of radius $\approx 80'$ and height $\approx 7.0'$ for downstream water levels same as for experiment I.

The object of running the maximum discharge with low water level downstream was to observe the depths of scour as such conditions are likely to occur in the river after the installation of automatic gates.

Though there is exposed rock in the bed of the river below the weir, pebbles of diameter varying from $3/16"$ to $1/2"$ were used in the bed of the model so as to compare qualitatively the efficiency of various measures tested in the model. For each experiment the bed was moulded to R. L. 310.0. In all the experiments a discharge of $4.67 \approx 92,000$ cusecs was run till scour stabilised. For each of the experiments the following data was collected:—

- (i) Water-profile downstream of the spillway.
- (ii) Profiles of scour downstream of spillway.
- (iii) Photos during the run of the experiments.

Fig. 91. Results of water-profile and profiles of scour are plotted in fig. 91 (See also photos).

Photos 5274, 5276, 5271, 5269, 5282 & 5284. Results of the scour are given below in table 1.

Table 1.

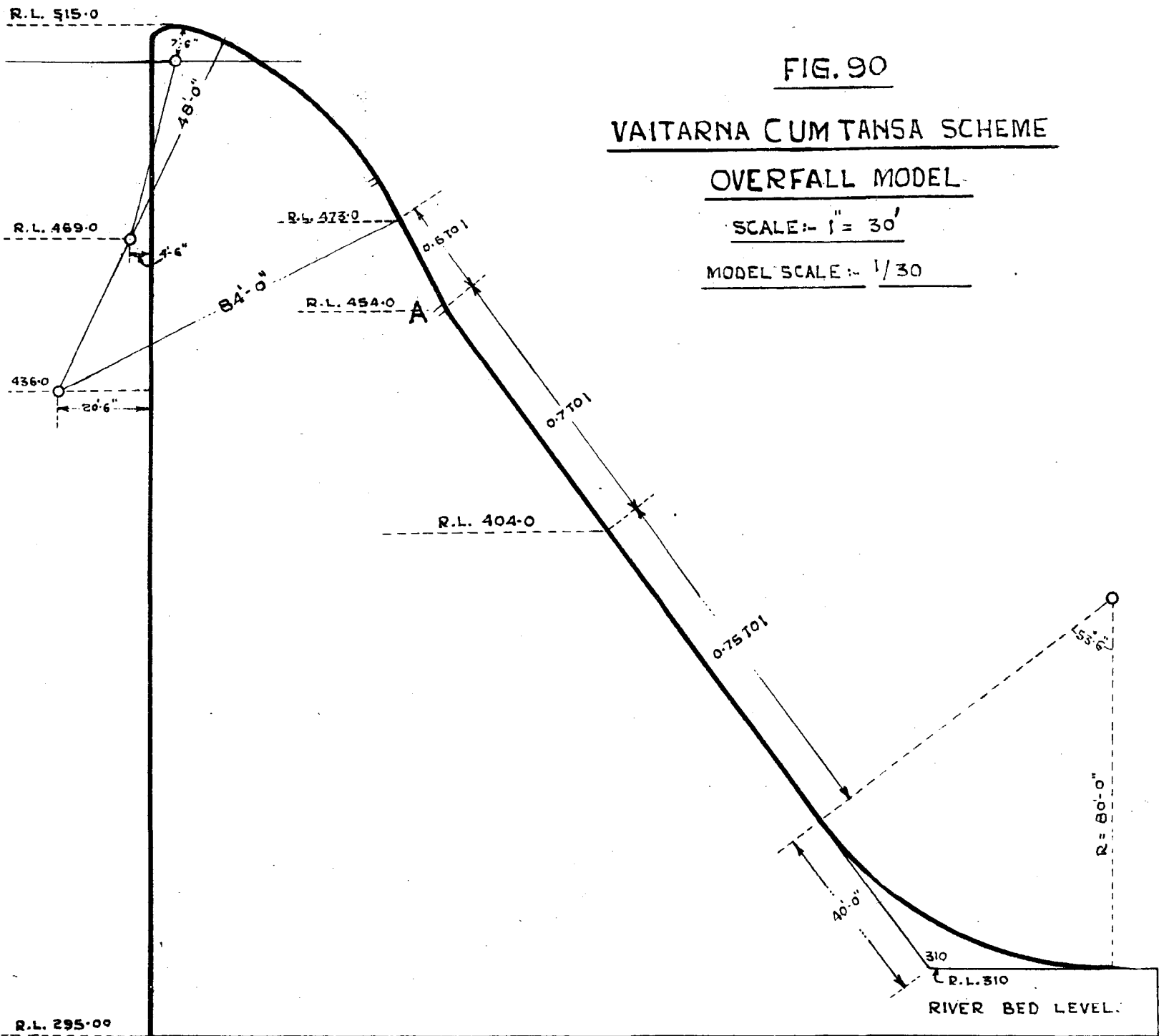
Expt No	Test No	Discharge (cusecs)	Downstream water level R. L.	Maximum scour in ft. below R. L. 3.0.0	Distance in ft. from toe of the weir	Conditions
I	1.	92,000	342.2	22.7	240.0	} Without Bucket.
	2.	92,000	326.7	33.0	150.0	
II	1.	92,000	342.2	18.6	405.0	} With 5 ft. Bucket.
	2.	92,000	326.7	33.0	255.0	
III	1.	92,000	342.2	22.2	420.0	} With 7 ft. Bucket.
	2.	92,000	326.7	33.9	300.0	

FIG. 90
VAITARNA CUM TANSA SCHEME

OVERFALL MODEL

SCALE :- 1" = 30'

MODEL SCALE :- 1/30



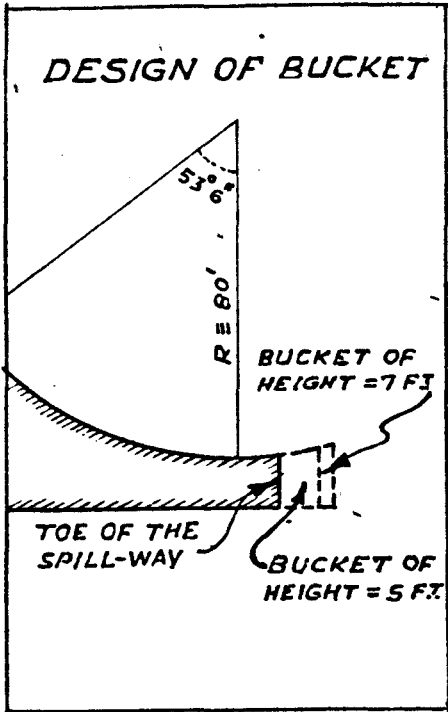


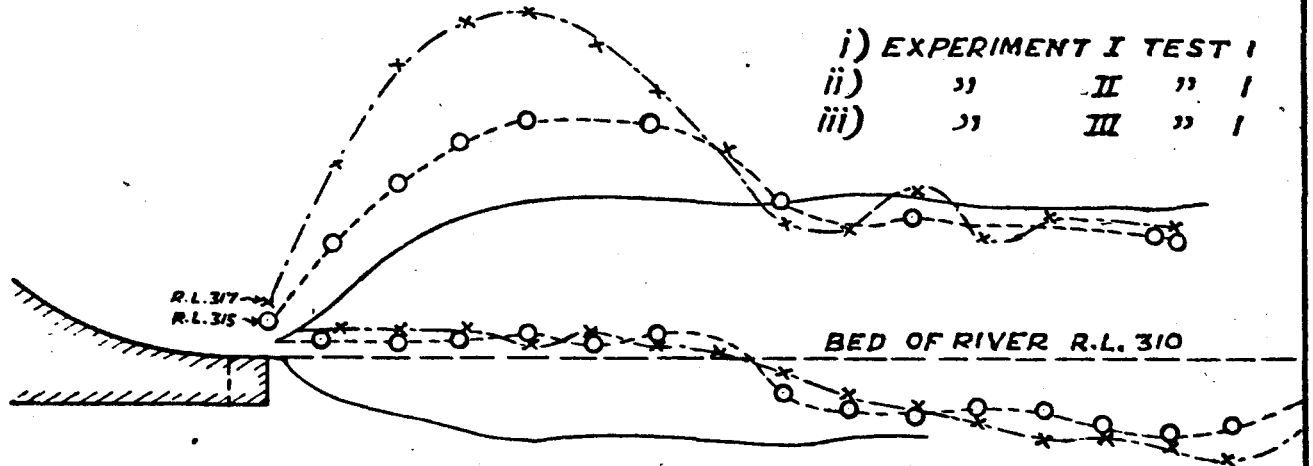
FIG. 91
SHOWING WATER PROFILES
AND SCOUR PROFILES D/S
OF THE SPILLWAY

SCALE:— HOR. 1" = 4' = 120'
VER. 1" = 2' = 60'

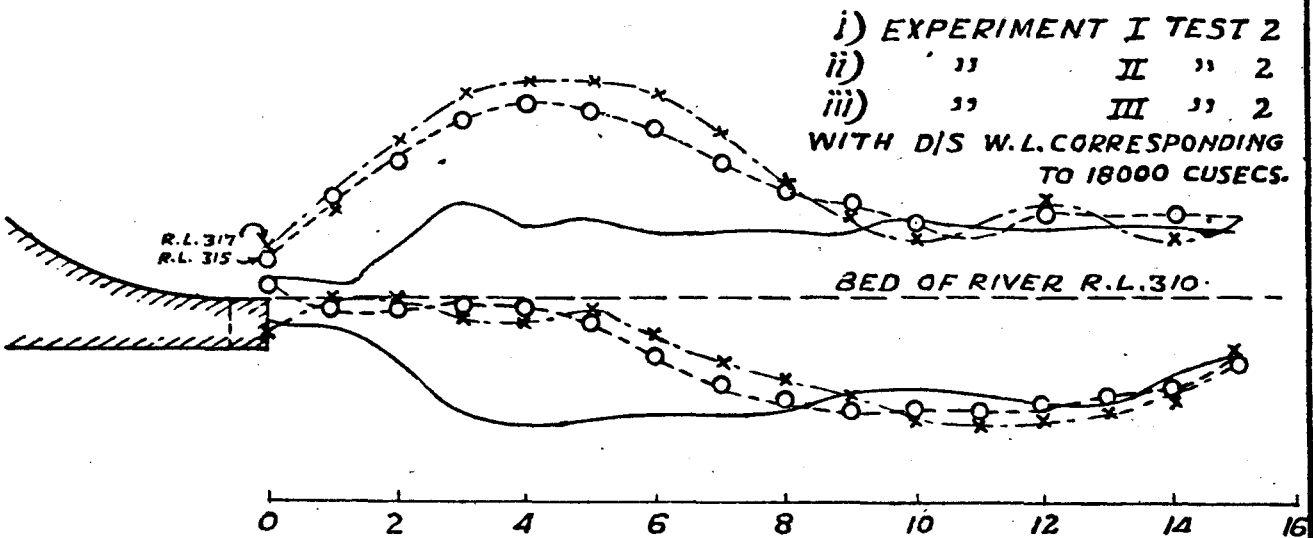
REFERENCES:—

- WITHOUT BUCKET
- -○- WITH 5' BUCKET
- x- -x- WITH 7' BUCKET
- EXISTING BED OF THE RIVER.

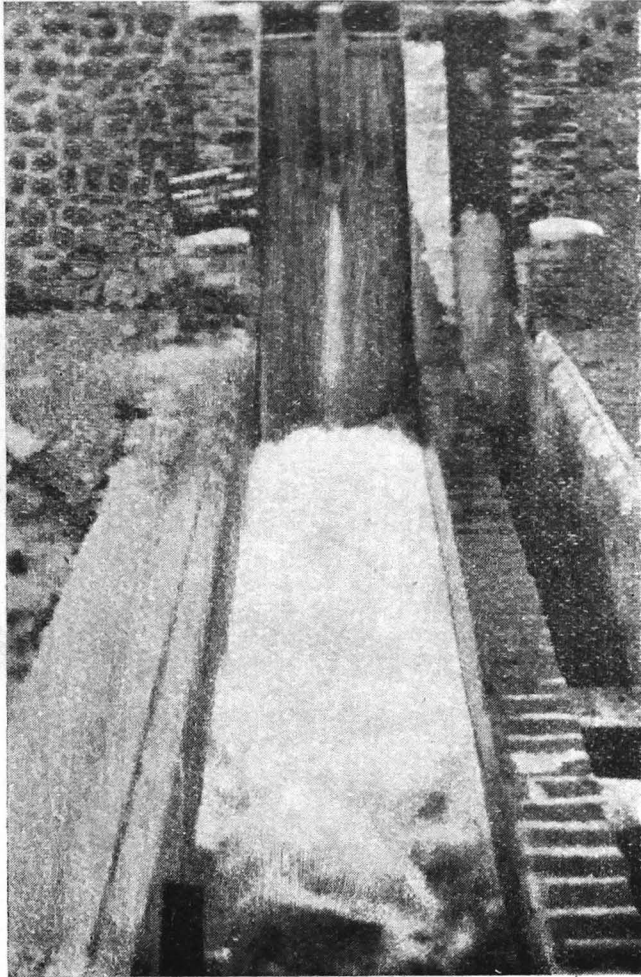
PLOTTINGS OF WATER PROFILES AND SCOURS D/S
RESULTS OF:—



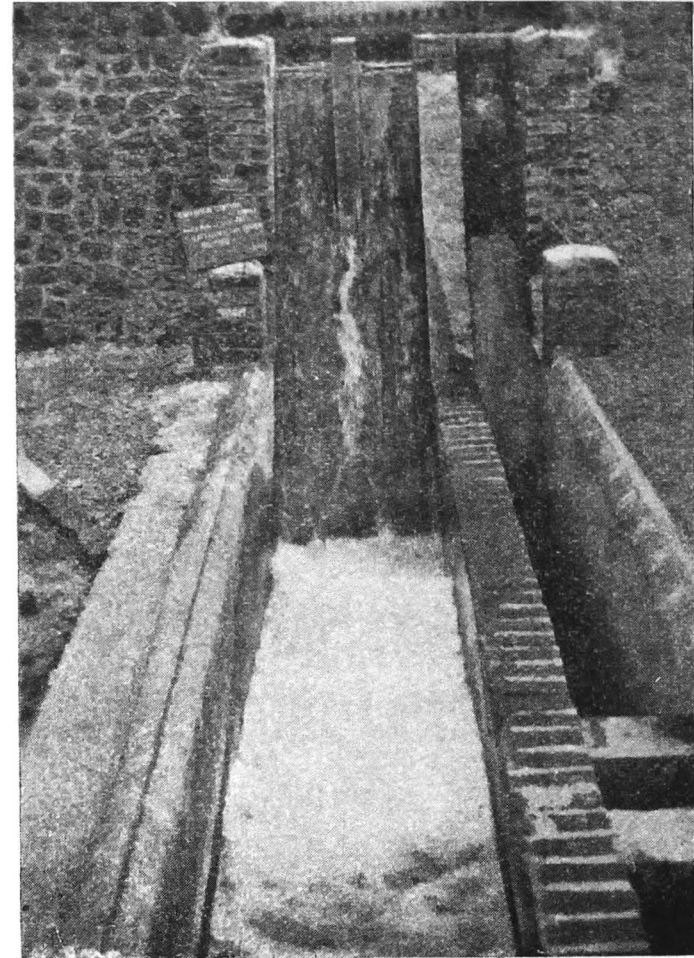
PLOTTINGS OF WATER PROFILES AND SCOURS D/S
RESULTS OF:—



VAITARNA DAM SPILLWAY
(BOMBAY).
EXPERIMENT I WITHOUT BUCKET

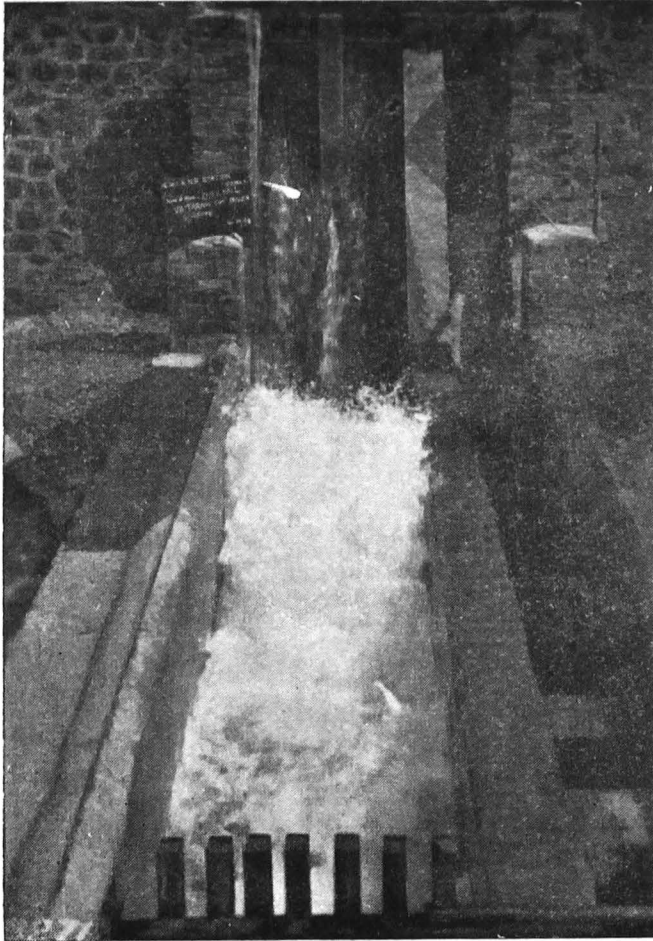


(Looking upstream) Test I.
PHOTO No. 5274. Downstream water
level corresponding to 92,000 cusecs.

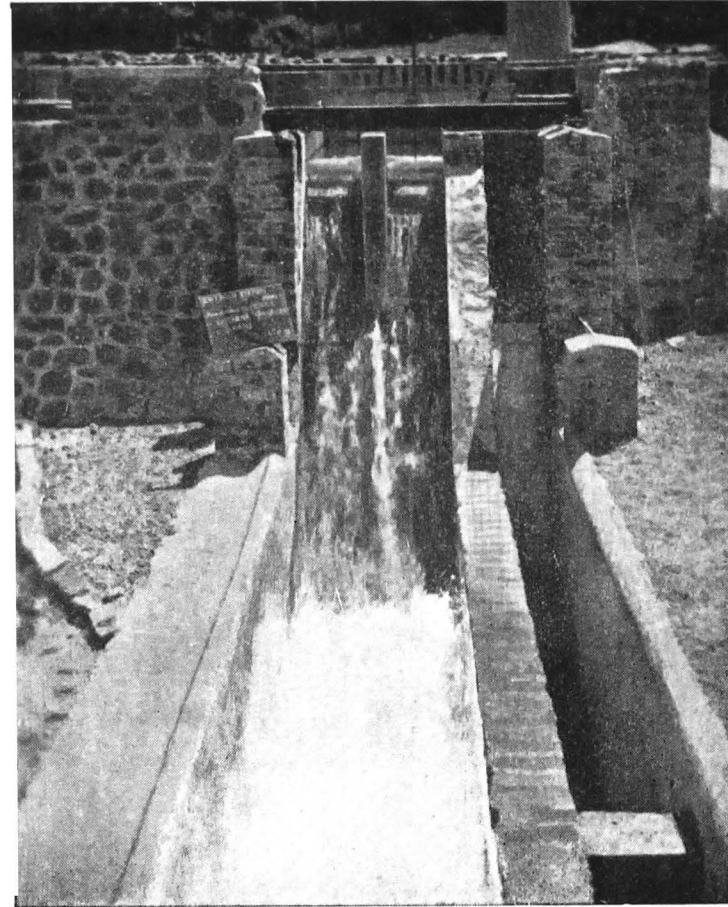


(Looking upstream) Test II.
PHOTO No. 5276. Downstream water
level corresponding to 18,000 cusecs.

VAITARNA DAM SPILLWAY
(BOMBAY).
EXPERIMENT II WITH 5 FT. HIGH BU

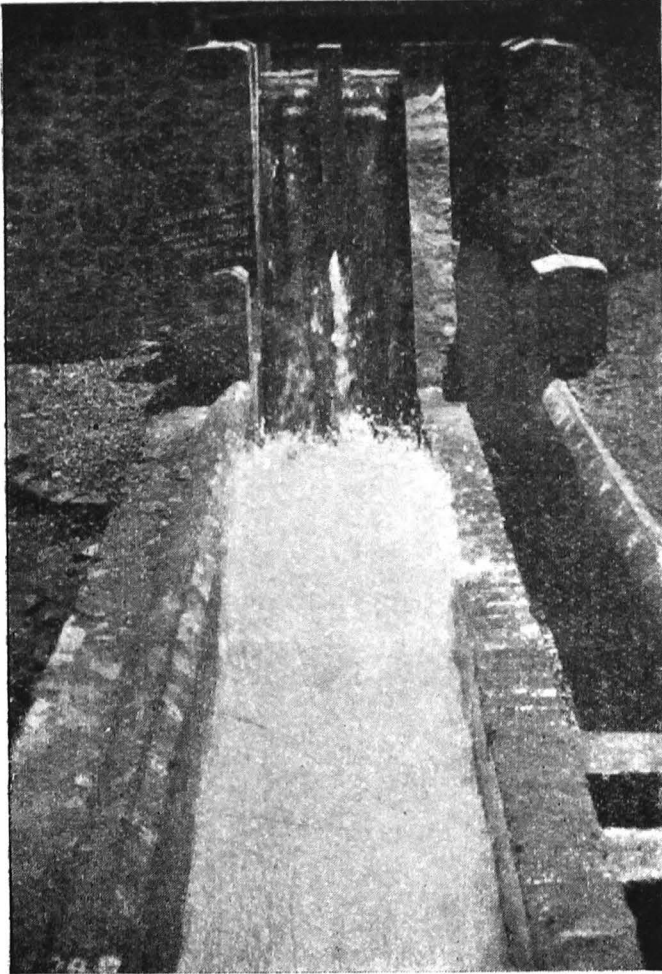


(Looking upstream) Test I.
PHOTO. No. 5271. Downstream water
level for 92,000 cusecs.

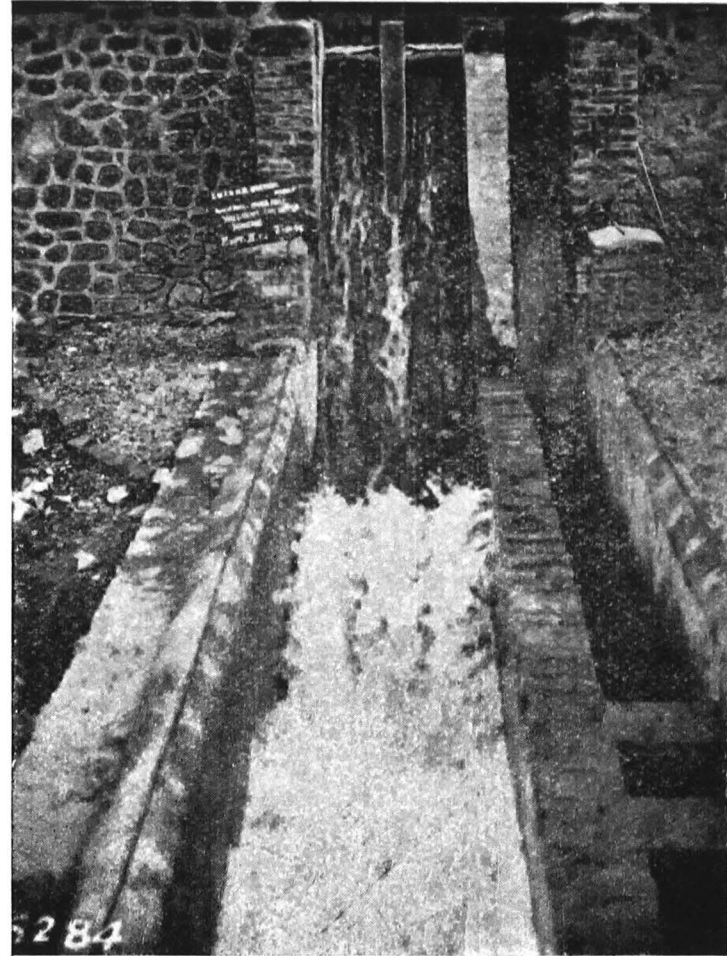


(Looking upstream) Test II.
PHOTO No. 5269. Downstream water
level for 18,000 cusecs.

**VAITARNA DAM SPILLWAY
EXPERIMENT III WITH 7 FT. HIGH
BUCKET.**



(Looking upstream) Test I.
PHOTO No. 5282. Downstream water
level corresponding to 92,000 cusecs.



(Looking upstream) Test II.
PHOTO No. 5284. Downstream water
level corresponding to 18,000 cusecs.

As there are no substantial marked advantages obtainable with 7.0 ft bucket, such as greater throw off from the toe of the dam, lesser depth of scour, effective back-roller at the toe etc., 5.0 feet bucket was considered more suitable.

Fig. 90.

Another 1/50 geometrically-similar model is now constructed to see the efficiency of the bucket with particular references to the position of the standing wave under various rising and falling conditions of water levels upstream and downstream and to study pressure distribution along the profile especially at the kink in the profile at A—fig. 90—where the upper curve meets the downstream slope.

28. Torque required to move the Tansa automatic gates (Bombay)

1. Introduction.

Experiments done on the half size geometrically-similar model of the titling type automatic gate as installed at Tansa⁽¹⁾ revealed that for ensuring the proper working of the gate according to the design, the design should be strictly based on the data collected on its model. Special treatment for each problem is necessary for provision of adequate aeration arrangement and devising suitable weir section to permit free movement of the gate. Model experiments are therefore indispensable in any specific case.

General design and design practice can however be standardised to some extent and data necessary for this was desired to be collected for Messrs. Duncan Stratton & Co of Bombay.

The conclusions arrived at from the results of the experiments made in this connection are given in 2 (i) below.

(1) Pages 153 to 165 of Annual Report (Technical) 1947 of C.W.I. & N. Research Station.

It was noticed in Experiment 2 of Series IV (described on pages 156 and 162 of 1947 A.R. (T)) that for a specific constant water level in the lake, the position of the gate was not fixed but varied within a range. Experiments were done to determine the torque in foot lbs. required to move the gate from the lower to upper position and vice-versa between these ranges. The results of these experiments are presented in 2 (ii) below. These were also carried out at the instance of M/S. Duncan Stratton & Co

2. Results of the experiments.

2. (i) Nappe profiles and pressures above and below the gate by piezometer tubes were observed at different inclinations of the gate, between $52^{\circ} - 30'$ and $0^{\circ} - 0'$ range, with two alternative upstream lake water level viz. 422.00 i.e. F.S.L. and 422.25 i.e. H.F.L.

The following conclusions were drawn from the experimental observations:—

(1) The pressure lines and nappe profiles are not concurrent and hence designing the counterweight on the assumption that pressure on the gate is equal to that given by nappe profile is not quite correct.

The pressure line is slightly divergent from the nappe profile at upper positions of the gate and appreciably different at lower positions of the gate.

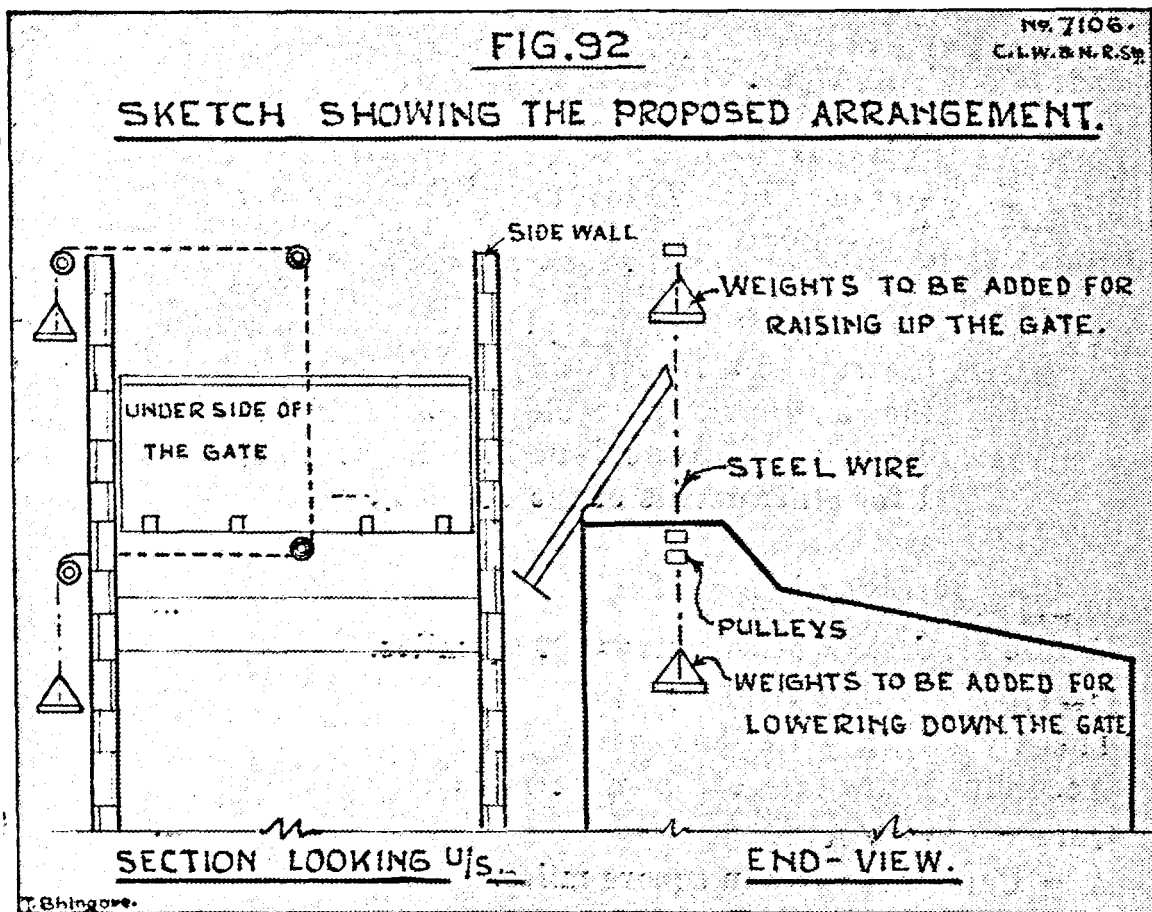
If therefore the counterweight is designed from nappe profiles, the actual pressures being less at the lower positions, the counterweight will become heavier and will prevent the gate from going down. If the counterweight be made lighter to allow the gate to go down, at upper positions it will become lighter and will open and close the gate at water levels below the retention level, which is not desirable.

A compromise thus appears necessary.

(2) For calculating net turning moment due to water pressure on the gate, pressures above as well as on the underside of the gate need to be algebraically summed up for all inclinations of the gate, when the nappe is unaerated.

2. (ii) Experiments were done to determine the torque required to move the gate between the ranges referred to in (1) above.

The method of observation of the torque was in fig. 92.



The arrangement consists of a pan for loading the necessary weights attached to the gate by means of a steel wire running over the pulleys.

Observations and results:—

Water level	Maximum opening of the gate in rising stage to max. closed position in the falling stage.	Torque in ft. lbs. to move the gate through the range in column (2)	Aeration	Remarks
1	2	3	4	5
422.144	17° to 8° - 10'	89.19	Full	In order to maintain a constant water level upstream of the gate, the water supply was required to be continuously adjusted and hence results were approximate.
422.174	14° - 0' to 7° - 30'	88.17	..	
422.250	13° to 6°	76.50	..	
422.324	8° - 30' to 5°	71.20	..	

29. Siphon spillway proposed for the Koilsagar dam project—Hyderabad (Deccan).

Introduction.

Appx. I

It has been proposed to construct a dam across the stream, Pedda Vagoo, near Mahboobnagar in Hyderabad. The catchment area of the stream at the site of the dam is 709 sq miles, but of this 502.5 sq. miles is intercepted and the rest i.e. 206.5 sq. miles area contribute directly to the run-off. Hyderabad P.W D. officers have estimated the maximum probable flood discharge of the stream by Nawab Ali Nawaz Jung Bahadur's formula (See Appendix I for other details of the scheme):—

$$Q_{\max} = C M^{a-b} \log M \quad \dots \quad (1)$$

Where M, area of catchment = 709 sq. miles.

$U = 1700$, $a = 0.92$ and $b = 1/14$

This gives $Q_{\max} = 1,87,500$ cusecs.

According to the fan catchment formula

$$Q_{\max} = 7,000 \sqrt{A} \quad \dots \quad (2)$$

A = Catchment area in square miles.

$Q_{\max} = 1,86,390$ cusecs.

01. Considering that nearly 2/3 of the catchment area does not directly contribute to the run-off the intensity of the maximum probable flood discharge is likely to be less than estimated on the basis of the assumption that the whole catchment contributes to the run-off. The normal maximum flood discharge according to

$$Q = 4000 \sqrt[3]{A} \quad (3)$$

gives a discharge of $Q = 1,06,500$ cusecs and the probable maximum flood discharge considering the peculiar catchment condition is not likely to exceed 1,60,000 cusecs. This point requires further study as it will result in saving over the cost of spillway construction.

1.0. It has been decided to construct siphon spillways in the gorge portion of the dam to pass the flood. Ordinary over-flow spillways are ruled out of consideration as the length of spillway required is considerable. The length of the spillway is necessarily limited by the width of the gorge which is about 700 feet.

1.1. *Siphon spillway designs.*

(a) The ordinary saddle type siphon (b) The volute siphon developed by Mr. Ganesh Iyer were therefore considered. The Ganesh Iyer's design

has a better coefficient of discharge but due to necessary spacing between two adjacent siphons, a spillway dam portion of about 1,000 feet is required. Results of the Jog volute siphons have disclosed that a priming depth required for the design is nearly 1.6 ft. In the saddle type siphon design though its coefficient of discharge is slightly less than the volute type, the length of spillway dam required is 750 feet due to close spacing of the batteries of the siphons possible in this design. The priming depth will be of the same order as in the volute design. It was decided to carry out experiments with small scale models and test both types to evolve a suitable design. The experiments with saddle type siphon with a 1/20 scale geometrically-similar model are given below.

2.0. Design of the saddle siphon.

In this design an easy curve is provided for the inner shell of the siphon to obtain better discharging capacity. A priming weir on the downstream of the siphon is also provided to cause sealing of the lower lip with a given depth of flow over the weir and to secure earlier priming. The lower lip of the siphon was kept 33 ft. below the crest of the siphon.

Fig. 93.

2.1. Four units of siphon to 1/20 scale were constructed in the 5 feet wide channel on the left bank of M.R.B.C. A Rehbock weir was constructed 40 feet downstream to measure the discharge through the siphons.

3.0. Experiments.

In order to determine the optimum position of the priming weir its distance was varied equivalent to 60 feet, 80 feet and 100 feet from the inner vertical leg of the siphon. In all these three positions the crest of the weir was kept at = R.L. 1309.0 or 1.5 ft. lower than the lower lip of the siphon.

3.1. Discharge Coefficients.

Varying discharges were run in each position and the depth of water upstream over the crest of the siphon was noted; from these observations the coefficient of discharges were worked out as shown in Table 1 below observed in the model for the three positions. Figure 94 shows D:Q and D:C curves for each siphon.

Fig. 94.

Table 1

(Discharge given in the Table is for 4 siphon units)

Sr. No.	D = Depth u/s over crest of siphon in ft.	Weir 3 ft. D/S = 60 ft.		Weir 4 ft. D/S = 80 ft.		Weir 5 ft. D/S = 100 ft.	
		Discharge Q in cusecs.	Coefficient C	Discharge Q in cusecs.	Coefficient C	Discharge Q in cusecs.	Coefficient C
1	2	3		4		5	
1.	0.0125 \equiv 0.25	4.800	0.517	4.976	0.539	5.184	0.561
2.	0.025 \equiv 0.50	4.845	0.521	5.920	0.542	5.223	0.563
3.	0.050 \equiv 1.00	4.942	0.528	5.114	0.547	5.309	0.567
4.	0.075 \equiv 1.50	5.043	0.535	5.207	0.553	5.392	0.571
5.	0.100 \equiv 2.00	5.140	0.541	5.300	0.559	5.473	0.575

Thus with weirs \equiv 80' from the siphon, the discharge of each siphon with upstream depth equivalent to 2.0 feet over the crest of siphon would be 2.370 cusecs. The low coefficients of discharge in these experiments are due to the relatively high crest of the priming weirs downstream. The crest was only 15 ft. lower than the lower lip of the siphon and may have caused back pressure.

3.2. On comparing the coefficients of discharges at the three positions it is seen that the discharge coefficient increased as the weir was moved downstream. This is according to what one would expect, but beyond a certain optimum distance downstream, though the coefficient may improve, the priming qualities would be affected; this is because due to bigger contents of the cistern, with a given rate of rise in the reservoir water level, the time required to fill the cistern would exceed the period required for the water level in the reservoir to rise to the priming level.

3.2. Priming depths.

The depth of water upstream over crest of siphon was observed with the three positions of the weir; in all cases the depth was observed to be the same. It was equivalent to 2.78 feet. It is an established fact that the priming depth in small scale model is relatively higher than in the prototype and to estimate the depth of priming in the full scales siphons, priming depths with at least three different scales models should be observed.

M.F.L. 1345.5
F.S.L. 1343.5

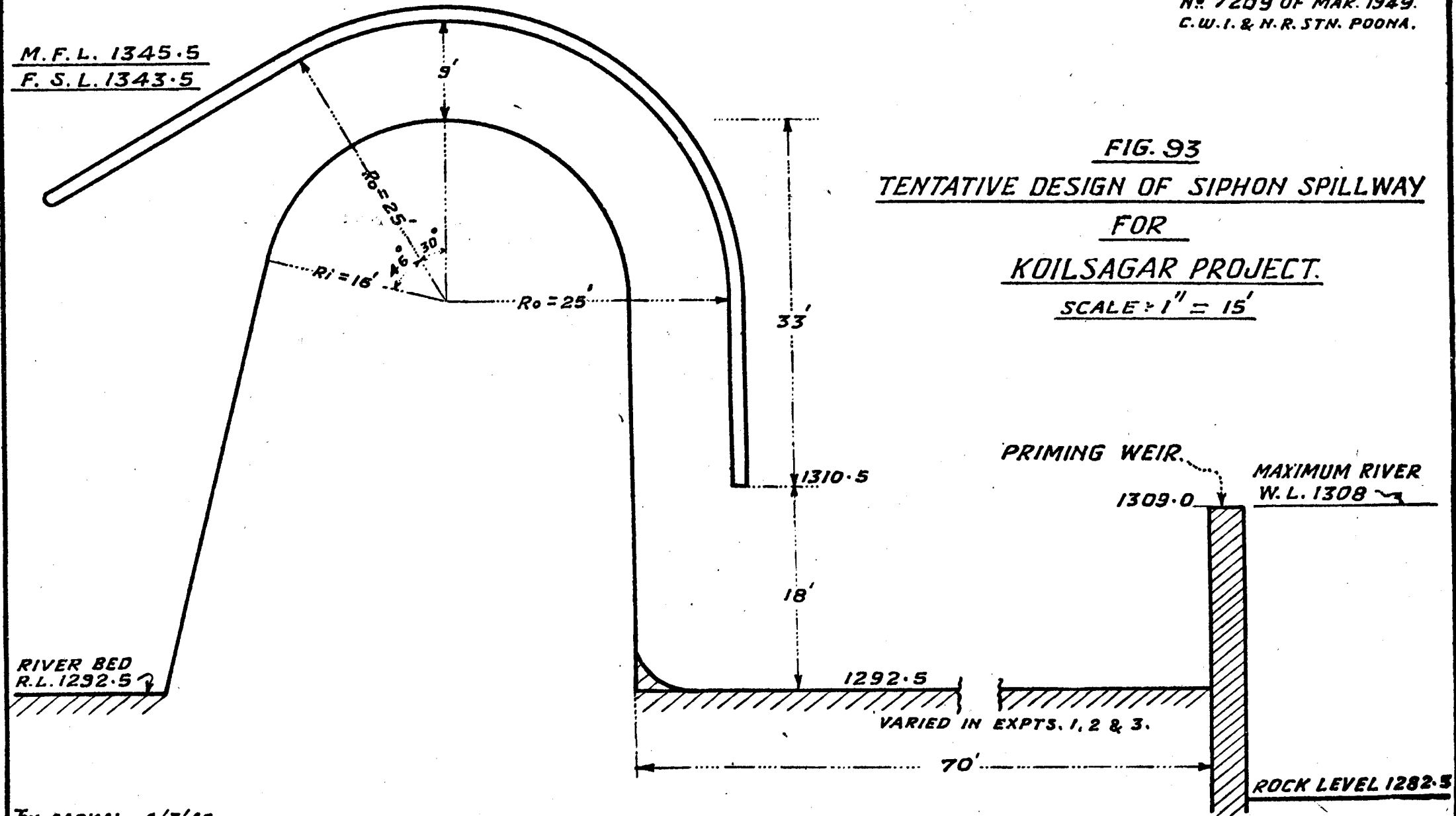


FIG. 93
TENTATIVE DESIGN OF SIPHON SPILLWAY
FOR
KOILSAGAR PROJECT.
SCALE: 1" = 15'

RIVER BED
R.L. 1292.5

PRIMING WEIR.

MAXIMUM RIVER
W.L. 1308

1309.0

1310.5

18'

1292.5

VARIED IN EXPTS. 1, 2 & 3.

70'

ROCK LEVEL 1282.5

By SAPKAL. 4/3/49

FIG. 94

KOILSAGAR SIPHON SPILLWAY EXPERIMENTS.

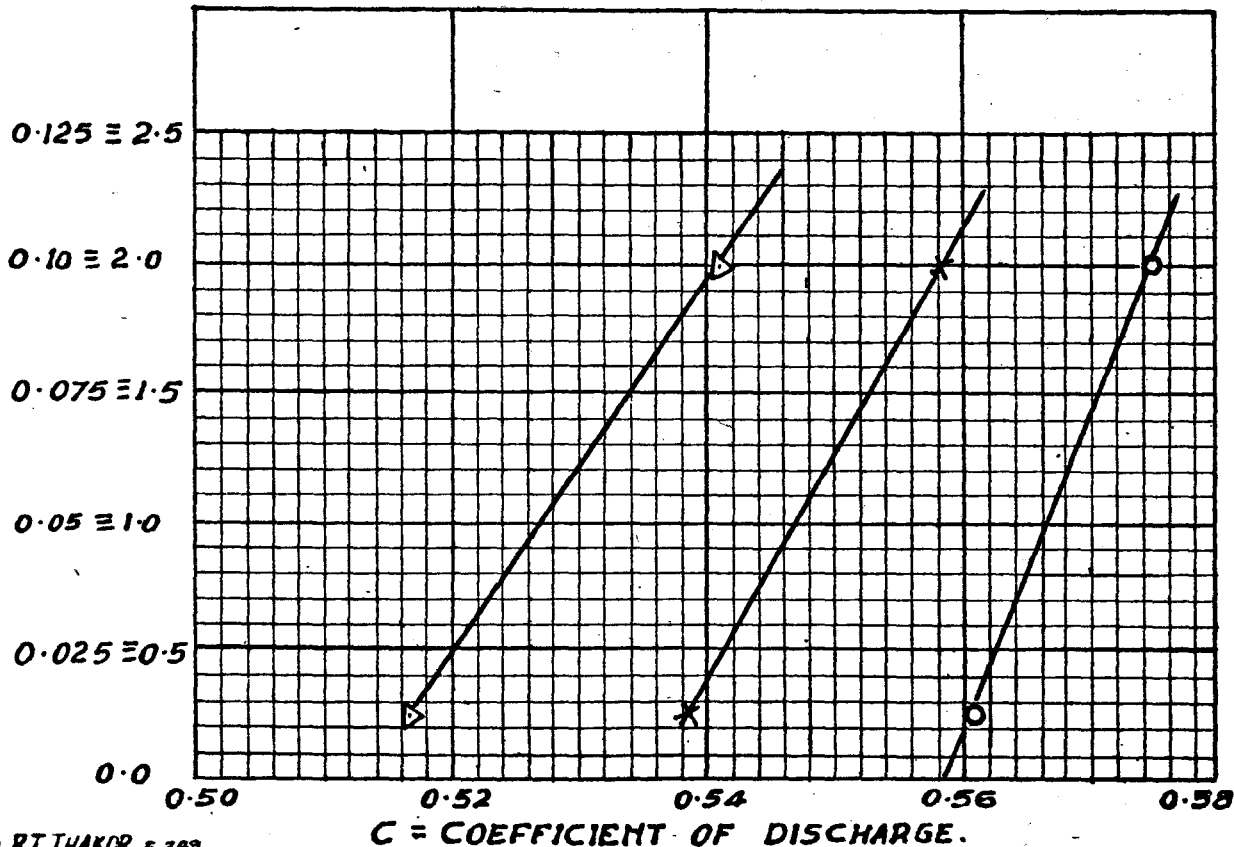
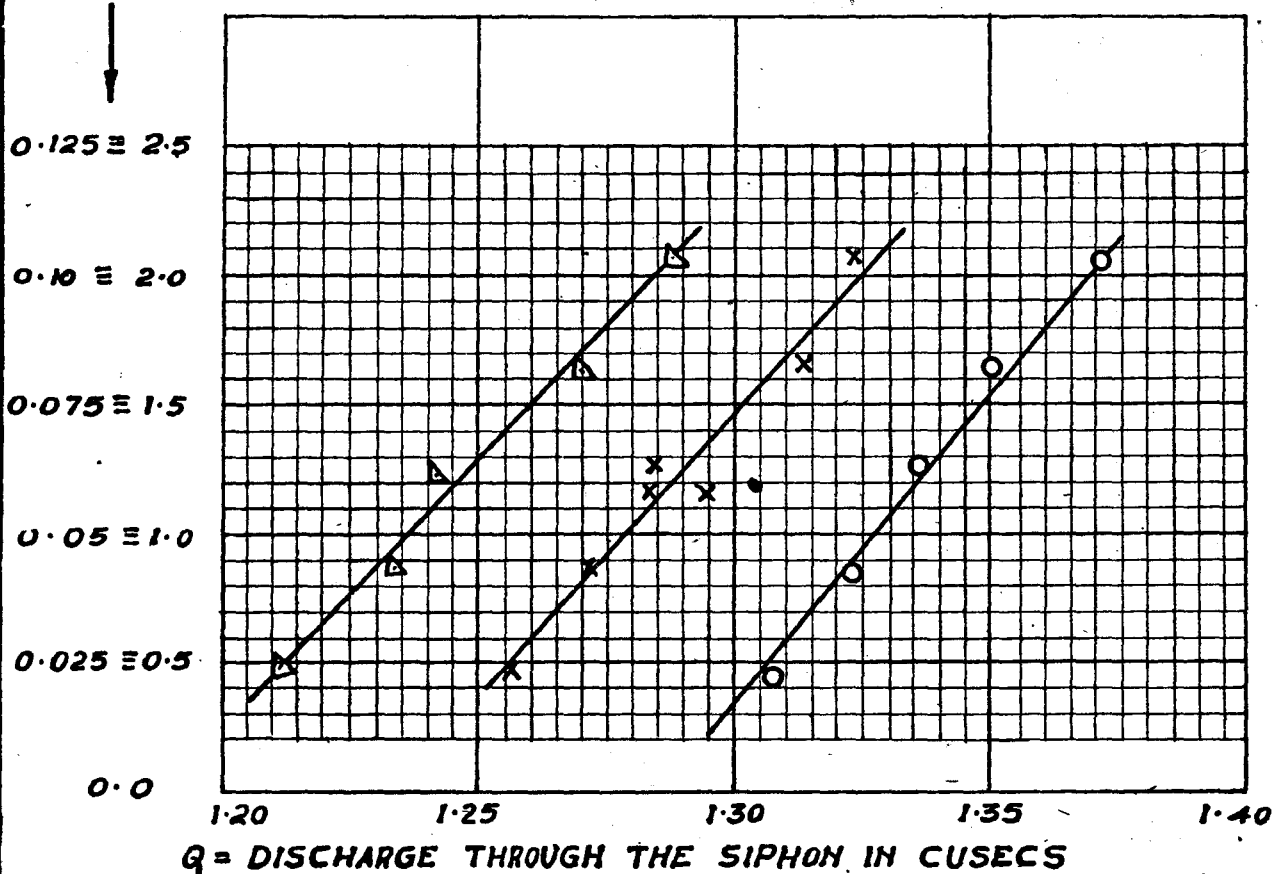
MODEL SCALE 1/20

SHOWING D : Q + D : C WITH DIFFERENT POSITION OF PRIMING WEIR

D/S OF SIPHON.

D = DEPTH OF WATER U/S
OVER CREST OF SIPHON IN FT.

Δ WEIR ≅ 60 FT. D/S
X " ≅ 80 FT. "
○ " ≅ 100 FT. "



APPENDIX 'A'

Principal features of Koilsagar Project, Hyderabad (Deccan)

1	Catchment area (206.5 sq. miles free & 502.5 sq. miles intercepted).	... 709.0 sq. miles.
2	Average monsoon rainfall	... 24.0 inches
3	Nature of catchment	... Average
4	Yield per square mile	... 8.15 M. cft.
5	Total yield.	... 2706.0 M. cft.
6	Fall of river bed above the reservoir H.F.L. of the stream at the dam	... 10.45 ft. /mile ... 1308.0
8	F.R.L. proposed	... 1343.5
9	M.W.L.	... 1345.5
10	T.B.L.	... 1351.5
11	Sill of sluices	... 1317.0
12	Capacity of reservoir at F.R.L.	... 1766.71 M. cft.
13	Capacity of reservoir at sill	... 174.0 M. cft.
14	Net capacity of reservoir at F.R.L.	... 1592.71 M. cft.
15	Length of composite dam	... 2600 feet.
16	Maximum height of reservoir above the pond level.	... 63.5 ft.
17	Maximum height of reservoir above the river bed	... 59 ft.
18	Estimated maximum flood discharge	... 1,87,309 cusecs
19	Lowest rock level in the river at dam	... R.L. 1278.5
20	Sound rock level	... R.L. 1270

30. The proposed raising of the waste weir level of the tank .
at Kalyanipura Village (Ajmer)

The Kalyanipura tank has a catchment area of 2.2 sq. miles. The crest level of the present waste weir (length 98 feet) is at R.L. 100.0 with M.F.L. R.L. 102.75. It is proposed to raise the F.S.L. by one foot by raising the crest of the waste weir to R.L. 101.0.

The Executive Engineer, Ajmer, required to know the increase in the spillway length so that the present M.F.L. is not exceeded even after the raising of the crest of the weir.

Probable maximum flood discharge at Kalyanipura Tank.

As the size of the catchment is small—only 2.2 sq. miles—the chances of the probable maximum flood discharge occurring in the catchment are very remote. In the absence of records of unprecedented floods in this locality, well-established empirical formulae were used to estimate its value.

(1.) Dicken's formula

$$Q_{\max} = CM^{3/4}$$

M = Catchment area in sq. miles

C = coefficient depends on the catchment characteristics; average value generally adopted is 825, with C = 825,

$$Q_{\max} = 1490 \text{ cusecs.}$$

If, however, the value of C is taken as 1600 as suggested for very small catchments;

$$Q_{\max} = 1600 (2.2)^{3/4} = 2890 \text{ cusecs.}$$

(2) Fan catchment formula.

The fan catchment formula developed at this Station and later modified to cover maximum run-offs observed in small catchments has been checked in many cases. The maximum probable discharge is given by

$$Q_{\max} = \frac{7000 A}{\sqrt{A + 4}}$$

where A = area of the catchment in sq. miles. Maximum discharge according to this formula is 5184 cusecs. A provision to pass this extraordinary flood discharge that may probably occur at extremely long intervals will require a much longer spillway and appears unnecessary. However, arrangement to pass the probable normal maximum flood discharge which would occur at more frequent intervals will have to be made.

The normal maximum discharge is given by

$$Q_{\max} = \frac{4000 A}{\sqrt{A+4}} = 3534 \text{ cusecs.}$$

Maximum flood discharge which could be passed with the existing weir

The inflow at the tank could be worked out from the outflow over the waste weir and the capacity of the Tank between the F.S.L. and M.F.L. The maximum flow over the weir with Water Level in the tank at R.L. 102.75 is

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

L = 98 feet and assuming
C = 3.30; with depth of water
upstream of 2.75 feet,

$$Q_{\max} = 3.3 \times 98 \times (2.75)^{3/2} = 1474 \text{ cusecs.}$$

Due to absorption capacity of the tank of 9.71 M. cft. between F.S.L. and M.F.L. higher rates of inflow than 1474 cusecs — the maximum outflow capacity of the existing weir — occurring for short durations, can be accommodated in the tank. The permissible duration of any such flood is, however limited by the fixed total capacity of the Tank and decreases with increase in the magnitude of the flood.

The probable duration of the maximum flood discharge varies with the size of the catchment and for Kalyanipura Tank this period according to Gurtu's formula :

$$T = 0.7 \sqrt{A + 4}$$

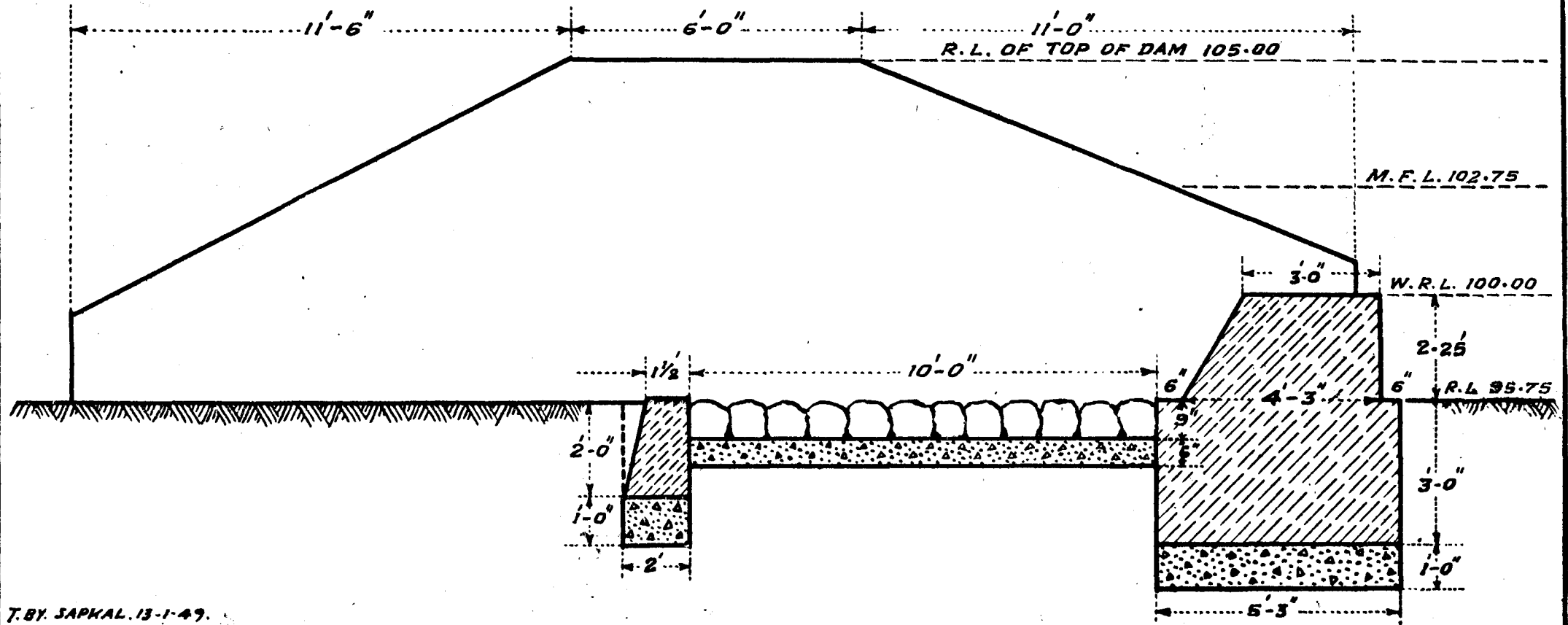
is 1.75 hours. With this duration the maximum inflow which can be passed through the Tank without exceeding the M.F.L. is computed to be 2200, cusecs (see Appendix I for calculations). If the duration of the probable maximum flood is smaller, an inflow of corresponding higher intensity could be accommodated in the Tank; thus if the flood lasts for 1 hour the maximum inflow which can be passed will be 3400 cusecs. Since the existing spillway has only an effective capacity of 3400 cusecs, it is presumed that the new spillway is required to pass only "normal" discharge i.e. 3534 cusecs.

The Design of high coefficient weir.

The L. Section of the existing weir is shown in fig. 95.
Fig. 95. The top of the weir is level, with length in the direction of flow of 3.0 feet. The height of the weir above the upstream bed level is 2.25 feet. Under free flow conditions down

FIG. 95
CROSS SECTION ON THE EXISTING WEIR
OF KALYANPURA TANK.

SCALE = 1" = 4 FT.



stream of the weir, the coefficient of discharge will not exceed 3.3 (with M.F.L. in the Tank). The coefficient can be improved to 4.0 by suitably modifying the profile of the crest. The exact value of the coefficient will have to be ascertained by model experiments. Even with the raised crest level the ratio of depth of flow over weir to height of weir will be $\frac{1.75}{3.25}$ or 0.54 and a higher coefficient is not likely to be attained.

The present weir crest may be modified as shown in Fig. 96. (The exact profile will have to be obtained by modifications to give best results on models). The coefficient of this profile for designing may be assumed as equal to 4.0.

Length of Spillway required after raising the crest.

With F.S.L. raised by 1 foot and H.F.L. same as before the absorption capacity of the tank between F.S.L. and H.F.L. will be reduced from 9.71 m. cft. to 7.01 m. cft. and it is safer to make a provision to pass the full maximum inflow over the waste weir viz. 3500 cusecs. The length of weir required to pass this discharge is given by

$$L = \frac{Q}{C.H.^{3/2}} = \frac{3500}{4.0 \times (3.75)^{3/2}} = 380 \text{ feet.}$$

Existing length = 98 feet.

i.e. additional weir length required is 282 feet.

This will still not pass the catastrophic super-flood of 5,184 cusecs but this risk may be taken as such floods are very rare and may be of a very short duration.

APPENDIX I: Computations to determine the time required for the Water Level in the Tank to rise from F.S.L. to M.F.L. R.L. 102.75 with inflow of 2200 cusecs.

Range of W.L. in the tank (sill of weir at R.L. 100.0)	Storage in m.cft.	Average outflow over the spillway cusecs.	Average rate of absorption in the tank in cusecs.	Period to fill the storage capacity within the range in col.1 minutes.	
1	2	3	4	5	6
1. 100 to 100.5	1.10	38	2162	8.48	The water level in the tank is assumed at F.S.L. R.L. 100.0 at the start of flood.
2. 100.5 to 101.0	1.59	197	2003	13.23	
3. 101.0 to 101.5	1.79	425	1775	16.81	
4. 101.5 to 102.0	2.02	703	1497	22.49	
5. 102.0 to 102.5	2.10	1025	1175	29.79	
6. 102.5 to 102.75	1.10	1292	908	20.19	
				110.99	

Total period of inflow = 1 hour and 51 minutes.

31. Remodelling of the Dimna Nala Overflow Spillway, Jamshedpur Water Supply Scheme

0.0 Introduction

When Mr. K.K. Framji late Director, Central Waterways Irrigation & Navigation Research Station, visited the Dimna Nala reservoir in May 1947, he observed, in progress the reconstruction of the R. C. C. bed slab above the cushion wall of the main spillway, which had been damaged during heavy rains of the preceding year; he also observed that the profile (crest length of 15 feet, high flood depth of 9 feet) was not of the high coefficient type. He, therefore, suggested that, if the case is referred to the Station, it would be possible to suggest a more efficient profile (i.e. one which would give a higher discharging capacity for the same flood level and better energy dissipation measures, the latter if, on a study of the case, it is found that the damage to the bed slab was a result of excessive dynamic action of the surface flow.

0.1. Later, Mr. Aga, the Chief Town Engineer, furnished all the data that had been asked for by the Director and suggested that, simultaneously, the questions (a) of fixing the crest level of the main escape, preferably in combination with as low automatic gates as were necessary and desirable, with a view to increase the storage capacity of the reservoir, and (b) of the maximum flood intensity which must be surplussed through the main and emergency escapes, should be reconsidered. Mr. Aga, therefore, desired that the profile of the main escape proposed by the Station should be for the *revised* crest level of the spillway which would permit increased storage, and at the same time, in combination with the emergency escape and the flood absorption capacity of the reservoir, dispose of estimated maximum flood occurring for the estimated duration without exceeding the safe level (i.e. top of masonry core wall, R. L. 533).

1.0. Existing Features.

It is useful to recapitulate some of the more important data of the existing Scheme which affect the proposals mentioned above.

(i) Earth Dam

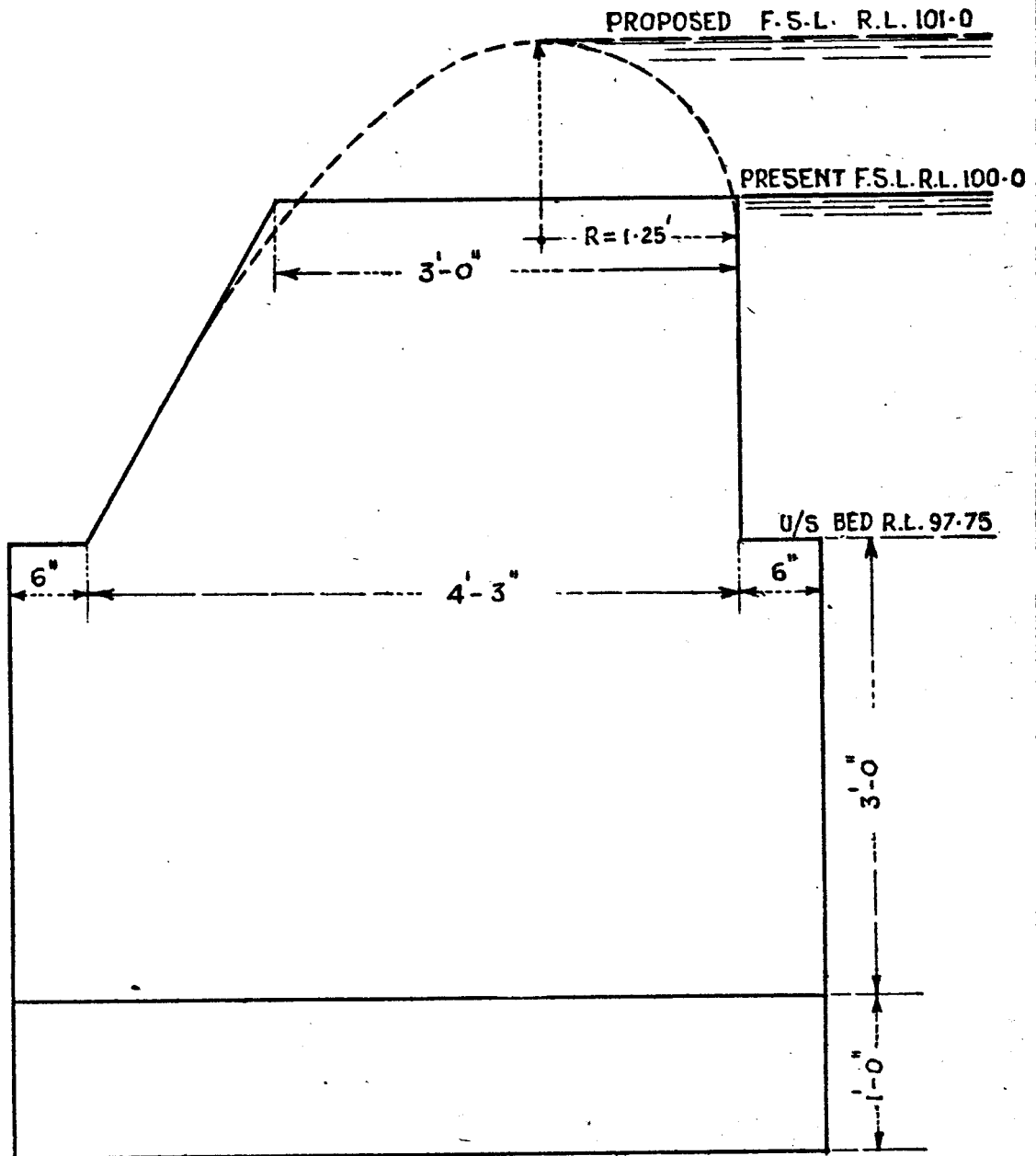
Top level of the dam	=	R L. 538.00
Top of hearting	=	R.L. 535.00
Top of masonry core wall i.e. safe H.F.L.	=	R.L. 533.00
Present F.S.L.	=	R.L. 524.00
Proposed F.S.L.	=	R.L. 527 to R.L. 530.

(ii) Main Escape.

Length of main escape	=	167 feet.
Existing crest level	=	R.L. 524.00
Ultimate crest level (Proposed by Mr. Muzumdar)	=	R.L. 522.00 (with 8 feet high gates).

FIG. 96
PROFILE OF WEIR AS EXISTING &
AS PROPOSED TO BE MODIFIED.

SCALE :- 1" = 1.5'



(iii) Emergency Escape.

Length of escape (aggregate)	=	600 feet.
Existing top of cuts.	=	R.L. 530
Ultimate top of cuts (in Mr. Muzumdar's final proposal)	=	R.L. 529

(iv) Reservoir

Catchment area intercepted by Dam.	=	36 square miles.
Dead storage capacity	=	53 m. cft.
Gross storage capacity at —		
R.L. 524	=	938 m.cft.
R.L. 525	=	990 „
R.L. 526	=	1042 „
R.L. 527	=	1094 „
R.L. 228	=	1150 „
R.L. 529	=	1208 „
R.L. 530	=	1267 „

2.0. Terms of Reference.

The Station has assumed the following issues as the terms of reference :—

- I. What should be the optimum F.S.L. — between R.L. 527 and R.L. 530 — upto which storage is practicable having regard to the existing need for the increased storage as well as to the supplies likely to be available ?
- II. What is the best estimate of the maximum catastrophic flood (i.e. super-flood) likely to occur in the Dimna Nala basin ?
- III. What should be the revised crest level (with or without automatic gates) of the Main and Emergency Spillways which will permit both—
 - (a) increased storage, as under I ; and
 - (b) the disposal of the super-flood discharge (vide II) allowing for the combined capacity of the two spillway — the Main Spillway having an improved high coefficient profile — and the absorption capacity of the reservoir between the proposed F.S.L. and the H.F.L. so that the water level shall not rise above the safe H.F.L. (R.L.533.00)
- IV. What measures can be suggested for the optimum dissipation of energy of flow over the Main Spillway and the reduction of damage to its floor ?

In answering these issues, it has been presupposed that even in 1944 since the Management rejected the proposal of Siphon Spillways — recommended by this Station (1) — for the disposal of the super-flood on considerations of the saving in cost, there is no likelihood of the Management, reopening the question now when the difference in cost will be more and a weir spillway has been already constructed, whatever be the merits of the Siphon on other grounds.

3. Issue No. 1: determination of the optimum storage level.

3.1. Estimated mean monsoon rainfall & corresponding run-off.

3.1.1. In his note dated 17-2-44 on "the Design of surplus arrangement, for the Dimna Nala Water Supply Scheme", Mr. S. C. Muzumdar, at that time, Special Engineer, has analysed the long-period rainfall data of four rain-gauge stations near the catchment (Chaibasa and Ranchi 55 years Purulia 44 years and Jamshedpur 35 years) for the period preceeding 1943. The rainfall data of the catchment basin of the Dimna Reservoir were available for only 8 years (1936-43) and so he has estimated the long-period monsoon (June to October) rainfall for the catchment from the ratio of

long-period to short period rainfall, particularly from that for the nearest Station (Jamshedpur). Table 1 shows the mean, maximum and minimum monsoon rainfall for the five stations considered to Mr. Muzumdar's note and the ratio of the long-period mean to the short-period mean.

Table 1.

3.1.2. Assuming that this ratio of 0.9 obtained for Jamshedpur will hold for the Dimna Catchment, the mean rainfall in the Dimna Catchment over a long-period of years is estimated to be 50 inches. This estimate is reasonable and may be accepted.

3.1.3. Over a long period the probability of the occurrence of rainfall in the Dimna Catchment may be taken to be similar to that at Jamshedpur, which is only 7 miles from the centre of the catchment and, therefore on the same analogy, the probability of occurrence of the rainfall (more or less than the long period mean by the same percentage) will be the same in the Dimna Catchment as at Jamshedpur. A probability curve of rainfall at Jamshedpur is plotted in fig. 97. Since

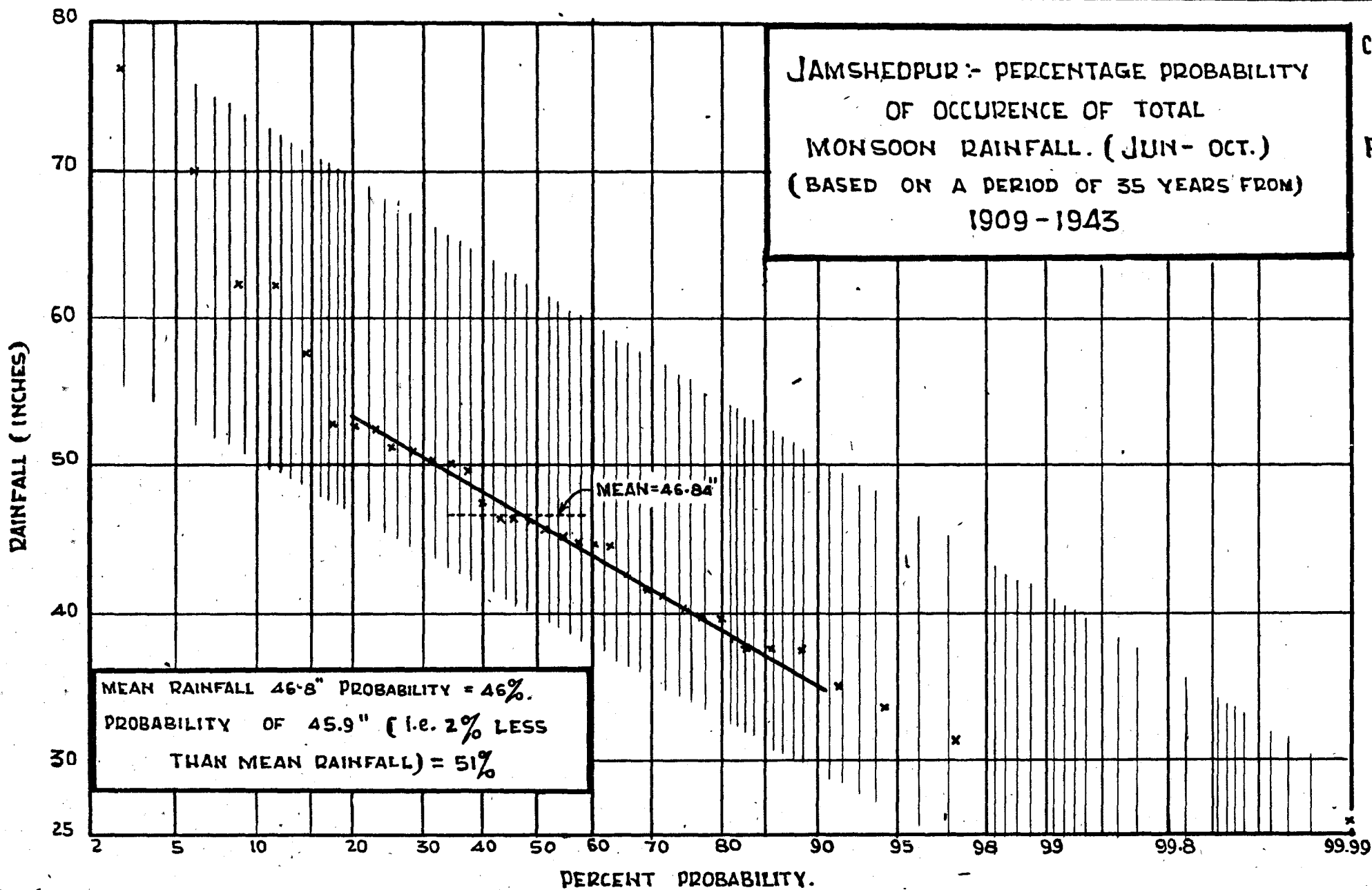
the long-period mean fall at Jamshedpur has a probability of 46%, it may be taken that estimated mean monsoon rainfall of 50 inches in the Catchment will have a probability of 46%; and the probability of a rainfall of 49.7 inches in the Dimna Catchment — which is 2% less the mean fall — will be 50%, because a rainfall 2% less than the mean at Jamshedpur, i. e. 45.9", has a probability of 50%

Fig. 97.

(1) See A.R. (T) for 1940-41; 1941-42 1942-43.

FIG. 97

JAMSHEDPUR :- PERCENTAGE PROBABILITY
OF OCCURENCE OF TOTAL
MONSOON RAINFALL. (JUN- OCT.)
(BASED ON A PERIOD OF 35 YEARS FROM)
1909-1943



Kulkarni 19-1-49.

i. e a frequency of 1 in 2 years. It will be seen later-
vide Table 3 in para 3.2.2 — that a mean monsoon rainfall
on 49.7" in the Dimna Catchment is required to produced
the yield necessary to fill the Storage to R.L. 529.0, thus
confirming the expectation of supplies being sufficient to

fill the stroage up to R.L. 529 with a frequency of 1 in 2 years.

3.1.4. Correlation between rainfall and monsoon-yield.

Having obtained the long-period mon soon rainfall, M. Muzumdar has
attempted to obtain the corresponding monsoon yiled by plotting along-side
a graph representing the relation between the monsoon rainfall and the
yield as given in Strange's table. This is because Mr. Muzumbar had only four
years data (i. e. for 1937, 1939, 1941 and 1943) which he correctly cosidered
insufficient to establish an independent relations between seasonal rainfall
and yield. Data of run-off and rainfall upto 1947 are now available and an

Fig. 98.

independent regression line can be fitted by the methods of
Statistics. This has been done in fig. 98 in which the curves
according to Strange's tables and Mr. Muzumdar's hypot-
hesis are also plotted for comparison. The regression
equation for the full data available is

$$\text{Yield} = 0.0116 (\text{Rainfall})^{2.0289}$$

with a high correlation ratio of 0.813. The fit is also good.

In this connection, it is pertinent to point out that, for the years
1936-40 when the run-off was measured at a point in the stream where the
catchment area is 40 square miles, Mr. Muzumdar had deducted 10% from
the actual run-off figures (assuming run-off proportional to area) to obtain
run-off corresponding to 36 square miles (when the subsequent run-off is
measured being appropriate to the Reservoir); since there is data available
to substantiate that run-off is proportional to \sqrt{A} (not to A); the appropriate
deduction would be

$$\frac{\gamma_{40} - \gamma_{36}}{\gamma_{40}} \equiv 5.14\%$$

This deduction is made in Table 2 showing the seasona
rainfall and corresponding yield data observed for the
years 1936 to 1947 and fig. 98 is the plot of the yield in
million cubic feet per square mile of catchment as comp-
uted in Table 2 against the season's rainfall.

Table 2.

Fig 98.

The yield corresponding to 50 inches monsoon rainfall is 32.48 mill-
ion cubic feet/square mile (according to the Statistical correlation) and so
the total yield intercepted by the dam would be $36 \times 32.48 = 1,169$ million
cubic feet, *against only 800 millon* cubic feet estimated by Mr. Muzumdar.

3.2. Required Storage capacity and F.S.L. of the Reservoir

Deducting dead storage of 53 million cubic feet below level of outlet sluices, the live storage capacity, corresponding to take levels of R.L. 529 and R.L. 530 are 1,155 and 1,214 million cubic feet, respectively.

3.2.2. Thus, storage to R.L. 529 (\approx 1,155 million c. ft.) can be wholly filled with the yield corresponding to the mean monsoon rainfall (viz. 1,169 million c. ft.), which has a frequency of about once in 2 years. Even in the remaining 45 years out of 190, by fixing the F. S. L. above R. L. 527 additional storage will be available, and the extra cost involved will be fully justifiable.

3.2.3. The actual figures of yield are available for recent years 1939 to 1947 (excepting 1941 and 1942). These years are said to be of more than normal monsoon activity. These figures — used in Table No. 2 — show that the tank with the proposed F.S.L. of R.L. 529 (or R.L. 530) would have filled for only 1 year (2 years) less, in the period of 12 years, than the existing tank:—

F.S.L.	Net Capacity (M. Cft.)	Net Capacity per sq. mile. M. Cft./Sq. ml.	Frequency with which obtained (From Table 2).
527	1,941	28.91	8 in 12
529	1,155	32.08	7 in 12
530	1,214	33.7	6 in 12

Note: In working out these frequencies it has been assumed, for the years 1941 and 1942 for which run-off data are not available, that the tank would have filled upto R.L. 527 in both these years, but only in one year for tank level upto R.L. 529 or R.L. 530, even though the rainfall in both these years is so much more than the normal rainfall required to fill the tanks upto these levels, (vide Table 3).

Table 3.

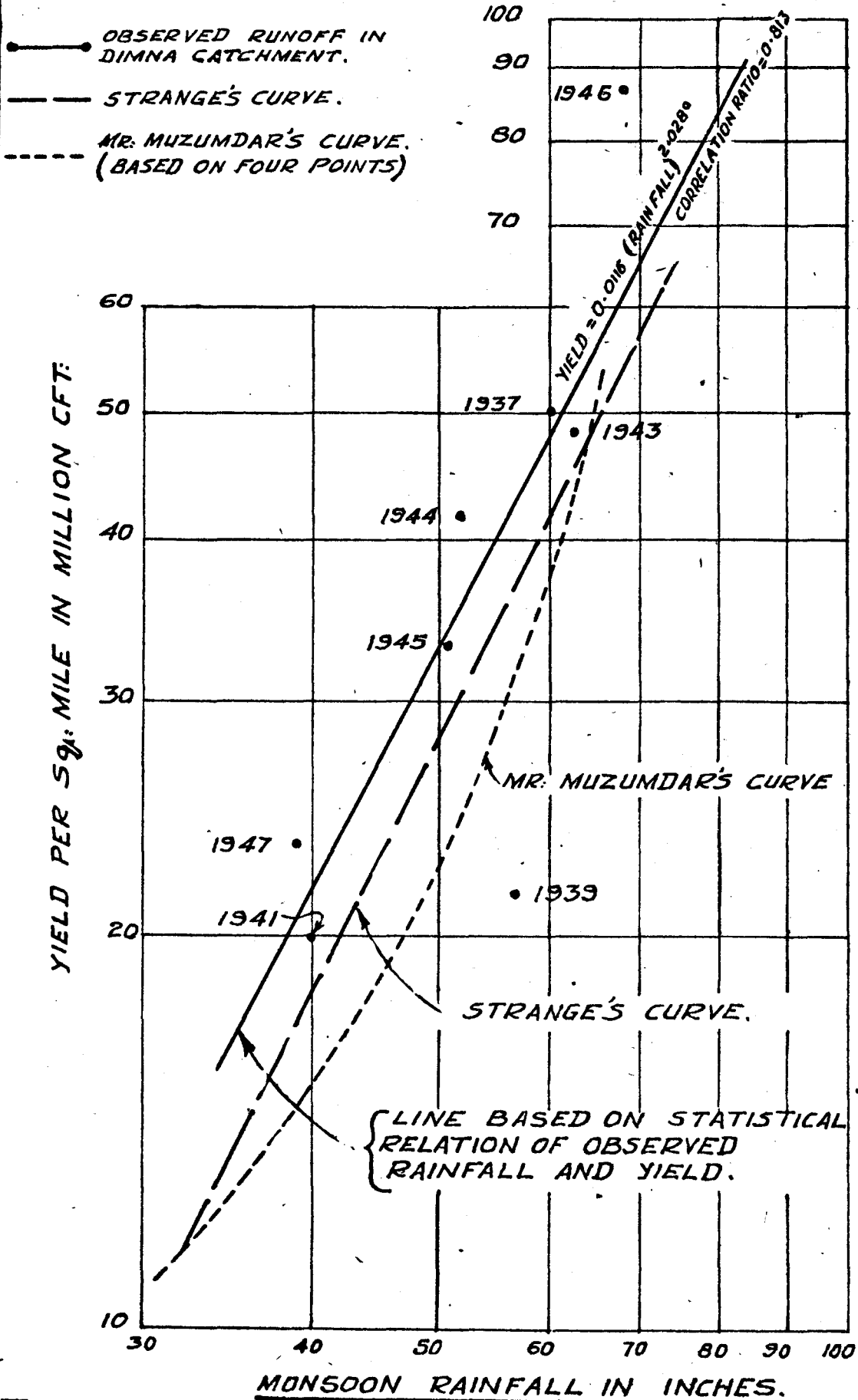
These frequencies also show that the probability of the overflowing in a long period of years, as given in Table 3 is reasonable.

3.2.4. To fix the storage capacity of the reservoir Mr. Muzumdar has prepared a Table showing rainfall necessary in the catchment to fill the tank to various storage capacities, as fixed by the sill level of the spillway adopted. Table No. 3 is prepared on analogous lines except that instead of the gross-storage, the net storage (excluding the dead storage below outlet

FIG: 98

NO: 7124.
C.W.J & N.R.S. POONA.

SHOWING CORRELATION BETWEEN
TOTAL MONSOON RAINFALL & SEASONAL YIELD
FOR DIMNA CATCHMENT.



By. MANCHUNDIA. 21.1.49

sluices) has been taken into consideration to find out the corresponding figures of monsoon rainfall and storage levels because after the first year of operation only the net storage will matter.

3.2.5. With sill at R.L. 529 and 530, respectively, the percentage probability of filling are 50% and 42% (1 in 2 and 1 in 2½ years) as compared to Mr. Muzumdar's figures 14.8% and 16.6% only (i.e. 1 in 7 years in either case). Mr. Muzumdar himself has stated—para 19 of his Note—that the storage capacity should be kept as high as possible only subject to the consideration that it can reasonably be expected to fill at least once in, say, 3 to 4 years. According to Mr. Muzumdar's own criterion, therefore, the F.S.L. of the reservoir should be fixed at about R.L. 529 or R.L. 530.

3.2.6. Considerations of satisfactory routing of abnormal floods by partial absorption in the reservoir in the conditions obtaining in the Dimna, however, require the F.S.L. to be not higher than R.L. 529—for reasons to be explained in para 5. Moreover, with storage up to R.L. 529, it is possible to two store year's requirements; even assuming the usual allowance of 1/6th for evaporation (although Mr. Muzumdar has stated in para 14 of his Note and in his comments on Sir Visweswaraya's note that it will be much less) and the city's yearly requirements are 430 million c. ft. corresponding to the *ultimate* consumption of 10 million gallons per day for 300 days, as estimated by Sir Visweswaraya.

3.2.7 For these reasons, it is considered sufficient and desirable to store normally up to R.L. 529.0.

4.0 Issue No. II.

The best estimate of the maximum catastrophic flood likely to occur in Dimna Nala Basin.

4.1.1. In making provision to pass the maximum flood discharge for designing the spillway, it is necessary to make a close estimate of the *maximum* rate of inflow in the reservoir with a certain degree of exactitude. In rare cases, however, sufficiently long data necessary to estimate the highest-ever discharges, on a rational basis, are available and, generally empirical formulæ are employed to ascertain the maximum possible flood from the catchment area. These formulæ are of the exponential form—

$$Q = CA^n$$

where Q is equal to the maximum flood discharge,

A is equal to the area of catchment, and

C and n are constants.

Such formulae are, however, limited in their accuracy and application by the variations in the constants assumed, which in turn depend on the nature and quality of and the fit to the data from which the formulae are

derived; and may lead to an under — or — over-estimation of the maximum flood discharge in other cases having different characteristics.

4.1.2. The index n has been variously taken as $3/4$, $2/3$ or $1/2$ in the Dicken's, Ryves' and the Station formula, but the bulk of the obtainable data tends to support the latest practice, of an index of 0.5, as evinced by the Station fan-catchment formula—

$$Q_{\max} = \frac{7,000 A}{\sqrt{A+4}}$$

4.1.3. This formula was evolved at the Station from data collected from all over India which covers a very wide range, and moderated to apply to very small catchments. It is to be noted that the equation is *not* the *statistical average* line fitting the data but the *outside envelope* line which thus gives weight to the points giving maximum discharge for given areas. (The statistical average line gives normal flood discharge about $4/7$ th this highest-ever discharge for the same area of catchment.)

4.1.4. The Station has also found that the shape of the catchment is a very important factor in the correct estimation of the intensity and duration of the maximum flood discharge. A truly fan-shaped catchment is one where the time of concentration from all the farthest points of catchment is the same and equal to or less than, the duration of rainfall of maximum intensity. In the present case, the area of catchment is only 36 square miles and therefore, no allowance for shape need be made.

4.1.5. According to the fan catchment formula, the maximum likely discharge from the Dimna Nala basin would be—

$$\frac{7,1000 \times 36}{\sqrt{36+4}} = 39,840 \text{ cusecs.}$$

4.2.1. For the other methods it is necessary to know the hourly rainfall intensity and run-off, the former requiring an automatic rain-gauge which is seldom, however, available; because a correlation between rainfall are only possible when these hourly intensity figures of rainfall and additions to storage during high floods are obtainable.

4.2.2. It is an established fact, however, that the rate of precipitation and the period over which it occurs vary inversely and that the rate for a shorter period bears a definite relation to the average rainfall over a longer period. On the basis of this assumption, the rainfall for a short duration can be estimated from the observed *daily* rainfall figures, data of which are available for the Dimna catchment.

4.2.3. The maximum daily rainfall observed in the Dimna Catchment during the last 10 years is 7.94". In fig.

Fig. 99. 99 are plotted observed run-off against the daily rainfall for high values of run-off. Scatter is inevitable and must be expected because of the variation in the intensity and duration of rainfall and in the varying conditions of the catchment, i. e. dry or saturated prior to the incidence of rain. The scatter is, however, immaterial as it is only in the envelope in which we are interested. The line enveloping the point may be taken to represent fairly the run-off of normal floods that might be expected corresponding to the given rainfall figures in a saturated catchment. See Table 4 and 4(A).

Table 4
& 4A.

4.2.4. In obtaining the average run-off from this curve — fig. 99 — it has to be remembered that the daily rainfall recorded at 8 A. M. each day may not be, and often is not, the actual rainfall occurring in the preceding 24 hours. The actual distribution of rainfall during the 24 hours is, however, not known and, for reasons explained by Mr. Muzumdar in his note on page 17 Para 21, it is often safer to assume that the rainfall of two consecutive days has actually fallen in a period of 24 hours. The maximum rainfall for two consecutive days observed so far has been 9.54" in 1939, that is, say 10". Since, however, this rainfall has occurred in the comparative short period of 10 years, it would be wiser to expect a maximum rainfall of 12.5" for 24 hours (i. e. 25% in excess of the maximum rainfall actually observed), particularly in view of the area being subject to cloud-bursts. From fig. 99, this will give an average run-off of 13,000 cusecs.

Fig. 99.

(Mr. Muzumdar has also estimated the intensity of normal high flood of 12,612 cusecs which is the observed maximum run-off in 1940, — paras 22 and 24 of his note).

4.2.5. *Peak discharge.* The ratios of peak discharge to average for each flood hydrograph is shown in column 8 of Table 4 and applying highest ratio of 3.366 observed in 1940 the peak discharge would be—

$$Q_{\max} = 13,000 \times 3.366 = 43,760 \text{ cusecs.}$$

4.3. *Maximum flood discharge based on rainfall intensities over shorter duration.*

4.3.1. For finding out the rainfall intensity over short-periods from the average maximum rainfall data for 24 hours the ratios given by Mr. Cochrane⁽¹⁾ have been used. Table No. 5 is, accordingly, prepared showing the inten-

Table 5.

(1) "Flood run-off from small areas" by Victor H. Cochrane, Engineering News Record Vol. 119, (1937).

sities of rainfall of various durations arising from an average rainfall of $19\frac{1}{2}$ " during 24 hours.

4.3.2. Maximum inflow occurs when the duration of rainfall of maximum intensity equals or exceeds the time required for flow from the furthestmost parts of the catchment, to reach the reservoir. Fig. 100 shows the Dimna Catchment. Table 5 is required to showing increment of areas contributing to the run off corresponding to increment in the distance from the reservoir (spillway) for the Dimna catchment. In fig. 101, these increments in areas are plotted against the mean distance of the zone considered from the spillway. It will be seen that the rate of

change of area with the distance drops for areas beyond 5 miles from the spillway.

Fig. 100.
Table 5.

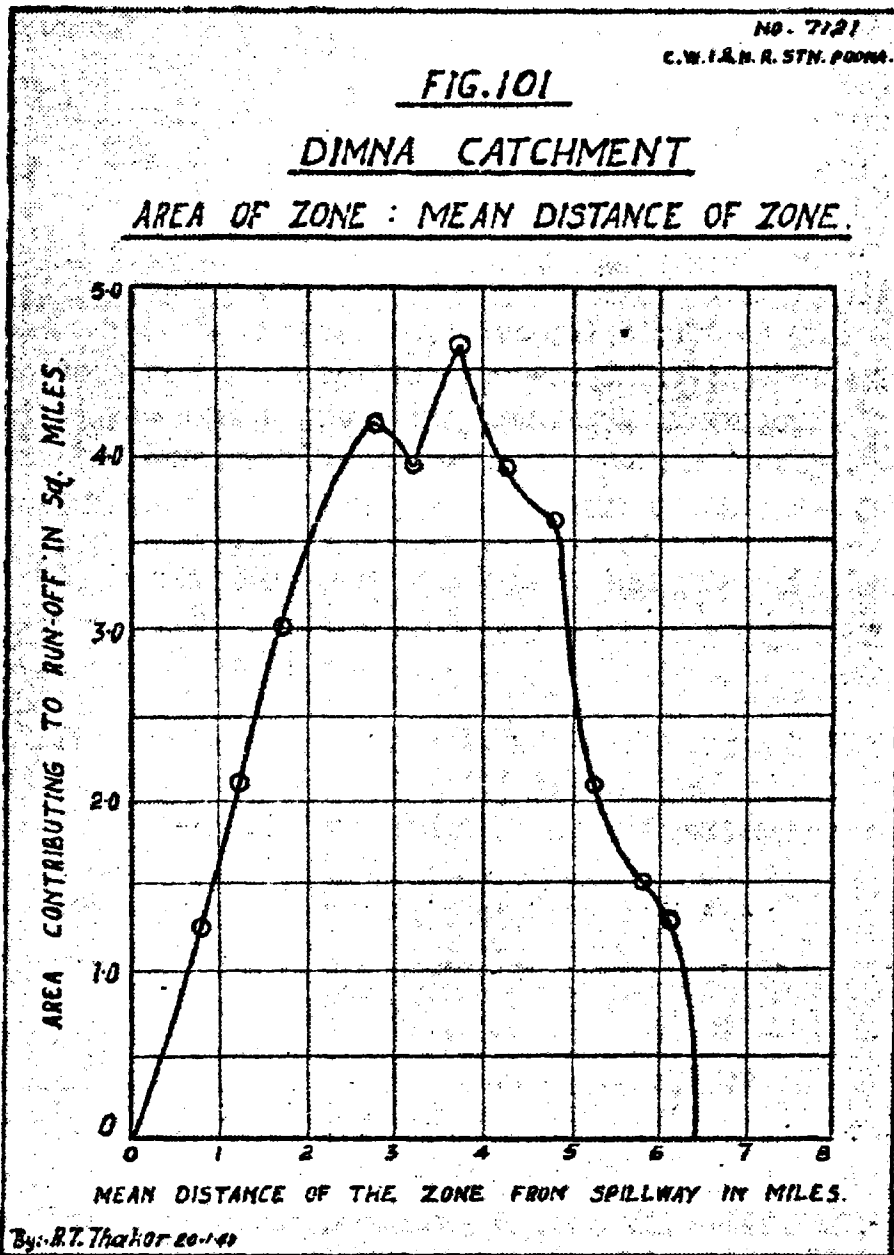


FIG. 99

DIMNA NALA CATCHMENT.

JAMSHEDPUR

DAILY RAINFALL : AVERAGE RUN-OFF.

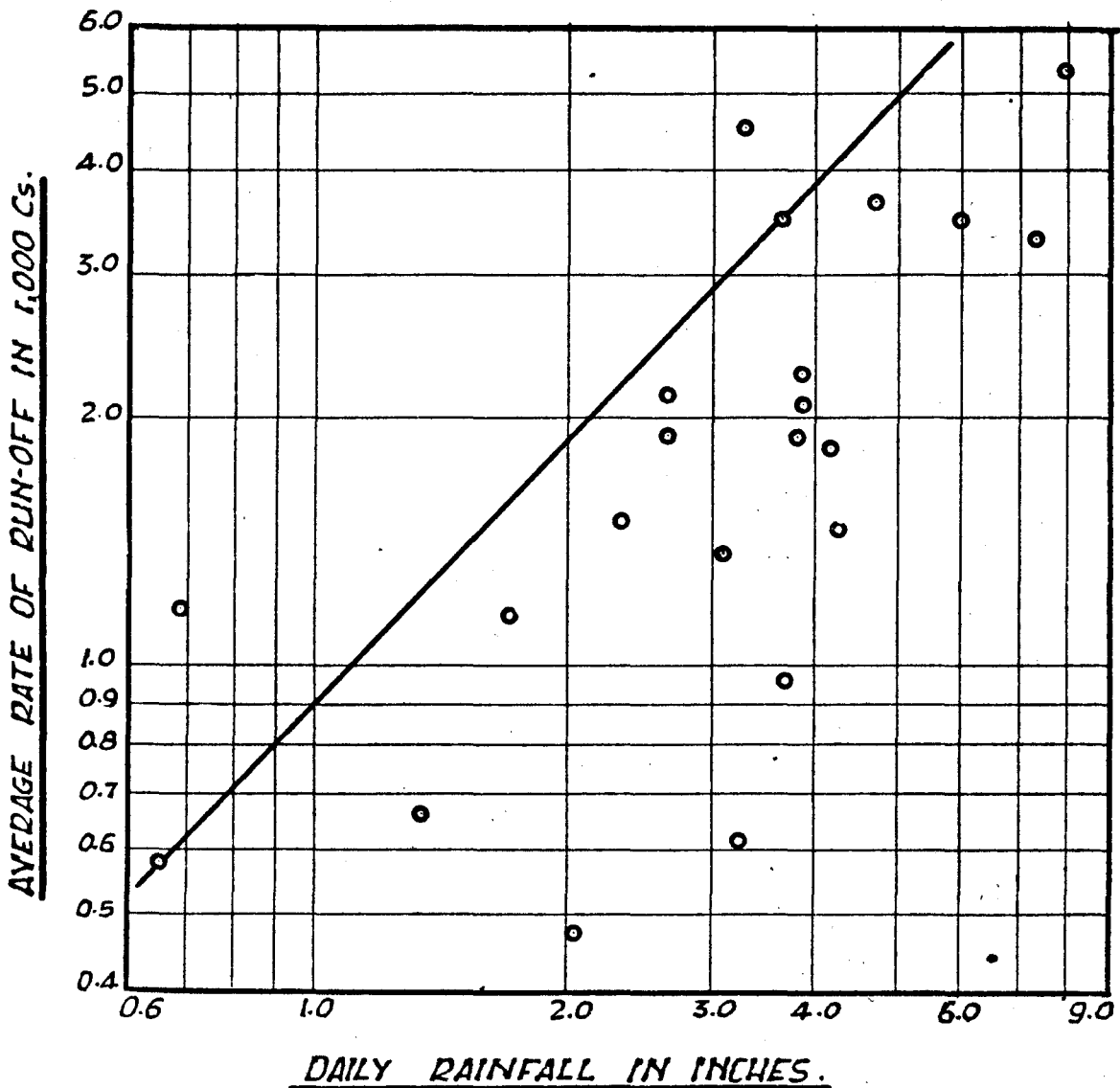
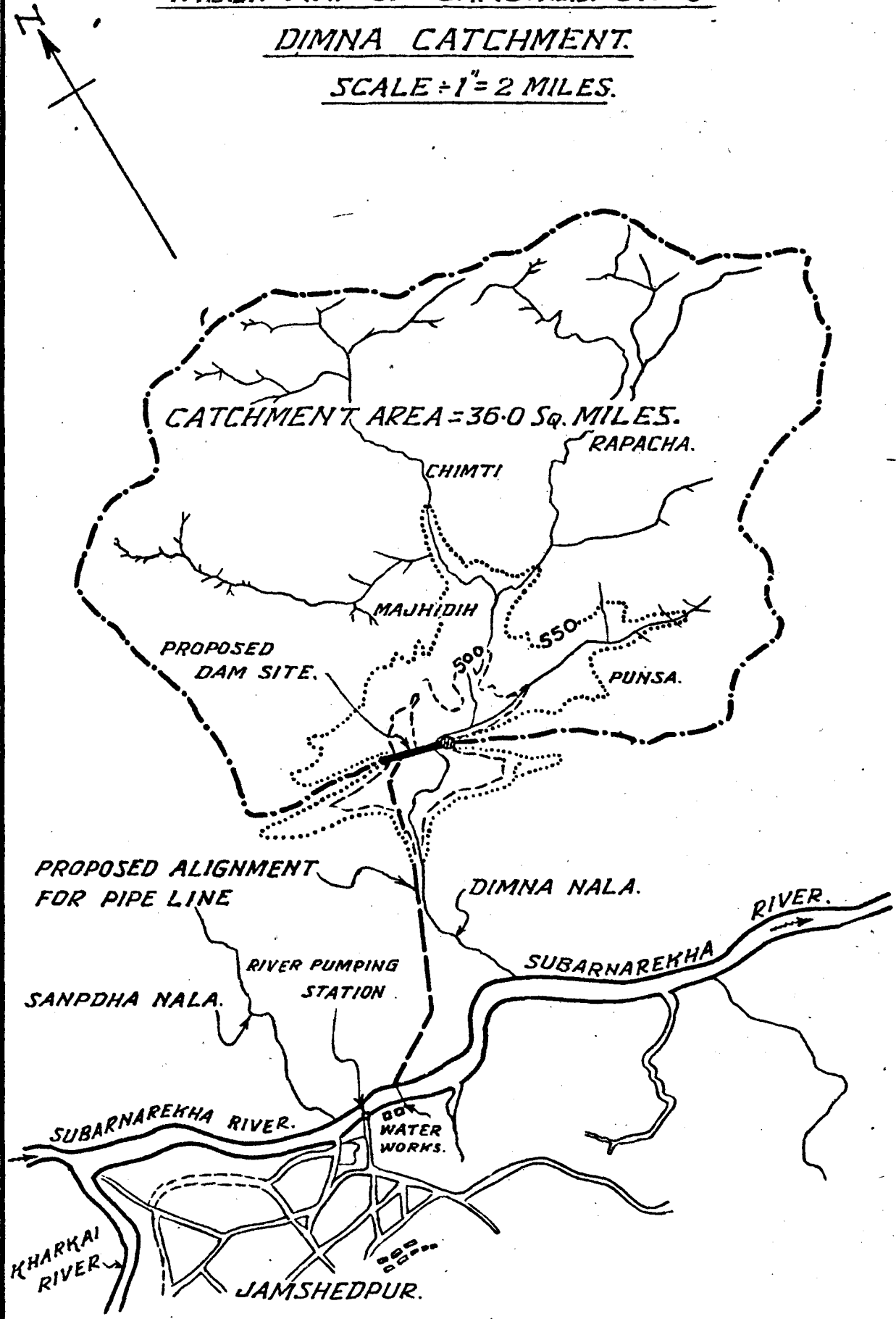


FIG. 100
INDEX MAP OF JAMSHEDPUR &
DIMNA CATCHMENT.
SCALE = 1" = 2 MILES.



By SAPKAL, 21-1-49.

4.3.3. In the Dimna catchment, however, maximum inflow is expected to occur when the duration of rainfall equal the time required for flow to cover a distance of 5 miles. It is difficult to assess the correct value of the average velocity of flow over, this distance in the absence of data of energy slope and rugosity, but a reasonable estimate of this velocity would be 4 feet per second or 2.73 miles per hour. (This allows for the sinuosity of the river and the modifications due to absorption in bank overflow; being in a rather steep area the assumption of 4 feet per second is considered reasonable) With this velocity of flow, the period of concentration from a 5 mile zone would be 1.833 hours and for the flow from the farthest part of the catchment to reach the spillway would be 2.38 hours.

4.3.4. Referring to Table No. 6, corresponding to a rainfall of 1.25" in 24 hours, the intensity of rainfall per hour for a rainfall of 1.83 hours duration is 3.59 inches, and the maximum run-off per square mile with this intensity is given according to formula (Cochrane).

$$q = 645 \times C \times 0.69 \times I \quad (I = \text{intensity of rainfall of } 5.208'' \text{ for a rainfall having one hour's duration corresponding to rainfall of } 12.5'' \text{ in 24 hours})$$

$$= 645 \times C \times 3.59$$

C is the rainfall to run-off coefficient of the catchment; at Dimna, a coefficient of 55% was experienced (generally for August as in latter period the ratio increases due to recuperation) and

$q = 645 \times .55 \times 3.59 = 1,270$ cusecs/square miles as the area of the zone considered, which only contributes to the maximum run-off is 90% of the total, i. e. about 32.4 square miles. The maximum run-off that may be expected is—

$$Q_{\max} = 1,270 \times 32.4 = 41,150 \text{ cusecs.}$$

4.3.5. This figure has, admittedly, been on an assumed velocity of flow in the catchment of 4 ft./sec., and the actual magnitude depends upon the correctness of this assumption. However, if the velocity were 3.5 ft./sec. or 4.5 ft./sec. the maximum flood, calculated in the same way, would be 37,300 and 45,200 cusecs respectively.

4.4. Thus, there is rational corroboration, to a remarkably close degree, of the estimate of the maximum flood discharge as obtained by the Station fan-catchment formula, the figures for the maximum flood according to the two other rational methods being 43,700 cusecs and 41,150 cusecs as against the estimate of 39,840 according to the Station fan-catchment formula.

It is accordingly, the Director's view that a maximum probable flood discharge of 40,000 cusecs is the best estimate that can be made from the available data.

4.5. *Duration of Flood.*

The duration of peak flood of $4\frac{1}{8}$ hours was suggested by the Station based on Gurtu's formula T (in hours) $= 0.7 \sqrt{A+4}$. This formula has not the authority or general acceptance based on verification from extensive data and the $4\frac{1}{8}$ hours figure need not, therefore, be rigidly enforced in design.

5.0. Issue No. III.—Revised crest levels (with or without automatic gates) required for flood routing in the Dimna Reservoir.

5.1.1. The proposed effective length of the main spillway is 150 feet with crest tentatively fixed at R. L. 524.0; it has a flat crest the length in the direction of flow being 15 feet and the depth of flow at Maximum flood level is 9 feet, i. e. the ratio of length of weir crest to depth of flow is 1.67. Under these conditions, the coefficient of discharge in the usual broad-crested weir formula—

Photoes
5396 & 5395

$$Q = CBD^{1.5}$$

would, at the most, be 3.20; against 3.09 for a broad crested weir; the coefficient of 3.3, as assumed by Mr. Muzumdar, is thus evidently high for depths of flow smaller than 6 to 7 feet.

5.1.2. It has been proposed to raise the F.S.L. in the reservoir to some level between R. L. 527 and 530 by either installing automatic gates over the present crest or by partly raising the spillway and partly by providing gates. In recent experiments at this Station with half scale model of the Tansa weir with over-shot gates the coefficient was found to be 3.16 for a depth of flow of 1.8 feet and length of gate 1.88 feet. A coefficient of 3.2 may be, therefore, reasonably assumed for the Dimna Nala prototype, if the profile of the spillway is not reshaped to that of a high coefficient weir.

5.1.3. If, however, there is no objection to convert the present broad-crested weir into a high coefficient profile by suitably modifying the crest and lowering the upstream bed down to R. L. 518 for a sufficient length upstream, the coefficient could possibly be improved to 4.50 with the crest at R. L. 524 and full tank level at R. L. 527; alternatively with F.S.L. at the reservoir to R.L. 529, requiring a crest level R.L. 522 (as proposed), unless the upstream bed could be lowered below R.L. 518 — which does not appear practicable it would be only safe to assume, in anticipation of model experiments, a coefficient of 4.0. This presupposes radial automatic gates of a suitable type which would be completely out of water (or wholly recessed into the profile) thereby offering no obstruction to flow over the weir in any way.

5.1.4. For routing the flood under the existing conditions, (with crest at R.L. 524 and F.S.L. at R.L. 527) the maximum discharging capacity will be given by

DIMNA NALLA WATER SUPPLY
SCHEME—JAMSHEDPUR.

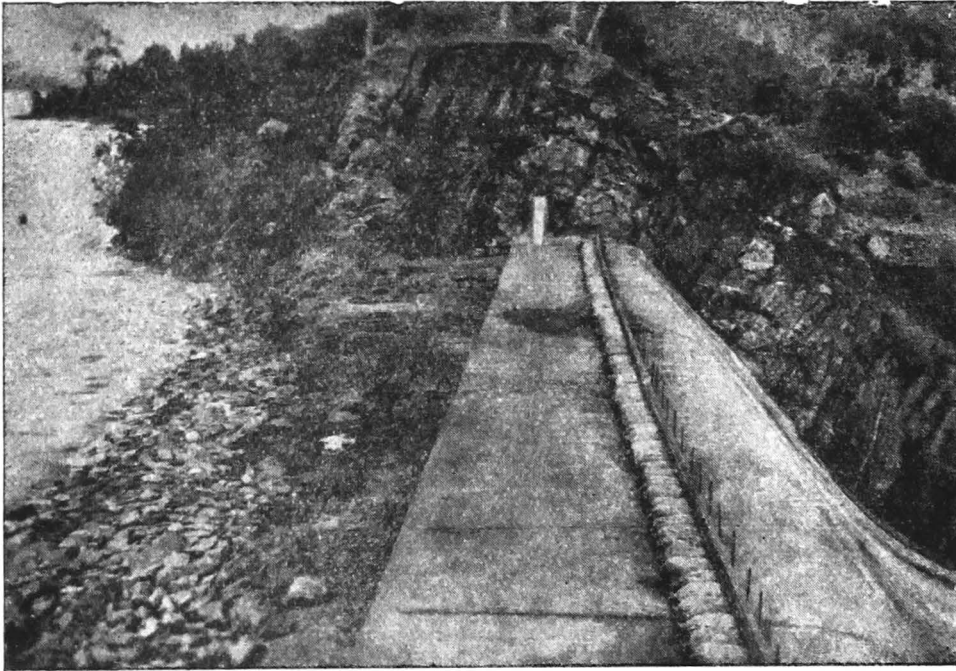


PHOTO No. 5396. Showing crest of the existing main spillway.

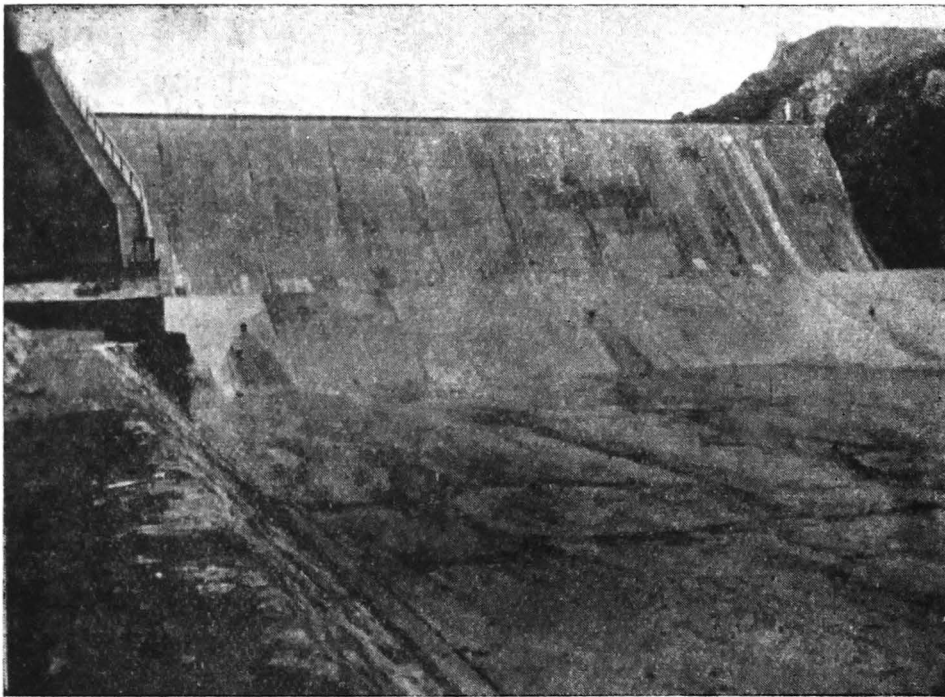


PHOTO No. 5395. Showing the main spillway and downstream cistern and baffle-Photo looking upstream.

$$Q = 3.2 \times 150 \times D^{1.5} \quad (1)$$

which with the maximum flood level of R.L. 533 in the reservoir would give $Q = 12,960$ cusecs. (A coefficient as 3.2, not 3.33 it may be noted, has been assumed).

With a high coefficient weir with crest at R.L. 524 and radial gates, the discharge over the weir would be given by

$$Q = 4.5 \times 150 \times D^{1.5} \quad (2)$$

i.e. with initial reservoir level of R.L. 527 and maximum flood level of R.L. 533 the main spillway would discharge 18,225 cusecs.

With the modified spillway crest at R.L. 522 the discharging capacity would be $Q = 4.0 \times 150 \times D^{1.5}$

$$= 21,892 \text{ cs.} \quad (3)$$

5.2. *The emergency spillway.*

5.2.1. The emergency spillway consists of an open cut in a saddle having a width of approximately 600 feet at the extreme western flank of the dam. Its present crest is at R.L. 530. As the spillway is an open cut with length in the direction of flow of nearly 350 feet, the flow over it will be of the drowned type and it is difficult to assess exactly — without model experiments — the discharge corresponding to various reservoir stages. Mr. Muzumdar has taken the discharge intensity per foot width, q , as given by Strange in the Standard book "Indian Storage Reservoirs". The discharge to depth relationship obtained from Strange's figure can be represented by the exponential formula:

$$q = 2.38 D^{1.6056} \quad (4)$$

where D is the upstream depth over sill.

It must, however, be pointed out that the conditions for which Strange's relationship applies are not satisfied in the present case; as in Strange's computations, no allowance has been made for losses in transit from the upstream to the downstream end of the crest. These may be negligible with crests of short lengths (such as perhaps those from which Strange's data are drawn) but must be taken into account when the crest is relatively long as in the present case; in order to avoid an over-estimate of the discharging capacity; the relation developed by Strange will have therefore to be modified to suit the conditions of flow at Dimna Reservoir.

5.2.2. *Results of Bhandardara Open Spillway experiments.*

Experiments (1) with 1/10 scale model of the Bhandardara Spillway (Bombay) were carried out at Poona Research Station to determine its dis-

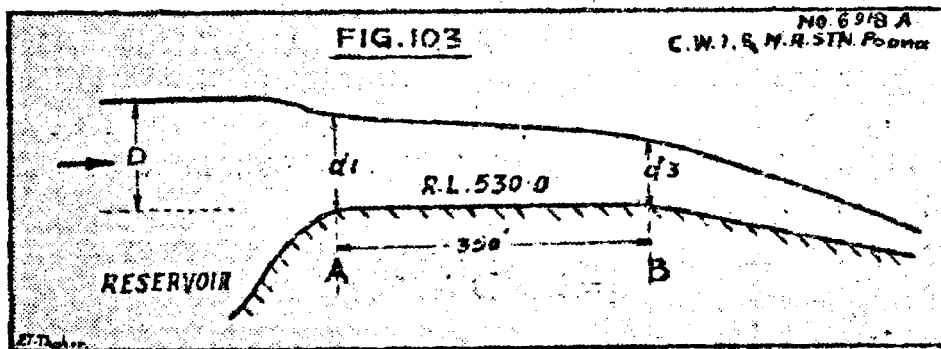
charging capacity and to evolve a suitable profile for a high coefficient weir. The spillway had a length of 650 feet and a nearly flat crest extending from 240 feet on the left flank to 600 feet on the right flank. The bed of the cut in the left half was extremely uneven. The roughness at site was reproduced to scale in the model as closely as possible. The discharge to depth relationship obtained for the prototype (i. e. after making due allowance in the model results for the scale effect) was

$$q = 0.77 D^{1.05} \quad (5)$$

Fig. 102. Photo 5394. Fig. 102 shows the water profiles obtained in the 1/10 scale model. At the Dimna emergency spillway the bed of the cut is, comparatively, less irregular and the spillway has a relatively lesser length of crest in the line of flow.

The discharging capacity for the Dimna conditions will therefore be somewhat more than that given by the relationship obtained from the Bhandardara experiments - vide (5.2.2.)

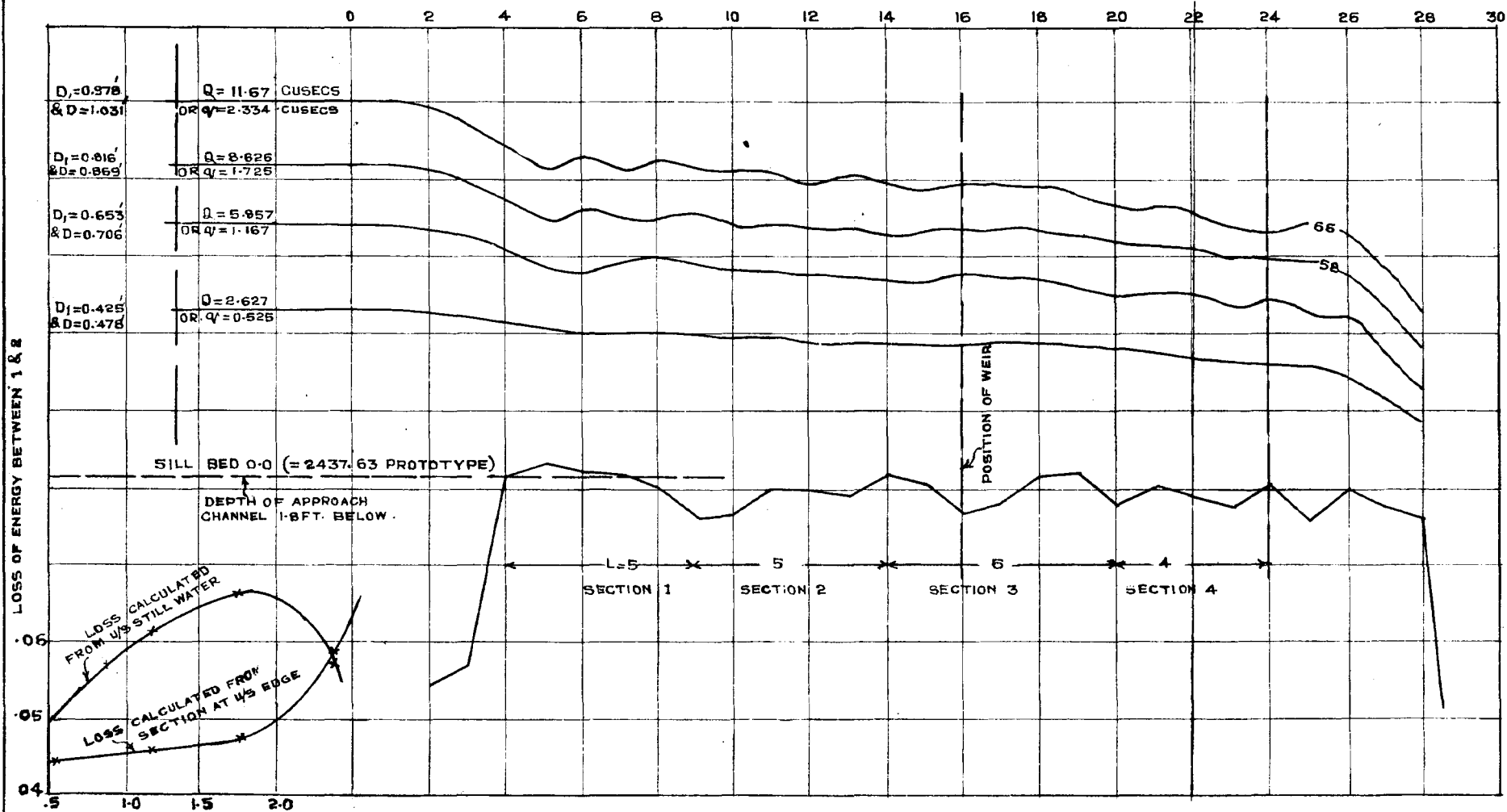
5.2.3 The discharge over the Dimna Reservoir Spillway can be worked out by making reasonable assumptions as shown below (fig. 103.)



The sketch above shows the L-Section of the spillway and water profile for an upstream depth D over the sill. If the slope of the bed immediately below A (in the sketch) is more than sufficient to compensate for frictional and other losses (i.e. more than the critical slope), the depth of flow at A would be critical; but in the present case the bed is nearly flat for a length of 350 feet and only thereafter i.e. from B, onwards, it begins to slope downstream, hence the depth of flow at A will be more than critical and flow will approach critical conditions downstream towards B. The depth at B, d_3 , would adjust itself so that $d_3 = KD$ where, theoretically $K = 2/3$ for maximum discharge and is independent of bed roughness.

FIG-102
WATER PROFILES IN SECTION A A
FOR VARIOUS DISCHARGES IN
1/10TH SCALE PART WIDTH MODEL OF
WASTE WEIR OF LAKE ARTHUR HILL (WILSON DAM)
AT BHANDARDARA

SCALE: $\frac{HOR}{VER} = \frac{1''}{4'} = \frac{1}{0.4}$



DIMNA NALLA WATER SUPPLY SCHEME—JAMSHEDPUR.

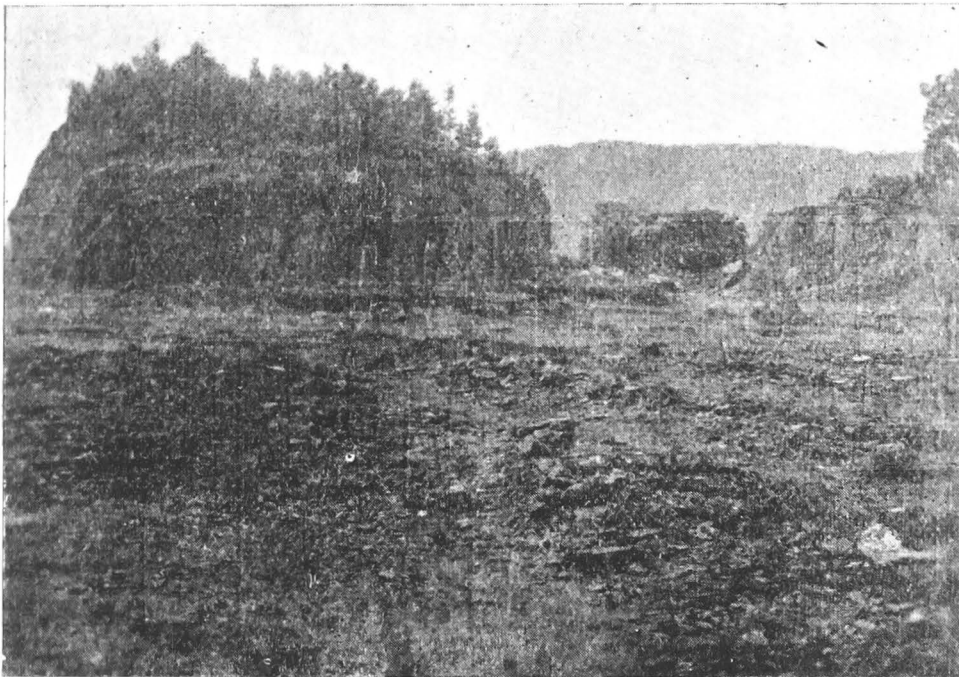
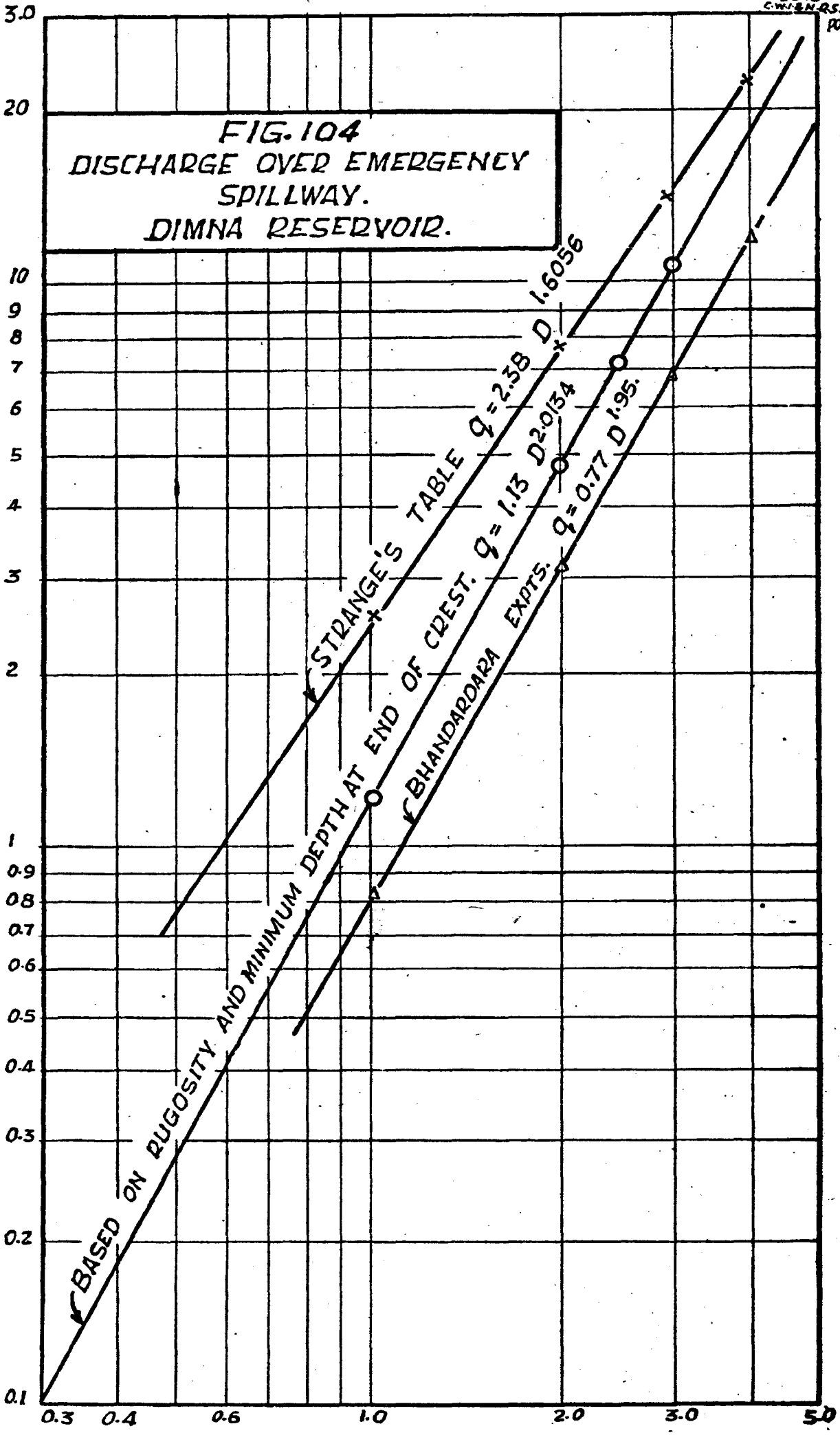


PHOTO No. 5394. Showing bed roughness in the open cut-emergency spillway.

FIG. 104
DISCHARGE OVER EMERGENCY
SPILLWAY.
DIMNA RESERVOIR.

Q_1 = DISCHARGE PER FOOT WIDTH OF SPILLWAY IN C.S.



D = DEPTH OF WATER U/S OVER SILL IN FT.

In practice, however, observations have shown that K is ordinarily always less than $2/3$, its actual value depending on the conditions of flow and of the crest at the entrance (i.e. whether sharp, rounded or smooth), For smooth entrance the value of K is 0.63 while for a very sharp entrance its value drops to below 0.6. In the Dimna spillway a value of 0.6. may be reasonably assumed, as it is neither very smooth nor sharp. This value of K has been found to hold good in the case of the Bhandardara Spillway.

Disregarding the velocity of approach at the entrance, the discharge at section A computed for a given drop in water level is

$$q = C_d \sqrt{2g(D - d_1)}$$

with an average value of 0.925 for C .

5.2.4. The usual value of Manning's rugosity coefficient for this type of boundary conditions is 0.03 (as given in Standard Works on Hydraulics) and the mean flow over the reach AB (in the sketch) would be given by

$$V = \frac{1.486 R^{2/3} S^{1/2}}{0.03}$$

5.2.5. The best estimate of discharge will be one that satisfies equations 6 and 7.

The table below shows the discharges worked out by this method. The statistical relation obtained for this estimate is $Q = 1.13 D^{2.0134}$

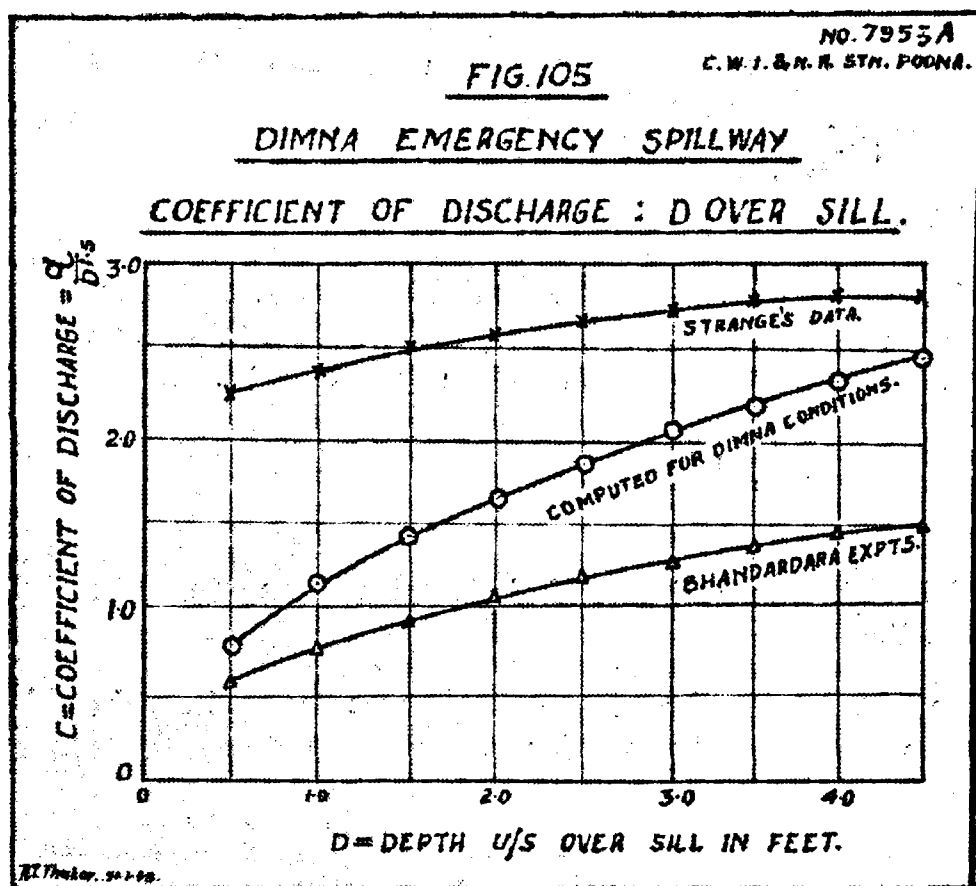
D in ft. :	q Statistical in cusecs	q (according to Strange) in cusecs.
1.0	1.122	2 380
2.0	4 650	7.285
2.5	7.128	10. 60
3.0	10.203	13.705

5.2.6. It will be observed from the table that the discharging capacity is less than that obtained from Strange's relationship. In fig.104 are plotted the discharges corresponding to various upstream depths according to the statistical relationship considered most proper for the Dimna conditions; the Strange's curve and the curve obtained from the Bhandardara experiments. The curve shows, at a glance, to what extent the discharges worked out by Strange's curve are more and those obtained from the

Fig. 104.

relationship fitting the Bhandardara data are less than the discharges to be expected for the Dimna Nala conditions. For reasons explained above in the subsequent calculations for routing the maximum flood the discharging capacity for the Dimna emergency spillway has been taken according to the Statistical relationship obtained above.

Fig. 105. The curve showing D.C for the three relations is shown in fig. 105.



5.27. Mr. Muzumdar has considered that, with crest of the Emergency Escape at R.L. 530 and the Main Escape as constructed (i.e. with crest R.L. 525) a flood of 40,000 cusecs can be passed without the water level exceeding the safe level R.L. 533. His anticipations, appear to be unduly optimistic in the following respects :—

- (1) He has assumed a higher coefficient for the Main Escape; at the most, a coefficient of 3.2 may be justified against 3.33 assumed by him.
- (2) He has used Strange's relationship for emergency escary which gives too high a discharge for a given D. The actual discharges are likely to be less for reasons explained; the difference is as much as 30% at H F.L. 533.0.
- (3) The peak flood of 40,000 cusecs can only be discharged, even under the optimistic assumptions made by him if its duration does

not exceed 3 hours 17 minutes. Although no rigidity is claimed for the estimate of the duration period of $4\frac{1}{8}$ hours, it is desirable to provide for a flood flowing for about that period.

Tables 7 and 8. 5.3.1. Tables 7 and 8 are prepared giving the outflow at various reservoir stages with a flood of 40,000 cusecs with the water level in the reservoir at F.S.L. 527.0 at the beginning of the flood. The method adopted is the same as used by Mr. Muzumdar in his Note.

Table 7. 5.3.2. From table 7 it will be seen that with a flood of 40,000 cusecs the water level in the reservoir would rise to M.F.L. (i.e. R.L. 533) within 3 hours and 12 minutes. This is not considered sufficient and a bigger spillway capacity is indicated. If the coefficient of the main spillway is increased from 3.2 to 4.5 by suitable modifications of its profile, an inflow of 40,000 cusecs could be accommodated for 3 hours and 40 minutes without the water level rising over the maximum safe level. As even this is not considered sufficient it would also be necessary to lower the top level of the Emergency Spillway.

5.3.3. Table 8 shows similar computations with the same reservoir F.S.L. of R.L. 527 and a high coefficient weir constructed at the Main Spillway (crest level R.L. 524) but with the top level of Emergency spillway lowered to R.L. 529. This Table shows that an inflow of 40,000 cusecs could be accommodated four hours and four minutes without allowing the water level to exceed R.L. 533; the main spillway will pass 18,225 cusecs and the emergency spillway 11,050 cusecs, the balance of 10,725 cusecs, being absorbed by the reservoir.

5.4.1. If however the F.S.L. in the reservoir is raised to R.L. 529 — since analysis of the available supplies shows it is desirable to keep the higher F.S.L. of R.L. 529 (vide para 3.2.5. above) — the flood absorption capacity between R.L. 527 and R.L. 529 would be lost. It would, therefore be necessary to provide for a bigger outflow over the Main Spillway. Higher intensities of flow over the emergency spillway are believed to be undesirable because there is no leading channel downstream. Provision is therefore, proposed for additional discharging capacity in the main spillway.

Table 9. 5.5.1. Table 9 is prepared, identically to Tables 7 and 8, giving the period over which a maximum flood of 40,000 cusecs can be passed over the Spillways and partially absorbed in the Reservoir without the water level in the reservoir exceeding R.L. 533 when the tank had been full up to R.L. 529 prior to the flood. For this purpose, however, the crest of the high coefficient weir in the main spillway is required to be lowered to R.L. 522.

It may be noted that the coefficient of discharge adopted is only 4.0 against 4.5 assumed in Table 8 because, in this case, the ratio of the height of the weir to the depth of flow of the weir is reduced. It is also assumed that the gates on the main spillway will come into operation when the water level in the reservoir is or exceeds R. L. 529 and are completely out of water, free from causing any obstruction to the profile. The top of the emergency spillway is, moreover, required to be lowered to R.L. 529.

5.512. From Table 9 it will be seen that with the above arrangement the maximum flood of 40,000 cusecs can be surplussed (with due allowance for flood absorption) provided its duration does not exceed 4 hours. As stated above, the duration of $4\frac{1}{8}$ hours obtained from Gurtu's formula need not be adhered to absolutely and arrangement suitable for surplussing a flood of 40,000 cusecs occurring for a period of 4 hours may be considered adequate, particularly in view of the fact that the duration of the normal flood has been estimated to be much less and the intensity of a flood and its duration are known to vary inversely, i.e. the higher the flood the lesser its duration.

5.6. *Conclusion.*

With the above proposals, maximum flood can be surplussed even if the tank was previously full at R.L. 529, which is the level desirable having regard to the supplies that will be available for filling the tank in the major number of years.

Although only safe and reasonable assumptions have been made in the above calculations it is, nevertheless, considered necessary to assess the exact values of the coefficients of discharge for both the spillways from model experiments.

6.0. Issue IV — Measures for the dissipation of energy of flow over the main spillway and the reduction of damage to its floor.

6.1. In this connection the Director would first of all suggest that an examination should be made whether the damage that has occurred was due to insufficient relief from uplift pressures caused by sub-soil flow or due to the tremendous velocity of surface flow. The data are not available before the Director to make this examination, specially in respect of uplift pressures.

6.2. With regard to the measures for dissipation of energy of surface flow model experiments are considered essential to give a sound and well-considered opinion. These experiments can be conveniently combined with the experiments for the optimum profile of the weir required in connection with the other issues and it is strongly recommended that these experiments should be done. The cost of the experiments will be only a fraction of the saving that will accrue in maintenance year after year.

Table No.1. Monsoon (June October) rainfall data.

Station	Distance from centre of Dim- na Catchment.	Short period mean rain- fall	Long period mean rain- fall	Maximum rainfall	Minimum rainfall	Period over which observed	Ratio
	miles.	inches	inches	inches	inches	years	$\frac{\text{Col. 4}}{\text{Col. 3}}$
1	2	3	4	5	6	7	8
1 Chaibassa	34	50.10	44.38	66.15	23.94	55 (1899-1943)	0.89
2 Purulia	32	50.36	46.07	66.77	32.03	44 ,,	0.91
3 Ranchi	65	55.83	50.92	77.61	32.72	55 ,,	0.91
4 Jamshedpur	7	51.80	46.84	76.91	26.10	35 (1909-1943)	0.90
5 Dimna Catchment	0	55.45	49.94* (estimated)	71.08	40.34	8 (1936-1943)	0.90 (Assumed)

NOTES:— Column 3 is the short-period mean for the 8 years 1936-43.

Column 4 is the long period mean for the period shown in column 7 except for the Dimna basin which has been estimated* from the Jamshedpur ratio 0.9 (vide column 8).

Column 8 ratio of long period mean (column 4) to the short period mean (column 3).

*Table No. 2
Monsoon Inflow at Dimna Reservoir observed between years 1936 to 1947.*

Year	Rainfall June to October in inches	Inflow in million gallons for A = 40 sq. miles	Inflow in million gallons for A = 36 sq. miles	Inflow in million gallons per square mile of catchment	Inflow per quare mile of catchment in million cft.	Remarks.
1	2	3	4	5	6	7
1933	46.33	9030.1	8566.0	237.95	38.07	Run-off observation started on 28-6-36 so inadmissible for correlation.
1937	60.51	11981.0	11355.38	315.70	50.51	
1938	44.93	4943.4	04889.39	130.26	20.84	
1939	56.14	5196.7	4929.68	136.94	21.91	
1940	40.34	4733.5	4490.28	124.73	19.96	
1941	60.56	Not recorded				
1942	71.03					
1943	62.74		10908.6	303.02	48.48	
1944	52.25		9539.2	264.98	42.40	
1945	51.10		7608.9	211.36	33.82	
1946	68.17		19457.2	540.48	86.48	
1947	38.95		5359.5	148.83	23.82	

Table No. 3.

								Remarks.	
1	Reservoir level.	524	525	526	527	528	529	530	
2	Gross storage Capacity (m. bft.)	938	990	1042	1094	1150	1208	1267	
3	Net storage capacity (excluding storage below sluice level),	885	937	989	1041	1097	1155	1214	
4	Yield per square mile of catchment required to fill the net capacity (m. cft.)	24.6	26.0	27.5	29.0	30.5	32.1	33.8	
5	Total monsoon rainfall required to produce the above yield (inches).	43.6	44.8	46.1	47.3	48.5	49.7	51.0	From the Statistical curve on fig. 98.
6	Percentage variation of (5) from mean monsoon rainfall of 50".	-12.8	-10.4	-7.8	-5.4	-3.0	-0.6	-2.0	
7	Frequency of the occasions when the reservoir can be expected to fill up to the level given in (1):— once in number of years.	1.5	1.5	1.5	2.0	2.0	2.0	2.5	
8	Percentage probability of filling.	72.5	68.0	62.0	58.0	52.0	50.0	42.0	

Table No. 4

Rate of average and maximum inflow at the Dimna Reservoir based on inflow hydrographs of 1937, 1939, 1940 and 1943.

Date.	Rainfall during the date as recorded on the next date at 8 A.M. in inches.	Area of inflow hydrograph in sq. inches.	Total inflow in hour cusecs.	Period of inflow in hours.	Average rate of inflow cusecs.	Maximum rate of inflow observed in cusecs.	Ratio of Max. inflow		Duration of Max. inflow observed in hours	Remarks.
							Average inflow			
28-7-37	0.56									Rainfall for 24 hours assumed= $7.33 + 1.29 = 8.32$
29-7-37	7.33	3.80	60,800	18	3378	8278	2.450	1.5		
30-7-37	1.29	4.09	65,440	24	2727	8278				
22-6-39	1.60									Run-off based on 10 a. m. 23-6 to 10 a. m. on 24-6-39.
23-6-39	7.94	4.03	64,480	12	5373	12074	2.247	1.0		
24-6-39	0.30	4.20	67,200	24	2800					
25-8-40	Nil.									Rainfall for 24 hours assumed to be= $3.53 + 4.71 = 8.24$.
26-8-40	3.53									
27-8-40	4.71	3.62	89,920	24	3750	12612	3.366	0.5		
3-8-43	1.73									Run-off hydrograph from 8 a. m. on 4-8 to 5-30 a. m. on 5-8-43. Run-off hydrograph from 5-30 a. m. on 5-8 to 8-00 a. m. on 5-8-43.
4-8-43	3.89	2.55	40,800	21.5	1900					
5-8-43	2.65	3.81	60,960	26.5	2300	6860	2.982	1.0		

Worst Ratio = 3.366

Table No. 4 (A)

Rainfalls and average run-off (24 hours) in Dimna Nala Catchment during
(Monsoon high run-off periods) years 1943 to 1946.

Date	Rainfall during the date 8 A.M. in inches	Total inflow computed \bar{E} outflow + shortage + supply to town during 24 hours in m. cft.	Average rate of inflow in cusecs	Remarks
1	2	3	4	5
1 8 43	1.42	14.63	169	
2 8 43	3.08	119.31	1380	
3 8 43	3.81	166.66	1925	
4 8 43	1.73	99.85	1185	
5 8 43	3.89	181.39	2000	
6 8 43	2.65	186.62	2160	
25 8 44	1.35	57.37	663	
26 8 44	5.96	307.40	3560	
27 8 44	2.07	41.04	475	
4 8 45	0.06	5.50	—	
5 8 45	3.26	53.27	616	
6 8 45	0.46	83.01	960	
29 8 46	0.07	21.29	246	
30 8 46	0.63	48.00	555	
31 8 46	4.27	128.03	1482	
14 9 46	nil	23.88	276	
15 9 46	4.14	159.88	1848	
16 9 46	2.36	129.87	1500	
17 9 46	3.67	308.48	3570	
18 9 46	0.69	99.73	1152	

Table No. 5.

Showing areas of zones contributing to run-off at various distance from the Dimna Reservoir.

	Limiting distances of the zone from the spillway in miles.	Area of the zone in square miles	Mean distan- ce of the zone miles.	Percentage area,	Summation per- centage.	Maximum distance of the zone in miles.	Col. 5 Col- 6	Remarks.
	1	2	3	4	5	6	7	8
1	0-0.5	0.445	0.25	1.239	1.239	0.5	2.478	
2	0.5-1.0	1.240	0.75	3.453	1.692	1.0	4.692	
3	1.0-1.5	2.075	1.25	5.778	10.470	1.5	6.981	
4	1.5-2.0	3.030	1.75	8.433	18.908	2.0	9.454	
5	2.0-2.5	3.820	2.25	10.638	29.546	2.5	11.818	
6	2.5-3.0	4.200	2.75	11.696	41.242	3.0	13.781	
7	3.0-3.5	3.970	3.25	11.005	52.297	3.5	14.942	
8	3.5-4.0	4.640	3.75	12.921	65.218	4.0	16.306	
9	4.0-4.5	3.960	4.25	11.028	76.246	4.5	16.944	
10	4.5-5.0	3.630	4.75	10.109	86.355	5.0	17.271	Maximum
11	5.0-5.5	2.090	5.25	5.820	92.175	5.5	16.760	
12	5.5-6.0	1.510	5.75	4.205	93.375	6.0	16.063	
13	6.0-6.5	1.300	6.25	3.620	99.999	6.5	15.384	
		<u>35.91</u>						

Table No. 6.

Showing average intensity of rainfall of different duration corresponding to 12.5" of rainfall in 24 hours (per hour = .5208".)

Duration in hours.	Average precipitation for period.		Total for the period in inches.
	Percent of intensity.	Intensity in inches/hours.	
0.25	169.2	8.812	4.406
1.0	100.0	5.208	5.208
1.83	69.0	3.590	6.570
2.0	64.7	3.370	6.740
2.5	55.0	2.864	7.160
3.0	47.7	2.484	7.452
4.0	38.5	2.005	8.020
6.0	28.4	1.470	8.874
12.0	16.9	0.880	10.560
24.0	10.0	0.520	12.480

Table No. 7

Range of reservoir level.	Storage capacity with the range in column (1) in m. cft.	Average rate of surplus discharge from the reservoir within the range in column (1)			Average rate of flood absorption within the reservoir—40,000—Col. 5 (cusecs)	Time taken to fill the storage capacity within range of column (2) Minutes.	Total time since the commencement of the Flood. Hrs. Mins.	Remarks.
		Over the main escape (cusecs)	Over the emergency escape crest R. L. 529 (cusecs)	Total average rate of surplus discharge (cusecs)				
1	2	3	4	5	6	7	8	9
527.0								
527.5	28	2813	Nil	2813	37187	11.06	0—11	Crest of main
528.0	28	3485	...	3485	36515	12.78	0—24	spillway R. L.
528.5	29	4206	...	4206	35794	13.50	0—37	524.0 length = 150.
529.0	29	4969	...	4969	35031	13.89	0—51	C = Coefficient of
529.5	29	5775	...	5775	34225	14.12	1—5	discharge for main
530.0	30	6619	...	6619	33381	14.98	1—20	spillway.
530.5	30	7501	42	7543	32457	15.40	1—36	= 3.2
531.0	31	8418	380	8798	31242	16.56	1—52	Top of emergency
531.5	31	9369	1063	9492	30568	16.90	2—9	escape at R. L. 530
532.0	31	10350	2091	12441	27559	18.75	2—28	length 600 feet.
532.5	31	11380	3470	14850	25150	20.54	2—48	Max. flood discharge = 40,000.
533.0	32	12420	5200	17620	22380	23.83	3—12	F. S. L. = 527.0

Table No. 8

Range of reservoir level.	Storage capacity within the range in column (1). in H. Cft.	Average rate of surplus discharge from the reservoir within the range in column (1)			Average rate of flood absorption within the reservoir = 40,000— Col. (5). (cusecs)	Time taken to fill the storage capacity within the range. Col. (2) Col. (6) Minutes.	Total time since the commencement of the flood.	Remarks.
		Over the main escape. (cusecs)	Over the emergency escape crest R. U. 529. (cusecs).	Total average rate of surplus discharge. (cusecs).				
1	1	3	4	5	6	7	8	9
527.0								H. C. Weir for main spillway.
527.5	28	3956	...	3956	36044	12.94	0—13	B = 150 ft.
528.0	28	4901	...	4901	35099	13.29	0—26	C = 4.5.
528.5	29	5900	...	5900	34100	14.17	0—40	and crest R. L.
529.0	29	6888	...	6988	33012	14.64	0—55	524.0.
529.5	29	8121	42	8163	31337	15.18	1—10	Emergency spill-
530.0	30	9309	380	9689	30311	16.50	1—27	way length = 600
530.5	30	10540	1063	11603	28937	17.61	1—41	feet. And top at
531.0	31	11840	2091	13931	26069	19.82	2—4	R. L. 529.0.
531.5	31	13170	3470	16640	23360	22.12	2—26	Maximum flood
532.0	31	14560	5200	19760	20240	25.53	2—52	discharge = 40,000
532.5	31	16000	7276	23276	16724	30.89	3—23	cusecs.
533.0	32	17470	9703	27173	12827	41.58	4—4	F. S. L. 527.0.

DIMNA NALA OVERFLOW SPILLWAY (JAMSHEDPUR)

Table 9

Range of reservoir levels R.L.	Storage capacity within the range in column (1) in m. cft.	Average rate of surplus discharge from the reservoir within the range in column No. (1)			Average rate of flood absorption within the reservoir with inflow rate of 40,000 cusecs	Time taken to fill the storage within the range in column (1) column (2) column (6) minutes.	Combined period since the commencement of the flood hrs.-mins.	Remarks
		Over the main escape (R.L. 722.0) (cusecs)	Over the emergency escape (R.L. 529.0) (cusecs)	Total average rate of surplus discharge. (cusecs)				
529.0-529.5	29	11,710	42	11,752	28,248	17.11	0 - 17	Crest of main spillway R.L. 522.0
529.5-530.0	30	12,940	380	13,320	26,680	18.74	0 - 36	
530.0-530.5	30	14,220	1063	15,283	24,717	20.29	0 - 56	F.S.L. R.L. 529.0 effective length of main spillway 250 feet and effective length of emergency spillway 600 feet
530.5-531.0	31	15,530	2091	17,621	22,379	23.09	1 - 19	
531.0-531.5	31	16,890	3470	20,360	19,640	26 31	1 - 46	
531.5-532.0	31	18,270	5200	23,470	16,530	31.26	2 - 17	Top at R.L. 629.0
532.0-532.5	31	19,690	7276	26,966	13,034	39 64	2 - 56	Maximum Inflow 40,000 cusecs.
532.5-533.0	32	21,140	9703	30,843	9,157	58.24	3 - 55	

Note:—Coefficient adopted for main spillway = 4.0

SECTION F—WATER UTILISATION PROJECTS

32 Meshwa Canal Project (Bombay)

(1) Introduction:

1.1. The monsoon in Gujrat practically ends by about the end of August. Any rice crop, therefore, planted on the early rains has a great danger of failing completely due to want of late rains if unaided by irrigation; whereas one or two waterings in September and October would give a bumper yield.

1.2. It is therefore proposed to construct a pick-up-weir across the Meshwa River which is a non-perennial and alluvial river, having a catchment area of 670 sq. miles at the pick-up-weir site near village Raska in Kaira District (Bombay Province).

The proposed scheme provides for only one canal, with full supply discharge of 334 cusecs, taking off from the right bank of the weir for irrigating areas between the Khari and Meshwa rivers, and also for filling two important tanks, viz. Galki and Ghoblaj, of capacities 50 and 123 m.cft. respectively.

2. Salient features and details of the Project are shown
 Statt. 1. in Statement I.

2.1. The Executive Engineer, Irrigation Project Division (North) Ahmedabad apprehends excessive draw-off of bed sand owing to the location of the head on the inner side of a gentle curve. It is, therefore desirable to orient the Head Regulator suitably to incorporate some form of sand-excluding device.

2.2. Table 1 shows the velocities at the pick-up-weir site, with and without the weir. It will be seen that the bed movement in the ponded reach upstream of the weir will start with discharge greater than 18,000 cusecs. As the average monthly discharges obtainable in September and October are short of the minimum discharge at which the bed movement begins in the ponded reach, there is little danger of the canal drawing excessive sand and as such sand-excluding devices do not appear to be necessary. But as the Meshwa river appears to be a flashy type of river there is the possibility that the discharge cycle of the year is such that the minimum discharge, at which the bed movement just starts, synchronises with

Table 1

Table showing the velocities at the pick-up weir site with and without weir.

Velocities at the pick-up-weir site without weir					Discharge in cusecs	Velocities at the pick-up-weir site with the weir				
W.L. at the site	Areas in sq. ft.	P in ft. (wetted perimeter)	R in ft.	V Velocity in ft./sec		W.L. at the site	Velocity in ft./sec.	V _L upstream of the weir	$\frac{V}{V_L}$	Remarks
108.0	1,100	340	3.26	3.75	4,120	114.50	0.84	2.67	0.31	No bed movement expected. Just upstream of the weir.
110.0	1,550	366	4.24	4.40	6,800	115.50	1.29	2.72	0.475	
112.0	2,500	396	6.33	5.35	13,400	117.25	2.30	2.96	0.78	
114.0	3,100	420	7.38	5.80	18,000	117.90	2.86	2.92	0.985	Bed movement may start up- stream of the weir.
116.0	3,820	440	8.65	6.27	24,000	118.50	3.65	2.92	1.27	Appreciable bed movement may start upstream of the weir.
118.0	5,200	549	9.60	6.55	34,000	120.70	4.47	3.04	1.45	
120.0	6,310	584	10.8	6.95	43,800	123.50	4.90	3.17	1.55	
121.0	6,900	610	11.30	7.00	48,000	124.00	5.25	3.17	1.66	

NOTE:—Velocity at the pick-up-weir site (without the weir) is worked out by the Mississippi formula.

$$V_L = 1.067\sqrt{fR}$$

$$V = \left[(225RS^{\frac{1}{2}})^{\frac{1}{4}} - 0.0388 \right]^2$$

the peak demand of the canal. Moreover, there is so much of heading up over the natural river water levels for commonly occurring discharges that there would be very heavy accretion within a few years owing to the substantial damping of velocities upstream of a weir for long periods of the season. This will seriously affect the smooth working of the canal. Under these conditions the problem needs further consideration in the matter of selection of the site for the weir, positioning the Head Regulator and the necessity of sand excluding devices.

(3) *Selection of site.*

3.1. The site selected by the Executive Engineer is shown in fig.2, the site being considered suitable for the reasons set out below:-

- (a) The river at the pick-up-weir site is stable for the last 20 years.
- (b) There is an overflow of the Khari river meeting about 2,000 ft. above the present site; the weir below the junction is considered preferable.
- (c) Shifting the site upstream would bring it within State Territory which is considered objectionable. (This reason will not now be valid.)

3.2. Though the stability of the river course and adequate command of the area proposed to be irrigated are important factors in choosing the site, advantages of design should not be sacrificed thereto, particularly the correct location of the offtake in an alluvial river.

Reliable, permanent and efficient silt or more correctly sand exclusion, can be obtained by correctly positioning the offtake towards the end of a fully established concave bank so as only to draw off relatively sand-free top water, the bed sand being deflected midstream.

This is nature's way of exclusion and experience as well as experiments have fully substantiated this form of exclusion to be highly efficient and lasting.

3.3. Tunnel type Excluders & Ejectors for excluding the sand from the head or ejecting it after its entry into the canal are also not feasible under the conditions obtaining at the site. Firstly because of the fineness of bed sand at the weir site ($m = .15 \text{ m. m}$) such Excluders or Ejectors would have very low efficiency; and, secondly, though sufficient head required for the Ejectors and Excluders is available at the site, the minimum escapage discharges of 33% and 20% respectively required for them would only be available at the cost of sacrificing the supplies vitally required for irrigation, when supplies are already short to the point of making the Project a precarious undertaking.

3.4. Moreover, because ponding counteracts curvature in this case where there will be excessive ponding it will not do to site the offtake near the transition of the curve.

For these reasons the best site, suited to the river conditions and the canal withdrawals, would be about 1/2 a mile upstream of the site selected by the Executive Engineer as sand exclusion can naturally be obtained there due to favourable curvature. But as crossing of the Khari river overflow is considered objectionable, the site 1,500 ft. upstream of the site selected by the Executive Engineer vide Fig. 2 is suggested as the next best alternative.

The all time advantages obtainable at this site would certainly repay any additional cost involved in locating the weir at the upper site.

(4) *Position of Head Regulator.*

In the Project the Head Regulator is proposed by the Executive Engineer to be constructed well away from the weir. It was not clear why the Head Regulator should be constructed so far away from the weir especially when the river banks are stated to be stable at the weir site. It is only when the river is unstable at the offtake of an *inundation canal* that it may be wise to retire the Regulator well inland, at an adequate distance inside the approach channel with provision to ensure that this length of the channel remains clear of sand.

At the site proposed by the Station the Head Regulator may be constructed on the right bank of the river. It is not so much the angle of offtake of the Head Regulator which arrests the excess influx of bed sand as the correct positioning of the offtake relative to the river curve upstream.

(5) *Recommendations ;*

The weir site should be shifted at least 1500 ft. upstream of the site selected by the Executive Engineer.

Even though the weir is located on the concave curve as proposed by the Station the excessive ponding inherent in the Project will lead to fairly rapid accretion upstream and subsequently also downstream. The river after accretion will adversely affect the smooth working of the canal and sand-exclusion

FIG. 1

MESHWA RIVER

GRAPH SHOWING GAUGE - DISCHARGE

RELATIONSHIP U/S OF THE -

PICK-UP - WEIR SITE . (WITH & WITHOUT WEIR)

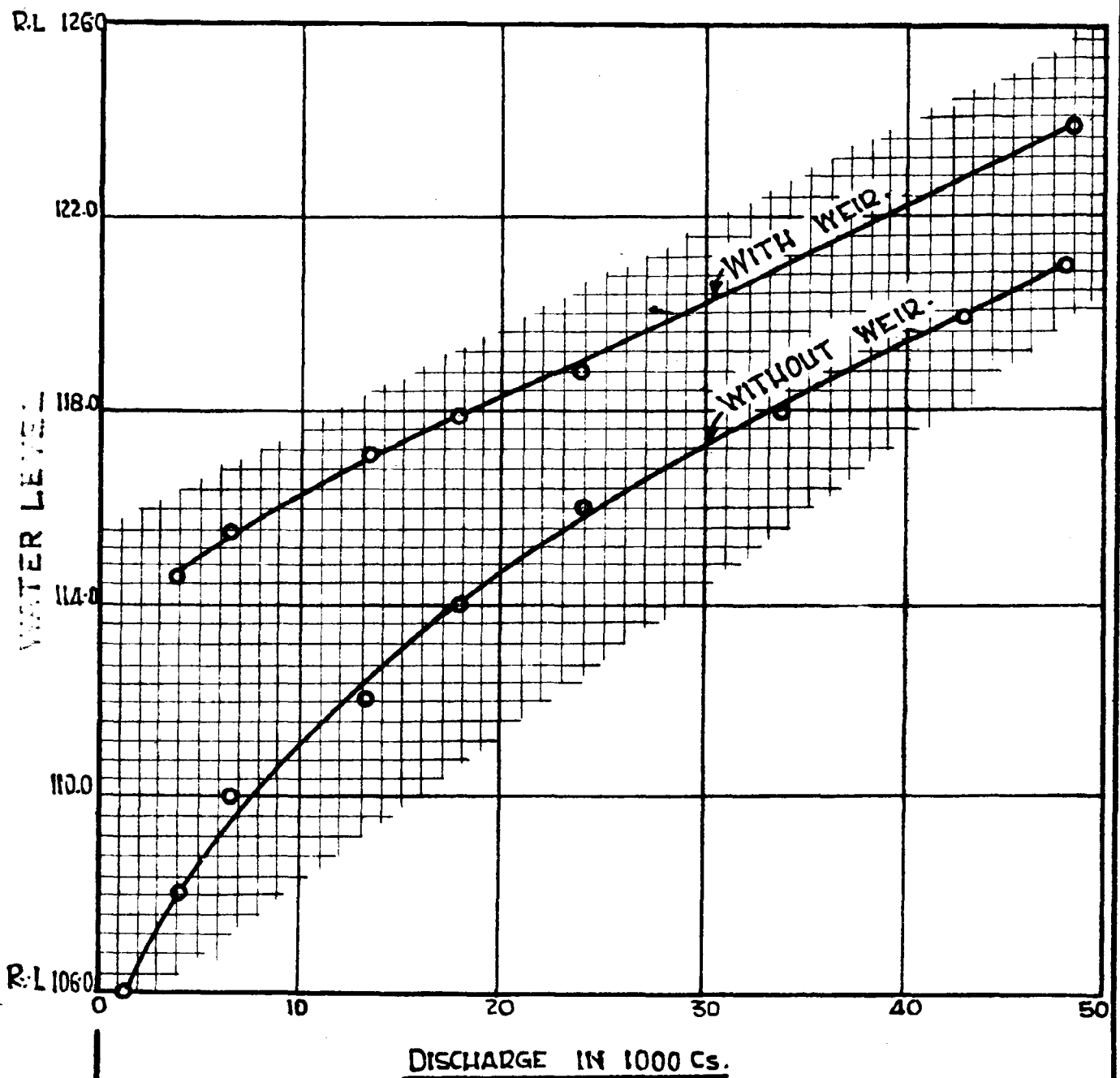
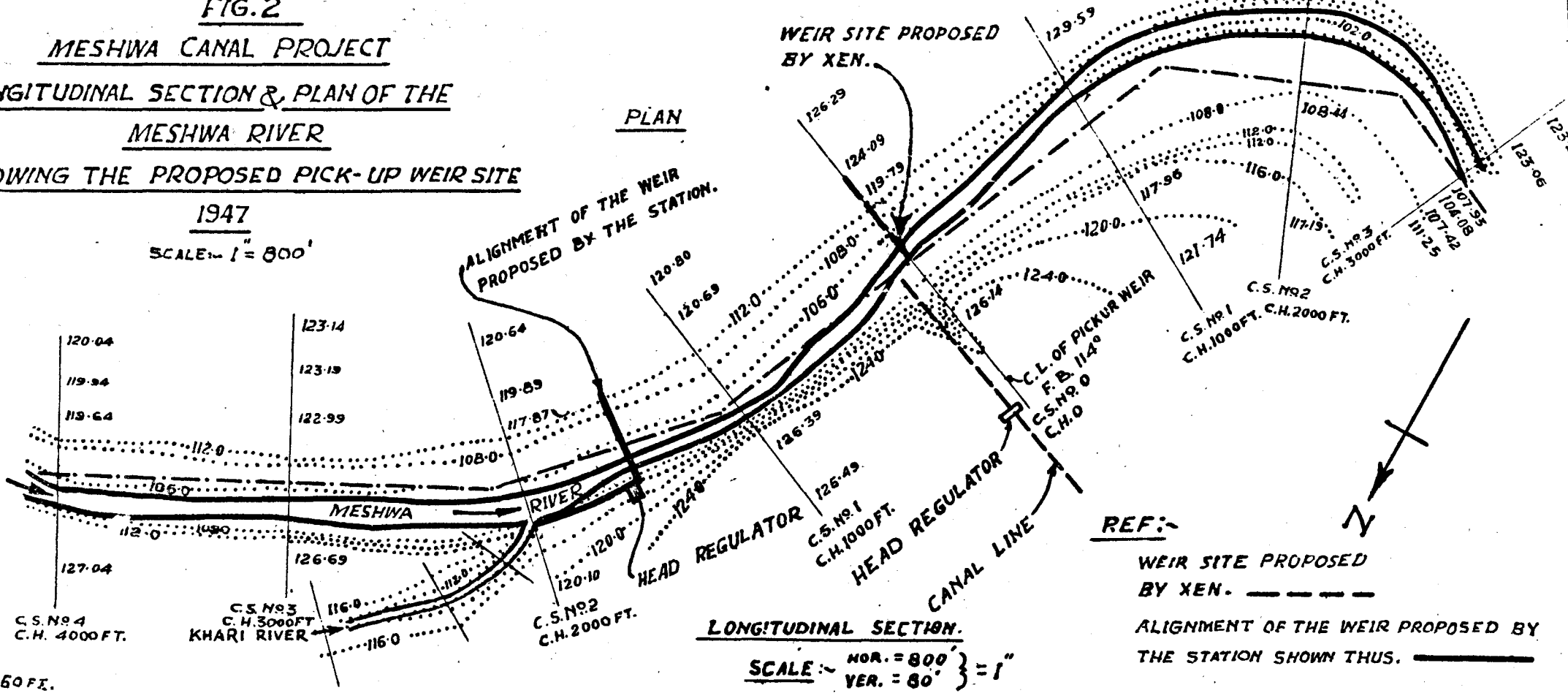


FIG. 2
MESHWA CANAL PROJECT
LONGITUDINAL SECTION & PLAN OF THE
MESHWA RIVER
SHOWING THE PROPOSED PICK-UP WEIR SITE

1947
SCALE: 1" = 800'



DISTANCE IN FEET.	4000	3800	3600	3400	3000	2800	2600	2400	2200	2000	1800	1600	1400	1200	1100	1000	800	600	400	200	0	200	400	600	800	1000	1200	1400	1800	2000	2200	2400	2600	2800	3000
GROUND LEVEL.	106.24	107.29	106.94	106.99	106.29	105.60	105.99	106.18	106.29	106.54	106.84	106.84	106.84	105.84	105.44	105.09	105.19	104.94	104.79	104.19	104.69	102.79	104.29	104.94	104.89	104.79	104.79	104.79	106.34	106.34	107.24	105.44	105.34	103.54	102.96

BY: RAJITSINGH. 3-3-49.

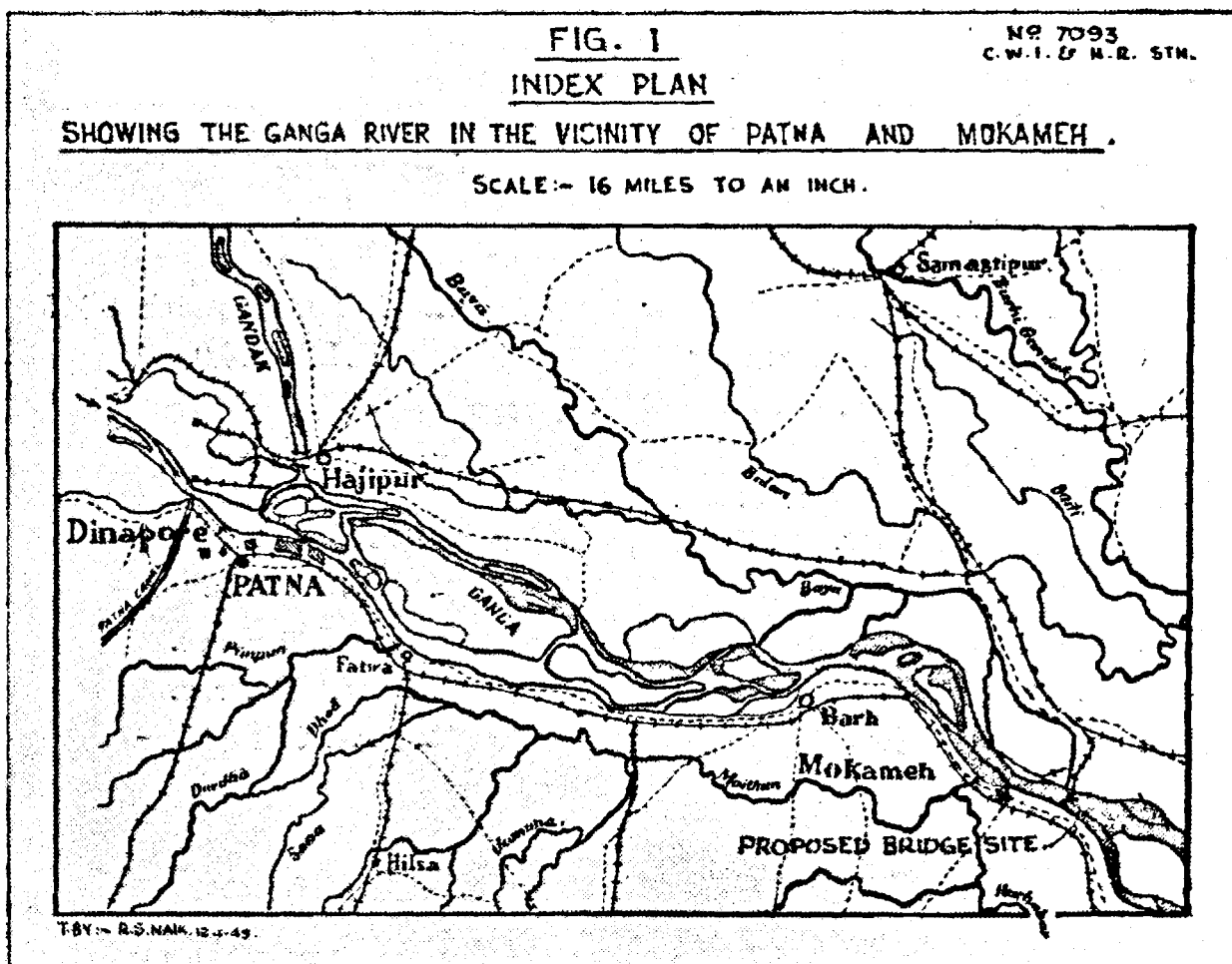
STATEMENT 1.*Salient features and details of the Meshwa Canal Project.*

(1)	Catchment area at the pick-up-weir site	...	670 sq. miles.
(2)	Average annual rainfall.	...	33.02 inches.
(3)	Maximum flood discharge from the catchment at the weir site.	...	44,683 Cusecs.
(4)	Highest flood level.	...	R.L. 121.00
(5)	Lowest bed of river at the weir site.	...	R.L. 103.0
(6)	Proposed crest of the pick-up-weir.	...	R.L. 112.50
(7)	Total length of the weir (including undersluices).	...	480 ft.
(8)	Length of the undersluices portion.	...	64 ft.
(9)	Sill of undersluices.	...	R.L. 106.0
(10)	No. and size of Head Regulator.	...	5 openings each 6'x5'
(11)	Crest of the Head Regulator.	...	R.L. 106.0
(12)	Average bed fall of the river at the weir site.	...	1 in 2,335
(13)	H.F.L. with afflux after construction of the pick-up-weir.	...	R.L. 123.70
(14)	Length of the weir canal.	...	9.15 miles.
(15)	Area under command.	...	18,000 acres.
(16)	Area proposed to be irrigated.	...	16,000 acres.
(17)	Cost of the Project.	...	Rs. 25,69,712/-

G—GENERAL

33. Submarine Cables across the Ganga at Patna and Mokameh - Best alignment for.

1. The Station was approached by the posts and Telegraph Department for suggesting suitable alignments for laying submarine telegraph cables across the Ganga river at Patna and Mokameh. So that damage to the cable by alternate silting and scouring of the river bed would be minimised.



At both these places, bridges for crossing the Ganga are contemplated and the proposal of the Mokameh bridge has already been approved by the Railway Board. When these bridges are constructed the cables will obviously be taken over them, but in the interim period cables will have to be run along the bed.

Fig. 2.

In the vicinity of Patna, the Ganga is no where so stable as to preclude the possibility of either silting or scouring. From data it was however apparent that the best alignment would be between sections AB and CD in fig. 2. Considerations guiding this selection of area between lines AB and CD were the following.

FIG. 2 SHOWING COURSES OF THE WINTER CHANNELS OF THE GANGA RIVER AT PATNA FOR DIFFERENT YEARS.

SCALE. 1.5 MILE TO AN INCH.

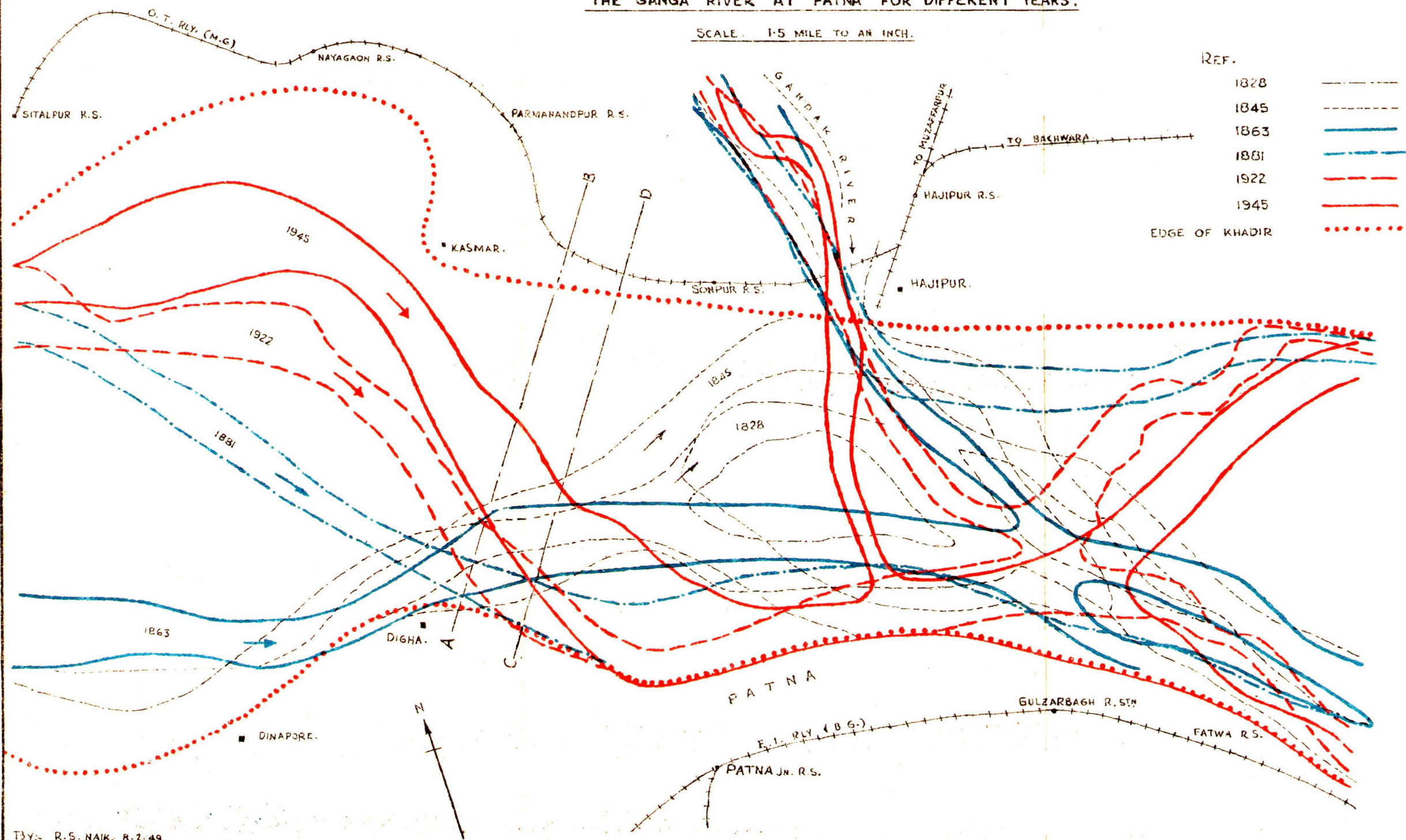
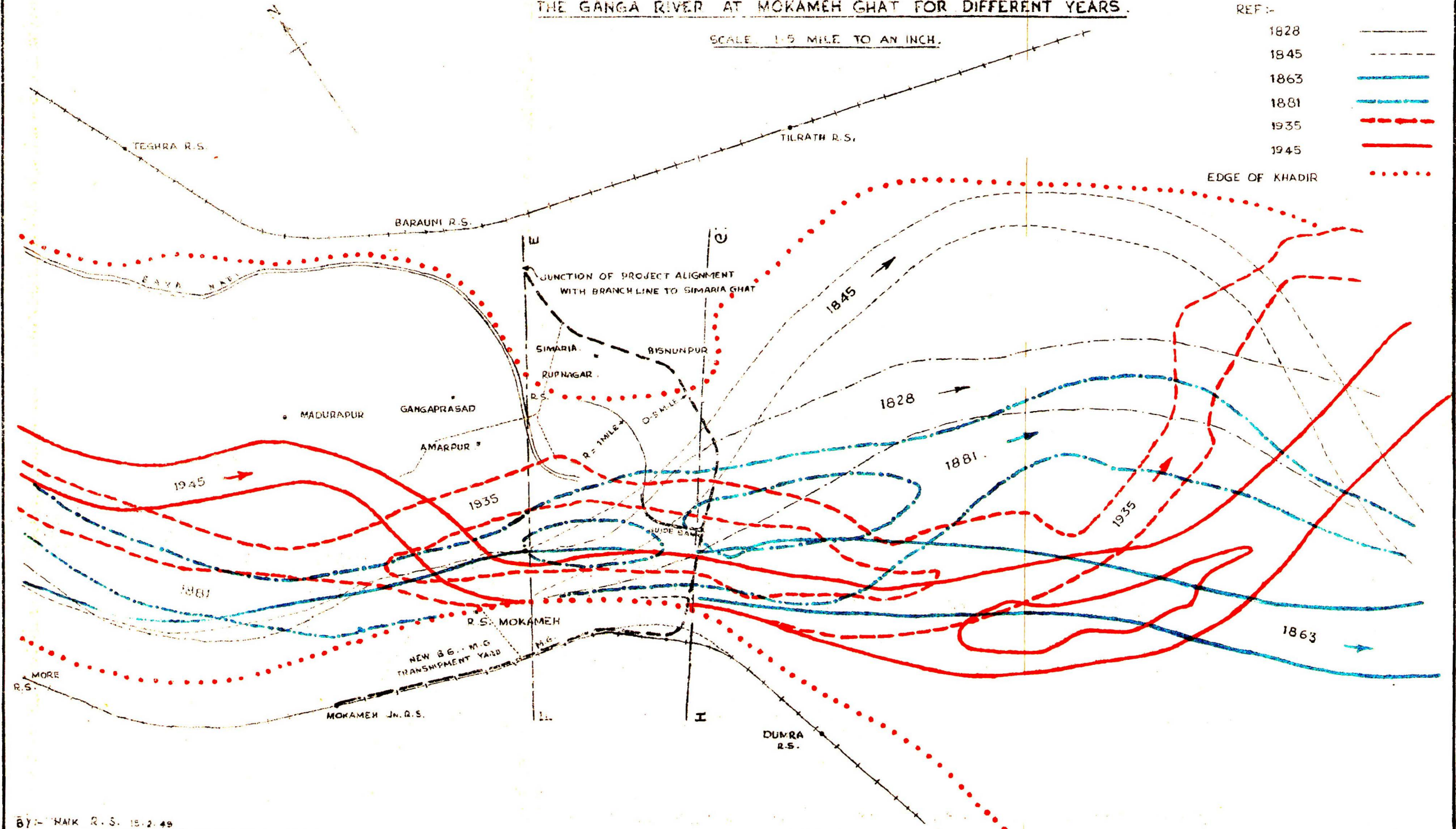


FIG. 3 SHOWING COURSES OF THE WINTER CHANNELS OF THE GANGA RIVER AT MOKAMEH GHAT FOR DIFFERENT YEARS.

SCALE: 1.5 MILE TO AN INCH.

REF:-

1828	—
1845	- - -
1863	— (blue)
1881	- · - · - (blue)
1935	- · - · - (red)
1945	— (red)
EDGE OF KHADIR	· · · · ·



(i) On both the sides, permanent high banks (Khadir limits) are stable since 1928, which indicates that further encroachment by the river is improbable.

(ii) The width of the river from bank to bank i.e. of the river Khadir, is about the minimum, thus requiring a minimum length of the cable to be laid.

(iii) The proposed area in which the cable may be aligned is sufficiently retired from the point of confluence of the Ganga and Gandak so as not to need frequent realignments of the cable due to shifting meanders of the Gandak.

(iv) The relatively stable region is across the 'node' of the various river courses of different years (1828-1945) so that the extent of the alternate silting and scouring of the bed is likely to be the least and limited only to a small part of the river width.

(v) The proposed area is not far removed from the alignments suggested by the Director General, Posts and Telegraphs on considerations perhaps, of availability of communication facilities and economy at the same time its right end is close to the Patna City.

3. At Mokameh the river is comparatively more stable than elsewhere in the neighbouring river reach and the alternate scouring and silting has been and will in all probability be least and much restricted. The river Khadir banks have been almost permanent and Khadir width very narrow. Thus Mokameh has always been looked upon as an ideal site for crossing the river.

Zone EFGH was recommended in this case as most secure. Alignment nearer EF was however reported to be preferable as it is further away from the proposed bridge line and will preclude the possibility of failure of the cable due to anchorage by steamers in the vicinity of ghats and wharves.

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1.	IHR 1.38	Annual Report of work done during 1937—38	1939	R . 7—8—0
2.	IHR 1.39	Annual Report of work done during 1938—39 <i>By C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff at Khadakwasla.</i>	1940	Rs, 7—8—0
3.	IHR 2	Note on "The theory, design and construction of Gibb Models which, without moving parts, give a constant discharge within working imits irrespective of variations in upstream and dawnstream water levels" <i>By C. C. Inglis and Rao Sahib D. V. Joglekar</i>	1941	Rs. 3—8—0
4.	IHR 1.40	Annual Report of work done during 1939—40 <i>By C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff at Khadakwasla</i>	1941	Rs. 7—8—0

No.	Serial No.	Publications & Authours	Year	Price
5.	IHR 1.41	<p>Annual Report (Technical) of work done during the year 1940-41 and index for 1937-41</p> <p><i>By C. C. Inglis, A. R. Thomas, Rao Sahib D. V. Joglekar and staff Khadakwasla.</i></p>	1942	Rs. 7—8—0
6.	IHR 1.42	<p>Annual Report (Technical) of work done during the year 1941—42</p> <p><i>By C. C. Inglis, J. S. Reid, Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i></p>	1944	Rs. 7—8—0
7.	IHR 1.43	<p>Annual Report (Technical) of work done during the year 1942—43</p> <p><i>By C. C. Inglis, J. S. Reid, Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i></p>	1945	Rs. 7—8—0
8.	IHR 1.43	<p>Annual Report (Technical) of work done during the year 1943.</p> <p><i>By C. C. Inglis, Rao Bahadur D. V. Joglekar and staff at Khadakwasla.</i></p>	1945	Rs. 7- 8—0

No.	Serial No.	Publications & Authours	Year	Price
9.	IHR 1.44	Annual Report (Technical) of work done during the year 1944. <i>By C. C. Inglis, Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i>	1945	Rs. 7—8—0
10.	IWE 1.45	Annual Report (Technical) of work done during the year 1945. <i>By Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i>	1948	Rs. 25—0—0
11.	IWE 1.46	Annual Report (Technical) of work done during the year 1946. <i>By K. K. Framji, Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i>	1947	Rs. 7—8—0
12.	IWE 1.47	Annual Report (Technical) of work done during the year 1947. <i>By K. K. Framji, Rao Bahadur D. V. Joglekar and Staff at Khadakwasla.</i>	1948	Rs. 25—0—0
13.	IWE 3-1	"The behaviour and control of Rivers and Canals" (with the aid of Models) (in two parts) <i>By C. C. Inglis.</i>	1949	Rs. 16—8—0

ERRATA

Page No.	Line No.	Incorrect	Correct
8	18	1486	1.486
10	9	13850 cusecs = 7460 m.g.p.d	13850 cusecs \equiv 7460 m.g.p.d
25	23	August	August 1948
29	23	upt	up-
29	24	experimen	experiment
29	30	Traiding	Training
33	17	$Q_{\max} = 136$	$Q_{\max} = 1.36$
33	24	R. L. 15675	R. L. 1567.5
33	29	photo 2227	photo 5227
36	14	But, is	But, it is
A5	3	h o e	hole
A6	10	L_2	L_2
A6	16	L_2	L_2
37	32	experiences	experienced
38	23	currcent	current
38	36	comprise	compromise
39	3	groyene	groyne
39	34	top of at	top at
41	Marginal note	photo 4249	photo 5249 & photo 5244
A7	28	abopted	adopted
50	11	090 ft.	0.90 ft
57	6	Erosion	17. Erosion.
58	7	Tidal Model	18. Tidal Model
	31	effecti s	effect is
64	Table 1, Col 4 Line 5	5,196	5.250
65	Table 1. Sub-heading of Col. 24.	$Q_{\text{Scale}} = \frac{L}{d^{3/2}}$	$Q_{\text{Scale}} = L \cdot d^{3/2}$
	Last line	$V = 1.20 (\phi - \phi)^{0.37}$ etc....	$V = 1.30 (\phi - \phi)^{0.27} L^{0.27} M^{-0.27} h^{0.23}$
		=	=
		(... x 0.2094 x 0.822)...	(... x 0.2084 ÷ 0.822)...
69	Marginal Note of sub-para 3,3	Fig 55	Fig 57
71	12	041	0.41
74	19	Menthod	Method
78	Marginal note of sub-para 3,1		photo 5409
78	Last marginal note of sub-para 3,3	photo 5400	Photo 5409

(ii)

Page No.	Line No.	Incorrect	Correct
83	Last line	5500 ft. parallel with guide bank...	5500 ft. parallel to and upstream of the barrage for $Q = 3,00,000$ cs., alternatively with guide bank...
84	Table 1, Col 2	070 et seq	0.70 et seq Decimal for whole column
86	11	afflux	afflux
88	16	R.L. 520	R.L. 52.0
88	Marginal note of sub-para 6.2.1	Fig 6	Fig 76
A12	Last line	$R = (\quad)$	$R_2 = (\quad)$
95	12	in 1941	in 1941 (see fig. 83),
97	5	par million	per million
98	4	Protuberances	Protuberances
103	Table 3, Col 7 & 8	2700 595	270.0 595
	Last 2 lines	27.00 6.33	270.0 63.3
122	9	to very	to very
129	10	$q = C_d \sqrt{2g(D-d_1)}$	$q = C_d \sqrt{2g(D-d_1)}$